PERFORMANCE ASSESSMENT OF STORM DRAINAGE SYSTEM, THE CASE OF WELKITE TOWN ,CENTRAL ETHIOPIA REGION,ETHIOPIA

THE DEGREE OF MASTERS OF SCIENCE IN WATER RESOURCE ENGINEERING AND MANAGEMENT

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A THESIS SUBMITTED TO THE

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ADVISOR'S THESIS SUBMISSION APPROVAL SHEET

This is to certify that the thesis entitled "Performance Assessment Of Storm Drainage System, The Case Of Welkite Town" submitted in partial fulfillment of the requirement for the degree of Master's with specialization in Water Resource Engineering, The graduate program of the school of Bio system and water resource Engineering, Institute of Technology, Hawassa university and has been carried out by Mr. Tamirat Amde. Therefore I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department for defense.

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We, the undersigned, members of the Board of Examiners of the final open defense by **TAMIRAT AMDE** have read and evaluated her thesis entitled "PERFORMANCE ASSESSMENT OF STORM DRAINAGE SYSTEM, THE CASE OF WELKITE TOWN ,CENTRAL ETHIOPIA REGION,ETHIOPIA", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfilment of the requirement for the degree .

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Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the Council of Graduate Studies (CGS) through the Department Graduate Committee (DGC) of the candidate's major department.

DECLARATION

I , TAMIRAT AMDE declare that this thesis entitled "Performance Assessment of Storm Drainage System Sustainability, the Case of Welkite Town" is my work and that all sources of materials used for this thesis have been duly acknowledged.

I solemnly declare that this thesis is not submitted to any other institution anywhere for the award of any academic degree, diploma, or certificate requairments**.**

Name: Tamirat Amde Wolde

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TABLE OF CONTENTS

LIST OF ABBREVIATION AND ACRONYMS

a.s.l ………………..above sea level

Cg………………….Center of gravity

ERA………………..Ethiopian road authority

EPA-SWMM……….Environmental Protection Agency Storm Water Management Model

GC………………….Gregorian calendar

GIS ………………..Geographic information system

HEC-RAS…………. Hydraulic Engineering Center River Analysis System

HEC-GeoHMS……...Hydraulic Engineering Center Geo spatial hydrologic modeling system

HEC-HMS………….Hydraulic Engineering Center Hydrologic Modeling Flood Hydrograph

HEC.........................GeoRAS - Hydraulic Engineering Center Geographical River Analysis System

Km………………… Kilo meter

MoWE……………… Ministry of Water and Energy

MSE………………….Micro and Small Enterprises

MoFED…………........Ministry of Finance and Economic Development

MoWE……………….. Ministry of Water and Energy

NGO…………………..Non-governmental organization

SCM…………………..Stormwater Control Measure

SNNPR………………..South Nation Nationalities & Regional Government

SWMM……………….Storm Water Management Model

SWCM…………..Storm Water Computer Model

UIIDP……………Urban Institutional and infrastructural Development Program

USWM …………..Urban Storm Water Management

WSPRO…………..Water Surface Profile Roadway/Bridge Over Flow

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDECES

ABSTRACT

Most storm water drainage problems investigated with respect to its hydraulic performance.In Wolikite town drainage structures are not well-designed to carryout the runoff due to blockage & capacity on size of the ditch.The objective of this study is ,to assess storm water drainage system performance of the town. The study employed both primary and secondary data collection. To achieve the specific objective HEC-RAS model analysiss has been used to calculate peak discharge and the averge rainfall using log pearson type III, in order to develop the Intensity Duration Frequency (IDF) curve for the rainfall intensities recorded in different duration and analyzed for a period of 1991-2021. Detail evaluation of drainage channels is required to control damages on high storm. For sizing of existing and proposed drainage channels manning formula is used and the hydrological peak flow was determined by using Rational Method. It was found the discharge using rational method is that 1.78mcu , and the result from the HEC RAS model 0.08mcu compared with existing hydraulic capacity of the channels . It shows that study area storm water was higher with respect to the drainage channels to carry the expected runoff during high rainfall events, make incapable to carry the runnoff for drainage ditches of the study area. To resolve these problems BMP's have been recommended and finally, the analysis from this study permits future work on re design of the town storm drainage ditches and resettlement of flood affected areas and amend land use of the town for residential should be considerd in connection with town master plan development .

Key words: storm water, HEC-RAS, storm water drainage, runoff, channel capacit

1 INTRODUCTION

1.1 Background

As human settlements rose higher, they quickly began influencing the natural hydrological processes. Ditches were dug, fields were under-drained, streams straightened and rivers were embanked in order to quickly take water from the land to the sea (Farjana et al., 2019).At this time, many watercourses running through towns and cities are encased in large pipes under the ground,and drainage open channel civil structure development becomes the basic infrastructure worldwide in the urban areas now demanded and historical (Steven J et al., 2002). In doing so, the natural water cycle has significantly disrupted landscapes and wildlife habitats have been destroyed. These impacts are related to quality and quantity variables of the hydrologic cycle (Buttler et al., 2018). In Ethiopia, small, medium and large town storm drainages are constructed with a tremendous amount of budget to minimize and control the damage of storm flooding in urban adminstrations. Due to their poor performance of functionality (Hawaz, 2020), assessment of system sustainability require finding out basic problem of conveyance, capacity and usability to regulate , amend and correct necessary input data during design. From this point of view,natural resource protection for all residents world wide entail urban waste and storm drainage disposal infrastructure development and plays the biggest role in the environment , after assessing the damage arisen.

Most developing countries like Ethiopia have low coverage of urban storm water management system (Adugnaa et al., 2019), providing neither on site management nor safe removal & discharge to natural receiving command. Drainage ditch designs should be consider number of problems that require effective structural drawing for construction purpose. Shape, alignments, slope, protection of debris and waste, mixing of waste with storm water challenge considered as criterion for new developed design.However the constructed ditches or drainage channels overlook most of the norms performed for safe removal of storm in the selected outlet.To handle and manage design principles, stake holder liability,capacity building and budget source are an activity to be done preliminarly.

As the process of urbanization accelerates, drains become increasingly overloaded and unable to cope with heavy rainfall,if appropriate study and design work not performed to the town /city standard. Urbanization along with its impermeable structures and improper design are the major causes of flooding in most developing urban areas in Ethiopia (Brook Legese.and Boneya Gumi, 2020), this urbanization affects the performance of the artificial drainage lines. Drainage system seek to manage rainfall in a way similar to natural processes, by using the landscape to control the flow and volume of surface water, prevent or reduce pollution of downstream development.

Storm water drainages channels in urban areas usually designed to collect excess run off to convey in appropriate end points to protect from disaster of flooding of the compound (Sambeto et al., 2023) but , this designs , after implementation face design gap, limitations of functionality and adequacy or carrying capacity of excess storm flow conveyance, serviceability through time, sedimentation and siltation, increasing in material roughness and structural failurity.Moreover climate and urbanization change aggravated the problem (Bassma Taher Hassan, Mohamad Yassine, 2022). This affect considered and utilized data variations followed by excess flood and peak flow rate. This means, implemented drainage system networks had been accepted state of functionality, capacity of drainage system networks are incapable for such phenomena result of excess flooding for extreme events of flow.

Hydrological modelling plays significant role in designing hydraulic structures for sustainable functionality and long service by lowering unexpected and environmental assessments. Sutable approaches for the control of calculated and expected run off starting from the beginning runoff are preferable than methodologies based on structural conveyance (Abraha, 2018). and structural conveyances play a big role for collecting excess run off and umber of studies had been done on improving of urban storm drainage system , such as ,case of mekele city, result on emphasizing faults on sizing of the drainage ditches, to control flooding.

The main objective of this study was to assess storm drainage system to perform better drainage system in Welikite town Bekur kifile ketema ,Tenatabiya sefer and Arsema area. For better collection and dispose of excess runoff, the appropriate performance of urban drainage system play a big role in collecting and disposal of high flood. In order to reach the goal, following the state of the art in drainage system of water resource, the study region simulated through HEC RAS model. Finally, system performance of drainage network assessed by means of indicators like reliability, durability and adequacy. At the end the total assessment result on adequacy and capacity by combining the above notified indicator index

1.2 Statement of the Problem

Drainage designs must integrate scientific attention with local protection methods to solve the harm in the towns through detail data collection inputs for satisfactory and acceptable conveyance activity and avoid construction adequacy limitation.

Welikite town is one of the old towns in the country, which has not sufficient drainage system in both storm and sewer lines. Significant proportion of the town is exposed to flooding during the rainy season (Tenatabiya Sefer and Arsema). This has been the cause for extensive loss of human residential and properties, serious environmental pollution, community health problem after high storm due to clogging of dry wast , sewerages line that pass through storm drainages dishes.

In Welkite Town storm water drainage system problems identified as they are filled by sediments, suspended material, sewer drainages directly added to storm drainages, that exposed some specific environment of the sub-catchment polluted and are drawback to serviceability of storm drainages,charactersticgs of the soil to cary on the structurs and in the other way round backward construction trend exercised to carry out the runoff properly is a serious case in the town.

This study aims will be evaluate capacity of the major storm water drainage ,assessing the hydraulic performance of selected drainage system, to determine the major problems for the performance of the drainage system of the existing drainage structures.

1.3 Objectives

1.3.1 General objectives

The main objective of this study was performance assessment of storm drainage system, the case of Welkite town.

1.3.2 Specific objectives

- To evaluate the capacity of existing urban storm water drainage system in the study area.
- To determine the hydraulic performance of the existing storm drainage system
- To identify the major problems associate in hydraulic performance of the storm drainage system

1.4 Significance of the study

Generally, managing urban storm water drainage system has a significant role for practical environmental management by keeping the service life of urban infrastructures, such as roads, buildings, telephone lines, water supply lines and the existing rivers, keeping human health safe. Therefore, benefits that will be draw from this study may contribute to current exertions by governments and other concerning body to solve the problem of drainage schemes that contribute for better service coverage assessing key problems to fascilitate for study and design purpose and maintence. It also contributes to the government design stakeholder by identifying the main and basic parameter to be included during design. This work will make drainage construction more feasible and acceptable during high flood time of the town and simplify storm drainage challenges.

The result of this study also may help in filling the data gaps by identifying problems to performance assessments by introducing the BMP option, taking proper designing of storm water drainage system and proper functioning of drainage schemes in the town and early protection. The town use it as a reference while they are preparing their annual plans for urban drainage system. The output of this work will benefit other sister towns to have lessons in case of new designs and also used as an input and base line documentation for other researchers as a reference.

1.5 Scope of the study

Evaluating the whole catchment of the town might be necessary, but representative sample is compulsory to come up with solution for the current storm water problem. Therefore, some representative major flood prone areas are selected. The research will be conducted in Welkite town drainage system, Central Ethiopia Region, Ethiopia. Which has a distance to Addis Ababa 153km and from Hawassa, 260km. The main work in this study was using clearly identified and analysis missed parameters, data presentation, optimize existing drainages, assessing the performance of the exixting drainage system, rainfall data collection, rate of flood, size and construction behavior of drainage capacities and soil type as a source information to reach the target objective of the research. Generally; it will address issues related to urban storm water drainage and give specific focus on evaluating the extent and performance of the existing drainage system and proposing best management practice for the existing problem.

2 LITERATURE REVIEW

2.1 Urban storm water drainage and its problem

Due to unstudied establishment of urban areas based on the natural drainage line,devastated hazards happened in the residents and abolish tremendous belongings of the community in cases with incapabale size of the channel, junction to junction conveyance problem and endpoint disposal conditions in the world. The research on urban flood demanded on properties of dry land concentration,built up environment,when the urban drainage and sewage system capacity cannot handle the storm water from extreme rainfall (Devis Buttler and John W,Devies, 2004).It found that, necessary to implement storm drainage infrastructure due to number of cases substentiating typical cases to excute in urban areas to take lessons on aggressive rise up of flooding, such as built up areas concentration in urban land territories, ground water lebel,soil type,catchment charcterstics,urban development,vegetation cover,etc. (Heidi Birch and Maria Bergman, 2008). Its alredy observed that flood ocuured not only the runoff occurrence but also the existance of enough collection mechanism with quality and quantity wise.

The practice of urban drainage in developing countries encounters more serious problems than those of developed countries because urban development occurs under more difficult socioeconomic,technological and climatic conditions.Developing countries experience accelerated urbanization without adequate investment & infrastructure. Urban concentrations have environmental consequences in the form of urban flooding and pollution of water courses, soil and air. In developing countries, concern for the environment, Legal or concealed urban development, runoff quality,climate engineering ‘know-how’, Population and administrators relations that yield lack of interaction to the situation. (Silveira, 2002) conclude that,inadequecies in the collection of runnoff and contaminants ,restrict the application of works to retaine and increase soil infilitretion of stormwater, usually only possible to control runoff volumes. These difficulties are further intensified by substantial volume of garbage and sediment entering in storm water channels and also by climataic and epidomological risks which may severly increased by socio – economic and terrestrial factors.

2.2 Urban storm water drainage and solid waste experience in Ethiopia and major problems

The reserches collected from 2016-2021 in Gregoriyan calender,in some Ethiopian towns ,which are affected by flooding and key problems identified and necessary measures that ought to be taken to safeguard the urban drainage network of the towns.

The researcher Asfaw, (2016) assess the Storm Water Drainage System of Kemissa town, a problem stated and that have been observed in the town were base failure, depressions, shoving, edge cracks, shoulder erosion, abatement damage, silted drainage ditches and flooding. He used hydrologic model of Arc GIS and DEM and rational method to calculate peak runoff and hydraulic analysis sized by manning equation. After such process he conclude and recommend that the storm drainage facility are inadequate to convey the peak discharge for required design period and the drainage system filled by sediment and other rubbish material. Based on the result these problems are due to the drainage design and construction practice adopted by ignoring hydrology and hydraulic analysis, type of drainage system provided and management problems of the concerned body and unawareness of the community.

The other researcher BESHIR, (2014) the major source of solid waste in welikite town are commercial institutions such as,bars,hotels and restourants.Due to no land fill sites ,peoples are forced to dispose the waste from home to other nearby areas of the town like road side,infront of residentials and commercial institutions irregularly.These causes Sanitary conditions of the area detriorate and certainly will have high potential environmental and human health risk and he recommend that landfill sites for the waste to be dispose in suitable place to decrease damage for sanitary and human health risk and as per detailed discription of Ketema, (2018) on sustainable Strom Water Management by Implementing Low Impact Development in Jemo Addis Ababa, identified a problem by visual assessment, along the drainage system, in many points, suggests there is a common flooding problem which creates a sever traffic crowding,especially in rainy season. The flooding also affects the asphalt pavement by eroding and creating a number of depressions and result in longer period impact even after a rain. The impact is too much on the socio economic activity of the area in which it affects day to day activities of the community. The researcher had seen mechanism of minimizing high traffic ocupations of roads during rainy season and protecting safty of the road by extra damage and he recommend that; evaluating the hourly rainfall and update the IDF curve,updating coeficients for design and revision of design document for execution, finaly reconstructing the drainage structures.

Storm water discharges are produced when the capacity of the land to retain precipitation is exceeded and run off occurs. Run off influenced by rainfall intensity and duration, antecedent storms and number of watersheds, land use characteristics such as slope, soil type and impervious surface as assessment provided by Worajo, (2021) and investigate that operational problem plays abigest rall and inadequacy of storm drainage conveyance structures. The researcher conclude that . rehabilitation and reconstruction of structures

2.3 Urban storm water distruction , disposal and management experiences in Ethiopia

Tremendous efforts have been taken to assess the existing performances of storm water drainage infrastructure different parts of Ethiopia , thire effects on human health thrugh drinking waterand the dangerous outcome disposal of wests in the storm drainages system in the town . Few of these studies are presented either as academic outputs or project reports.

In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of the excess water/runoff through proper drainage facilities become essential as discussed and identified by Tafete, (2013). Drainage problems in Ethiopian urban centers include flooding, deterioration of roads, land degradation, sedimentation, water logging, blockage of drainage facilities and the like. With urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.), drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future environmental conditions of urban centers as report stipulated ERA, (2013).

After its inception, Federal Urban Planning Institute has been involving in planning and design of urban storm water drainage facilities as part of the master/development plan of a city/town with the objective of keeping the life of urban infrastructure and to protect the urban environment like water pollution from non-point sources of storm water, air pollution from stagnated water and Soil from erosion and degradation. The federal urban planning institute under the Ministry of works and urban development has been trying to put a considerable effort in controlling runoff, which is produced as a result of urban structural pavements and external sources, like flooding from entoto and yeka mountains in Addis Ababa tht assessed by Belete, (2009).

Before the three decades ago, there has been no formal working organization in the area of urban storm water drainage system. Even now a day the attention towards urban storm water system is at its immature stage that is why most of the urban storm water drainage structures get blocked with solid waste of various types after huge amount of money has been invested on them. In some areas they by themselves are sources of environmental problems as briefed Werkneh., (2017). The technologies in handling the environmental problems of urban storm water drainage in Ethiopia, which have been practiced, are not in a position to utilize the flood/runoff for various uses, like the treatment/sedimentation of runoff water, construction of detention ponds and other perforated structures for the water to be infiltrated in to the soil, rather the primary aim of urban storm water drainage system in the country is to safely discharge the storm/run-off out of the urban centers, Belete, (2009).

2.4 History and policy of urban storm water drainage In Ethipia

Number of studies has been done in Ethiopia to evaluate the status of the drainage system ,starting from 2010-2021 in gregorean calender.As per Besha, (2016) has conducted case study aboute the challenges of urban drainges system in some developing towns of the country.The case study of (Assosa Town ,Benishangulgumz) To look at the availability,coverage and management of drainage system.The study identified that ,current drainage channels are not adequate,theire maintenance work not complete.The main flooding of the town has raised ,due to poor maintenance work, lack of solid disposal and illegal connection of sewers are the main cause of flooding in the town.

According to Melesse et al., (2023) concerns aboute the rapid spatial expantion of urban areas and their adverse consquences continue to focus on livable and susutainable urban development strategies and policies that ensure self sustaining urban communities.Such type of explanatory findings make the country to develop policies and strategies to handle the above problems before incidence of of storm disasters , due to lack on land use management strategies in urban areas.

The Ethiopia land allocation policy follows an approach that can be termed ”part to whole” according to the urban planning proclamation of Ethiopia.The approach explains,the application of land area allocation stnadards in the development of urban area from piece of urban plot.General proportion of urban center for road and related infrastructure proportion is 25-30% dedicated for road right of way which include pedstrain road and crossing,vehicular ways(major collector and local),bikeways,green areas along (side in the middle),road ways,utilities (water,drainage,electricity).

2.5 Development of modern urban storm drainage practices

The beginning of modern urban drainage practices was initiated in european cities during the nineteenth century. One critical turning point in urban drainage occurred during the middle of the nineteenth century.Due to this,urbanization is the changing of land use from forest or agricultural uses to sub urban and urban areas, as report prepared ,COUNCIL, (2008). As the nineteenth century progressed the concept of urban drainage changed with the incorporation of water-carriage sanitary waste collection into the urban drainage systems. Sanitary connections to the sewers were made legal and new sewers were constructed to drain storm water and sanitary wastewater.

2.6 Current urban drainage perspectives

Engineers continued to improve design concepts and methods. Extensive monitoring efforts immeasurably improved the understanding of urban drainage quantity and quality characteristics. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. As per Schou et al., (2018) identified that, addressing the challenges of increased runoff and simpler legislations,Preventing storm water from entering the urban sewer drainage sytem ,typically by using locally placed stormwater control measure (SCM) that utilize combination of the hydrological process storage,infilitretion,evapotranpiretion and delayed runnoff.

Methods to design and construct well performed urban drainage systems are currently being researched and tested. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area expeditiously to include the consideration of social, economic, political, environmental, and regulatory factors.

2.7 Storm water management

Stormmwater drainage in urban areas has become achellenge due to the rapid and random growth of urban areas,removal of vegetation,reduction in the effectiveness of drainage infrastructure ,and climate change by Lakshmi Raghu Nagendra Prasad Rentachintala, M. G. Muni Reddy,and

Pranab Kumar Mohapatra, (2022).This involves the development and implementation of a combination of structural and nonstructural measures to reconcile the conveyance and storage function of storm water systems, with space and related needs of expanding urban populations. It also involves the development and implementation of a range of measures or best management practices to improve the quality of urban storm water runoff prior to the discharge into receiving waters. Before reviewing storm water management system it is important; knowing the type, characteristics and function of the storm water drainage system.

2.8 Urban drainage systems

The Primery purpose of urban drainage system is to manage stormwater effectively. It aims to collect,convey and control the runoff generated during rainfall events,preventing flooding and minimizing damage to urban areas.By efficiently channeling stormwateer away from built-up areas,the drainage system helps maintain the safety and functionality of the urban environment by Jamal, (2017) classified as, a drainage system can be either natural or artificial. Many areas have some natural drainage which means the excess water flow to the lakes and rivers. Natural drainage, however, is often inadequate and artificial drainage (surface & subsurface) is required for safely removal of water from road pavements and its surroundings as assessement done by Asfaw, (2016).

Urban drainage systems can be thought of consisting of two main parts: the convenience-oriented, or the minor system, which contains the components that accommodate frequent, small runoff events; and, the emergency, or major system, which comprises the components that control infrequent but large runoff volume. Although many of the components are common to both of convenience and emergency systems, their relative importance in the two systems varies significantly, Hassen, (2016).

2.8.1 Types of storm water drainage system

A drainage system will include all the components needed to ensure that the substructure is properly drained, and may be formed of components such as; open ditches, closed ditches with pipe drains, drainage through storm water drainage pipes, channels and culverts as Asfaw, (2016).

2.8.2 Functions of storm water drainage system

One of the drainage system's functions is to collect surface water and/or ground water and direct it away, thereby keeping the ballast bed drained ,Hailemichael, (2015). The drainage system must also protect the substructure from erosion, from becoming saturated and losing its load-bearing capacity and stability.

Drainage systems are provided in order to prevent spread of disease by contact with fecal and other waterborne waste, to protect drinking water sources from contamination by waterborne waste and to carry runoff and surface water away while minimizing hazards to the public reserched by Blom, (2015). She conclude that, drainage syastems and its maintenance ,if negelected ,could pose a threat in both community and health care causing infections as well as emergence of multi-resistant bacteria that could unpredictable clinical manifestaions.

2.9 Excess garbage management

The amount of garbage entering the drainage network is reduced corresponding to a production of 0.4 to 0.8% of total garbage produced as concluded by Adane, (2019). For developing countries, the rate of garbage accumulation in the streets is certainly higher, since in some parts of the cities the storm- drain network is used for garbage disposal and achieve high sediment and garbage loads, no modern solution to urban drainage is viable without special retention structures upstream or rigorous maintenance procedures with dredging or mechanical removal of the large volumes carried after every storm. This is a peculiar feature of developing countries which makes control works for storm runoff control even more expensive to implement.

2.10 Community participation

Lack of community participation for vital solutions for urban drainage problems is one of the main obstacles preventing the success of modern storm runoff control measures, whether by structural or non-structural measures ,in reseach of Mulualem Bekele, (2018). In most denveloping countries, this has been a problem for sustainable storm water drainage management. Lack of community participation leads to the repetition of earlier errors in solving drainage problems, to the discredit of public action, and lack of concern with environmental questions in the discussion paragraph of the research Asfaw, (2016). It can also bring about low investment in urban facilities.

2.11 Uncontrolled urban settlement

Impermeable surfaces and the construction of drains for rapid storm-water removal are the major causes of urban floods due to traditional urban settlement, pursued without regard for the environment as Tafete, (2013). Such urbanization patterns make it difficult to control urban drainage, since it not only causes or aggravates local flooding but can also create problems downstream. The extent of impermeable cover is directly correlated with runoff coefficients and also with population density, so that an indirect method of evaluating the impact of urbanization on drainage is to relate population density with runoff coefficients. There is evidence world-wide that higher urban population density commonly results in greater storm-water generation, by Abraham, (2018) but many urban planners take no account of this important effect and neglect the wider costs of their storm- water control procedures.

Modern urban drainage calls for detention and infiltration areas, contrary to the philosophy of higher population density. Many cities in developing countries have a density index which already causes critical drainage situations. Besides the problems of control in legal settlements, socioeconomic problems lead to the invasion of public areas, forming slums with high population density and high rates of impermeable soil surface by Hassen, (2016).

2.12 Hydrologic considerations

2.12.1 Urban hydrology and storm water disposal considerations

Urban stormwater hydrology includes the information and procedures for estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems for conveyance of surface runoff and structural stormwater controls for quality and quantity. In the hydrologic analysis of a site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that must be considered include: Rainfall amount and storm distribution ,drainage area size, shape, and orientation , ground cover and soil type , slopes of terrain and stream channel(s) antecedent moisture condition , Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.) , Watershed development potential , Characteristics of the local drainage system identified parameters **by** Design Manua,(2013)

2.12.2 Hydrologic effects of urban areas

Due to urbanization development, dense metworks of ditches and culverts in cities reduce the difference that runoff must travel overland or through subsurface flow paths to reach streams and rivers. Once water enters adrainge network ,it flows faster than either overland or subsurface flow during high precipitation and urbanization generally increase the size and frequency of floods and may expose communities to increasing flood hazards by Konrad,(2016) and concluded that, current stream flow information provides a scientific foundation for flood planning and management in urban areas. Because flood hazard maps based on stream flow data from afew decades ago may no longer be accurate today,flood palin managers need new peak stream flow data to update flood frequency analysiss and flood maps in areas with recent urbanization. Stream flow gaging stations provide a continous record of streams that can be used in the design of new urban infrastructure including roads,bridges,culvert,channels and detention structures.

With less storage capacity for water in urban basines and more rapid runoff,urban streams rise more quickly during storms and have higher peak discharge rates than do rural streams. In addition,the total volume of discharging a flood tends to be larger for urban streams than rural streams (Stephens, 2022). As with any comparison between streams,the difference in stream flow can not be attributed solely to land use, but may also reflect diferences in geology, topography,basin size and shape and storm ppattern.

2.13 Hydrologic modeling for storm drainage

Proper alignment is also particular importance to prevent outlet scour or excessive sediment buildup in the culvert barrels. A culvert placed too low in relation to the channel bottom may lose hydraulic performance if the channel aggrades. In addition, a culvert placed at a slope different from the natural channel slope may have problems related to both sediment deposition and bed scour, and this affects hydraulic performance.

2.13.1 Hydrological modelling in urban areas

Hydrological Modeling used to answer environmental transport questions where water excess, scarcity or dissolved or solid content is of primery importance.Inorder to engage mathematical modelling a class of issues that actually illustrates complex models as a model-eliciting activity can be utilized to deal with the need to develop models by expressing ,examining and refining theire mathematical idea.

According to Chow, (1984) All hydrologic models are approximation of reality ,so the out put of the actual system can never be forcast with certainity ; like wise ,hydrologic phenomena vary in all three space dimentions ,and in time,but th e simultaneous considerations of all five sources of variations (randomness,three space dimentions and time) has been accomplished for only afew idealized cases. Apractical model usually considers only one or two sources of variations.

2.14 Hydraulic model of storm drainage

Hydraulic Modelling Takes the quantity of water and the shape of the land scape and stream Channeland determine how deep and fast water will be and what are it will cover. Hydraulic model is acollabretion of mathematical equations that give simple representative of reality.They estimate :-

- Flow,Water Level and velocity in river channel
- Pipe network
- Tidal system
- Flood plains

After establishing flow boiundery conditions ,you need too apply them to hydraulic model. The bhydraulic model represent path way receptors of fluvial pipe safty.

2.14.1 Hydaulic modeling procedure in urban areas for storm drainage

According to the requirement of the software used, the designer will first assemble and carefully check all the required data on design rainfall, drainage geometry, hydraulic roughness,roughness coefficient and rainfall obstruction parameter. After the preliminary analysis and according to the design objectives, drainage complexity, available data, computer facilities and other futures. The designer should be able to select an appropriate modeling procedure &software to suit the designed purpose. (ICARET, 2016).

2.14.2 Selection criteria of a hydraulic model in urban storm drainage

In this study the hydrological model selection process are under taken in three stages including 1/initial selection based on modeling purpose 2/selection of the model based on inter comparison and 3/ final selection of the model based on influencing criteria. (Diba Ghonchepour, Amir

Sadoddin, Abdolreza Bahremand, Jackeman, Abdolrasquel Salman-Mahiny; August 2021; AFHMSP)

2.14.3 Performance of hydrologic and hydraulic modeling in urban storm drainage

Some kind of storm water model is needed whenever an estimate of the expected volume, rate or growth of storm is desired. Modeling is also often necessary for the proper design of storm water. If monitoring data exists for the specific combination of precipitation and site condition under consideration, modeling may not be necessary. However, in many cases the condition to be analyzed does not fit precisely with monitoring condition and modeling will be necessary.

2.14.4 Hydraulic performance evaluation of storm drainage system analysis using HEC-RAS

HEC-RAS is hydraulic modeling software developed by the U.S. Army Corps of Engineer´s Hydrologic Engineering Center. In this study, version 4.1.0 of HEC-RAS was used. The software is capable of performing one-dimensional (1-D) steady and unsteady-flow simulations and comprises a graphical user interface, separate hydraulic analysis components, data storage and management capabilities as well as graphics and reporting facilities. In inundation analysis, flow modeling is used to simulate the flow of a flood wave through a river reach and its floodplains. In one-dimensional flow routing, flow through the river channel and the floodplains is treated only in the longitudinal direction parallel to the conduit. Even though in reality, the flow in a natural channel is never truly 1-D, these flow models were found to deliver acceptable results for predicted hydraulic parameters in many applications. In the 1-D HEC-RAS flow model, the geometry of the channel and the floodplains is represented by a series of cross sections along the reach.

2.14.5 Hydraulic Performance evalaution of urban storm draianges using HEC-RAS 1D and 2D model

One dimentional model and two dimentional model in HEC-RAS can be used to simulate floods in reverien system. However each modeling approach has its own advantage and disadvantage.The main objective of 1D model is to compute water surface elevation at location of interest for a given boundery conditions.The flow underlying 1D can be steady and unsteady state conditions.Main input to the model include flow data,manning n value,crossectional geometry and reach length. The 1D model just computes,how deep that water is going to get.It doesn't determine direction. 2D modeling can be can be very usfulto actually determine direction the flow going to go.HEC-RAS has a GIS interfaceand applies the finish volume method to solve unsteady flow equetionsfor the two dimention as presented and published by Brunner, (2016).

HEC-RAS model usually works upstream from down boundery(for sub critical flow,which is the nost common in natural channekl),and this down stream level needs to be carefully assessed. The main parameter used for HEC-RAS analysis is manning's n amount of channel roughness.

The discharge is a function of the return period, runoff coefficient, drainage density, longest path, rainfall intensity and catchment area. (Abd-Elhamid et al., 2020) In general, climate, topography, geology, scientific knowledge, engineering and construction capabilities, societal values, religious beliefs, and other factors have influenced the local perspective of urban drainage. The climatic factor and the soil-vegetation complex are variables that exercise their principal influence on the volume of runoff. The topography of drainage basins is a sensibly permanent characteristic which influences mainly the concentration or time distribution of the discharge from a drainage basin (LANGBEIN, 1941). For proper functioning of adrainege system, the above constraints and inputs are evaluated properly for suitable adrainege structure. Conventional drainage systems designs are in appropriate because they faile to take the potential for flooding in to account (Parkinson, 2003). Historical accounts provide sights of many interesting and unique urban drainage techniques.Therefor,its possible to take account consideration of input data, environmental conditions, geolojical and sociological formatons and development in to account.

2.14.6 Hydraulic Performance ecvalaution of urban storm draianges using HEC-HMS Model

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems -HEC ‑HMS is a product of the Hydrological Engineering center within the U.S Army Corps of Engineering. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. The software features a completely integrated work environment including a database, data entry utilities, computation engine, and results .

2.14.7 Performance evalauation of **urban** hydrologic model using Rational method

The rational method may be considered as a conceptual model employed for rainfall-runoff modeling. A conceptual model attempts to simulate the behaviour of the catchment action on the rainfall to convert it to runoff in a simplified form by lumping the distributed action of a real watershed. While in the real runoff process the runoff is generated in a distributed manner by of simultaneous action of translation and attenuation on any small unit of input, the same actions are executed by the conceptual model by two separate actions of translation and attenuation in alumped manner.

Therefore, to evaluate the performance of the rational method either the actual hydrologic data over a number of small urban watersheds for various number of storm events should be available; alternatively, one may generate such data set using hypothetical approach of runoff generation based on a physically based mathematical model applied on a number of hypothetical watersheds of varying sizes suitable for the application of the rational method.

2.14.8 Performance evaluation of urban hydrological model using SCS method SCS-CN method of estimating runoff

SCS-CN method developed by Soil Conservation Services (SCS) of USA in 1969 is a simple, predictable and stable conceptual method for estimation of direct runoff depth based on storm rainfall depth. It is a versatile and widely used procedure for runoff estimation. The SCS approach involves the use of simple empirical formulas and readily available tables and curves, developed by the Soil Conservation Service SCS, (1985). The curve numbers are varied for different soil, land use/land cover and hydrologic conditions. Although the method is designed for a single rainfall event, it can be scaled to find average runoff as published by Amstutz et al, (2008).

2.15 Performance indicators for storm water drainage systems

There are afew works specifically developed for performance assessment of storm water system. performance metrics that were developed by different authors and organizations,whose focus was to assess storm water system performance.They organized urban storm water collection and conveyance performance in four groups of indicators and other metrics applicable such as; conventional system(combined and separate channel system),sustainable urban drainage system (SUD), both cpnventional system snd SUD and waste water system. Given in the wide range of application focused on performance metrics,namly performance indicator,that are only related to system performance (hydrologic.environmental.ecological,social, and economic catagories). By Liliana Ferreira Santos, (2019).

2.16 Performance indicator for storm water conveyace challenges

The identified challenges can be grouped as planning, design and construction, monitoring and evaluation, collaboration and regulatory challenges, as discussed below.

I Planning challenges

As Welikite has no city-wide stormwater network master plan, the process of drains planning is not led by draingae master plan. Consequently, drains planning is based on traditional and fragmented approaches. The option of integrating other sustainable stormwater management systems (e.g. rainwater harvesting, retention and detention based solutions) is no inccorporeted and abscent. In addition, most industries, institutions and residences prefer to increase impermeable surfaces with out any guiding rule and reguletions. Permeable areas are not identified and no institution is accountable to for hinderance and guidance of fresh constructions.Following longer roads most drains are installed from upstream (initial point of a road) to downstream (final point of a road) without distributing into nearby receiving system which would reduce the volume of stormwater travelling downstream. Moreover,Welikite has not execute integrated planning approaches from the context of stormwater management. For example, integrating stormwater management with urban land-use planning,since landscape plans is absent at any level. The components of urban water (water supply, waste water & stormwater) managed separately by separate institutions.

II Design and construction challenges

Based on field survey, the design of drains in Welikite is carried out through segmental or fragmented approaches resulting in flash flooding. It was found that drains are usually designed without hydraulic and hydrologic analysis. Moreover, the new constructed ditch are destroyed with out expected service delivery,they are filled by silt and dry waste,workmanship problem is an outstanding issue. .

III Monitoring and evaluation challenges

Drains are commonly provided by welikite town municipality, but regarding monitoring and evaluation the town has weak evaluation system. This shows that the town focuses only on provision than on the management of the provided drains and associated facilities. Moreover, no monitoring and evaluation on the hydraulic performance and need of stormwater management facilities.

2.18 Performance indicator assessment of storm water drainage

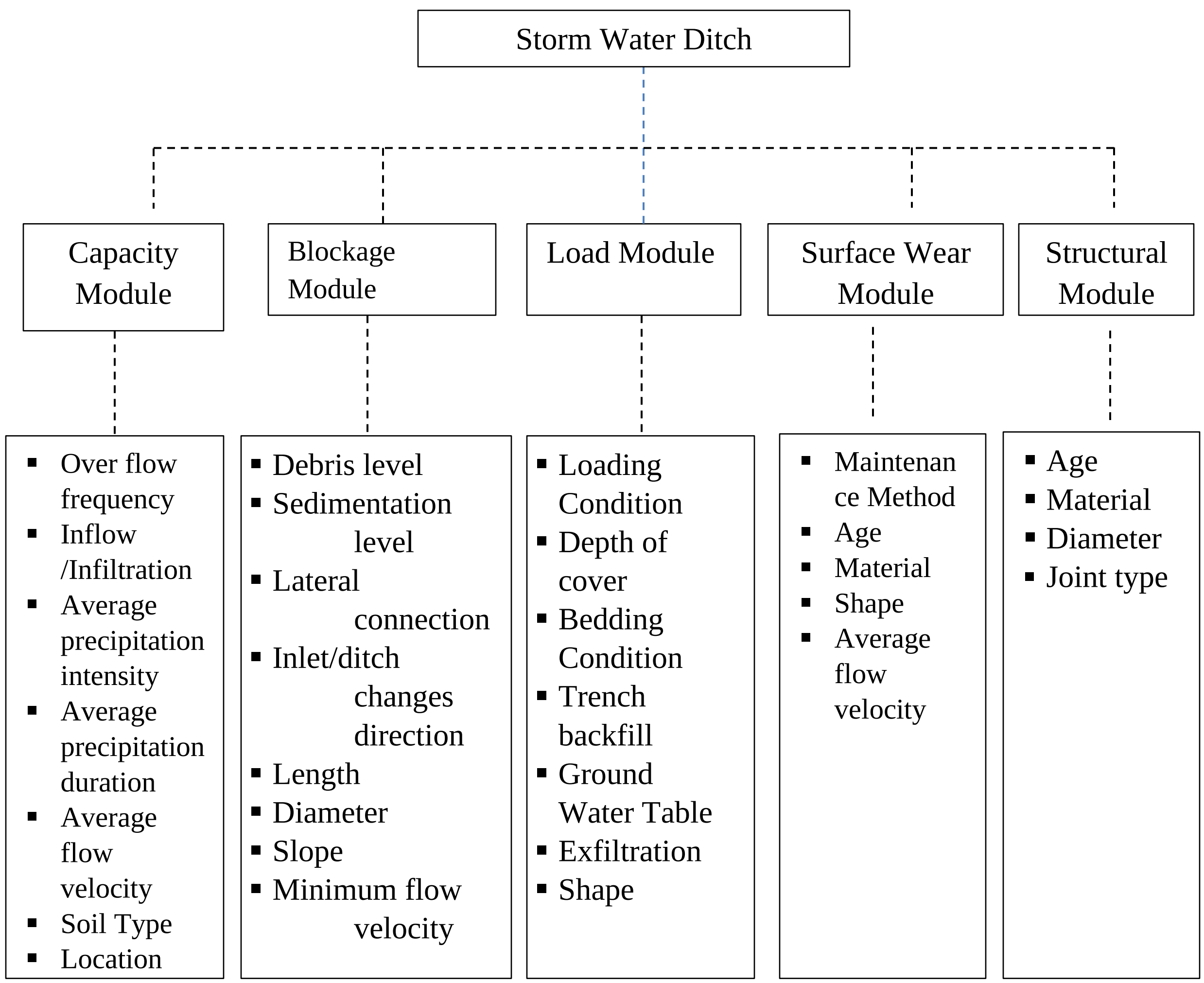
The need to have an adequately performing urban drainage network would seem to be an obvious requirement, as would be its link with asset maintenance and rehabilitation.The performance objectives are relatively straight forward and have already been highlighted in this research to efficiently carry away stormwater from the total area map of the town has taken the sub catchment area. Of course, consistently achieving these objectives is less forthright, as is measuring and demonstrating that it has been done.

The most important factor in designing stormwater drainage system is the physical storage volume that need to be achieve flood control and minimize the pollution impact of urban storm water runoff (Ydnekachew, 2019).and examine performance of current condition of the drainage system by using ERA standards based on the field survey and model simulation and observed that some the drainage lines have lower capacity and canot safly discharged through the system.

During assessment of conveyance system in urban drainage system ,there are typical indicators like, availability and abscence of natural river in the area, healthiness of conveyance systems i.e damping of suspended and clogging wastes in side the drainage ditches , damaged drainage structures,lack of adequate clearance and availabilityof best ngt practices (Ydnekachew, 2019).

2.17 Performance evaluation of indices and indicators for convetional system

There are performance indexs developed that affect separate storm water channels;to repair,rehabilitate and replacement decistions. The index was obtained by applying the weighted factor method and consists evaluating all relevant attributes of the problem and allocating ascore to each attirbute based on rating scheme.For this,the attributes were the potential failer module that may affect the performance of astorm water channel ,i.e capacity,blockage, load surface wear and structure (Liliana Ferreira Santos,Ana Fonseca Galvão and Maria Adriana Cardoso, 2019) and it summarized the module as under that evaluate the performance of the channel.



Storm Water Ditch
Capacity Module
Blockage Module
Load Module
Surface Wear Module
Structural Module
Over flow frequency
Inflow /Infiltration
Average precipitation intensity
Average precipitation duration
Average flow velocity
Soil Type
Location
Debris level
Sedimentation level
Lateral connection
Inlet/ditch changes direction
Length
Diameter
Slope
Minimum flow velocity
Loading Condition
Depth of cover
Bedding Condition
Trench backfill
Ground Water Table
Exfiltration
Shape
Maintenance Method
Age
Material
Shape
Average flow velocity
Age
Material
Diameter
Joint type

Figure 1:Performance Indicator (Liliana Ferreira Santos, 2019)

The proposed index focused on several important aspects of pipe performance,namely operational and structural performance,that are less considered in most works.The effect of climate change on convetional storm water system is on relevant motivation to assess their performance assessment.the indicated were classified interms of description of system performance ,i.e description of consquences because of capacity exceeding.The authors concluded that in separet system performance indicators identified and marked as i.e

Table 1:Performance Assessment Indicators (Jefferies, 2004)

Performance Domains	**Performance indicators**
Hydraulic performance (at site and catchment)	Flow attunetion of the outlet
	Volume reduction at the outlet
	Lag time
	Overflow frequency
	Drainage frequency
Hydraulogic performance(at site scale)	Reduction in mean annual runnoff volume back to natural volume
	Runoff frequency
	Similarity b/n the pre-developed volume of base flow and the volume of storm flow released
Economic aspect	Preliminary costs
	Construction cost
	Operational cost
	Saving /return on investment
Life span and long term efficiency	Long term functionality

Social aspects	Number of complaints
	Kind of odour smelled
	Percentage of satisfaction

Table 2:Quantitative Performance Indicators For Storm Water Measures (Matzinger , 2014)

Effect category	Performance indicators
Building physics and services	Rainwater recycling and reuse for service water [%] Energy saving from cooling (or heating) [kWh m—2]
Landscape quality	Variance in plant height [m]
	Dry plant volume [kg m—2]
	Proportion of open water surfaces [%]
	Scale from 0 to 3: the higher, the better
Urban climate	Evaporation rate [%]
	Green volume per connected impervious area [m3 m—2]
	Surface albedo [-]
Biodiversity	α- and β-diversity (floristic and faunistic) [number of species]
	Structural diversity of vegetation [number of different elements]

	Average distance between measures [m] and species dispersal among measures[%]
	Proportion of rare species [%]
Groundwater	Change in groundwater recharge rate [mm]
	Change in electric conductivity, chloride, sulphate, zinc and biocide concentrations [%]
Surface water	Reduction of one-year peak runoff rate [%] (compared to situation without the measure)
	Reduction in P and N loading [%] (depending on total annual flow reduction and/or cleaning effect)
	Reduction of 30-year peak runoff rate [%] (compared to situation without the measure)
Direct costs	Present value of annual costs per connected impervious area [€ year—1 m—2]
Resource use	Cumulative energy demand of non-renewable fuels [MJm—2]
	Abiotic resource depletion of minerals [kg Fe-eq m—2]
	Abiotic resource depletion of minerals [kg Fe-eq m—2]

3 MATERIAL AND METHODS

3.1 Study area description

3.1.1 Location

Wolkite town is found in the Guraghe zone of Sothern Nations, Nationalities and Peoples Regional State (SNNPR's). The city is a Zonal administrative capital as well as an important business center for the region. The town is located at a distance of 153 km south of Addis Ababa, the capital city of Ethiopia. The geographic coordinates of the town are; approximately 8º16’20’ to 8º18'30" N Latitude and 37º45'30" to 37º49'0" E Longitude.

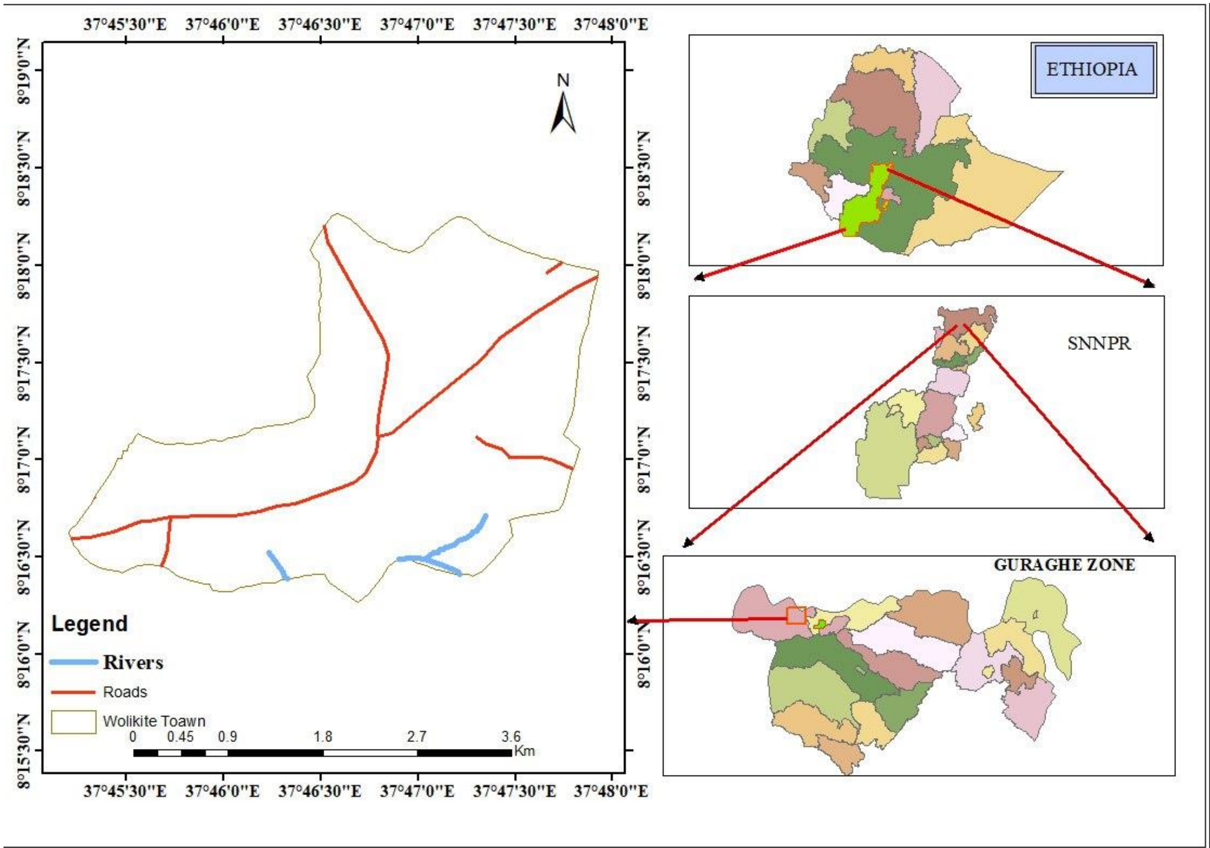


Figure 2:Location Map The Study(Shape Map From MUD and Arc GIS)

The town covers an area of approximately 10,297 ha(welikite town municipality,2010) and the altitude ranges from 1910 m to 1935 m above sea level. In different literatures the area of the city is reported differently. The town has three sub cities and Bekur Kifle Ketema is the oldest sub city and it has dense livelihood condition and poor drainage capability.

Inside the drainage ditches in the town, plastic material are found , remain of chat leaf, rubbish and rugged materials, due to tremendous shops on road side of the town,waste of industrial baggage material thrown and others dropped inside the open ditch which is one of the constraints for easily flow of storm. Lack of close follow up for constructed ditches storm drainage condition are another observed issue. During inspection of the drainage ditches ,they were Broken, depressed, shoved and found as silted drainages ditches in the targeted study area.

3.1.2 Urban Settlement

Unmanageable population system is another outstanding issue for healthy infrastructure operation. To provide capacity building trainings in each keble how they operate and give care and ownership. Majority of the road pavements are red ash and compacted earth which lacks the proper drainage system. As a result, flooding is the main causes of structural damage to the roads during rainy season and dust is a problem in the dry season. The road pavements are deteriorating at a faster rate than they can be maintained. It is important to note that this problem will increase further as the increase in population and population density, the increase in paved areas, create an increase in storm water runoff. Drainages in the majority of town discharge into gully which are eroded badly.

3.1.3 Topography and drainage pattern of the area

Welkite Town located south east of the Ethiopia rift valley, characterized by platue feature from west to east. Welkite extends for a considerable area over the rolling hills in the western section and flat areas towards south to west. The altitude of the city ranges from 1908-1975 meter a.s.l.

The slope of the watershed is one of the driving parameter on runoff generation. According to the DEM based terrain analysis results, the slope gradient of the majority of the watershed ranges from 1^0 to $23^{\,0}$ in degree rise. (Fig-6)\

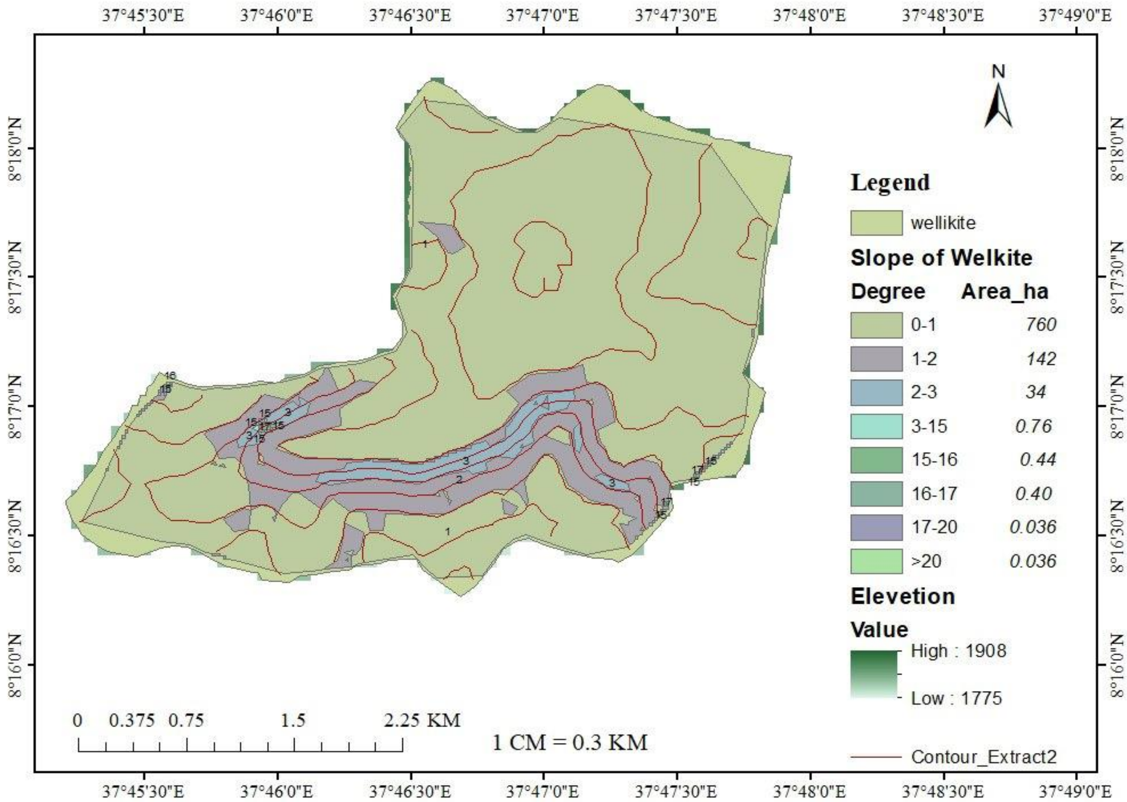


Figure 3:Slope Map Of Welikite Town(shape map from ARD using Arc GIS)

Slope analysis contributes for the selection of preferable landscape for urban expansion and determining different urban uses in the urban centers.

The slope of the town is analyzed using xyz coordinates through GIS tools. In the town plain slope which is less than 2^0 occupies aboute an area of 850ha of the total area (Tabel-2). These are floodable areas and require appropriate drainage channel structure to convey the storm in anear bye river. so they require extra fascility land manegmnet for infrastructure and for different constructions development and also exposed for storing stagnate water and that may negatively affect the health and safe movement of the town dwellers. There is no excessive steep slope that hinders the development of urban infrastructures like road network in the town.

Table 3:Slope Classification And Area Covered(Arc GIS out put result)

Slope classification in degree of the town	**Area cover(ha)**
0 -1^0	760
1^0-2^0	142
2^0-3^0	34
3^0-15^0	0.76
15^{0-}16^0	0.44
16^0-17^0	0.4
17^0-20^0	0.036
>**20^0**	0.036

3.1.4 Population size of the town

Based on the 2007 census conducted by central statistics Agency, this town has a total population of 28,866 of whom 15,074 men and 13,793 women. From this the current projected till 2014, the total population of the own reaches to 38,750 with a yearly growth rate of 2.2%.

3.1.5 Land Use Land Cover

The land use land cover data will be an essential input for the calculation of run off coefficient for the determination runoff by HEC RAS model and after determining Curve Number by the category of the town which it fall. LU-LC map is produced using 10 X 10 m resolution sanital-2 2021 raster image and the town has atoltal area of aboute 1062ha.

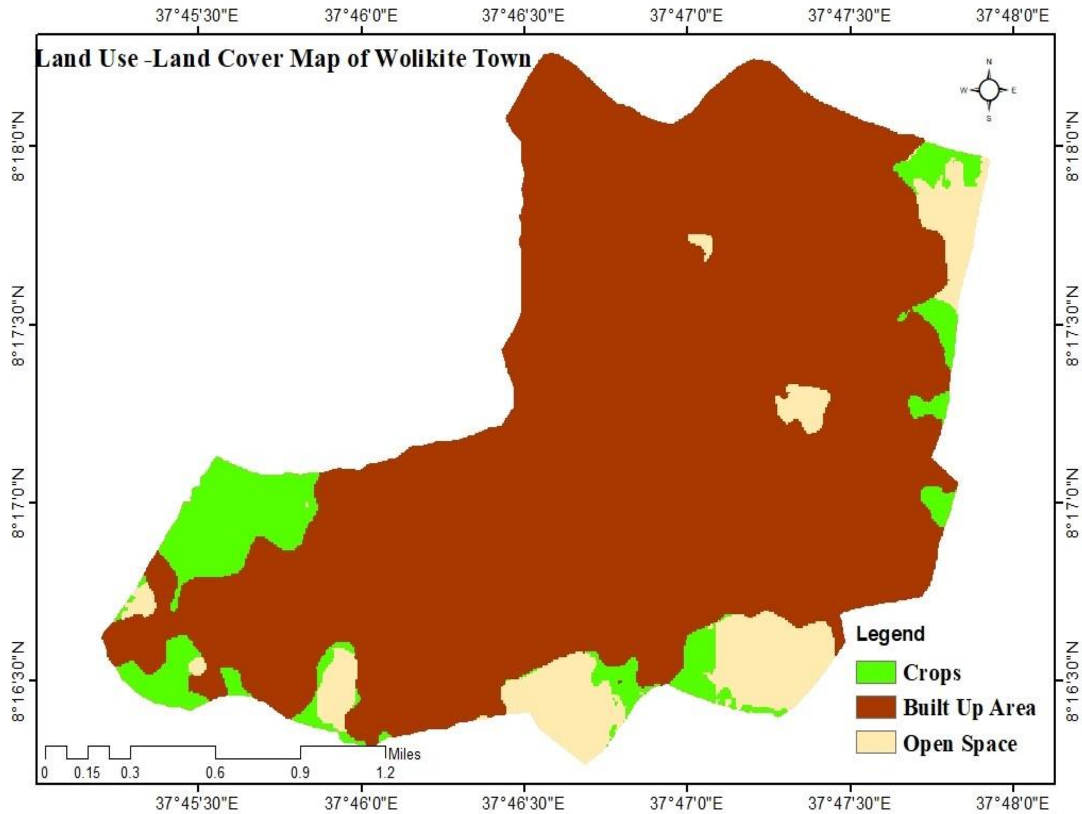


Figure 4:Land Use Land Map Of Welikite Town (ESRI lulc,WTM and Arc GIS)

The natural vegetation surrounding the town is depleted and causes land degradation. Most of the study area was covered with Residential area, annual and perennial crops, shrub land, dense and sparse forests, grass cover land in some compounds of individuals, religious and government institutions (Fig-4).

Table 4:Classification By Area Land Use - Land Cover Map of The Town(Calculeted result from arc Map)

Classification	Area _ sqkm	,Area_percentage
Crop cover area	0.8	7.76
Built Up Raea	8.68	84.24
Open Space	0.82	8

	=10.62	=100%
	=1062 ha	

A. **Crops Cover Area-** Human planted cereals ,grasses,and crops not at atreee height ,examples corn,wheat,soy,fallow plots of structural land

B. **Buit up area-**Human made structure.major road,larg,etce homogenous surfaces includingparking structure,office building and residential housing, for example,house,dense village, cities coble stone road, main asphalt road

3.1.6 Soil type

The development of soils depends primarily on geologic and climatic conditions. The FAO Soil Map of Ethiopia classifies 19 soil units. According to FAO‟s soil classification, about 1 major types of soils are found,which is Pellic Vertisols . Pellic Vertisols have Hydrologic Soil group of D which are very much affected by Runoff and are washed during high storm.

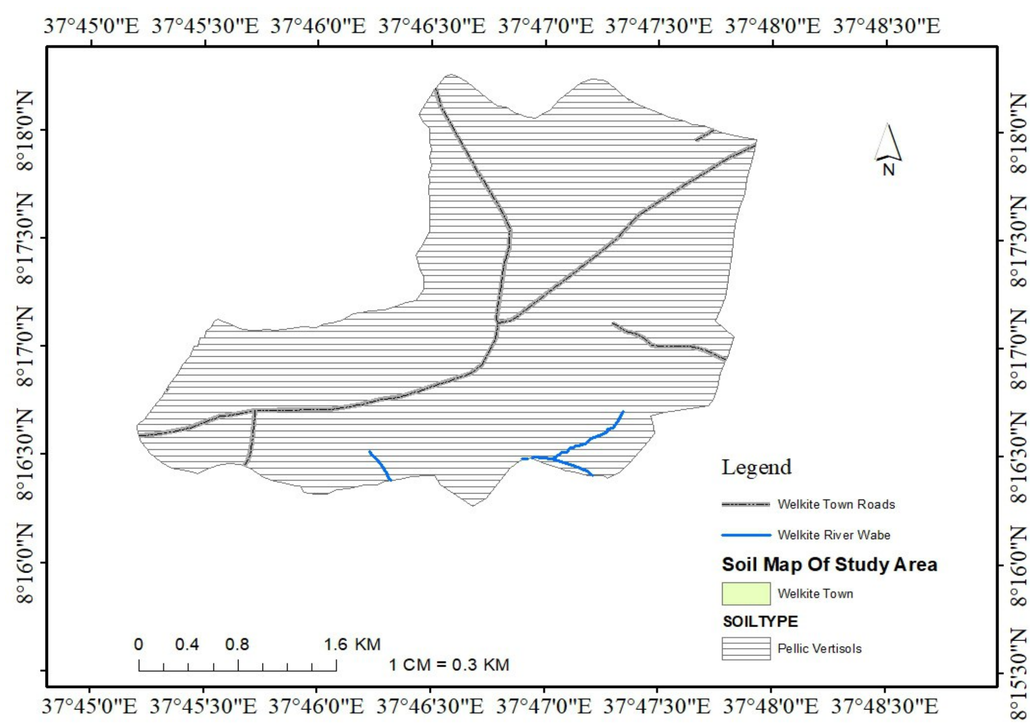


Figure 5:Soil Map Welikite Town(ARD & using Arc GIS)

The above figure shows that , in welikite town pellic veristol soils are found in abouindant through out the town aand this soil are clayey soils that shiornk and swell extensively upon changing soil moisture conditions. They occur globally under various parent material and environmental conditions (Element B.colons, 1996).The shirinking and swelling of versitols can cause extensive subsidence in building and roads. Veristols (often known as ‘ black cotton soil’ 10% of the estiopian land mass coveribfc about 13million hectyars ,of which 8 miliion in central hignoands.This shows that the life cycle of drainage structure,any other infrastructure due to the characterstics of the soil nature.some typical cracks and shrinkage character of soil has shown as under

Due to the soil nature, the town drainege structures are settle and cracked in the picture shown as under and the storm and the sewers liquid disposals drain under the foundation of the drage structures

Figure 6: Damaged drainage structure due to settlement of soil and constructed drainage structure at bottom side of residential, tenatabiya sefer(site visiting time)

Foundetion of any structure disturbed by soil type and it is one measurement of the performance of servvisability to keep the design periode safe. Pellic Veristols (often known as ‘black cotton soils) 10% of Ethiopian landmass covering aboute 13 million hectars of which 8 million in central highlands.They accounts for aboute70% of all high lands soil with slope 0-8%.The high clay content of the veristols is responsible for their heavy water logging in highland areas with abundant rainfall and relatively low exoansion rates.

3.1.7 Socio-economic activities

In Welkite Town there are large and diversified modern investments activities such as hotels and tourism, social services, real estate development, small and large industries including manufacturing. There is one university with additional independent faculties, one teachers training institute and others.Since the town is high traffic area due to center of passengers rest on their travel of the southern part of the country initially from addis ababa.

3.1.8 Natural and artificial drainage networks of the town

In welikite naturally produced drainage due to erosion and others are man made which plan to built to convey storm water during rainy season. However ,most drainage were built in the town through municipality by associations to encourage un employment, and this drainage built aimed to solve current problem of the youths but already miss the target which needs to built. Due to incapability and other situation storm water in the area not collected easily due defect of construction guided by knowledge and most drainage channels looks ppor construction capability. Welkite town and its vicinity is part of the areas which are found in Rift valley. There are three sub cities named as Bekur, Gubre and Addis Sub Cities

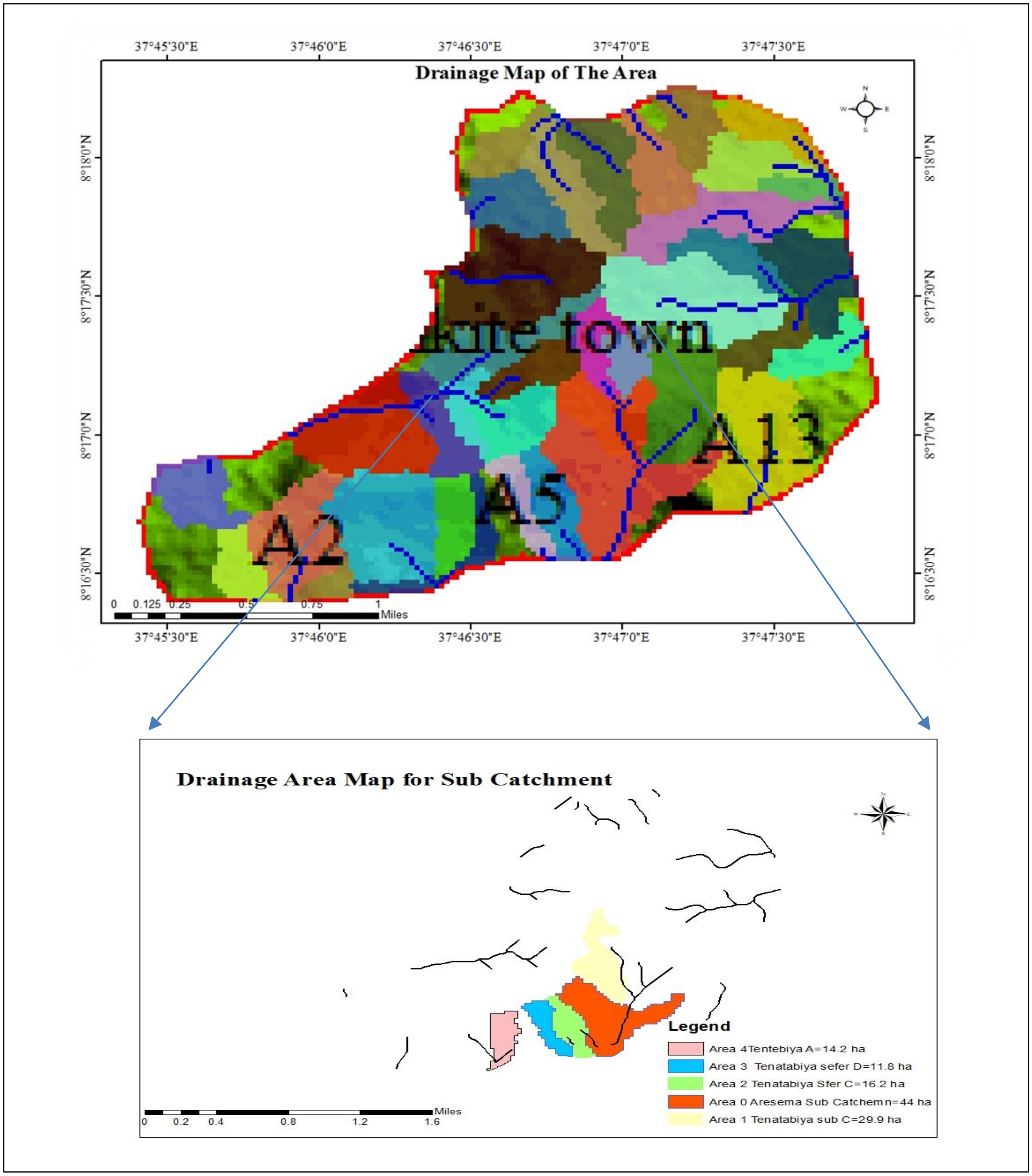


Figure 7:Natural drainage of sub basine and drainage line of the study area(ARC GIS from CER,Urban Dev't)

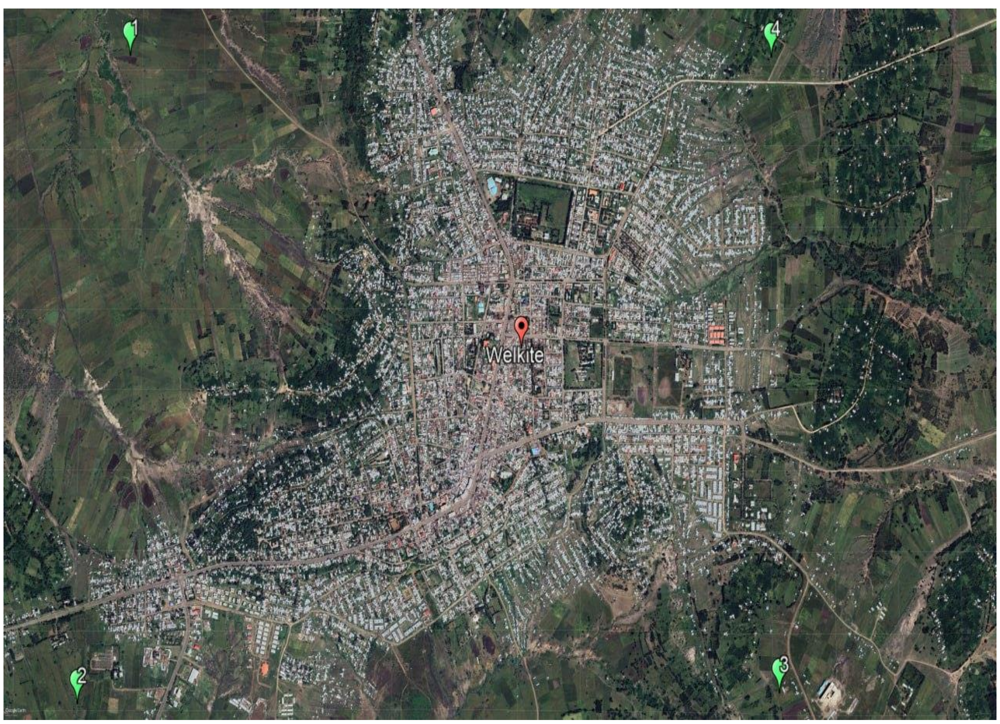


Figure 8 Artificial Drainege Of Study Area(G. Earh,W .T municipality)

Figure 9 **:Solid Wastes Inside Drainage Systems(site visiting time)**

: **Figure 10 :Placement ,Mis Use, poor alignment of drainage structurs(site visiting time)**

3.2 Data collection

3.2 Climate

3.2.1 Monthly Rainfall

Based on Welkite meteorological station, the long term annual rainfall amount is computed to be 1324 mm/year over the period of 1991-2021. The maximum annual rainfall observed in a year 1992 with an amount of 2331 mm and the minimum value observed in a year 2004 with a value of 1026 mm. The annual maximum daily rainfall value of 83.7 mm depicted in a year 1995 and the minimum annual maximum value of 38.5 mm observed in a year 2007 over the 31 years (1991-2021).

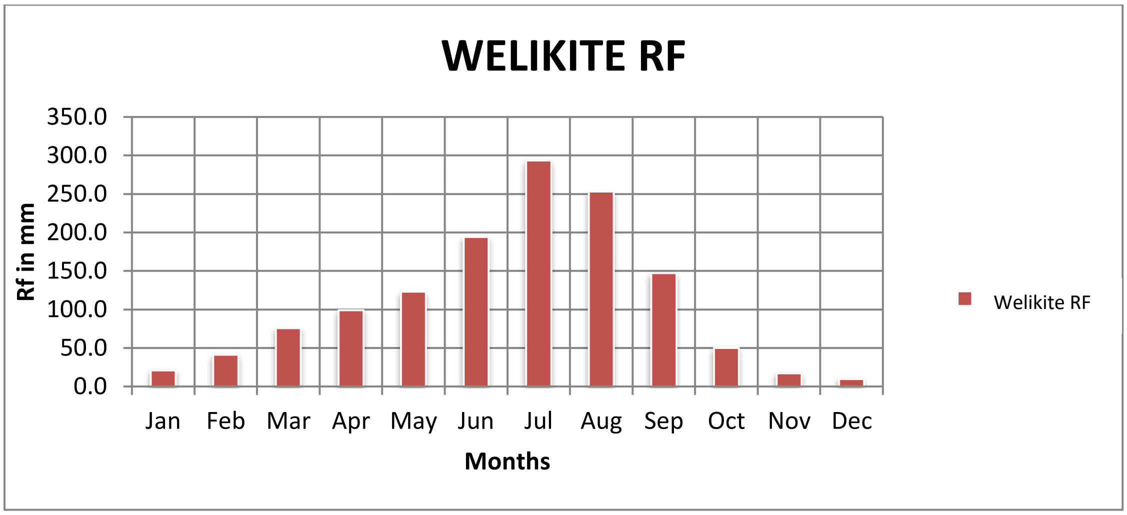


Figure 11:Bar graph of ave.rainfall in each months of the year

3.2.2 Temperature

These elements are very useful meteorological data for estimating the water balance of a given river basin. They are also very important for estimating the Evapo Transpiration of the area. The raw data elements are presented in the following table. From the figure we can see that Minimum temperature is 23℃ and Maximum temperature is 29.3 ℃.

3.3 Primary data collection

Primary data were collected at on site measurement storm water flow in the ditches, total ditches in meter with full of sediment, suspended material and other debris and those ditches free from problems, interviewing the service situation of drainage ditches in the compound, representative public meeting fallen suspended material, discussion with the town political leaders about drainage design and construction, and other method decided at on site situation at field work.

To find out the solution ,the hydrologic modeling HEC RAS incorporated with Log Pearson type III method was used to calculate the discharge and the hydraulic analysis to size the conveyance and the inlet used manning and different empirical formulas to avoid excess runoff along the road side and transportation problem and identified inlet sizing and conveyance a big problem and recommend that correcting after updating the ERA manual by hourly rainfall intensity, possible to increase the conveyance diameter and adjust the inlet types and size moreover considering the sub catchment rainfall influences for the discharge are substantiated as main points.

So, generally these are the data needed

- Site Observation the overall of drainage system condition and booked camera.
- Measured the drainage dimensions using GPS and TAPE meter for the input of HEC RAS model simulation.
- Measured flow depth in drainage system that used for comparing the result

Figure 12:Primery data collection during the study(**site visite)**

3.4 Secondary Data Collection

3.4.1 Climate data collection

A. Meteorological Data

From the National Meteorological Services Agency collected climate data of rainfall in daily manner ,from different stations of Welkite, Werabe, Bui and Emdibir Observatory gauged station.

3.4.2 Topography and shape file

From Ministry of water, Irrigation and energy,welikite Town municipality ,ministry of Urban Development and Agricultura and Rural Development Bureau used shape file all Central Ethopia incorporetd with ESRI Soil and landuse landcover maps.global mapper and google earth for slope calculations, finally using atool Arc GIS to locate the exact location of the study area .

3.4.3 Data Source

Table 5:Institutions Of Secondery Data Colleection(CER,WMEDB)

Data Type	Data Source
Rainfall and Temrature,	Ethiopian Metrological Agency,Hawassa Branch
Shape Files	MoUD,WTM,ARDB
Soil Map	Global Soil Data Center ,USGS,ESRI 2021soil map,using ArcGIS
LULC Map	Global Land Use Map,ESRI 2021 lans use land cover map and using atool ArcGIS
Drainge Map	Google Earth,Google Map,ArcGIS
Existing Drianage Status Report	Gurage Zone,Wolikite Adminstretion
Surving Data	Existing drainage using Total Station,GPS,Tapmete
Master Plan Of The Towm	From Wolikite Town Municipality

3.4 Material used

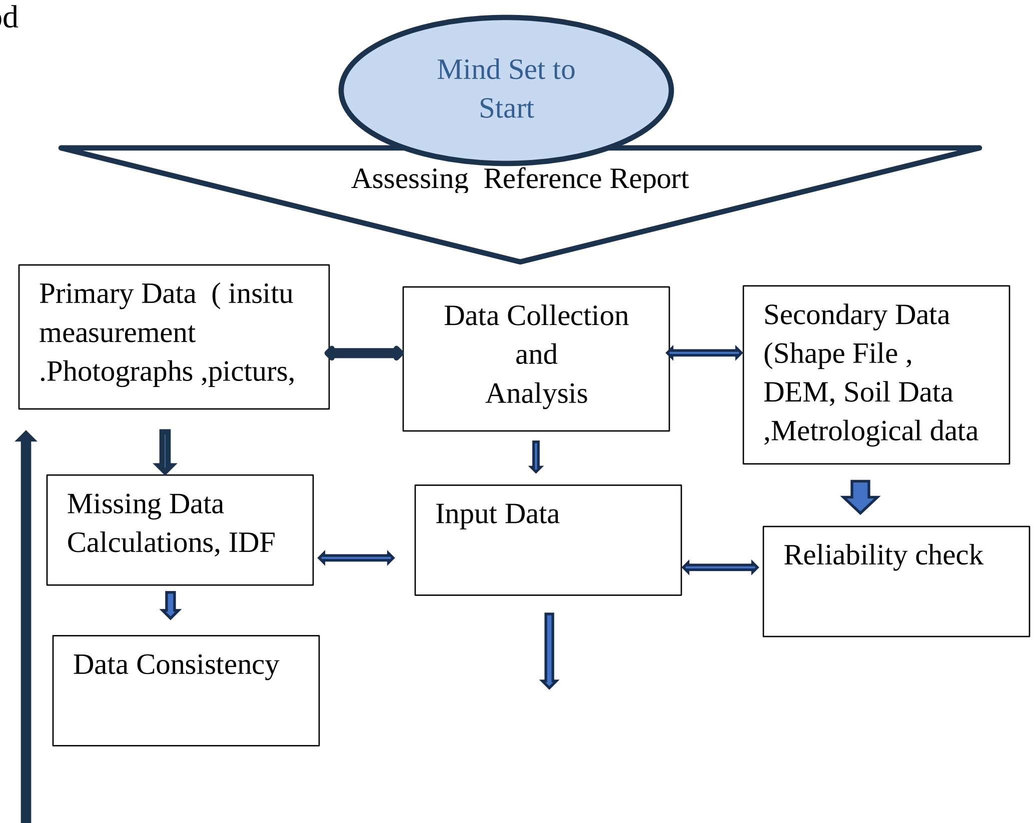
All rainfall data recorded daily intervals were, collected from National meteorological Agency.

The data recorded period was for 31 year (1991 – 2021 G.C). Generally the data collection and material used for each results are as follows:

- Shape file data is used as an input for ARC-GIS software for catchment delineation and estimation of catchment characteristic
- ARC-GIS to obtain hydrological and physical parameters and spatial information of the catchments of the study area.
- Google Earth 2019 Software to verify water shed and divides of catchments of the study area.
- Hydrological and hydraulic data where used as input for HEC RAS tool
- GPS and TAPE meter to measure elevation of nodes and drainage demission that input for HEC RAS tool.
- HEC RAS model to determine the peak runoff
- All the primary and secondary data organized used the study

Referring different journals/thesis, books, design documents and manuals used as guideline

3.5 Method



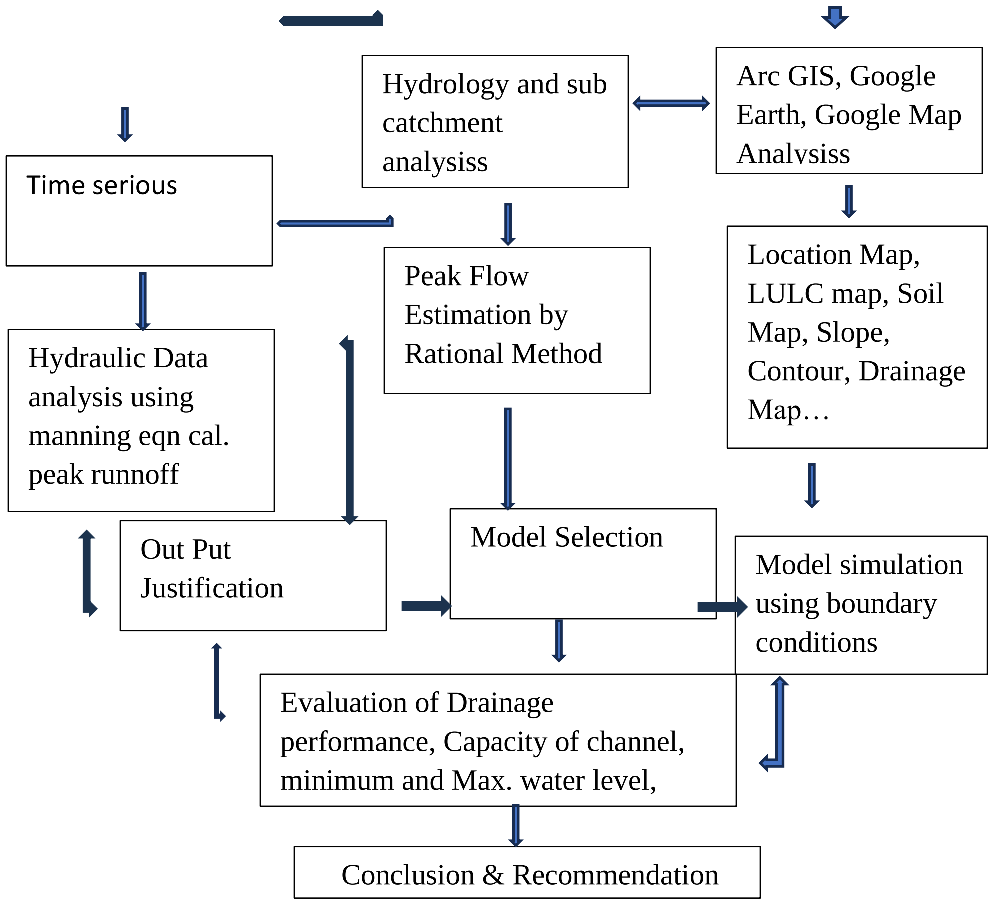


Figure 13: Methodology of the research

3.6 Data Analysis

The existing drainage of the town is constructed by stone masonry walls along the primary and secondary roads of some parts of the town. Most of the constructed drainage ditches are filled by unnecessary wastes, plastics and during high rain runoff occurrence floods directly inter in the residence of peoples compound and disrupt and damage resources and hinders normal movement of the area. The infra-structure construction situation of the town very weak and unplanned due to management and such study reports for the concerned bodies. The figure below shows Welkite town study aerial photo map.

Figure 14:Sample of uncovered drainage ditches in the study area(site visiting time)

3.7 Rainfall Data Analysis

Climatic variables are important for the design and operation of different projects. The most important variables include; Precipitation over the catchment and evapotranspiration estimates for the design of the water balance. Moreover, precipitation and potential evaporation (PE) (or climatic variables to estimate PE, temperature, wind speed, humidity and so on) of the catchment are required.

Welkite has one class-A met station located at Welkite which is found in the town boundary. But this station represents only the higher portion of the catchment. However, there are four more meteorological stations with long daily precipitation data records in the vicinity of Welkite. The precipitation data in these stations has many gaps and they are not of the same length. Moreover, some stations are not spatially consistent with the surrounding stations. Therefore, data have undergone the routine missing daily data filling, stationary and randomness check before using for variable estimation

3.8 Missing Data filling, Data quality assessment and adjustments

All data were checked for reliability and quality through data consistency and stationary tests. Since design of hydraulic structures normally assume and require the stationarity of hydrologic design parameters, meteorological variables also need to be stationary. For this purpose, a hydrological data analysis software tool called HYDROGNOMON was used. HYDROGNOMON is capable of carrying out spatial homogeneity test that is Double mass curve analysis.

Rainfall data records occasionally are incomplete due to different reasons like the absence of observer, instrumental failure and social disorders. In such cases, one can estimate the missing data by using the nearest stations rainfall data. There are a number of methods to fill the gaps of the records.The following four methods are suggested for different conditions. Optimal distribution

Table 6:Mathimatical Approaches Of Missing Data Calculetions (Irrigation & Drainage Handbook)

It No	Name	Formula	Condition of Applicability
1	Arithmetic mean	$Px = \frac{1}{M}[P1 + P2 + \cdots + Pm]$	For the coefficient of variation among stations less than 10%
2	Normal Ratio	$Px = \frac{N}{M}\left[\frac{P1}{N1} + \frac{P2}{N2} + \frac{P3}{N3} + \cdots \frac{Pm}{Nm}\right]$	If the amount of variation has no significance if the stations are in the same rainfall regimes
3	Inverse distance weightage	$Px = \sum_{i=1}^{n} Wipi$ $Wi = \frac{1/di2}{\sum_{i=1}^{n} 1/di2}$	If the amount of variation has no significance and the stations are in short distance
4	Regression equation	Locally developed	If there are significant correlation

Since the amount of variation has no significance and also the stations are in the same rainfall regimes we have used Normal ratio Method to fill the missing data's.

3.9 Tests for Consistency

Rainfall data a station may not be consistent over the period of observation. Inconsistency may occur due to due to different reasons: shifting of the rain gauge to new location; changes in the ecosystem due to calamites, such as forest fires, landslides…etc.; significant construction works that may change the surrounding or occurrence of observational error of a certain data. Therefore, testing and adjusting the inconsistency have to be made in the rainfall data processing, which is often carried out with double mass curve technique. The double mass curve technique is a graphical approach where the accumulated rainfall at the doubtful gauge will be plotted as ordinate versus the concurrent accumulated mean rainfall of nearby. Therefore we have used HYDROGNOMON software to test the consistency of the data.

3.10 Outlier Test

Check on outliers has been undertaken on the recoded rainfall and flow data to identify any low or high outliers. Outliers are data points, which depart significantly from the trend of the remaining data. The retention, modification, deletion of these outliers can significantly affect the statistical parameters computed from the data, especially for small samples. All procedures for treating outliers ultimately require judgment involving both mathematical and hydrologic considerations. The procedure followed for detection and treatment of high and low outliers for this project are summarized in the next section based on statistical technique described here under:

For this study we have used two methods to check the quality of data's, Outlier Test, Higher outlier

$Y_H = \bar{Y} + KN*S_y$ KN values are obtained from tables

Maximum rainfall X = 10 YH

Lower outlier

$Y_L = \bar{Y} + KN*S_y$ KN values are obtained from tables

Minimum rainfall X = 10 Yl

Where $\bar{Y}$ is mean

Sy is standard deviation

From Welkite Station for the data from 1991-2021 (Appendix-3)the outlier test result shown below, which indicate the data is ok or there are no outliers

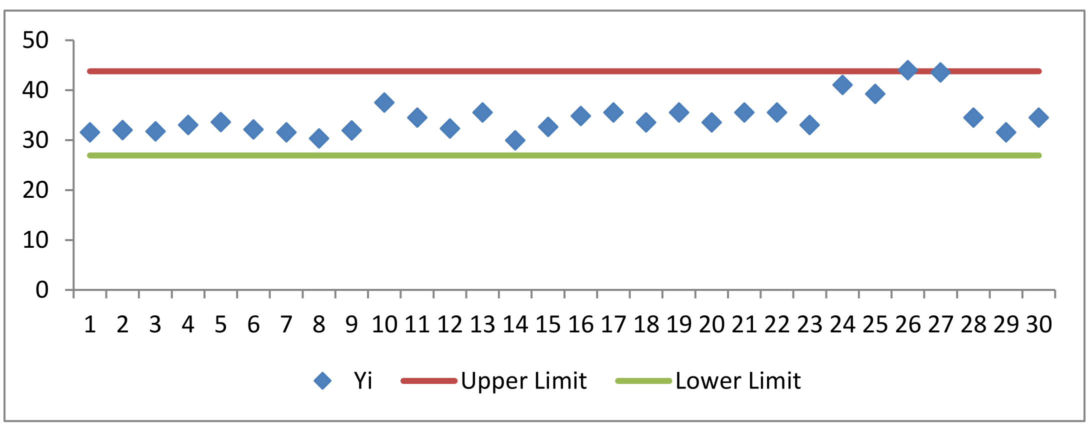


***Figure 15*:Graphical Representation Of Outliers.**

3.11 Variance Test

For variability the formula used is α = (Standard deviation/√Number of data* Mean)

Where $\boldsymbol{\delta}$n-1 is standard deviation

N is number of Data

Mean is X mean

α is Standard error

For variability from the formula mentioned in the methodology we got

3.12 Frequency analysis of rainfall

The term frequency analysis refers to the techniques whose objective is to analyze the occurrence of hydrologic variable (e.g. rainfall) within statistical framework. In many hydraulic-engineering applications, the probability of occurrence of a particular extreme rainfall, e.g. a 24-h maximum rainfall, will be of paramount importance. There is a definite relation between the frequency of occurrences and magnitude; the ordinary events occurring almost regularly than the severe storms. The reasonable length of record for frequency analysis should be more than 30 years. However, a record of up to 20 years can be used as sample data set for frequency analysis if data for longer record is not available.

i. Statistical selecting method

Probanility distribution are basic concepts in statistics .The result of statistical expermrnts and their probabilities occurrence   are linked by probability distribution .Rainfall data from Wolikite were evaluated with four probability models to find the best fit model.The probability model ,used include ,the Normal (N),Log –Normal(LN),Log-Pearson Type III(LP-III)and Gumbel probability model.

ii. Normal distribution

The normal distribution is the most useful continuous distribution of all the distributions. The probability density factor and cumulative distribution ofnormal are calculated using eqn-1 and eqn-

$$f(x) = \frac{\exp\left(-1/2(\frac{x-\mu}{\sigma})^2\right.}{\sigma\sqrt{2\pi}} \text{--} \qquad \text{Equation 1}$$

$$F_{(x)} = \varphi\left(\frac{x-\mu)}{\sigma}\right) = \frac{1}{2}\left[1+\text{erf}\,(\frac{x-\mu}{\sigma\sqrt{2}})\right] \text{--} \qquad \text{Equation 2}$$

Where $\mu = Location\ parameter$,the point where max.rainfal located to the horizontal axis

σ= Scale parameter

φ = Lapalace integral

In the normal distribution maximum value of expected rainfall :

X_T= x $(1 + CvK_T)$-- Equation 3

Where X_T= maximum value of Expeted Rainfall

$\bar{X}$ = Mean

C_V = Coffiecient of variation

K_T = Frequency factor ,which depends on the return period

$K_T = \frac{(X_T - \mu)}{\sigma}$-- Equation 4

σ = Stanadard deviation

The frequency factor (K_T) is the same as the standard normal variate (z).which is calculated using eqn(5)

$$Z = \omega - \frac{2.515517 + 0.802853 + 0.0110321\omega^2}{1\ 1.432788\omega + 0.189269\omega^2 + 0.001308\omega^3}$$ ----------------------Equation 5

From (eqn-5) can expressed as follows

$\omega = (\ln (\frac{1}{p^2}))^{1/2}$ ($0 < P \leq 0.5$)---Equation 6

Where p is the exceedence probability

P= $\frac{1}{T}$,when P > 0.5 , 1-P is substituted for ‘p’ in eqn 6

iii. Log normal distribution

The log-normal distribution is a distribution of random variables with a normally distributed logarithm. The lognormal distribution model includes a random variable Y, and Log(Y) is normally distributed. The probability density function (PDF) and cumulative distribution function (CDF) of the log-normal distribution are calculated using eqn -7) and (eqn-8), respectively:

$f(x) = \frac{\exp\left(-1/2\left(\frac{\ln(x-\mu)-\mu}{\sigma}\right)^2\right.}{(x-\gamma)\sigma\sqrt{2\pi}}$--Equation 7

$F_{(x)} = \varphi\left(\frac{x-\mu)}{\sigma}\right) = \frac{1}{2}\left[1+\text{erf}\left(\frac{x-\mu}{\sigma\sqrt{2}}\right)\right]$ ---Equation 8

Where $\mu = Shape\ parameter$,the point where max.rainfal located to the horizontal axis

σ= Scale parameter

φ = Lapalace integral

γ = lacation parameter

The log-normal distribution assumes that Y=In(X); therefore, the maximum value of expected rainfall (X_T) corresponding to any return period (T) can be calculated using Eq. (9):

X_T= exp $(1/T)$)---Equation 9

YT=Y(1+CvgKT)============---Euation 10

KT=(Yt-Py)/σ --Equation 11

Where X_T= maximum value of Expeted Rainfall

$\bar{X}$ = Mean

C_V = Coffiecient of variation

K_T = Frequency factor ,which depends on the return period

iv. Log pearson type III distirbution

The log-Pearson type-III distribution has been widely and frequently used in hydrology and for hydrologic

$$f(x) = \frac{1}{\sigma(\beta)\gamma(\alpha)} \left(\frac{ln(X)^{\alpha-1}}{\beta}\right) - y \frac{\exp\left(\frac{\ln(x)-\mu)-\mu}{\sigma}\right)}{\beta}$$ --Equation 12

$$F_{(x)} = P \frac{\left(\frac{lnx - y(\alpha))}{\beta}\right)}{I\alpha};=$$ --Equation 13

Where β= Scale parameter

φ = Shape

γ = lacation parameter

The log-normal distribution assumes that Y=In(X); therefore, the maximum value of expected rainfall (X_T) corresponding to any return period (T) can be calculated using Eq. (9):

X_T= Anti log x--Equation 14

Log x= X +KTSd --Equation 15

KT=2/Cs$\{2 - Cs/6)Cv/6 + 1\}$---Equation 16

KT=(Yt-Py)/σ--Equation 17

Where $\bar{X}$ = Mean

C_s = Coffiecient of skewness

Sd=Standard deviation

v.Gumbel distribution

Also known as the Extreme Value Type I (EV I) distribution, is a continuous probability distribution... This distribution can be applied to model maximum or minimum values (extreme values) of a random variable. The probability density function (PDF) and cumulative distribution function (CDF) of the Gumbel distribution are calculated using Eqs. (17) and (18), respectively:

$f(x) = \frac{1}{\sigma}\exp(-x - \mu) - exp(-x - \mu)/\sigma)$--Equation 18

$F_x = exp - \exp(-x - \mu)/6$ ---Equation 19

Where

σ= Scale parameter

$\mu =$ location parameter

he Gumbel distribution can be used to calculate the maximum value of expected rainfall (X_T) corresponding to any return period (T) using Eq. (19):

X_T= Xmean(1+CvKT)--Equation 20

KT=$\frac{\sqrt{6}}{\pi}$ $(0.5772 + \ln$ (ln ($\frac{5}{T-1)}$)--Equation 21

Where

$\bar{X}$ = Mean

Cv = Coffiecient of Variation

VT=Frequency factor

Sd=Standard deviation

General frequency equation (Chow (1951) can be used to analysis the frequency of annual 24-hr maximum rainfall of the project area.

$X_T = X_{av} \pm \sigma K_T$ ---Equation 22

Where:

XT: - the event (magnitude) at return period of T years

Xav =the mean of the sample data

σ =the standard deviation and

K= frequency factor, which is different for different distribution and is dependent on the recurrence interval, sample number, and skewnes of the distribution.

Therefore, the major question in this frequency analysis is getting the fitted distribution for the catchment rainfall dataset. Follow procedure to compute design or maximum 24-hr rainfall:

Procedure

- Extract the annual maximum time series data from the daily rainfall dataset (it has to be > 15 years of data record)
- Compute the mean (Xav) and standard deviation (σ) of the data
- If there is outlier in the annual maximum data series, remove it (them) from the data series
- Find the fitted probability distribution for the available sample data. The fitting of the probability distribution can be evaluated with statistical goodness fit tests (Kolmogorov Smirnov, Anderson Darling, Chi-Squared) using Hydro gnomon tool.
- Using the parameters of the best fitted probability function, compute the KT values for different return periods using the KT equations of each probability distribution
- Compute the XT for the different return periods using general frequency equation

3.13 Peak flood estimation

I. Rational method

Provides peak runoff rates for small urban and rural catchment areas, less than 50 hectares, but is best suited to urban storm drain systems and rural ditches. It shall be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input. The Rational Method is most accurate for estimating the design storm peak runoff for areas up to 50 hectares (0.5 km 2). This method, while first introduced in 1889, is still widely used. Even though it has come under frequent criticism for its simplistic approach, no other drainage design method has achieved such widespread use.

$$Q = 0.00278\ CIA$$ ……………………………………………………………Equation 23

Where

Q = maximum rate of runoff, m3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a given duration and A = Watershed area

II. SCS synthetic unit hydrograph

The U.S. Soil Conservation Service has developed a synthetic unit hydrograph procedure that has been used widely for developing rural and urban hydro graphs. The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions.

Techniques developed by the U. S. Soil Conservation Service for calculating rates of runoff require the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess.

A catchment area is determined from topographic maps and field surveys. For large catchment areas it might be necessary to divide the area into sub-catchment areas to account for major land use changes, obtain analysis results at different points within the catchment area, or locate storm water drainage structures and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems shall be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub catchment areas.

The SCS method is based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a ’typical" time distribution which the SCS has prepared from rainfall records. It is applicable for interior rather than the coastal regions and should be appropriate for Ethiopia. The Type II rainfall distribution will usually give a higher runoff than a Type I distribution. Figure shows this distribution. To use this distribution, it is necessary for the user to obtain the 24-hour rain fall value.

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental catchment areas were included. The equation was developed mainly for small catchment areas for which daily rainfall and catchment area data are ordinarily available. It was developed from recorded storm data that included total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall.

The equation is

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)}$$ ………………………………………………. ……..Equation 24

Where

Q = Direct runoff in mm

P = Rainfall depth in mm

S = potential difference

But

$$S = \frac{25400}{CN} - 254$$ ……………………………………………..Equation 25

Where

CN is Curve number

III. Estimating time of concentration (TC)

Time of concentration has been calculated by taking the stream profile of the longest streamline and dividing it in to different elevation. Kirpich formula is adopted for computation.

For channel-flow, Kirpich equation is:

$$T_{ch} = KL^{0.770}S^{-0.385}$$ ……………………………………-------------…Equation 26

Where: T_{ch} is as defined above and is given in minutes

L = the overland-flow length, in meters;

S = slope of main channel (m/m)

K = a unit conversion coefficient (K = 0.0195 for SI units)

For the existence of a low slope or a transitional slope condition, an adjusted slope should be used in calculating the time of concentration.

For the channel flow use the Kirpich

$$T_{ov} = K(L * N)^{0.467} S^{-0.235}$$ ……………………………......Equation 27

Therefor the total Time of concentration is given by

Tt =Tov + Tch **……………………………………………………**Equation 28

So, to use the above formula we have to first calculate the longest flow path length in (Km) and the elevation difference in the watershed from Arc GIS maps.

IV. Rainfall profile and area rainfall

Rainfall profile is the distribution of design rainfall with respect to time in the whole watershed area. It needs developed models for the selected drainage area. But there is sufficient modeling data in the vicinity and adaptation of standard curves is the only option. Designers of this project have adopted the standard curve from IDD Manual and used to compute rainfall profile of the project area.

As the area of the catchment gets larger, coincidence of all hydrological incidences becomes less and less. This can be optimized by changing the calculated point rainfall to aerial rainfall. The conversion factor is taken from standard table that relate directly with the size of watershed area and type of the gauging station. Aerial Rainfall = (Point Rainfall) x (Conversion factor).

V. Rainfall – Intensity – Duration (IDF)

The IDF relationship is a mathematical relationship between rainfall intensity, duration and return period that was developed in several parts of the world for its practical use. The design rainfall-intensity-duration relationships are obtained directly from the time distribution of rainfall, simply by converting the rainfall during a given duration to rainfall intensity in millimeters per hour. Maximum Intensity in mm per hour is derived for various durations and return periods for short durations of rainfall ranging from 5 minutes to 120 minutes. The value of rainfall intensity is computed by considering the proportion of the rainfall at the time of concentration using the of 24-hr rainfall. Rainfall intensity is the average rainfall rate during the time of concentration. Based on this definition, it can be calculated with the equation.

$$i = \frac{R_{tc}}{t_c}$$ ……………………………………Equation 29

Where: I= rainfall intensity (mm/hr),

Rtc=the amount of rainfall during the time of concentration (mm)

Tc = the time of concentration for the sub basin (hr) for its detail refer section5.3.2.2

An analysis of rainfall data collected by (Hershfield,1961) for different durations and frequencies showed that the amount of rain falling during the time of concentration was proportional to the amount of rain falling during the 24-hr period.

$$R_{tc} = \alpha_{tc} R_{day}$$ ……………………........................Equation 30

Where Rtc = the amount of rain falling during the time of concentration (mm of water),

αtc = the fraction of daily rainfall that occurs during the time of concentration,

Rday = the amount of rain falling during 24-hr (mm of water).

The Transport and Road Research Laboratory (TRRL, UK), Laboratory Report 623, Prediction of Storm Rainfall in East Africa (D. Fiddes, J.A. Forsegate and A.O. Grigg) give a valuable regional study on storm rainfalls in East Africa. The regional study is very useful for hydrologists and engineers involved in determination of peak discharges for bridges and culverts. The research study used the following model to fit a large number of stations data.

$$i = \frac{a}{(b+t)^{n}{}_{c}}$$

……………………………....------------------.Equatio 31

Where i = the intensity or rainfall in mm/hr,

t = the duration in hours, a, b and n are constants.

The best fit was found when b equals 0.33 hours and the index n varies from 0.78 to 1.09.

Adopting 24 hour rainfall value as I24

$$i_t = \frac{(b+24)}{(b+t)^{n}{}_{c}} * I_{24}$$

……………….,……………….------------Equation 32

Therefore, in this case, we adopt the value of “b” is 0.33 and “n” equals 0.9 (the average of n is 0.78 to 1.09 for East Africa).

3.14 HEC-RAS modeling approach

To apply the HEC-RAS model to the study area, the following datasets were necessary:

- Preparation and extraction of geometric data and Manning’s roughness coefficient at the streams cross sections and defining the flow discharge input data.
- The cross sections geometric data were extracted from Surveying data of the channel
- The Channel subsurface relief was represented by simplified rectangular channel cross section

In order to define the Manning’s roughness coefficient on the cross sections, a land use and cover map of the study area was created. For this, the aerial images of the area were processed in digital images processor Software.

The flow discharge input data for the simulations were estimated by Gamble /pearson type III distribution using the historic rainfall data of Wolkite station. According to Naghettini & Pinto (2007), this distribution is appropriate for estimation of extreme flood events studies. In this study flow discharge with return periods of 10, 25, 50 and 100 years were simulated. In order to define the discharge flow at the non-gauged watercourses, an on-site discharge flow measurement was performed to establish a discharge proportion between the gauged streams,

Finally, the extraction of geometric roughness coefficient data was performed in ArcGIS GIS, by HEC GeoRAS extension tools. After importing GIS processed data into HEC-RAS, steady flow state simulations were performed entering the discharge flow data at the upstream cross section of each reach, characterizing it as normal,supercritical and Subcritical.

The computational procedure is as follows:

1. Assume a water surface (WS) elevation at the upstream cross section (or downstream cross section if a supercritical profile is being calculated).
2. Based on the assumed water surface elevation, determine the corresponding total conveyance and velocity head.
3. With values from step 2, compute Sf and solve Equation for he.
4. With values from steps 2 and 3, solve Equation for WS2.

The model application assessment was done by comparing the simulation of a real event with flooded areas from a historical record, documented in Welkite Town.

3.15 Geometry

A 1D/2D coupled model was set up for the drainage channel as shown in Figure below, covering a 0.45 km reach of the Welkite drainage extending from the station 0 to 450 recorder site to the end of the lower gorge.

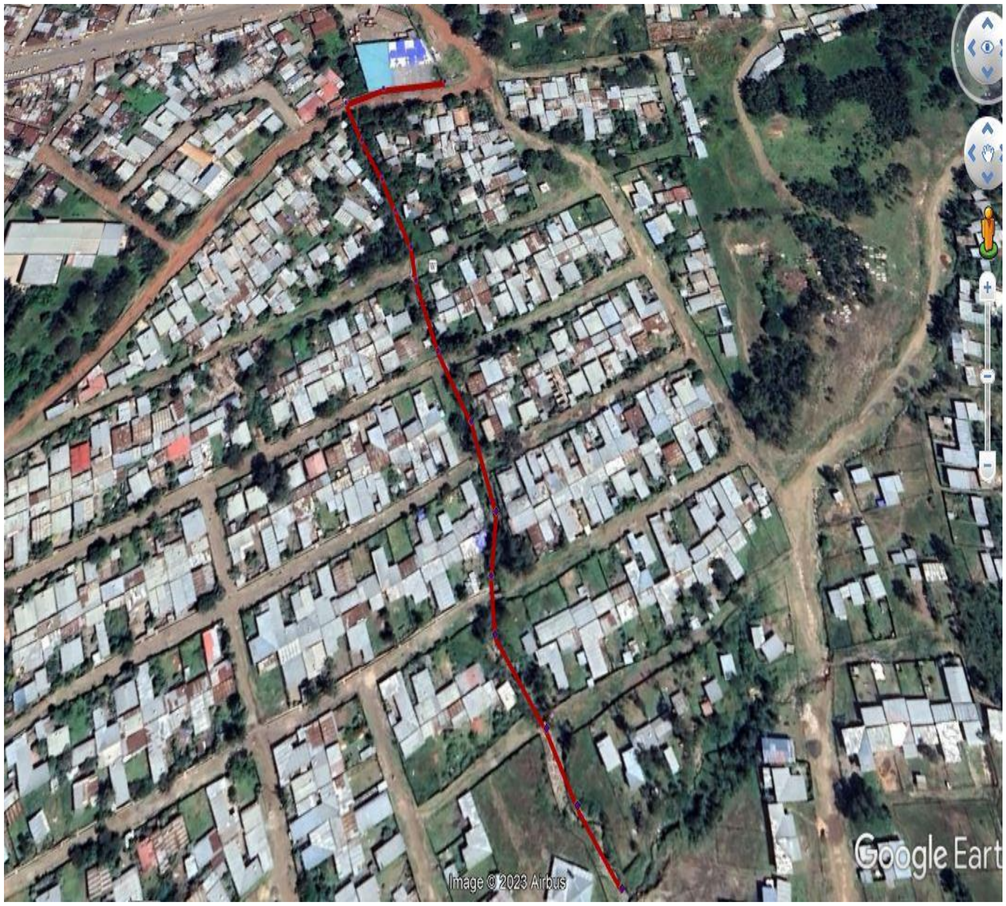


Figure 16:Allignment of the drainage chanel in the study area(Welikite municipal)

3.16 Terrain

A 30 m resolution DEM (Digital Elevation Model) was used to generate elevation data and also surveying data or cross sectional data of the drainage channels have been used. This DEM was interpolated from council LiDAR data, supplemented with survey data to provide accurate channel crest levels and river cross-sections in the vicinity of Welkite Town.

3.17 Hydraulic Model

Hydraulic modelling are normally only suitable for the puppose they were built however minor changes to this models may make them suitable for projects. HEC-RAS 1D model is used to simulate the water level of the the sytem .It make different,from others software.

3.18 Boundary Conditions

Bounderi conditions are required in order to perform the calculations. If a sub critical flow analysiss is going to be performed , the downstream bounderi conditions are required.If asuper critical flow analysiss is going to be performed, then only the upsteam bounderi cionnditions are required. If amixed flow regime calculation may perform, then both upstream and downstream boundery conditions are requaired.The upstream boundary condition used was normal depth in the channel and the downstream boundary condition used was critical depth in the channel.

3.19 Boundery Conditions in HEC-RAS

Several & sifferent type boundery conditions are available

Table 7:Boundery Conditions For HEC_RAS Analysis(Tutorial,HEC-RAS)

Boundery Condition	US or D/S?	Internal or External	Steady or Unstea dy)
Normal Depth	Both for steady D/s for unsteady	External	Both
Critical depth	Both	External	Steady
Known water surface elevation	Both	External	Steady
Rating Curve	Both for steadyD/s for steady	Extrernal	Both
Stage hydrograph	Both	External/	Unsteady
Flow hydrograph	Both	Exterbnal	External
Stage/flow hydrograph	Both	External	Unsteady

Laternal inflow hydrogqah	N/A	Internal unsteady	Unsteady
Unknown lateral inflow	N/A	intrnal	Unsteady
Elevetion controlled gate	N/A	internal	Unstady
Internal Boundery Conditions	N/A	internal	Unsteady
Stage/flow hydrograph	N/A	N/A	Unsteady

3.19.1 Normal Depth

The most widely used boundery condition for both steady and unsteady flow analyisis .For this type of boundery conditions ,the user is required to enter the energy slope value .This Value is used to calculate normal depth (manning eqn) at that location. For example most of the time:

Table 8:Bounery Conditions For Normal Depth(Tutorial,HEC-RAS)

Profile name	Boundery	Boundery
25 yr	Normal dpth	Slope=0.05
100 yr	Normal dpth	Slope=0.05

3.10.2 Critical Depth

In critival depth boundery conditions,the user is not required to enter any further information and uses the softwere automaticaly calculate critical depth for each of the profile uses that at the boundery conditions. However critical depth does not occur very often in stream or channel . Using critical depth is only appropriate, if there is any significant elevetions change or drop structure. It doesn’t matter if you use critical depth as long as your model exceeds for enough upstream or d/s the area of interest. For instance:-

Table 9:Boundery Conditions For Critical Depth(Tutorial,HEC-RAS)

Profile name	**Boundery**	**Boundery**
25 yr	Critical depth	Critical depth

3.19. 3 Known water surface elevation

The known water surface elevetion boundery condition is typically based on observed data. Alternativly,the user may enter known water surface elevation to make the current model consistent with another existing model.

Table 10:Boundery conditions for known water surface elevetion(Tutorial,HEC-RAS)

Profile name	**Boundery**	**Boundery**
25 yr	Water surface elev	Known ws elevtion
1ooyr	Water surface elev	Known ws elevtion

3.19.3 Rating curve

The rating curve boundery conditiond are typically used where achannel or stream flows in to a poned or lake, Rating curve can be constructed the data available from metrological stations in the country,in our case Welikte Town.

3.20 Goodnnes of fit for station rainfall data

Gooddness of fit is astatistical measure that define how well amodel fits as set of observetuons .It summerises the discrepancy beteewn observed values and the values expected under the model in question. Chai squared goodness test and kolmogurov-smirnov goodness test report is used under the software hydrognomon ‘

Chai-Square (x^2) goodness of fit test is atype of pearson chi –square test you can use it to test whether the observed distribution of categorical variables different from your expectations. Kolmogurov –smirnov goodness of fit test (k-S test) compare your data with aknown distribution and let you know if they have the same distribution. Although the test is non parametric, it doesn't assume any particular underlying distribution it is commonly used as attest for normality to see if your data is normally distributed.

Frequency Analysis

Step 1:- Extract the annual maximum time series data from the daily rainfall dataset (it has to be > 15 years of data record). From Welkite meteorological station we have got 31 years of data which is good for our analysis;

Step 2:- Compute the mean (Xav) and standard deviation (σ) of the data

For a flood computation of for 31 years maximum one-day rainfall data of Welkite station have been used to compute statistical analysis as follows(tabel 6)

Step 3:- If there is outlier in the annual maximum data series, remove it (them) from the data series We have checked the outlier test the section above and the data is ok and there are no outliers to be removed

Step 4:- Find the fitted probability distribution for the available sample data. The fitting of the probability distribution can be evaluated with statistical goodness fit tests. By using HYDROGNOMON software and by using Kolmogorov-Smirnov test have found that Pearson and Gamble max methods fit the data distribution however above all Pearson distribution fits best.

Step 5:- Using the parameters of the best fitted probability function, compute the KT values for different return periods using the KT equations of each probability distribution

Step 6:- Compute the XT for the different return periods using general **fre**quency equation

As shown in the Appendix 3 Goodness Tabuleted Report shaded,the data enterd is aaccepeted and with minmum error in the GEV=Max and EV1-Max (Gumble .L-moment)

As shown under Appendix-4 on kolmogno-smirnov goodnes test shaded , like that of chai square EV1-Max(Gumble ,L-moment) and GEV(Gumble) accepted with minimum error to use the data Graphically we see the goodness to fit in such away,

4. RESULTS AND DISCUSSIONS

4.1 Goodns of fit for station rainfall data

Gooddness of fit is astatistical measure that define how well amodel fits as set of observetuons .It summerises the discrepancy beteewn observed values and the values expected under the model in question. Chai squared goodness test and kolmogurov-smirnov goodness test report is used under the software hydrognomon ‘

The fitting of the probability distribution can be evaluated with statistical goodness fit tests. By using HYDROGNOMON software and by using Kolmogorov-Smirnov test have found that Pearson and Gamble max methods fit the data distribution however above all Pearson distribution fits best.

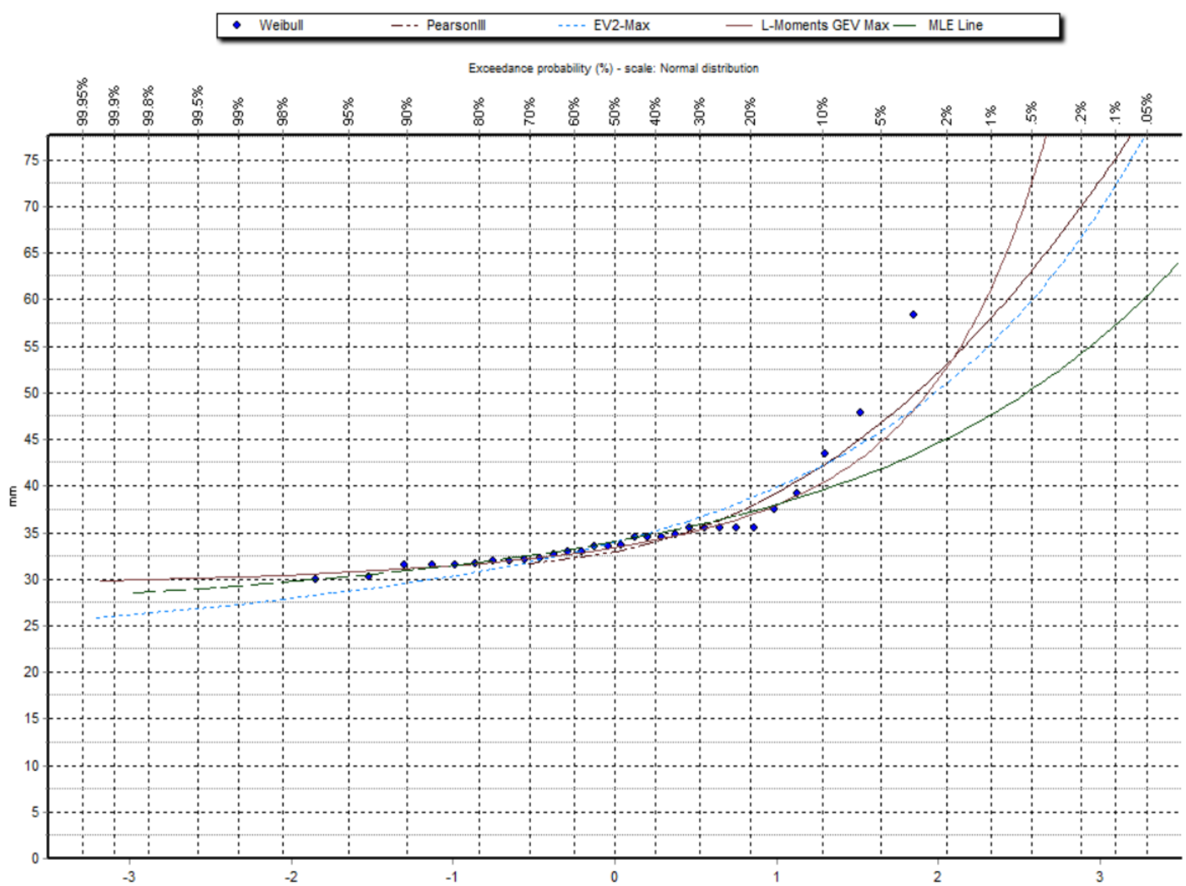


Figure 17:Goodness of fit curve

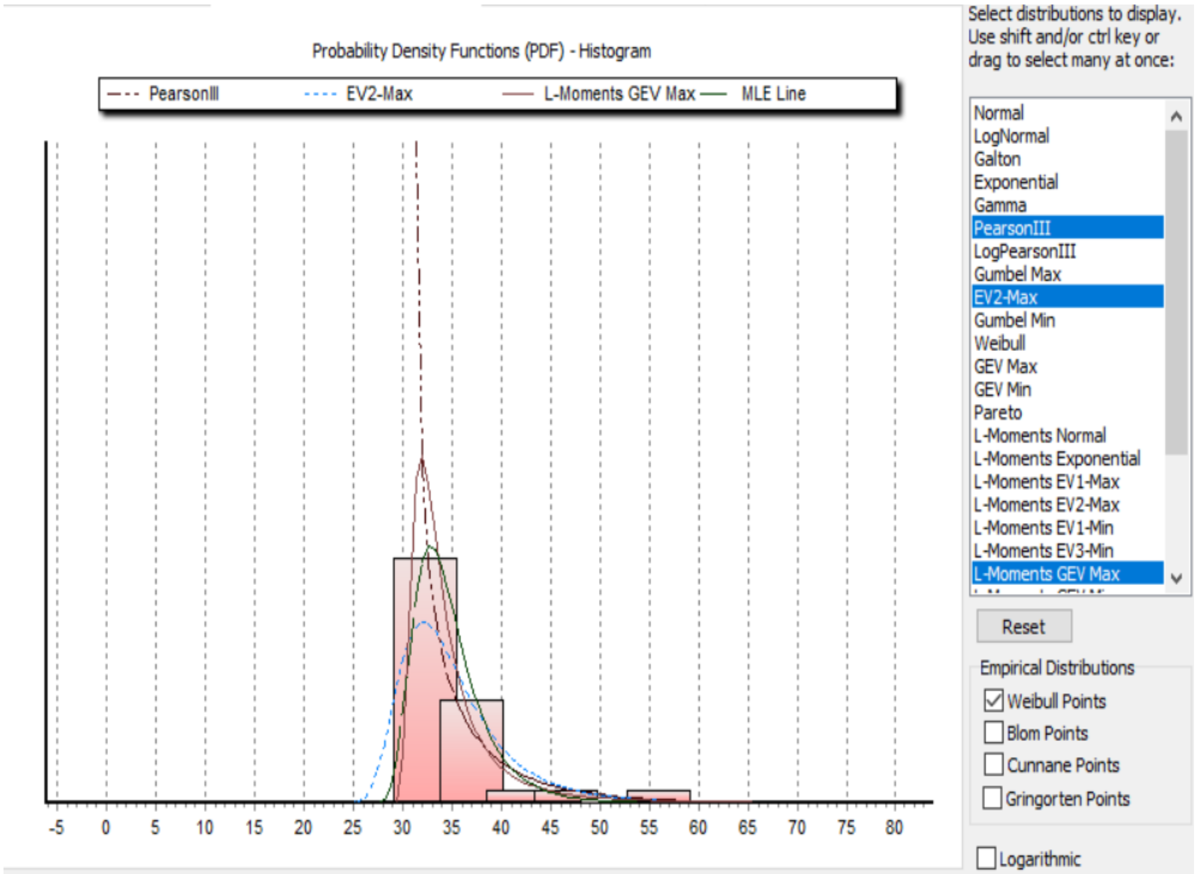


Figure 18:Bar Graph for used data estimation/return periode(Hysrognomon analysiss test)

As the graph shown as the rainfall return period increases some additional years and the rainfall used for flow estimation is good.

I .The Best to Fit test

The Quantile test result tell us which parameter is the best fit to use the data referring the outlier ,since in the oulier the max rainfall returned will around 45mm depth.

From the appemdex 2 GAM and pearson III is the best parameter can be used.

II .IDF Curve Development

The intensity duration frequency curve developed from daily rainfall data of 31 years from 1991 to 2021 obtained in Ethiopia meteorological agency rainfall gauge station located in Welkite city. For this study the rainfall intensity obtains from IDF curve is used .(Tabel-8)

All the probability distribution functions were compared by Kolmogorov-Smirnov test of goodness of fit and the selecting the function that gave the smallest value determined the best probability function. From the final result the best fit probability is Pearson distribution and that used to developed IDF curve of Welkite town as shown Tabulated result in Appendix 5 and figure below

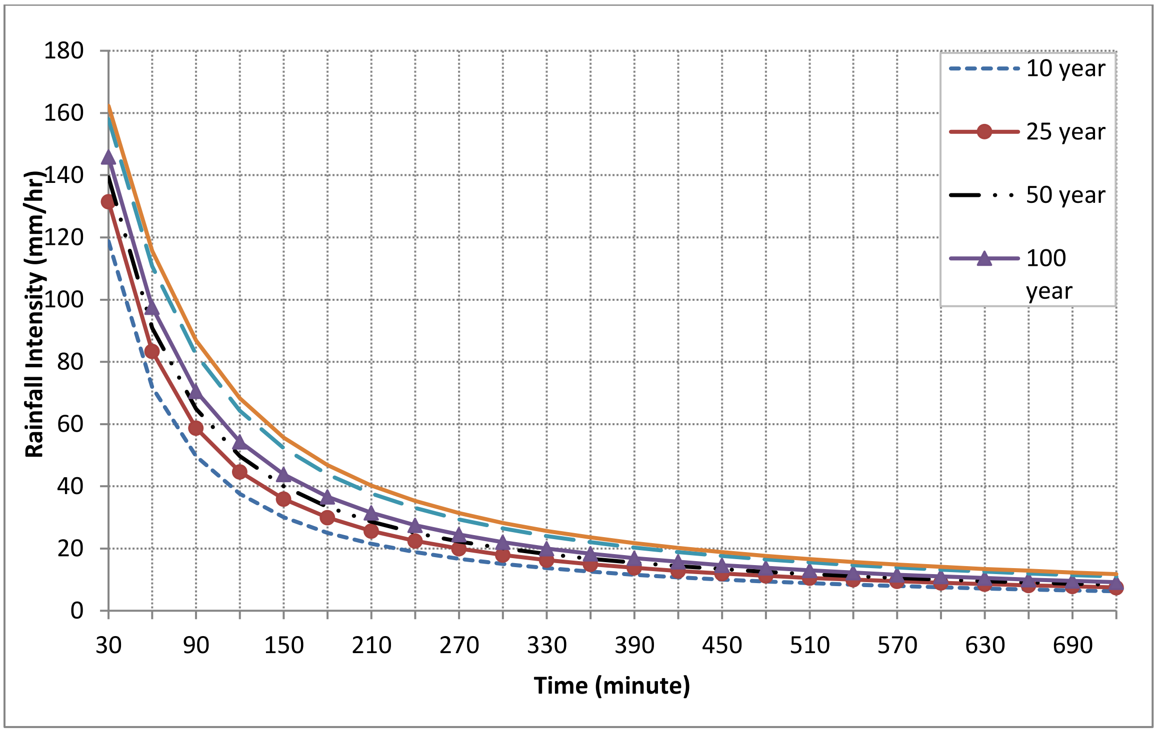


Figure 19:Wolikite **town IDF curve**

4.2 Design flood estimation and evaluation of the capacity of channels

Since our study area has less than 0.5 km2 area and it is urban area rational flow estimation was selected. Accordingly the result is shown below.

In the sub catchment shown that there are 5 identified drainage areas where the drainage chanela are evaluated ,and area of Tenataniya sefer sub catchment covers an area of 20.9 ha,area of arsema sub catchment covers 44 ha ,area of Tenatabiya sefer C covers area of 16.2ha,area of tenatabiya sefer D covers area of 11.4ha and area of Tenatabiya Sefer A covers area of 14.2ha.

Using SCS method. and

$$T_C = \frac{0.0195 * \left[\frac{L}{S^{0.5}}\right]^{0.77}}{60} \qquad S = \frac{25.4[1000 - 10]}{CN}$$

Table 11:Tc Calculation

No. Reach	Distance from watershed divide(km)	Elevation (m)	Slope	Tc (channel)	Tc (overland)	Tc (hrs)
1		1874				
2	1.2	1818	0.05	0.23	1.18	1.41
					Tc in minutes	84.55

So for the given Tc which is 41.06 min we can get Rainfall intensity from the IDF table as

$$I = \frac{(1 - \exp(2Tc * \ln(1 - \alpha 0.5))) * Rday}{Tc}$$

Table 12:I (Intensity) Calculation

	10 year	25 year	50 year	100 year	RP	
I	60.82	65.11	67.97	70.59	**41.06**	minutes rainfall intensity, P mm

***Table 13*:Sensetivity Parameter**

Slope factor	Cs	Soil permeability factor	CP	Land use/cover	Cv
< 3.5% Flat	0.05	Well drained soil e.g. sand and gravel	0.05	Dense forest/thick bush	0.05
3.5% - 10% Soft to moderate	0.1	Fair drained soil e.g. sand and gravel with fines	0.1	Sparse forest/dense grass	0.1
10% - 25% Rolling	0.15	Poorly drained soil e.g. silt	0.15	Grassland/scrub	0.15
25% - 45% Hilly	0.2	Impervious soil e.g. clay, organic silts and clay	0.25	Cultivation	0.2
>45% Mountainous	0.25	Rock	0.4	Space grassland	0.25
		Water-logged black cotton soil	0.5	Barren	0.3
C= Cs+Cp+Cv					

Source: Chow, 1964, p. 21-38; ASCE, 1996, p. 590

C=Cs+Cp+Cv=0.2 +0.1+0.25=0.55

Table 14:Rational Method Flood Result

S N	Name	Runoff coefficient (C)	Area, Ha	Flow Length, m	**Design Discharge (m^3s^{-1}) with return period**					Formula used
				1200	10	25	50	100		
1	A	0.55	20.9		1.94	2,08	2.17	2.25		Q=0.278 CIA
				450						
	B	0.55	**44**		4.09	4.57	4,57	4.75		Q=0.278 CIA
				300						
	C	0.55	**16.2**		1,5	1.61	1.68	1.74		Q=0.278 CIA

Table 15:Hyraulic Dimention And Capacity Of The Channel Using Manning Equation

Type of Channel	Observrd Dimention		n	A= b y (msq)	$P = \frac{by}{b+2y}$	$R = \frac{by}{b+2y}$	S	$\frac{1}{n}AR^{2/3}\sqrt{S}$	
	-	-		(m^2)	(m)	(m)	-	(m3s-1)	
Caannel A	b	0.9	0.013	0.45	0.195	0.195	0.052	**1.66**	
	y	0.5		-	-	-		-	-
Channel B	b	0.95	0.013	0.6175	0.3859375	0.3859375	0.035	**1.32**	
	y	0.65		-	-	-	-	-	-
Channel C	b	0.55	0.013	0.275	0.261904762	0.261904762	0.055	**0.34**	
	y	0.5		-	-	-	-	-	-

Table 16:Comparison Of Flow Summary

Sub Catchemnt Name	Peak Flow using Rational Method (m^3/s)				Existing Rectangular Drianage structure using Manning Eqn
	Q_{10}	Q_{25}	Q_{50}	Q_{100}	Q of Drainge Channels in each sub catchment in (m^3/s)
Catchemnt A Tenatabiya sefer	1.94	2,08	2.17	2.25	Q_A=1.66
Catchemnt B Tenatabiya sefer	4.09	4.57	4,57	4.75	Q_B=1.32
Catchemnt C Tenatabiya sefer	1,5	1.61	1.68	1.74	Q_C=0.34

4.3 Model result

HEC-RAS analysis requires several parameters such as the left and right bank elevetion, contraction and expansion coefficient, Manning's coefficient, segment length between two adjacent cross-sections,slope, and flow of the catchment and others for the process of hydraulic model analysis. Manning's coefficient, n of 0.045 for each cross-section and for the channel 0.013 is taken since the channel constructed by massonery foundetion. Those parameters were used to obtain required dpth of the channel ,channel power,velocity of flow to keep smooth flow longitudinally finally to calculate the capacity of the channel to be done by controlling the the slop along the channel and increase servisability. From the out put of the model result ,flow velocity ,depth of the channel,station crossections ,rating curve, channel profile ,slope and iterated results.

This study result of the model fully understands the town drainage system performance under multiple working condition one to identify the critical condition and related problem with drainage system, secondly to assess the hydraulic performance of storm water drainage infrastructures and finally to Evaluate capacities for drainage problem and mitigation measures by using hydrological model.

The total sub catchment of the study area is generating runoff from daily precipitation taken as extreme events from each year from 1991-2021 rainfall data.

Therefore, from the result , we identify and relate the model result with the existing constructed channels are under size and not carry and convey the runoff at maximum rainfall , they are exposed to distruction due stream power, slope of the channls are not adapted as per the demand and landform of the town,slope of ditches was above the requairment leads to environmental ditruction and exposed to erosion, ditches that have high slope produce sound turbulence ,disturb and distroy the ditch itself and displace residents and destroy the properties at the periphery of ditch area ,crack down of ditches and produce differential settlement, since the size of ditches under the requirement and over top during annual minimum rainfalls because from analysis we could see that the peak flood data's used as an input for the HEC RAS steady state analysis the depth of the channel very small. Hence we can descide that the ditch cross sectional design is inappropriate for the catchment discharge. Unless and other wise channel depth extended vertically to carry out the computed flow to control the water surface elevation found ,a yearly flooding and distruction of properties expected.

During detaile desaign, differnrt parameters should be comsidered such as, slope, appropriate selection of residential buildings and delocation of households in appropriate risk free places in the town, considering the existing flood risk areas using for BMP to eliminate and control flooding, conveyance ratio (upstream conveyance divided by downstream conveyance) shall be controlled , need for design revision and additional depth of channels for smooth normal conveyance of flow inside , using the flood for water balance and other development pupose.

4.3.1 The Channel Conveyance

Result from the geometry and flow data shows tthat physical state of the available cross-section in each station. The physical state of each cross section is different from each other depending on the depth and width of the channel station. Figure below shows example of HEC-RAS cross-sectioa up steam and bdown stream plot Station.

I. **Channel A ,Geometry Crossection,and Profile (Tenataniya Sefer)**

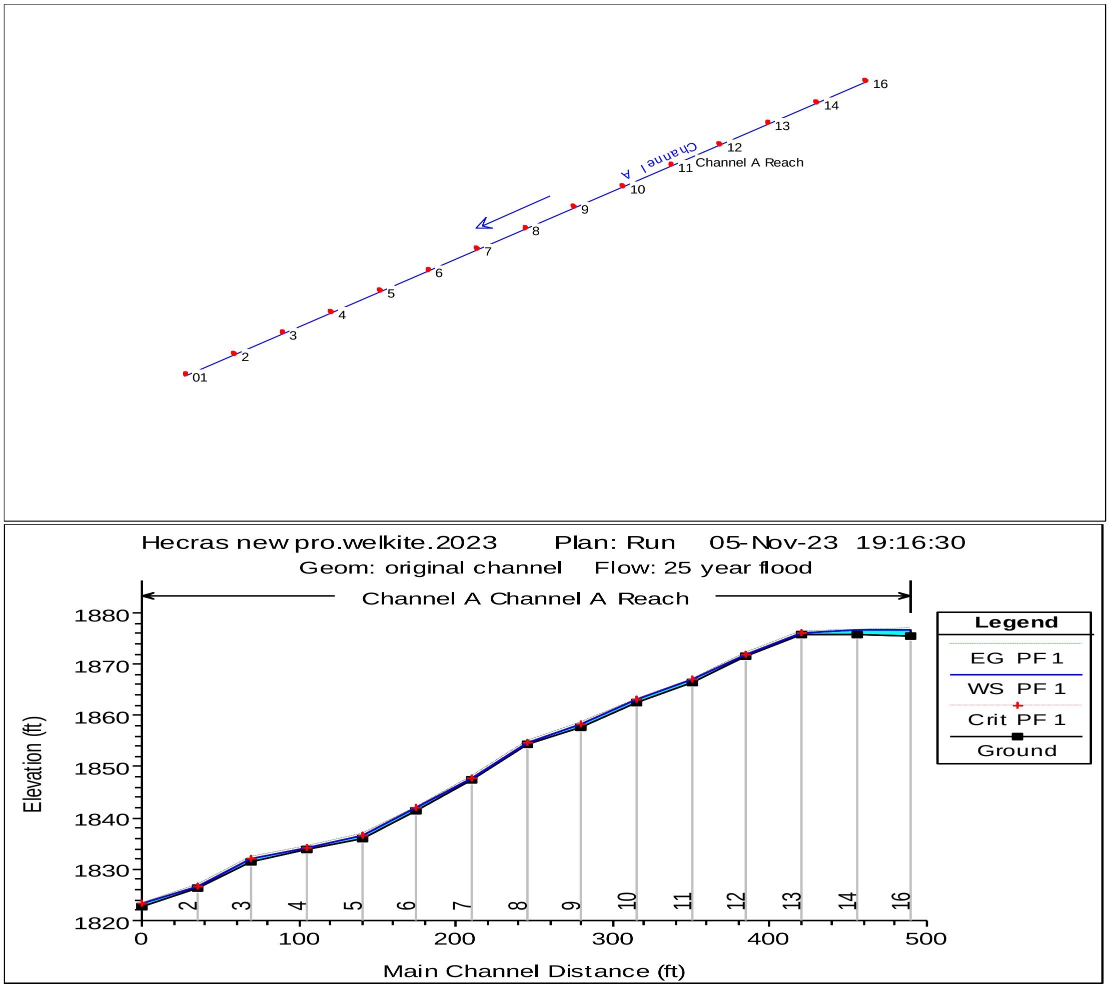


Figure 20:Upstream And Down Stream Crossection Of Channel A Tenatabiya Sefer For 10 Yr Return Period

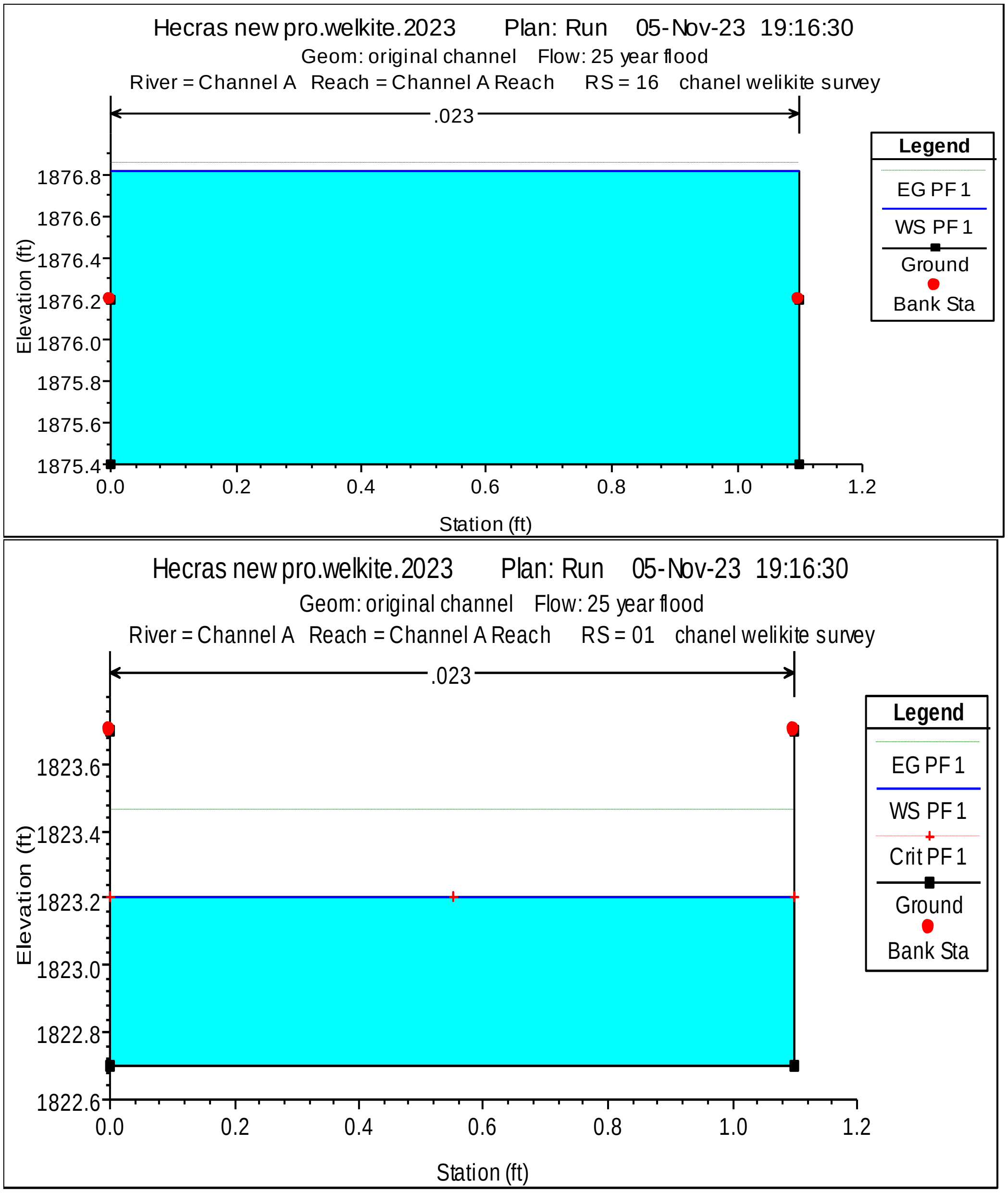
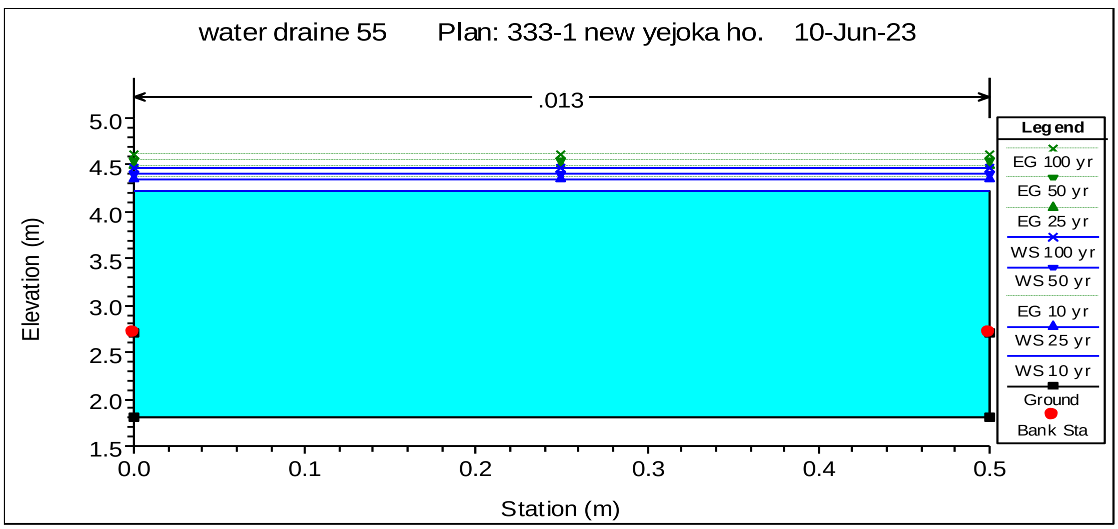


Figure 21:Upstream And Down Stream Crossection Of Channel A Tenatabiya Sefer For 10 Yr Return Period

The above figure shows that ,at upstream and down stream of the channel the width was different and the channel depth is similar and the result is under capacity the sub catchment flow and needs additional vertical extention. The result shows that, during maximum rainfall the copmpund area is flooding and from the flow was above the level of existing structure and requires additional 1.38m depth with a 0.1m free board atotal of 1.48m depth vertical extention. Which means from the channel bed 2.43 m depth should be construted to control flooding of the area since existing depth of the channel less than Δd of flow and under capacity for calculated subcatchemnt discharge.However,the down stream channel looks under the bank level of channel due to the supercritical flow and no vertical extention requaired for 10 years return periode.

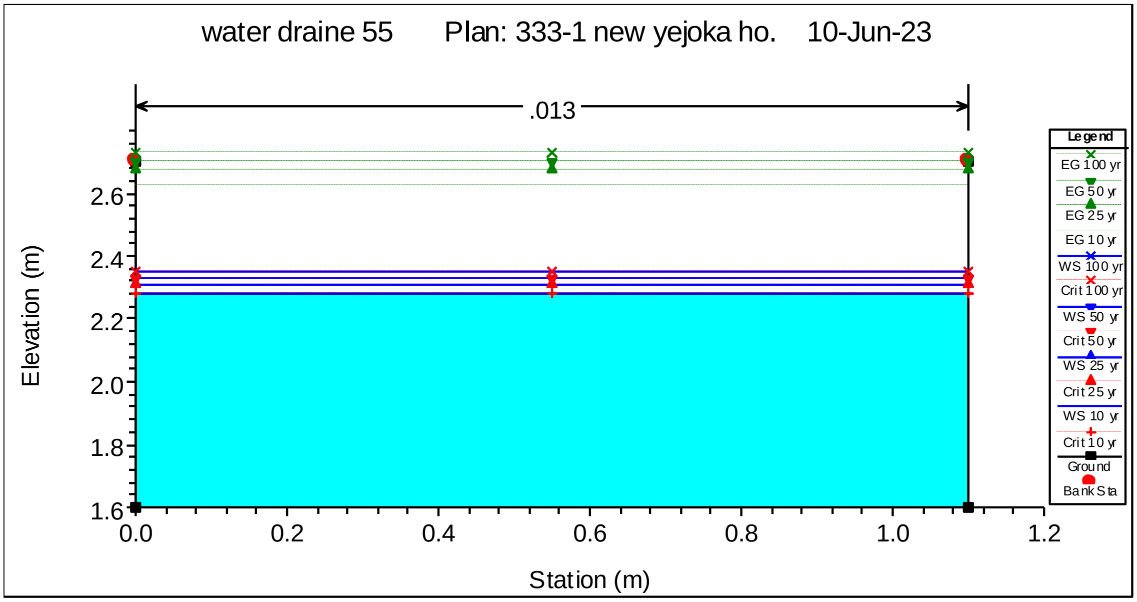
The under crossectiom attached was additional a 10,25,50 and 100 years return periode occurs of flow in the sub catchment area and considering a 50 years check point for design pupose and observed that a water surface change in elevation of 2.61 from the bed of the channel and with afree board of 0.1m atotal of 2.71m depth of channel should be constructed and indentified with the existing level of channel we obtained a difference of 1.56m additional vertical extention for 50 years return periode with the same width.

The implication of the result tell us ,the design periode of the exising drainage desaign periode less than 3 years calcelted by linear interpolation. From this point additional research has to done based on the cost benefit economic analysiss of the town construction of drainage structure



a)upstream

Figure 22 Crossection of channel A- Tenatabiya sefer from 10-100yr period storm



b)Down stream

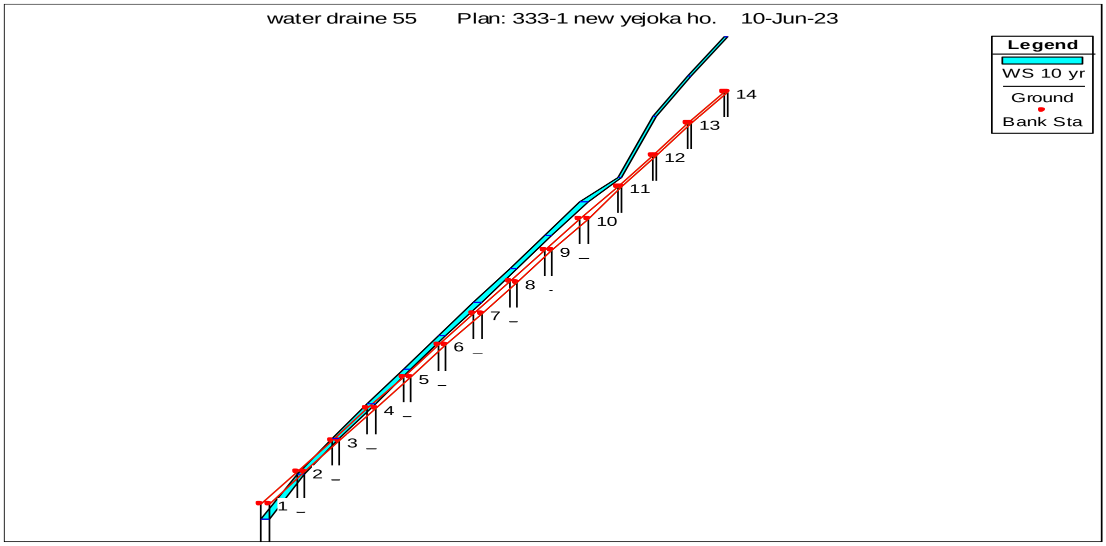
Figure 23 Crossection of Channel A -Tenatabiya sefer from 10-100yr periode storm

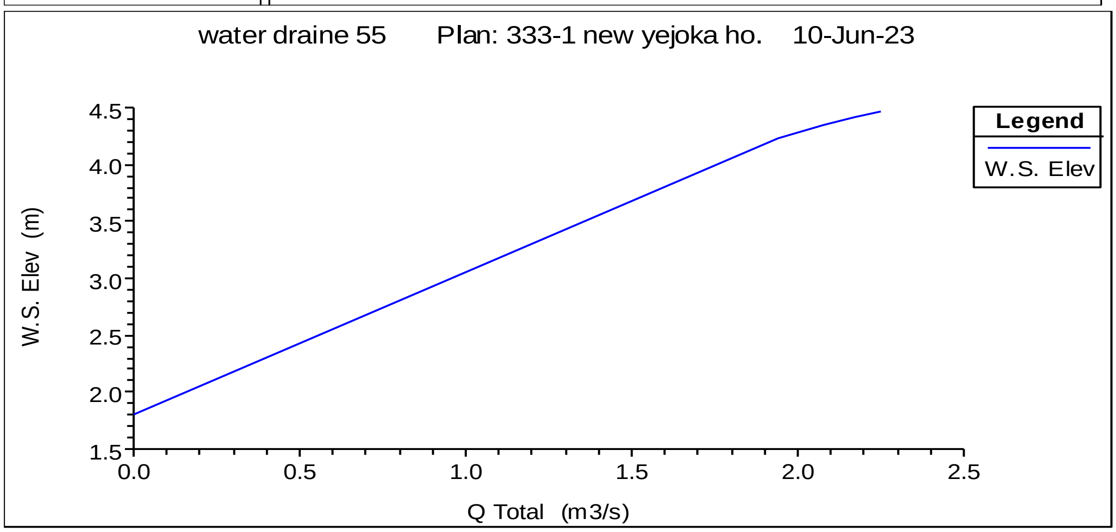
As discussed during the ten year crossection,the croosection of the 10 -100 year time periode indicates that ,the town existing structure capacity was not perform the subcatchment discharge among the maximum precipitation of yearly occurrence.

4.3.2 Drainage Ditch Profile ,Rating Curve,profile and Detailed out put results of Channel A

From the surveying data of the drainage ditch the cross sectional profile can be drawn as follows and demostretes the depth of the channel in relation with the flow as shown as (b- rating curve) of the figure and the longitudinal three dimentional view demonstrates, the model water surface elevation compared with the existing structural level.

As per the data taken from up stream level and downstream end it start 1818 at higher point and ends at 1874 at lower end with ground surface length of 1200m having approximated slope of 0.045. The perpendicular crossection the channel relate the linear relation ship of depth and discharge along the channel and tell increasing the depth of the channel be unquestionable as per the the top level of the channel. The figer under ilustrete the function depth(m)with Q in mcu.





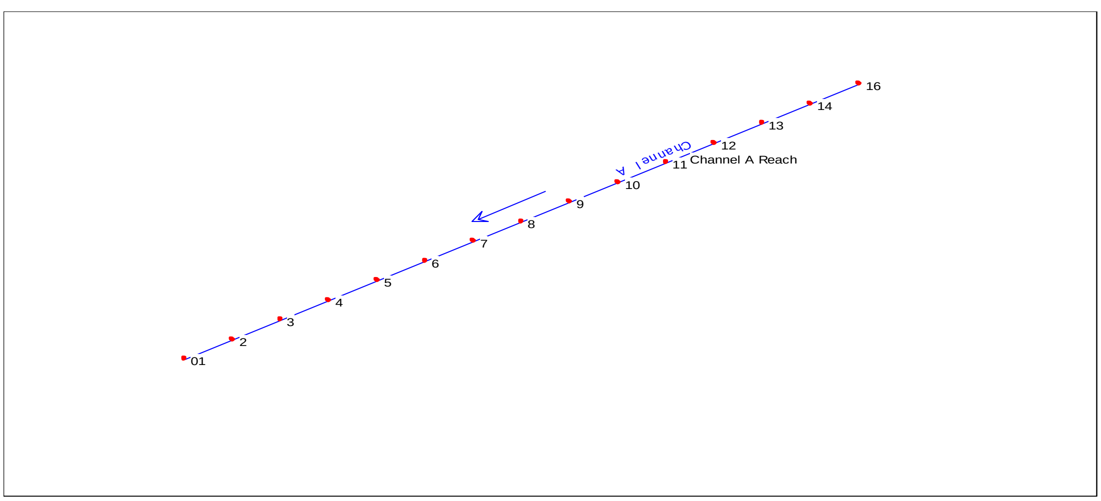


Figure 24:Profile and Rating curve of Channel B

From the Appendix 6 table 21 shown the critical water surface elevation of the channel for the sub catchment flow was 2.65 meter however the existing depth 1.05   and water overtops and flooding the sub catchement during maximum rainfall occurrence. When we observe the result in each stations of along the channel looking the result of the profile . which means awater surface elevation of 4.23<Y<2.28 meter ,since Yis a water surface elevation and the channel is below the water surface elevation.

As shown under in Appendix 6 the summary of table 22,the required water surface elevation for the width stipulated in the input data in the average depth of Δd=3.65-surface elevation=5.65-1.65=2m depth for the ten year desaign period with constant width of the channel. In down stream end Δd=2.28-surface elevation=2.28-1.65=0.63m that is less than the depth which currently in the channel.

When we come to velocity of flow,above the standard stipulated in ethipian roads athourity underlining velocity of flow in open channel should be not less than 0.5m/s and not greater than 1m/s to make normal through the channel and the resuul frm the model much greater than the velocity stipulated. So, amendement of slope and size along the channel and proper site selection is required to control the flow.

The result from Appendix - 6 Table 23 ,10-100 yr maximum flow calculation result shown as under for channel A of Tenatabiya Sefer. The figure shown under Crosssection of 10-100 year return Periode for Channel A

The under shown table shows that the 10-100 year return periode profile out put table, indicating necessary dimentions to be done based on the flow occurred.

II Channel B conveyance Crossection (Tenatabiya Sefer)

The same procedure has been followed that , channel B crosses number residential buildings with in side of coble stone road that collect not only storm water , but also connected to sewer line tha drain to in the drainage line around downstream of the town. This channel has 350m in length ,adepth of 0.65 meter and awidth of 0.95meter the sub catchment runoff calculated by rational method and 4.09 mcu/s in a ten year maximu rainfall annual occurecnce.

Based on the data describe aboevethe existing drainage dich capacity has calculated by manning equation and the subcatchment runoff calculated by rational method.,HEC-RAS tool is used to model the chnnel and simulate the described input datas to justify the physical problem by mathimathical result.After the model simulation basic output datas are to describe ,to define and to indicate the basic problem across the existing channel.the following crossectional outputs are found for beter conclusion and recomendaation.

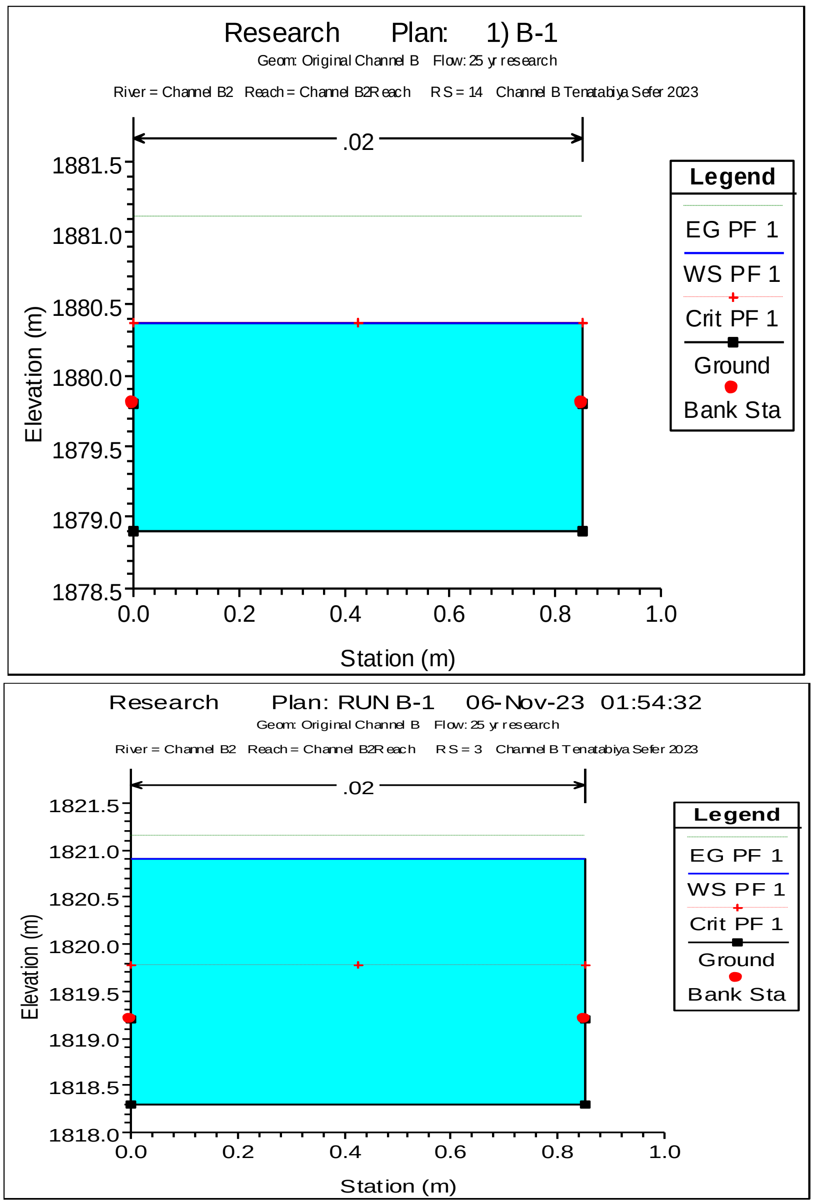


Figure 25: Crossection of **Upsteam of 10year return peiod & Down Stream of 10 year Channel B**

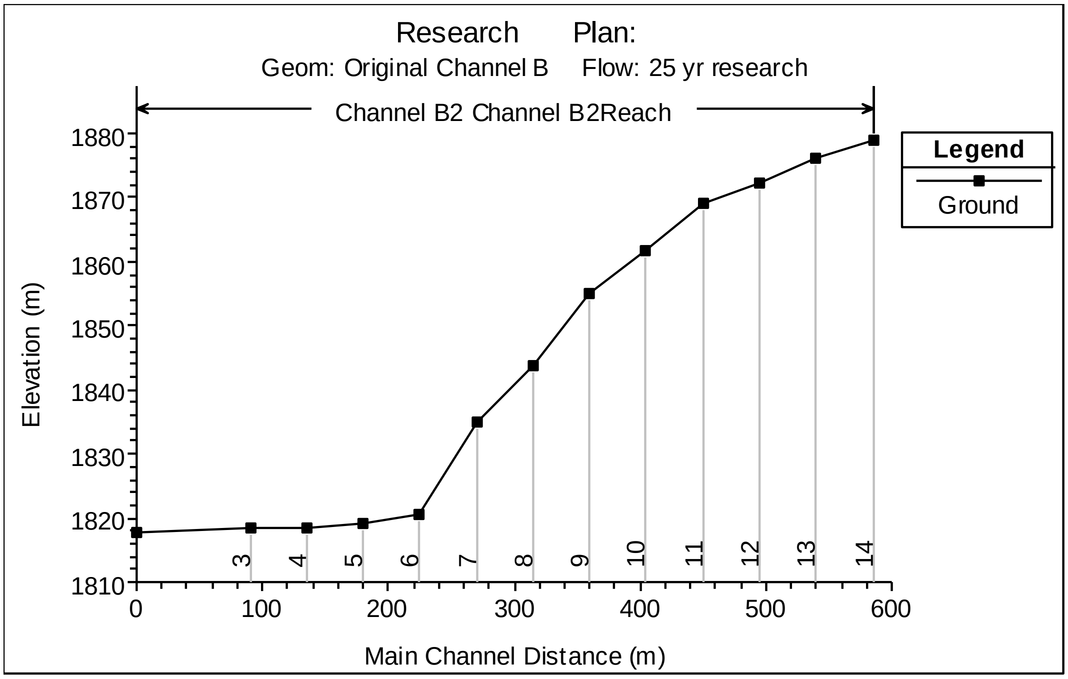


Figure 26:Crossesction Of Channel B Upstream And Down Stream Profile

From the figure above in up stream and down stream cases ,the existing depth of channel B much lower than the required return peride depth for the flow.The water surface level of the channel during analysiss reaches to achenge in depth of 63.3-61.15=2.15 above the bank of the channel .Abig concern is required to overcome the flood occurred around the compound.The main issue was tha cappcity of the ditches which is not perform the flow. Up stream of the channel area runoff not collected and flooded the compound and destroy resourcses of the area.

This is directly pass through how much the drainage structure of the town constructed with out any precondition of thechnical guide.

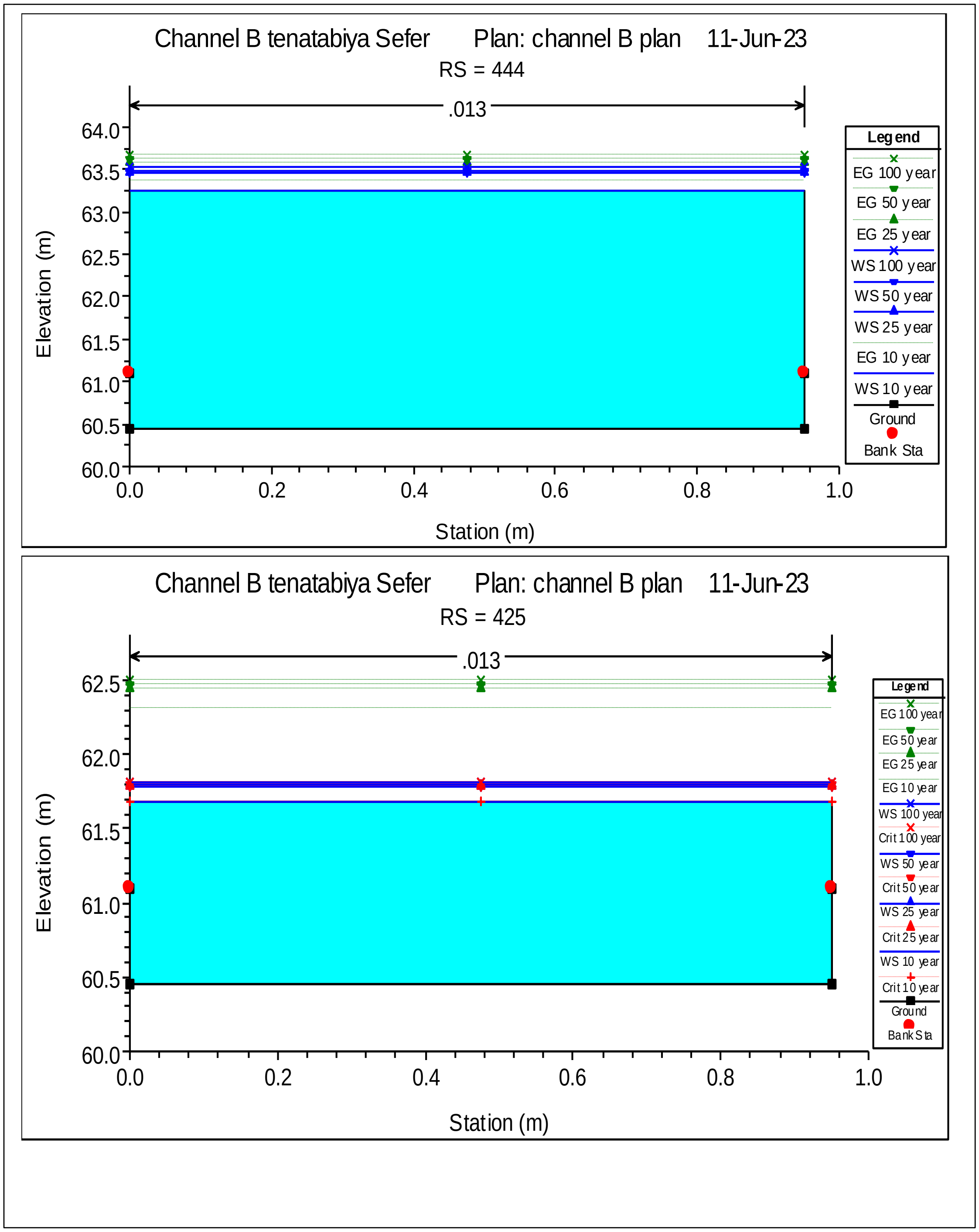
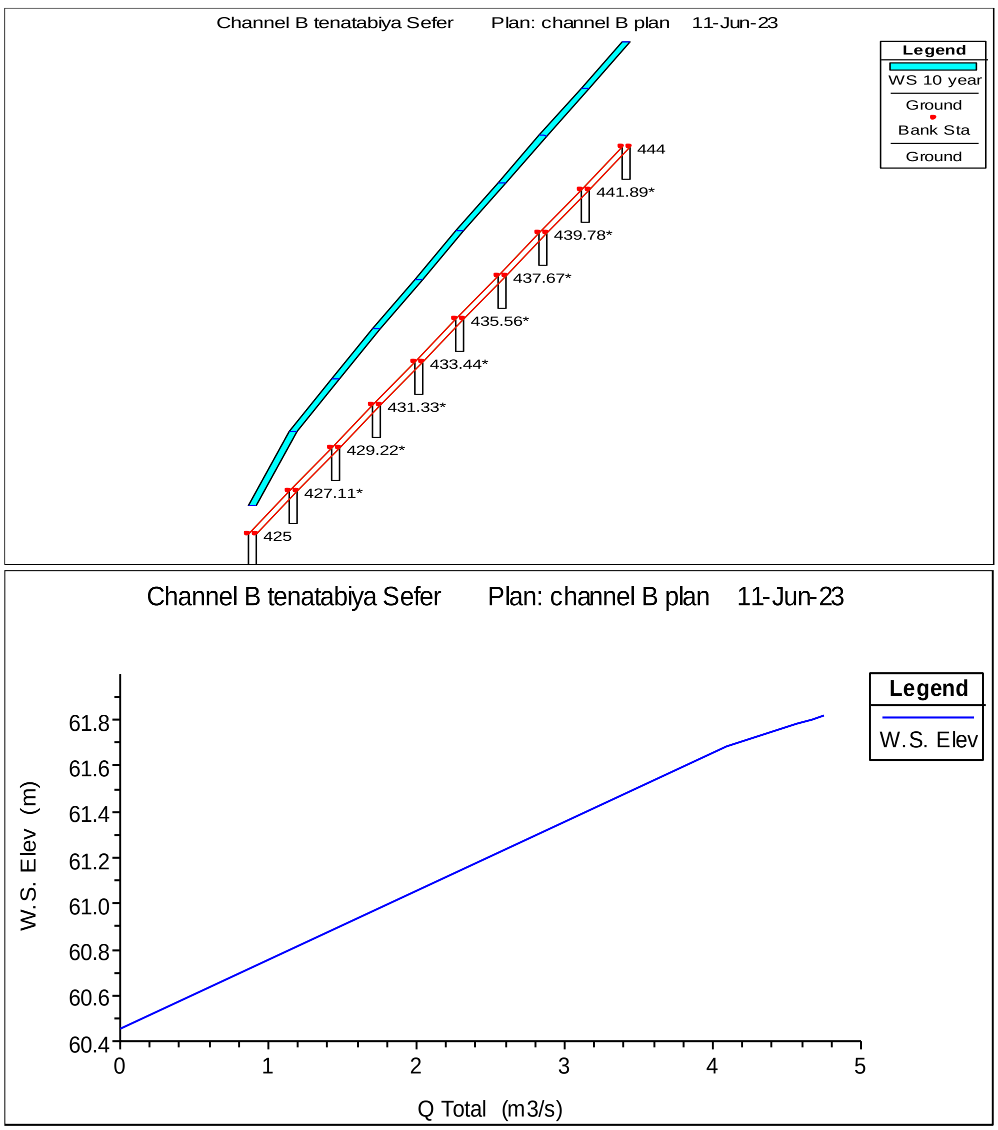


Figure 27:Crossection of upstrteam and down Stream 10y-100r-yr profile

From the figure,ober flow of the channel above the bank of the channel indicate that the design periode of the channels are bellow 10 year that means the existing driange dtructure meets in the ration of depth versus flow 0.65/2.90 with a 0.1m free board reach to 22.4% and by interpoletin with the service year below 2 year and has an implication of redundant construction cost westage.

4.3.3 Drainage Ditch Profile ,Rating Curve,profile and Detailed out put results of Channel B

AS shown the rating curve of the graph based the crossection of channel B at upstream end ,the depth versus Q linearly increase and the depth of the channel not enogh to convey the flow.The graph advices and show increasing of the depth channel make the catchment runnocc collection smooth and minimize property distruction of the community.



a) X-Y-Z perspective profile plot *b)Rating curve*

Figure 28:Channel B Profile and Rating Curve

Therefore, within 450 m distance there is 61m elevation difference which is very high head that can create diatructuon and due to this aproper number of dicision for channels are required speciallt from Joke hotel front face to via arsema there need to be require total design correction and diche line adjustment to minimize the head stipulated. That means the slope of the channel was above the standard recommendation for drainage comveyance i.e 61/450=0.13 and slope must ne between 0.01 to o.0.05 .

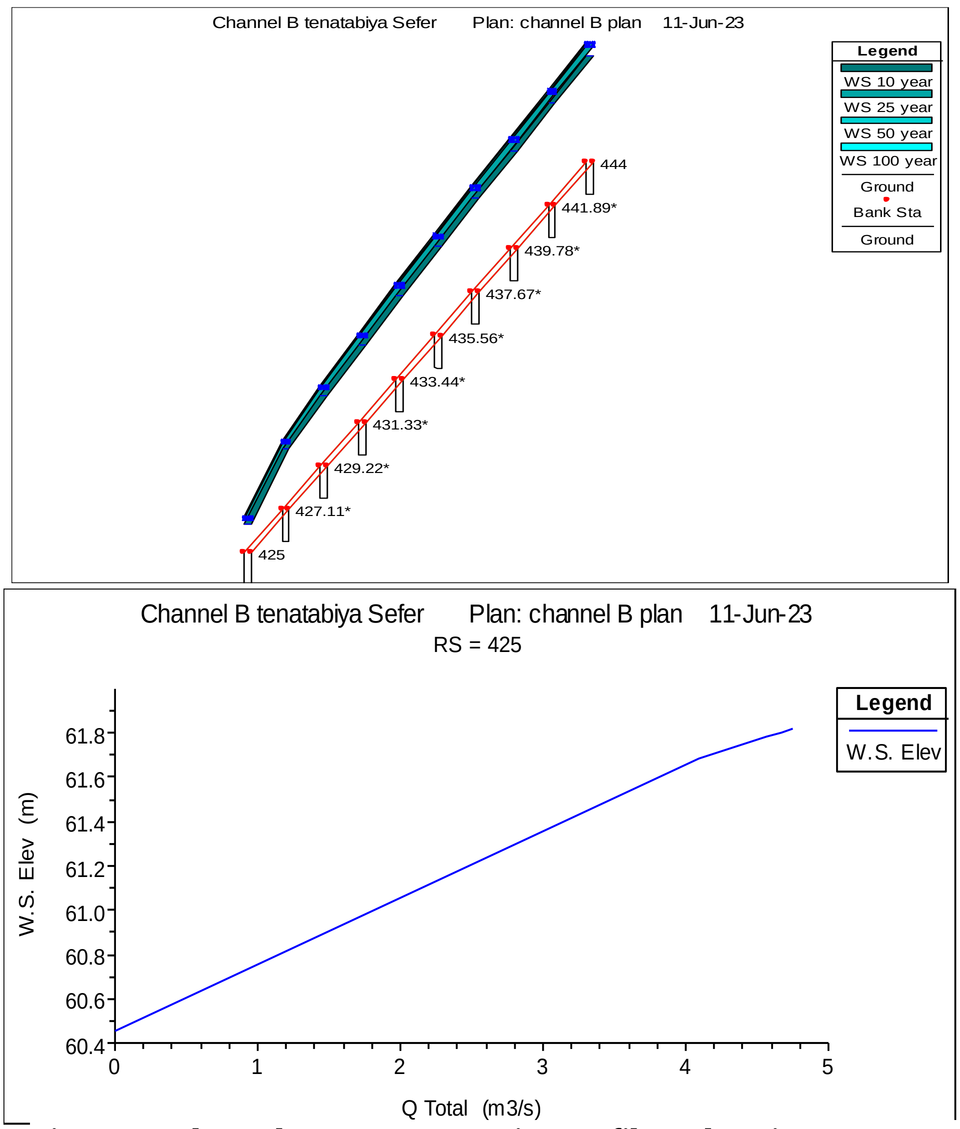


Figure 29 Channel B XYZ Perspective Profile and Rating Curve

:

The profile along the channellength shows that depth of the existing depth of channel much shorter than to accomodete the flow result from maximum rainfall of each year of flow predicted at a ten – hundred year return period. Its un questionable that additional depth amddement during detaile design is the first assignment to be done.

At upstream of the channel,as depth of the channel and flow has a direct relation ship with liner function.

As we know, the rating curve is a relation between water surface elevation and channel flow (discharge). At upstream location as depth of the channel reaches to maximum ,the flow ranges from 0-4.6mcs. and vary along the channel length shown above.The out puts results of hecras attached for 10 up 100 yers design periode.

Appendix-7 Table 24 channel B below anexed shows the result of water overflow in all 10 stations. From the table, it can be concluded that the maximum level of water overflow is 1.23 meter and the existing observed depth was 0.65m . the remaing 0,58m the overflow of water for each station and correction of depth required which mean 47,2 % of the channel hight must be added in order to accomodete the 10 year return periode flow.

This indicate that Channel B of tenatabiya sefer complete its design periode even if it constructed early 2012 periode of time ,and number consruction probems may mentioned during detaile design or research on the issue..

Appendix 7 Table 25 below annexed of Channel B Tenetabiya Sefers ,shows the result of cross sectional output in all stations. From the table , , it can be concluded that the cross section end points

Various element correction and the existing drainage dich was under capacity to convey flow in side in it.

From the Appendix – 7 table 26 detailed out put of channel B shown for 100 year return perode the required maximum depth and the observed and measured esisting channel dpth was compared and problem of flooding is identified as limitation on depth of the channen to carryout the runoff of the catchemnt .Also ,for sustainability of the channel,velocity of flow considerd and above the limit,and slope of the channel should be modified

III Channel C Crossection (Arsema Sefer)

Its known that from our manning caculetion above to check the capcity of of the channel and Rational method to know the catchment discharge ,the following out are found by the under attached model result.

As we saw from the ccrossection the bank the existing chnnel under the water surface of the simulation.this means channel C is in capable to carry out the discharge in sub catchment since this channel is located possible location of inlet zone of the sub catchment eventhogh there are many channels around the area.

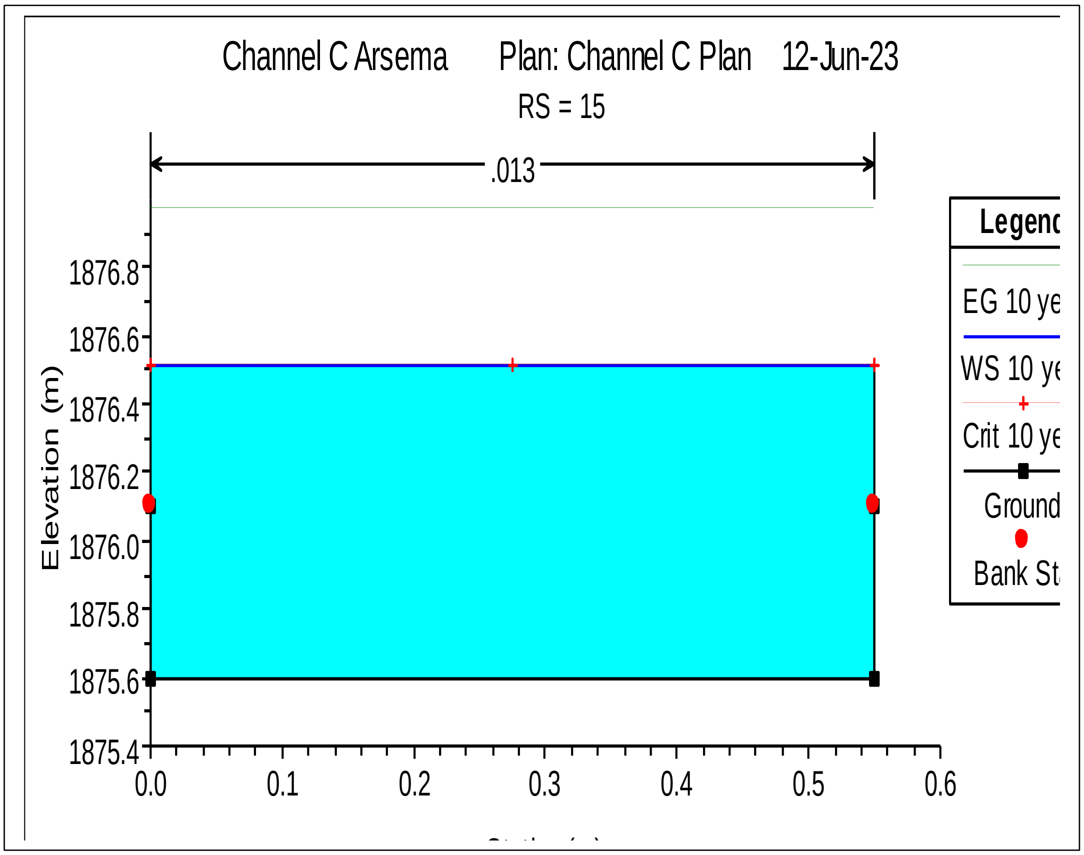


Figure 30 up stream of Channel C crossection arsema sefer

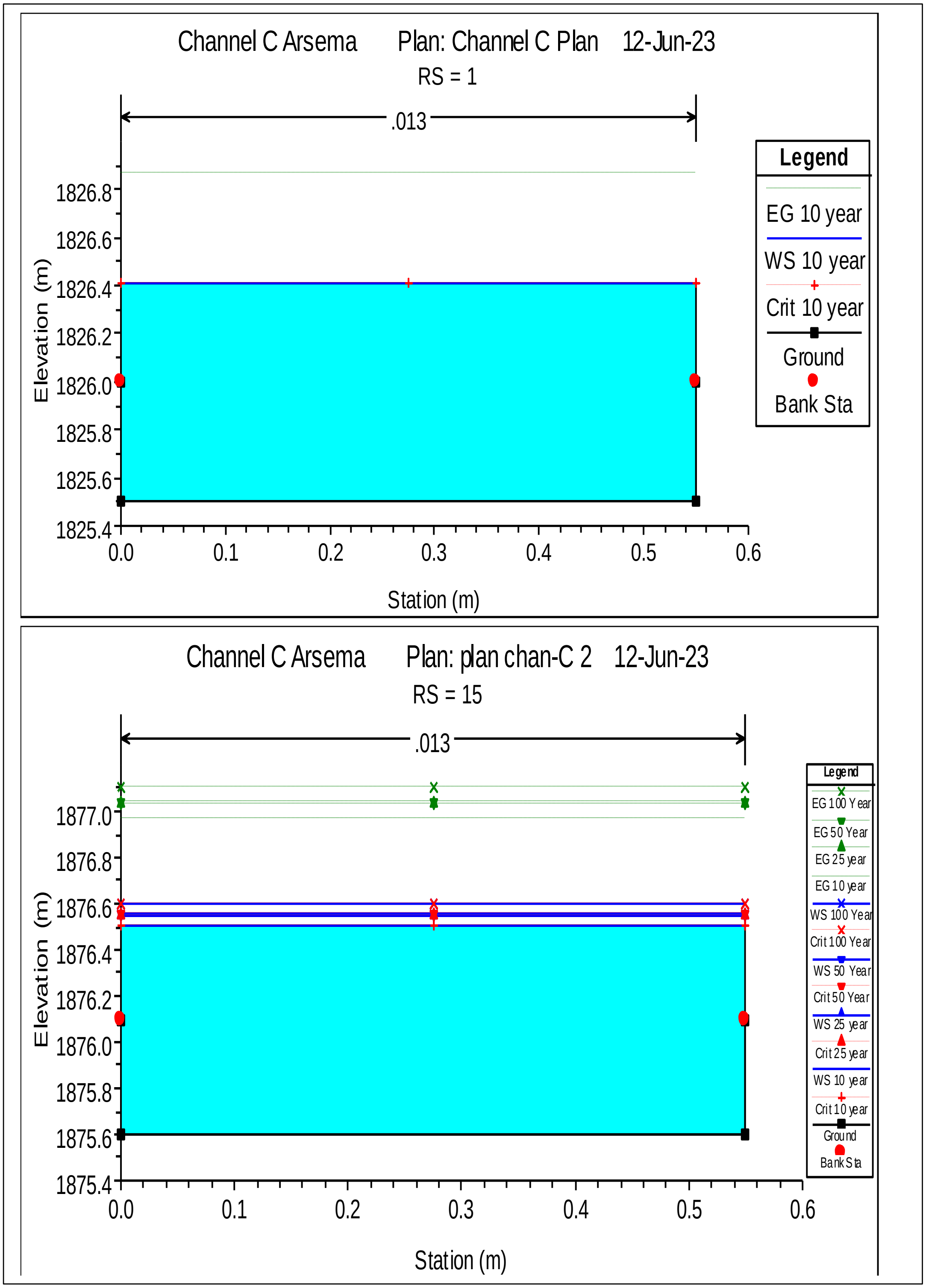
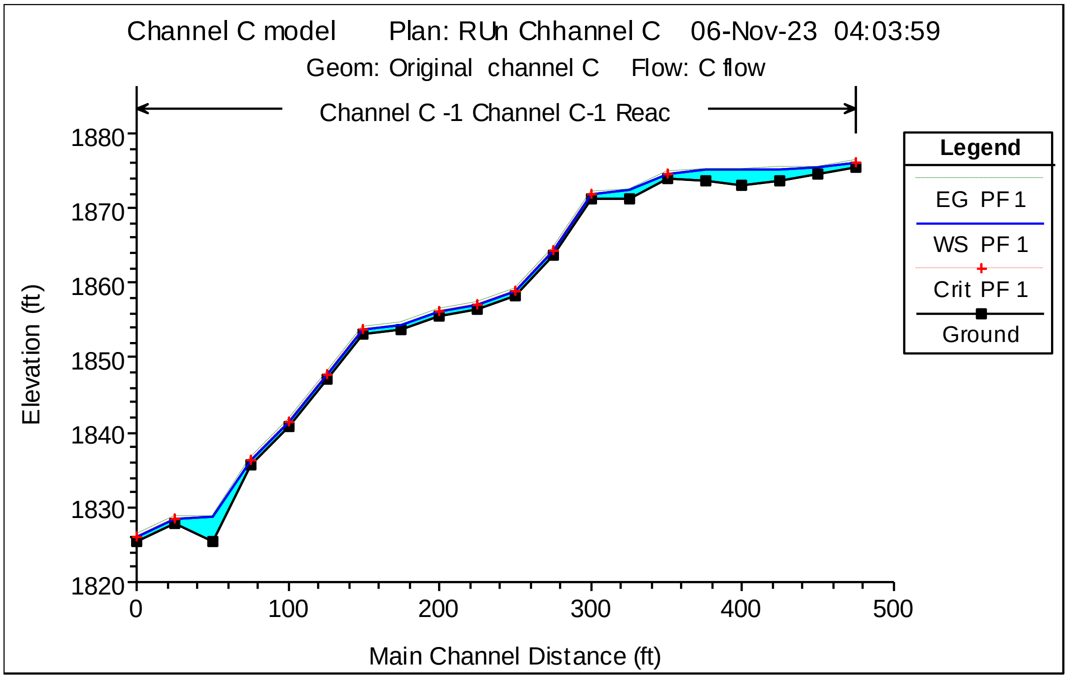


Figure 31:Channel C arsema sefer from 10-100 yr storm depth crosection at up stream



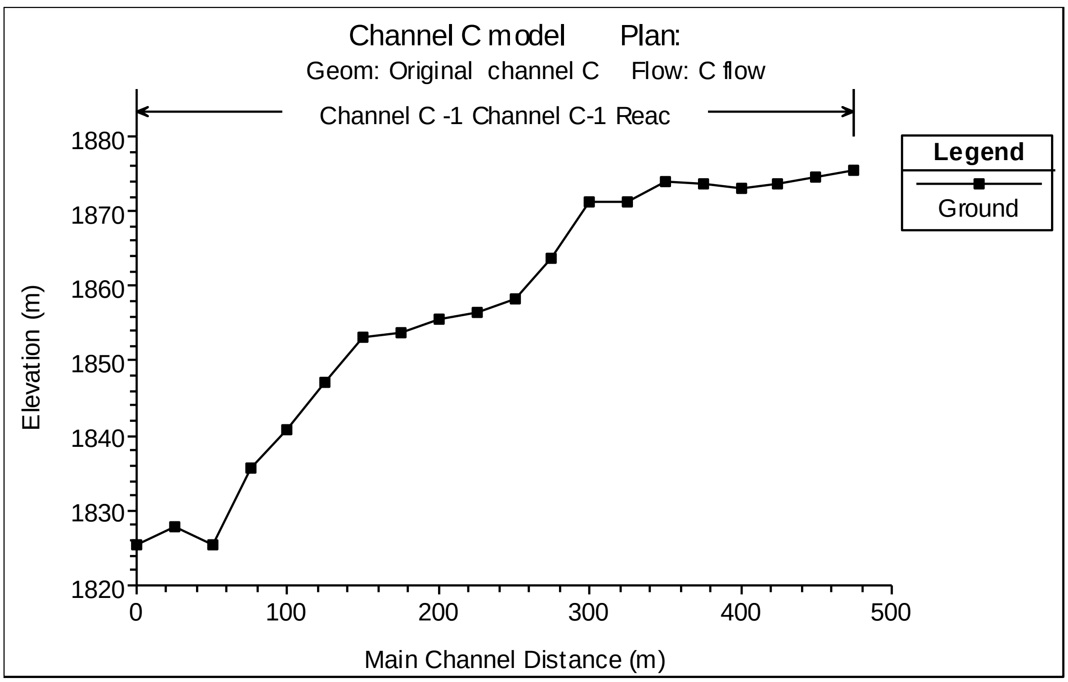


Figure 32Channel C longitudinal crosection over floofing obcserved

From the figure above shown that chanel c capacity to carry out the runoff was insufficieAnd from the figure the linear increament of the flow and depth ditch help us to estimate the accurate return periode depth to protect overflooding. the upstream level of the channel,making deep the dich better than the downstream favours normal flow.

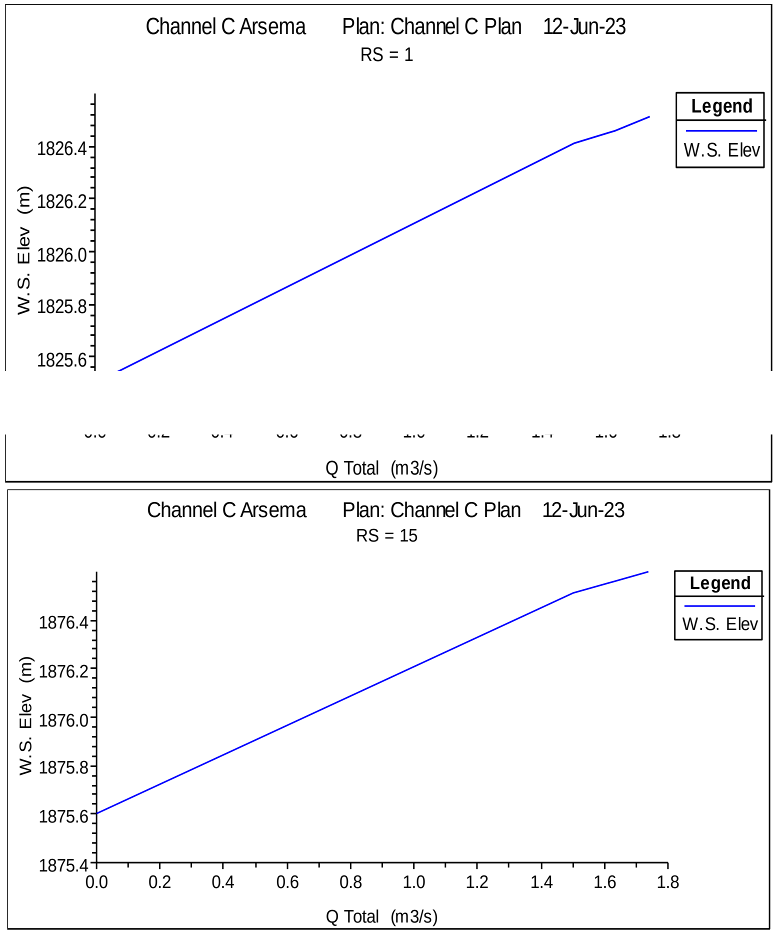
4.3.4 Drainage Ditch Profile ,Rating Curve,profile and Detailed out put results of Channel C(aresema sefer)

The table in appendix 8 under shows the results based on the input datas in the situ for ten years period and profile summary table for 10 to 100 years consquetivly.

the result of water overflow in all 10 stations. From the table, it can be concluded that the maximum level of water overflow is 1.23 meter and the existing observed depth was 0.65m . the remaing 0,58m the overflow of water for each station and correction of depth required which mean 47,2 % of the channel hight must be added in order to accomodete the 10 year return periode flow.

This indicate that Channel C of Arsema sefer complete its design periode even if it constructed early 2012 periode of time ,and number consruction probems may mentioned during detaile design or research on the issue..

Appendix 8 Table 28 below annexed of Channel C Arsema Sefers ,shows the result of cross sectional output in all stations. From the table , it can be concluded that the cross section end points



a)Downstream rating curve b)up stream Rating Curve

Figure 34 Channel C Profile and Rating Curve

Various element correction and the existing drainage dich was under capacity to convey flow in side in it.

From the Appendix – 8 table 29 detailed out put of channel C shown for 100 year return perode the required maximum depth and the observed and measured esisting channel dpth was compared and problem of flooding is identified as limitation on depth of the channen to carryout the runoff of the catchemnt .Also ,for sustainability of the channel,velocity of flow considerd and above the limit,and slope of the channel should be modified.

From the above figures, we can see that different stations have different water level or flooding status when rainfalls, this is because the cross section differs at each stations and also the catchment area increases at the downstream stations. We can see that at the upstream stations the water level is within the ditch or cross section of the ditch that is it is below 1 m, however further downstream the water spills above the ditch and flood the area.

4.4 Performance Capacity Evaluation and The Sub- Catchment peak Flow

4.4.1 performance of the drainage ditch in in the selected area

From the indicators idetifiea which are the couses for flooding and overtoping ,described as under the table

***Table 17*:Performance of Drainage Ditch of the selected area**

Performance domains	Performance
Hydraulic performance (At tenatabiya sefer) Drainage Ditch A	Flow attenuation at the outlet damaged
	Volume reduction at the outlet
	Lag-time very high
	High Overflow frequency
	high Drainage duration frequency
Drainage duration frequency(at Channel B)	high in mean annual runoff volume increase to natural volume
	High Runoff frequency
	high model volume of base flow and the volume of storm water incresed

Economic aspects	Require high design reriew Budget,
	High Construction costs
	Operational costs
Lifespan and long-term effectiveness	short time functionalities
	Monitoring and maintenance check-list
Social aspects	All tenatabiya and Arsema sefer residents complained to construct size and quality wise for drainage structures
	The town solid wastes driven by the storm and deposited there and pollute both communities.

From the above table - 18 and Table -19,the capacity of existing drainage channels calculated using manning formula and subcachment peak discharge calculated using rational method . the under specified table stipulate the subcatchment peak flood as shown.

Table 18:Proposed peak discharge in the sub catchment

Sr,No	Name of the catchemnt	Proposed Discharge			
		Q_{10}	Q_{25}	Q_{50}	Q_{100}
1	Sub Catchment A Tenatabiya Sefer	1.94	2,08	2.17	2.25
2	Sub Catchment B Tenatabiya Sefer	4.09	4.57	4,57	4.75
3	Sub Catchment C Tenatabiya Sefer	1,5	1.61	1.68	1.74

Existing drainage channel and carryibg capacity using manning equation sumerized as under the table

Table 19:Capacity of Existed Drainag Channel

Sr.No	Catchment Name	Sub Catchment A Channel	Sub catchment B Channel	Sub Catchment C Channel
	Drainge type by shape	Rectangular	Rectangular	Rectangular
1	Calculated Discharge from table 11 in(m^3/s)	1.66	1.32	0.34

From the computed proposed peak discharge which is stated in appendix table 31 and existed channel capacity of the drainage structure shown in the appendix table 32,the existed drainage structure are fully hydraulically insufficient and most of the drainage structure need rehabilitation rework.

From tha table under which has shown clearly the comparison between existed drainage structure and the proposed maximum design discharge on each sub catchment

Table 20:Over flow discharge

Sr.No	Catchment Name	Sub cat catchment Q(m^3/s)	Existed Q(m^3/s)	Proposed Discharge (m^3/s)				Difference in (m^3/s)			
				Q10	Q25	Q50	Q100	**Q10**	**Q25**	**Q50**	**Q100**
	Sub Catchment A	Q_A	1.66	1.94	2,08	2.17	2.25	**0.28**	**0.42**	**0.51**	**0.59**
	Sub Catchment B	Q_B	1.32	4.09	4.57	4,57	4.75	**2.77**	**3.25**	**3.35**	**3.43**
	Sub Catchment C	Q_C	0.34	1,5	1.61	1.68	1.74	**1.16**	**1.27**	**1.34**	**1.4**

Percentage of flood which overtoped and flow on the compound

Table 21:**Percentage of Over Flooding**.

Catchment	% of flood over flow			
	Q_{10} %	Q_{25} %	Q_{50} %	Q_{100} %
Sub Catchment A Tenatabiya Sefer	14.43	20.19	23.50	26.22
Sub Catchment BTenatabiya Sefer	67.73	71.12	71.73	72.21
Sub Catchment C Arsema Sefer	77.33	78.88	79.76	80.46

From the table degree of flooding more sound in the catchment that, Sub catchment C>Sub catchment B>Sub catchment A. This implies that the selected study area affected by flooding during maximum rainfall precipitation and hydraulically not perform to collect the runoff occurences fully and under capacity.

Due to this ,we are going to describe why this situation is coused and underlay possible indicators for the issue . The under table describe ,indicators of quantitative performance .

4.4.2 Major Probelems Associated with The Existing Drainage System

Major problems associated with welikite town drainage system is identifying the catchement storm capacity,selection of residential location, appropriate engineering design and consructionin in the problem area. The design period is under estimated due to manpower and budget. However ,capacity building to control the existing drainage channel gap is the other issue. In other wey round unpremeditated drainage ditches are going construct still. Blockage of chat leaf,disposal of dry sewer ,expansion of residential home,increamnet of percentage impermeability rise up,etc.

4.4.2.1 The Town Storm Drainage Lay out Guide Plan

Since the wey of treating the problem as we observe ,to have planned and studied document abscenece make the town to suffer a lot and the knowledge gap between the problem and the excuters make it undermine and senseless. They saw and they may see it to tie the hunds of the community economy by a yearly occurecce of bad and heavy rainfall at atime.

From this point of view, the town perspective to upgrade and solve the high flooding of the specified areas is very week and during the groundwork of this research paper, the town has not updated aguide plan for protection of tenatbiya and aresema areas.Lack of healthy and appropriate construction of drainage ditch problem and to build this structure,for the town ,similar work procedure as that of sister towns expriances must be shared (mobilizing fundrising team,proposal prparetion and submittal team for different donors, and the like,) and further reconiscence of the drainage dithes in addition to best management practices shall be selected.

4.4.2.2 Storm Drainage Management Condition

Tentabiya Sefer Drainage structures are looks disappointing,they are full of chate remainents,plastics, residential dry west,animal remainents after consmptions,and the like are the basic probelems ,in additions to the size of the channel to perform the runoff.

Ther are a UNIDP project office to construct the town storm conveyance, by constructing coble roads and road side drainage ditches. However every construction guided by simple method with out any studied document, and the town is avictim of acopy pest trend of documents application for constructiom.

4.5 Discussion

As we calculate and ilaborate the storm of drainage capacity of Tenatabiya Sefer and Arsema area,there need to be requair of re-construction as per the model result , to make vertical extention of all the drainage diches of sample taken.

Channel A,Channel B and C are selected better in size diches in the problem area , however after hydrologic and hydraulic examination , this drainage ditches are incapable to carry out the storm.

Detaile feasibility and engineering design documents of the town drainage system and the updated guide plan incorporated with masterplan are the town basic work to manege ,monitor and control the town storm drainage system.

Welikite Town is one of the town which is in the scarce of potable water ,but high storm during rainy season destroy afew residentials, road network,vulnerable area community properties,and newly constructed earthen and coble stone roads.

For the town drainage network, there need to be organized institution under the miniscipality to identify,plan and find budget source for healthy and economoic lively hood of the town residents.

Due to such find out and budget source,the town exercise best management prctices to reduce damage of storm drainage ditches from siltation,accumulation dry west, soil erosion and making safe the community from unnecessary flood damage and thire properties.

4.6 Best Management Practices (BMPs)

In storm water drainage system Best Management Practices are reducing the amount of impervious surface areas to reduce storm water runoff before it leaves the development site.

BMPs siting tool identifies potential suitable locations for implementing different type of of BMPs. It supports users with selecting suitable locations that meet the desired site suitability criteria, such as drainage area ,slope, hydrological soil group, ground water soil depth, road, stream and building buffers. It is designed to be used by anyone interested in reducing runoff from a property, including site developers, land scape architects and urban planners.

4.7 Benefit of Using Best Management Practics (BMPs)

In urban areas there are different methods to control flood before occurrence of exhaustive property damage, implementing the ways to control and there are benefits handed to the environment and social affairs

- Promote the natural movement of water instead of allowing it to wash into streets and down town drains
- Can blend in with land scape and infrastructure
- Can be infra structure, such as permeable parking lot Can provide aesthetic effect
- Can reduce the amount of pollutant entering the water collection system
- Can improve water quality and the water treatment process at water treatment plots

4.8 Wolikite Town Selected Best Management Practics (BMPs)

It was primarily drafted in 1972 as part of the Clean Water Act in the US and Canada to prevent pollution using structural approaches. It has a historical basis in the management of centralized storm water management systems (Fletcher et al., 2014) until it was matured to pollution prevention activities. BMPs encompasses both non-structural and structural measure including schedules of activities, prohibitions of practices, maintenance procedures and practices to control plant site runoff, leaks, and sludge disposal (USEPA, 2011).

The best management practice for welikite town be assumend that acclimate change mitigation measures that control overflooding the town in the specific area due to deafforestation, bad behavior of dry waste disposal and the bearing of soil characterstics, need an over look. From this point, to encourage infiltration and locally control the flow rates and volumes for frequent events. Subsequently, these trends have profound implications for the planning, design, and financial and operational aspects of drainage networks. In a context of climate change conditions the urban drainage system may change given the spatiotemporal changes expected in precipitation. An intensification of extreme events will result in a higher flood risk in urban areas and deterioration of the quality of receiving environments. It will be noted, as shown by the results presented in the detailed study of welikite flood estimation, that the impact of the increase of precipitation on the flows and volumes of runoff is not similar. Thus, for areas with land used for residential building, increases in flows rates and runoff volumes obtained by modeling are larger than those for precipitation.

Moreover, regardless of the method used to evaluate future climate in rainfall intensities, all impact studies reported in the literature using these intensities as input to a model of hydro- logic / hydraulic simulation of urban drainage networks to estimate the impact of these increases on network flows and runoff volumes. It should be noted, in addition to expected variations in rainfall intensities, some studies also consider the potential growth of urban areas in the coming decades ,growth areas connected to the network and increase the per- centage of impervious drained areas.

4.9 Possible Measures

Since the calculated and model of runoff relatively large in welikite town, so it is necessary to context certain measures that can mitigate these changes. These measures can take many forms and can be considered as part of a control function as illustrated in Figure 35, with interventions that may be closer to the source and others in the drainage networks .

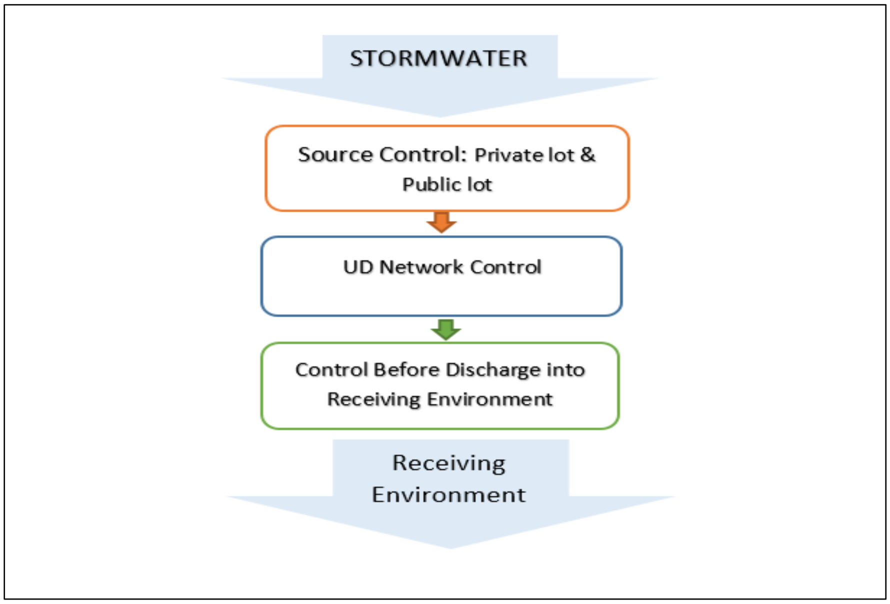


Figure 35. Runoff control mechanisms chain (adapted from GGEP).

These measures must also be defined taking into account a number of principles for the management of stormwater in urban areas.Control of stormwater is often less for residential areas (especially when there is no hydraulic undercapacity of existing networks downstream) where it is now more common for large paved areas that are associated with commercial and industrial sectors. As the increases of precipitation, can be anticipated for residential areas are sub- stantial, we will include more controls to be able to offset these increases . A re definition of urban planning involv development concepts, landscaping and design engineer of drainage systems will have to be better disseminated in practice. In this context, the source controls are no longer an alternative technique but should be a basic approach for new networks to compensate for the longer term effects of climate change, even when there is no network over- load problem observed for current conditions.

Frequent flows (return period of 2 years and 5 years as shown in IDF or slightly less)are typically associated with erosion in the earthen channels as these rates are known to contribute substantially to the definition of the hydraulic section of arrund drainage channel area. The increase in these rates can therefore result in a significant increase in erosion and degradation of ecosystems, especially as for residential areas, runoff flow can increase by in large percentage.

On the other hand, the types of measures that can be considered will necessarily differ depending on the case with an existing network or a planned network. In the case of a planned network, it is obviously easier to adequately plan networks with controls that are needed, so that increasing flow and volumes can be compensated. Most of the measures included in the diagram in Figure 1 can be applied alone or in combination, keeping in mind that it is often cheaper to control the flow and volumes as possible upstream, near the source.

Practices (BMPs) to control stormwater at source and with a more comprehensive approach involving better integration of green spaces in urban areas.Regarding the design criteria, they are fundamentally attached to the selected return periods and rainfall data. Two approaches can be selected as adaptation measures.

- If we continue to use current information from the IDF (Intensity-Duration-Frequency) curves can be used for designing a longer return period. For example, the return period that is most often used for network design using a traditional approach in 5 years. To maintain this level of service, it is therefore appropriate to raise this return period to 10 years, if current rainfall data are used for design. So, if you want to keep a level of service from 1 in 5 years to a situation envisaged in 50 years (so below the useful life of networks) could take (based on current data) return period 10 years to the design. Similarly, the retention volumes expected for different return periods should be increased.

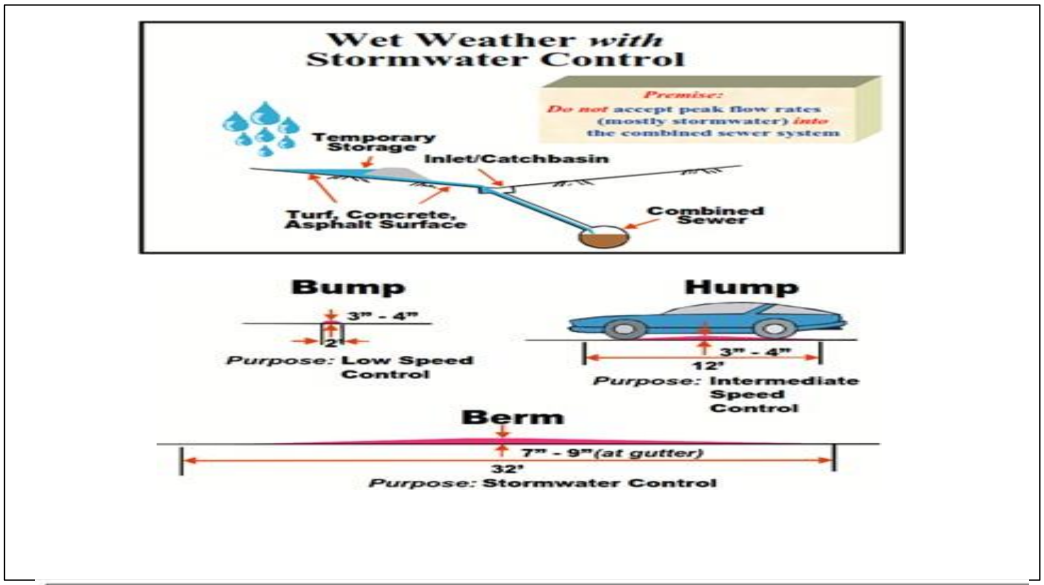
- Another approach would be to consider, case by case, rain percentage increases that have been defined by the HECRAS model shown.

4.10 Sorce Control Techniques

Several types of source control can be considered appropriate for different types of urban areas. For commercial, industrial and institutional sectors, retention of local areas can easily be arranged, either on the parking areas or sub parking (Fig- 36). For residential areas, controls may be made on private lots or on public properties (ie the streets)On private lots, the most interesting measures are adding absorbent soil or using infiltration facilities (e.g. bioretention areas) that are designed to retain runoff and provide time for it to infiltrate. This

Figure 36 Example of planning for parking area and road sides

type of control will help to eliminate the direct runoff by maintaining the capacity of infiltration and evaporation in impervious surfaces (Figure 36) on development parcels (driveways, parking lots) and roads (pavedOn the streets, a redevelopment like those shown in Figures 5 will allow a certain level of control for the more frequent rains. For areas with streets with low longitudinal slopes, we can also use the volumes available at low points with sumps to control which may limit the entry of water to the network (Walesh, 1999) used this approach, by adding berms on the pavement, so as to create larger storage basins .Figures 37 illustrate these approaches. However, in this type of application, it is ne- cessary to make field validations to ensure that problems



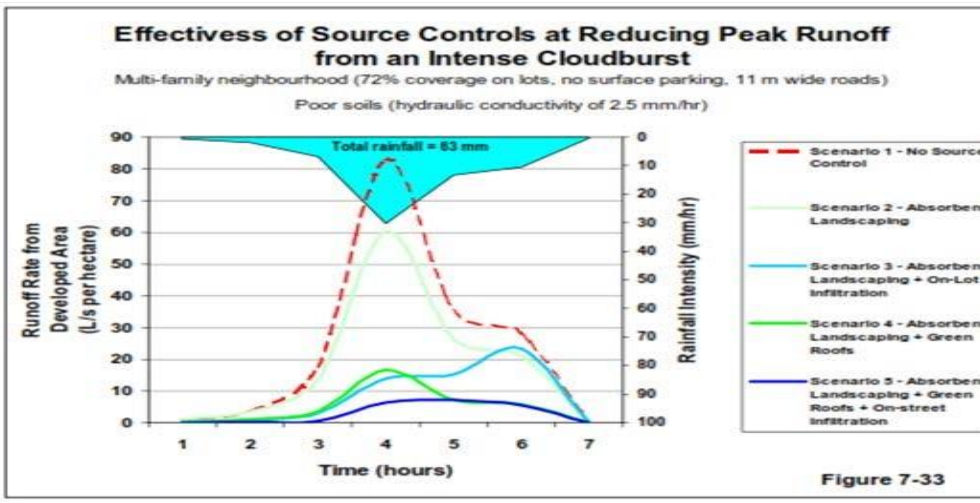


Figure 37 :Effectiveness of source controls at reducing peak runoff from intense cloudburst

5 CONCLUSIONS AND RECOMMENDATION

5.1 Conclusion

Storm water drainage problem of welkite town had been perceived as acause of over flooding in two identified sub kebeles (Tenatabiya and Arsema sefer). Therefor,to remove the runoff ,it is possible to evaluate and assess the performance of the existing drainage system in the problem area. The objective of this thesiss was ,to evaluate the capacity of existing urban storm water drainage system,to determine the hydraulic performance of the existing storm drainage system and to identify the major problems associate in hydraulic performance of the storm drainage of Welikite town Bekur Sub city, Tenatabiya Sefer and Arsema). HEC-RAS model is used to identify water profile elevation toward the channel (A,B and C) and to over see the performance capacity of the channel minimum and maximum water level whether over flooded or not. For calculleted result of the runoff using manning formula channel A found as 1.66 mcu/s ,Channel B found as 1.32 mcu/s and channel C found o.34 mcu /s and for design period 25 year return periode, for the sub basine drainage area using rational method, drainage area as the out fall of channel A found 2.08 mcu/s , drainage area as the out fall of channel B found 4.57 mcu/s and drainage area as the out fall of channel C found 1.61 mcu/s.The difference of discharge,which is overfloded found in channel A and Channel B 0.42 mcu/sec and 3.25mcu/sec consecutively and channel C 1.27mcu.sec.The HEC- RAS simulation model shows that all channels are in capable and above the maximum flow level and over flooding the compound in flooding percent of 20.19%,71.12% and 78.88% channel A,B and C in order0all channel dimentions are not sufficient to perform the generated runoff from the subcatchment.From the field survey,rational method and HEC-RAS simulation result ,the existing storm water drainage system of welikite town (Specially Bekur Sub Town) generally can be considered inadequate both quality as well as coverage to carry out the runoff , unstudied and provided size of existing channels are constructed with out any input data and problem of solid waste disposal of the town require serious look on its managment.

5.2 Recommendation

In order to alleviate the problems that has been hindering the drainage systems in this study area, the following recommendations are made for better and sustainable urban storm water drainage system for Welikite Town Storm Drainage System.

1. **1.** Create awareness within the community to use the drainage systems in a way that the drainage systems could be able to serve as their life span and the community should also know how to manage solid and liquid wastes.
2. **2.** The different Governmental and non-governmental organization, who are involved in storm water drainage and related activities, should work in consultation and preparation of with the municipality by feasibility and detaile design report with budget source.
3. **3.** Future expansion plan of the city should be done by take in to account identified locations for storm water prone areas by resettling the resisdents and practicing best Mnagement systems (e.g. designing flood control techniques and land fill processing for the dry waste) and make new residential settlements free of flood pron locations.
4. **4.** Detail feasibility and design study should be done before approval of the drainage canal construction during early stage of the project.
5. **5.** Supervision of drainage canals during construction should be mandatory to avoid wrong dimensions and quality of the work as a whole.
6. **6.** Capacity building aboute dry disposal system for different insttutuins,hotels,and any marketing areas are shall be practiced
7. **7.** In addition to this finding, future studies are recommended to include sustainable urban drainage system (SuDS).

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APPENDICES

Appendix 1:Quantile Test For The Most Commonly Used Distribution

T	L_T	NRM	EVI	EXP	GAM	LN2	LOG	LN3	PE3	LP3	GPAR	U_T
2	34.29	34.50	32.47	33.42	33.72	34.32	34.50	34.41	33.72	33.93	32.18	72.01
5	35.79	37.45	34.89	36.64	36.95	37.38	37.18	37.76	36.95	35.96	34.52	158.88
10	36.16	39.00	36.49	39.07	39.15	39.09	38.75	40.05	39.15	37.41	35.75	187.02
20	36.36	40.28	38.03	41.51	41.27	40.56	40.20	42.29	41.27	38.86	36.65	202.51
25	36.40	40.65	38.51	42.29	41.94	41.00	40.65	43.01	41.94	39.33	36.89	205.82
50	36.49	41.71	40.02	44.73	44.01	42.28	42.03	45.25	44.01	40.82	37.49	212.60
100	**36.52**	**42.67**	**41.51**	**47.16**	**46.04**	**43.47**	**43.40**	**47.53**	**46.04**	**42.34**	**37.94**	215.55
200	36.53	43.55	42.99	49.60	48.07	44.58	44.75	49.86	48.07	43.91	38.26	215.93
500	36.50	44.61	44.95	52.81	50.74	45.97	46.53	53.03	50.74	46.07	38.56	213.78
1000	36.46	45.35	46.43	55.25	52.75	46.97	47.87	55.51	52.75	47.77	38.72	210.81
2000	36.42	46.06	47.91	57.68	54.77	47.94	49.22	58.06	54.77	49.53	38.84	207.05
5000	36.34	46.93	49.87	60.90	57.45	49.17	50.99	61.56	57.45	51.97	38.95	201.28

10000	36.28	47.56	51.35	63.34	59.47	50.07	52.33	64.30	59.47	53.90	39.01	196.52
100000	36.07	49.48	56.27	71.43	66.27	52.92	56.79	74.07	66.27	60.89	39.11	179.90

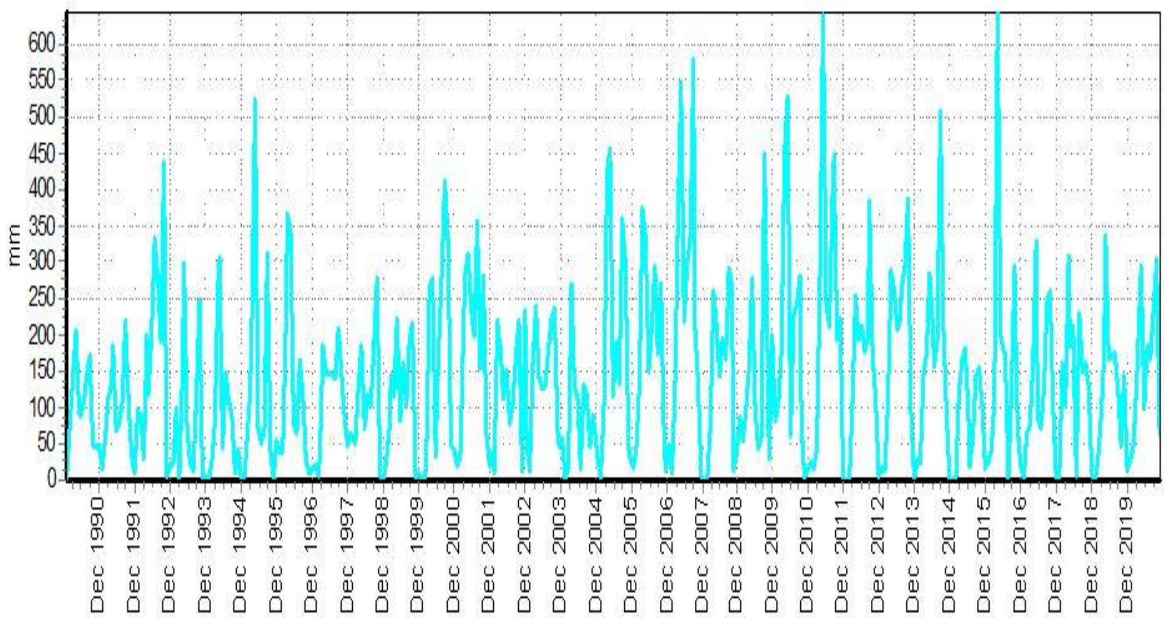
Appendix 2: Annual Daily maximum rainfall data at the selected stations (Wolikite)

1991	62.1
1992	79.4
1993	56.3
1994	48.3
1995	83.7
1996	80.6
1997	33.1
1998	33.7
1999	65.4
2000	42.5
2001	53
2002	51
2003	39.4
2004	38.9
2005	39
2006	40.7
2007	38.5
2008	58
2009	67.3
2010	45
2011	43.5
2012	53.2
2013	58.1

2014	65.5
2015	52.5
2016	40
2017	19.5
2018	39
2019	47
2020	19.5
2021	64

***Appendix* 3:** Maximum daily rainfall data series computation of Welkite station

Year	Yi	logYi	Rank	$(Yi - \bar{Y})^2$	$(Yi - \bar{Y})^3$	$(Yi - \bar{Y})^4$
1991	62	4.13	3	140.46	1664.69	19729.22
1992	79	4.37	4	849.82	24773.52	722188.14
1993	56	4.03	5	36.62	221.62	1341.17
1994	48	3.88	6	3.80	-7.40	14.41
1995	84	4.43	7	1119.01	37432.70	1252184.29
1996	81	4.39	8	921.22	27960.53	848647.04
1997	33	3.5	9	294.07	-5042.78	86475.51
1998	34	3.52	10	273.85	-4531.76	74993.34
1999	65	4.18	11	229.57	3478.38	52703.02
2000	43	3.75	12	60.04	-465.19	3604.50
2001	53	3.97	13	7.57	20.83	57.33
2002	51	3.93	14	0.56	0.42	0.32
2003	39	3.67	15	117.69	-1276.72	13850.35
2004	39	3.66	16	128.79	-1461.51	16585.81
2005	39	3.66	17	126.53	-1423.22	16008.88
2006	41	3.71	18	91.17	-870.54	8312.28
2007	39	3.65	19	138.02	-1621.57	19050.79
2008	58	4.06	20	60.09	465.78	3610.51
2009	67	4.21	21	290.76	4957.88	84539.93
2010	45	3.81	22	27.55	-144.57	758.76
2011	44	3.77	23	45.54	-307.33	2073.96
2012	53	3.97	24	8.71	25.71	75.90
2013	58	4.06	25	61.65	484.03	3800.45
2014	66	4.18	26	232.61	3547.70	54108.20
2015	53	3.96	27	5.07	11.42	25.70
2016	40	3.69	28	105.03	-1076.38	11031.18
2017	20	2.97	29	945.46	-29071.47	893900.87
2018	39	3.66	30	126.53	-1423.22	16008.88
2019	47	3.85	31	10.55	-34.28	111.35
2020	20	2.97	32	945.46	-29071.47	893900.87
2021	64	4.16	33	189.11	2600.52	35761.40


mm
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50
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150
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250
300
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Dec 2014
Dec 2015
Dec 2016
Dec 2017
Dec 2018
Dec 2019

***Appendix* 4**:*Chai Squared Goodness Tabuleted Report* Chai Squared Goodness Tabuleted Report

X-Square test for All data	**a=1%**	**a=5%**	**a=10%**	**Attained a**	**Pearson Param.**
Normal	REJECT	REJECT	REJECT	0.02%	21.8
Normal (L-Moments)	REJECT	REJECT	REJECT	0.18%	17.1333
LogNormal	REJECT	REJECT	REJECT	0.00%	25.5333
Galton	ACCEPT	REJECT	REJECT	4.08%	8.26667
Exponential	REJECT	REJECT	REJECT	0.15%	17.6
Exponential (L-Moments)	ACCEPT	ACCEPT	REJECT	8.23%	8.26667
Gamma	REJECT	REJECT	REJECT	0.01%	23.2
Pearson III	ACCEPT	REJECT	REJECT	2.67%	9.2
Log Pearson III	ACCEPT	REJECT	REJECT	4.08%	8.26667
EV1-Max (Gumbel)	ACCEPT	REJECT	REJECT	1.16%	12.9333
EV2-Max	ACCEPT	ACCEPT	REJECT	9.92%	7.8
EV1-Min (Gumbel)	REJECT	REJECT	REJECT	0.00%	25.5333
EV3-Min (Weibull)	REJECT	REJECT	REJECT	0.01%	23.2
GEV-Max	ACCEPT	ACCEPT	ACCEPT	11.49%	5.93333
GEV-Min	ACCEPT	REJECT	REJECT	2.67%	9.2
Pareto	ACCEPT	REJECT	REJECT	2.67%	9.2
GEV-Max (L-Moments)	ACCEPT	ACCEPT	ACCEPT	17.18%	5
GEV-Min (L-Moments)	ACCEPT	REJECT	REJECT	4.08%	8.26667
EV1-Max (Gumbel, L-Moments)	ACCEPT	ACCEPT	ACCEPT	17.12%	6.4
EV2-Max (L-Momments)	ACCEPT	ACCEPT	ACCEPT	17.12%	6.4
EV1-Min (Gumbel, L-Moments)	REJECT	REJECT	REJECT	0.08%	19
EV3-Min (Weibull, L-Moments)	REJECT	REJECT	REJECT	0.08%	19
Pareto (L-Moments)	ACCEPT	REJECT	REJECT	2.67%	9.2
GEV-Max (kappa specified)	ACCEPT	ACCEPT	ACCEPT	20.42%	5.93333
GEV-Min (kappa specified)	REJECT	REJECT	REJECT	0.01%	23.2
GEV-Max (kappa specified, L-Moments)	ACCEPT	ACCEPT	ACCEPT	20.42%	5.93333
GEV-Min (kappa specified, L-Moments)	REJECT	REJECT	REJECT	0.34%	15.7333

Appendix 5:**Kolmognov-Smirnov goodness to fit test report**

Kolmogorov-Smirnov test for:All data	**a=1%**	**a=5%**	**a=10%**	**Attained a**	**DMax**
Normal	ACCEPT	REJECT	REJECT	1.14%	0.28613
Normal (L-Moments)	ACCEPT	REJECT	REJECT	1.39%	0.28049
LogNormal	ACCEPT	REJECT	REJECT	3.34%	0.25383
Galton	ACCEPT	ACCEPT	ACCEPT	40.35%	0.1556
Exponential	ACCEPT	ACCEPT	ACCEPT	14.13%	0.20285
Exponential (L-Moments)	ACCEPT	ACCEPT	ACCEPT	41.83%	0.1537
Gamma	ACCEPT	REJECT	REJECT	2.38%	0.26451
Pearson III	ACCEPT	ACCEPT	ACCEPT	51.72%	0.14181
Log Pearson III	ACCEPT	ACCEPT	ACCEPT	74.09%	0.11722
EV1-Max (Gumbel)	ACCEPT	ACCEPT	ACCEPT	10.18%	0.21547
EV2-Max	ACCEPT	ACCEPT	ACCEPT	22.74%	0.18299
EV1-Min (Gumbel)	REJECT	REJECT	REJECT	0.07%	0.35561
EV3-Min (Weibull)	REJECT	REJECT	REJECT	0.30%	0.32162
GEV-Max	ACCEPT	ACCEPT	ACCEPT	31.91%	0.1674
GEV-Min	ACCEPT	ACCEPT	ACCEPT	40.28%	0.1557
Pareto	ACCEPT	ACCEPT	ACCEPT	32.18%	0.16699
GEV-Max (L-Moments)	ACCEPT	ACCEPT	ACCEPT	94.68%	0.0883
GEV-Min (L-Moments)	ACCEPT	ACCEPT	ACCEPT	81.68%	0.10839
EV1-Max (Gumbel, L-Moments)	ACCEPT	ACCEPT	ACCEPT	11.48%	0.21092
EV2-Max (L-Momments)	ACCEPT	ACCEPT	ACCEPT	22.29%	0.18388
EV1-Min (Gumbel, L-Moments)	REJECT	REJECT	REJECT	0.09%	0.35076
EV3-Min (Weibull, L-Moments)	REJECT	REJECT	REJECT	0.28%	0.32344
Pareto (L-Moments)	REJECT	REJECT	REJECT	0.04%	0.37101
GEV-Max (kappa specified)	ACCEPT	ACCEPT	ACCEPT	28.58%	0.17263

GEV-Min (kappa specified)	REJECT	REJECT	REJECT	0.34%	0.31888
GEV-Max (kappa specified, L-Moments)	ACCEPT	ACCEPT	ACCEPT	30.23%	0.17
GEV-Min (kappa specified, L-Moments)	REJECT	REJECT	REJECT	0.42%	0.31335

Appendix 6: Welikite Station IDF Tabulated result by pearson type III

		5 year	10 year	25 year	50 year	100 year	200 year
	24hr RF	61.90	71.78	83.06	90.76	97.97	104.81
	0.5hr MRF	0.846	0.804	0.758	0.729	0.703	0.680
10	0.167	172.17	180.38	187.87	192.13	195.62	198.57
20	0.333	132.26	142.61	152.46	158.23	163.07	167.22
30	0.500	104.69	115.38	125.95	132.31	137.74	142.47
40	0.667	85.16	95.39	105.82	112.26	117.83	122.75
50	0.833	70.98	80.43	90.32	96.55	102.02	106.91
60	1.000	60.42	69.02	78.20	84.09	89.33	94.05
70	1.167	52.38	60.15	68.60	74.09	79.03	83.53
80	1.333	46.11	53.14	60.88	65.97	70.59	74.83
90	1.500	41.11	47.49	54.59	59.30	63.60	67.58
100	1.667	37.07	42.88	49.40	53.75	57.75	61.47
110	1.833	33.73	39.05	45.06	49.09	52.82	56.29
120	2.000	30.93	35.84	41.39	45.13	48.60	51.85
130	2.167	28.56	33.10	38.25	41.74	44.98	48.03
140	2.333	26.52	30.75	35.55	38.81	41.84	44.70
150	2.500	24.76	28.71	33.20	36.25	39.10	41.78
160	2.667	23.21	26.91	31.13	34.00	36.68	39.21
170	2.833	21.85	25.33	29.31	32.01	34.54	36.93
180	3.000	20.63	23.93	27.68	30.24	32.63	34.90
190	3.167	19.55	22.67	26.23	28.65	30.92	33.07
200	3.333	18.57	21.53	24.92	27.22	29.38	31.43
210	3.500	17.69	20.51	23.73	25.93	27.99	29.94
220	3.667	16.88	19.58	22.65	24.75	26.72	28.58
230	3.833	16.15	18.73	21.67	23.68	25.56	27.34
240	4.000	15.47	17.95	20.76	22.69	24.49	26.20

Appendix 7**: Model Tabuleted Result Crossection A**

Channel A Tenatabiya Sefer Detailed Out put and profile summary table for !0 year

Cross Section Output

File Type Options Help

River: joka front Profile: 10 yr

Reach joka reach chann RS: 2 Plan: 333--3

Plan: 333--3 joka front joka reach chann RS: 2 Profile: 10 yr

E.G. Elev (m)	2.79
Vel Head (m)	0.14
W.S. Elev (m)	2.65
Crit W.S. (m)	2.28
E.G. Slope (m/m)	0.001863
Q Total (m3/s)	1.94
Top Width (m)	1.10
Vel Total (m/s)	1.68
Max Chl Dpth (m)	1.05
Conv. Total (m3/s)	44.9
Length Wtd. (m)	50.00
Min Ch El (m)	1.60
Alpha	1.00
Frctn Loss (m)	0.15
C & E Loss (m)	0.02

Element	Left OB	Channel	Right OB
Wt. n-Val.		0.013	
Reach Len. (m)	50.00	50.00	50.00
Flow Area (m2)		1.15	
Area (m2)		1.15	
Flow (m3/s)		1.94	
Top Width (m)		1.10	
Avg. Vel. (m/s)		1.68	
Hydr. Depth (m)		1.05	
Conv. (m3/s)		44.9	
Wetted Per. (m)		3.20	
Shear (N/m2)		6.59	
Stream Power (N/m s)		11.09	
Cum Volume (1000 m3)		0.05	
Cum SA (1000 m2)		0.06	

Errors, Warnings and Notes

Warning:	The velocity head has changed by more than 0.5 ft (0.15 m). This may indicate the need for additional cross sections.
Warning:	The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

Select Profile

Appendix 8:Summary of Result Chnnnel A

HEC-RAS Plan: 333--3 River: joka front Reach: joka reach chann Profile: 10 yr

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
joka reach chann	14	10 yr	1.94	1.80	4.23		4.36	0.003117	1.60	1.22	0.50	0.33
joka reach chann	13	10 yr	1.94	1.80	4.03		4.19	0.003730	1.74	1.12	0.50	0.37
joka reach chann	12	10 yr	1.94	1.80	3.78		3.97	0.004857	1.96	0.99	0.50	0.45
joka reach chann	11	10 yr	1.94	1.80	2.95	2.95	3.53	0.015903	3.38	0.57	0.50	1.01
joka reach chann	10	10 yr	1.94	1.60	3.16		3.22	0.000719	1.13	1.71	1.10	0.29
joka reach chann	9	10 yr	1.94	1.60	3.12		3.19	0.000766	1.16	1.67	1.10	0.30
joka reach chann	8	10 yr	1.94	1.60	3.07		3.15	0.000821	1.20	1.62	1.10	0.32
joka reach chann	7	10 yr	1.94	1.60	3.02		3.10	0.000888	1.24	1.57	1.10	0.33
joka reach chann	6	10 yr	1.94	1.60	2.97		3.06	0.000972	1.29	1.51	1.10	0.35
joka reach chann	5	10 yr	1.94	1.60	2.91		3.00	0.001081	1.34	1.44	1.10	0.37
joka reach chann	4	10 yr	1.94	1.60	2.84		2.95	0.001231	1.42	1.37	1.10	0.41
joka reach chann	3	10 yr	1.94	1.60	2.76		2.88	0.001457	1.52	1.28	1.10	0.45
joka reach chann	2	10 yr	1.94	1.60	2.65	2.28	2.79	0.001863	1.68	1.15	1.10	0.52
joka reach chann	1	10 yr	1.94	1.60	2.28	2.28	2.62	0.005571	2.60	0.75	1.10	1.01

Appendix 9:Channel A Profile Out Put Result For 10-100 Year Return Periode

:Channel A Profile Out Put Result For 10-100 Year Return Periode.

HEC-RAS Plan: 333--3 River: joka front Reach: joka reach chann

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
joka reach chann	14	10 yr	1.94	1.80	4.23		4.36	0.003117	1.60	1.22	0.50	0.33
joka reach chann	14	25 yr	2.08	1.80	4.34		4.48	0.003259	1.64	1.27	0.50	0.33
joka reach chann	14	50 yr	2.17	1.80	4.41		4.55	0.003348	1.66	1.31	0.50	0.33
joka reach chann	14	100 yr	2.25	1.80	4.47		4.62	0.003426	1.68	1.34	0.50	0.33
joka reach chann	13	10 yr	1.94	1.80	4.03		4.19	0.003730	1.74	1.12	0.50	0.37
joka reach chann	13	25 yr	2.08	1.80	4.14		4.30	0.003897	1.78	1.17	0.50	0.37
joka reach chann	13	50 yr	2.17	1.80	4.20		4.37	0.004003	1.81	1.20	0.50	0.37
joka reach chann	13	100 yr	2.25	1.80	4.26		4.43	0.004095	1.83	1.23	0.50	0.37
joka reach chann	12	10 yr	1.94	1.80	3.78		3.97	0.004857	1.96	0.99	0.50	0.45
joka reach chann	12	25 yr	2.08	1.80	3.87		4.07	0.005072	2.01	1.03	0.50	0.45
joka reach chann	12	50 yr	2.17	1.80	3.92		4.14	0.005206	2.04	1.06	0.50	0.45
joka reach chann	12	100 yr	2.25	1.80	3.97		4.19	0.005324	2.07	1.09	0.50	0.45
joka reach chann	11	10 yr	1.94	1.80	2.95	2.95	3.53	0.015903	3.38	0.57	0.50	1.01
joka reach chann	11	25 yr	2.08	1.80	3.00	3.00	3.61	0.016467	3.45	0.60	0.50	1.00
joka reach chann	11	50 yr	2.17	1.80	3.04	3.04	3.67	0.016837	3.50	0.62	0.50	1.01
joka reach chann	11	100 yr	2.25	1.80	3.07	3.07	3.71	0.017161	3.55	0.63	0.50	1.01
joka reach chann	10	10 yr	1.94	1.60	3.16		3.22	0.000719	1.13	1.71	1.10	0.29
joka reach chann	10	25 yr	2.08	1.60	3.22		3.29	0.000749	1.16	1.79	1.10	0.29
joka reach chann	10	50 yr	2.17	1.60	3.27		3.34	0.000768	1.18	1.83	1.10	0.29
joka reach chann	10	100 yr	2.25	1.60	3.30		3.38	0.000785	1.20	1.87	1.10	0.29
joka reach chann	9	10 yr	1.94	1.60	3.12		3.19	0.000766	1.16	1.67	1.10	0.30
joka reach chann	9	25 yr	2.08	1.60	3.18		3.25	0.000798	1.20	1.74	1.10	0.30
joka reach chann	9	50 yr	2.17	1.60	3.22		3.30	0.000818	1.22	1.79	1.10	0.30
joka reach chann	9	100 yr	2.25	1.60	3.26		3.34	0.000835	1.23	1.82	1.10	0.31
joka reach chann	8	10 yr	1.94	1.60	3.07		3.15	0.000821	1.20	1.62	1.10	0.32
joka reach chann	8	25 yr	2.08	1.60	3.14		3.21	0.000855	1.23	1.69	1.10	0.32
joka reach chann	8	50 yr	2.17	1.60	3.18		3.26	0.000876	1.25	1.73	1.10	0.32
joka reach chann	8	100 yr	2.25	1.60	3.21		3.29	0.000895	1.27	1.77	1.10	0.32

Appendix 10:Channel B Profile Out Put Result For 10-100 Year Return Periode

Channel B Profile Out Put Result For 10-100 Year Return Periode

HEC-RAS Plan: 333--3 River: joka front Reach: joka reach chann

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
joka reach chann	7	10 yr	1.94	1.60	3.02		3.10	0.000888	1.24	1.57	1.10	0.33
joka reach chann	7	25 yr	2.08	1.60	3.09		3.17	0.000925	1.27	1.63	1.10	0.33
joka reach chann	7	50 yr	2.17	1.60	3.12		3.21	0.000947	1.29	1.68	1.10	0.33
joka reach chann	7	100 yr	2.25	1.60	3.16		3.25	0.000967	1.31	1.71	1.10	0.34
joka reach chann	6	10 yr	1.94	1.60	2.97		3.06	0.000972	1.29	1.51	1.10	0.35
joka reach chann	6	25 yr	2.08	1.60	3.03		3.12	0.001011	1.32	1.57	1.10	0.35
joka reach chann	6	50 yr	2.17	1.60	3.07		3.16	0.001036	1.34	1.61	1.10	0.35
joka reach chann	6	100 yr	2.25	1.60	3.10		3.19	0.001057	1.36	1.65	1.10	0.36
joka reach chann	5	10 yr	1.94	1.60	2.91		3.00	0.001081	1.34	1.44	1.10	0.37
joka reach chann	5	25 yr	2.08	1.60	2.97		3.07	0.001124	1.38	1.51	1.10	0.38
joka reach chann	5	50 yr	2.17	1.60	3.00		3.10	0.001151	1.41	1.54	1.10	0.38
joka reach chann	5	100 yr	2.25	1.60	3.03		3.14	0.001174	1.43	1.58	1.10	0.38
joka reach chann	4	10 yr	1.94	1.60	2.84		2.95	0.001231	1.42	1.37	1.10	0.41
joka reach chann	4	25 yr	2.08	1.60	2.90		3.00	0.001279	1.46	1.43	1.10	0.41
joka reach chann	4	50 yr	2.17	1.60	2.93		3.04	0.001308	1.48	1.46	1.10	0.41
joka reach chann	4	100 yr	2.25	1.60	2.96		3.08	0.001334	1.50	1.50	1.10	0.41
joka reach chann	3	10 yr	1.94	1.60	2.76		2.88	0.001457	1.52	1.28	1.10	0.45
joka reach chann	3	25 yr	2.08	1.60	2.81		2.93	0.001511	1.56	1.33	1.10	0.45
joka reach chann	3	50 yr	2.17	1.60	2.84		2.97	0.001545	1.59	1.37	1.10	0.46
joka reach chann	3	100 yr	2.25	1.60	2.87		3.00	0.001575	1.61	1.40	1.10	0.46
joka reach chann	2	10 yr	1.94	1.60	2.65	2.28	2.79	0.001863	1.68	1.15	1.10	0.52
joka reach chann	2	25 yr	2.08	1.60	2.69		2.85	0.001929	1.73	1.20	1.10	0.53
joka reach chann	2	50 yr	2.17	1.60	2.72		2.88	0.001970	1.76	1.24	1.10	0.53
joka reach chann	2	100 yr	2.25	1.60	2.75		2.91	0.002006	1.78	1.26	1.10	0.53
joka reach chann	1	10 yr	1.94	1.60	2.28	2.28	2.62	0.005571	2.60	0.75	1.10	1.01
joka reach chann	1	25 yr	2.08	1.60	2.31	2.31	2.67	0.005672	2.66	0.78	1.10	1.00
joka reach chann	1	50 yr	2.17	1.60	2.33	2.33	2.70	0.005738	2.69	0.81	1.10	1.00
joka reach chann	1	100 yr	2.25	1.60	2.35	2.35	2.73	0.005798	2.73	0.83	1.10	1.00

Appendix 11:

: Model Tabuleted Result Crossection B Tenatabiya Sefer, Channel B Down Stream Detailed Report

Cross Section Output

File Type Options Help

River: Channel B Profile: 10 year

Reach Tenatabiya sefer RS: 425 Plan: 44-1

Plan: 44-1 Channel B Tenatabiya sefer RS: 425 Profile: 10 year

E.G. Elev (m)	62.30
Vel Head (m)	0.62
W.S. Elev (m)	61.68
Crit W.S. (m)	61.68
E.G. Slope (m/m)	0.008588
Q Total (m3/s)	4.09
Top Width (m)	0.95
Vel Total (m/s)	3.49
Max Chl Dpth (m)	1.23
Conv. Total (m3/s)	44.1
Length Wtd. (m)	
Min Ch El (m)	60.45
Alpha	1.00
Frctn Loss (m)	
C & E Loss (m)	

Element	Left OB	Channel	Right OB
Wt. n-Val.		0.013	
Reach Len. (m)			
Flow Area (m2)		1.17	
Area (m2)		1.17	
Flow (m3/s)		4.09	
Top Width (m)		0.95	
Avg. Vel. (m/s)		3.49	
Hydr. Depth (m)		1.23	
Conv. (m3/s)		44.1	
Wetted Per. (m)		3.42	
Shear (N/m2)		28.88	
Stream Power (N/m s)		100.85	
Cum Volume (1000 m3)			
Cum SA (1000 m2)			

Errors, Warnings and Notes

Appendix 12*:*

Summary Of Profile: Sumary of Profile

HEC-RAS Plan: 44-1 River: Channel B Reach: Tenatabiya sefer Profile: 10 year

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Tenatabiya sefer	444	10 year	4.09	60.45	63.25		63.37	0.001328	1.54	2.66	0.95	0.29
Tenatabiya sefer	441.89*	10 year	4.09	60.45	63.17		63.30	0.001410	1.58	2.59	0.95	0.31
Tenatabiya sefer	439.78*	10 year	4.09	60.45	63.09		63.23	0.001508	1.63	2.51	0.95	0.32
Tenatabiya sefer	437.67*	10 year	4.09	60.45	63.00		63.15	0.001626	1.69	2.43	0.95	0.34
Tenatabiya sefer	435.56*	10 year	4.09	60.45	62.91		63.06	0.001772	1.75	2.33	0.95	0.36
Tenatabiya sefer	433.44*	10 year	4.09	60.45	62.80		62.97	0.001961	1.83	2.23	0.95	0.38
Tenatabiya sefer	431.33*	10 year	4.09	60.45	62.67		62.86	0.002218	1.94	2.11	0.95	0.42
Tenatabiya sefer	429.22*	10 year	4.09	60.45	62.52		62.74	0.002598	2.08	1.97	0.95	0.46
Tenatabiya sefer	427.11*	10 year	4.09	60.45	62.32	61.68	62.59	0.003268	2.30	1.78	0.95	0.54
Tenatabiya sefer	425	10 year	4.09	60.45	61.68	61.68	62.30	0.008588	3.49	1.17	0.95	1.00

Appendix 13:

Detailed Out Put Channel B for 100 Year Return Periode: Detailed out put for Channel B for 100 Year Return Periode

Cross Section Output

File Type Options Help

River: Channel B Profile: 100 year

Reach Tenatabiya sefer RS: 425 Plan: 44-1

Plan: 44-1 Channel B Tenatabiya sefer RS: 425 Profile: 100 year

E.G. Elev (m)	62.50
Vel Head (m)	0.68
W.S. Elev (m)	61.82
Crit W.S. (m)	61.82
E.G. Slope (m/m)	0.009081
Q Total (m3/s)	4.75
Top Width (m)	0.95
Vel Total (m/s)	3.66
Max Chl Dpth (m)	1.37
Conv. Total (m3/s)	49.8
Length Wtd. (m)	
Min Ch El (m)	60.45
Alpha	1.00
Frctn Loss (m)	
C & E Loss (m)	

Element	Left OB	Channel	Right OB
Wt. n-Val.		0.013	
Reach Len. (m)			
Flow Area (m2)		1.30	
Area (m2)		1.30	
Flow (m3/s)		4.75	
Top Width (m)		0.95	
Avg. Vel. (m/s)		3.66	
Hydr. Depth (m)		1.37	
Conv. (m3/s)		49.8	
Wetted Per. (m)		3.68	
Shear (N/m2)		31.39	
Stream Power (N/m s)		114.83	
Cum Volume (1000 m3)			
Cum SA (1000 m2)			

Errors, Warnings and Notes

Warning:	The cross section had to be extended vertically during the critical depth calculations.
Warning:	The parabolic search method failed to converge on critical depth. The program will try the cross section slice/secant method to find critical depth.

Appendix 14:Summary of Profile 10-100 Year: Summaryof profile 10-100 Year

HEC-RAS Plan: 44-1 River: Channel B Reach: Tenatabiya sefer

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Tenatabiya sefer	444	10 year	4.09	60.45	63.25		63.37	0.001328	1.54	2.66	0.95	0.29
Tenatabiya sefer	444	25 year	4.57	60.45	63.45		63.58	0.001423	1.60	2.85	0.95	0.30
Tenatabiya sefer	444	50 year	4.67	60.45	63.49		63.63	0.001443	1.61	2.89	0.95	0.30
Tenatabiya sefer	444	100 year	4.75	60.45	63.53		63.66	0.001458	1.62	2.92	0.95	0.30
Tenatabiya sefer	441.89*	10 year	4.09	60.45	63.17		63.30	0.001410	1.58	2.59	0.95	0.31
Tenatabiya sefer	441.89*	25 year	4.57	60.45	63.37		63.51	0.001511	1.65	2.78	0.95	0.31
Tenatabiya sefer	441.89*	50 year	4.67	60.45	63.41		63.55	0.001532	1.66	2.81	0.95	0.31
Tenatabiya sefer	441.89*	100 year	4.75	60.45	63.44		63.59	0.001548	1.67	2.84	0.95	0.31
Tenatabiya sefer	439.78*	10 year	4.09	60.45	63.09		63.23	0.001508	1.63	2.51	0.95	0.32
Tenatabiya sefer	439.78*	25 year	4.57	60.45	63.28		63.43	0.001615	1.70	2.69	0.95	0.32
Tenatabiya sefer	439.78*	50 year	4.67	60.45	63.32		63.47	0.001637	1.71	2.73	0.95	0.32
Tenatabiya sefer	439.78*	100 year	4.75	60.45	63.35		63.50	0.001654	1.72	2.76	0.95	0.32
Tenatabiya sefer	437.67*	10 year	4.09	60.45	63.00		63.15	0.001626	1.69	2.43	0.95	0.34
Tenatabiya sefer	437.67*	25 year	4.57	60.45	63.19		63.35	0.001741	1.76	2.60	0.95	0.34
Tenatabiya sefer	437.67*	50 year	4.67	60.45	63.23		63.39	0.001764	1.77	2.64	0.95	0.34
Tenatabiya sefer	437.67*	100 year	4.75	60.45	63.26		63.42	0.001783	1.78	2.67	0.95	0.34
Tenatabiya sefer	435.56*	10 year	4.09	60.45	62.91		63.06	0.001772	1.75	2.33	0.95	0.36
Tenatabiya sefer	435.56*	25 year	4.57	60.45	63.08		63.25	0.001897	1.83	2.50	0.95	0.36
Tenatabiya sefer	435.56*	50 year	4.67	60.45	63.12		63.29	0.001922	1.84	2.54	0.95	0.36
Tenatabiya sefer	435.56*	100 year	4.75	60.45	63.15		63.32	0.001942	1.85	2.56	0.95	0.36
Tenatabiya sefer	433.44*	10 year	4.09	60.45	62.80		62.97	0.001961	1.83	2.23	0.95	0.38
Tenatabiya sefer	433.44*	25 year	4.57	60.45	62.97		63.15	0.002098	1.91	2.39	0.95	0.38
Tenatabiya sefer	433.44*	50 year	4.67	60.45	63.00		63.19	0.002126	1.93	2.42	0.95	0.39
Tenatabiya sefer	433.44*	100 year	4.75	60.45	63.03		63.22	0.002147	1.94	2.45	0.95	0.39
Tenatabiya sefer	431.33*	10 year	4.09	60.45	62.67		62.86	0.002218	1.94	2.11	0.95	0.42
Tenatabiya sefer	431.33*	25 year	4.57	60.45	62.83		63.04	0.002370	2.02	2.26	0.95	0.42
Tenatabiya sefer	431.33*	50 year	4.67	60.45	62.86		63.08	0.002402	2.04	2.29	0.95	0.42
Tenatabiya sefer	431.33*	100 year	4.75	60.45	62.89		63.10	0.002426	2.05	2.32	0.95	0.42

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Tenatabiya sefer	437.67*	10 year	4.09	60.45	63.00		63.15	0.001626	1.69	2.43	0.95	0.34
Tenatabiya sefer	437.67*	25 year	4.57	60.45	63.19		63.35	0.001741	1.76	2.60	0.95	0.34
Tenatabiya sefer	437.67*	50 year	4.67	60.45	63.23		63.39	0.001764	1.77	2.64	0.95	0.34
Tenatabiya sefer	437.67*	100 year	4.75	60.45	63.26		63.42	0.001783	1.78	2.67	0.95	0.34
Tenatabiya sefer	435.56*	10 year	4.09	60.45	62.91		63.06	0.001772	1.75	2.33	0.95	0.36
Tenatabiya sefer	435.56*	25 year	4.57	60.45	63.08		63.25	0.001897	1.83	2.50	0.95	0.36
Tenatabiya sefer	435.56*	50 year	4.67	60.45	63.12		63.29	0.001922	1.84	2.54	0.95	0.36
Tenatabiya sefer	435.56*	100 year	4.75	60.45	63.15		63.32	0.001942	1.85	2.56	0.95	0.36
Tenatabiya sefer	433.44*	10 year	4.09	60.45	62.80		62.97	0.001961	1.83	2.23	0.95	0.38
Tenatabiya sefer	433.44*	25 year	4.57	60.45	62.97		63.15	0.002098	1.91	2.39	0.95	0.38
Tenatabiya sefer	433.44*	50 year	4.67	60.45	63.00		63.19	0.002126	1.93	2.42	0.95	0.39
Tenatabiya sefer	433.44*	100 year	4.75	60.45	63.03		63.22	0.002147	1.94	2.45	0.95	0.39
Tenatabiya sefer	431.33*	10 year	4.09	60.45	62.67		62.86	0.002218	1.94	2.11	0.95	0.42
Tenatabiya sefer	431.33*	25 year	4.57	60.45	62.83		63.04	0.002370	2.02	2.26	0.95	0.42
Tenatabiya sefer	431.33*	50 year	4.67	60.45	62.86		63.08	0.002402	2.04	2.29	0.95	0.42
Tenatabiya sefer	431.33*	100 year	4.75	60.45	62.89		63.10	0.002426	2.05	2.32	0.95	0.42
Tenatabiya sefer	429.22*	10 year	4.09	60.45	62.52		62.74	0.002598	2.08	1.97	0.95	0.46
Tenatabiya sefer	429.22*	25 year	4.57	60.45	62.67		62.91	0.002774	2.17	2.11	0.95	0.46
Tenatabiya sefer	429.22*	50 year	4.67	60.45	62.70		62.94	0.002810	2.18	2.14	0.95	0.47
Tenatabiya sefer	429.22*	100 year	4.75	60.45	62.72		62.97	0.002838	2.20	2.16	0.95	0.47
Tenatabiya sefer	427.11*	10 year	4.09	60.45	62.32	61.68	62.59	0.003268	2.30	1.78	0.95	0.54
Tenatabiya sefer	427.11*	25 year	4.57	60.45	62.46		62.75	0.003485	2.40	1.90	0.95	0.54
Tenatabiya sefer	427.11*	50 year	4.67	60.45	62.48		62.78	0.003529	2.42	1.93	0.95	0.54
Tenatabiya sefer	427.11*	100 year	4.75	60.45	62.50		62.81	0.003563	2.43	1.95	0.95	0.54
Tenatabiya sefer	425	10 year	4.09	60.45	61.68	61.68	62.30	0.008588	3.49	1.17	0.95	1.00
Tenatabiya sefer	425	25 year	4.57	60.45	61.78	61.78	62.45	0.008950	3.61	1.26	0.95	1.00
Tenatabiya sefer	425	50 year	4.67	60.45	61.80	61.80	62.48	0.009014	3.64	1.28	0.95	1.00
Tenatabiya sefer	425	100 year	4.75	60.45	61.82	61.82	62.50	0.009081	3.66	1.30	0.95	1.00

*Appendix*15*:*

: Model Tabuleted Result Crossection C Arsema Sefer, Detaile profile out put

Cross Section Output

File Type Options Help

River: Channel C Profile: 10 year

Reach Chammel C Reach RS: 15 Plan: C2

Plan: C2 Channel C Chammel C Reach RS: 15 Profile: 10 year

E.G. Elev (m)	1876.97
Vel Head (m)	0.46
W.S. Elev (m)	1876.51
Crit W.S. (m)	1876.51
E.G. Slope (m/m)	0.012100
Q Total (m3/s)	1.50
Top Width (m)	0.55
Vel Total (m/s)	3.00
Max Chl Dpth (m)	0.91
Conv. Total (m3/s)	13.6
Length Wtd. (m)	30.00
Min Ch El (m)	1875.60
Alpha	1.00
Frctn Loss (m)	0.36
C & E Loss (m)	0.00

Element	Left OB	Channel	Right OB
Wt. n-Val.		0.013	
Reach Len. (m)	30.00	30.00	30.00
Flow Area (m2)		0.50	
Area (m2)		0.50	
Flow (m3/s)		1.50	
Top Width (m)		0.55	
Avg. Vel. (m/s)		3.00	
Hydr. Depth (m)		0.91	
Conv. (m3/s)		13.6	
Wetted Per. (m)		2.37	
Shear (N/m2)		25.05	
Stream Power (N/m s)		75.15	
Cum Volume (1000 m3)		0.15	
Cum SA (1000 m2)		0.16	

Errors, Warnings and Notes

Warning:	The energy loss was greater than 1.0 ft (0.3 m). between the current and previous cross section. This may indicate the need for additional cross sections.
Warning:	During the standard step iterations, when the assumed water surface was set equal to critical depth, the calculated water surface came back below critical depth. This indicates that there is not a valid subcritical answer. The program defaulted to critical depth.

Appendix16:

Cannel C Summarry of Profile For 100 Year

HEC-RAS Plan: C2 River: Channel C Reach: Chammel C Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Chammel C Reach	15	10 year	1.50	1875.60	1876.51	1876.51	1876.97	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	15	25 year	1.61	1875.60	1876.55	1876.55	1877.03	0.012541	3.08	0.52	0.55	1.01
Chammel C Reach	15	50 Year	1.63	1875.60	1876.56	1876.56	1877.05	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	15	100 Year	1.74	1875.60	1876.60	1876.60	1877.11	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	13.600*	10 year	1.50	1870.59	1871.50	1871.50	1871.96	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	13.600*	25 year	1.61	1870.59	1871.54	1871.54	1872.02	0.012545	3.08	0.52	0.55	1.01
Chammel C Reach	13.600*	50 Year	1.63	1870.59	1871.55	1871.55	1872.04	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	13.600*	100 Year	1.74	1870.59	1871.59	1871.59	1872.10	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	12.200*	10 year	1.50	1865.58	1866.49	1866.49	1866.95	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	12.200*	25 year	1.61	1865.58	1866.53	1866.53	1867.01	0.012541	3.08	0.52	0.55	1.01
Chammel C Reach	12.200*	50 Year	1.63	1865.58	1866.54	1866.54	1867.03	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	12.200*	100 Year	1.74	1865.58	1866.58	1866.58	1867.09	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	10.800*	10 year	1.50	1860.57	1861.48	1861.48	1861.94	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	10.800*	25 year	1.61	1860.57	1861.52	1861.52	1862.00	0.012545	3.08	0.52	0.55	1.01
Chammel C Reach	10.800*	50 Year	1.63	1860.57	1861.53	1861.53	1862.02	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	10.800*	100 Year	1.74	1860.57	1861.57	1861.57	1862.08	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	9.400*	10 year	1.50	1855.56	1856.47	1856.47	1856.93	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	9.400*	25 year	1.61	1855.56	1856.51	1856.51	1856.99	0.012541	3.08	0.52	0.55	1.01
Chammel C Reach	9.400*	50 Year	1.63	1855.56	1856.52	1856.52	1857.01	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	9.400*	100 Year	1.74	1855.56	1856.56	1856.56	1857.07	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	8.000*	10 year	1.50	1850.55	1851.46	1851.46	1851.92	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	8.000*	25 year	1.61	1850.55	1851.50	1851.50	1851.98	0.012545	3.08	0.52	0.55	1.01
Chammel C Reach	8.000*	50 Year	1.63	1850.55	1851.51	1851.51	1852.00	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	8.000*	100 Year	1.74	1850.55	1851.55	1851.55	1852.06	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	6.600*	10 year	1.50	1845.54	1846.45	1846.45	1846.91	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	6.600*	25 year	1.61	1845.54	1846.49	1846.49	1846.97	0.012541	3.08	0.52	0.55	1.01
Chammel C Reach	6.600*	50 Year	1.63	1845.54	1846.50	1846.50	1846.99	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	6.600*	100 Year	1.74	1845.54	1846.54	1846.54	1847.05	0.013114	3.17	0.55	0.55	1.01

Chammel C Reach	5.200*	10 year	1.50	1840.53	1841.44	1841.44	1841.90	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	5.200*	25 year	1.61	1840.53	1841.48	1841.48	1841.96	0.012545	3.08	0.52	0.55	1.01
Chammel C Reach	5.200*	50 Year	1.63	1840.53	1841.49	1841.49	1841.98	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	5.200*	100 Year	1.74	1840.53	1841.53	1841.53	1842.04	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	3.800*	10 year	1.50	1835.52	1836.43	1836.43	1836.89	0.012100	3.00	0.50	0.55	1.00
Chammel C Reach	3.800*	25 year	1.61	1835.52	1836.47	1836.47	1836.95	0.012541	3.08	0.52	0.55	1.01
Chammel C Reach	3.800*	50 Year	1.63	1835.52	1836.48	1836.48	1836.97	0.012546	3.08	0.53	0.55	1.00
Chammel C Reach	3.800*	100 Year	1.74	1835.52	1836.52	1836.52	1837.03	0.013114	3.17	0.55	0.55	1.01
Chammel C Reach	2.400*	10 year	1.50	1830.51	1831.42	1831.42	1831.88	0.012086	3.00	0.50	0.55	1.00
Chammel C Reach	2.400*	25 year	1.61	1830.51	1831.46	1831.46	1831.94	0.012545	3.08	0.52	0.55	1.01
Chammel C Reach	2.400*	50 Year	1.63	1830.51	1831.47	1831.47	1831.96	0.012577	3.08	0.53	0.55	1.00
Chammel C Reach	2.400*	100 Year	1.74	1830.51	1831.52	1831.52	1832.02	0.012764	3.13	0.56	0.55	0.99
Chammel C Reach	1	10 year	1.50	1825.50	1826.41	1826.41	1826.87	0.012095	3.00	0.50	0.55	1.00
Chammel C Reach	1	25 year	1.61	1825.50	1826.45	1826.45	1826.93	0.012532	3.07	0.52	0.55	1.01
Chammel C Reach	1	50 Year	1.63	1825.50	1826.46	1826.46	1826.95	0.012581	3.08	0.53	0.55	1.00
Chammel C Reach	1	100 Year	1.74	1825.50	1826.51	1826.51	1827.01	0.012760	3.13	0.56	0.55	0.99

Appendix 17:In situ Measurement Of Length For Selected Draiange Channels In problem Area

Channel A					Algnment	Chainage(m)	Bank Level avg(m)		Avg.Depth (m)
Easting (m)	Northing(m)	Elevetion(m)	Length(m)	Total lenrth(m)			LB	RB	
366011	915938	1874.9	0	0	Up stream	35	0	0.8	0.8
365971	915934	1876.2	40.20	40.20		35	0	0.8	0.8
365948	915924	1876.5	25.08	65.28		35	0	0.8	0.8
365961	915900	1872.3	27.29	92.57		35	0	0.8	0.8
365976	915876	1867.2	28.30	120.88		35	0	0.8	0.8
365985	915858	1863.3	20.12	141.00		35	0	0.8	0.8
365995	915837	1858.6	23.26	164.26		35	0	0.8	0.8
366000	915820	1855.1	17.72	181.98		35	0	0.8	0.8
366014	915785	1848.1	37.70	219.68		35	0	0.8	0.8
366032	915752	1842.2	37.59	257.27		35	0	0.8	0.8
366046	915715	1836.9	39.56	296.83		35	0	0.8	0.8
366043	915691	1834.6	24.19	321.01		35	0	0.8	0.8
366045	915670	1832.3	21.10	342.11		35	0	0.8	0.8
366067	915639	1827	38.01	380.12		35	0	0.8	0.8
366080	915616	1823.7	26.42	406.54		35	0	0.8	0.8

366097	915593	1820.5	28.60	435.14	Down Stream	35	0	0.8	0.8
Channel B									
Easting(m)	Northing(m)	Elevetion(m)	Length(m)	Cumulative Length(m)	Algnment	Chainage(m)	Bank Level avg(m)		Avg.Depth (m)
365316	915533	1879.8	0	0	Up stream		LB	RB	
365334	915518	1877.2	23.43	23.43		25	0	0.85	0.9
365351	915495	1873.3	28.60	52.03		25	0	0.85	0.9
365363	915478	1869.9	20.81	72.84		25	0	0.85	0.9
365379	915446	1862.7	35.78	108.62		25	0	0.85	0.9
365388	915419	1855.8	28.46	137.08		25	0	0.85	0.9
365417	915378	1844.6	50.22	187.30		25	0	0.85	0.9
365441	915332	1835.8	51.88	239.18		25	0	0.85	0.9
365496	915234	1821.5	112.38	351.56		25	0	0.85	0.9
365460	915203	1819.9	47.51	399.07		25	0	0.85	0.9
365417	915171	1819.3	53.60	452.67		25	0	0.85	0.9
365375	915143	1819.4	50.48	503.15		25	0	0.85	0.9
365328	915114	1819.2	55.23	558.37		25	0	0.85	0.9

365289	915093	1818.7	44.29	602.67	Down Stream	25	0	0.85	0.9
Channel C					Algnment	Chainage(m)	Bank Level avg(m)		Avg.Depth (m)
					0		LB	RB	
Easting	Northing	Elevetion	Length	Cumulative Length					
365852	915835	1876.1	0	0	Up stream		0	0.65	0.5
365831	915803	1875.1	38.28	38.28		45	0	0.65	0.5
365808	915776	1874.3	35.47	73.74		45	0	0.65	0.5
365793	915754	1873.6	26.63	100.37		45	0	0.65	0.5
365772	915734	1874.3	29.00	129.37		45	0	0.65	0.5
365759	915720	1874.6	19.10	148.48		45	0	0.65	0.5
365771	915712	1871.9	14.42	162.90		45	0	0.65	0.5
365772	915712	1871.8	1.00	163.90		45	0	0.65	0.5
365804	915688	1864.3	40.00	203.90		45	0	0.65	0.5
365815	915656	1858.9	33.84	237.74		45	0	0.65	0.5
365801	915634	1857.1	26.08	263.81		45	0	0.65	0.5
365807	915634	1856	6.00	269.81		45	0	0.65	0.5

365776	915589	1854.2	54.64	324.46		45	0	0.65	0.5
365768	915578	1853.6	13.60	338.06		45	0	0.65	0.5
365780	915551	1847.5	29.55	367.60		45	0	0.65	0.5
365788	915519	1841.2	32.98	400.59		45	0	0.65	0.5
365795	915492	1836.3	27.89	428.48		45	0	0.65	0.5
365795	915465	1832.2	27.00	455.48		45	0	0.65	0.5
365805	915440	1828.4	26.93	482.41		45	0	0.65	0.5
365812	915423	1826	18.38	500.79	Down Stream	45	0	0.65	0.5