

Stormwater Characterization and Detention Pond Modeling for Effective Stormwater
Management: The Case of Adama Science and Technology University Campus, Ethiopia



Shoangezaw Shumi Bereda

A Thesis Submitted to
The Department of Water Resources Engineering
School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Water
Resources Engineering (Water Supply and Environmental Engineering)

Office of Graduate Studies
Adama Science and Technology University

September, 2023
Adama, Ethiopia

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Advisor: Andinet Kebede (PhD)

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Declaration

I declare that this thesis entitled “Stormwater Characterization and Detention Pond Modeling for Effective Stormwater Management: The Case of Adama Science and Technology University Campus, Ethiopia” is my original work. That is it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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I, the advisor of this, thesis, hereby certify that I have read the revised version of the thesis entitled “Stormwater Characterization and Detention Pond Modeling for Effective Stormwater Management: The Case of Adama Science and Technology University Campus, Ethiopia” prepared under my guidance by Shoangezaw Shumi Bereda submitted in partial fulfillment of the requirements for the degree of Master of Science in Water Supply and Environmental Engineering. Therefore, I recommend the submission of the Thesis to the department for further review and evaluation.

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I, the advisor of the thesis entitled “Stormwater Characterization and Detention Pond Modeling for Effective Stormwater Management: The Case of Adama Science and Technology University Campus, Ethiopia” and developed by Shoangezaw Shumi Bereda, hereby certify that the recommendation made by the board of examiners are appropriately incorporated into the final version of the thesis.

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We, the undersigned, members of the Board of examiners of the thesis by Shoangezaw Shumi Bereda have read and evaluated the thesis entitled “Characterization of Stormwater Drainage System and Modeling of Its Detention Pond: The Case of Adama Science and Technology University Campus, Ethiopia” and examiner the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Water Supply and Environmental Engineering.

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List of Acronyms and Abbreviations

ASTU	Adama Science and Technology University
CA	Total Rational Coefficient
CAD	Computer Aided Design
CAT	Total Catchment
CB	Catch Basin
CH	Channel
CHI	Computational Hydraulics Inc.
CM	Catchment
CO	Conduits
CS	Cross Section
DEM	Digital Elevation Model
DGC	Department Graduate Council
ERA	Ethiopian Roads Authority
GIS	Geographical Information System
GU	Gutter
GUID	Gutter Identification
IDF	Intensity Duration Curve
LULC	Land Use Land Cover
MH	Manhole
NA	Not Applicable
OPGS	Office of Postgraduate Studies
PCSWMM	Personal Computer Storm Water Management Model'
PO	Pond
POS	Pond Outlet Structure
SewerGEMS	Sewer Geospatial Engineering Modeling
SGC	School Graduate Committee
SUDS	Sustainable Urban Drainage Systems
SWMM	Storm Water Management Model
UNESCO	United Nations Educational, Scientific and Cultural Organization
US	United States
UTM	Universal Transverse Mercator

Abstract

Urban runoff if not managed properly is a major catastrophic trait of many cities around the world, which is a result of manifold and uncertain factors such as, hydrological factors, urbanization, climate change and infrastructure inadequacy and failures that result in property damage, critical infrastructure distraction and loss of lives. Adama Science and Technology University (ASTU), with approximately 150 ha area, sits on high ground relative to the surrounding flood-prone neighborhoods. Although the university campus is not prone to prolonged flooding, the excess stormwater drains to the downstream catchments without provision of proper stormwater management and stormwater reuse causing erosion and pollutant migration to the downstream areas. The main objective of this research was to assess the stormwater characteristics of ASTU campus and to model its detention ponds for effective stormwater management. To achieve the objective, the ASTU campus catchment was delineated into 18 sub-catchments, and the Sewer Geospatial Engineering Modeling (SewerGEMS) software was used to determine the peak runoff from the campus catchment and to analyze the hydraulics of the drainage system on campus. Additionally, manning equation was used to assess the existing drainage network of ASTU campus. Model simulation for pre- and post-development scenarios was also performed to identify the effects of land use land cover (LULC) change on runoff flow. Moreover, using hydrology studio modeling software, storage ponds were routed to attenuate peak runoff flow. Based on the results of the research, for a 100-year return period, the total peak stormwater runoff flow of the entire catchment was estimated to be 14.64 m³/s. The hydraulic analysis revealed that six pipe sizes with diameters ranging from 150 to 1200 mm, covering a total distance of 10.28 km could securely convey 11.25m³/s 25-year return period peak runoff flow from the campus catchment. The existing drainage network assessment result revealed that channels type 1, 2, 4, and 6 and conduit 42-inch diameter in sub-catchment CM-1, CM-7, and CM-15 were over-designed while channels type 1 and conduit 30-inch in sub-catchment CM-13 and CM-16 were found to be under-designed. According to the modeling results for the pre- and post-development scenarios, 38.23% of what was once cropland with open woodland land cover was converted into impervious land covers, causing the study area's peak runoff flow to rise by 47%, from 5.33 m³/s to 11.25 m³/s. It is also found that, two detention ponds with volumes of 6,408 and 9,384m³ can successfully attenuate 50.9% of the study area's average peak runoff flow. It can be concluded that the peak runoff flow has significantly increased as a result of the establishment of the ASTU campus and the proposed ponds could potentially reduce the risk of flooding and its related problems on the society downstream of the campus. Furthermore, the stored water can be used as a supplementary source of water for the campus.

Keywords: ASTU, Drainage network modeling, Detention Pond, SewerGEMS, Stormwater management

1 Introduction

1.1 Background of the Study

Storm can be defined as the violent disturbance of atmosphere with strong wind and usually rain, thunder or snow and the stormwater has the origin during heavy precipitation events (Manoj et al., 2020). Stormwater discharges are produced when the capacity of the land to retain precipitation is exceeded and run-off occurs. A run-off will be influenced by rainfall intensity (millimeter of rainfall per hour)) and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces (Teferra, 2020).

Urban runoff if not managed properly is a major catastrophic trait of many cities around the world, which is a result of manifold and uncertain factors such as, hydrological factors, urbanization, climate change and infrastructure inadequacy and failures that result in property damage, critical infrastructure distraction and loss of lives. It highly affects the local economy and overall urban environment (Morita, 2014; Mugume & Butler, 2016). Due to these challenges of human environment, the interest in urban flood risk management is increasing (Marie et al., 2011). Generally, structural and non-structural measures are taken to best manage the urban flood risk (Eui et al., 2016).

The pattern of urbanization and modernization in Ethiopia has meant increase densification along with urban infrastructure development. With urbanization, impermeability increases with the increase in impervious surfaces such as residential houses and commercial buildings and drainage pattern is bound to create increasing environmental problems (Elias, 2015).

The traditional civil engineering approach which most cities and universities still employ conveys stormwater from paved surfaces directly into storm drains, carrying it offsite as quickly as possible. This approach, while expedient, only exacerbates flash flooding downstream, affecting lower-lying catchments and causing flash flood, erosion and pollution of nearby local waterways, water bodies and catchments. Universities should be responsible to generate innovative approaches to stormwater management on their campuses, helping to position them as stewards of their environment. Universities, as large land-owners, are uniquely positioned to lead the way in reducing their contribution to flash flood events by exploring creative ways of

handling stormwater onsite. Many urban universities are concerned not only about minimizing flooding on their own campuses but also about being good neighbors to surrounding communities by minimizing their impact on those downstream in the watershed (Sasaki Associates Inc, 2018). ASTU with approximately 150 ha area, which sits on high ground relative to surrounding, flood-prone neighborhoods is an example of such universities. Although the University campus is not prone to prolonged flooding, the excess stormwater drains to the downstream catchments without provision proper stormwater management and stormwater reuse.

Managing stormwater will significantly help to maintain the water quality of nearby water bodies and reduce waterlogging and flooding of downstream areas. Stormwater management can be simply defined as controlling and using stormwater runoff in other activities where water quality is not of much concern.

To manage stormwater runoffs, ranges of methods are used to store and slow down the flow of water. There are a number of terms used to describe these methods, including nature-based solutions, blue-green infrastructure, sustainable drainage system, green-infrastructure and stormwater control measures. Storage ponds or wet storage ponds are a type of nature-based solutions or stormwater control measures often incorporated into new housing developments in urban catchments. Unlike storage ponds, these ponds include a permanent pool of water in their design. They are designed to store some excess water, with an outlet structure that gradually allows discharge to offset potable water use for applications like irrigation, habitat and recreational benefit (Stephen & Vladimir , 2022).

This study aims to characterize the stormwater drainage system and to model its detention ponds for effective stormwater management of ASTU campus. SewerGEMS which is a comprehensive modeling software for the design and analysis of stormwater systems and Hydraulic Studio which is a suite for stormwater design and modeling mainly used for storage pond design, storm sewer design and analysis, were used to estimate the storm water flow of the area, to design drainage network and detention ponds routing and sizing with the aim of improved and sustainable stormwater drainage.

1.2 Problem Statement

Universities usually have substantial amounts of land; therefore, it tend to generate a lot of runoff flow. However, the conventional civil engineering method presently used by most universities transfers rainfall from paved surfaces straight into storm drains, moving it offsite as rapidly as feasible. Despite being practical, this method just makes flash flooding worse downstream, affecting lower-lying catchments and resulting in erosion, pollution, and flash flooding in neighboring local streams, water bodies, and catchments (Lei et al., 2017).

Since its establishment, ASTU has grown to become one of Ethiopia's fastest growing and expanding institutions (www.astu.edu.et, 2020). Rapid growth of the campus transformed part of the once vegetated porous land to hard impervious surfaces. Increased imperviousness causes frequent flooding, erosion, degradation of water quality and waterlogging issues (Yao et al. 2017). Furthermore, ASTU is located at higher altitude relative to the surrounding area, which also contributes to stormwater flooding risk, pollutant migration and pollution of nearby water bodies of downstream areas. The flood emanating from ASTU campus and its surroundings has left large eroded gorges with more than 4 m deep in the campus which continued to the downstream areas. In addition to the erosion, the stormwater runoff ASTU's campus catchment can contain contaminants such as oil, organic matter and toxic metals which are by products from workshops and laboratories. This contaminated surface runoff can result in poor water quality in rivers and streams, adversely affecting biodiversity and amenity value of the downstream areas.

Urban run-off contamination may sometimes be inevitable, and it is impracticable to treat water at each outfall. However, by regulating flows and filtering run-off, Sustainable Urban Drainage Systems (SUDS) may considerably decrease the harm to water resources and enhance the quality of the environments. In order to attenuate and store excess stormwater, which lowers the risk of flooding and flood-related challenges for the campus and surrounding downstream area, this study will simulate the stormwater flow of the campus and design and locate appropriate detention ponds. Additionally, the stored stormwater can be the supplementary source of water for the campus that will be redirected for different purposes. Additionally, there may be an opportunity to use the property as an outdoor classroom.

1.3 Objective of the Study

1.3.1 General objective

The general objective of the study was to assess the stormwater characteristics of ASTU campus and to model its detention pond for effective stormwater management.

1.3.2 Specific objectives

To realize the general objective, the following specific objectives were used as operational steps.

- To develop IDF curve of ASTU campus catchment
- To determine potential peak runoff from ASTU campus catchment
- To model the stormwater drainage network of ASTU campus
- To select and design appropriate detention pond

1.4 Research Questions

In line with the specific objectives stated above, the study tried to get answers to the following research questions.

- What type of IDF Curve can be generated for the ASTU campus?
- How much would the ASTU campus catchment runoff be?
- What is the most suitable stormwater drainage system for conveying runoff from the ASTU campus catchment?
- What is the appropriate type and size of detention pond for the ASTU campus catchment?

1.5 Significance of the Study

Stormwater management is an increasingly important consideration in the design of urban drainage systems. Stormwater management practices, when properly selected, designed, and implemented, can be utilized to mitigate the adverse hydrological and hydraulic impacts caused by drainage facilities, thereby protecting downstream areas from increased flooding, erosion, and water quality degradation.

The technical and scientific explanations for the causes of flood are provided and future solutions are proposed that has the potential to help minimize potential damage to ASTU campus infrastructure from flooding through proper and sustainable drainage systems. In addition, the proposed best stormwater management method and stormwater reuse may benefit the campus and other areas with similar environments. The study will also create an additional water source for the campus that will be used for various purposes. The study can also be used as a reference for the campus in preparing annual spatial plans and budget for roads and stormwater drainage infrastructure, and it can also be used to mitigate flooding and other flood-related risks such as erosion, environmental degradation, water pollution, and health security issues on campus and downstream areas, and finally serves as a general reference for anyone involved in stormwater drainage and stormwater management.

1.6 Scope and Limitations of the Study

The study was constrained by space, time, and theme. It was geographically limited to ASTU campus catchment. It was carried out in order to characterize the stormwater of the campus and to provide the detention ponds storage mechanisms for peak runoff attenuation. The study developed an IDF, evaluated the peak of runoff flow for various return periods using a rational method, assessed the existing drainage network and modeled new drainage network of the study area. Finally, it has designed and located detention ponds that effectively reduce the peak runoff in the study region. However, due to time and resource limitations, the study did not assess alternatives for stormwater quality and treatment to reduce the study area's increased water demand. For the same reasons, it also did not include an evaluation of runoff generation from the campus's upstream area, which is another factor in the challenges of flooding and the risks associated with it for the campus and areas downstream.

Due to the existing network is disintegrated, it was difficult to use it as a conveys system to transport stormwater to the proposed ponds; instead, the study used the existing drainage network layout and modeled a closed pipes new drainage network to convey stormwater to the proposed ponds.

2 Literature Review

2.1 Stormwater

Stormwater is the water that is received from infrastructures like rooftops, roadways, parking lots, etc. after a rainfall event; and runs over the surface of the earth, drainage lines. It ultimately joins the nearby water bodies. Its properties in terms of quantity and quality are determined by the nature and characteristics of both the rainfall and the catchment.

Urban drainage network channels try to overcome the runoff to much extent by accommodating the generated runoff within it. But the increased human activity and their produced conditions such as imperviousness and manmade water courses lead faster rainfall to runoff transformation (Rupak et al., 2009). A sustainable urban water system, or stormwater system, is not only a question of problems and avoiding unwanted content in the water, it can also be a question of its potential usability as a water resource in society. Stormwater drainage may not only be considered as systems to divert undesired water from urban areas, but also as a valuable element for landscaping the surrounding of buildings and roads (Nivedita & Pratap , 2015).

2.1.1 Runoff Generation

Rain falling over an urban watershed will strike either a pervious surface or an impervious surface. On pervious surface most of the rainfall infiltrated to the subsurface and some remains runoff as over land flow and depression storage. Depression storages are small pores on land surfaces which temporarily store water. Some portion of the runoff water may be evaporated (Rupak et al., 2009). Impervious covers, such as asphalt, concrete or a bare soil highly consolidated by traffic, contribute to high runoff rates because of low hydrologic losses (Geiger et al, 1987). The water that was formerly able to infiltrate soil now flows off surfaces, creating a greater volume of runoff, which ends up converging to regions of a lower topographic level, thus generating areas of flooding. The increase of surface runoff may also lead to overflow of streams, gullies, ditches, and rivers, creating problems of riverside flooding. Flooding results in risks to people's health and quality of life, in addition to social and economic losses (Poletto & Tassi, 2012)

2.1.2 Flood

Floods are extreme natural event and can occur in a wider geographical scale like river basin or in a smaller scale such as catchment and watershed (Ochoa-Rodriguez, et al., 2015). Flooding is typically coupled to water moving faster than normal, in part because of the weight of an increased amount of water upstream, leading to an increase in the pressure gradient that drives the flow. In contrast to rural flood, urban floods can have both area-wide and localized origin. Impacts of urbanization could entail a substantial increase in frequency and magnitude of flooding (Zhou, 2014). In case of inland cities far away from shores, urban floods have become more severe and occur in the built-up areas as a result of heavy rain and subsequent rainfall runoff. They are further exacerbated by the high intensity rainfall in cities combined with inappropriate sewer system and diverse land cover (Salvan et al., 2016).

Floods produce damage through the immense power of moving water and through the deposition of dirt and debris when flood waters finally recede. The energy of that moving water goes up as the square of its speed; when the speed doubles, the energy associated with it increases by a factor of four. In most cases, the damage potential of the flood is magnified by the debris that the waters carry: trees, vehicles, boulders, buildings, etc. When the waters move fast enough, they can sweep away all before them, leaving behind scenes of terrible destruction (Zhou, 2014).

2.1.3 Stormwater Quantity

Rain falling over an urban watershed will strike either a pervious surface or an impervious surface. On pervious surface most of the rainfall infiltrated to the subsurface and some remains runoff as overland flow and depression storage. Depression storages are small pores on land surfaces which temporarily store water. Some portion of the runoff water may be evaporated (Rupak et al., 2009). It has long been recognized that covering land with impervious surfaces, such as roofs and roads, reduces both the volume of water infiltrating into soils and the volume of water lost to the air through evapotranspiration, thus increasing the volume of runoff following rain events (Leopold, 1968).

2.1.4 Stormwater Quality

Sources of urban storm water runoff are related to untreated wastewater, households, road network, fuel stations, parking lots, and parks. However, some other factors also play a crucial

role in urban run-off pollution: atmospheric pollutants deriving from urban transport and deposited in urban surface; pollutants deposited in surfaces like roads, roofs and parking lots; pollution deriving from construction fields; the use of pesticides in parks and green space; and deicing materials used in roads during winter. The most important pollutants, included in urban runoff, are sediments, nutrients (nitrogen and phosphorus), chlorides, heavy metals, diesel hydrocarbons, microbial pollution, organic compounds and litter (Yannopoulos et al., 2013). Based on the study by Alemu (2018), regarding the impact of urbanization on surface water quality in Gonder Town, Kecha River was labeled as a river highly polluted due to anthropogenic activities. Land use change has great impact on Kecha River so that water quality has decreased in the past 15 years. The quality of river water decreased from upstream to downstream.

Based on wet and dry season sampling of water from three rivers in the Little Akaki catchment in Addis Ababa, the rivers have a poor water status when compared to international standards. This includes low oxygenation, too high salt concentrations and tendency to too alkaline pH, high nutrient status of both nitrogen and especially phosphorus, and severe heavy metal contamination with chromium and copper. Even when compared to the less strict Ethiopian standards phosphate is still problematic, and in two of the three rivers also chromium and manganese. (Adugna et al., 2019).

2.2 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 (Poletto & Tassi, 2012).

The combined effects of population growth, urbanization and industrializing change adversely the hydrological response of the affected area and cause various environmental impacts. The natural environment is changed to a man-made one, with concomitant quantitative and qualitative changes of the water cycle. Urban drainage is part of this cycle and is related to the urban hydrological system in a very complex way (Geiger et al., 1987). Changes in the local

hydrologic cycle and its consequences due to the urbanization process, directly affect the environmental balance and, thus, the quality of life of the resident population. Problems arise both due to the inflows to the system, which in future will be less certain, and increasing downstream hydraulic and regulatory constraints on outflows (Ashley et al., 2012).

The quantitative evidence from the analysis of the three period maps of the Addis Ababa indicated that expansion of built environment is significant and continues at an alarming rate of modifying the flood regime of the city. The mean runoff transformation potential (runoff coefficient) of the city has increased from 28% in 1984 to 45% in 2002. The mean increase in flood volume between 1984 and 2002 was more than 60%. The peak flood discharge of selected watersheds exceeds from 180% to 300% in some part of Addis Ababa (Moges et al., 2019). A typical impact of urban development on recipient waters is increased amounts of water entering them, and a lower time of concentration. The combination of these two events can result in altered stream paths due to erosion of streambeds and stream banks, and more rapid runoff. Rapid runoff and impermeable surfaces also prevent infiltration to soil and aquifer recharge, which can lead to lower sustained Stream flows, especially during dry periods (Hawaz, 2020).

2.3 Components of stormwater drainage system

2.3.1 Catchment Areas

A catchment is the geographical area that "catches" the rainfall and directs it towards a common discharge point within the storm collection network.

2.3.2 Runoff coefficients

The runoff coefficient (C) is a dimensionless coefficient relating the amount of runoff to the amount of precipitation received. It is a larger value for areas with low infiltration and high runoff (pavement, steep gradient), and lower for permeable, well vegetated areas (forest, flat land). It is important for flood control channel construction and for possible flood zone hazard delineation. A high runoff coefficient (C) value may indicate flash flooding areas during storms as water moves fast overland on its way to a river channel or a valley floor. It is measured by determining the soil type, gradient, permeability and land use. The larger values correspond to higher runoff and lower infiltration. (ERA, 2013) (Table 2.1)

Table 2.1: Runoff coefficient for various land uses (source: ERA, 2013)

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential: Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: 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
Woodland, Sandy Loam Soils	0.1-0.3
Sports Fields	0.2-0.35
Parks	0.1-0.25
Asphalt	0.7-0.95
Concrete	0.7-0.95
Concrete Blocks, uncemented Joints	0.5-0.7
Grass, Poor Condition	0.32-0.62
Grass, Fair Condition	0.25-0.6
Grass, Good Condition	0.21-0.58

Most catchments are comprised of more than one type of ground cover. For example, a roadside drainage inlet may accept flow from the paved roadway, the curbside grass, and a nearby wooded area. To account for the effects of each of these areas, multiply each corresponding sub-catchment area and rational coefficient, then add the values to obtain the total rational coefficient (CA) for the entire catchment.

Since the rational coefficient is unit less, CA values have units of area. A weighted value for the rational coefficient can be determined by dividing the catchment's total CA by the total catchment area. Rather than tracking area and weighted rational coefficients separately, rational loads are often described solely by using the total CA value. Since the C coefficient and area are multiplied together in the rational formula anyway, there is no adverse effect of this simplification on flow determination.

$$CA_T = \sum_{i=1}^n (C_i \times A_i) \dots\dots\dots \text{Eq. 2.1}$$

Where: CA_T = Total Catchment CA (ha)

C_i = Individual sub catchments rational coefficients (unitless)

A_i = Individual sub catchment areas (ha)

2.3.3 Catch Basins

Catch basins convey surface water into a storm sewer pipe system. A catch basin (a.k.a., storm drain inlet, curb inlet) is an inlet to the storm drain system that typically includes a grate or curb inlet where stormwater enters the catch basin and a sump to capture sediment, debris and associated pollutants. They are also used in combined sewer watersheds to capture floatable and settle some solids (Bentley Systems, 2019).

2.3.4 Manholes

Manholes are placed in a sewer system to provide access for inspection, maintenance, and emergency service. Manholes should be placed at sewer junctions (i.e., tees, wyes, and crosses), upstream terminal ends of sewers, and locations where there is a change in sewer grade or direction. Manholes are locations where loads enter the gravity portion of the sewer system (Bentley Systems, 2019).

2.4 Runoff determination

Several methods will be appropriate for predicting peak flood rates and volumes. Comparison of hydrologic prediction methods on recurrence interval curves should be performed in selection of peak flow rates for a drainage design. The most widely methods that can be used

in determining peak flood magnitudes for design of road drainage structures are rational method (ERA, 2013)

2.4.1 Rational Method

The Rational Method is most accurate for estimating design storm peak runoff for areas up to 50 hectares (0.5 square km). This method, while first introduced in 1889, is still widely used. Even though it has come under frequent criticism for its simplistic approach, no other drainage design method has achieved such widespread use.

Characteristics of the Rational Method, it generally limit its use to 50 hectares include, the rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or longer than the time of concentration, the frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration, the fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume and the peak rate of runoff is sufficient information for the design.

The rational formula estimates the peak rate of runoff at any location in a catchment area as a function of the catchment area; runoff coefficient; and mean rainfall intensity, for a duration equal to the time of concentration. The rational formula is expressed as: (ERA, 2013)

$$Q = 0.278 C I A \dots\dots\dots \text{Eqn. 2.2}$$

Where: Q = Maximum rate of runoff, m^3/s

C = Runoff coefficient representing a ratio of runoff to rainfall (Table 3-2)

I = Rainfall intensity for a duration equal to the time of concentration and for design return period, mm/hr

A = catchment area tributary to the design location, km^2

2.4.2 Intensity Duration Curve

The rainfall intensity–duration–frequency (IDF) curves are graphical representations of the probability that a given average rainfall intensity will occur within a given period of time. Providing mathematical relationship between the rainfall intensity the duration, and the return period T (or equivalent to the annual frequency of exceedance), the IDF curves allow for the estimation of the return period of an observed rainfall event or conversely of the rainfall intensity corresponding to a given return period. Design storms derived from IDF curves are

commonly adopted in water resources engineering for designing of urban drainage systems, evaluating the endurance of hydraulic structures, and assessing regional flood vulnerabilities (Sun et al., 2019)

The basic principle behind the IDF relationship is finding how to relate the intensity of a rainfall event and its duration to its expected frequency. Precipitation intensity is the depth rainfall per unit of time, and is usually expressed in the units of mm/hr and in/hr. The average intensity is frequently applied, which is expressed as the rainfall depth for a particular precipitation event divided by the duration of that event. The frequency is often described in terms of return period, which is the average period of elapsed time between rainfall events that are equal to or more than the magnitude of design. The return period is the inverse of the annual probability of exceedance of an event. The return periods usually represented are for 2, 5, 10, 25, 50 and 100 years (Elsebaie, 2012). Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves.

2.4.3 Return Period and Frequency

The return period and frequency are statistical descriptors of the severity of a storm event. The return period is the expected length of time between two rainfall events that exceed a specific magnitude. Frequency, or exceedance probability, is the inverse of the return period. As you might expect, the higher the return period, the more infrequent the storm event, and the higher the intensity of the rainfall.

For example, a storm with a 5-year return period represents an event that is expected to be exceeded once every five years (on average). The frequency is $1/5$, which means that there is a 20% probability of a storm exceeding that magnitude occurring in any given year. The return period does not mean that two storm events exceeding a given magnitude will not occur in the same year, nor does it guarantee that a storm event exceeding this magnitude will occur within any given five-year span. It just means that these storms will occur at an average rate of once every five years (ERA, 2013).

2.4.4 Estimation of Short Duration Rainfall

Design and analysis of drainage structures require rainfall intensity of shorter duration. However, the available rainfall depth is for 24 hr duration so that it is mandatory to convert it

to shorter duration. Short-duration rainfall is critical for the small catchments and urban drainage systems. There is always a shortage of short-duration rainfall data as it requires automatic rain gauges to record such data. On the contrary, daily rainfall values are generally available due to the use of cheap manual Instruments. Ethiopian Road Authority Drainage Design Manual (2013), suggests the following equation.

$$R_{Rt} = \frac{t}{24} \frac{(b+24)^n}{(b+t)^n} \dots\dots\dots \text{Eq. 2.3}$$

Where: R_{Rt} = Rainfall depth Ratio R_t : R_{24}

R_t = Rainfall depth in a given duration 't'

R_{24} = 24 hr rainfall depth

b and n = coefficients $b=0.3$ and $n=(0.78-1.09)$.

The methods employed to develop IDF curve for the shorter duration events using the above Equations were as follows:

Using Gumbel-Extreme Value Distribution Type I method of frequency analysis, R_{24} were calculated for 2, 5, 10, 25, 50 and 100-year return period.

Rearranging the above Equations. gives

$$R_t = \frac{t}{24} \frac{(b+24)^n}{(b+t)^n \times R_{24}} \dots\dots\dots \text{Eq. 2.4}$$

Intensity (mm/hr) $I_t = \frac{R_t}{t}$. Substituting in the above Eq. gives

$$I_t = \frac{R_{24}}{24} \frac{(b+24)^n}{(b+t)^n} \dots\dots\dots \text{Eq. 2.5}$$

The resulting table is graphed for each return period.

2.4.5 Gumbel Distribution Method

According to (Elsebaie, 2012), the Gumbel distribution is the most widely used distribution for IDF analysis owing to its suitability for modeling maximum. It is relatively simple and uses only extreme events (maximum values or peak rainfalls). The Gumbel distribution calculates the 2, 5, 10, 25, 50 and 100 years return intervals for each duration period and requires several calculations. Frequency precipitation P (in mm) for each duration with a specified return period T_r (in year) is given by

$$P_T = P_{av} + K_T S \dots\dots\dots \text{Eq. 2.6}$$

Where K_T is Gumbel frequency factor given by:

$$K_T = \frac{-\sqrt{6}}{\pi} \left\{ 0.5772 + \ln \left[\ln \left(\frac{T_r}{T_r - 1} \right) \right] \right\} \dots\dots\dots \text{Eq. 2.7}$$

and P_{av} is the average of the maximum precipitation corresponding to a specific duration. In utilizing Gumbel's distribution, the arithmetic average is used:

$$P_{av} = \frac{1}{n} \sum_{i=1}^n P_{(t)} \dots\dots\dots \text{Eq. 2.8}$$

Where,

n is the number of events or years of record and the standard deviation S is calculated by:

$$S = \sqrt{\frac{(P - P_{ave})^2}{n-1}} \dots\dots\dots \text{Eq. 2.9}$$

Where; P Maximum precipitation depth corresponding to a specific duration.

Then the rainfall intensity (mm/h) for return period T is obtained from:

$$I_T = \frac{P_T}{T_d} \dots\dots\dots \text{Eq. 2.10}$$

2.4.6 Time of concentration

Hydrologic methods require an estimation of the time of concentration. This section provides guidance on ways to estimate time of concentration and covers the following topics: description, flow components, and procedure to estimate time of concentration.

The time of concentration (T_c) is used in the Rational Method to determine the critical rainfall duration, which can then be combined with an appropriate rainfall IDF relation to establish the required design rainfall intensity. The T_c is the time required for water to flow from the most remote point of the basin to the location being analyzed.

A storm equal to this duration will permit direct runoff to arrive from all points in the watershed concentrating at the outlet. This time measure is taken to be the critical time by many flood-estimating approaches, in that it is assumed that the use of any other time would result in a lower flood estimate. A shorter time, although resulting in higher rainfall intensity, will not

permit the entire basin to contribute flow simultaneously. A longer duration allows the entire basin to contribute, but with a lower intensity (ERA, 2013).

Many different T_c formulas are available, and it is through the selection and use of these formulas result in greatest error that typically occurs in applying the Rational Formula to non-urban watersheds. While some error occurs in assigning runoff coefficients, as there is a large range of possible values for each surface condition, the realistic determination of the response time of the basin is the greatest challenge, particularly for natural (rural) basins.

Overland flow is the type of flow that occurs in small, flat or in upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface. The Kerby formula is recommended for the calculation of T_c in this case. It is only applicable to parts where the slope is fairly even (ERA, 2013).

$$T_c = 0.604 \left(\frac{rL}{S^{0.5}} \right)^{0.467} \dots\dots\dots \text{Eq. 2.11}$$

Where: T_c = time of concentration (hours)

r = roughness coefficient obtained from Table 5-6

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

S = Slope of the catchment = (m/m)

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the United States (US) Soil Conservation Service.

$$T_c = \left(\frac{0.87L^2}{1000S_{av}} \right)^{0.385} \dots\dots\dots \text{Eq. 2.12}$$

Where: T_c = time of concentration (hours).

L = hydraulic length of catchments measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

S_{av} = average slope (m/m).

In most cases, the longest water path includes both overland and channel flow. In large catchments, the channel flow is usually dominant, but in small catchments, it may be necessary to determine T_c as the sum of flow times, for overland and channel flow. To obtain a broad indication, it may be accepted that a defined watercourse exists when the average slope of the catchment is greater than 5 percent, and the catchment itself is larger than 5 km² (ERA, 2013).

2.5 Sustainable drainage systems

In urban areas, where many surfaces are sealed by buildings and paving, natural infiltration is limited. Instead, drainage networks consisting of pipes and culverts divert surface water to local watercourses. In some cases, this has resulted in downstream flooding and deterioration in river water quality. This is caused when foul sewers are overwhelmed by surface water, leading to a release of dirty water into rivers. SUDS are drainage solutions that provide an alternative to the direct channeling of surface water through networks of pipes and sewers to nearby watercourses (British Geological Survey, 2022)

By mimicking natural drainage regimes, SUDS aim to reduce surface water flooding, improve water quality and enhance the amenity and biodiversity value of the environment. SUDS achieve this by lowering flow rates, increasing water storage capacity and reducing the transport of pollution to the water environment (British Geological Survey, 2022). SUDS use a sequence of techniques that together form a management train. As surface water flows through the system, flow velocity is controlled and pollutants are removed. The management train may include the following stages: (British Geological Survey, 2022)

- Source control methods that decrease the volume of water entering the drainage/river network by intercepting run-off water on roofs for subsequent re-use (e.g. for irrigation) or for storage and subsequent evapotranspiration (e.g. green roofs)
- Pre-treatment steps, such as vegetated swales or filter trenches, that remove pollutants from surface water prior to discharge to watercourses or aquifers
- Retention systems that delay the discharge of surface water to watercourses by providing storage within ponds, retention basins or wetlands
- Infiltration systems, such as infiltration trenches and soak ways, that mimic natural recharge, allowing water to soak into the ground

SUDS Site control describes those SUDS features within or at the edge of developments that provide a second or third treatment stage including storage for run-off that has been conveyed from source control structures. The most common features are detention and retention basins, swales and small urban ponds, providing cost effective and easily managed landscape structures for temporary storage of water and to trap and treat pollutants before they move further through the site (Poletto & Tassi, 2012). Some SUDS site control features are discussed below.

1. Retention Ponds

Retention Ponds can be particularly effective during high intensity rainfall events as they can provide attenuation and treatment for rainwater and runoff. To reduce the risk of flooding in areas, some ponds are designed to slowly release the runoff and rainwater they have collected and stored over time, allowing for controlled flow rates.

This retention method reduces peak flows when an area is at risk from flooding, reducing the overall risk. Retention ponds should function in both dry and wet conditions, and submerged aquatic vegetation should be used. They are only suitable for properties which have enough land available, and this land will most likely need re-landscaping if the retention pond is being retrofitted (Poletto & Tassi, 2012).

2. Wetlands

Wetlands are quite similar to retention ponds. They are shallow ponds and marshy areas filled mostly with aquatic vegetation. They treat surface water runoff through sedimentation and filtration. Wetlands slow down the flow of runoff for a period of time to allow for sediment settlement and filtering. Wetlands remove fine sediments, dissolved nutrients, metals and particulates by filtration through vegetation and aerobic decomposition. Overall, they are very useful for the attenuation and treatment of runoff. This will have a positive effect on the water quality of the runoff which will eventually enter local watercourses (Poletto & Tassi, 2012).

3. Bioretention Strips

Bioretention strips are vegetated areas with sand and gravel beneath. They are designed to channel, filter and cleanse runoff vertically. The runoff can either infiltrate into the ground below or drain to a pipe which carries the water elsewhere. Often, they are vegetated areas

providing some aesthetic value and can be of varying size. Their surface area is around 5-10% of the drained area with storage below. The storage of runoff and rainwater can reduce peak runoff rates and encourage infiltration which reduces the overall flood risk (Poletto & Tassi, 2012).

4. Detention Pond

Detention ponds are storage basins that control the flow of water and the attenuation of runoff. As a result, they can reduce the local flood risk of an area by storing runoff and rainwater. They also allow for the settlement of some sediment and particulate pollutants.

Detention ponds are usually put in place if the treatment of runoff is required, this will improve the overall water quality of the stored water. They can also be used as landscaping and for wildlife habitats.

They can be useful in varying magnitudes of rainfall events and are easy to design and construct. They are easy to maintain and can be used for a number of different purposes not just for drainage and flood risk. Detention basins can serve as both a sustainable drainage system and a recreational area. When wet, they can be used as a pond for wild life, this will increase the biodiversity of the area and may be useful as an educational resource (Poletto & Tassi, 2012).

2.6 Detention Pond attenuation

Attenuation design is often undertaken as part of a larger Sustainable Drainage System (SUDS) design. Attenuation systems are large storage facilities included in a drainage system so that rainfall from storm events can be stored onsite and slowly discharged instead of discharging the whole storm event as it happens. These systems are required where there is a limitation to the quantity of rainwater which can be discharged from a site. This is common when discharging from a large site to a small outfall or public sewer.

The quantity of rainfall must be calculated for rain events of different durations to determine the worst-case design storm. The design outflow from the system is then used to determine the maximum water storage volume required. Then a tank of appropriate dimensions can be designed to store the water while it is slowly discharged. Note that the void ratio of the tank must be taken into account when designing the storage. (CivilWeb Spreadsheets, 2019)

2.7 Selection Criteria of Hydrological Model

Each model type serves a purpose, and a particular model type may not categorically be considered more appropriate than others in all situations. Choice of a suitable model structure relies heavily on the function that the model needs to serve. Herrera-Granados (2010) suggested four criteria that can be used to choose between alternative models:

- 1) Accuracy of prediction: Accuracy of prediction of system output is obviously very important; it is desired when all other factors being equal, the model with minimum error variance would be superior.
- 2) Simplicity of the model: refers to the number of parameters that must be estimated and the ease with which the model can be explained to clients or public bodies.
- 3) Consistency of parameter estimates: If the optimum values of the parameters are very sensitive to the particular period of the record used, or if they vary widely between similar catchments, the model will probably be unreliable.
- 4) Sensitivity of results to changes in parameter values: models should not be extremely sensitive to input variables that are difficult to measure.

2.8 Comparison and selection of hydrological model

1. Stormwater management model (SWMM-5)

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 is a complete rewrite 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 (Teamir, 2018).

2. *PCSWMM*

PCSWMM is a third-party interface for SWMM and is developed by Computational Hydraulics Inc(CHI). It is a GIS integrated model that uses SWMM 5.0 as the model computational engine for hydrologic and hydraulic calculations. It is a stand-alone modeling tool with all necessary GIS- tools included and it has support for various CAD and GIS formats (Ydnekachew, 2019).

3. *CivilStorm*

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 Low Impact Development Structures as well as studying the water quality (Ydnekachew, 2019).

4. *SewerGEMS*

SewerGEMS is a program for the design, analysis and planning of gravity flow and pressure low through pipe networks. The operational behavior of various gravity and pressure network elements (manholes, outlets, junction chambers, pressure junctions, wet wells and pumping stations) can be simulated. Prototype elements can be defined with default characteristics, reducing data entry requirements if a group of network elements share common data (Freni et al, 2008). SewerGEMS software is used to analyses the pipe networks, design the flow, velocity, and surface features topography in somehow to delineate the paths and drawn the profiles to show the ground surface, elevations. SewerGEMS can run both Steady State Analyses, modeling a single instant in time, and extended period analyses, modeling a network over a specified duration of time; moreover, the program allows to automatically design gravity piping and structures, specifying the elements to be designed, from a single pipe size to the entire system, or anything in between, intending the program's design only as a preliminary step (Freni et al, 2008).

Table 2.2: Comparison some features of the modeling tools (Ydnekachew, 2019).

Some features of the modelling tools		SWMM	PCSWMM	StormCAD	CivilStorm	SewerGEMS
Developer/ Publisher		EPA	CHI	Bentley	Bentley	Bentley
Water systems	Stormwater	Yes	Yes	Yes	Yes	Yes
	Wastewater	Yes	Yes	No	No	Yes
	River systems	No	Yes	No	No	Yes
Area of use	Water quantity	Yes	Yes	Yes	Yes	Yes
	Water quality	Yes	Yes	No	Yes	Yes
	Sewer system	Yes	Yes	No	No	Yes
	SUDS	Yes	Yes	No	Yes	Yes
	Long term predictions	Both	Both	No	No	Both
	Simulation of 1D pipe flow	Yes	Yes	Yes	Yes	Yes
	2D Overland flow	No	Yes	No	No	Yes
Import/ Export/ Connections	GIS	Interchange	Integration	Some conversion utilities from GIS	Some conversion utilities from GIS	Import/ Export
	CAD	No	Yes (various formats)	can be used within AutoCAD	can be used within AutoCAD	Import/ Export

After the above comparisons, it is recommended to select SewerGEMS as it is user friendly, applicable anywhere, requires minimal data, use flexible scale, relevant to many kinds of decisions, allows multi-service assessment, considers landscape context, low data availability, low temporal resolution and scenarios analysis and large-scale planning while other water models need relatively high data availability, improve process representation, high temporal resolution, and need high confidence in results.

3 Materials and Methods

3.1 Description of the Study Area

3.1.1 Location

ASTU is a university in Ethiopia, found in Adama town East Shewa zone in Oromia regional state, at 100 km road distance from the capital city of Ethiopia, Addis Ababa. Geographically, it is located at $8.5644743^{\circ}\text{N}$ and 9.29208°E (Figure 3.1). The university was established in 1993 as Nazareth Technical College, and was later renamed as Nazareth College of Technical Teacher Education, specializing in training technical teachers until 2003. The same college became a university, namely Adama University, in 2006. Five years later, the university once again changed its name to Adama Science and Technology University (ASTU) (Website-www.astu.edu.e, 2020)

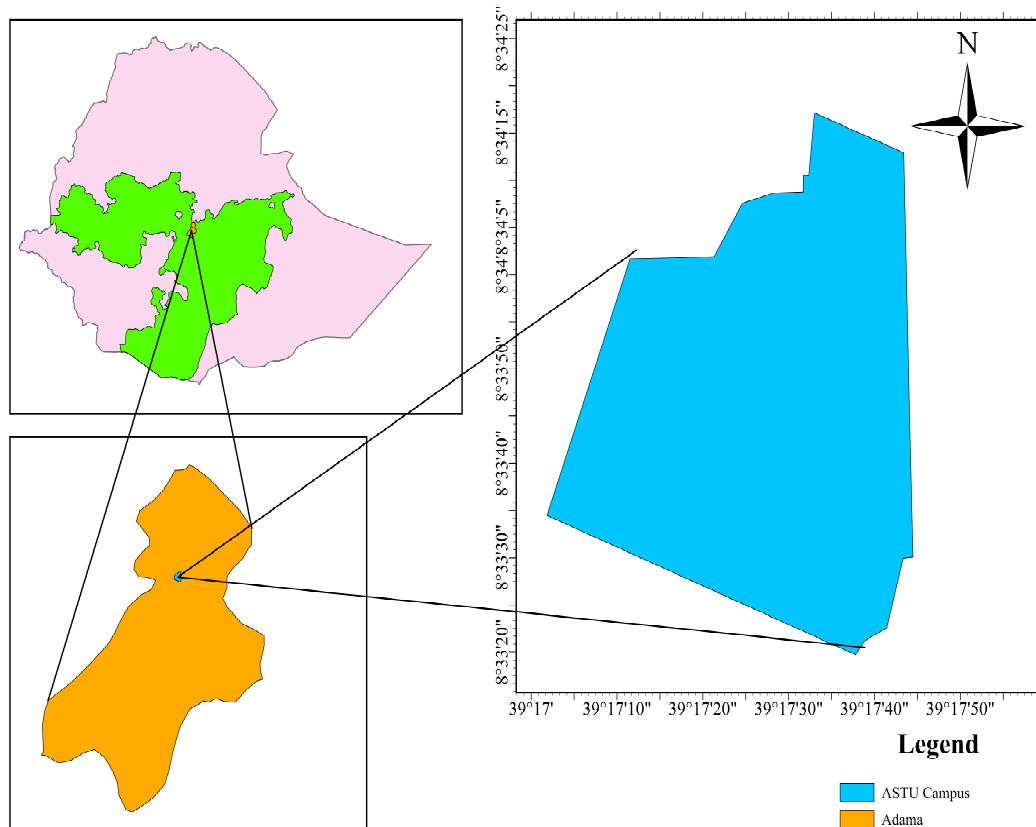


Figure 3.1: Location of Study Area (Source: ASTU construction project office)

3.1.2 Topography

ASTU is located at an elevation of 1712m a.s.l, The city sits between the base of an escarpment to the west, and the Great Rift Valley to the east. The elevation map of ASTU is generated using digital elevation map (DEM) from NASA's 30 m resolution Shuttle Radar Topography Mission data (Figure 3.2).

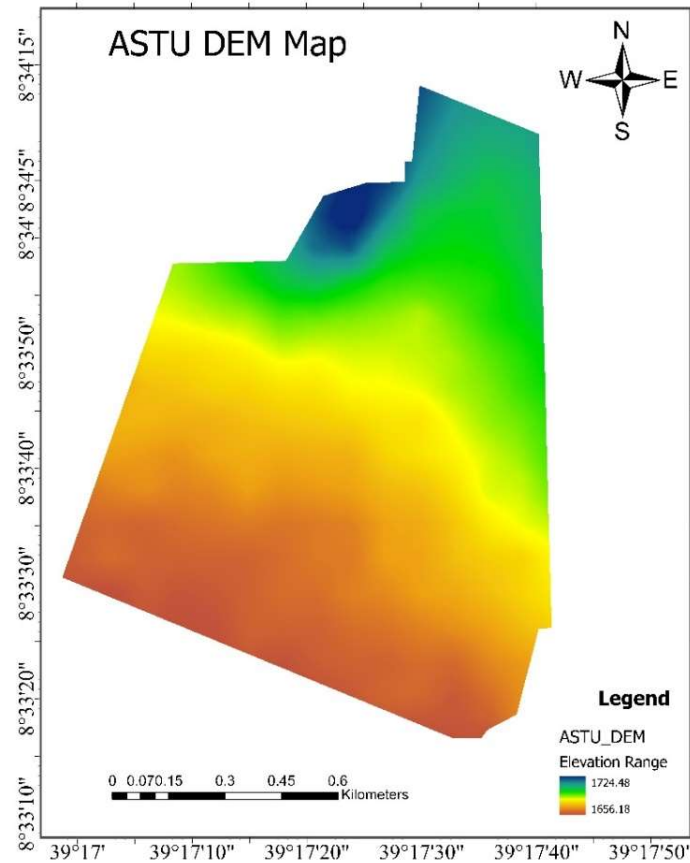


Figure 3.2: Topography map of Study Area

3.1.3 Land use /land cover

Land use land cover (LULC) assessment was carried out for the ASTU campus area. Based on satellite images interpretation, Master plan map of the campus and field observations, the campus is covered with 8 types of land cover (Figure 3.3). The most three dominant LULC in the area are grassland, woodland and roof catchments (buildings) with 43.14%, 18.63% and 15.22% coverage respectively (Table 3.1).

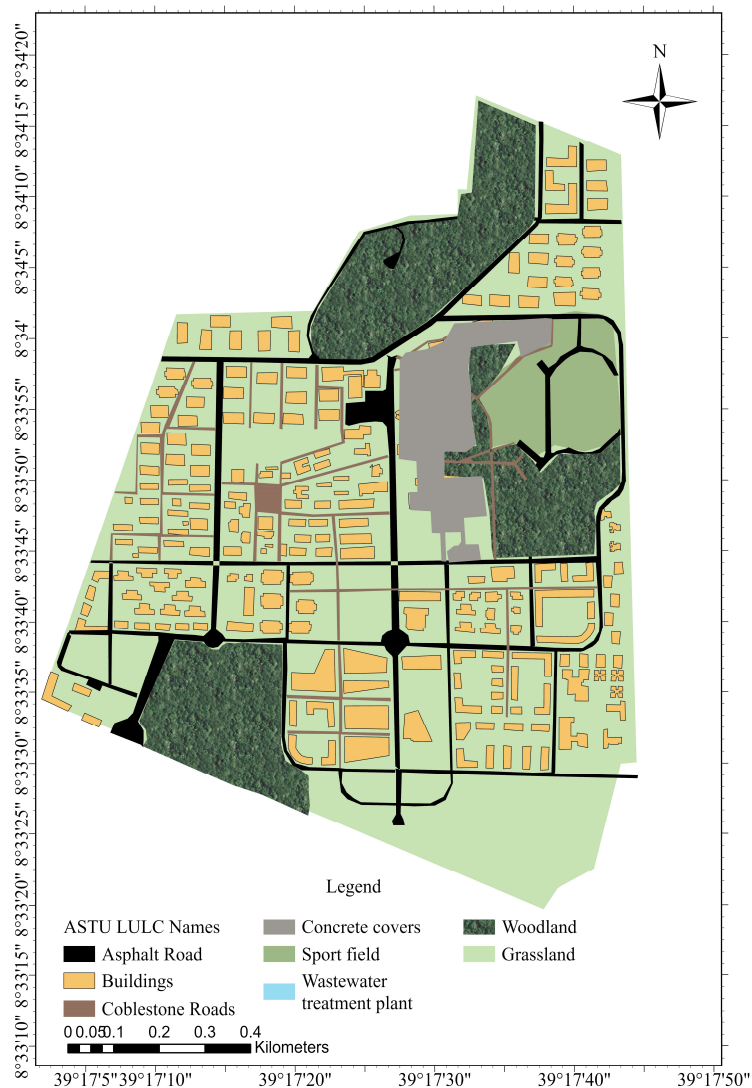


Figure 3.3: LULC map of Study Area (source: Construction project office of ASTU)

Table 3.1: LULC of ASTU campus

<i>S.no</i>	<i>Land Use Land Cover</i>	<i>Area (ha)</i>	<i>Coverage (%)</i>
1	Roof Catchment	22.38	15.22
2	Cobblestone Road	4.06	2.76
3	Asphalt Road	12.24	8.33
4	Grassland	63.41	43.14
5	Woodland	27.39	18.63
6	Concrete cover	7.1	4.83
7	Sports Fields	6.99	4.76
8	Waste water treatment area	3.43	2.33

3.1.4 Climate of the Study Area

I. Rainfall

The rainfall recorded at Adama meteorological station for the past years indicated that the average annual rainfall is about 866.25 mm. The maximum monthly average rainfall is about 200 mm (Table 3.1). Most of the rain occurs in summer season (June to September). (Rainfall intensity data is not available for Adama town. Thus, the data recorded at the nearby station and the average of maximum intensity of rainfall in the three stations is about 39.7 mm/hr.

Table 3.2: Monthly total rainfall of Adama (mm) 20011-2021

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall (mm)	2.3	28.5	63.6	54.2	3.9	57.6	196.1	220	108	17.4	12	14

II. Temperature

The mean annual temperature is 21⁰C. It can be classified as semi-humid to semi-arid climate, which characterizes the altitude range between 1300 to 1800m a.s.l. In Adama the hottest month with the maximum mean temperature of 31 ⁰C is May. The monthly minimum mean temperature is November and December with the temperature of 11.5 ⁰C. The maximum temperature varies between 25.8 and 31 ⁰C while the minimum monthly values vary between 11.5 and 17 ⁰C.

III. Wind speed and direction

The available wind speed measured five times a day at Adama station indicates that wind with minimum speed occurs during September (1.65 m/s), while the wind with high velocity occurs during December, January and February (3.05-3.2 m/s). The values calculated over 10 years (1990 to 1999 E.C) indicate that mean is 2.31 m/s. Generally, the wind speed at Adama is high. During summer month the wind speed portrays decreasing trend. In ten years, period (1990-1999 E.C), the maximum wind speed recorded was about 10m/s.

3.2 Data Collection

The initial phase of work began with the collection of the necessary data for modeling and an evaluation of existing data. Digital elevation model (DEM), Daily precipitation, Catch Basin and Manholes coordinates and elevation, Flow Length, Average Slope, LULC, Runoff Coefficient "C," and Catchment & Sub-catchments map are the input data required for the SewerGEMS model. The Meteorological Agency, the ASTU university project management office, and secondary web data sources provided the majority of this data.

A. Contour map of the study area

A DEM which is a grid representation of elevation contour lines with a resolution of 30 m by 30 m was acquired from Ministry of Water and Energy and then projected to Universal Transverse Mercator (UTM) on the spheroid of World Geodetic System 84 Zone 37N. The contour map of the campus was created using the projected DEM (Figure 3.3). According the contour map, the catchment's lowest and highest altitudes, respectively, are 1656 m and 1725 m above sea level.

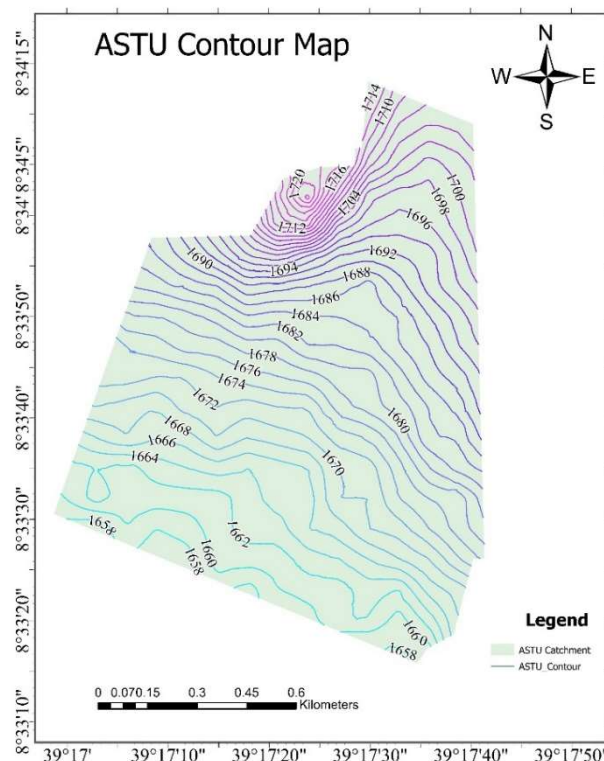


Figure 3.4: Contour map of ASTU campus

B. Daily maximum precipitation

Meteorological data sets are the key inputs for hydrological modeling purpose. The National Meteorological Agency provided the meteorological data, which included daily precipitation, over the 30-year period from 1992 to 2022. Then, using the acquired rainfall data, a maximum daily precipitation dataset was created for the IDF curve formulation. The IDF was then used as an input for the SewerGEMS modeling software, which was used to determine the total runoff generated from the study area.

C. Catch Basin and Manholes

A catch basin (CB) or storm drain is a curbside drain with the sole function of collecting rainwater from our properties and streets and transporting it to local waterways through a system of underground piping, culverts and/or drainage ditches. While a Manhole (MH) is a belowground round or square structure which is used on storm-water sewer lines for the conveyance of rainwater. MH have a widely used as an access point for making connections, inspections, valve adjustments, or to perform maintenance. The invert elevations and ground elevations are principal input parameters for both CB and MH. ASTU's catchment CB are located at the end of each sub catchments following the contour lines to catch stormwater that falls on that specific sub catchments. The MHs are assigned in the network following the criteria in ERA drainage manual and following the existing drainage network line. Accordingly, there are 16 catch basins and 20 manholes as depicted in the Figure 3.5.

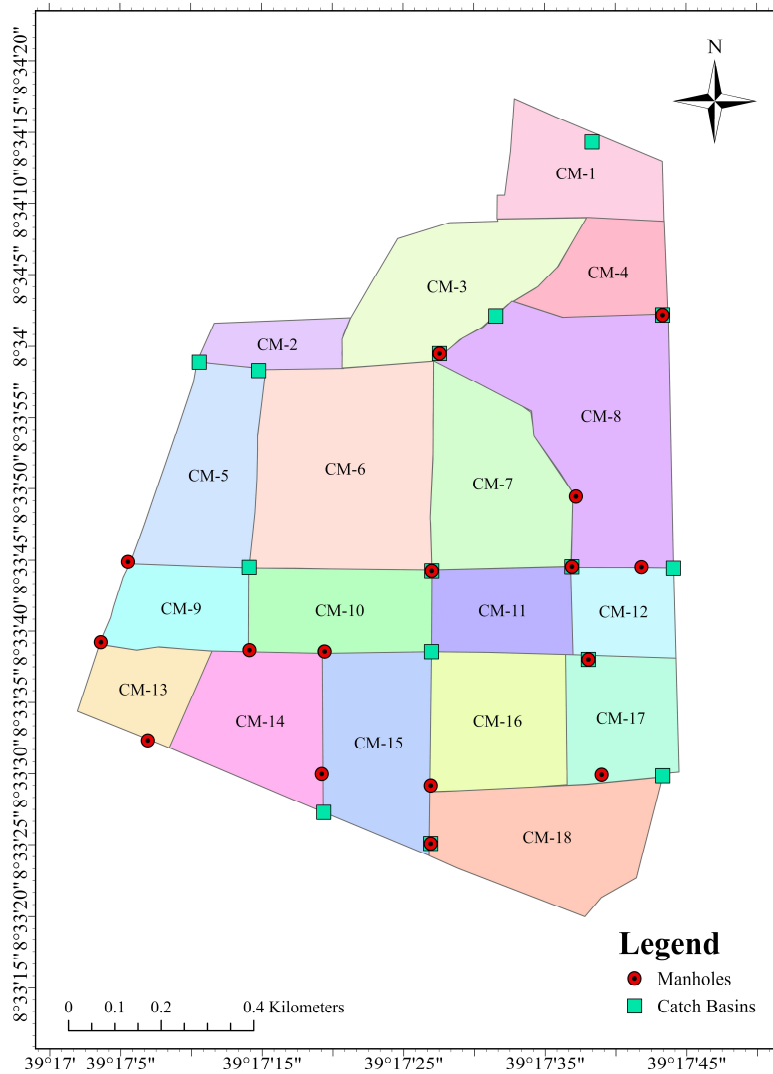


Figure 3.5: CBs and MHs of ASTU campus

D. Average Slope and hydraulic flow length

For each sub-catchment, the average slope and hydraulic flow length were used as an input to calculate the time of concentration. The area's contours were used to estimate the sub-catchment's hydraulic flow length, and a Google Earth application tool was used to calculate the average slope of the flow length. Table 3.7 shows the hydraulic flow length and its average slope for each sub-catchment of ASTU.

Table 3.3 Hydraulic flow length and average slope of sub catchments

Sub catchments	Scaled Area (ha)	Hydraulic Length (km)	Slope (%)
CM-1	6.457	0.496	2.00
CM-2	3.039	0.349	1.50
CM-3	10.022	0.641	3.20
CM-4	5.058	0.539	2.20
CM-5	8.661	0.453	2.00
CM-6	16.496	0.666	1.20
CM-7	10.442	0.462	1.40
CM-8	17.080	0.498	1.90
CM-9	5.255	0.664	2.30
CM-10	7.147	0.639	2.90
CM-11	5.528	0.399	1.40
CM-12	4.266	0.470	2.40
CM-13	4.194	0.540	1.40
CM-14	8.191	0.391	2.30
CM-15	9.083	0.460	1.90
CM-16	8.611	0.511	2.20
CM-17	6.420	0.533	4.30
CM-18	10.410	0.466	0.90

E. Land use/land cover (LULC)

A key component for hydraulic models is a LULC map. The land use information that is necessary for the simulations was obtained from satellite images (Figure 3.6) and the future master plan of the campus (Figure 3.7) that were later classified by the GIS (Geographic Information System) program. Then runoff coefficient was determined for each LULC as per stated in the ERA drainage design manual.

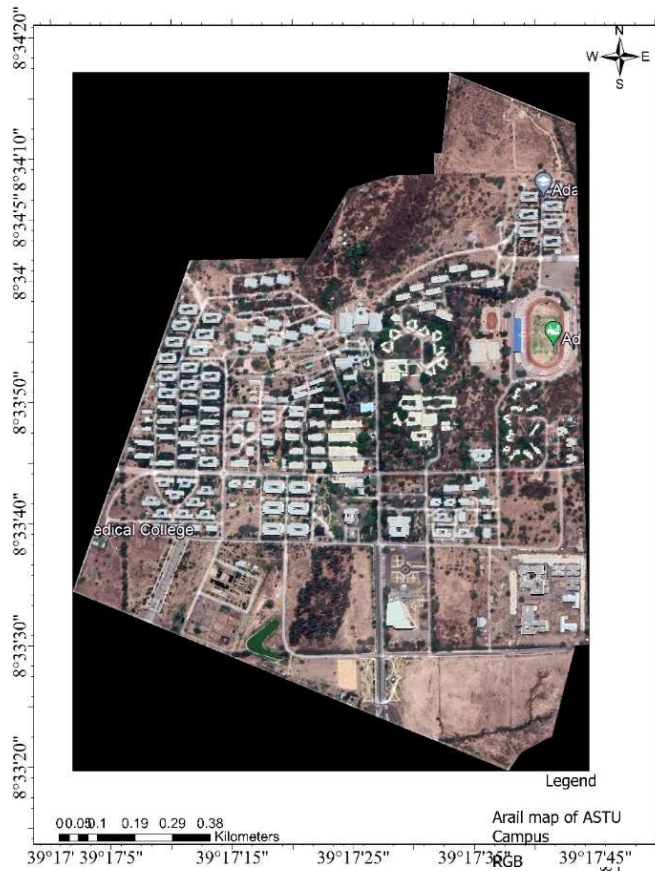


Figure 3.6: Current Aerial Map ASTU campus (2023)



Figure 3.7: Future Master Plan of ASTU Campus

F. Runoff Coefficient "C"

This variable represents the ratio of runoff to rainfall. It is the most difficult input variable to estimate. It represents the interaction of many complex factors, including the storage of water in surface depressions, infiltration, antecedent moisture, ground cover, ground slopes, and soil types. In reality, the coefficient may vary with respect to prior wetting and seasonal conditions. However, in this study the use of average values has been adopted to simplify the determination of this coefficient. (Table 3.4)

Table 3.4: Composite Runoff Coefficients of ASTU Campus

<i>Sub catchments</i>	<i>Area (ha)</i>	<i>Composite Runoff Coefficient</i>
CM 1	7.183	0.492
CM 2	3.039	0.656
CM 3	9.768	0.317
CM 4	5.641	0.488
CM 5	8.661	0.565
CM 6	16.496	0.749
CM 7	10.442	0.378
CM 8	16.539	0.470
CM 9	5.255	0.672
CM 10	7.147	0.728
CM 11	5.528	0.666
CM 12	4.266	0.706
CM 13	4.194	0.634
CM 14	8.191	0.192
CM 15	9.083	0.341
CM 16	8.611	0.714
CM 17	6.420	0.648
CM 18	10.410	0.562

G. Catchment and Sub-catchments map

The catchment and sub-catchment were generated in SewerGEMS using the catchment delineation tool based on the boundaries of the campus as defined by the master plan and digital elevation model (DEM). The campus boundaries served as the vertices of the catchment polygon used to define the catchment. The generation of sub-catchments, which offers a more

advantageous path to install the sewers that collect storm water and convey it to a common outlet, took into consideration the similarities in the existing stormwater drainage network, LULC, the arrangement of blocks and roadways. As a result, the campus catchment was delineated into 20 sub-catchments which are CM 1, CM 2, CM 3, CM 4, CM 5, CM 6, CM 7, CM 8, CM 9, CM 10, CM 11, CM 12, CM 13, CM 14, CM 15, CM 16, CM 17 and CM 18, with their area coverage depicted in Table 3.4. The sub-watersheds maps is shown in Figures 3.8.

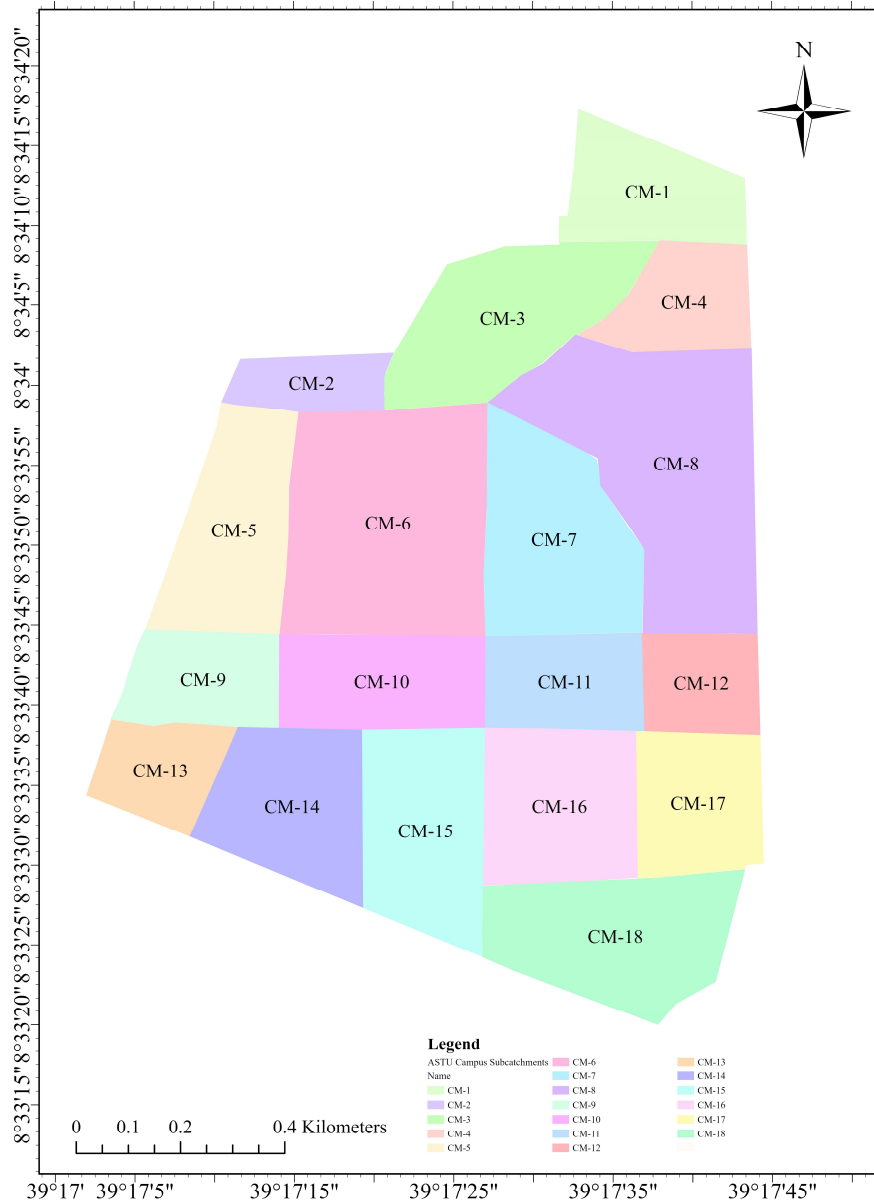


Figure 3.8: Sub catchments of ASTU

H. Existing drainage network of ASTU campus

As part of the ongoing development of the ASTU campus, the existing drainage network has been redesigned and is currently under construction. The new system has taken into account the primary, secondary, and tertiary roads of the campus and is designed to effectively manage stormwater runoff. The drainage network is comprised of rectangular, trapezoidal, and circular-shaped channels, which are planned to be constructed along both the left and right sides of the roads. Additionally, there are three outlets that have been strategically placed to direct the stormwater to downstream areas (Figure 3.9). The channels come in six different sizes, with a total length of 9.9km (Table 3.4). While the new system is expected to effectively drain the stormwater from the campus, it has also been identified that it poses a potential flood threat to the downstream areas. As a result, further stormwater management measures will need to be taken to ensure the safety of the surrounding areas.

Table 3.5 Types of existing drainage channel of ASTU campus

U-ditch Type	Shape	Bottom Width (m)	Depth (m)	m=H/V	Total (m)
Type-1	U-Shape	0.6	0.6		1,229.00
Type-2	U-Shape	0.6	0.8		2,721.00
Type-3	U-Shape	0.8	1.2		2,588.00
Type-4	U-Shape	1	1.2		560.00
Type-5	Trapezoidal	0.5	1	1.50	2,463.30
Type-6	U-Shape	1.2	1.3		339.00
Total					9,900.30

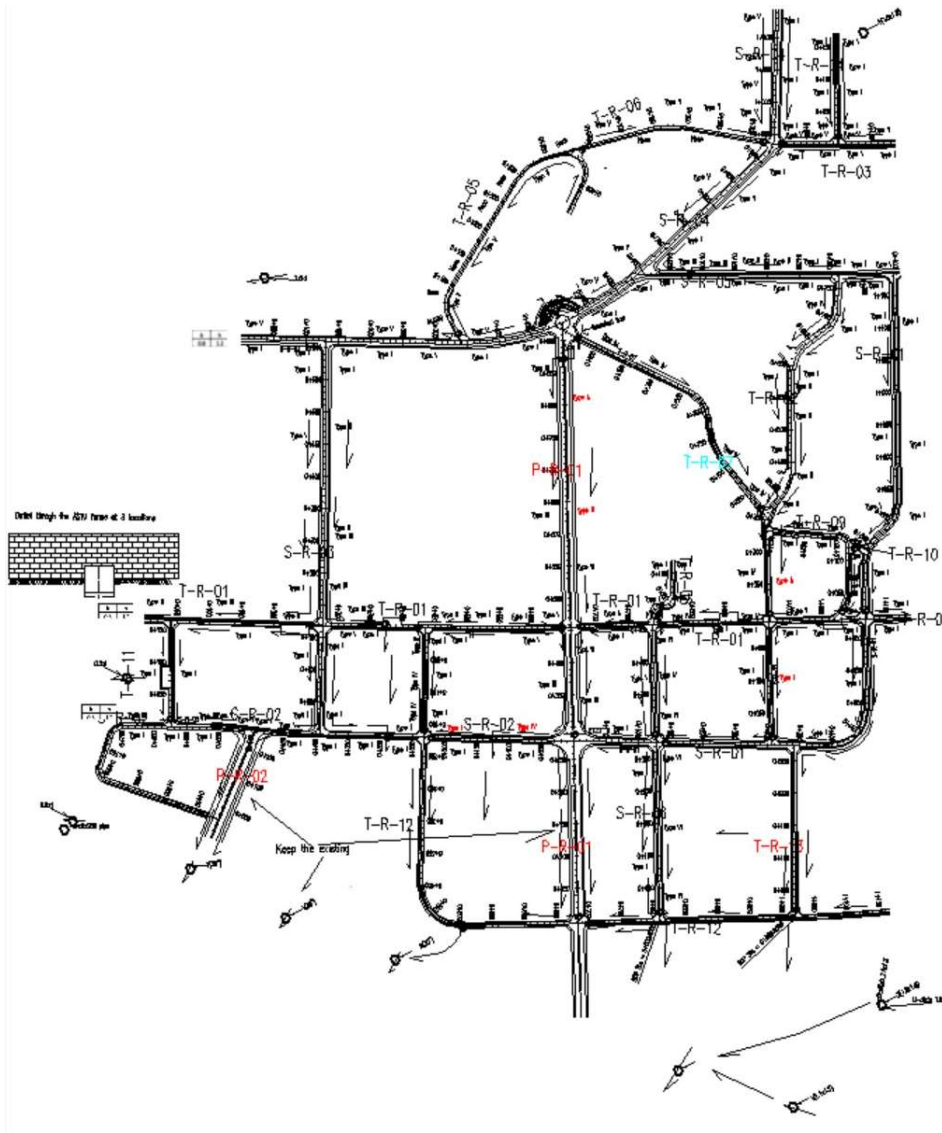


Figure 3.9: The existing drainage network of ASTU campus

3.3 Data Analysis Methods

3.3.1 Runoff determination

For stormwater modeling, the Bentley SewerGEMS modeling application employs two types of solvers. The first is hydrological solvers, which were used for runoff determination and also to characterize the catchment in terms of its properties.

The hydrologic component of the model is responsible for runoff generation and flow routing from the drainage sub-catchment to the receiving drainage system (culverts, ditches, stormwater mains). For this study, the Rational Method is used for runoff generation with the flow routing

component captured in the time of concentration calculations completed for each sub catchments. The runoff flow is then applied directly to the receiving pipes which also transport it to the designed detention ponds.

The following list describes the approach used to delineating the drainage sub-catchments in the study area as well as the runoff calculations completed for each of the sub-catchments.

A. Catchment delineation

Sub-catchments are hydrologic units of land whose topology and drainage system elements direct surface runoff to a single discharge point. The natural catchment area was delineated using ArcGIS Pro based on DEM and the Campus master plan CAD file as input, then the shape file was imported in to the SewerGEMS software and overlayed with the master plan map, the sub-catchments was then traced based on blocks polygon in the hydraulic model. Accordingly, since rational method works best for smaller areas, the ASTU campus catchment is subdivided into 18 sub catchments. These areas were used for computation of discharge using rational method. As shown in the Figure 3.8.

B. Runoff coefficient (C) assigning

In the Rational method of converting rainfall to runoff, the runoff coefficient is the most crucial factor. Table 2.1 was used to assign the C values for each LULC in their particular sub catchments using the weightage approach. Then, using Eq. 2.1, composite runoff coefficients were calculated for each sub catchment.

C. Time of concentration determination

The time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. In order to calculate the Time of Concentration (T_c) parameter, the sub-catchment slope, length and roughness coefficient need to be determined. The slope was estimated using the contour. The length was estimated by tracing the drainage flow path from the furthest point in the sub-catchment to the discharge point. The roughness coefficient was estimated based on pipes material type and assigning the value from the software catalog.

Many empirical equations are available for calculating time of concentration for a watershed. For this study overland time of concertation was calculated by the Kerby formula using Eq.

2.11 and time of concentration for equations channel flow was calculated by the US Soil Conservation Service using Eq. 2.12. Finally, a composite time of concentration was prepared using from the result of the above two equations.

3.3.2 IDF Curve formulation

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. For this study, Eq. 2.1 is used as suggested by ERA Drainage design manual, 2013. In addition, Eq. 2.8, Gumbel distribution Type I is used for IDF curve formulation.

3.3.3 Stormwater drainage network modeling

The second solver in the Bentley SewerGEMS modeling program for modeling stormwater is the hydraulic solver, which computes and models flows in pipes and channel networks. The physical network of the drainage system, which conveys the peak discharge to common outlet, was analyzed following the hydrological analysis. A drainage network system's main components include various-sized drainage pipes, catch basins, manholes, outfalls, and storage structures. Table 3.4 lists the different Bentley SewerGEMS modeling components that were utilized to represent these drainage components in the model.

Table 3.6: SewerGEMS Modeling Elements of ASTU Catchment

<i>Bentley SewerGEMS elements</i>	<i>Used to Represent</i>	<i>Symbol</i>	<i>Elements (by number)</i>
Junctions	Catch basin	CB	16
	Manhole	MH	20
	Transition	T	NA
	Cross sections for connecting	CS	NA
Layout	Pipes	CO	38
	Channel	CH	NA
	Gutter	GU	NA
Outfall	Discharge location such us to water bodies	O	2
Catchment	Catchments	CM	18
Pond	Pond	PO	2
	Pond Outlet Structure	POS	2

Each modelling element has a specific set of input data requirements, most of the data was available from the Master plan CAD files, DEM map, Google earth and slope map of the ASTU catchment which was the primary input for developing the drainage model. The existing drainage network, master plan CAD files, the Google earth maps were first projected to UTM World Geodetic System 84, Zone 37 and imported into Bentley SewerGEMS software as background and the model elements were developed. Field survey data were used as supplementary sources of data to develop the model. The following subsections provide further descriptions of the approach used and assumptions applied to each of the modelling elements.

A. Pipes

The primary purpose of pipes is conveying the stormwater to an out flow. The principal input parameters for pipes are invert start, invert stop elevations, length, and Manning's roughness coefficient (n) and material type. Approximately 10.2 km of pipes of different sizes were modelled. The pipes cross sections were assumed to be circular and the material type is concrete. The size of the pipes is computed by the SewerGEMS software from the software's catalog; the catalog has pipes diameters ranging from 200mm to 3800mm. Design constraints are used as an input to select the sizes of the pipes in every node. The model was computed for 2, 5, 10, 25, 50 and 100 return period peak discharges. To be economical, the pipes are sized based on the 25-year return period discharge.

B. Nodes

SewerGEMS model consists of a set of nodes that are combined to simulate the behavior of the entire system. Each node represents a hydrograph operation, such as generating runoff from a sub catchment or routing the hydrograph through a reach or pond. The individual nodes are interconnected by the routing diagram, which specifies how the hydrographs are routed from one node to another.

In this study the main nodes are catch basins, manholes, outfalls and outlet devices for a pond. The principal input parameters of these nodes are the ground elevation and invert elevation. For this study the elevation was set using TRex feature tool using contour shape file map of the campus. TRex is a feature that can be used to load elevation data into the model (nodes) from a variety of file types - including both vector and raster files. To use raster files as the data source, the ArcGIS platform must be used which is compatible in SewerGEMS. With a vector

data source, it is possible to use any platform. Vector data must consist of either point with an elevation or contours with an elevation.

Design constraints

In SewerGEMS, a set of design constraints were used to design pipe diameters, invert elevations, nodes, and inlets. To improve the design's economy and structural soundness, it is also possible to independently alter these values for each pipe or other components. The Default Design Constraints dialog is divided into the three following tabs:

- I. Gravity Pipe
- II. Node
- III. Inlet

I. Gravity Pipe Tab

The Gravity Pipe tab allows you to enter default constraints to be used for the design of pipes when performing a calculation run in design mode. In this section, there is a Velocity tab, a Cover tab, a Slope tab, the following default constraints can be specified to be used for the design of gravity pipes:

Velocity and slope tabs consists of the maximum and minimum allowable velocity and slope controls while Cover tab has cover along pipe length control which considers cover along the pipe length by checking against the active terrain model, maximum and minimum allowable cover controls.

II. Node Tab

This tab lets you specify the design constraints to be used by default for all gravity structures when performing calculations in design mode. During an automatic design, the program will adjust the elevations of the pipes adjacent to the structure according to the structure's matching constraints. The two choices for matching are Inverts and Crowns. Additionally, the downstream pipe can be offset from the upstream pipe(s) by a specified amount. This value is called the Matchline Offset. Optionally, the program supports the design of drop structures. In some situations, drop structures can minimize pipe cover depths while maintaining adequate hydraulic performance.

III. Inlet Tab

This tab lets you specify the design constraints to be used for all inlets when performing a calculation run in design mode. During an automatic design, the program will adjust the length of the inlet in order to meet the design constraints.

For an inlet in sag, the Default In Sag Design Constraints consist of maintaining the gutter spread and water depth under a given value. For an inlet on a grade, the Default on Grade Design Constraints consist of ensuring that at least a given percentage of the gutter flow is intercepted.

Default In Sag Design Constraints Section

This section lets you specify the design constraints to be used for all inlets located in sag when performing calculations in design mode. During an automatic design, the program will adjust the length of the inlet in order to meet both design constraints:

- Maximum Spread in Sag–The maximum allowed spread of water at the inlet, measured from the curb.
- Maximum Gutter Depth in Sag–The maximum depth of water allowed at the inlet.

The table below depicts the design constraints used in this study. The constraints are set based ERA the drainage design manual 2013 as summarized in Table 3.5.

Table 3.7: Design Constraints of the Study Area

Design constraint components	Minimum	Maximum
Velocity (m/s)	0.6	3
Cover (m)	0.8	2.5
Slope (%)	1	10
Spread in Sag (m)	N/A	4
Gutter Depth in Sag (m)	N/A	2

3.3.4 Existing drainage network assessment

The purpose of the assessment was to determine whether the current drainage system on campus was capable enough to transfer stormwater from all the sub-catchments. The assessment was carried out by using the Manning equation (Equation 3.1) to calculate the maximum discharge that a channel can transmit. The equation takes into account the planned shape and attributes of the channel.

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \dots \dots \dots \text{Eq 3.1}$$

Where: 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 dimensionless

R =hydraulic radius in m

$$R = \frac{A}{P} \dots \dots \dots \text{Eq 3.2}$$

Where: Q =flow discharge in m^3/s

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

Once the maximum discharge was calculated, it was compared to the peak runoff flow of each sub-catchment that the channel was expected to carry during a 25-year return period. This helped to determine if the channels were adequate or not.

The findings of the assessment were used to determine the adequacy of the channels. However, since the channels were not adequate enough to prevent further degradation of water quality, they were eventually redesigned into closed conduits and redirected to carry stormwater to the suggested ponds.

Overall, the assessment provided detailed information about the drainage system on campus, and the redesign of the channels has helped to improve the water quality.

3.3.5 Pre and Post development scenario modeling

The natural state of the land before to the establishment of the institution was referred to as pre-development. According to Ethiopia's LULC map from 2000 (Annex 2), the region where the institution was established is cropland with open woodland. Consequently, the pre-development

landcover was assumed to be cropland with open woodland. For pond routing purposes, it was assumed that the pre-development catchments would drain to the outlets at the left and right ends of the campus.

Post-development describes the future condition of the land when the institution fully developed, and it includes any ground that has been cleared of vegetation or that has been graded, grubbed, filled, bulldozed, or dug at any stage during the site preparation for the university. Buildings, roads, and other development result in the conversion of once-naturally pervious and vegetated land into impermeable land, which increases the amount of impervious surfaces in the drainage area. As a result, the response of an urbanized catchment to precipitation is significantly different from the response of a natural watershed. Post-developed peak runoff is expected to exceed pre-developed runoff from a similar storm event.

Two models were built: one that represents the catchment in its previous pre-developed condition and one that represents the future catchment after it is fully developed. Because this is an initial estimation of the discharges at the outlets of the catchment under its previous and future conditions, no channelized flows will be defined and only runoff as overland flow will be simulated.

The objective is to estimate the stormwater discharges at the catchment's outlet and compare them to the ones generated prior to urbanization. The approach is to compute the hydrologic response of the catchment to a series of synthetic design storms associated with different return periods. Most of the parameter values remains the same except the rational coefficients and the slope of the catchment.

3.3.6 Detention pond routing and sizing

The detention pond routing was conducted using Hydrology studio model. The most widely used method of determining the required storage volume in detention ponds is the Storage-Indication Method which is a level pool routing procedure for calculating the outflow hydrograph of a system with horizontal water surface, given its inflow hydrograph, and storage outflow characteristics and is the method used by Hydrology Studio. This routing procedure consists of an iterative process based upon the Continuity Equation.

The basic premise is that, over a given time interval, the volume of water entering the pond minus the volume of water leaving the pond equals the remaining storage volume. The process begins at time zero and progresses in steps equal to the time interval, until the entire time span of the inflow hydrograph has been reached.

The method begins with a stage-storage and a stage-discharge relationship (basically a pond created with an outflow device). The stage-storage relationship indicates how much storage or volume is in the pond at a given depth or stage. Similarly, the stage-discharge relationship tells us how much flow is exiting the pond at a given depth or stage. To this adding an inflow hydrograph is required. Hence, to perform a pond routing, three things; namely, Inflow hydrograph, stage versus storage relationship and stage versus discharge relationship, were conducted as described above:

I. Inflow and Target hydrographs

Inflow and Target hydrographs are a hydrograph that will be used for a pond routing. In this study the inflow hydrograph is a post-development hydrograph while the target hydrograph is the pre-development. The required storage volume of the pond was determined from the difference of the two hydrographs.

II. Stage vs. Storage relationship

After determining the physical detention pond volume, it is required to know how much volume it can hold at any given depth or elevation, or stage. Stage is the absolute distance from the water surface to the pond bottom. At Stage-Storage relationship stage the depth of the pond was determined.

III. Stage versus Discharge Relationship

After determining the depth of the pond in the Stage versus Storage relationship phase, the next step is to determine how much water will in-turn exit the pond due to the determined depth. In this phase the out let type, structure and dimension of the pond was modeled and designed.

So, once the relationship between the inflow volume from the runoff hydrograph and the flow out of the pond is known, monitoring this across each individual time step was done. Finally, the final outflow hydrograph was produced. The volume in the pond at any point along the way was also determined. This is what the Storage-Indication calculation method provides.

On the final hydrograph, it was shown that the maximum volume stored in the pond was the difference between the inflow hydrograph and the outflow hydrograph from the beginning up to the peak of the outflow hydrograph.

4 Results and Discussion

4.1 Formulation of IDF curve for ASTU

The IDF curve for ASTU Campus was created using the Gumbel Type I distribution method and is based on 30-year rainfall data from the Ethiopian Meteorological Agency for short periods of 0.5, 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5, 5, 6, 12, and 24 h with 2, 5, 10, 25, 50, and 100 years return periods. The results showed that rainfall intensity reduced as the duration of the storm increased, and if the return period of the rainfall was large, rainfall of any specific duration showed a higher intensity which confirmed the study by Shamkhi et al (2022). The maximum rainfall intensity of 246.76mm/hr was happened at the duration 0.08hr of the return period 100yr, whereas the minimum rainfall intensity of 1.72mm/hr was happened at the duration 24hr of the return period 2yr. As reported Twagirayezu et al., (2019) the obtained results of research, showed that the intensity of rainfall increased with the increment in return periods, but decreased with the increment in duration. The result is displayed in Table 4.1, and the curve is illustrated in Figure 4.1.

Table 4.1 IDF table of ASTU campus

Time (hours)	Return Period (years)				
	2	5	25	50	100
Rainfall I (mm/hr)					
0.08	75.04	151.45	189.91	218.44	246.76
0.17	47.27	95.41	119.64	137.61	155.45
0.25	36.07	72.81	91.30	105.02	118.63
0.50	22.73	45.87	57.51	66.16	74.73
1.00	14.32	28.89	36.23	41.68	47.08
2.00	9.02	18.20	22.82	26.25	29.66
12.00	2.73	5.51	6.91	7.95	8.98
24.00	1.72	3.47	4.35	5.01	5.66

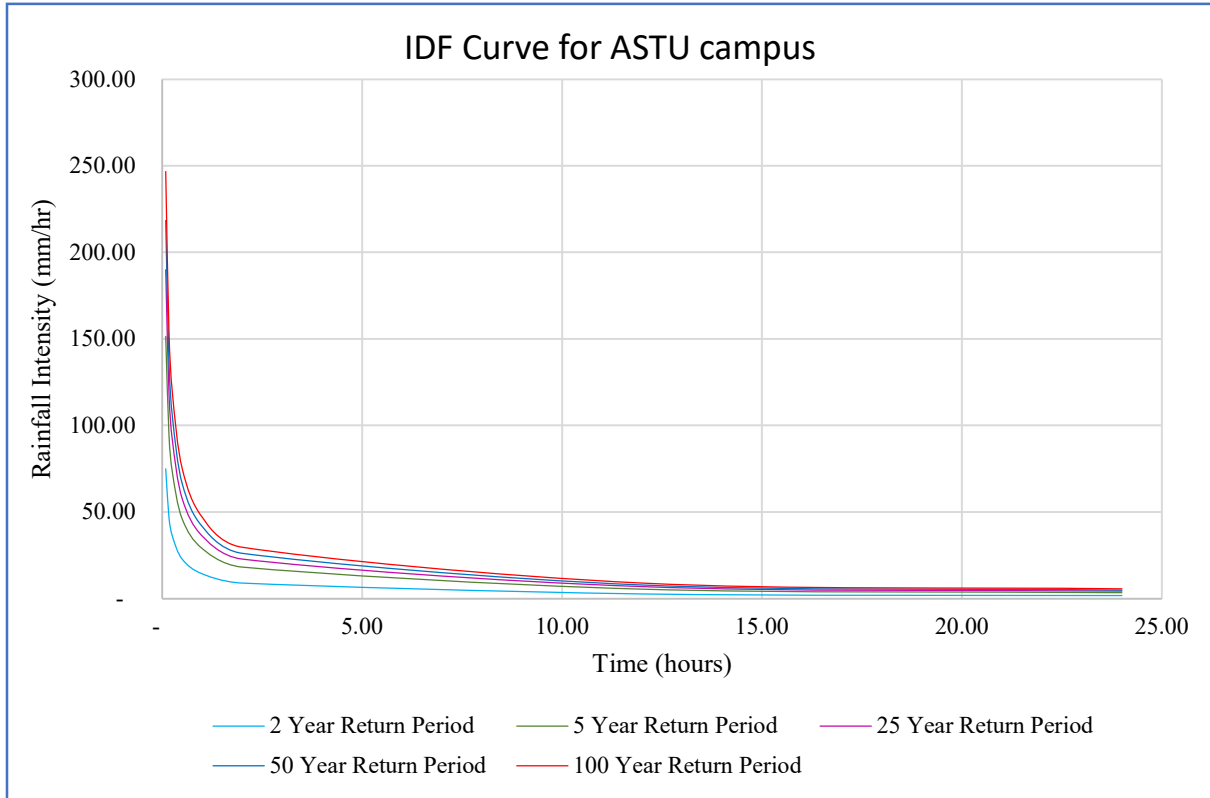


Figure 4.1: IDF Curve of ASTU Campus

4.2 Runoff Determination of the study area

Rational method to determine the runoff generated in the study area, Since the rational method performs best in smaller areas, the total catchment was divided into 20 sub-catchments (CM 1 to CM 20) by taking into account the catchment contour lines, which will aid in the development of the sewer layout, and similarity of land cover, which aids in the determination of runoff coefficients. In accordance with this, as shown in Table 4.2, of all the sub-catchments, CM 3 had the largest area (11.9 ha), which was mostly covered by open wood land and is also the highest point of the campus. CM 2 had the smallest area (4.17 ha), which was primarily made up of Men's Freshman Dormitory buildings.

4.2.1 Runoff coefficient (C) determination

The percentage of precipitation that emerges as runoff is described by the runoff coefficient. After splitting the catchment into sub catchments, runoff coefficients for each LULC were determined, and composite runoff coefficients were calculated for each sub catchment. As a

result, the results indicated that the CM-12 sub-catchment had the highest runoff coefficient (0.749), meaning that this area has low infiltration and high runoff, and that CM-14 has the lowest runoff coefficient (0.192), meaning that this area has high infiltration and low runoff. (Table 4.2)

Table 4.2: Composite Runoff coefficients of ASTU sub catchments

<i>Sub catchments</i>	<i>Area (ha)</i>	<i>Composite Runoff Coefficient</i>
CM 1	6.457	0.492
CM 2	3.039	0.656
CM 3	10.022	0.317
CM 4	5.058	0.488
CM 5	8.661	0.565
CM 6	16.496	0.749
CM 7	10.442	0.378
CM 8	17.080	0.470
CM 9	5.255	0.672
CM 10	7.147	0.728
CM 11	5.528	0.666
CM 12	4.266	0.706
CM 13	4.194	0.634
CM 14	8.191	0.192
CM 15	9.083	0.341
CM 16	8.611	0.714
CM 17	6.420	0.648
CM 18	10.410	0.562

4.2.2 Time of concentration

Time of concentration (T_c) is the time required for an entire watershed to contribute to runoff at the point of interest for hydraulic design. As recommended by ERA drainage design manual the overland time of concertation was calculated by the Kerby formula and channel flow time of concentration was calculated by the US Soil Conservation Service. Since, Time of concentration will vary depending upon slope and character of the watershed and the flow path; the slope flow Length is determined and composite time of concentration for each sub

catchments was calculated. The results of the study demonstrated that the time of concentration ranges from 0.87 to 0.37 hours, with the sub-catchment CM-6 having the longest time of concentration (0.87 hours), indicating that the peak discharge for that sub-catchment occurred at 0.87 hours of rainfall intensity; and the CM 18 sub-catchment has the shortest time of concentration (0.37 hours), meaning that its peak discharge occurs at 0.37 hours of rainfall intensity (Table 4.3).

Table 4.3: composite time of concentration of ASTU campus sub catchments

Label	Outflow Element	Slope	Catchment Flow Time (min)	Time of Concentration (Composite) (hour)
CM-1	CB-2	0.020	27.000	0.450
CM-2	CB-16	0.025	52.200	0.810
CM-3	CB-4	0.032	32.400	0.540
CM-4	MH-1-1	0.032	48.600	0.810
CM-5	MH-10	0.020	51.600	0.860
CM-6	CB-15	0.012	48.000	0.870
CM-7	MH-3	0.014	45.600	0.760
CM-8	CB-6	0.019	40.200	0.670
CM-9	MH-11	0.023	36.000	0.600
CM-10	MH-9	0.029	49.800	0.830
CM-11	CB-11	0.014	42.000	0.700
CM-12	CB-7	0.024	34.800	0.580
CM-13	PO-1	0.014	27.000	0.450
CM-14	MH-12	0.023	44.400	0.740
CM-15	CB-18	0.019	27.600	0.460
CM-16	MH-6	0.022	40.200	0.670
CM-17	MH-3	0.043	47.400	0.790
CM-18	PO-2	0.029	22.200	0.370

4.2.3 Peak rate of Runoff

Drainage systems should be designed to convey based on a predetermined peak runoff magnitude so as to avoid significant flood hazards. After the simulation of SewerGEMS, peak rate of runoff for different return periods is obtained for each sub catchment. Accordingly, sub-catchment CM-6 which contributed 13.69% of the total peak runoff of the entire catchment, was found to have the highest peak runoff flow, as shown in Table 4.4, with values of (0.6,

1.22, 1.54, 1.77 and 1.99) m³/s for the return periods of 2, 10, 25, 50, and 100 years, respectively. The sub-catchment CM-14, however, had the lowest peak runoff flow, contributing an average of only 1.85% of the total peak runoff of the entire catchment.

Table 4.4: Peak Runoff flow (m³/s) of sub catchments for different return periods

<i>Sub Catchments</i>	<i>Return Period (year)</i>				
	<i>2</i>	<i>10</i>	<i>25</i>	<i>50</i>	<i>100</i>
CM-1	0.22	0.45	0.57	0.65	0.74
CM-2	0.09	0.18	0.23	0.27	0.30
CM-3	0.19	0.39	0.49	0.57	0.64
CM-4	0.12	0.24	0.30	0.35	0.39
CM-5	0.23	0.46	0.57	0.66	0.74
CM-6	0.61	1.22	1.54	1.77	1.99
CM-7	0.20	0.41	0.51	0.58	0.66
CM-8	0.44	0.89	1.12	1.29	1.46
CM-9	0.21	0.42	0.52	0.60	0.68
CM-10	0.25	0.50	0.63	0.72	0.82
CM-11	0.20	0.40	0.50	0.58	0.65
CM-12	0.18	0.36	0.45	0.52	0.59
CM-13	0.19	0.38	0.47	0.55	0.62
CM-14	0.08	0.16	0.21	0.24	0.27
CM-15	0.21	0.43	0.54	0.62	0.70
CM-16	0.34	0.68	0.86	0.99	1.12
CM-17	0.21	0.42	0.52	0.60	0.68
CM-18	0.48	0.97	1.22	1.40	1.59

The flow of all contributing sub-catchments was added to produce the net peak runoff flow at the catchment level (the campus). According to Figure 4.2, the campus' total flow was 4.45 m³/s during a 2-year return time and 14.64 m³/s at a 100-year return period. As per ERA manual recommendations, the 25-year peak runoff flow result was used as an input for pond sizing and routing.

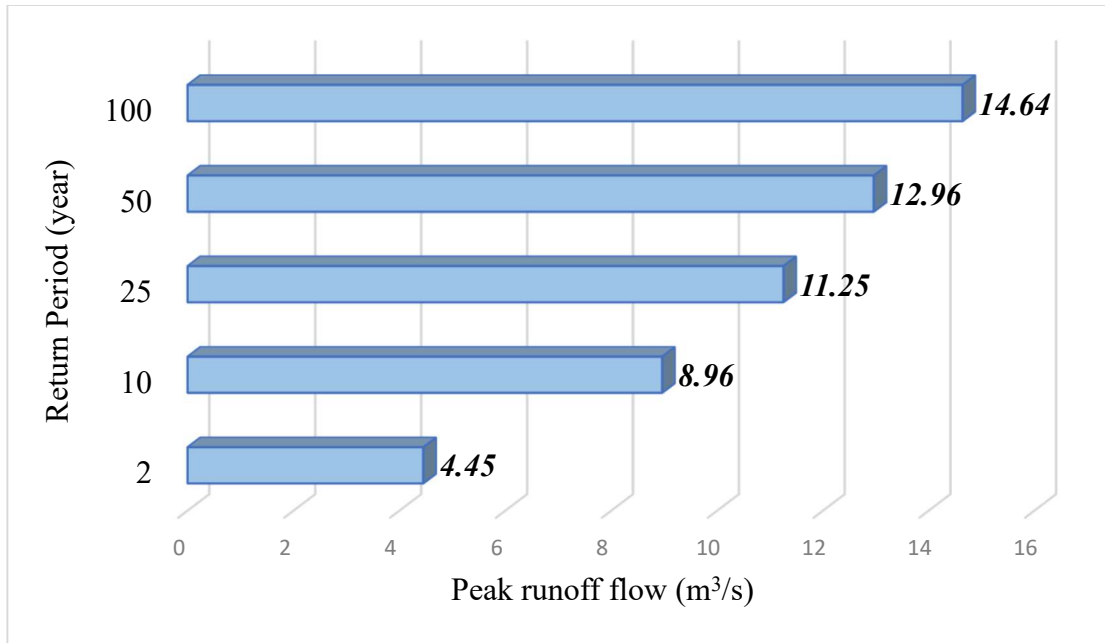


Figure 4.2: The total peak runoff flow of ASTU campus

4.3 Assessment of existing drainage system

The existing stormwater drainage channels were evaluated based on the geometry, slope, and dimensions of the channels provided in the design document by ASTU's construction project office. To calculate the velocity and discharge of the channels, these parameters were taken into consideration. The velocity was then compared with the design criteria specified in the ERA drainage manual. The results indicated that the velocity in channels type 1, 2, 3, and 4 and conduit 30-inch diameter in sub-catchment CM-1, CM-2, CM-3, and CM-11 exceeded the design criteria.

The next step involved comparing the discharge capacity of each channel with the modelled 25-year return period peak runoff. The results revealed that channels type 1, 2, 4, and 6 and conduit 42-inch diameter in sub-catchment CM-1, CM-7, and CM-15 were over-designed. This means that their discharge capacity is higher than the peak runoff during a 25-year return period.

On the other hand, channels type 1 and conduit 30-inch in sub-catchment CM-13 and CM-16 were found to be under-designed. This means that their discharge capacity is lower than the peak runoff during a 25-year return period. Therefore, it is necessary to make the required

modifications to these channels to ensure that they meet the design criteria and can handle the expected stormwater runoff.

Table 4.5 Existing drainage channel capacity assessment result

Sub Catchments	Existing Drainage Canal Type	Designed Velocity (m/s)	Calculated Discharge (m ³ /s)	25-Return Period Modeled Discharge (m ³ /s)
CM-1	Type 1 and 4	4.67	2.73	0.47
CM-2	Type 4	3.81	1.23	0.72
CM-3	Type 1 and 4	2.85	0.86	0.96
CM-4	Type 1	2.90	0.76	0.3
CM-5	Type 3 and conduit 30'	3.76	3.48	2.24
CM-6	Type 1, 3 and 5	2.54	3.36	2.8
CM-7	Type 2 and 4	3.22	5.39	1.67
CM-8	Type 1, 2 and 4	1.81	2.12	1.69
CM-9	Type 1 and 3	2.43	3.39	2.87
CM-10	Type 1, 2, 3 and 4	2.88	3.23	0.63
CM-11	Type 1, 3 and 4	3.67	2.52	3.44
CM-12	Type 1	2.09	1.23	1.37
CM-13	Type 1 and Conduit 30'	2.99	3.32	4.41
CM-14	Type 1	2.07	0.87	0.65
CM-15	Type 1 , 6 and conduit 42"dia	2.43	2.25	0.44
CM-16	Type 1	2.61	2.47	4.72
CM-17	Type 1	2.04	0.96	0.87
CM 18	None			5.94

4.4 Drainage network modeling

Hydraulic design is about ensuring that drainage structures are selected to avoid flooding, that there is sufficient space for the water to flow at times of flood. It is also about avoiding potential problems such as erosion and siltation. The hydraulic analysis of a pipes determines the depth and velocity at which a given discharge will flow in a pipes of known geometry, roughness, and slope.

After determining the sub catchments and their peak runoff flow, nodes (catch basins and manholes) were assigned to each sub catchments to connect the drainage pipes that convey the

stormwater to an outfall, in this case to the ponds. The ground elevations of each node were assigned using TRex tool using the contour map of the study area. The hydraulic analysis was conducted using automated design alternative option of the software. The automated design alternative can automatically size pipes, set node invert elevations and determines the size of inlets and pipes to pass a storm design while meeting the specified constraints. To ensure there is sufficient space for the stormwater to avoid flooding, the pipes are sized based the flow at 25-year return period.

In order to convey storm water to the proposed ponds 1 and 2, two main stormwater drainage networks have been created based on the contour lines of the study area. The overall peak runoff flow of the first drainage network, which conveys stormwater from sub-catchment CM-2 CM-5 CM-6 CM-9 CM-10 CM-13 CM-14 and CM-15 to the first pond (PO 1), was determined to be 4.71 m³/s. While the second drainage network conveys water to second pond (PO 2) with a total peak runoff flow of 6.54 m³/s from sub-catchments CM-, CM-3, CM-4, CM-7, CM-8, CM-11, CM-12, CM-16, CM-17 and CM-18.

Using SewerGEMS modeling software, drainage network modeling was done for both networks. According to the findings, the first drainage network's total length of pipes was 4.08km, while the second drainage network's total length was 6.21km. All of the pipes are circular concrete types, and the pipe sizes required for the first drainage networks to reliably transport stormwater to PO1 ranged from 300 to 1200 mm in diameter and for second drainage network it ranged from 150 to 900mm in diameter. As it should be, the smaller diameter was at the network's entrance and the larger was at its exit because the smaller diameter carries stormwater from the adjacent sub-catchment while the larger carries the combined flow from all the pipes. (Figure 4.3 and 4.4).

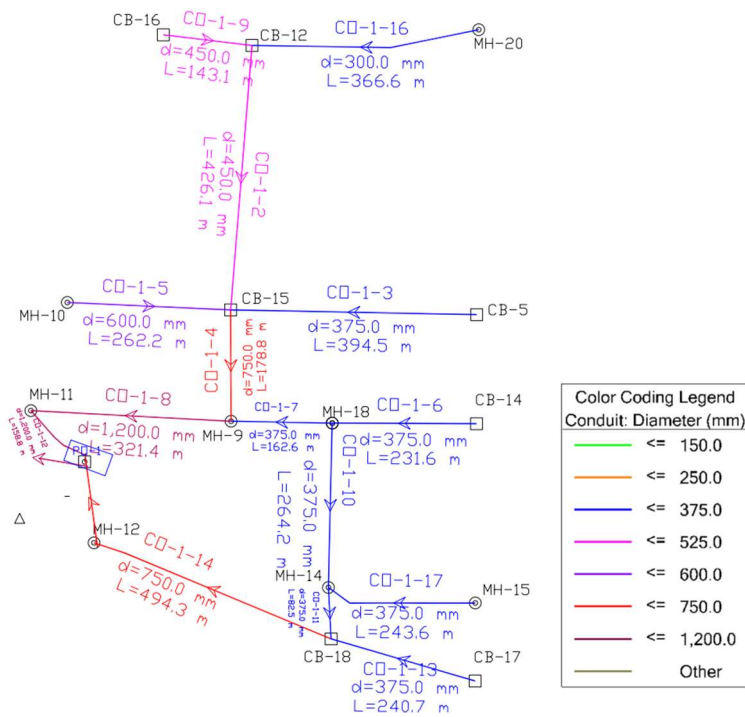


Figure 4.3: The first Drainage Network to PO1

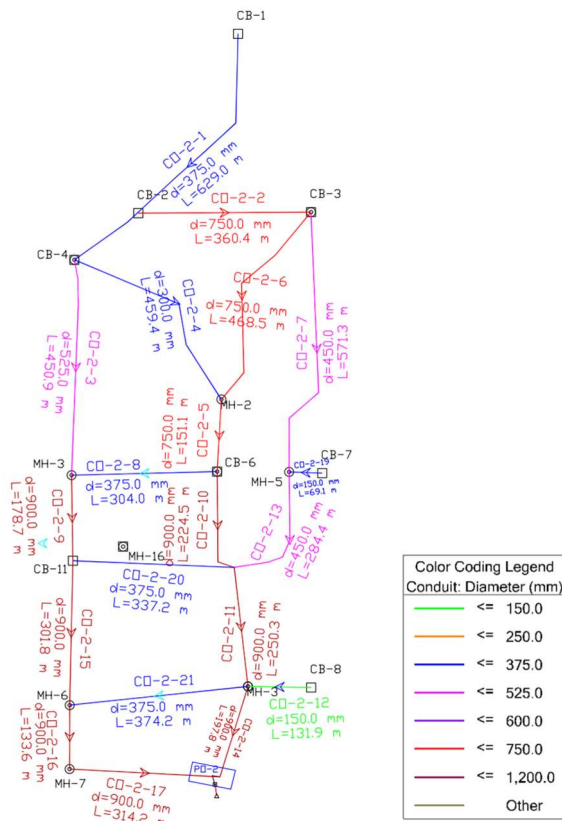


Figure 4.4: The Second Drainage Network to PO2

In the first network, the velocities ranges from 0.7 to 3m/s in pipes CO-1-1 and CO-1-15, respectively. Additionally, the slope ranges between 0.01 and 0.09 m/m in pipes CO-1-1 and CO-1-12, respectively, while the cover varies between 0.87and 2.49 m in pipes CO-1-1 and CO-1-16. (Table 4.5). Similarly, the velocities in the second drainage network vary from 2.99 to 0.6 m/s in pipes CO-2-18 and CO-2-15, respectively. Furthermore, the slope ranges from 0.01 to 0.092m/m in pipes CO-2-18 and CO-2-14, while the cover ranges from 0.5 to 2.67m in pipes CO-2-4 and CO-2-6, respectively. (Table 4.6). which conforms all the design criteria were meet.

Table 4.6: Properties of the first drainage network

Label	Start Node	End Node	Length (m)	Diameter (mm)	Slope (m/m)	Velocity (m/s)	Cover (m)
CO-1-1	POS-1	O- F1	130.7	900	0.01	0.7	2.43
CO-1-2	CB-12	CB-15	426.1	450	0.037	2.63	1.81
CO-1-3	CB-5	CB-15	394.5	375	0.011	1.75	1.85
CO-1-4	CB-15	MH-9	178.8	750	0.025	2.96	2.22
CO-1-5	CB-15	MH-10	262.2	600	0.01	2.68	1.85
CO-1-6	CB-14	MH-13	231.6	375	0.01	1.68	0.98
CO-1-7	MH-18	MH-9	162.6	375	0.01	1.64	0.87
CO-1-8	MH-9	MH-11	321.4	900	0.01	2.83	2
CO-1-9	CB-12	CB-16	143.1	450	0.01	2.19	2.46
CO-1-10	MH-13	MH-14	264.2	375	0.014	1.9	1.65
CO-1-11	MH-14	CB-18	82.5	375	0.024	2.69	1.55
CO-1-12	MH-11	O-1	101.3	900	0.093	2.74	2.15
CO-1-13	CB-17	CB-18	240.7	375	0.01	1.68	0.88
CO-1-14	CB-18	MH-12	410.3	750	0.01	2.78	2.08
CO-1-15	MH-12	O-1	225.7	750	0.017	3	2.03
CO-1-16	MH-20	CB-12	366.6	250	0.03	2.37	2.49
CO-1-17	MH-15	MH-14	243.6	375	0.01	1.64	1.17

Table 4.7: Properties of the Second drainage network

Label	Start Node	End Node	Length (m)	Diameter (mm)	Slope (m/m)	Velocity (m/s)	Cover (m)
CO-2-1	CB-1	CB-4	629	375	0.019	2.45	0.5
CO-2-2	CB-2	CB-3	360.4	750	0.01	2.63	2.67
CO-2-3	CB-4	MH-3	450.9	525	0.039	2.38	0.68
CO-2-4	MH-19	MH-2	459.4	300	0.017	1.99	0.5
CO-2-5	MH-2	CB-6	151.1	750	0.026	2.81	0.5
CO-2-6	CB-3	MH-2	468.5	750	0.012	2.82	2.67
CO-2-7	MH-5	MH-1-1	571.3	450	0.02	2.81	1.5
CO-2-8	MH-4	MH-3	304	375	0.01	1.64	0.68
CO-2-9	MH-3	CB-11	178.7	900	0.012	2.23	0.76
CO-2-10	CB-6	CB-7	224.5	900	0.019	2.27	0.5
CO-2-11	CB-7	MH-3	250.3	900	0.035	2.64	0.5
CO-2-12	CB-8	MH-3	131.9	150	0.048	1.65	0.5
CO-2-13	CB-7	MH-5	284.4	450	0.042	2.69	1.5
CO-2-14	MH-3	O-2	197.8	900	0.092	2.5	2.5
CO-2-15	CB-11	MH-6	301.8	900	0.03	2.99	1.66
CO-2-16	MH-6	MH-7	133.6	900	0.027	2.28	1.58
CO-2-17	MH-7	O-2	314.2	900	0.036	2.94	2.5
CO-2-18	POS-2	O-F2	23.5	900	0.01	0.6	2.12
CO-2-19	CB-7	MH-5	69.1	150	0.012	1	1.5
CO-2-20	MH-16	CB-11	337.2	375	0.01	1.64	0.58
CO-2-21	MH-17	MH-6	374.2	375	0.01	1.64	1.58

Comprehensive modeling was used to securely convey the 25-year return period peak discharge that emanated from the whole catchment of the ASTU campus to the proposed ponds. The model's results showed that 8 various sizes of concrete circular pipes with diameters ranging from 150mm to 1200mm and covering more than 10.27 km in length are sufficient to carry the aforementioned discharge. (Table 4-7). To provide a better understanding of the drainage network and pond locations, Figure 4.6 offers a clear depiction of these elements superimposed on the master plan of the ASTU Campus.

Table 4.8: Pipes properties of ASTU's campus catchment

S. No	Pipes Description	Count	Length (All Materials) (m)
1	Circle - 150.0 mm	2	201
2	Circle - 300.0 mm	2	851.7
3	Circle - 375.0 mm	11	3,241.80
4	Circle - 450.0 mm	2	855.7
5	Circle - 525.0 mm	3	1,021.90
6	Circle - 750.0 mm	6	1,749.50
7	Circle - 900.0 mm	9	1,777.20
8	Circle - 1,200.0 mm	3	580.3
Total Length		38	10,279.20



Figure 4.5: Drainage Network Layout of ASTU Campus

4.5 Comparison of Pre- and Post-Development Peak Runoff Flow

The SewerGEMS model was further employed to simulate impact of pre and post development change on the stormwater peak flow. The ASTU campus catchment was analyzed using the pre-development land cover of 2000, which is cropland with open woodland, and the post-development which is future land cover after the campus fully developed. With the exception of the runoff coefficients, slope, and hydraulic flow length, which are directly related to land cover and influence the time of concentration of the sub-catchments, all the factors used to calculate the peak runoff flow stay the same.

Peak runoff flow was predicted for 2, 10, 25, 50, and 100-year return periods after the parameters were adjusted. Bajracharya, et al., (2016) has stated that, At the post development context, the total runoff is estimated to be almost 40 million m³ per annum. When comparing this figure with that of the base-scenario (pre-development), it is found to be 2.7 times higher. The difference is found to be 25 million m³ per annum, and this is the excess annual run-off, due to urbanization in line with, this research report, the peak runoff flow of ASTU campus for each return period increased by 47%, indicating that the establishment of the ASTU campus increased the runoff by almost half, becoming the root flood problem, particularly for the downstream adjacent communities (Figure 4.6). The pre and post development result each sub catchments are shown in Appendix 4.

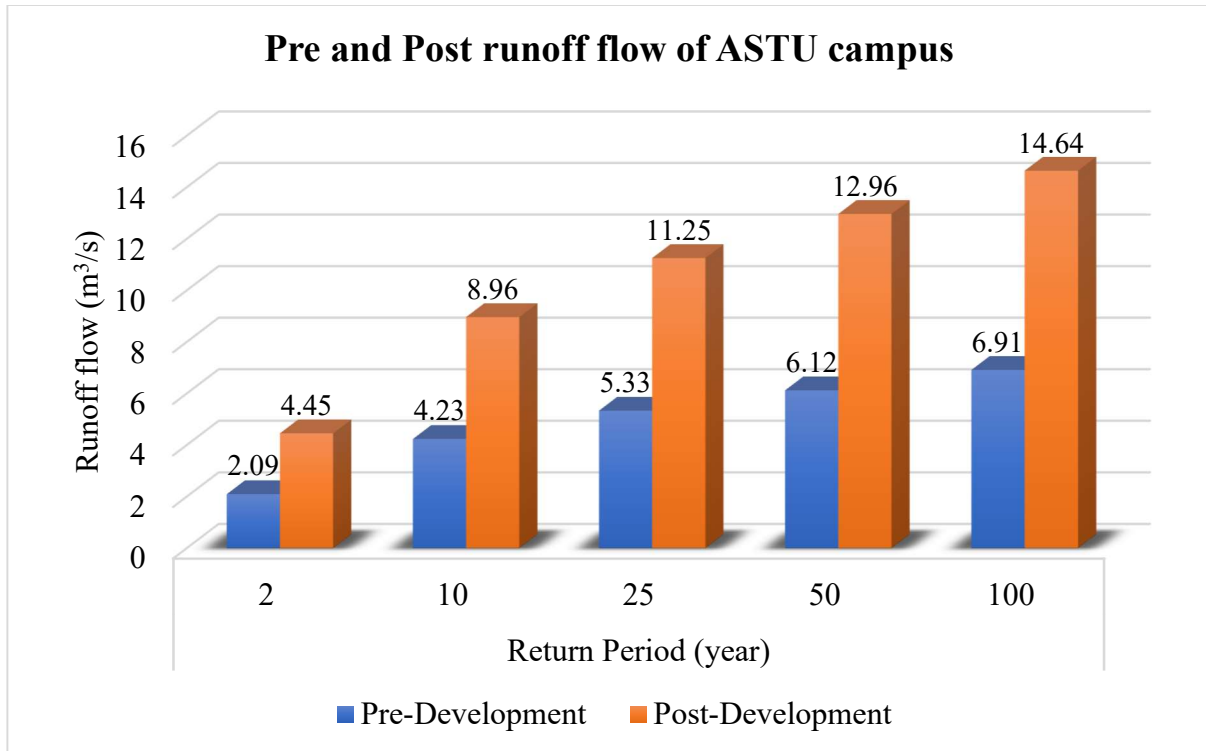


Figure 4.6: Pre and Post Development Peak flow of ASTU catchment

4.6 Detention pond sizing and routing

4.6.1 Formulation of pre and post hydrographs

Hydrology studio modeling software was used to simulate the detention pond routing. Prior to doing the ponds routing analysis, the pre and post development hydrographs were created using the SewerGEMS model's 25-year return period peak runoff flow rate results for the first drainage network which conveys the stormwater emanated from sub-catchments CM-2, CM-5, CM-6, CM-9, CM-10, CM-13, CM-14 and CM-15 to PO-1 located at $X=531332.42\text{m}$, $Y=946187.15\text{m}$ 37N UTM coordinates and second drainage networks which convey stormwater from sub-catchments CM-1, CM-3, CM-4, CM-7, CM-8, CM-11, CM-12, CM-16, CM-17 and CM-18 to PO-2 located at $X=532303.20\text{m}$ and $Y=945814.48$ 37N UTM coordinates. These hydrographs were then used as input for the ponds routing analysis. The post-development hydrograph was regarded the inflow, whereas the pre-development hydrograph was considered the target hydrograph, and the difference between the two hydrographs gave the ponds' storage volume (Figure 4.7 and 4.8).

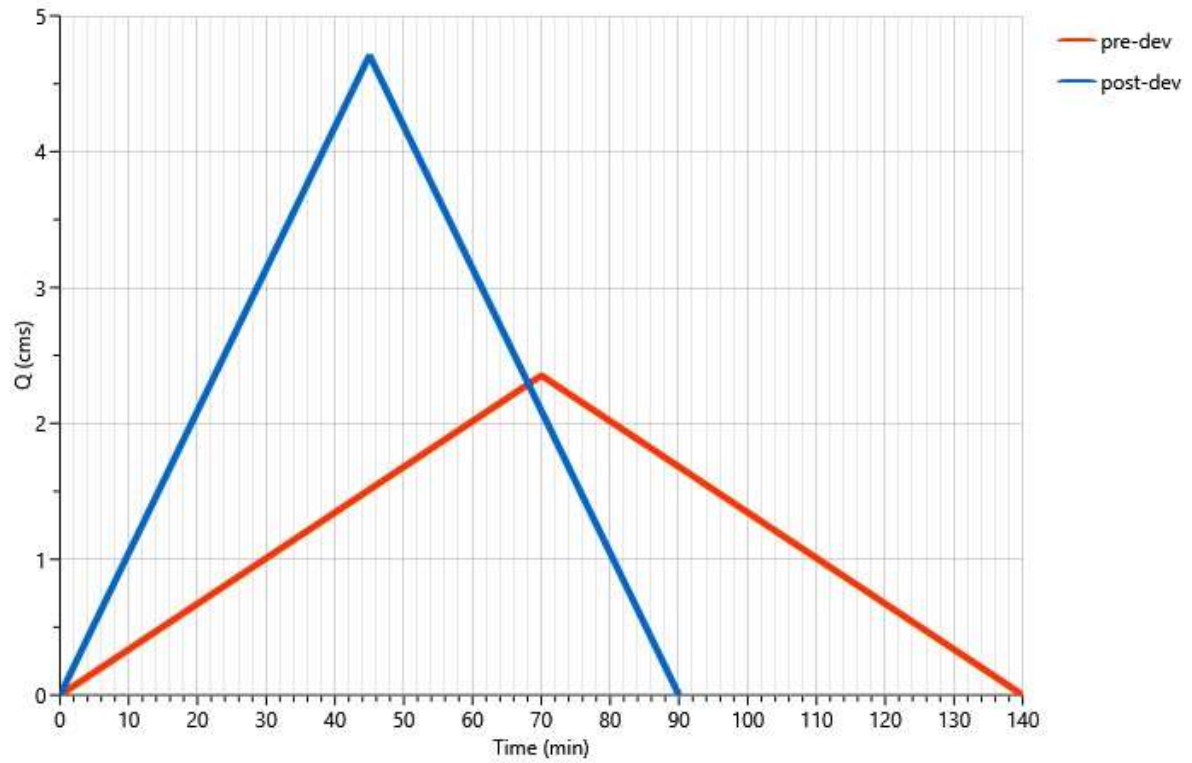


Figure 4.7: Pre and Post Hydrograph of 1st Drainage Network

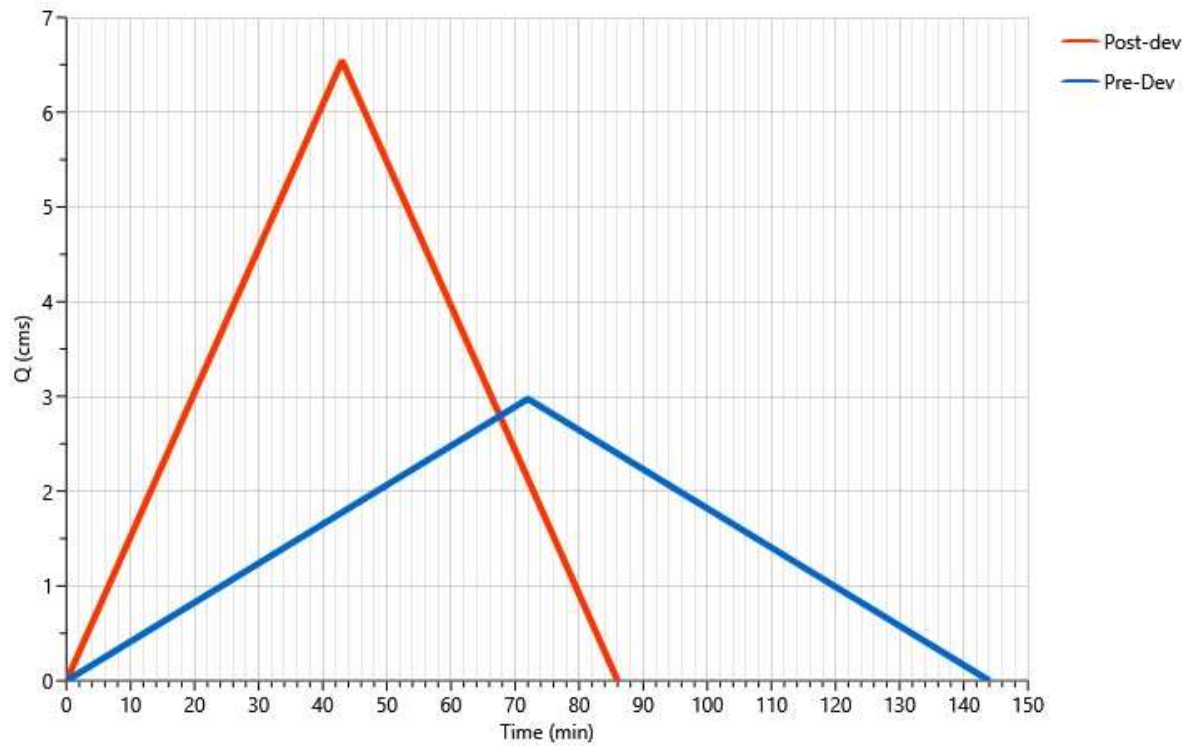


Figure 4.8: Pre and Post Hydrograph of 2nd Drainage Network

As depicted in Figure 4.9, The pre and post development runoff flow rate study results revealed a 50% increase in the first and second drainage networks and 45% in the second drainage network. These are the flow rates used for pond routing and flow attenuation.

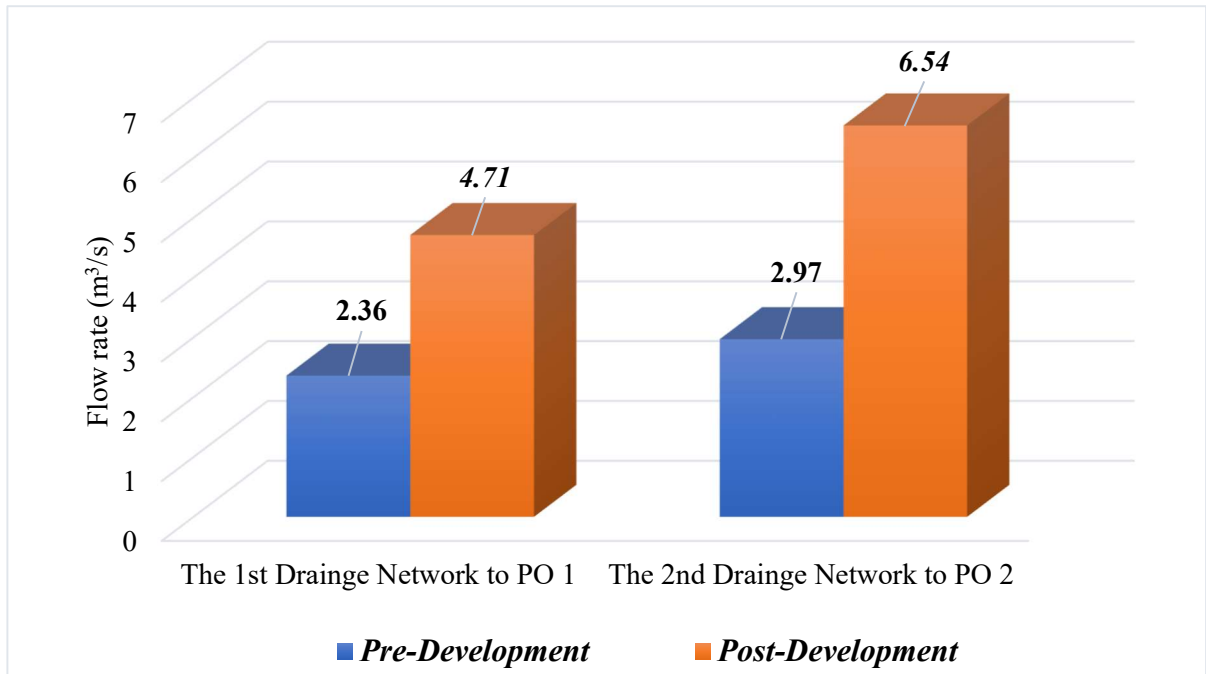


Figure 4.9: Pre and Post flow rates of 100-year return period

4.6.2 Storage determination using hydrographs

To determine the storage capacity of PO-1 and PO-2, the difference between the pre (target) and post (inflow) hydrographs of sub-catchments contributing to PO-1 and PO-2 was calculated, and the result showed that the estimated storage, which is shaded in Figures 4.10 and 4.11, was found to be $6,364 \text{ m}^3$ and $9,206 \text{ m}^3$ respectively.

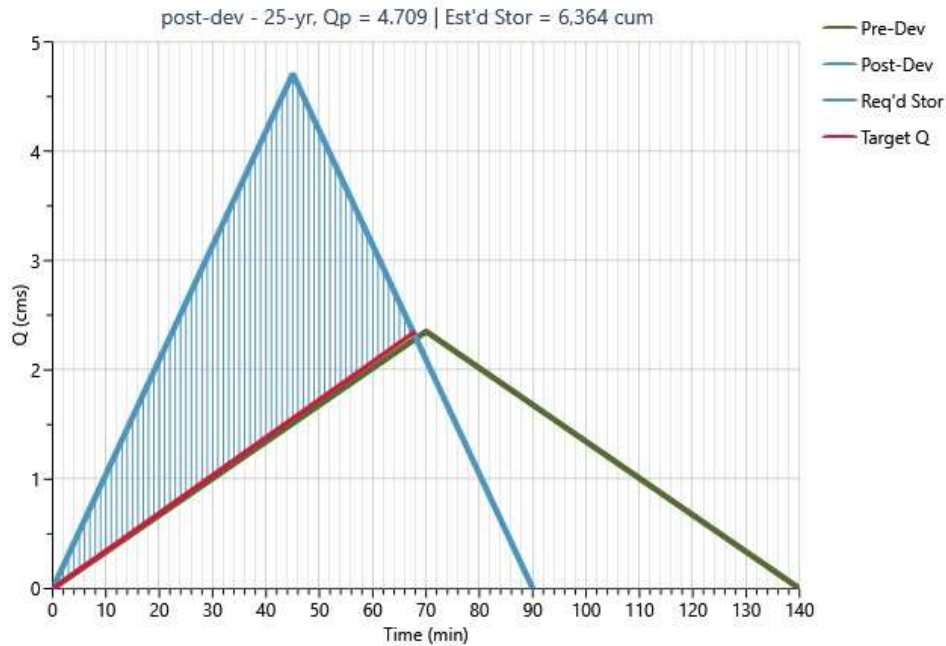


Figure 4.10: Storage estimate of PO 1

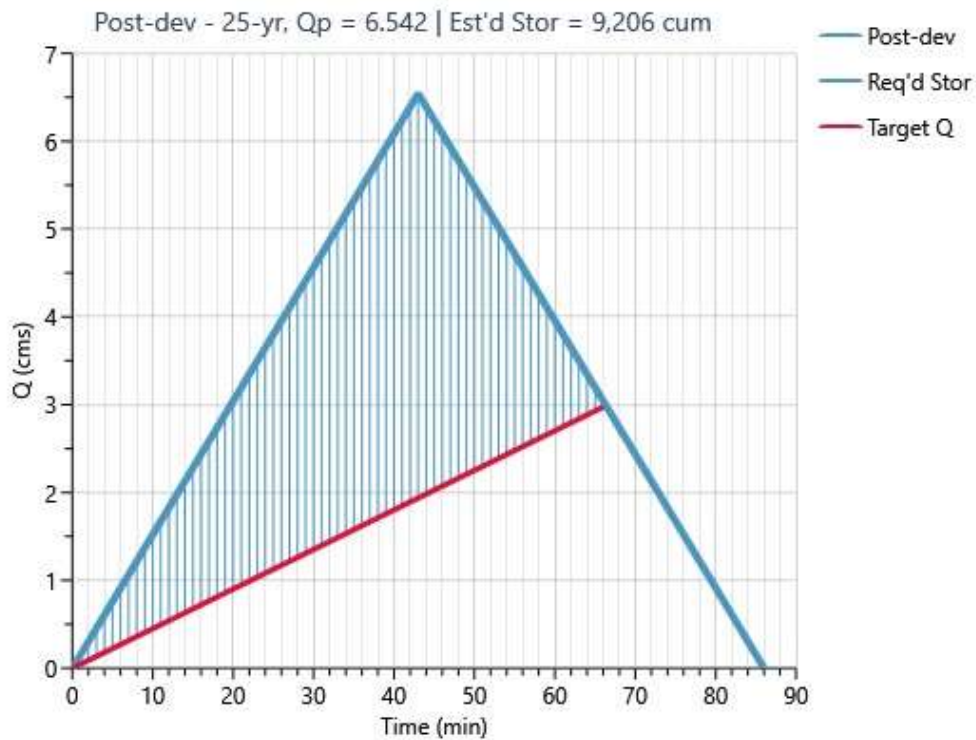


Figure 4.11: Storage estimate of PO 2

Following the determination of the storage volume, the ponds were designed using the Hydrology studio modeling program, For PO-1 and PO-2, trapezoidal sections with 2:1 slope, 53m length, 30m width with 3m depth and 70m length, 35m width with 3m depth were

modeled, respectively. Stage-Storage curves for PO-1 and PO-2 are shown in Figures 4.12 and 4.13, respectively. The storage available at every particular elevation or Stage is shown by the green line. The maximum storage that the PO-1 and PO-2 were estimated to have, based on the aforementioned measurements, was 6,408m³ and 9,384m³, respectively. Which surpasses the calculated storage volume for both ponds. When the ponds are full, the water level will rise to Stage 3m (1660m), which is the highest level possible for both ponds

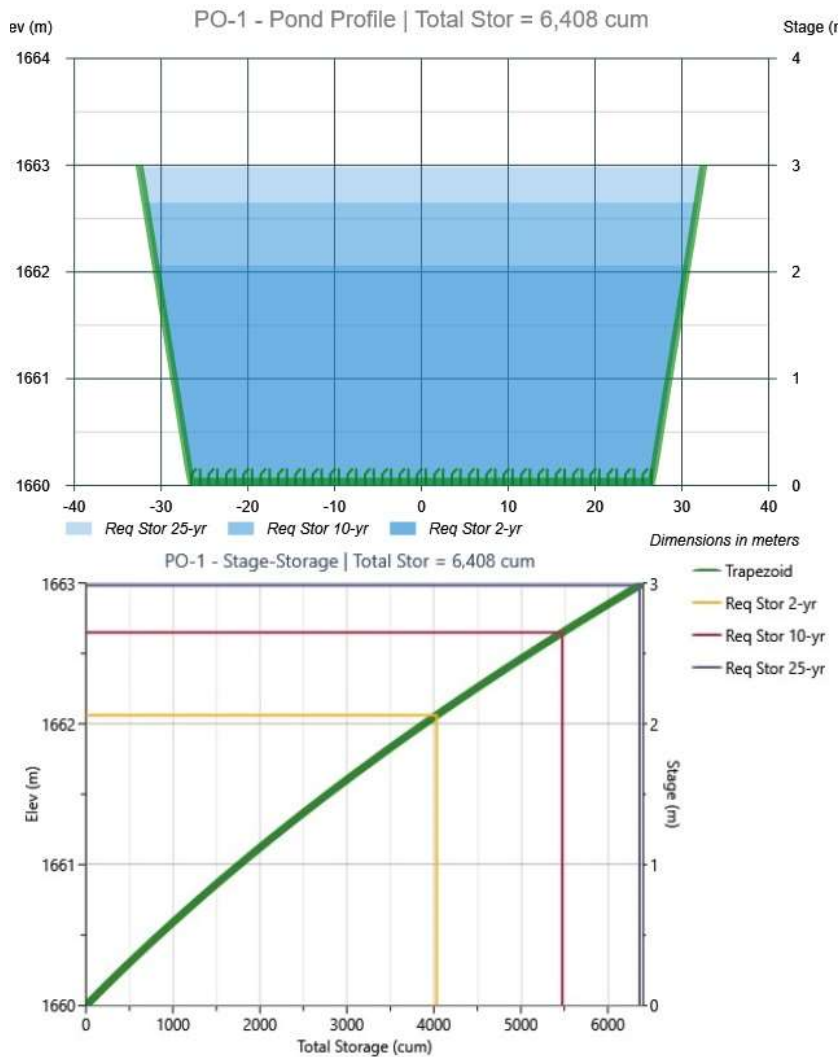


Figure 4.12: Stage-Storage Curve for PO 1

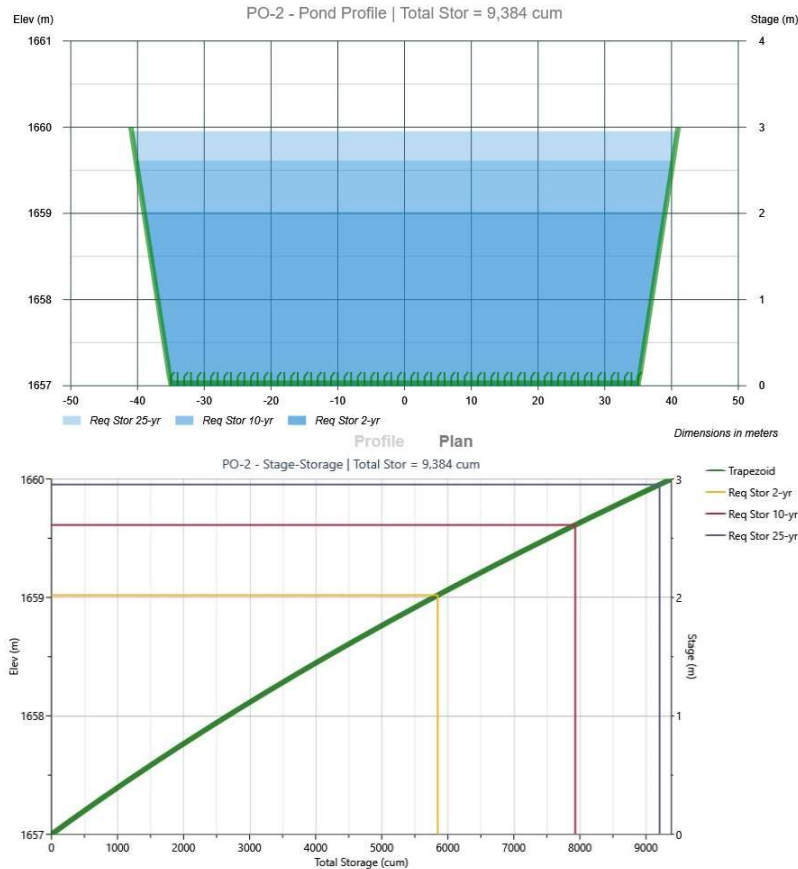


Figure 4.13: Stage-Storage Curve for PO 2

4.6.3 Outlet Structures

Outlet structure is component of a pond which purpose is to release water at a controlled rate. The pond's outlet devices were modeled using a multi-stage configuration in order to control three return period 2, 10 and 25-year. The structure selected for this pond was a composite pond outlet structure that uses a Culvert and a Rectangular Riser, an Orifice and a Rectangular Weir. The Orifice controls the 2-year while the Rectangular Weir controls both the 10 and 25-year events. The Culvert's sole purpose is to convey water out of the pond.

The Figure 4.14 and 4.15 presents the stage-discharge relationship of pond. The blue line was the target stage-discharge of the PO 1 and PO 2, the red curve is the expected outflow through composite outlet structures. The outlet structure stage-discharge curve should closely follow the target rating curve to better estimate the outflow as depicted below.

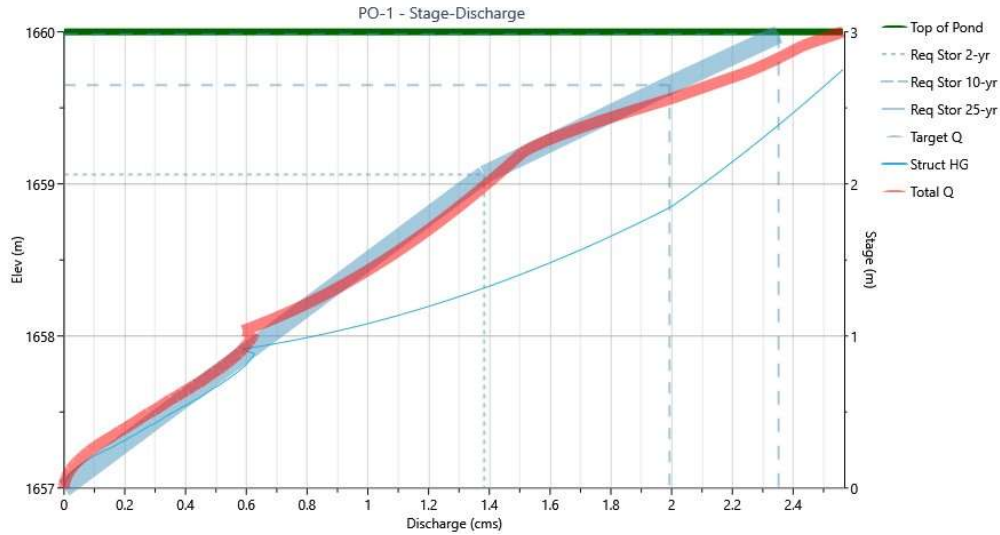


Figure 4.14: Stage-Discharge and Outlet Structures Curve of PO 1

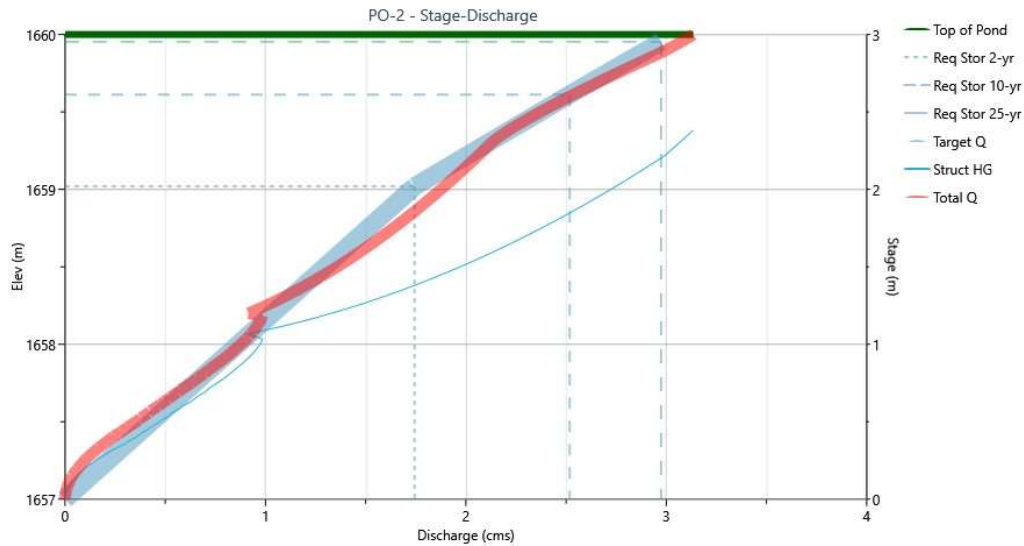


Figure 4.15: Stage-Discharge and Outlet Structures Curve of PO 2

Therefore, PO 1 has an outlet structure composed of culvert with 900mm diameter at invert elevation of 1657m, a rectangular riser with crest elevation 1660m and crest length 3.5m, an orifice with 900mm diameter with an invert elevation of 1657m and a rectangular weir with crest elevation of 1659.2m and 0.8m crest length. (Figure 4.16) and PO 2 has outlet structure composed of culvert with 1050mm diameter at elevation of 1657m, a rectangular riser with crest elevation 1660m and crest length 5m, an orifice with 1100mm diameter with an elevation of 1657m and a rectangular weir with crest elevation of 1659.3m and 0.6m crest length. (Figure 4.17)

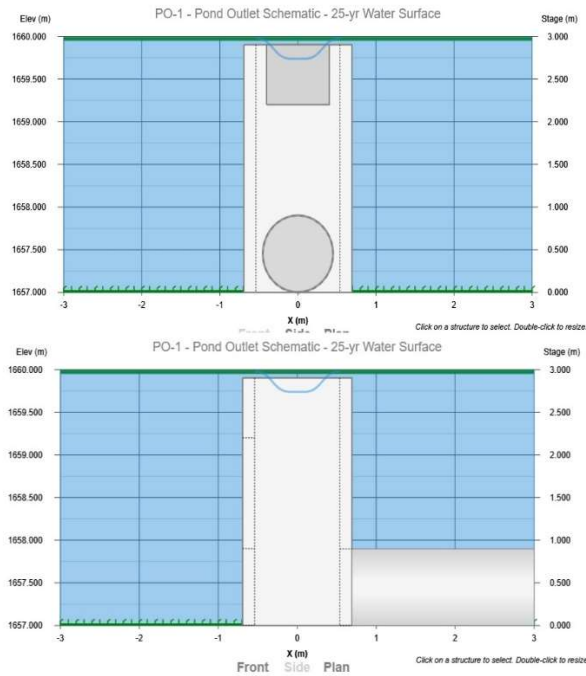


Figure 4.16: Outlet Structure of PO 1

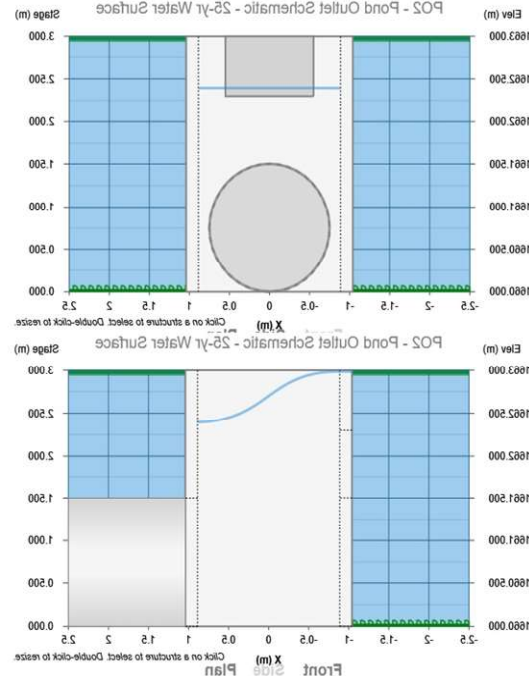


Figure 4.17: Outlet Structure of PO 2

A study by Nicholson et al., (2019) reported that physically-based model has been created to demonstrate the impact of a network of offline storage areas to enable assessment of storage thresholds required to mitigate design storm events, thus enabling design of a natural flood management scheme. The modelling results have shown that peak flow can be reduced by more than 30% at downstream receptors. Similarly, using Hydro Studio modeling software, in this study, it was determined how much the ponds attenuated the post-development peak runoff flow to reduce the occurrence and impact of floods. The results showed that PO-1 reduced the peak runoff flow from the post-development, which was $4.71 \text{ m}^3/\text{s}$, to $2.547 \text{ m}^3/\text{s}$, a 54.08% reduction over 5.6 hours, and PO-2 reduced the peak runoff flow from the post-development, which was $6.54 \text{ m}^3/\text{s}$, to $3.12 \text{ m}^3/\text{s}$, a 47.7% reduction over 6.7 hours. (Figure 18 and 19)

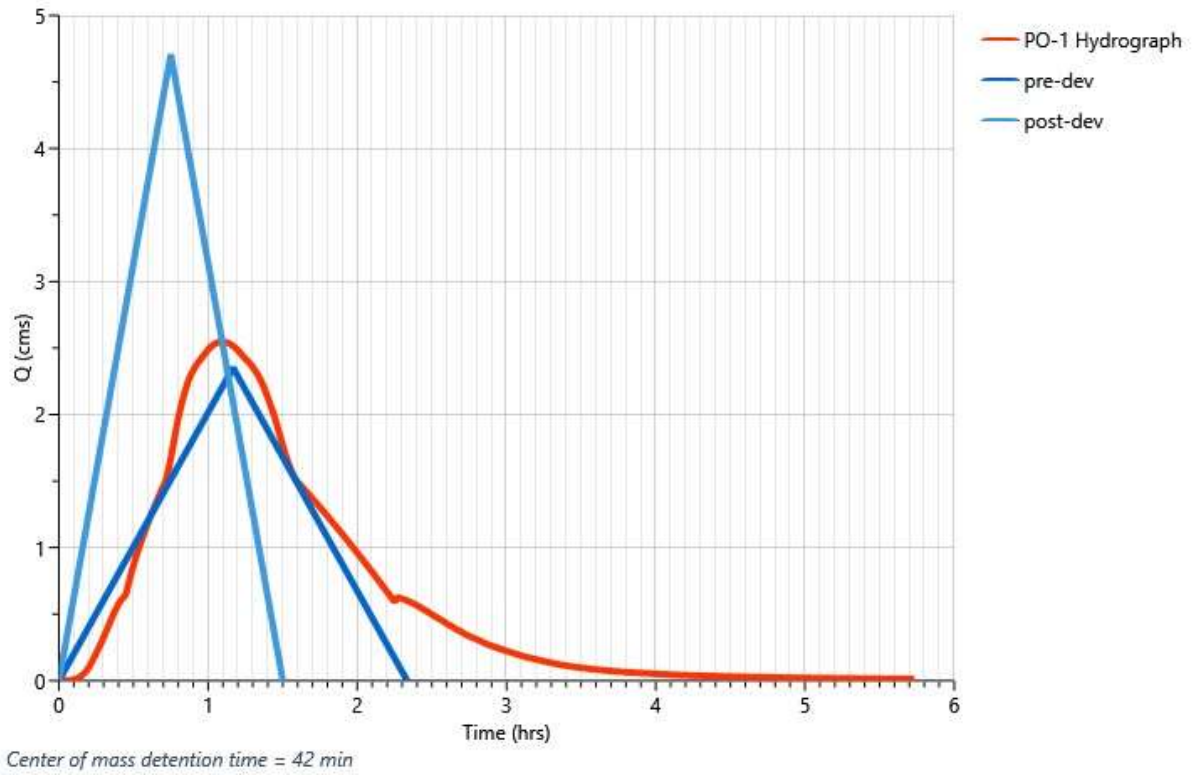


Figure 4.18: Attenuated Hydrograph of PO 1

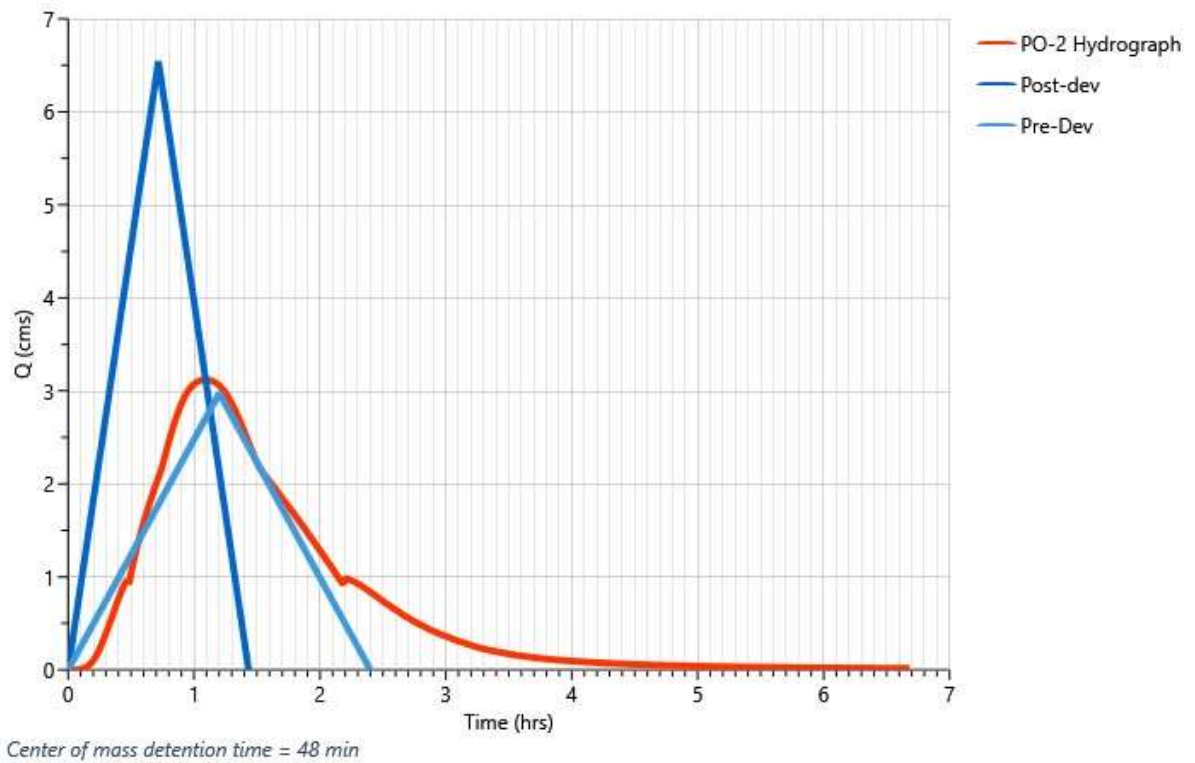


Figure 4.19: Attenuated Hydrograph of PO 2

5 Conclusion and Recommendation

5.1 Conclusions

The goal of this study was to characterize the stormwater and model detention ponds for effective stormwater management of ASTU campus. SewerGEMS model simulation setup was started by first delineating the ASTU catchment into 18 sub-catchments which were coded from CM-1 to CM-18. The rainfall data for model input were taken from Metrological service central for eastern and central Oromia from 1992-2022 which was used to formulate IDF curve of the study area. The composite time of concentration of the sub catchments was calculated using Kerby and US Soil Conservation Service equations. Based the LULC composite runoff coefficients were determined for each sub catchments of ASTU campus. Following the IDF curve formulation, determination of time of concertation and runoff coefficients, the model was run for 2, 5, 10 25-, 50- and 100-year return to obtain the peak flow for each sub catchments of ASTU campus. After analyzing the results, the following conclusions were drawn:

The peak runoff flow rate from the ASTU campus varied from 4.45 m³/s during the 2-year return period to 14.64 m³/s during the 100-year return period, and it is this flow that is flooding and other flood-related risks occur for the downstream neighboring areas.

Based on the hydraulic analysis findings for conveying the runoff flow from ASTU, it has been concluded that a network of circular concrete pipes with 10.28km total length and diameters ranging from 150mm to 1200mm can securely covey the 25-year return period stormwater flows to the specified detention ponds.

Based the existing drainage network assessment result, channels type 1, 2, 4, and 6 and conduit 42-inch diameter in sub-catchment CM-1, CM-7, and CM-15 were over-designed while channels type 1 and conduit 30-inch in sub-catchment CM-13 and CM-16 were found to be under-designed furthermore the drainage network is disintegrated with 3 outlets to different areas. Which lead to the conclusion that network requires significant revision both in capacity and layout.

Pre-development, which refers to the natural LULC, was compared to post-development, which refers to the future LULC as per the master plan of the campus. The results showed that 38.23%

of what was once cropland with open woodland land cover was converted into roof catchments, asphalt and cobblestone roads, concrete covers and sports fields, which led to the conclusion that the study area's 47% increase in peak runoff flow was caused by the change of once pervious land cover to relatively impervious one.

Two detention ponds were proposed to attenuate the peak runoff flow during the 100-year return period based on the contour lines of the study area. According to the findings, detention ponds PO-1 and PO-2, respectively, can attenuate and store a runoff volume of $6,408\text{m}^3$ and $9,384\text{m}^3$ to be used for various purposes. A $53\text{m} \times 30\text{m} \times 3\text{m}$ PO-1 and a $70\text{m} \times 35\text{m} \times 3\text{m}$ PO-2 detention ponds were modeled to hold the aforementioned amount. As a result, PO-1 and PO-2 may attenuate 54.08% and 47.7%, respectively, of the peak flow. Thus, it can be concluded that a total average peak runoff flow of 50.8%, which represented the risk of flooding to the downstream areas, can be reduced.

5.2 Recommendations

Traditional development of the landscape with impervious surfaces such as buildings, roads, and parking lots, as well as storm sewer systems and other man-made features, alters the hydrology of a catchment and has the potential to adversely affect water quality. As a result of development, vegetated and forested land that consists of pervious surfaces is largely replaced by land uses with impervious surfaces. This transformation increases the amount of stormwater runoff from a site, decreases infiltration and groundwater recharge, and alters natural drainage patterns. By mimicking natural drainage regimes, this the research aims to reduce surface water flooding, and enhance the amenity and biodiversity value of the environment by increasing water storage capacity. Therefore, based on this study findings, the following are recommended.

- ❖ Construction of two trapezoidal Detention Ponds with a combined storage capacity of at least 15,792 m³ be taken into consideration in order to reduce surface water flooding and flooding-related risks to the campus and downstream areas as well as to obtain additional water for the campus landscaping project or other uses.
- ❖ It is advised to choose closed concrete stormwater drainage pipes in order to prevent further deterioration of the water quality. To transport the peak runoff flow from the ASTU Campus to the proposed detention ponds, networked pipes with a diameter ranging from 150 mm to 1200 mm are suggested.
- ❖ Further research on water quality analysis with water treatment options for stored water to be relevant to many applications such as domestic consumption, irrigation, and so on.
- ❖ Further research on the assessment of stored storm water on both amount and quality is advised to reuse it for irrigation or landscape development.
- ❖ Further studies in a stormwater management approach which seeks to mimic a site's predevelopment hydrology by using design techniques that infiltrate, filter, store and evaporate stormwater runoff at or close to its source is recommended.
- ❖ There is no organized data regarding stormwater drainage structures in the campus which is essential to investigate the stormwater. Therefore, it is recommended that organized data to be available to all users online, in libraries and other easily accessible areas.

- ❖ Since time and shortage of data were the primary limitations of this study, the runoff determination was done using only the Rational Method. However, it advised more studies on alternative runoff determination methods for a better outcome.

References

- Ashley, W., Bentley, M., & Stallins, J. (2012). Urban-induced thunderstorm modification in the Southeast United States. *Climatic Change*.
- Adama Science and Technology University. (2020). *Background of ASTU*. Retrieved May 12, 2023, from <https://www.astu.edu.et/about-us/background-of-astu>
- Adugna, D., Mamaru, B. L., Bergen, M. J., & Sahilu, G. G. (2019). Evaluating the hydraulic capacity of existing drain systems and the management challenges of stormwater in Addis Ababa, Ethiopia. *Journal of Hydrology Regional Studies* 25:100626.
- Alemu, A. (2018). *Assessment of Impacts of Urbanization on Surface Water Quality, The Case of Keha River, Gondar Town*. Msc Thesis, Addis Ababa University, Ethiopia.
- Bajracharya, A., Rai, R. R., & Rana, S. (2016). Effects of Urbanization on Storm Water Run-off : A Case Study of Kathmandu Metropolitan City, Nepal. *Journal of the Institute of Engineering*.
- Bentley Systems. (2019, February 13). Retrieved May 14, 2023, from Bentley SewerGEMS CONNECT Edition Help:
<https://docs.bentley.com/LiveContent/web/Bentley%20StormCAD%20SS5-v1/en/GUID-689B704BA4064162B2EE6FD879FFC8AB.html>
- British Geological Survey. (2022, February 04). *Sustainable drainage systems (SuDS)*. Retrieved June 05, 2023, from BGS Research: <https://www.bgs.ac.uk/geology-projects/suds/>
- CivilWeb Spreadsheets. (2019, February 23). *Attenuation Design Guide*. Retrieved May 4, 2023, from <https://civilweb-spreadsheets.com/drainage-design-spreadsheets/attenuation-design/attenuation-design-guide/>
- Elias, M. (2015). Highway Drainage Facilities Problems (Case Study – Assessment of Drainage Problems in Adama). *Msc Thesis, Addis Ababa University, Ethiopia*.
- Elsebaie, I. (2012). Developing rainfall intensity–duration–frequency relationship for two regions in Saudi Arabia. *Journal of King Saud University - Engineering Sciences*.

- Ethiopia Roads Authority (ERA). (2013). *Drainage Design Manual*. Addis Ababa, Ethiopia.
- Eui, H. L., Yong , S. L., Jin, G. J., Donghwi , J., & Joong , H. K. (2016). Flood Reduction in Urban Drainage Systems: Cooperative Operation of Centralized and Decentralized Reservoirs. *Water* 8(10).
- Freni, G., Maglionico, M., Mannina, G., & Viviani, G. (2008). Comparison between a detailed and a simplified integrated model for the assessment of urban drainage environmental impact on an ephemeral river. *Urban Water Journal* 5(2).
- Geiger, W. F., Marsalek, J., Rawls, W., & Zuidema, F. C. (1987). *Manual on drainage in urbanized areas*. France: UNESCO.
- Hawaz, M. (2020). *Assessment of Storm Water Drainage System Performance of Mojo Town*. Msc Thesis, Addis Ababa University, Ethiopia.
- Herrera-Granados, O. (2010). Hydraulic modelling : choosing the most appropriate tool in the decision-making process for water resources management. *VIII Konferencja Naukowa Studentów* At. Szklarska Poręba, Poland: Researchgate.
- Lei , Y., Li-Ding , C., & Zhang , W. T. (2017). Exploring the Linkage between Urban Flood Risk and Spatial Patterns in Small Urbanized Catchments of Beijing, China. *International Journal of Environmental Research and Public Health*.
- Leopold, L. B. (1968). *Hydrology for Urban land Planning - A Guidebook on the Hydrologic Effects of Urban land Use*. Washington: Geological Survey Circular.
- Li , L., Xing , L., Gaoyuan, X., Juliang , J., & Guowei , C. (2016). Spatial fuzzy clustering approach to characterize flood risk in urban storm water drainage systems. *Natural Hazards*.
- Manoj, N., Keshav, B., & Ganesh, P. P. (2020). Modeling of Urban Storm Water Drainage using Hykas for Lamachaur, Pokhara, Nepal. *Proceedings of 8th Ioe Graduate Conference*. Nepal.
- Marie-Claire, T. V., Francois, C., & Pieter, v. G. (2011). Quantitative fault tree analysis for urban water infrastructure flooding. *Structure and Infrastructure Engineering* 7(11).

- Moges, S., Sahilu , G. G., Habtamu, G., & Raschid-Sally, L. (2019). Built Environment, flooding regime change, and its effect on adaptation to climate change in the cities of developing countries: Learning a lesson from Addis Ababa City, Ethiopia. In *Extreme Hydrology and Climate Variability*. Sri Lanka : Researchgate.
- Morita, M. (2014). Flood Risk Impact Factor for Comparatively Evaluating the Main Causes that Contribute to Flood Risk in Urban Drainage Areas. *Water* 6(2):253-270.
- Mugume, S., & Butler, D. (2016). Evaluation of functional resilience in urban drainage and flood management systems using a global analysis approach. *Urban Water Journal*.
- Nicholson, A., O'Donnell, G., Wilkinson, M., & Quinn, P. (2019). The potential of runoff attenuation features as a Natural Flood Management approach. *Flood Risk Management*.
- Nivedita , G., & Pratap , R. (2015). dentifying objectives for sustainable stormwater management in urban Indian perspective: a case study. *International Journal of Environmental Engineering*.
- Ochoa-Rodriguez, S., Wang, L.-P., Gires, A., Pina, R., Reinoso-Rondinel, R., Bruni, G., . . . Ten Veldhuis, M.-C. (2015). Impact of Spatial and Temporal Resolution of Rainfall Inputs on Urban Hydrodynamic Modelling Outputs: A Multi-Catchment Investigation. *Journal of Hydrology*.
- Poleto, C., & Tassi, R. (2012). Sustainable Urban Drainage Systems. In M. S. Javaid, *Drainage Systems* (pp. 52-70). Brazil: InTechOpen.
- Rupak , A., Jeevetha , K., S , V., Ravi , N., & S.H. , L. (2009). Review of Stormwater Quality, Quantity and Treatment Methods: Stormwater: Quality Modelling. *Environmental Engineering Research* 14(2).
- Salvan, L., Abily, M., Gourbesville, P., & Schoorens, J. (2016). Drainage System and Detailed Urban Topography: Towards Operational 1D-2D Modelling for Stormwater Management. *Procedia Engineering*.

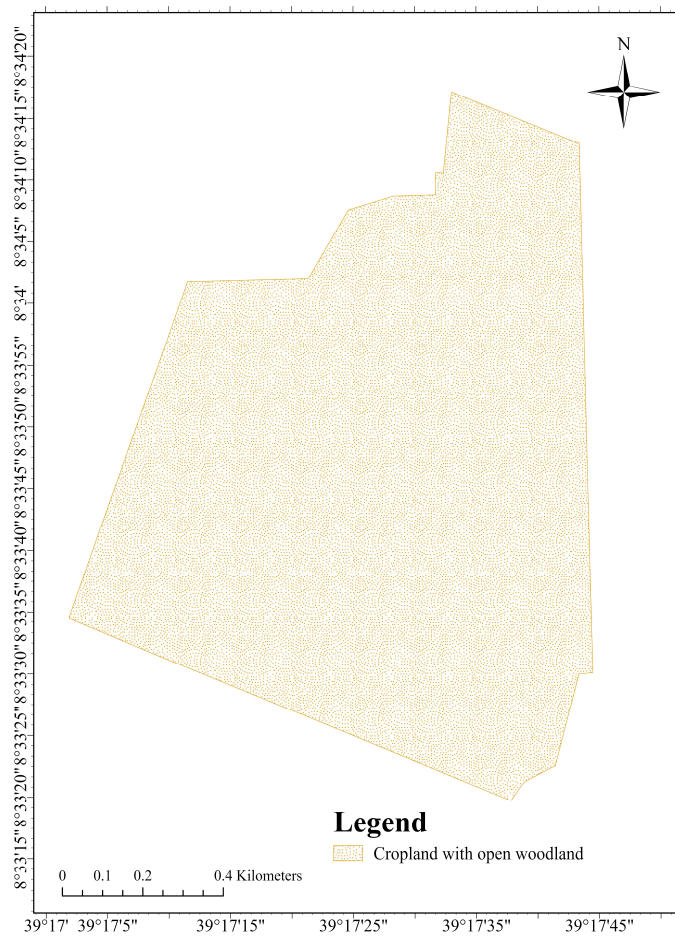
- Sasaki Associates Inc. (2018, February 2). *Stormwater as an Asset on Urban Campuses*. Retrieved June 3, 2023, from <https://www.sasaki.com/voices/stormwater-as-an-asset-on-urban-campuses/>
- Shamkhi, M., Azeez, M., & Obeid, Z. (2022). Deriving rainfall intensity–duration–frequency (IDF) curves and testing the best distribution using EasyFit software 5.5 for Kut city, Iraq. *Open Engineering*.
- Stephen , J., & Vladimir , K. (2022). Evaluating the Effect of the Location and Design of Retention Ponds on Flooding in a Peri-Urban River Catchment. *Land*.
- Sun, Y., Wendi, D., Eon Kim, D., & Liong, S.-Y. (2019). Deriving intensity–duration–frequency (IDF) curves using downscaled in situ rainfall assimilated with remote sensing data. *Geoscience Letters*.
- Teamir, A. (2018). *Sustainable Urban Drainage Options for Mekelle City*. Msc Thesis, Addis Ababa University, Ethiopia.
- Teferra, D. (2020). Assessment and Modeling of Urban Drainage System , a Case Study of Mojo Town. *Msc Thesis, Addis Ababa University, Ethiopia*.
- Twagirayezu, G., Judith , M. K., Bakunzibake, P., Bunani, P., & Luc , J. H. (2019). Development of Rainfall Intensity Duration Frequency (IDF) Curves for Hydraulic Design Aspect. *Ecology & Natural Resources*.
- Yannopoulos, S., Grivaki, G., Giannopoulou, I., Basbas, S., & Oikonomou, E. (2013). Environmental impacts and best management of urban stormwater runoff: Measures and legislative framework. *Global Nest Journal*.
- Ydnekachew, A. (2019). *Performance Evaluation of Stormwater Drainage System: The Case of Hawassa City*. Msc Thesis, Addis Ababa University, Ethiopia.
- Zhou, Q. (2014). A Review of Sustainable Urban Drainage Systems Considering the Climate Change and Urbanization Impacts.

Appendices

Annex 1: Gorges formed by floods from the ASTU watershed



Appendix 2: 2000 LULC map of ASTU campus (MoWE)



Appendix 3: Catch Basin coordinates and elevation

Label	X (m)	Y (m)	Elevation (Ground) (m)	Elevation (Rim) (m)	Elevation (Invert) (m)
CB-1	532,339.86	947,234.32	1,689.96	1,689.96	1,689.09
CB-2	532,511.09	947,024.14	1,698.72	1,698.72	1,697.49
CB-3	532,517.60	946,740.21	1,694.83	1,694.83	1,692.94
CB-4	532,526.62	946,488.69	1,690.21	1,690.21	1,684.42
CB-5	532,309.65	946,483.71	1,663.37	1,663.37	1,662.27
CB-6	532,525.25	946,281.12	1,683.08	1,683.08	1,682.43
CB-7	532,304.83	946,294.40	1,661.05	1,661.05	1,659.80
CB-8	532,503.50	946,030.10	1,674.94	1,674.94	1,674.29
CB-9	531,899.33	946,885.14	1,681.62	1,681.62	1,680.52
CB-10	531,997.59	946,674.40	1,679.05	1,679.05	1,677.48
CB-11	532,010.47	946,311.85	1,674.79	1,674.79	1,673.54
CB-12	531,913.76	946,788.38	1,684.12	1,684.12	1,683.47
CB-13	531,951.00	946,469.65	1,666.49	1,666.49	1,665.84
CB-14	531,963.19	946,301.38	1,661.85	1,661.85	1,661.20
CB-15	531,610.15	946,479.82	1,663.04	1,663.04	1,661.23
CB-16	531,502.21	946,921.58	1,684.11	1,684.11	1,683.36
CB-17	531,954.48	945,907.72	1,660.00	1,660.00	1,659.35
CB-18	531,769.55	945,950.42	1,655.00	1,655.00	1,653.98
CB-19	532,249.82	947,029.23	1,688.96	1,688.96	1,688.31

Appendix 4: Catch Basin coordinates and elevation

Label	X (m)	Y (m)	Elevation (Ground) (m)	Elevation (Invert) (m)	Flow (m³/s)
MH-1	532,297.89	947,039.66	1,675.96	1,674.94	0.56
MH-2	532,316.07	946,687.23	1,671.93	1,670.78	1.12
MH-3	532,287.77	946,023.41	1,657.37	1,656.12	2.07
MH-4	531,995.22	946,782.81	1,680.40	1,678.72	0.51
MH-5	532,004.40	946,472.32	1,676.52	1,675.27	1.49
MH-6	531,998.68	945,998.10	1,666.62	1,665.37	2.66
MH-7	532,001.07	945,897.12	1,661.78	1,660.53	2.65
MH-8	531,622.58	946,774.92	1,668.26	1,667.39	0.53
MH-9	531,609.83	946,292.84	1,658.58	1,656.36	1.76
MH-10	531,348.18	946,491.82	1,661.48	1,660.61	0.79
MH-11	531,289.79	946,318.10	1,652.45	1,651.20	2.89
MH-12	531,455.64	946,118.79	1,650.93	1,649.83	0.67

Appendix 5: Runoff flow for Pre and Post Development of ASTU sub catchments

Label	Post Development Flow (m ³ /s)					Pre-Development Flow (m ³ /s)				
	2	10	25	50	100	2	10	25	50	100
CM-1	0.09	0.18	0.23	0.27	0.3	0.09	0.18	0.23	0.27	0.3
CM-2	0.1	0.21	0.26	0.3	0.34	0.04	0.08	0.1	0.12	0.13
CM-3	0.15	0.31	0.39	0.45	0.51	0.15	0.29	0.37	0.42	0.48
CM-4	0.21	0.42	0.52	0.6	0.68	0.08	0.16	0.21	0.24	0.27
CM-5	0.17	0.34	0.43	0.5	0.56	0.1	0.2	0.25	0.29	0.33
CM-6	0.16	0.32	0.4	0.46	0.52	0.06	0.13	0.16	0.18	0.2
CM-7	0.1	0.2	0.25	0.29	0.32	0.04	0.08	0.11	0.12	0.14
CM-8	0.17	0.35	0.43	0.5	0.56	0.07	0.14	0.17	0.2	0.22
CM-9	0.22	0.44	0.55	0.63	0.72	0.11	0.23	0.29	0.33	0.38
CM-10	0.21	0.42	0.53	0.61	0.69	0.08	0.15	0.19	0.22	0.25
CM-11	0.13	0.27	0.33	0.38	0.43	0.05	0.1	0.13	0.15	0.17
CM-12	0.17	0.35	0.44	0.5	0.57	0.06	0.12	0.15	0.17	0.19
CM-13	0.19	0.39	0.49	0.56	0.64	0.07	0.14	0.17	0.2	0.22
CM-14	0.17	0.35	0.44	0.51	0.57	0.06	0.12	0.15	0.18	0.2
CM-15	0.17	0.34	0.43	0.5	0.56	0.06	0.13	0.16	0.19	0.21
CM-16	0.24	0.49	0.61	0.7	0.8	0.1	0.19	0.24	0.28	0.31
CM-17	0.18	0.37	0.46	0.53	0.6	0.13	0.25	0.32	0.37	0.41
CM-18	0.06	0.13	0.16	0.19	0.21	0.09	0.17	0.22	0.25	0.28

Appendix 6: Outlet structures of ASTU catchment

Label	X (m)	Y (m)	Elevation (Ground) (m)	Elevation (Invert) (m)
O-1	531,330.02	946,200.76	1,649.79	1,647.13
O-2	532,313.19	945,843.36	1,650.32	1,648.17

Appendix 7: The Highest Daily Peak Rainfall (mm)

S.No	Year	Highest Daily Peak mm
1	1992	41.1
2	1993	70.0
3	1994	51.0
4	1995	77.5
5	1996	47.5
6	1997	61.4
7	1998	59.8
8	1999	41.5
9	2000	82.2
10	2001	44.3
11	2002	82.2
12	2003	70.4
13	2004	43.3
14	2005	42.3
15	2006	62.8
16	2007	55.1
17	2008	72.5
18	2009	54
19	2010	56.1
20	2011	46.9
21	2012	69.6
22	2013	59.4
23	2014	46.6
24	2015	52.6
25	2016	56
26	2017	68.4
27	2018	56.4
28	2019	56.3
29	2020	57
30	2021	69.1
31	2022	45.4

Appendix 8: Reduced Standard deviation of mean of Gumbel's distribution

TABLE -1 - REDUCED MEAN $\bar{y}_n$ IN GUMBEL'S EXTREME VALUE DISTRIBUTION

N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

TABLE -2 - REDUCED STANDARD DEVIATION S_n IN GUMBEL'S EXTREME VALUE DISTRIBUTION

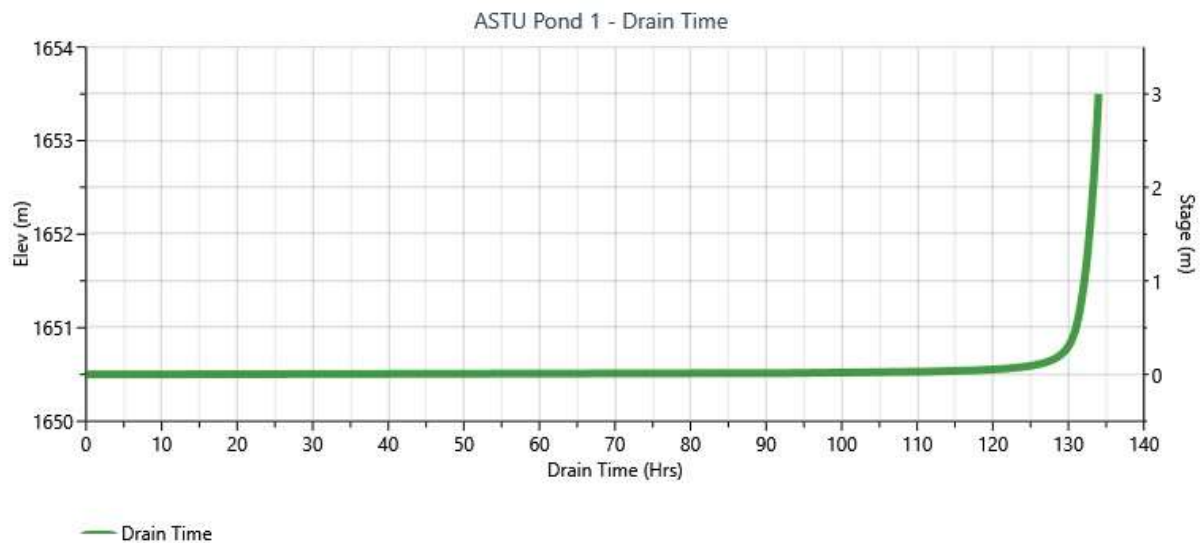
N = sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Appendix 9: Stage vs Storage Relationship of PO 1

Stage (m)	Elevation (m)	Contour Area (sqm)	Incr Stor (cum)	Total Stor (cum)
0	1650.5	3,400	0	0
0.15	1650.65	3,475	516	516
0.3	1650.8	3,551	527	1,043
0.45	1650.95	3,628	538	1,581
0.6	1651.1	3,706	550	2,131
0.75	1651.25	3,784	562	2,693
0.9	1651.4	3,863	574	3,266
1.05	1651.55	3,943	585	3,852
1.2	1651.7	4,023	597	4,449
1.35	1651.85	4,104	610	5,059
1.5	1652	4,186	622	5,681
1.65	1652.15	4,269	634	6,315
1.8	1652.3	4,352	647	6,961
1.95	1652.45	4,436	659	7,620
2.1	1652.6	4,521	672	8,292
2.25	1652.75	4,606	684	8,976
2.4	1652.9	4,692	697	9,674
2.55	1653.05	4,779	710	10,384
2.7	1653.2	4,867	723	11,107
2.85	1653.35	4,955	737	11,844
3	1653.5	5,044	750	12,594

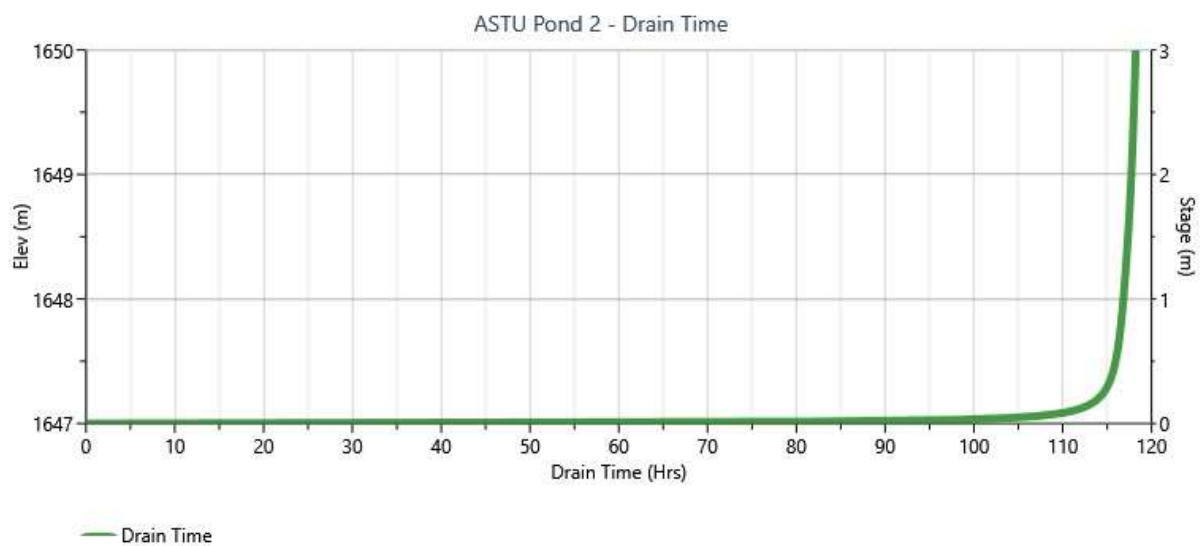
Appendix 10: Drain time of PO 1



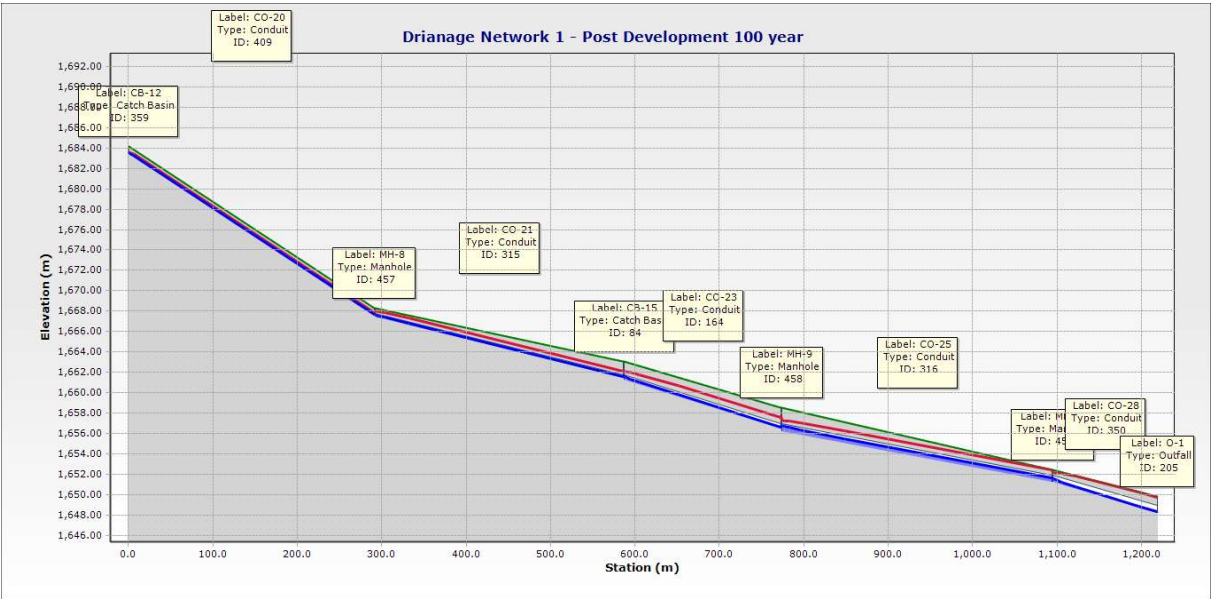
Appendix 11: Stage vs Storage Relation of PO 2

Stage (m)	Elevation (m)	Contour Area (sqm)	Incr Stor (cum)	Total Stor (cum)
0	1647	3,600	0	0
0.15	1647.15	3,678	546	546
0.3	1647.3	3,757	558	1,104
0.45	1647.45	3,837	570	1,673
0.6	1647.6	3,918	582	2,255
0.75	1647.75	3,999	594	2,849
0.9	1647.9	4,081	606	3,454
1.05	1648.05	4,164	618	4,073
1.2	1648.2	4,247	631	4,704
1.35	1648.35	4,331	643	5,347
1.5	1648.5	4,416	656	6,003
1.65	1648.65	4,502	669	6,672
1.8	1648.8	4,588	682	7,354
1.95	1648.95	4,675	695	8,048
2.1	1649.1	4,763	708	8,756
2.25	1649.25	4,851	721	9,477
2.4	1649.4	4,940	734	10,211
2.55	1649.55	5,030	748	10,959
2.7	1649.7	5,121	761	11,720
2.85	1649.85	5,212	775	12,495
3	1650	5,304	789	13,284

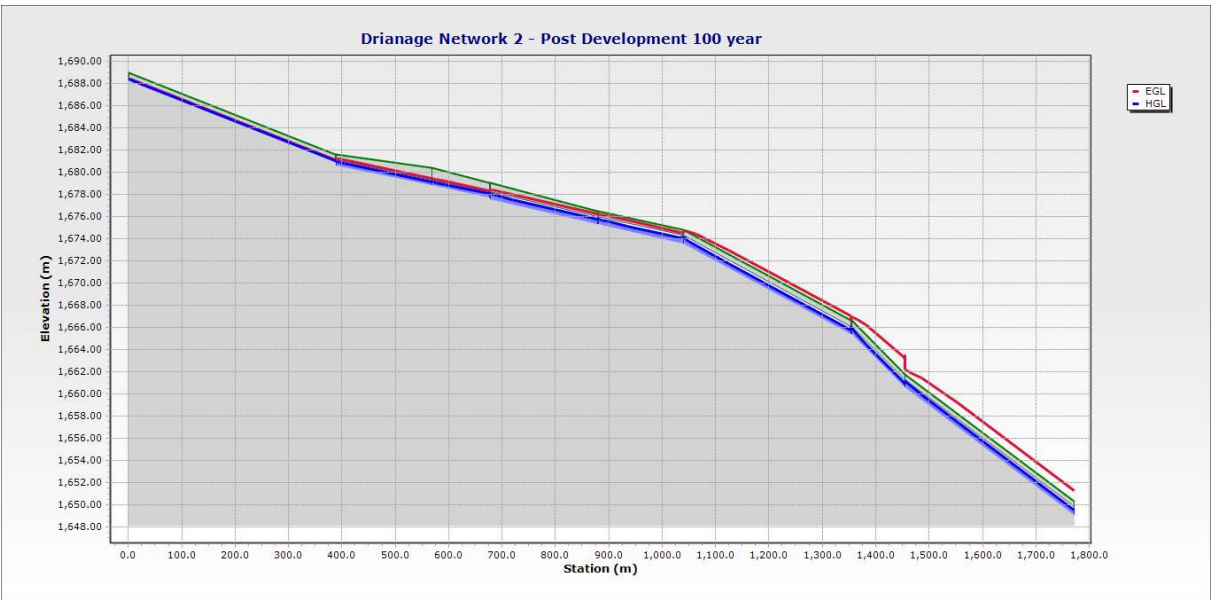
Appendix 12: Drain time of PO 2



Appendix 13: Pipe Profile for Drainage Network 1



Appendix 14: Pipe Profile for Drainage Network 2-a



Appendix 14: Pipe Profile for Drainage Network 2-b

