Performance Assessment of Storm Water Drainage System by Stormwater Management Model

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Abstract

As urban population numbers increase, it is evident that urbanization areas continue to expand. This expansion of urban areas mainly changes the natural land surface to the artificial landscape. The urban landscape is partly impervious, and it will decrease infiltration and in reverse increase the volume of surface runoff which causes several destructions in the town. Thus, the study was to assess the performance of the stormwater drainage system of Ambo town. The data have been used for this study were both spatial and temporal data. Rainfall data for 1987 to 2018 years and digital elevation models, land use, and soil data were used. Additionally, canal dimensions and sub-catchment data were also used. For this study, SWMM for simulating rainfall-runoff, the water level in the junctions, and flow depth in the canal were used. ARC-GIS was used to obtain spatial information and delineate the catchment of the study area. Intensity duration frequency curves of Ambo were developed using Gumbel distribution methods to analyze peak runoff for different return periods. The total runoff from the whole catchment using the rational method is 60.32m ^{3} /s, whereas the stormwater management model (SWMM) is 60.87m ^{3} /s. The model performance was evaluated using performance measuring techniques including Nash Sutcliff Efficiency (NSE) and Coefficient of Determination (R2), and the values were RNS = 0.997 and R2 = 0.997 respectively for a ten-year return period which shows the model gives good results. From simulation results, 22% of junctions are insufficient to accommodate maximum flooding, and 21% of canals are surcharged. So, from the simulation results, most of the junction and canal dimensions are insufficient to drain the generated runoff from the sub-catchments. To alleviate these problems, clearing, on-time maintenance, redesign, and constructing the adequate canal are recommended for Ambo town.

Introduction

According to(Abeje and Ortiz 2014), the urban population in Ethiopia is expected to nearly triple from 15.2 million in 2012 to 42.3 million in 2037, with an annual growth rate of 3.8 percent. According to the findings of this analysis, the rate of urbanization will be considerably quicker, at around 5.4 percent each year (Dorosh et al. 2018). As the urban population continues to increase from time to time, urban space also continues to expand to accommodate a growing global population. This expansion of urban areas leads to alterations in natural processes, environmental quality, and landscape (McGrane 2016). The impact of urban developments on runoff aggravates after construction, as roads (gravel, cobblestone, and asphalt), parking, driveways, rooftops, and other different impervious surfaces partly decrease the infiltration of rainfall into the ground. So, it is evident that the result is increased surface runoff and significantly reduced groundwater recharges (Wadhwa and Pavan Kumar 2020).

Drainage systems are needed in urban areas because of the interaction between human activity and the natural water cycle. This interaction has two primary forms. The first one is that, abstraction of water for water supply, and diverting rainwater away from the local natural drainage system. These two interactions imply that two types of water require drainage. The first is wastewater, which is water that has been provided to the user for a variety of purposes. If not adequately drained after use, it may pollute the environment and provide a health concern. As a result, urban drainage systems deal with these two types of water to reduce the issues they pose to human life and the environment (Lund et al. 2018). Stormwater becomes one of the most critical issues when there is excessive stormwater runoff, and cannot be controlled(Srishantha and Rathnayake 2017) .

If stormwater were not drained properly, it would cause flooding, which leads to several damage and health problems(Pale et al. 2018). To safely dispose of stormwater, the capacity of drainage must be adequate. So, to assess the capacity of drainage, different hydrologic models can be used for assessing the capacity of drainage systems.

The Stormwater Management Model (SWMM) is widely used for urban drainage system planning, analysis, and design. The Stormwater Management Model (SWMM) is a dynamic rainfall-runoff simulation model that is widely used to simulate the quantity and quality of runoff that is generated from urban areas for single events as well as long-term (continuous) simulations (Huber et al. 1975; U.S. Environmental Protection Agency (EPA) 2015). Control of runoff at the source, flood protection, and safe disposal of excess water/runoff through proper drainage facilities become important in Ethiopian contexts, where watersheds of many urban centers receive significant amounts of annual rainfall, and rainfall intensity is produced at a high level (Adugna et al. 2019). Inadequate urban stormwater drainage is one of the most prevalent reasons for citizen complaints in many Ethiopian towns, and the problem is getting worse with towns were recording a continuous high urbanization rate (Kebede Warati 2015).

For the last few years, it has been noticed that Ambo town has been associated with the rapid conversion of land from rural to urban areas like buildings, different types of roads, and other artificial structures where most of them are partly impervious. So it is evident that it has a significant role in increasing surface runoff. Like much of the towns in Ethiopia, Ambo town is the town that has been facing different challenges regarding stormwater drainage. To identify drainage-related problems, the determination of peak runoff generated from the urban area is a crucial step in determining the hydraulic capacity of urban drainage systems (Rabori and Ghazavi 2018).So, the purpose of this article was to assess the capacity of the existing stormwater drainage system using the Stormwater Management Model (SWMM), to identify significant problems with the stormwater drainage system, and recommend possible mitigation measures for Ambo town.

Material And Methods

2.1 General Description of the study area

Ambo town is located in the Western Shoa Zone of Oromia National Regional State (ONRS) of Ethiopia and is found at a distance of 126km from Addis Ababa. Ambo town, founded in 1889, is one of Ethiopia's oldest towns. It covers a total area of 1320 hectares (Srilaxmi Shanmugham, BekeleTekle 2011). In terms of geographic coordinates, the town has a latitude and longitude of 8°59'N 37°51'E and an elevation of 2101 meters above sea level, and the location of Ambo town is shown in Fig. 1.

2. Materials and software used

For proper implementation of the study, some equipment, materials, and software were used for data collection, processing, and evaluation. Software and materials used for this study were listed one by one below.

SWMM

is a dynamic rainfall-runoff simulation model used for a single event or long-term (continuous) simulation of runoff quantity, the water level at the junction, and link.

ARC-GIS

to obtain hydrological and physical parameters and spatial information of the catchments of the study area.

Excel spreadsheet

for pre and post-processing of data and results

Contour Map

To successfully delineate a watershed boundary, it is needed to visualize the landscape as represented by a topographic map. This map helps to examine the elevation, determine flow direction and flow length of the catchment areas.

Master plan

To look into the overall conditions of land use land cover of the study area.

Handle GPS

To measure the elevation of the existing drainage system.

Tape meter

To measure the existing drainage dimensions like its depth, width, and length.

Digital camera

To take HD pictures of the field

2.3 Data type, source, and collection process

2.3.1 Data type

There are two major categories of data types that have been used for this research. Those data types are briefly discussed in the following.

i. Primary data: the most reliable and relevant data for this study. The primary data were the dimensions of the existing drainage system measured at the field and its current drainage conditions, as well as information on which areas were more prone to flooding.

ii. Secondary data: Rainfall data, DEM data, and land use map, as well as soil map, were gathered from various related agencies.

2.4 Data analysis and quality checking

Before using the collected data immediately, it must be discussed and analyzed.

2.4.1 Checking for missing rainfall data

Lack of good quality rainfall data or the presence of missing data will have dire implications on the process of analysis and subsequently, lead to biased results of the analysis (Lebay and Le 2020). So, it is a must to assess for missing data. Rainfall data for this study were obtained specifically from the NMSA. However, the collected data have missing rainfall data for each station. When the normal annual precipitation of an index station differs by more than 10% from that of a precipitation station, the normal ration approach is applied (De Silva et al. 2007; Lebay and Le 2020). If normal precipitation varies a lot, P_x is calculated by weighting the precipitation at several stations by the normal annual precipitation ratios (De Silva et al. 2007). The normal ration method gives P_x as:

$$\mathrm{Px} = \frac{\mathrm{Nx}}{\mathrm{m}}\left[\frac{\mathrm{P1}}{\mathrm{N1}} + \frac{\mathrm{P2}}{\mathrm{N2}} + \cdots + \frac{\mathrm{Pm}}{\mathrm{Nm}}\right] \quad (1)$$

Where Px is estimated for the ungagged station; Pi is the Rainfall values of rain gauges used for estimation; Nx is the Normal annual precipitation of X station; Ni is the Normal annual precipitation of surrounding stations; m is the Number of surrounding stations.

2.4.2 Checking for data quality

2.4.2.1 Checking for outlier

Outliers are data points that depart significantly from the trend of the remaining data. The following formula is used to calculate the station skewness coefficient (Chow 2007).

The skewness coefficient is given by:

$$\mathrm{Cs} = \mathrm{n}\sum_{\mathrm{k}=0}^{\mathrm{n}} \frac{(\mathrm{Y}-\mathrm{Ym})^3}{(\mathrm{n}-1)(\mathrm{n}-2)*\mathrm{Sy}^3} \quad (2)$$

Where n is the sample size; Y is the log transform value; Ym is Log transformed mean; Sy is the Standard deviation of y.

The outliers of data were checked by the following frequency equations:

Low outliers

$$\mathrm{YL} = \mathrm{Ym} - \mathrm{Kn}^{*}\mathrm{Sy} \tag{3}$$

Where: Y_m is the mean of data in log unity; K_n is from the table for sample size N.

Higher outliers

$$\mathrm{YL} = \mathrm{Ym} + \mathrm{Kn}^{*}\mathrm{Sy} \tag{4}$$

Where Y_m is the mean of data in log unity; K_n is from the table for sample size N.

From the calculated result, the station skew (Cs) is -0.03902, which is below − 0.4. Therefore, it needs checking for lower outliers (Chow 2007; Assefa and Moges 2018). The lowest recorded daily heaviest rainfall data was 18.9 mm in the 1994 year, which is higher than the threshold value of lower outliers (16.41). Hence, the daily heaviest rainfall data recorded concerning lower outliers is within a reasonable range. So, all the available rainfall data can satisfy the condition or there is no rejection of data.

2.5 Rainfall frequency analysis

One purpose of frequency analysis is to use probability distributions to relate the magnitude of extreme occurrences to their frequency of recurrence (Alam et al. 2018).

2.5.1 Rainfall frequency analysis method

So, to analyze the maximum discharge expected in T years four of the commonly used frequency distribution functions were used for this study(Jacob et al. 2020; Caissie et al. 2022).

2.5.1.1. Normal distribution method

$$\mathrm{XT} = \mathrm{Xm} + \mathrm{K}_{T}{}^{*}\sigma_{n-1} \tag{5}$$

Where XT is Annual Maximum rainfall T years return Period; Xm is Mean rainfall data; KT is Frequency factor;σ_{n-1} are Standard deviations

$$KT = w - \frac{2.51557+0.01033w^2}{1+1.143279w+0.1992w^2+0.00131w^3} \quad (6)$$

$$w = \left(\ln\left(\frac{1}{p^2}\right)\right)^{0.5} \quad (7)$$

$$p = \frac{1}{p} \quad (8)$$

2.5.1.2. Lognormal distribution method

$$YT = Ym + K_T{*}Sy \quad (9)$$

Where, the variation of K_T =f (CS, T) is from the table. But, in the case of log-normal distribution the Cs = 0.

2.5.1.3. Gumbel method

$$XT = Xm + K_T{*}\sigma n - 1 \quad (10)$$

Where,

$$K_T = \frac{YT - Yn}{Sn} \quad (11)$$

YT = reduced variate, a function of T and it’s given by.

$$Y_T = -\left[\ln * \ln \frac{T}{T-1}\right] \quad (12)$$

Where Y_T is reduced mean in Gumbel’s extreme value distribution for N sample size; Sn is Reduced standard deviation in Gamble’s extreme value distribution for N sample size and; T is Return period; σn-1 is Standard deviation of annual rainfall, and Xm is Mean of all values annual rainfall.

2.5.1.4. Log Pearson type III distribution method

Y = log X is obtained, for this Y series for any recurrence interval T.

$$YT = Ym + K_T{*}Sy \quad (13)$$

Where K_T is the Frequency factor which is a function of T and Cs.

$$Sy = \sqrt{\frac{\sum (Y - Ym)^2}{(N-1)}} \quad (14)$$

Where Sy is the standard deviation of the Y verity sample,

$$Cs = \frac{N \sum (Y - Ym)^3}{(N-1)(N-2)Sy^3} \quad (15)$$

The variation of KT = f (CS, T) is from the table.

From the results obtained by these statistical analysis methods, the best fit was adopted for developing the IDF curve.

2.5.2 Goodness of fit

The reliability of the distributions is checked by the goodness of fit tests. The goodness of fit test analyzes whether, if a random sample fits a theoretical probability distribution function (ERA 2013). So, the goodness fit test enables us to determine the best-fit probability distribution (Basheer 2016; Alam et al. 2018) The selection of the best fit method is based on the ranks given by the three fitness methods.

2.6 Development of rainfall intensity-duration-frequency curve (IDF)

IDF curves are usually represented as a graph with rainfall duration (D) plotted on the horizontal axis, intensity (I) on the vertical axis, and a series of curves, one for each design return period (Gebreigziabher 2020). Because rainfall data of a shorter duration are unavailable, an appropriate IDF derivation for the shorter duration is required (ERA 2013), suggesting the following equation.

$$R_{Rt} = \frac{t}{24}\left(\frac{b+24}{b+t}\right)^n \quad (16)$$

Where, is Rainfall depth Ratio Rt: R24; Rt is Rainfall depth in a given duration, t; R24 is 24 hr. rainfall depth, b and n is coefficient b = 0.3 and n= (0.78–1.09) (ERA 2013).

The methods employed to develop the IDF curve for the shorter duration events using the above equations for this study, Gumbel distributions were selected and R24 was calculated for 2, 5, 10, 25,50, and 100 year return periods.

Rearranging the above equation:

$$R_t = \frac{t R_{24}}{24}\left(\frac{b+24}{b+t}\right)^n \quad (17)$$

Substituting Intensity (mm/hr.) in the Eq. (17).

$$I_t = \frac{R_t}{t} \quad \textbf{(18)}$$

$$I_t = \frac{R_{24}}{24}\left(\frac{b+24}{b+t}\right)^n \quad (19)$$

Using b = 0.3 and n = 0.92 as suggested by (ERA 2013) and results are tabulated for rainfall durations 5, 10, 15, 20, 25, 30 up to 180 minutes within 5 minutes interval.

The maximum peak flood computed using the return period recommended in (ERA 2013; Greening 2014) considering the road standard and the design life span of the structure, ten years for the design return period were adopted.

2.7 Computation of peak runoff

Hydraulic and drainage engineers frequently use the rational method to estimate design discharges, which are used to size a variety of drainage structures for small urban (development) and rural (undeveloped) watersheds (AlSubih et al. 2021). The Rational Method is most accurate for estimating design storm peak runoff for small urban and rural watershed areas up to 50 hectares (ERA 2013; Amatya et al. 2021)

2.7.1 Rational method

The rational method is widely used in engineering practice to estimate peak flood discharge for the design of hydraulic structures in small watersheds (Young et al. 2009). The rational formula is expressed as:

$$Q = 0.00278 C_f C I A \quad (20)$$

Where, Q is the maximum rate of runoff, m3/s; C is runoff coefficient representing a ratio of runoff to rainfall; C_f: is frequency factor; I is average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr.; A is a catchment area tributary to the design location, ha.

Lastly, peak discharge is calculated for each sub-catchment for the ten-year return period using the Eq. (20). The parameters used for the rational method are as follows.

A. Catchment area (A)

Once the delineation of the catchment was done from DEM and shape file of town, its catchment area was determined from the topographic map with help of google earth. Therefore, the study area is divided into 40 sub-catchments. The catchment is divided based on the drainage line and the outlet that is more appropriate for SWMM software.

B. Runoff coefficient (C)

Weighted average runoff coefficient values for each catchment area were determined under the rational method as follows(Young et al. 2009; Pale et al. 2018).

$$C_{weighted} = \frac{\sum(A_i * C_i)}{A_T} \tag{21}$$

Where: Ci is the runoff coefficient for a given hydrologic soil group area; Ai is the area under each hydrologic soil group and A_T is the total catchment area considered.

C. Design rainfall intensity (I)

Once the design return period and duration are determined, the design rainfall intensity can be determined from the developed intensity duration-frequency curve for the location of the drainage area (AlSubih et al. 2021).

D. Time of concentration

Time of concentration consists of two components: the first one is the time for the surface flow to reach the first inlet (To), and the second one is that, time to flow through the storm drainage system to the point of consideration (Huber et al. 1975; U.S. Environmental Protection Agency (EPA) 2015) To determine the time of concentration, the following formulas that are widely used were selected for this study.

i. Time of surface flow (To)

The formula to calculate the time of a surface flow is given as follows (U.S. Environmental Protection Agency (EPA) 2015).

$$T_o = \frac{0.218*(1.1-C)L^{0.5}}{S^{0.333}} \tag{22}$$

Where, To is Time of Surface flow (in minutes); C is Rational Method Runoff Coefficient; L is the length of surface flow (m) and S is Surface slope (in ratio)

ii. Time of flow in a conduit (Tf)

Time of flow in a conduit or channel flow time can be estimated from the hydraulic properties of the conduit or channel using Manning’s equation (Chen and Goldscheider 2014; U.S. Environmental Protection Agency (EPA) 2015).

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \quad (23)$$

Where, V is the mean velocity of flow, m/s; n is Manning's roughness coefficient; R is the hydraulic radius (m) and S is the slope of the hydraulic grade line, decimal.

Once the velocity of flow is computed from manning's equation, the time of flow in a conduit is calculated from the Eq. (24).

$$\mathrm{T_f} = \frac{\mathrm{L}}{\mathrm{V}}_{\mathrm{Conduit(canal)}} \quad (24)$$

Where, Tf is time for flow in a canal, in minutes; L is the length of the canal, m; V is the velocity of flow through the canal, m/s.

Finally, after both times, that means time for surface flow and time of flow in a canal calculated, the total time of concentration is the summation of both times. That mean,

$$T_C = T_O + T_f \quad (25)$$

Where, Tc is Total time of concentration, in minutes; T_o is time for surface flow, in minutes and Tf is Time for flow in canal, in minutes

Results And Discussion

3.1 Rainfall Frequency Analysis

In this study, the rainfall frequency analysis was determined by four commonly used probability distributions of hydrology. The chosen methods were Normal, Log-Normal, Extreme Value Type-I (Gumbel), and Log Pearson Type III. However, the ERA manual recommended Gumbel and log person type III for rainfall frequency analysis. The summary of Extreme rainfall for all methods for different return periods is as tabulated in Table 1.

To identify which distribution fits the theoretical probability distribution, the GOF test was conducted using Easy Fit 5.6 professional software. As a result, the Gumbel distribution fits better than the rest of the method. In this study, in addition to the Easy-fit software, the correlation coefficient was used to identify the best fit distribution. So, Gumbel has a better R^2 than the other methods which is 0.998.

3.2 Intensity-duration-frequency curve (IDF)

The IDF curve was developed from a 24-hour rainfall data of 32 years duration that is from 1987 to the 2018 year, which was obtained from the Ethiopian Meteorological Agency were the gauge station located at Ambo Agricultural center in Ambo town. Since the shorter rainfall data is not available, the Reduction equation was used to calculate the shorter rainfall data needed for the development of the IDF Curve. The rainfall intensity was calculated for 5minutes intervals up to 180 minutes, and its result was shown in the tables 2. From the rainfall intensity result table 2, the Intensity duration frequency curve was developed for the Ambo station as shown in figure 2.

3.3 Peak discharge determination using the rational method

Since all sub-catchment areas of this study are less than 50ha it's recommended to use rational methods to determine the peak runoff from the sub-catchment. The weighted runoff coefficients were determined from the land-use composition for each sub-catchment using the runoff coefficient for each land use as discussed in the methodology part. Time of concentration consists of inlet time plus time of flow in a channel as discussed under the methodology part of this study and its result as shown in a table for each sub-catchment. So, using the above input parameters, the peak discharge from the catchment was determined using the rational method as shown in table 3.

3.4 Stormwater management model simulation result

The performance of the stormwater drainage network of the study area was assessed using the stormwater management model. To assess the performance of the stormwater drainage system of the town, the software needs input data for modeling runoff quantity and others. The details of input data used for physical catchment were collected using field surveying and other adopted from different hydrologic kinds of literature. As well as the detail of conduits properties and Junction properties were measured at the field. The maximum, minimum infiltration rate and decay constant values were adopted from (Gilliom et al. 2019; Liu et al. 2022) depending on the study area's soil type.

Finally, after the input parameters of the model were collected, processed, and inserted, the model simulates successfully. Then, once the SWMM is successfully simulated, the simulated result were discussed in the following.

3.4.1 Drainage network mapping

The total study area is divided into 40 sub-catchments and the network consists of 73 nodes or junctions, 8 outfalls, 75 conduits links, and one rain gage station. For this study the sub-catchments area is denoted as Sc, the nodes as J, the conduit as C, and the rain gage denoted as RG. The sub-catchment was classified depending on the area. As seen from the modeling map, most of the sub-catchment were less than 25ha and only two sub-catchments were greater than 25ha. The node in the modeling map was classified depending on the inverted elevation. Since invert elevation for the whole junction is greater than 100, it is displayed in red color. Again the link was classified depending on the link depth. From the modeling map, the majority of link depth ranges from 0.5 to 1m, whereas six links were less than 0.5m.

The total model map of the study area includes sub-catchments, nodes, and the link was shown in figure 3.

3.5 Calibration and validation of the model

Modeling study typically requires calibration and validation to evaluate model performance. For SWMM, some model parameters are directly measured and their value was fixed through onsite investigation. Others which are the only conceptual representation of catchment features necessarily be determined through a trial and error process until a satisfactory matching is obtained between the model output and measured data(Perin et al. 2020). Since there is no measured data in the study area, the calculated runoff using the rational methods was used as measured data for fixing the sensitive parameters of the model, and Sensitive parameters value range for SWMM hydrology and hydraulic parameters was given in table 4 according(Akdoğan and Güven 2016).

3.5.1 Model performance evaluation

The performance of SWMM was evaluated using runoff calculated using the rational method and the simulated result of the software. According to the calculated and simulated value of discharge for a 10year return period were R_{NS} value is 0.997 and R^2 is 0.997. Since the R_{NS} and R^2 are 0.99 it shows that the model gives a good result.

3.5.2 Comparison of discharge results

A comparison was done between the discharge determined by rational methods for each sub-catchment and the discharge from the SWMM or the simulated results.

From figure 4; the correlation between both results is 0.998 which implies that the simulated result with SWMM well matches the rational method. So, the calibration and validation of hydrological parameters gave an excellent result.

3.7 Water depth and flow in the canal

The water depth and flow in a canal were also simulated to assess the adequacy of the canal. The water depth simulation results for a canal were presented as follows.

3.7.1 Water elevation profile toward outfall 1

Figure 5; shows that the water profile in canals 3 and 4 is the design water depth. That means canals 3 and 4 are sufficient to carry the generated runoff. So there are no flooding injunctions 2, 3, and 6.

3.7.2 Water elevation profile toward outfall 2

From the simulation result, figure 6; shows that the water profile at junction 5 is insufficient to carry the generated runoff from the sub-catchment and there is a flood at junction 5 and outfall 2.

3.7.3 Water elevation profile toward outfall 3

As the simulation result indicates, the drainage canal toward outfall 3 means junction 9 to outlet 3 is sufficient to carry the generated runoff. So, there is no flooding in canal 9 as shown in figure 7.

3.7.4 Water elevation profile toward outfall 4

As shown in figure 8 the drainage canal toward outfall 4 means the junction 10, 11, 19 to outlet 4 is sufficient to carry the generated runoff. So, there is no flooding in canals 10, 11, 19, and 23.

3.7.5 Water elevation profile toward outfall 5

From the simulation result of SWMM, the majority of the canal that dispose of stormwater toward the outfall 5 is surcharged. As shown in figure 9, the junctions 14, 15, 16, 17, 24, 30, 32, and outfall 5 are highly flooded, whereas the rest of the junctions are sufficient to carry the generated runoff from the sub-catchment.

3.7.6 Water elevation profile toward outfall 6

As the simulation result indicates, the flow level in drainage canal 45 is highly flooded. As shown in figure 10, both junctions 38 and 43 are insufficient. Again from the simulation, Junction 45, 46, 50, and 51 are sufficient to safely dispose of the generated runoff from the sub-catchment

3.7.7 Water elevation profile toward outfall 7

From the simulation result as shown in figure 11, the drainage canal toward outfall 7 means that junction 57, 60, and 69 is insufficient to carry the generated runoff. So, there is flooding in canals 58 and 70. Except for the junction listed above, it's sufficient or there is no surcharge in the canal.

3.7.8 Water elevation profile toward outfall 8

Figure 12, shows the water flow depth in the canal toward the outfall 8. As can be from the figure below that, junctions 73 and 72 which connect to canal 73 are highly flooding due to insufficient canal depth.

But, both canals 74 and 75 are sufficient to carry the generated runoff. So there are no flooding injunctions 71 and 79.

Conclusion

Drainage problems are noticed in the study area (Ambo Town) as a cause of flooding. So, to safely dispose of surface runoff, it's necessary to assess the performance of the existing stormwater drainage system of the town. Therefore, the main objective of this study is to assess the performance of the stormwater drainage system of Ambo town. The SWM model has been successfully used to estimate the amount of runoff from each sub-catchment, water depth at the junction, and in the link or canal. Since the study area is large and to effectively determine the runoff from the catchment, the total catchment was divided into 40 sub-catchments and it consists of 73 junctions, 8 outfalls, 75 conduits, and one rain gauge. From the simulation results of SWMM, the total runoff from the whole catchment is $60.87m^3/s$ and the result using the rational method is $60.32m^3/s$. The result of total runoff from SWMM is greater than the result obtained using the rational method. So, the difference in the result implies that the SWMM parameters need more calibration for a more reliable result.

The simulation results indicate that 22% of junctions are insufficient to accommodate the maximum flooding that occurred. Since the design water level in most of the junctions is less than the flooding level, there is flooding at the junction and it causes flooding. In addition to that, 21% of the canals are surcharged and it causes an overflow in the canal. From this, it's possible to conclude that some junction and canal dimensions are not sufficient to drain the generated runoff from the sub-catchments. From the field surveying and the simulation results, the existing stormwater drainage system of Ambo town generally can be considered inadequate both in quality as well as in coverage.

Declarations

Declaration of Competing Interest

The authors had participated in this research and institutions have no complaint of interest that could have appeared to influence the work reported in this paper.

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Author Contribution Statement

All authors contributed to the study through material preparation, data collection, and analysis.

Walabuma Oli Emama: Conceptualization, Methodology, and Writing-original draft preparation

Haile Dida Edae: Conceptualization, Software, Data curation, and visualization

Tolera Abdisa Fayisa(Assistance Professor): Supervision and Software Validation.

Deme Betele Hirko: Writing, Reviewing and Editing and Software Validation

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Data Availability

All data generated, or analyzed during this study are included in this manuscript. The raw data that support the findings of this study were collected from the Ministry of Water, Irrigation, and Energy, the National Metrology Service Agency(NMSA), and the Ethiopian Map Agency. The authors have no authority to openly distribute those data.

Ethical Approval

Not applicable

Consent to participate

Not applicable

Consent to publish

Not applicable

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Tables

Table1.Extreme rainfall depth results for 2, 5, 10, 25, 50, and 100 years return period.

Return period	Extreme rainfall depth (mm)			
	Gumbel	Log person type III	Log normal	Normal
2	39.15	38.99	39.12	44.28
5	53.31	45.76	45.97	56.75
10	62.69	59.70	60.13	63.13
25	74.53	69.67	70.36	69.84
50	83.32	76.91	77.89	74.13
100	92.04	84.04	85.32	77.97

Table 2. Duration of rainfall Vs their corresponding Intensity

	Intensity (mm/hr.)					
Duration (t) in Minutes	T=2	T=5	T=10	T=25	T=50	T=100
	39.15	53.31	62.69	74.53	83.32	92.04
5	74.2	101.0	118.8	141.3	157.9	174.4
10	61.9	84.3	99.1	117.9	131.8	145.6
15	53.2	72.5	85.2	101.3	113.3	125.1
20	46.8	63.7	74.9	89.0	99.5	109.9
25	41.7	56.8	66.8	79.4	88.8	98.1
30	37.7	51.3	60.4	71.8	80.2	88.7
35	34.4	46.9	55.1	65.5	73.3	80.9
40	31.7	43.1	50.7	60.3	67.4	74.5
45	29.4	40.0	47.0	55.9	62.5	69.0
50	27.4	37.3	43.8	52.1	58.2	64.3
55	25.6	34.9	41.1	48.8	54.6	60.3
60	24.1	32.8	38.6	45.9	51.3	56.7
65	22.8	31.0	36.5	43.4	48.5	53.6
70	21.6	29.4	34.6	41.1	45.9	50.8
75	20.5	27.9	32.9	39.1	43.7	48.2
80	19.6	26.6	31.3	37.2	41.6	46.0
85	18.7	25.4	29.9	35.6	39.8	43.9
90	17.9	24.4	28.6	34.0	38.1	42.0
95	17.2	23.4	27.5	32.7	36.5	40.3
100	16.5	22.4	26.4	31.4	35.1	38.8
105	15.9	21.6	25.4	30.2	33.8	37.3
110	15.3	20.8	24.5	29.1	32.5	36.0
115	14.8	20.1	23.6	28.1	31.4	34.7
120	14.3	19.4	22.9	27.2	30.4	33.6
125	13.8	18.8	22.1	26.3	29.4	32.5
130	13.4	18.2	21.4	25.5	28.5	31.5

135	13.0	17.7	20.8	24.7	27.6	30.5
140	12.6	17.2	20.2	24.0	26.8	29.6
145	12.2	16.7	19.6	23.3	26.1	28.8
150	11.9	16.2	19.1	22.7	25.3	28.0
155	11.6	15.8	18.6	22.1	24.7	27.3
160	11.3	15.4	18.1	21.5	24.0	26.5
165	11.0	15.0	17.6	21.0	23.4	25.9
170	10.7	14.6	17.2	20.4	22.9	25.2
175	10.5	14.3	16.8	20.0	22.3	24.6
180	10.2	13.9	16.4	19.5	21.8	24.1

Table 3.Discharge computed for different return period

SC	Area (ha)	C Weighted	Tc (min)	I=10 mm/hr.	I=25 mm/hr.	Q=10 m^3/s	Q=25 m^3/s
SC 1	2.90	0.72	12.9	90.5	97.7	0.53	0.62
SC 2	15.40	0.59	13.7	88.3	105.0	2.25	2.94
SC 3	6.50	0.68	17.0	80.7	98.3	0.99	1.33
SC 4	11.80	0.62	24.0	68.3	81.2	1.39	1.82
SC 5	6.39	0.65	6.7	111.2	132.2	1.28	1.67
SC 6	28.30	0.62	12.5	91.7	109.0	4.47	5.85
SC 7	0.59	0.90	10.1	98.7	117.4	0.15	0.19
SC 8	6.21	0.66	9.4	101.1	120.2	1.16	1.51
SC 9	6.80	0.59	5.3	117.6	139.8	1.31	1.71
SC 10	4.14	0.75	9.5	100.7	119.8	0.87	1.14
SC 11	20.39	0.62	18.0	78.7	93.6	2.75	3.59
SC 12	3.49	0.65	9.0	102.6	121.9	0.65	0.85
SC 20	7.82	0.78	15.9	83.2	98.9	1.41	1.84
SC 13	1.96	0.63	4.2	122.5	145.7	0.42	0.55
SC 14	5.77	0.62	18.1	78.5	93.3	0.77	1.01
SC 15	2.59	0.75	8.7	103.4	123.0	0.56	0.73
SC 16	3.07	0.75	6.5	111.9	133.1	0.72	0.94
SC 17	1.74	0.64	3.5	126.6	150.5	0.39	0.51
SC 18	4.23	0.62	9.9	99.4	118.2	0.72	0.95
SC 19	6.41	0.63	7.3	108.7	129.2	1.21	1.59
SC 21	4.90	0.65	11.7	93.8	111.5	0.84	1.09
SC	4.90	0.66	9.8	99.9	118.8	0.89	1.17

22							
SC 23	4.84	0.67	20.3	74.3	88.3	0.67	0.87
SC24	19.00	0.62	13.1	90.1	107.1	2.94	3.85
SC26	7.24	0.68	13.9	87.9	104.5	1.20	1.57
SC27	3.37	0.78	4.4	121.8	144.8	0.89	1.16
SC25	4.70	0.64	10.2	98.4	117.0	0.82	1.08
SC28	5.63	0.80	12.0	93.1	110.7	1.16	1.52
SC 29	7.37	0.70	13.3	89.6	106.5	1.28	1.68
SC 30	21.03	0.63	14.7	85.8	102.1	3.15	4.12
SC 31	8.11	0.74	9.8	99.8	118.6	1.66	2.17
SC 32	13.36	0.60	20.8	73.4	87.3	1.65	2.15
SC 33	36.40	0.56	23.1	69.6	82.8	3.96	5.18
SC 34	19.70	0.63	12.2	92.5	110.0	3.19	4.17
SC 35	7.50	0.65	8.7	103.6	123.1	1.40	1.83
SC 36	7.68	0.65	9.8	99.7	118.5	1.38	1.80
SC 37	9.95	0.70	19.8	75.3	90.9	1.46	1.94
SC 38	19.34	0.64	23.1	69.7	82.9	2.40	3.14
SC 39	10.88	0.64	13.3	89.4	106.3	1.74	2.28
SC 40	21.04	0.66	11.6	94.1	111.9	3.65	4.77

Table 4. The sensitive parameters value range for SWMM hydrology and hydraulic parameters.

Sensitive parameters for calibration from kinds of literature				
Name of Parameters	Meaning	Value range	Initial Value	Used Values
N-Impart	Manning’s roughness coefficient for Impervious area	0.011-0.15	0.014	0.012
N-pervious	Manning’s roughness coefficient for pervious area	0.05-0.8	0.2	0.171
Dstore-Impervious	Depth of depression storage on the impervious area (mm)	0-3	1.5	1.3
Dstore-pervious	Depth of depression storage on the pervious area (mm)	3-10	3	3.1
Conduit Roughness	Manning’s roughness coefficient for conduit	0.011-0.024	0.02	0.015

Figures

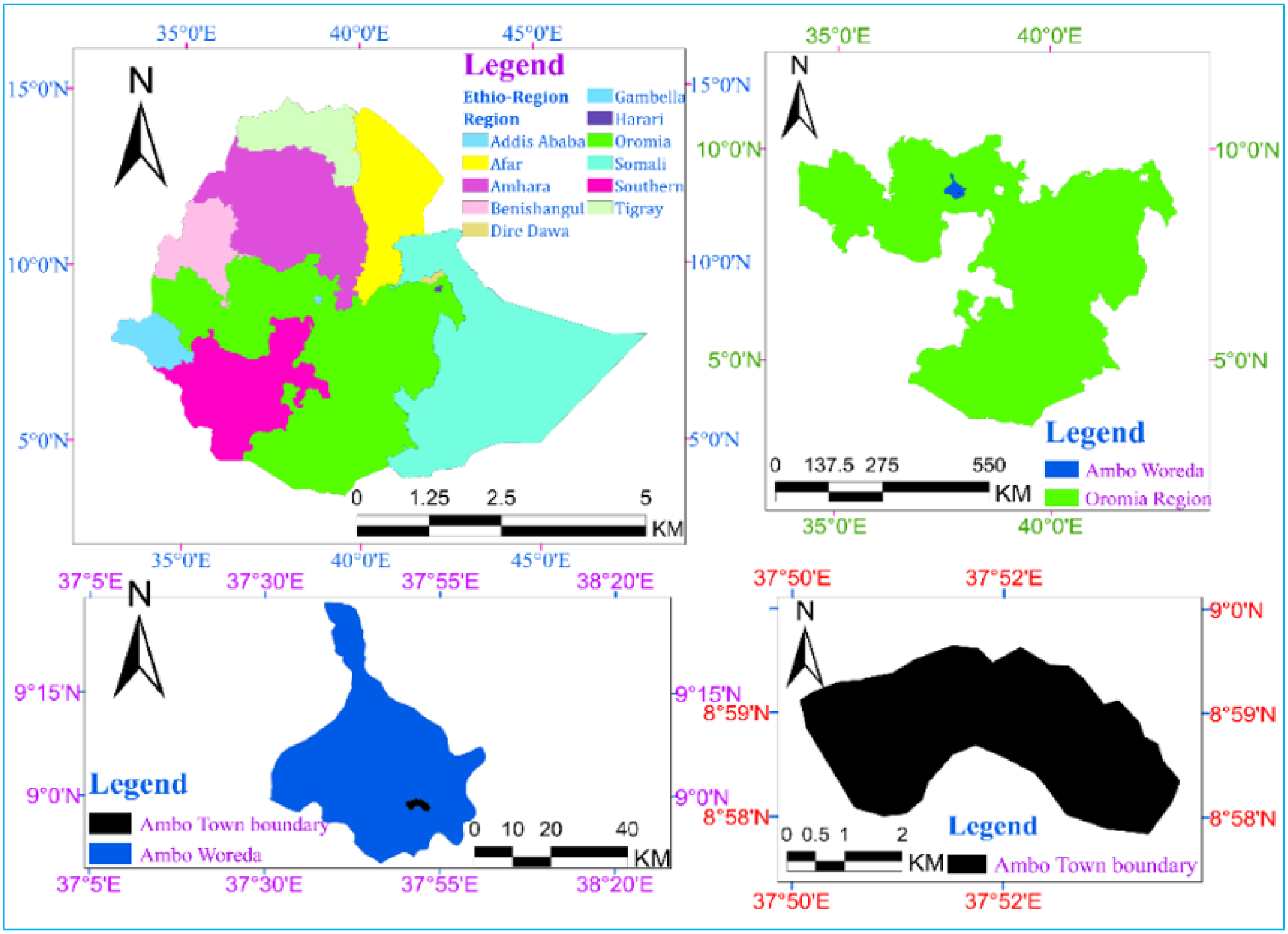


Figure 1

Location map of the Study Area

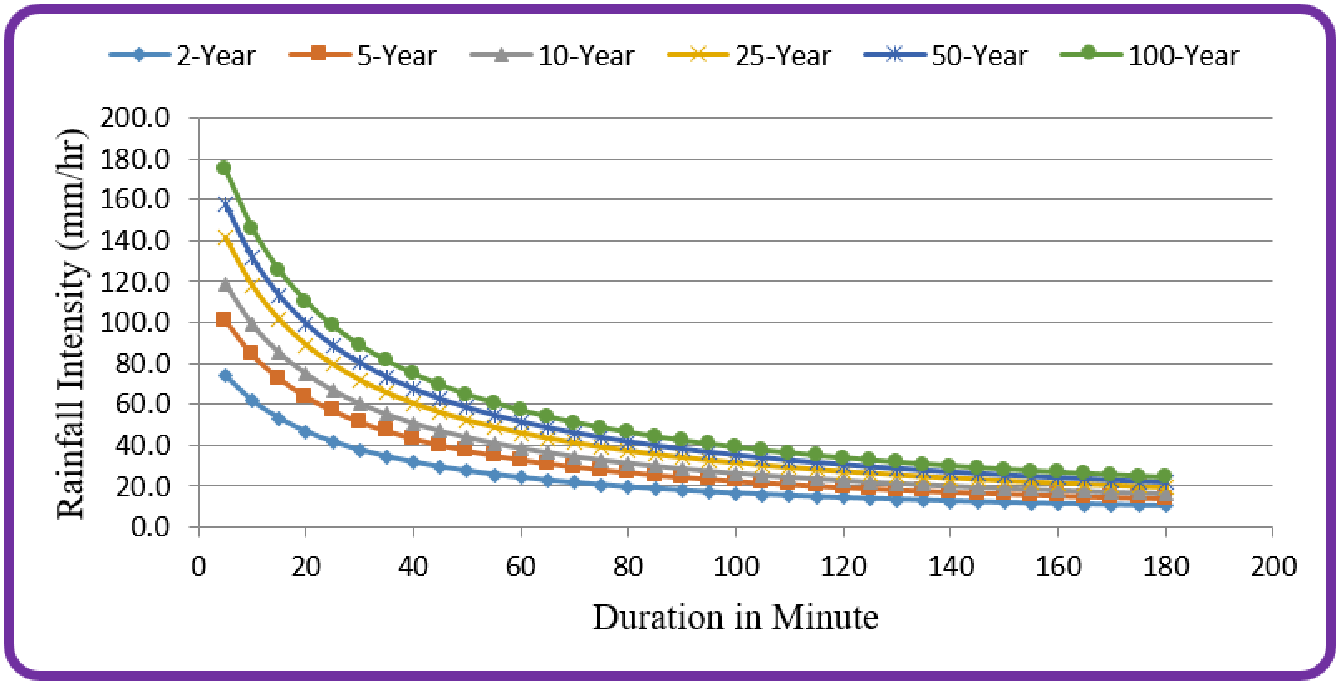


Figure 2

IDF curve developed of Ambo Town

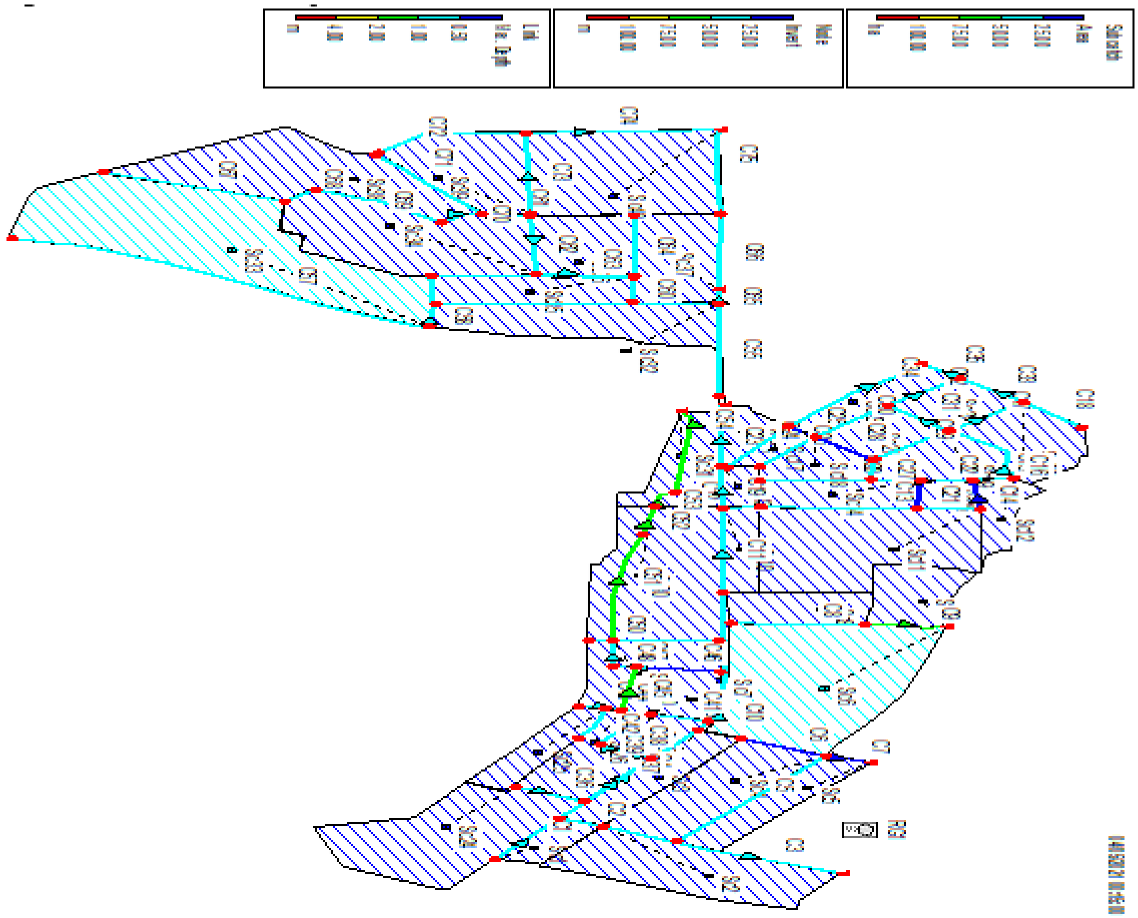


Figure 3

Sketch of model map

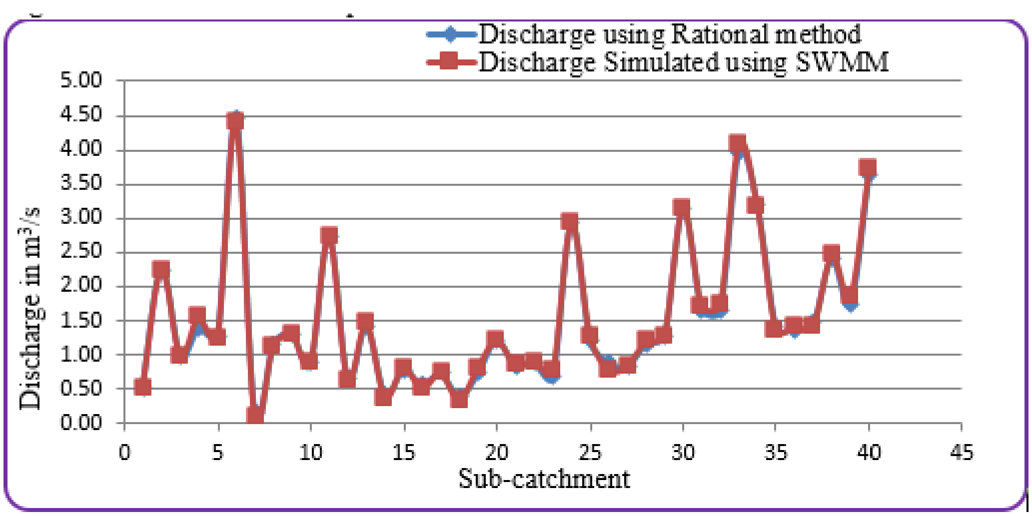


Figure 4

Comparison between the calculated and simulated result

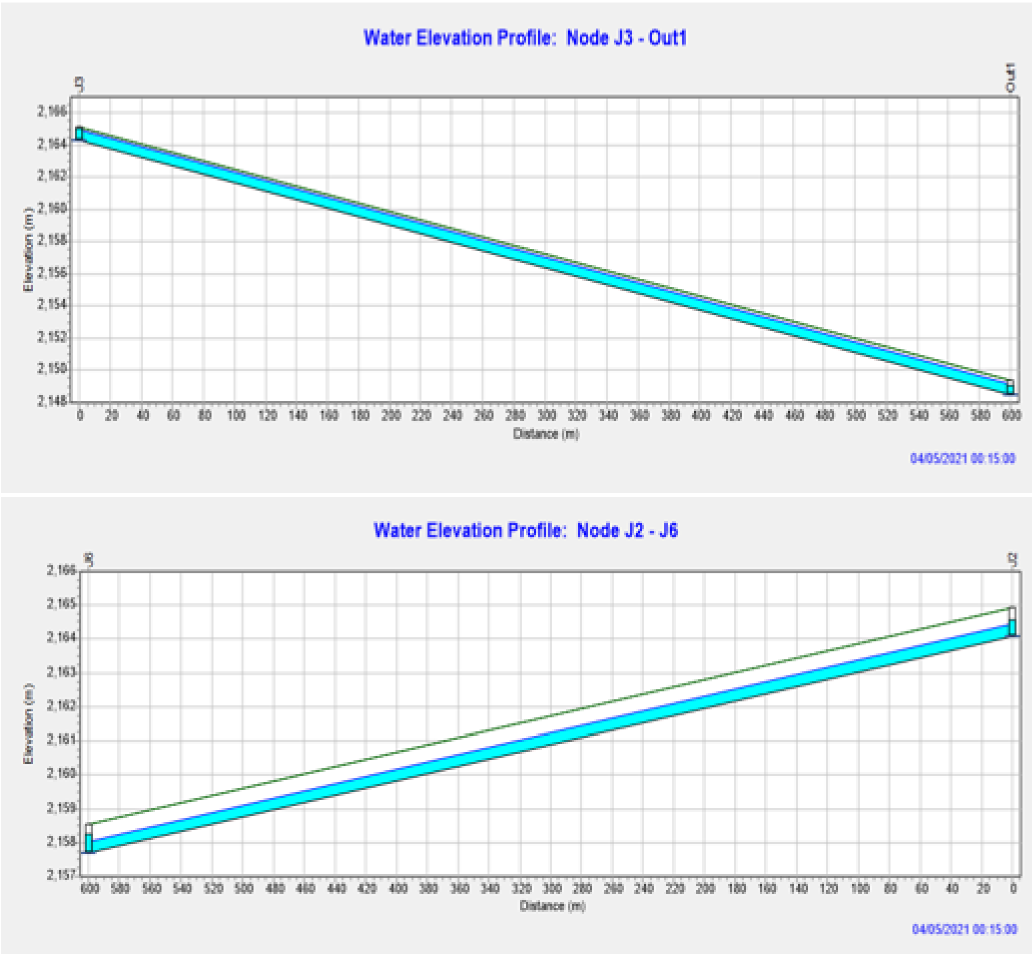


Figure 5

Water depths toward outfall 1

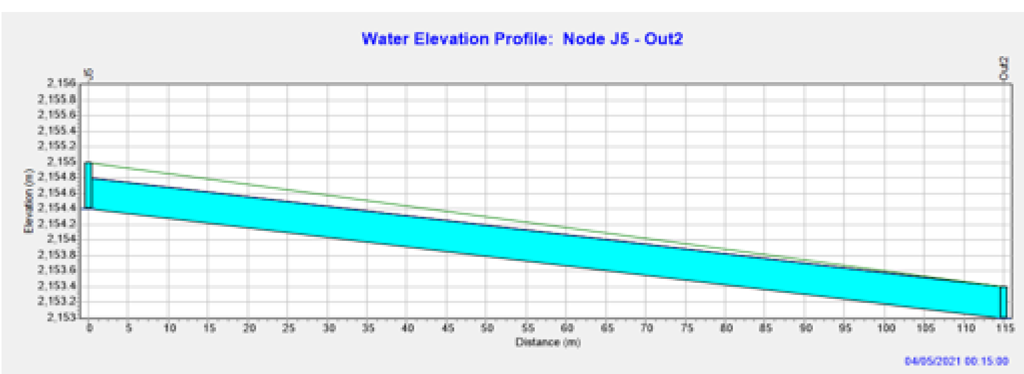


Figure 6

Water depths toward outfall 2

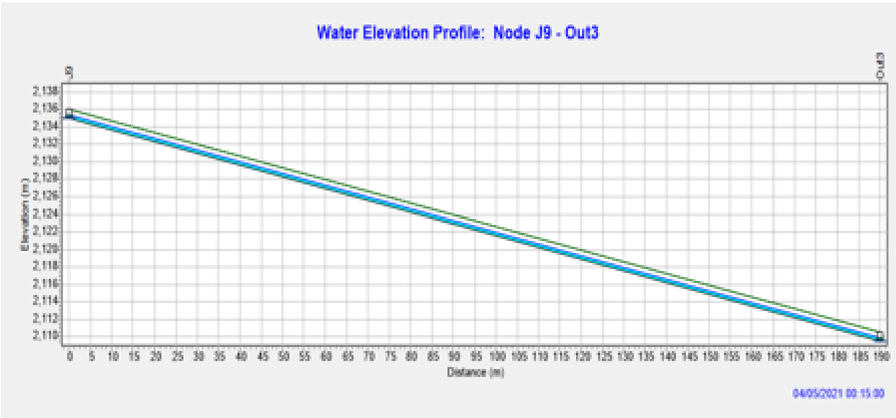


Figure 7

Water depths toward outfall 3

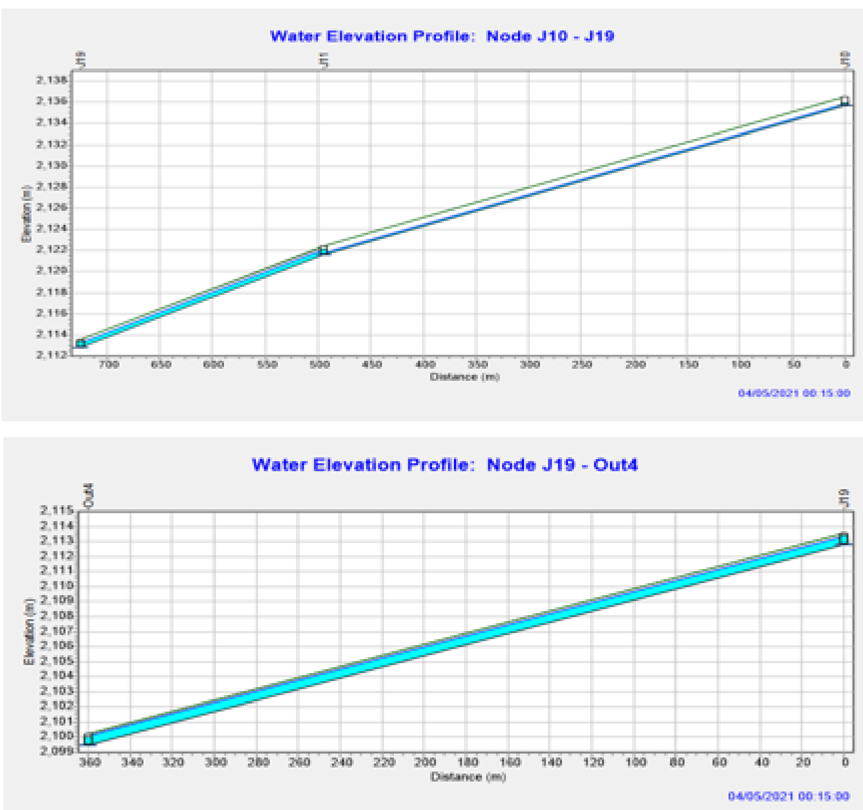
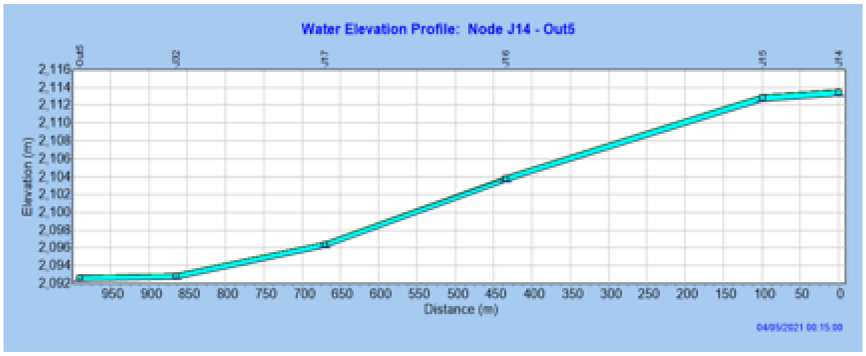


Figure 8

Water depths toward outfall 4



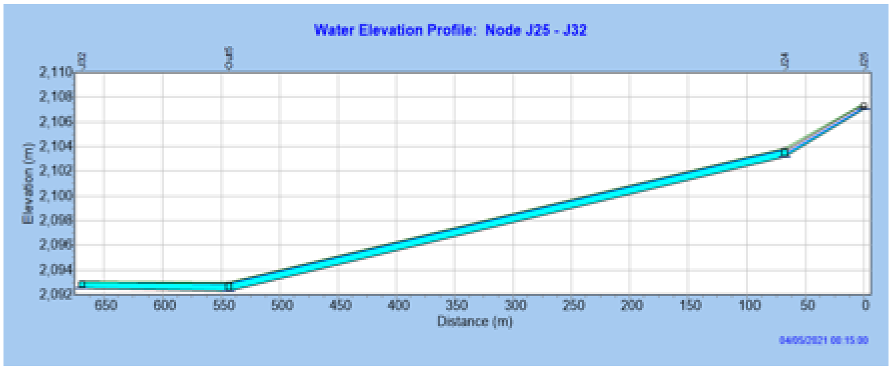
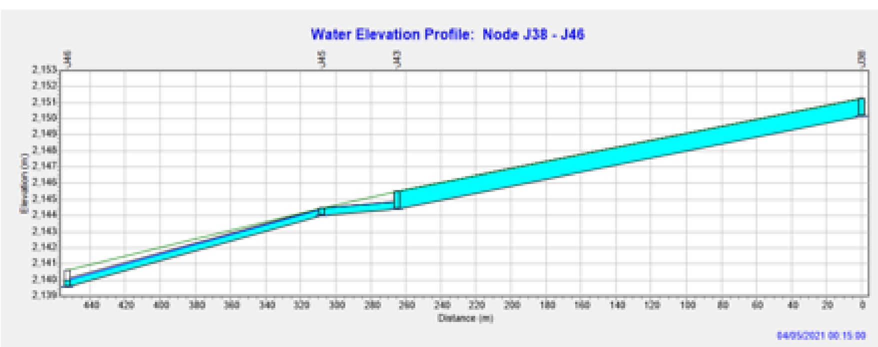


Figure 9

Water depths toward outfall 5



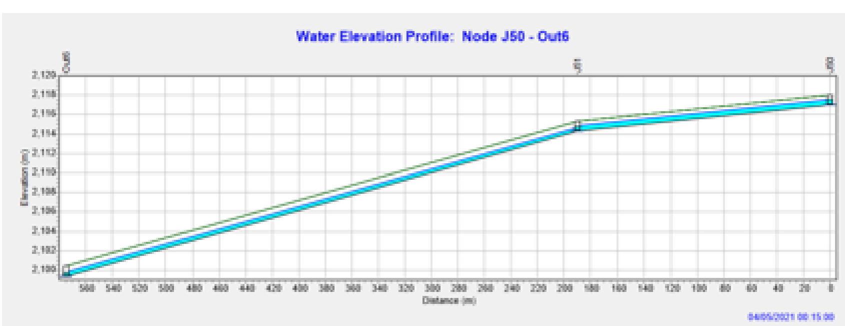
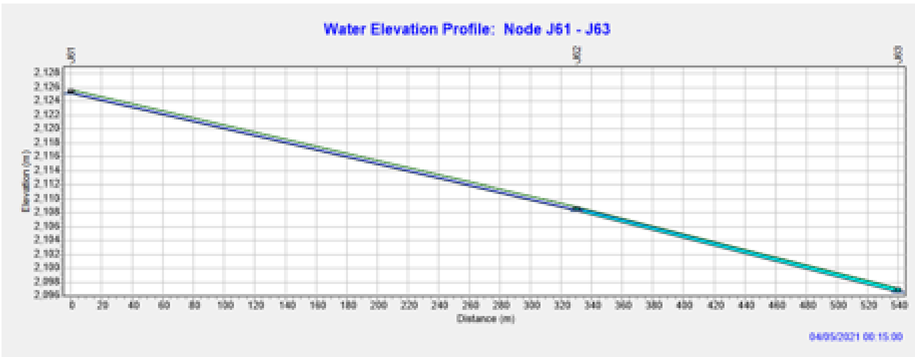


Figure 10

Water depths toward outfall 6



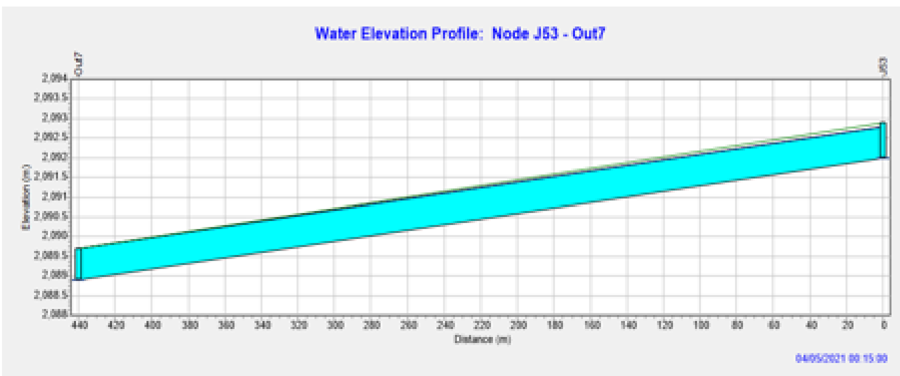


Figure 11

Water depths toward outfall 7

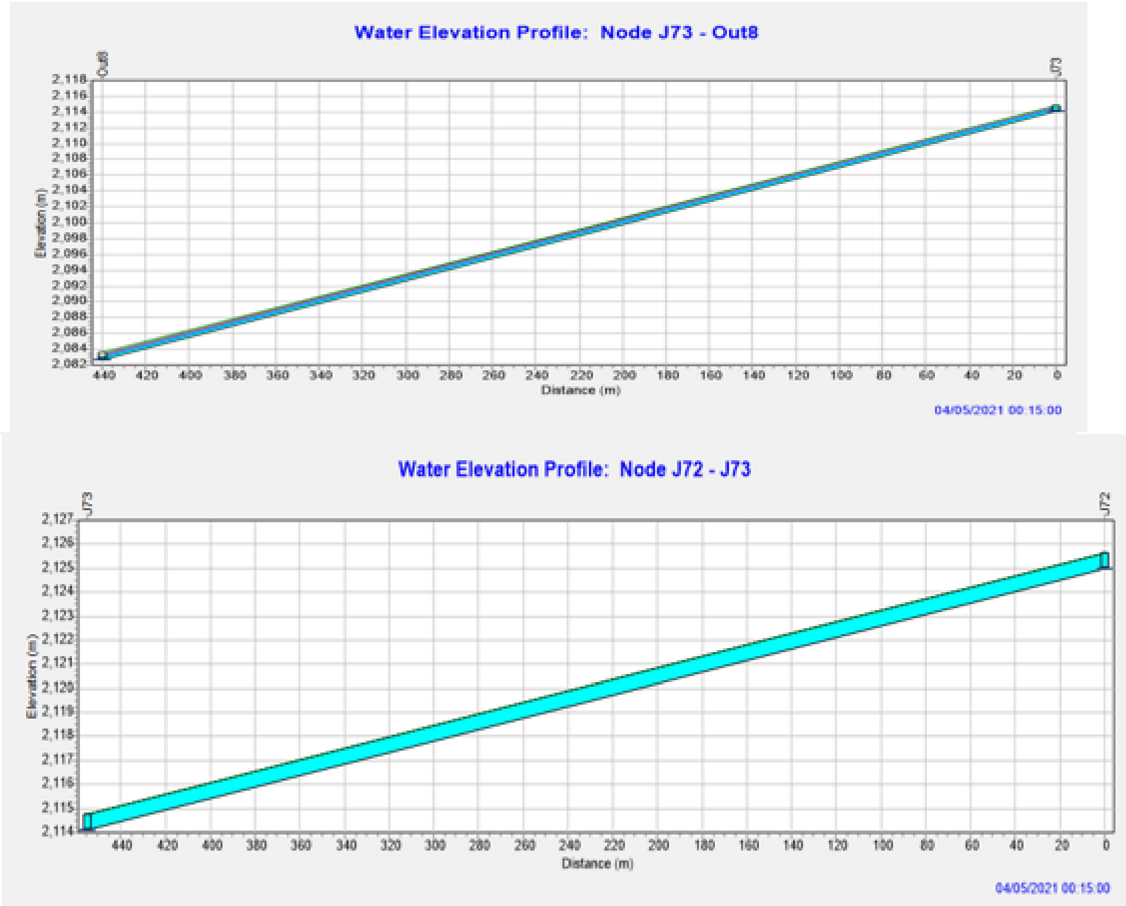


Figure 12

Water depths toward outfall 8