3.3.3 Estimation of Effective Rainfall Intensity

The design hyetograph described in the sub-section 3.2.6 should be converted to the hyetograph of effective rainfall intensity with using the preliminary runoff rate, the saturated runoff rate and the saturation rainfall depth.

Explanation:

The basin's infiltration and other storage function cause a certain loss between the actual rainfall intensity and the effective rainfall intensity which contributes to the basin runoff discharge. In order to express the loss, the following three (3) factors could be considered; (a) preliminary runoff rate, (b) saturation rainfall depth and (c) saturated runoff rate. The effective rainfall intensity is estimated from the design hyetograph multiplied with the preliminary runoff rate, until the cumulative rainfall depth in the model hyetograph reaches to the saturation rainfall depth. After reaching to the saturation rainfall depth, the effective rainfall intensity could be in equal to the rainfall intensity of the model hyetograph multiplied with the saturated runoff rate. The three (3) factors on the rainfall loss vary according to the land use and the geological conditions in the basin. Determination of the factors should be based on the empirical data in the model basins.

Reference

Among the above three (3) factors, the Rational Formula solely apples the saturated runoff rate for estimation of the design peak discharge. The actual values of the saturated runoff rate are as described in the existing guideline in Malaysia*[3-1]. This Guideline applies the Quasi-linear Runoff Simulation Model in order to develop the discharge hydrograph. In case of the Quasi-linear Runoff Simulation Model applied, determination of all of the three (3) factors are required. The following standard values for these factors are applied in Japan*[3-4]:

Table 3.5 Standard Values of Factors for Rainfall Loss

Land Use	Preliminary Runoff Rate	Saturation Rainfall Depth (mm)	Saturated Runoff Rate
Paddy Field	0.00	50	1.00
Dry Drop Land	0.15	150	0.60
Forest	0.25	300	1.00
Urban Area	0.60 –0.90	50	1.00

3.3.4 Estimation of Flood Concentration Time

The flood concentration time should be estimated through either (a) the theoretical method by summation of over flow time and drain flow time or (b) the empirical method based on the actual observation.

Explanation:

The flood concentration time is defined as a time required until the rainfall which has fallen in the farthest point of a drainage area reaches to the exit of the drainage area. The flood concentration time is the essential factor for the linear type of runoff simulation models such as the "Rational Formula Model" and "Quasi-Linear Model". Both of the models are usually applied to the study on the urban drainage improvement.

The principal methods to estimate the flood concentration time could be divided in to (a) the method by summation of overland flow time and drain flow time and (b) the method based on the actual observation. In the first method of (a), the flood concentration time is assumed to consist of (1) the time required for runoff to flow over the ground surface to the nearest drain and (2) the time of flow in the drain to the exist of the drainage area. Details of this method could be referred to the existing guideline in Malasia*[3-1].

The above method of (a) requires information on the slopes of basin surface as well as the drainage channels. When the such information is hardly obtained, the empirical method of (b) should be applied. This sub-section describes the following prominent formula proposed by Kadoya*[3-5] among the several formulas proposed as method of (b).

$$tc = C \times A^{0.22} \times re^{-0.35} \quad \text{(Eq. 3.8)}$$

Where;
- tc : Flood concentration time (hr)
- C : Coefficient of basin characteristics varied according to land use
- A : Catchment Area (km^2)
- re : Effective rainfall intensity for a time duration of "tc" (mm/hr)

The linear relationship between the flood concentration time "tc" and the effective rainfall intensity could be also expressed by the rainfall intensity-duration curve as described in the foregoing sub-section 3.2.4. That is, the flood concentration time "tc" could be solved as a value to satisfy both equations of (Eq. 3.8) and the most conformable one selected among the equations of (Eq. 3.5) to (Eq. 3.7).

In the formula of (Eq. 3.8), the constant "C" expresses the land use characteristics of the basin varying according to land use classification. The value of "C" for mountainous area was firstly estimated through observation in 18 model basins in Japan. Then, "C" value for other land use classification was estimated from that for mountainous area through Kinematic Wave Theory. The following are the standard "C" values experimentally estimated in Japan*[3-5].

Table 3.6 Coefficient of "C" Value

Land Use Item	"C" Value
Mountainous area	250 ~350 ≒ 290
Pasture	190 ~210 ≒ 200
Golf Course	130 ~150 ≒ 140
On-going land development area	90 ~120 ≒ 100
Established urban area	60 ~ 90 ≒ 70

As shown above, the "C" value tends to decrease (that is, the flood concentration time "tc" tends to be shortened), as the land use is shifted to the built-up areas (such as the on-going land development area and the established urban area) from the non-built-up areas (such as mountainous area, pasture and golf course). This tendency could well conform to the known actual runoff conditions.

The "C" values could be provisionally applied as the standard in Malaysia. However, The values have a certain range, and some of them could be different from those in Malaysia due to different land use conditions between Japan and Malaysia. Accordingly, it is recommended to carry out the observation works for estimation of "C" several model basins in Malaysia.

The formula of (Eq. 3.8) also defines that the flood concentration time "tc" has linear relationships with the catchment area of the basin "A" and the effective rainfall intensity "re" for a time duration of "tc". These linear relationships were given from observation in the model basins as shown in Figs. 3.7 and 3.8.

Fig. 3.7 Relationship between Concentration Time and Catchment Area

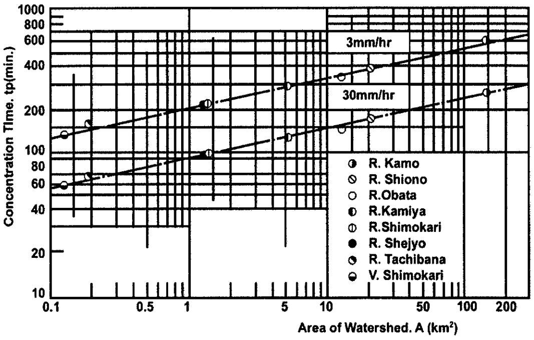
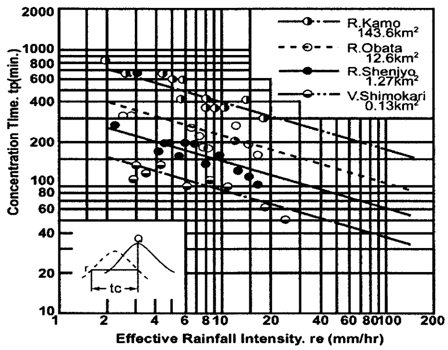


Fig. 3.8 Relationship between Concentration Time and Effective Rainfall Intensity



3.3.5 Estimation of Basin Run-off Discharge

The basin runoff discharge should be estimated as a part of the standard project flood through the following simulation models:

(1) Estimation for only the peak discharge : Rational Formula Model

(2) Estimation for both of the peak discharge as well as the discharge hydrograph : Quasi-Liner Runoff Simulation Model

Explanation:

The flood runoff from an urban drainage basin tends to immediately respond to rainfall and, therefore floods in the basin could have a close linear relationship between the runoff discharge and the rainfall intensity. Due to the linear relationship, the flood peak discharge for urban drainage plan has been conventionally estimated by "Rational Formula Model", which is expressed by the following equation*[3-1]:

q_p = re.. (Eq. 3.9)

Where; q_p : Peak runoff depth (mm)
re : Maximum average rainfall intensity observed during a flood concentration time (mm/hr)

In addition to the above peak discharge, the Rational Formula could generate the flood hydrograph through the following equation:

$$q(t) = (1/tc) \int_{\tau - tc}^{\tau} re(\tau) d\tau \quad \text{.......... (Eq. 3.10)}$$

Where, q(t) : Runoff depth at time t (mm)
tc : Flood concentration time (hour) (assumed to be constant)

$\int_{\tau - tc}^{\tau} re(\tau) d\tau$: Effective rainfall during a time of tc (mm)

The above Rational Formula has a distinct disadvantage such that the effective rainfall which occurs before "tc" does not contribute to the runoff discharge. Due to the feature, the equation could hardly express the flood hydrograph, its diminution in particular.

The flood hydrograph is essential for plan formulation of detention and retention facility, and in order to retrieve the above disadvantage of Rational Formula and to simulate more appropriate flood hydrograph, this sub-section prescribes the "Quasi-Liner Runoff Simulation Model". The basic formula of the model is composed of the equation of continuity (Eq. 3.11) and the equation of motion (Eq. 3.12)

$$q(t) = = re - dh(t)/dt \quad \text{(Eq. 3.11)}$$

$$h(t) = K \times q(t) \quad \text{(Eq. 3.12)}$$

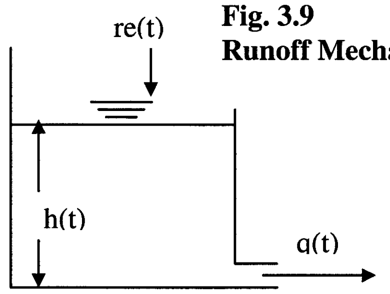
Where; h(t) : Storage depth of basin (mm)
re(t) : Effective rainfall intensity (mm/hr)
q(t) : Runoff depth (mm/hr)
K : Storage Constant (Inverse number of recession constant)

The Model expresses the runoff from an imaginary tank as illustrated in Fig. 3.9. The runoff is proportional to the storage depth above the outlet, and presents a unit hydrograph expressed by an exponential diminution with time as below

$$q(t) = \int_0^t u(\tau)(re(t-\tau)d\tau \quad \text{(Eq.3.13)}$$

$$u(\tau) = 1/K \times e^{-\tau/K} \quad \text{(Eq. 3.14)}$$

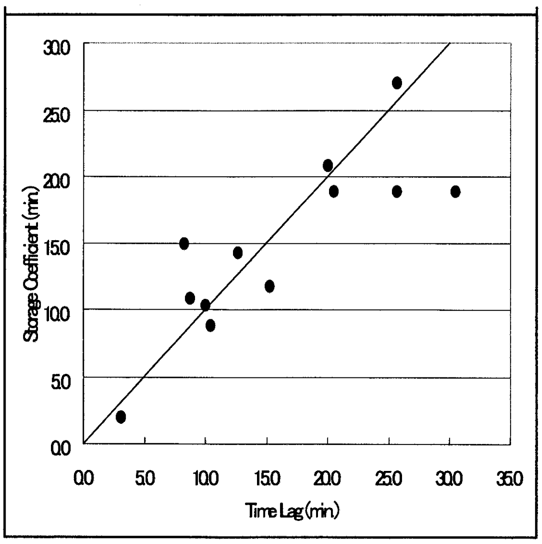
Fig. 3.9
Runoff Mechanism



Yoshino[*3-6] proposed that the storage coefficient "K" in the equation (Eq. 3.14) is in equivalent with the flood lag time "tl". This is based on an experiment for the relationship between actual "K" and "tl" in Japan (refer to Fig. 3.10).
It is herein noted that the flood lag time is defined as the time difference from the peak rainfall intensity to the peak runoff discharge (refer Fig. 3.11).

Fig. 3.10 Results of Experiment on Relationship between Storage Coefficient and Flood Lag Time in Japan

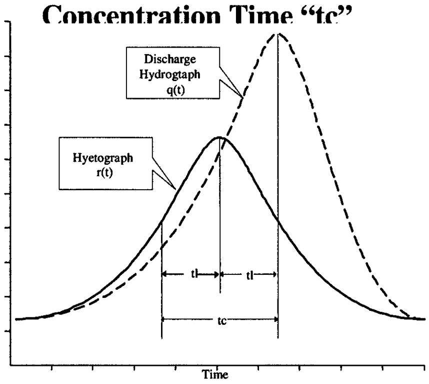
Name of River	Recession Constant (K)	Storage Coefficient (1/K) (min.)	Time Lag (min.)
Yabata (upper)	0.113 (21)	8.8	10.4
Yabata (lower)	0.053 (9)	18.9	25.7
Momozono (upper)	0.097 (3)	10.3	10.0
Momozono (lower)	0.053 (14)	18.9	20.4
Senri NT No.1	0.503 (2)	2.0	3.0
Senri NT No. 2	0.092 (9)	10.9	8.7
Senri NT No. 3	0.070 (8)	14.3	12.6
Sennen Nanbu	0.048 (4)	20.8	20.0
Chubu Daiichi	0.053 (5)	18.9	30.5
Tamitsu	0.067 (3)	14.9	8.3
Yamachi	0.085 (3)	11.8	15.2
Shimizu	0.037 (1)	27.0	25.7



As illustrated in Fig. 3.11, the flood lag time of "tl" could correspond to twice of the flood concentration time "tc". This relationship is primarily realized for the flood hydrograph around the peak discharge, but also applied to the entire portion of the hydrograph. Based on this assumption, the storage coefficient "K" is expressed by the flood concentration time "tc" as below:

$$K = tl = tc/2 \qquad \text{(Eq. 3.15)}$$

Fig. 3.11 Flood Lag Time "Tl" and Concentration Time "tc"



The flood concentration time "tc" could be estimated from the Kadoya's formula (Eq. 3.8), and therefore, the storage coefficient "K" could be estimated through formulas of (Eq. 3.8) and (Eq. 3.15).

3.3.6 Estimation of Channel Flow Discharge

The basin runoff discharges from sub-basins flows into the river channels and/or the trunk drains and propagated toward the downstream channels. These channel flow discharges should be estimated at every object control points and every object facilities.

Explanation:

The main parameters to be considered for the channel flow discharge are the channel storage function and the flood travelling time on the channel. These parameters should be ideally determined on the basis of the observed data. The observed data is, however, not available in many cases. The data such as channel cross-sections and longitudinal sections and roughness coefficients are often used as bases to determine the parameters as described below:

(1) Estimation of Channel Storage Function

In order to express the channel storage function, the "Storage Function Model" could be applied as the principal channel flow method. The application of this model is, however, subject to a rather large channel section where the channel width is more than 10 times of channel depth. As for the small channel section, the storage function is regarded to be nil. The outflow discharge "Qo" at the downstream end of the rather large channel section could be expressed by the following formulas:

$$dS/dt = Qi(t) - Qo(t) \qquad \text{(Eq. 3.16)}$$

$$S = K \cdot Qo(t)^{P} \qquad \text{(Eq. 3.17)}$$

Where; S : Storage volume of the basin ($m^3 \cdot hr/s$)
Qi(t) : Inflow discharge of the channel at time "t" (m^3/s)

Qo(t)	:	Outflow discharge of the channel at time "t" (m^3/s)
K, P	:	Constant parameters

The parameters of "K" and "P" are expressed by Manining's Formula for Steady Flow where "P" takes the constant value of 0.6 and "K" takes the value estimated from the following equation;

$K = L \cdot B \cdot^{0.4} (n/I^{0.5})^{0.6}/3.6$ .... (Eq. 3.18)

Where; L : Channel Length (km)
B : Average channel width (assumed from the existing channel width)
n : Manning's roughness coefficient (=0.035 for non-improved channel, and 0.02 for improved channel)
I : Average channel slope (assumed from the existing channel slope)

(2) Estimation of Channel Flood Travelling Time

The flood travelling time on the channel means the elapsed time for the inflow discharge for the channel to reach to the downstream end of the channel. Many empirical formulas for estimation of the flood travelling time have been proposed and many of them are expressed by the channel length and slope. Among the formula, this Guideline introduces the following Karaven's formula which are applied in many of the urban drainage plans.

$T = L/(W \cdot 3.6)$ .... (Eq. 3.19)

Where; T : Flood travelling time (hr)
L : Channel length (km)
W : Flood travelling velocity (m/s)

In the above equation, the following is applied as the flood travelling time.

Channel slope (I)	over 1/100	1/100 – 1/200	below 1/200
Flood travelling velocity (W)	3.5 m/s	3.0 m/s	2.1 m/s

3.4 Analysis on Regulation Effect by Flood Detention and Retention Facilities

3.4.1 Purpose of Analysis

The analysis aims at estimating the following two (2) items:

(1) Design flood discharge at every object control points and every object facilities; and

(2) Hydraulic dimensions of the flood detention and retention facilities such as the storage capacity and the size of outlet.

Explanation:

The standard project flood discharge prescribed in the foregoing sub-section 3.3 is subject to no regulation effect by any flood detention and retention facilities and regarded as the inflow discharge into the flood detention and retention facilities. On the other hand, the design flood discharge is subject to regulation effect by the flood detention and retention facilities and therefore, defined as the outflow discharge from the flood detention and retention facilities. The estimation on the design flood discharge could facilitate to determine the design flow capacity of the drainage channels and/or the river channels. This sub-section prescribes a methodology to estimate the design flood discharge at every object control points and every object facilities. This sub-section further prescribes a methodology to determine hydraulic dimensions of the flood detention and retention facilities based on the standard project flood discharge as the inflow discharge to the flood detention and retention facilities. The conceptual procedures of the analysis are as shown in Fig. 3.12.

Fig. 3.12 Conceptual Procedures for Estimation of Design Flood Discharge and Hydraulic Design for Flood Detention and Retention Facilities

(1) Estimation of Standard Project Flood at Ever Control Point
(refer to sub-section 3.3)

↓

(2) Hydraulic Design of Flood Detention and Retention Facilities

↓

(3) Estimation of Design Flood Discharge at every Control Points

3.4.2 Classification of Flood Detention and Retention Facilities

The flood detention and/or retention is made by a combination of flood storage and infiltration in the drainage basin. The detention by flood storage is classified into the off-site storage and on-site storage. The flood infiltration is also classified into the diffusion type and the well type. These various types of flood detention facility contain their own typical hydraulic effect on the flood runoff discharge from a drainage basin.

Explanation:

The flood detention facility is classified into various types as shown in Table 3.7. Among these, the storage type includes the off-site and on-site detention facility. The off-site detention facility is such as the basin retarding basin and the flood detention pond which controls the runoff discharge from a rather extensive catchment area. On the other hand, the on-site detention facility collects the rainfall within its compound, and has a smaller detention capacity as compared with that of the off-site detention facility. The typical facilities of the storage type are such as (a) storage tank in an individual house lot and (b) storage facility in a public open space (ground, car park, etc.).

The infiltration type is also divided into diffusion type and concentration type. The diffusion type lets the storm rainfall to penetrate into the site filled up by crushed stones diffusing the penetrated water, while the concentration type pours the rainfall into wells. The basin flood detention and/or retention could be made by a combination of the flood storage and infiltration. That is, the infiltration facilities could be attached to both of the off-site and on-site storage facilities.

Table 3.7 Classification of Flood Detention Facility

Primary Classification	Secondary Classification	Facility in the Classification
Storage Type	Off-site Storage	Flood retarding basin
		Flood detention pond
	On-site Storage	Storage tank in an individual house lot
		Storage facility in a public open space
Infiltration Type	Diffusion type	Penetration Trench
		Penetration gutter
		Permeable pavement
	Well Type	Dry well
		Wet well

3.4.3 Hydraulic Design for On-site Storage Type of Detention Facility

The on-site storage detention facility stores the storm rainfall and regulate its outflow discharge. The hydraulic design of the facility is subject to the design hyetograph as the design inflow and should be made to determine the design outflow discharge and hydraulic dimensions of the facilities such as the design storage capacity and size of the outlet.

Explanation

The hydraulic design for the on-site storage detention facilities should be made to determine the following items on the premises of the design hyetograph as the inflow discharge to the facility, the available space for the facility and the social requirement to the facility (such as allowable storage depth and allowable storage time duration of the facility).

(a) Design outflow discharge from the on-site detention facility against the design hyetograph as the inflow of the facility;

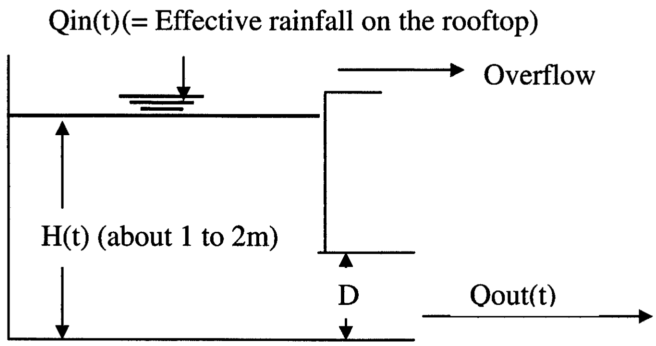
(b) Hydraulic dimensions such as the storage capacity and the size of the outlet of the on-site detention facilities

The hydraulic designs for principal types of storage detention facility are as below:

(1) Hydraulic Design of Storage Tank in House Lot

A storage tank with a small outlet hall at side bottom is installed in an individual house lot to collect the rainfall from the rooftop of the house and regulate outflow discharge from the tank (refer to Fig. 3.13). The size of the storage tank is determined on the basis of the available open space of a house lot and usually has a range of 2 to 4 m^3.

Fig. 3.13 Concept of Storage Tank in a House Lot



The size of the outlet hole is also determined is on the premises that the water tank does not allow overflow against the inflow of the design hydrograph with the target design scale for urban drainage. When the design hyetograph, the size of storage tank, and the width of outlet hole (usually 3 to 5 cm) are given as known-values, the depth of outlet holes and the maximum outflow discharge could be estimated through trial calculation with using the following equations. Trial calculation is made until the depth of the outlet holes could satisfy the condition that the calculated maximum storage volume becomes equal to the size of the storage tank.

$dS(t)/dt = Qin(t) - Qout(t)$ .... (Eq. 3.20)

$Qout(t) = 1.7 \cdot B \cdot H(t)^{1.5}$ (If H>1.2D) .... (Eq. 3.21)

$Qout(t) = C \cdot B \cdot D \cdot (2gH(t) - D/2)^{0.5}$ (If H<1.8D) .... (Eq. 3.22)

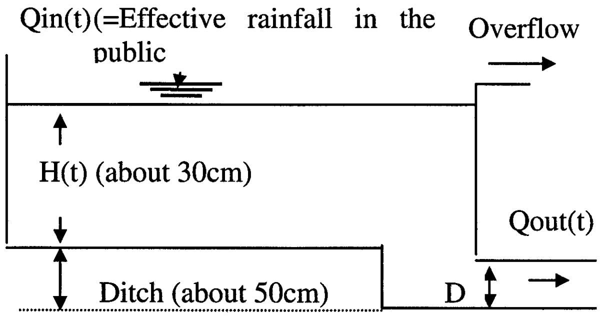
Where; S(t) : Storage Volume at time t,
Qin(t) : Inflow (i.e., design hyetograph) at time t
Qout(t) : Outflow discharge at time t
H(t) : Depth of storage tank at time t

B : Width of outlet hole
D : Depth of outlet hole
C : Coefficient (=0.6 in case of bell-mouth is not attached to the outlet hole, or = 0.85 in case of bell-mouth is attached)

(2) Hydraulic Design of Storage Facility in Public Open Space

The storage measure of this type is such that a public open space (such as a sport ground and a car parking area) is enclosed by a low wall with a surrounding side drain and a outlet to collect the rainfall from an entire public compound (refer to Fig. 3.14). The maximum storage depth and storage time length should be limited in due consideration of the original purpose of the storage space as public utility.

Fig. 3.14 Concept of Storage Facility in Public Open Space



Most of the facilities of this type in Japan are designed to have the maximum storage depth of 30cm and the maximum storage time of 2 to 12hours due to the original purpose of the storage space as the public utility. The size of the outlet should be determined on the premises that the storage will meet to the requirement of the maximum storage depth and storage time against the design hydrograph of the target design scale for urban drainage (i.e., 5-year return period), not allowing any overflow. When the inflow discharge hydrograph (i.e., the standard project flood) and the width of outlet hole (usually about 50 cm to 1m) are given as known-values, the depth of outlet holes could be estimated through trial calculation with using the foregoing equations (Eq. 3.20) to (Eq. 3.22).

(3) Integration of Hydraulic Design

The on-site flood detention and retention facilities area scattered in a objective drainage basin. In order to evaluate the gross regulation effect of the facilities, the storage tanks in house lot and the storage facilities in public open space could be respectively integrated into one group. The integrated hydraulic dimensions could be determined through the following equations:

Storage Volume : $V = \Sigma Vi$ .... (Eq. 3.23)

Area of Storage : $A = \Sigma Ai$ .... (Eq. 3.24)

Storage Depth : $H = V / A$ .... (Eq. 3.25)

Height of Outlet Hole : $D = \Sigma (Qi \cdot Di) / \Sigma Qi$ .... (Eq. 3.26)

Area of Outlet Hole : $a = \Sigma Qi / [C \cdot \{2g \cdot (H + dH - D/2)\}^{0.5}]$ .... (Eq. 3.27)

Width of Outlet Hole : $B = a / D$ .... (Eq. 3.28)

Where; Vi : Storage volume of each facility
Ai : Area of Storage of each facility
Qi : The outflow discharge from each of facility against the basic project flood (refer to sub-section 3.4.2)
Di : Height of outlet hole for each of facility
dH : Depth of ditch (≑ 50cm for the storage facility in a public open space and 0cm for other facilities)

3.4.4 Hydraulic Design for Off-site Storage Type of Detention Facility

The design inflow discharge to the off-site detention facility should be estimated by summing up of (a) the standard project flood discharge without regulation by any on-site flood detention and retention facility and (b) the design outflow discharge from the on-site storage and infiltration facilities scattered in the upper reaches. Based on the design inflow discharge, determined are the design outflow discharge and the hydraulic dimensions of the off-site facility such as the design storage capacity and size of the outlet.

Explanation

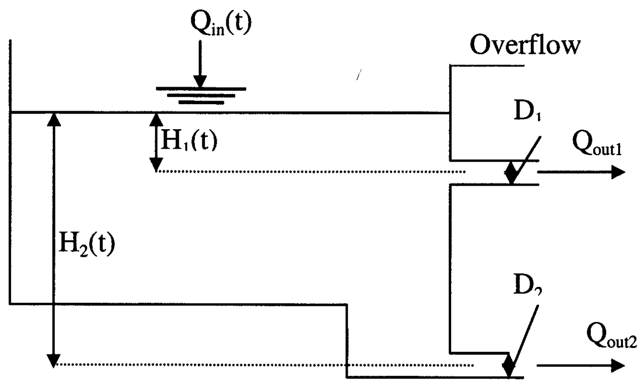
The storage capacity of the off-site detention facility could be far larger than the on-site detention facilities. Due to such larger storage capacity, the flood detention pond could regulate the basin runoff discharge from a new land development area, and reduce the peak discharge to be same as before the land development. This reduction of peak discharge is given to both of the following two (2) types of standard project flood .

(a) The standard project flood of 100-year return period which is set as the target design level for prevention of overflow from the river channel with a catchment area of more than 4km².

(b) The standard project flood of 5-year return period which is set as the target design level for urban drainage.

In order to perform the above regulation effect, the flood detention pond should have two outlet holes as illustrated in Fig. 3.15. The probable flood hydrograph of less than 5-year return period is discharged through only the lower outlet, while the probable flood of 5 to 100 year return period is discharged through both of the lower and upper holes. When the flood exceeds the probable scale of 100-year return period, the overflow starts to flow out from the spillway.

Fig. 3.15 Concept of Flood Detention Pond



When the inflow discharge hydrograph (i.e., the hydrograph of the standard project flood), the width of outlet and the depth of pond are given as known-values, the depth of outlet holes and height of the outlet could be estimated through trial calculation with using the following equations.

$$dS(t)/dt = Q_{in}(t) - Q_{out1}(t) - Q_{out2}(t) \quad \text{(Eq. 3.29)}$$

$$Q_{out\text{-}1}(t) = 1.7 \cdot B_1 \cdot H_1(t)^{1.5} \quad (\text{If } H_1 > 1.2D_1) \quad \text{(Eq. 3.30)}$$

$$Q_{out\text{-}1}(t) = C \cdot B_1 \cdot D_1 \cdot (2gH_1(t) - D_2/2)^{0.5} \quad (\text{If } H_1 < 1.8D_1) \quad \text{(Eq. 3.31)}$$

$$Q_{out\text{-}2}(t) = 1.7 \cdot B_2 \cdot H_2(t)^{1.5} \quad (\text{If } H_2 > 1.2D_2) \quad \text{(Eq. 3.32)}$$

$$Q_{out\text{-}2}(t) = C \cdot B_2 \cdot D_2 \cdot (2gH_2(t) - D_2/2)^{0.5} \quad (\text{If } H_2 < 1.8D_2) \quad \text{(Eq. 3.33)}$$

Where; $S(t)$: Storage Volume at time t,
$Q_{in}(t)$: Inflow discharge (i.e., design discharge) at time t
$Q_{out1}(t)$, $Q_{out1}(t)$, : Outflow discharge sat time t for upper and lower outlet

$H_1(t)$, $H_2(t)$	:	Depth of storage tank at time t from the centers of upper and lower outlet
B_1, B_2	:	Width of upper and lower outlet hole
D_1, D_2	:	Depth of upper and lower outlet hole
C	:	Coefficient (=0.6 in case of bell-mouth is not attached to the outlet hole, or = 0.85 in case of bell-mouth is attached)

3.4.5 Hydraulic Design for Infiltration Type of Flood Retention Facility

The design infiltration capacity of the facility should be determined through the field infiltration test and/or with referring to the existing related data such as soil data, land use data, ground water level data and geological data.

It is firstly required to determine the average infiltration capacities in the catchment areas of the infiltration facility based on the results of the actual field infiltration test and/or the existing available data such as soil data and the previous boring data. Then, these infiltration capacities are averaged as a weighted value by the size of the catchment area as expressed in (Eq. 3.34).

$$Iave = \Sigma (Ai \cdot Ii) / \Sigma Ai \quad \text{(Eq. 3.34)}$$

Where;

	Iave	:	Average infiltration capacity for the integrated infiltration facility (mm/hr)
	Ai	:	Catchment area for each of the scattered infiltration facilities (ha)
	Ii	:	Infiltration capacity for each of the scattered infiltration facilities (mm/hr)

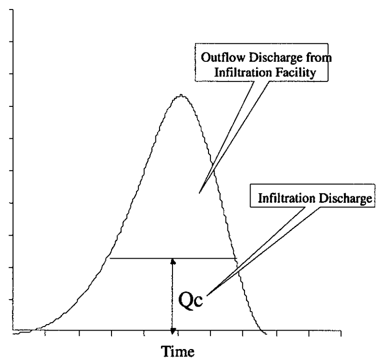
The above weighted average could be regarded as the infiltration capacity of the integrated infiltration facility, and the infiltration discharge by the integrated facility could be estimated by the following equation (refer to Fig. 3.16) .

$$Qc = Iave \cdot \Sigma Ai / 360 \quad \text{(Eq. 3.35)}$$

Where; Qc: Infiltration discharge (m3/s)

The infiltration facility could be installed together with the flood storage facility. In such case, the outflow discharge from the flood storage facility should be regarded as a total of the outflow discharge from the outlet hole and the above infiltration discharge.

Fig. 3.16 Outflow and Infiltration Discharge from an Infiltration Facility



3.4.6 Estimation of Design Flood Discharge

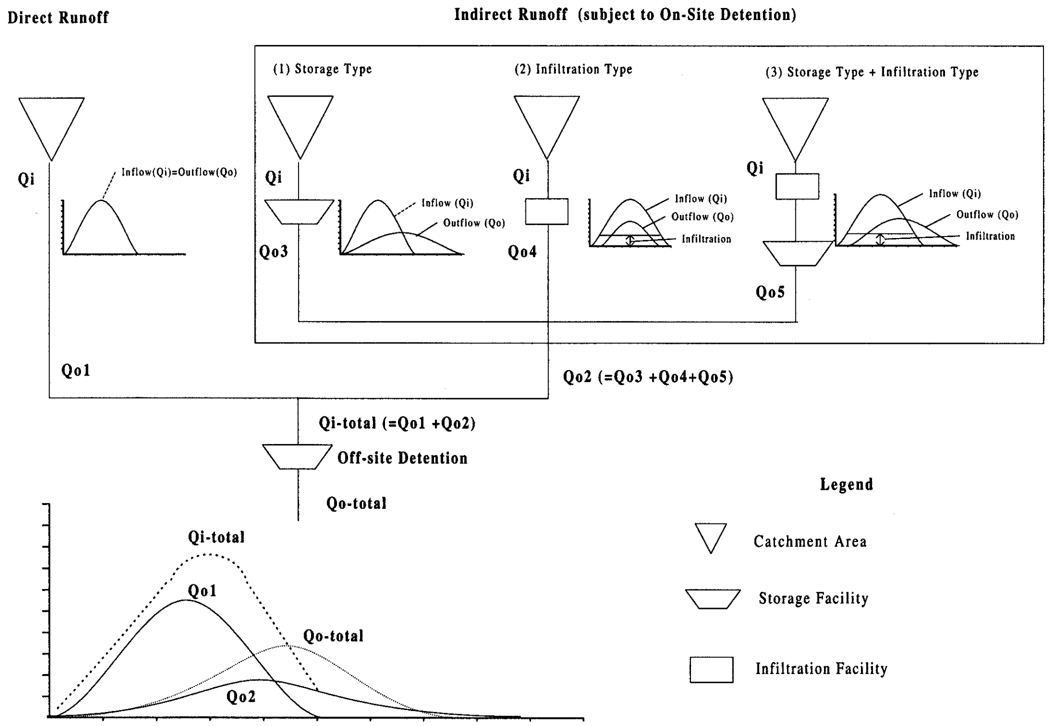
The design discharge at every control point should be estimated by accumulation of (a) direct runoff discharge without regulation by any flood detention and retention facility and (b) indirect runoff discharge regulated by all proposed flood detention and retention facilities. The accumulation should be made one after another from the upstream toward the downstream.

Explanation

In order to estimate the design discharge, the sub-basin described in the sub-section 3.3.2 should be further divided into a direct runoff area and an indirect runoff area, and an off-site detention facility is placed at the exit of the sub-basin as shown in Fig. 3.17. The runoff from the direct runoff area is not filtered by any detention facility. On the other hand, the runoff from the indirect runoff area is regulated by the on-site detention facility and/or the infiltration facilities.

All sub-basins incorporated into the flood runoff simulation model described in the foregoing sub-section 3.3.2 are assumed to have a runoff mechanism as shown in Fig. 3.17. When a detention facility is proposed to a certain sub-basin, its hydraulic effect to the lower reaches could be evaluated through the runoff simulation model..

Fig.3.17 Conceptual Diagram for Estimation of Design Flood Discharge



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