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Evaluation of Urban Drainage System by SWMM : A Case Study At Alaba Kulito Town, SNNPRS, Ethiopia

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ABSTRACT: *The general objective of the study is Evaluation of Urban Drainage System of Alaba Kulito Town by Storm Water Management Model (SWMM). The study employed the collection of primary data like measuring the existing drainage size of the flood-prone region and asking stakeholders and secondary data which was obtained from National Meteorology Agency (NMA) of Ethiopian (33 years meteorological data), Ethiopian Map Agency (topographic map and soil map data), Ministry of Water, Irrigation and Energy (soil data) and Alaba Town Municipality (historical data of flood and organizational structure of city administration). 3.7% of Missed RF data of Alaba Kulito station was filled by Normal Ratio Method (NRM). The Generalized Pareto Distribution (GPD) method which was followed by Peak Over Threshold (POT) extreme value determination method was used to predict the probability of flood occurrence due to the best fit and approach of study. For analysis of hydrology and hydraulics done by the Soil Conservation Service Curve Number (SCS-CN)/rational method and Stormwater Management Model (SWMM) are used. It has been found that Alaba-Kulito town is geographically nearly plain which was between class 1 and class 4 slope classification and located in foothill which contributes much amount of runoff to the town and some of the drainage lines are incapable to convey runoff generated from rural catchment. In addition to this, limited landscape based mitigation strategies in the study area with insufficiency of drainage canals, limited collector and feeder drainage lines, lack of awareness of community while disposing of household wastes together have worsened the impacts of flooding. The overall result of the study is terminated by distinguishing and pointing both structural methods: diverting the upper catchment(which shares more than 64% runoff load), providing collector and feeder drainage lines through the flood-prone section of town and constructing a lined canal at the common outlet to Bilate river which is about 1.5km from ST.Gabriel church ; and non-structural managing systems depending on the degree of the flood. The peak runoff load of each junction and nodes are obtained by summing up the runoff magnitude of all upper contributing catchments and accordingly the outlet point received about 49.45*m^3/s *and 29.1*m^3/s *without divertion work and if diversion work was provided for 10 years return period respectively. The coefficient of correlation between simulated and estimated peak discharge becomes greater than 0.99.*

Keywords: *Alaba Kulito Town, Urban Drainage, POT, GPD, IDF, SWMM*

1 INTRODUCTION

Natural hazards have caused considerable danger to the progress and development of the human population on the surface of the earth. Some of the natural hazards include volcanic eruption, earthquake, temperature extremes, hurricane, tropical cyclones and flooding. Among the natural hazards, floods hold the leading position in the world (Birehanu, 2018) and in practice, flooding occurs when the volume of water in a waterway exceeds the capacity of the channel. Biplab and Aditya, (2012); defined flooding as a general and temporary condition of partial or complete inundation of normally dry land areas from the usual and rapid runoff of surface waters which may result from rainfall, rivers, ice melt and so on.

Commonly; flooding is caused by rainfall intensity, duration, soil condition, nature of the topography, ground cover, antecedent moisture condition, climate change and other natural and manmade factors. Construction of buildings and roads without providing adequate side canals, main canals and appropriate outlets in the urban area leads to urban flooding as urbanization results in considerable changes in hydrological processes due to an increase in built-up and decrease in infiltration.

The Global Facility for Disaster Reduction and Recovery, (2019); reports that Ethiopia is one of the African countries which are experiencing flooding which occurs at irregular intervals and varies in duration, magnitude.

Studies reveal that the increase in artificial surfaces due to urbanization increases flooding frequency and intensity because of the resulting poor infiltration and reduction in flow resistance and it is an accelerating trend throughout the world (Zhang et al., 2007). Likewise, the portion of permeable land of Alaba Kulito Town is changing to impermeable surfaces due to the increasing of population number and built-up which aggravates the occurrence of urban flooding with the combined effect of hydrologic condition and hydraulics of existing drainage system.

During intense rainfall of Summer season (Juley to September), flooding is suspected at the town section as the town is geographical located between Rekame Hill and Bilate River which has been historically affecting the town resulting damage to properties and inundation of stormwater on flat areas malfunctions the road and reduce the aesthetic view of the town since the establishment of the town. Hence, the climatic change and expansion of urbanization are not able to be avoided, we need to control nature and to shape the environment we are living in.

The main objective of this research is to assess the flood hazard and drainage system of Alaba Kulito Town using Stormwater Management Model (SWMM) by developing IDF curve, identifying flooding lines and flood casue with proposing appropriate flood hazard mitigation methods.

2 MATERIAL AND METHODS

2.1 Description of the Study Area

- **Location**

Alaba Kulito (also known as Halaba Kulito, or Kuliito) is found between Shashamane and Wolayta Sodo Town and crossed by two cross country main federal asphalt roads at a distance of 313 km and 243 km from Addis Ababa, via Shashamane and Butajira/Hulbareg/Sankura/ towns. Its absolute location is between 7°17′19″ and 7°19′25″ N of latitude and 38°4′10″ and 38°6′17″ E of longitude. The town sits on the left bank of the Bilate River, with an elevation of 1726 meters above the mean sea level.

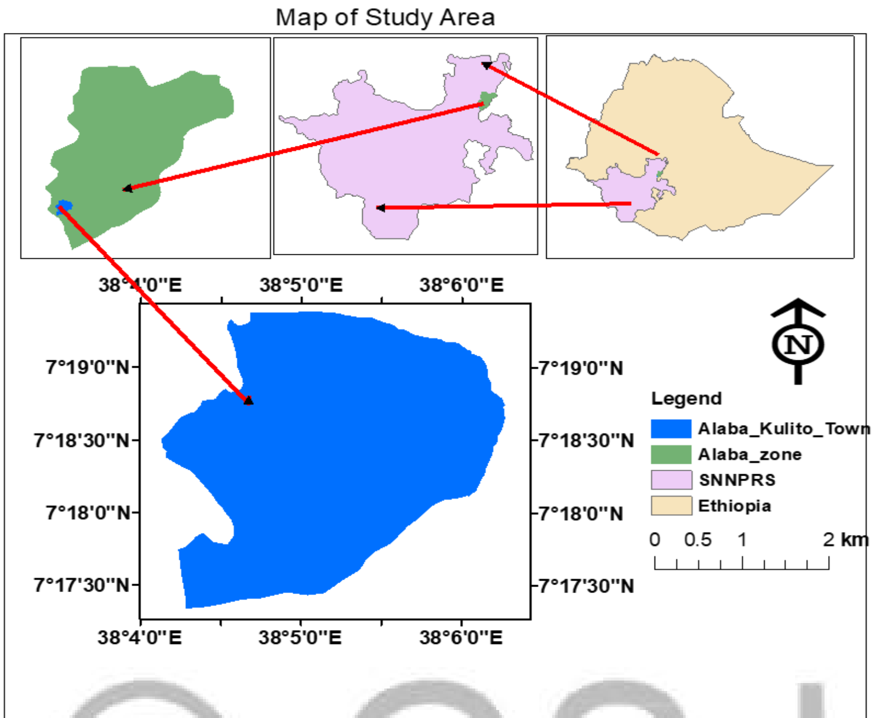

Figure 2.1: Map of the study area

❖ Climate

The average temperature and rainfall are 19.4 °C and 89.7mm respectively. Annual Precipitation here averages 1043 mm. February is the warmest month with an average of 20.6 °C. The lowest average temperatures in the year occur in August (18.4 °C). The climatic data of kulito station is summarized in the following table (table 2.1)

Table 2.1: Climate data of Alaba Kulito Station

	Jan.	**Feb.**	**Mar.**	**Apr.**	**May**	**Jun.**	**Jul.**	**Aug.**	**Sept.**	**Oct.**	**Nov.**	**Dec.**
Avg. T° (°C)	19.6	20.6	20.6	20.3	19.8	19.1	18.5	18.4	19.1	19	18.7	18.6
Min. T° (°C)	11.1	12.3	12.6	13.2	12.7	12.8	13.2	12.9	12.8	11.2	10	9.1
Max. T° (°C)	28.2	28.9	28.6	27.5	27	25.5	23.8	24	25.5	26.9	27.5	28.1
RF (mm)	27.5	49.7	91.3	137.1	126.4	93.4	115.3	154.8	121.0	73.8	63.2	23.0

❖ Vulnerability to flood

Flooding has been a problem in Alaba Kulito Town having the natural topography, which varies from the mountainous (Rekame) to the flatlands. The catchment of Alaba Kulito Town is flat land and hence it is severely affected by flooding hence the formation of the Town.

❖ Soil Type

According to FAO soil map classification of Ethiopia, ERA's manual (2002) and soil property information of World Harmonized Soil Database (WHSD) version 1.2 software; the soil type of the study area is classified as mollic andosol which is B hydrologic soil group.

❖ Slope and Topography of the Study Area

Here; 20 by 20 DEM is used to calculate the slope of the study area which is reduced to five classes of slope ranging from 0 up to 33.25 percent. The town is most probably near to flat whose slope is less than 6%; and slopes following drainage lines are relatively moderate from 6% - 20% and slope around Bilate river is about 33.25%. According to FAO slope classification, the most portion of Alaba Kulito Town is between class 1 and class 4 with the remaining classes are found around Bilate River.

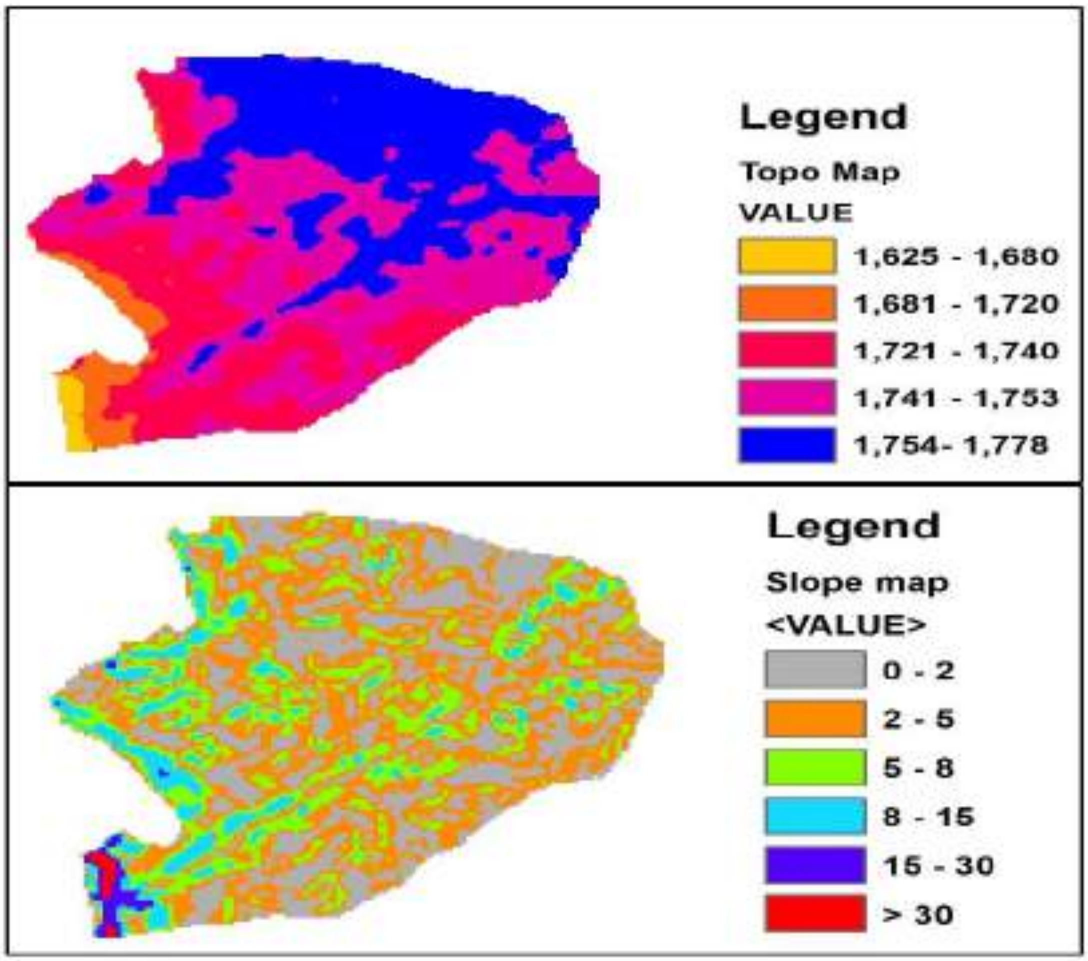

Figure 2.2: Slope and Topographic map of the Alaba Kulito Town

❖ Land-Use Land-Cover Classes

As the basis of hydrologic impact evaluation and to introduce the current land use coverage of Alaba Kulito Town; land use classification is carried out.

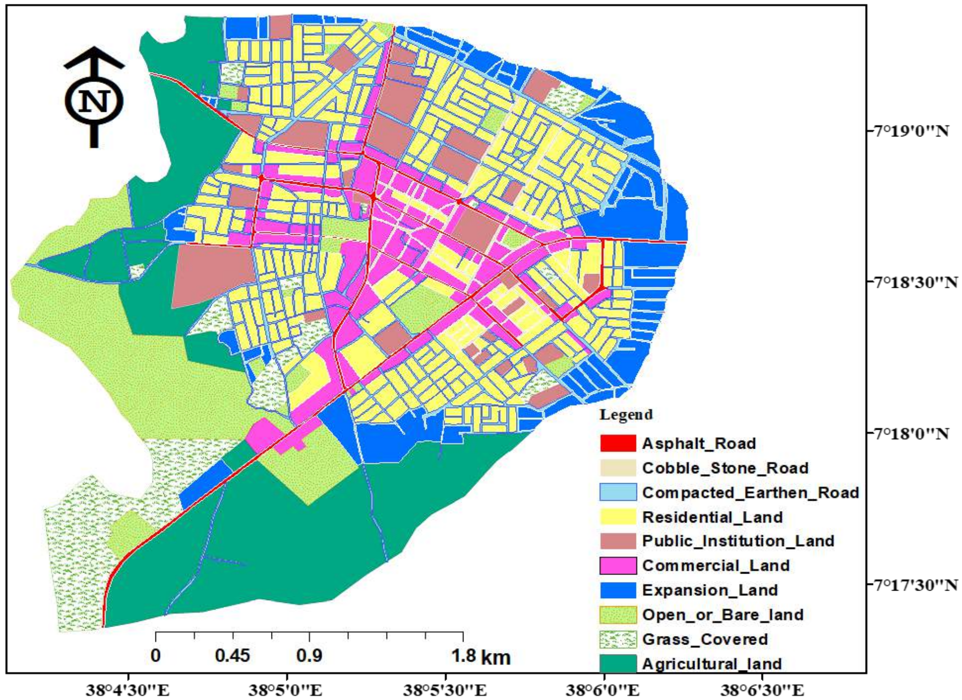

Figure 2.3: land-use map of the whole Alaba Kulito Town

2.2 Data Collection and Analysis Methods

For this study, daily rainfall data for 33 years (from 1987-2019) is obtained from the National Meteorological Agency (NMA) of Ethiopia. For further hydraulic simulation works, the maximum daily precipitation was selected and used as an input. The annual rainfall values are not within 10%, (for both Durame and Hosana), the stations are not evenly spaced, hilly regions and they are not near to each other (distant) it is not satisfactory to use the Inverse distance method and arithmetic mean method. Therefore, this paper considers the Normal Ratio Method (NRM) to fill missing precipitation data.

$$Px = \frac{Nx}{3}\left[\frac{P1}{N1} + \frac{P2}{N2} + \frac{P3}{N3}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Where; P_x is the rainfall value of the weather station with missing precipitation.

P_1, P_2, and P_3 are the current rainfall values for station 1, station 2, and station 3 respectively.

N_1, N_2, N_3 and N_x, are the annual normal precipitation values for station 1, station 2, station 3 and station X respectively.

The double mass curve technique was used to adjust precipitation records to take account of non-representative factors such as a change in location or exposure of rain gauge.

2.2.1 Determination of extreme value

- **Peak over Threshold (POT) extreme value Determination Approach**

The POT method is more time consuming than block maxima (BM), but has been found to yield better results in many studies for both extreme events of flood and drought (Yilmaz and Perera, 2015). An important advantage of the POT approach is that it produces larger sample sizes (data points), that enables one to use observed data more efficiently by considering more than one sample per year. However, the number of extreme data points to be used for analysis depends on the selected threshold value; on average, 1.65–3.0 extreme events per year are considered for further analysis (Niguse et al., 2018).

A widely used method of determining the threshold value from a time series is the use of a plot called Mean Residual Life Plot (MRLP) (Hafid and Mohamed, 2019). The MRL plot displays the mean excess against a range of different threshold values. The mean residual life plot should be approximately linear above a threshold, u, at which the GPD provides a valid approximation to the excess distribution and the linearity is the basis for deciding the threshold value (Saeed & Abd Wahab, 2016). The mean of the excesses of the threshold, u, represented as $(X - U|X > U)$, can be estimated by the sample mean of the threshold excesses and, thus, the mean residual life plot is plotted by using the locus of points as:

$$\left\{\left(U, \frac{1}{n_u}\sum_{i=1}^{n_u}(Xi - U)\right); U < Xmax\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Where; n_u is the number of observations that exceed threshold u.

3.2.1.1 Generalized Pareto Extreme Value Distribution Method

The probability density function for GPD with a shape parameter k ≠ 0, a scale parameter σ, and a threshold (location) parameter μ, is given as:

$$Y = f(x|k, \sigma, \mu) = \left(\frac{1}{\sigma}\right)\left(1 + k\frac{(x-\mu)}{\sigma}\right)^{-1-\frac{1}{k}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Range of x: $\mu < x \leq \mu + \sigma/\kappa$ if $\kappa > 0$; $\mu \leq x < \sigma$ if $\kappa \leq 0$

Graphical and statistical methods are also used to test the goodness-of-fit of the distribution of the extreme rainfall values to the selected test distribution (GPD in this study). Among the several statistical tests available in the literature, the Kolmogorov-Smirnov (KS) statistic is the most frequently used; the most widely used graphical tests are the P-P and QQ plots (Mohamed, 2015). In this study, EasyFit 5.6 Professional software was used to develop these plots.

The KS test is a nonparametric supremum test based on the empirical cumulative distribution function (CDF). The empirical distribution function Fn for n number of iid observations of Xi is expressed as:

$$F_n(x) = \frac{1}{n}\sum_{i=1}^{n} I_{[-\infty,x]}(X_i) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Where: $I_{[-\infty, x]}$ is the indicator function. $I_{[-\infty, x]} = 1$ if $X_i \leq x$ and $I_{[-\infty, x]} = 0$ if $X_i > x$.

The Kolmogorov-Smirnov statistic (D_n) is then computed as the largest vertical distance between the empirical CDF ($F_n(x)$ and the expected CDF (F(x) as:

$$D_{n=} =_{x}^{sup} |F_n(x) - F(x)| \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Where: sup_x is the supremum of the set of distances. The KS statistic is a procedure for testing whether two samples of a dataset are from the same distribution. In these statistics, the null hypothesis, which states that the two samples were drawn from the same distribution, is rejected if the p-value is less than the significance level.

2.2.2 Assessment of Hydraulic Capacity of the Existing Drainage System

The measured canal dimension helps us to analyze the hydraulic capacity of the existing drainage system to convey the runoff generated from sub-catchments by using the Manning method. Further, the output of EPA SWMM 5.1, rational equation (for less than 50 ha area) and soil conservation service curve number (SCS-CN) method (for more than 50 ha area) are used for comparison.

The urban drainage system can be represented as a network consisting of catchments and sub-catchments, nodes, links and outlets. Therefore, data of catchment property and data of existing drainage canals are necessary to know the runoff volume generated from the catchment area and capacity of drainage lines. For this study, 20x20 meter of DEM of USGS is obtained from Ethiopia Mapping Agency and the location of the study area was clipped by ArcGIS 10.4.1 data management tools.

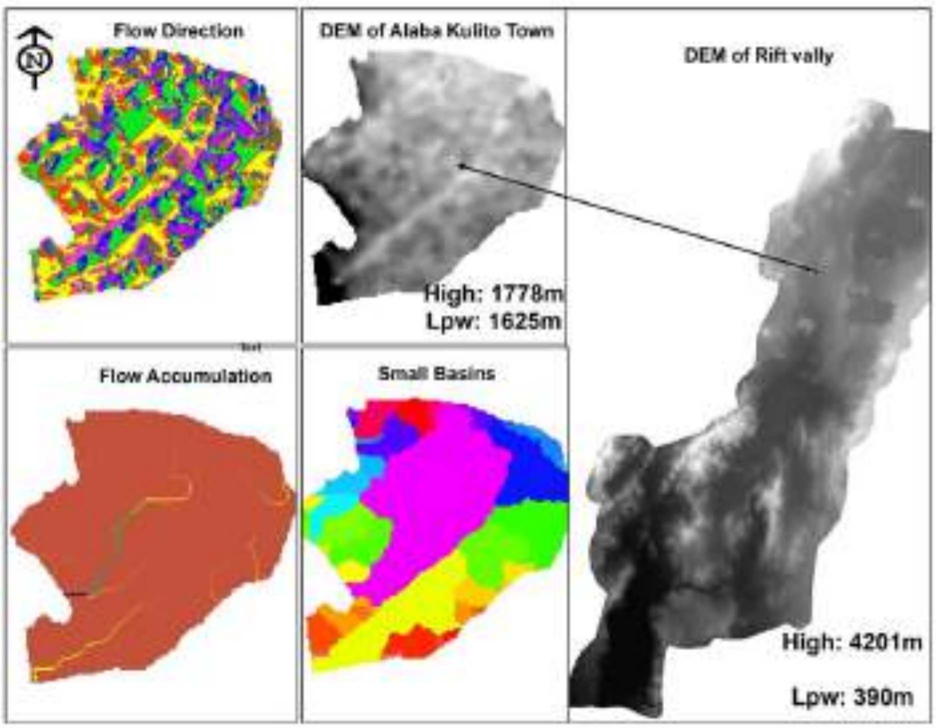

Figure 2.4: Clipped DEM for the study area

Soil data was taken from the Ministry of Agriculture to know the types of soil for the study area. The soil type of the study area is mollic andosol which is B hydrologic soil group.

2.2.2.1 Drainage System Network Data

Data of the drainage system network for the drainage network is collected on-site by direct measuring. The following information of storm drain system are collected:

- Dimension, slope, type and location of links
- Dimension and location of manholes and junctions
- X, Y, Z Coordinates of junctions

3 RESULTS AND DISCUSSION

3.1 Developing Intensity Duration Frequency (IDF) Curve of Alaba Kuito Town

❖ Rainfall Data

Table 3.1: Spatial description of weather stations which are found near to Alaba Kulito

Name of stations		Alaba kulito	Durame	Hosana	Awassa
Geographic position	Latitude (N)	7.31058°	7.2°	7.5673°	7.065°
	Longitude (E)	38.09392°	37.95°	37.8538°	38.48306°
	Elevation (m)	1772	2000	2307	1694
Distance from A/ Kulito weather station (km)		0	20	38.8	50.8
Elevation(m) with respect to A/Kulito		0	228	535	-78
Annual normal rainfall		1043	1156	1173	965

❖ Checking consistency by Double-mass curve (DMC)

Here, the average cumulative precipitation of neighboring stations (Durame, Awassa, Hosana and Alaba Kulito) is consecutively arranged in the reverse chronological order and plotted with the yearly cumulative of precipitation of Alaba Kulito station for the corresponding years.

Table 3.2: Annual precipitation and Cumulative annual precipitation in mm for double mass curve

Year	Annual precipitation of stations					Cumulative annual precipitation of stations				
	A. Kulito	Awassa	Durame	Hosana	Mean	A. Kulito	Awassa	Durame	Hosana	Mean
1987	1550.3	958.7	1330.6	1280.5	1280.0	1550.3	958.7	1330.6	1280.5	1280.0
1988	1498.0	957.0	1255.1	1216.2	1231.6	3048.3	1915.7	2585.7	2496.7	2511.6
1989	1083.5	1082.0	1580.8	1199.7	1236.5	4131.8	2997.7	4166.5	3696.4	3748.1
1990	968.0	756.7	1047.9	1058.9	957.9	5099.8	3754.4	5214.4	4755.3	4706.0
1991	975.1	846.8	1277.3	1079.1	1044.6	6074.9	4601.2	6491.7	5834.4	5750.6
1992	1162.6	962.3	1326.7	1387.6	1209.8	7237.5	5563.5	7818.4	7222.0	6960.4
1993	1272.2	928.4	1117.2	1413.5	1182.8	8509.7	6491.9	8935.6	8635.5	8143.2
1994	806.1	861.5	812.8	920.4	850.2	9315.8	7353.4	9748.4	9555.9	8993.4
1995	975.6	1004.4	935.9	1160.7	1019.2	10291.4	8357.8	10684.3	10716.6	10012.5
1996	1160.5	1189.1	949.7	1168.2	1116.9	11451.9	9546.9	11634.0	11884.8	11129.4
1997	1193.5	1054.1	1190.4	1442.5	1220.1	12645.4	10601.0	12824.4	13327.3	12349.5
1998	1252.7	1148.3	1378.6	1556.4	1334.0	13898.1	11749.3	14203.0	14883.7	13683.5
1999	763.4	808.9	1066.0	1011.3	912.4	14661.5	12558.2	15269.0	15895.0	14595.9
2000	876.0	821.5	1162.2	991.9	962.9	15537.5	13379.7	16431.2	16886.9	15558.8
2001	944.2	1021.7	1323.7	1145.5	1108.8	16481.7	14401.4	17754.9	18032.4	16667.6
2002	796.0	919.3	1140.4	1346.4	1050.5	17277.7	15320.7	18895.4	19378.8	17718.1
2003	943.3	888.9	1053.5	1140.2	1006.5	18221.0	16209.6	19948.9	20519.0	18724.6
2004	992.6	897.7	1064.7	1185.1	1035.0	19213.6	17107.3	21013.6	21704.1	19759.6
2005	866.5	1002.6	1460.9	1179.0	1127.3	20080.1	18109.9	22474.5	22883.1	20886.9

2006	873.8	1197.9	939.8	1201.7	1053.3	20953.9	19307.8	23414.3	24084.8	21940.2
2007	1081.6	1152.7	1033.1	1098.8	1091.6	22035.5	20460.5	24447.4	25183.6	23031.7
2008	950.3	915.0	1003.8	1202.8	1018.0	22985.8	21375.5	25451.2	26386.4	24049.7
2009	845.8	703.7	853.8	1247.2	912.6	23831.6	22079.2	26305.0	27633.6	24962.3
2010	1800.4	1032.3	1024.7	1121.5	1244.7	25632.0	23111.5	27329.7	28755.1	26207.1
2011	956.0	922.9	1201.2	1124.5	1051.1	26588.0	24034.4	28530.9	29879.6	27258.2
2012	752.3	785.4	758.9	981.6	819.6	27340.3	24819.8	29289.8	30861.2	28077.8
2013	2017.6	1054.1	1373.7	1145.1	1397.6	29357.9	25873.9	30663.4	32006.3	29475.4
2014	1212.7	1154.6	1564.8	1541.5	1368.4	30570.6	27028.5	32228.2	33547.8	30843.8
2015	644.6	778.4	815.7	1070.0	827.2	31215.1	27806.9	33043.9	34617.8	31670.9
2016	1275.7	1001.9	1656.6	1133.1	1266.8	32490.8	28808.8	34700.5	35750.9	32937.7
2017	802.1	1210.4	1206.9	952.0	1042.9	33292.8	30019.2	35907.4	36702.9	33980.6
2018	1000.5	1180.6	1703.2	1454.8	1334.8	34293.3	31199.8	37610.6	38157.7	35315.4
2019	1180.0	1076.2	1932.9	1543.4	1433.1	35473.3	32276.0	39543.5	39701.1	36748.5

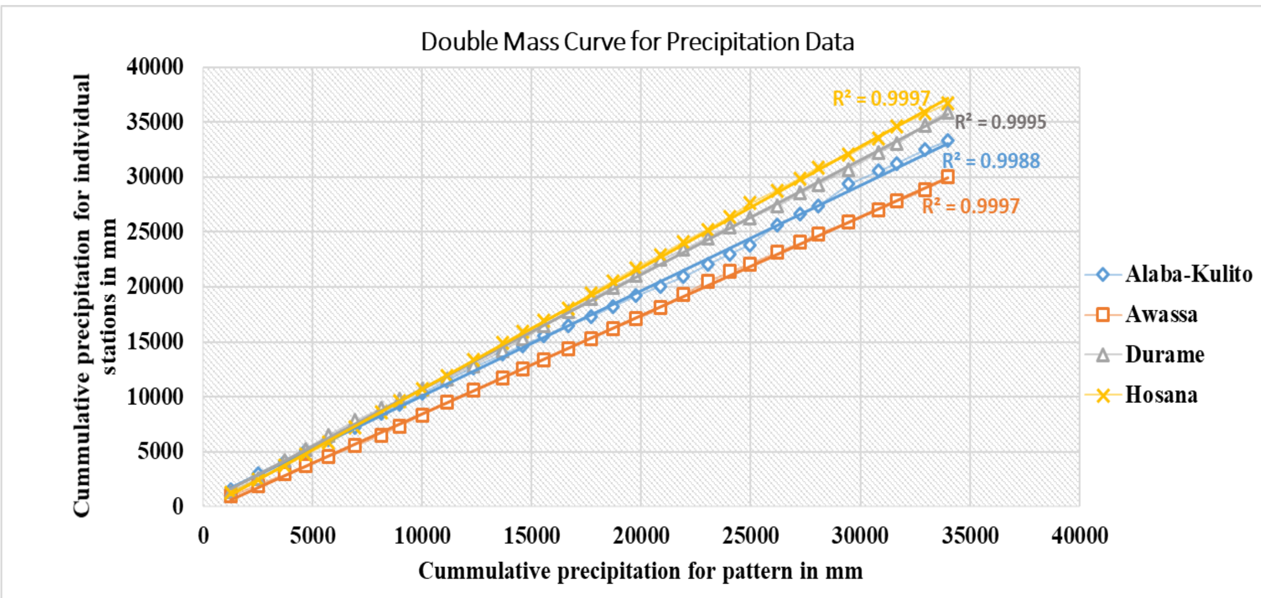

Figure 3.1: Double Mass Curve of rainfall data of individual stations

In this study, the result of the applied consistency checking method (double mas curve)shows that, all stations results in good consistency having a straight line and R-square values near to unit (0.9988, 0.9997, 0.9995 and 0.9997 for Alaba Kulito, hosanna, Durame and Awassa respectively).

❖ **Homogeneity test**

According to Homogeneity test analysis, the selected stations were plotted for comparison with each other. The result is displayed in the figure below (figure 3.2).

Table 3.3 Average monthly rainfall (mm) of 33 years of four stations

Months		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Stations	A/kulito	27.5	49.7	91.3	137.1	126.4	93.4	115.3	154.8	121.0	73.8	63.2	23.0
	Durame	24.0	45.3	86.2	148.8	154.3	113.3	158.6	160.3	141.6	97.3	44.7	23.9
	Awassa	26.1	35.9	75.0	111.7	125.8	103.3	122.1	122.7	118.6	72.8	38.2	25.9
	Hosana	26.8	43.1	99.4	141.7	149.5	127.7	152.3	176.7	155.5	72.5	31.3	26.6

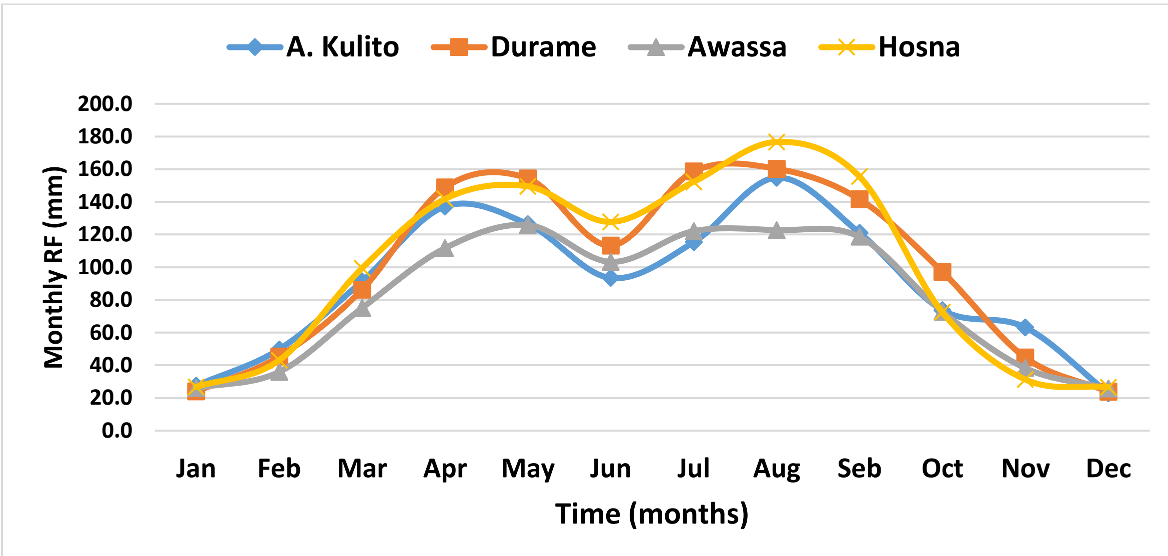

Figure 3.2: Homogeneity of the rainfall areas of the representative stations

This shows that the result of homogeneity analysis resulting Same-mode (bi-modal) and pattern of the stations are observed and hence group stations selected are homogenous since all shows likely similar patterns.

- ❖ **Determination of extreme value by Peak Over Threshold Method**

According to section 3.2.1.5; the maximum number of observations considered for further analysis are 99 (i.e. 33*3) data sets. The data sorted in ascending order for maximum daily rainfall of possible maximum number (99) is shown in table 3.5.

By using equation 3.3, the MRL plot can be developed as shown by figure 3.3.

Table 3.4: Mean residual versus threshold values

Threshold	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54
Mean Residual	12.8	12.3	12.8	12.8	13	12.7	12.3	12.1	11.6	11.2	10.6	10.9	12	11.2	11.5	11	11
Threshold	55	56	57	59	60	61	62	63	64	65	66	68	71	73	74	75	79
Mean Residual	10.7	11.5	12.8	12	11.7	11.5	11.2	9.6	9.9	8.9	8.6	10.3	7.3	6.2	6.2	6.2	3.7

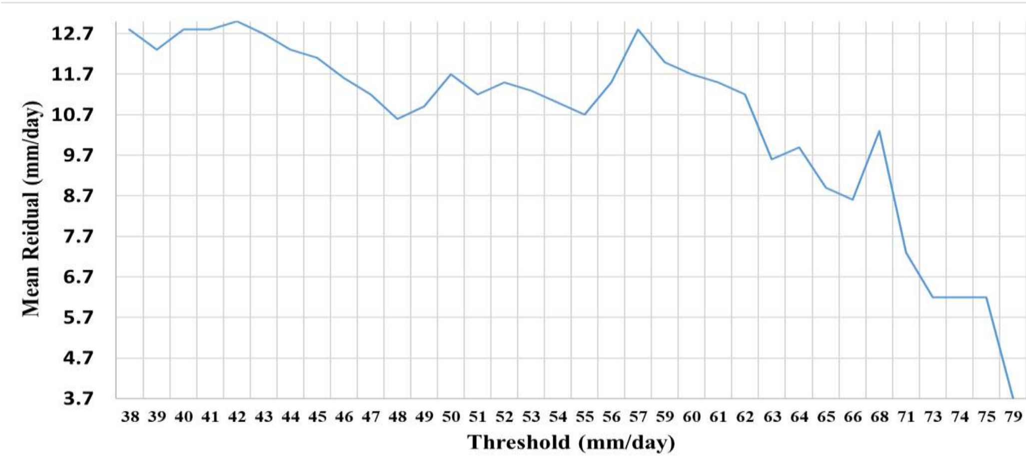

Figure 3.3: Mean Residual Life Plot (MRLP) of the reference period rainfall data

The graph (figure 3.3) shows that there is a sort of slight linear relationship between MRV and threshold in the range between (48 and 50), (55 and 57) and (66 and 68) relatively.

Here; by trial and error, values above 48.4 mm/day fits best the GPD ranking first, second and third-order for Chi-Squared, Kolmogorov Smirnov and Anderson Darling respectively than others.

Table 3.5: Annual maximum daily precipitation for POT method

Annual maximum daily precipitation for POT method												
Rejected												
Number	1	2	3	4	5	6	7	8	9	10	11	12
P(mm)	38	38.2	38.3	38.4	38.5	39.4	39.5	39.5	39.5	39.5	39.5	39.6
Number	13	14	15	16	17	18	19	20	21	22	23	24
P(mm)	40	40	40	40	40.2	40.3	40.7	40.9	41	41	41.4	41.4
Number	25	26	27	28	29	30	31	32	33	34	35	36
P(mm)	41.7	42	42	42	42	42.2	42.4	42.4	42.8	43.2	43.5	43.5
Number	37	38	39	40	41	42	43	44	45	46	47	48
P(mm)	44.1	44.2	44.4	44.5	45.4	45.7	46.3	46.7	46.8	47.6	47.6	48.2
Accepted												
Number	49	50	51	52	53	54	55	56	57	58	59	60
P(mm)	48.4	48.8	49	49	49	49.3	49.3	50	50	50	50	50
Number	61	62	63	64	65	66	67	68	69	70	71	72
P(mm)	51	51	51.2	51.3	51.4	52	52.4	52.4	53.8	54	54.8	55
Number	73	74	75	76	77	78	79	80	81	82	83	84
P(mm)	55.3	55.4	55.7	55.7	56.4	56.6	56.7	57	59	59	60	60.5
Number	85	86	87	88	89	90	91	92	93	94	95	96
P(mm)	62	64	65.3	66.5	66.8	68	68	71.8	73.7	74.7	75.4	79.4
Number	97	98	99									
P(mm)	79.4	86	86									

❖ **Detecting outlier**

Outlier detection is summarized in the table below (table 3.6) for POT approach.

Table 3.6: Summary of outlier detection

Type of Ext. Value determination	N	Mean	Standard deviation	Lower limit	Upper limit	Rejected	
						value/s	Reason
POT	51	58.1	10.24	-28.26	89.7	None	(48.4 & 86) are within (-28.26 & 89.7)

Accordingly; out of 99 sample data (3* number of years (33)), 48.2 mm/day is selected as the threshold value of POT based on MRLP and the remaining values (48.4mm/day up to 86mm/day which are 51 sample data) are used. Furthermore with $\mu = 58.1$, $\sigma = 10.24$ and N = 51, none of data deviates from range (-28.26 and 89.7). Remarkably, all values fall under the limit as illustrated in table 3.6 above. In the same manner, all data sets (33 years daily maximum RF data) lays between lower and upper limits. Therefore, the selected data can be used for the remaining hydraulic and hydrologic analyses.

❖ **Generalized Pareto Distribution Method**

IDF curve is developed by combining the RRt value of the study area for different durations with 24 hr Rainfall depth of Alaba Kulito station for different return periods.

Table 3.7: Return period, percentage of probability and 24 hr RF depth (mm) of GPD

T	2	5	10	25	50	100
$P = 1 - \frac{1}{T}$	0.5	0.8	0.9	0.96	0.98	0.99
R24 of A. Kulito of GPD	55.49	65.626	73.131	82.845	90.041	97.108

Table 3.8: IDF values of GPD for Alaba Kulito station

Intensity I(mm/hr)										
Duration (hr)		0.0833	0.1667	0.25	0.5	1	1.5	2	2.5	3
Duration (min)		5min	10min	15min	30min	60min	90min	120min	150min	180min
Return period (T)	2	105.2	87.8	77.7	53.5	34.2	25.3	20.2	16.9	14.5
	5	124.4	103.8	91.9	63.2	40.4	30.0	23.9	20.0	17.2
	10	138.6	115.7	102.4	70.4	45.1	33.4	26.7	22.2	19.1
	25	157.0	131.0	116.0	79.8	51.0	37.8	30.2	25.2	21.7
	50	170.6	142.4	126.1	86.7	55.5	41.1	32.8	27.4	23.5
	100	184.0	153.6	136.0	93.5	59.8	44.4	35.4	29.5	25.4
	1000	226.8	189.3	167.6	115.3	73.8	54.7	43.6	36.4	31.3

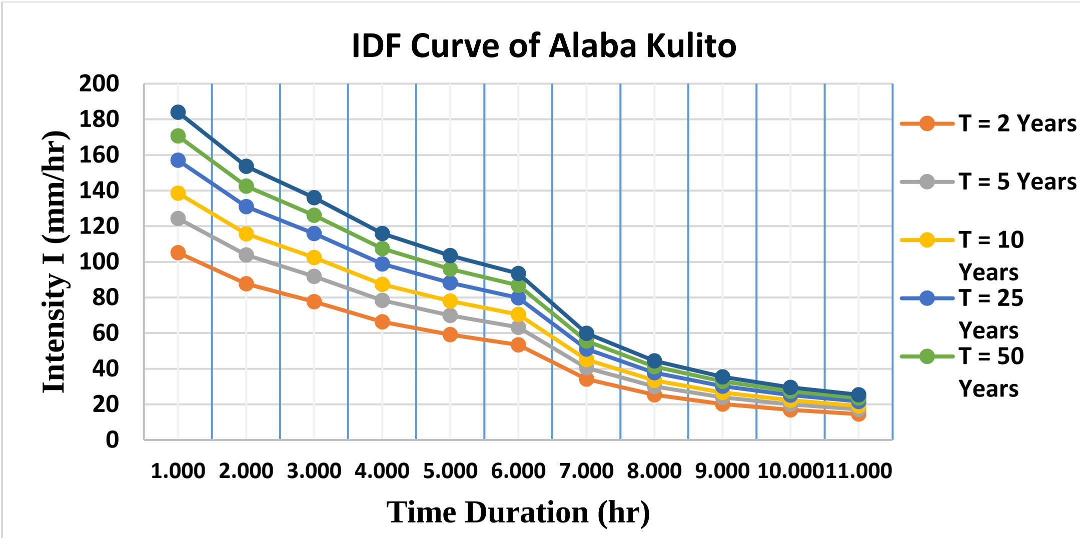

Figure 3.4: IDF curves of 33 years of rainfall data for Alaba Kulito by GPD

3.2 Assessment of Hydraulic Capacity of Existing Drainage canal and Hydrologic Response of Catchment to Rainfall Intensity

❖ **Land-use map of the study area**

As the basis of hydrologic impact evaluation, the land-use study was carried out with the help of Google Earth pro and ArcGIS 10.4.1 software with detail physical observation of the considerable study area including land-use types and existing drainage lines.

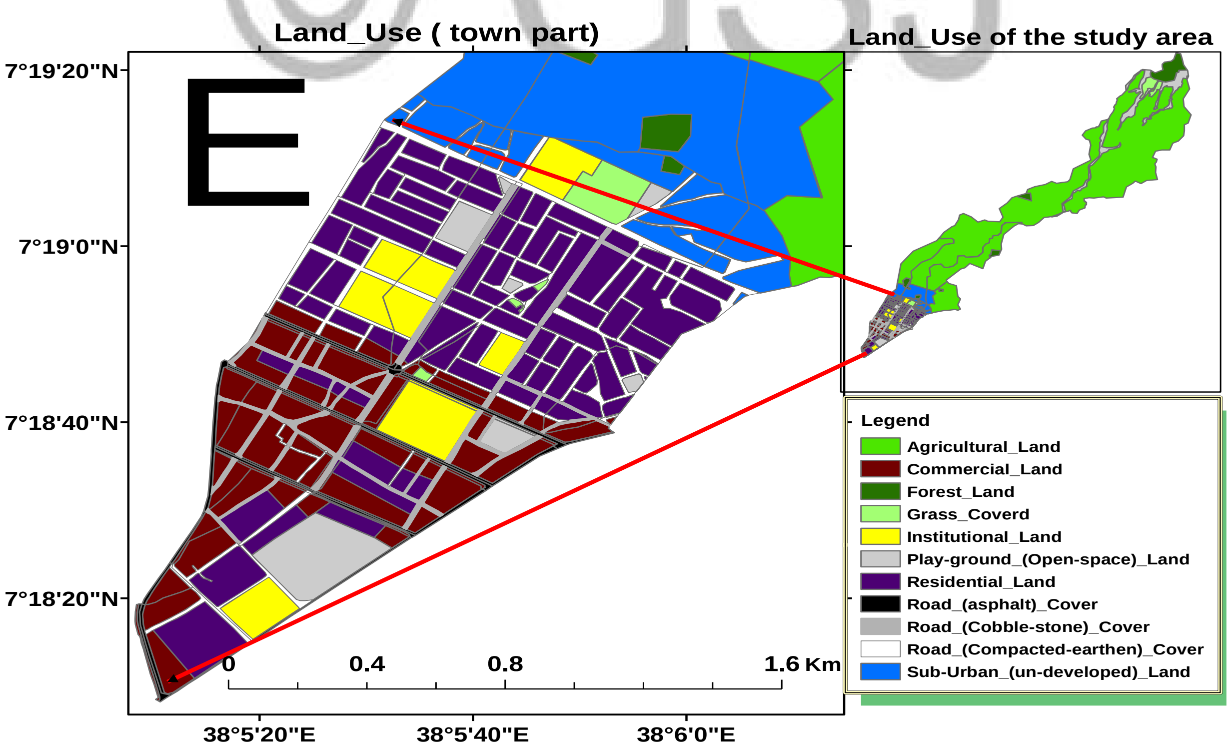

Figure 3.5: Study Area Land Use/Land Cover

According to the study; eleven land use/land covers are distinguished and coded as shown in table 4.10 with their respective area (ha), land-use percentage from coverage of study area (%), and representative runoff coefficients (C).

Table 3.9: Land Use/land cover Composition and Runoff coefficient of study Area

Land-Use/Cover	Land-use code	Area (Ha)	Percentage (%)	Runoff cof.(C)
Agricultural	LU-AG	1318.6	76.8	0.4
Commercial	LU-CM	37.0361	2.2	0.8
Forest	LU-FR	61.7704	3.6	0.3
Grass	LU-GR	23.8986	1.4	0.35
Institutional	LU-IT	16.9837	1.0	0.5
Play-ground (Open Space)	LU-OS	102.4137	6.0	0.3
Residential	LU-RS	66.32	3.9	0.5
Road (Asphalt)	LU-RA	5.5805	0.3	0.9
Road (Cobble-Stone)	LU-RC	7.64219	0.4	0.7
Road (Compacted Earth)	LU-RE	18.19991	1.1	0.45
Sub-Urban (Un-Developed)	LU-SU	59.0185	3.4	0.4
Sum		1717.46	100.00	

❖ **Flow Direction and Drainage Network of Study Area**

Both Lenda Ber and Mahal Arada kebele are connected by the same drainage lines by which Lenda Ber being the upper catchment and drains toward outfall located at Mahal Arada around ST Gabriel Church.

The drainage network follows the geographical topography/contour with necessary modifications made according to the land-use and road pattern of the town. The existing drainage lines are used to divide sub-catchments where SC01 (sub-catchment 01) being the uppermost sub-catchment and SC30 (sub-catchment 30) is found at the lower with the coded links. Accordingly, there are 37 drainage lines with respective sub-catchments connected as shown in the figure below (figure 3.6).

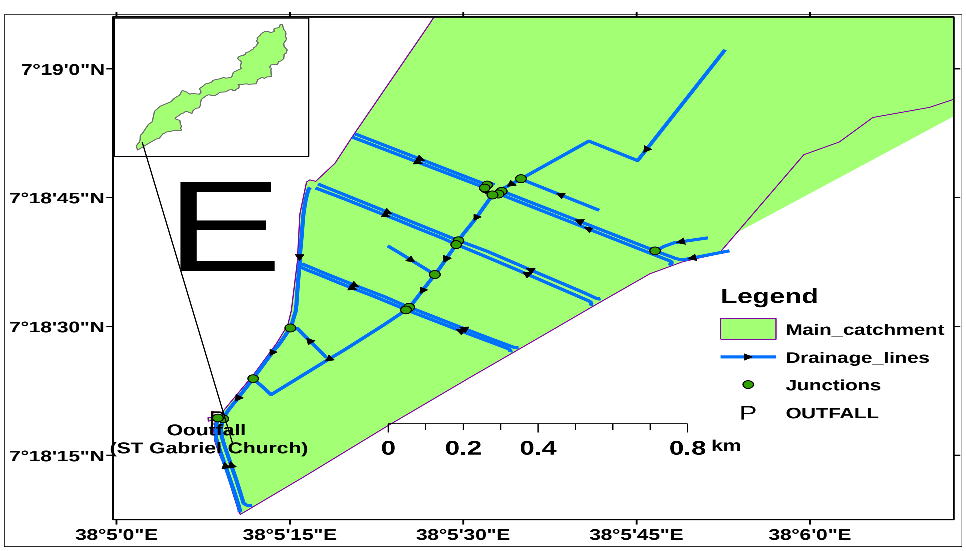

Fig 3.6: Flow Direction of links

❖ Sub-catchment, LandUse Compositions and Weighted Runoff Coefficient (Cw)

Initially, raw data from Elevation Model (DEM) was analyzed by ArcGIS10.4.1 terrain processing tool with an input stream network. The output 30 sub-catchments were obtained with the input stream network which is an existing link network of study area since the urban drainage is not as such a natural way, sub-catchments are done by the pattern of elevation differences following the consecutive drainage lines. LandUse Compositions and Weighted Runoff Coefficients of each sub-catchments are summarized in table 3.10 below.

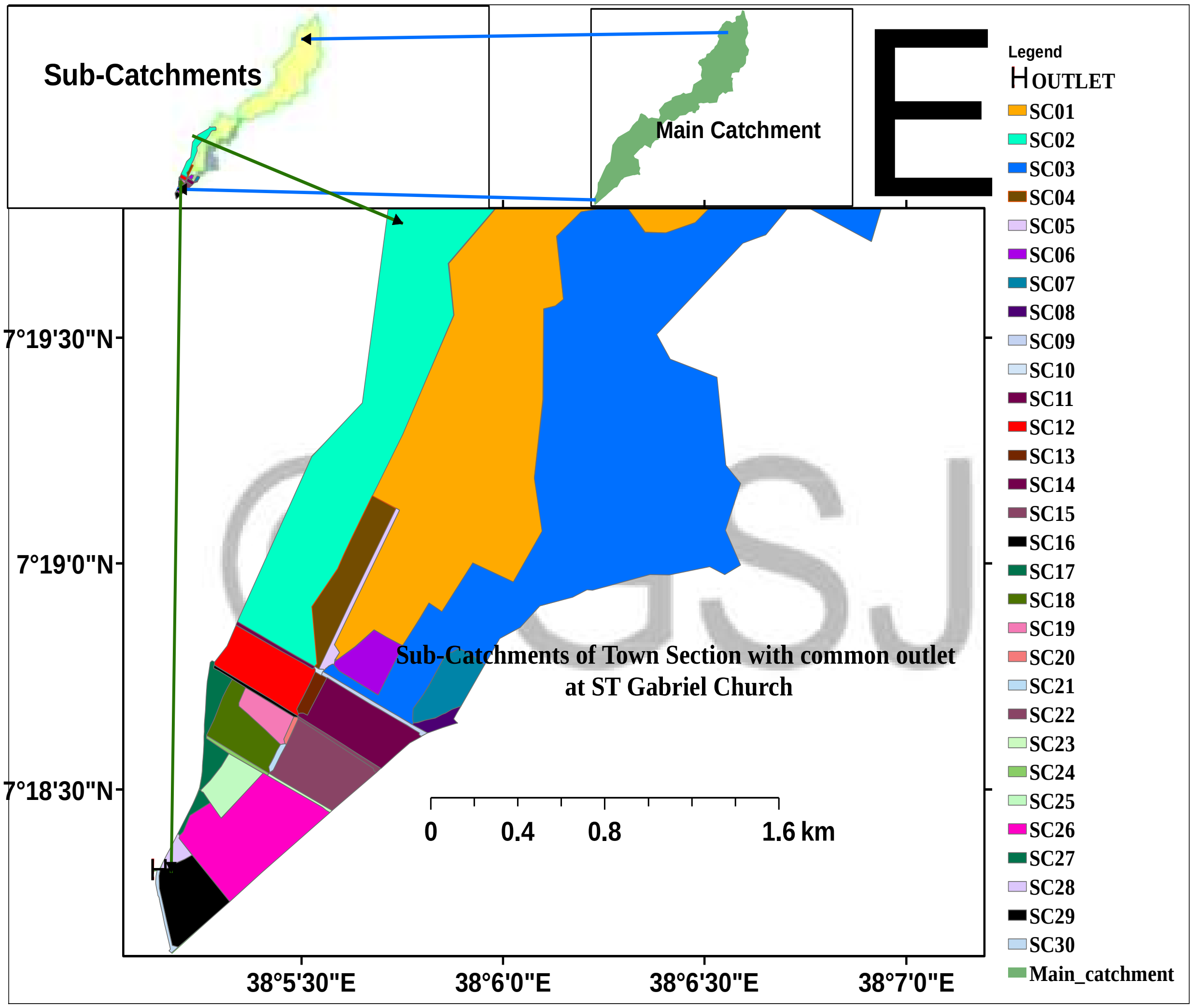

Figure 3.7: Sub-Catchments of the Study Area section

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Table 3.10: Each Sub-catchment Land Use Compositions and Weighted Runoff Coefficient

Sub-Catchments	Composition of LU/LC for each Sub-catchments (ha)												Cw
	LU-AG	LU-CM	LU-FR	LU-GR	LU-IT	LU-OS	LU-RS	LU-RA	LU-RC	LU-RE	LU-SU	Sum	
SC01	1086.		52.55	17.42	2.72	92.61	15.34		0.72	4.79	33.09	1305.24	0.39
SC02	69.60	1.81	3.99	6.33	3.13		14.11	0.26		5.29	12.21	116.73	0.42
SC03	163.0	1.55	5.23	0.15			14.39	0.25	0.52	3.78	13.72	202.58	0.41
SC04		0.10			2.95	1.69	1.93	0.03	0.51	0.90		8.12	0.47
SC05		0.06					0.66	0.04	0.68	0.10		1.54	0.61
SC06		0.54			1.04		2.21		0.11	0.48		4.38	0.54
SC07		0.97				0.27	3.43		0.22	0.87		5.76	0.54
SC08								0.12	0.03	0.04		0.19	0.77
SC09		0.10			0.18	0.10		0.46	0.03			0.87	0.73
SC10		0.01						0.07				0.08	0.89
SC11		0.27						0.26	0.03			0.56	0.84
SC12		5.29					1.33	0.26	1.18			8.06	0.74
SC13		0.70							0.31			1.02	0.77
SC14		2.53			4.43	1.04		0.51	0.73			9.23	0.60
SC15		0.29						0.30	0.01	0.01		0.61	0.84
SC16		0.24						0.25	0.02			0.51	0.85
SC17		2.47						0.50	0.22			3.18	0.81
SC18		4.85						0.20	0.41	0.21		5.67	0.78
SC19		2.08							0.50	0.00		2.59	0.78
SC20		0.03					0.05		0.17			0.24	0.67
SC21		0.10					0.04		0.19			0.33	0.71
SC22		4.98					3.03	0.41	0.71	0.08		9.20	0.69
SC23		0.08					0.08	0.19				0.35	0.79
SC24		0.17						0.22	0.02	0.01		0.42	0.84
SC25		1.65					1.60		0.31			3.56	0.66
SC26		2.31			2.54	6.70	4.38	0.31		1.15		17.38	0.47
SC27		0.47						0.13				0.60	0.82
SC28		0.71						0.13		0.06		0.90	0.79
SC29		2.49					3.74	0.42		0.44		7.09	0.63
SC30		0.20						0.26				0.46	0.86
Sum	1318.6	37.04	61.77	23.90	16.98	102.41	66.32	5.58	7.64	18.20	59.02	1717.46	

❖ **Estimation off Peak Runoff by Rational Method**

As stated in metheodology section, the peak runoff rate for small sub-catchment areas less than 50 ha are done by the rational method. As depicted in Table 3.10 above, about 27 sub-catchments are less than half a kilometer square.

Here is a sample calculation is done for sub-catchment twelve (SC12);

Step 1.Sub-catchment area of SC12: A_{sc12} = 8.06ha (from table 3.10)

Step 2. Longest flow path and elevation: length of overland flow is 0.185 km, length of the defined canal is 0.425km, elevation difference for overland flow is1m.

Step 3. Catchment property: Hydrologic soil group is B, land cover is composed of five land-use type as illustrated in table 3.10 above, rainfall region is grouped under B2 but for analysis Intensity done by GPD is selected.

Step 4. Time of concentration:

i. For overland flow; for C_{v12} = 0.74, L= 0.185km and S= 0.54054%

$$T_{c1} = 0.604\left(\frac{C_{vi}*L_i}{S^{0.5}}\right)^{0.467} = 0.2753\text{hr}$$

ii. For canal flow; L = 425m, V= 2.117m/s (done by manning equation)

$$T_{c2} = \frac{L}{3600V} = 0.05576\text{hr}$$

$$T_c = T_{c1} + T_{c2} = 0.05576\text{hr} + 0.2753\text{hr} = 0.3311\text{hr} = 20\text{min}$$

Step 5. Rainfall intensity: The rainfall intensity of different return period is done. Accordingly; Intensity for sub-catchment 12 is found to be: I_2 = 66.48, I_5 = 78.62, I_{10} = 87.61mm/hr, I_{25} =99.25mm/hr, I_{50} = 107.87mm/hr and I_{100} = 116.33mm/hr.

Step 6. The runoff coefficient were depended upon vegetation cover (if there is), land use type and inclination of respective sub-catchment slope and it is 0.74 for SC12.

Step 7. Peak flood, Q =C_f*C*I*A (all parameters by SI unit);

T (year)	2	5	10	25	50	100
C_f	1	1	1	1.1	1.2	1.25
I (mm/hr)	66.48	78.62	87.61	99.25	107.87	116.33
Q (m^3/s)	1.10	1.30	1.45	1.81	2.14	2.41

Hydrological analysis done for the remaining sub-catchments is depicted in table 3.11 below.

Table 3.11: Peak Discharge by Rational Method

Return Period (T)			2	5	10	25	50	100
Frequency Factors (Cf)			1	1	1	1.1	1.2	1.25
SC	C	A(ha)	Q (m^3/s)					
SC04	0.471	8.12	0.65	0.77	0.86	1.07	1.27	1.42
SC05	0.605	1.54	0.14	0.17	0.19	0.23	0.27	0.31
SC06	0.537	4.38	0.50	0.59	0.66	0.82	0.97	1.09
SC07	0.541	5.76	0.57	0.68	0.75	0.94	1.11	1.25
SC08	0.769	0.19	0.03	0.04	0.04	0.06	0.07	0.07
SC09	0.728	0.87	0.16	0.18	0.20	0.25	0.30	0.34
SC10	0.886	0.08	0.02	0.02	0.02	0.03	0.04	0.04
SC11	0.841	0.56	0.12	0.15	0.16	0.20	0.24	0.27
SC12	0.739	8.06	1.10	1.30	1.45	1.81	2.14	2.41
SC13	0.769	1.02	0.21	0.25	0.28	0.35	0.42	0.47
SC14	0.598	9.23	1.11	1.31	1.47	1.83	2.16	2.43

SC15	0.843	0.61	0.12	0.15	0.16	0.21	0.24	0.27
SC16	0.846	0.51	0.12	0.14	0.15	0.19	0.23	0.25
SC17	0.809	3.18	0.56	0.66	0.73	0.91	1.08	1.21
SC18	0.784	5.67	0.84	0.99	1.10	1.37	1.63	1.83
SC19	0.780	2.59	0.46	0.55	0.61	0.76	0.90	1.01
SC20	0.674	0.24	0.05	0.06	0.06	0.08	0.10	0.11
SC21	0.709	0.33	0.07	0.08	0.09	0.11	0.13	0.15
SC22	0.695	9.20	1.20	1.42	1.58	1.97	2.34	2.63
SC23	0.785	0.35	0.08	0.09	0.10	0.13	0.15	0.17
SC24	0.841	0.42	0.10	0.11	0.13	0.16	0.19	0.21
SC25	0.656	3.56	0.47	0.55	0.61	0.76	0.91	1.02
SC26	0.467	17.38	1.50	1.77	1.98	2.46	2.92	3.28
SC27	0.821	0.60	0.12	0.14	0.15	0.19	0.23	0.26
SC28	0.791	0.90	0.16	0.18	0.21	0.26	0.30	0.34
SC29	0.626	7.09	0.78	0.92	1.03	1.28	1.52	1.71
SC30	0.857	0.46	0.08	0.10	0.11	0.13	0.16	0.18
$Q(m^3/s) = C*Cf*A(ha)*I(mm/hr)/360$								

- **Estimation of Peak Runoff by SCS-CN Method**
  - ✓ **Area and Land-use/cover percentage**

As the area of three sub-catchments (SC01, SC02 and SC03) is greater than 50ha, they are done by SCS-CN method. For each sub-sections, their respective catchment area (table 3.12), land-use percentage (table 3.12) and weighted runoff coefficient (table 3.13) are calculated as shown in each respective tables.

Table 3.12: Area and Land-use/cover percentage of Sub-Catchments done by SCS

Sub-catchments		LU/LC percentage (%) for each sub-catchments											SUM
		LU-AG	LU-CM	LU-FR	LU-GR	LU-IT	LU-OS	LU-RS	LU-RA	LU-RC	LU-RE	LU-SU	
SC01	A (ha)	1086.00	0.00	52.55	17.42	2.72	92.61	15.34	0.00	0.72	4.79	33.09	1305.24
	LU(%)	83.20	0.00	4.03	1.33	0.21	7.10	1.18	0.00	0.06	0.37	2.54	100.00
SC02	A (ha)	69.60	1.81	3.99	6.33	3.13	0.00	14.11	0.26	0.00	5.29	12.21	116.73
	LU(%)	59.62	1.55	3.42	5.42	2.68	0.00	12.09	0.23	0.00	4.53	10.46	100.00
SC03	A (ha)	163.00	1.55	5.23	0.15	0.00	0.00	14.39	0.25	0.52	3.78	13.72	202.58
	LU(%)	80.46	0.77	2.58	0.07	0.00	0.00	7.10	0.12	0.26	1.86	6.77	100.00

✓ Weighted Curve Number

Weighted runoff coefficients are obtained by dividing the sum of the product of each sub-catchment area by LU percentage to the summation of land-use percentage. Curve number values are selected by considering soil group, slope, and vegetation.

Table 3.13: Weighted Curve Number

Sub-catchments	Product of percentage of land-use/land-cover by curve number												Sum	CN_w
	LU-type	LU-AG	LU-CM	LU-FR	LU-GR	LU-IT	LU-OS	LU-RS	LU-RA	LU-RC	LU-RE	LU-SU		
	CN	71	92	55	61	70	61	68	98	85	82	69		
SC01	LU(%)	83.2	0.0	4.0	1.3	0.2	7.1	1.2	0.0	0.1	0.4	2.5	100.0	69.47
	CN*LU%	5907.4	0.0	221.4	81.4	14.6	432.8	79.9	0.0	4.7	30.1	174.9	6947.3	
SC02	LU(%)	59.6	1.6	3.4	5.4	2.7	0.0	12.1	0.2	0.0	4.5	10.5	100.0	70.20
	CN*LU%	4233.4	142.7	188.2	330.9	187.4	0.0	822.0	22.2	0.0	371.6	721.7	7020.0	
SC03	LU(%)	80.5	0.8	2.6	0.1	0.0	0.0	7.1	0.1	0.3	1.9	6.8	100.0	70.67
	CN*LU%	5712.8	70.4	141.9	4.5	0.0	0.0	482.9	12.0	21.9	152.9	467.2	7066.5	

The 24 hr rainfall depth of table 3.8 is used to calculate accumulated runoff (Q), initial abstraction (I) and maximum potential retention (S).

Table 3.14: Accumulated precipitation (P) in mm and Ia/P

catchments	Number (CN)	S (mm)	Ia (mm)	Accumulated precipitation (P) in mm for return period of 2, 5, 10, 25, 50 and 100 respectively											
				55.5	65.63	73.1	82.8	90.04	97.11	55.5	65.6	73.1	82.85	90.041	97.11
				Accumulated direct rain fall (Q) in mm						*Ia/P*					
SC01	69.47	111.63	22.33	7.4	11.9	15.6	21.0	25.3	29.7	*0.13*	*0.18*	*0.21*	*0.25*	*0.28*	*0.31*
SC02	70.20	107.82	21.56	8.0	12.7	16.6	22.1	26.5	31.0	*0.14*	*0.19*	*0.23*	*0.27*	*0.29*	*0.32*
SC03	70.66	105.47	21.09	8.6	13.4	17.4	23.0	27.5	32.1	*0.16*	*0.20*	*0.24*	*0.28*	*0.31*	*0.33*

✓ Unit peak discharge, (m3/s/km2)/mm

Table 3.15: unit peak discharge, (m3/s/km2)/mm

Sub-catchment	Tc (min)	Tc (hr)	Log (tc(hr))	qu ((m3/s)/km2)/(mm) for return period of 2, 5, 10, 25, 50 and 100 respectively					
				2	5	10	25	50	100
SC01	69.46	1.16	0.064	0.136288	0.1289	0.1308	0.1206	0.1170	0.1123
SC02	51.22	0.85	-0.069	0.162883	0.1542	0.1487	0.1445	0.1401	0.1328
SC03	62.04	1.03	0.015	0.141505	0.1368	0.1303	0.1257	0.1206	0.1156

✓ Peak discharge (qp)

Table 3.16: Peak discharge (qp) of Sub-catchments done by SCS-CN

Sub-cat chments	Area (km^2)	Peak Discharge qp = (Q *A * qu) in m^3/s						
		T	2	5	10	25	50	100
SC01	13.052	Q	7.27	11.68	15.39	20.69	24.92	29.29
		qu	0.136	0.129	0.131	0.121	0.117	0.112
		qp	***12.94***	***19.65***	***26.28***	***32.57***	***38.05***	***42.92***
SC02	1.167	Q	7.98	12.59	16.46	21.95	26.31	30.81
		qu	0.163	0.154	0.149	0.145	0.140	0.133
		qp	***1.51***	***2.25***	***2.84***	***3.68***	***4.28***	***4.75***
SC03	2.026	Q	8.72	13.55	17.57	23.25	27.75	32.37
		qu	0.142	0.137	0.130	0.126	0.121	0.116
		qp	***2.50***	***3.76***	***4.64***	***5.92***	***6.78***	***7.58***

❖ Estimation of Drainage Capacity by Manning Equation

The existing drainage canals of both Lenda Ber and Mahal Arada Kebele are considered in this study as these kebeles are prone to flood than the remaining three kebeles (Wanaja, Denebefama and Murasa Kebele). Most of the drainage facilities are open canals constructed by masonry and concrete along the main road, sub-main and local roads; whereas concrete made closed canals are found only along the main road of Hossana-Alaba Kulito-Sodo road way.

The physical parameters of existing canals like shape, type and size are used to analyze the hydraulic capacity of conduits under the study area. A tape meter is used to measure the depth and width of the canals. According to a field survey; all canals are open rectangular masonry except L20L, L21L, L22L, L23L and L23R which are closed links and found along the main asphalt road of Mehal Arada kebele. perimeter, hydraulic radius, and slope of the terrain are derived from canal depth, canal width and an elevation difference of starting and ending of successive links. The values obtained by the manning equation is shown in table 3.17 and under the section of adequacy analysis which is compared with the output of the SWMM model, and rational/SCS method.

Table 3.17: Existing drainage cannels capacity by manning equation

Conduit code	Area (m^2)	Slope (%)	n	R(m)	V (m/s)	Q(m^3/s)
L01R	1.000	0.361	0.020	0.3333	1.4443	2.152
L02R	0.930	0.422	0.020	0.3252	1.5358	2.128
L03L	1.170	0.834	0.020	0.3503	2.2697	3.957
L04R	1.500	0.242	0.015	0.4054	1.7947	4.011
L05L	0.618	0.336	0.020	0.2422	1.1253	1.035
L06R	0.694	0.976	0.015	0.2550	2.6487	2.737
L06L	0.646	0.562	0.015	0.2504	1.9851	1.911
L07L	0.740	0.251	0.015	0.2701	1.3962	1.539
L07R	0.700	0.278	0.015	0.2593	1.4286	1.490
L08R	0.595	1.000	0.020	0.2479	1.9732	1.749
C01	0.900	0.833	0.015	0.3333	2.9258	3.923

C02	0.805	2.500	0.015	0.2683	4.3853	5.260
C03	1.200	2.500	0.015	0.3750	5.4815	9.801
L09L	1.960	1.093	0.020	0.4667	3.1448	9.184
L10L	2.250	1.613	0.020	0.5000	4.0003	13.411
L11R	0.989	0.463	0.020	0.3130	1.5683	2.311
C04	2.000	0.714	0.015	0.5000	3.5494	10.577
L12L	0.945	0.471	0.020	0.3150	1.5879	2.236
L12R	0.850	0.471	0.020	0.2982	1.5311	1.939
L11L	0.972	0.469	0.020	0.3115	1.5742	2.279
L13L	3.200	0.331	0.020	0.5714	1.9796	9.439
L14L	0.848	0.617	0.020	0.2904	1.7227	2.177
L15L	3.200	0.735	0.020	0.5714	2.9524	14.077
L16L	0.706	0.309	0.020	0.2522	1.1089	1.167
L16R	0.706	0.313	0.020	0.2522	1.1158	1.174
C05	2.400	0.833	0.015	0.5455	4.0628	14.529
L17R	0.680	0.615	0.020	0.2537	1.5720	1.593
L17L	0.653	0.625	0.020	0.2511	1.5732	1.530
L18L	4.000	0.405	0.020	0.6667	2.4288	14.475
L19R	0.490	0.132	0.020	0.2333	0.6874	0.502
L20L	1.276	1.183	0.015	0.3376	3.5160	6.685
L21L	3.120	0.097	0.015	0.5612	1.4098	6.554
L22L	4.000	1.014	0.015	0.6667	4.0132	23.919
L23R	2.700	0.030	0.015	0.5294	0.7538	3.032
C06	1.766	1.429	0.015	0.7500	6.5776	17.310
L23L	1.000	0.029	0.015	0.3333	0.5457	0.813
L24	1.766	1.800	0.015	0.7500	7.3833	19.431

❖ **Simulation of Drainage Network by EPA SWMM 5.1.**

In this study, the parameters inputted are from sub-catchment, node, conduit, and rain gage. The sub-catchment area was divided into 30 sub-catchments, 37 junctions, 37 conduits and 1 outfall as shown in the figure below (figure 3.8).

Table 3.18: Sub-Catchment Property

Name	Outlet node	Area (ha)	Width (m)	Slope (%)	Imperv. %	N-Imperv	N-Perv	Dstore.Imperv (mm)	Dstore.perv (mm)
SC01	J01	1305.24	1155.08	1.91	6	0.029	0.1	1.65	3.01
SC02	J05	116.73	323.36	0.66	10	0.029	0.1	1.65	3.01
SC03	J11	202.58	386.60	0.65	10	0.029	0.1	1.65	3.01
SC04	J04	8.12	109.76	0.81	42	0.029	0.1	1.65	3.01
SC05	J03	1.54	21.07	0.55	58	0.029	0.1	1.65	3.01
SC06	J02	4.38	224.81	1.03	48	0.029	0.1	1.65	3.01
SC07	J09	5.76	215.84	0.37	55	0.029	0.1	1.65	3.01
SC08	J10	0.19	22.35	1.19	64	0.029	0.1	1.65	3.01
SC09	J13	0.87	385.86	0.88	75	0.029	0.1	1.65	3.01

SC10	J14	0.08	23.09	0.88	95	0.029	0.1	1.65	3.01
SC11	J07	0.56	561.04	2.00	75	0.029	0.1	1.65	3.01
SC12	J16	8.06	435.90	0.54	65	0.029	0.1	1.65	3.01
SC13	J15	1.02	234.57	2.31	70	0.029	0.1	1.65	3.01
SC14	J18	9.23	423.38	1.38	55	0.029	0.1	1.65	3.01
SC15	J19	0.61	225.27	0.74	80	0.029	0.1	1.65	3.01
SC16	J20	0.51	389.90	2.31	80	0.029	0.1	1.65	3.01
SC17	J30	3.18	303.08	0.95	79	0.029	0.1	1.65	3.01
SC18	J24	5.67	219.90	1.55	74	0.029	0.1	1.65	3.01
SC19	J22	2.59	186.00	2.16	76	0.029	0.1	1.65	3.01
SC20	J21	0.24	153.00	1.87	60	0.029	0.1	1.65	3.01
SC21	J23	0.33	203.56	1.25	35	0.029	0.1	1.65	3.01
SC22	J25	9.20	375.65	0.82	63	0.029	0.1	1.65	3.01
SC23	J28	0.35	219.81	2.50	75	0.029	0.1	1.65	3.01
SC24	J27	0.42	419.36	2.00	75	0.029	0.1	1.65	3.01
SC25	J31	3.56	140.26	1.57	58	0.029	0.1	1.65	3.01
SC26	J33	17.38	538.21	0.62	41	0.029	0.1	1.65	3.01
SC27	J32	0.60	93.09	1.56	70	0.029	0.1	1.65	3.01
SC28	J33	0.90	92.41	0.52	69	0.029	0.1	1.65	3.01
SC29	J34	7.09	239.63	1.69	54	0.029	0.1	1.65	3.01
SC30	J36	0.46	139.73	3.03	75	0.029	0.1	1.65	3.01

Table 3.19: Conduit Properties

Condui code	Conduit Type/Shape/Surface	Inlet node	Outlet node	length (m)	Width (m)	Depth (m)	Slope (%)	Roug hness	Location/Near to
L01R	Open/ Rectangular/Masonry	J01	J03	831	1.00	1.00	0.361	0.020	Haymale Mewcha
L02R	Open/ Rectangular/Masonry	J02	J03	237	1.00	0.93	0.422	0.020	Haymale Mewcha
L03L	Open/ Rectangular/Masonry	J03	J12	71.9	1.00	1.17	0.834	0.020	Edget Jerba
L04R	Open/ Rectangular/Concrete	J10	J11	207	1.20	1.25	0.242	0.015	Shashene Mewcha
L05L	Open/ Rectangular/Masonry	J09	J11	149	0.65	0.95	0.336	0.020	Dashn Bank
L06R	Open/ Rectangular/Concrete	J11	J12	461	0.68	1.02	0.976	0.015	Menaharya
L06L	Open/ Rectangular/Concrete	J13	J14	534	0.68	0.95	0.562	0.015	Menaharya
L07L	Open/ Rectangular/Concrete	J05	J06	398	0.74	1.00	0.251	0.015	Hayat Dabo Megagerya
L07R	Open/ Rectangular/Concrete	J07	J08	396	0.70	1.00	0.278	0.015	Hayat Dabo Megagerya
L08R	Open/ Rectangular/Masonry	J04	J06	20	0.70	0.85	1.000	0.020	Haymale Adebabay
C01	Closed/ Rectangular/Concrete	J06	J08	12	1.50	0.60	0.833	0.015	Haymale Adebabay

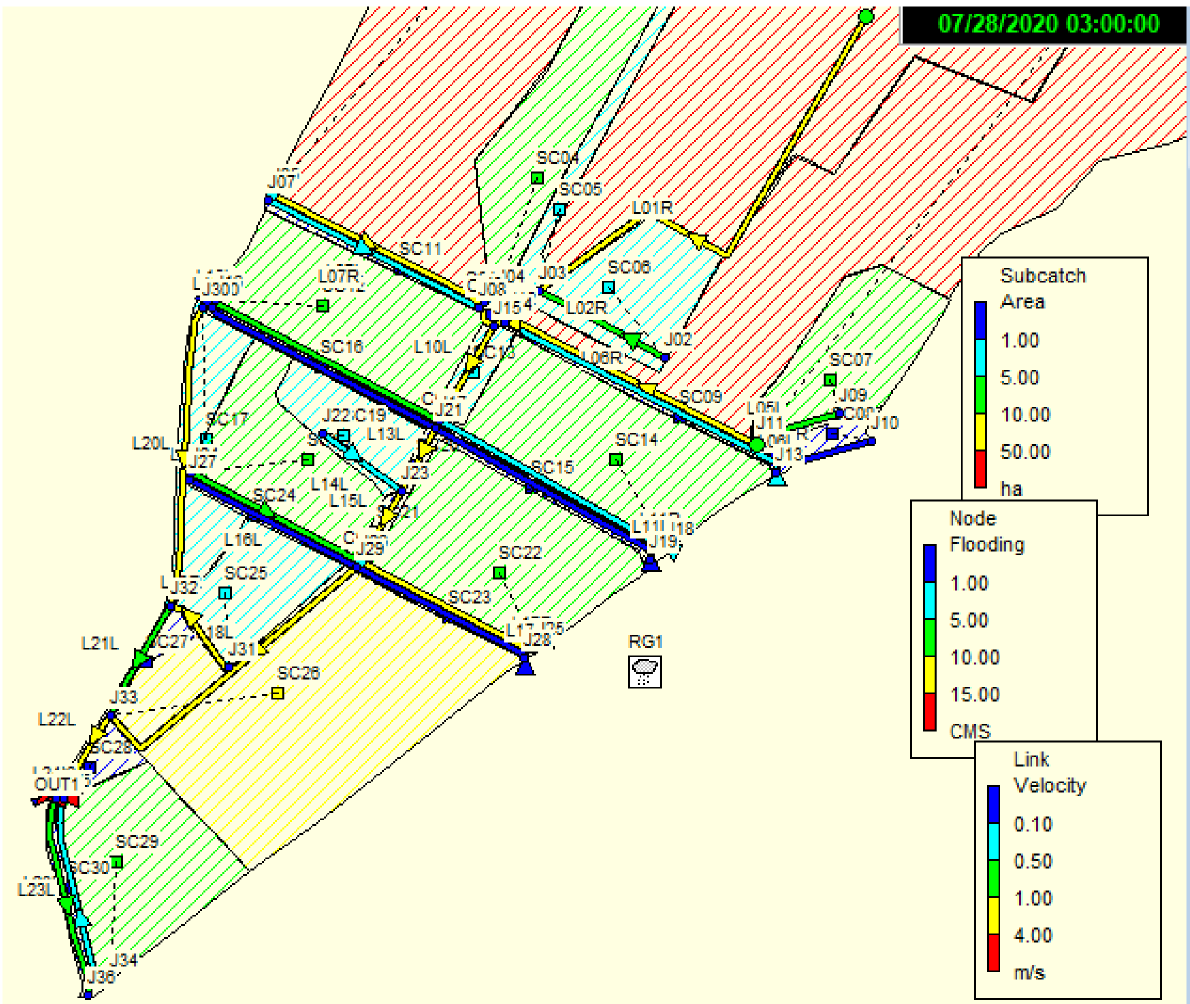
C02	Closed/ Rectangular/Concrete	J08	J15	36	0.70	1.15	2.500	0.015	Haymale Adebabay
C03	Closed/ Rectangular/Concrete	J12	J14	12	1.20	1.00	2.500	0.015	Haymale Adebabay
L09L	Open/ Rectangular/Masonry	J14	J15	18.3	1.40	1.40	1.093	0.020	Haymale Adebabay
L10L	Open/ Rectangular/Masonry	J15	J17	186	1.50	1.50	1.613	0.020	Haymale Adebabay
L11R	Open/ Rectangular/Masonry	J18	J17	432	0.86	1.15	0.463	0.020	Jemal Res. House
C04	Closed/ Rectangular/Concrete	J17	J21	14	2.00	1.00	0.714	0.015	Dubay one fashion
L12L	Open/ Rectangular/Masonry	J16	J17	425	0.90	1.05	0.471	0.020	Dubay one fashion
L12R	Open/ Rectangular/Masonry	J20	J21	425	0.85	1.00	0.471	0.020	Ommo Micro finance
L11L	Open/ Rectangular/Masonry	J19	J17	426	0.86	1.13	0.469	0.020	Jemal Res. house
L13L	Open/ Rectangular/Masonry	J17	J23	121	1.50	1.90	0.331	0.020	lowee Condominium
L14L	Open/ Rectangular/Masonry	J22	J23	162	0.80	1.06	0.617	0.020	Deneke Molla Res. House
L15L	Open/ Rectangular/Masonry	J23	J26	136	1.50	1.90	0.735	0.020	Gubae Egziabiher Church
L16L	Open/ Rectangular/Masonry	J24	J26	324	0.66	1.07	0.309	0.020	Ker Tej House/Enkutatash mgb
L16R	Open/ Rectangular/Masonry	J27	J29	320	0.66	1.07	0.313	0.020	Ker Tej House/Enkutatash mgb
C05	Closed/ Rectangular/Concrete	J26	J29	12	2.00	1.20	0.833	0.015	Green View Hotel
L17R	Open/ Rectangular/Masonry	J28	J26	325	0.68	1.00	0.615	0.020	Green View Hotel
L17L	Open/ Rectangular/Masonry	J28	J29	320	0.68	0.96	0.625	0.020	Green View Hotel
L18L	Open/ Rectangular/Masonry	J29	J33	543	1.60	2.00	0.405	0.020	Mender 39 Bridge
L19R	Open/ Rectangular/Masonry	J31	J32	152	0.70	0.70	0.132	0.020	Eden Wuha Akefafay
L20L	Closed/ Rectangular/Concrete	J30	J32	507	0.88	1.45	1.183	0.015	Commercial Bank/Market
L21L	Closed/ Rectangular/Concrete	J32	J33	207	1.56	2.00	0.097	0.015	Zagol Nedaj Madeya
L22L	Closed/ Rectangular/Concrete	J33	J35	163	1.60	2.00	1.014	0.015	Halaba View Hotel
L23R	Closed/ Rectangular/Concrete	J34	J35	335	1.50	1.80	0.030	0.015	Nafkot Pension
C06	Closed/ Circular/Concrete	J35	J37	14	Dia = 1.5		1.429	0.015	ST Gabriel
L23L	Closed/ Rectangular/Concrete	J36	J37	345	1.00	1.00	0.029	0.015	Nafkot Pension
L24	Closed/ Circular/Concrete	J37	OUT1	14	Dia = 1.5		2.857 1	0.015	ST Gabriel

Table 3.20: Node property

Name	Type	X(m)	Y(m)	Max. Depth(m)	Invert elvn. (m)
J01	Junction	400433.08	808923.33	0.90	1783.1
J02	Junction	400098.7	808349.21	0.90	1780.1
J03	Junction	399890.72	808462.62	0.90	1779.1
J04	Junction	399818.82	808448.95	0.80	1779.2
J05	Junction	399449.83	808624.76	1.00	1780
J06	Junction	399802.64	808440.4	1.00	1779
J07	Junction	399444.03	808624.76	1.00	1780
J08	Junction	399795.58	808430.67	1.20	1778.8
J09	Junction	400386.16	808250.65	0.50	1783
J10	Junction	400443.59	808204.41	0.50	1783
J11	Junction	400249.54	808204.17	0.80	1782.2
J12	Junction	399840.04	808419.54	1.40	1778.6
J13	Junction	400282.54	808152.41	1.00	1782
J14	Junction	399832	808409	1.60	1778.4
J15	Junction	399815	808405	2.00	1778
J16	Junction	399351.64	808446.03	1.00	1778
J17	Junction	399725.54	808241.99	2.00	1775
J18	Junction	400102.12	808032.42	1.00	1779
J19	Junction	400072.24	808008.67	1.00	1778.5
J20	Junction	399344.81	808430.9	1.00	1779
J21	Junction	399717	808227	2.10	1774.9
J22	Junction	399535.87	808224.11	1.00	1776
J23	Junction	399662	808121	1.50	1774.5
J24	Junction	399309.48	808159.57	1.00	1776
J25	Junction	399883.61	807857.68	1.00	1776
J26	Junction	399593	808004	1.50	1773.5
J27	Junction	399305.15	808148.07	1.00	1775
J28	Junction	399861.84	807842.56	1.00	1776
J29	Junction	399585	807995	1.60	1773.4
J30	Junction	399330.84	808432.07	1.60	1777.4
J31	Junction	399374.92	807827.55	1.00	1773
J32	Junction	399278	807932	1.60	1772.4
J33	Junction	399177	807746	1.80	1771.2
J34	Junction	399176.01	807296.81	1.00	1768
J35	Junction	399100	807606	2.10	1767.9
J36	Junction	399139.93	807271.58	0.80	1768.2
J37	Junction	399085	807609	2.20	1767.8
OUT	Outfall	399072.18	807607.69		1767.4

As there is no simulation made by any model for the study area, the peak discharge value of rational and SCS is used to adjust the sensitive parameters. Accordingly; D-store-impervious, D-store-pervious, N-impervious and N- pervious are adjusted to be 1.65, 3.01, 0.029 and 0.1 respectively. The simulation was done by using 3hours rainfall intensity with a 5minute time interval and the remaining parameter values of sub-catchment, link and node used for simulation are used accordingly.

(a) Full study area (b) Study area after diversion work is done



(c) Detail part of town

Figure 3.8: Map of Sub-Catchment, link and Conduit property by SWMM

Model outcomes

- **Peak discharge of each sub-catchments for return period of 2, 10, 25, 50 and 100**

The statistical analysis by SWMM is performed for the study area and peak discharge generated from sub-catchments is shown in the table below (Table 3.21).

Table 3.21: Peak discharge result of SWMM for return periods of 2, 5, 10, 25, 50 and 100 years

SC-Code	Q m^3/s					
SC01	15.42	21.37	26.27	33.17	38.69	44.36
SC02	2.2	2.99	3.63	4.51	5.21	5.91
SC03	3.18	4.31	5.23	6.52	7.54	8.58
SC04	0.55	0.69	0.8	0.95	1.07	1.18
SC05	0.12	0.16	0.18	0.21	0.24	0.26
SC06	0.43	0.54	0.62	0.74	0.82	0.91
SC07	0.55	0.68	0.79	0.93	1.04	1.14
SC08	0.03	0.03	0.04	0.04	0.05	0.05
SC09	0.16	0.19	0.22	0.26	0.28	0.31
SC10	0.02	0.02	0.02	0.03	0.03	0.03
SC11	0.12	0.14	0.16	0.18	0.2	0.21
SC12	0.97	1.2	1.37	1.61	1.79	1.98
SC13	0.16	0.19	0.22	0.26	0.29	0.32
SC14	1.02	1.27	1.46	1.72	1.92	2.12
SC15	0.12	0.14	0.16	0.19	0.21	0.22
SC16	0.11	0.14	0.15	0.17	0.19	0.21
SC17	0.51	0.63	0.72	0.84	0.93	1.02
SC18	0.77	0.96	1.1	1.29	1.43	1.57
SC19	0.4	0.49	0.57	0.66	0.73	0.8
SC20	0.04	0.05	0.06	0.06	0.07	0.08
SC21	0.06	0.07	0.08	0.09	0.1	0.11
SC22	1.1	1.36	1.57	1.83	2.04	2.25
SC23	0.07	0.09	0.1	0.11	0.12	0.13
SC24	0.09	0.11	0.12	0.14	0.15	0.16
SC25	0.41	0.5	0.58	0.68	0.76	0.84
SC26	1.33	1.67	1.94	2.31	2.6	2.89
SC27	0.09	0.11	0.13	0.15	0.16	0.18
SC28	0.12	0.15	0.18	0.21	0.23	0.25
SC29	0.75	0.93	1.07	1.26	1.41	1.56
SC30	0.09	0.11	0.12	0.14	0.15	0.17

- **Flooding Results**

The flooding volume and flooding hours of flooded junctions for different return periods is summarized in the table below (Table 3.22).

Table 3.22: Flooded junctions for return period of 2, 5, 10, 25, 50 and 100 years

No. of flooded junctions	flooded nodes	Flooding hours	Flood volume (10^6 ltr)	No. of flooded junctions	flooded nodes	Flooding hours	Flood volume (10^6 ltr)	No. of flooded junctions	flooded nodes	Flooding hours	Flood volume (10^6 ltr)
2 Years return period				**5 years return period**				**10 years return period**			
1	J01	2:55	114.9	1	J01	2:55	158.6	1	J01	2:55	194.6
2	J05	2:55	10.5	2	J05	2:55	16.7	2	J05	2:55	21.8
3	J11	2:51	15.7	3	J11	2:52	25.3	3	J09	0:30	0.1
4	J17	0:25	2.1	4	J17	0:25	5.7	4	J11	2:55	33.2
5	J18	0:05	0.0	5	J18	0:04	0.0	5	J17	0:25	8.9
6	J25	0:25	0.2	6	J21	0:15	0.0	6	J18	0:04	0.0
7	J26	0:20	15.4	7	J25	0:25	0.8	7	J21	0:14	0.0
8	J34	0:26	1.6	8	J26	0:18	19.8	8	J24	0:20	0.0
9	J36	0:24	7.7	9	J34	0:23	4.7	9	J25	0:25	1.4
25 years return period				10	J36	0.015	8.3	10	J26	0:17	22.3
1	J01	2:55	245.6	**50 years return period**				11	J34	0:21	7.5
2	J05	2:55	28.9	1	J01	2:55	286.5	12	J35	0:18	0.0
3	J09	0:30	0.5	2	J05	2:55	34.6	13	J36	0.01	8.7
4	J11	2:54	43.9	3	J09	0:30	0.9	**100 years return period**			
5	J17	0:26	13.5	4	J11	2:53	52.4	1	J01	2:55	328.7
6	J18	0:04	0.2	5	J17	0:25	17.1	2	J05	2:55	40.4
7	J21	0:12	0.0	6	J18	0:03	0.6	3	J09	0:30	1.3
8	J24	0:20	0.3	7	J21	0:12	0.0	4	J11	2:53	61.1
9	J25	0:25	2.3	8	J24	0:20	0.6	5	J17	0:25	20.8
10	J26	0:15	24.6	9	J25	0:25	3.1	6	J18	0:03	1.0
11	J31	0:20	0.0	10	J26	0:15	25.9	7	J21	0:11	0.0
12	J34	0:25	11.5	11	J31	0:20	0.2	8	J24	0:20	0.9
13	J35	0:16	0.0	12	J34	0:24	14.5	9	J25	0:20	4.0
14	J36	0:17	9.1	13	J35	0:15	0.1	10	J26	0:14	26.8
				14	J36	0:17	9.3	11	J31	0:20	0.3
								12	J34	0:23	17.6
								13	J35	0:14	0.1
								14	J36	0:16	9.5

As we can see from table 3.22, the number of junctions which are flooded is 9, 10 and 13 for a return period of 2, 5, and 10 years respectively and for remaining return periods (25, 50 and 100 years) 14 junctions are flooded with varying time and volume amount.

Among different paths of links path, J01-OUT1 and J26-OUT1 of 10 years return period is shown in the figure below (figure 3.9 and figure 3.10 respectively).

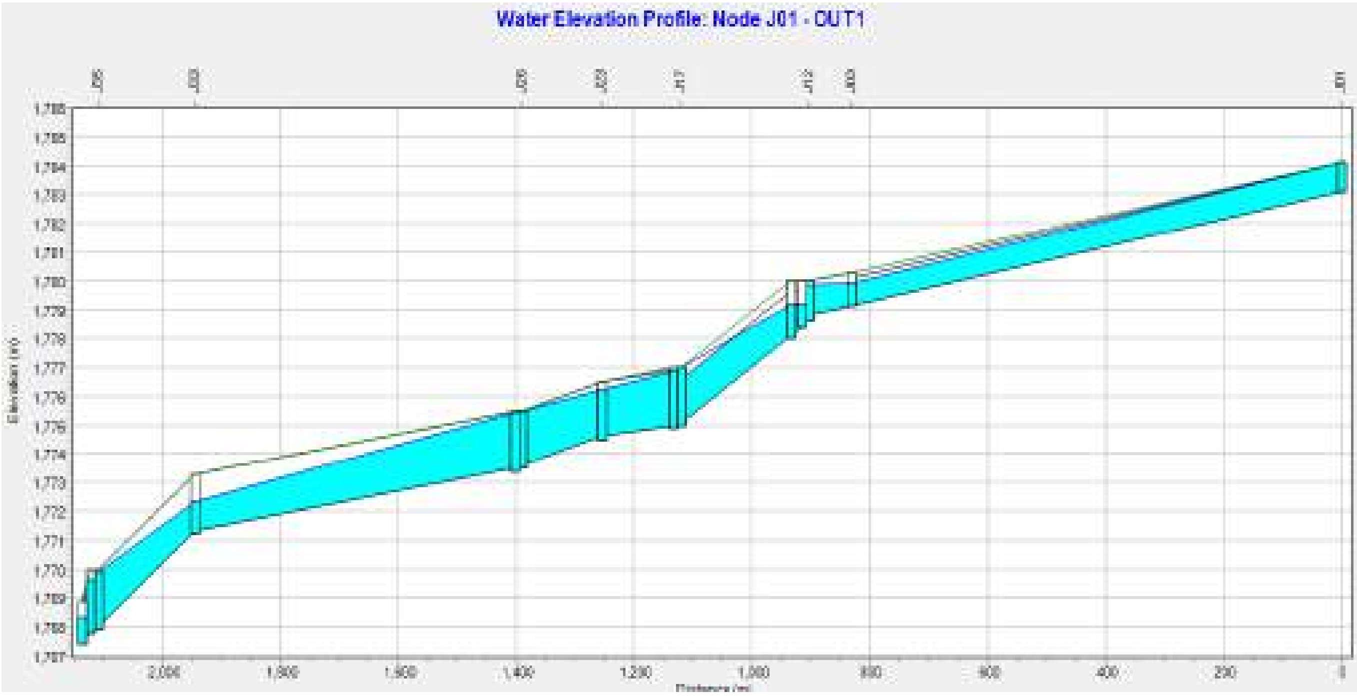


Figure 3.9: 10 years return period flooded junctions of path J01-OUT1

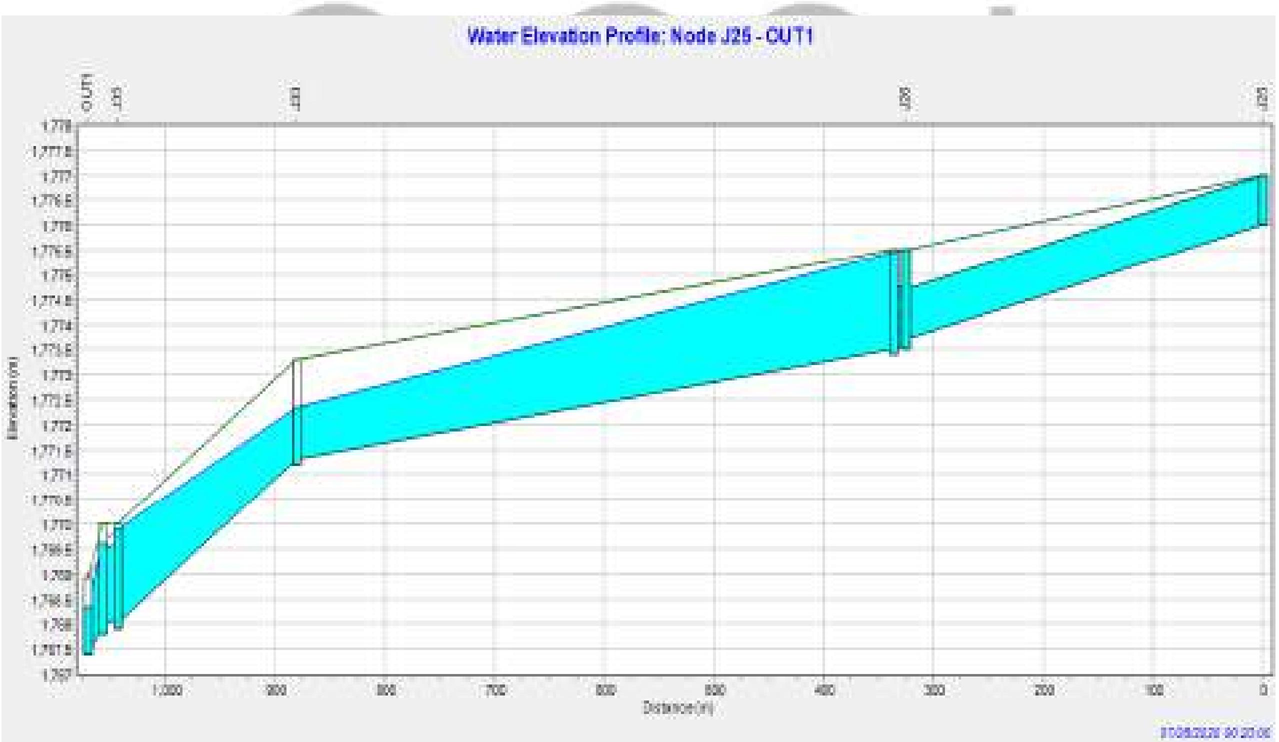


Figure 3.10: 10 years return period flooded junctions of path J26-OUT1

3.2.1 Simulated and Estimated Peak Discharge Comparison

The peak discharge result of EPA SWMM 5.1 and rational/SCS method for a different return period of each sub-catchment shows nearly the same result which indicates that the sensitive parameters are adjusted in a well manner having correlation coefficient greater than 0.99 for return periods of 2, 5, 10, 25, 50 and 100 years as shown in the table below (table 3.23) and figure 3.11.

Table 3.23: Peak discharge comparison for return period of 10, 25 and 50 years

SC Code	Generated Q_{10} (m^3/s)		Generated Q_{25} (m^3/s)		Generated Q_{50} (m^3/s)	
	SCS/ Rational	SWMM	SCS/ Rational	SWMM	SCS/ Rational	SWMM
SC01	26.28	26.27	32.57	33.17	38.05	38.69
SC02	2.84	3.63	3.68	4.51	4.28	5.21
SC03	4.64	5.23	5.92	6.52	6.78	7.54
SC04	0.86	0.8	1.07	0.95	1.27	1.07
SC05	0.19	0.18	0.23	0.21	0.27	0.24
SC06	0.66	0.62	0.82	0.74	0.97	0.82
SC07	0.75	0.79	0.94	0.93	1.11	1.04
SC08	0.04	0.04	0.06	0.04	0.07	0.05
SC09	0.2	0.22	0.25	0.26	0.3	0.28
SC10	0.02	0.02	0.03	0.03	0.04	0.03
SC11	0.16	0.16	0.2	0.18	0.24	0.2
SC12	1.45	1.37	1.81	1.61	2.14	1.79
SC13	0.28	0.22	0.35	0.26	0.42	0.29
SC14	1.47	1.46	1.83	1.72	2.16	1.92
SC15	0.16	0.16	0.21	0.19	0.24	0.21
SC16	0.15	0.15	0.19	0.17	0.23	0.19
SC17	0.73	0.72	0.91	0.84	1.08	0.93
SC18	1.1	1.1	1.37	1.29	1.63	1.43
SC19	0.61	0.57	0.76	0.66	0.9	0.73
SC20	0.06	0.06	0.08	0.06	0.1	0.07
SC21	0.09	0.08	0.11	0.09	0.13	0.1
SC22	1.58	1.57	1.97	1.83	2.34	2.04
SC23	0.1	0.1	0.13	0.11	0.15	0.12
SC24	0.13	0.12	0.16	0.14	0.19	0.15
SC25	0.61	0.58	0.76	0.68	0.91	0.76
SC26	1.98	1.94	2.46	2.31	2.92	2.6
SC27	0.15	0.13	0.19	0.15	0.23	0.16
SC28	0.21	0.18	0.26	0.21	0.3	0.23
SC29	1.03	1.07	1.28	1.26	1.52	1.41
SC30	0.11	0.12	0.13	0.14	0.16	0.15

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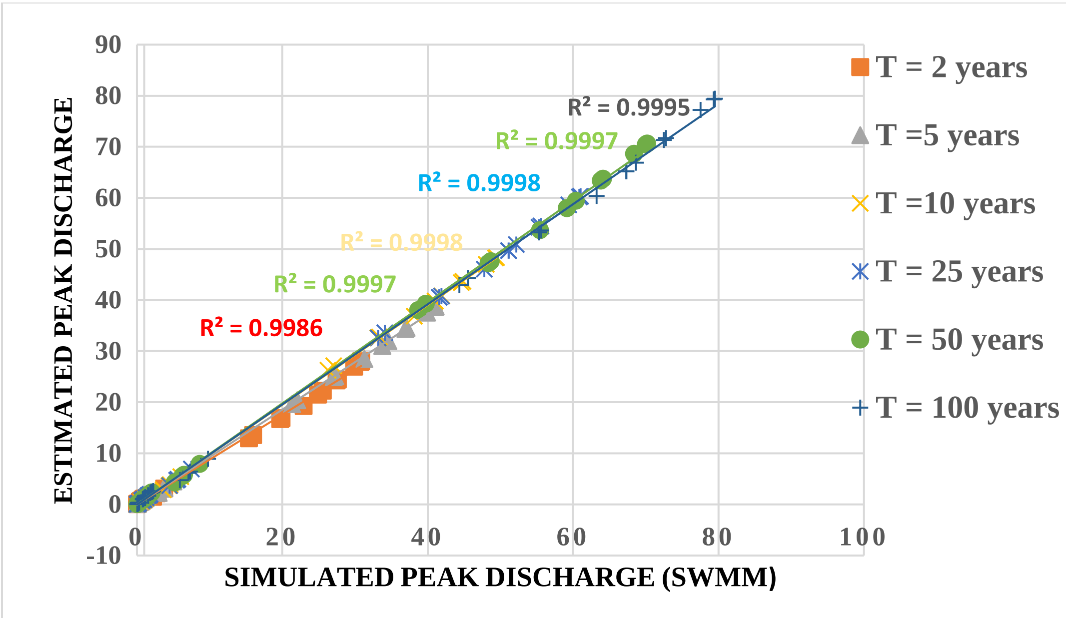


Figure 3.11: Correlation Value of Simulated and Estimated Peak Discharges

3.2.2 Adequacy of Existing Drainage System

Here there are canals which carry the runoff from more than one sub-catchment. The canals conveying runoff from single sub-catchment (according to the sub-catchment classification that I made) are; L01R, L02R, L04R, L05L, L06L, L07L, L07R, L08R, L11R, L12R, L12L, L11L, L14L, L16L, L16R, L17L, L17R, L19R, L20L, L23L and L23R. The remaining canals carry the runoff which comes from the upper sub-catchments and for comparison purposes, their values are added together and compared whether they are capable enough to convey the coming runoff or not.

The canals hydraulic capacity is calculated by using Manning's equation and compared with the SWMM model output and SCS/Rational method of estimated peak runoff of the given sub-catchment contributing to their respective coded canals. The comparison for the return period of 10 and 25 is done for SWMM and SCS/Rational methods with hydraulic capacity results done by Manning's method as shown in table 3.24.

Table 3.24: Comparison of Manning's Peak discharge with output of SWMM and Rational/SCS Method

Conduit code	Contributing Sub-catchment/junction	Q10 (CMS)		Q25 (CMS)		Manning's	Remark
		SCS/ Rational	SWMM	SCS/ Rational	SWMM		
L01R	J01	26.28	26.27	32.57	33.17	2.15	Inadequate
L02R	J02	0.66	0.62	0.82	0.74	2.13	adequate
L03L	J03	27.13	27.07	33.62	34.12	3.96	Inadequate
L04R	J10	0.04	0.04	0.06	0.04	4.01	adequate
L05L	J09	0.75	0.79	0.94	0.93	1.04	adequate
L06R	J11	3.64	4.46	4.68	5.48	2.74	Inadequate
L06L	J13	0.20	0.22	0.25	0.26	1.91	adequate
L07L	J05	4.64	5.23	5.92	6.52	1.54	Inadequate
L07R	J07	0.16	0.16	0.20	0.18	1.49	adequate

L08R	J04	0.86	0.80	1.07	0.95	1.75	adequate
C01	J06	5.50	6.03	6.99	7.47	3.92	Inadequate
C02	J08	10.14	11.26	12.91	13.99	5.26	Inadequate
C03	J12	30.76	31.53	38.30	39.60	9.80	Inadequate
L09L	J14	30.99	31.77	38.58	39.89	9.18	Inadequate
L10L	J15	41.41	43.25	51.84	54.14	13.41	Inadequate
L11R	J18	1.47	1.46	1.83	1.72	2.31	adequate
C04	J17	44.33	46.08	55.48	57.47	10.58	Inadequate
L12L	J16	1.45	1.37	1.81	1.61	2.24	Inadequate
L12R	J20	0.14	0.14	0.14	0.14	1.94	Inadequate
L11L	J19	0.16	0.16	0.21	0.19	2.28	adequate
L13L	J17	44.70	46.44	55.91	57.86	9.44	Inadequate
L14L	J22	0.61	0.57	0.76	0.66	2.18	adequate
L15L	J23	45.40	47.09	56.78	58.61	14.08	Inadequate
L16L	J24	1.10	1.10	1.37	1.29	1.17	adequate
L16R	J27	0.13	0.12	0.16	0.14	1.17	Inadequate
C05	J26	48.08	49.76	60.12	61.73	14.53	adequate
L17R	J28	1.58	1.57	1.97	1.83	1.59	adequate
L17L	J28	0.10	0.10	0.13	0.11	1.53	adequate
L18L	J29	50.28	51.92	62.87	64.29	14.48	Inadequate
L19R	J31	0.61	0.58	0.76	0.68	0.50	Inadequate
L20L	J30	0.73	0.72	0.91	0.84	6.68	adequate
L21L	J32	1.43	1.38	1.78	1.61	6.55	adequate
L22L	J33	51.92	53.48	64.91	66.11	23.92	Inadequate
L23R	J34	1.03	1.07	1.28	1.26	3.03	adequate
C06	J35	52.95	54.55	66.19	67.37	17.31	Inadequate
L23L	J36	0.11	0.12	0.13	0.14	0.81	adequate
L24	J37	53.06	54.67	66.32	67.51	19.43	Inadequate

As shown in the table 3.24, it is clear that there are drainage lines which are incapable to convey the runoff generated from both the town and rural area (out of the town). SC01 contributes the majority of flood as it emanated from Rekame hill and crosses about 10km of rural agricultural land and gets the inlet around Zonal Hospital of Alaba Kulito. Next to SC01; SC03 and SC02 hold successive ranks in which most of their runoff is also generated from the rural region near to suburban areas of North of Alaba Kulito Town.

3.3 Flood causes of Alaba Kulito Town identified by this research

- **Incapable Drainage Structure to Convey the Coming Runoff from Rural Area**

The hydrologic and hydraulic analysis of the study area results all main and some sub-main canals of the drainage network are not able to convey the peak discharge which can be produced possibly. This is due to the incapability of the size of links which are loaded to convey much runoff from rural area crossing both Lenda Ber (upper part) and Mahal Arada (lower part) through seriously connected drainage networks. The drainage lines which are not capable enough to convey even 2 year return period peak discharge are lines out of LO2R, L04R, L05L, L06L, L07R, L08R, L11R, L11L, L14L, L16L, L17R, L17L, L20L, L21L, L23R and L23L.

❖ Neglecting collector and feeder Drainage Structures

In addition to the insufficiency of the existing drainage canals, the drainage service of the town is not well connected which means by, collector drainage services are neglected to allow the overcoming flood to pass arbitrarily through the town. This impedes the comfort of dwellers: by inundating in depressions, by entering the individual blocks, by creating pondages which is safe for production of insects causing malaria and related diseases, hindering harmonious movement after heavy rain (in the town and upper catchment of Rekame hill) and related effects on both Lenda Ber and Mahal Arada kebele dwellers. The access roads are also acting as canals with washing the upper layer of soil (erosion) and aggravating flood occurrence.

Figure 3.12: Effect of neglecting collector Drainage Systems around Lenda Ber(left side) and Mahal Arada (right side)

❖ Blockage of drainages by dry waste materials and maintenance problem

From field visit, some drainage lines of open and closed canals are field by dry wastes of households, shops and street debris materials which are able to reduce the capability of canals conveyance system by the amount of volume they occupied. Some canals are damaged which lets the chance of runoff to deviate from the conveyance line to the street and then to houses.

Figure 3.13: Blocked closed canal (right side), Blocked open canal (left side) and Damaged canal (middle)

The other problem observed along closed canals of main streets is insufficient opening and closure of manhole openings by mud and garbage materials which prevents the entrance of runoff from the street into drainage canal through provided manhole openings.

Figure 3.14: Manhole opening closed by garbage and less depth of opening

3.4 Remedial Measures to be taken

3.4.1 Diverting the Upper Catchment Runoff to Bilate River

The sub-catchments which are reduced to diversion work are SC01 and SC03 while others are not affected due to diversion. Here both values of area and percentage coverage are changed. The area of SC01 and SC03 are reduced from 1305.24ha and 202.57ha to 272.309ha and 197.653ha respectively with total area reduced from 1717.46ha to 673.306ha that is the runoff of about 1044ha land can be reduced from flowing to town.

By doing this, runoff magnitude of SC01 and SC03 of 10 years return period can be reduced from 26.28 m3/s to 6.51 and 5.23 to 4.64 respectively which is 20.35 m^3/s of total peak discharge load is diverted to Bilate River at the North-West of Alaba Kulito Town as shown in the figure below (figure 3.15).

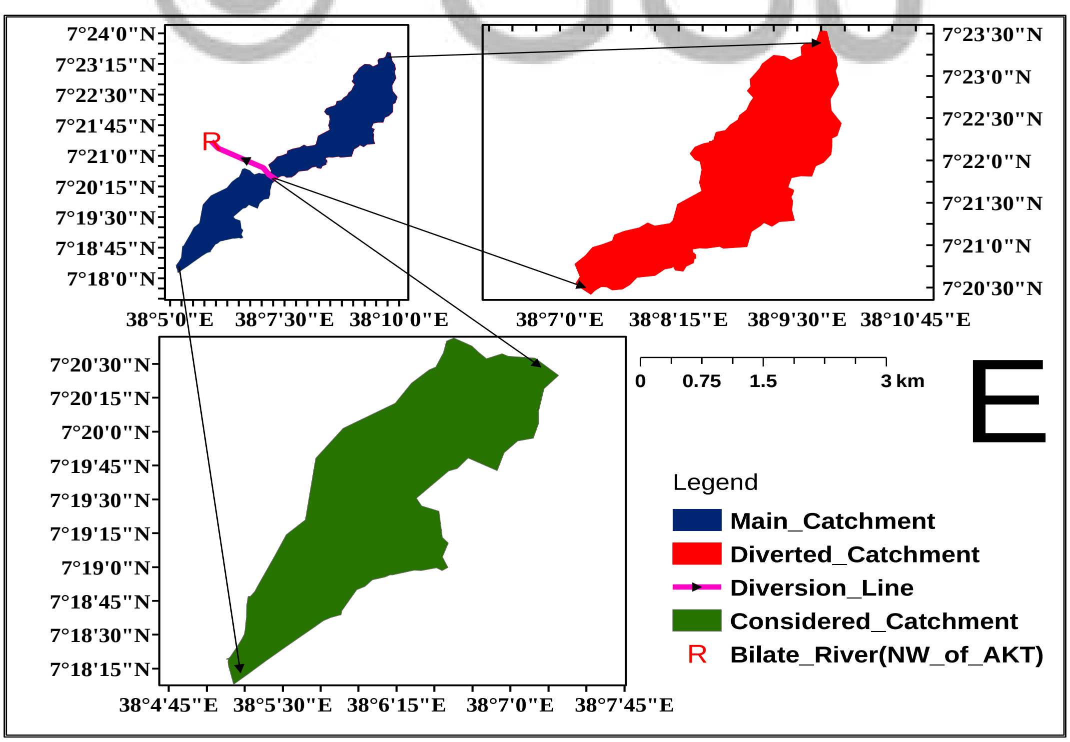


Figure 3.15: Diverted and considered Catchment of study area

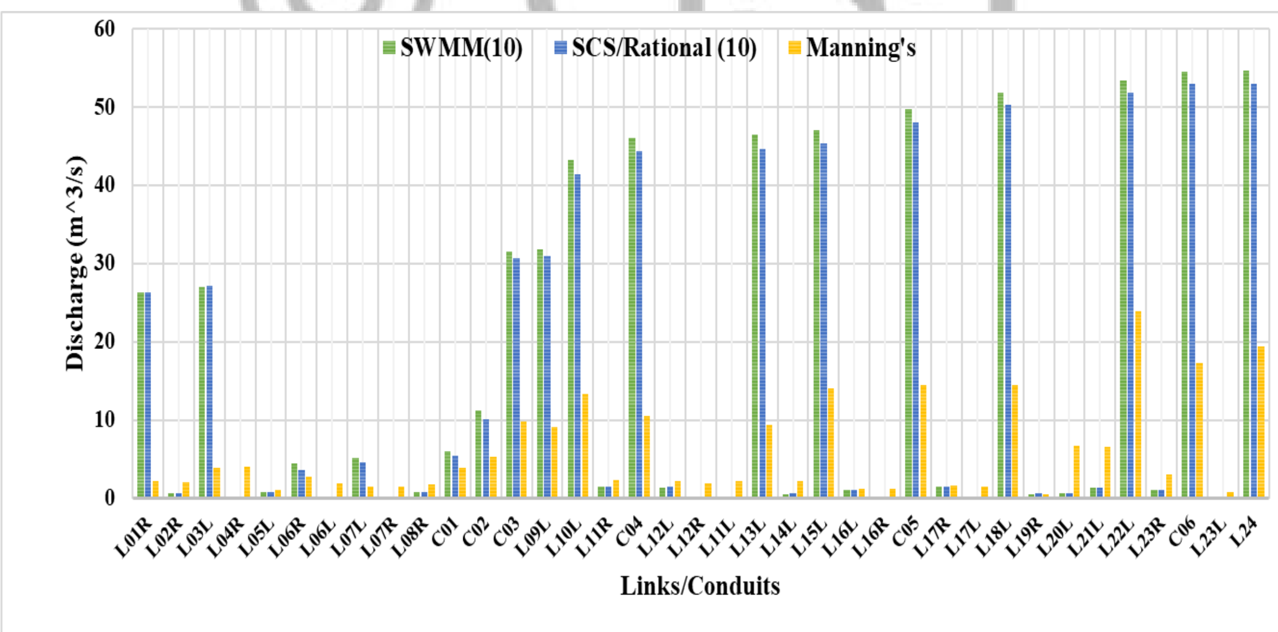
The total difference which can be obtained by diverting the upper catchment is illustrated in the table below (Table 3.25).

Table 3.25: Change of Peak discharge due to Diversion

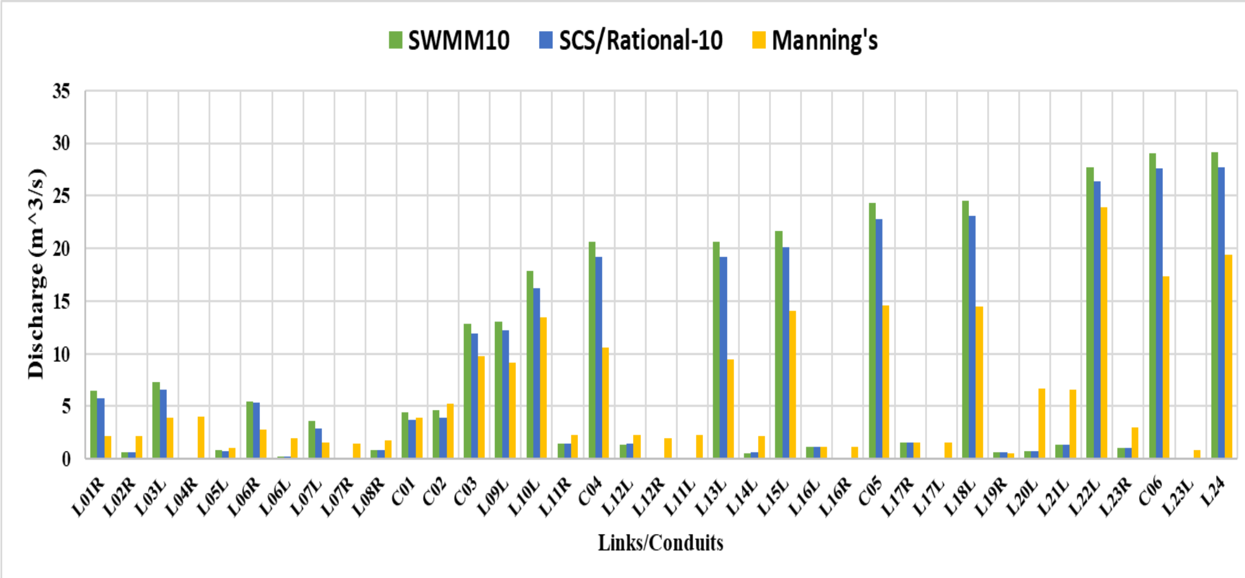
Return period (yr)		2	5	10	25	50	100
SC01	Qo (m^3/s)	15.42	21.37	26.27	33.17	38.69	44.36
	Qr (m^3/s)	3.84	5.32	6.51	8.15	9.45	10.75
	Qd (m^3/s)	11.58	16.05	19.76	25.02	29.24	33.61
Ssc03	Qo (m^3/s)	3.18	4.31	5.23	6.52	7.54	8.58
	Qr (m^3/s)	2.8	3.82	4.64	5.77	6.66	7.57
	Qd (m^3/s)	0.38	0.49	0.59	0.75	0.88	1.01
Total Qd		11.96	16.54	20.35	25.77	30.12	34.62
Diverted Discharge (%)		64.30	64.41	64.60	64.93	65.15	65.39
Where:- Qo, Qr & Qd are peak discharge of original, remaining and diverted respectively							

Without diverting the upper catchment runoff, the existing canal capacity and peak discharge values to be conveyed deviates by much amount. As we have seen from table 3.25, diversion work shares more than 64% of runoff load which indicates that it is a sounded solution.

The following figure shows a clear change of runoff load on links due to proposed diversion work. The load difference for all conduits, for all methods (SWMM, SCS/Rational methods and Manning's method) and return periods of 10 yeasr. Figure 4.18 is plotted by using 10 years return period of runoff load done by SWMM and SCS/Rational method of discharge analysis comparing with the hydraulic capacity of existing drainage conduits.



a) Without diversion work



b) If diversion work is done

Figure 3.16: Discharge result comparison of SWMM, SCS/Rational and Manning's method

The amount of peak discharge (total Qd of table 3.25) are used and canal dimensions are calculated by using the maximum permissible velocity method of erodible canals at which the design canal can function with compromised velocity without exposing to erosion.

3.4.2 Strengthening Soil Conservation and Afforestation Work at the upper catchment

As runoff is the combined effect of the response of land-use /land-cover type, soil type, terrain or slope and the precipitation condition; there is the opportunity to change land cover of the land which is not irrigable and still it is possible to do soil conservation work on both cultivated and uncultivated land throughout the catchment by sustaining the started green legacy program in a strong manner. By doing so, the curve number and/or runoff coefficient magnitude can be reduced which is directly linked to runoff generation.

3.4.3 Constructing Additional Drainage Canals

As collector canals are the canals which collects and dispose the runoff to the main drainage canal, it is reasonable to provide such canals through the town. The upper Lenda Ber kebele (above Aymale Adebabay) needs a number of well-connected collector drainage canals to keep the town and dwellers from the adverse effects mentioned above. Similarly, both wings of the zonal hospital of Alaba Kulito town need adequate feeder drainage canal which can share the load from link coded as L01. Likewise, the Eastern part of Mahal Arada (including sides of the main road of Shashemene-Alaba Kulito-Sodo way) needs collector canals and main canals which can convey runoff from SC02 and roadsides drainage to Bilate River. By doing these the town can be kept from stormwater retention and environmental pollution with keeping an aesthetic view of the developing zonal city (ALaba Kulito Town).

3.4.4 Adapting appropriate dry waste material removal

Illegal dumping of solid wastes like plastics, chat remains, and other waste of households should not be disposed into stormwater drainage lines instead it should be collected separately and disposed into the prepared waste disposing place. Therefore, awareness should be created regarding to waste disposing and environmental protection, there must be a collaborated approach of appropriate dry waste removing mechanism among the dwellers and government body and individuals must be responsible for the aesthetic view of the environment that they are living.

In fact, Alaba Kulito Town administration do have appreciable dry waste collecting and removal trend. It do have a cart and manpower which is intended to clean the drainage canal. However, the town administration only cannot keep the whole drainage lines clean always. It should be an assignment of each dweller and town administration together with a cooperative manner.

3.4.5 Connecting Outfall with Bilate River by Lined Trapezoidal canal

The high amount of runoff generated from the upper catchment is allowed to pass through the outfall located near to ST Gabriel Church. The peak discharge which should be conveyed through outfall is the summation of each sub-catchments i. e. Q_2 = 18.88m^3/s, Q_5 = 24.57 m^3/s, Q_{10} = 29.10 m^3/s, Q_{25} = 35.25 m^3/s, Q_{50} = 40.06 m^3/s and Q_{100} = 44.88m^3/s.

There is a naturally existing gully which is being eroded due to high flood coming from the upper catchment and eroding the bed and banks of the formed gully. I observe that there are buildings and live on sides of the gully which needs to be protected. Therefore, outfall starting from ST Gabriel Church up to Bilate River should be changed from natural earthen canal to manmade masonry trapezoidal canal which can save the assets and life on both sides of canal, the soil which is being eroded can be conserved and the stormwater can be removed into the natural river (Bilate River) in a safe manner.

For this case (non-erodible trapezoidal canal); Manning's equation is used and to calculate the dimension of section factors (AR$^{2/3}$) containing bed width (b in m), flow depth (d in m), side slope (z in m/m), wetted perimeter (p in m), freeboard which is 20 percent of flow depth (d' in m), total depth (D in m) and top width (B in m) are done by using constants (manning's roughness coefficient (n) and side slope) and longitudinal slope (S in m/m) from elevation difference and canal length with fixing bed width and solving for flow depth by iteration until the cross parameters agree to each other and able to convey the design discharge.

Table 3.27: Discharge of corresponding return periods of canal dimension of common outlet

b	d	z	A	P	R	R^2/3	n	S	V	Q	Comment	d'	D	B
Q_2 = 18.88m3/s														
1	2	1	6	3.83	1.57	1.35	0.03	0.0141	5.35	32.08	too much	0.40	2.40	5.80
1	1	1	2	2.41	0.83	0.88	0.03	0.0141	3.49	6.99	too small	0.20	1.20	3.40
1	1.5	1	3.75	3.12	1.20	1.13	0.03	0.0141	4.48	16.79	slightly small	0.30	1.80	4.60
1	1.58	1	4.08	3.23	1.26	1.17	0.03	0.0141	4.62	18.84	O.K.	0.32	1.90	4.79
Q_5 = 24.57m3/s														
1.2	2	1	6.40	4.03	1.59	1.36	0.03	0.0141	5.39	34.53	too much	0.40	2.40	6.00
1.2	1	1	2.20	2.61	0.84	0.89	0.03	0.0141	3.53	7.77	too small	0.20	1.20	3.60
1.2	1.7	1	4.93	3.60	1.37	1.23	0.03	0.0141	4.88	24.07	slightly Small	0.34	2.04	5.28
1.2	1.72	1	5.00	3.63	1.38	1.24	0.03	0.0141	4.91	24.57	O.K.	0.34	2.06	5.32
Q_{10} = 29.10m3/s														
1.5	2	1	7.00	4.33	1.62	1.38	0.03	0.0141	5.46	38.21	too much	0.40	2.40	6.30
1.5	1.8	1	5.94	4.05	1.47	1.29	0.03	0.0141	5.12	30.40	slightly big	0.36	2.16	5.82
1.5	1.75	1	5.69	3.97	1.43	1.27	0.03	0.0141	5.03	28.61	slightly small	0.35	2.10	5.70

1.5	1.77	1	5.76	4.00	1.44	1.28	0.03	0.0141	5.06	29.14	O.K.	0.35	2.12	5.74
Q_{25} = 35.25m3/s														
2	**2**	**0.5**	**6.00**	**3.12**	**1.92**	**1.55**	**0.03**	**0.0141**	**6.13**	**36.78**	**slightly much**	**0.40**	**2.40**	**4.40**
2	**1.8**	**0.5**	**5.22**	**3.01**	**1.74**	**1.44**	**0.03**	**0.0141**	**5.72**	**29.88**	**too small**	**0.36**	**2.16**	**4.16**
2	**1.9**	**0.5**	**5.61**	**3.06**	**1.83**	**1.50**	**0.03**	**0.0141**	**5.93**	**33.23**	**slightly small**	**0.38**	**2.28**	**4.28**
2	**1.96**	**0.5**	**5.83**	**3.09**	**1.88**	**1.53**	**0.03**	**0.0141**	**6.05**	**35.26**	**O.K.**	**0.39**	**2.35**	**4.35**
Q_{50} =40.06m3/s														
2.3	2.3	0.5	7.94	3.59	2.21	1.70	0.03	0.0141	6.73	53.39	too much	0.46	2.76	5.06
2.3	2.1	0.5	7.04	3.47	2.03	1.60	0.03	0.0141	6.34	44.62	slightly big	0.42	2.52	4.82
2.3	1.9	0.5	6.18	3.36	1.84	1.50	0.03	0.0141	5.94	36.69	slightly small	0.38	2.28	4.58
2.3	2	0.5	6.60	3.42	1.93	1.55	0.03	0.0141	6.14	40.55	O.K.	0.40	2.40	4.70
Q_{100} = 44.88m3/s														
2.5	2.5	0.5	9.38	3.90	2.41	1.80	0.03	0.0141	7.11	66.68	too big	0.50	3.00	5.50
2.5	2	0.5	7.00	3.62	1.93	1.55	0.03	0.0141	6.15	43.06	slightly small	0.40	2.40	4.90
2.5	2.1	0.5	7.46	3.67	2.03	1.60	0.03	0.0141	6.35	47.34	slightly big	0.42	2.52	5.02
2.5	2.04	0.5	7.18	3.64	1.97	1.57	0.03	0.0141	6.23	44.75	O.K.	0.41	2.45	4.95

4 CONCLUSION

- ❖ IDF was developed by GPD (in this paper) which can be used for Hydrologic analysis and any design of hydraulic structure around Alaba Kulito by water resource professionals and concerned institutions. Because of IDF curve of GPD best represents the study area as it is developed by using long years RF data which is updated and uses the rainfall data of Alaba station and nearby stations, and that of IDF developed by ERA for Alaba Kulito is not updated and it is done by stations of Sodo, Arbaminch and Hawassa which are distant stations from Alaba Kulito, IDF done by GPD was used in this paper.
- ❖ The flood prediction made for all return periods of 2, 5, 10, 25, 50 and 100 years showed that most of the canals which carry runoff collected from upper catchments (more than one sub-catchment) and drainage lines which received runoff from wide sub-catchment coming from the rural area were totally incapable to convey the peak discharge.
- ❖ It has been found that insufficiency of existing canals size, canal length and canal number with inappropriate waste disposing system all together allows the generated runoff from a rural area (emanating from Rekame Hill) causes flood at lower catchment of Alaba Kulito Town during high intensity of rainfall with favorable antecedent moisture condition of the catchment.
- ❖ Runoff which is generating from the rural areas coming from Rekame Hill (North-West of Alaba Kulito Town) and passes through Alaba Kulito Town and then to Bilate River accounts for more than 64% of total runoff. Therefore it shall be diverted to Bilate river before it riches the town at about 3km from Alaba Zonal Hospital.
- ❖ Adequate and enough drainage canals which receives runoff from the upper catchment and convey to outfall should be added at Lenda Ber kebele (at both wings of the existing canal from Hospital to Aymale Adebabay) and Eastward of Mahal Arada along the main road of Bus station to Sodo way and should be well connected to dispose of runoff in a safe manner. The existing common outlet (from ST. Gabriel to Bilate River) should be lined canal instead of natural waterway which is going to be gully due to high amount of runoff from the upper catchment which is allowed to pass through it.
- ✓ Finally, I recommend the Alaba Zone Municipality to construct drainage structures by using the currently developed IDF curve and should have to store data like dimensions of constructed

canals, images and videos of flood events and hazard level of the flood to do a flood risk assessment. It is necessary to invest money, time, labor, knowledge and share the experience of dwellers, government body and non-governmental organizations altogether to keep the town from threat and to sustain life and property.

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