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Drainage System and Storm Water Management for Ndola Central Business Area, Zambia

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Division of Water Resources Engineering

Department of Building and Environmental Technology

DRAINAGE SYSTEM AND STORM WATER MANAGEMENT FOR NDOLA CENTRAL BUSINESS AREA, ZAMBIA

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ABSTRACT

Flooding of urban areas is a worldwide problem as cities grow and the amount of impermeable surfaces increase generating more surface runoff. In the Zambian city of Ndola, the capital of the Copperbelt Province, flooding occurs in some parts of the city especially on Dag Hammarskjöld Drive at the underbridge, which has been flooding every rainy season consecutively for 8 years now. The main objective of the study was to assess the effectiveness of the storm water drainage system and management for Ndola Central Business Area (CBA) and to determine the possible changes, to avoid or minimise yearly flooding of the underbridge area.

Both hard and soft solutions approach were employed. The procedure comprised of field study, data collection, modelling and analysis.

It was concluded that the major problem with the system are blockages resulting from garbage being thrown into the system as well as silt build-up that accumulates overtime. Additionally, changing the size of the draining water pipe (P27=600mm) from the underbridge area would improve the drainage systems' rainfall events' handling capacity to more than 10yrs return period without flooding.

Sustainable Urban Drainage Systems (SUDS) should be incorporated into the system in order to reduce and delay the peak flow downstream and subsequently in conduit P36 which contributes towards flooding by causing back water effects. Further studies should be conducted to ascertain specific SUDS adapted for the region owing to climatology and industrial activities.

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1 INTRODUCTION

1.1 Background and Problem Statement

Flooding of urban areas is a worldwide problem as cities grow and the amount of impermeable surfaces increase generating more surface runoff. Existing storm water drains are typically not capable of handling such increases in runoff, therefore, greater volumes of water are left on the surface. The reason for flooding in most areas can be attributed to the increased runoff resulting from land-use change, particularly the increase in impermeable surfaces. Also, lack of any upgrade or maintenance of the drainage system implies that the system can no longer sustain the runoff volumes. In the Zambian city of Ndola, the capital of the Copperbelt Province, flooding occurs in some parts of the city. The city is among the cleanest, well-planned and most-organised cities in Zambia with an approximate population of 455,194 in 2010 as compared to 374,757 in 2000 [4]. Lately, Ndola has been facing challenges with pluvial floods and flash floods caused by short-lived intense convective rainfall and/ or prolonged frontal rainfall events, where the proximity to the equator influences the climatology.

Below is Figure 1 presenting the monthly precipitation in Ndola together with other meteorological parameters.

Precipitation Table	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Annual
Average Precipitation mm (in)	0.1 (0)	0.4 (0)	2.9 (0.1)	31.5 (1.2)	130.3 (5.1)	305.9 (12)	292.9 (11.5)	249 (9.8)	170.1 (6.7)	45.5 (1.8)	3.5 (0.1)	0.7 (0)	1232.8 (48.5)
Precipitation Litres/m² (Gallons/ft²)	0.1 (0)	0.4 (0.01)	2.9 (0.07)	31.5 (0.77)	130.3 (3.2)	305.9 (7.5)	292.9 (7.18)	249 (6.11)	170.1 (4.17)	45.5 (1.12)	3.5 (0.09)	0.7 (0.02)	1232.8 (30.24)
Number of Wet Days (probability of rain on a day)	0 (0%)	0 (0%)	0 (0%)	4 (13%)	12 (40%)	19 (61%)	20 (65%)	19 (67%)	15 (48%)	5 (17%)	0 (0%)	0 (0%)	94 (26%)
Percentage of Sunny (Cloudy) Daylight Hours	82 (18)	82 (18)	75 (25)	72 (28)	49 (51)	35 (65)	35 (65)	31 (69)	48 (52)	68 (32)	76 (24)	77 (23)	61 (39)

Figure 1: Precipitation and other meteorological parameters [13]

The geographical position of Ndola in Zambia, in South Central Africa is shown in Figure 2 below

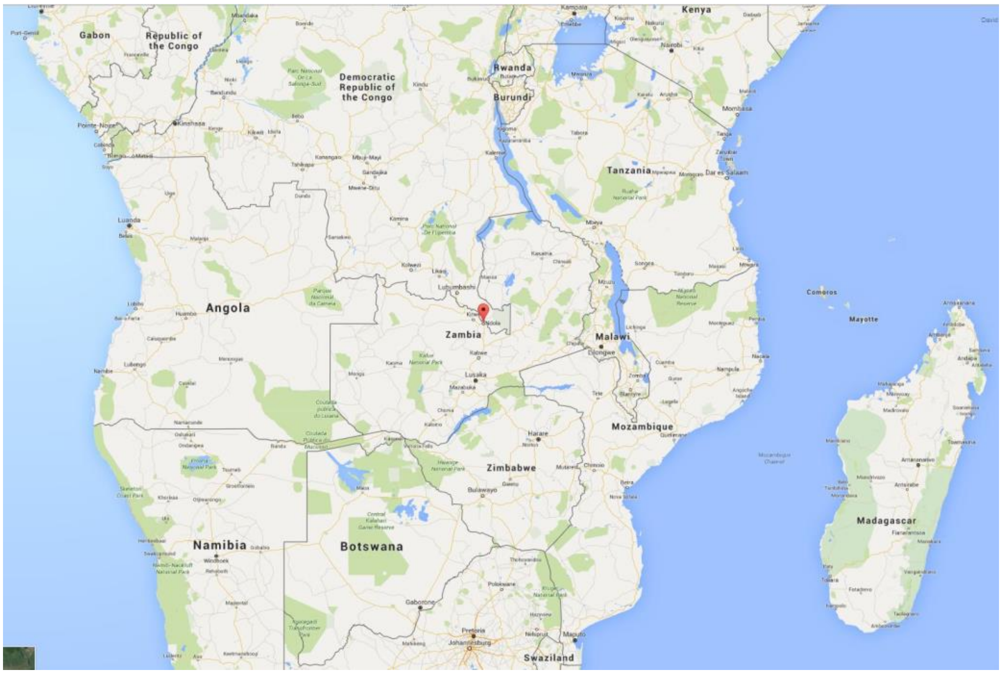

Figure 2: Drop pin showing Position of Ndola in Zambia

Located on -12.96 latitude and 28.64 longitude, the average altitude of Ndola is 1270m, but its central business area (CBA) is located on a slightly lower altitude as compared to the other residential areas. Thus, much of the runoff is directed towards the CBA, which adds to the runoff from impervious areas within the CBA. The result is the flooding of several important roads. The most affected road is Dag Hammarskjöld Drive at a point commonly known as the underbridge. The overbridge is the point where the great north road goes under the national railway line. The underbridge is a mini tunnel that dips underneath the railway line.

(Dag Hammarskjöld Drive was named in honour of the Swedish diplomat serving as second United Nations secretary general at the time, who died in a plane crash in Ndola on 18th September 1961).

The underbridge has been flooding every rainy season consecutively for 8 years now and has caused a lot of traffic jams being one of the major busy roads in the city [1] [2]. After each flood, the road gets damaged and this has the potential of weakening the whole structure with time especially that on top of the bridge is a functional railway line that is used in the transportation of heavy goods like copper cathodes and bars. The main problem is the yearly flooding of the underbridge which lies on the downstream end of the city's drainage system. Below is Figure 3 showing the southern side of the underbridge when it is flooded.

Figure 3: Southern side of Underbridge on Dag Hammarskjöld Drive

1.2 Objectives

The main objective of this study was to assess the effectiveness of the existing storm water drainage and management system in place for the Ndola CBA and determine the possible changes in order to avoid or minimise yearly flooding of the underbridge.

In order to achieve the overall objective, below is a sequence of specific objectives carried out as presented in this paper.

- To make on-site investigations and collect necessary data about the existing drainage and storm water management system for the Ndola CBA
- To model the existing drainage system for the Ndola CBA to ascertain its performance
- To identify the shortcomings of the drainage system for the Ndola CBA that contribute towards yearly flooding of the under-bridge
- To modify and model different scenarios to come up with the possible solution to any shortcomings in the drainage system for the Ndola CBA
- To recommend the most feasible and sustainable solution based on model results and observations for the Ndola CBA drainage system that will minimise flooding at the underbridge

1.3 Procedure

In order to fulfil the above-stated objectives, a dual approach to the problem was employed. One was through hard solutions by modelling followed by subsequent proposed changes to the existing design. Secondly, soft solutions through sustainable urban drainage systems (SUDS) that utilises nature to solve the problem, partly through employing geographic information system (GIS) for land use analysis.

The procedure was divided into four main categories namely field study, data collection, modelling and analysis in order to achieve the specific objectives.
After a preliminary understanding of the drainage system of Ndola CBA through study visits, data was then collected on-site much of which served as input to the stage of modelling. Modelling was done using the Storm Water

Management Model (SWMM) and preliminary model results were compared to observed conditions and adjustments were made to the model to ensure it was representative of observations on-site. Then different scenarios were modelled and simulated for possible solutions.

The other aspect was to look for potential applications of the blue-green solutions which basically integrates nature and engineering. The Ortho-photo from google combined with observations through field visitation were used in identifying potential sites and features for the purpose of implementing sustainable urban drainage systems.

1.4 Report Overview

Below is Figure 4 presenting the flow of events in this report.

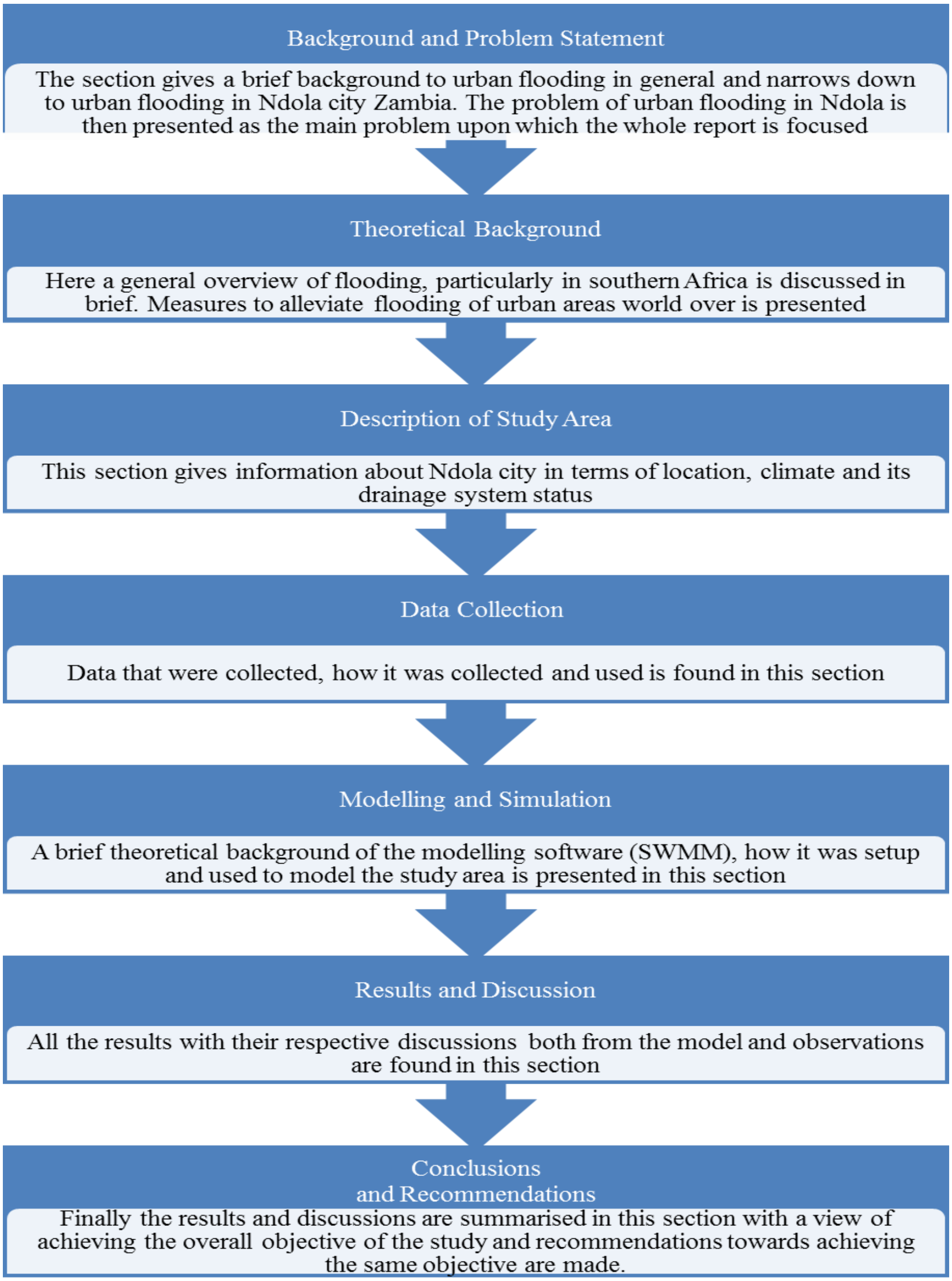

Figure 4: Report overview Flow chart

2 URBAN FLOODING

2.1 Overview of Problems

> *"Floods are natural disasters that have been affecting human lives since time immemorial. Throughout history, nature has shown little respect for man's unwise occupancy of nature's right-of-way and has ensured that the message has been clearly understood by sporadically flooding people's properties and taking their lives* (Ivan Andjelkovic) [10].*"*

Although, in layman's language, flooding can be defined as too much water in the wrong place, it is also true that people could have settled in the wrong place. Flooding can be caused by natural factors such as extremes in meteorological and hydrological conditions as well as interference from human activities dimmed as artificial factors [11]. Floods come in different types such as urban, pluvial and overland, coastal, groundwater, semi-permanent and flash floods.

Flooding in urban areas can be categorised into four main kinds namely localised flooding, small streams in urban areas, major rivers and wet season flooding [16]. Localised flooding results from increased imperviousness due to constructions leaving less room for runoff infiltration into the ground and also under-capacity drains within the urban area hence the path ways between residential areas become pathways for the runoff [16]. Small streams within urban areas add beauty to the cities but when there is a heavy down pour at increased intensities these streams tend to overflow beyond their initial design capacity partly because there could be some foreign materials in their channels leading to reduced effective flow area if they are not well maintained [16]. Some major rivers pass through cities and tend to be major sources of energy in form of damming and subsequent hydropower generation for example. Dam failures can release large volumes of water beyond the river channel capacity hence flooding downstream urban areas. In wet season, the most affected urban areas or cities are those in low laying areas or coastal zones [16]. Naturally these areas tend to have a raised water table hence even very little prolonged rainfall can cause flooding for longer periods due to saturated ground [16].

The cause to all these kinds and types of floods can be natural and/ or human induced. Most of the floods resulting from human activity interference involve collapsing of a dam, developments in low-lying areas and coastal zones, land use changes due to urbanisation leading to increasing imperviousness and subsequent runoff increase. The drainage systems and dams cannot handle the increasing volumes of runoff which can be attributed to design and maintenance aspects. Of the natural factors, heavy rainfall, earthquakes, volcanic eruptions, high water table coupled with heavy rainfall, sea level rise and land subsidence can be among the major catastrophic causes. Therefore, flooding can occur anytime regardless of the season of the year.

2.2 Flooding in Urban Areas in Southern Africa

Southern African region can be said to be comprising of 15 countries namely Angola, Botswana, Democratic Republic of Congo, Lesotho, Malawi, Mauritius, Mozambique, Namibia, Swaziland, Tanzania, Zambia, Zimbabwe, South Africa, Seychelles and Madagascar. According to the Southern African Development Community (SADC) Drought Monitoring Centre, flooding continues to occur in Southern African countries [18]. Amid the predicted El Niño, in Malawi 230, 000 people were displaced by floods in January 2015 [17]. Along major rivers in the region such as the Zambezi river, flash floods occur in urban areas especially where there is poor infrastructure and drainage system. Most floods in SADC region bring about communicable diseases such as cholera. So they do not only destroy property but human life either directly through floods or indirectly through diseases. In 2015, those affected by floods were about 1.8 million, displaced 280, 000 and killed more than 600 people [17]. The secondary effect of cholera killed 176 of the 20, 000 cases reported [17].

Below is Figure 5 showing a summary of flooding in the SADC region as compiled by the United Nations in their Humanitarian bulletin for southern Africa issue number 18 in May 2015.

Year	No. of people affected	No. of deaths	No. of countries affected
2007/8	1,049,516	Not known	9
2008/9	1,369,463	212	8
2009/10	368,581	7	8
2010/11	708,000	477	9
2011/12	553,773	160	8
2012/13	519,000	176	10
2013/14	453,256	117	14
2014/15	1,822,030	539	8

Source: OCHA Situation Reports & Updates. Data partial and incomplete

Figure 5: Summary of flooding in the SADC region [17]

2.3 Measures to Alleviate Flooding

No two flood risk scenarios or events are the same and as such it follows that there can never be the same strategy put forth for both [11]. This is because in any given two scenarios, either the source of flood risk, the type of the flood, chances of flooding, extent of damage or damage associated cost is different. Given the same outflow from two different catchments does not mean they are as a result of identical rainfall but instead complex combinations of hydrological and hydraulic factors. It is impossible to totally prevent flooding but usually measures aim at reducing its impact on human life, environment and associated economy. This then calls for an integrated approach towards flood risk reduction, management and mitigation [3]. Mitigation is usually an on-going process supporting flood management whilst preventive measures aim at directly tackling the problem. Hence, preventive measures are viewed as structural measures whilst mitigation measures as non-structural. Then follows the analogy that if structural measures are the bones then non-structural measures the flesh, signifying that the two are not distinct but complementary [10]. In the case of the Southern African region, the measures have been put in place such as disaster risk management organisations or units in most countries. The organisations seem to be operating well at nation level but not so much at local levels where flooding occurs mainly due to lack of financial and human resource support [17].

2.3.1 Sustainable Urban Drainage System (SUDS)

Sustainable urban drainage systems advocate for utilisation of nature's way of handling storm water through infiltration, percolation, surface runoff, slow drainage, detention ponds and wetlands [12]. Four categories according to Stahre (2006) for sustainable urban drainage facilities are shown in Figure 6 below

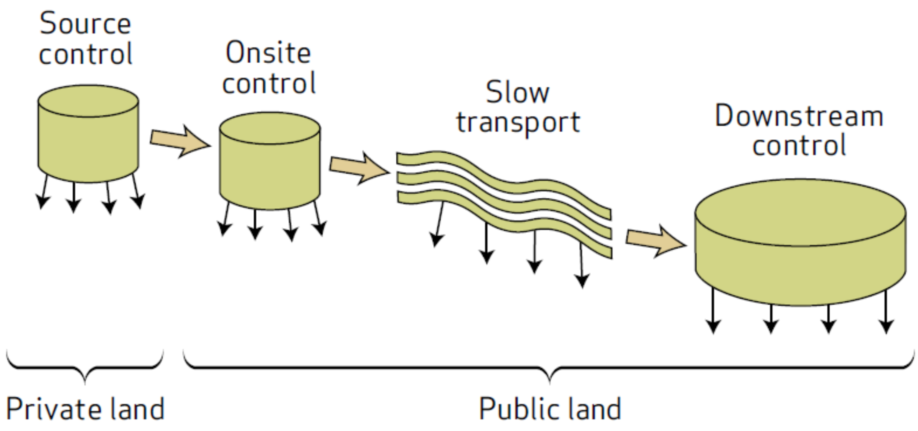

Figure 6: Categories of sustainable urban drainage system (SUDS) [12]

Source controls aims at reducing the runoff right from the source on private land through green roofs, infiltration lawns, pervious surfaces, rain gardens, localised ponds, rain harvesting, etc [12]. The remainder of the runoff is then handled onsite through pervious surfaces, ponds, green filter strips, rain gardens, deliberately and temporarily flooding designated areas etc [12]. Whilst onsite, conveyance of storm water is delayed to avoid flash flooding by allowing the runoff to flow on open drainage system through swales, ditches/ creeks, canals etc [12]. As the water reaches downstream, the volumetric flow might be too high to be discharged into a water course hence, downstream control measures such as large ponds, wetlands, lakes etc. are employed. The last three categories from onsite control to downstream control happens on public land and it is the responsibility of the municipality in Sweden for instance [12], but it is also true for Zambia.

Unlike the traditional way of urban drainage system, SUDS take an integrated approach in that the water is sometimes availed to both the environment and people [12]. Although this is not really practiced in Zambia particularly on this study area, the approach, places the responsibility of taking care of the overall environment on both the municipality and the people in general since the system comes along with valuable amenities to both the citizens and the environment. Below is the Figure 7 showing some of the values brought about by SUDS if an integrated approach to planning is considered.

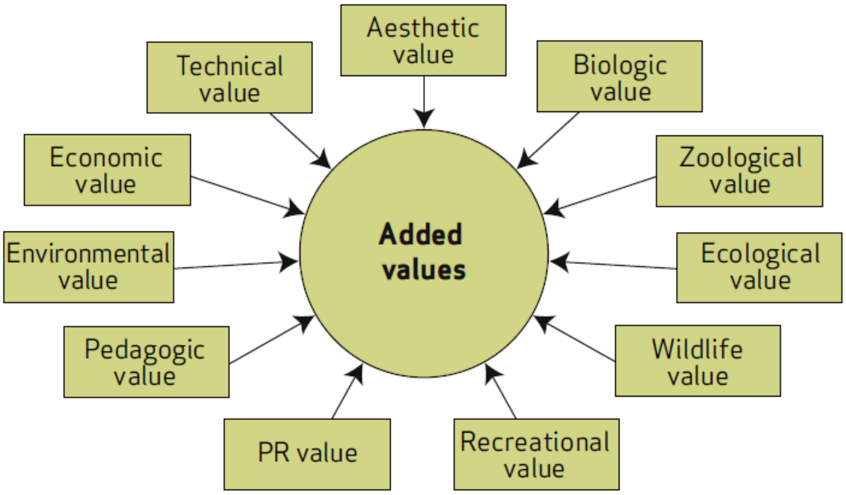

Figure 7: Values of sustainable urban drainage systems [12]

3 NDOLA STUDY AREA

3.1 Geographic Setting

Ndola is Located on -12.96 latitude and 28.64 longitude in Zambia, south central of Africa with an average altitude of 1270m. It is the provincial capital for the Copperbelt province of Zambia. According to Zambia Agricultural Research Institute (ZARI), the predominant soil type for Ndola is Acrisols [8] belonging to group C [15]. Hydrologically, Acrisols type of soil is not good due to its poor water infiltration capacity. The size of the catchment under study is approximately 1% of the total area of Ndola district. Figure 8 below show the catchment location in Ndola.

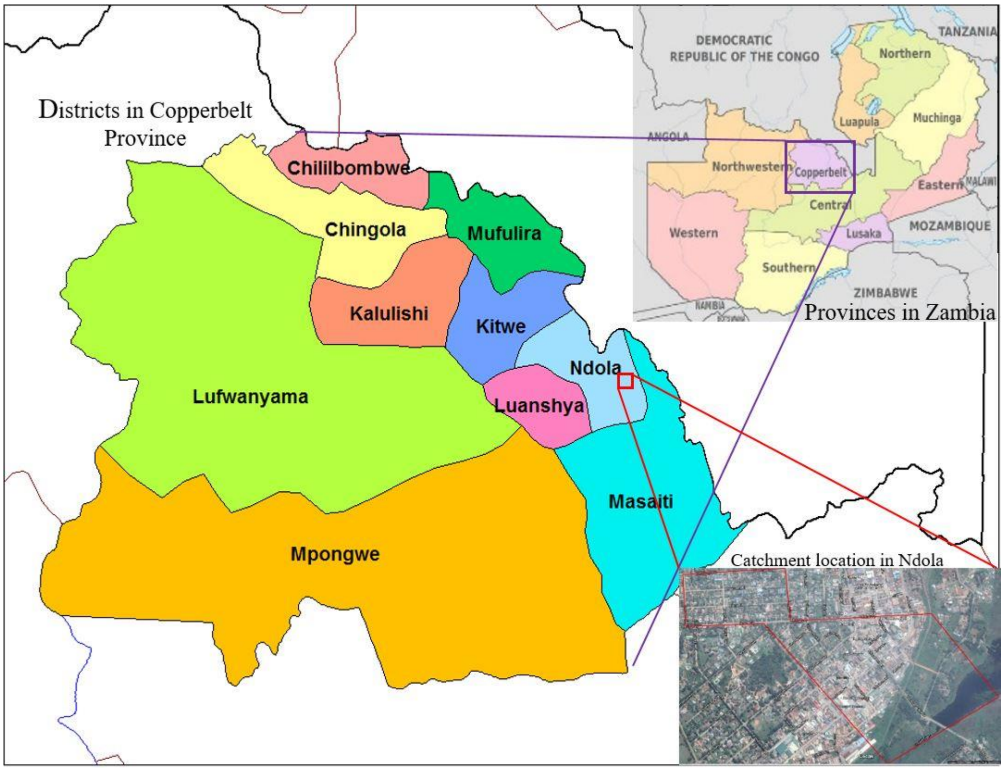

Figure 8: Catchment location in Ndola, Copperbelt, Zambia [22].

Ndola CBA has been growing continuously in terms of infrastructure while the drainage system is still the same. Below are Figure 9 and Figure 10 showing images of part of the catchment in 1969 and 2014.

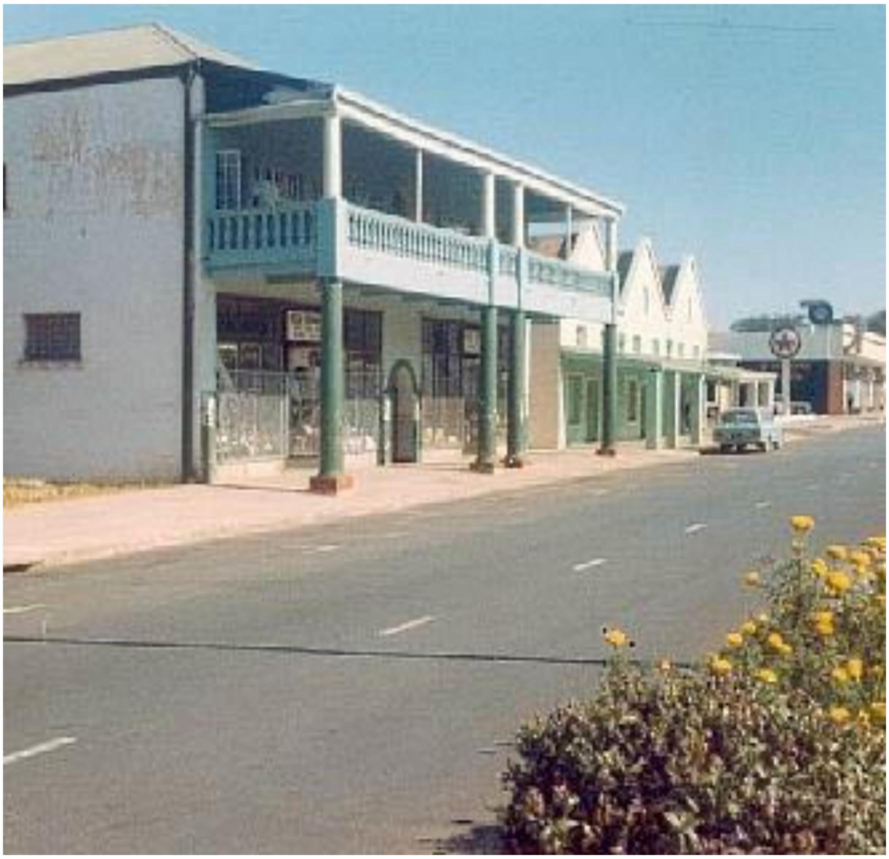

Figure 9: President Avenue, Ndola, 1969 [23].

Figure 10: President Avenue, Ndola, 2014 [23].

3.2 Climatology

Ndola being in a sub-tropical region and slightly north in Zambia is affected by the intertropical convergence zone (ITCZ) due to its proximity to the equator. The seasonal pattern is influenced by the ITCZ therefore; the rainy season is usually from October to April with its peak in January when the ITCZ is in the southernmost position [20]. The average temperature during the rainy season is 19˚C. Below is Figure 11 showing a climate graph for Ndola.

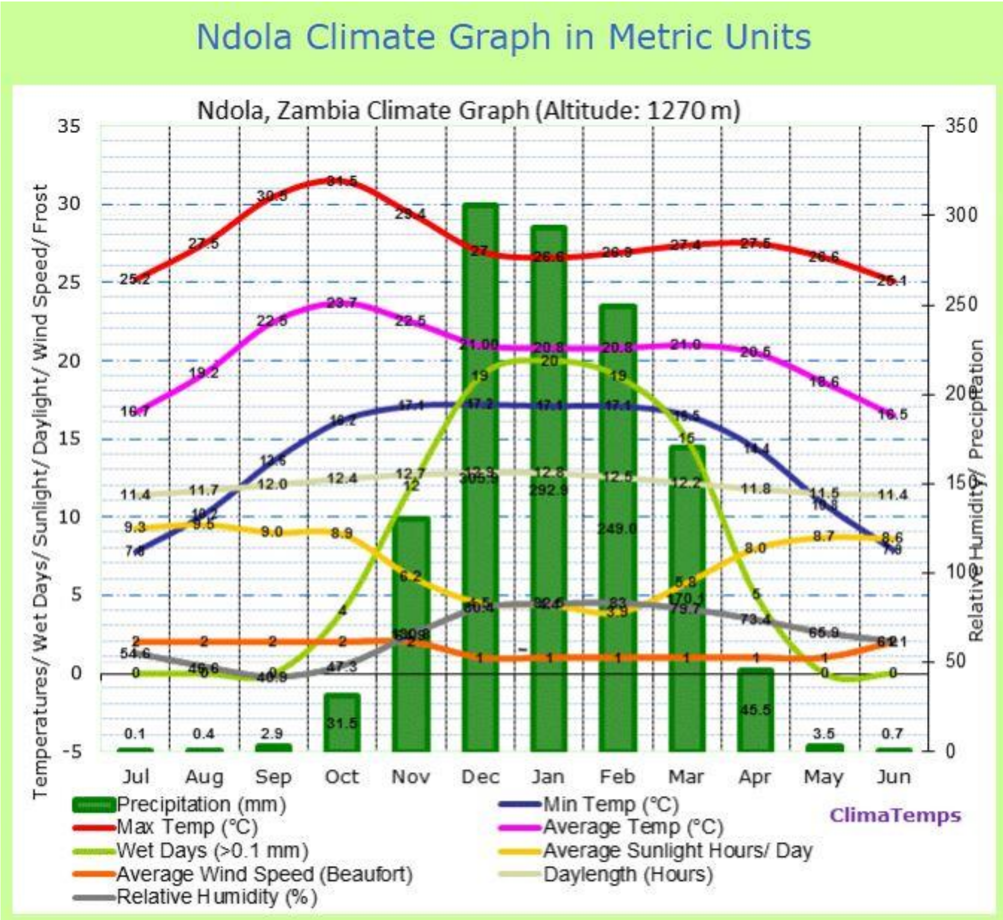

Figure 11: Graph showing climate in Ndola [19]

3.3 Drainage System

The drainage system in Ndola is well designed and organized though not fully maintained as evidenced from blockages in some parts of the city center. The city council does not have updated drawings of the drainage system and hence this affects prioritization of maintenance works. Most of the existing drawings were done between 1950 and 1970 when the system was initially constructed but maintenance works have since been carried out on most parts of the drainage system without any updates to the drawings. Lack of updated drainage system database translates into inefficiency towards maintenance and rehabilitation works especially by new members of staff. Some drainage pipes and channels are fully blocked while others are partially blocked. The CBA drains its runoff into Itawa river.

3.4 Existing Problems

Aside from flooding of the underbridge, other roads within the Ndola CBA flood as well. The CBA has been fully developed with less consideration to blue-green solutions. There has been less integration of SUDS facilities with almost no operational parks within the CBA leading to increased runoff within short periods of rainfall. Figure 12 below shows the land-use of the catchment under current study. The area within the triangle formed by boundary lines in the figure below represent the actual area of study.

Figure 12: Land-use of the catchment under current study within the triangle

4 DATA COLLECTION AND ANALYSIS

4.1 Field Study

Seeing that there were no complete and updated drawings for the Ndola CBA drainage system, the field studies were aimed at making investigations of the drainage system and ownership of property upon which it lay. The city council staff aided in this undertaking as they had experience through maintenance works from the time they started working for the municipality. It was verified that the Ndola CBA has a separate drainage system for storm water and that most of its network lay on public land. It was also established that much of the public land is built-up. Field studies continued throughout the entire period of data collection as challenges kept popping up. Below is Figure 13 showing a glimpse of the field study of the Ndola CBA drainage system. In the image are three Ndola city council staff and the author of this report.

Figure 13: Three Ndola city council staff and the researcher during Field study of the Ndola city centre drainage system

4.2 Field Work

The data that needed collected included the outlook of the existing drainage system network, the type of the network, the dimensions of the drainage elements, the elevations and slopes, the size of the study area, the land-use of the study area, rainfall data and images of study area. With reference to the weather pattern of Ndola, it required that data were collected during rainy season so as to have a general idea on the ground. Therefore, Collection of data was done on the actual site in Ndola Zambia in January and February 2016 during part of the peak of the rainy season. Ideally much of the detailed data such as dimensions were supposed to be extracted from the drawings. Unfortunately for the area of study, the drawings were missing which only left the option of going on the ground to physically collect the data. Lack of drawings also meant time constraints therefore, the study area was limited or confined to only the underbridge section unlike the initial plan for the whole CBA. It was not possible to collect every needed data as there was limited access to some other hydraulic elements, either they could not open or could not be traced. The use of ArcGIS software made the data collection easier and time saving as it was possible to digitize, edit, build on existing database and update.

With the help of the ortho-photo from google and ArcGIS, a georeferenced map for the area of study was produced. And this served as a fundamental basic data upon which the rest were digitised. All linear and areal calculations were automatically handled by some tools within the software. From the furthest sub-catchment to the point of flooding is about 2km. Surface elevation contours spaced at 1m resolution were generated through the use of a digital elevation model (DEM) of 30m by 30m resolution obtained from the ministry of lands in Zambia. An observed natural slope from the surface contours on the map helped in determining the system of pipe/ channel network and the directions of flow. However, where possible, verifications were done through accessible hydraulic elements.

Inaccessibility of some hydraulic elements made it difficult to determine the slopes of the sections in the drainage network system hence in this study grounnd surface slopes are assumed. But the depth of most manholes and conduit dimensions were measured. It was during these field visits that the land-use was observed though combined with the help of from satellite imagery. This in turn coupled with literature guidelines was a good input in estimating the imperviousness of sub-catchments. Throughout the entire period and process, images were taken including during the flood event on 31st January, 2016. Figure 14 below show the map of the area under study and some other catchment details developed using ArcGIS. The map further shows photos of particular locations within the map to give a vivid visualisation of the area under study.

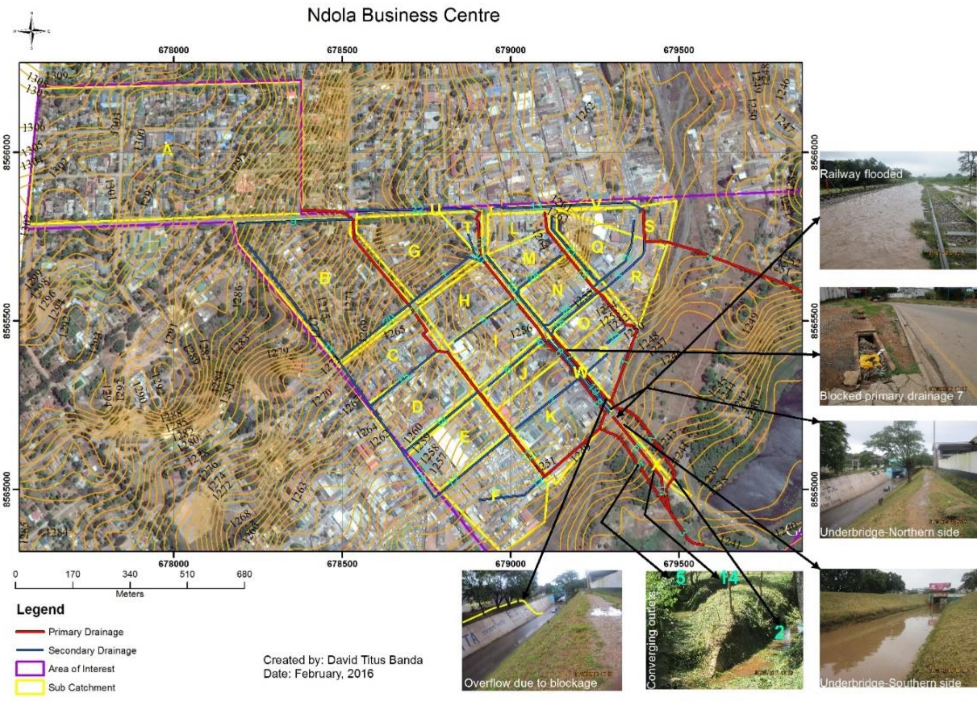

Figure 14: Map showing area of study with hydrological and hydraulic elements

4.3 Rainfall

Rainfall data was obtained from the provincial metrological department in Ndola which records data for Ndola. The weather station is located at Simon Mwansa Kapwepwe International airport within the city. Studies show that for rainfall data to be representative, the weather station should be within a radius of 5km in the area of interest [9]. The area of study in this report satisfies the condition above because from the furthest sub-catchment to the weather station is about 4km. The meteorological department in Ndola measures only average daily values of rainfall hence, data obtained were only daily averages in millimetres of precipitation. Rainfall data collected was from October 2007 to 31st March 2016.

The rainfall data was used to generate the Intensity Duration Frequency (IDF) curves from which inputs to the model were extracted. During a rainfall duration of 45min corresponding to the time of concentration for the catchment, average rainfall intensities of 22, 25, 40 and 50mm/hr for 1, 2, 4 and 8 year return periods were determined although it is uncertain as data used was for a short period only. For future designs, it would be better to have IDF curves derived for each weather station throughout the country. This will improve the accuracy on further modelling and designs as it will localise the climatic conditions thus being more representative for a particular catchment. The step by step procedure of how to generate IDF curves is shown below

4.3.1 Procedure for Generating IDF Curves from Daily Rainfall Data

For each year, maxima for different durations in days were determined and used as inputs to Table 1 below.

A table containing duration in consecutive days and number of years for the collected rainfall data as shown below was created.

Table 1: Duration in consecutive days and number of years for the collected rainfall data

	DURATION						
Year	**1 day**	**2days**	**3days**	**4days**	**5days**	**6days**	**7days**
1							
2							
3							
4							
5							
6							

For each duration in Table 1, the biggest volume to the smallest were determined hierarchically. Then to come up with a specific rainfall event return period, a formula $T = \frac{N}{n}$ was used, where T is the rainfall event return period in years, N is the total number of years used to extract data from e.g. 6 from Table 1 and n is any number from 1-6. n should give the desired T when divided into N and it also represents the n^{th} largest volume/ value for each duration in Table 1. For example, if the desired rainfall event return period is 2yrs, then using the formula T=2 since its already known, N=6 which is always fixed as it represents the total number of years from which data is extracted, then *n*=3 in this case. Therefore, it also follows that *n* represents the third largest volume/ value for each duration in Table 1. This leads to another Table 2 that contains values for *T* and *n* for each duration as shown below.

Table 2: Values for T and n for each duration

	DURATION						
Period (T)	**1 day**	**2days**	**3days**	**4days**	**5days**	**6days**	**7days**
1	n	n	n	n	n	n	n
2	n	n	n	n	n	n	n
3	n	n	n	n	n	n	n
4	n	n	n	n	n	n	n
5	n	n	n	n	n	n	n
6	n	n	n	n	n	n	n

From Table 2, a log-log plot of duration and volume represented by n for each return period (*T*) is done. From excel a power trend line is inserted to generate a straight line and the equation of the line determined. The reason for a power trend line is because the relationship between the amount of precipitation *n* and duration *t* is given by a power equation $h = at^{n}$ [14]. Where *h* is the precipitation amount in millimetres, *a* is constant, *t* is the duration in hours and *n* is the n^{th} largest value. Once the equation has been determined, its derivative I= nat^{n-1}gives the instantaneous intensity while the average intensity is given by the change in precipitation (*h*) over the entire duration (*t*) I= at^{n-1}. Therefore, for each return period, the intensities can be determined for different durations using the equation. The return period too can be changed to frequency by getting its inverse hence, Intensity-Duration-Frequency (IDF) curves can be generated [14].

Below is Figure 15, the graph showing the IDF curves produced for the purpose of this study.

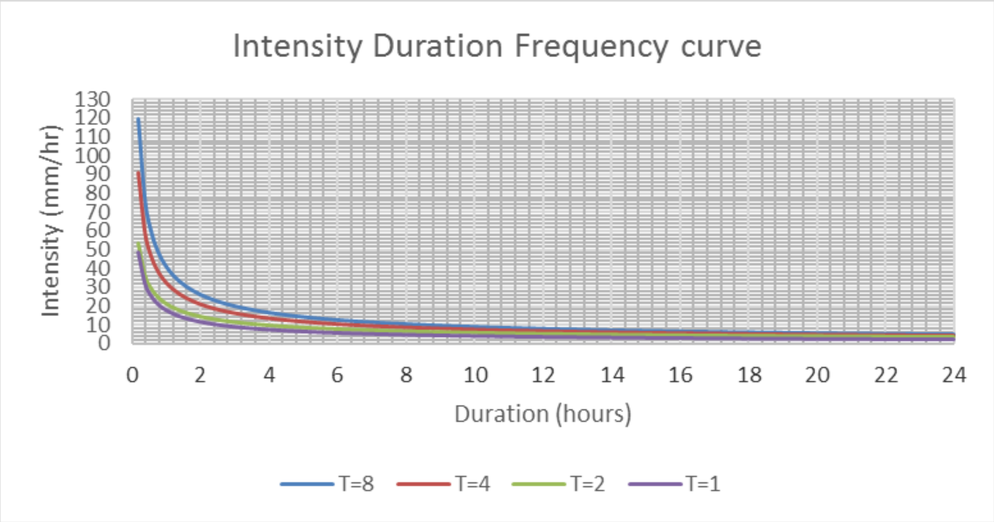

Figure 15: Intensity Duration Frequency curve (IDF) for the area of study

4.4 Catchment Characteristics

4.4.1 Hydrology Properties

The whole study area was divided into 35 sub-catchments depending on the flow direction of its runoff and the manhole to which the sub-catchment was connected. As explained earlier, its land-use was determined and a percentage imperviousness assigned. In SWMM, the infiltration capacity was considered to be decreasing with increasing rainfall period due to saturation of the soil and therefore the infiltration method chosen in the model was curve number (CN) system. The CN is basically a coefficient expressing total precipitation in terms of runoff potential therefore, a higher curve number signifies a higher potential of runoff of each sub-catchment and is dependent on its land-use cover type and soil type. Each soil group has a CN from which a particular sub-catchment's CN can be determined. Soil types can be more complex but it is assumed to be almost the same under this study due to relatively small area under study. With information on soil type and land-use cover, CNs for all the 35 sub-catchments were determined from the Technical Release Manual number 55 (TR 55) on Urban Hydrology for Small Watersheds [15]. Since an IDF curve was developed, the rain format in this case was intensity with mm as units.

4.4.2 Time of Concentration

The time of concentration is basically time taken for runoff to travel and reach the outlet from the hydraulically most distant point in the catchment. Several methods are available for computing the time of concentration but the simplest is used in this study. The furthest point in the catchment was determined and this was in sub-catchment C1. The water particle is assumed to flow in two segments. First overland/ unpaved in sub-catchment C1 to a point where it gets into the channel and then from there to the outlet called out1 in the model. From the TR-55 chapter 3, using an average slope of 0.028 for sub-catchment C1, the velocity for the first segment was determined to be

0.78m/s over a length of 1000m and using an average slope of 0.034 for the other sub-catchments where the channel passes, the velocity for the second segment was determined to be 1.2m/s over a length of 1700m. Therefore, the total time of concentration T_c=45min was determined. Taking the time of concentration to represent the critical rainfall duration, then the average intensity from the IDF curves is 22, 25, 40 and 50mm/hr for 1, 2, 4 and 8 year return periods respectively. Below is Figure 16 showing an image of the catchment with segment-1 representing overland flow and segment-2 representing flow on paved surface. Figure 17 is the graph for average velocities used to estimate travel time [15].

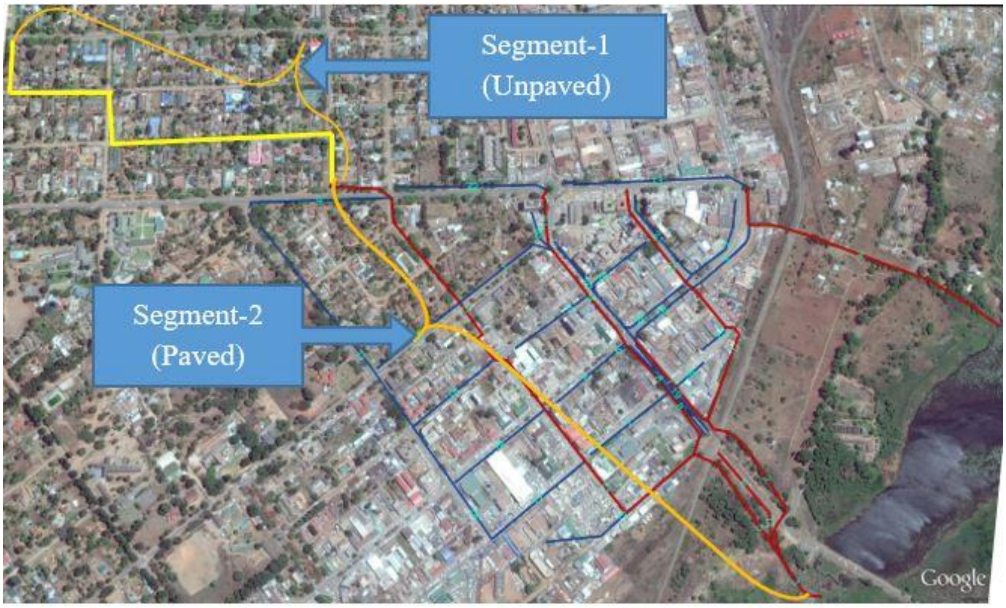

Figure 16: Segments used in determining the time of concentration

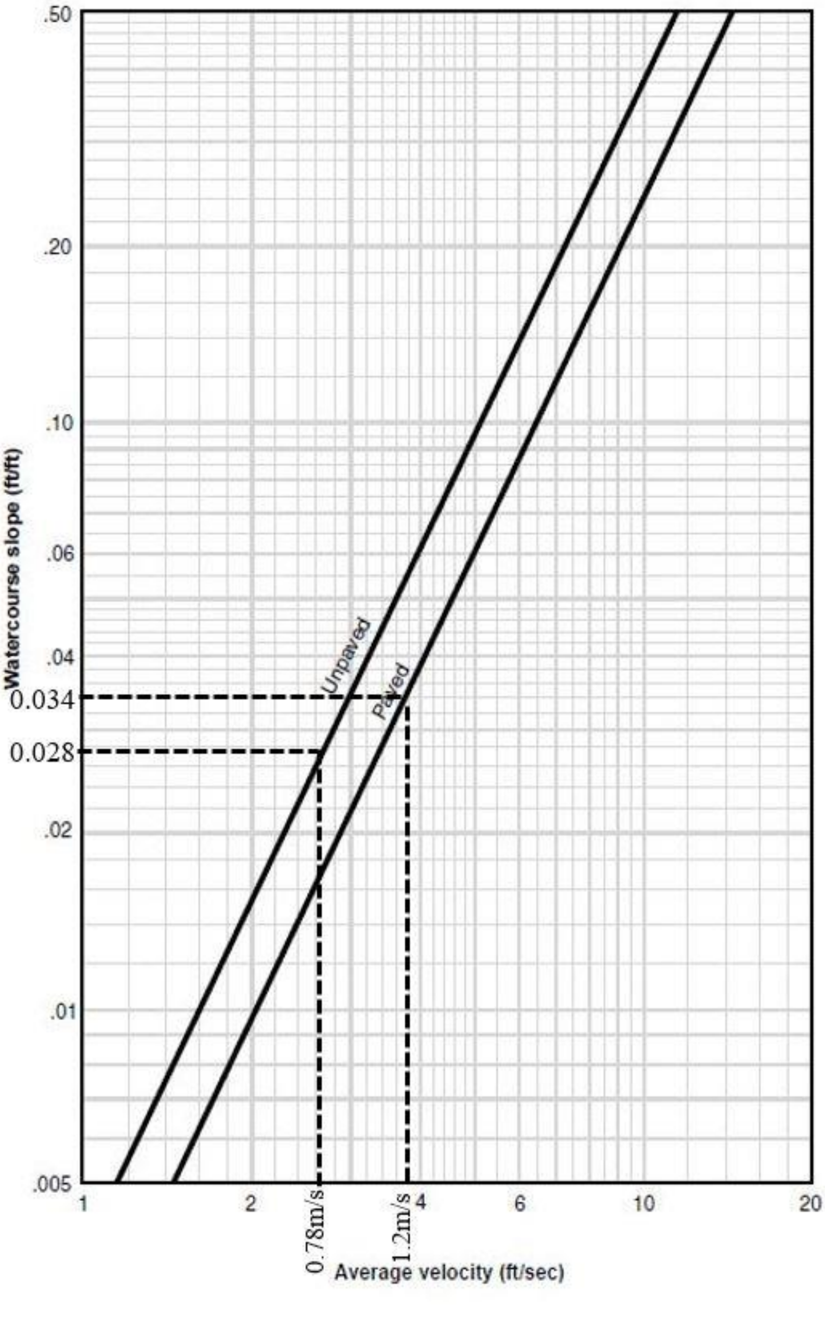

Figure 17: The graph used to estimate the velocity for each segment time [15]

4.4.3 Hydraulic Properties

Some of the depths of the manholes were measured while others were generated with reference to the surface slope. And wherever the manhole was accessible, the conduit properties were also measured. The drainage system presented herein, has a mixture of different types of conduits that is rectangular, circular and trapezoidal of which some are open channel in nature whilst others closed and underground mainly made out of concrete material. Manning's equation in SWMM was opted for due to unpressurised flow system with an average manning's roughness of 0.01. Since the hydraulic properties developed in SWMM assumed the slope of the surface, the drainage system is seen to have a very high slope which need verification in future studies. A more complex scenario was considered for this model, therefore, dynamic wave routing method was chosen. Below is Figure 18 showing one of the accessible manhole for measuring some hydraulic properties' parameters such as diameter, depth etc.

Figure 18: Measuring hydraulic properties' parameters

4.5 Flooding

Flooding at the underbridge has occurred at least once a year for the past 8 consecutive years with the latest on 31st January, 2016. The daily rainfall recorded on the 31st of January, 2016 was 71.4mm. However, on 15th December, 2014, the meteorological department recorded 120.5mm for the same area but no flooding was reported according to residents and the Ndola City Council. This suggests that ground saturation leading to raised ground water table contributes towards flooding or the intensity on 31st January 2016 was higher than that on 15th December 2015. No life has been reportedly lost over the underbridge floods but damages to cars due to high water depth of up to half a meter approximately. And the road has been damaged every after each flood such that the Ndola city council have to undertake some repair works every year.

5 STORMWATER MANAGEMENT MODEL (SWMM) AND ITS IMPLEMENTATION

5.1 Model Structure and Theory

"The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas (Lewis A. Rossman,2009) [6]".

The model utilises sub-catchments from which runoff is generated and keeps track of it at every stage in the routing process [6]. Hydraulic elements such as pipes, channels, and many more, are utilised in the routing process. Its main governing equation in the routing system relating flow rate, flow depth and bed slope is Manning, while the Hazen-Williams or the Darcy-Weisbach equations are used for pressurised flow [6]. The main inputs to the model include hydrologic parameters such as sub-catchment properties in form of size and imperviousness needed to generate runoff and subsequent hydraulic properties for routing such as pipe network system [6]. The output from the model include among others, time series graphs and tables, profile plots, runoff from each sub-catchment, flow in each conduit, flooding and associated time and duration [6].

SWMM is capable of modelling a variety of scenarios, among them are; Rainfall varrying in time, snow accumulation and melting, rainfall infiltration in unsaturated soil layer, flow between ground water and drainage system, including complex ones like non linear reservoir routing of overland flow [6]. The model is so flexible hyraulically in that it can accommodate an unlimited size of a network, it can handle a combination of closed, open and natural channels in the same system at once [6]. SWMM accounts for water quality inputs and external flows from various sources such as rainfall, sanitary, ground water flow, surface flow and even user defined [6]. The software can use either kinematic or dynamic wave in routing, therefore, it handles backwater, surcharge, reverse flow and surface ponding [6]. It can also handle pollutant loads and keep track of it at every stage [6]. Some specific applications of SWMM are designing and sizing the drainage system, designing the control measures to reduce the combined sewer over-flow and mapping of flood plains in natural channel systems [6]. The software is open source and as such it has undergone several major upgrades from the time it was first developed in 1971 [6]. The software can be accessed by anyone which makes it easier for any further improvements on the system currently under study.

5.2 System Schematization

Figure 19 below shows a conceptual model diagram of the connections in the pipe network. It emphasises on junctions, conduits and showing the number of sub-catchments connected to each junction/ node/ manhole. The respective numbering of both junctions and conduits are not necessarily hierarchical due to editing at the developing stage in SWMM but are presented the same way both in SWMM model and the schematic diagram. To avoid overcrowding information, the sub-catchments are not named in the schematic diagram but can be viewed from Figure 21 below.

Ideally, the model should have been developed with the secondary pipes as shown in blue or thinner lines in Figure 20. All the sub-catchments should have been connecting to the secondary pipes and not directly to the primary/ main pipes leading to the outlet. But due to lack of data on secondary pipes owing to their inaccessibility, the model in SWMM was simplified as shown in Figure 21.

Since the main problem and focus in this model lies at the underbridge downstream of the catchment, therefore, even if the model in SWMM is simplified without secondary pipes, the amount of water reaching the downstream section is still the same and would still be representative in terms of overall behaviour at the underbridge. The only problem that arises would be smoothening of water flow which may also affect the shape of the hydrograph.

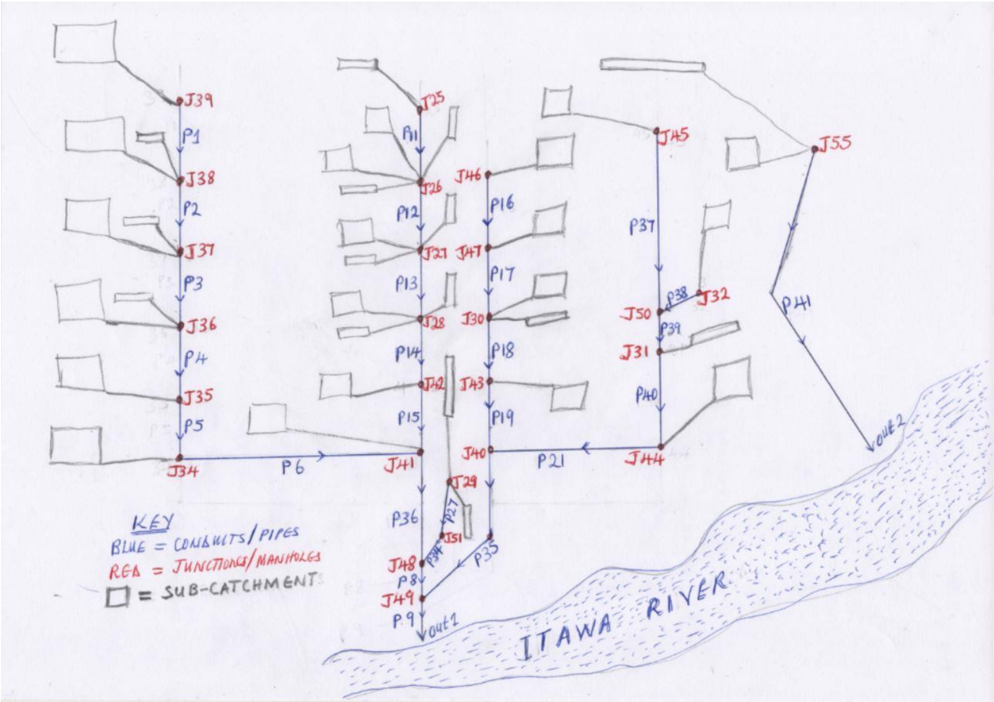

Figure 19: Schematic diagram showing conduits, junctions and sub-catchments

5.3 Model Setup

The basic inputs to SWMM under such a study can be classified as hydrology (sub-catchment properties and rain gauge properties) and hydraulic (conduit properties, node properties). In this study, sub-catchments were categorised as either, single-family area, apartment dwelling area, business area and/ or asphalt road. All these categories have a range of imperviousness from literature [7]. Figure 20 below developed in ArcGIS and overlaid with surface contours was used as a background when setting up the model in Figure 21 in order to conserve the shape, linear and areal parameters. Therefore, the area of each sub-catchment was calculated within the software while the average width was measured using an inbuilt tool in SWMM. The average slope for each sub-catchment was determined from the elevation contours of Figure 20 coupled with Figure 21.

Figure 20: Both secondary (blue/ thin lines) and primary (red/ thick lines) drainage

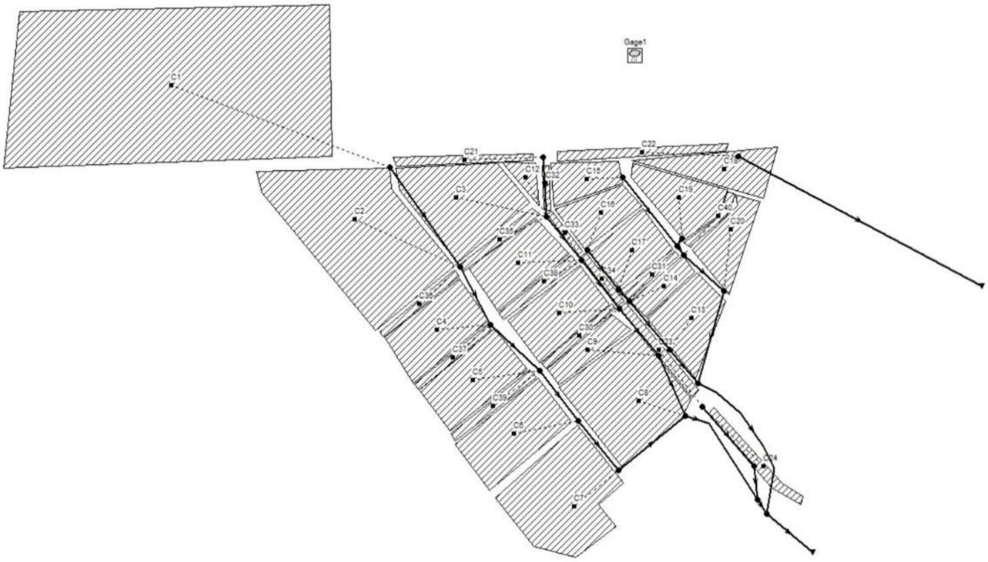

Figure 21: Outlook of the model of the area of study in SWMM

In both the model in SWMM and schematic diagram above, outlet 2 is also shown though it is independent from the rest of the system. During the flood that occurred on 31st January, 2016, much of the flood waters on top of the underbridge (overbridge) were as a result of the blockage of the channel leading to outlet 2. This brought about double flooding of both the top and bottom of the bridge which in turn poses a threat to the strength of the structure. Below is the effect of outlet 2 blockage as at 31st January, 2016. The two images of Figure 22 and Figure 23 below were taken at the same time (less than a minute time difference) and position (Right above the underbridge) but facing opposite directions (East-West).

Figure 22: Flooding along the rail right above the underbridge in western direction

Figure 23: Flooding along the rail right above the underbridge in eastern direction

6 MODEL APPLICATION

6.1 Calibration

It was difficult to calibrate the model as no flow measurements have been done or any other data recorded for the study area. The rainfall intensity also was not measured except the daily average values which could not be used directly into the model. The only form of calibration that was done on the model was to observe flooding at the underbridge when existing (blockages) conditions were simulated using rainfall intensity of one year return period since the area under study floods at least once every year. The rainfall intensity was extracted from IDF curves generated from short term rainfall period of 2007 to 2016.

6.2 Validation

The model could not be validated as there were no measured benchmarks to refer to. Nonetheless, some observed scenarios on the ground were reproduced. Whenever ideal conditions were simulated, very little or no flooding was observed but with the existing (blockages) conditions, flooding was observed exactly in the same area in the model as in reality.

The other observation that was reproduced was the fact that conduit P36 had more flow hindering conduit P34 from effectively discharging its water into conduit P8. This was seen through the conduit profile and confirmed by the hydrograph of two conduits.

6.3 Sensitivity Analysis

Since data was collected manually on site, it was of great importance to perform a sensitivity analysis. Sensitivity analysis involves holding all variables constant except small variations in one. If the system shows significant changes with the small variations in a particular variable, then the system is said to be sensitive to that parameter and the more accurate that parameter should be [5]. In this study, sensitivity analysis can be categorised in two types that is hydrological and hydraulic.

6.4 Simulation of Scenarios

Up to 7 different scenarios were simulated in order to investigate the sensitivity of the system. Conduit size, slope and roughness are classified as hydraulic and the rest namely infiltration rates, sub-catchment Manning's roughness, percent of imperviousness and width of overland flow are classified as hydrology. The criteria used on whether the variable was sensitive or not was critical intensity. The critical intensity herein means the maximum intensity the system could handle without flooding.

Infiltration Rates

The infiltration model chosen for the current study is CN. Therefore, the CN values for individual sub-catchments were reduced by 50% but holding the rest of the variables constant to see what happens to runoff. This increased the infiltration rate as 27% of the total precipitation over the entire catchment was infiltrated as compared to 10% before the CN values were changed. In the overall system, the critical intensity was also raised from 35 to 37mm/hr.

It can be concluded that the system is slightly sensitive to CN values for each sub-catchment.

Sub-catchment Manning's Roughness

Manning's roughness for both the impervious and pervious areas of the sub-catchments were increased by 50% each. The difference in the total runoff before and after the increment was 0.4% resulting into a raised critical intensity from 35mm/hr to 38mm/hr. signifying that the system is more sensitivity to sub-catchment manning's roughness compared to infiltration rates.

Percent of Imperviousness

A 50% reduction in the percent imperviousness for each sub-catchment reduced the runoff by 6.84% thereby raising the critical intensity to 43mm/hr from 35mm/hr. The system was considered to be sensitive to percent imperviousness.

Width of Overland Flow Path

The overland flow path was reduced by 50% resulting into a difference of 0.74% in runoff before and after. The reduction in runoff from 89.99% to 89.25% raised the critical intensity to 40mm/hr from 35mm/hr which is very significant.

Conduit Size

The size of conduit P27 at the under-bridge was increased by 50% and this raised the critical intensity to 52mm/hr from 35mm/hr. The same increase in critical intensity can be achieved if the same conduit is increased by 33% signifying that there is a maximum to which changes in conduit dimension can affect the system above which other variables are responsible.

Conduit Slope

Conduit P27 was chosen because that was where the problem occurred first in the system. Increasing the slope by 50% did not produce any significant changes to the system as the critical intensity just remained the same at 35mm/hr.

Conduit Manning's Roughness Coefficient

When manning's roughness for all the conduits in the system are increased by 50%, the critical intensity is reduced to 25mm/hr. from 35mm/hr. This could be as result of reduced flow due to too much roughness and was considered to be significant.

6.5 Sensitivity Analysis Results

Three of the scenarios investigated in sensitivity analysis namely percent imperviousness, width of overland flow and conduit roughness were considered to be sensitive for the system because they altered the critical intensity by more than 10%. Great care was used in determing the values of the above sensitive parameters.

7 RESULTS AND DISCUSSION

Up to 8 different scenarios were simulated under this study. But all the 8 scenarios can be classified into only two categories namely ideal conditions and existing/reality conditions. Ideal conditions mean that all sub-catchments are connected to their intended manhole as per initial design while existing/reality conditions assumes blockages and hence some sub-catchments are not connected to the intended manhole as per initial design, instead connected to other manholes to mimic the actual flow of runoff as observed. Modifications were made to the model under each of these two categories and simulated and these included changing slopes and dimension of certain portions in the drainage system. The aim of simulating all the 8 scenarios was to come up with the best possible solution to the underbridge flooding by observing the behaviour of the system under different scenarios.

The model in SWMM for the area of study was developed and ran for a maximum of 12hrs at varying intensities but constant duration of precipitation into the catchment of 3hrs. Flooding is observed at the underbridge or junction 29 in the model before anywhere else could flood and no other part in the system floods simultaneously with the underbridge during the simulations in this study. In reality (31/01/16) when flooding occurs at the underbridge, it takes at least 5hrs from the time it stops raining for all flood waters to drain. At the deepest point, it can be up to half a metre of flood depth. In the tables below, *critical* means that any value higher than the critical one, causes the system to flood. *Surcharge* means that the water level in the node/ manhole is above the crown of the highest conduit/ pipe, or in other words the conduit/ pipe is completely submerged due to increased volume of water. When it is a conduit/ pipe that is surcharged, then it means that it is full either upstream, downstream or along its entire length. And the annotations *D* for dry means *not flooding* and *F* means *flooding*.

The rainfall intensities (22, 25, 40 & 50mm/hr) and their respective return periods (1, 2, 4 & 8yrs.) were derived from the generated IDF curves as shown again in Figure 24 below.

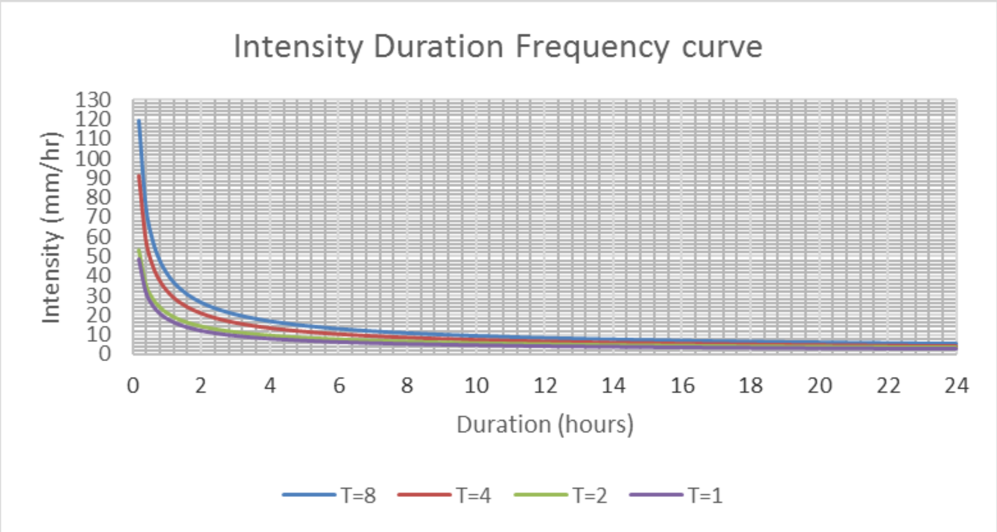

Figure 24: Intensity Duration Frequency curve (IDF) for the area of study

scenario 1: Simulating ideal conditions

As mentioned above, Ideal condition mean that storm water runoff from every sub-catchment goes to its designated outlet/manhole as planned in the drainage system design. The aim of this scenario is to observe the system behaviour according to the initial design and this saved as a reference point for the rest of the scenarios under the ideal conditions category. Table 3 below show results from scenario 1 simulation.

Table 3: Simulation of ideal condition (scenario1)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
1	22	D		
2	25	D		
Surcharge(J29)	28	D		
Critical	35	D		
4	40	F	0.01	
8	50	F	3.02	421

According to this scenario, the critical intensity 35mm/hr is between 2yrs and 4yrs return period. This signifies that the system can handle a rainfall event of at least 2yrs return period without flooding and therefore, under these conditions no yearly flooding of the underbridge should be excepted. But since flooding occurs in reality, this leads to scenario 2.

Scenario 2: Simulating Existing (Blockages) Conditions

As of January, 2016, some parts of the drainage system were blocked hence the runoff did not go to its designated manhole but instead to other manholes.

In some cases, the drainage was fully blocked while in some, little flow was observed. In this scenario, only some road sub-catchments or the runoff from some roads ends up at the underbridge. From observations, the names of the roads that lead their storm water runoff to the underbridge under blocked conditions are parts of President Avenue, that is the stretch between Chimwemwe Road and Moffat Road, Buteko Avenue, the stretch between Chimwemwe Road and Moffat Road, Independence Avenue the stretch between Chimwemwe Road and Maina Soko Road and finally Maina Soko Road stretching from Broadway Roundabout to the bridge on Dag Hammarskjöld Drive. The above mentioned roads also shown in Figure 25 below could be prioritised when it comes to maintenance especially prior to rainy season.

The aim of simulating this scenario is to investigate the influence of blockages towards the yearly flooding of the underbridge.

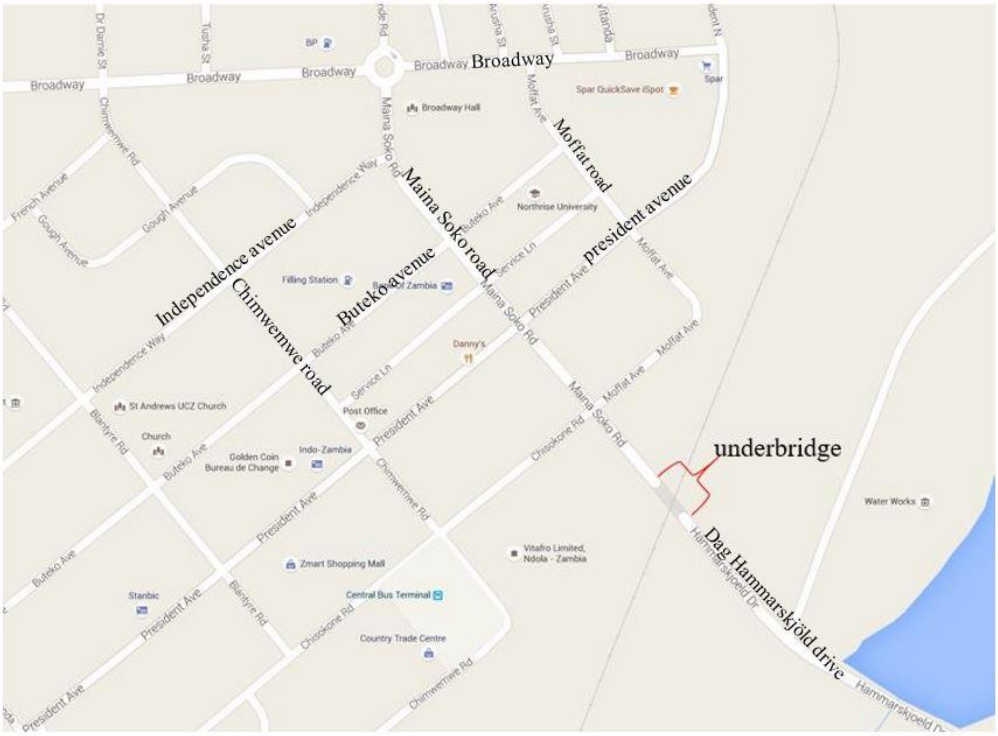

Figure 25: Map showing roads within the catchment of the study area

Figure 26 below show a blocked drainage on the junction of President Avenue and Maina Soko Road within the catchment under study.

Figure 26: Fully blocked drainage at the junction of Maina Soko road and president avenue

Table 4 below show simulation results for scenario 2

Table 4: Simulation of existing condition (scenario 2)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
Surcharge(J29)	13	D		
Critical	15	D		
1	22	F	1.31	38
2	25	F	3.17	340
4	40	F	3.24	2680
8	50	F	3.26	4292

Scenario 2 show a drastic change and shift of the critical intensity to below one year return period rainfall event. A one year return period rainfall event according to scenario 2 causes the system to flood which could be the reason why the system floods at least once a year.

Changes in slopes: Background to scenario 3 and scenario 4

Since the most, if not the only, sensitive part in the system is the underbridge hence changes to slopes were only done on the network from the underbridge (J29) to outlet 1 bearing in mind the maximum allowable slope of 10%. The slopes in the ideal model were based on the ground surface slopes and this serves as a basis or reference for these changes. The motivation behind changing slopes was from the back water effect observed on open channel P34. The slope of the pipe from the underbridge (P27) was increased by 150%, and that of the semi-circular open channel (P34) connecting the underbridge pipe to the other open channel (P36) was reduced by 48%, the open channel (P8) connecting (P36) to the junction where the other open channel (P35) joins was increased by 36% and finally the last segment (P9) leading to the outlet was decreased by 55%. Modifications to the slopes was done with the aim of improving the system hence manhole/ junction 29 and outlet 1 are fixed as in ideal conditions. Below is Figure 27 showing the conduits mentioned above.

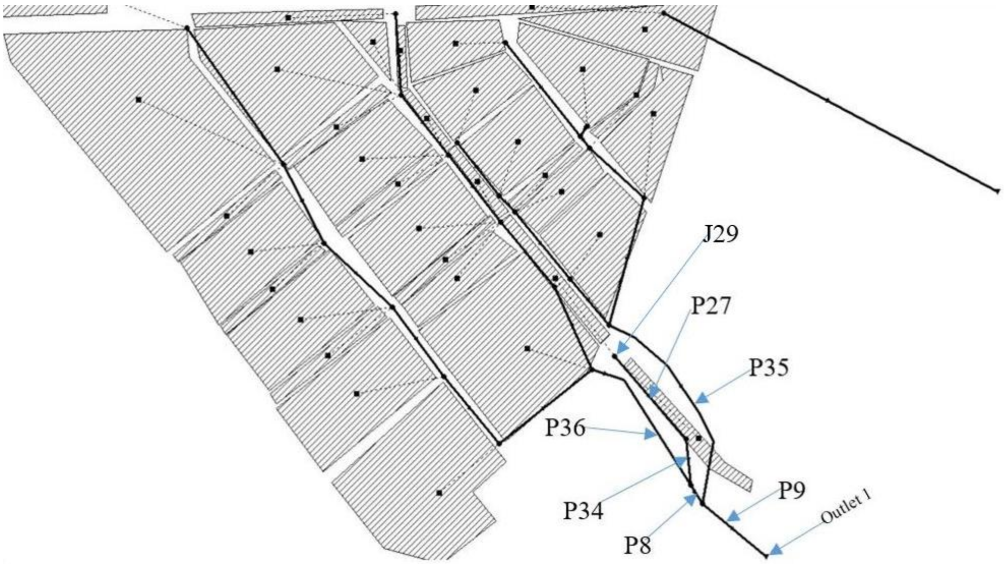

Figure 27: Part of model showing conduits and nodes

scenario 3: Simulating ideal conditions with changes in slopes

In scenario 3, the conditions are the same as in scenario 1 except for the changed slopes as mentioned in the background above. The results for scenario 3 are shown in Table 5 below.

Table 5: Simulation of ideal condition with changes in slopes (scenario 3)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
1	22	D		
2	25	D		
Surcharge(J29)	36	D		
4	40	D		
Critical	43	D		
8	50	F	2.46	985

In comparison to scenario 1, modifications to the slope as shown in Table 5 above improves the system performance by raising the critical intensity to above 4yrs but below 8yrs return period rainfall event. This means that if slopes were to be as presented in scenario 3, then the underbridge would not flood every year under the current intensities.

scenario 4: Simulating existing (blockages) conditions with changes in slopes

In scenario 4, the conditions are the same as in scenario 2 except for the changed slopes as mentioned in the background above. The results for scenario 4 are shown in Table 6 below.

Table 6: Simulation of existing condition with changes in slopes (scenario 4)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
Surcharge(J29)	19	D		
Critical	19	D		
1	22	F	0.01	
2	25	F	1.96	48
4	40	F	3.23	2205
8	50	F	3.27	4585

No much improvement is seen in scenario 4 as compared to scenario 2. Both being blocked conditions; it seems the modification of slopes is not really playing an important role because the critical intensity (19mm/hr) in scenario 4 as seen in Table 6 above is still below one year return period. This means the system will continue to flood at least once a year. The major difference between scenario 2 and 4 is the flood volume and flood duration which is higher in scenario 2.

Changed dimension Conditions: Background to scenario 5 and scenario 6

Only the size of the underbridge pipe (P27) was varied because that is where the major problem is as compared to the rest in the system. And in reality it is the most feasible compared to the open channels made of concrete lining. The actual size of the pipe is 450mm, lying on a total length stretch of approximately 200m and it was changed to 600mm.Variations were made under both ideal and real existing conditions.

scenario 5: Simulating ideal conditions with changed dimension (P27=600mm)

In scenario 5, the conditions are the same as in scenario 1 except for the changed dimension of the underbridge pipe (P27) as mentioned in the background above. The results for scenario 5 are shown in Table 7 below.

Table 7: Simulation of ideal condition with change in dimension (scenario 5)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
1	22	D		
2	25	D		
Surcharge(J27)	40	D		
Surcharge(J29)	42	D		
8	50	D		
Critical (J29)	52	D		
Underbridge(J29)	53	F	0.53	104

Comparing scenarios 3 and 5 where there is modification to the system in both cases but under the same category of ideal condition, scenario 5 as seen in Table 7 shows a greater improvement in the critical intensity of above 8yrs return period rainfall event as compared to between 4yrs and 8yrs return period rainfall event for scenario 3.

scenario 6: Simulating existing (blockages) conditions with changed dimension (P27=600mm)

In scenario 6, the conditions are the same as in scenario 2 except for the changed dimension of the underbridge pipe (P27) as mentioned in the background above. The results for scenario 6 are shown in Table 8 below.

Table 8: Simulation of existing condition with change in dimension (scenario 6)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
1	22	D		
2	25	D		
Surcharge(J29)	30	D		
Critical (J29)	31	D		
4	40	F	2.18	155
8	50	F	3.21	2007

Scenarios 4 and 6 also present modifications to the system but under similar conditions. However, scenario 6 as can be seen in Table 8 shows greater improvement as the critical intensity (31mm/hr) is now between 2yrs and 4yrs return period rainfall event as compared to scenario 4 which is below one year.

scenario 7: Simulating ideal conditions with changes in both slopes and dimensions (P27=600mm)

Scenario 7 is the combination of scenarios 3 and 5 conditions both modifications to slopes and dimension but under ideal conditions. The results for scenario 7 are shown in Table 9 below.

Table 9: Simulation of ideal condition with changes in both slopes and dimension (scenario 7)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
1	22	D		
2	25	D		
Surcharge(J27)	40	D		
4	40	D		
Critical	43	D		
8	50	F	1.68	356

Comparing scenarios 3, 5 and 7 in which all cases there was modification to the system but similar ideal conditions, scenario 3 and 7 show the same improvement in terms of critical intensity (43mm/hr) being between 4yrs and 8yrs return period rainfall event. The major difference as can be seen from both Table 9 and Table 5 above is that different nodes get surcharged. Node 29 in scenario 3 gets surcharged at an intensity of 36mm/hr but it does not get surcharged in scenario 7 because the pipe is big enough to accommodate the increased volumes of water. This is also seen in the flood duration and volume for scenario 7, it is capable of handling more runoff than scenario 3.

scenario 8: Simulating existing (blockages) conditions with changes in both slopes and dimensions (P27=600mm)

Scenario 8 is the combination of scenarios 4 and 6 conditions both modifications to slopes and dimension but under existing (blockages) conditions. The results for scenario 8 are shown in Table 10 below.

Table 10: Simulation of existing condition with changes in both slopes and dimension (scenario 8)

Return period (yr.)	**Intensity (mm/hr)**	**Status D= dry F= flood**	**Hours flooded (hrs)**	**Flood volume (m^3)**
1	22	D		
2	25	D		
Surcharge(J29)	36	D		
4	40	D		
Critical (J29)	42	D		
8	50	F	3.10	2776

Comparisons are made on scenarios 4, 6 and 8 all being modifications to the system under similar conditions. Scenario 8 as shown in Table 10 above exhibits the best improvement of the three scenarios with the critical intensity (42mm/hr) being between 4yrs and 8yrs return period rainfall event.

By deductive reasoning, scenario 5 presents the best improvement as compared to the rest. Below is a summarised table of all the 8 scenarios discussed above.

Table 11: Summary of 8 scenarios

Type of simulation	**Node surcharged**	**Node surcharge intensity(mm/hr)**	**Critical intensity(mm/hr)**	**Node first flooded**
Ideal conditions **(scenario 1)**	J29	28	35	J29
Existing Conditions **(scenario 2)**	J29	13	15	J29
Ideal conditions with changes in slopes **(scenario 3)**	J29	36	43	J29
Existing conditions with changes in slopes **(scenario 4)**	J29	19	19	J29
Ideal conditions with changed dimension (P27=600mm) **(scenario 5)**	J27	40	52	J29
Existing conditions with changed dimension (P27=600mm) **(scenario 6)**	J27	30	31	J29
Ideal conditions with changes in both slopes and dimensions (P27=600mm) **(scenario 7)**	J27	40	43	J29
Existing conditions with changes in both slopes and dimensions (P27=600mm) **(scenario 8)**	J29	36	42	J29

Underbridge conduit P34 and Overbridge conduit P36

From Figure 28 below, node J29 is the only inlet point for storm water for the underbridge network that runs through conduits P27 and P34. Whilst conduit P36 is the downstream part of a large network as seen from the figure and it runs over the bridge.

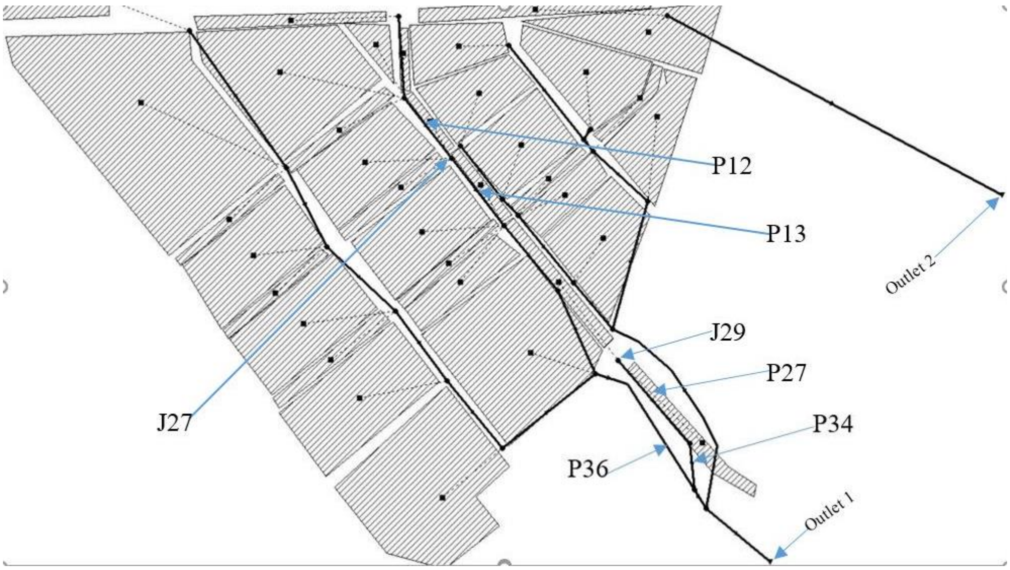

Figure 28: Some hydraulic elements in the catchment

Figure 29 below show an image of conduits P34, P35 and P36 junctions. Conduit P36 is the main, leading to the outlet as shown in Figure 28 above while conduits P34 and P35 discharge into P36.

Figure 29: Conduits P34, P35 and P36 junctions

Figure 30 below show a velocity-time graph of conduits P36 and P34 for a simulation at critical intensity 35mm/hr.

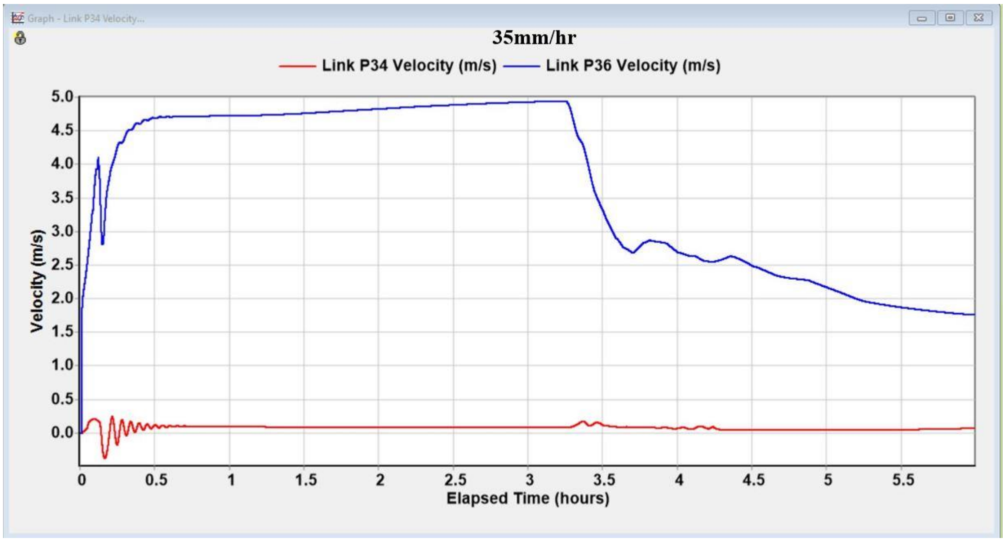

Figure 30: Scenario 1 Graph showing the velocity (m/s) in conduit P34 & P36 at the intensity of 35mm/hr

In simulating with the critical intensity 35mm/hr, there is no flooding but node J29, conduit P13 and conduit P27 are surcharged. From Figure 30, conduit P36 show a rise and sudden drop and another rise again before the velocity stabilises within 45min. The velocity is stable up to 3.25hrs after which it begins to drop. But in conduit P34 there is a sinusoidal shape within the first 45min. Thereafter, just like in conduit P36, the velocity is stable for approximately 2.5hrs after which it fluctuates before descending to zero. **Error! Reference source not found.** Table 12 below show surcharged conditions for conduits P27 and P13 for the same simulation conditions as for Figure 30 and Figure 31.

Table 12: Surcharged conduits and associated durations at 35mm/hr

Conduit	**Hours Both Ends Full**	**Hours Upstream Full**	**Hours Downstream Full**	**Hours Above Normal Flow**	**Hours Capacity Limited**
P27	2.56	2.73	2.58	2.87	2.51
P13	0.01	0.01	0.01	4.02	0.01

From the schematic diagram of Figure 19, is can be seen that conduit P34 receives its water from conduit P27. Table 12, shows that conduit P27 is surcharged for a duration approximately equal to the duration of velocity stability in Figure 30. In the same vein, from scenario 1 in Table 3, node J29 which is the upstream end of conduit P27 is surcharged for 2.73hrs. Therefore, the sinusoidal shape occurs before both conduit 27 and node J29 get surcharged. The velocity becomes stable mainly because both conduit (P27) and node (J29) upstream are surcharged.

Figure 31 below show a graph of flow against time in conduits P36 and P34 for a simulation at critical intensity 35mm/hr.

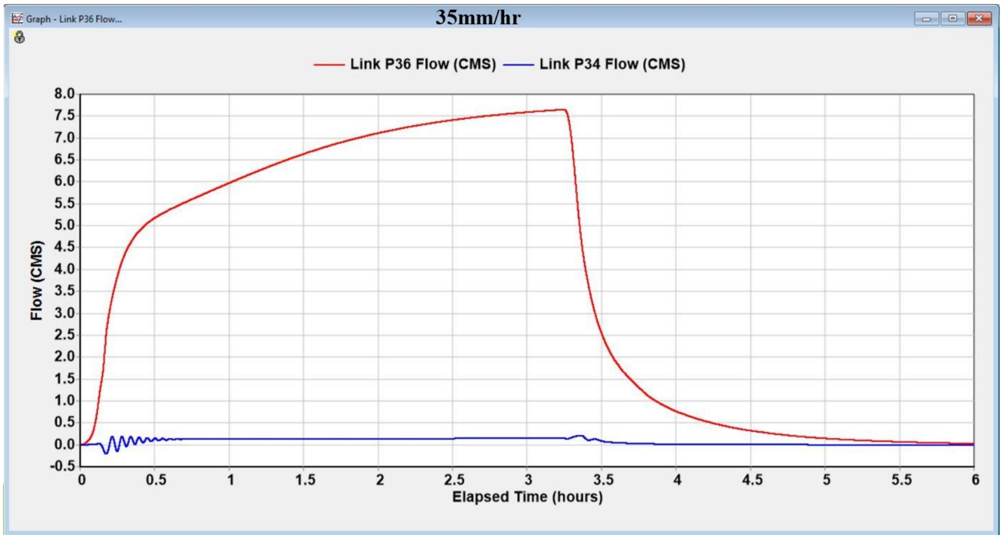

Figure 31: Scenario 1 Graph showing the flow (m^3/s) in conduit P34 & P36 at the intensity of 35mm/hr

Conduit P36 in Figure 31 shows a sharp and steady continuous rise in flow until a rainfall duration elapses then a sharp drop for the next 15min and steady for the last 2hrs before getting to zero. Conduit P34 again show a sinusoidal pattern of flow for the first 45min just like with the velocities in Figure 30.

In both Figure 30 and Figure 31 above there is a sharp drop on the recession side of the hydrograph and this can be attributed to hydrological and hydraulic factors such as conduit roughness, overland flow path width and the size of the catchment. The sudden decrease in rainfall due to block rain being fed into the system in combination with the above factors can influence the shape too. From the hydrograph it can be seen that there is rapid response immediately the simulation begins in within 15min. This is generally due to the small overland flow path width of the sub-catchments and the steep slope. The catchment under study is small with almost similar characteristics in all sub-catchments. In addition, the block rain being fed into the catchment could influence the outlook of the hydrograph. The total area of the catchment is approximately 100ha over an average slope of 3.5% and longest flow path length of less than 2km of conduit.

Oscillations in Hydrograph

Oscillations in the model occur in conduit P34 only. Generally, oscillations indicate model instability which is usually rectified by adjusting the time step. In Figure 30 and Figure 31, time steps were adjusted to as low as one second, but that did not eliminate the oscillations from the hydrograph. According to the Canadian Professional Engineer Todd S. Wyman, oscillations in SWMM can occur in the hydrograph when the following conditions exist [21]:

- If the flow in a conduit is between 95% full and surcharge condition
- When there is a small flow in a large pipe
- When there is a flow restriction downstream

Going by the above assumptions, the cause of oscillations is discussed below. Figure 32 below show a magnified graph of the flow oscillations in conduit P34

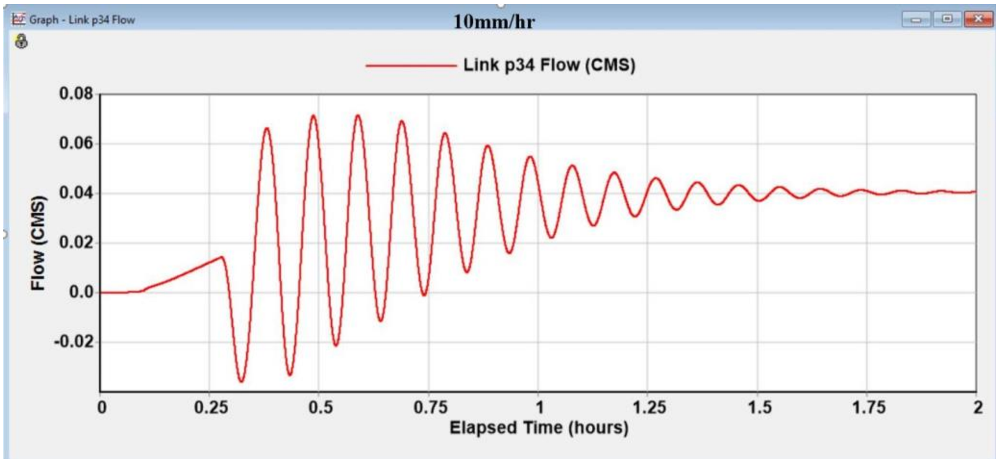

Figure 32: Flow oscillations in conduit P34 at an intensity of 10mm/hr

In Figure 32 above, it can be seen that oscillations begin after about 17min into the simulation and begin to smoothen after 105min. There seems to be a correlation between the oscillations in Figure 32 and the water elevation profiles shown in Figure 33 and Figure 34 below.

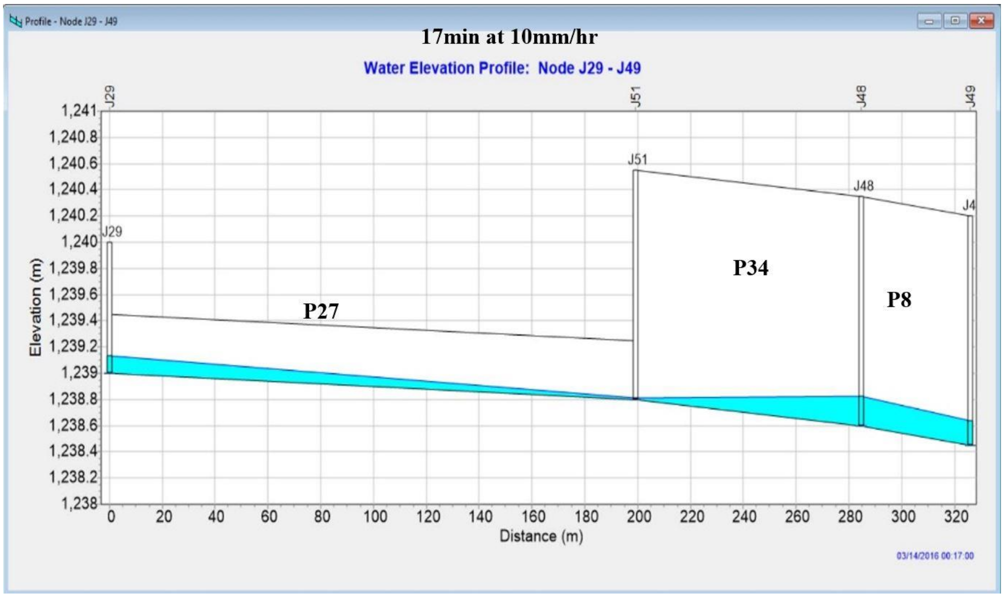

Figure 33: Water elevation profile for conduits P27, P34 & P8 17min into simulation at an intensity of 10mm/hr

The water profile in conduit P34 of Figure 33 imply a subcritical flow caused by restricted flow downstream. The profile is more prominent at 17min duration into the simulation which suggests the cause of oscillations at the same duration as seen in Figure 32. Figure 34 below on the other hand, show almost an even profile suggesting the beginning of stability in the flow thus the observed smoothening of oscillations in Figure 32 after 105min into the simulation.

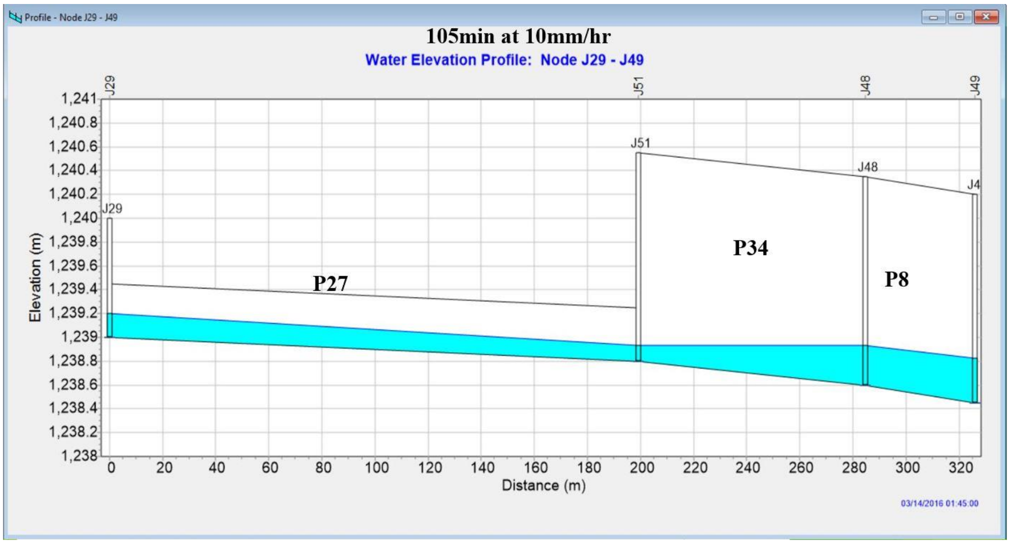

Figure 34: Water elevation profile for conduits P27, P34 & P8 105min into simulation at an intensity of 10mm/hr

Below is Figure 35 showing similar oscillations but at a different and higher intensity of 35mm/hr.

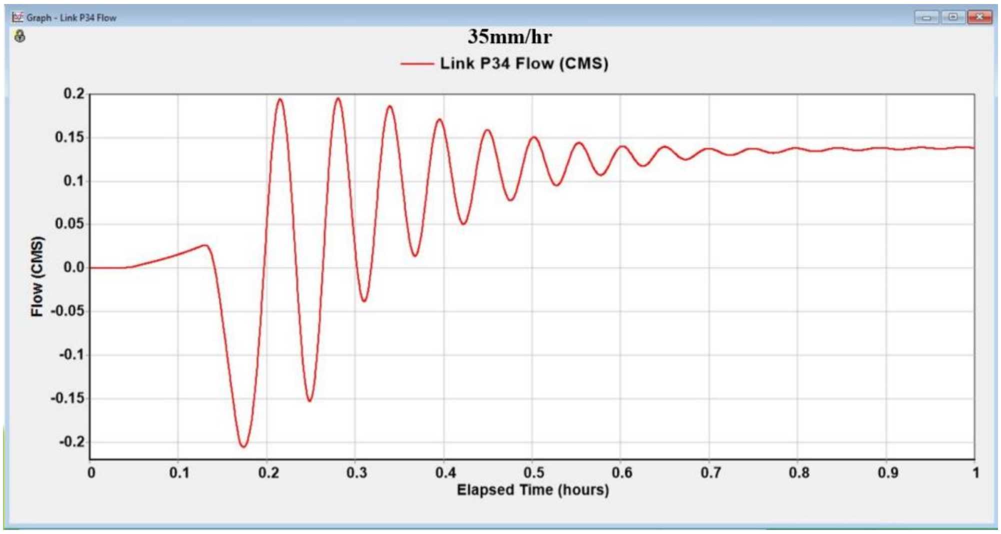

Figure 35: Flow oscillations in conduit P34 at an intensity of 35mm/hr

As opposed to Figure 32, simulating at an intensity of 35mm/hr shifted both the start of oscillations and subsequent smoothening. Figure 35 show that oscillations begin 9min into the simulation and begin to smoothen 41min later. The oscillatory duration at higher intensity is shorter (41min) than at lower intensity (88min) as exhibited by Figure 32 and Figure 35 respectively. This could vindicate some of the assumptions above about the causes of oscillations in conduits. It could be that the different intensities influence the rate at which flow restriction downstream occurs or the rate at which the conduit attains a flow between 95% full and surcharge condition. Similarly, Figure 36 and Figure 37 show the water elevation profiles how they correlate to the oscillations at an intensity of 35mm/hr.

Figure 36: Water elevation profile for conduits P27, P34 & P8 9min into simulation at an intensity of 35mm/hr

The major difference between Figure 33 and Figure 36 is the elevation of water in the conduit and the time it takes to develop a similar water elevation profile in terms of shape and slope. The same difference applies for Figure 37 below and Figure 34. Therefore, the same explanation holds.

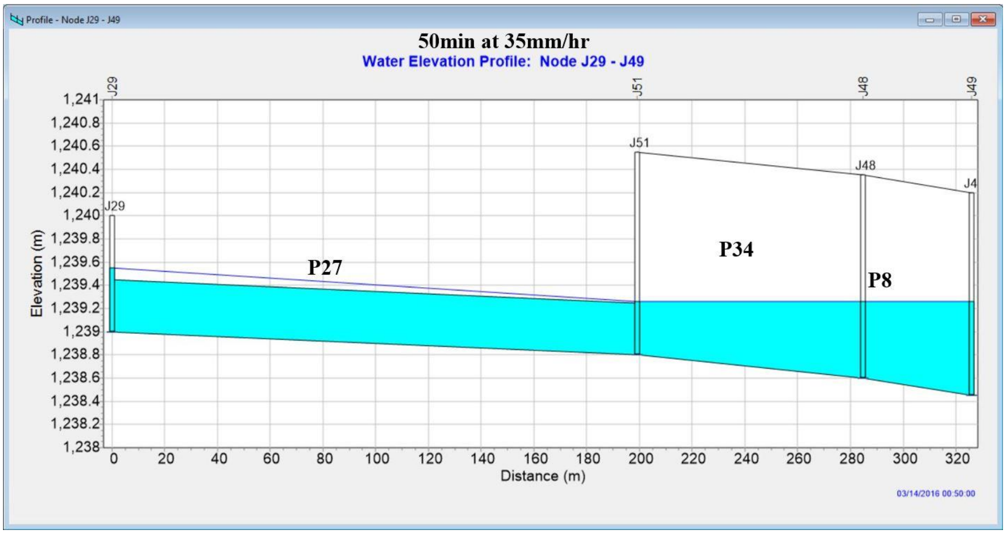

Figure 37: Water elevation profile for conduits P27, P34 & P8 50min into simulation at an intensity of 35mm/hr

The increasingly huge flow in conduit P36 as seen from Figure 31, can be said to be inducing backwater effects in conduit P34 by inhibiting it to discharge properly into conduit P36. As a result, there is reduced flow in conduit 34 and subsequent build-up of water resulting into flooding at junction 29 at the underbridge.

Siltation

By inspection and observation during field visits, silt build-up was noticed as one of the problems which could contribute to reduced flow area and blockages. Particularly on the southern side of the underbridge, one of the causes could be the steep side slope of earthen embankments which when it rains, soil is washed away along with runoff. But also, there seem to be a small slope for the pipe network from the underbridge which could affect the required minimum velocity for pipe self-cleansing of about 0.5-0.7m/s.

Figure 38: Silt build-up 2 days after the flood (31/01/16) causing blockage to the gulley and eventually pipes as shown in Figure 39 if not maintained

Figure 39: Image showing silt build-up inside the pipe due to lack of maintenance

Three images below show the same location (southern side of underbridge) taken at different times.

Figure 40: Earthen embankments with two gullies when there is no flooding

Figure 41: Flooding but two gullies cannot be seen due to depth of water (surcharge)

Figure 42: Two gullies when there was flooding (31/01/16) about an hour after Figure 41 image was taken

The model was simple to use requiring not much of complex input data. Its output was also quite substantial for understanding the behaviour of the system. It may not give accurate results in reality due to some uncertainties especially in the limited input data but gives a general overview of the behaviour of the system upon which decisions can still be made. The main uncertainties emanate from input data which was limited especially slopes for both hydrological and hydraulic elements, size of sub-catchments, length of conduits, actual drainage system network path, representative rainfall and hydrological parameters for sub-catchment rainfall infiltration. The model

also requires that whenever there is a connection between two conduits, there must be a node, which is not true in reality especially for open drainage surface flow.
In this catchment since the main focus and problem is downstream, this reduces the degree of uncertainty particularly for this study because all the runoff upstream still finds itself downstream regardless of the path as long as the system is closed.

7.1 Possible Solutions

From the simulated scenarios, the main problem seems to lie in the blockage of the drainage system which results into large volumes of runoff towards the underbridge drain which cannot handle the flows hence leading into flooding. There is also insufficient slope from the underbridge up to where it joins the rest of the outlet drain which in turn promotes silt build-up. In addition, backwater effects occur downstream the underbridge at a point/ junction where the underbridge drain joins the main line to the outlet.

Scenario 2 results show that the system floods with the rainfall intensity of less than a year return period which is also true in reality from observation. Usually such storm water drainage systems are designed to handle more than one year return period events and as such this may not be acceptable. Seeing this flooding occurs under scenario 2 due to blockages especially resulting from garbage thrown into the open drainage system which eventually finds its way into the closed system too buried underground, it is expedient that a comprehensive cleaning and maintenance program be put in place. The program may involve developing a good database for the whole system and regularly updating the status of individual hydraulic elements at fixed intervals with increased (monthly) updates prior to and during rainy season. These needed updates will bring about the formation of an inspection team which can even take an integrated approach by involving all stakeholders and bring awareness.

It is difficult to remove the garbage in closed systems hence preventive measures should be put in place to avoid garbage reaching the closed parts of the drainage system. One way would be to install meshes near the inlets to closed systems so as to trap the foreign objects but to be backed up by routine maintenance to avoid the trappings from reducing the flow area and blocking the system. The city council should place garbage bins in most strategic places within the city centre where people can throw their garbage to avoid throwing it anyhow. Policy should be reinforced by the relevant authorities to charge everyone that throws garbage onto undesignated places.

The aspect of looking for potential applications of blue-green solutions was explored. The Ortho-photo from google combined with observations through field visitation were used in identifying potential sites and features for the purpose of implementing sustainable urban drainage system facilities. From the observed land-use, the area seems built up already except in some portions of residential areas. The possibility is to incorporate such solutions right on private land of each household and on public land whenever there is maintenance. SUDS come in different kinds, forms, types and sizes, therefore, regardless of how built-up an area can be, they can still be implemented. This only calls for studies to ascertain which SUDS facilities are suitable. Recommendations should be made to utilise SUDS solutions for any new development and renovations. Figure 43 below shows the land-use of the catchment under current study.

Figure 43: Land-use of the catchment under current study

8 CONCLUSION

The drainage system for Ndola CBA was assessed and modelled from the data collected. It can be concluded that the major problem with the system is blockages resulting from mainly garbage being thrown into the systems as well as silt build-up overtime if no adequate maintenance is done. If the blockages alone can be done away with, the drainage system can handle rainfall events of more than 2yrs return periods basing on the generated IDF curves, the model setup and simulations. Priority should be given to a comprehensive cleaning and maintenance program. The program may involve developing a good database for the whole system and regularly updating the status of individual hydraulic elements at fixed intervals with increased (monthly) updates prior to and during rainy season. These required updates will automatically bring about the formation of an inspection team which should take an integrated approach by involving all stakeholders and thus promote awareness.

From the modelling and simulations, it can also be concluded that changing the size of the pipe (P27=600mm) draining water from the underbridge would improve the drainage systems' rainfall events' handling capacity to more than 10yrs return period without flooding. The pipe to be changed lies on a 200m length stretch which may not be too costly as it lies outside the CBA on the downstream part of the catchment.

SUDS should be incorparted into the system in order to reduce and delay the peak flow downstream the catchment and subsequently in conduit P36 which contributes towards flooding by causing back water effects. Furthermore, any future developments and maintenance of existing infrastructure should take into consideration the SUDS facilities. Source control in this case will be cardinal as there is little space for a variety blue-green solutions except on private land as shown by the land-use map in Figure 43. SUDS come in different kinds, forms, types and sizes, therefore, regardless of how built-up an area can be, they can still be implemented. Thus, studies should be conducted to ascertain specific SUDS adapted for the region owing to climatology and industrial activities.

9 RECOMMENDATIONS

- In order to verify model results, further studies on the drainage system should be done. These studies should address missing data for the Ndola CBA drainage system as well as rainfall data for the catchment. More data should be collected and the IDF curves should be generated for longer periods of up to 50yrs if possible. And this should be done for every weather station nationwide for future use. Some sort of measurements should be done on the study area in order to properly calibrate the model and validate it.

- The Ndola City Council should consider developing strategies of implementing and reinforcing the policy on garbage which should be fully supported at all levels. If the City Council should effectively implement the policy; garbage bins should be made availed to the public in strategic places in order to culture the mind-set in people throwing garbage into designated places.

- Some parts of the drainage system should be covered to avoid garbage and silt getting into the system which ends up getting blocked. The grass on the earthen embankments should not be cut to grassroots in order to prevent erosion and subsequent silt build-up. The grass trimmings should be removed from the slopes of the open drainages. Then the city council should consider updating the database for the drainage system network for easier and accurate maintenance and modelling in future.

- It would be good also to introduce a warning system whenever the area floods so that motorists can use alternative routes unlike having to reach the scene and cause congestion in trying to make a U-turn

10 REFERENCES

1. Times of Zambia. 2014: "Sort Out Ndola Underbridge Fast"- Available at http://www.times.co.zm/?p=15156 [Accessed 11 September 2015]
2. Times of Zambia. 2014: "Poor Underbridge Works Anger Ndola Residents"- Available at http://www.times.co.zm/?p=14112 [Accessed 11 September 2015]
3. Ramboll Group. 2016. Cloudburst Concretization Masterplan – Available at http://www.ramboll.com/projects/germany/copenhagen-cloudburst. [Accessed 17 December 2015].
4. Central Statistical Office, Zambia. 2011: "2010 Census of Population and Housing"- Available at http://unstats.un.org/unsd/demographic/sources/census/2010_phc/Zambia/PreliminaryReport.pdf [Accessed 12 September 2015]
5. Storm water Modelling 1976, page 13. Academic press, Network Donald E. Overton & Michael E. Meadows
6. Storm Water Management Model User's Manual 2009, EPA, Lewis A. Rossman
7. Water and Wastewater Technology. Pearson New International edition of 7th revised Edition. Pearson Education Limited.
8. ZARI. 2015: "Status, priorities and needs for sustainable soil management in Zambia "-Stalin Sichinga Available at http://www.fao.org/fileadmin/user_upload/GSP/docs/South_east_partnership/GSP_Zambia.pdf [Accessed 12 march 2016]
9. Effect of rain gauge density over the accuracy of rainfall: a case study over Bangalore, India | SpringerPlus | 2016. Available at

http://springerplus.springeropen.com/articles/10.1186/2193-1801-2-311. [Accessed 17 April 2016].

10. IHP-V, Technical Documents in Hydrology, No. 50. UNESCO, Paris, 2001. “guidelines on non-structural measures in urban flood management”-Ivan Andjelkovic Available at http://unesdoc.unesco.org/images/0012/001240/124004e.pdf [Accessed 17 April 2016].
11. World Bank-Cities and Flooding. “A Guide to Integrated Urban Flood Risk Management for the 21st Century”-Abhas K Jha, Robin Bloch & Jessica Lamond
12. VASYD 2008: “Blue-Green fingerprints in the city of Malmö, Sweden”-Peter Stahre
13. Rainfall in Ndola, Zambia Average Precipitation and Wet Days. 2016. Available at: http://www.ndola.climatemps.com/precipitation.php. [Accessed 10 July 2015].
14. Hydrology-principles, analysis & design. H. M. Raghunath- New Age International (P) Limited 2006 New Delhi.
15. USDA: Urban Hydrology for small watersheds-TR-55, Second Ed., June 1986.
16. Climate change, urban flooding and the rights of the urban poor in Africa- October 2006- Action Aid International, Gideon Mendel.
17. SOUTHERN AFRICA HUMANITARIAN OUTLOOK 2015/2016 Special Focus on El Niño-UN-OCHA
18. World Meteorological Organization. 2016. Floods in Southern Africa. Available at: http://public.wmo.int/en/media/news-from-members/floods-southern-africa#sthash.NF4slKPV.dpuf. [Accessed 16 April 2016].

19. Climate Graph for Ndola, Zambia. 2016. Available at: http://www.ndola.climatemps.com/graph.php. [Accessed 10 May 2016].
20. Encyclopaedia - Britannica Online Encyclopaedia. 2016. Available at http://academic.eb.com/EBchecked/topic/655568/Zambia. [Accessed 16 May 2016].
21. SWMM Knowledge Base. 2016. Available at: https://www.openswmm.org/Forum. [Accessed 30 May 2016].
22. Wikipedia. 2016. Districts of Zambia. Available at:https://en.wikipedia.org/wiki/Districts_of_Zambia#/media/File:Copperbelt_districts.png. [Accessed 07 June 2016].
23. Ndola Photographs - Great North Road. 2016. Available at: http://www.greatnorthroad.org/boma/Ndola_Photographs. [Accessed 07 June 2016].

11 APPENDICES

11.1 Appendix 1: Model Structural properties

The drainage system structure in form of input parameters to the model is shown below from Figures 43 to Figure 49

Project Data

Data Category
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[JUNCTIONS]
[OUTFALLS]
[CONDUITS]
[XSECTIONS]
[TIMESERIES]
[REPORT]

Name	Elevation	MaxDepth	InitDepth	SurDepth	Aponded
J25	1262.45	2.5	0	0	0
J26	1260.75	2.55	0	0	0
J27	1256.7	2.3	0	0	0
J28	1256.7	2.1	0	0	0
J29	1239	1	0	0	0
J30	1254	0.8	0	0	0
J31	1251.94	2.86	0	0	0
J32	1253.85	1.65	0	0	0
J34	1247.2	2.5	0	0	0
J35	1252.2	1.5	0	0	0
J36	1256	1.5	0	0	0
J37	1260.5	1.5	0	0	0
J38	1264	1.5	0	0	0
J39	1273.5	1.5	0	0	0
J40	1247	2.5	0	0	0
J41	1246.7	2.5	0	0	0
J42	1250	1.2	0	0	0
J43	1250.4	0.8	0	0	0
J44	1248.44	2	0	0	0
J45	1261.14	2.86	0	0	0
J46	1258.2	0.8	0	0	0
J47	1255.2	0.8	0	0	0
J48	1238.6	1.75	0	0	0
J49	1238.45	1.75	0	0	0
J50	1252.14	2.86	0	0	0
J51	1238.8	1	0	0	0
J55	1255	1	0	0	0

Figure 44: Model Junction parameters

Project Data

Data Category

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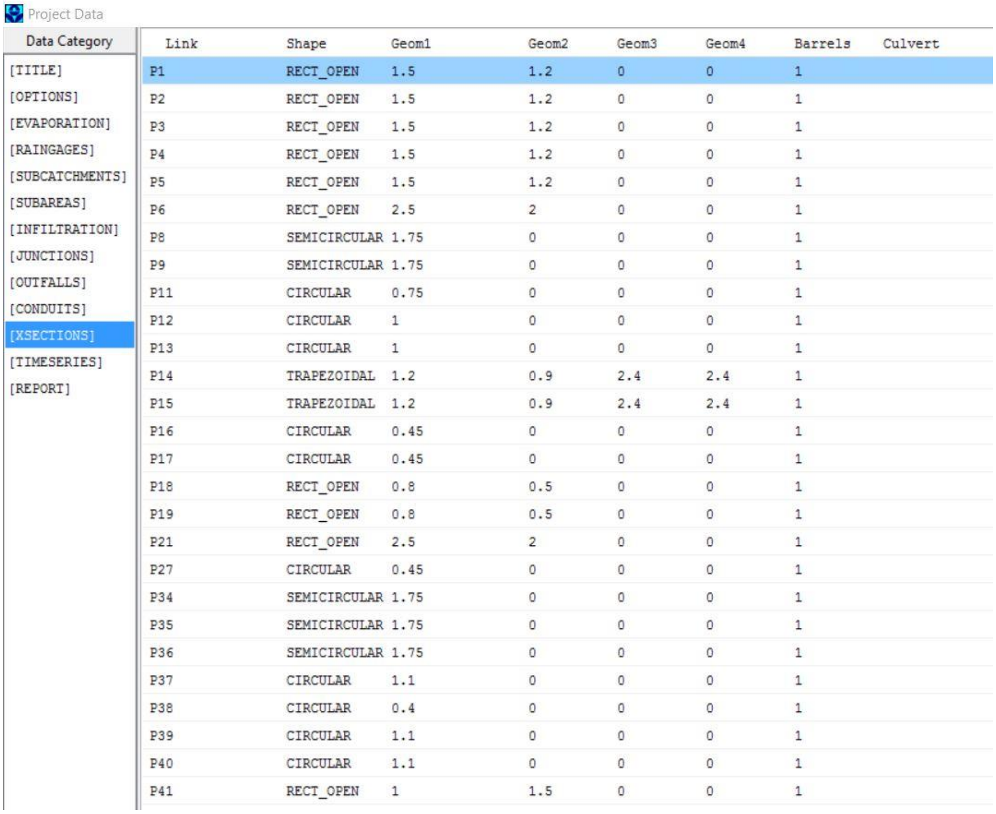
[REPORT]

Name	From Node	To Node	Length	Roughness	InOffset	OutOffset	InitFlow	MaxFlow
P1	J39	J38	309.60	0.01	0	0	0	0
P2	J38	J37	164.74	0.01	0	0	0	0
P3	J37	J36	172.71	0.01	0	0	0	0
P4	J36	J35	160.40	0.01	0	0	0	0
P5	J35	J34	161.01	0.01	0	0	0	0
P6	J34	J41	220.21	0.01	0	0	0	0
P8	J48	J49	41.12	0.01	0	0	0	0
P9	J49	Out1	153.19	0.01	0	0	0	0
P11	J25	J26	150.63	0.01	0	0	0	0
P12	J26	J27	142.61	0.01	0	0	0	0
P13	J27	J28	156.93	0.01	0	0	0	0
P14	J28	J42	156.26	0.01	0	0	0	0
P15	J42	J41	167.74	0.01	0	0	0	0
P16	J46	J47	125.59	0.01	0	0	0	0
P17	J47	J30	41.64	0.01	0	0	0	0
P18	J30	J43	161.01	0.01	0	0	0	0
P19	J43	J40	111.34	0.01	0	0	0	0
P21	J44	J40	244.29	0.017	0	0	0	0
P27	J29	J51	199.28	0.01	0	0	0	0
P34	J51	J48	85.44	0.01	0	0	0	0
P35	J40	J49	414.79	0.01	0	0	0	0
P36	J41	J48	292.36	0.01	0	0	0	0
P37	J45	J50	223.01	0.01	0	0	0	0
P38	J32	J50	24.03	0.01	0	0	0	0
P39	J50	J31	28.18	0.01	0	0	0	0
P40	J31	J44	135.75	0.01	0	0	0	0
P41	J55	Out2	702.02	0.01	0	0	0	0

Figure 45: Model conduit parameters



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Link	Shape	Geom1	Geom2	Geom3	Geom4	Barrels	Culvert
P1	RECT_OPEN	1.5	1.2	0	0	1	
P2	RECT_OPEN	1.5	1.2	0	0	1	
P3	RECT_OPEN	1.5	1.2	0	0	1	
P4	RECT_OPEN	1.5	1.2	0	0	1	
P5	RECT_OPEN	1.5	1.2	0	0	1	
P6	RECT_OPEN	2.5	2	0	0	1	
P8	SEMICIRCULAR	1.75	0	0	0	1	
P9	SEMICIRCULAR	1.75	0	0	0	1	
P11	CIRCULAR	0.75	0	0	0	1	
P12	CIRCULAR	1	0	0	0	1	
P13	CIRCULAR	1	0	0	0	1	
P14	TRAPEZOIDAL	1.2	0.9	2.4	2.4	1	
P15	TRAPEZOIDAL	1.2	0.9	2.4	2.4	1	
P16	CIRCULAR	0.45	0	0	0	1	
P17	CIRCULAR	0.45	0	0	0	1	
P18	RECT_OPEN	0.8	0.5	0	0	1	
P19	RECT_OPEN	0.8	0.5	0	0	1	
P21	RECT_OPEN	2.5	2	0	0	1	
P27	CIRCULAR	0.45	0	0	0	1	
P34	SEMICIRCULAR	1.75	0	0	0	1	
P35	SEMICIRCULAR	1.75	0	0	0	1	
P36	SEMICIRCULAR	1.75	0	0	0	1	
P37	CIRCULAR	1.1	0	0	0	1	
P38	CIRCULAR	0.4	0	0	0	1	
P39	CIRCULAR	1.1	0	0	0	1	
P40	CIRCULAR	1.1	0	0	0	1	
P41	RECT_OPEN	1	1.5	0	0	1	

Figure 46: Model conduit cross-section parameters

Project Data

Data Category

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[REPORT]

Name	Rain Gage	Outlet	Area	%Imperv	Width	%Slope	CurbLen	SnowPack
;Single-family area								
C1	Gage1	J39	31.76	40	370	3.1	0	
;Single-family area								
C2	Gage1	J38	10.97	50	190	3.96	0	
;Apartment dwelling area								
C3	Gage1	J26	5.08	50	165	3.87	0	
;Apartment dwelling area/ Business area								
C4	Gage1	J37	3.26	80	105	3.02	0	
;Apartment dwelling area/ Business area								
C5	Gage1	J36	3.28	80	105	2.5	0	
;Business area								
C6	Gage1	J35	3.88	90	200	2.5	0	
;Business area								
C7	Gage1	J34	6.14	90	230	2.96	0	
;Business area								
C8	Gage1	J41	6.16	80	230	2.59	0	
;Business area								
C9	Gage1	j42	1.23	95	90	2.67	0	
;Business area								
C10	Gage1	j28	3.01	90	120	2.8	0	
;Apartment dwelling area/ Business area								
C11	Gage1	J27	2.84	70	105	2.22	0	
;Business area								
C12	Gage1	J26	0.46	80	30	1.88	0	
;Business area								
C13	Gage1	J43	2.65	90	120	2.17	0	
;Business area								
C14	Gage1	j30	0.95	95	50	1	0	
;Business area								
C15	Gage1	J45	1.47	85	75	0.5	0	
;Business area								
C16	Gage1	J46	1.80	85	90	1.95	0	

Figure 47: Model sub-catchment parameters

Project Data

Data Category

- [TITLE]
- [OPTIONS]
- [EVAPORATION]
- [RAINGAGES]
- [SUBCATCHMENTS]
- [SUBAREAS]
- [INFILTRATION]
- [JUNCTIONS]
- [OUTFALLS]
- [CONDUITS]
- [XSECTIONS]
- [TIMESERIES]
- [REPORT]

Name	Rain Gage	Outlet	Area	%Imperv	Width	%Slope	CurbLen	SnowPack
;Business area								
C17	Gage1	j47	2.16	90	170	6.15	0	
;Business area								
C18	Gage1	J55	2.36	70	70	2.65	0	
;Business area								
C19	Gage1	J32	2.41	80	100	3.2	0	
;Business area								
C20	Gage1	J44	1.88	85	85	1.74	0	
;Asphalt Road								
C21	Gage1	J25	0.73	95	20	2.86	0	
;Asphalt Road								
C22	Gage1	J55	0.98	95	25	1.74	0	
;Asphalt Road								
C23	Gage1	J29	0.67	95	24	4.66	0	
;Asphalt Road								
C24	Gage1	J29	0.82	95	30	0.17	0	
;Asphalt Road								
C30	Gage1	j28	0.40	90	20	1.2	0	
;Asphalt Road								
C31	Gage1	j30	0.29	90	15	0.1	0	
;Asphalt Road								
C32	Gage1	j26	0.20	90	20	1	0	
;Asphalt Road								
C33	Gage1	j27	0.20	90	15	3.6	0	
;Asphalt Road								
C34	Gage1	j28	0.30	90	20	2.7	0	
;Asphalt Road								
C35	Gage1	j26	0.50	95	20	0.2	0	
;Asphalt Road								
C36	Gage1	j38	0.46	95	19	2.8	0	
C37	Gage1	j37	0.43	95	18	2	0	
;Asphalt Road								
C38	Gage1	j27	0.57	95	23	1	0	
;Asphalt Road								
C39	Gage1	j36	0.63	90	26	0.8	0	
;Asphalt Road								
C40	Gage1	j31	0.29	90	15	0.1	0	

Figure 48: Model sub-catchment parameters

Project Data

Data Category

- [TITLE]
- [OPTIONS]
- [EVAPORATION]
- [RAINGAGES]
- [SUBCATCHMENTS]
- [SUBAREAS]
- [INFILTRATION]
- [JUNCTIONS]
- [OUTFALLS]
- [CONDUITS]
- [XSECTIONS]
- [TIMESERIES]
- [REPORT]

Subcatchment	CurveNum		DryTime
C1	83	0.5	4
C2	83	0.5	4
C3	90	0.5	4
C4	92	0.5	4
C5	92	0.5	4
C6	94	0.5	4
C7	94	0.5	4
C8	94	0.5	4
C9	94	0.5	4
C10	94	0.5	4
C11	92	0.5	4
C12	94	0.5	4
C13	94	0.5	4
C14	94	0.5	4
C15	94	0.5	4
C16	94	0.5	4
C17	94	0.5	4
C18	94	0.5	4
C19	94	0.5	4
C20	94	0.5	4
C21	98	0.5	4
C22	98	0.5	4
C23	98	0.5	4
C24	98	0.5	4
C30	92	0.5	7
C31	92	0.5	7
C32	98	0.5	7
C33	98	0.5	7
C34	98	0.5	7
C35	98	0.5	7
C36	98	0.5	7
C37	98	0.5	7
C38	92	0.5	7
C39	92	0.5	7
C40	92	0.5	7

Figure 49: Model sub-catchment infiltration parameters

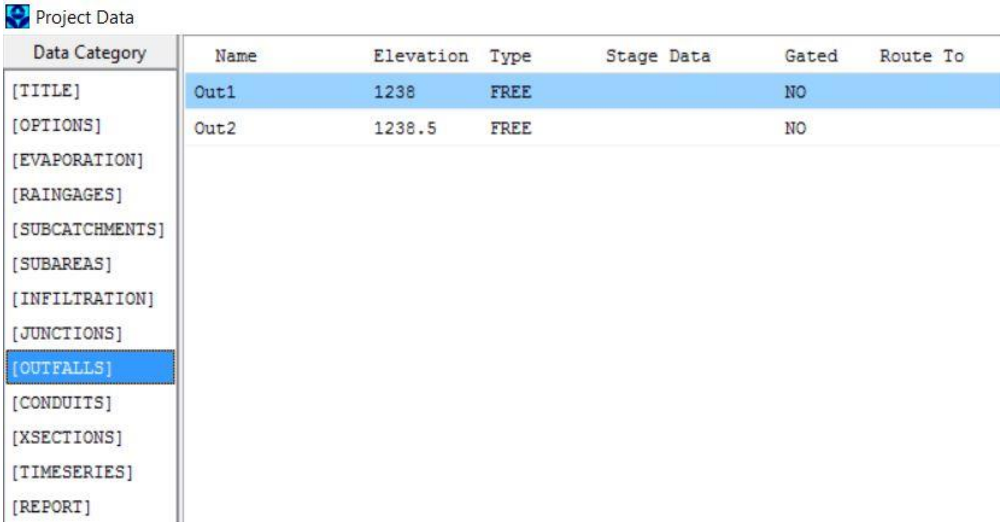

Figure 50: Model outfalls parameters