PERFORMANCE EVALUATION OF KEWADO TOWN, DRAINAGE SYSTEM IN SIDAMA REGIONAL STATE.

M.Sc. THESIS

BY

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PERFORMANCE EVALUATION OF KEWADO TOWN, DRAINAGE SYSTEM IN SIDAMA REGIONAL STATE.

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ADVISORS’ APPROVAL SHEET

This is to certify that the thesis entitled “Performance Evaluation of Kewado town Drainage Systems in Sidama regional state” submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Irrigation and drainage Engineering, the Graduate Program of the Department Water Resource and irrigation Engineering, and has been carried out by Gizaw Hayiso ID. No. GP/IrDrR/ 0005/13, under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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EXAMINERS’ APPROVAL SHEET

We, the undersigned, members of the Board of Examiners of the final open defense by Gizaw Hayiso Gundula have read and evaluated his thesis entitled “Performance Evaluation of Kewado town Drainage System in Sidama regional state”, and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of Master of Science in irrigation and drainage Engineering.

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LIST OF ABBREVIATIONS AND ACRONYMS

CSA	Central Statistical Agency
DEM	Digital Elevation Model
ERDAS	Earth Resources Data Analysis System
GIS	Geographic Information system
ILWIS	Integrated Land and water information system
RS	Remote Sensing
DEFRA	Department for Environment Flood and Rural Affairs
SUDS	Sustainable Urban Drainage System
BMPs	Best Management Practices
LIDs	Low Impact Developments
WSUD	Water Sensitive Urban Design
USEPA	United States Environmental Protection Agency
CIRIA	Construction Industry Research and Information Association
SWMS	Storm water management system
USGS:	United State Geological Survey
ASL**:**	Above sea level
LULC:	Land Use Land Cover Change
NSC	Nash–Sutcliffe model efficiency
EPA	Environmental Protection Agency
IDF	Intensity duration Frequency
GTZ	Germen technical corporation agency
SC	Sub Catchment

CONTENTS

ABSTRACT

Developing countries experience accelerated urbanization without adequate investment in infrastructure. kewado town, like other towns of Ethiopia, have a lot of problems including inadequacy and poor-quality drainage infrastructure. This study was aimed to assess the Performance Evaluation of Kewado town Drainage Systems focused on the western and eastern part of the main road of the town. Both field and historical climatic data were collected and analyzed. Rainfall data for a period of 1990-2019 were used for the study. The methodology employed in this study starts with identifying parent probability distribution for the rainfall intensities with various return periods. This enabled to develop the Intensity Duration Frequency Curve (IDF). The rainfall frequency analyses depicted that the log-Pearson III distribution best fits the rainfall intensities as confirmed by the goodness of fit tests. . Rainfall quintiles for a duration of 24hr with a magnitude of 52.4, 62.9, 75.6, 86.9, 95.1 and 103.3 mm/hr. was predicted for a return period of 2,5,10,25, 50 and 100 years respectively The hydraulic dimensions of existing structures were measured in the field using tape meter.

The catchment that contributes runoff areas, weighted runoff coefficient was delineated using Arc GIS 10.7 tool. Considering the current land use, rainfall intensity and catchment area, the peak discharge was estimated using the rational methods. The Manning equation was used for fixing the dimensions of the storm drainage canals. The adequacy of the existing drainage systems were checked by comparing the estimated runoff with the existing drainage capacity. Hence, most of drainage lines were inadequate to safely dispose of the runoff. Based on the GTZ standard, 60 %, 46 %, 18% and 48% of the existing canals were performed with the severe and light conditions around Mazegaja, Ethiotele, TVT college and Darancho respectively. Illegal disposal of solid & liquid waste into drainage systems, lack of sewer line, unavailability of drainage for some part of town, low community awareness, low mitigation measures and dis- connectivity of drainage were the major problems.. The Stormwater Management Model (SWMM 5.2) was applied to simulate the water levelin the links and junctions by considering the current land use condition.

Keywords:-*Kewado Town, Drainage Systems, Storm water, SWMM5.2*

1. INTRODUCTION

1.1. Background

Metropolitan drainage systems have been built in cities all over the world for more than a century to quickly remove runoff from metropolitan areas and minimize the annoyance of flooding. As populated areas grew larger, they soon started to have an impact on the hydrological cycle. To quickly transport water from the land to the sea, ditches were created, fields were under-drained, streams were straightened, and rivers were embanked (Abraha, 2018). In several regions of the world, combined storm water and waste water drainage systems were also offered with a similar goal.

When urbanization progresses more quickly, drains become more and more overloaded and unable to handle significant rainfall. Most developing urban areas in Ethiopia experience flooding as a result of rapid urbanization, which also has an impact on the performance of the drainage systems due to its impermeable structures and poor design. (Belete, 2009)Developing countries experience accelerated urbanization without adequate investment in infrastructure, and against a background of deficient public services for water treatment, collection and treatment of foul sewage, garbage collection, urban drainage, transport and health. urbanization impervious surfaces increase, drainage pattern changes, overland flow becomes speedy, flooding and environmental problems such as land degradation increases.

The problem in urban stormwater drainage network is also the other challenge in urban areas, because the run-off produced with in a particular urban area could not safely be discharged in to the final receiving system (G/Wahed, 2016). Urban concentrations have environmental consequences in the form of urban flooding and pollution of water courses, soil and air. Settlements are established inappropriate areas such as thoseoriginallysetas

Environmental preservation and on steep hillsides and areas liable to flooding. Therefore estimates of peak rate of runoff, runoff volume, and the time distribution of flow provide the basis for all planning, design, and construction of drainage facilities. The major causes of flooding as the blockage of urban storm water drainage lines along with inadequate integration between road and urban storm water drainage infrastructures. In addition, with urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads drainage pattern changes) the overland flow gets faster flooding and environmental problems like degradation increases. It is a crucial problem facing the existing and future road infrastructure (Belete,2011). In general road construction without adequate provision of drainage is a major cause of flooding. Lack of urban storm water drainage (USWD) management represents one of the most common sources of complaint from the residents in many urban centers of Ethiopia, and this problem gets worse and worse with the rate of urbanization. Currently, Kewado town is the one facing drainage problem due to inadequate provision of drainage structure in conveying the runoff to the outlet points and road drainage structures are not properly functioning. As a result, a change in the run-off characteristics within the town, increased runoff and greater susceptibility to flooding.

1.2. Statement of the problem

Road construction without adequate provision of drainage is a major cause of gully erosion. The road that has improper drainage systems which results in gully erosion may occur anywhere in the world. The problem is particularly severe in developing countries due to neglect in maintenance and lack of provision of safe outlets to the excess runoff. In Sidama regional state Kewado town, road drainage structures are not properly functioning due to insufficient capacity of road ditches, unavailability of organized (networked) drainage structures at proper place, and illegal dumping of household solid and liquid wastes. As a result, kewado town is currently, facing serious environmental problems. There is a change in the run-off characteristics within the town, increased runoff and greater susceptibility to flooding.Thedrainagesystemsthroughthetownareinadequatetoconveythe.

Runoff to the outlet points properly. Additionally the absence of appropriate maintenance of the existing drainage facilities (either partially or fully filled with silts or corrupted and causing the storm water to flow over the road surface) and insufficient capacity of road ditches.

1.3. Objectives of the Study

1.3.1. General Objective

The general objective of the study was to evaluate the Performance of Drainage Systems of Kewado town in sidama regional state.

1.3.2. Specific objective

The following were the specific objectives of the study:

- .To assess the current status of the existing drainage system.
- To develop the intensity duration frequency curve for various return period
- To evaluate the hydraulic capacity of the existing drainage structures of the town.

1.4. Research questions

- Does the existing drainage system perform well?
- What are the hydraulic properties of kewado town storm water drainage system Contributing to the flooding problem?
- What are the hydrologic characteristics of the catchments contributing to flooding?
- What are the major challenges in the stormwater drainage system in the study area?
- What are the possible best management practices to improve the problems?

1.5. Significance of the Study

The benefit that draws from this study may contribute to current efforts by governments and other concerning bodies to solve the problem drainage schemes that contribute to better service coverage. To understand problems of damage and preserve the structures by avoiding further deterioration for taking corrective measures as well as to reduce any inconvenience and disruption to travel due to the overflow of water in the main road due to flooding. Also may help in filling the gaps by identifying problems to Sustainability, taking proper designing of Storm water drainage system and proper functioning of drainage schemesithetow

In general, Kewado town is the part of which facing the drainage problems, so further investigation is contributed the solution for storm water drainage problems and sustainable drainage systems for future use in the area. Also, the study contributes the following significances to Ethiopia Road Authority (ERA), Dara Kewado Administration, academicians, researchers, and other stakeholders who will conduct similar researches on other road drainage structures. Preserving the road network and save the asset value, identifying the existing situation in order to know the real problems of the drainage Systems, finding possible solutions based on the recommendations to avoid the problems.

1.6. Scope of study

This research express and observe the determinants of the performance evaluation of drainage network system, peak flows of selected segment, operations of outlets and discussing the existing management responses to mitigate the problems sampled in the Kewado town. The study conducted to performance evaluation and improvement option of drainage system of the study area. And also the study bounded to possible engineering solution to the problem.

2. LITERATURE REVIEW

2.2. Historical Perspectives of Urban Drainage System

Urban drainage systems have historically been viewed from a number of angles. It has been viewed as a crucial natural resource, a practical cleaning mechanism, an effective waste transport medium, a concern for floods, and a disease transmitter during various historical eras and in various locales (Asfaw, 2016). The local perspective on urban drainage has generally been impacted by climate, geography, geology, scientific understanding, engineering and construction skills, social values, religious views, and other variables (Burian and Edwards, 2002). These variables have influenced and limited the creation of urban drainage systems for as long as people have been building cities (Parkinson, 2013).

Current Urban Drainage Systems Perspectives

Urban drainage in the early parts of the twentieth century was firmly established as a vital public works system. Engineers continued to improve design concepts and methods. During the second half of the twentieth-century regulatory elements were promulgated in the United States, Europe, and other locations addressing urban drainage issues.

Extensive monitoring efforts vastly improved the understanding of urban drainage quantity and quality characteristics. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from a public health and nuisance flooding concern during the first half of the twentieth century into a public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability (UD and FC, 2011).

During the secondhalf of the twentieth century regulatory elements were promulgated in the United States, Europe,and other locations addressing urban drainage issues (Tafete, 2013). In addition, alternative on-site wastewater management strategies are being touted as more suitable than centralized wastewater management for some situations. Communities are searching for innovative techniques to capture, detain, and use rainwater within the watershed instead of constructing massive drainage structures (Graham, 2012)

Many communities are developing watershed-wide storm water quality management plans to meet the dual objectives of flood prevention and water quality control. 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 (Burain and Edwards,2002).

2.1. Urban Drainage System Problems

The problem of urban drainage systems is that as long as road structures and subgrade soil do not have excess water the road will work well. But increase water content reduces the bearing capacity of the soil, which will increase the rate of deterioration and shorten the lifetime of the road. In such cases, the road will need rehabilitation more often than a well-drained road structure. Mainly the problem was observed on poorly working structures, such as culverts and ditches (Saara and Saarenketo, 2006).

The problems created by inadequate drainage channels have attracted a varying dimension of environmental problem. However, the most prominent among such problems being that the absence of drainage channels easily translate into environmental deterioration(Jimoh, 2003). Settlements are established in inappropriate areas such as those originally set aside for environmental preservation and on steep hillsides and areas liable to flooding (Hassen, 2016). Generally, drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, waterlogging. Factors affecting drainage performance: inadequate capacity of the infrastructures, clogging, storage capacity, waste disposal, poor integration with road, unexpected rate of urbanization, land use change, population pressure, lack of periodical inspection and maintenance, carelessness of the community, Poor workmanship, solid wastes and sediments are also the causes of resulting in negative impacts of the existing drainage structure (Dibaba, 2018).

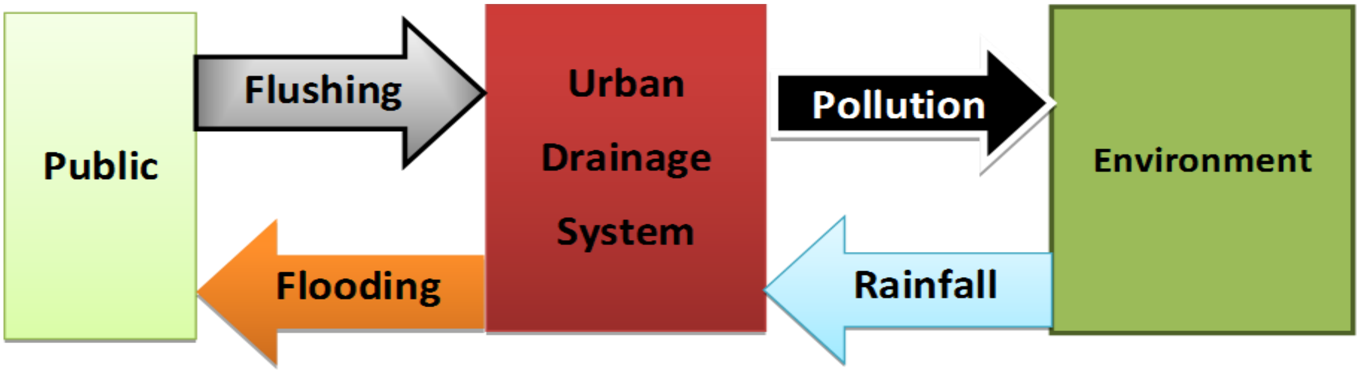

Figure.1 : Interfaces with the public and the environment (Butler and Davies, 2011).

2.1.1. Effects of Urbanization on Urban Drainage

There are a large number of human and non-human activities that can destabilize the hydrologic cycle. Among human activities that have the greatest impact on the natural hydrologic cycle is the urbanization process. Along with urbanization, new engineering works arise such as buildings, paving of streets, sidewalks, and consequent removal of original plant cover from the environment, which causes a change in the natural permeability of these areas (Poleto & Tassi, 2012)

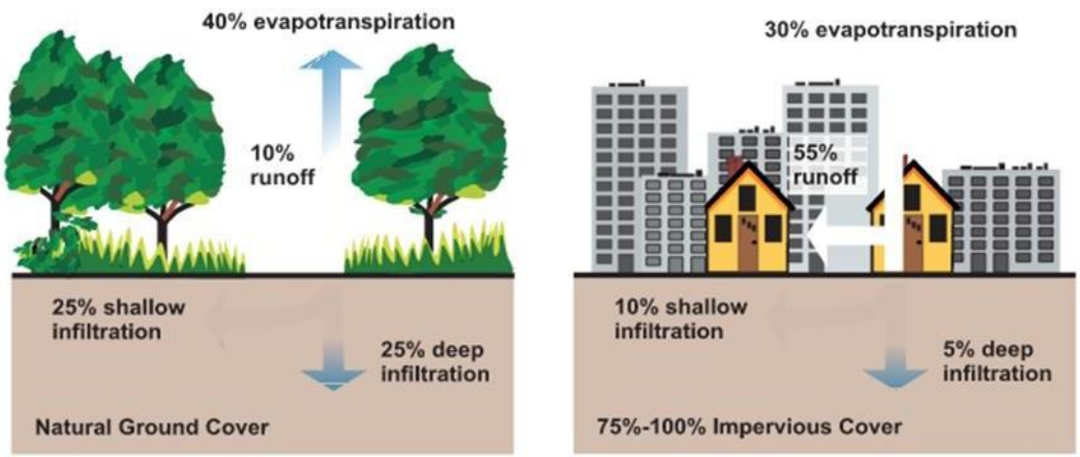

Figure 2: Relationship between Impervious Cover and Surface Runoff (Source: U.S. EPA)

Proper drainage is an important factor that should be given due consideration in the design of a highway since inadequate drainage facilities can lead to premature deterioration of the highway and the development of adverse safety conditions (Mukherjee, 2014).

In this connection, the design of pavement drainage is of great importance and should be given due consideration when designing drainage. Both groundwater and surface runoff are likely to find their way to a river, but surface runoff arrives much faster. (David et al, 2018). Channel shapes are generally determined for a location by considering the terrain, flow regime and the quantity of flow to be conveyed. Typically, ditch geometry is either rectangular, V-shaped and trapezoidal. Roadway channels should provide recoverable slopes, thereby minimizing the impact of errant vehicles. This can be accomplished by designing ditch cross-sections with mild side slopes. Depending on the side slopes used, both V-shaped and trapezoidal ditches can provide driver safety and be economical to construct. When mild side slopes are used, the shape tends to approach a parabolic shape, which is recognized as being the most hydraulically efficient shape (AASHTO, 1990).

2.2. Urban Drainage Systems Practice in World

According to United Kingdom's Department for Environment Flood and Rural Affairs (DEFRA) research framework for implementing integrated urban drainage, it is defined as management of the risk arising from drainage and flooding in urban areas through a portfolio of approaches. The Worldwide number of different countries is working toward integrated drainage, either on its own or as part of a broader approach to managing the water cycle. In the United Kingdom the term Sustainable Urban Drainage System (SUDS) is used to mean the integrated urban drainage system, as integration of drainage system with the environment ultimately lead to sustainability (Balmforth, et al., 2009).

Drainage systems are needed in developed urban areas because of the interaction between human activity and the natural water cycle. This interaction has two main forms: the abstraction of water from the natural cycle to provide a water supply for human life, and the covering of land with Impermeable surfaces that divert rainwater away from the local natural system of drainage (Butler and Davies, 2000).

A complete drainage system design includes consideration of both major and minor drainage systems. The minor system sometimes referred to as the "convenience" system, consists of the components that historically considered as part of the "storm drainage system". These components include curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, detention basins, and water quality control facilities (Alderson, 2006).

2.2.1. Urban Drainage Practice in Africa

Different types of structures are employed in the drainage systems are open channels whether artificial or natural convey the flows of water surface and sub-surface drainage systems: culverts and bridges convey flows under road cross-section; energy dissipaters used to control the velocities of flows, especially at culvert outlets (kalantari,2011).

In addition noted that the main challenge in developing countries has to do with the lack of the adequate number of skilled personnel who are able to plan and implement urban drainage in a timely manner and the lack of funding needed to pay for the work (Armitage, 2010).

2.2.2. Urban Drainage Practice in Ethiopia

In Ethiopia context, where watersheds of many urban centers receive a significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of excess water/runoff through proper drainage facilities is very essential (MoWUD,2008).

the major causes of flooding as the blockage of urban storm water drainage lines along with inadequate or poor integration between the road and urban storm water drainage infrastructure. In addition, with urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots.), drainage pattern changes, the overland flow gets faster flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure. (Belete, 2011)

As FUPCoB (2008), the drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, water logging With urbanization, impermeability increases with the increase in impervious surfaces.

Urban storm water influences the service life of urban infrastructures and the rainfall intensity and characteristics of the catchment area are the major factors for designing urban storm water drainage facilities. These facilities have a paramount advantage to safely dispose of the generated floods to ultimate receiving systems. According to Urban storm water drainage design Manual, storm water management is concerned with the collection, conveyance, storage, treatment and disposal of storm runoff in a way that minimize accelerated channel erosion, increased flood damage (Meraf, 2015). Adequate drainage facility is an important factor that should be given paramount importance in the design of roads since it greatly determines the roads serviceability and useable life. Drainage problems in urban areas include flooding residential areas deterioration of roads, land degradation, sedimentation and blockage of drainage facilities as well as waterlogging (Mulualem, 2016).

2.3. Integrated Urban Drainage Systems

Drainage system are challenged by continued urbanization ,accelerated climate change and aging infrastructure, resulting in hydraulic overloading of drainage system and increased flooding.to address these challenges and to contribute to ensuring safe and attractive urban environments ,by taking an integrated urban drainage management approach. Integrated urban drainage is defined to mean different thing by different people. But every definition includes the fact that integrated urban drainage must recognize the complexity of managing risk arising from drainage and flooding in urban areas. Furthermore, it utilizes coordinated effort from different stakeholders to deliver full suite of techniques, adaptation and resilience measures that integrate in a sustainable manner to manage flood risk.

According to United Kingdom's Department for Environment Flood and Rural Affairs (DEFRA) research framework for implementing integrated urban drainage, it is defined as management of the risk arising from drainage and flooding in urban areas through a portfolio of approaches (Balm forth, et al., 2009). United States Environmental Protection Agency (USEPA) calls the practices for controlling storm water runoff volume and reducing pollutant loadings to receiving waters as LID. According to the agency LID is a site design strategy with a goal of maintaining or replicating the predevelopment hydrologic regime through the use of design techniques to create a functionally equivalent hydrologic landscape (US Environmental Protection Agency, 2000).

- Construction Industry Research and Information Association (CIRIA) of United Kingdom uses the term Sustainable Urban Drainage System (SUDS) to refer to drainage methods that can mitigate adverse effects of urban storm water runoff on the environment. According to CIRIA SUDS mitigate storm runoff effects through the following ways (CIRIA, 2007).

- acting as a buffer for accidental spills by preventing direct discharge of high concentrations of contaminants to the receiving water body
- Contributing to the enhanced amenity and aesthetic value of developed areas
- Providing habitats in urban areas and opportunities for biodiversity enhancement.
- Reducing runoff rates by attenuation
- Reducing excess runoff volumes that result from urbanization
- encouraging natural groundwater recharge
- Reducing pollutant concentrations in storm water, and protecting the quality of the receiving water body

2.4. Explanation of Model (SWMM)

The storm water management model was first developed in 1971, at Washington DC and has undergone several major upgrades. Since then, it continues to be widely used throughout the world for planning, analysis and design related to storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas. SWMM is a physically based, discrete-time simulation model. It is a dynamic rainfall-runoff simulation model used for a single event or long term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels storage/treatment devices, pumps, and regulators (Lewis and Huber, 2016).

SWMM is a full dynamic wave simulation model used for single event or long-term simulation of runoff quantity and quality, primarily from urban areas. Version 5.2 is a completer-write of the previous release, running under Microsoft Windows and providing an integrated environment for editing data, running hydrologic, hydraulic and water quality simulations, and viewing the results. It conceptualizes a drainage system as a series of water and material flows between several major environmental compartments, namely: the atmosphere compartment, from which precipitation falls and pollutants are deposited onto the land surface compartment; the land surface compartment, which is represented by sub-catchment objects; the groundwater compartment, which receives infiltration from the land surface compartment and transfers a portion of this inflow to the transport compartment; and the transport compartment, which contains a network of conveyance, storage, regulation and treatment elements. Not all compartments need appear in a particular SWMM model (Abraha, 2018).

2.4.1. Selection criteria of SWMM model

There are several methods for assessing stormwater runoff that are based on the water balance equation, empirical equation, and practical models like CivilStorm, STORM model, XPSWMM, PCSWMM and SWAT by such strategies calculations for penetration, overflow of surface, routing of flow, and slacking of surface overflow have been re arranged to permit simulation of flow with a hydraulic and Hydrologic Simulation Program For each one of those number of techniques they have some limitations fixing urban drainage systems networks (Sidek ,et al., 2011).

2.4.2. PCSWMM

PCSWMM is a third-party interface for SWMM and is developed by Computational Hydraulics. It is a GIS integrated model that uses SWMM 5.2 as the model computational engine for hydrologic and hydraulic calculations. It is a stand-alone modeling tool with all necessary IS- tools included and it has support for various CAD and GIS formats (Lind, 2015).

2.4.3. XPSWMM

XPSWMM is another hydraulic and hydrologic modeling tool from XP-solutions and has been used for analysis, design and simulations for over 25 years. Like XP Storm, XPSWMM includes stormwater and river systems/flood plains. Additionally, it also includes wastewater management. The model simulates 1D network flows in combination with 2D overland flows, LID structures and stormwater quality. The tool can be used for natural systems like for example ponds, rivers and lakes and manmade environments like pipes, conduits and streets (Lind, 2015).

2.4.4. Civil Storm

Another product from Bentley is Civil Storm. It is stormwater modeling software that models more aspects of the system than Storm CAD. Civil Storm is a dynamic model that accounts for storage, detention and flows over time and is therefore a more advanced modeling tool than Storm CAD. It is used for master planning, modeling the effect of LID Structures as well as studying the water quality (Lind, 2015).

2.5. Application of SWMM

SWMM applications have been used in sewer and storm water studies throughout the world. Its typical applications include:- designing and sizing of drainage system components for flood control, sizing detention facilities and their appurtenances for flood control and water quality protection, mapping flood plains of natural channel systems, designing control strategies for minimizing combined sewer overflows, evaluating the impact of inflow and infiltration on sanitary sewer overflows, generating non-point source pollutant loadings for waste load allocation studies, controlling site runoff and evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings(Rossman, 2004).

2.6. Best management practice (BMP)

This section provides guidance on factors that should be considered when selecting BMPs for Storm water drainage design. BMP selection involves many factors such as physical site Characteristics, treatment objectives, aesthetics, safety, maintenance requirements, and cost. Typically, there is not a single answer to the question of which BMPs should be selected for a site; there are usually multiple solutions ranging from standalone BMPs to treatment trains that combine multiple BMPs to achieve the water quality objectives. Factors that should be considered when selecting BMPs are the focus of this section (Rossman,2004).

2.6.1. Physical site characteristics

The first step in BMP selection is identification of physical characteristics of a site including

Topography, soils, contributing drainage area, groundwater, base flows, wetlands, existing drainage ways, and development conditions in the tributary watershed (e.g., construction activity) (UD and FC, 2011).

2.6.2. Space constraints

Space constraints are frequently cited as feasibility issues for BMPs. In some cases, constraints due to space limitations arise because adequate spaces for BMPs are not considered early enough in the planning process (UD and FC, 2011). This is most common when a site plan for roads, structures, etc., is developed and BMPs are embraced into the remaining spaces. The most effective and integrated BMP designs begin by determining areas of a site that are best Suited for BMPs (e.g., natural low areas, areas with well-drained soils) and then designing the layout of roads, buildings, and other site features around the existing drainage and water quality resources of the site (Zewdu, 2015).

2.6.3. BMP Processes

The physical and chemical characteristics of storm water runoff change as urbanization occurs, requiring comprehensive planning and management to reduce adverse effects on receiving waters. As stormwater flows across roads, rooftops, and other hard surfaces, pollutants are picked up and then discharged to streams and lakes (UD and FC, 2011). Additionally, the increased frequency, flow rate, duration, and volume of stormwater discharges due to urbanization can result in the scouring of rivers and streams, degrading the physical integrity of aquatic habitats, stream function, and overall water quality.

Stormwater drains traditionally lead to local creeks and waterways where the stormwater is dispersed without treatment.

Unmanaged stormwater systems can result in pollutants such as oil, sediment, nutrients and rubbish entering waterways. Physical changes can also occur, such as waterway channel erosion, due to the reduced stormwater infiltration which typically occurs with urbanization, and consequent increased velocity and extended duration of flow entering the natural water system. If stormwater is left unmanaged, pollution and physical changes caused by stormwater can cause considerable damage to the environment and, in particular, to waterways (Zewdu, 2015).

There is Four Step Process pertains to management of smaller, frequently occurring events, as opposed to larger storms for which drainage and flood control infrastructure are sized. Implementation of these four steps helps to achieve a well-developed stormwater management practice (UD and FC, 2011)

2.7. Techniques of Drainage Improvement System

Property damage and human injury caused by flooding have been considerable in recent decades worldwide, and it is expected that flood risk will increase continuously because of climate change and human growth among the sustainable systems that are being developed and implemented, the deferent devices or techniques may be used. These devices adopted in sustainable systems seek to mitigate the effects of urbanization through reduction of runoff and provide for significant increases in infiltration rates. To obtain these results, permeable areas and rainwater retention areas must be created as part of a larger complex, which is water resource management.

Such systems may be used jointly or separately according to the proposed project or the local needs and/or possibilities, providing a good cost/benefit ratio, in addition to social, economic, and environmental gains. Currently there are a large number of sustainable drainage systems being tested or even already consolidated in innumerable countries. In following, we will present the principal systems that may be adopted in management of urban watersheds as a way for improving or supplementing conventional drainage systems.

2.7.1. Storm water drains

The detailed design of storm water drains should be carried out by engineers and take into account climatic and hydrological data. These data may be scarce, or may not cover the community where work is to be carried out. In such cases, the community can help by describing where major flood problems occur in the village and providing information about previous floods. Storm water drains should be designed to collect water from all parts of the community and lead it to a main drain, which then discharges into a local river.

The size of the drains should be calculated according to the amount of water they would be expected to carry in a storm. More extreme floods occur relatively infrequently; to provide a safety margin.

2.7.2. Combined drains

Combined drains are designed to carry both stormwater and sullage. Unless a combined drain is well designed and maintained, however, sullage will pool within the drain and form insect breeding sites. These problems can be overcome by using a system with a small insert drain that carries the sullage into a larger drain for carrying stormwater. As with all drainage systems, it is essential that the drains are properly operated and maintained, and that refuse is cleared from the drains.

Infiltration trenches are shallow excavations filled with rubble or stone. They allow water to infiltrate in to the surrounding soil from the bottom and side of the trench enhancing the natural ability of the soil to drain the water. Ideally they should receive lateral inflow from the adjacent impermeable surface, but point source inflows may be acceptable with some design adaptation. Reservoirs full of rock to which rain water is directed for initial storage and from which water gradually infiltrates the soil.

According to Silva (2007) infiltration trenches (percolation and/or draining) are linear structures in which length is preponderantly greater than width and depth. Their geometry depends on the infiltrates of the soil and the area available for infiltration to occur. Depending on local conditions and the volume to infiltrate, the design may give priority to infiltration, storage, or both. Generally trenches are planned for large volumes of water to be infiltrated, are closed, and allow landscaping use in harmony with other structures.

According to Silveira (2008), the main function of infiltration trenches is to reduce runoff and promote recharging of aquifers, but another important function is to promote water treatment by means of soil infiltration.

Waste filters (also known as French drains) are widely used by authorities along highways. They are similar to infiltration trenches, but use a perforated pipe, which carries the flow along the gulley. This allows storage, filtration, and infiltration of water that passes from the collection area to the final point. Pollutants are removed by absorption, filtering and microbial decomposition in the surrounding soil. That way, these systems may be successfully designed so as to incorporate infiltration as well as to serve as efficient filtration systems.Thus, according to Nascimento (1996), the advantages in the use of this type of structure are:

- Reduction or even elimination of the local micro drainage network
- It avoids reconstruction of the downstream network in the case of saturation
- Reduction of risks of flooding
- Reduction in pollution of surface waters
- Recharging of underground waters
- Good integration with the urban area.

2.7.3. Infiltration ditch and gully

Gully is a small temporary or permanent dam constricted across a drainage ditch or channel to lower the speed of concentrated flow for a certain design range of storm events and to prevent erosion and to settle sediments and pollutants.

Ditches and gullies are compensatory techniques consisting of simple depressions dug in to the ground for the purpose of collecting rainwater, making temporary storage and favoring infiltration (Silva, 2007).

The depressions are linear and on permeable land, where generally there is a grassy covering. In addition, the infiltration gulley may incorporate small deceleration dams that favor infiltration and protect against erosion. Nevertheless, according to Silveira (2002), this system may cause commutation and allow the passage of pollutants and it has a propensity to retain stagnant water. Therefore it needs frequent maintenance.

Ditches are characteristically works of large width and low slope in the longitudinal direction dug into the earth. Gullies, for their part, are ditches that are not very deep (Brito, 2006). In both cases, they may also be used as one of the management resources within gardened areas of the urbanized watershed, or may even be incorporated in leisure areas.

These systems provide temporary storage for rainwater, reducing peak flows of waters and also aid in filtering of pollutants (deposited in the impermeable areas). They are often installed as part of a drainage network, where they help to connect it to a lagoon or wetland before final discharge into a natural waterway. Even so, they have often been installed along avenues, streets, and roadways to substitute conventional drainage systems Silveira (2002),

2.7.4. Underground reservoir

The water flowing from the saturated soil downward to deeper layers, feeds the ground water reservoir feeds the ground reservoir.

According to Silveira (2002), an underground or buried reservoir is a type of impervious tank built below the ground (with impermeable concrete walls) allowing utilization of the surface for another purpose. In general, an underground reservoir operates as an impermeable open air detaining reservoir. Therefore, it weakens the runoff introduced in it through the controlled effect of water release through the orifice and valve at the bottom. Nevertheless it should have a mechanism to avoid the accumulation of pollutants and sediment, it requires frequent maintenance, and it is restricted to areas with more frequent rainfall.

2.7.5. Reuse of rainwater

Rain water harvesting is an expertise for collection and efficient storage of rain water to reuse from different basement area like roof tops of residential buildings, ground surface, rock and catchment. The main objective of reuse of rain water is to minimize the flood and soil erosion. According it is worth emphasizing that all captured rainwater will reduce the waters that may cause flooding. Thus, this application is highly important for controlling water balance and it may play an important social role in areas subject to drought. When there is the occurrence of rainfall and there is no system for capturing this water, it will end up in storm drains. Until this water arrives at the storm drain, it traces a route where it comes in contact with innumerable type of residues, contaminating it and making it inappropriate for consumption. Nevertheless, this water may potentially be used in many day-to-day situations Fendrich (2003)

According to through Brazilian legislation, all water derived from rains is treated as sewage, creating the need for this water to be treated before its use, even without having been used Pereira (2008),.

2.7.6. Permeable pavement

Permeable pavement is an alternative infiltration device where surface runoff is diverted through a permeable surface into a rock reservoir located under the same surface (Urbonas & Stahre, 1993). According to the same authors, permeable pavement may be classified into three types:

- Porous asphalt pavement: has its upper layer constructed in a similar way to conventional pavements with a difference in the fraction of fine sand, which is removed from the mixture of the aggregates used in construction of the pavement;
- Porous concrete pavement: just as in porous asphalt pavement, it only has a difference in the sand fraction as compared to conventional pavements;
- Semipermeable pavement (described below).

2.7.7. Semi-permeable pavement

According to Urbonas & Stahre (1993), semi-permeable pavements are constructed of hollowed concrete blocks filled with granular material like sand or undergrowth, like grass, placed over a granular base layer and this layer is covered by geotextile filters to avoid migration of the granular base. For studies of Araújo *et al.* (2000), semipermeable pavements may be industrial concrete blocks, as well as paving blocks, among other items. According to Cruz *et al.* (1999), they have the function of providing for reduction in runoff volumes and in the time for recharging the watershed.Moreover, according to Araújo *et al.* (2000), in their studies performed on different types of semipermeable pavements (concrete, concrete blocks, compacted soil, paving blocks, and hollowed blocks), after simulations of similar precipitation levels, the following runoffs were obtained, respectively: 17.45 mm, 15.00 mm, 12.32 mm, 10.99 mm, and 0.5 mm. These studies were able to prove the efficiency of different types of pavements in reduction of runoff through increase of infiltration rates.

Furthermore, according to Araújo *et al.* (2000), the use of permeable pavements eliminates the need for collection boxes and water conduits because the device practically does not generate runoff. In addition to implementation costs of permeable pavements, there are maintenance costs, which consist of cleaning the pores of porous pavements (porous concrete) with water jets and machines for vacuuming of dirt and sediment. These costs were not estimated due to the non-existence of companies specialized in maintenance of this type of device in the country. Nevertheless, to have an idea, average expense on maintenance in the United States is in the order of 1% to 2% of the implementation cost of the device.

3. MATERIAL AND METHODS

3.1. General Description of Study Area

3.1.1. Location of study area

This study is conducted in the Kewado town Dara woreda administration in Sidama regional state. Sidama regional state is bordered by, Gedeo zone in the south, Wolaita in the west and Oromia in the north and south east. Its regional capital is Hawassa, which is the largest city in the region. Dara is one of the woredas in the Part of the Sidama regional state, Dara is bordered on the south by the Gedeo zone and on either side of it by the Oromia region, on the northwest by Chuko, on the north by Aleta Wondo, and on the northeast by Hula. Kewado town is the administrative center of Darra Woreda. The location of Kewado is 76 km from Hawassa the capital city of Sidama regional state and 350 km from Addis ababa which is capital of Ethiopia. Kewado Town is Latitude of: 6°28’22’’N-6°34’41’’N and Longitude of: 38°20’31”E-38°27’45’’E .The altitude ranges from 1644m to 1861m.

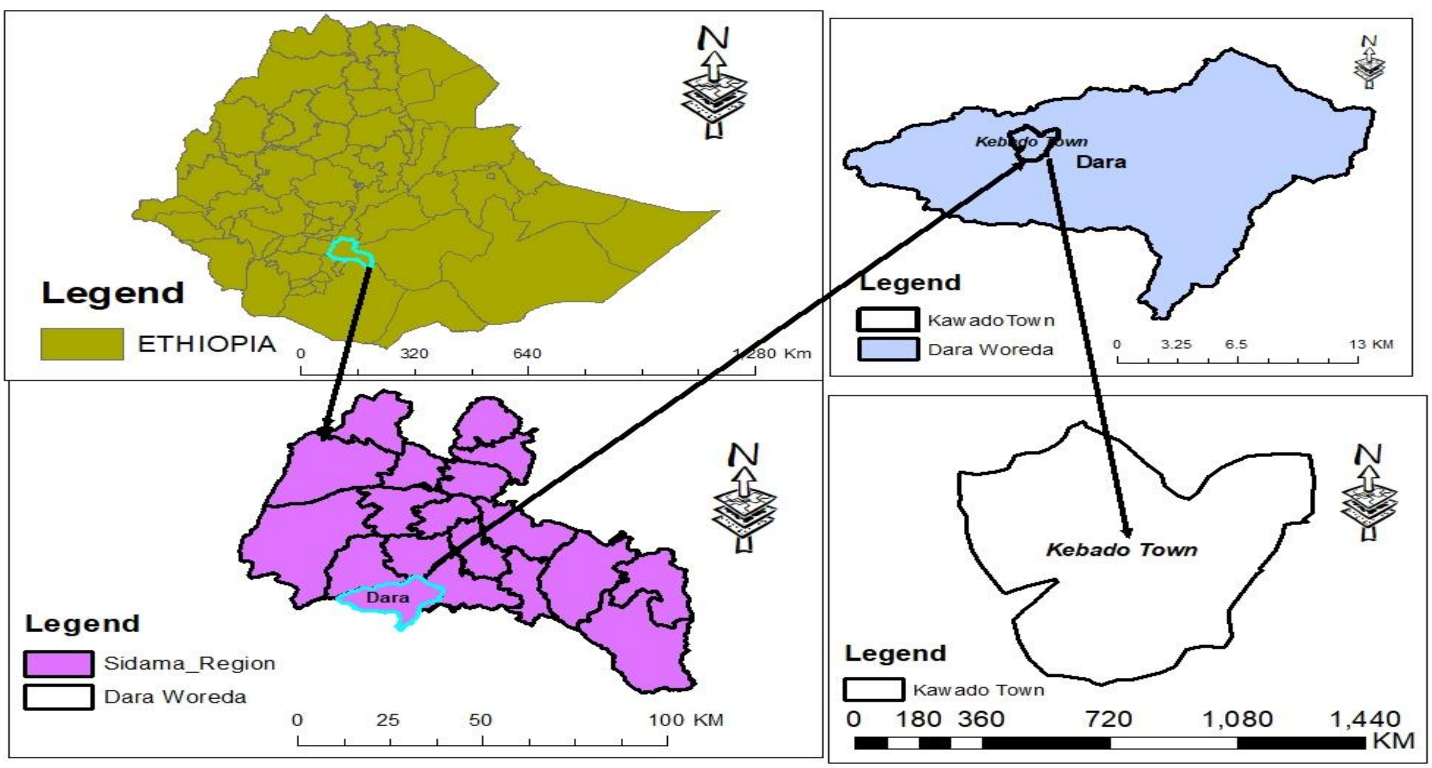

Figure 3.1: Location map of study area

3.1.2. Population

Based on CSA 2007 Kewado town has total population of 9,817 of whom 5,175 are men and 4642 are women.

3.1.3 Climate

The sources of moisture for almost all rains in Ethiopia are the Indian and Atlantic Oceans (Degefu, 1987). Based on the general classification of Agro- climate zone (on the bases of annual rainfall, temperature, length of growing period and plant types), the year is divided into two seasons: a rainy season mainly is on the months of June to September, and a dry season is from October to March. The amount of rainfall in Ethiopia is influenced by the location of the place relative to the source of moisture, the direction of winds and topographical relief. Regarding the climatic conditions of Kewado town**,** Kewado has a Woina Dega agro-climate, According to development Partners **(**2007**)**

3.1.3.1 Rainfall

According to meteorological data the year 1990-2019 the highest rainfall is registered in July and is 95 mm while, the lowest is in April and is 7.8 mm approximately 4 rainy day in the month

Table 3.1: Annual maximum rainfall in daily (mm/day) basis for 1990 to 2019

Year		1	2	3	4	5	6	7	8	9
1990	46.1	51.5	95	44.8	42.4	67.1	76	56.2	65.7	65.4
2000	34.5	41.3	55.6	38.9	43	30.9	44.3	63	62.4	52.4
2010	56	80	38	55	42.6	30.6	55.1	65	79.5	64.2

3.1.3.2 Temperature

The minimum and maximum daily temperatures are observed in the month of March are 12.5 and 29.3 degree centigrade respectively.

3.1.4 Topography features of the town

3.1.4.1 Slope of the town

Slope is rise or fall of land surface and it is defined as the ratio of difference in height by horizontal distance between two points.

Generally the topography of the town is sloping south north with a gentle slope ranging between 0-18%.

3.1.4.2 Altitude of the town

The altitude of the study area ranges from 1644 meters to 1861 meters above mean sea level and the average altitude of Kewado town is 1752. m.a.s.l. This altitudinal condition categorizes Kewado to be a Woyna Dega zone of medium altitude and climate.

3.1.4.3 Land Use/Land Cover

The land use land cover data is an essential input for the calculation of the runoff coefficientin the determination of runoff rate using the rational method. The Classification of different land use land cover was done bythe use of Google Earth pro and verified by field survey. The land use land cover of the study area was classified as Residential, Commercials, Administrations, Public Institutions (Schools, Health centers, Churches, Mosques), Open spaces, Roads (Asphalt, Cobbles, Red Ash, and Earth Pressed), Undeveloped Areas, Agricultural and Forest Areas.

3.1.5. The soil of Study Area

Soil data is another important parameter for the determination of runoff coefficient to compute peak flow. According to FAO Soil Map of Ethiopia classification (FAO, 1998) and ministry of agriculture, the Soil type of study area was classified as pellic vertisols with D hydrological soil group.

Pellic Vertisols: Heavy clay soils with a high proportion of swelling clays. These soils form deep wide cracks from the surface downward when they dry out are medium.

Hydrological soil group D: Clay loam, silty clay loam, sandy clay, silty clay or clay.Soils having a high runoff potential due to very slow infiltration rates. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a clay layer at or near the surface, and shallow soils over nearly impervious parent material.

3.1.6 Existing drainage networks of the town

The existing storm drainage systems in Kebado town can be divided into closed and open drainage lines. There are some areas with closed drainage systems, particularly along the town major routes. Open drainage channels, constructed by masonry are found along town mains and local roads. The stormwater drainage system collects the runoff and discharges to neighborhood through north directions. The drainage systems overflow as a result of being filled with solid waste and soil; and insufficient capacity of the drainage infrastructures

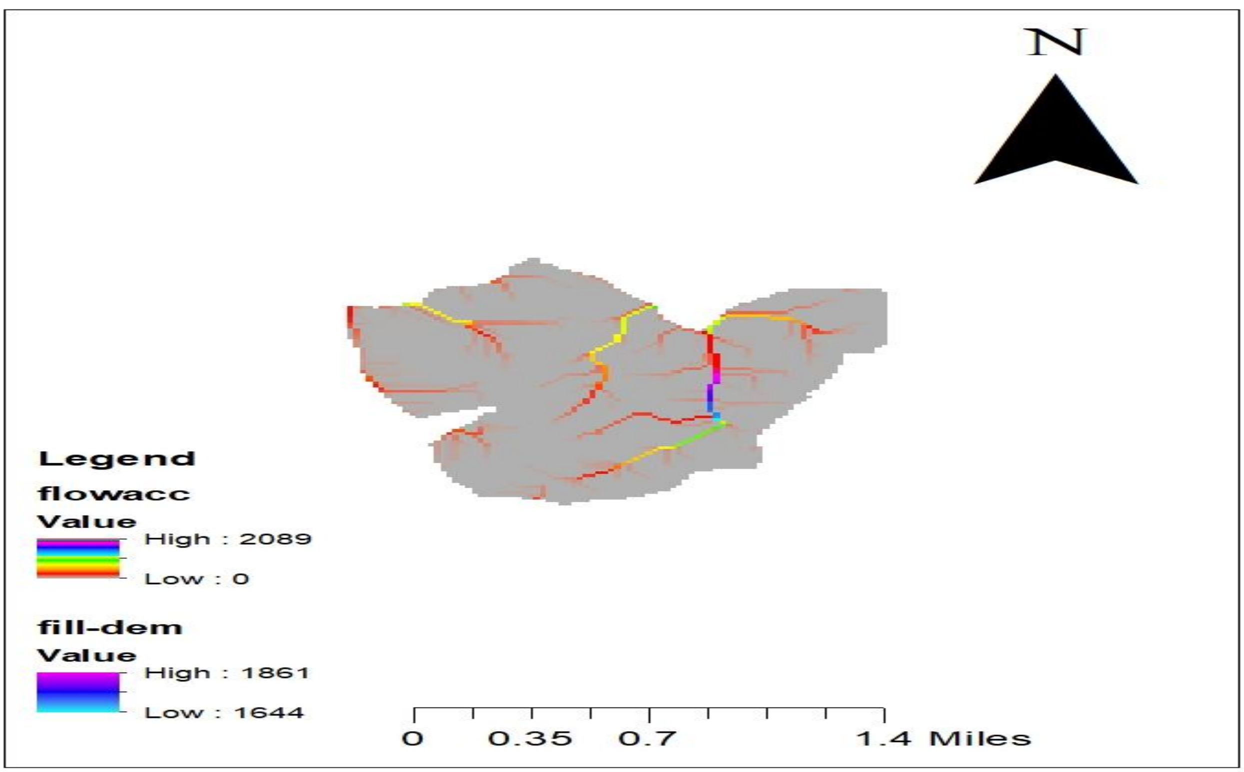

Figure 3.0: The existing natural drainage networks of Kebado town (Source: ArcGIS)

3.2 Data Collection and Analyses

The primary task was collecting relevant information and data of the study area.

This section identifies and discusses the types of data and sources used for the study

- **Data necessary for urban drainage system and their sources**

- Hydro-meteorology data from Ethiopian Meteorological service Agency
- Slope, elevation and drainage density-derived from DEM using spatial analyzing tools.
- Elevation map of Kewado town
- Land use land cover from Kewado town plan study.

3.2.1 Primary Data

Topography survey (visit) of the study area is carried out to determine the existing performance condition of drainage structures.The field servey includes observing flood marks, measuring the size of the existing drainage structures, measuring the elevation difference between river/stream bed and flood mark as well as gathering information about the overall performance of drainage structures during the rainy season.

The materials that are used for the study of the research are digital camera, GPS device and measuring tape. All these materials are used during field visit of the study area for getting my primary data.

3.2.2 Secondary data

Secondary data were collected from various sources, which include:

Table 3.2 List of secondary data collected

Data type	Source	Purpose
Meteorological data ❖ Daily Rainfall data (1990-2019)	National Meteorological agency	To formulate IDF curves
Land surface data ❖ Digital Elevation Model of the area (Resolution of 30 meter X 30 meter)	Satellite image from USGS (United States Geological Survey)	To make watershed & topographic map of the area.
Master plan of the town	kewado Municipality	To examine the existing land use with respect to the structural plan
Soil data for the town (texture)	kewado master plan Socio economic report	To use as an input for the model
Contour map	Kewado town municipality	to delineate a watershed boundary

- **Equipment**
- In the process, Digital-Camera, Global Positioning System (GPS) and meter were frequently used to collect data from the field and to take photographs in some important locations. GPS was used to collect data about location(X & Y coordinates) of drainage infrastructure as well as elevation (Z).The GPS was working with ±3m accuracy. The meter was used to measure dimensions of drainage infrastructures without a significant accuracy problem

- **Software Package**

This research is completely dependent on computer software and applications. Software and computer application which have been used in the research process are listed in Table3-3.

Table 3.3: Lists of Software and computer applications.

Software/Application	Used for
Google Earth pro	To select alternative model areas. To do previous & existing land use of the model area. To draw road/drainage network of the study area.
Arc-Gis (10.7 version)	To identify stream and to do flow direction & watershed of the area. To do topographic map of the study area. To do land use classification of the study area.
Digital Elevation Model (DEM)	An input for Arc-GIS to do watershed and related issues.
Global mapper	To do contour map of the study area.
Microsoft excel, Microsoft word	Large data manipulation, data analysis and manuscript writing.
SWMM	Storm water quantity simulation

3.2

3.3 Hydrologic Analysis

3.3.1. Rainfall data

Hydrological modeling has been widely used in numerous fields such as flood forecasting, water resource estimation, water project planning, groundwater fluctuations, hydrological station planning, and water quality monitoring and sediment load estimation. At present, a significant goal of existing research efforts in hydrological modeling is to improve the accuracy of runoff simulation and prediction. Rainfall is regarded as the key driving input of hydrological modelsand it is impossible to produce accurate runoff predictions if forced with inaccurate rainfalldata (Beven, 2001).

Rainfall is an important part of the hydrological cycle. One of the first steps in any hydrological and meteorological study is accessing reliable data. However, precipitation data is frequently incomplete. The incompleteness of precipitation data may be due to damaged measuring instruments, measurement errors and changes to instrumentation over time. In countries where it has not been possible to accurately and consistently record precipitation data in a particular time section, it is necessary to use methods to estimate the missing precipitation data and apply it in hydrological models. Daily Rainfall data of the study area were obtained from the Ethiopian National Meteorological Service agency for 29 years. But, due to the incompleteness of rainfall data, two neighboring rainfall stations were also taken for estimating the missing data. The summary of the missing percentage is presented in Table3.3

3.3.2 Filling Missing Data

Table 3.4 : The percentage of missed data at each stations

Stations	Observed Year	Available data (day)	Observed data (day)	Missing data (day)	% of missing
Kewado	29	10585	10440	145	1.37
Teferkela	29	10585	10440	145	1.37
Aleta Wendo	29	10585	10250	335	3.16

Kewado Station and Teferkela have adequate daily records and a few missed data. The station missed information in percentage and the total missed data is less than 10%,

the available data is representative. Therefore, the missed values were filled for the given stations by simple Regression method using equation:

PX = bo+b1P1+ b2P2+ …. + bnPn……………………………………………3.1

Where

PX is the missing precipitation value for station X for certain time period

P1, P2, …, Pn are precipitation values at the neighboring stations for the same period

b0, …, bn are coefficients calculated by least-squares methods

n is the number of nearby gages.

3.3.3 Data processing and quality checking

3.3.3.1 Data quality checking for consistency

The consistencies of the data set of the given stations were checked by the double mass-curve method in-reference to their neighborhood stations. The double mass curve was plotted by using the annual cumulative total rainfall of the base station as ordinate and the average annual cumulative total of neighboring stations as abscissa.

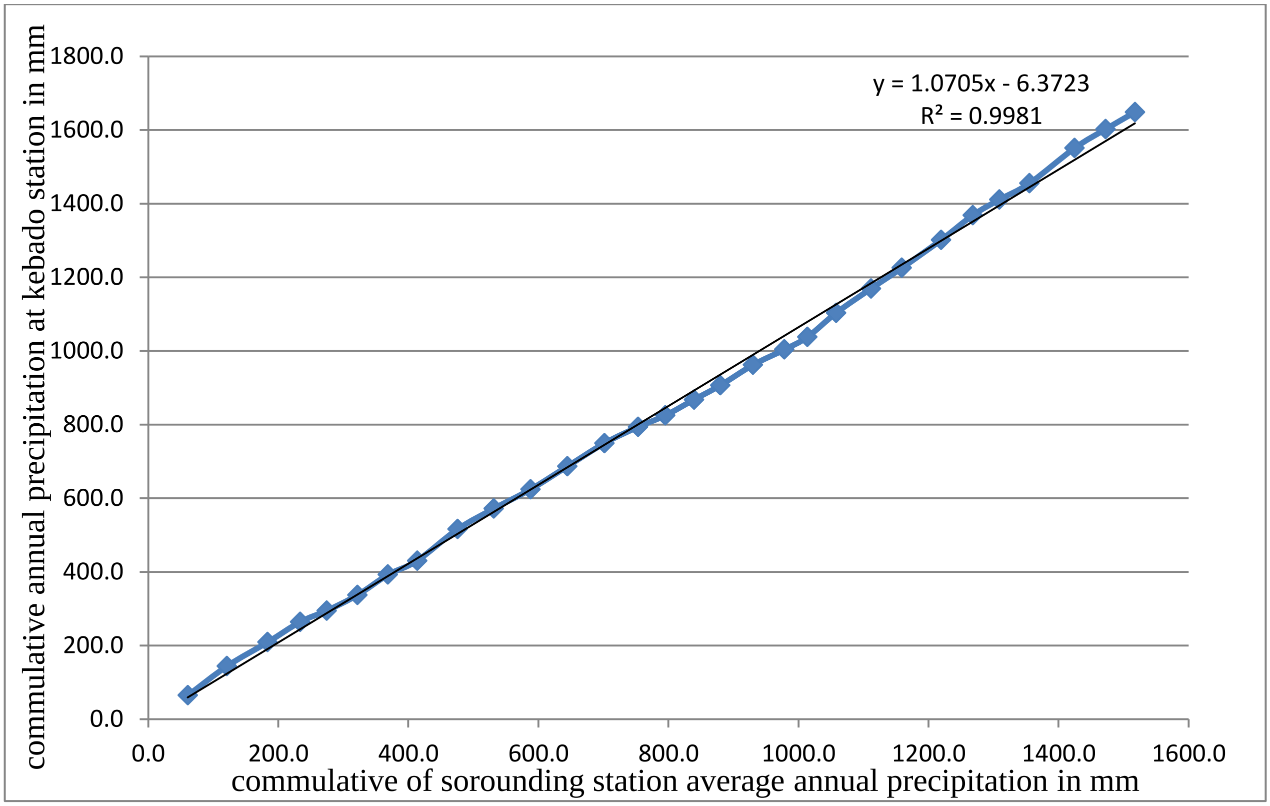

Figure 3.3: Double Mass Curve

3.3.3.2 Data quality checking for higher and lower outlier

From the annual daily maximum rainfall data, the highest and lowest values are **110.20**mm and **2.64** mm respectively those values are less than the higher outlier and greater than lower outlier. Therefore, no higher and lower outlier eliminated. finding of Those value are explained in appendix part.

3.3.4 Rainfall Frequency Analysis

Hydrologic processes such as floods are complex natural events. To analyze the maximum discharge expected in T years we can use the frequency distribution function listed below, but the data in hand may fit only one of them. Some of the commonly used frequency distribution functions for the prediction of extreme maximum values. (Subramanya, 1994):

3.3.4.1 Log-Pearson Type III Distribution Method

It is a three-parameter gamma distribution with a logarithmic transform of the variable. Mean, standard deviation, and coefficient of skewness were the three necessary parameters that are necessary to describe the distribution. In this method, the flow data is first transformed into logarithmic form (base ten) and the transformed data were analyzed. If X is the variety of random flow series, then the series of Z varieties were obtained for this series for any recurrence interval T. The logarithms of daily maximum values are calculated, Z = log$X$. The average (=average ()), standard deviation (=stdev ( )) and standardized skew (=skew ( )) of the data set are obtained using the respective excel functions.

Z = log X…………………………………………………………………...3.2

ZT= $\bar{a}$+KZ* δ………………………………………………………………..3.3

Where, KZ = a frequency factor which is a function of

T and the coefficient of skewness, Cs= standard deviation of Z variety sample.

$$\text{Standard deviation}(\delta z) = \frac{\sqrt{\sum(z-zm)^2}}{N-1} \text{……………………………………………………...3.4}$$

$$\text{Skweness coefficient}(Cs) = \frac{N\sum(z-zm)^3}{(N-1)(N-2)\delta z^3} \text{…………………….………………3.5}$$

The variation of Kz = f (CS, T) is given in appendix 4,

Lastly, convert to XT as follows for comparison with the others

XT =10^zt ………………………………………………………………………3.6

The relation adopted for IDF development at a given station, all probability distribution function were compared by chi-Square test of goodness fit and selecting the best probability function can be used for this is research is **log Pearson type III distribution method**.

3.3.5 The goodness of fit tests (GoF) of the probability distributions

The best fit probability distributions were identified based on the result of GoF obtained from different tests. There are several tests of goodness of fit tests that exist in determining which distribution is the best model for this study.

3.3.5.1 Hydrognomon 4 software

Rainfall frequency analysis of 24 hours daily maximum rainfall records of the study area was carried out for different distributions. Comparison among the goodness of fit assessments for the different distributions indicates that station data is found to be more appropriate for fitting the empirical points for Kebado meteorological and it is used for computation of magnitude of maximum. Hydrognomon 4 software shows how the recorded data is related at a different level of significance and how the record data deviates from the mean value.

3.3.5.2 Easy Fit 5.5 professional software

This software is used to identify which distribution fits the theoretical probability distribution. Kolmogorov-Smirnov test, Chi-Squared test, and Anderson-Darling were used in this study for the GoF test using Easy Fit 5.5 professional software.

- **Kolmogorov-Smirnov Test**

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). When comparing different distribution, lower statistics mean a better fit.

- **Anderson-Darling Test**

The Anderson-Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. When comparing different distribution, lower statistics mean a better fit.

- **Chi-Squared Test**

The Chi-Squared test is used to determine if a sample comes from a population with a specific distribution. This test is applied to binned data, so the value of the test statistic depends on how the data is binned. When comparing different distribution, lower statistics mean a better fit.

3.3.5.3 Correlation Coefficient (R^2)

Correlation Coefficient is Statistic showing the degree of relation between two variables. It measures the nature and strength between two quantitative variables, i.e. the sign of $\mathbf{R^2}$ denotes the nature of association while the value of $\mathbf{R^2}$denotes the strength of association.

3.4 Intensity–Duration–Frequency Curves (IDF)

Rainfall intensities of various frequencies and durations are important parameters for the hydrologic design of storm sewers, culverts, and other hydraulic structures. This can be achieved by the rainfall Intensity–Duration–Frequency (IDF) relationship, which is determined through rainfall frequency analysis. The order statistics approach is applied for the determination of distributional parameters in order to estimate rainfall and develop IDF. Relationships for different return periods. Goodness-of-Fit (GoF) tests are employed for Checking the adequacy of fitting the distributions to the recorded data.

3.4.1. Design rainfall of shorter duration

The rainfall depths obtained from gauging station of 24hr duration depth. Design and analysis of drainage structures require rainfall intensity duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable appropriate IDF derivation for shorter duration is required. Ethiopian Road Authority Drainage Design Manual of 2013 suggests the following equation.

$$R_{Rt} = \frac{t}{24}\frac{(b+24)^n}{(b+t)^n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.10$$

Whereas:

RRt = Rainfall depth ratio (R_t: R_{24})

Rt = Rainfall depth in a given duration t

R24 = 24hr rainfall depth and coefficients b = 0.3 and n = 0.78 - 1.09

Using b = 0.3 and n = 0.92 as suggested by ERA manual results are tabulated for rainfall durations 5, 20, 30 … 180 minutes. The maximum peak flood computed using the return period recommend in ERA Drainage Design Manual 2013 considering the road standard and the design life span of the structure,10 years for design return period were adopted (design storm frequency(yrs) by Geometric Design Criteria (ERA DDM, 2011).

3.4.2 Peak Discharge

The hydrological study is the basic step that should be done carefully in every flood management and road drainage facilities. The hydrological investigation was dealt with analyzing rainfall data, water level depth observations, topographical maps, satellite image and aerial photographs for the estimation of design discharge. The investigation has also been supplemented by field inspection of the study area and interviews. It is known that there are many methods used for peak flood calculation, but their applicability depends mainly on the availability of hydrological data. Based on the aforementioned concepts, rational methods were used for this study according to the area of the catchment

3.4.3 Evaluating Existing Drainage Capacity by Manning Formula

Manning's formula was used for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for the flow of a specified depth in a canal of known diameter. The formula was applicable for a constant flow rate through the channel with a constant slope, size, and shape, roughness, also geometry and canal type. The hydraulic design of the existing drainage system would be checked using the Manning formula as equation 3.11

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \quad \text{..........3.11}$$

Where, Q = the volumetric flow rate passing through the channel reach in m^3/sec.

A = the cross-sectional area of flow normal to the flow direction in m^2

S = the bottom slope of the channel in m/m (dimensionless).

P = the wetted perimeter of the cross-sectional area of flow in m.

n = a dimensionless empirical constant called the Manning roughness coefficient

R = the hydraulic radius, which is the cross-sectional area of flow normal to the flow direction in m^2 divided by the wetted perimeter of the cross-sectional area of flow in m

Manning's Formula can be used to compute the average flow velocity (V) in any channel ornatural stream with uniform flow.

Manning's equation was used for calculating the cross- sectional area, wetted perimeter, and hydraulic radius for the flow of a specified depth in a pipe of known diameter and/or stream channel cross-section.

To calculate average flow velocity:

V=1/n*R^2/3*S^1/2…………………………………………………..3.12

Whereas:

S = Channel slope in meter per meter ($^{m}/_{m}$)

R = hydraulic radius in meter (m)

n=manning's roughness coefficient

V = Average flow velocity in meter per second ($^{m}/_{s}$)

Roughness Coefficient (n) Varies considerably, depending on the characteristics of a channel or the smoothness of a canal, pipe, etc. Roughness coefficients represent the resistance to flood flows in channels and flood plains. Smooth, open stream channels with gravel bottoms have values around 0.035-0.055. Very winding, vegetated, or rocky channels have values around 0.055 to 0.075. Smooth earth or rock channels have values of 0.020 to 0.035. Roughness values typically increase as channel vegetation and debris increase, as channel sinuosity increases, and as the mean size of channel materials increases. The value decreases slightly as flow depth increases (ERA DDM, 2013). .

3.4.4 Design flood Estimation using Rational Method

The rational method is the most commonly used model for storm drainage systems, is primarily based on the concept that peak discharge from watershed would occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow. In the design of stormwater drainage systems, the main purpose of the hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that can be accumulated at certainstorm drainage outlet (usually a ditch) along a highway.

The rational method is one of the deterministic methods and recommended for the size of the catchment area less than 80ha (US Department of Transportation 2009).

This method was used in this thesis to calculate the peak runoff of the existing carrying capacity of the main drainage canal system for a certain return period of rainfall intensity as equation 3.13.

$$Qp = 0.00278CIA \qquad (3.13)$$

Where: Qp - Peak run-off the catchment (m3/sec),

C=Runoff coefficient of the catchment

I = rainfall intensity in (mm/hr.)

A = catchment area in (ha).

urban environment especial land use and land cover, in order to obtain Land use area A (ha), Runoff coefficient (Ci), Weighted runoff coefficient (CW), Catchment area (ha) and intensity(I) . It is also used to have the most recent data of the study area in the urban drainage system of the catchment as well as sub-catchment. The main input variable of the rational method: Rainfall intensity, Time of concentration, Rainfall duration, Rainfall frequency, Catchment area.

a) Runoff Coefficient (C)

The runoff coefficient C is the function of the landuse land cover of the study area and used as input for peak discharge estimation. The more the surface is impervious the higher the runoff would be as the infiltration decreases. The rainfall intensity directly affects the runoff coefficient. Vegetation cover reduces the impact of a raindrop on the ground and intercepts some of the rain on its leaves and branches letting them evaporate. This directly decreases the runoff coefficient. A weighting method is employed to obtain the representative runoff coefficient i.e. the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area (Ven Te Chow, *et al*, 2012).

Cw = (A1C1 + A2C2 +…-+ AnCn) / (A1 + A2 +---+ An)………………………………3.14

Where: Cw= Weighted Runoff Coefficient, C1, C2,

Cn = coefficient of runoff for parts of the drainage area. A1, A2,

An = parts of drainage areas with different runoff coefficients.

b) Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. Rainfall Intensity is a function of geographic location, design exceedance frequency (or return interval), and storm duration. The magnitude was carried out by using return period maps and intensity-duration-frequency (IDF) Curves. The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period (the annual frequency of exceedance). For this research, The IDF Curves developed by ERA and by this study were compared.

c) Time of Concentration (Tc)

The time of concentration is the time required for runoff to flow from the most remote point of the basin to the point of interest. In a rational method, it was used to determine the rainfall duration which would result in maximum runoff. The required design rainfall intensity was established from the IDF curve for the required recurrence interval.

Tc = Tc inlet + Tt………………………………………………………………………3.15

Where: Tc = time of the concentration (minute),

Tci = time of concentration inlet (hrs.) and

Tt = the travel time (hr)

- **Time of concentration inlet (Tci)**

The time of concentration of drainage area is the time required for runoff from the farthest partof the drainage area to reach the outlet used as the duration for the design storm. The inlet time for the time of concentration is estimated using Airport or Federal Aviation Administration (1970) methods. This method could desirably be used for inner areas especial for developed areas of urban centers as equation 3.16.

Tci=(3.64(1.1-c)L/1000)/H^0.33……………………………………………..3.16

Where: Tci = time of concentration inlet (hrs.),

C = runoff coefficient

L = flow length from the remotest point to the point t of interest (km).

H = elevation difference between the upstream and downstream of the canal (m)

.

d) Travel Time

Water moves through a catchment area as sheet flow, shallow concentrated flow, open channel flow or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection. Travel time is the ratio of flow length to flow velocity as equation 3.17.

T_t=L/(3600V)……………………………………………………………………..3.17

Where: T_t = travel time, hr

L = length of the main drainage canal (m)

V = flow velocity in the channel (m/s)

3600= conversion factor from seconds to hours.

e) Catchment Area (A)

The catchment area can be determined considering land cover with runoff coefficient depending on land types of the study area from topographic maps that contribute to the canal. For large catchment areas, it is 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 and for locating stormwater drainage systems and assess their effects on the flood flows

3.5. Catchment Delineation

Sub-catchments are hydrologic units of land whose topology and drainage system elements direct surface runoff to a single discharge point. Evaluating the whole catchment is not necessarily important to come up with solution for stormwater drainage problem. In this study representative major flood prone area contributing to around Kewado mazegaja were analyzed. The natural catchment area was delineated using Arc Hydro tools 10.3 based on DEM as input and the shape file was importedin to the software and overplayed with the CAD flow direction of drainage line, the sub-catchments was traced based on blocks polygon in the hydraulic model. The area which is 72.8 ha delineated to investigate the hydraulic performance of the existing drainage system with respect to hydrology. This area was used for computation of discharge using both rational method and swmm model.

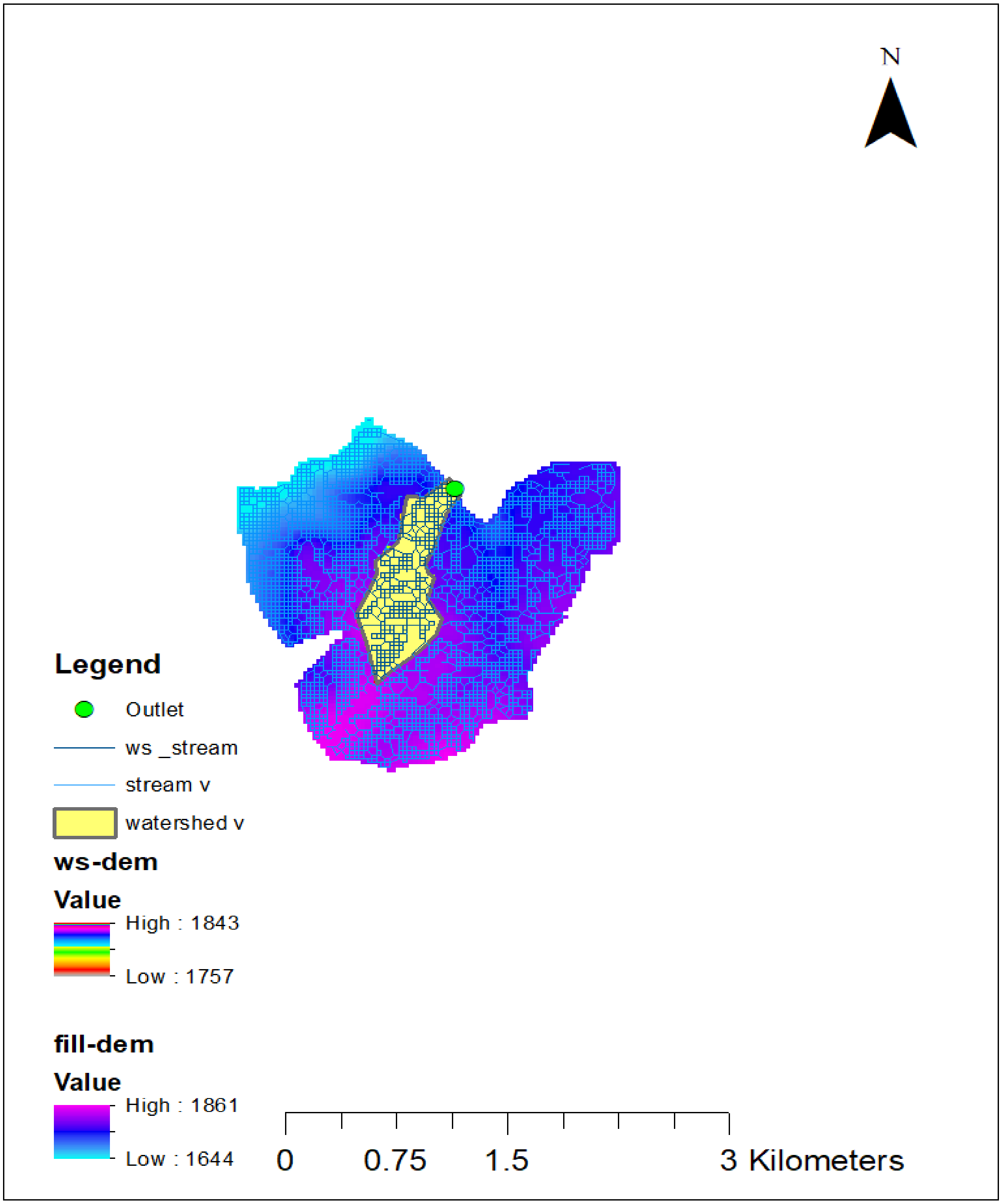

Figure 3.4 Location of Delineated model area

3.6. Storm Water Management Model (SWMM)

In this study, the rational method and Stormwater management model (SWMM) was used forthe design flood computation and its analysis. The rational method would be compared with the existing carrying capacity of the main drainage canal system for a certain return period of rainfall intensity. Whereas the stormwater management model is one-dimensional models which allow the flow properties to vary along or within the channel only rather than to account the changes across the channel (Rossman, 2004).

3.6.1. Model Governing Equation

Flow routing within a conduit link in SWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e., the Saint Venant flow equations). The Saint-Venant equations consist of mass Conservation equation (Equation (20)) and momentum Conservation equation (Equation (21)) for gradually varied, unsteady flow.

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \quad \text{3.18}$$

$$\frac{\partial h}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial X} + \frac{1}{g}\frac{\partial V}{\partial t} = S_o - S_f \quad \text{3.19}$$

Where : Q is discharge, A is area of flow cross section, h is depth of water, x is longitudinal distance, t denotes time, v is velocity, g is gravitational acceleration, S_0 is channel slope term and S_f is friction term (W. Chen & G. Huang, 2017).

a) Steady Flow Routing

Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The Manning equation is used to relate the flow rate to the flow area (or depth). This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing is insensitive to the time step employed and is really only appropriate for preliminary analysis using long-term continuous simulations.

b) Kinematic Wave Routing

This routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit. The maximum flow that can be conveyed through a conduit is the full-flow Manning equation value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available.

Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However, this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow, and is also restricted to dendritic network layouts.

c) Dynamic Wave Routing

Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flows in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices (Rossman, 2006).

3.6.2. Model setup

The simulation software package selected for this study was the EPA Storm Water Management Model (SWMM 5.2). Due to the fact that the design represents a drainage system of a new development with no calibration (measured data not available), all the model parameters had to be estimated either from information included in the consultancy design,the SWMM manual, existing literature or similar case studies (Tsihrintzis and Hamid 1998; Rossman 2010).

The first step in the model setup was building a conceptual model divided into sub-catchments that represent the study area itself as well as other contributing catchments. Sub catchment information such as area and slope were obtained, while typical values from the literature were taken for Manning for pervious and impervious areas. In order to be able to alter the width parameter, the average maximum length of each sub-catchment was chosen to be altered, according to the equation below (Rossman, 2010):

W = A/L………………………………………………………………….... (3.20)

Where: A is the area of each sub-catchment (m2)

L is the average maximum length of each sub-catchment (m)

3.6.3. Input parameters of the model and their properties

Parameters included in Sub-catchments, junctions, links, and outfalls were used for the simulation of the model. the following parameters were used

Table 3.5: some key parameters used for sensitivity analysis in this study and Their allowed range of change

Parameter	Description	allowed range
N-Imperv	Manning's roughness coefficient for impervious areas	0.005-0.05
N-Perv	Manning's roughness coefficient for pervious areas	0.05-0.5
Dstore-Imperv	Depth of surface storage in impervious areas (mm)	1.3-2.5
Dstore-Perv	Depth of surface storage in pervious areas (mm)	2.5-7.6

The above parameters were fixed based on the flow rate estimated with the manning equation and flow rate simulated by the model. Hence, the flow rate obtained for the same sub- catchment was used to compare the estimated flow rate with the simulated.

Table 3.6: Rain gage Properties

Name	User assigned rain gage name
Rainfall data type	The format in which the rain data are supplied. **Intensity**-each rainfall value is an average rate over the recording interval (mm/hour). **Volume**- each rainfall value is the volume of rain that falls in the recording interval (mm). **Cumulative** – each rainfall value represents the cumulative rainfall that has occurred(mm).
Data source	Source of rainfall data; either TIME SERIES for user-supplied time series data orFILE for the external data file.

Table 3.7: Sub-catchment properties

Name	User assigned sub catchment name
Rain gage	Name of the rain gage associated with the sub-catchment.
Outlet	Name of the node or sub catchment which receives the sub-catchment runoff.
Area	Area of the sub- catchment in acres or ha.
Width	Characteristic width of the overland flow path for sheet flow runoff (m).
% Slope	Average percent slope of the sub catchment.
% Imperv	Percent of the land area which is impervious.
N-Imperv	Manning’s n for overland flow over the impervious portion of the sub catchment.
Infiltration	Used to edit infiltration parameters for the sub-catchment.
Land uses	Uses to assign land uses to the sub-catchment.

Table3.8: Junction properties

Name	User assigned junction name
Invert El.	Invert elevation of the junction (m).
Max. Depth	Maximum depth of junction (m).
Initial Depth	Depth of water at the junction at the start of the simulation (m).

Table 3.9: conduit properties

Name	User assigned conduit name
Inlet Node	Name of the node on the inlet end of the conduit at a higher elevation.
Outlet Node	Name of the node on the outlet end of the conduit at the lower elevation.
Shape	Used to edit the geometric properties of the conduit’s cross section.
Length	Conduit length (m).
Roughness	Manning’s roughness coefficient for opened conduit and closed channel values.
Inlet Offset	Height of the conduit invert above the node inverts at u/s end of the conduit.
Outlet Offset	Height of the conduit invert above the node inverts at d/s end of the conduit.
Initial Flow	The initial flow of the conduit at the start of the simulation (flow units)

Table 3.3.4.1 : Outlet property

Name	User assigned outlet name
Inlet Node	Name of the node on the inflow side of the outlet
Outlet Node	Name of the node on the discharge side of the outlet.
Height	Height of outlet above inlet node invert (m)
Flag Gate	YES, if a flag gate exists which prevents backflow through the outlet, or NO if no flap gate exists.

3.6.4. Model Simulation

For the constant current land use condition with different rainfall intensity (especial heavy rainfall intensity of short durations), the flooding and other flow characteristics are high or cause flooding problems at the junctions and links. This is true when the maximum designed water level of the drainage canal is less than flood Level in the junctions and links of both upstream and downstream parts of the system.

3.7.1. Calibration and validation data

Flow data is required in order to calibrate and validate the model. However, flow data was not available due to the fact that flow gages were not installed to measure the quantity of flow generated from the drainage system of the town. Four conduits were selected for this purpose. Those were C2, C4, C5 and C7 which have an open rectangular channel with SC8, SC13,SC16 and SC20 sub catchments. This flow depth was recorded at different interval for eight (8) days from June to july to calibrate of sensitive parameters and validate SWMM model for the area.

Table 3.11: Recorded flow depth for calibration and validation model

Gauged area	Channel type	Date of observed data	Recorded average flow depth(m)	Method calculation flow discharge $Q = \frac{1}{n} A R^{2/3} S^{1/2}$
For calibration	Rectangular	June 26 2023	0.85	Each Values are filling in result and discussion part
		June.29 2023	0.64	
		July 06 2023	0.9	
		July 10 2023	0.86	
for validation	Rectangular	June. 28 2023	0.68	
		July. 9 2023	0.56	
		July 12 2023	0.82	
		July 15 2023	0.62	

The model was calibrated using the maximum flow depth (m) obtained from the previous table 3.6 for four days of rainfall data, and the remaining four days were used to validate the model.

The manning equation is used as a calculating tool for calibrating and validating observed data.

$$Q = \frac{1}{n} * A * R^{2/3} * S^{1/2} \text{.........} 3.14$$

Whereas: - Q=flow discharge in m^3/s

n=manning roughness coefficient

A=cross-sectional area in m^2 that derived from recorded flow depth (m) with given drainage dimension

R (A/P) =hydraulic radius in m

P=wetted parameter in m

S=channel slope in m/m fraction

3.7.2. Model Performance Evaluation

The reliability of the model was tested using Coefficient of Determination (R^2) and Nash– Sutcliffe efficiency coefficient (NSE).

$$\text{Coefficient of determination}(R^2) = \left[\frac{\sum_{t=1}^{n}(q_t^{obs} - q_t^{Av.obs})(q_t^{sim} - q_t^{Av.sim})}{\sqrt{\sum_{t=1}^{n}(q_t^{obs} - q_t^{Av.obs})^2}\sqrt{\sum_{t=1}^{n}(q_t^{sim} - q_t^{Av.sim})^2}}\right]^2 \text{.........} 3.15$$

The R2 value, which ranges from 0 to 1, indicates how much of the observed dispersion is explained by the prediction. A value of zero indicates that there is absolutely no correlation, while a value of one indicates that the dispersion of the prediction and the observation is the same (J.Y. Wu et al., 2013).

$$\text{Nash} - \text{sutcliffe coefficient(NSE)} = 1 - \frac{\sum_{t=1}^{n}(q_t^{obs} - q_t^{sim})^2}{\sqrt{\sum_{t=1}^{n}(q_t^{obs} - q_t^{Av.obs})^2}} \text{.........} 3.16$$

The value of NSE ranges from 1 to - ∞.When NSE value is 1, the model performance is perfect. If NSE value is negative, it implies that the performance of the model is poor.

Whereas q_t^{obs} and $q_t^{Av.obs}$ = are the calculated and average flow respectively

q_t^{sim} and $q_t^{Av.sim}$ = are the simulated and average flow respectively

t=at time t and

n= is the total number of time steps.

Table 3.12: Performance evaluation criteria for both methods (Moriasi, 2015).

Method	Output Response	Temporal Scale	Performance Evaluation Criteria			
			Very Good	Good	Satisfactory	Not satisfactory
R^2	Flow	Day/Month / Year	R^2>0.85	0.75<R^2≤0.85	0.60< R^2≤0.75	R^2≤0.60
NSE	Flow	Day/Month / Year	NSE>0.85	0.75<NSE≤0.85	0.60<NSE≤0.75	NSE ≤0.60

4. RESULTS AND DISCUSSION

4.1 Fitting probability distribution

The goodness of fit tests was ranked from one (best fit) to four (least fit) for all probability distributions. The selection of the best fit probability distribution is based on the total score from all the goodness of fit tests. All developed probability distributions are ranked with each selection tool (rank one is the best fit). The three ranking results are summed to yield a ranking score. The distribution model with the smallest ranking score is selected as the best- fit distribution model. The results of the goodness of fit tests for each probability distribution for this study and for the others are shown in below,.

Table 4. 1: Goodness of fit tests with Easy fit 5.5 software

Distribution Model	Kolmogorov Smirnov test		Chi-squared test		Anderson Darling Test		Total
	Statistic	Rank	Statistic	Rank	Statistic	Rank	
Normal	0.1721	4	3.1543	4	1.5308	4	12
Log normal	0.1372	3	2.2334	3	0.78984	3	9
Log-Pearson III	***0.1023***	***1***	***0.8865***	***1***	***0.39723***	***1***	***3***
Gumbel	0.1142	2	1.2858	2	0.5765	2	6

To conclude, for selecting the best fit distribution ranking system was made, with rank 1 being the best, two the second best and so on. The distribution type with the lowest sum of the three rankings was selected as the best fit distribution for the station. Log-Pearson type- III provide the best fit at the rainfall gauging station analyzed in this study

4.2. Intensity-Duration-Frequency Curve (IDF)

The resulting IDF curve of Kewado town was developed from the steps shown above section 3.4. The IDF Curve of Kewado town was developed from 24-hour estimated rainfall data from 1990 to 2019, which was obtained from the computation of design rainfall from Ethiopian Meteorological agency rainfall gauge data located in Kewado town. The appropriate reduction equation described in the methodology section has been applied to estimate rainfall for shorten duration (Rt) and the corresponding rainfall intensities (It) as presented in appendix. Hence, the following intensity duration frequency curve was developed for this study from the computation results.

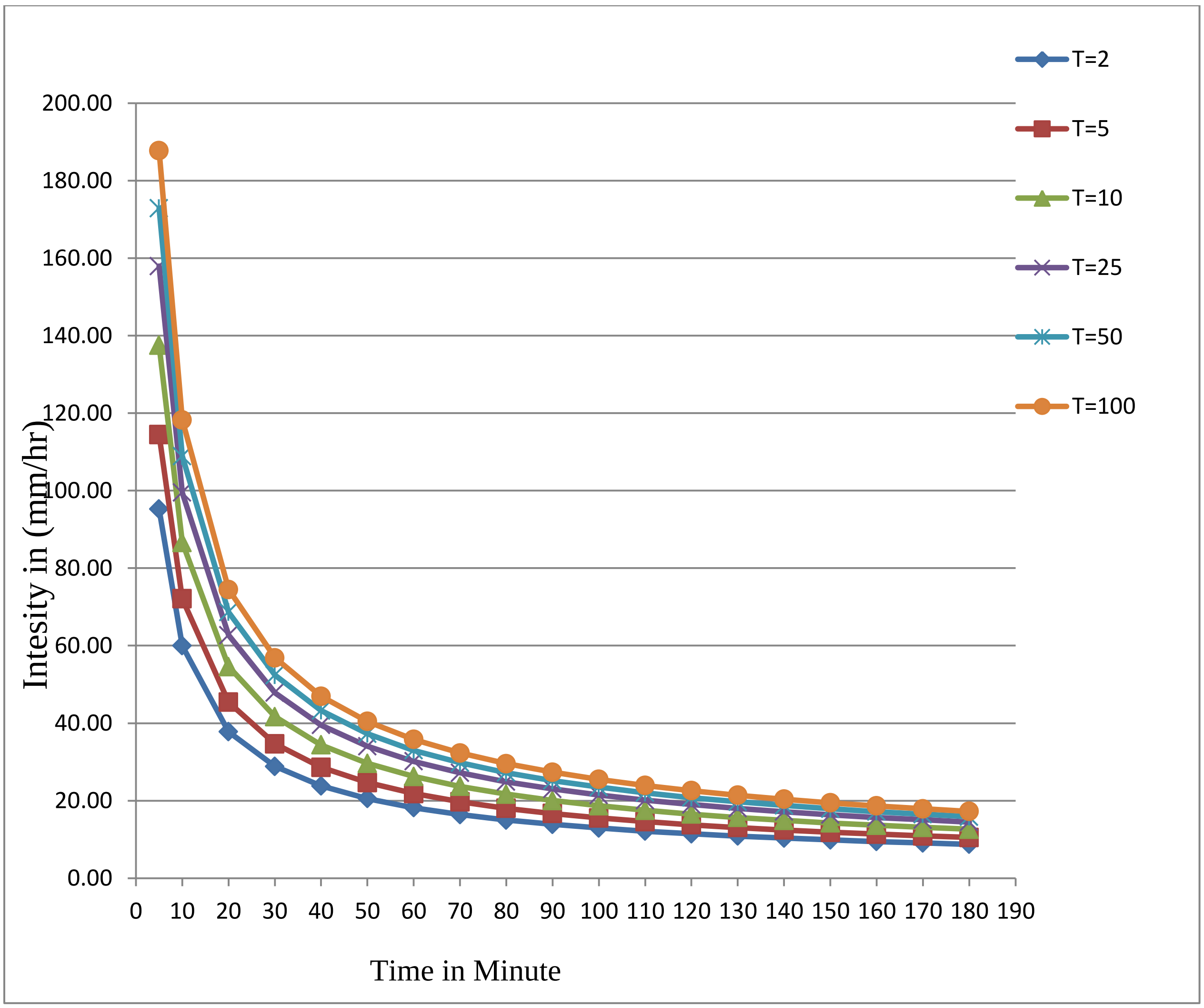

Figure 4.1: IDF curve of kebado town

Table 4.2: Comparison of IDF curves

Return period	*Duration in min.*											
	10		30		60		90		120		180	
	Kebado	ERA	Kebado	ERA	Kebado	ERA	Kebado	ERA	Kebado	ERA	Kebado	ERA
2	59.99	74	28.84	48	18.17	31	13.87	25	11.45	21	8.73	15
5	72.10	100	34.66	61	21.83	40	16.66	31	13.76	26	10.5	18
10	86.65	114	41.65	70	26.24	45	20.03	35	16.53	30	12.62	21
25	99.51	120	47.84	80	30.14	52	23.00	40	18.99	34	14.29	25
50	108.93	145	52.37	89	32.99	57	25.18	45	20.78	36	15.86	26
100	118.27	157	56.86	97	35.82	66	27.34	49	22.56	40	17.22	30

From the Table 4-5, the rainfall intensities developed by this study for the study area have smaller numerical values than ERA.

Kewado town is categorized hydrological section of B2 rainfall region. This is because ERA considered relatively the averaged values for the all zones which is included in the hydrological rain fall region B2 like Hawasa, Wolaita Sodo,arbamich and jimma..For this case Kewado town have lower value than ERA.

The engineer must undertake further research on category of rainfall region classification and revise the data is needed. Based on the statistical analysis undertaken as part of the current manual review work, the country could be further divided into more than 8 rainfall pattern regions. This task is outside the current scope of works and the current regional classification has been retained but this should be revised when the manual is updated in the future. The methodology adopted in the (ERA, 2002) manual to generate the IDF curves has also been adapted to update the IDF curves with the latest rainfall data (ERA, 2013). Therefore, the IDF developed in this study was used for this study.

4.3. Model Calibration and Validation

The calibration and validation of SWMM model in this work depends on sensitive parameter value modifying till to suit the observation is comparable to that obtained in the calibrate by using goodness of fit measured using performance simulation model.

4.3.1. Calibration of model

For the modeled accuracy and effective calibration of SWMM, the calibration process has been verified by altering sensitive parameters to varied model parameter input and checking the output model result until it matches closely within the required range in comparison with the observed results.

Table 4.3: Sensitivity parameter used to calibrate the model

Parameters	Manning Description	Recommended Value Range	Initial Used Proceed Sensitive Parameters	Final used Sensitive Parameters
N-Imperv	Manning roughness for impervious area	0.011- 0.015	0.012	0.013
N-perv	Manning roughness for pervious area	0.05-0.8	0.2	0.5
Dstore-Imperv	Depth of depression storage on impervious area	0-3	1.5	3
Dstore-Perv	Depth of depression storage for pervious area	3-10	3	7.6
Smooth concrete roughness	Manning roughness coefficient for open smooth concrete	0.0122	0.0122	0.0122

The input parameters needed to calibrate the model's final output are listed in table 4.3 above. Peak discharge flow rate determined using manning equations is 4.248m^3/s but peak discharge flow rate simulated using calibration values from the SWMM model is 4.01m^3/s.

4.3.2. Validation of model

The table 4.4 below lists the velocity and the discharge flow rate in the drainage systems along with a validation of this model's fit to the observed at the outlet one drainage lines.

Table 4.4: Validation flow depth observed result

Channel type	Record depth (m)	Manning roughness used(n)	channel slope (S)	Width (m)	Area(m^2)	Perimeter (m)	Hydraulic radius (m)	Velocity (m/s)	Discharge $Q(m^3/s) = \frac{1}{n}AR^{2/3}S^{1/2}$
Rectangular	0.68	0.013	0.0122	1.5	1.02	2.86	0.36	4.3	4.36
	0.56		0.0894	0.6	0.34	1.72	0.20	7.7	2.60
	0.82		0.0041	1.2	0.98	2.84	0.35	2.4	2.39
	0.62		0.003	1	0.62	3.24	0.19	1.62	1.46

According to result table 4.4, the channel velocity of 4.3 m/s and 7.7 m/s is high values. The material for the trapezoidal open channel is made of stone masonry with the floor stone pavement joined by concrete material. The material for the rectangular open channel is made of high smooth quality concrete reinforced and the channel has a high channel slope. Due to the great strength of the channel material used in construction, the high value of velocity shown in table 4.4 has no impact on the channel.

Table 4.5: Summary of the model result and observed values

Date of observed data	Recorded average flow depth(m)	Determine flow rate using manning equation $Q = \frac{1}{n} AR^{2/3} S^{1/2}$	Simulated flow rate using SWMM5.2 $(\boldsymbol{m^3/s})$
June. 28 2023	0.68	4.36	4.46
July. 09 2023	0.56	2.60	2.72
July 12 2023	0.62	1.46	1.79
July 15 2022	0.82	2.39	2.55

4.3.3. Model performance evaluation

With the help of the methods of Coefficient of Determination (R^2) and Nash-Sutcliffe Efficiency Coefficient (NSE) in the detailed evaluation shown in Table 4.6 below, the model performance was evaluated by comparing the observed and simulated data.

Table 4.6: model performance evaluation

Recorded average flow depth(m)	Calculated (observed) flow rate (using manning's) CMS	Simulated flow rate (using SWMM5.2) CMS	$q_t^{obs} - q_t^{Av.obs}$	$q_t^{sim} - q_t^{Av.sim}$	$(q_t^{sim} - q_t^{Av.sim}) * (q_t^{sim} - q_t^{Av.sim})$	$(q_t^{obs} - q_t^{Av.obs})^2$	$(q_t^{sim} - q_t^{Av.sim})^2$	$(q_t^{obs} - q_t^{sim})^2$
0.68	4.36	4.46	-0.02	0.126	-0.0020	0.0003	0.0159	0.0841
0.56	2.60	2.72	1.74	1.756	3.0625	3.0415	3.0835	0.0256
0.62	1.46	1.79	-0.35	-0.314	0.1086	0.1197	0.0986	0.0324
0.82	2.39	2.55	-0.23	-0.344	0.0777	0.0511	0.1183	0.0009
Total	10.81	11.52			4.66178	4.5489	4.8145	0.1494
Average	2.70	2.88						

$$\text{Coefficient of Determination } (R^2) = \left(\frac{4.66178}{\sqrt{4.54892}\sqrt{4.81452}}\right)^2$$

R^2= **0.99**

$$\text{Nash} - \text{Sutcliffe Efficiency (NSE)} = 1 - \left(\frac{0.1494}{\sqrt{4.66178}}\right)$$

NSE=**0.929**

Overall, the calibration achieved an excellent fit with observed daily flows for the simulation period, with **R^2= 0.99** and **NSE = 0.93.** This calibration and validation result showed that the model structure and parameters matched the runoff production pattern, and the calibrated model was adequate for simulating runoff in the research area.

4.3 Peak Discharge

The peak discharge is adequate for the design of conveyance systems such as storm drains, open channels, culverts, and bridges. Therefore, a number of methods were listed under the methodology Section for estimating a design discharge. Hence, based on the criteria listed there a rational method was used in this study for the computation of maximum discharge. Land use land cover, Runoff coefficient, the intensity of rainfall and drainage are the important parameters to compute peak/maximum discharge

The peak discharge of the study area estimated by using a rational method from currently estimated runoff for design 10yrs considering the sub-catchments that contributes drainage area with the outfall of the study area .excel result of peak discharge shown in table below.

Table 4.7: Result of runoff by rational method.

SC ID	Area(ha)	TC-min	I 10YR	I 25YR	I 50YR	I 100yr	W runoff	Q(m^3/s) 10 yr	Q(m^3/s) 25 yr	Manning Q(m^3/s)
SC1	7.2	14.33	54.12	64.90	68.05	79.09	0.743	2.273	2.725	2.185
SC2	5.4	20.39	42.31	49.97	55.49	57.85	0.766	1.740	2.054	1.437
SC3	1.3	20.41	37.28	42.81	46.86	50.91	0.683	1.063	1.221	1.814
SC4	2.9	13.16	67.72	78.83	84.74	92.22	0.800	0.726	0.845	1.451
SC5	3.3	11.74	76.73	89.56	96.62	106.07	0.900	2.140	2.498	1.075
SC6	1.5	8.18	113.80	131.58	146.93	160.71	0.872	1.000	1.156	0.807
SC7	3.8	18.33	30.27	36.27	37.73	40.28	0.500	0.990	1.186	1.307
SC8	3.0	13.94	58.28	69.80	73.27	84.92	0.800	1.432	1.715	1.759
SC9	4.2	18.99	38.02	44.64	48.22	52.26	0.683	2.015	2.366	1.751
SC10	1.5	16.92	46.28	53.25	64.88	69.68	0.661	1.313	1.511	0.986
SC11	1.8	13.97	55.23	66.50	75.17	79.00	0.735	0.812	0.978	1.102
SC12	2.6	14.30	44.16	50.07	54.81	59.58	0.598	1.355	1.537	2.064
SC13	5.3	16.71	39.71	45.61	49.93	54.32	0.610	2.048	2.352	1.284
SC14	3.0	18.14	34.32	39.42	43.15	47.00	0.605	1.355	1.556	1.614
sc15	2.1	17.62	35.48	40.76	44.61	48.56	0.567	1.853	2.129	2.128
SC16	5.3	13.24	51.29	59.39	65.01	70.64	0.677	2.230	4.581	2.123
SC17	7.0	19.16	32.10	36.87	40.36	43.96	0.552	2.130	2.446	2.038
SC18	2.0	17.63	35.34	37.74	44.44	48.38	0.580	1.007	1.075	0.944
SC19	3.2	17.10	34.57	39.71	43.47	47.32	0.559	1.416	1.627	1.603
SC20	6.4	14.82	37.63	43.22	47.31	51.42	0.517	1.769	2.032	1.906

4.4 Performance evaluation and current situation of the existing Drainage System in the Study Area

4.4.1 Performance evaluation of the existing Drainage System

The current performance (conditions) of the drainage systems were evaluated by using GTZ standards (GTZ, 2006).

Table 4.8: GTZ standards for evaluation of drainage performance

Classification Indicators	Surface condition
Very good	Shapes of USWD lines as still in original design condition
Good	No significant depressions, undulations, and deformation
Light	The shape of the USWD lines deteriorate, but still sheds water
Severe	The total collapse of the USWD lines structure and barely Passable

Table 4.9: Urban drainage System condition around CBE

Drainage	D pave	X-(Gps/map)	Y-(Gps/map)	Existing condition	Length(m)	Present from total%
Closed	Masonry	427071.3	715143.2	v.good	652.12	35.50
Closed	Masonry	427079.4	715124.3	Good	145.25	7.91
Closed	Masonry	427048.3	715118.1	Light	250.45	13.63
open	Masonry	427062.5	715132.5	Good	142.25	7.74
open	Masonry	427080.7	715142.2	Sever	245.65	13.37
open	Masonry	427060.4	715139.8	Good	186.58	10.16
open	Masonry	427068.8	715139.4	Good	214.65	11.69

From the table 4-8 total drains, about 13.37% is severely degraded, 13.63% is the light 37.5% good and 35.5% is very-good.

Hence, the major drainage systems were Safely passes the runoff i.e needs reconstruction severe condition and maintenance for light and good conditions case

Table 4.10: Urban drainage System condition around mazegaja

Drainage	Drainage pavement	X-(Gps/map)	Y-(Gps/map)	Existing condition	Length(m)	Percentages from total%
close	Masonry	426939.4	715153.1	Good	256.39	11.64
close	Masonry	426945.5	715141.3	Good	254.68	11.56
open	Masonry	426948.2	715138.4	Sever	189.26	8.59
open	Masonry	426928.5	715132.4	Good	179.56	8.15
open	Masonry	426935.4	715136.2	Light	224.61	10.20
open	Masonry	426935.4	715140.4	Good	183.65	8.34
open	Masonry	426951.2	715137.2	Sever	337.45	15.32
open	Masonry	426958.3	715133.4	Sever	145.68	6.61
open	Masonry	426955.2	715143.2	Sever	236.165	10.72
open	Masonry	426947.2	715136.2	Sever	195.34	8.87

From the Table 4-9 total drains, about 50.11% is severely degraded, 10.20% is the light, 39.69% good. That is the flood generated around this location cannot safely discharge into the proposed drainage system condition around Ethiotele

Table 4.11: Urban drainage system condition around mazegaja

Drainage	Drainage pavement	X-(Gps/map)	Y-(Gps/map)	Existing condition	Length(m)	Percentages from total%
Closed rectangular	Masonry	427200.0	715272.1	V.Good	243.21	11.32
Open rectangular	Masonry	427209.2	715289.4	Good	196.21	9.14
Open rectangular	Masonry	427221.3	715278.5	Sever	171.25	7.97
Open rectangular	Masonry	427218.2	715266.3	Sever	144.33	6.72
Open rectangular	Masonry	427212.2	715259.6	Sever	125.36	5.84
Open rectangular	Masonry	427216.3	715251.3	Good	174.98	8.15
Open rectangular	Masonry	427206.2	715244.6	Light	211.34	9.84
Open rectangular	Masonry	427219.3	715253.2	Good	269.38	12.54
Open rectangular	Masonry	427211.3	715253.3	Sever	345.98	16.11
Open rectangular	Masonry	427208.3	715247.1	Good	265.69	12.37

On the other hand, run over road surfaces to cause flood hazards. The drainage system around this has been damaged by erosion and overflow. So the major part of drainage systems around mazegaja shows degraded..

- ✓ From the Table 4-10 total drains, about 36.64% is severely degraded, 9.84% is the light, 42.2 good and 11.32% v.good. That is the flood generated around this location cannot safely discharge into the proposed drainage system.
- ✓ On the other hand, run over road surfaces to cause flood hazards. The drainage system around this has been damaged by erosion and overflow. So the major part of drainage systems around Ethiotel shows degraded.

Table 4.12: Urban drainage System condition around TVT College

Drainage	Drainage pavement	X-(Gps/map)	Y-(Gps/map)	Existing condition	Length(m)	Percentages from total%
Open rectangular	Masonry	427058.1	714656.0	v.good	115.36	5.92
Open rectangular	Masonry	427048.2	714656.1	Good	245.3	12.59
Open rectangular	Masonry	427095.1	714642.5	Good	358.45	18.40
Closed rectangular	Masonry	427101.3	714649.6	v.good	256.98	13.19
Open rectangular	Masonry	427108.3	714638.2	Good	458.36	23.53
Closed rectangular	Masonry	427116.5	714629.5	v.good	147.69	7.58
Open rectangular	Masonry	427136.5	714668.2	Sever	256.34	13.16
Open rectangular	Masonry	427149.2	714655.3	Sever	109.25	5.61

- ✓ From the Table 4.11 conditions of the urban drainage system around TVT college were 18.77% sever, 54.52% good and 26.29% with very good condition. Hence, the major drainage systems were Safely passes the runoff i.e needs reconstruction severe condition and maintenance for good conditions case.

Table 4.13: Urban drainage System condition around darancho

Drainage	drainage pavement	X- (Gps/map)	Y- (Gps/map)	Existing condition	Length(m)	Percentages from total%
Open rectangular	Masonry	427473.4	715474.4	God	259.32	9.80
Open rectangular	Masonry	427478.5	715463.3	Light	365.96	13.83
closed rectangular	Masonry	427480.2	715455.2	v.good	256.38	9.69
Open rectangular	Masonry	427466.1	715442.7	Sever	721.03	27.25
Closed rectangular	Masonry	427459.3	715439.5	v.good	423.36	16.00
Open rectangular	Masonry	4274540.3	715428.6	Light	189.36	7.16
Closed rectangular	Masonry	427442.1	715480.2	God	136.32	5.15
Closed rectangular	Masonry	427492.8	715513.2	v.good	294.38	11.13

- ✓ From the Table 4.12 conditions of the urban drainage system around darancho were 27.25% sever, 20.99% light, 14.95% good and 36.82% with very good condition. Hence, the major drainage systems were Safely passes the runoff i.e needs reconstruction severe and light condition and maintenance for good conditions case.

4.5 Current Situation of Existing Drainage System in the Study Area

- ✓ From the result of a field survey, interview and visit, the existing storm drainage facilities are generally classified into closed and open drainage lines. Closed drainage lines are found in some areas especially along main roads. Most of the open drainage channels were constructed by masonry and found along sub-mains and local roads. Drainage services in the town were inadequate in terms of quality and coverage. Currently, the stormwater drainage management of the town is not efficient as a result of managing problems.
- ✓ Drainage systems are not well connected and do not have the capacity to carry large amounts of water, hence resulting in overflowing. In some areas, drainage systems were not provided, and some of the existing drainages have been silted by sand, other rubbish, and waste materials. Especially the great problem in the study area were lack of waste management techniques (like manholes and trash bin).
- ✓ In the case of Kebado town, the existed manholes were out of service and have been clogged with waste and blocked due to lack of clearance. Additionally, at different places, they were not constructed. As well as a trash bin in the town not adapted. As a result, the runoff that is generated in that sub-basin overflows with a higher velocity which erodes the ditches as well as the road and walkways.

4.5.1. Dumping of solid wastes into storm drainage Systems

- ✓ Dumping solid waste materials into the drainage system was the other challenge of the stormwater drainage system in kewado town. Urban litter (alternatively called trash, debris, junk, floatables, gross pollutants, rubbish or solid waste) has become a major problem through the town. It typically consists of manufactured materials such as bottles, cans, plastic and paper wrappings, newspapers, shopping bags, cigarette packets and remains of chat. As a result of dumping such solid wastes into drains the drainage systems have been clogged and cause flooding over streets and drainage failures.

Figure 4.2: Blockage by Liquid and solid waste in the drainage system

4.5.2. Lack of Awareness and Understanding

During site observation, illegal dumping of household solid and liquid wastes blocks the drainage systems. Awareness creation is one of the best active measures for environmental protection and for managing urban drainage structures. Understanding the problem at a time is rare, once the structure is constructed response of governmental body for maintenance is weak, no immediate solution is taken, and they wait until the structure is fully destroyed. There should be a strong relationship between the community and governmental body in awareness creation, environmental protection and participating in the community to keep their environment.

4.5.3. Increment of impervious surface with the change of land use

Growing populations and migration towards built areas are driving land-use change in the form of urbanization across the town, per-urban development upon a previously rural area has led to significant increases in peak flows. When the land-use changes from semi-urban areas to urban with the construction of different infrastructures the runoff coefficient increased and most of the surface changes from previous to impervious. Due to residential increased through the town without a drainage system, the existing drainage systems become inadequate to safely pass runoff through the town. Therefore, while designing and construction the effect of stormwater from sub-urban should be taken into consideration

4.5.4. Lack of Community responsiveness to Environmental Management inhabitant

Known that community awareness is one of the best active measures for the maintainable urban drainage management. Unfortunately, from the interview, it was studied that about 48% (19 out of 40 people) of the inhabitants thought that dumping wastes into storm water drainage systems is the right thing. The other 52% of the residents knew that dumping liquid wastes into existing drainage system is erroneous however; they have been enforced to dump the wastes into the drainage systems regardless of the effects that could cause to their environment. They have implied that there is no proper sewage system and trash bin to collect wastes extracted from each household. As a result blockage of drainage systems by solid waste material and failed. In general, it was realized that even though few people have the responsiveness, the authority should create awareness among the communities and also should provide proper waste management techniques.

4.6. Model result from SWMM

This study result of the model simulation of the kebado town drainage system and performance under multiple working condition one to identified stormwater management issues and existing urban drainage, secondly to assess the hydraulic performance of storm water drainage infrastructures. The total sub catchments of the study model area are generating runoff from daily precipitation taken as extreme events from each year from 1990-2019 rainfall data.

The project of sub catchment used as input for drainage systems connected through each nodes and simulate runoff in the network system of swmm model in a outfall study.

4.6.1. Runoff from sub catchment

Summary Results

Topic: Subcatchment Runoff — Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
SC01	600.37	0.00	0.00	45.94	123.40	429.22	552.62	5.25	2.33	0.920
SC02	600.37	0.00	0.00	45.94	122.58	435.52	558.10	3.63	1.68	0.930
SC03	600.37	0.00	0.00	45.94	121.79	441.80	563.60	2.82	1.36	0.939
SC04	600.37	0.00	0.00	45.94	121.45	444.65	566.09	1.84	0.93	0.943
SC05	600.37	0.00	0.00	45.94	122.37	436.70	559.06	4.67	2.19	0.931
SC06	600.37	0.00	0.00	40.20	185.13	383.90	569.03	2.56	1.28	0.948
SC07	600.37	0.00	0.00	40.20	184.58	386.39	570.96	2.46	1.27	0.951
SC08	600.37	0.00	0.00	45.94	122.42	437.00	559.41	3.69	1.72	0.932
SC09	600.37	0.00	0.00	45.94	123.34	430.13	553.48	4.85	2.16	0.922
SC10	600.37	0.00	0.00	45.94	121.97	440.46	562.43	2.31	1.09	0.937
SC11	600.37	0.00	0.00	40.20	183.18	391.95	575.13	2.08	1.16	0.958
SC12	600.37	0.00	0.00	45.94	122.35	437.29	559.64	3.09	1.45	0.932
SC13	600.37	0.00	0.00	40.20	186.47	379.21	565.69	5.12	2.36	0.942
SC14	600.37	0.00	0.00	40.20	184.04	388.80	572.84	2.61	1.39	0.954
SC15	600.37	0.00	0.00	25.84	343.08	251.41	594.49	3.17	1.96	0.990
SC16	600.37	0.00	0.00	34.46	251.00	320.93	571.93	5.39	2.55	0.953
SC17	600.37	0.00	0.00	34.46	250.13	325.49	575.62	4.87	2.47	0.959
SC18	600.37	0.00	0.00	43.07	153.48	411.28	564.76	2.84	1.36	0.941
SC19	600.37	0.00	0.00	40.20	184.95	384.96	569.91	2.51	1.27	0.949
SC20	600.37	0.00	0.00	40.20	182.54	394.20	576.75	3.13	1.79	0.961

Figure 4.*3*: sub catchment runoff result

According to figure 4.3's simulation results, all 20 sub catchments together have a total area of 72.8 hectares. The total runoff from all 20 sub catchments is 33.77 m^3/sec, with sub catchment (S16) generating the highest peak runoff at 2.55 m^3/sec.

4.6.2. Channels and junctions flooded

Summary Results

Topic: Node Flooding Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J01	0.15	1.397	0	00:15	0.538	0.000
J02	0.80	3.606	0	00:15	3.649	0.000
J03	0.33	2.170	0	00:15	1.241	0.000
J04	1.56	6.543	0	00:15	9.548	0.000
J05	2.63	5.415	0	00:10	13.210	0.000
J06	2.89	4.134	0	00:10	9.779	0.000
J07	2.11	1.826	0	00:10	1.859	0.000
J08	2.87	3.908	0	00:10	8.272	0.000

Summary Results

Topic: Conduit Surcharge Click a column header to sort the column.

Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Downstream Full	Hours Above Normal Flow	Hours Capacity Limited
C01	0.15	0.15	0.86	0.01	0.01
C02	0.34	0.86	0.34	0.87	0.34
C03	0.34	0.34	2.40	0.01	0.01
C04	1.66	1.66	2.67	1.56	1.58
C05	2.66	2.67	2.89	0.01	0.01
C06	2.88	2.89	2.88	2.89	2.88
C07	2.81	2.81	2.88	0.01	0.01
C08	0.01	2.87	0.01	0.01	0.01

Figure 4.4: flooded junctions and channels

According to the results of Table 4.3, sections between Junctions J1 to J8 are nodes flooded. This is because the drainage system channels and nodes overflow.

There have been overflows in the following channels: C01 through C02, C03, C04, C05, C06, C07 and C08, which have limited the capacity of the constructed channel infrastructure to convey flow.

4.6.3. Water Elevation profile and outfall

Water Elevation Profile at around darancho river Outfall

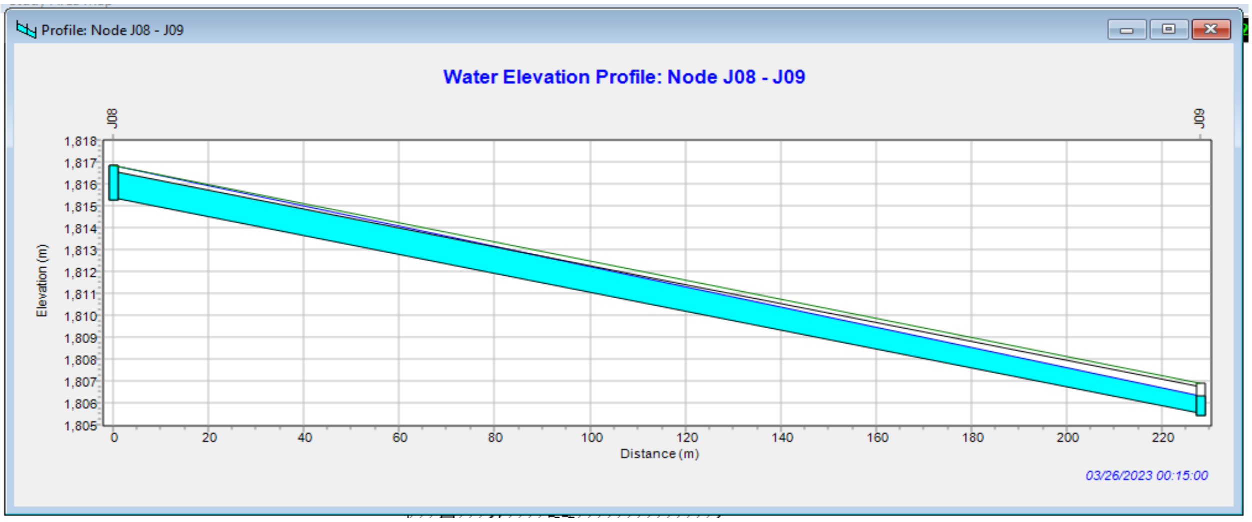

Figure 4.5 Water elivation profile around darancho river

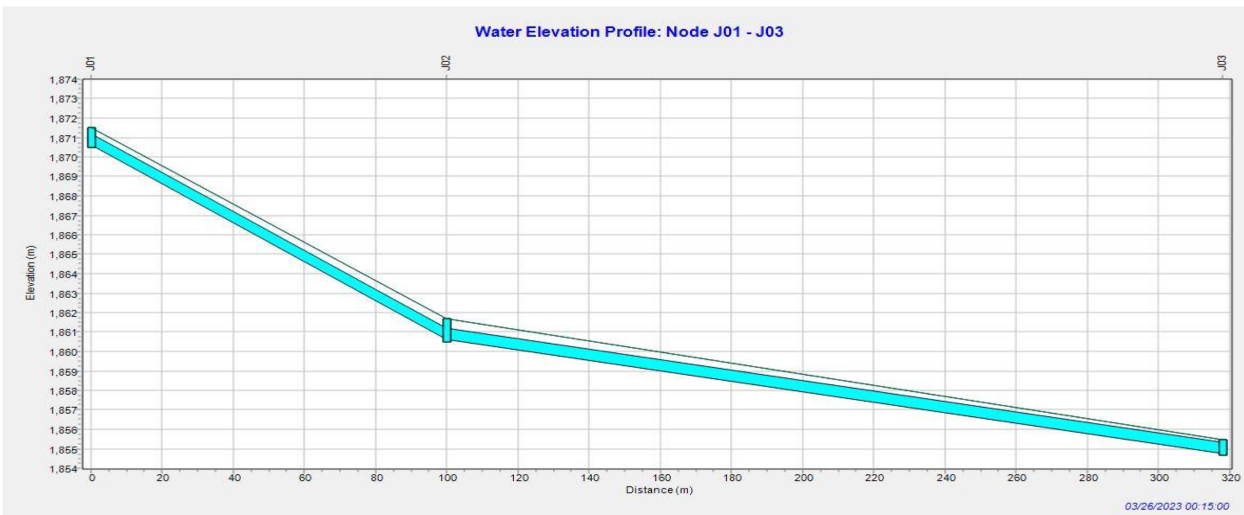

Figure 4.6 Water elevation profile CBE branch

Summary Results

Topic: Outfall Loading   Click a column header to sort the column.

Outfall Node	Flow Frequency %	Avgerage Flow CMS	Maximum Flow CMS	Total Volume 10^6 ltr
OF01	98.72	2.006	4.447	20.108

Figure 4.7: water elevation profile and discharge

According to figure 4.7 results of the simulation result indicates that the average flow rate is $2.006 m^3/s$, the maximum flow rate is $4.447\ m^3/s$ at the ends of outlet and the total runoff volume is $20.108*10^3\ m^3$.

The hydraulic performance capacity of the entire drainage system is assessed using the swmm model. The systems simulate the runoff generated from each sub catchment and flow routed via the network of channels and junction infrastructure constructed under varied operating situations the outcomes of flooded drainage system are well evaluated.

According to result of model simulation the channels have limited capacity and some channels have unable to carry flow with in the channel during high rain season.

Table 4.14: comparisons of swmm, manning and rational result

		10years	10 years
sub catch	Manning Q(M^3/s)	Swmm Q(m^3/s)	Rational Q(m^3/s)
SC1	2.185	2.330	2.273
SC2	1.437	1.680	1.740
SC3	1.814	1.360	1.063
SC4	1.451	0.930	0.726
SC5	1.075	2.190	2.140
SC6	0.807	1.280	1.000
SC7	1.307	1.270	0.990
SC8	1.759	2.720	2.432
SC9	1.751	2.160	2.015
SC10	0.986	1.090	1.313
SC11	1.102	1.160	0.812
SC12	2.064	1.450	1.355
SC13	1.284	4.460	4.248
SC14	1.614	1.390	1.355
SC15	2.128	1.960	1.853

SC16	2.123	2.550	2.230
SC17	2.038	2.470	2.130
SC18	0.944	1.360	1.007
SC19	1.603	1.270	1.416
SC20	1.906	1.790	1.769

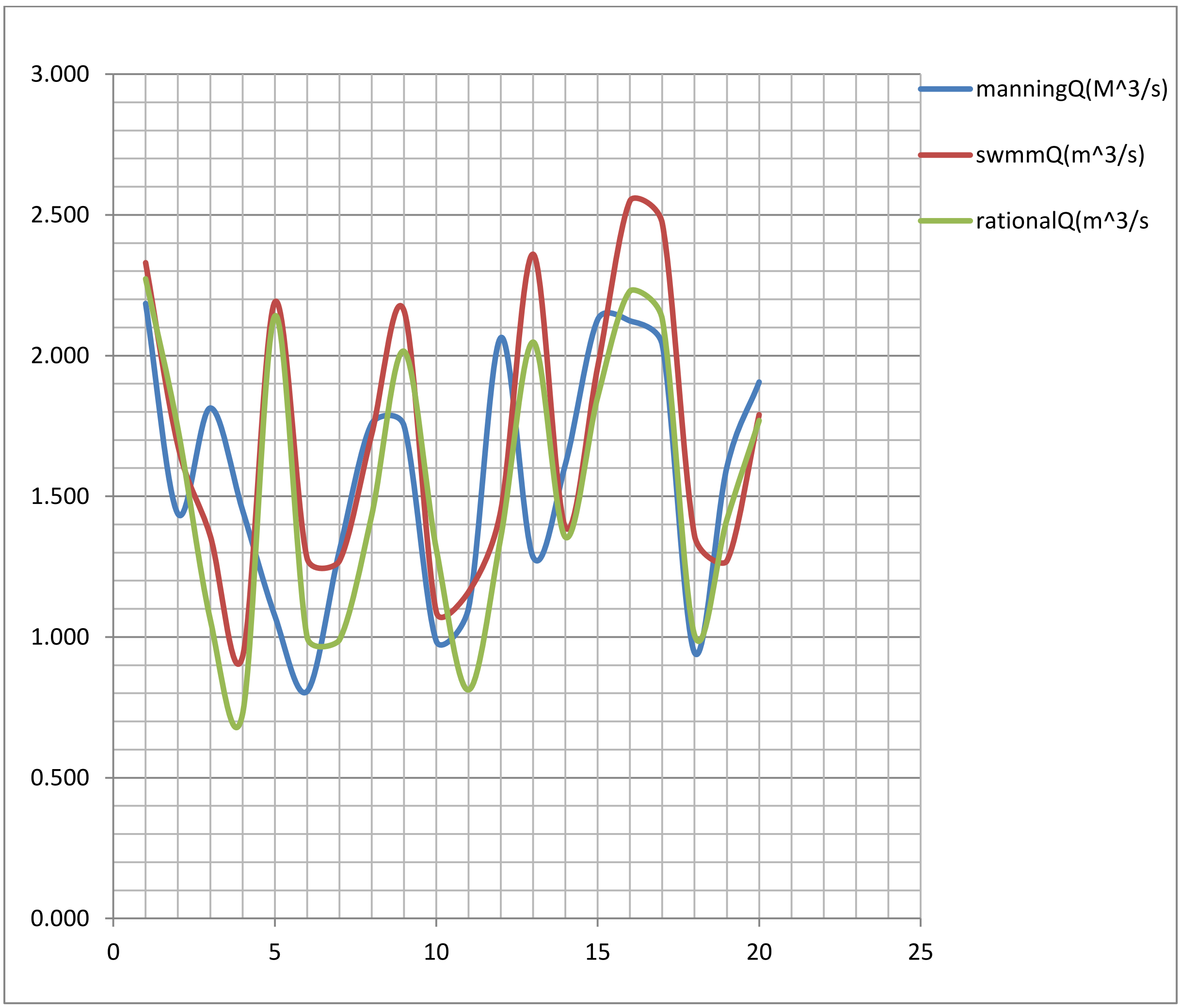

Figure 4.8: Discharge result by Rational Method, SWMM and Manning Formula

Figure 4.8 show that, the result of discharge by SWMM and rational method with in design period of 10 year were 33.77 and 30.66 m3/s, It show that, the performance of the SWMM Model 5.2 with Rational method was well matched and acceptable in this study.

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In the study a storm water drainage network performance of Kebado town is examined using Stormwater Management model. Thus, it is hoped that the study results are helpful for a better planningand management of urban drainage system of the town as well as for other towns with similar situations. From the study the following conclusions are deduced as general and specific to the Kebado town.

General

Urban drainage infrastructure development and management were not given attention as other public infrastructures and public utilities. Absence of urban drainage master plan and an organized existing drainage data are evidences for the reluctance being observed in the sector.

Specific

- ✓ Study evaluated the performance assessment of existing urban storm drainage systems for the case of Kewado town by using NMA data, SWMM with version 5.2, rational method used to estimate the peak flow of current runoff, Manning equation, and IDF curve for kewado town. I have developed, DEM, GIS, and different junction and outfall point are identified in order to transport flows through the given canal dimensions.
- ✓ According to simulation of SWMM for 10 years return period, the existing drainage system of the town is insufficient to accommodate the runoff being generated in the town. Out of the total drainage system, 22.7 % and 26.4% of conduits and junctions donot provide proper service which could be a cause for flood induced problems. As the return period increases, the malfunctioning proportion of conduits, junctions & other related drainage infrastructures supposed to be increased.
- ✓ The performance of these storm water drainages was not satisfactory. Therefore, it is recognized that its capacity has shown lower results which needs some adjustment or improvement to give the best service, and needs a serious of regular maintenance and also provide drainage networking for the areas without drainage systems for its complete service.

- ✓ The main drainage problem in the existing system is, the capacities of the drainage system can'thandle the current runoff that flows over the area. This is due to urbanization and lack of good assessment of the hydrology of the area.

5.2 . Recommendation

Based on the findings of the study, the following recommendations are given as general and specific to the Kewado town.

General

As a Municipality, Kewado town is expected to implement a paramount urban drainage infrastructure development along with other urban infrastructures. The town needs to have drainage master plan as soon as possible that enhances proper urban drainage management and green infrastructure development.

Specific

- ✓ Frequently clearance of the drainage canals and repairs before it is damaged at non-rainfall season
- ✓ Establish well-connected drainage lines, waste management techniques to safely release wastewater, to properly manage and use stormwater drainage lines as it were their objective.
- ✓ Adaptation of different Best Management Practices (BMP) which will not only reduce the peak runoff but also have aesthetic values and improve the environment. Even though urbanization cannot be avoided, the runoff that can be generated due to impervious areas can be infiltrated in to the ground through different best management practices that is supported by strong institutional setup, policy framework, and the public at large.
- ✓ In addition to this finding, future studies are recommended to include sustainable urban drainagesystem (SuDS).

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7. APPENDICES

Appendices 1 maximum rainfall for to test outlier data

Year	max rainfall	log(RF)
1990	46.1	1.66
1991	46.1	1.66
1992	95.2	1.98
1993	44.8	1.65
1994	42.4	1.63
1995	67.1	1.83
1996	76.0	1.88
1997	56.2	1.75
1998	65.7	1.82
1999	65.4	1.82
2000	34.5	1.54
2001	41.3	1.62
2002	55.6	1.75
2003	38.9	1.59
2004	43.0	1.63
2005	30.9	1.49
2006	44.3	1.65
2007	63.0	1.80
2008	62.4	1.80
2009	52.4	1.72
2010	56.0	1.75
2011	135.4	2.13
2012	38.0	1.58
2013	55.0	1.74

2014	42.6	1.63
2015	30.6	1.49
2016	55.1	1.74
2017	65.0	1.81
2018	79.5	1.90
2019	64.2	1.81
Average	56.4233	1.7275
Std	21.0979	0.1419
Kn	2.5490	0.4064
Higher	**110.2018**	1.7851
Lower	**2.6448**	1.6698

Return period	X (mean)	δz	Kz	Kz * δz	ZT	XT
2	1.721	0.1220	-0.017	-0.002	1.719	52.408
5	1.721	0.1220	0.6375	0.078	1.799	62.983
10	1.721	0.1220	1.292	0.158	1.879	75.692
25	1.721	0.1220	1.785	0.218	1.939	86.931
50	1.721	0.1220	2.107	0.257	1.978	95.159
100	1.721	0.1220	2.4	0.293	2.014	103.320

duration (min)	2Yr	5yr	10yr	25yr	50yr	100yr
5	7.94	9.54	11.46	13.16	14.41	15.65
10	10.00	12.02	14.44	16.59	18.15	19.71
20	12.60	15.14	18.19	20.90	22.87	24.84
30	14.42	17.33	20.83	23.92	26.18	28.43
40	15.87	19.07	22.92	26.33	28.82	31.29
50	17.10	20.55	24.69	28.36	31.04	33.71
60	18.17	21.83	26.24	30.14	32.99	35.82
70	19.13	22.99	27.62	31.73	34.73	37.71
80	20.00	24.03	28.88	33.17	36.31	39.42
90	20.80	24.99	30.04	34.50	37.76	41.00
100	21.54	25.89	31.11	35.73	39.11	42.47
110	22.24	26.72	32.12	36.89	40.38	43.84
120	22.89	27.51	33.06	37.97	41.56	45.13
130	23.51	28.25	33.96	39.00	42.69	46.35
140	24.10	28.96	34.80	39.97	43.76	47.51
150	24.66	29.63	35.61	40.90	44.77	48.61
160	25.20	30.28	36.39	41.79	45.75	49.67
170	25.71	30.90	37.13	42.65	46.68	50.69
180	26.20	31.49	37.85	43.47	47.58	51.66

Duration (Min)	Duration (Hr)	2yr	5yr	10yr	25yr	50yr	100yr
5	0.0833	95.23	114.45	137.54	157.96	172.92	187.74
10	0.1667	59.99	72.10	86.65	99.51	108.93	118.27
20	0.3333	37.79	45.42	54.58	62.69	68.62	74.51
30	0.5000	28.84	34.66	41.65	47.84	52.37	56.86
40	0.6667	23.81	28.61	34.39	39.49	43.23	46.94
50	0.8333	20.52	24.66	29.63	34.03	37.25	40.45
60	1.0000	18.17	21.83	26.24	30.14	32.99	35.82
70	1.1667	16.39	19.70	23.68	27.19	29.77	32.32
80	1.3333	15.00	18.02	21.66	24.88	27.23	29.57
90	1.5000	13.87	16.66	20.03	23.00	25.18	27.34
100	1.6667	12.93	15.53	18.67	21.44	23.47	25.48
110	1.8333	12.13	14.58	17.52	20.12	22.02	23.91
120	2.0000	11.45	13.76	16.53	18.99	20.78	22.56
130	2.1667	10.85	13.04	15.67	18.00	19.70	21.39
140	2.3333	10.33	12.41	14.92	17.13	18.75	20.36
150	2.5000	9.86	11.85	14.25	16.36	17.91	19.45
160	2.6667	9.45	11.35	13.65	15.67	17.16	18.63
170	2.8333	9.07	10.90	13.11	15.05	16.48	17.89
180	3.0000	8.73	10.50	12.62	14.49	15.86	17.22

Appendices 2: Existing Drainage Capacity

	code	shape	material	width(m)	depth(m)	length(m)	S	manning(n)	permeter(m)	Area(m^2)	R(m)	R^2/3	S^1/2	1/n	Q(m/s)
manning															
method															
	KHsc	Rectangulary	masonery	0.5	0.85	45.63	0.040	0.013	2.2	0.425	0.193182	0.334178	0.2	76.92308	2.185008
	KHsc	Rectangulary	masonery	0.5	0.65	16.61	0.040	0.013	1.8	0.325	0.180556	0.319453	0.2	76.92308	1.597266
	KHsc	Rectangulary	masonery	0.5	0.55	111.47	0.040	0.013	1.6	0.275	0.171875	0.30913	0.2	76.92308	1.30786
sc1	**KHsc**	Rectangulary	masonery	0.5	0.6	67.63	0.040	0.013	1.7	0.3	0.176471	0.314617	0.2	76.92308	1.452076
															2.185008
	DAsc	Rectangulary	masonery	0.5	1.4	119.14	0.03	0.03	3.3	0.7	0.212121	0.355677	0.173205	33.33333	1.437452
	DAsc	Rectangulary	masonery	0.4	0.95	104.88	0.03	0.05	2.3	0.38	0.165217	0.301095	0.223607	20	0.511685
	DAsc	Rectangulary	masonery	0.3	1.25	90.91	0.025	0.05	2.8	0.375	0.133929	0.261767	0.223607	20	0.438997
	DAsc	Rectangulary	masonery	0.25	1.25	122.77	0.03	0.025	2.75	0.3125	0.113636	0.234609	0.158114	40	0.463687
sc2	**DAsc**	Rectangulary	masonery	0.3	0.85	190.21	0.026	0.03	2	0.255	0.1275	0.253322	0.173205	33.33333	0.372952
															1.437452
	DARsc	Rectangulary	masonery	0.28	1.5	103.12	0.020	0.02	3.28	0.42	0.128049	0.254049	0.141421	50	0.754486
	DARsc	Rectangulary	masonery	0.5	1.3	51.04	0.025	0.02	3.1	0.65	0.209677	0.35294	0.158114	50	1.813654
	DARsc	Rectangulary	masonery	0.4	0.8	100.76	0.020	0.05	2	0.32	0.16	0.294723	0.141421	20	0.266752
	DARsc	Rectangulary	masonery	0.35	1.5	45.57	0.028	0.05	3.35	0.525	0.156716	0.290676	0.167332	20	0.510714
	DARsc	Rectangulary	masonery	0.25	0.93	235.88	0.032	0.05	2.11	0.2325	0.11019	0.229841	0.178885	20	0.191185
	DARsc	Rectangulary	masonery	0.3	1.35	141.28	0.030	0.03	3	0.405	0.135	0.263162	0.173205	33.33333	0.615342
sc3	**DARsc**	Rectangulary	masonery	0.35	1.5	176.29	0.025	0.05	3.35	0.525	0.156716	0.290676	0.158114	20	0.48258
															1.813654
	ETsc	Rectangulary	masonery	0.2	1.23	62.76	0.028	0.025	2.66	0.246	0.092481	0.204504	0.167332	40	0.336726
	ETsc	Rectangulary	masonery	0.2	1.09	67.28	0.0250	0.025	2.38	0.218	0.091597	0.203198	0.158114	40	0.28016
	ETsc	Rectangulary	masonery	0.3	1.28	187.27	0.0340	0.025	2.86	0.384	0.134266	0.262207	0.184391	40	0.742633
sc4	**ETsc**	Rectangulary	masonery	0.5	1.3	109.26	0.0250	0.025	3.1	0.65	0.209677	0.35294	0.158114	40	1.450923
															1.450923
	Msc	Rectangulary	masonery	0.25	1.3	44.05	0.0250	0.05	2.85	0.325	0.114035	0.235158	0.158114	20	0.241681
	Msc	Rectangulary	masonery	0.25	1.05	36.07	0.0400	0.04	2.35	0.2625	0.111702	0.231939	0.2	25	0.30442
	Msc	Rectangulary	masonery	0.5	0.95	128.02	0.0400	0.03	2.4	0.475	0.197917	0.339616	0.2	33.33333	1.075451
sc5	**Msc**	Rectangulary	masonery	0.4	0.98	37.59	0.0350	0.05	2.36	0.392	0.166102	0.302169	0.187083	20	0.4432
															1.075451
	Hsc	Rectangulary	masonery	0.25	0.95	34.50	0.0200	0.04	2.15	0.2375	0.110465	0.230224	0.141421	25	0.193316
	Hsc	Rectangulary	masonery	0.25	0.85	63.85	0.010	0.04	1.95	0.2125	0.108974	0.228148	0.1	25	0.121203
	Hsc	Rectangulary	masonery	0.3	1.5	32.71	0.010	0.04	3.3	0.45	0.136364	0.264931	0.1	25	0.298047
	Hsc	Rectangulary	masonery	0.45	1.25	48.79	0.030	0.04	2.95	0.5625	0.190678	0.331284	0.173205	25	0.806907
sc6	**Hsc**	Rectangulary	masonery	0.35	1	74.84	0.025	0.04	2.35	0.35	0.148936	0.280974	0.158114	25	0.388727
															0.806907
	TVTsc	Rectangulary	masonery	0.4	1.6	67.18	0.025	0.025	3.6	0.64	0.177778	0.316168	0.158114	40	1.279759
	TVTsc	Rectangulary	masonery	0.3	1.36	198.84	0.025	0.013	3.02	0.408	0.135099	0.263291	0.158114	76.92308	1.306539
	TVTsc	Rectangulary	masonery	0.35	1.24	78.78	0.035	0.020	2.83	0.434	0.153357	0.286507	0.187083	50	1.163133
	TVTsc	Rectangulary	masonery	0.2	1.085	64.60	0.025	0.020	2.37	0.217	0.091561	0.203146	0.158114	50	0.348504
sc7	**TVTsc**	Rectangulary	masonery	0.2	1.4	16.40	0.035	0.013	3	0.28	0.093333	0.205759	0.187083	76.92308	0.829099
															1.306539
	CBE sc	Rectangulary	masonery	0.4	1.3	79.08	0.020	0.013	3	0.52	0.173333	0.310877	0.141421	76.92308	1.758584
	CBE sc	Rectangulary	masonery	0.4	1.2	46.84	0.020	0.013	2.8	0.48	0.171429	0.308595	0.141421	76.92308	1.611394
sc8	**CBE sc**	Rectangulary	masonery	0.3	1.5	73.54	0.025	0.025	3.3	0.45	0.136364	0.264931	0.158114	40	0.754006
															1.758584
	HMsc	Rectangulary	masonery	0.25	1.05	50.32	0.015	0.025	2.35	0.2625	0.111702	0.231939	0.122474	40	0.29827
	HMsc	Rectangulary	masonery	0.35	0.95	82.77	0.0600	0.013	2.25	0.3325	0.147778	0.279516	0.244949	76.92308	1.751177
	HMsc	Rectangulary	masonery	0.3	0.9	112.13	0.026	0.030	2.1	0.27	0.128571	0.25474	0.161245	33.33333	0.36968
sc9	**HMsc**	Rectangulary	masonery	0.3	0.85	182.37	0.030	0.040	2	0.255	0.1275	0.253322	0.173205	25	0.279714
															1.751177
	CHsc	Rectangulary	masonery	0.35	0.95	182.37	0.045	0.020	2.25	0.3325	0.147778	0.279516	0.212132	50	0.985766
	CHsc	Rectangulary	masonery	0.35	0.8	100.44	0.042	0.040	1.95	0.28	0.14359	0.274209	0.204939	25	0.393373
	CHsc	Rectangulary	masonery	0.25	0.9	96.29	0.035	0.040	2.05	0.225	0.109756	0.229238	0.187083	25	0.241236
	CHsc	Rectangulary	masonery	0.3	0.98	94.91	0.012	0.050	2.26	0.294	0.130088	0.256739	0.109545	20	0.165371
sc10	**CHsc**	Rectangulary	masonery	0.3	1.24	75.75	0.024	0.050	2.78	0.372	0.133813	0.261617	0.154919	20	0.301539

	MCsc	Rectangulary	masonery	0.4	1.03	114.73	0.040	0.025	2.46	0.412	0.16748	0.303838	0.2	40	1.001448
	MCsc	Rectangulary	masonery	0.35	1.23	63.34	0.050	0.025	2.81	0.4305	0.153203	0.286315	0.223607	40	1.10246
	MCsc	Rectangulary	masonery	0.35	0.78	82.41	0.028	0.025	1.91	0.273	0.142932	0.273371	0.167332	40	0.499522
	MCsc	Rectangulary	masonery	0.35	0.75	94.55	0.020	0.030	1.85	0.2625	0.141892	0.272044	0.141421	33.33333	0.336637
sc11	**MCsc**	Rectangulary	masonery	0.25	1.05	182.37	0.034	0.320	2.35	0.2625	0.111702	0.231939	0.184391	3.125	0.035083
															1.10246
		Rectangulary	masonery	0.4	0.85	120.24	0.035	0.013	2.1	0.34	0.161905	0.297057	0.187083	76.92308	1.453481
		Rectangulary	masonery	0.35	0.95	84.15	0.035	0.013	2.25	0.3325	0.147778	0.279516	0.187083	76.92308	1.337483
sc12		Rectangulary	masonery	0.4	1.25	96.45	0.030	0.013	2.9	0.5	0.172414	0.309776	0.173205	76.92308	2.063646
															2.063646
		Rectangulary	masonery	0.35	0.85	110.25	0.035	0.020	2.05	0.2975	0.145122	0.276157	0.187083	50	0.768504
		Rectangulary	masonery	0.4	0.8	85.21	0.035	0.020	2	0.32	0.16	0.294723	0.187083	50	0.882201
sc13		Rectangulary	masonery	0.45	0.95	65.47	0.035	0.020	2.35	0.4275	0.181915	0.321055	0.187083	50	1.283864
															1.283864
		Rectangulary	masonery	0.4	0.75	125.32	0.03	0.02	1.9	0.3	0.157895	0.292132	0.173205	50	0.75898
		Rectangulary	masonery	0.4	0.85	145.36	0.03	0.020	2.1	0.34	0.161905	0.297057	0.173205	50	0.87468
sc14		Rectangulary	masonery	0.45	1.25	180.65	0.03	0.020	2.95	0.5625	0.190678	0.331284	0.173205	50	1.613814
															1.613814
		Rectangulary	masonery	0.45	1.5	85.54	0.035	0.02	3.45	0.675	0.195652	0.337021	0.187083	50	2.127964
		Rectangulary	masonery	0.45	1.25	140.63	0.035	0.02	2.95	0.5625	0.190678	0.331284	0.187083	50	1.743118
sc15		Rectangulary	masonery	0.45	1.4	96.54	0.035	0.02	3.25	0.63	0.193846	0.334943	0.187083	50	1.973859
															2.127964
		Rectangulary	masonery	0.5	1.5	48.56	0.025	0.02	3.5	0.75	0.214286	0.358093	0.158114	50	2.123229
		Rectangulary	masonery	0.5	1.25	96.45	0.025	0.02	3	0.625	0.208333	0.35143	0.158114	50	1.736438
sc16		Rectangulary	masonery	0.45	0.95	123.25	0.025	0.02	2.35	0.4275	0.181915	0.321055	0.158114	50	1.085063
															2.123229
		Rectangulary	masonery	0.5	1.25	142.25	0.02	0.03	3	0.625	0.208333	0.35143	0.141421	33.33333	1.035411
sc17		Rectangulary	masonery	0.5	1.4	122.31	0.02	0.03	3.3	0.7	0.212121	0.355677	0.141421	33.33333	1.173675
		Rectangulary	masonery	0.5	1.6	95.64	0.02	0.02	3.7	0.8	0.216216	0.36024	0.141421	50	2.037826
															2.037826
		Rectangulary	masonery	0.35	0.85	125.32	0.03	0.025	2.05	0.2975	0.145122	0.276157	0.173205	40	0.569198
sc18		Rectangulary	masonery	0.35	0.98	58.65	0.03	0.025	2.31	0.343	0.148485	0.280407	0.173205	40	0.666351
		Rectangulary	masonery	0.35	1.35	45.98	0.03	0.025	3.05	0.4725	0.154918	0.288448	0.173205	40	0.944257
															0.944257
		Rectangulary	masonery	0.45	1.25	142.32	0.025	0.03	2.95	0.5625	0.190678	0.331284	0.158114	33.33333	0.982136
		Rectangulary	masonery	0.45	1.35	125.36	0.025	0.02	3.15	0.6075	0.192857	0.333803	0.158114	50	1.60316
sc19		Rectangulary	masonery	0.45	0.95	126.45	0.025	0.03	2.35	0.4275	0.181915	0.321055	0.158114	33.33333	0.723375
															1.60316
		Rectangulary	masonery	0.4	1.25	145.63	0.02	0.013	2.9	0.5	0.172414	0.309776	0.141421	76.92308	1.68496
		Rectangulary	masonery	0.4	1.4	85.26	0.02	0.013	3.2	0.56	0.175	0.312866	0.141421	76.92308	1.90598
sc20		Rectangulary	masonery	0.4	1.25	132.25	0.02	0.013	2.9	0.5	0.172414	0.309776	0.141421	76.92308	1.68496
															1.90598

Appendix 3: Hydrology

Kn value for different sample size (used for outlier)

S/ size (n)	Kn	S/ size (n)	Kn	S/ size (n)	Kn	S/ size (n)	Kn
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.7	80	2.94
15	2.247	29	2.549	43	2.71	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3

18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.76	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129 ,
23	2.448	37	2.65	55	2.804		

Source: (Chow, 1988)

Appendices 4: Kz=F (Cs, T) for use in Log-Pearson Type III Distribution Used for R24

Cs	Return Period T in years											
	2	5	10	25	50	100	Cs	2	5	10	25	50	100
3.0	-0.396	0.392	1.180	2.278	3.152	4.051	0.1	-0.017	0.6375	1.292	1.785	2.107	2.400
2.5	-0.360	0.445	1.250	2.262	3.048	3.845	0.0	0.00	0.641	1.282	1.751	2.054	2.326
2.2	-0.330	0.477	1.284	2.240	2.970	3.705	-0.1	0.017	0.6435	1.270	1.716	2.000	2.252
2.0	-3.07	-0.88	1.302	2.219	2.912	3.605	-0.2	0.033	0.6455	1.258	1.680	1.945	2.178
1.8	-0.282	0.518	1.318	2.193	2.848	3.499	-0.3	0.050	0.6475	1.245	1.643	1.890	2.104
1.6	-0.254	0.537	1.329	2.163	2.780	3.388	-0.4	0.060	0.6455	1.231	1.606	1.834	2.029
1.4	-0.225	0.556	1.337	2.128	2.706	3.271	-0.5	0.083	0.6495	1.216	1.567	1.777	1.955
1.2	-0.195	0.572	1.340	2.087	2.626	3.149	-0.6	0.099	0.6495	1.200	1.528	1.720	1.880
1.0	-0.164	0.588	1.340	2.043	2.542	3.022	-0.7	0.116	0.6495	1.183	1.488	1.663	1.806
0.9	-0.148	0.595	1.339	2.018	2.498	2.957	-0.8	0.132	0.649	1.166	1.448	1.606	1.733
0.8	-0.132	0.602	1.336	1.998	2.453	2.891	-0.9	0.148	0.6475	1.147	1.407	1.549	1.660
0.7	-0.116	0.608	1.333	1.967	2.407	2.824	1.0	0.164	0.646	1.128	1.366	1.492	1.588
0.6	-0.099	0.614	1.328	1.939	2.359	2.755	-1.4	0.225	0.633	1.041	1.198	1.270	1.318
0.5	-0.083	0.62	1.323	1.910	2.311	2.686	-1.8	0.282	0.6135	0.945	1.035	1.069	1.087
0.4	-0.066	0.625	1.317	1.880	2.261	2.615	-2.2	0.330	0.607	0.884	0.888	0.900	0.905
0.3	-0.050	0.629	1.309	1.849	2.211	2.544	-3.0	0.396	0.528	0.660	0.666	0.666	0.667
0.2	-0.033	0.634	1.301	1.818	2.159	2.472							

Source: (Chow, 1988)

Appendecis:5 Cumulative of each station for check Consistency

Kebado	Tefrkela	Aleta Wendo	Average	Avrg comlative	Kebado comulative
64.2	54.6	64	60.9	60.9	64.2
79.5	49.3	52	60.3	121.2	143.7
65.0	64.5	58	62.5	183.7	208.7
55.1	44.6	50.2	50.0	233.7	263.8
30.6	42.2	50	40.9	274.6	294.4
42.6	36.2	63.5	47.4	322.0	337.0
55.0	32.5	52.8	46.8	368.8	392.0
38.0	37	59.5	44.8	413.6	430.0
85.4	48	52.6	62.0	475.6	515.4
56.0	47.5	64	55.8	531.5	571.4
52.4	47.5	70	56.6	588.1	623.8
62.4	42.2	65	56.5	644.6	686.2
63.0	50.5	58	57.2	701.8	749.2
44.3	54.5	56.6	51.8	753.6	793.5
30.9	48.3	45.2	41.5	795.1	824.4
43.0	45.2	44.5	44.2	839.3	867.4
38.9	42.3	41	40.7	880.0	906.3
55.6	49.4	45.9	50.3	930.3	961.9
41.3	36.4	66.3	48.0	978.3	1003.2
34.5	39.2	33.2	35.6	1014.0	1037.7
65.4	30.1	36.8	44.1	1058.1	1103.1
65.7	36.3	58.3	53.4	1111.5	1168.8
56.2	26.1	60.1	47.5	1159.0	1225.0
76.0	40.1	65.2	60.4	1219.4	1301.0
67.1	42.7	36.2	48.7	1268.1	1368.1

42.4	31.7	48.2	40.8	1308.8	1410.5
44.8	25.6	69.7	46.7	1355.5	1455.3
95.2	42.3	69.7	69.1	1424.6	1550.5
51.5	41.3	50.6	47.8	1472.4	1602.0
46.1	39.9	49.2	45.1	1517.5	1648.1

Appendices: 6 .Runoff coefficients for different land use land cover

Type of Drainage Area	Runoff Coefficient (C)
Business:	
Commercial areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, Cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Lawns:	
Sandy soil, flat, <2%	0.05-0.10

Sandy soil, average, 2 to 7%	0.10-0.15
Sandy soil, steep, >7%	0.15-0.20
Heavy soil, flat,<2%	0.13-0.17
Heavy soil, average 2 to 7%	0.18-0.22
Heavy soil, steep, >7%	0.25-0.35
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and Walks	0.75-0.85
Roofs	0.75-0.95

Appendecise:7 GTZ Standards of drainage condition

Indicators classification	Surface condition
Very good	Shapes of USWD lines as still in original design condition
Good	No significant depressions, undulations and deformation
Light	Shape of the USWD lines deteriorate, but still sheds water
Severe	Total collapse of the USWD lines structure and barely passable

Source: (GTZ, 2006.)

Appendices 8: Manning's n for Overland Flow

Surface	N	Surface	N
Smooth Asphalt	0.011	Vitrified clay	0.015
Smooth Concrete	0.012	Cast iron	0.015
Ordinary concrete lining	0.013	Corrugated metal pipes	0.024
Good wood	0.014	Cement rubble surface	0.024
Brick with cement mortar	0.014	Fallow soils (no residues)	0.05
Cultivated soils Residue cover < 20%Residue cover > 20%	 0.06 0.17	Woods Light underbrush Dense under brush	 0.40 0.80
Grass Short, prairieDensegras	 0.15 0.24 0.41	Range (natural)	0.13

Source: McCuen, R.et al. (1996). Hydrology, FHWA-SA- 96-067, Federal Highway Administration, Washington, DC

Appendices 9 Manning's n for open channels

Channel types	Manning's n	Channel types	Manning's n
Lined channels		Excavated or dredged	
Asphalt	0.013-0.017	Earth, Straight & uniform	0.02-0.03
Brick	0.012-0.018	Earth, winding, fairly uniform	0.025-0.04
Concrete	0.011-0.020	Rock	0.03-0.045
Rubble or Riprap	0.020-0.035	Unmaintained	0.05-0.14
Vegetal	0.030-0.40		
Natural channels (minor streams, top width at flood stage <100ft)			
- Fairy regular section			0.03-0.07
-Irregular section with pools			0.04-0.100

Source: ASCE (1982). Gravity Sanitary sewer Design and Construction, ASCE Manual of Practice No.60, New York, NY

Appendices 10 Manning's n for closed conduits

Conduit material	Manning's n	Conduit material	Manning's n
Asbestos- cement pipe	0.011-0.015	Spun asphalt lined	0.011-0.015
Brick	0.013-0.017	Plastic pipe (smooth)	0.011-0.015
Cast iron pipe		Vitrified clay	
Cement-lined & seal coated	0.011-0.015	Pipes	0.011-0.015
Concrete (monolithic)		Liner plates	0.013-0.017
Smooth forms	0.012-0.014		
Rough forms	0.015-0.017	Corrugated-metal pipe (1/2-in.* 22/3in.corrugations)	
Concrete pipe	0.011-0.015	Plain	0.022-0.026
		Paved invert	0.018-0.022

Source: ASCE (1982). Gravity Sanitary Sewer Design and Construction, ASCE Manual of practice No.60, New York, NY

Appendices 11: Depression Storage

Impervious Surface	0.05-0.10 inches
Lawns	0.10-0.20 inches
Pasture	0.20 inches
Forest litter	0.30 inches

Source: ASCE, Design and Construction of Urban Storm Management Systems, New York, NY