DETENTION TANKS AS A MEANS OF URBAN STORM WATER MANAGEMENT

Gashaw Yayehyirad
Department of Civil Engineering
Addis Ababa University

ABSTRACT

Urban storm sewers, be it for separate or combined systems, are designed (dimensioned) for a specific rainfall intensity, duration and frequency. But, during the life of these sewers, rainfall of different intensities, duration and frequency can occur. Besides new settlements may develop. All these result in exceeding the design discharges of the sewers causing flooding.

Thus, in this paper, some means af reducing such risks are indicated. In particular, the application and design principles of storm water detention tanks is presented. The general principles af dimensioning sewers is also described. In order to elaborate and for better understanding, a simplified illustrative example of dimensioning a storm water detention tank is worked out.

APPLICATION AND DESIGN PRINCIPLES

Storm water collectors, for a certain sewerage system, are designed for a given intensity of rainfall. The design discharge is, therefore, based on an intensity of a certain duration, which can be exceeded per given period of time [4].

There are different methods of determining the amount of storm runoff, on the basis of which sewers can be designed. Out of the many and beyond certain packages, the method based on the time coefficient is mostly used. This method is selected as it can be easily computed using tabular procedures using a spreadsheet as well.

In the method, the amount of storm runoff coming at an inlet point of the sewer is calculated. For every inlet point, the respective rainfall and its period are fixed and, based on the catchment characteristics and length, the flow time is determined. The design value will be then the maximum amount of runoff computed.

The amount of runoff at the inlet point can be computed by [1]:

$$Q_R = \varphi_{x,n=k} * \sum_{1}^{m} \left\{ \frac{r_{i,n=k}}{\varphi_{i,n=k}} * \psi * AE \right\} \qquad (1)$$

where: Q_R = Runoff [l/sec]
φ = time factor taking care of duration. The value of φ, based on the value of the 15 min. rain and frequency of 1 ($r_{15,n=1}$) can be computed for other durations and frequency as:

$$\varphi = \frac{38}{T+9}\left(\frac{1}{\sqrt[4]{n}} - 0.369\right) \qquad (2)$$

x = longest flow time or time of concentration. Time of concentration is the time required for the maximum runoff to develop [4]. For one partial catchment area, this is the time from the farthest point to the inlet point at the outlet of the particular area. However, for more partial catchment areas, this can include the runoff time plus the flow time within the sewer upto the point under consideration.
n = frequency of occurrence
T = duration of rain dependent on the rainfall-runoff relationship. As long as the flow time $x \leq T$, then $\varphi_{i,n=k} = 1$.

(Example: $r_{15,n=1}$, = 60 l/s, ha means that a rainfall of duration 15 min having a depth of 60 l/s, ha occurs ones a year.)

or

φ can be read from the graph of fig. 1

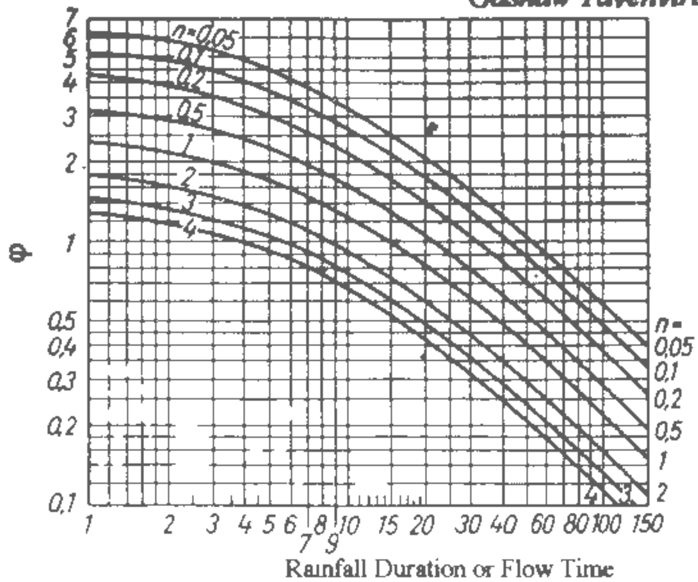


Figure 1 Time factor curve based on $r_{15,n=1}$ [1]

flow time = time of concentration
T* for calculating *r
***t* for calculating runoff volume**

ψ = coefficient of discharge dependent on duration of rain, frequency of occurrence and runoff reduction due to other factors related to the area, (infiltration, slope, etc) temperature, etc. It is mainly dependent on the percent imperviousness of the area from which the runoff originates.

The value used for ψ, actually, differs for designing sewers and detention tanks. That is, for sewers the peak value, ψ_P, and for detention tanks the total, ψ_{total}, is used, which are computed as follows:

$$\psi_P = \frac{\text{Max } q}{\text{Max } r} \tag{3}$$

and

$$\psi_{total} = \frac{\int_{t_b}^{t_e} q \, dt}{\int_0^T r \, dt} \tag{4}$$

q = volume of runoff [l/sec, ha]
r = $166.7i = 166.7i$ [l/sec, ha]
t_b = time at the start of runoff [min]
t_e = time at which runoff ends [min]
T = duration of rain [min]
I = N/T = rainfall intensity [mm/min]
N = Rainfall depth [mm]

The coefficient of discharge for various surfaces with different % of imperviousness have been established and tabulated. One of these is the one made by Kuichling [4], is dealing with similar types of surfaces.

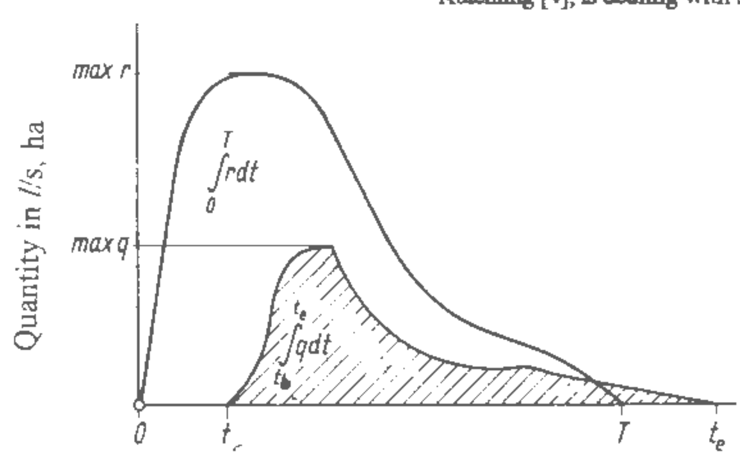
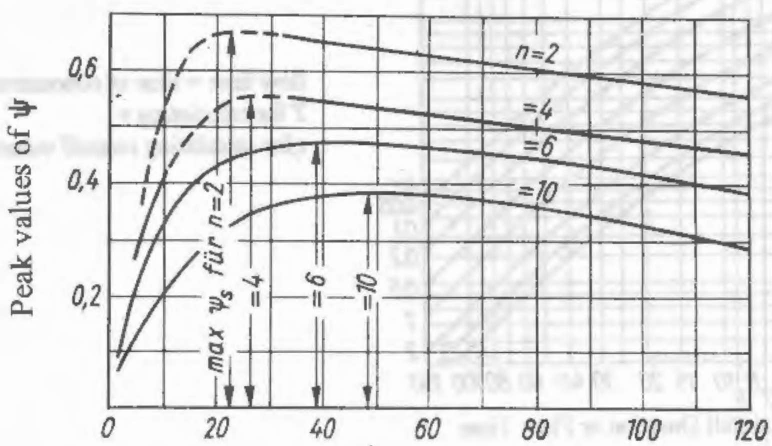


Figure 2 Sample demonstration of q and r versus time [1]

For determining the peak factor, one can use fig. 3



Duration of Rain T in min

Figure 3 Peak values of the runoff coefficient for various durations and frequencies of rain [1]

A_g = partial catchment Area [ha]
m = number of catchment Areas

But, still, due to the great variability of the rainfall-runoff process resulting in long periods a very low or zero discharge and relatively short periods of very large discharge, overflows resulting flooding can take place, resulting damages.

If such incidents are, in particular, in combined systems, the overflowing sewage can cause disastrous effects.

Besides flooding and the various sanitation problems, it can paralyze urban functions such as traffic and information networks.

A study made for Tokyo City, where the intensity used for designing sewers was 50 mm/hr and a runoff coefficient of 0.5, indicated a flood damage as indicated in Table 1.

But, all the above problems can be better handled by providing different storm water releaving or detention systems, within the network, for rainfall intensities greater than the design value and incorporation of new settlements.

One of the mechanisms or systems to relieve temporary effects of storm waters, exceeding the design value, is the provision of storm water detention tanks, at appropriate locations of the network. Such a detention tank is a practical method of storm water management.

In particular, these detention tanks are constructed at locations, where the trunk sewer of the newly developed area is to be connected to an existing trunk sewer or in case of rehabilitation works, at those particular points, where rehabilitation is required.

The detention tank is more advantageous, compared to other systems, like the overflow by pass, when the location is in the city area, where it should not be exposed and there is no natural system for receiving excess flows. Since they are also buried underground, they can be covered with appropriate material and be used on top, as tenn's court, for example.

The detention tank can be located along the line, for new networks, or on the side of the sewer supplied by an overflow wier as shown in Figure 4.

Such storage tanks, attenuate the peak-runoff rate and permit rational reduction in the capacity of the downstream sewer system.

Table 1: Flood Damage in Tokyo [3]

Date (Yr/Mo)	Rainfall intensity (mm/hr)	Total Rainfall (mm)	Number of Households	Flooded Roads (No. of points)
1981/07	80	86	10407	85
10	51	204	41498	48
82/09	58	255	20459	27
11	49	90	5174	51
83/06	50	50	9722	31
85/07	91	102	8506	0
86/08	58	264	6129	3
87/07	73	81	3546	29
07	60	60	2197	46
08	58	61	551	147
88/08	51	211	152	41
89/08	70	280	3620	170
08	92	93	594	61
90/08	39	51	60	60

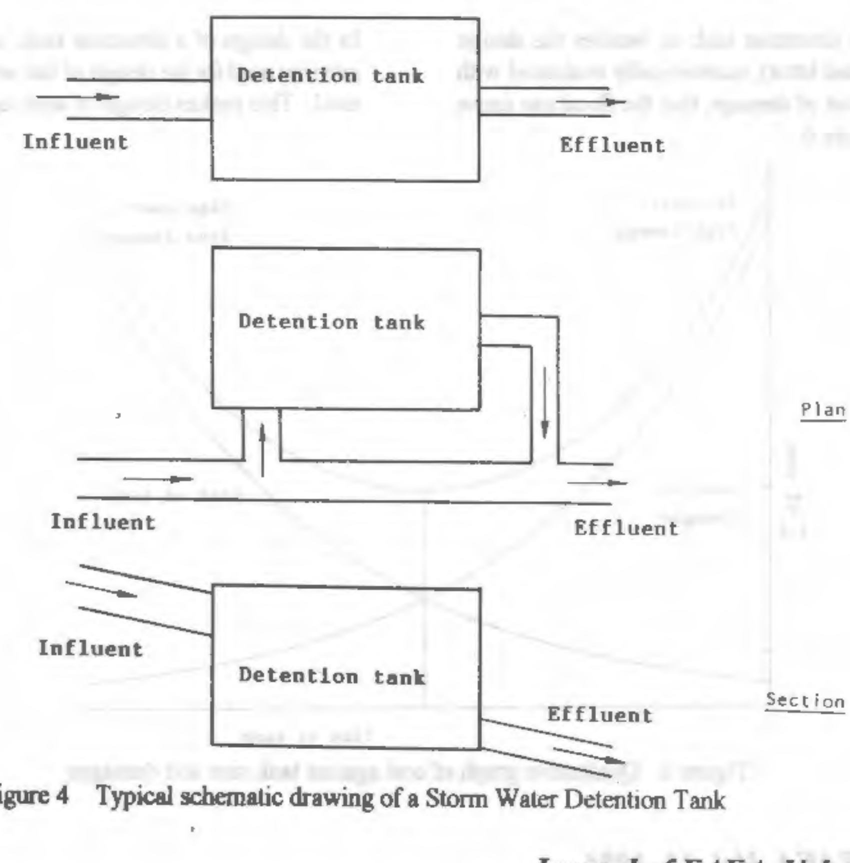


Figure 4 Typical schematic drawing of a Storm Water Detention Tank

The final decision on the application of a sewer and/or a detention tank is mostly made using economic analysis as demonstrated in figurs 5 and 6.

DESIGN

In order to determine the size of the tank required, proper design has to be made.

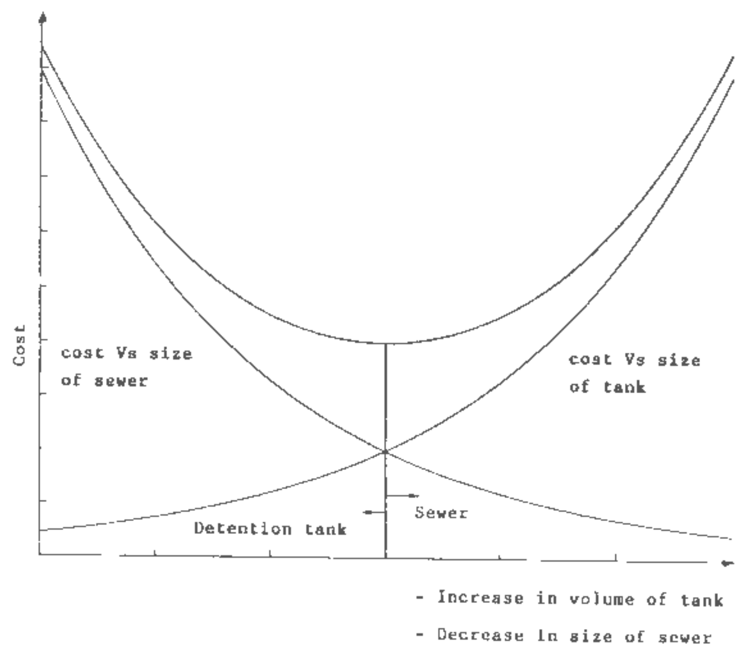


Figure 5 Qualitative graph of cost against size of detention tank and sewer

The size of the detention tank is, besides the design value (as presented latter), economically evaluated with respect to the cost of damage, that the flood can cause as shown in figure 6.

In the design of a detention tank, usually, the rainfall intensity used for the design of the sewer network is not used. This makes design of such tanks more difficult.

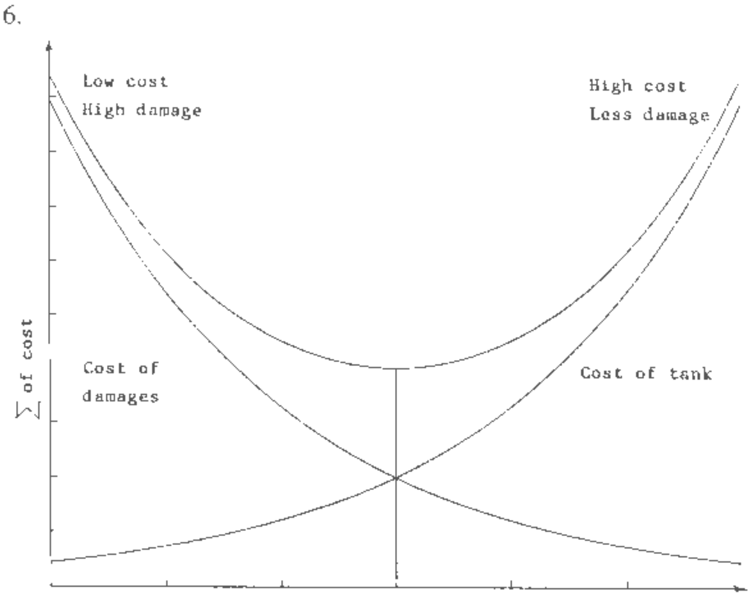


Figure 6 Qualitative graph of cost against tank size and damages

The maximum capacity of such tanks can be determined by different iteration technics. But the most widely used method is the mass balance, which can be established for different rains, with different periods and frequency [2]. From these, an estimate of the inflow into the detention tank is made and the maximum rate of outflow is set. The outflow is, actually, equal or less than the maximum capacity or flowrate of the sewer corresponding to the design storm of the sewer downstream.

In determining the size of the detention tank, the total volume of runoff of the catchment is calculated first. This volume is calculated by:

$$Q_{inf} = \varphi_{x,n=y} \sum_{1}^{m} \left\{ \frac{r_{i,n=k}}{\varphi_{i,n=k}} * \psi * A_E \right\} \qquad (5)$$

The difference between eqn. 5 and eqn. 1 is in the values that are used for x and y.

In eqn. 1, as it is used to design the sewer and that the sewer capacity can be exceeded, the longest possible flow time is used as the value for x. But, on the other hand, as detention tanks are not meant to be flooded, the, value of x is taken to be equal to the minimum value, which is usually, the duration of rain. The value of y for the determination of sewer sizes is taken to be equal to k. But, based on the fact, that runoffs of equal duration but difference frequencies differ in magnitude and that less frequent rains with the same duration produce more discharge, the value of y for determining size of detention tanks is taken to be the minimum duration ($y \leq k$), to avoid risk of over flooding.

The max. effluent volume from the detention tank, Q_{eff}, is equivalent to the max. capacity of the sewer, downstream of the tank. This effluent is not constant, as the head of water with in the tank is variable. Thus, it is maximum, when the tank is full and minimum when the tank begins to fill. To make the design simpler and more practical, the average of the two values is used. Hence,

$$Q_{eff} = \frac{1}{2} \left\{ min\ Q_{eff} + max\ Q_{eff} \right\} \qquad (6)$$

In order to determine the required volume of the detention tank a certain dimensionless parameter, based on the ratio of the effluent to the influent and the flow time is used. That is

$$\eta = \frac{Q_{eff}}{Q_{inf}} \qquad (7)$$

where η = dimensionless parameter
Q_{eff} = effluent [l/sec]
Q_{inf} = influent [l/sec]

Assuming a design value of $r = r_{15,\, n=1}$ for the influent, which is most of the time the case, a design chart is established from which the dimensionless design parameter, DP, can be read (fig. 7).

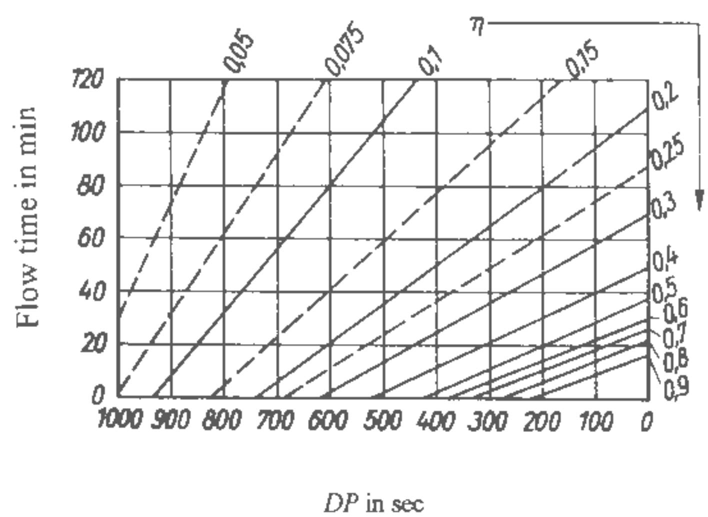


Figure 7 Design Parameter based on η and flow time [1]

Hence, once the influent, the effluent and the flow time (time of concentration) are known, one can read the design parameter *DP* from the chart and compute the required volume of the detention tank as:

$$V_T = \frac{DP * Q_{inf}}{1000} \ [m^3] \qquad (8)$$

where: V_T = volume of detention tank [m^3]

Q_{inf} = influent flow [l/sec]

DP = design parameter established based on the flow time and the ration (η) of the effluent from the tank to the influent. The value of this parameter has been plotted for various values of flowtime and η.

ILLUSTRATIVE EXAMPLE

A certain city *x* has an existing storm sewer system, whose trunk sewer has an extra design capacity of 300 *l*/s. Now a new settlement of 40 hectares has been developed which will be incorporated into the sewerage system. This new area has an average runoff coefficient of 0.30 and flow time of 30 min. to the inlet point.

Check if the existing system is sufficient to take care of the new settlement, if rainfall intensity N/T = 0.6 mm/min, duration = 15 min and

a) $n = 1.0$

b) $n = 0.2$

SOLUTION

Checking for the sewer, first:

$$Q_R = \varphi_{x,\, n=k} \sum_1^m \left\{ \frac{r_{i,\, n=k}}{\varphi_{i,\, n=k}} * \psi * AE \right\} \quad \text{---Eqn. 1}$$

$$\varphi_{30,\, n=0.2} = \frac{38}{30+9} \left(\frac{1}{\sqrt[4]{0.2}} - 0.369 \right) \quad \text{--- Eqn. 2}$$

$$= \frac{38}{39} (1.495 - 0.369) = 1.097$$

$$r = 166.7 * i = 166.7 * 0.6 = 100 \ l/s, ha$$

$$r_{i=15,\, n=1} = \varphi_{15,\, n=1} * r = 1 * 100$$

$$= 100 \ l/s, ha$$

$$\varphi_{15,\, n=0.2} = \frac{38}{24} \left(\frac{1}{\sqrt[4]{0.2}} - 0.369 \right)$$

$$= 1.783$$

$$\therefore \ \varphi_{inf} = 1.097 * \frac{100}{1.783} * 0.3 * 40$$

$$= 738.46 \ l/sec$$

amount that can be used for dimensioning the sewer but Q_R > effluent capacity of the existing sewer.

$\therefore$ a detention tank is required

Hence, inorder to compute the volume of the detention tank eqn. 5 will be used with $y = 1$ and 0.2 i.e.,

$$Q_{inf} = \varphi_{x,\, n=y} \sum_1^m \left\{ \frac{r_{i,\, n=k}}{\varphi_{i,\, n=k}} * \psi * A_E \right\} \quad \text{---Eqn. 5}$$

Case a) $y = 1$

$$Q_{inf} = Q_{(r_{15,\, n=1})}$$

$$= \varphi_{15,\, n=1} \sum \frac{r_{15,\, n=1}}{\varphi_{15,\, n=1}} * \psi . A_E$$

$$= \varphi_{15,\, n=1} = \frac{38}{24} \left(\frac{1}{\sqrt[4]{1}} - 0.369 \right) \approx 1$$

$$\therefore \ Q_{inf} = 1 * \frac{100}{1} * 0.3 * 40$$

$$= 1200 \ l/sec$$

$$\eta = Q_{eff} / Q_{inf} \quad \text{--- Eqn. 7}$$

$$\eta = \frac{300}{1200} = 0.25$$

from fig. 7, $DP = 450$ for $t_f = 30$ min

$$V_T = DP \, \frac{Q_{inf}}{1000} \quad \text{--- Eqn. 8}$$

$$\therefore \; V_T = 450 * \frac{1200}{1000} = 540 \text{ m}^3$$

Case b) $y = 0.2$

$$Q_{inf} = Q_{(T_{15,n=0.2})}$$

$$= \varphi_{15,n=0.2} \sum \frac{r_{15,n=1.0}}{\varphi_{15,n=1.0}} * \psi * A_E$$

$$= 1.783 * \frac{100}{10} * 0.3 * 40$$

$$= 2140 \; l/sec$$

$$\therefore \; \eta = \frac{300}{2140} = 0.14$$

for $t_f = 30$ min, $DP = 700$ (fig. 7)

Hence,

$$V_T = 700 * \frac{2140}{1000}$$

$$= 1498 \approx 1500 \text{ m}^3$$

The relative size (length, width and depth) can be decided on the actual topography and other site conditions.

CONCLUSION

Due to urbanization, the size and number of towns and cities is growing day to day. This demands a rehabilitation of the existing or totally new infrastructure, like water supply, power supply, storm and wastewater management, etc.

A change or new development in the mode of settlement of a certain town will directly affect the volume of runoff that is generated due to rainfall in that catchment. Excessive size of sewers or augmenting the existing system with parallel lines may not be technically and/or economically feasible.

Thus, a method has been presented to reduce the effect of sewer overflows in urban storm water management. To use the method described one has only to carefully establish the runoff coefficient and $r_{15,n=1}$. Ones these parameters are set, the method can be used without any problem.

Eventhough one can really consider the variation of the runoff coefficient with respect to the intensity and duration of rain, the method assumes a constant value, which, for practical engineering designs, is good enough.

Otherwise for more refined estimation of Q_R and Q_{inf}, one can consider the variation of ψ with respect to intensity and duration and apply a factor, taking care of the variation. But, the author feels, that assuming a constant value is sufficient for design purposes, provided that the partial catchment area, under consideration, is not very large.

REFERENCES

[1] Bischof, Wolfgang
Abwassertechnik
9., neubearbeitete und erweiterte Auflage
B.G. Teubner, Stuttgart, 1989

[2] Fair, G.M.; Geyer, J.C.; Okun, D.A.
Elements of Water Supply and Wastewater Disposal
John Wiley and Son's, Inc, 1971

[3] Higuchi, Kazuyuki; Maeda, Masahiro; and Shintani, Yasuyuki
Sewer System for Improving Flood Control in Tokyo
Water Science and Technology, Vol. 29
Number 1-2, 1994, pp 303-310

[4] Steel, Ernest W.
Water Supply and Sewerage
McGraw-Hill Book Company, 1960

[5] Stephenson D., Stormwater Hydrology and Drainage
Elsevier Scientific Publishing Company, 1981