Optimal Design of Detention System using Incremental Dynamic Programming

Lee, Kil Seong* · Lee, Beum Hee**

ABSTRACT : The purpose of this study is to develop an efficient model for the least cost design of multi-site detention systems. The IDP (Incremental Dynamic Programming) model for optimal design is composed of two sub-models: hydrologic-hydraulic model and optimization model. The objective function of IDP is the sum of costs; acquisition cost of the land, construction cost of detention basin and pumping system. Model inputs include channel characteristics, hydrologic parameters, design storm, and cost function. The model is applied to the Jung-Rang Cheon basin in Seoul, a watershed with detention basins in multiple branching channels. The application results show that the detention system can be designed reasonably for various conditions and the model can be applied to multi-site detention system design.

1. Introduction

The detention pond becomes more necessary in the city which is vulnerable to the concentrated rainfall. The pond stores the runoff which exceeds the pumping capacity and reduces the peak flows. However, the design of a single detention pond can not consider the relations between the detention ponds and sub-watersheds, the design of an integrated detention system is more required. The optimized detention system design needs to consider not only the mitigation of the internal and external floods, but also economic conditions and interrelations between detention ponds. To develop an efficient and rational design model of detention system and to present the applicability of the model developed to the real case, it is applied to the Jung-Rang Cheon basin in Seoul, a watershed with detention basins in multiple branching channels.

Since Lakatos (Ormsbee et al., 1987) developed such an optimized model, Mays and Bedient (1982) developed an optimized model using DP (Dynamic Programming) and also there are some other computer models such as ILSD, ISS, Ti-59 (Voorhees, 1978). Moreover, Ormsbee and Lansey (1994) considered the environmental factors in their model.

---

* Prof., Department of Civil Engineering, Seoul National University

** Graduate Student, Department of Civil Engineering, Seoul National University

There are several studies in Korea (Lee et al., 1992a, b, c; Wone and Yoon, 1993; Lee et al., 1993). Among them, Lee et al. (1992b) IDP model, which is a modified model of DP, was applied to the operation of multi-reservoirs, but no models have been applied to the design of a real detention basin.

This study is especially performed to develop a multi-site design model. The multiple detention/retention facilities may cause a timing problem that the peak flow downstream would partially increase due to the coincidence of the flow arrivals from upstream reservoirs.

It may also cause another problem that the flood with short return period is hard to control, even if the flood with long return period is easily controlled. Therefore, the design of multi-detention system needs some different analysis from the design of a single detention pond (Walesh, 1989).

This study finds an optimum model which circumvents the above two problems through the sensitivity analysis with different return periods. Moreover, to consider the continuing urbanization and the following changes of channel conditions, the developed model can simulate the changes of both detention system and the hydrologic phenomena due to the changes of safety channel water level.

2. Hydrologic-Hydraulic Model

Hydrologic-hydraulic models for the design of detention system are described as Fig. 1 and a schematic diagram with stages (m-1) and (m), a part of the detention system indicated as A, is presented as Fig. 2. It can be analyzed using some sub-modes. The outflows $O_{m,t}$ of the basin m is calculated from design storm (rainfall-runoff model). If the inflow to the detention pond is equal to $O_{m,t}$ and the outflow of the detention pond $U_{m,t}$ can be determined, then the storage of the detention pond V_m is determined using $P_{m,t}$ which is difference between $O_{m,t}$ and $U_{m,t}$ and $\phi_{m-1}X_{m-1,t}$ can be calculated by the channel routing model considering the safety water levels.

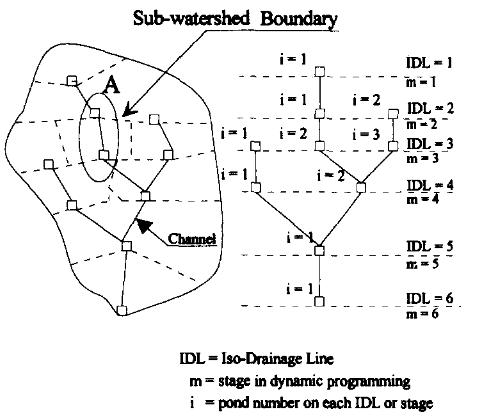

Fig. 1. Iso-Drainage Line Applied to Detention Basin Systems (Taur et. al, 1987)

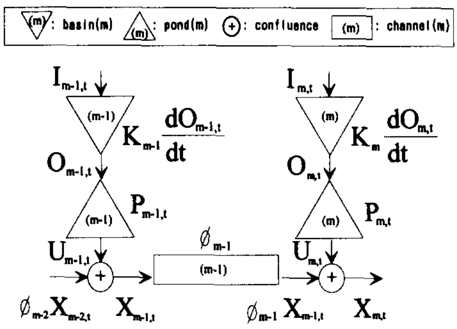

Fig. 2. Hydrologic-Hydraulic Schematic Diagram of Inter-Basin Detention System

2.1. Design Rainfall

A design storm or design rainfall, which is the proposed rainfall event to design of a special purposed hydraulic structure, is the artificial rainfall event by the rainfall depth–duration–frequency analysis.

In this study, design storm is estimated from the probable precipitation data of Korea (Rainfall frequency atlas of Korea, 1988) with Huff's rainfall distribution in Seoul (Time distribution of regional design rainfall, 1989). The tables of the design storm and Huff's rainfall distribution in Jung–Rang Cheon are given in Tables 1 and 2.

Table 1. Probable Precipitation in Jung–Rang Basin (mm)

Frequency (years) \ Duration (hr)	0.5	1	2	3	6
5	39.5	59.5	81.5	97.0	129.0
10	47.5	72.0	95.0	114.5	154.0
20	54.5	83.5	110.0	132.0	175.0
50	65.0	96.5	131.0	155.0	205.0

Table 2. Huff's Rainfall Distribution in Seoul Basin (50% probability)

Type \ Dur.(%)	10	20	30	40	50	60	70	80	90	100
1st	20.0	44.1	61.5	71.5	77.8	83.6	87.9	92.9	96.6	100.0
2nd	8.2	18.5	35.0	53.5	72.6	82.2	89.6	93.9	97.3	100.0
3rd	3.9	7.9	15.2	22.3	33.3	53.1	74.4	89.4	96.5	100.0
4th	4.8	9.7	15.4	20.0	28.6	35.5	45.1	63.6	86.0	100.0

2.2. Rainfall–Runoff Model

There are many methods for simulation of rainfall input and complex process in sub–basin as SWMM, ILLUDAS (Terstriep and Stall, 1974). However, the objective of this study is to expand the design model using IDP method with IDL concept, and this study modifies the Clark model and combines with design model for the efficiency.

The Clark model simulates both the translation effect and the storage effect by routing its time–area–concentration curve through an assumed single linear reservoir. The runoff coefficient in this study has referenced the general standard of runoff coefficient for landuse in Seoul sewerage facilities' standards (Table 3; '87 report of flood disaster white paper, 1988).

The Clark model assumes a linear reservoir with the storage S, outflow O, and storage coefficient K. If the inflows and outflows at the beginning and ending time for the intervals Δt are denoted as I_1, I_2, O_1, O_2, then the Eq. of continuity and its coefficients are determined as

$$\frac{I_2+I_1}{2}-\frac{O_2+O_1}{2}=\frac{K\cdot(O_2-O_1)}{\Delta t} \quad (1)$$

$$O_2=C_0\cdot I_2+C_1\cdot I_1+C_2\cdot O_1 \quad (2)$$

$$C_0=\frac{0.5\cdot\Delta t}{K+0.5\cdot\Delta t},\ C_1=\frac{0.5\cdot\Delta t}{K+0.5\cdot\Delta t},\ C_2=\frac{K-0.5\cdot\Delta t}{K+0.5\cdot\Delta t} \quad (3)$$

If hydrograph data are not available, K can be equated with the basin lag time using any empirical equation (Viessman, 1977). From these equations, rainfall–runoff of each sub–watershed and inflow hydrograph of design detention pond are calculated.

A watershed characteristic parameter t_g(min.), time of concentration, can be calculated by the Rizha's equation and Kraven's equation in Design standards for small facilities (1990)

$$\text{Rizha} : t_g=0.833\times\frac{L_m}{h_m^{0.6}} \quad (4a)$$

$$\text{Kraven} : t_g=0.444\times\frac{L_m}{h_m^{0.515}} \quad (4b)$$

where h_m =Slope of the watershed m, *and* L_m =Length of the channel m (km).

Rizha's equation is to the upper region of the natural channel ($h_m \geq 1/200$), and Kraven's equation is to the mid–lower region of the natural channel ($h_m \leq 1/200$).

Table 3. General Standard of Runoff Coefficient for Landuse

Landuse Classification	Runoff Coefficient
Woodland and similar residence zone	0.80
Factory district and general residence zone	0.65
An intermideate grade residence zone and detached house zone	0.50
A high grade residence zone and the suburbs with field	0.35
A park and the green areas	0.20

2.3. Channel Routing Model

The Muskingum method has been one of the most popular methods for channel routing. However, the approximation using Manning's Eq. is very useful, when the length of channel m is short and well maintained. It can also useful when the storage effect is smaller than the transport effect, and the cross–sectional area $A(y_{m,t})$, the mean flow velocity $u(y_{m,t})$, the hydraulic radius $R_h(y_{m,t})$ can be represented by the Eq. of water depth $y_{m,t}$ for a given discharge $X_{m,t}$ (Mays, 1978).

$$X_{m,t}=A(y_{m,t})\cdot u(y_{m,t})=A(y_{m,t})\cdot\frac{1}{\mathrm{n}}\cdot \mathrm{R_h}(y_{m,t})^{\frac{2}{3}}\cdot h_m^{\frac{1}{2}} \tag{5}$$

If the channel discharge $X_{m,t}$, channel sectional shape, slope h_m and Manning's n are given, then water depth $y_{m,t}$ and cross-sectional area $A(y_{m,t})$ in Eq. (5) are determined by Newton–Raphson method. The channel length L_m, the mean flow velocity $u(y_{m,t})$, and the shift time $t_y(X_{m,t})$ are expressed as Eqs. (6) and (7).

$$u(y_{m,t})=\frac{X_{m,t}}{A(y_{m,t})} \tag{6}$$

$$t_y(X_{m,t})=\frac{L_m}{u(y_{m,t})}=L_m\cdot\frac{A(y_{m,t})}{X_{m,t}} \tag{7}$$

Inversely, if the probable highest water level $y_{m,max}$ is given from the channel section data, then maximum allowable discharge Q_m can be decided at the channel m.

$$Q_m=A(y_{m,max})\cdot u(y_{m,max})=A(y_{m,max})\cdot\frac{1}{n}\cdot R_h(y_{m,max})^{\frac{2}{3}}\cdot h_m^{\frac{1}{2}} \tag{8}$$

Therefore, t_y can be calculated from Eqs. (6) and (7) with the changes of the channel flows on time (Mays and Bedient, 1982) and the downstream flow $X_{m,t}$ can be represented by the linear transformation of the upstream flow $X_{m-1,t-t_y}$ when there is no lateral inflow.

From these relations, $X_{m-1,t}$ and $X_{m,t}$ can be represented by the shift operator ϕ_{m-1} that means the time shift is t_y.

$$X_{m,t}= X_{m-1,t-t_y} = \phi_{m-1}\cdot X_{m-1,t} \tag{9}$$

This study has two sub–programs (NEWTON and CHR) for the Newton–Raphson approximation and for the channel routing, respectively.

2.4. Reservoir Routing Model

To estimate the detention volume, the rising limbs of the inflow and outflow hydrographs are assumed to coincide until the limiting discharge rate(α) is reached as Abt and Grigg method (McCuen, 1989) and, excess inflows are stored. All inflows are discharged when the recession of the hydrograph is less than the maximum outflow rate. If the rule for the control of storm drain pump is given, then the volume of storage can be decided from the given inflows $O_{m,max}$ and maximum outflow rate $U_{m,max}$. In this study, also, the detention volume $V_m(O_{m,t},U_{m,max})$ can be calculated from given pumping capacity $U_{m,max}$, where $U_{m,max}$ is not smaller than 0 and not larger than $O_{m,max}$. If there is no natural drainage, then storage–runoff relations can be given as Eqs. (10), (11) and (12).

$$0 \leq U_{m,max} \leq O_{m,max} \tag{10}$$

$$P_{m,t}=O_{m,t}-U_{m,t} \tag{11}$$

$$U_{m,t} \left[\begin{array}{l} = O_{m,t}, \text{ when } O_{m,t} \leq U_{m,max} \\ = U_{m,max}, \text{ when } O_{m,t} \geq U_{m,max} \end{array} \right.$$

$$V_m(O_{m,t}, U_{m,max}) = \int_0^T P_{m,t}\, dt \tag{12}$$

Eventually, the downstream (m stage) flow $X_{m,t}$ can be expressed as the summation of the transformation of $X_{m-1,t}$ and $U_{m,t}$, the pumping rate in the m stage watershed, in Fig. 2.

In the upper stream, it can be expressed as $X_{m,t}=U_{m,t}$, where the $X_{m-1,t}=0$.

$$X_{m,t} = \phi_{m-1} \cdot X_{m-1,t} + U_{m,t} \tag{13}$$

3. Optimization Model

There are three major optimization techniques, namely, linear programming (LP), non–linear programming (NLP), and dynamic programming (DP). Among them, DP technique needs more memories as the number of stages and states increase, but, the complex problems that have non–linearity, stochastic state, and many parameters can be reduced to the multi–stage form. Therefore, DP has usually been used for multi–stage water resource system problems.

3.1. Dynamic Programming Model

The physical arrangement of the detention facilities in this model is described by hypothetical iso–drainage lines (IDL), as shown in Fig. 1. The iso–drainage lines are constructed by starting at the most downstream detention basin location and proceeding upstream, and are numbered starting upstream and proceeding downstream (stage, m).

Assuming a detention pond shape as Fig. 3, then the detention volume V_m is described as a function of depth D_m ($V_m=825\times D^3_m$).

In stage m (=1, M), the state variable and decision variable are defined as the channel flow $X_{m,max}$ and the maximum pumping capacity $U_{m,max}$, respectively. Time t is discretized by a uniform interval Δt (= 5 min.) and is discretized into 120 time steps (120×5 min. = 600 min.). During these time steps, the changes of channel flows and water levels are simulated on all stages.

For the computer application of DP model, the state variable $X_{m,max}$ can be discretized by the finite number L of vector set (Discrete Dynamic Programming; DDP) which defined as $\{X^l_{m,max}\}_{l=1,L}$. If the state variables at the stage $(m-1)$ and (m) are $X^j_{m-1,max}$ and $X^l_{m,max}$, the outflows of these state variables are $X^j_{m-1,max}$ and $X^l_{m,max}$ and the pumping capacity $U_{m,max}$ of these two states are $U^{j,l}_{m,max}$. Then the relation between these variables is given as Eq. (14).

$$U^{j,l}_{m,max} = \underset{t}{\text{Max}}\ [X^l_{m,t} - \phi_{m-1} \cdot X^j_{m-1,t}] \tag{14}$$

Where the $X^j_{m-1,max}$ and $X^l_{m,max}$ may not occur at the same time, so the decision variable $U^{j,l}_{m,max}$ can not be decided directly. For this reason, $U^{j,l}_{m,max}$ should be determined by an iteration method, that is,

<Step 1> First, let $U^{j,l}_{m,max}$ be the difference between two state variables.

$$U^{j,l}_{m,max} = X^l_{m,max} - X^j_{m-1,max} \tag{15}$$

<Step 2> $$X^l_{m,max} - \underset{t}{Max}\ [\phi_{m-1}\ X^j_{m-1,t} + U^{j,l}_{m,t}] \equiv du \tag{16}$$

$$\begin{aligned} U^{j,l}_{m,t} &= U^{j,l}_{m,t}\ (U^{j,l}_{m,max},\ O_{m,t} \\ &= O_{m,t} \qquad (O_{m,t} \leqq U^{j,l}_{m,max}) \\ &= U^{j,l}_{m,max} \quad (O_{m,t} \geqq U^{j,l}_{m,max}) \end{aligned}$$

If $du=0$, then obtain the decision variable $U^{j,l}_{m,max}$ corresponding to the state trajectory (j,l) and stop this iteration.

<Step 3> If $du \neq 0$, then,

$$U^{j,l}_{m,max} = U^{j,l}_{m,max} + du \tag{17}$$

If $\{0 \leqq U^{j,l}_{m,max} \leqq O_{m,max}\}$, then go to <Step 2>,

Otherwise, this state trajectory (j,l) can not satisfy the constraints, and thus it has to be removed from this iteration.

The stage return $R^{j,l}_m$ is a cost function of the detention system that consists of the land cost (CL), construction cost (CM), and cost of pumping system (CP) (Reports for supplementary design, 1991).

$$R^{j,l}_m(X^l_{m,max},\ U^{j,l}_{m,max}) = CL + CM + CP \tag{18}$$

where

$R^{j,l}_m$: The cost of detention system that have pump capacity $U^{j,l}_{m,max}$ and detention pond with depth $D_m(U^{j,l}_{m,max})$, when the channel discharge $X^l_{m,max}$ at stage m.

CL	: The land cost to construct the detention pond with depth $D_m(U^{j,l}_{m,max})$ at stage m, $CL(D_m)=CE_m \times D^2_m \times 900$
CE_m	: The land cost per unit area of the basin at stage m, (Won/m^2).
CM	: The construction cost of the detention pond with depth $D_m(U^{j,l}_{m,max})$ at stage m, $CM(D_m)=CV_m \times D^3_m \times 825$.
CV_m	: The construction cost per unit volume of the basin at stage m, (Won/m^3).
CP	: The cost of acquisition of the pumping system that have the capacity $U^{j,l}_{m,max}$, when the channel discharge is $X^l_{m,max}$ at the stage m. The pump capacity–cost relation is given at the section "4.1. Study Area."

Finally, $V^{j,l}_m$ is determined from $U^{j,l}_{m,max}$ given all trajectory (j,l) that constitutes of the $X^l_{m,max}$ and its allowable $X^j_{m-1,max}$ using Eqs. (11) and (12). $D^{j,l}_m$ and $R^{j,l}_m$ can be calculated and the optimum path j can be determined using the following recursive equation.

$$f_m(X^l_{m,max}) = \underset{U^{j,l}_{m,\max}}{Min} \left[R^{j,l}_m(X^l_{m,max}, U^{j,l}_{m,max}) + f_{m-1}(X^j_{m-1,max}) \right] \tag{19}$$

$f_m(X^l_{m,max})$ means the minimum cost of detention system, when, $X^l_{m,max}$ is the state value at the stage m, and obtain the $V^{j,l}_m$, $D^{j,l}_m$, $R^{j,l}_m$, as V^l_m, D^l_m, R^l_m, then its discharge $X^l_{m,t}=[\phi_{m-1} X^j_{m-1,t}+U^{j,l}_{m,t}]$.

3.2. IDP Method

The IDP is an improved method of DP that has a problem of "curse of dimensionality" (Larson and Casti, 1982) and is an iteration method that applied recursive Eq. to the initial feasible solution for selection of a new solution. The selection of initial feasible solution and the reduction ratio are very important factors that influence the accuracy and computation time. An example of these studies reported is an LP–DP model (Grygier and Stedinger, 1985) that is composed of the selection of initial solution using LP and the computation using DP.

In this study, the initial feasible solution can be calculated from the simple simulation from hydraulic–hydrologic conditions are as follows.

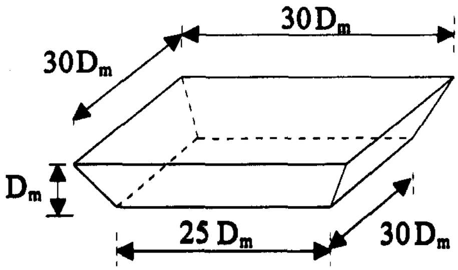

Fig. 3. Assumed Detention Basin Variables

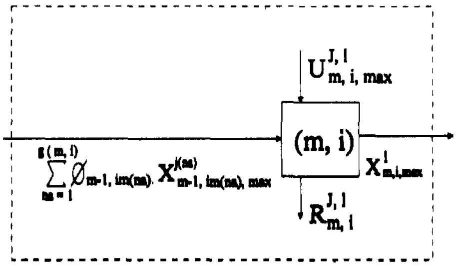

Fig. 4. Typical Stage-wise Structure for Sequential Decision Processes

<Step 1> Calculate the initial solution, and define it as $\{X^0_{m,k}, m=1, M\}$. That is $X^0_{m,max}$ shows $X^0_{m,k}$ on iteration number k ($k=0$).

<Step 2> Solve the recursive Eq. on 3 states, $X^0_{m,k}$, upper allowable state $X^+_{m,k}$, and lower allowable state $X^-_{m,k}$ that have increments $\Delta X_{m,k}$. These values are defined as $\{X^0_{m,k+1}, m=1, M\}$.

$$X^+_{m,k}=X^0_{m,k}+\Delta X_{m,k} \tag{20}$$

$$X^-_{m,k}=X^0_{m,k}-\Delta X_{m,k} \tag{21}$$

<Step 3> If the change rate of $X^0_{m,k}$ and $X^0_{m,k+1}$ $(m=1, M)$ is less than given change criteria $CRIT^{Max}_{m=1,M}$ $|(X^0_{m,k}-X^0_{m,k+1})/X^0_{m,k}| \leq CRIT$, then stop the computation. Else, let $k = k+1$ and store the $X^0_{m,k}$. Thereafter, the $\Delta X_{m,k}$ be reduced as much as the reduction ratio DEC and the feasible solutions are adjusted as $(\Delta X_{m,k}=DEC\times\Delta X_{m,k-1})$, then recomputes the <Step 2> procedure for $X^0_{m,k}$.

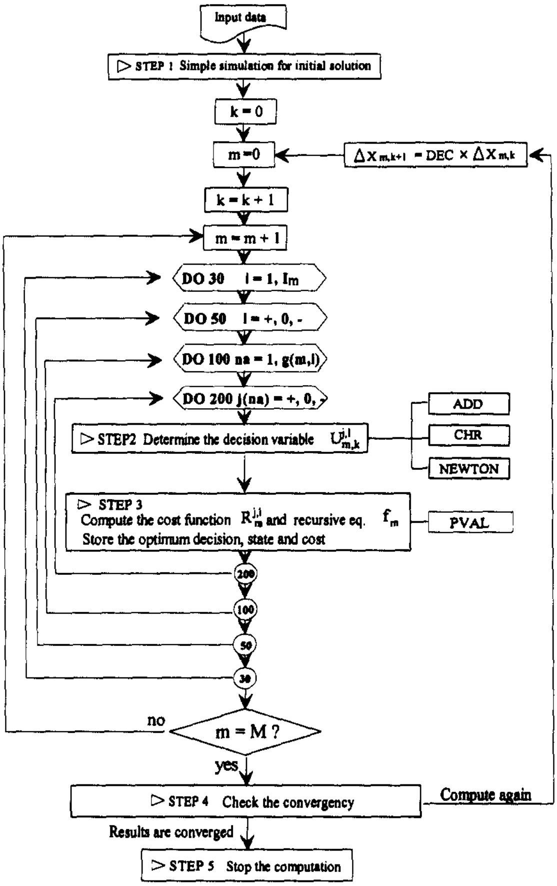

Fig. 5. Flow Chart for Optimum Design Model

If X_m can be described as a vector, $X_m=(X_{m,1}, X_{m,2}, \cdots\cdots, X_{m,i}, \cdots\cdots, X_{m,Im})$, and there are I_m basins in each stage, then the real model can be expanded to consider the flows and confluents of the tributaries. Therefore, Eqs. (13), (14) and (18) can be expanded and the recursive Eq. (19) can be rearranged as Eq. (25).

$$X^{l}_{m,i,t}=\sum_{na=1}^{g(m,i)} (\phi_{m-1,im(na)}\cdot X^{j(na)}_{m-1,im(na),t})+U^{j,l}_{m,i,t}, \ (i=1,I_m) \tag{22}$$

where

$g(m,i)$: number of channels in stage $(m-1)$ connected to the (m,i) confluence.
$im(na)$: channel numbers in stage $(m-1)$ connected to the (m,i) confluence, $na=1,g(m,i)$.
$X^{j(na)}_{m-1,im(na),t}$: runoff on time, when the state of $(m-1, im(na))$ channel be $X^{j(na)}_{m-1,im(na),\max}$.
$U^{j,l}_{m,i,\max}$: pumping capacity of (m,i) detention pond that are linked with two states $X^{j}_{m-1,\max}$ and $X^{l}_{m,i,max}$. Where the states of (m,i) channel connected to the confluence are defined $\{X^{j(na)}_{m-1,im(na),max}\}_{na=1,g(m,i)}$, as $\{X^{j}_{m-1,\max}\}$.

$$U^{j,l}_{m,i,\max}=\underset{t}{\mathrm{Max}}\left[X^{l}_{m,i,t}-\sum_{na=1}^{g(m,i)} \phi_{m-1,im(na)}\cdot X^{j(na)}_{m-1,im(na),t}\right], \ (i=1,I_m) \tag{23}$$

$$R^{j,l}_{m,i}(X^{l}_{m,i,\max}, U^{j,l}_{m,i,\max})=\mathrm{CL}+\mathrm{CM}+\mathrm{CP} \tag{24}$$

$$f_{m,i}(X^{l}_{m,i,\max})=\underset{U^{j,l}_{m,i,max}}{Min}\left[\mathrm{R}^{j,l}_{m,i}(X^{l}_{m,i,max}, U^{j,l}_{m,i,max})+\sum_{na=1}^{g(m,i)} f_{m-1,im(na)}(X^{j(na)}_{m-1,im(na),\max})\right] \tag{25}$$

Therefore, the optimization procedure in Eqs. (23), (24) and (25) and the procedure of programming can be described as in Figs. 4 and 5.

4. Model Application and Analysis of Results

4.1. Study Area

The study area of this study is Jung-Rang Cheon basin in Seoul, a watershed with 6 detention ponds. The Jung-Rang Cheon streams through Uijonbu City, and the downtown areas of Seoul. It confluences with Cheong-gye Cheon and flows into the Han River. Its basin area is 288 km² and channel length is 34.8 km. This basin is very suitable to study for the disaster mitigation system because it has some internal problems of urban flood due to the global urbanization. The physical arrangement of the detention facilities in the basin is described by hypothetical iso-drainage lines, as shown in Fig. 6.

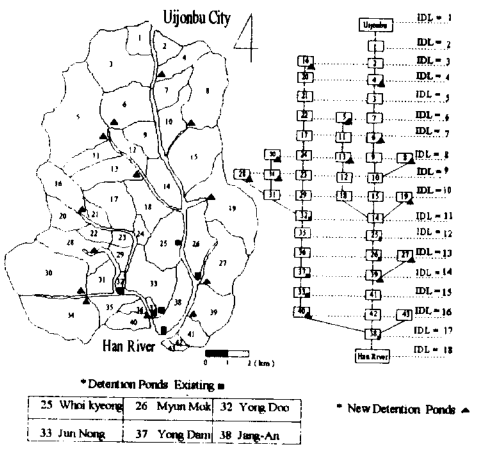

Fig. 6. IDL Applied to Jung-Rang Cheon Detention Basin Systems.

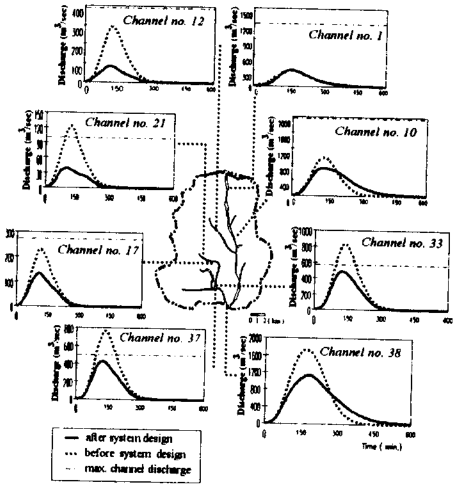

Fig. 7. Hydrologic Phenomena of Total Basin with New Detention System (Case 342)

For the assessment of the detention system in each sub-basin, IDP model is applied as belows:

(1) Assuming that the case of small hydraulic structure required safety, this model simulates with the standard design conditions ('87 report of flood disaster white paper, 1988) that have 20 return years and 0.1 m of safety water level. The rainfall duration to estimate the size of a single detention pond is commonly 2 hours ('87 report of flood disaster white paper, 1988), but, the 3 hours rainfall duration is assumed for the wider area and multi-detention case as in this study.

(2) Moreover the standard design conditions, this study simulates for various designs which are 4 cases of the rainfall frequency (5, 10, 20, 50 years), 5 cases of the rainfall duration (30 min., 1, 2, 3, 6 hours), and 4 cases of the safety water level (0, 0.1, 0.2, 0.3 m). One result compared with the standard design condition is described as case 342 (20 years return, 3 hours duration, and 0.1 m safety water level).

(3) The changes of the detention system and its hydrologic phenomena with the changes of the design conditions and the applicability of this model using IDP method are also examined. The applicability is expressed in terms of both the computation time and memories required.

The land price, one of the important factors to design an optimum detention system, is referenced from the data of Land price statistics (1991) and it be assumed that becomes lower as the land is located farther from the city hall (Table 4). The land acquisition to construct a detention pond near the center of city is very difficult. Therefore, the land cost of the center of city (15×10^5 Won/m^2) is assumed to be 3 times more expensive than the land cost of the border of city (5×10^5 Won/m^2), and it can be classified into 5 groups.

The construction cost per unit volume is assumed 2,250,000 Won, and the pumping system is uniformly classified by the pumping discharges related types and capacities. The pumping capacity clas-

Table 4. Land Cost of Detention Ponds

Rank	Price ($\times 10^5$ Won/m^2)	Basin Number
1	15.0	21,22,28,30,31,32,34,35
2	12.0	11,12,13,14,17,18,20,23 24,25,29,33,36,37,38,40
3	10.0	39,41,42,43
4	7.5	3,5,6,7,9,10,15,16,19, 26,27
5	5.0	1, 2, 4, 8

Table 5. Pump Capacity-Cost Relations

	Pump Capacity (m^3/sec)	Cost ($\times 10^4$ Won)
1	- 0.5	0.0
2	0.5- 10.0	7266.0
3	10.0- 30.0	17532.0
4	30.0- 50.0	30388.2
5	50.0- 70.0	50284.0
6	70.0- 90.0	63105.0
7	90.0-110.0	77926.0
8	110.0-130.0	93140.0
9	130.0-150.0	114961.0
10	150.0-170.0	132782.0
11	170.0-190.0	153603.0
12	190.0-250.0	180086.8

sified into 7 groups limited up to 6,000 m^3/min. (= 100 m^3/sec), where the maximum pumping capacity is 5,340 m^3/min. at Shin-Jung pumping station in Seoul ('87 report of flood disaster white paper, 1988). Moreover this condition, this model can simulate for the exceeded pumping capacity up to double values (12,000 m^3/min. = 200 m^3/sec) which classified into 12 groups [Table 5].

The subprograms, the ADD for simulation of inflow, outflow and confluence at a reservoir and the PVAL for cost estimating of system, are used in this study.

4.2. Results of Design Model Application

The data of channel characteristics (sectional shapes, slope, connection of channels, Manning's coefficient), basin characteristics (time-area, runoff coefficient, land cost), the existing detention ponds and probable precipitation are needed, and they are referenced from many reports (Atlas for safety analysis, 1992; report for supplementary design, 1991).

The computation of this study started with the initial $\Delta X_{m,0}$ that is the 1/3 of the maximum allowable flow Q_m, the reduction ratio DEC of 0.8, and change criteria CRIT of 0.1.

For the result of case 342, the new detention system is given as Fig. 6 and the hydrologic phenomena of total basin is given as Fig. 7. Where the maximum allowable channel discharge is 12,300 m^3/sec, which is not shown in Fig. 7. To decrease the peak flows of channel number 21, 33, 37 shown as the danger zones in the design conditions, the channel discharges of the upper streams (channel number 10, 12) are decreased. The reason why new detentions are gather together in the upper area of each tributary is due to the design conditions of multi-detention system and the channel safety.

The change of hydrologic phenomena with the change of channel safety water level is described as Fig. 8 which means the change of the maximum allowable channel discharge. However, the channel flow is even lesser than the maximum allowable channel flow in channel number 17, the effects of the lower danger zone are delivered upward and the real discharges are controlled.

In Fig. 9 , the change of hydrologic phenomena with the rainfall frequency in channel number 37 is shown, which means the channel discharge is increasing when the maximum allowable channel discharge is fixed.

The results show that the maximum discharge is controlled under the maximum allowable channel discharge by the new detention system as given in Fig. 6 . The results of Table 6 also show the deten-

Table 6. Depth of Detention Determined by New Detention System (unit : m)

Basin No. \ Case	312	322	332	342	352
1	2.7 (–)	–	–	–	2.0 (–)
2	2.5 (–)	2.9 (–)	–	–(19.1)	–(7.5)
3	2.4 (–)	2.2 (–)	–(99.1)	–(130.2)	–(107.9)
4	–	–	–	2.3 (–)	2.5 (–)
5	2.1 (–)	2.5 (–)	2.8 (–)	3.0 (–)	3.4 (–)
6	–(74.7)	2.0 (–)	2.3 (–)	2.5 (–)	–(88.3)
7	–	–(38.5)	–	–	2.1 (–)
8	–	2.1 (–)	2.3 (–)	2.5 (–)	2.8 (–)
9	–(48.0)	–(61.9)	–(58.6)	–(54.5)	2.1 (–)
13	–	2.1 (–)	2.1 (–)	2.3 (–)	–(13.9)
16	–	–	–	2.1 (–)	2.3 (–)
19	–	2.1 (–)	2.4 (–)	2.6 (–)	2.9 (–)
24	–	–	–	–	2.2 (–)
27	–	2.1 (–)	2.3 (–)	2.5 (–)	2.8 (–)
28	–	–	2.2 (–)	2.4 (–)	2.7 (–)
30	2.2 (–)	2.6 (–)	2.9 (–)	3.2 (–)	3.6 (–)
31	–(14.5)	2.1 (–)	–(63.0)	–(33.2)	–(52.4)
34	–(59.6)	–(149.4)	2.3(30.8)	2.5(23.0)	–(81.8)
35	–(12.5)	3.0 (–)	–(15.0)	–(11.3)	–
39	2.5(44.0)	2.4(34.2)	–(45.0)	2.1 (–)	–(25.4)
40	2.4 (–)	2.5 (–)	–(29.4)	3.0 (–)	2.6 (–)
42	2.5 (–)	–	–	–	–
Total Cost ($\times 10^{11}$ Won)	1.60	1.90	2.60	6.64	9.16

( ) : pump capacity (m^3/sec)

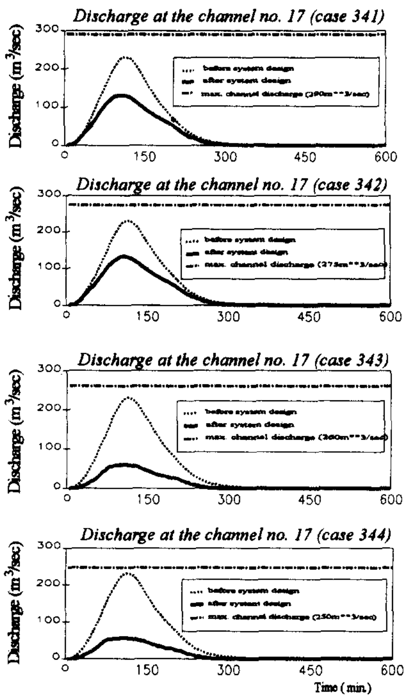

Fig. 8. Hydrologic Phenomena with Channel Safety Water Level.

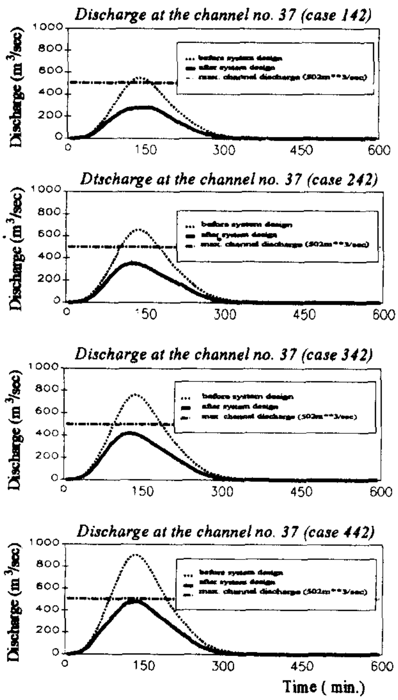

Fig. 9 Hydrologic Phenomena with Rainfall Frequency in Channel No. 37.

tion ponds and pumps are designed complementary. The computation time is about 3 min. per each case using 80486 DX-66.

This model can be used efficiently by the coupling with simulation model to estimate the initial feasible solution, while the estimation of initial feasible solution is very difficult due to the various design conditions.

5. Conclusions

This study suggests an optimization model which determines the optimum size and location of detention-pumping system, efficiently. Moreover, for the successful storm management, this model includes sub-models that are the rainfall-runoff model, the channel and the reservoir routing models. It can design the urban multi-detention system which considers the urbanization by the comparison of the changes of hydrologic phenomena due to the change of design conditions that are the rainfall frequency, the duration and the channel safety water level.

For the application of this model to Jung-Rang Cheon basin, the data of probable precipitation with the various of frequencies and durations, and Huff's rainfall distribution are used. The change of detention system as compared with existing detention system is estimated using the Clark's model for rainfall-runoff model and the Abt-Grigg model for reservoir routing model.

The results show that the locations of new detentions are gather together in the upper area of each tributary due to the design condition of channel safety, but the existing ponds are gather together in the lower area. From the results, this model can be used to design successfully a detention system connecting with the existing detention ponds and can be applied to the basins with complex channel network.

For a supplement to this model, the expansion to the continuous event model or operational design model will be needed, and sensitivity analysis to the economic or legal factors will give more important information for real design guides.

Acknowledgment

This study was supported by funds for open study themes in 1991 provided by the Korea Research Foundation.

References

Atlas for safety analysis of river levees in Seoul and comprehensive countermeasures for flood control. (1992). Seoul Metropolitan Government.

Design standards for small facilities. (1990). Ministry of Construction, Korea.

Grygier, J.C., and Stedinger, J.R. (1985). "Algorithms for optimizing hydropower system operation." *Water Resources Research*, Vol. 21, No. 1, pp. 1-10.

Land price statistics ('90 4/4 session). (1991). Ministry of Construction, Korea.

Larson, R.E., and Casti, J. L.(1982). *Principles of dynamic programming.* Marcel Dekker.

Lee, W.H., Cho, W.C., and Shim, J.H. (1992a). "A new control technique of drainage pump based on fuzzy control." *Korean Society of Civil Engineers*, Vol. 12, No. 3, pp. 107–114.

Lee, J.H., Lee, K.S. and Jeong, D.K. (1992b). "An optimal operation of multi-reservoirs for flood control by incremental DP." *Journal of Korean Association of Hydrological Science*, Vol. 25, No. 2, pp. 47–59.

Lee, W.H., Park, S.D., and Shim, J.H. (1992c). "A safety evaluation of detention reservoirs at Seoul by new pumping criteria." *Proc. Korean Society of Civil Engineers*, Vol. 12, No. 1, pp. 141–150.

Lee, J.T., Yoon, S.E. Lee, J.J., and Yoon, Y.N. (1993). "Design of detention pond and critical duration of design rainfall in Seoul." *Journal of Korean Association of Hydrological Science*, Vol. 26, No. 1, pp. 115–124.

Mays, L.W. (1978). "Sewer network scheme for digital computations." *J. Environmental Eng. Div.*, Vol. 104, No. EE3, pp. 535–539.

Mays, L.W., and Bedient, P.B. (1982). "Model for optimal size and location of detention." *J. Water Resources Planning and Management Div.*, Vol. 108, No. WR3, pp. 270–285.

McCuen, R.H. (1989). *Hydrologic analysis and design.* Prentice–Hall, Inc.

Nemhauser, G.L. (1966). *Introduction to dynamic programming*. John Wiley and Sons, Inc.

Ormsbee, L.E., and Lansey, K.E. (1994). "Optimal control of water supply pumping systems." *J. Water Resources Planning and Management*, ASCE, Vol. 120, No. WR2, pp. 237–252.

Ormsbee, L.E., Houck, M.H. and Delleur, L.W. (1987). "Design of dual purpose detention systems using dynamic programming." *J. Water Resources Planning and Management*, ASCE, Vol. 113, No. WR4, pp. 471–484.

'87 report of flood disaster white paper. (1988). Seoul Metropolitan Government.

Rainfall frequency atlas of Korea. (1988). Report on Development of Techniques for Water Resources Management, Vol. 2, Ministry of Construction, Korea.

Report for supplementary design of Kye–Bong storm drain pumping station. (1991). Kuro–Gu.

Taur, C.K., Toth, G. Oswald, G.E. and Mays, L.W. (1987). "Austin detention basin optimization model." *J. Hydraulic Eng.*, ASCE, Vol. 113, No. 7, pp. 860–878.

Terstriep, M.L., and Stall, J.B. (1974). *"The Illinois urban drainage area simulator, ILLUDAS." Illinois State Water Survey Bulletin 58.*

Time distribution of regional design rainfall (1989). Korea Institute of Construction Technology.

Viessman, W. (1977). *Introduction to hydrology.* IEP–A Dun–Donnelley.

Voorhees, M.L. (1978). "TI–59 calculator program for storm sewer design using rational method." *Storm Sewer System Design*, Univ. of Illinois at U–C, pp. 257–272.

Walesh, S.G. (1989). *Urban surface water management.* John Wiley & Sons.

Wone, S.Y., and Yoon, Y.N. (1993). "A comparative study of urban runoff models." *Proc. Korean Society of Civil Engineers*, Vol. 13, No. 5, pp. 135–146.