



Hydraulic Analysis of Storm Water Drainage System in Alamata Town, South Tigray, Ethiopia

A. Zinabie^{1*}, B. Kebede²

¹ Department of Hydraulic and Water Resources Engineering, University of Gondar, Gondar, Ethiopia

² School of Civil and Water Resources Engineering, Bahir Dar University (BIT), Bahir Dar, Ethiopia

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ABSTRACT

With urbanization impervious surfaces increase, drainage pattern changes, overland flow becomes speedy, flooding and environmental problems such as land degradation increases. The objective of this study is to evaluate the hydraulic performance of storm water drainage system of the study area. The drainage channels are filled with or blocked by silt and garbage, the blockage of drainage structures is due to flat areas and siltation of drainage system which leads to failures on roadways. Spatial analyses were conducted using Arc GIS, Bentley Flow master, Google Earth pro and Auto CAD software's as analytical tools. The hydrological analysis was computed by rational method. The existing depth and width of the storm water drainage channel were measured for all sub catchments of the town and used to determine the existing drainage capacity using Bentley Flow master. The existing condition drainage channel depths of sub-catchments 6,7 and 8 increased by 68.40%, 160.20% and 121.07%; and the widths by 55.44%, 116.83% and 100.98%, and the design discharge was also 9.29m³/s, 16.47m³/s and 9.13m³/s, respectively. Therefore, this study recommends improvement in the integration of road and drainage structure; integration of solid waste management system in order to prevent the over flowing of storm water as a result of blockage of drains; the stations of the road requires proper maintenance and suitable measures should be taken in order to make the road and storm water drainage structures serve for the intended purposes sustainably to the stakeholders.

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INTRODUCTION

Urbanization is characterized by a marked increase in built structures [1] such as streets, walkways, parking lots and rooftops creating sealed surfaces. Drainage networks are an important part of the infrastructure of any society. The main purpose of providing the drainage network is to carry away sanitary waste and storm water from a municipal area in such a way that it does not cause any public health related problems [2].

Storm water is the water generated from rainfall, it can infiltrate the ground surface and join with the ground water table or contribute to subsurface flow and the remaining portion ends up as surface runoff which ultimately joins with the streams, rivers, etc. Under natural condition, most of the storm water gets infiltrated into the soil due to higher permeability of the surface [3]. Storm water is rainwater or melted snow that runs off streets, lawns and other sites. When storm water is absorbed into soil, it is filtered and ultimately replenishes aquifers or flows into streams and rivers [4].

Storm water drainage system is designed to drain excess rain and ground water from impervious surfaces such as

paved streets, car parks, parking lots, footpaths, sidewalks, and roofs [5]. In modern cities the storm water drainage system is the essential part of the urban infrastructures. Basically, the most important use of the storm water drainage system is to carry the excess rain water in the monsoon season from the roads, streets and roofs and make the city free from the problems of back flow and also from the flood. In developing countries, the factors which are affecting the storm water drainage system classified into two categories: 1) Natural 2) Human activity. Rainfall patterns, Properties of catchment areas, soil type, presence of water course, climate change etc are the natural factors. Extension of urban areas, impermeability, degradation of plant cover, lack of planning and poor management and such are the Human activities that impacts on natural storm drainage [6]. Storm water runoff can result in flood damage when large amounts of precipitation collect on the ground glide over impervious surfaces without being absorbed [7].

The absence of proper drainage negatively affects the economy of the community (car crash, delay in time or time wastage, wastage of gasoline) as well as social activities such as facilitation of trade and businesses, access to basic services,

*Corresponding Author Email: ashezinabie@gmail.com (A. Zinabie)

safe transportation. Surface drainage involves collecting the water from the road surface, road shoulders, side slopes and adjacent areas and carrying it away via downhill slopes, roadside ditches and pipes [8].

The pattern of urbanization and modernization Ethiopia has meant increase densification along with urban infrastructure development. This has led to deforestation, use of corrugated roofs and paved surfaces. The combined effect of this results in higher raindrop intensity and consequently accelerated and concentrated runoff. Due to inadequate integration between road and urban storm water drainage, infrastructure provision and poor management significant proportion of the area is exposed to flooding hazards/risks. Appropriate design of the surface drainage system is an essential part of road design [8].

A well planned, operated and maintained storm water system should drain storm water runoff effectively during normal periods and during floods. However, with rapidly expanding cities, unanticipated problems from changing land usage, system overloading, pollution and deteriorating environment cause problems to storm water assets and contribute to its failure. These issues are more problematic in larger and older cities where replacement is costly. Alamata town is anticipated to face many of the challenges this problem poses asset manages the urban water cycle system in an integrated manner to protect, restore and enhance the storm water assets. With absence of proper drainage line along roads in the town, seasonal floods and other environmental problems are most frequently occurring especially on downstream parts of the town. This has resulted in negative impacts on sustainable urban drainage system provision and management. Thus, there is a need to study the adequacy of the existing storm water drainage system of Alamata town to handle the yearly repeating urban storm water drainage problems with overloading the drainage system and the serious consequences on the environment. In the current research, there are few sources comparing the results of evaluating hydraulic performance of storm water drainage system, even at site scale. This article seeks to bridge this gap by examining this difference. The article highlights the hydraulic performance of the existing and proposed drainage system based on the hydrological analysis. So, the study focuses on the hydraulic Analysis of storm water drainage system of the study area.

MATERIALS AND METHODS

Location of study area

Alamata town is found at a distance 600km from Addis Ababa and 180 km from Mekelle. The Town stretches from 39°55'Latitude 12°42'longitude and shares borderlines on all directions with Raya Alamata Woreda. According to the Woreda Atlas; the total area coverage of the Woreda is 1050 hectare or 10.5 km². The study area has a variable rainfall pattern with a short rainy season and a long rainy season with an average annual rainfall of 790 mm. The Location map of the study area is described in Figure 1.

Model selection

When choosing a suitable model, it should first be considered if it is possible to use the model in respect of investments in

time and money. It should then be considered whether the model gives the desired output data required for the project and if the input data required is possible to obtain within a reasonable amount of time and price. A complex model often requires more input data than a simple model, while a simple model with fewer input data instead may not be specific enough for the current study. Steps of model selection were described on Figure 2.

Based on the above model selection criteria were described on Figure 2 for this Paper Arc GIS, Google earth pro, Auto CAD and Bentley flow master were used.

A. Arc GIS: The first step in doing any kind of hydrologic modeling involves delineating streams and watersheds, and getting some basic watershed properties such as area, slope, flow length, stream network density. With the availability of digital elevation models (DEM) and GIS tools, watershed properties can be extracted by using automated procedures. The processing of DEM to delineate watersheds is referred to as terrain pre-processing. There are several tools available online for terrain pre-processing. In this thesis, Arc Hydro tools were used to process a DEM to delineate stream network and some other watershed characteristics that collectively describe the drainage patterns of a basin [9].

B. Bentley Flow Master: Bentley Flow Master is an easy-to-use program that helps civil engineers/hydraulic engineer to undertake hydraulic analysis for analysis of pipes, ditches, open channels, and more and it computes flows related parameters (depth, discharge and velocity) in a given flow system i.e. open channel flow, pipe flow by using well-known formulas such as Darcy-Weisbach, Hazen-Williams and Manning's formula [10].

C. Google Earth Pro: Google earth is an additional source for deriving spatial information on drains androids.

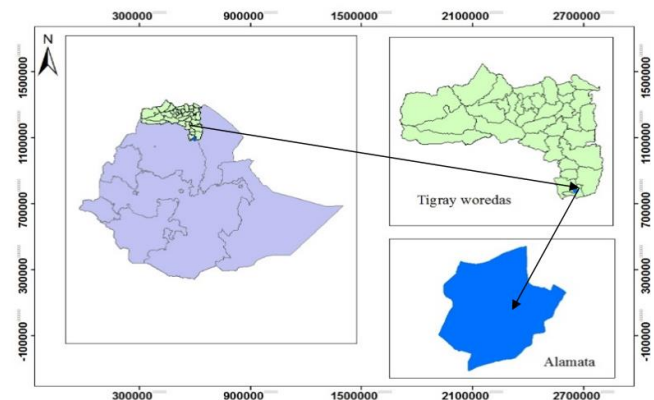


Figure 1. Location map of the study area

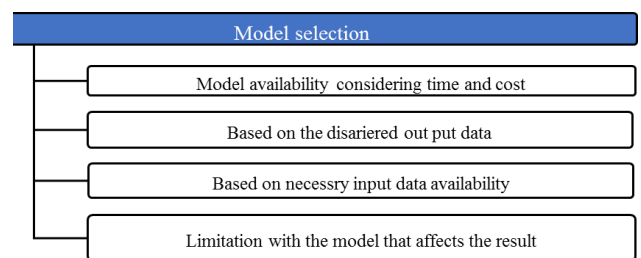


Figure 2. Steps to choose a proper model

DATA COLLECTION

Spatial data for Alamata town were acquired from a variety of sources depending on types of data. This paper involves collection of both primary and the secondary data and was also include information from respective organizations. Primary data was collected from personal field observation/site investigation and Google earth data with the help of a base map and check list. Secondary data was collected through from different organizations, literature studies and document analysis.

Digital Elevation Models (DEM)

In this study, a Digital Elevation Model (30m x 30m resolution) which was taken from Ministry of water, Irrigation and Electricity Addis Ababa Ethiopia was used.

Storm sewer and drainage network

Drainage map of Alamata town data regarding existing storm sewers along with their directions and outlets were collected from Alamata town municipality and field survey to collect channel depth and width of the drainage network.

Land use maps

Land use/Land cover maps of Alamata town during 2018 were collected from Ministry of Agriculture, Addis Ababa, Ethiopia.

Metrological data

Metrological Data was collected from National Meteorological Service Agency Addis Ababa, Ethiopia.

DATA ANALYSIS

For accomplishing the objectives of the study and to answering the research questions, the collected raw data was edited, coded, classified, and tabulated in order to make it ready for analysis. Information from primary and secondary sources was analyzed by using qualitative and quantitative methods. Data that have quantitative nature such as average and percentage was computed with Microsoft Excel. Furthermore, spatial analyses were conducted using Arc GIS, Bentley flow Master, Google earth pro and Auto CAD software as analytical tools. Figure 3 is described the methodology for determination of the actual and design discharge of Alamata town to evaluate the adequacy of existing storm water drainage system.

Hydraulic and hydrological analysis

Meteorological data availability In order to apply flood estimation models for peak discharge computation using available rainfall data, the rainfall depth-duration-frequency relationship is required. Available rainfall data on these stations has been collected and analyzed using two methods of distribution analysis namely Log Pearson-III and Gumbel's Methods. Due to the rain gage station in the study area is one the outlier test is appropriate to check the quality of the data.

Estimating missing rainfall data Due to the absence of observer or instrumental failure rainfall data record

occasionally are incomplete. In such a case one can estimate the missing data by using the nearest station rainfall data. There are different approaches for estimating missing rainfall data varying with and based on the effect of orography on rainfall, distance between the rainfall stations and the variation of rainfall amount recorded on the stations.

Among different method Normal ratio method was one of them which was recommended to estimate missing rainfall data in regions where annual rainfall between stations differ by more than 10% [11].

$$P_z = \frac{1}{3} \left(P_1 \frac{N_z}{N_1} + P_2 \frac{N_z}{N_2} + P_3 \frac{N_z}{N_3} \right) \quad (1)$$

where:

P_z : missing rainfall data (daily, monthly or yearly)

P_1, P_2 and P_3 : rainfall data at nearest different station (daily, monthly or yearly)

N_z : mean annual rainfall at missed station

$N_1, N_2,$ and N_3 : mean annual rainfall at different nearest station.

Check the quality of data

Check on outliers has been undertaken on the recoded rainfall data to identify any low or high outliers. Outliers are data points, which depart significantly from the trend of the remaining data. The outlier test is done to check whether the adopted data is within the limited range or not. The daily heaviest rainfall data of Alamata meteorological station from 1995 to 2019 was taken for the design. Hence, 25 years of daily heaviest rainfall data is available. These data was also checked for its consistency by higher and lower outlier testes using the equations [11] described in Table 1.

Test for higher outlier

$$\text{Higher outlier: } YH = Y_{avg} + K_n * \delta n - 1 \quad (2)$$

where:

Y_{avg} : mean of data in log unity

K_n : from Table 2 for sample size N

$\delta n - 1$: standard deviation from Table 1.

Higher outlier test = 10^{YH}

For data $Y_{avg}=1.719, N=25, \delta n - 1=0.101$ from Table 1 and $K_n=2.563$ from Table 2.

Higher outlier test $YH = Y_{avg} + K_n * \delta n - 1 = 1.719 + 2.563 * 0.101 = 1.966$

Higher outlier test = $10^{YH} = 10^{1.966} = 92.533$

The highest recorded value from meteorological station is (80.2 mm) as described in Table 1 which was less than the higher outlier test (92.5337 mm). Therefore, no higher outlier data eliminated.

Test for lower outlier

$$\text{lower outlier: } YL = Y_{avg} - K_n * \delta n - 1 \quad (3)$$

Lower outlier = 10^{YL}

$YL = 1.7119 - 2.563 * 0.101 = 1.454$

$YL = 10^{1.454} = 28.43$

The lower recorded value from meteorological station is (27.7 mm) as described in Table 1 which was less than the lower outlier test (28.425 mm). Therefore, the lowest value from recorded data (27.7 mm) was excluded from the hydrological analysis.

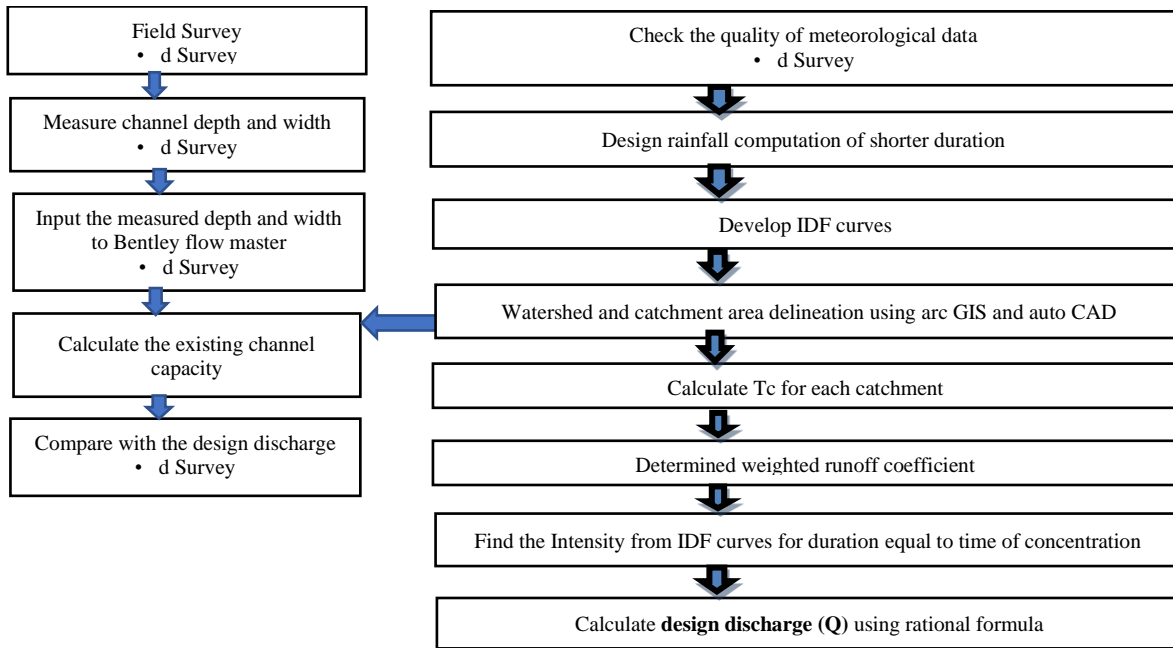


Figure 3. Determination of the actual and design discharge of Alamata town to evaluate the adequacy of existing storm water drainage system

TABLE 1. Max rainfall data from 1995-2019 and its statistical calculation

S. No	Year	Max Daily Rainfall	Descending Order	Rank	Logarithmic Value/Yo/	New Rainfall(X)	log(X)
1	1995	55.4	80.2	1	1.904	80.2	1.904
2	1996	39	73.7	2	1.867	73.7	1.867
3	1997	50	73.7	3	1.867	73.7	1.867
4	1998	46	73.7	4	1.867	73.7	1.867
5	1999	62	66.2	5	1.821	66.2	1.821
6	2000	58.5	64.7	6	1.811	64.7	1.811
7	2001	66.2	62	7	1.792	62	1.792
8	2002	44.5	62	8	1.792	62	1.792
9	2003	64.7	58.5	9	1.767	58.5	1.767
10	2004	53.1	58	10	1.763	58	1.763
11	2005	55.2	55.4	11	1.744	55.4	1.744
12	2006	48	55.2	12	1.742	55.2	1.742
13	2007	58	53.1	13	1.725	53.1	1.725
14	2008	37.3	53	14	1.724	53	1.724
15	2009	62	50	15	1.699	50	1.699
16	2010	33.3	48	16	1.681	48	1.681
17	2011	27.7	47.8	17	1.679	47.8	1.679
18	2012	47.8	46	18	1.663	46	1.663
19	2013	80.2	45.3	19	1.656	45.3	1.656
20	2014	45.3	44.5	20	1.648	44.5	1.648
21	2015	73.7	42.7	21	1.63	42.7	1.63
22	2016	73.7	39	22	1.591	39	1.591
23	2017	73.7	37.3	23	1.572	37.3	1.572
24	2018	53	33.3	24	1.522	33.3	1.522
25	2019	42.7	27.7	25	1.442		
Mean			51.556		1.719	52.725	1.73
Standard deviation			11.257		0.101	10.523	0.101
Skewnesscoefficient					-0.163		-0.163

Checking data reliability

For data N=24 and $\delta n - 1 = 10.52$ Mean (\bar{X}) = 52.556 from Table 1.

$$\text{Standard error of mean } (\delta n - 1) = \frac{10.52}{\sqrt{24}} = 2.15$$

Relative standard (δn) = $\frac{\delta n - 1}{\bar{X}} * 100 = \frac{2.145}{52.556} * 100 = 4.09\% < 10\%$ hence the data series could be regarded as reliable.

Design rainfall computation of shorter duration

These rainfall analyses and processing is aimed at determination of appropriate Intensity-Duration Frequency relationship. Extreme rainfall depth at town station for different return periods was determined using Log Pearson Type III distributions and Gumbel method analysis [11]. The rainfall depths obtained from gauging station are of 24 h duration depth. Design and analysis of drainage structures require rainfall intensity duration relationship of shorter duration. Because rainfall data of shorter duration is an available appropriate IDF derivation for shorter duration is required [12] suggests the following equation.

Log Pearson Type III distributions

$$YT = Y_{avg} + KT * Sy \tag{4}$$

where:

YT: Log XT –logarithm of Rainfall depth (X T) at return period T years [mm]

Yavg: Mean value of logarithmic rainfall data (daily) [mm]

Sy: Standard deviation [mm] (Table 3).

Gumbel method analysis

$$XT = X_{mean} + K * Sy \tag{5}$$

$$y = \frac{\sqrt{\sum y^2 - 1/n(\sum y)^2}}{n-1} \tag{6}$$

Sy: Stander deviations of sample size N

K: frequency factors expressed as $K = \frac{Yt - Yn}{Sn}$, Yn and Sn reduced standard deviations, as sample N in Table 4 [13].

INTENSITY-DURATION-FREQUENCY (IDF) CURVES

The IDF relationships are used when designing drainage works for any engineering project, and allow the engineer to design safe and economical flood control measures.

The rainfall depths obtained from gauging station are of 24h duration depth. Design and analysis of drainage structures require rainfall intensity duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable, appropriate IDF derivation for shorter duration is required.

$$RRt = \frac{t}{24} \left[\frac{(b+24)^n}{(b+t)^n} \right] \tag{7}$$

where:

RRt: Rainfall depth ratio Rt:R24

Rt: Rainfall depth in a given duration t

R24 : 24h rainfall depth

Coefficients b = 0.3 and n = 0.78 - 1.09

Substituting Intensity (mm/h) in the above equation:

$$It = \frac{Rt}{t} = \frac{R24}{24} \left[\frac{(b+24)^n}{(b+t)^n} \right] \tag{8}$$

The methods employed to develop IDF curve for the shorter duration events using the above equations are as follows. Among many frequencies analyses Log Pearson type III better R² value, so for this paper Log Pearson type III distribution was selected. Using the trend line equation obtained from Log Pearson type III distribution method of frequency analysis, that is $y = 0.2715X + 59.841$ where y is 24-hour rainfall depth (R₂₄) of a return period x under consideration, R₂₄ is calculated for 2, 5, 10, 25, 50- and 100-year return period.

TABLE 2. Outlier test Kn value

Sample size	Kn	Sample size	Value	Sample size	Value	Sample size	Value
			Kn	N	Kn	N	Kn
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.467	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.7	80	2.94
15	2.247	29	2.549	43	2.71	85	2.917
16	2.309	30	2.563	44	2.719	90	2.961
17	2.309	31	2.577	45	2.727	95	2.981
18	2.361	32	2.591	46	2.736	100	3
19	2.385	33	2.604	47	2.744	110	3.017
20	2.408	34	2.619	48	2.744	120	3.078
21	2.408	35	2.628	49	2.753	130	3.107
22	2.429	36	2.639	50	2.76	140	3.214
23	2.448	37	2.65	55	2.768		

TABLE 3. Yearly extreme series and frequency analysis Log-Pearson Type III distribution

Return period(T)	X ⁻	δn - 1	KT	YT =X ⁻ + KT *δn - 1	XT = 10 ^{YT}
2	1.711	0.0854	-0.017	1.710	51.233
5			0.836	1.782	60.587
10			1.292	1.821	66.269
25			1.751	1.861	72.527
50			2.054	1.886	76.979
100			2.472	1.922	83.572

TABLE 4. Yearly extreme series and frequency analysis calculations Gumbel distribution

Return Period (T)	X ⁻	δn - 1	Yn	Sy	YT	KT	XT=X ⁻ + KT*Sy
2	52.3793	10.498	0.5353	1.1086	0.3665	-0.1522	50.7808
5					1.4999	0.8701	61.5147
10					2.2503	1.5470	68.6214
25					3.1985	2.4023	77.6008
50					3.9019	3.0368	84.2623
100					4.6001	3.6666	90.8746

CATCHMENT AREA AND STREAM NETWORK DELINEATION USING ARC GIS AND ARC SWAT

This tool allows the user to delineate sub watersheds based on an automatic procedure using Digital Elevation Model (DEM 30mx30m) data. User specified parameters provide limits that influence the size and number of subwatersheds created. In addition, users have the option of importing pre-defined watershed boundaries and an associated stream network. Figure 4 described catchment delineation of Alamata Town. Procedures to delineate watershed using Arc SWAT

- Load the DEM (30mx30m)
- Define the working area (Mask)
- Load the stream network to be used for the delineation
- Preprocess the DEM
- Specify the minimum sub-watershed area (critical source area)
- Review and edit the stream network points
- Run the calculation of the subbasin parameters

HYDROLOGICAL EQUATIONS FOR DETERMINING PEAK RUNOFF

Rational method

This method is widely used for peak discharge determination from smaller catchments (less than 50 ha or 0.5Km² area) as recommended by ERA drainage design manual. The equation of rational formula is function of catchment area, runoff coefficient and time of concentration. The equation is expressed as:

$$Q = 0.00278 CIA \tag{9}$$

where:

Q: Discharge at outlet (m³/s)

C: Runoff Coefficient

I: Maximum probable rainfall Intensity (mm/h)

A = Catchment Area (hectares)

Runoff coefficient

A weightage method is employed to obtain the representative runoff coefficient i.e. the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area.

$$C_{weighted} = \frac{\sum(Ai * Ci)}{AT} \tag{10}$$

where:

Ci: Runoff coefficient for a given hydrologic soil group area

Ai: Area under each hydrologic soil group

AT: Total catchment area considered of town

Time of concentration

The time of concentration is the time required for the rain water to flow over the ground surface from the farthest point in the drainage basin and reach the point under consideration (point of entry in the drain) [14].

Sheet flow time

It usually occurs in the headwater of the streams (usually for the first 100mrun), but from topographic map obtained 100m Sheet flow, natural range and short grass slope of 0.015 m/m, and length of 100 m and

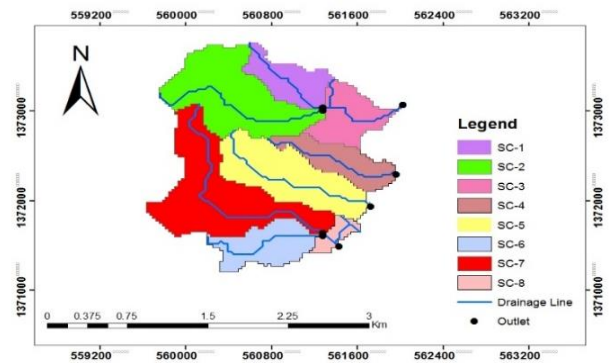


Figure 4. Catchment delineation of Alamata town

Manning’s roughness coefficient is 0.015. The 2-year, 24-hour rainfall depth is calculated to be 33.466 mm. Hence, from Equation (11) travel time for sheet flow is determined as:

$$Tt1 = \left\{ \left(\frac{0.091 * (n * L)^{0.8}}{(P2)^{0.5} + S^{0.4}} \right) \right\} \quad (11)$$

where:

Tt: travel time, hr

n: Manning's roughness coefficient

L: flow length, m

P2: 2-year, 24-hour rainfall, mm

S: slope of hydraulic grade line (land slope), m/m based on topographic map

Open channels flow For channel flow, natural stream channel, winding with weeds and pools, slope is 0.02m/m and length is 3394m. By direct measuring the average bottom width of the stream channel is 1.2 m.

$$\text{Velocity through the channel: } V = \frac{R^{\frac{2}{3}} * S^{\frac{1}{2}}}{n} \quad (12)$$

The time of concentration is the sum of all Tt values for the various consecutive flow segments.

$$\text{Total Time of Concentration: } (Tc) = Tt1 + Tt2 \quad (13)$$

By the same processes, Time of concentration for the other catchments is determined in Table 5.

Capacity of existing drainage structures

The current conditions of hydraulic calculation are performed and evaluated by collecting the data of existing storm water drainage structure using field survey. The existing depth and width of the storm water drainage channel were measured for all sub catchments of the town and used to determine the existing drainage capacity using Bentley Flow master.

Adequacy of existing drainage structures and its proposed

Based on the hydraulic calculation of the result drainage capacity of existing system were checked to compare with proposed discharge this process also done first by determining the peak discharge for each existing catchment from the measured depth and width of the storm water drainage channel for all sub catchments of the town. The design capacity of the sub catchments of the town was calculated by taking the intensity for 25-year return period from the IDF was determined by rational formula. This discharge was also compared with existing discharge of the town was determined

from the measured depth and width of the storm water drainage channel for all sub catchments of the town. Then for this Existing and Design discharge drainage size (channel depth and width) was determine.

Drainage size determination for proposed

By used manning equation determined the size of the drainage channel based on the parameter for most economical rectangular channel for the proposed condition.

$$Q = \frac{A * R^{\frac{2}{3}} * S^{\frac{1}{2}}}{n} \quad (14)$$

Area of rectangular channel: $A = b * y$

For most economical rectangular channel: $b = 2 * y$

Area of rectangular channel $A = 2y^2$ $P = 2y + 2y = 4y$

$$\text{and } R = \frac{2y^2}{4y} = 0.5y, \frac{Q * n}{S^{\frac{1}{2}}} = y^{\frac{8}{3}} y = \left(\frac{Q * n}{S^{\frac{1}{2}}} \right)^{\frac{3}{8}}$$

$$\% \text{ Additional Depth} = \left(\frac{\text{Proposed Depth} - \text{Existing Depth}}{\text{Existing Depth}} \right) * 100 \quad (15)$$

$$\% \text{ Additional width} = \left(\frac{\text{Proposed Width} - \text{Existing Width}}{\text{Existing width}} \right) * 100 \quad (16)$$

RESULTS AND DISCUSSION

Topography and impervious surface

As illustrated in the map (Figure 5) the upper part of the watershed is characterized by steep type of slope (>20 %).

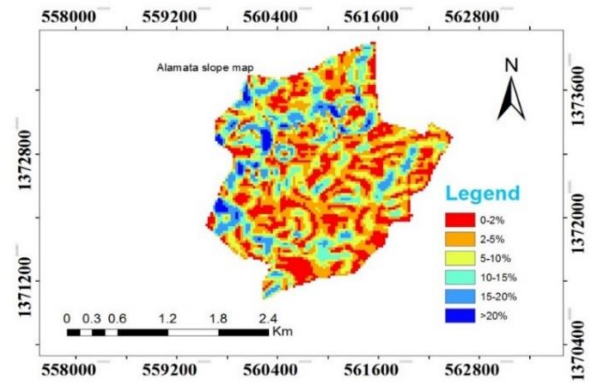


Figure 5. Slope map of Alamata town

TABLE 5. Time of concentration result in drainage system

Sub-catchment	Sheet flow time TC ₁ (Hr)	Open Channels Flow TC ₂ (Hr)	Total Tc(Hr)	Total Tc(min)	Total Tc(hr)
SC-1	0.1	0.07	0.18	10.52	0.18
SC_2	0.12	0.14	0.26	15.54	0.26
SC-3	0.12	0.15	0.27	16.24	0.27
SC-4	0.11	0.12	0.23	13.81	0.23
SC-5	0.12	0.15	0.27	16.02	0.27
SC-6	0.11	0.1	0.21	12.5	0.21
SC-7	0.12	0.22	0.34	20.19	0.34
SC-8	0.13	0.04	0.17	10.34	0.17

Consequently, the middle part of the watershed is categorized as a flat (0 - 5 %) kind of topography which covers. The characteristic of the slope has impact on the velocity and infiltration rate of storm water. Storm water from the upper part of the watershed flows with high velocity causing fragmentation of cobblestone and damaged the drainage channel. According to site observation, the drainage system provided along the local streets of this area doesn't consider this slope character of the area. Therefore, storm water in this area flows with high velocity in the provided drainage systems. Figure 5 described Slope map of Alamata town with the effect of topography.

HYDROLOGICAL AND HYDRAULIC ANALYSIS

Intensity–duration–frequency (IDF) curves

As described in the methodology the resulting IDF curve was used the daily maximum rainfall from Ethiopian Meteorological Agency rainfall gauge located in Alamata town, 24-hour rainfall data of 25 years was calculated using Log Pearson type- III distribution method because it has better R² values as shown in Figure 6 (and also see Table 6).

Rainfall intensity for the design storm was used to calculate peak runoff rate from a drainage area for the design of storm water structures using the rational method. For this study IDF curve from ERA could be used, however, we developed IDF curve using the local rainfall data and compared with IDF curve from ERA drainage design manual and found rainfall intensity values much different from ERA design manual. The difference in the two intensity values as described in Figure 7 (and also see Table 7). is due to differences in the data used to develop the IDF curve in the two studies.

Catchment area and stream network delineation using arc GIS and arc SWAT

As described in the methodology part to delineate sub watersheds based on an automatic procedure using Digital Elevation Model of (30m x 30m). Figure 4 described Catchment delineation of Alamata Town.

Time of concentration

Sheet flow time It usually occurs in the headwater of the streams (usually for the first 100m run). As described in the methodology sheet flow time was calculated as follows:

$$Tt1 = \left\{ \frac{0.091 * (0.015 * 100)^{0.8}}{(33.466)^{0.4} + 0.02^{0.4}} \right\} = 0.10h \quad (17)$$

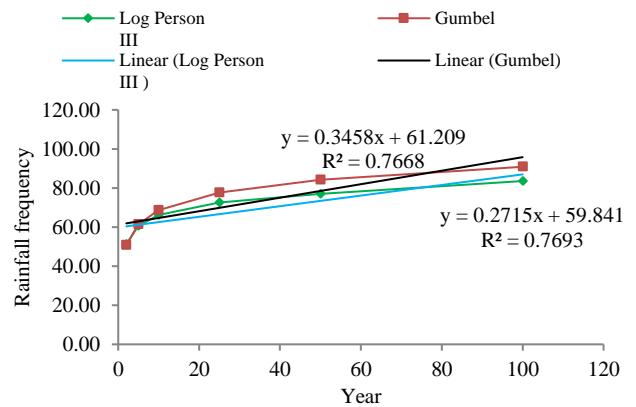


Figure 6. Plots of frequency analysis results based on Table 6 calculated design point rainfall

TABLE 6. Design point rainfall and return period

Return Period (Year)	Design Point Rainfall	
	Log Person III	Gumbel
2	51.23	50.78
5	60.59	61.51
10	66.27	68.62
25	72.53	77.6
50	76.98	84.26
100	83.57	90.87

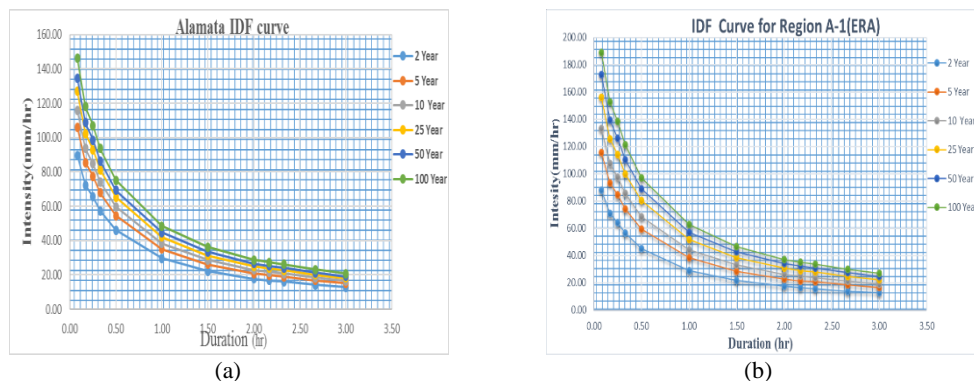


Figure 7. (a) Alamata IDF curve, (b) IDF curve for region A-1(ERA,2013) based on calculated values on Table 7

TABLE 7. Comparison of IDF curve results for Alamat with IDF curve from ERA for station(A1)

Duration (mint)	ERA T-2	SELF T-2	ERA T-5	SELF T-5	ERA T-10	SELF T-10	ERA T-25	SELF T-25	ERA T-50	SELF T-50	ERA T-100	SELF T-100
5	89.7	88.0	115.5	106.0	133.5	116.0	156.0	126.9	172.6	134.7	189.1	146.3
10	72.3	71.0	93.2	85.5	107.7	93.6	125.8	102.4	139.2	108.7	152.6	118.0
15	65.6	64.4	84.5	77.6	97.6	84.8	114.1	92.8	126.2	98.5	138.3	107.0
20	57.4	56.4	74.0	67.9	85.5	74.3	99.9	81.3	110.6	86.3	121.2	93.7
30	46.1	45.3	59.4	54.5	68.7	59.6	80.2	65.3	88.8	69.3	97.3	75.2
60	29.7	29.2	38.3	35.1	44.2	38.4	51.7	42.1	57.2	44.6	62.7	48.5
90	22.2	21.8	28.6	26.3	33.1	28.7	38.6	31.4	42.7	33.4	46.8	36.2
120	17.7	17.4	22.8	20.9	26.3	22.9	30.8	25.0	34.0	26.6	37.3	28.8
130	16.8	16.5	21.6	19.8	25.0	21.7	29.2	23.7	32.3	25.2	35.4	27.3
140	16.1	15.8	20.7	19.0	23.9	20.8	27.9	22.7	30.9	24.1	33.9	26.2
160	14.2	13.9	18.3	16.8	21.1	18.4	24.7	20.1	27.3	21.3	29.9	23.2
180	12.8	12.6	16.5	15.1	19.1	16.6	22.3	18.1	24.7	19.2	27.0	20.9

Open channels flow For channel flow, natural stream channel, winding with weeds and pools, slope is 0.02m/m and length is 3394m.

$$\text{Velocity through the channel } V = \frac{R^{\frac{2}{3}} * S^{\frac{1}{2}}}{n} \quad V = \frac{0.33^{\frac{2}{3}} * 0.02^{\frac{1}{2}}}{0.015} = 4.23m/s$$

$$Tt2 = \frac{L}{3600 * V} = \frac{1085}{3600 * 4.23} = 0.071h$$

The time of concentration is the sum of all Tt values for the various consecutive flow segments. Total Time of Concentration (Tc) = $Tt1 + Tt2 = 0.10h + 0.07h = 0.17h$
By the same processes, Time of concentration for the other catchments is determined in Table 5.

HYDRAULIC ANALYSIS

Adequacy of existing drainage structures

The existing depth and width of the storm water drainage channel were measured for all sub catchments of the town and used to determine the existing drainage capacity using Bentley Flow master described in Table 2 below for all the respective sub catchments and Figure 8 also described Cross section for Rectangular Channel Sub- Catchment – 1.

Adequacy of existing drainage structures and its proposed

Based on the hydraulic calculation of the result drainage

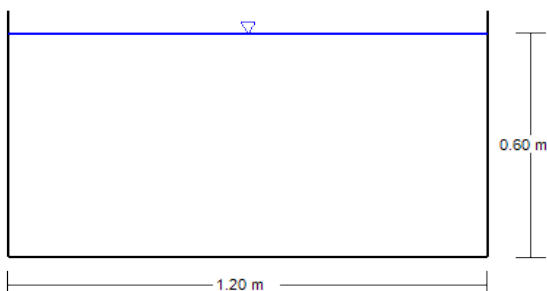


Figure 8. Cross section for rectangular channel sub-catchment-1

capacity of existing system were checked and presented in Table 8 to compare with proposed discharge as described in the methodology part. The design capacity of the sub catchments of the town was calculated by taking the intensity for 25-year return period from the IDF was determined using rational formula.

Drainage size determination for proposed

As described in the methodology by used manning equation (Equation (14)) determined the size of the drainage channel based on the parameters.

Area of rectangular channel $A = b * y$ for most economical rectangular channel $b = 2 * y$

$$\text{Area of rectangular channel} = 2y^2P = 2y + 2y = 4y \text{ and } =R = \frac{2y^2}{4y} = 0.5y$$

$$\frac{Q * n}{S^{\frac{8}{3}}} = y^{\frac{8}{3}} = \left(\frac{Q * n}{S^{\frac{8}{3}}}\right)^{\frac{3}{8}}$$

$$y = \left(\frac{8.69 * 0.015}{0.02^{\frac{8}{3}}}\right)^{\frac{3}{8}} = 0.97m \text{ and width } b = 2 * y = 2 * 0.97 = 1.94m$$

$$\% \text{ Additional Depth} = \left(\frac{0.97 - 0.60}{0.60}\right) * 100 = 61.58\%$$

$$\% \text{ Additional width} = \left(\frac{1.94 - 1.2}{1.2}\right) * 100 = 61.58\%$$

Based on the hydraulic calculation of the result drainage capacity of existing system were checked and presented in Table 8 to compare with proposed discharges described in the methodology part subtracted existing discharge from the proposed this step is important know over flow peak (excess discharge) for each catchments and for this over flow peak discharge calculate additional depth and width to control the over flow. The existing condition drainage channel depths of catchments 6, 7 and 8 Increase by 68.40%, 160.20% and 121.07%; and the widths by 55.44%, 116.83% and 100.98%, respectively, and the Design discharge was also 9.29m³/s, 16.47m³/s and 9.13m³/s respectively as described in the above Table 9.

TABLE 8. Current capacity of the drainage systems of Alamata town

Lable	Normal Depth (m)	Bottom Width (m)	Channel Slope (m/m)	Flow Area (m ²)	Roughness Coefficient	Wetted Perimeter (m)	Hydraulic Radius (m)	Velocity (m/s)	Discharge (m ³ /s)
SC-1	0.60	1.20	0.02000	0.72	0.015	2.40	0.30	4.23	3.04
SC-2	0.60	1.30	0.01500	0.78	0.015	2.50	0.31	3.76	2.93
SC-3	0.30	0.60	0.01400	0.18	0.015	1.20	0.15	2.23	0.40
SC-4	0.50	1.00	0.01400	0.50	0.015	2.00	0.25	3.13	1.57
SC-5	0.50	1.20	0.01500	0.60	0.015	2.20	0.27	3.43	2.06
SC-6	0.50	1.20	0.01800	0.60	0.015	2.20	0.27	3.76	2.26
SC-7	0.50	1.10	0.01500	0.55	0.015	2.10	0.26	3.34	1.84
SC-8	0.50	1.10	0.01100	0.55	0.015	2.10	0.26	2.86	1.57

TABLE 9. Existing, proposed, additional depth and width results

Sub-Catchment	Existing Condition			Proposed Condition				
	Depth (m)	Width (m)	Existing Discharge (m ³ /s)	Design Discharge (m ³ /s)	Depth (m)	Width (m)	% Additional Depth	% Additional Width
SC-1	0.60	1.20	3.04	8.69	0.97	1.94	61.58	61.58
SC-2	0.60	1.30	2.93	7.71	0.98	1.96	63.09	50.55
SC-3	0.30	0.60	0.40	8.05	1.01	2.02	235.86	235.86
SC-4	0.50	1.00	1.57	8.56	1.01	2.01	101.13	101.13
SC-5	0.50	1.20	2.06	8.37	1.02	2.03	103.14	69.28
SC-6	0.60	1.30	2.26	9.29	1.01	2.02	68.40	55.44
SC-7	0.50	1.20	1.84	16.47	1.30	2.60	160.20	116.83
SC-8	0.50	1.10	1.57	9.13	1.11	2.21	121.07	100.98

Best management paractices (BMPs)

If Gra-kahsu Mountain were fully covered by forest the value of run-off coefficient "C" would decrease, this allows flood to infiltrate in to the soil instead of flood generation. Applying different Structural measures of BMPs like terraces, in the Mountain, retaining walls on the river banks like gabion, detention systems, infiltration controls, Porous Pavement Systems and Gabion. Non-structural BMPs that are recommended to control the storm water from the Mountain are; Reduce Man-made impacts, Grassed Swales and covering the mountain with vegetation, area ex closure, gully treatment.

CONCLUSIONS

The existing drainage system has extensive defects and requires immediate rehabilitation or reconstruction, and also maintaining major drainage works. There is inadequate and low coverage of drainage system coupled with poor physical condition and in electiveness of drainage system development in the town. Capacities of most of the drainage system can't handle the current runoff that flows over the area. The design capacity of the catchments of the town was calculated by taking the intensity for 25-year return period from the IDF and was determined by rational formula. This discharge was also compared with existing discharge of the town was determined

from the measured depth and width of the storm water drainage channel for all sub catchments of the town. Then for this existing and design discharge drainage size (channel depth and width) was determine. The existing condition drainage channel depths of catchments 6,7 and 8 Increase by 68.40%, 160.20% and 121.07%; and the widths by 55.44%, 116.83%and 100.98%, and the design discharge was also 9.29m³/s, 16.47m³/s and 9.13m³/s, respectively.

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Persian Abstract

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چکیده

با افزایش سطح شهرها سطح ناپایدار، الگوی زهکشی تغییر می‌کند، جریان در سطح زمین تسریع می‌شود، جاری شدن سیل و مشکلات زیست محیطی مانند تخریب زمین افزایش می‌یابد. هدف از این مطالعه ارزیابی عملکرد هیدرولیکی سیستم زهکشی آب طوفان منطقه مورد مطالعه است. کانال‌های زهکشی توسط سیلت و زباله پر شده و یا مسدود شده‌اند، انسداد سازه‌های زهکشی به دلیل مناطق مسطح و آبیگری سیستم زهکشی است که منجر به خرابی در معابر می‌شود. تجزیه و تحلیل مکانی با استفاده از Arc GIS، استاد بنتلی Flow، نرم افزار Google Earth Pro و Auto CAD به عنوان ابزار تحلیلی انجام شد. تجزیه و تحلیل هیدرولوژیکی با روش منطقی محاسبه شد. عمق و عرض موجود کانال زهکشی آب طوفان برای کلیه حوضه های زیر آب شهر اندازه گیری شد و برای تعیین ظرفیت زهکشی موجود با استفاده از استاد بنتلی جریان استفاده شد. عمق کانال زهکشی وضعیت موجود در زیر حوضه ۶،۷ و ۸ افزایش یافته است ۶۸/۴۰، ۱۲۰/۲۰ و ۱۲۰/۰۷ درصد و عرض‌های آن ۵۵/۴۴، ۱۱۶/۱۸۳ و ۱۰۰/۱۹۸ درصد و میزان نشتی نیز به ترتیب ۹/۲۹ m³/s، ۱۶/۴۷ m³/s و ۱۶/۴۷ m³/s بود. بنابراین، این مطالعه بهبود در ادغام ساختار جاده و زهکشی را توصیه می‌کند. ادغام سیستم مدیریت پسماند جامد به منظور جلوگیری از جاری شدن آب طوفان در نتیجه انسداد زهکشی‌ها است. ایستگاه‌های جاده نیاز به نگهداری مناسب دارد و باید اقدامات مناسبی انجام شود تا سازه‌های زهکشی آب و طوفان جاده و طوفان برای اهداف در نظر گرفته شده بطور پایدار در اختیار ذینفعان باشد.