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DESIGN OF IMPROVED STORMWATER MANAGEMENT SYSTEM FOR THE FEDERAL UNIVERSITY OF TECHNOLOGY AKURE

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ABSTRACT
Stormwater management is becoming a problem in many cities due to rapid urbanization and poor infrastructure. Proper drainage system conveys stormwater from the road to a suitable disposal area. FUTA is well planned with good networks of roads. However, many of them are without functioning drains. The paper presents a pragmatic approach to the design of an improved stormwater management in FUTA. The entire catchment area was divided into subcatchments that form the designated units for data collection. The result of the field work showed that over half (57.31%) of the drainage area are grass land and the paved road covered 10.96%. the unpaved road and built up area covered 10.29% and 9.76% respectively. Others areas are thick forest (7.66%) and rock area (4.02%). The design of stormwater collection systems were based on the principles of hydraulics as explained by Manning. The drain size was based on the maximum discharge from each subcatchment and the time of concentration computed with Kirpich's formula. If this design is implemented, common flash flood and siltation along roads in FUTA will be completely eliminated.

Keywords: *Stormwater, Subcatchments, Pervious, Impervious, Drains*

1. INTRODUCTION

Poor stormwater management poses a serious challenge in urban and suburban areas worldwide. This problem is highly compounded in developing countries because of the poor stormwater drainage infrastructure and lack of maintenance culture. Throughout humanity's history, rain was considered a blessing that replenished springs, watered agricultural fields, fed streams and rivers, supported game and fish, and made travel possible [1]. But with the advent of urbanization, our relationship with rain changed from that of friend to foe, flooding kept causing economic loss, streams continued eroding, aquatic habitats kept dwindling. Pollution of surface water with toxic chemicals and excessive nutrients, resulting from a combination of stormwater runoff, point and non-point leaching and groundwater discharges has become an issue of environmental concern worldwide [2]. Stormwater pollution has been attributed to the process of industrialization and urbanization that has progressively developed over time without any regard for environmental consequences [3]. This development has eventually resulted in the deterioration of physical, chemical and biological properties of surface water [4]. Also, the high runoff rates which can occur, if unchecked, cause erosion problems in receiving streams and re-entrain polluted sediment from the riverbed. It is now recognized that surface water system can be a major cause of river pollution.

According to [5] stormwater is defined as the water from rain, snowmelt or melting ice that flows across the land surface. Stormwater runoff refers to all water that flows off developed or urbanized areas, including construction sites, and into the municipal sewerage system. This runoff may result from rainfall events, landscape irrigation, or other human activities such as car washes. Studies have shown that urban runoff is a significant source of nonpoint source pollution. In the United State of America, the US Environmental Protection Agency (USEPA) estimated that about 30 percent of known pollution to the country's waters is attributable to stormwater runoff [6]. This pollution has negative impacted on

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rivers and streams by decreasing and limiting their use or value. Large peak flows (floods) usually impact man and his property [7]. Thus, the management of a stormwater through effective and efficient conveyance system in an urban area is necessary to minimize flood damages or traffic hazards, lessen the aesthetic, physical, chemical and biological impacts to existing receiving water. In order to maximize the potential of streams and rivers for recreational and aesthetic benefits, it is necessary that the stormwater flowing into such rivers and streams must be properly channelled from road pavements and parking lots [5].

Since water is a vital and finite resource necessary for maintaining good health and sanitation, food security and ecological system, it is necessary to manage our urban stormwater so that it does not pose danger to the sustainability of water source [8]. In meeting the objective of maintaining a healthy and sound environment that promotes academic excellence, this study aims at design an improved stormwater management system in the developed area of the Federal University of Technology, Akure, Nigeria (FUTA). Since FUTA is still developing, the same approach can be applied to the drainage system of the undeveloped area based on the slope/contour analysis of the University master plan in future and a good connection made with the existing ones

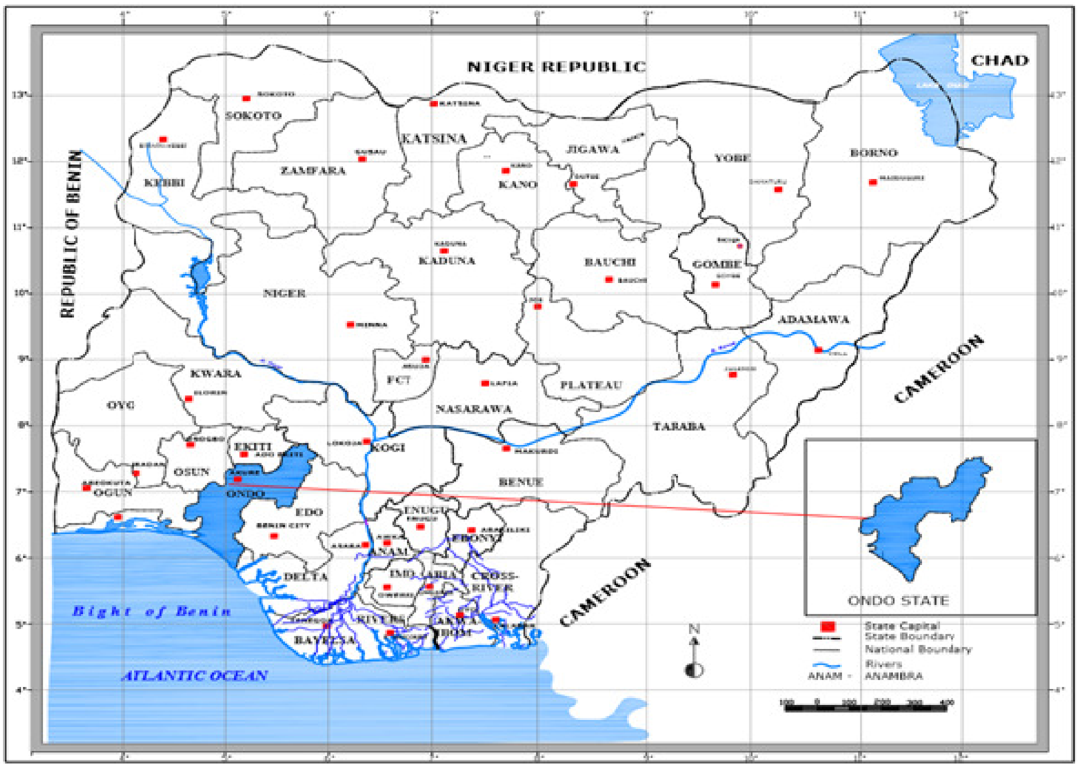

Figure 1: Map of Nigeria Showing Ondo State

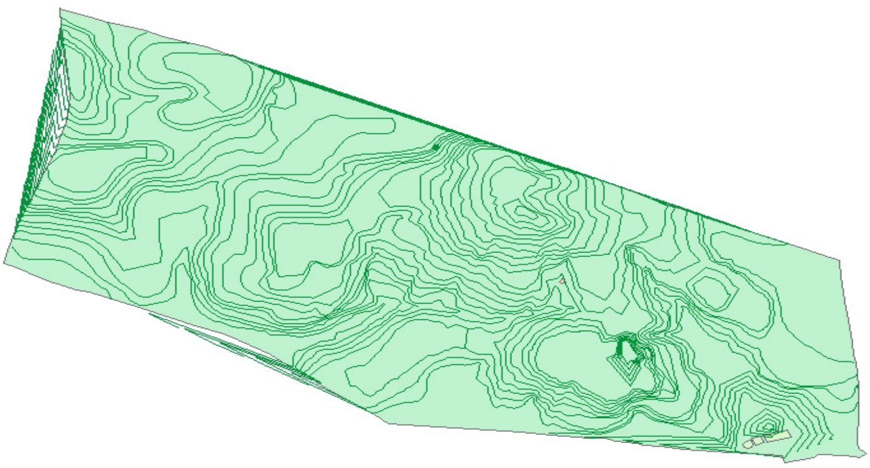

Figure 2: Digitized Map Showing the Contour Analysis

. 2. RESEARCH METHODOLOGY

2.1 Description of the Study Area

Akure lies on latitude 7°15′North of the Equator and on longitude 5°15′ East (Figure 1). Rain falls throughout the whole year but the onset (substantial rainfall amount) is during the month of March and sharp decrease in amount is during the month of November. The mean annual temperature is 24°C - 27°C, whiles the annual rainfall varies between 1500mm and 3500mm. The mean relative humidity is over 75%.

2.2 Reconnaissance Survey and Digitalization of Map

Topographic map (Figure 2) of the university was carefully studied. Reconnaissance survey of the area was carried out to know the actual location of some physical features. The slope analysis was generated using computer software (Arc GIS) to indicate the ground elevation of the area. Other features were digitized to provide better understanding of the stormwater flow direction. It also helps in determining the slope which is a vital parameter in the drainage design.

In this study, a map prepared for The Federal University of Technology, Akure (FUTA) by the Regional Centre for Training in Aerospace Survey (RECTAS) in 2006 was acquired from Centre for Space Research and Applications (CESRA) and carefully studied. The information contained in the map was acquired from IKONOS satellite image (2000) with extensive field work and field verification.

Rainfall data was collected from Nigerian Meteorological Agency (NIMET) to determine the maximum rainfall and its frequencies. The rainfall data was used to estimate the rainfall intensity in addition to the rainfall intensity duration frequency relationship to determine the maximum discharge using Rational formula as explained in section 2.6.

2.3 Drainage Area and Drains

The entire study area was divided into subcatchment based on flow characteristics and as shown in Figure 3 for easy analysis and data collection based on flow characteristics and topographic map. The land use of the catchment such as building, roads, grasses, forest etc was obtained through field survey and measurements. The reconnaissance survey was also carried out to ascertain the adequacy and effectiveness of the existing drains.

Figure 3: Layout of Stormwater Subcatchment Areas and the Proposed Stormwater Collection Networks

Reconnaissance survey revealed that many roads do not have functioning drains and many drains are completely silted, thereby reducing the channel capacity. Subsequently, new drains were proposed to follow the network of roads within the study area as shown in Figure 3

Using an approach developed by [9] to compute the drainage density (average length of streams within the basin per unit area), FUTA is well drained with drainage density of 0.86 as described below:

Length of main drainage stream (L) = 2 km

Length of tributary streams (L) = 1.5 km

Area (A) = 4.05 Km^2

Drainage density, $D_d = \frac{\sum L}{A} = \frac{3.5}{4.05} = 0.86$

However, many roads within the campus lack good and functioning drains.

2.4 Runoff Coefficient

Estimating the runoff coefficients for stormwater flow in a given area depends on the level of imperviousness (built and un-built areas), paved and unpaved roads, forest and grasses. These values are obtainable from extensive field measurement using odometer and interpretation using standard published manuals [9]. The values used in this study are expressed in Table 1

Table 1: Runoff Coefficient Values Used

Drainage Area	Runoff Coefficient (c)
Rock area	0.9
Paved area	0.8
Built-up area	0.7
Unpaved area	0.6
Grasses	0.3
Thick forest	0.2

Source: [10]

2.5 Peak Discharges

The study area was divided into thirty-six (36) subcatchment areas based on the topographical analysis carried out using topographical map on a scale of 1: 5,000 (Figure 3). The delineation of each subcatchment area was done using the topographical map to determine the flow direction based on the elevation or contour heights. To ensure that the subcatchment areas designated via the topographical map correspond with the flow direction, field check through physical survey was carried out on several occasions during and after rain storms to ascertain the direction of flow.

The area of each of the thirty-six subcatchments defined was computed using the area tool in AutoCAD environment. The level of imperviousness of the subcatchments derived from the proportion of built up areas, vacant plots (grasses), paved and unpaved roads, rocky and thick forest areas were obtained from the land use maps and physical measurements. Because the land use within the subareas of the catchment are different, their runoff coefficients are also different; and in order to get the composite runoff for the sub area, the runoff coefficients are weighted in proportion to the area of the sub catchments and the used to estimate peak discharge.

The slope of each subcatchment was obtained as the ratio of the difference in elevation derived from topographic map and the length of each subcatchment. In determining the catchment rainfall-runoff response, time of concentration in each subcatchment was determined using Kirpich's formula, [11]

$$t_c = \frac{0.00032L^{0.77}}{S^{0.385}} \quad (1)$$

In (1), T_c is time of concentration (hours), L is Maximum length of water travel (m), S is surface slope, given by H/L (m/m), H is difference in elevation between the remotest point in the drainage basin and the outlet (m). The velocity of flow in m/s was computed using equation 2.

$$V = \frac{L}{t_c \times 60} \quad (2)$$

2.6 Rainfall Intensity

The monthly rainfall data collected for the year 1980 to 2006 (26 years) obtained from Akure Airport gauging station was examined for maximum rainfall. However, the rainfall intensity needed to determine the peak flow could not be calculated from these data. As an alternative, the rainfall intensity duration curve prepared for Akure by [12] and shown in Figure 4 was available to select appropriate rainfall intensity values. Equation 3a was used in computing the peak flow in all the subcatchment of the study area.

$$Q_p = CiA \quad (3)$$

$$Q = 0.278CiA \quad (equation\ 3\ in\ S.I\ Unit) \quad (3a)$$

In (3), i is the rainfall intensity in mm/hr which was estimated using the weighted function on the Ondo, Ibadan and Benin data obtained from the technical report on the climate of Akure area by [12] as illustrated in Figure 4, A is the catchment area in km^2, C is a dimensionless runoff coefficient, whose value depends on catchment characteristics and Q_p is the peak discharge (m^3/s) due to the particular rainstorm and assumed to occur after time, Tc

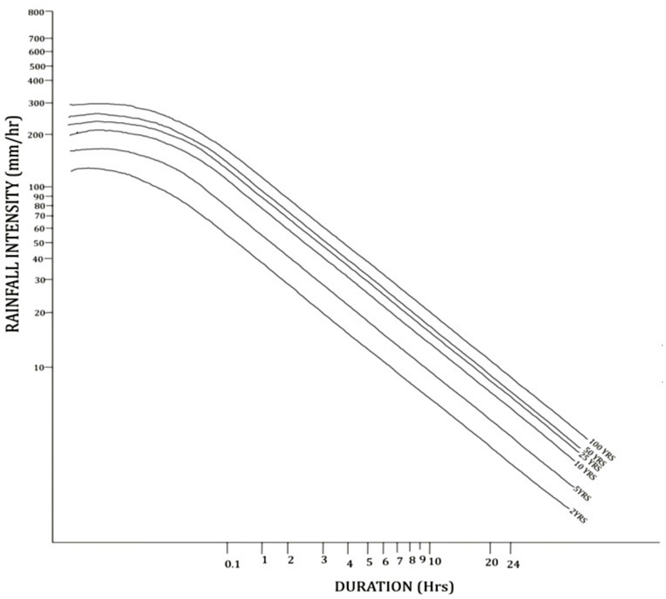

Figure 4: Rainfall Intensity Duration Curve Developed for Akure Township [11]

Taking subcatchment a_1 as an example. From Figure 3, area of a_1 was obtained to be 16,262 m^2 using the area tool in AutoCAD. Horizontal length along the flow direction passing through the measured area was 190.12 m. The difference in elevation or contour height of the highest and lowest point was found to be 10. Using this information, the following computations were made:

$$Slope = \frac{difference\ in\ contour\ height}{Length} = \frac{10}{628.78} = 0.01603 \quad (4)$$

Applying equation 1, time of concentration is computed as $t_c = \frac{0.0078 \times 623.78^{0.77}}{0.0526^{0.385}} = 5.44$ minutes

2.7 Sizing of the Drains

Stormwater flowing through the drains is to be discharged at various locations into the streams. Size of drains and the point of discharge into the receiving streams were selected and their peak discharges were computed by simple addition of discharges from the subcatchments flowing into the streams as indicated in Figure 5. The sizes of the drains were computed using Manning's equation to determine the maximum flow depth based on an assumed valued of width to be 0.5 m, 0.75 m and 1.0 m. ideally channel depth are suppose to be greater than 300 mm (0.3 m). Deeper channel drains also needs to be covered. It has been proposed in [13] that where there is space constraint, assumed width of 0.5 m can be used while where there is high runoff based on gravitational flow, assumed width of 1.0 m can be employed. The square drains fall within the range of the assumed width 0.5 m and assumed width of 1.0 m is more economical [13].

Computation example is as follows:

The depth of drain at point A assuming a width of 1.0m, Manning's n of 0.013, slope, S of 0.0160 and discharge Q = 1.62m^3/s.

$$Q = \frac{A}{n} R^{2/3} S^{1/2} \quad 5$$

where

$$R = \frac{A}{P} = \frac{bd}{b + 2d} \quad 5a$$

$$Q = \frac{bd}{n} \left(\frac{bd}{b + 2d}\right)^{2/3} S^{1/2} \quad 5b$$

$$1.62 = \frac{d}{n} \left(\frac{d}{1 + 2d}\right)^{2/3} (0.016)^{1/2} \quad 5c$$

$$d^{5/2} - 0.136d - 0.068 = 0 \quad 5d$$

$$\therefore d = 0.45\ m \quad 5e$$

The values of the depth d are computed by a trial and error method using assumed width 0.5m, 0.75m and 1.0m

2.8 Stormwater Quality

The stormwater quality was determined in the laboratory by measuring physical, chemical and bacteriological characteristics of a typical stormwater sample from a storm that preceded a two weeks break. This is to enable sufficient non-point source pollution within the study area to accumulate before determining the pollution strength of the stormwater. This is necessary to know if the self cleansing of the receiving stream is sufficient or simple treatment is required. The appropriate laboratory analysis was carried out using the standard method of water quality measurements at the Regional Water Laboratory, Federal Ministry of Water Resources in Akure, Ondo State.

3. RESULTS AND DISCUSSION

3.1 Pervious and Impervious Area

The result of the field work on the pervious and impervious area of the entire catchment (Figure 7) showed that over half (57.31%) of the entire drainage area are grass land and the paved road covered 10.96%. the unpaved road and built up area covered 10.29% and 9.76% respectively. Other areas are thick forest (7.66%) and rock area (4.02%).

Breaking down the information in Figure 6 into subcatchment and the application of runoff coefficient (Table 1) into subcatchment characteristics, Table 2 was obtained.

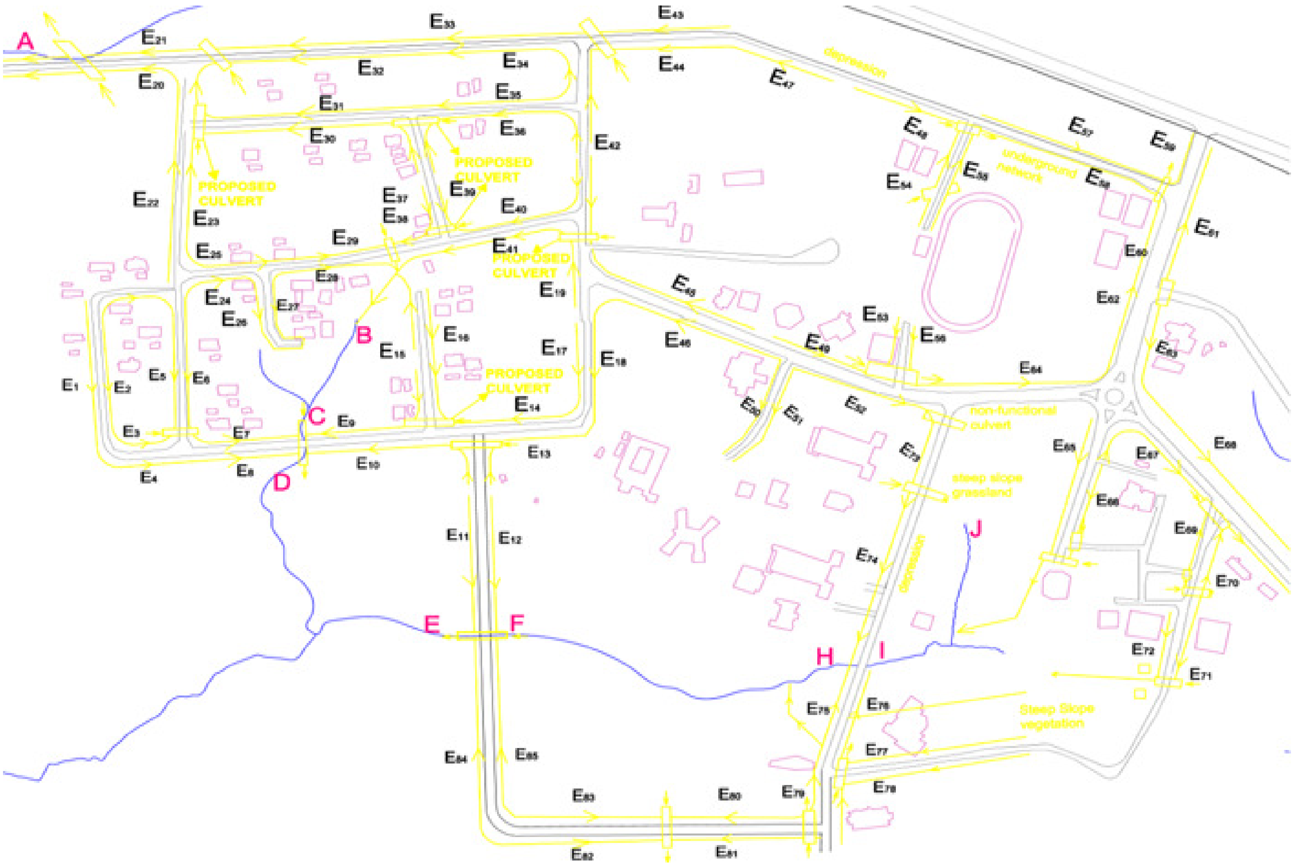

Figure 5: Layout of the Stormwater Collection Network showing the Selected Receiving Stream Discharge Points

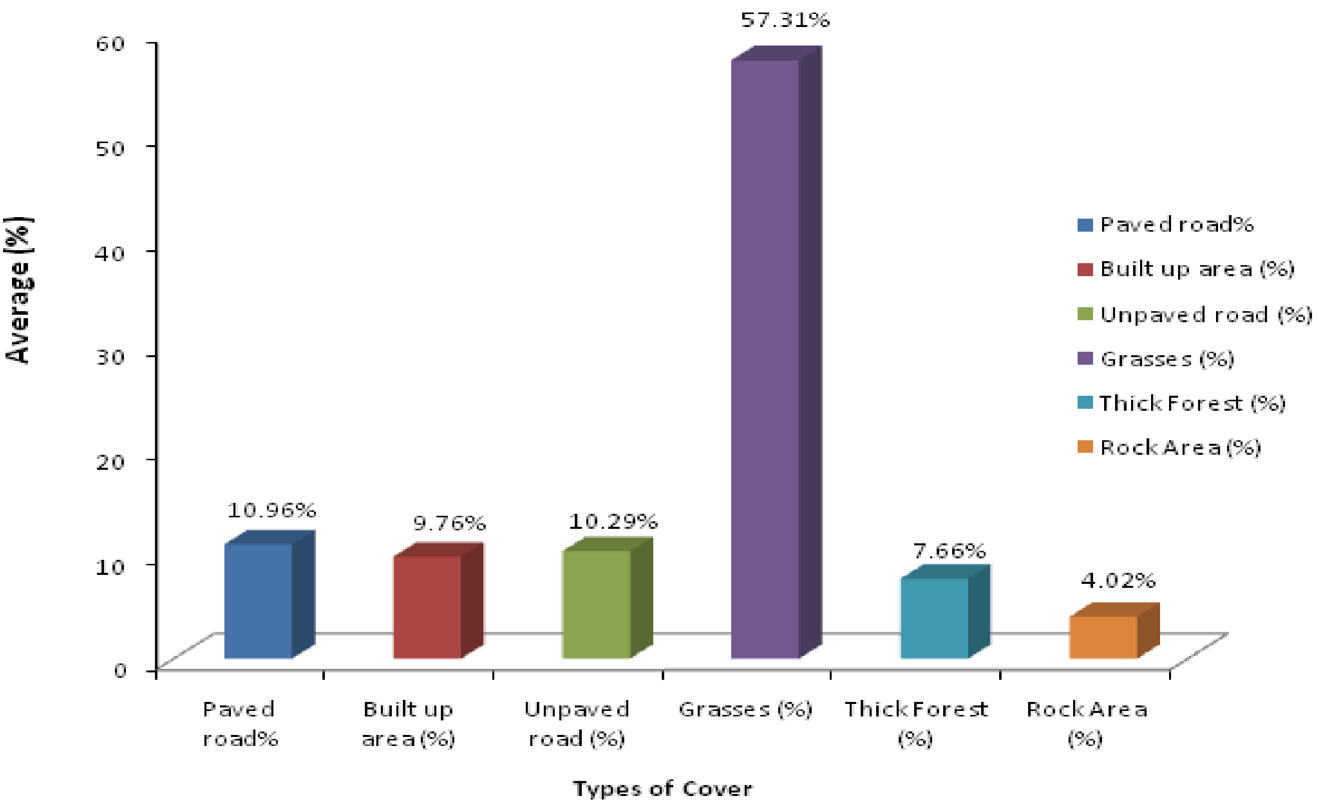

Figure 6: Pervious and impervious Area

Table 2: Derivation of Runoff Coefficient [14]

Catchment Designation	Paved road%	Built up area (%)	Unpaved road (%)	Grasses (%)	Thick Forest (%)	Rock Area (%)	Runoff Coefficient (C)
a_1	-	-	10	58	32	-	0.298
a_2	-	15	10	75	-	-	0.390
a_3	-	10	5	85	-	-	0.355
a_4	-	-	5	95	-	-	0.315
a_5	-	12	25	63	-	-	0.423
a_6	-	30	10	60	-	-	0.450
a_7	-	5	10	85	-	-	0.350
a_8	-	10	25	65	-	-	0.415
a_9	-	30	10	60	-	-	0.450
a_{10}	-	10	20	70	-	-	0.400
a_{11}	-	10	10	75	-	5	0.400
a_{12}	-	-	10	8	82	-	0.248
a_{13}	-	-	15	8	77	-	0.268
a_{14}	-	10	30	55	-	5	0.460
a_{15}	-	5	30	60	-	5	0.440
a_{16}	-	10	12	68	-	10	0.436
a_{17}	15	5	5	75	-	-	0.420
a_{18}	10	25	10	50	-	5	0.510
a_{19}	45	15	-	40	-	-	0.585
a_{20}	-	10	5	80	-	5	0.385
a_{21}	5	-	5	80	-	10	0.400
a_{22}	15	20	10	55	-	-	0.485
a_{23}	30	20	10	50	-	-	0.590
a_{24}	-	3	-	97	-	-	0.312
a_{25}	-	7	3	90	-	-	0.337
a_{26}	5	15	10	70	-	-	0.415
a_{27}	25	15	20	35	-	5	0.560
a_{28}	10	5	-	65	10	10	0.420
a_{29}	30	10	-	45	-	15	0.580
a_{30}	10	10	36	34	-	10	0.568
a_{31}	-	-	-	5	75	20	0.345
a_{32}	20	-	8	60	-	12	0.496
a_{33}	60	10	-	30	-	-	0.640
a_{34}	35	10	10	45	-	-	0.545
a_{35}	40	-	-	40	-	20	0.620
a_{36}	40	15	2	30	-	8	0.599
Total	395	352	371	2066	276	145	
Average (%)	10.96	9.76	10.29	57.31	7.66	4.02	

3.2 Peak Discharges

Using subcatchment characteristics, the application of equations 1, 2 and 3, resulted in Table 3. From Table 3, it can be seen that the time of concentration in any of the subcatchments was less than 0.12 hour (7 minutes). Hence, from Figure 5, a rainfall intensity of 200 mm/hr that corresponds to about 5 minutes duration (equal to the time of concentration) for a frequency of 25 years was selected to compute peak discharge. This peak discharge values are summarised in Table 3.

Applying equation 2, the velocity of flow can be computed as follows, taking subcatchment a_1 as an example

$$V = \frac{L}{t_c \times 60} = \frac{190.12m}{5.44\,min \times 60} = 0.58\,m/s$$

While applying equation 3a, peak discharge is calculated as follows

$$Q = 0.278 \times 0.298 \times 200\frac{mm}{hr} \times 0.016262\,km^2$$
$$= 0.27\,m^3/s$$

Table 3: Peak Discharge Values for all Subcatchments [13]

CD	Area (m^2)	Area (km^2)	RC	Elevation Difference	Elevation Height	Length (m)	Slope	T_c(mins)	Velocity (m/s)	Q (m^3/s)
a_1	16,262	0.016262	0.298	374 – 364	10	190.12	0.0160	5.44	0.58	0.27
a_2	8,097	0.008097	0.390	376 – 370	6	150.41	0.0122	5.05	0.50	0.18
a_3	3,604	0.003604	0.355	380 – 376	4	64.94	0.0188	2.24	0.48	0.07
a_4	3,104	0.003104	0.315	382 – 378	4	67.49	0.0181	2.34	0.48	0.05
a_5	10,518	0.010518	0.423	376 – 372	4	132.14	0.0092	5.08	0.43	0.25
a_6	10,393	0.010393	0.450	376 – 374	2	107.96	0.0056	5.26	0.34	0.26
a_7	10,527	0.010527	0.350	384 – 376	8	112.19	0.0217	3.22	0.58	0.20
a_8	7,123	0.007123	0.415	378 -374	6	108.11	0.0169	3.45	0.52	0.16
a_9	13,883	0.013883	0.450	376 – 366	10	136.13	0.0224	3.70	0.61	0.35
a_{10}	11,440	0.011440	0.400	372 – 366	6	129.77	0.0141	4.26	0.51	0.25
a_{11}	17,879	0.017879	0.400	382 – 372	10	174.61	0.0175	4.93	0.59	0.40
a_{12}	21,207	0.021207	0.248	368 – 358	10	153.15	0.0199	4.24	0.60	0.29
a_{13}	26,025	0.026025	0.268	368 – 358	10	193.83	0.0157	5.56	0.58	0.39
a_{14}	17,983	0.017983	0.460	392 – 384	8	175.14	0.0139	5.39	0.54	0.46
a_{15}	13,863	0.013863	0.440	396 – 392	4	165.70	0.0074	6.60	0.42	0.34
a_{16}	9,414	0.009414	0.436	394 – 388	6	124.95	0.0146	4.08	0.51	0.23
a_{17}	13,789	0.013789	0.420	388 – 378	10	163.42	0.0187	4.57	0.60	0.32
a_{18}	6,823	0.006823	0.510	394 – 388	6	124.90	0.0146	4.07	0.51	0.19
a_{19}	6,580	0.006580	0.585	396 – 386	10	92.61	0.0329	2.37	0.65	0.21
a_{20}	18,010	0.018010	0.385	394 – 382	12	128.22	0.0285	3.22	0.66	0.39
a_{21}	7,534	0.007534	0.400	386 – 378	8	113.79	0.0214	3.27	0.58	0.17
a_{22}	14,470	0.014470	0.485	388 – 374	14	137.96	0.0309	3.30	0.70	0.39
a_{23}	15,077	0.015077	0.590	384 – 376	8	128.49	0.0190	3.77	0.57	0.49
a_{24}	12,925	0.012925	0.312	382 – 372	10	125.54	0.0243	3.37	0.62	0.22
a_{25}	17,026	0.017026	0.337	374 – 362	12	166.45	0.0220	4.35	0.64	0.32
a_{26}	12,183	0.012183	0.415	374 – 364	10	135.33	0.0225	3.67	0.61	0.28
a_{27}	6,792	0.006792	0.560	376 – 372	4	64.91	0.0188	2.24	0.48	0.21
a_{28}	9,549	0.009549	0.420	372 – 364	8	85.36	0.0286	2.35	0.61	0.22
a_{29}	4,842	0.004842	0.580	380 – 378	2	78.77	0.0077	3.65	0.36	0.16
a_{30}	2,338	0.002338	0.568	378 – 376	2	59.34	0.0103	2.63	0.38	0.07
a_{31}	8,577	0.008577	0.345	372 – 380	8	106.03	0.0230	3.02	0.59	0.16
a_{32}	10,728	0.010728	0.496	378 – 368	10	98.24	0.0310	2.54	0.65	0.30
a_{33}	11,656	0.011656	0.640	370 – 366	10	218.28	0.0140	6.38	0.57	0.41
a_{34}	16,225	0.016225	0.545	370 – 362	8	156.36	0.0156	4.73	0.55	0.49
a_{35}	3,830	0.003830	0.620	372 – 368	4	75.65	0.0161	2.67	0.47	0.13
a_{36}	4,862	0.004862	0.599	384 – 374	10	85.57	0.0356	2.16	0.66	0.16

Note: RC – Runoff Coefficient, T_c – Time of Concentration

3.4 Drain Sizes

Table 4 and 5 shows the results of the drain of various sizes obtained when equation 5 to 5e was applied to the subcatchment flow at the point of discharge into stream and along the road side respectively.

Table 4 shows that discharge point E and B receives the highest (4.77 m^3/s) and minimum (0.65 m^3/s) stormwater discharge. This implies that the drain sizes of these points are the biggest and smallest among all the drains at the stormwater exits. Table 5 also shows that the flow through all the drains are less than 1.0m^3/s except drain number 60 (i.e. E_{60}) which receives flow of 1.15 m^3/s.

3.5 Stormwater Quality

The results of the stormwater quality are summarised in Table 6. The stormwater can be classified as moderately polluted and need no further treatment. This is because the self cleansing of the stream is sufficient to maintain a clean stream quality with adequate dissolved oxygen.

Table 4: Drain Sizing of Stormwater Discharge Points in the Study Area for width,b = 1.0m [14]

Discharge Point	Discharge m^3/s	Slope	Depth D (m)	Width b (m)	Depth D (m)	Width b (m)	Depth D (m)	Width b (m)	Square drain D	Square drain b
A	1.62	0.0160	0.45	1.0	0.65	0.75	0.98	0.5	0.66	0.66
B	0.65	0.0141	0.30	1.0	0.35	0.75	0.50	0.5	0.48	0.48
C	1.81	0.0224	0.45	1.0	0.57	0.75	0.94	0.5	0.65	0.65
D	1.98	0.0200	0.48	1.0	0.65	0.75	1.07	0.5	0.69	0.69
E	4.77	0.0199	0.95	1.0	1.37	0.75	2.45	0.5	0.97	0.97
F	4.09	0.0156	0.91	1.0	1.30	0.75	1.85	0.5	0.95	0.95
G	3.00	0.0225	0.62	1.0	0.87	0.75	1.46	0.5	0.79	0.79
H	2.33	0.0286	0.48	1.0	0.65	0.75	1.05	0.5	0.68	0.68
I	1.49	0.0310	0.35	1.0	0.45	0.75	0.70	0.5	0.57	0.57
J	0.71	0.0243	0.25	1.0	0.3	0.75	0.45	0.5	0.45	0.45

Table 5: Road Side Drains Design in the Study Area [14]

Side Drains	Discharge, m^3/s	Slope	Depth D (m)	Width b (m)	Depth D (m)	Width b (m)	Existing Drains b	Existing Drains D	Square Drain D	Square Drain b
E_1	0.16	0.0160	0.20	1.0	0.20	0.75	No drain		0.28	0.28
E_2	0.16	0.0160	0.20	1.0	0.20	0.75	No drain		0.28	0.28
E_3	0.16	0.0160	0.20	1.0	0.20	0.75	No drain		0.28	0.28
E_4	0.16	0.0160	0.20	1.0	0.20	0.75	No drain		0.28	0.28
E_5	0.16	0.0160	0.20	1.0	0.20	0.75	No drain		0.28	0.28
E_6	0.35	0.0224	0.25	1.0	0.24	0.75	No drain		0.36	0.36
E_7	0.51	0.0224	0.27	1.0	0.25	0.75	No drain		0.41	0.41
E_8	0.51	0.0224	0.27	1.0	0.25	0.75	No drain		0.41	0.41
E_9	0.65	0.0141	0.30	1.0	0.35	0.75	No drain		0.49	0.49
E_{10}	0.65	0.0199	0.28	1.0	0.33	0.75	No drain		0.46	0.46
E_{11}	0.29	0.0199	0.20	1.0	0.20	0.75	0.52	0.50	0.34	0.34
E_{12}	0.32	0.0220	0.22	1.0	0.23	0.75	0.52	0.45	0.34	0.34
E_{13}	0.17	0.0220	0.15	1.0	0.18	0.75	No drain		0.27	0.27
E_{14}	0.40	0.0175	0.23	1.0	0.24	0.75	No drain		0.39	0.39
E_{15}	0.25	0.0141	0.15	1.0	0.20	0.75	No drain		0.34	0.34
E_{16}	0.40	0.0175	0.23	1.0	0.24	0.75	No drain		0.39	0.39
E_{17}	0.40	0.0175	0.23	1.0	0.24	0.75	No drain		0.39	0.39
E_{18}	0.17	0.0214	0.13	1.0	0.15	0.75	No drain		0.27	0.27
E_{19}	0.40	0.0175	0.23	1.0	0.24	0.75	No drain		0.39	0.39
E_{20}	0.27	0.0160	0.15	1.0	0.23	0.75	No drain		0.34	0.34
E_{21}	1.62	0.0160	0.45	1.0	0.65	0.75	No drain		0.67	0.67
E_{22}	0.27	0.0160	0.15	1.0	0.23	0.75	No drain		0.34	0.34
E_{23}	0.25	0.0092	0.20	1.0	0.25	0.75	No drain		0.37	0.37
E_{24}	0.35	0.0224	0.20	1.0	0.24	0.75	No drain		0.36	0.36
E_{25}	0.25	0.0092	0.20	1.0	0.25	0.75	No drain		0.37	0.37
E_{26}	0.35	0.0224	0.20	1.0	0.24	0.75	No drain		0.36	0.36
E_{27}	0.35	0.0224	0.20	1.0	0.24	0.75	No drain		0.36	0.36
E_{28}	0.25	0.0141	0.15	1.0	0.20	0.75	No drain		0.34	0.34
E_{29}	0.46	0.0056	0.30	1.0	0.35	0.75	No drain		0.51	0.51
E_{30}	0.25	0.0056	0.24	1.0	0.25	0.75	No drain		0.41	0.41
E_{31}	0.18	0.0122	0.15	1.0	0.20	0.75	No drain		0.31	0.31
E_{32}	0.30	0.0122	0.18	1.0	0.21	0.75	No drain		0.38	0.38
E_{33}	0.8	0.0188	0.30	1.0	0.42	0.75	No drain		0.50	0.50
E_{34}	0.12	0.0181	0.13	1.0	0.15	0.75	No drain		0.25	0.25
E_{35}	0.30	0.0181	0.15	1.0	0.24	0.75	No drain		0.35	0.35
E_{36}	0.20	0.0217	0.14	1.0	0.22	0.75	No drain		0.29	0.29
E_{37}	0.26	0.0056	0.25	1.0	0.25	0.75	No drain		0.41	0.41
E_{38}	0.46	0.0056	0.30	1.0	0.35	0.75	No drain		0.51	0.51
E_{39}	0.20	0.0217	0.14	1.0	0.22	0.75	No drain		0.29	0.29
E_{40}	0.20	0.0217	0.14	1.0	0.22	0.75	No drain		0.29	0.29
E_{41}	0.20	0.0217	0.14	1.0	0.22	0.75	No drain		0.29	0.29
E_{42}	0.40	0.0175	0.23	1.0	0.24	0.75	No drain		0.39	0.39
E_{43}	0.46	0.0139	0.24	1.0	0.26	0.75	No drain		0.43	0.43

Side Drains	Discharge, m^3/s	Slope	Depth D (m)	Width b (m)	Depth D (m)	Width b (m)	Existing Drains		Square Drain	
							b	D	D	b
E_{44}	0.46	0.0139	0.24	1.0	0.26	0.75	No drain		0.43	0.43
E_{45}	0.80	0.0139	0.30	1.0	0.40	0.75	0.54	0.58	0.53	0.53
E_{46}	0.19	0.0146	0.15	1.0	0.20	0.75	No drain		0.31	0.31
E_{47}	0.19	0.0309	0.15	1.0	0.18	0.75	No drain		0.27	0.27
E_{48}	0.34	0.0074	0.20	1.0	0.26	0.75	No drain		0.43	0.43
E_{49}	0.23	0.0146	0.18	1.0	0.20	0.75	No drain		0.33	0.33
E_{50}	0.21	0.0309	0.15	1.0	0.18	0.75	0.38	0.20	0.28	0.28
E_{51}	0.39	0.0309	0.22	1.0	0.24	0.75	0.44	0.55	0.35	0.35
E_{52}	0.39	0.0309	0.22	1.0	0.24	0.75	0.34	0.42	0.35	0.35
E_{53}	0.49	0.0190	0.25	1.0	0.25	0.75	0.35	0.39	0.42	0.42
E_{54}	0.21	0.0329	0.15	1.0	0.18	0.75	0.53	0.51	0.27	0.27
E_{55}	0.23	0.0146	0.18	1.0	0.20	0.75	0.56	0.60	0.33	0.33
E_{56}	0.32	0.0187	0.22	1.0	0.24	0.75	0.41	0.38	0.36	0.36
E_{57}	0.39	0.0285	0.22	1.0	0.24	0.75	No drain		0.35	0.35
E_{58}	0.32	0.0187	0.22	1.0	0.24	0.75	No drain		0.36	0.36
E_{59}	0.55	0.0187	0.24	1.0	0.26	0.75	No drain		0.44	0.44
E_{60}	1.15	0.0187	0.35	1.0	0.45	0.75	0.42	0.47	0.57	0.57
E_{61}	0.32	0.0190	0.22	1.0	0.24	0.75	No drain		0.35	0.35
E_{62}	0.60	0.0190	0.25	1.0	0.33	0.75	0.57	0.34	0.45	0.45
E_{63}	0.39	0.0285	0.22	1.0	0.24	0.75	No drain		0.35	0.35
E_{64}	0.32	0.0190	0.22	1.0	0.24	0.75	No drain		0.35	0.35
E_{65}	0.60	0.0285	0.25	1.0	0.25	0.75	No drain		0.42	0.42
E_{66}	0.22	0.0243	0.15	1.0	0.22	0.75	No drain		0.29	0.29
E_{67}	0.16	0.0077	0.18	1.0	0.20	0.75	No drain		0.32	0.32
E_{68}	0.23	0.0077	0.23	1.0	0.25	0.75	No drain		0.37	0.37
E_{69}	0.23	0.0077	0.23	1.0	0.25	0.75	0.33	0.49	0.37	0.37
E_{70}	0.07	0.0103	0.10	1.0	0.13	0.75	0.31	0.63	0.22	0.22
E_{71}	0.07	0.0103	0.10	1.0	0.13	0.75	No drain		0.22	0.22
E_{72}	0.16	0.0103	0.18	1.0	0.18	0.75	No drain		0.31	0.31
E_{73}	0.16	0.0103	0.18	1.0	0.18	0.75	No drain		0.31	0.31
E_{74}	0.49	0.0190	0.25	1.0	0.25	0.75	No drain		0.42	0.42
E_{75}	0.21	0.0188	0.18	1.0	0.18	0.75	0.51	0.44	0.30	0.30
E_{76}	0.41	0.0286	0.22	1.0	0.24	0.75	0.75	0.55	0.36	0.36
E_{77}	0.46	0.0310	0.23	1.0	0.25	0.75	0.62	0.65	0.37	0.37
E_{78}	0.30	0.0310	0.18	1.0	0.20	0.75	0.86	0.71	0.32	0.32
E_{79}	0.16	0.0356	0.13	1.0	0.15	0.75	0.49	0.40	0.24	0.24
E_{80}	0.13	0.0161	0.15	1.0	0.18	0.75	0.58	0.73	0.26	0.26
E_{81}	0.13	0.0161	0.15	1.0	0.18	0.75	0.55	0.51	0.26	0.26
E_{82}	0.13	0.0161	0.15	1.0	0.18	0.75	0.54	0.39	0.26	0.26
E_{83}	0.41	0.0140	0.25	1.0	0.25	0.75	No drain		0.41	0.41
E_{84}	0.41	0.0156	0.24	1.0	0.25	0.75	No drain		0.40	0.40
E_{85}	0.39	0.0157	0.23	1.0	0.25	0.75	No drain		0.40	0.40
E_{86}	0.49	0.0156	0.2	1.0	0.26	0.75	No drain		0.43	0.43

Table 6: Stormwater Quality and Receiving Stream Quality [14]

S/N	Parameter	Unit	Stormwater (roof)	Stormwater (parking lot)	Stream before rainfall	Stream after rainfall
1	Appearance/Colour	Clear	Clear	Turbid	Clear	Clear
2	Odour	UNO	UNO	Objectionable	UNO	Objectionable
3	Temperature	^oC	25.6	25.8	25.6	25.6
4	pH	pH	7.93	5.22	7.77	7.46
5	Turbidity	NTU	1.36	74.3	1.21	279
6	Conductivity	µs/cm	20.0	80.0	130	400
7	Total Dissolved Solid	mg/l	13.4	53.6	87.1	268
8	Total Hardness $CaCO_3$	mg/l	64.0	44.0	204	70.0
9	Ca Hardness $CaCO_3$	mg/l	12.0	18.0	136	26.0
10	Mg Hardness $CaCO_3$	mg/l	52.0	26.0	68.0	44.0

S/N	Parameter	Unit	Stormwater (roof)	Stormwater (parking lot)	Stream before rainfall	Stream after rainfall
11	Aluminium (Al)	mg/l	ND	ND	0.002	0.003
12	Nitrate (NO_3)	mg/l	1.87	1.23	2.56	3.23
13	Iron (Fe)	mg/l	0.01	0.01	0.02	0.02
14	Alkalinity	mg/l	12.0	52.0	208	38.0
15	Fluoride	mg/l	ND	ND	ND	ND
16	Manganese (Mn)	mg/l	0.01	0.02	0.02	0.02
17	Calcium (Ca^{2+})	mg/l	4.81	72.1	54.5	10.4
18	Magnesium (Mg^{2+})	mg/l	12.7	6.34	16.6	10.7
19	Chloride (Cl^-)	mg/l	9.99	13.0	11.0	15.9
20	Sodium (Na)	mg/l	6.49	8.45	7.15	10.4
21	Bicarbonate (HCO_3)	mg/l	12.0	52.0	208	38.0
22	Ammonia (NH_4)	mg/l	ND	ND	0.04	0.08
23	Total Bacterial Count	mg/l	9	10	18	25
24	E-coli	mg/l	0	0	2	3
25	BOD	mg/l	0.06	1.20	0.4	1.05
26	Dissolved Oxygen	mg/l	1.00	1.10	0.80	0.50

Note: ND = Not Detected, TNTC = Too Numerous to Count, UNO = Unobjectionable, NTU = Nephelometric Turbidity Unit

4. PRESENT STATUS OF THE EXISTING DRAINAGE

FUTA is well planned with good networks of roads and plot layout. The present state of the stormwater collection network is not quite adequate because most of the roads and their side drains are not well maintained. There is blockage of drainage channels by refuse and other wastes as a result of poor drain management. Proper and regular channelization of the existing streams in this study area would go a long way to increase the stormwater carrying capacity of the drains. This will reduce overflow of the drains during heavy downfall.

5. CONCLUSION

Proper stormwater management involve collection, conveyance and disposal in a safe and environmentally sustainable manner. This paper presents a holistic approach to the improved design of stormwater management drains in a semi urban area of developing country using Federal University of Technology, Akure as a case study. This is to prevent environmental degradation that may result from poor stormwater management and provide an aesthetically pleasing environment suitable for academic excellence.

6. RECOMMENDATIONS

As a result of the outcome of this study, the following recommendations are made:

1. Proper and regular channelization of the existing streams in this study area would go a long way to increase the stormwater intake thereby reducing water overspill during heavy rainfall.
2. Planting of grasses and trees at the riverbank to improve flow velocity and reduce soil erosion and sedimentation.

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