EXPERIMENTAL ESTIMATION OF DETENTION IN STORM SEWER SYSTEM

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1. Introduction

The essential object of sewer systems has been the improvement of local environments by removing sewage and storm water rapidly from the district. Sewer system, however, is being signified more positively as an element of the wider system including receiving public streams, where quality pollution is taking general attention. The design criterion of sewer system should, therefore, be changed to realize the more reasonable and more economical system concerning with water quality and quantity in addition to the problem of selection of separate system or combined.

The conventional formulas for storm runoff calculation, the rational method and empirical method, deal with only the maximum rate of runoff at the downstream end of any sewer reach. Runoff hydrographs may be very important to design and control the sewer system, especially they are indispensable with respect to dynamic control of water quality. The storm runoff hydrographs should be based on sufficient investigation of the flow detention, retardation, storage and flooding in sewer, which can not be estimated in detail by the conventional formulas. The detention formula[1] formulated the maximum rate of runoff introducing a concept for these phenomena. This formula seems to be generally utilized in actual design, because it results in the moderate design flow between the values of the two methods above mentioned.

Laboratory experiments were performed for the investigation of these phenomena and for making a step toward a model of runoff mechanism.

In this paper we will define the concept of these phenomena and will point out appricable limits of several existing methods from the experiments. The difference between the detention phenomena in the gentle slope area and the steep slope area are analized and we, then, make clear that the return period for design storm will be discussed in relation to the occurrence of flooding.

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2. Detention, Retardation, Storage and Flooding[2]

"Storage" will include both of the static storage such as the depression storage and the dynamic storage as change of water depth profile with respect to time. When the rate of runoff exceeds the dynamic storage capacity, flooding will occur in the drainage area and sewer conduit. On the other hand, "retardation" will represent the phenomenon when the flow down period is longer than the rainfall duration if the average rainfall intensity is the basis of calculation. The flow down period, however, may be influenced by the characters of drainage area and sewer conduit even if the rainfall is with the same duration time. Accordingly, the retardation, storage and flooding are said to be the phenomenon which is represented with the detention in a wide sense.

There are several methods to estimate the detention behavior; estimation of retardation width in the drainage area[3], the storage fuction[4] and the method introducing a new coefficient[1]. Any of these methods still needs intensive runoff survey in actual drainage area in order to obtain better agreement.

3. Discussion of Detention Formula

The detention formula was proposed by Dr. Itakura in 1955 for the revision of the conventional rational method that had a tendency to result in the excess amount of storm runoff compared with the actual one. As he considered, it may be the fact that the maximum rate of runoff calculated by the rational method is naturally depressed in many cases caused by the storm flow detention, and therefore, the detention capacity should be positively utilized for storm runoff calculation. The contents of his paper were the depression of drainage pump capacity and the reduction of sewer conduit section introducing the detention coefficient for an apparent flow down period.

3-1. Theoretical Studies of the Detention Formula

When a storm sewer is designed by the conventional formula of rational method, the duration of design storm at any point along the sewer is shown in **Fig. 1** (**a**) as the sum of inlet time and flow down period, where no inlet flow through the upstream end of sewer, constant flow velocity in sewer and a rectangular drainage area are assumed. The maximum rate of runoff in each section is then calculated by $Q=CiF$. In spite of gradually decrease of design storm intensity as in **Fig. 1** (**b**), the increase of the tributary drainage area will result in the $Q_{\max}$ and $A_{\max}$ distributions represented by the solid lines in **Figs. 1** (**c**) and (**d**), respectively. The sewer section $A_{\max}$, however, is not occupied by maximum flow at the same time. When $Q_{L,\max}$ occurs at the lower end with the uniform storm i_L throughout the whole area, the theoretical distribution of flow and the necessary sewer section change linearly as shown by the dotted lines in **Figs. 1** (**c**) and (**d**). Therefore, $[A_{\max}-A]$ that is the clearance of the designed sewer section will allow for the detention volume per a unit length. Applying the Talbot type formula of rainfall intensity, $i=a/(t+b)$, the total volume for possible detention V_S is calculated as follow;

$$\left.\begin{aligned} V_S&=\int_0^L (A_{\max}-A)dx=\frac{CaF_L}{T_L}\\ &\quad\times\int_0^{T_L}\left(\frac{t}{t+b}-\frac{t}{T_L+b}\right)dt,\\ &=CaF_L\left[1-\frac{b}{T_L}\log_e\frac{T_L+b}{b}-\frac{T_L}{2(T_L+b)}\right] \end{aligned}\right\} \quad \cdots\cdots(3.1)$$

where C is the runoff coefficient, F_L is the total area of drainage and the inlet time is ignored to express $F=F_Lt/T_L$.

Dr. Itakura considered that the rate of maximum runoff could be depressed to the limit of drainage pump capasity at the downstream end if the detention volume V_S was taken into account to be efficiently utilized. The depression rate X as in Eq. (3.2) was, therefore, calculated with the assumption that no flooding could occur in the lower area even if V_S was fully occupied by the excess runoff.

$$\left.\begin{aligned} V_S&=X^2T_LQ_{L,\max}\\ X&=\sqrt{\frac{T_L+b}{T_L}\left[1-\frac{b}{T_L}\log_e\frac{T_L+b}{b}-\frac{T_L}{2(T_L+b)}\right]}. \end{aligned}\right\} \quad \cdots\cdots(3.2)$$

For the convenience of understanding, 1.0 for the runoff coefficient C and no inlet time are assumed. Accordingly, rainfall with the volumetric intensity of $Q_{L,\max}(=iF)$ concentrates to the sewer of length L within the duration T_L as is represented by EAFO in **Fig. 2** (**a**). Following to the usual idea of rational method, the rainfall mass of OAE in **Fig. 2** (**a**) will occupy (or correspond to) the sectional area

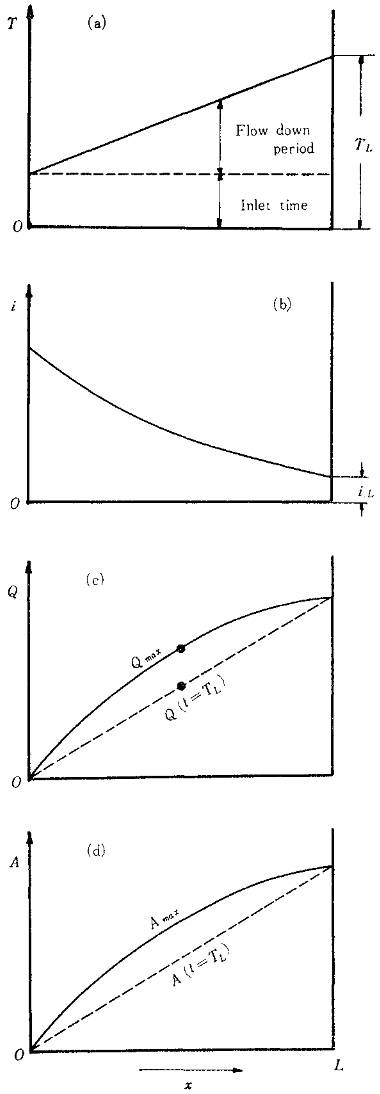

Fig. 1 Storage capacity in sewer by the detention formula.

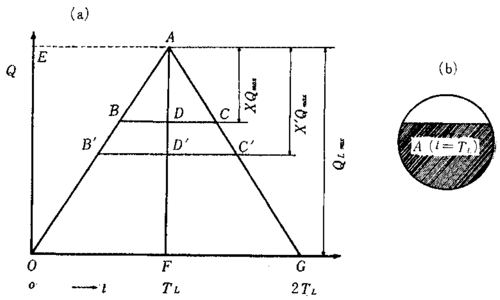

Fig. 2 Depression of maximum rate of runoff by drainage pump.

$A(t=T_L)$ in **Fig. 2 (b)** which can be calculated by $\frac{CaF_L}{T_L}\int_0^{T_L}\frac{t}{T_L+b}dt$. Immediately after the duration T_L, there is no inlet flow to the sewer. The discharge volume AFG in the period of T_L to $2\,T_L$ is the transformation of OAE ever resided in $A(t=T_L)$. Therefore, the storage capacity in the sewer becomes maximum at the time T_L and then it begins to decrease. For this reason, the geometrical space which was provided in the sewer to store the half volume ADC of the excess runoff will remain without being used. Consequently, if the excess space can be used in 100% storage as the assumption by Dr. Itakura, V_S in Eq. (3.1) should be equal to $AB'D'$, resulting in a new depression rate X' larger than X;

$$\left.\begin{aligned}V_S&=\frac{1}{2}X'^2T_LQ_{L,\max},\\ X'&=\sqrt{2}X.\end{aligned}\right\}\quad\cdots\cdots\cdots\cdots\cdots\cdots(3.3)$$

The design pump capacity can also be depressed as low as $(1-X')Q_{L,\max}$. In this case, however, a doubt concerning to discharge after $t=T_L$ may occur. The rational formula only shows that $Q_{L,\max}$ is the possible maximum rate at $t=T_L$ and does not show it must be completely drained. As far as the assumption of no flooding along the sewer before becoming full flow is accepted, it is possible to reduce the discharge after $t=T_L$, less than $(1-X')Q_{L,\max}$ by more compression of pump operation. Of course, the drainage must be finished before the next storm peak comes to appear.

Fig. 3 shows the variation of X or X' with a parameter of Talbot's constant b. In spite of utilizing only half of storage capacity, Eq. (3.2) tends to give excessive value to X for smaller b. For this reason, the design pump capacity does not always seem in safety side. When the full storage capacity is utilized as above, this tendency becomes greater. If b is equal to zero, the designed sewer section becomes uniform which has just the equivalent volume to the amount of rainfall. The storm needs not, therefore, theoretically be discharged, or in another words, all rain water can be stored in the sewer. The rational formula upon which the present consideration is based has no regard with naturally occurring retardation. The actual discharge hydrograph can be damped lower than the one in **Fig. 2 (a)**. The storage capacity which has been ascribed to the flow out depression by drainage pump actually includes the natural detention volume. Then the contribution of pump becomes less or there would be the case that the discharge could not be depressed by pump. From this, the relation between the discharge and the detention capacity should be investigated hydraulically even in gentle slope area.

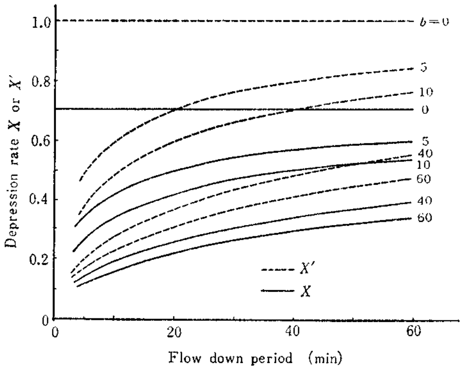

Fig. 3 Depression rate vs. flow down period with the parameter of Talbot's constant b.

Dr. Itakura extended the above concept of flow detention directly to the usual storm sewer design without drainage pump. When the time of concentration T_L and hereafter the rainfall intensity i' were selected by the usual manner of rational method, he supposed the possibility to decrease the sewer section as to satisfy the runoff from the lower rainfall i of duration αT_L. The detention coefficient $\alpha(\alpha>1)$ was determined letting the total storage volume in sewer be arithmetically zero even when the maximum runoff from i' in T_L was loaded on the sewer;

$$V_S=\frac{CaF_L}{T_L}\int_0^{\alpha T_L}\left(\frac{t}{\alpha t+b}-\frac{t}{T_L+b}\right)dt=0,\quad\cdots\cdots\cdots(3.4)$$

$$\therefore\ \alpha^2-\frac{b}{T_L}\log_e\frac{\alpha^2T_L+b}{b}-\frac{\alpha^4T_L}{2(T_L+b)}=0.\quad\cdots\cdots\cdots(3.5)$$

It has been commonly accepted for ease of understanding that the maximum rate of runoff will actually occur with the storm intensity i for the period αT, while the conventional rational formula is still used for calculation. However, even if a pump is constructed at the downstream end of sewer designed with duration time αT, it is doubtful whether the runoff can be drained without flooding in the drainage area as the hydraulic significance of Eq. (3.5) differs from that of Eq. (3.2). Moreover, the original form of Eq. (3.4) is $\int_0^L(Q_{\max}-Q)dx/U$, where U is the flow down velocity, and if the rainfall i' of duration time T_L occurs actually, the value of $(Q_{\max}-Q)dx/U$ becomes negative at the downstream reach of sewer because the sewer becomes full at $x=L$ before $t=T_L$. In such a case, the flow mechanism naturally changes and storm water from the lower area cannot always flow freely into sewer. It is not reasonable that the more complicated phenomena than the one represented by Eq. (3.1) is geometrically analyzed based upon only the consideration of flow profile at the instant of $t=\alpha T_L$.

Now, in the practical calculation of actual serial sewers, it is regarded that the excess flow is stored in upstream side as in **Fig. 4 (a)** if the detention formula is applyed for the entire length L, but the flow and storage pattern becomes as in **Fig. 4 (b)** if L is divided into several reaches. These are evidently inconsistent. If the latter case is concerned, the detention coefficient α may be different in each reach, and therefore, the apparent storage capacity also differs from the former. It may become also evident from the experiment explained in the next section that Eq. (3.5) can not be applied at steep slope area because the inner space of sewer is not utilized as expected storage capacity.

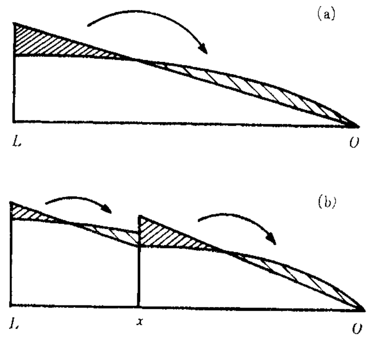

Fig. 4 Apparent replacement and storage of storm water.

It is recommended in application of the detention formula to take the average value of local velocity with the same rainfall. While, the average of local maximum which corresponds to the each design rainfall is used generally. Practical comparison is made between Q_{max} and Q of the middle point of sewer as in **Fig. 1 (c)** and about thirty per cent reduction of flow down velocity is supposed from that of the usual rational method. Storm runoff, however, is a kind of flood waves. Therefore, maximum runoff flows down with the maximum velocity in each section which can be also confirmed with the hydraulic study of flow down mechanism by the one of the present authors[3]. The average intensity i including the peak of hyetograph is a simplified model of rainfall which exists only at a particular point of flow down period T. Accordingly, it may not be correct to calculate a velocity with an average rainfall intensity for the entire sewer length.

3-2. Discussion on the Detention Formula by the Experimental Data

The basis of the drainage pump capacity depression is the assumption that the maximum volume of dynamic storage appears as the static storage in sewer at an appropriate time. The experiment herein performed, the detail of which will be explained in the next chapter, was also to confirm the validity of this basis. As the flow rate of runoff in the downstream end of the experimental sewer was controlled with a valve, the water surface rised almost uniformly in the sewer, as the runoff rate increased. If the runoff is controlled with a pump capacity, the flow condition may be slightly changed. But it must be noted that the valve control is more advantageous than the control with the pump for the utility of storage capacity. **Fig. 7** shows the relationship between time and detention capacity utilized in the case of sewer slope of 1/200, and **Fig. 9** the case of 1/1 000, both of these are with the water depth and opening ratio of the valve as the parameters. The "full flow line" which shows the sewer becomes full at the downstream end is nearly equal to the line of the total sewer capacity ($50.6\times10^3\text{cm}^3$) in the case of 1/1 000. This fact indicates that the dynamic storage is nearly equal to the possible static storage in this case and that the drainage pump capacity can be depressed as expected by the detention formula. On the other hand, the "full flow line" is lower than the sewer capacity line in the case of 1/200, and the depression of the drainage pump is not always possible, because the dynamic storage is less than the static capacity.

It is interesting to see in these figures that the conventional rational method with the Manning's roughness coefficient $n=0.016$ ($\text{m}^{-1/3}\text{sec}$) has the similar detention volume–time relationship with the case of actual sewer model (with the valve opened fully) except for the considerable difference between the maximum detention volumes finally attained. In **Fig. 9** for the slope 1/1 000, the full flow line is rising in left-ward and will have a intersection point B with the maximum storage capacity line. Accordingly, the difference between the possible maximum detention and the one from the rational method (the vertical distance of points B and C) is composed of the natural increase of detention up to the case of actual sewer without valve control (the vertical distance of A and C) and the artificially utilized detention volume with the reduction of valve opening (between B and A). Although a certain amount of natural increase of detention seems to be expectable compared with the rational method calculation by an assumed sewer roughness, in the case of steep slope sewer, the artificial detention is rather negative not allowing for the effective depression of drainage pump capacity without flooding because the full flow line has the opposite tendency of the above.

The detention coefficient α for the design of sewer

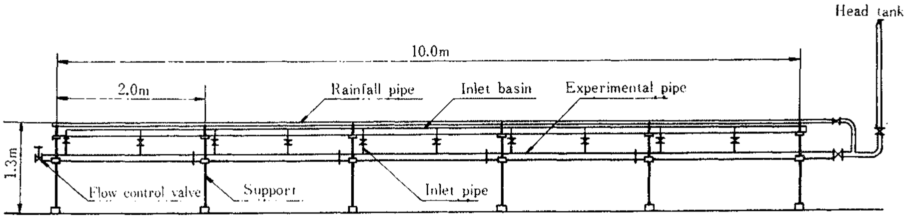

Fig. 5 Experimental equipments for sewer system.

conduits will be discussed in the next chapter as it is closely related to the conventional rational method.

In this chapter, the principle of detention formula was investigated. The concept of the method is not inconsistent, but the natural detention, which should be more or less analysed to revise the conventional rational method, is confused with the artificial by intended detention. At the present stage, this method may be available for the estimation of depression rate X for drainage pump capacity, provided that the limits for b in Talbot type rainfall intensity formula and the length of sewer are carefully selected. The design of sewer with the detention coefficient α still leaves several drawbacks so far as it is based on the conventional rational method, and the further investigation is necessary even if the gentle slope area is concerned.

4. Experimental Studies of the Detention Phenomena

4-1. Equipments and Method of Experiments

The experimental equipments are shown in **Fig. 5**. The polyvinyl chloride pipe (rainfall pipe) with 3.8 cm diameter and 13 m long, with 1.5 mm diameter openings at intervals of 6~7 cm, was provided for rainfall supply to 10 pieces of drainage area (inlet basin). The rainfall pipe was designed to yield uniform intensity, which was controlled by the valve at the upper end of this pipe. The rainfall can be turned to a side gutter so that the rectangular rainfalls are obtained with respect to time. The drainage area, each of which is 1.0 m long and 25 cm wide, connected with the experimental pipe as a sewer conduit through each inflow pipe. The experimental pipe is 7.8 cm diameter and 10.6 m long with the inflow pipes at intervals of 1.0 m. Measurement of water level at every 20 cm along the pipe was made by the manometers, the readings of which were taken from the series of photographs. A sluice valve was set at the downstream end of experimental pipe in order to control the runoff rate. Essentially, such a control would better be made by pump than valve. Many problems, however, would be occurred in this case; referring to drainage pump capacity, starting time of pump operation and operating period, etc. It was not necessarily operated by pump for the basic observation of general tendency of detention.

Rainfall intensity, opening ratio of flow control valve and slope of sewer conduit were changed as variables in this experiment. The rainfall supply was continued until the time when the flow became steady or the appropriate time after flooding occurred.

The rate of runoff was measured by a triangular weir. Though the weir essentially measures flows kept near the steady state, the runoff rate was calculated by the following formula.

$$Q_I = Q_O + ds/dt = f_n(h_w) + A_w \times dh_w/dt,$$

where Q_I is the inlet flow to the weir or runoff rate from the experimental pipe, Q_O the outlet flow from the weir, s the storage volume in the weir, $f_n(h_w)$ the function of the weir, A_w the surface area of the weir box, h_w the waterlevel in the weir. The rate of runoff can be calculated if the increase of the weir is measured. Measurement of level was made by the inclined manometer. The rate of runoff was measured at the upstream end and the downstream

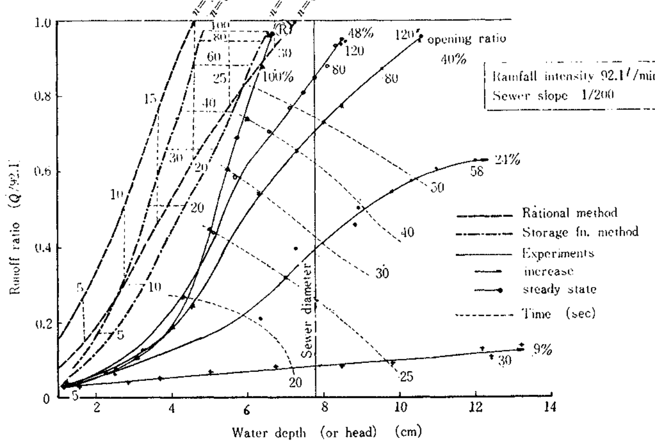

Fig. 6 Runoff rate vs. water depth or head with the parameter of flow valve's opening ratio in sewer slope 1/200.

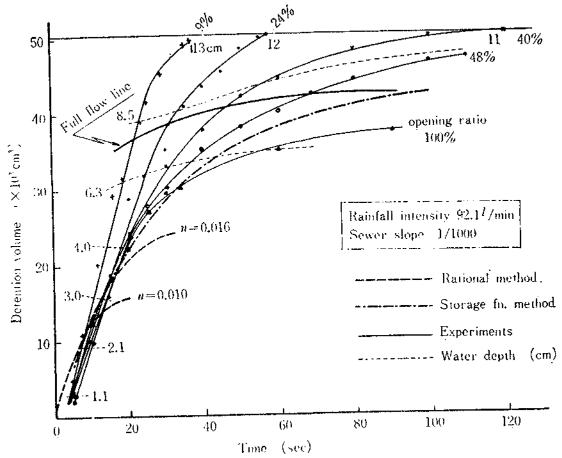

Fig. 7 Detention volume vs. time the with the parameter of flow valve's opening ratio in sewer slope 1/200.

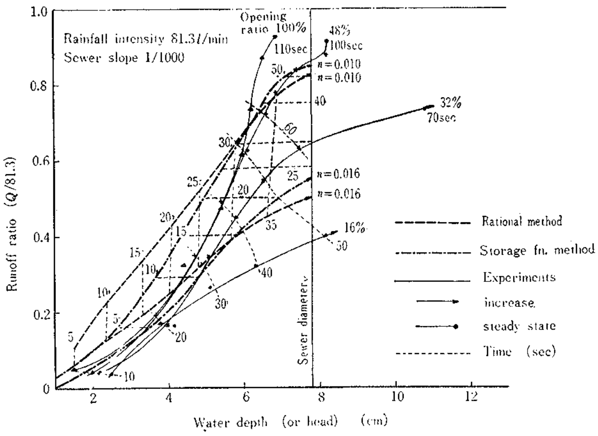

Fig. 8 Runoff rate vs. water depth or head with the parameter of flow valve's opening ratio in sewer slope 1/1 000.

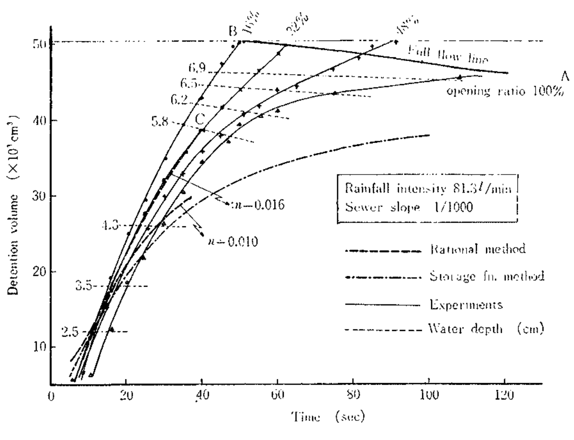

Fig. 9 Detentio volume vs. time with the parameter of flow valve's opening ratio in sewer slope 1/1 000.

end. In any intermediate point, however, measurement of flow rate could not be made.

4-2. Discussion on the Detention Phenomena by the Experimental Data

Slopes of the experimental sewer were 1/200 and 1/1 000 and volumetric rainfall intensities were 60 ~120 l/min. **Figs. 6** and **7**, and **Figs. 8** and **9** show the representative experimental results in the cases of slopes of 1/200, 1/1 000, respectively.

The similar analyses of depth-flow rate-detention volume relationships in arbitrary sewer section as illustrated in these figures can be made using the data obtained. This might be rather necessary on the detention phenomena accompanied with flooding which might possibly occur at any section other than the lowest point in sewer. In this case, however, the flow rate in the intermediate section includes certain error due to indirect measurements. Therefore, the present discussion will be concentrated upon the comparison of calculation formulas applied for the flow detention in the overall sewer system.

It has been clear that the volumetric intensity has influence on the phenomenal speed, but very little on the changes of water profile and the runoff rate. The volumetric intensity was selected so as to obtain the steady state with nearly same depth in each slope; 92.1 l/min in the slope of 1/200 and 81.3 l/min in 1/1 000.

Figs. 6 and **8** show the relationship between the ratio of runoff rate and depth with a parameter of the flow control valve's opening ratio. The pipe diameter is represented by the vertical straight line at depth 7.8 cm. **Figs. 7** and **9** show the relationship between time and the detention capacity. The total sewer capacity, that is the static storage capacity (50.6×10^3cm^3), is represented by the horizontal straight line. The points becoming full flow are shown by the "full flow line". Water depth represents the maximum along whole length of the pipe. The flooding begins to occur when the water head exceeds the pipe diameter. It is seen from these figures that the smaller the opening ratio of the valve is set, the more quickly the flow becomes full, but the less the rate of runoff.

The results of calculation using the conventional rational method are also shown in these figures for the two cases of roughness $n=0.016$, $n=0.010$, the former is evaluated so as to obtain the same condition for the maximum runoff and the rainfall intensity with the experimental, and the latter may be considered as the ordinary value for the test pipe. The triangle hydrographs are assumed to calculate the detention volume for the rational method. It is matter of cause that the runoff rate decrease and the detention increase are followed to increase of roughness. The relationships between time and water depth is not influenced by the roughness as a result. In the case of 1/200, the flow becomes steady at the point R. But, in the case of 1/1 000, the rational method does not inform steady state below the depth of pipe diameter, and the depth and runoff increase still more after the sewer becomes full. The designed sewer section may result in much bigger in this case.

The hydrographs using the storage function[4],

$$S=K[xI+(1-x)Q] \quad \cdots\cdots(4.1)$$

are also obtained for comparison, where S is the storage volume, K the coefficient having the dimension of time, x the dimensionless factor which defines the relative weights given to inflow and outflow in determining storage, I the rate of inflow, Q the rate of outflow. K and x are determined from the observed hydrograph and rainfall intensity; the results are shown in the **Table 1**. Theoretical hydrographs can be successively calculated by the next equations;

$$\left.\begin{aligned} &Q_2=C_0I_2+C_1I_1+C_2Q_1, \\ &C_0=-\frac{Kx-0.5\,\varDelta t}{K-Kx+0.5\,\varDelta t}, \\ &C_1=\frac{Kx+0.5\,\varDelta t}{K-Kx+0.5\,\varDelta t}, \\ &C_2=\frac{K-Kx-0.5\,\varDelta t}{K-Kx+0.5\,\varDelta t}, \\ &C_0+C_1+C_2=1 \end{aligned}\right\} \quad \cdots\cdots(4.2)$$

where suffix 1 in I or Q shows the biginning of any short time interval $\varDelta t$ and suffix 2 shows the end of the interval. Water depths are again calculated by Eq. (4.1) and roughness coefficient $n=0.010 \sim 0.016$. The relationship of the runoff rate versus time is unchanged for the given conditions of K, x, irrespective to the roughness. The water depth, however, is varied with roughness in this case.

Table 1. Coefficients, K and x, in storage function method.

slope of sewer	K (sec)	x
1/200	45	0.370
1/1 000	40	0.285

Let compare the rational method with the experiments. If the flow down velocity and roughness coefficient are reasonably predicted for every instant and every section, calculated hydrograph would coincide with the actual one. If a hydrograph is calculated with the flow down velocity at the time of the maximum runoff rate in the downstream end, design of sewer section is in safety side compared with the necessary one. Retardation would occur even if the same velocity with the actual is used because the retardation is the phenomenon occurring when the rainfall duration is shorter than the flow down period either in the gentle or the steep slope area. As mentioned formerly, if the actual phenomena are not observed for the velocity prediction in rational method, it is naturally said that the assumed velocity may happen to be different from the actual one. Therefore, it is generally possible to classify so called detention in two kinds as above mentioned; one is retardation which is inevitable so far as the idea of the average rainfall intensity is adopted and the other is the non-essential problems due to the assumption of constant velocity or the insufficient data of the field observation. Both of them can be generally defined as the detention described in Chapter **2**.

In **Fig. 6**, it is understood that the actual runoff (with the valve opened fully) at the first stage is behind the one calculated by the conventional rational method. In **Fig. 8**, the rate of runoff by the rational method is much different from the actual one. The correction of this difference is difficult even if the roughness is less assumed, say $n=0.010$. This fact indicates that the Manning's formula which is the equation of motion in the rational method is no longer established. The sewer diameter to discharge the runoff rate, 81.3 l/min, in this case requires at least 8.4 cm (with $n=0.016$) if the velocity is calculated conventionally by the Manning's formula. Therefore, the sewer diameter will be over-designed than the actually required by the rational method used in the gentle slope. The only exception may be possible to represent the actual depth-runoff relation by the Manning's formula with very small value of n, but it should be remembered that the flow down and therefore detention are distorted far from the actual phenomena. If the present experimental sewer has been designed by the conventional rational method ($n=0.010$), the maximum capacity is to be for the rainfall intensity of only 67.7 l/min; nevertheless 81.3 l/min is completely drained indicating around 16.7% depression of runoff by the natural detention above mentioned.

Applying the detention formula to the cases of **Figs. 6** and **8**, the detention coefficient α can be evaluated experimentally. α is generally averaged as 1.27, although it varies, strictly speaking, by the constants of rainfall intensity formula. When α is 1.27, 15.0% depression of runoff rate may be expected and depression ratio will be changed proportionally with α in ordinary range. As a result, α is nearly 1.30 in the slope of 1/1 000. In the slope of 1/200, because the conventional rational method can design the same sewer diameter with the actual if proper value of n is selected, detention will not occur and α becomes unity. Although further experiments are necessary as for intervening slopes between these, the value of α will be presumed to be changed gradually. The excess value of α is calculated actually because the experimental sewer has constant diameter even in the upstream reach where the rate of runoff is low.

Full discussion on the detention formula is quite difficult for the resultant hydrograph of runoff is not shown. This is why the detention volume measured as in **Figs. 7** and **9** is not so useful for the evaluation of the detention formula. α is defined to express the depression of runoff by the sewer and is calculated taking dynamic storage equal to static storage.

Considering the runoff-depth state before and after becoming full flow referring to **Figs. 6** and **8**, the trend of increase of runoff with the depth is kept almost same for any opening ratio of the valve. Rather, there is a tendency of rapid ascent of the depth after the full flow, so that, it may be concluded that the depression of runoff rate is unable by the reduction of sewer section. A part of sewer volume is remained not occupied even after being full flow at the downstream end as seen in **Fig. 7**, when the slope of sewer is steep. This will verify the dynamic storage is not coincident to the static one. But in the slope of sewer 1/1 000, both are in comparable agreement as in **Fig. 9**. If any of control structure is attached, the runoff hydrograph can be flattened as **Fig. 6**, but the dynamic storage is reduced as seen in the full flow line of **Fig. 7**. Even in slope of sewer 1/1 000, the dynamic storage is hardly increased by the control structure artificially. The effect of construction of any control structure should not be to increase the dynamic storage, and should be intended only to reduce the runoff rate at the downstream end. According to this fact, it is better to understand that the detention coefficient α of detention formula represents the difference between the hydrograph by the conventional rational method and the actual one. Such control structure, then, has the negative effects except when the flooding in lower drainage area is concerned or when the drainage pump capacity is designed to be depressed.

Let consider the storage function method in **Figs. 6** and **7**. As K and x in Eq. (4.2) has been determined from the experimental hydrograph, it is rather natural to obtain the resultant curves better fitting to the actual phenomena. In the first stage of runoff, however, not so good agreement can be seen even for $n=0.016$. One of the reasons may be that the time-depth relationship at the maximum runoff is referred to the point of correspondence in this case. Another reason is as follows. Considering a stepwise rainfall with a constant intensity i as in the experiment, $i=I_1=I_2$ in Eq. (4.2). The runoff rate is then calculated by

$$Q_2=(C_0+C_1)i+C_2Q_1,$$

taking the initial condition as $Q=0$. Because $C_0+C_1+C_2=1$, the runoff rate Q_n after n-th time intervals is written as

$$Q_n=i[1-(C_2)^n]. \quad\cdots\cdots\cdots(4.3)$$

The second derivative of Q_n with respect to n becomes as follows.

$$Q_n''=-i\times C_2^n(\log_e C_2)^n<0,$$

which results in the convex type hydrograph obviously suggesting the inconsistency with the actual one. Besides, the infinite time is apparently required in Eq. (4.3) to attain the steady state, $Q_n=i$. Therefore, the expression of hydrograph by the storage function method may also contain a certain limit of application.

In **Figs. 8** and **9**, for the slope of 1/1 000, conformity of the storage function method seems worse than in the steeper slope, and the constant values of storage coefficients K and x for the whole ranges of runoff might not be established any more owing to the fundamental flow profile assumed. The form of Eq. (4.1), therefore, should be altered or treatment of K and x as a variables with time may be, at least, necessary in the gentle slope sewer. The storage function method, however, still holds the practical utility if the value of K and x are reasonably determined. The characteristics of these coefficients are of more importance when a composite drainage area of sewers and tributaries is concerned and the further study[5] is being progressed especially in relation to the equivalent roughness of drainage area.

As above clarified, the runoff phenomenon in gentle slope sewer differs essentially from that in the steep sewer. There may be two kinds of differences; one of them is the phenomenal difference and another occurs theoretically due to calculation procedures. The latter is strictly related to the former and will be of no significance when the complete calculation method is established. The most distinct differences appear in the state after the sewer becomes full flow or the flooding begins. Although the unchanged depth-time relationship holds approximately in the slope of sewer 1/200 even after the sewer becomes full flow, the rate of increase of depth (pressure head) is so remarkable in the slope of 1/1 000 after becoming full that the substantial flooding to the street area is more probable. As the static storage in sewer is larger than the dynamic storage in the slope of 1/200 as is clear from **Fig. 7**, the full flow state does not go up the sewer even if the flooding occurs locally in the lower section. On the other hand, in the slope of sewer 1/1 000, the flooding will occur through the wide extent and the amount flooded is also large for the static storage is almost occupied by the dynamic storage once the flow becomes full flow anywhere.

It can be, therefore, concluded that any of the existing storm runoff formulas including the detention formula is not verified to be effective for the maximum flow design since the maximum flow is probable itself and is followed by flooding.

5. The Design Formula and Return Period of Storm Intensity

Ordinarily, the constant storm intensity for the duration which is equal to the flow down period is adopted to design sewer by the conventional rational method, so as to calculate the maximum runoff without retardation. Even if the duration is prolonged with the same intensity, resulting the increase of return period as shown by the point A in **Fig. 10**, the maximum rate of runoff is unchanged keeping steady state for the prolonged period. In case of the increase of return period caused by heavier rainfall but without duration change as B in **Fig. 10**, the flooding will always occur. On the other hand, there may be also the case to flood with the heavier intensity and shorter duration but the same return period (C in **Fig. 10**). The equivalent roughness method[3] and detention method can cover the sewer design in such a case, but the maximum rate of runoff is not in steady and the flooding will definitely occur when any type of longer return period is actualized.

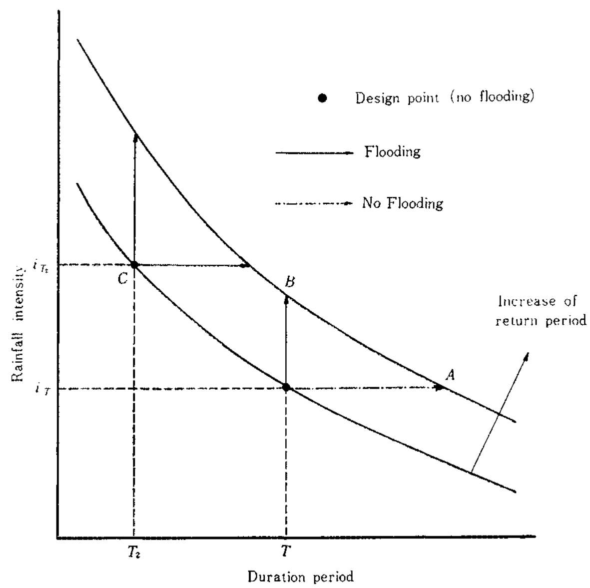

Fig. 10 Rainfall intensity vs. duration period with the parameter of return period.

The occurrence of flooding caused by the longer return period is rather essential, but it should be noted that the sewer designed by the method considering retardation and detention in narrow sense may have a chance of flooding more frequently than the one designed by the conventional rational method, if the design return period is kept constant as in the rational method. This fact may be one of the reasons why the sewer section designed by the rational method is generally said to allow for the excess amount of storm runoff. In another words, this tendency comes from the poor design formula which is not based on the full consideration of natural detention.

Furthermore, it is clear from the results in the previous chapter that neither enough space for dynamic storage nor artificial detention is expected much in the gentle slope sewer. For this reason, if the sewer becomes to be designed on the actual phenomena as possible, the gentle slope sewer will tend to have little additional room for the storm over the design return period, compared with the steep slope sewer. As the gentle slope area mostly has the steep slope area at the upstream side in urban area, the damage of flooding in the gentle slope area will be accumulated much larger. Once the concept of stochastic return period is proposed, the possibility of flooding should be accepted for the heavy rainfall over the design return period. The method to determine the return period should rather be an important question. In view of the reasonable sewer storm system design, the return period should be determined so as to minimize the sum of construction cost and damage by flooding. It is meaningless to simply select three or five years so far.

Let discuss the difference between the gentle slope area and the steep slope area from the experimental data as for the damage of flooding. Now, assume each experimental sewer has been designed in the state that the dynamic detention is utilized in full. Then design storm intensity formula to fit both cases becomes as the result;

$$i=\frac{8\,920}{t+26.8},$$

where i is volumetric intensity in l/min, t the duration in minutes and three years for the assumed return period The volumetric intensity is roughly proportional to the 0.2 power of the return period[6]. The average depth of flooding may be the half of the maximum flooding depth which can be obtained from **Figs. 6** and **8** as the difference between the depth (pressure head) and sewer diameter. The area of flooding is calculated dividing the maximum flooding depth by the slope of drainage area (slope of sewer). The cost of damage is considered to be proportional to the second power of depth[7] and will be expressed as

$$y=\gamma A_f h_f^2,$$

where y is the cost of damage by flooding, γ a constant, A_f areal flooding and h_f the average depth of flooding.

Table 2. Damages by 5 years rainfall

Sewer slope	1/1 000	1/200
Control structure	exists	exists
Dynamic storage	100% utilized	100% utilized
Inevitable space in sewer (cm³)⁽¹⁾	0	12×10³
Rainfall intensity (l/min)	81.3	92.1
Duration time in 3 years return period (sec)⁽²⁾	83	70
Duration time in 5 years return period (sec)	94	80
Increased duration of rainfall (sec)	11	10
Maximum depth of flooding (cm)⁽³⁾	0.8	0.3
Areal flooding (cm²)	4 850	3 140
Cost ratio of damage⁽¹⁾	11	1

(1) : calculated from Figs. 7 and 9.
(2) : assumed return period.
(3) : calculated by subtracting sewer diameter from water head.
(4) : calculated by dividing the cost of flooding damage in sewer slope 1/1 000 by the flooding cost in 1/200.

The result calculated under these conditions is summarized in **Table 2**. The flooding occurs in each case of slopes as the storm of 5 years return period is loaded on to the sewer designed with the rainfall of 3 years return period. It is, therefore, shown in the table that the gentle slope area has the damage as large as 11 times of the steep slope area. In order to minimize the sum of cost and damage, the return period should be increased in the gentle slope area, or if the factor such as the safety coefficient is introduced, this factor should be weighted much more in the gentle slope area. It is emphasized again that application of conventional rational method to design the upstream side sewers with steeper slopes would definitely charge the lower area with an extraordinary burden of flooding.

6. Summary

In this paper, the results of experimental study on the storm runoff in urban area were described. The former investigations have been performed mainly based on theories or practical observations. The practical survey, however, is extremely difficult because of the problems of provision for sudden storm, scarcety of design storm occurrence and also the selection of proper gauge station in sewers. These difficulties may be the principal reasons why the progress of investigations of urban storm runoff is so slow. The model configuration of sewer system is not unable because the drainage area and sewer include comparatively many artificial elements. The present laboratory model herein used is only the system model of storm-plain basin-single sewer and will not be sufficient for the complete study, but would make a step further toward the better model to study the storm detention and runoff. The results of the present investigation will be summarized as follows.

i) It was pointed out that the phenomena called as the flow detention, retardation, storage and flooding were the hydraulically identical phenomena and might be called "detention" in a wide sense. The detention formula may be available for the estimation of depression rate X for drainage pump capacity, provided that limits for b in Talbot's formula and the length of sewer are carefully selected, but the theoretical depression rate would become $X'(=\sqrt{2}X)$. The detention coefficient α is of different significance hydraulically from X and several questions are remained concerning with the theoretical procedure.

ii) Experiments were made for the sewer slopes of 1/200 and 1/1 000. Applying the rational method, the rate of runoff becomes the maximum more rapidly than the actual. Sewer section design, therefore, is in safety side. The hydrograph agrees comparatively well with the actual one in case of the steep slope sewer if the roughness is selected properly. But the hydrograph is much different in the gentle slope, and the correction is difficult in the practical range of the roughness. This fact is ascribed to the so called natural detention and suggests that the Manning's formula which is the equation of motion of the rational method is no longer established in the gentle slope. If this detention is evaluated experimentally by the term of detention coefficient α, α becomes 1.30 in the slope of 1/1 000 but is unity in 1/200, so that α should be understood as the correction factor representing the

difference between the hydrographs by the rational method and the actual one.

iii) Any of control structure is understood to have the negative effects upon the capacity of dynamic storage except when the flooding in lower area is concerned or when the drainage pump capacity is designed to be depressed.

iv) When the storage function method is used, the calculated hydrograph for steep slope agrees with the actual hydrograph. In the gentle slope area, the hydrograph is also not so good to be fitted. This method would be limited theoretically and the determination of the storage coefficients should be noted to be quite difficult.

v) The flooding does not always occur by the increasing of return period because the maximum rate of runoff becomes steady state when calculated by the rational method. On the contrary, when calculated by any design method considering detention, the maximum rate of runoff is not necessarily in steady state and the flooding certainly occurs by the increase of return period. The return period should be determined in order to minimize the sum of cost and damage based on the possibility of flooding due to heavier rainfall.

In the present investigation, the hydraulic approach is not yet established. But the concept of so-called detention has been signified more clearly. The further investigations are now in progress on the problems of the law of similitude, transition of partial flow to full flow and flooding mechanism.

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