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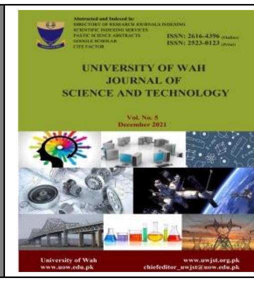

Sustainability in Storm Water Management and Conservation of Rainwater: A case study of an Educational Institute in Lahore, Pakistan.

Asif Naseer[1], Syed Taseer Abbas Jaffar[1], Hira Sattar[1], Abdullah[1], Fatima Ashfaq[1], Muhammad Bilal Zahid[1], Shakir Ahmad[1] and Abdul Waqar Akhtar[1]

***Abstract*— The urban area of Lahore is facing serious problem for the management of storm water due to heavy rainfalls especially in the summer (monsoon) season. The accumulation of storm water is due to the high level of water in the adjacent housing colonies during or after heavy rainfall events. The ongoing and newly proposed construction projects inside the city are supposed to take an unknown period of time for completion and have become an additional reason for water stagnation. Due to the improper or inadequate drainage facilities, the storm water gets stagnant and creates hurdles for the normal on-going activities all over the city. The existing conditions for the drainage facilities are unable to cope up with the aftermaths of heavy rainfalls. In addition, the present storm water drains are becoming insufficient due to rapid developments and need serious attention. This research takes into consideration the current situation of storm water drains in a university campus and proposes sustainable solutions for storm water management and conservation of rainwater. Using the rainfall intensity, suitable runoff coefficient was estimated and by computing, the area based on survey the rational equation was then used to estimate the runoff and design discharges for different return period rainfall events. From point of view of draining facility for managing the storm water as consequences of rainfall events two types of systems were proposed i.e., closed conduit (underground pipe) system based on commercial pipe sizes available in market and open channel system with drains with 30 cm bottom width.**

***Index Terms*— Friction Factor, Reynold's Number, Energy Line Slope, Plain Bed, Gravel Bed**

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Asif Naseer (email: asif.naseer@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan.

Syed Taseer Abbas Jaffar (email: taseer.jaffar@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan.

Hira Sattar (email: hira.sattar@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan.

Abdullah (email: Abdullah.nazir@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan.

Fatima Ashfaq (email: fatima.ashfaq@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan.

M. Bilal Zahid (email: bilal.zahid@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan.

Shakir Ahmad (email: shakir.ahmad@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan

Abdul Waqar Akhtar (email: abdul.akhtar@umt.edu.pk) is affiliated with the department of civil engineering, University of Management & Technology, Lahore Pakistan

Corresponding Author: abdullah.nazir@umt.edu.pk

I. Introduction

TO control, understand and utilize the waters available in their various forms within the hydrological cycle, the knowledge of stormwater management is applied. The major part of the work is carried out for surface waters and groundwaters generated by rainfall events [11]. The stream water flow perceived during and just after a certain precipitation storm is actually yielded by surface runoff and groundwater. However, the volume of storage and the groundwater flow for a longer period of time are connected to rainfall events [10]. The utilizations of the concepts, maintenance rules and design methods for the management of stormwater are evident for many of diverse fields, like water supply, agricultural drainage, flood control, urban runoff, lake management, ecological impact studies and forest management [13]. Undeniably, the usages of storm water management are very much wide and are found in subjects related to hydrology, environmental engineering, hydraulics, and water resources engineering [2]. Since early ages, it is believed that the storm water coming out of constructed areas must be managed one way or another. Pollution and waste material transferred by the storm water is contained with quality and quantity issues [4, 5], which significantly affects the public health along with the environmental quality [9]. Sanitation systems and infrastructures in urban areas have various development stages and the understanding about storm water changed significantly during the decades, especially in some recent years. However, there exists a clear heterogeneity worldwide while the shortages of studies are analyzed for urban storm water when conducted for some African or Asian countries [8]. Techniques for a maintainable storm water management at decision stages (regional, local or political scale) are required, but all these strategies need the data related to possibilities and a clear understanding about each and every consequence connected to all these decisions [15]. One of the solid approaches for the management of storm water must be flexible, either based on local features and should be incorporating the temporal, spatial and managerial factors and regulations among other problems. Technical or economic limitations define various scenarios for decision-making [14].

Best approaches for the management of storm water must be perceived as a prospect for the establishment and rectification of social, environmental and educational conditions for surrounding and urban areas. Consequently, they demand a bunch of opportunities and the participation from various concerned stakeholders [7]. A good quality decision requires time and a just overview of the issue under consideration [6]. The aim of this article is to take part in the sustainable management of storm water and to address the most important associated factors that must be evaluated along with their interaction [3].

II. METHODOLOGY

An institute located in Johar Town, Lahore as shown in Figure 2 was facing serious problem for the management of storm water due to heavy rainfalls especially in the summer (monsoon) season. The accumulation of storm water was due to the high level of water in the adjacent colony during or after heavy rainfall events. The projects in development and proposed project consisted new buildings expected to be developed over an unknown period of time, were additional reason for such consequences. Due to the improper or inadequate drainage facilities, the storm water got stagnant and created hurdles for the normal on-going activities all over the campus. In order to meet the objectives first there was need for the review of existing approaches to estimate the runoff due to precipitation. Then based on the area and data availability we needed to select certain method that was most appropriate for the estimation of runoff discharge [16]. Once the design runoff discharge was computed the new modified scheme for the storm water drainage system was possible. Based on the design discharge the layout for the drainage system was proposed. The storage facility was also based on the design runoff. The schematic layout for the methodology is shown in the figure 1.

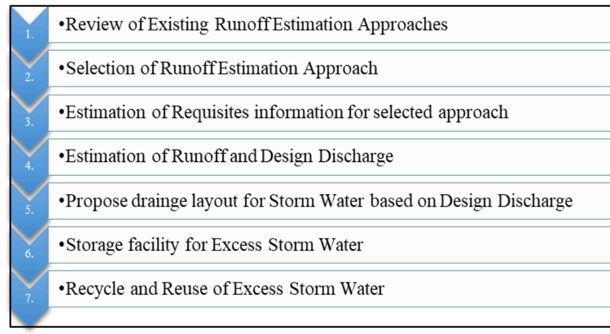

Fig. 1. Schematic Layout based on the methodology

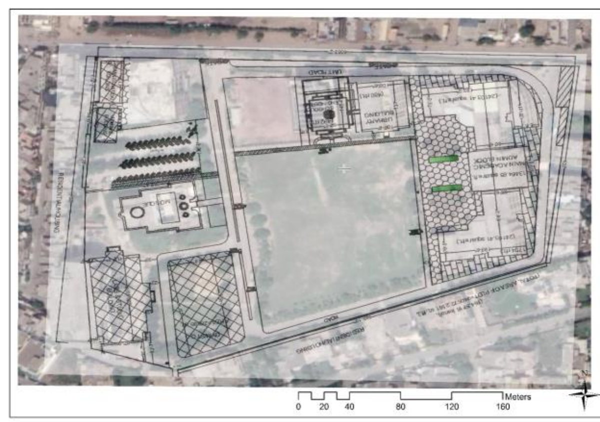

Fig. 2. Layout map of institute, Lahore

On the other hand, based on the innovative vision the institute has plans for re-using this water for multiple purposes such as irrigating plants, grassy land to keep the environment clean and from water conservation point of view. Therefore, there is a strong need to resolve the existing issues the management and propose possible schemes for the re-use of storm water. The existing conditions for the drainage facilities are unable to cope up with the aftermaths of heavy rainfalls. The existing storm water drains are becoming insufficient due to rapid developments over the campus and need serious attention. The situation just after the rainfall can be seen in figure 3.

On the basis of existing storm water management issues in different parts of the campus there was immense need for revising the existing draining scheme and propose some additional drainage system. In addition to modified drainage facilities the excess storm water need attention to deal with the consequences of water accumulation and stagnation. Some suitable place was essential within the campus to store the water as multipurpose reservoir. Once the water was stored it needs to have certain level of treatment for if it was stored for various purposes. Based on the above discussions, primarily there were three major objectives of this study i.e., to upgrade the existing storm water management system/scheme based on the hydrology of institute, to propose the facility for storing the excess storm water and to propose the re-cycle or re-use methodology for the excess water.

Fig. 3. Post rainfall situation near the institute

A. Review of Existing Runoff Estimation Approaches

Methods for the Runoff Estimation can broadly be classified as; Direct Methods and Indirect Methods. Direct Methods are based on the measurements and observations at some specific point of the study area. Whereas, the proposal and design of the storm water drainage facilities mostly depend on the indirect methods. Most of the indirect methods are based on the equations derived and are also known as lumped approaches [17]. Some of the popular approaches are; Empirical Equations, Unit Hydrograph Technique and Rational Method. Empirical relationships or equations are developed using past peak flow information or available data. Generally, the peak flow is correlated with basin characteristics including area and slope [17]. Sometimes they are strictly region specific and applicable to the particular region or study area within particular region. However, with certain modifications they can be applied to other regions if they have similar hydro-meteorological characteristics. Various Flood discharge formulae are; Dicken's Formula, Ryves Formula, Inglis Formula, Fuller's Formula, Baird & McIllwraith Formula, Hafiz Asif Arshad Formula and Usman Naeem Formula [18, 19].

Unit hydrograph is a direct runoff hydrograph resulting from one unit (one inch or one cm) of constant intensity uniform rainfall occurring over the entire watershed [19] (Eq. 1). It assumes the watershed response to rainfall input as linear:

$$Q_{Peak} = P^{*} \times q_{Unit} \tag{1}$$

Where, QPeak is peak runoff estimated, QUnit is peak discharge in response to a unit of rainfall input that lasted over same time and P* is actual rainfall. The main task is to estimate the unit-hydrograph for a given rainfall duration (e.g., 2.5hr in Figure 4 is illustrated as below). Major limitation is linear assumption that may induce errors as the response also depends on antecedent conditions.

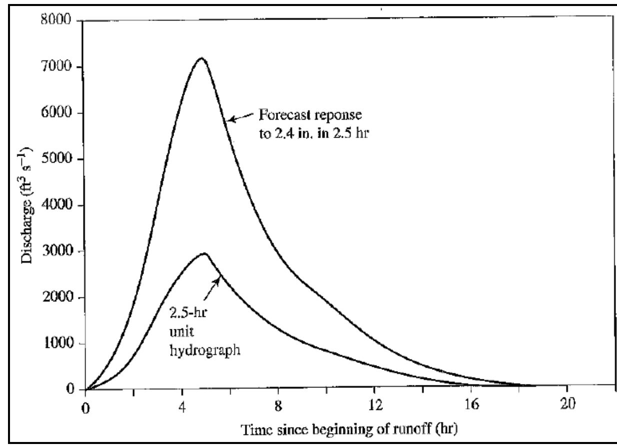

Fig. 4. Post rainfall situation near the institute

Mulvaney first outlined the rational method, which assumes that a steady, uniform rainfall rate will produce maximum runoff when all parts of the watershed are contributing to outflow. This occurs when the storm event lasts longer than the time of concentration. The time of concentration is the time it takes for rain in the most hydrologically remote part of the watershed to reach the outlet. The method assumes that the runoff coefficient remains constant during a storm. This method postulates a simple proportionality between peak discharge and rainfall intensity. The expression for the runoff from the Rational Method is given in Eq. 2 [19].

$$Q = C.I.A \tag{2}$$

Where, Q is Peak discharge (cusecs), C is Coefficient of runoff (dimensionless), I is intensity of rainfall (in/hr) and A is area in acres. However, for the practical field applications generally we use System International (SI) units therefore, the rational formula can be re-written (Eq. 3).

$$Q = \frac{1}{3.6}.C.i.A \tag{3}$$

Where, Q is Peak discharge (cumecs), C is Coefficient of runoff (dimensionless), i is mean intensity of rainfall (mm/hr) and A is area in km2

B. Selection of Runoff Estimation Approach

Surface runoff can be calculated by many methods as discussed above but every method has some specification and limitation, depending on the area, soil condition and topography. The empirical approaches are site specific and used when hydro-meteorological conditions for the study area somehow match which is practically very difficult. However, based on similarity these equations are useful. In case of runoff estimation from the Unit Hydrograph method, primary limitation is linear assumption that may induce significant margin of errors as depending upon antecedent conditions. Conversely, the use of the rational method for drainage system design in small urban areas is appropriate due to multiple reasons. Firstly, the subareas are larger than approximately 40 acres. Secondly, practically there is more than one drainage channel usually for the urban areas and thirdly this approach considers the hydrologic properties of different areas and lastly the time of concentration is usually greater than 30 minutes for small urban areas [7]. The rational method fulfils the site conditions. When rainfall starts then surface runoff starts, just after refill the surface retention. Discharge from various locations of the watershed will differ due to difference in their time interval which is the time of concentration. When storm duration is less than time of concentration, only some parts of the catchment contribute to total discharge which is closer to the outlet. When storm duration is greater than the time of concentration, then total catchment area contributes to total discharge of the outlet [20].

i. Requisites for Rational Method

As described above the rational method depends on the rainfall intensity, runoff coefficient and area of the basin. Most important these three parameters are estimation of the rainfall intensity. In the subsequent sections, estimation of these parameters of the rational method are discussed in details.

a) Rainfall Intensity

Average monthly precipitation data of Lahore is available from 1987 to 2013. Maximum precipitation occurs from June to September due to the monsoon season. The maximum monthly rainfall is 180 mm which is observed in the month of August. The design rainfall for this project is therefore taken as 180 mm. As the available data length is 31 years, the rainfall intensity at return period of 5, 10 and 25 years is taken as the design value. Kirpich formula for the calculation of Time of concentration (Eq. 4) is used.

$$t_c = 0.0078\,(L^{0.77}/S^{0.385}) \tag{4}$$

Where, t_c is time of concentration, L is Length of flow (m) and S is the slope (m/m).

If the rainfall data from a self-recording rain gauge is available for a long period, the frequency of occurrence of intensity occurring over a specified duration can be determined. A knowledge of maximum intensity of rainfall of specified return period and of duration equal to the critical time of concentration is of considerable practical importance in evaluating peak flows related to hydraulic structures. Briefly, the procedure to calculate the intensity-duration-frequency relationship for a given station is as follows (Eq. 5):

$$i = \frac{KT^x}{(D+a)^n} \tag{5}$$

Where, i is maximum intensity (cm/h), T is return period (years) and D is duration (hours). While, K, x, a and n are coefficients for the area represented by the station. [15] presented the values of these coefficients for various cities of India. Due to geographical closeness and similarity of terrain, values of Amritsar are selected for the Lahore area.

The Duration of the rainfall was kept 1hr to ensure the maximum intensity rainfall for the various return periods. Data in Table 2 was used for the rainfall intensity calculation. The probability of rainfall event is calculated as m/(N+1) where m is the order of rainfall depth when arranged in descending order, and N is the number of years of the record. The time period T is calculated as the reciprocal of the probability. Intensity is then calculated using the intensity-duration-frequency relationship. Intensity calculations are given in the Table 3.

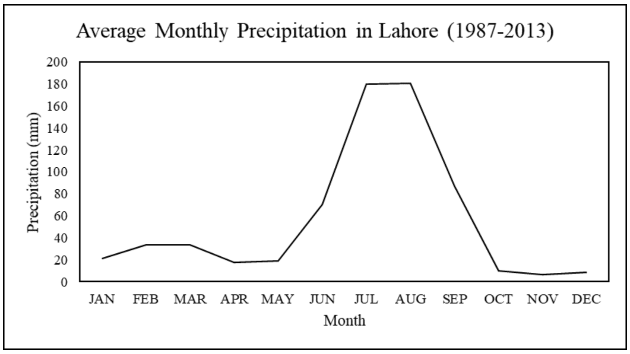

Fig. 5. Monthly average precipitation values in Lahore (base period: 1987-2013)

TABLE I
TYPICAL VALUES OF COEFFICIENTS K, x, a AND" IN ABOVE EQUATION

Zone	Place	K	x	a	n
Northern	Amritsar	14.41	0.1667	0.25	0.6293

TABLE II
PARAMETERS FOR THE RAINFALL INTENSITY CALCULATIONS

Duration of rainfall D	*1 hr*
N	*number of years of record*
Probability of occurrence P	*m/(N+1)*
Time Period T	*1/P*
K	*14.41*
x	*0.1304*
a	*1.4*
n	*1.2963*

Another parameter in the Rational Method equation is the runoff coefficient, C. The major factors affecting the rational method runoff coefficient value for a watershed are the land use, the soil type and the slope of the watershed. The physical interpretation of the runoff coefficient for a watershed is the fraction of rainfall on that watershed that becomes storm water runoff. Thus, the runoff coefficient must have a value between zero and one.

For Non homogenous catchments, weighted equivalent runoff coefficient (Eq. 6) is calculated using following formula;

$$C_e = \frac{\Sigma_{i=1}^{N}(C_i A_i)}{A} \tag{6}$$

Where, N is Number of sub areas in the catchment.

Using the rainfall intensity, suitable runoff coefficient and by computing the area based on survey the rational equation was then used to estimate the runoff. For the simplicity the institute was divided into different sub-areas (Table 5) (sub-catchments) as shown in the Figure 6.

TABLE III
CALCULATION FOR THE RAINFALL INTENSITIES AT DIFFERENT RETURN PERIOD

Sr. No.	Precipitation (mm)	P	T	Intensity (cm/hr)
1	1232.5	0.031	32.000	7.28
2	1188.5	0.063	16.000	6.65
3	955.2	0.094	10.667	6.31
4	902.8	0.125	8.000	6.08
5	856.2	0.156	6.400	5.90
6	852.9	0.188	5.333	5.76
7	826	0.219	4.571	5.65
8	815.3	0.250	4.000	5.55
9	806.5	0.281	3.556	5.47
10	785.9	0.313	3.200	5.39
11	749.8	0.344	2.909	5.32
12	658.3	0.375	2.667	5.26
13	652.2	0.406	2.462	5.21
14	629.2	0.438	2.286	5.16
15	628.1	0.469	2.133	5.11
16	627.5	0.500	2.000	5.07
17	615.7	0.531	1.882	5.03
18	613	0.563	1.778	4.99
19	582.9	0.594	1.684	4.96
20	557.3	0.625	1.600	4.93
21	541.8	0.656	1.524	4.89
22	540.7	0.688	1.455	4.86
23	535.7	0.719	1.391	4.84
24	520.1	0.750	1.333	4.81
25	495.2	0.781	1.280	4.78
26	492.6	0.813	1.231	4.76
27	491.1	0.844	1.185	4.74
28	473.7	0.875	1.143	4.71
29	374.9	0.906	1.103	4.69
30	371.2	0.938	1.067	4.67
31	333.7	0.969	1.032	4.65

III. Results And Discussion

Rainfall data for use as design storms may be presented in various forms. These include a relationship between rainfall intensity, duration and frequency (IDF) curves which are based on historic records as described in above sections [21]. It provides an average rate of rainfall corresponding to a given storm duration and specified return period, a synthetic design storm profile which describes the variation of rainfall intensity with time throughout the duration of the event. Short duration and high intensity rainfalls are of great importance in the urban area in general, with particular reference to design. Indeed, in urban watersheds, whose times are almost less than or equal to 1-hour, the estimate of extreme values of rainfall depth at high time resolutions (short duration) is crucial [1].

TABLE IV
SUB-AREAS (BASINS) NOTATIONS AND DESCRIPTION

Notation	Area (km^2)
A1	0.00025
A2	0.00061
A3	0.000219
A4	0.000526
A5	0.000294
A6	0.0003
A7	0.000443
A8	0.000412
A9	0.000217
A10	0.000191
A11	0.00051
A12	0.000473
A13	0.000305

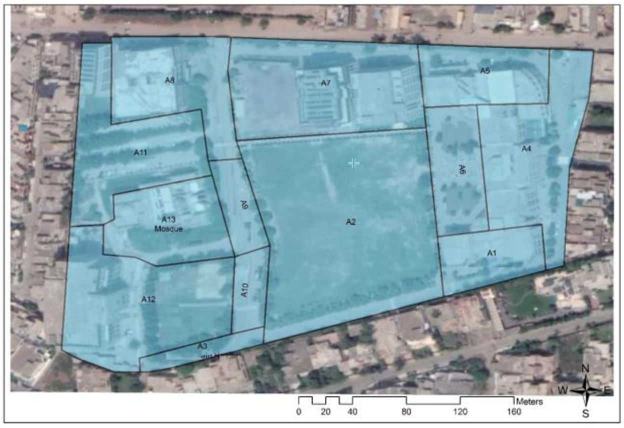

Fig. 6. Layout Map for the institute with Sub-Areas

The design discharge calculated by applying rational method are shown in Table 6. These discharges are estimated for 5 years return periods, while the discharges for 10, 15, 20, 25 and 30 years return period are given in attached supplementary file. As it is evident from the Table 6 that the total flow on the basis of 5 years return period is 0.0594 m3/sec with the help of rational method approach. The remaining scenarios gave 0.0648, 0.0676, 0.0747, 0.0748 and 0.0821 m3/sec. The design for conduit system was carried out for all the scenarios.

TABLE IV
CALCULATION OF DESIGN DISCHARGE (5-YEAR RETURN PERIOD)

Discharge Designation	Area	Runoff Coefficient	Rainfall Intensity	Peak Discharge
	A (Km^2)	C	i (mm/hr)	Q = (1/3.6).C.i.A (m^3/sec)
Q1	0.00025	0.95	57.1219	0.0038
Q2	0.00061	0.35	57.1219	0.0034
Q3	0.000219	0.95	57.1219	0.0033
Q4	0.000526	0.95	57.1219	0.0079
Q5	0.000294	0.95	57.1219	0.0044
Q6	0.0003	0.75	57.1219	0.0036
Q7	0.000443	0.75	57.1219	0.0053
Q8	0.000412	0.95	57.1219	0.0062
Q9	0.000217	0.95	57.1219	0.0033
Q10	0.000191	0.95	57.1219	0.0029
Q11	0.00051	0.75	57.1219	0.0061
Q12	0.000473	0.75	57.1219	0.0056
Q13	0.000305	0.75	57.1219	0.0036
			Total Q_p	0.0594

Based on design discharge, considering the layout for provision of the drainage facilities, two approaches were used and described i.e., Closed Conduit Underground System (CCUS) and Open Channel Layout System (OCLS). Following design procedure is used for the design of closed channel i.e., the calculation of discharge 'Q' by rainfall analysis and using continuity equation (Eq. 7) to calculate the Area of sewer to cover the particular discharge flowing in that sewer.

$$Q = AV \tag{7}$$

Where, Q is discharge of water (m3/s), V is Velocity for storm water flow (Self-cleansing Velocity), m/s and A is Area of Sewer, m2 Calculation of diameter 'd' of sewers is done as A=(πd^2)/4. Then obtaining slope is obtained using Manning's Equation V=1/n R^(2/3) S^(1/2). Where, R is hydraulic radius in meters; whereas, R = d/4, S is Slope of sewer for the flow of water with velocity V. n is Manning's coefficient, 0.013 for new pipes. After that, calculation fall of each sewer which would help to calculate the invert level of the sewers. Calculation of Invert Levels is made using the below formula (Eq. 8).

$$I.L\ of\ U/E = Ground\ Level - Earthcover - Thickness\ of\ pipe - Dia\ of\ pipe \tag{8}$$

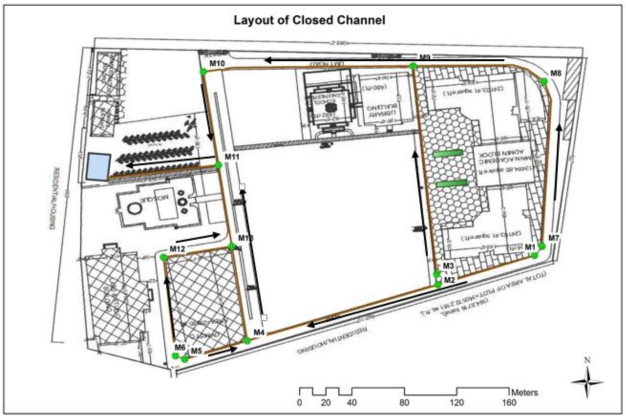

Fig. 7. Layout for the Closed Channel Conduit Drainage System

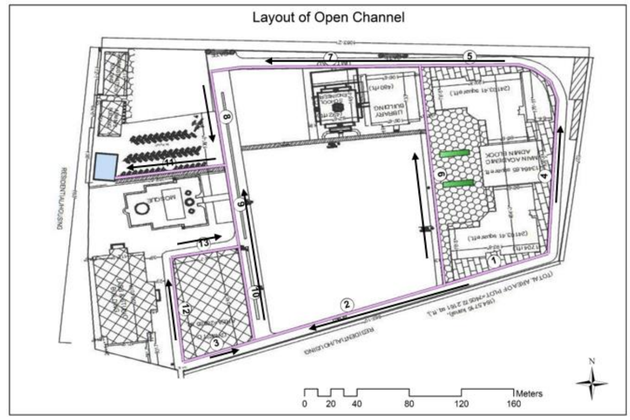

Fig. 8. Layout for the Closed Channel Conduit Drainage System

TABLE V
DESIGN CALCULATIONS FOR THE CLOSED CHANNEL SYSTEM (5-YEARS RETURN PERIOD)

Sr. No.	From Manh ole	To Manh ole	length	Flo w Desi g	Local Flow	Previ ous Flow	Qa	Vfull	Area	Dia	Co mm erci al Dia	slope	Fall	Invert Level	
			m		m^3/s	m^3/s	m^3/s	m/s	m^2	mm	mm	m/m		U/IL	L/IL
1	1	2	70.3	Q1	0.003	0	0.003	0.7	0.005	82.8	225	0.0038	0.270	58.72	58.45
2	2	4	135.7	Q2	0.003	0.003	0.007	0.7	0.010	114.1	225	0.0038	0.521	58.45	57.93
3	5	4	67.3	Q3	0.003	0	0.003	0.7	0.004	77.55	225	0.0038	0.258	58.50	58.24
4	7	8	145.4	Q4	0.007	0	0.007	0.7	0.011	120.0	225	0.0038	0.558	58.72	58.16
5	8	9	79.2	Q5	0.004	0.007	0.012	0.7	0.017	149.9	225	0.0038	0.304	58.16	57.86
6	3	9	172	Q6	0.003	0	0.003	0.7	0.005	80.63	225	0.0038	0.660	58.62	57.96
7	9	10	131.6	Q7	0.005	0.015	0.021	0.7	0.030	196.4	225	0.0038	0.505	57.86	57.35
8	10	11	80.1	Q8	0.006	0.021	0.027	0.7	0.039	223.3	225	0.0038	0.307	57.35	57.04
9	4	13	69.2	Q10	0.002	0.010	0.013	0.7	0.019	155.8	225	0.0038	0.265	57.93	57.66
10	6	12	61.6	Q12	0.005	0	0.005	0.7	0.008	101.1	225	0.0038	0.236	58.65	58.41
11	12	13	70	Q13	0.003	0.005	0.009	0.7	0.013	129.8	225	0.0038	0.268	58.41	58.14
12	13	11	72.1	Q9	0.003	0.022	0.025	0.7	0.036	217.0	225	0.0038	0.277	57.66	57.39
13	11	Reser voir	92.7	Q11	0.006	0.053	0.059	0.7	0.084	328.6	380	0.0019	0.177	56.89	56.71

The first two steps of procedure for open channel were same as that for closed channel, the next step was the calculation of depth of the channel by assuming any width of the channel as per the space available on the site $Depth = \frac{Area}{Width}$. The factor of safety of 0.5ft was added in depth to avoid the overflowing of water. Perimeter 'P' of channel was calculated by the formula $Perimeter = (2 \times depth) + Width$. The slope was obtained from Manning's formula like previous case applying the value of n as 0.02 (for barren land).

Figure 7 & 8 are showing the two proposed channel systems for the management of storm water i.e., open channel and closed conduit systems. First purpose was to transport the water effectively to the desired spot, which can be a reservoir or bigger tank, where further water treatment and screening processes can be applied to make that stored water useful for community. In coming sections some of these scenarios will be discussed in details.

TABLE VI
DESIGN CALCULATIONS FOR THE CLOSED CHANNEL SYSTEM (5-YEARS RETURN PERIOD)

Sr. No	**Channel**	**length**	**length**	**Flow Designation**	**Local Flow**	**Previous Flow**	**Actual Q**	**V full**	**Area**	**Width**	**Depth**	**Depth with F.B**	**Perimeter**	**R**	**slope**
		m	**ft**		**m^3/s**	**m^3/s**	**m^3/s**	**m/s**	**m^2**	**m**	**m**	**m**	**m**	**m**	**m/m**
1	1	70.3	230.6	Q1	0.0037	0	0.0037	0.7	0.005	0.3	0.0180	0.5180	2.5360	0.0021	0.7167
2	2	135.75	445.3	Q2	0.0033	0.0037	0.0071	0.7	0.010	0.3	0.0341	0.5341	2.5682	0.0040	0.3105
3	3	67.35	220.9	Q3	0.0033	0	0.0033	0.7	0.004	0.3	0.0157	0.5157	2.5315	0.0019	0.8538
4	4	145.47	477.2	Q4	0.0079	0	0.0079	0.7	0.011	0.3	0.0377	0.5377	2.5755	0.0044	0.2722
5	5	79.21	259.8	Q5	0.0044	0.0079	0.0123	0.7	0.017	0.3	0.0588	0.5588	2.6176	0.0067	0.1540
6	6	172	564.3	Q6	0.0035	0	0.0035	0.7	0.005	0.3	0.0170	0.5170	2.5340	0.0020	0.7706
7	7	131.63	431.8	Q7	0.0052	0.0159	0.0211	0.7	0.030	0.3	0.1009	0.6009	2.7019	0.0112	0.0781
8	8	80.15	262.9	Q8	0.0062	0.0211	0.0274	0.7	0.039	0.3	0.1305	0.6305	2.7610	0.0142	0.0571
9	10	69.21	227.0	Q10	0.0028	0.0104	0.0133	0.7	0.019	0.3	0.0636	0.5636	2.6271	0.0073	0.1395
10	12	61.66	202.2	Q12	0.0056	0	0.0056	0.7	0.008	0.3	0.0268	0.5268	2.5536	0.0031	0.4248
11	13	70	229.6	Q13	0.0036	0.0056	0.0092	0.7	0.013	0.3	0.0441	0.5441	2.5882	0.0051	0.2226
12	9	72.14	236.6	Q9	0.0032	0.0226	0.0258	0.7	0.037	0.3	0.1232	0.6232	2.7465	0.0135	0.0612
13	11	92.7	304.1	Q11	0.006	0.0532	0.0593	0.7	0.084	0.3	0.2826	0.7826	3.0652	0.0277	0.0234

The procedure for the design of rectangular storage tank was, calculation of discharge 'Q' by Rainfall Analysis and then measuring the capacity of tank by using duration of rainfall using Capacity = Q × Duration of rainfall. Where, capacity is in m3, Q = discharge in m3/s and duration of rainfall is in seconds.

TABLE VII
DESIGN CALCULATIONS FOR THE CLOSED CHANNEL SYSTEM (5-YEARS RETURN PERIOD))

Return Period (Years)	**Flow (m3/s)**	**Duration of Rainfall**		
		1 hr	**2 hr**	**3 hr**
		3600 sec	**7200 sec**	**10800 sec**
		Capacities (m^3)		
5	0.0594	213.6640	427.3280	640.9920
10	0.0648	233.2973	466.5945	699.8918
15	0.0676	243.3063	486.6125	729.9188
20	0.0747	268.8609	537.7219	806.5828
25	0.0748	269.3622	538.7243	808.0865
30	0.0821	295.4180	590.8361	886.2541

Calculation of the height of tank by fixing the Height (H) and Width (W) of the tank constant was done by Area=H×W. Then by using Hit and Trial methods for suitable length width and height according to the space available. Here the Factor of Safety (FOS) of 1.5 times was considered for deciding the final dimensions for the length of the proposed reservoir Length=Capacity/Area. The details for the finalizing the geometry of the reservoir are presented for different during of rainfall are given in Table 9 to 11.

TABLE VIII
RESERVOIR DIMENSIONS FOR THE 1-HOUR DURATION RAINFALL EVENT

Return Period (Years)	**Flow (m3/s)**	**Fixed**		**1hr Duration**		
		Height	**Width**	**Area**	**Length**	**Length with FOS**
		(m)	**(m)**	**(m2)**	**(m)**	**(m)**
5	0.0594	3	30	71.22	2.37	3.6
10	0.0648	3	30	77.77	2.59	3.9
15	0.0676	3	30	81.10	2.70	4.1
20	0.0747	3	30	89.62	2.99	4.5
25	0.0748	3	30	89.79	2.99	4.5
30	0.0821	3	30	98.47	3.28	4.9

TABLE IX
RESERVOIR DIMENSIONS FOR THE 2-HOUR DURATION RAINFALL EVENT

Return Period (Years)	**Flow (m3/s)**	**Fixed**		**2hr Duration**		
		Height	**Width**	**Area**	**Length**	**Length with FOS**
		(m)	**(m)**	**(m2)**	**(m)**	**(m)**
5	0.0594	3	30	142.44	4.75	7.1
10	0.0648	3	30	155.53	5.18	7.8
15	0.0676	3	30	162.20	5.41	8.1
20	0.0747	3	30	179.24	5.97	9.0
25	0.0748	3	30	179.57	5.99	9.0
30	0.0821	3	30	196.95	6.56	9.8

TABLE X
RESERVOIR DIMENSIONS FOR THE 3-HOUR DURATION RAINFALL EVENT

Return Period (Years)	Flow (m3/s)	Fixed		3hr Duration		
		Height (m)	Width (m)	Area (m2)	Length (m)	Length with FOS (m)
5	0.0594	3	30	213.66	7.12	10.7
10	0.0648	3	30	233.30	7.78	11.7
15	0.0676	3	30	243.31	8.11	12.2
20	0.0747	3	30	268.86	8.96	13.4
25	0.0748	3	30	269.36	8.98	13.5
30	0.0821	3	30	295.42	9.85	14.8

Founded on the above calculations the higher value for the length of the reservoir was selected (i.e., Height = 3m, Width = 30m and Length = 15m) to ensure the safety and to cater the effect of any possible extreme condition or situation for the management of rain water. Storm water picks up and carried numerous pollutants into waterways. Porous areas, such as parks and lawns, soak up most of the rain that fell there. Therefore, there was much more polluted storm water runoff in areas with large amounts of impervious surfaces. Storm water from developed areas was 'nonpoint source' pollution (pollution that came from many diffuse sources), so storm water picked up all the pollutants along its pathway [24]. Pollutants in storm water may include grease, oil, and heavy metals from cars; fertilizers, pesticides and other chemicals from gardens, homes and businesses; bacteria from pet wastes and failing septic systems; and sediment from poor construction site practices [22].

The quality of storm water collected from institute would slightly be different as it is a small area, which does not include pet wastes, fertilization etc. The pollutants of storm water from institute may include leaves and cutting from plants, grease and oil from cars and sediments from construction sites (buildings). The treatments suggested for the storm water collected according to the pollutants mentioned above can be screening, coagulation or flocculation, sedimentation, filtration and disinfection [23].

IV. Conclusions

In the light of above explanations and calculation, following are concluding remarks for establishing the storm water management scheme with recycle/reuse purposes:

1. Based on the available data from conducted engineering survey the estimation of runoff and design discharges for different return period rainfall events were conducted.
2. From point of view of draining facility for managing the storm water as consequences of rainfall events two types of systems were proposed i.e., closed conduit (underground pipe) system based on commercial pipe sizes available in market and open channel system with drains with 30 cm bottom width.
3. In order to consider the recycle or reuse concept for the rainfall water, a reservoir was proposed under the parking area adjacent to institute Mosque and Human Resource Department.
4. In order to make water drinkable the water quality and treatment was also described which would be expansive option. Alternatively, the water could be utilized for the irrigating the plants and grass within and outside the university campus.
5. By enlarging the underground reservoir and utilizing it as underground parking facility for the institute staff as there are shortage for the parking facilities for the faculty and staff. In rainy monsoon season it could be closed for parking but could be effectively used for the collection and storage of rain water for appropriate and safe usage on or off the campus premises.

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Dr. Asif Naseer is an Assistant Professor in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He has obtained his Ph.D. in Civil Engineering from National Graduate Institute for Policy Studies, Japan in 2019.

Dr. Syed Taseer Abbas Jaffar is an Assistant Professor in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He has obtained his Ph.D. in Civil Engineering from Shanghai Jiao Tong University, China in 2018.

Hira Sattar is a Lecturer in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He received his BS degree in Civil Engineering from UET, Lahore in 2014, and MS degree in Civil Engineering from AIT, Thailand in 2018.

Abdullah is a Lab Engineer in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He received his BS degree and MS degree in Civil Engineering from UET, Lahore, Pakistan in 2017 and 2021 respectively.

Fatima Ashfaq is a Lecturer -cum - Lab Engineer in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He received his BS degree and MS degree in Environmental Engineering from UET, Lahore, Pakistan in 2017 and 2021

M. Bilal Zahid is a Lecturer in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He received his BS degree in Civil Engineering from UET, Lahore and MS Degree from Fast NUCES, Lahore in 2019.

Shakir Ahmad is a Lecturer – cum - Lab Engineer in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He received his BS degree in Civil Engineering from UET, Taxila and MS Degree from Fast NUCES, Lahore in 2019.

Abdul Waqar Akhtar is a Lecturer in the Department of Civil Engineering at the University of Management and Technology, Pakistan. He received his BS degree and MS degree in Civil Engineering from UET, Lahore, Pakistan in 2015 and 2019 respectively.