KSCE Journal of Civil Engineering (2012) 16(4):660-666
DOI 10.1007/s12205-012-0991-0

Water Engineering
www.springer.com/12205

Optimal Design of Stormwater Detention Basin using the Genetic Algorithm

Minkyu Park*, Gunhui Chung, Chulsang Yoo***, and Joong-Hoon Kim******

Received September 15, 2009/Revised 1st: March 20, 2011, 2nd: June 27, 2011/Accepted July 28, 2011

Abstract

The urbanization of an undeveloped area often shortens outflow travel time and increases peak discharge from the basin, thereby increasing the downstream flood frequency. Stormwater detention ponds are the most common measure to maintain the outflow from the post-developed basin to a flow similar to that under the pre-developed condition. The design criteria of stormwater detention ponds are to minimize construction cost while achieving the flood control purpose. The tedious and time-consuming, trial-and-error method is commonly used to determine the optimal size and location of the pond and outlet structure for a design storm period. In this study, a stochastic search algorithm, a Genetic Algorithm (GA), is used to optimize the detention pond design. The decision variables are the pond storage, and the pipe diameters and number of pipes for the service outlet. The flood control objective considered in this study is that the peak discharge in the post-developed condition does not exceed that under the pre-developed condition and that the maximum water level in the pond during the flood remains below the allowable water level. The proposed optimization algorithm method was applied to the real design of two detention ponds in South Korea, where it generated better design options comprising smaller pond storage and smaller outlet standpipe dimensions than those of the traditional trial-and-error method, and in a much shorter computational time. Therefore, the stochastic search algorithm, GA, can be successfully applied in the design of a stormwater detention basin to improve accuracy and convenience. The engineers can accordingly assess the development plan in terms of the potential basin disaster more efficiently than is possible when using the tedious computation method.

Keywords: *optimal design, stormwater detention basins, service outlet dimension, standpipe, riser-and-barrel*

1. Introduction

Urbanization changes the response of watersheds to precipitation. Peak discharge increases and the time to peak is shortened, resulting in frequent floods and severe channel erosion. Detention basins, retention basins, roof top storage, infiltration basins, and dry wells can be used to reduce the peak discharge and delay the time to peak to levels similar to those under the pre-developed condition. Although different flood control facilities may be used under different circumstances, detention basins (ponds) are the most common. A detention pond can be created by excavating a pond in the existing ground. Often, ponds are constructed by a combination of cut and fill. The outflow from the detention basin depends on the type and size of the outlet structure. A detention basin has at least one primary (service) outlet, such as an orifice, weir, or riser type outlet, to pass the regulated flow from the basin and one secondary outlet (emergency spillway) to pass the overflow above the basin storage during rainy seasons. The secondary outlet is often a weir-type control, separate from the service outlet structure and configured as a part of the detention basin embankment. The detention basin with multiple outlets can also be used to achieve more effective control of stormwater runoff from multi-stage outlet control. Fig. 1 shows an example of a double-outlet detention basin (Akan *et al.*, 2003).

Conventional design method to determine detention basin structure is trial-and-error method which is time consuming, also, the optimal design for the minimum cost is not guaranteed by the conventional approach. Assumptions for outlet dimension by the engineering judgment are also required to the conventional design method. Therefore, Mays and Bedient (1982) recognized the need for optimality in detention basin networks. Taur *et al.* (1987) showed successfully optimized individual basin outlet structures. Papa and Adams (1997) and Behera *et al*. (1999) analyzed stormwater detention basin using an analytical probabilistic model. Zhao (1999) proposed an optimization algorithm using MATLAB's optimization tool box to minimize the total cost including land acquisition, construction, and maintenance. Zhen *et al*. (2004) surveyed optimal location and size of

*Member, Professor, Dept. of Disaster Mitigation and Safety Science, Faculty of Convergence Science, Jungwon University, Chungchung-bukdo 367-805, Korea (E-mail: mkhoin@jwu.ac.kr)
**Member, Senior Researcher, Water Resources Research Division, Korea Institute of Construction Technology (KICT), Kyeonggi-do 306-020, Korea (E-mail: gunhui@kict.re.kr)
***Member, Professor, School of Architectural, Civil and Environmental Engineering, Korea University, Seoul 136-713, Korea (E-mail: envchul@korea.ac.kr)
****Member, Professor, School of Architectural, Civil and Environmental Engineering, Korea University, Seoul 136-713, Korea (Corresponding Author, E-mail: jaykim@korea.ac.kr)

stormwater basins using heuristic optimization techniques and a scatter search algorithm. Travis and Mays (2008) tried to optimize retention basin networks using discrete dynamic programming. These optimization researches focus on the volume or displacements of storm water detention basins. It may be useful in the phase of a master plan for watershed development. However, the detailed design and construction required more specific procedure to determine both volume of ponds and the dimension of outlet facilities. In this case, optimal location and design capacities for the detention basins are decided from the algorithm, but the algorithm was case specific and hard to be generalized.

In this study, the optimal design method to determine the dimensions of pond storage and service outlet, especially for the standpipe (riser-and-barrel) outlet, is suggested. As a stochastic search technique, a Genetic Algorithm (GA) is used in the optimization technique to determine the optimal pond's structure while achieving the flood control objectives. The validity of the proposed optimization approach is evaluated in the actual design of detention basins in South Korea and the results are analyzed.

2. Design of Detention Basins

Specific design criteria for detention basins usually vary from state to state in the U.S., but the general design criteria are common in all cases. Some guidelines can be found in the literature (Yu and Kaighn, 1992; Brown *et al.*, 1996; Stahre and Urbonas, 1990; ASCE, 1996; Loganathan *et al*., 1996; Urbonas and Stahre, 1993).

Figure 1 shows the general structure of a detention basin with two primary outlets and one secondary outlet. Regulated outflow from the primary outlets is discharged from the lower outlet and as the water level rises, the larger outflow is discharged from both lower and higher outlets. When the inflow is too large to discharge through the two outlets and water elevation in the basin reaches the maximum allowable level, an emergency spillway is used to drain the excessive volume of water to downstream.

The conventional procedure for the hydraulic design of a detention basin is a trial-and-error process consisting of the following steps:

1. Calculate the inflow hydrograph from post-developed condition to the detention basin for a rainfall of the design return period being considered. A rainfall-runoff computation model can be used for this purpose.
2. Establish the appropriate hydraulic design criteria. In most cases, the outflow peak from the detention basin in the post-developed condition is required to be reduced to the magnitude of the pre-development peaks for the design return period rainfall. Also, the maximum water surface elevation in the detention basin for the design return period rainfall is required to be lower than the allowable elevation for the structural safety of the detention basin.
3. The initial (trial) design is proposed to satisfy the proposed hydraulic design criteria. The location and size of the detention basin and the sizes and elevations of the outlet structures have to be decided for this initial design. Once this is done, one stage-storage relationship of the detention pond and another one of the outlet structure have to be calculated.
4. The inflow hydrographs are routed through the proposed detention basins and the design criteria have to be checked. If the criteria are not satisfied, go to step 3 and the pond size and outlet dimensions must be modified.
5. The iteration procedure is repeated until the design criteria are met. If the criteria are met, but the outflow peaks are much smaller than that of the pre-developed condition, then the detention basin is over designed. As this is normally not acceptable due to excessive cost, the detention pond design has to be modified again to reduce the total construction cost.

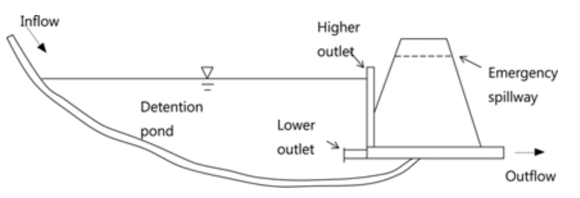

Fig. 1. Schematic Figure of a Detention Basin (Akan, 1993)

In the computationally tedious and time-consuming steps of the traditional trial-and-error method, the design of a detention basin can become a tedious and lengthy task if the trial basin and outlet structures are not chosen properly. In Korea, the critical storm duration for a design return period is recommended to be used in the design of a detention basin. The critical storm duration is defined as the design-storm duration that tends to maximize the detention storage volume for a given return period. The critical storm duration can be found by multiple trial-and-error steps using the local Intensity-Duration-Frequency curves. Therefore, the possible combinations of the design return period and storm durations have to be evaluated to determine the critical storm duration. This complicates the design procedure and hinders the production of appropriate results. Moreover, in practice, the evaluation of various design return periods for the downstream safety greatly increases the number of calculations.

3. Methodology

3.1 Design Considerations of Detention Basins

After determining the size and location of a detention pond, the outflow hydrograph is calculated from the known inflow hydrograph, initial condition, and pond's characteristics. The inflow hydrograph is the runoff from the post-developed basin caused by the rainfall with the design return period and the critical storm duration. The initial condition is the water elevation in the downstream of the detention pond. Those two data are assumed to be fixed and cannot be controlled by the designer. However, the detention pond's characteristics are normally prescribed as the stage-storage relationship of the pond and stage-discharge (outflow) relationship of the outlet structures, which can be controlled from the design procedure. If the stage-storage relationship is fixed in the land-use planning for the detention pond and unchangeable, then the remaining key factor is the stage-discharge relationship that is related to the design of service outlet structure.

3.2 Stage-Discharge Relationship

The outflow from a detention basin is decided by the relationship between the stage and discharge calculated from the outlet structures. The most common type of primary outlet in Korea is the standpipe type of structure which is described by the higher outlet in Fig. 1. This type of structure is also called by riser-and-barrel or morning glory.

The hydraulics of the standpipes is complex but this is well documented in the Guideline of the assessment for disaster impact in Korea (2005). Fig. 2 shows the types of flow condition in the standpipe outlet. The flow conditions are classified into weir flow (Fig. 2a), orifice flow (Fig. 2b), pipe flow with free outfall (Fig. 2c), and pipe flow with submerged tail water (Fig. 2d), depending on the inlet head, flow condition inside the pipe, and outlet head. Discharge from the outlet pipe can be accordingly calculated by the following governing equations:

$$Q = C_w L_w H^{3/2} \qquad \text{weir equationnce} \qquad (1)$$

$$Q = C_o A_o \sqrt{2gH} \qquad \text{orifice equation} \qquad (2)$$

$$Q = \frac{A_g\sqrt{2gH'}}{\sqrt{K_e+K_b+K_f+K_o}} \qquad \text{pipe equation} \qquad (3)$$

When the inlet head is low enough to form a weir flow in the standpipe outlet structure, Eq. (1) is used to calculate the outflow from the outlet, Q. In Eq. (1), C_w is the weir coefficient (1.7 ~1.8), L_w the weir length, which is the corresponding wetted perimeter of the outlet structure, and H the inlet head. As the inlet water level rises, the flow condition changes from a weir flow, and Eq. (2) is used to calculate the discharge from the outlet structure under the orifice condition, where C_o is the orifice coefficient (0.6), A_o the orifice cross-section area, i.e., the area of the vertical pipe in the standpipe, and g the gravitational acceleration. Under the weir and orifice flow conditions, the flow inside the horizontal pipe is assumed as an open-channel flow. As the inlet water level rises further, however, the flow condition inside the horizontal outlet pipe becomes a pressurized pipe flow, which has to be calculated by Eq. (3). Depending on the tail water elevation, the pipe flow can be divided into two conditions, as shown in Figs. 2(c) and (d). If the tail water has a free outfall, the outflow discharge from the pipe is calculated without tail-water effect, whereas if the tail water elevation is higher than the outlet pipe, it substantially affects the discharge from the outlet. To consider the tail water elevation in the governing equation, H' is used. This is the difference between the inlet water level and the center of outlet pipe (0.6D) in the case of Fig. 2(c) and the difference between the inlet water level and the tail water level when the tail water elevation is higher than the outlet of the stand pipe described in the case of Fig. 2(d). The pipe flow condition considers the head losses from the entrance (K_e), bending (K_b), friction (K_f), and outlet (K_o) losses. The cross-section area of the stand pipe for the pipe flow condition is the area of the horizontal pipe (A_g).

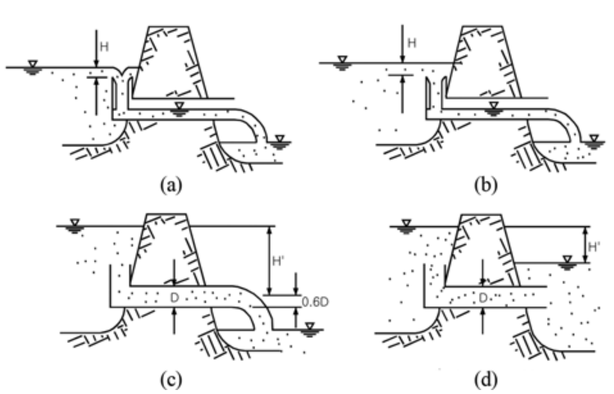

Fig. 2. Flow Classification of Riser-pipe in a Detention Basin (The guideline of the assessment for disaster impact in Korea, 2005): (a) Weir Control, (b) Orifice Control, (c) Pipe Flow with Free Outfall, (d) Pipe Flow with Tailwater Control

The stage-discharge relationship calculated by Eqs. (1)-(3) is shown in Fig. 3. Among the three stage-discharge relationship curves in a stage, the smallest outflow in each stage is selected as the discharge from the outlet.

3.3 Pond Routing

Detention ponds are usually built to reduce downstream flooding, minimize stream bank erosion, and improve water quality. A level-pool routing procedure is typically used to investigate the effects of a detention pond on a runoff hydrograph. As described earlier, the inflow hydrograph, the stage-storage-discharge relationship, and the initial condition of the detention pond are known a priori. The initial condition is the water level in the pond at the time when the incoming flood reaches the detention basin. The change of the storage in a pond is described by the hydrologic routing equation, which is the mass balance equation over the time given the incompressibility of water:

$$I - O = \frac{dS}{dt} \qquad (4)$$

where I is the inflow rate, O the outflow rate, S the storage, and t

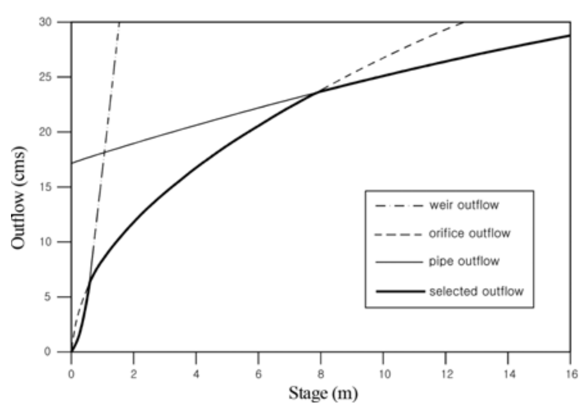

Fig. 3. Stage-discharge Relationship of the Stand-riser Pipe in a Detention Basin

the time.

For a finite time period, Δt, Eq. (4) can be written in finite difference form and rearranged as:

$$(I_1+I_{22})+\left(\frac{2S_1}{\Delta t}-O_1\right)=\frac{2S_2}{\Delta t}-O_2 \tag{5}$$

Equation (5) is the mass balance during time period Δt, that is, the storage difference (S_2-S_1) during time period Δt is the same as the difference between the average inflow to the basin ($(I_1+I_2)\Delta t$) and the average outflow from the basin ($(O_1+O_2)\Delta t$) during time period Δt. As mentioned earlier, as the initial condition of the detention pond (O_1 and S_1) and inflow hydrograph into the pond is assumed to be known, S_2 and O_2 can be calculated from the stage-discharge relationship and Eq. (5). However, in most cases, the storage-discharge relationship is not in equation form but in graph form, as in Fig. 3. Thus, a graphical or semi-graphical procedure is needed, and is summarized as follows (Akan, 1993):

1. From the given stage-storage and stage-discharge relationship, obtain a storage-discharge (S versus O) relationship for the designed outlet structure (Fig. (3)).
2. Select a time increment Δt for outflow calculation. Then, calculate the quantity $(2S/\Delta t)+O$ from the stage-discharge relationship as a function of O and prepare a plot between O and $(2S/\Delta t)+O$.
3. For any time step computation, calculate I_1+I_2 from the inflow hydrograph, and $(2S_1/\Delta t)-O_1$ from either the initial conditions or previous time step calculations.
4. Calculate $(2S_2/\Delta t)+O_2$ from Eq. (5).
5. Obtain O_2 from the graph developed in step 2. This will be the outflow rate at time t_2.
6. To proceed to the next time step, calculate $(2S_2/\Delta t)-O_2$ by subtracting $2O_2$ from $(2S_2/\Delta t)+O_2$ calculated in step 4 and go back to step 3 again. Obviously, the value of $(2S_2/\Delta t)-O_2$ calculated at any time step will become $(2S_1/\Delta t)-O_1$ for the next time step.
7. Repeat the same procedure until the routing is completed.

3.4 Optimization Algorithm to Determine Outlet Dimensions

The optimal design of a stormwater detention basin often equates to minimizing the total cost, which can be related to the land acquisition and construction costs. Therefore, if the minimum pond storage and pipe diameter of the service outlet are obtained while meeting the hydraulic constraints, the cost is assumed to be minimized. In order to determine the optimal pond storage and outlet dimensions using GA, the objective function and constraints are defined as follows:

$$\text{Minimize Cost}\left(V+\sum_{i=1}^{n}(D_1^i+D_2^i)\right) \tag{6}$$

subject to:

$$Q'_p \le Q_p \tag{7}$$

$$Z'_p \le Z_p \tag{8}$$

In the objective function, COST( ) is the annual cost of stormwater detention basin construction. This refers to research results of Sample *et al.* (2003). We used the diameters of the pipe (vertical pipe diameter (D_1) and horizontal pipe diameter (D_2) of standpipe i) and the number of standpipes (n) as the only decision variables. Pond storage volume (V) is considered in the pond routing calculation to satisfy constraint conditions. The number of pipes and the diameter of pipes are integer because the commercial sizes of pipe diameters are considered in the study. Since GA can handle integer variables, the decision variables can be implemented in GA simultaneously. Once these variables are selected, the outflow from the detention basin can be calculated using the stage-discharge relationship (Fig. 3). Pipe diameters of the standpipes are selected from the set of commercial pipe dimensions. The peak outflow from the detention basin (Q'_p) should be less than or equal to the peak discharge from the predeveloped condition (Q_p). Also, the maximum water level in the detention basin during the flood (Z'_p) has to be lower than or equal to the allowable water level (Z_a), which is defined as the elevation below the freeboard from the top crest of the embankment of the basin. The freeboard ensures that the detention basin is safe under the inflow hydrograph with a higher return period than the design return period. Due to the nonlinearity of the routing procedure to calculate the outflow from the detention pond, the problem is solved using a nonlinear programming with discrete decision variables.

Solutions to be mentioned by Eqs. (7), (8) and (9) can be obtained using the GA algorithm. Genetic algorithm is a global search technique, modeled after the process of natural selection, which can be used to find near optimal solutions to highly nonlinear optimization problems. It has become one of the most widely used techniques for solving a number of hydrology and water resources problems (Wang, 1997; Khu *et al.*, 2001) The general application of this algorithm is as follows. First, an initial population of solutions to the problem is generated and ranked according to their objective function values. Subsequently, these solutions undergo a selection process to identify those that will generate the next generational population through crossover and mutation. This cycle is repeated until a termination criterion is met. In this study, the selection process has two-step procedure.

Figure 4 shows the detailed procedure of the proposed GA algorithm-based method to determine optimal pipe dimension and pond size. The detailed procedure into the following two steps:

Step 1: *Initialization*. GA algorithm parameters are initialized. Genotypes (i.e., solution matrix of pipe diameters) are then randomly generated from the possible variable bounds. Here, one component of initial genotypes consists of the first horizontal pipe diameter, the first vertical pipe diameter, the second horizontal pipe diameter and the second vertical pipe diameter. In this study, the maximum number of pipes is assumed as two. If necessary, this maximum number may set to be adjusted. Initial value for the storage of the pond shall be given by the designer according to Korean design practices, this size value of pond will not change under the constraints are satisfied. If the

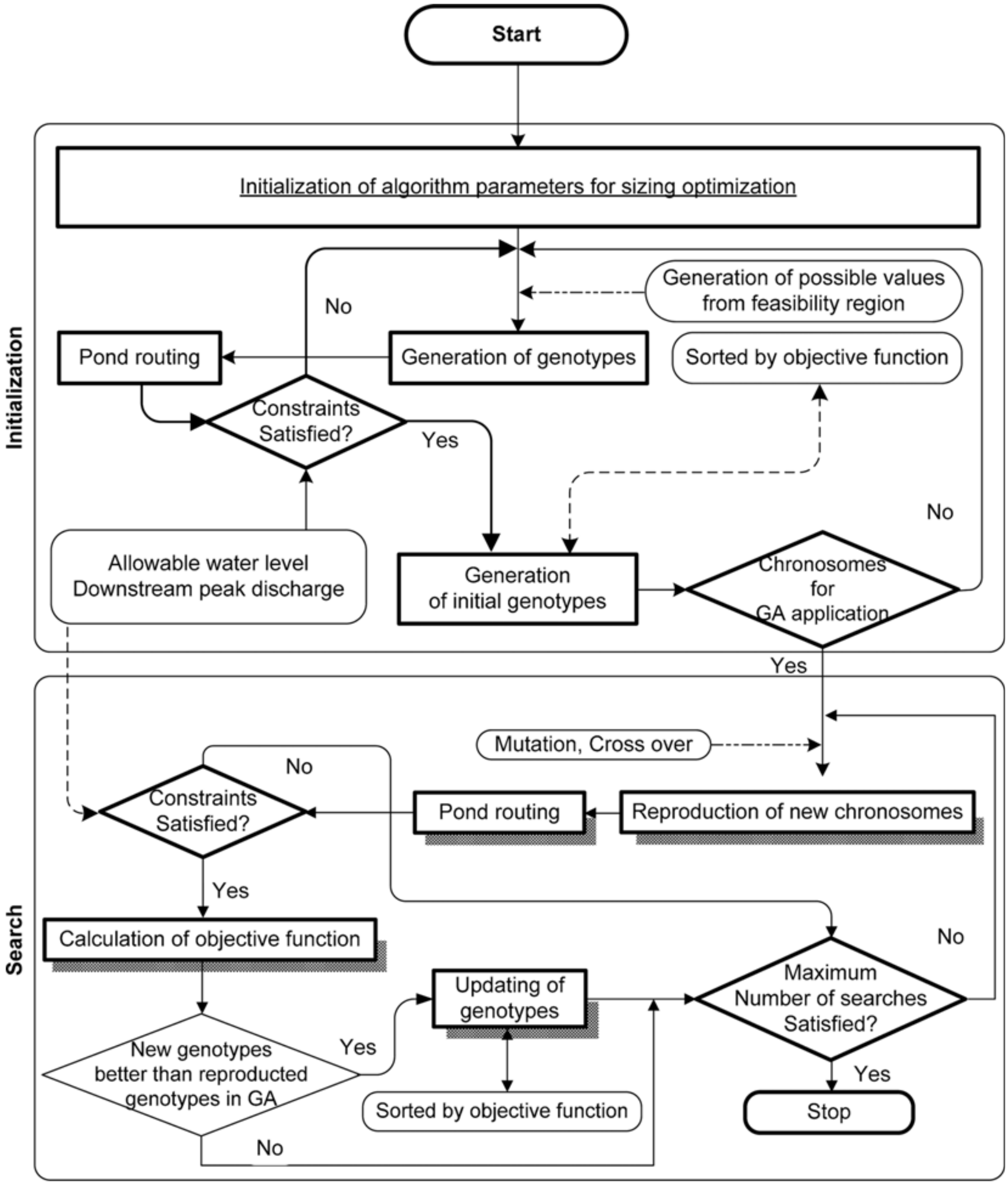

Fig. 4. GA Algorithm for Optimal Stormwater Detention Basin Design Procedure

optimal values are not obtained because of the constraint violation in spite of lots of trials, then the storage can be adjusted. The initial genotypes are sorted by the objective function values of Eq. (6) subjected to the satisfaction of the constraints.

Step 2: *Search*. A new genotypes is improvised from the initially generated genotypes or possible variable values using crossover and mutation. These parameters are introduced to allow the solution to escape from local optima and to improve the global optimum prediction in the GA algorithm. The new genotype is analyzed using the pond routing, and its fitness is evaluated using the constraint functions. If satisfied, the objective function is applied. If the new genotype is better than the previous worst genotype, the new genotype is included in the population and the previous worst harmony is excluded from the population. The population is then sorted by the objective function value. The computations terminate when the maximum number of the search criterion is satisfied. If not, this step is repeated.

4. Application

4.1 Study Area

In order to test the proposed optimization scheme, two previously designed detention ponds located in Chungchugbuk-do, Umsung-gun, South Korea (Fig. 5) were selected. The construction of a county club was planned in this area and based on the assessment of the disaster impact on the area due to this development, two detention ponds were suggested to reduce the impact. Table 1 lists the change of peak discharge from the sub-

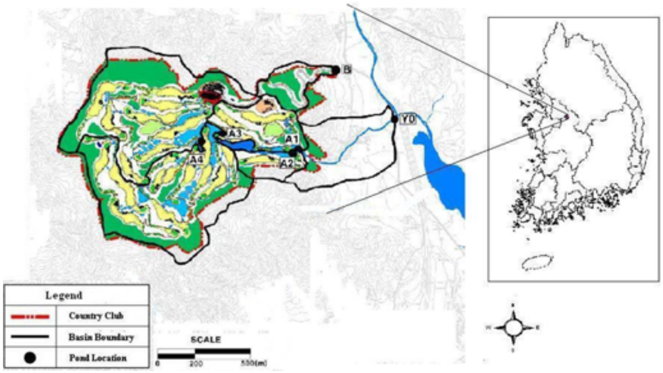

Fig. 5. Location Map of the Study Area

Table 1. Assessment Results of the Disaster Impact of the Study Area

Pond	Pre-development		Post-development		
	Watershed area (km^2)	Peak discharge (m^3/s) ①	Watershed area (km^2)	Peak discharge (m^3/s) ②	① - ②
A1	0.165	4.53	0.159	5.83	(+)1.30
A2	0.131	3.28	0.072	2.32	(-)0.96
A3	0.686	19.46	0.575	19.04	(-)0.42
A4	0.360	10.90	0.544	15.28	(+)4.38
B	0.110	3.11	0.087	2.52	(-)0.59

basins for possible locations of the detention basins. After the development, the sub-basin boundaries were changed due to the change in the flow paths. The peak discharges in the post-development condition increased in the location of ponds A1 and A4, but decreased in the area of ponds A2, A3, and B due to the reduction of the watershed area. Therefore, only the detention ponds A1 and A4 were subject to the optimization using the proposed method to reduce the peak discharge.

4.2 Results and Discussion

To optimize the detention pond design, the parameters for the GA such as the population size, crossover rate, and mutation rate have to be selected. Since these optimization parameters substantially influence the efficiency of the optimization algorithms, the sensitivity of these parameters was analyzed to determine the best value. As a result, the total population size was 200, and the crossover and mutation rates were selected as 0.86 and 0.05, respectively.

The proposed pond size and service outlet dimensions from the actual design are shown in Table 2. These design dimensions satisfied the two predefined constraints. Although, these results may not be the optimal design in terms of cost-effectiveness, this level of results is acceptable in practical aspect. Table 2 compares the real and optimized designs. The optimized pond storage was smaller than that of the actual design and the optimized service outlet dimensions were also smaller than the actual ones. According to Table 2, the optimized construction cost was 5.4% less than that of the actual trial-and-error design in pond A1. In pond A4, the results showed more economic than the trial-and-error design by 12.0%. Although limited cases, the application of optimization techniques showed greater effect in the larger pond capacity. In general, the trial and error method is difficult in consideration of the cost-effectiveness because of complex design constraints during the design process. Also, conservative consideration of the safety in large ponds makes those facilities to be more uneconomical. In contrast, the method proposed in this study demonstrated significant reduction in the time and efforts as well as cost-effectiveness in the design. Therefore, this approach can provide guidelines to the developer on the minimum feasible cost under the development condition within a much faster computation time than the traditional, trial-and-error method.

5. Conclusions

Stormwater detention basins are the most common flood control measures to reduce the peak outflow hydrograph and shorten the peak time. However, in practice, determining the location and size of ponds and outlet structures is tedious and time-consuming due to the repetitive nature of the conventional trial-and-error method. The additional complexity introduced by the combination of return period and storm duration further complicates the computation. In order to overcome these limitations of the traditional methods, a stochastic search algorithm, GA, was used to optimize the pond size and outlet structure with minimum cost. The decision variables were the storage of detention ponds, the number of outlet structures and the pipe diameter of the standpipe outlet. Two hydraulic constraints were applied in the optimization algorithm: (1) the peak discharge from the detention pond must be less than or equal to that from the pre-developed condition, and (2) the maximum water level of the detention ponds during a flood has to be less than or equal to the maximum allowable level. The proposed optimization approach was applied in an area containing two detention ponds to control

Table 2. Comparison Results

Pond	Actual design results			Optimization results		
	Pond size (m^3)	Service outlet dimension (D_1, D_2: mm, N: EA)	Construction cost ($\times 10^6$ Won)	Pond size (m^3)	Service outlet dimension (D_1, D_2: mm, N: EA)	Construction cost ($\times 10^6$ Won)
A1	5,322	D_1 1,200 D_2 1,000 N 1	817.4	4,911	D_1 1,000 D_2 900 N 1	773.3 (5.4% reduction)
A4	24,045	D_1 2,000 D_2 1,500 N 1	2,313.9	20,011	D_1 1,800 D_2 1,500 N 1	2,036.5 (12.0% reduction)

a potential flood after the development of a country club. The proposed method optimized a design with smaller detention ponds than those of the actual designed pond. In addition, a comparison between the actual designed values and those of the proposed optimization method revealed that the GA optimization results can minimize the cost whiling meeting the hydraulic constrains within a faster time than the conventional trial-and-error method.

Acknowledgements

This work was supported by the National Research Foundation of Korea (NRF) grant funded by the Korea government(MEST) (No. 20100026519).

References

Akan, A. O. (1993). *Urban stormwater hydrology-A guide to engineering calculations*, Technomic, Lancaster, PA, ISBN:0-87762-966-6.

Akan, A. O. and Houghtalen R. J. (2003). *Urban hydrology, hydraulics, and stormwater quality*, Technomic, Lancaster, PA, ISBN:0-471-43158-3.

American Society of Civil Engineers (1996). *Chapter 9 urban hydrology in hydrology handbook*, ASCE Manuals and Reports on Engineering Practice No. 28, New York, NY.

Behera, P. K., Papa, F., and Adams, B. J. (1999). "Optimization of regional storm-water management systems." *Journal of Water Resources Planning and Management*, Vol. 125, No. 2, pp. 107-114.

Brown, S. A., Stein, S. M., and Warner, J. C. (1996). *Urban drainage design manual*, Federal Highway Administration, Hydraulic Engineering Circular No. 22, Washington, D.C.

Khu, S. T., Liong, S. Y., Babovic, V., Madsen, H., and Muttil, N. (2001). "Genetic programming and its application in real-time runoff forecasting." *Journal of the American Water Resources Association,* Vol. 37, No. 2, pp. 439-451.

Longanathan, G. V., Kibler, D. F., and Grizzard, T. J. (1996). *Urban stormwater management in handbook of water resources engineering*, Mays, L. (ed.), McGraw-Hill, New York, NY.

Mays, L. W. and Bedient, P. B. (1982). "Model for optimal size and location of detention." *Journal of Water Resources Planning and Management*, Vol. 108, No. 3, pp. 270-285.

National Emergency Management Agency (2005). *Guideline of the assessment for disaster impact in Korea*, NEMA Guideline No. 2005.

Papa, F. and Adams, B. J. (1997). "Application of derived probability and dynamic programming techniques to planning regional stormwater management systems." *Water Science and Technology,* Vol. 36, No. 5, pp. 227-234.

Sample, D. J., Heaney, J. P., Wright, L.T., Fan, C-Y., Lai, F-H., and Field, R. (2003). "Cost of best management practices and associated land for urban stormwater control." *Journal of Water Resources Planning and Management*, Vol. 129, No. 1, pp. 59-68.

Stahre, P. and Urbonas, B. (1990). *Stormwater detention,* Prentice-Hall, Englewood Cliffs, NJ.

Taur, C., Toth, G., Oswald, G. E., and Mays, L. W. (1987). "Austin detention basin optimization model." *Journal of Water Resources Planning and Management*, Vol. 113, No. 7, pp. 860-878.

Travis, Q. B. and Mays, L. W. (2008). "Optimizing retention basin networks." *Journal of Water Resources Planning and Management,* Vol. 134, No. 5, pp. 432-439.

Urbonas, B. and Stahre, P. (1993). *Stormwater: Best management practices and detention for water quality, drainage and CSO management*, McGraw-Hill, New York, NY.

Wang, Q. J. (1997). "Using genetic algorithms to optimize model parameters." *Environmental Modeling and Software*, Vol. 12, No. 1, pp. 27-34.

Yu, S. L. and Kaighn, Jr. (1992). *VDOT Manual of Practice for Planning Stormwater Management*, Virginia Transportation Council, Charlottesville, VA.

Zhao, B. (1999). "Cost minimization for detention basin system design." *29th Annual Water Resources Planning and Management Conference*, Tempe, Arizona, USA.

Zhen, X.-Y., Yu, S. L., and Lin, J.-Y. (2004). "Optimal location and sizing of stormwater basins at watershed scale." *Journal of Water Resources Planning and Management*, Vol. 130, No. 4, pp. 339-347.