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A Comparative Study of Storm Water Drainage Methods for Urban Storm Water Management

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Abstract

Urban storm water management is an important aspect of any urban area development, planning and expansion. Urbanization of an area invariably leads to increase in overall imperviousness of the area. When land becomes impervious, storm water will stagnate on the surface thereby affecting the infrastructure, transportation and causing inconvenience to general populace. One way to minimize these effects is to provide a proper storm drainage system. In this paper, therefore, a small urban region was selected and for that region suitability of three different storms sewer systems were considered. The urban area considered for case study lies in the northern part of the Vellore town, and has a total area of about 25 km^2. For this region three different storm drainage systems were proposed, namely, construction of a new underground circular sewer system (Alternative 1), repair and expansion of existing surface sewer system (Alternative 2) and construction swales (Alternative 3). Of the three options, Alternative 2 was found to be economical; however it can be argued that from efficiency and aesthetics point of view 1 and 3 will be preferred alternatives. Alternative 3 can be used as a management measure for recharging groundwater aquifers, along with storm water drainage. The final selection of the option however, will depend on the suitability of the method, budgetary constraints and space availability.

Keywords: Storm Water Management, Storm Sewer Design, Urban Hydrology

1. Introduction

Population growth and urban development can create potentially severe problems in urban water management. One of the most important facilities in preserving and improving the urban water environment is an adequate and properly functioning storm water drainage system1. Construction of houses, commercial buildings, parking lots, paved roads and streets increases the impervious cover in a watershed and reduces infiltration. Also, with urbanization, the spatial pattern of flow in the watershed is altered and there is an increase in the hydraulic efficiency of flow through artificial channels, curbing, gutters and storm drainage and collection systems. These factors increase the volume and velocity of runoff and produce larger peak flood discharges from urbanized watersheds than occurred in the pre-urbanized condition. Many urban drainage systems constructed under one level of urbanization are now operating under a higher level of urbanization and have inadequate capacity. A typical urban drainage system can be considered as having two major types of elements: location dements and transfer elements.Location elements are the places where the water example, water storage, water treatment, water use and wastewater treatment. Transfer elements connect the location elements; these elements include channels, pipelines, storm sewers, sanitary sewers and streets. The system is fed by rainfall, influent water from various sources and imported water in the pipes or channels. The receiving water body can be a river, a lake, or an ocean. A storm sewer systemis a network of pipes used to convey storm runoff in a city. The design of storm sewer systems involves the determination of diameters, slopes and crown or invert elevations for each pipe in the system. The crown and invert elevations of a pipe are, respectively, the elevations of the top and the bottom of the pipe circumference. The selection of a layoutor network of pipe locations, for a storm sewer system requires a considerable amount of subjective judgment. Hydrologists are usually able to investigate only a few of the possible layouts. Generally, manholes are placed at street intersections and at major changes in grade, or ground surface slope

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and the sewers are sloped in the direction of the ground surface, so as to connect with downstream sub-mains and trunk sewers. Once a layout has been selected, the rational method can be used to select pipe diameters.In the present project three alternatives are being suggested for the drainage of storm water. The three alternatives are: repair and expansion of the existing surface storm sewer, construction of an underground circular storm sewer and construction of swales. Each of the alternatives have their own merits and demerits and the best alternative will depend on the volume of storm water to be carried, area available for storm sewer layout, economy and aesthetics. For example, Hamel et al.[2] and Davis et al.[3] noted that vegetated swales can help in reducing the magnitude of peak flows during a storm and swales can also help in pollutant removal from storm runoff as suggested by Lucke et al[4].

2. Study Area Description

The area selected for storm sewer drainage lies within the larger part of the Vellore town. Figure 1 shows a schematic map of the study area. There is already an existing storm drain, which is a surface drainage system, partially open and is rectangular in shape. The drain begins roughly from Chittoor bus stand and ends near Palar River Bridge. The width of the existing drain is 1.30 m and the depth is 0.7 m. The total length of the drain is 2.42 km. The existing storm sewer drains water only from one side of the road. The existing drain however, is filled with sediment and debris at various places along its length and at some places it is also being used as a solid waste dumping site. One of the alternatives is to repair the existing drain however there is no guarantee that it will not lead to same disuse in future as it is now. Also, the existing drain was constructed quite a few years back for a certain carrying capacity and since there is increase in impervious area and thereby surface runoff, the existing drain will not be sufficient for carrying the future discharges. Figures 2 (a), 2 (b) and 2 (c) show the condition of the existing storm sewer. For an efficient storm water discharge, an efficient storm water management option is necessary. The goal of any typical storm water management technique is to efficiently collect the flood water and discharge it to some sink such as a river or a lake. For this purpose a study area in Vellore town was selected. Three alternative storm water management options were considered in the project, which are:

Alternative 1 (A1): Construction of an underground storm sewer network.

Alternative 2 (A2): Repair and expansion of the existing rectangular storm drain.

Alternative 3 (A3): Construction of swales as a Best Management Practice (BMP).

Figure 1. Schematic map of the study area for which storm sewer network is to be designed.

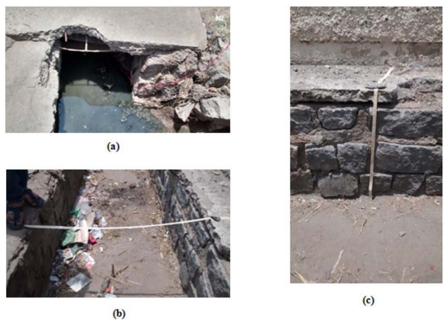

Figure 2. Schematic view of the existing storm sewer network.

3. Design Approach and Details of Underground Circular Storm Sewer Network (Alternative 1)

A storm sewer is designed to drain excess rain and groundwater from paved streets, parking lots, sidewalks and roofs. Storm drains vary in design from small residential dry wells to large municipal systems. They are fed by street gutters on most roadways, freeways and other busy roads, as well as towns in areas which experience heavy rainfall, flooding and coastal towns which experience regular storms. Many storm drainage systems are designed to drain the storm water, untreated, into rivers or streams.

3.1 Determination of Hydrologic Parameters of the Drainage Basin

The first and foremost step in the design of storm sewer network is to delineate the larger watershed into sub-watersheds. The delineation can be done by identifying natural boundaries (such as streams or rivulets) or artificial boundaries like road, channel etc. Once the watershed delineation is completed, the next step is to calculate the runoff from each of the sub-watershed. The discharge from the watershed can be calculated using rational formula, which is expressed as:'

$$Q = CiA \tag{1}$$

In which, Q is the surface runoff in ft3/sec, i is the rainfall intensity in in/h and A is the drainage basin area in acres. In the above equation C is called runoff coefficient which is in general the function of land use land type characteristics of a drainage basin. For a drainage basin with a varied land use characteristics, a composite CA value is calculated:

$$(CA)_c = \sum_{i=1}^{k} C_i P_i A_i \tag{2}$$

Where, Ci is the runoff coefficient value for land use type i; Pi is the percentage area of the land use type i; and Ai is the area of the land use type i in the sub-watershed. The rainfall intensity i in Equation (1) can be determined through the IDF (Intensity-Duration-Frequency) curves or the IDF equation available for the region. If IDF curves for the region are not available, then IDF equation can be used for determining the design rainfall intensity value. A typical IDF equation may be expressed as:

$$i = \frac{aT^m}{(b+t_c)^n} \tag{3}$$

where a, b, m and n are the IDF equation parameters; i is the rainfall intensity (mm/hr); T is the design return period, and for an urban storm sewer design it is typically taken as 2 years or 5 years; tc is the time of concentration for the watershed for which discharge is required (in minutes). Many empirical relationships are available for determining the tc value, however, the most commonly used formula is the Kirpich[5] Equation, which is given by:

$$t_c = 0.0078L^{0.77}S^{-0.385} \tag{4}$$

Where, L is the length of the watershed (in feet) and S is the slope of the watershed (ft/ft) and tc is obtained in minutes. Substituting these values in Equation (1), the design discharge from the watershed can be calculated.

3.2 Calculating the Pipe Diameter

The design discharge from the watershed is used as input for determining the size of the storm sewer. The required size of the pipe can be determined using the Manning's equation (6). The Manning's equation is an empirical equation that applies to uniform flow in open channels and is a function of the channel velocity, flow area and channel slope. Mathematically, Manning's Equation is expressed as:

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{5}$$

Where n is Manning's coefficient, which is a function of pipe material; R is the hydraulic depth expressed as the ratio of pipe area to the wetted perimeter; and S is the slope of the pipe alignment. For a circular pipe, hydraulic depth is D/2, where D is the pipe diameter. Substituting this value in the above equation, the desired pipe diameter can be calculated as follows:

$$D = \left(\frac{2.02Q}{\sqrt{S}}\right)^{\frac{3}{8}} \tag{6}$$

3.3 Estimation of Storm Sewer Network Cost

Once the pipe diameter for the corresponding runoff from all the sub-watersheds is determined, the cost of the storm sewer network can be determined from:

$$TC = \sum_{i=1}^{n} c(d) \mathrm{x} d_i \mathrm{x} L_i \tag{7}$$

where c(d) is the unit cost of pipe in Rs/m, as a function of pipe diameter; di and Li are the diameter and length of the pipe i.

3.4 Sample Design Calculations

A line diagram of the study area for which the storm sewer is being designed is shown in Figure 3. The Figure shows the hydrologic and geometric details for all sub-catchments. A sample calculation for storm sewer design is explained here. Consider the pipe segment P1 which receives discharge from the sub-watershed A1 and A2. In the design of storm sewer network, an inherent assumption being made is that all the rainfall on the watershed will drain towards the storm sewer. In lieu of this assumption, discharge from A2 will drain into A1 and from there it will drain into P1. The drainage area of A1 and A2 are, 6.11 ha and 6.57 ha, respectively. The next step is to identify the net area under each land use type. For A1, the land use type area division is as follows: residential – 0.611 ha, grass land – 2.44 ha and open land – 3.05 ha. Similarly, for A2, the land use type area division is: residential – 3.28 ha, grass land – 2.63 ha and open land – 0.657 ha, respectively. These areas are multiplied with their respective runoff coefficients (for residential C = 0.75, for grass lands C = 0.35 and for open land C = 0.3) to get the factored watershed area.

For A1, $\mathbf{\Sigma CA = 0.75 \times 0.611\ \textit{ha} + 0.35 \times 2.44\ \textit{ha} + 0.30 \times 3.05\ \textit{ha} = 2.23\ \textit{ha}}$

For A2, $\mathbf{\Sigma CA = 0.75 \times 3.28\ \textit{ha} + 0.35 \times 2.63\ \textit{ha} + 0.30 \times 0.657\ \textit{ha} = 3.58\ \textit{ha}}$

Next, the rainfall intensity for the region was determined by using the IDF equation developed by Chawathe[5] and is given by:

$$i = \frac{6.126T^{0.16_6}}{(t+0.5)^{0.80_3}} \tag{8}$$

Where i is the rainfall intensity in mm/hr; T is the return period in years, which was taken 2 years for the present study; t is the time of concentration in minutes. The time of concentration is determined using Kirpich formula. For watershed A1, the slope and watershed length are, 0.0055 m/m and 275.9 m, similarly for watershed A2, the slope and watershed length are 0.06 and 76.2 m respectively. Hence the time of concentration for A1 is:

$\mathbf{t_{A1} = 0.0078 \times 275.9^{0.77} \times 0.0055^{-0.385} = 10.91\ \textit{min}}$

Similarly for A2, the time of concentration is:

$\mathbf{tA2 = 0.0078 \times 76.2^{0.77} \times 0.06^{-0.385} = 1.62\ \textit{min}}$

Once the time of concentration is determined, the design rainfall intensity for the watersheds can be determined from the IDF formula. The design rainfall intensity for A1 is:

$$i = \frac{6.126(2)^{0.16_6}}{(10.91+0.5)^{0.80_3}} = 9.3\frac{mm}{hr}$$

Similarly, the design rainfall intensity for A2 is:

$$i = \frac{6.126(2)^{0.16_6}}{(1.62+0.5)^{0.80_3}} = 11.5\frac{mm}{hr}$$

Table 1. Hydraulic (slope, watershed length, time of concentration) and hydrologic (rainfall intensity, discharge) parameter values for left side of the road

Sub watershed ID	Total area (m^2)	Land use area (m^2) Residential ($C_1 = 0.75$)	Grass land ($C_2 = 0.35$)	Open land ($C_3 = 0.3$)	ΣC_iA_i	Slope (m/m)	Watershed length (m)	Time of concentration tc (min)	Rainfall intensity, i (mm/h)	Discharge (m^3/sec)
A1	61092.31	6109.231	24436.92	30546.16	22298.7	0.0055	275.9	10.91	9.3	0.06
A2	65740.59	32870.3	26296.24	6574.059	35828.6	0.0600	76.2	1.62	11.5	0.11
A3	279659.42	265676.4	0	13982.97	203452.2	0.0106	228.7	7.33	10.1	0.56
A4	228408.89	216988.4	11420.44	0	166738.5	0.0013	481.7	29.56	6.9	0.32
A5	186046.11	167441.5	9302.306	9302.306	131627.6	0.0017	731.7	36.68	6.3	0.23
A6	98218.09	93307.19	4910.905	0	71699.2	0.0078	427.4	13.36	8.9	0.18
A7	151466.58	90879.95	45439.97	15146.66	88607.9	0.0080	152.4	5.99	10.4	0.25
A8	42252.06	0	25351.24	16900.82	13943.2	0.0044	205.8	9.47	9.6	0.04
A9	184198.97	147359.2	36839.79	0	123413.3	0.0060	152.4	6.69	10.2	0.35

Table 2. Hydraulic (slope, watershed length, time of concentration) and hydrologic (rainfall intensity, discharge) parameter values for left side of the road

Sub-watershed ID	Total area (m^2)	Land use area (m^2) Residential ($C_1 = 0.75$)	Grass land ($C_2 = 0.35$)	Open land ($C_3 = 0.3$)	ΣC_iA_i	Slope (m/m)	Watershed length (m)	Time of concentration tc (min)	Rainfall intensity, i (mm/h)	Discharge (m^3/sec)
A10	151497.84	7574.892	136348.1	7574.892	55675.5	0.0089	411.6	12.36	9.1	0.14
A11	207949.76	197552.3	10397.49	10397.49	154922.6	0.0222	82.3	2.52	11.2	0.48
A12	46895.39	0	2344.77	44550.62	14185.9	0.0085	216.5	7.69	10.0	0.04
A13	188687.23	179252.9	0	9434.362	137270.0	0.0107	457.3	12.50	9.1	0.34
A14	53849.86	0	2692.493	51157.37	16289.6	0.0160	76.2	2.69	11.2	0.05
A15	255825.21	243033.9	0	12791.26	186112.8	0.0091	233.8	7.92	9.9	0.51
A16	164745.42	156508.1	8237.271	0	120264.2	0.0108	198.2	6.54	10.2	0.34
A17	71955.79	0	7195.579	64760.21	21946.5	0.0159	268.3	7.11	10.1	0.06
A18	235011.25	211510.1	23501.13	0	166858.0	0.0092	463.4	13.36	8.9	0.41
A19	107122.31	0	42848.92	64273.39	34279.1	0.0100	182.9	6.33	10.3	0.10
A20	110019.45	99017.51	0	11001.95	77563.7	0.0160	76.2	2.69	11.2	0.24
A21	83668.82	75301.94	4183.441	4183.441	59195.7	0.0037	329.6	14.60	8.7	0.14

Table 3. Calculation of pipe diameter, pipe crown elevation and pipe invert elevation for the left side of the road

Pipe ID	Total discharge into the pipe (m3/sec)	Pipe length (m)	Slope (m/m)	Manning's n	Pipe diameter (m)	Velocity in the pipe (m/sec)	Pipe crown elevation (m)	Pipe invert elevation (m)
P1	0.17	90	0.0002	0.012	0.53	0.77	195.97	195.44
P2	0.73	321	0.0011	0.012	0.79	1.49	195.62	194.83
P3	1.06	654	0.0022	0.012	0.80	2.12	194.18	193.38
P4	1.29	482	0.0016	0.012	0.91	1.98	193.41	192.50
P5	1.47	327	0.0011	0.012	1.03	1.77	193.05	192.02
P6	1.72	196	0.0006	0.012	1.22	1.47	192.93	191.71
P7	2.11	352	0.0012	0.012	1.16	2.01	192.51	191.35

Table 4. Calculation of pipe diameter, pipe crown elevation and pipe invert elevation for the left side of the road

Pipe ID	Total discharge into the pipe (m^3/sec)	Pipe length (m)	Slope (m/m)	Manning's n	Pipe diameter (m)	Velocity in the pipe (m/sec)	Pipe crown elevation (m)	Pipe invert elevation (m)
P8	0.14	136	0.00021	0.012	0.49	0.76	195.97	195.48
P9	0.66	353	0.00057	0.012	0.86	1.14	195.77	194.90
P10	1.06	375	0.00061	0.012	1.01	1.31	195.54	194.52
P11	1.57	316	0.00051	0.012	1.22	1.35	195.38	194.16
P12	1.91	322	0.00052	0.012	1.30	1.43	195.21	193.90
P13	1.97	206	0.00033	0.012	1.44	1.21	195.14	193.70
P14	2.38	423	0.00068	0.012	1.35	1.67	194.86	193.50
P15	2.48	167	0.00027	0.012	1.63	1.19	194.81	193.18
P16	2.73	699	0.00113	0.012	1.29	2.10	194.02	192.72
P17	2.87	331	0.00054	0.012	1.51	1.61	193.84	192.32

After calculating factored watershed area and design rainfall intensity, the runoff from the watersheds can be calculated using rational formula. Runoff from the watershed A1 is:

$$Q_{A1} = 2.23\text{x}10^4\text{x}9.3 = 0.06\frac{m^3}{\text{sec}}$$

Similarly, the discharge from watershed A2 is:

$$Q_{A2} = 3.58\text{x}10^4\text{x}11.5 = 0.11\frac{m^3}{\text{sec}}$$

The total discharge to the storm sewer is $Q = Q_{A1} + Q_{A2} = 0.06 + 0.11 = 0.17$ m^3/sec. This discharge will be input to the storm sewer. The storm sewer diameter can be determined using Manning's equation. The diameter of pipe P1 will be:

$$D = \left(\frac{2.02\text{x}0.17}{\sqrt{0.0002}}\right)^{\frac{3}{8}} = 0.53m$$

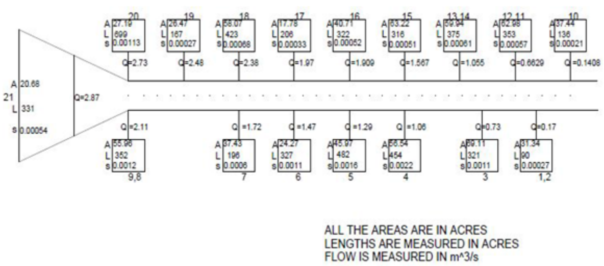

Figure 3. Line diagram of the case study area for which storm sewer network is designed.

Similar calculations can be carried out for determining the pipe diameters for other pipes. The final calculations and results for all the sub-watershed are shown in Tables 1, 2, 3 and 4.

4. Repair and Expansion of Existing Storm Sewer (Alternative 2)

As mentioned earlier in the report the region for which the analysis is being carried out has an existing partially covered surface storm sewer system. The shape of the existing storm sewer is rectangular with a width of 1.3 m and depth of 0.8 m. The sewer is laid out on either sides of the road, and runs parallel to the road up to a distance of 2.07 km starting from Chittoor bus stand. The existing storm sewer was constructed for a different land use pattern, whereas at present due to rapid increase in the urbanization, there is a significant change in the land use pattern. The change observed was mostly in terms of increase in the total impervious area. When impervious area increases, surface runoff volume increases, and therefore the existing storm sewer will not be able to carry the increased discharge. Second alternative for storm water management is therefore to revamp the existing surface storm sewer system. The calculations for determining the size of the revised storm sewer network were done in a similar fashion to underground circular storm sewer network. The final results are given in Table 5. The channel shape for the revamped storm sewer is assumed to be square. From the table the total cost of revamping and constructing a new surface storm sewer system will be around Rs. 38.29 Million. Below a sample calculation for channel C4 is shown. Manning's formula is used for calculating the channel dimensions. For a square channel B (channel width) = y (channel depth). Hence, channel cross section area, A = B × y = y^2, and channel wetted perimeter, P = B + 2y = 3y. Assuming the channel is lined with cement concrete, Manning's n = 0.024. The Manning's Equation for a channel can be written as:

Table 5. Dimensions for the revised surface storm sewer network

Channel Id	Total discharge (m^3/sec)	Velocity (m/s)	Slope (m/m)	Channel width (m)	Channel depth (m)	Channel length (m)	Costof sewer (Rs. Million)
C1	0.31	1.68	0.00021	1.02	1.02	136	0.46
C2	1.39	2.38	0.00057	1.49	1.49	353	2.53
C3	2.12	2.63	0.00061	1.72	1.72	375	3.59
C4	2.86	2.46	0.00051	1.99	1.99	316	4.05
C5	3.38	2.53	0.00052	2.11	2.11	322	4.65
C6	3.69	2.27	0.00033	2.37	2.37	206	3.77
C7	4.49	3.14	0.00068	2.23	2.23	423	6.82
C8	2.5	1.20	0.00027	2.13	2.13	167	2.45
C9	2.48	1.90	0.001137	1.62	1.62	699	5.97
C10	2.73	1.52	0.00054	1.93	1.93	331	4.01

$$Q = A - \frac{1}{n}R^{\frac{2}{3}}S^{\frac{1}{2}} \tag{9}$$

For C4, the total discharge is 2.86 m3/sec, and channel slope is 0.00051 m/m. Substituting these values in Equation (9), including the values of A, n and R, the simplified equation obtained will be:

$$y = \frac{0.325Q^{\frac{3}{8}}}{S^{0.187}} \tag{10}$$

Where y is the channel depth in meters. Solving Equation (10), for C4 we get the channel depth as 1.99 m and since the channel is assumed to be square, the width will also be 1.99 m. The calculation for other channels was done in a similar fashion. A sample design of proposed surface storm sewer drain after repair and expansion is given in Figure 4.

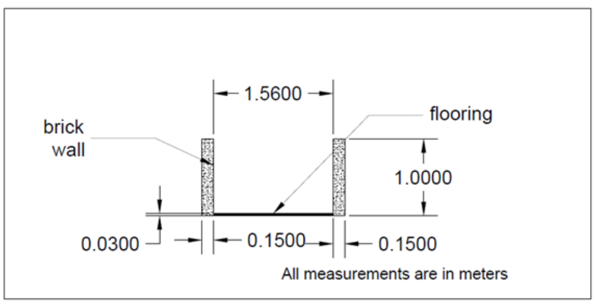

Figure 4. Cross section details of surface storm sewer after expansion.

5. Design of Swales

A vegetated swale is a broad, shallow channel with a dense stand of vegetation covering the side slopes and bottom.

Swales can be natural or manmade, and are designed to trap particulate pollutants (suspended solids and trace metals), promote infiltration, and reduce the flow velocity of storm water runoff. Vegetated swales can serve as part of a storm water drainage system and can replace curbs, gutters and storm sewer systems. Therefore, swales are best suited for residential, industrial, and commercial areas with low flow and smaller populations. The third alternative in this project aims at studying the efficacy of a vegetated swale as a storm water management option for the study area.

5.1 Sample Design Calculation

A sample design calculation of swale is shown here. The first step in design of swales is to determine the hydraulic depth of the swale using Manning's equation, assuming that all other parameters such as discharge, bottom width, n and slope are known. For the present case the channel bottom width is taken as 1.3 m. Discharge for the sample calculation is 1.29 m^3/sec, n is 0.027, and channel slope is 0.0016. Substituting these values in Equation (9) will give R value as 0.76 m. The channel depth is then calculated as 0.1R, i.e. y = 0.076 m. Channel side slope is taken as 1 in 10. From this the side width of channel will be 0.763 m. The cross section detail of a typical vegetated swale is shown in Figure 5. In Table 6 and 7, the summary of the calculations for all the sub areas is shown. From tabulated results it can be seen that the maximum depth of the proposed vegetated swale is 1.83 m, for a maximum discharge of 2.48 m^3/sec. The flow velocity in the channel will be 0.914 m/sec, which is greater than the limiting value of 0.7 m/sec, which is required for maintaining minimum flow in the channel. The final cost which includes the capital cost and operation and maintenance cost for the proposed vegetated swales on either sides of the road is shown in Table 8 and 9. The unit capital cost was taken as Rs. 6240/m length of swale, and the operation and maintenance cost was taken as Rs. 130/m length of swale.

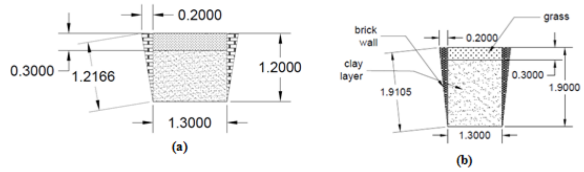

Figure 5. Cross section view of the proposed vegetated swale.

Table 6. Summary of swale dimensions for water draining on left side of the road

Swale ID	Cumulative discharge (m^3)	Channel slope	Channel depth, y (m)	Channel sides, my (m)	Channel side depth (m)
S1	0.17	0.00027	0.407	0.041	0.41
S2	0.73	0.0011	0.612	0.061	0.61
S3	1.06	0.0022	0.624	0.062	0.63
S4	1.29	0.0016	0.76	0.076	0.76
S5	1.47	0.0011	0.88	0.088	0.88
S6	1.72	0.0006	1.19	0.12	1.19
S7	2.11	0.0012	1.1	0.11	1.11

Table 7. Summary of swale dimensions for water draining on right side of the road

Area ID	Cumulative discharge (m^3/s)	Channel slope (m/m)	Channel depth, y (m)	Channel sides,	Channel side depth (m)
S8	0.14	0.00022	0.37	0.037	0.37
S9	0.66	0.00057	0.69	0.069	0.70
S10	1.05	0.00061	0.89	0.089	0.90
S11	1.56	0.00051	1.18	0.12	1.19
S12	1.90	0.00052	1.31	0.13	1.32
S13	1.97	0.00033	1.51	0.15	1.52
S14	2.38	0.00068	1.37	0.13	1.38
S15	2.48	0.00027	1.83	0.18	1.84
S16	2.72	0.00113	1.29	0.13	1.30
S17	2.86	0.00053	1.64	0.16	1.65

6. Comparison of the Alternatives

The project considers three alternatives which are:

Table 8. Cost summary for the proposed vegetated swale on the left side of the road

Swale ID	Cumulative discharge (m3)	Channel slope	Channel length (m)	Capital cost (Rs. Million)	Annual O&M cost (Rs. Million)	Total cost (Rs. Million)
S1	0.17	0.00027	90	0.56	0.012	0.57
S2	0.73	0.0011	321	2.01	0.042	2.05
S3	1.06	0.0022	654	4.08	0.085	4.17
S4	1.29	0.0016	482	3.01	0.063	3.07
S5	1.47	0.0011	327	2.04	0.043	2.08
S6	1.72	0.0006	196	1.22	0.025	1.25
S7	2.11	0.0012	1.1	0.11	1.11	

Table 9. Cost summary for the proposed vegetated swale on the right side of the road

Area ID	Cumulative discharge (m3/s)	Channel slope (m/m)	Channel length (m)	Capital cost (Rs. Million)	Annual O&M cost (Rs. Million)	Total cost (Rs. Million)
S8	0.14	0.00022	136	0.85	0.02	0.87
S9	0.66	0.00057	353	2.20	0.05	2.25
S10	1.05	0.00061	375	2.34	0.05	2.39
S11	1.56	0.00051	316	1.97	0.04	2.01
S12	1.90	0.00052	322	2.01	0.04	2.05
S13	1.97	0.00033	206	1.29	0.03	1.32
S14	2.38	0.00068	423	2.64	0.05	2.69
S15	2.48	0.00027	167	1.04	0.02	1.06
S16	2.72	0.00113	699	4.36	0.09	4.45
S17	2.86	0.00053	331	2.07	0.04	2.11

Alternative 1 (A1) is the storm water management through the construction of underground circular storm sewer; Alternative 2 (A2) is the expansion of the existing storm sewer network; Alternative 3 (A3) is the storm water management through the construction of vegetated swales. For the project the three alternatives were compared economically, and the alternative which gives the least cost will be the economically best alternative.

6.1 Economic Analysis of the Alternatives

For economic comparison of the alternatives, all the costs were projected for a period of thirty years which is usually the life period of a storm sewer network. The discounting rate was taken to be 10%. The total cost of a project will be the sum of capital cost and the annual operation and maintenance cost. Table 10 shows the cost comparison for the three alternatives. From results tabulated in Table 10, it is apparent that A2 i.e., the expansion of existing storm network is the most economical option followed by A3 and A1. However, when selecting a best storm water management option, economy is not the only criteria to be considered. For instance, although the total cost of a circular storm sewer is quite high, it requires relatively very less operation and maintenance. Also, once laid it will not interfere with any future expansion of road or roadside developments. The storm sewer can also be used for draining domestic sewage, especially during dry weather periods. Alternative 2 although economically better option, will need regular operation and maintenance. Also since the sewer is laid above the surface it is prone to damage and misuse more frequently. It can also interfere with the road expansion and future roadside developments. Alternative 3, i.e., vegetated swales are the latest development in storm water management. The swales act as natural filters such that water infiltrating from a swale into underground drain will be of a better quality. However, the swales need a large open space and are in general suitable only for carrying relatively small discharges. Apart from that they have the disadvantages similar to a surface storm sewer system.

Table 10. Economic comparison of the alternatives

Cost component	Alternative 1	Alternative 2	Alternative 3
Capital cost (Rs. Million)	140	11.60	30.83
Annual O&M cost (Rs. Million)		0.741	0.63
Total cost projected to 30 years (Rs. Million)	140	25.28	42.44

7. Conclusion

To summarize the project work, a catchment area located in the Vellore town was selected for design of urban storm sewer system. In order to select the best option for the storm water management, three alternatives were considered, namely: underground circular storm sewer network, repair and expansion of the existing surface storm sewer system, and vegetated swales. The alternatives were compared economically and it was found that alternative AII, i.e. repair and expansion of existing sewer system was found to be most economical. However, before finalizing on AII it is important to consider other aspects of a storm sewer system, like regular O and M requirements, space availability, long term usability, aesthetics and environmental issues.

8. References

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