

Evaluation of the Complicated Storm Sewer Networks Design Method and the Rational Method Using a Risk-Based Simulation-Optimization Approach

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Evaluation of the complicated storm sewer networks design method and the rational method using a risk-based simulation-optimization approach

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Abstract

In this study, optimal designs with minimum costs are obtained for various storm return periods. Then the risk analysis is used to determine the return period in which the design cost plus the damage risk cost is minimum. SWMM software was used to handle the simulation and the Network optimization was performed by using the genetic algorithm. The non-linear reservoir model to convert the rainfall into runoff and the dynamic wave model to perform the network hydraulic simulation in this software are utilized as a complicated simulation model. The results showed that the 10-year return-period storm in which the summation of the design and the damage risk costs are minimum is the one that should be selected. Also, the rational method of the software was applied as the simplest method of rainfall-runoff and the hydraulic calculations were performed using a Manning equation without considering the flow travel time. The results show that the return period of the risk analysis is the same as the first one whereas the total design costs are greater by 16.6%. Afterward, the classical rational method in which the flow travel time is considered was used to design the same network. The peak flows of the pipes were remarkably reduced, causing the design costs to be only 4.7% greater than the complicated precise method. It can be concluded that the simple classic rational method considering the flow travel time may be used in the design of storm sewer networks due to its acceptable accuracy and costs.

Keywords: Genetic Algorithm, SWMM Model, Complicated Method, Rational Method.

1. Introduction

Population growth, urban development, agricultural and industrial activities, and the conversion of natural lands to residential areas result in a change in the natural hydrologic cycle of the

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region (Coffman, 1999). Also, the increase of impervious areas significantly has altered the pre-urban hydrology and increased the surface run-off in urban environments compared to the undeveloped natural environments. Implementation of the storm sewer networks is one of the effective ways to manage water resources. The optimal design of storm sewer networks is one of the most important issues in water engineering since they are very costly. Any attempt to reduce the construction costs of these networks can lead to significant savings. The design of storm sewer networks is always associated with risk. In a proper design, there must be a balance between the cost of the design and the risks that may occur in the future. Therefore, both optimization and risk analysis should be considered in the optimal design. The mathematical formulation of the storm sewer network optimization leads to the conversion of the problem into a constrained nonlinear optimization problem. Therefore, appropriate methods should be used. In recent years, the optimization of storm sewer networks has received remarkable attention. Sadat Darbandi et al. (2011) presented a new model for the optimal design of storm sewer networks. They combined the genetic algorithm as the optimizer and the transport module from the SWMM software as the simulator. The successful application of the model in the examples considered by various researchers proves the efficiency of the developed model. Afshar (2006) used two constrained methods of ant colony optimization algorithm to design storm sewer networks, considering node elevation as a decision variable. His results showed that the proposed method is not sensitive to the population and the search space. Pan and Kao (2009) combined a genetic algorithm and second-order programming methods to propose a model called GA-QP for the optimal design of storm sewer networks. They considered the diameter of the pipes and the location of the pumping stations as the decision variables. Their results demonstrated good performance. Zeferino et al (2010) presented a multi-objective model for the problem of optimal

design of regional wastewater systems plane. They handled the model by a weighting method and solved it through a simulated annealing algorithm. The proposed multi-objective model includes three objectives: minimization of capital costs, minimizing maintenance costs and maximizing of dissolved oxygen. The result of their research showed that the model has good convergence speed and was able to optimize the wastewater network well. Sun et al (2011) developed a risk-based optimal design approach for storm sewer networks with a multi-objective genetic algorithm. The results of their optimization proved a good performance of the model. Haghghi and Bakhshipour (2012) followed the optimal design of a large-scale sewer network with 79 branches and 80 nodes, which covered a residential area of 260 hectares with an adaptive genetic algorithm. To satisfy the constraints, they used the loop-by-loop cutting algorithm in which the operators of the genetic algorithm were compatible with the constraints of the problem. The occurrence of urban floods and the resulted damages in Iran is increasing, and consequently, the required budget to control and reduce the damages also increases. On the other hand, due to financial constraints, the dedicated budget for flood control may not be enough. Therefore, it is necessary to find the best optimal alternative with the reasonable highest efficiency in the design of storm sewer networks. Also, one of the objectives of this study is to develop proper relationships to evaluate the green area damage and the creation of traffic damages due to runoff. These relationships are used in the objective function to evaluate damage costs in the genetic algorithm optimization analysis. The decision variables are the diameters and slopes of the pipes. The optimization solver is linked to the simulation SWMM model to obtain the optimal design.

One of the precise methods to calculate the runoff in storm sewer design is the nonlinear reservoir model used in the SWMM program in which non-linear differential equations have to

be solved. This method is regarded as a complicated one compared to the others. The rational method to calculate the runoff in storm sewer design systems is very simple in concept and application compared to the complicated one. The rational method available in the SWMM software does not consider the flow travel time in the conduits. Thus, it produces large flow discharges in the conduits. The classic rational method considers the flow travel time in the conduits which results in greater time of concentration and consequently lower flow discharges.

There are three common methods to handle the hydraulic analysis of the flow in the conduits; the dynamic wave, the kinematic wave, and the Manning equation methods. These methods are also available in the SWMM software. Let's call the method in which the runoff is calculated by the nonlinear reservoir model and the conduit hydraulic analysis is done by the dynamic wave method as the complicated method, the rational method in which the flow travel time in pipes is regarded as the simple one, and the rational method considering the travel time as the classic one. The complicated method results in the most precise design. The rational method leads to greater runoffs and consequently larger conduit sizes which in its turn increases the costs whereas, the classic rational method results in lower runoffs due to the increase in the time of concentration because of considering the flow travel time in the conduits. This yields smaller conduit sizes compared to those obtained by the rational method in which the travel time is not regarded. How much is the difference in the design costs of these methods compared to the complicated ones? This is investigated in this research by using a case study in a local region in Tehran city.

The damage costs are required for the risk analysis. Relationships for the damage costs of the land use, infrastructure, and traffic, are provided for the mentioned case study. The genetic algorithm is utilized as an optimization tool to find the decision variables (pipe diameters, and slopes) so that the summation of the design costs will be minimized. In this regard, the

complicated method and both the rational methods (with and without) considering the flow travel time in the conduits are used and the results are compared

2. Introducing the SWMM model

The SWMM software was developed by the United States Environmental Protection Agency in collaboration with Metcalfe & Eddy Engineering and the University of Florida from 1969 to 1971 to simulate the quantitative and qualitative associated aspects with runoff pipes.

The required data is very extensive and includes physiographic data of sub-catchments, specifications of system structures, system maintenance information, discharge intensity in the dry period, rainfall information, flood hydrographs, and flow quality in conduit. The surface Water Management Model is a dynamic model of rainfall-runoff simulation and can simulate the quality and quantity of runoff for urban areas for one single event or continuous one. It can also be combined with other models. Since this model is widely used to design, analyze and estimate the cost of constructing a drainage network system in urban areas, in this study, this hydrological-hydraulic model was selected. Because SWMM is a distributed model it allows a study area to be subdivided into any number of irregularly shaped sub-catchment areas to best capture the effect that spatial variability in topography, drainage pathways, land cover, and soil characteristics have on runoff generation.

2.1. Calculation of surface runoff in SWMM model

In the complicated method, SWMM uses a nonlinear reservoir model to estimate the surface runoff produced by rainfall over a sub-catchment. Using a mass balance, the net change in depth

d per unit of time t is simply defined as the difference between inflow and outflow rates over the sub-catchment:

$$\frac{\partial d}{\partial t} = i - e - f - q \quad (1)$$

Where:

i is the rainfall intensity including snowmelt (m/s), e is the surface evaporation rate (m/s), f is the infiltration rate (m/s), q is the runoff rate $= \frac{Q}{A_s}$ (m/s), A_s is a sub-catchment area (m²), and Q is the runoff's flow rate (m³/s).

The runoff flow rate is obtained using the Manning equation given in Equation (2):

$$Q = W \cdot \frac{1}{n} (d - d_p)^{5/3} S^{1/2} \quad (2)$$

Where W is the width of the sub-catchment in meters, n is the Manning roughness coefficient, d_p is the depression storage depth in meters, d is the depth, and S is the slope of the sub-catchment. Figure 1 illustrates these variables (Rossmann, 2016).

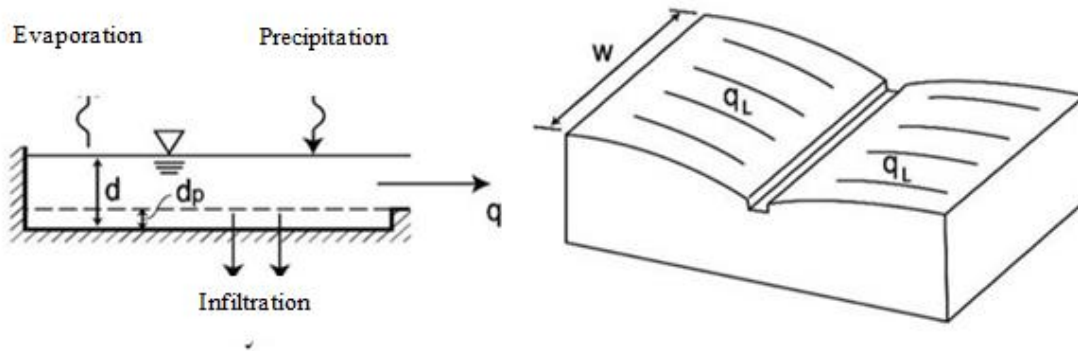


Fig. 1 Nonlinear reservoir model of a sub-catchment

By combining equations 1 and 2, a nonlinear differential equation (Equation 3) is obtained.

$$\frac{\partial d}{\partial t} = i - e - f - \alpha (d - d_p)^{5/3}, \quad \alpha = \frac{wS^{1/2}}{A_s n} \quad (3)$$

Equation (3) is an ordinary nonlinear differential equation. This equation is solved by an iterative numerical method in the SWMM model (Rossman, 2016). The runoff is then determined by using the Manning equation for every time step.

2.2. Hydraulic analysis methods in SWMM

The data given to the model are the rainfall histogram, meteorological input data, basin system, and the drainage network. These data are employed to produce the output hydrograph (Sin et al, 2014). This applies to all sub-catchments with the necessary routing using the continuity, and momentum equations. To solve these equations, the SWMM user can use three methods of steady flow, dynamic wave routing, and kinematic wave. The equations of St. Venant are presented below:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \quad \text{Continuity} \quad (4)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (\frac{Q^2}{A})}{\partial x} + gA \frac{\partial H}{\partial x} + gAS_f + gAH_L = 0 \quad \text{Momentum} \quad (5)$$

In the above relations, x is the distance from the beginning of the conduit, A is the flow cross-sectional area, Q is the flow rate, t is the simulation time, H is the water head in the conduit, S_f is the friction slope, H_L is the local head-loss, and g is the acceleration due to gravity. The dependent variables in these equations are the flow rate, Q , and the water head, H , which are functions of the distance, and time, t . Solving the equations of St. Venant in an independent channel requires the initial conditions of H and Q at zero time and the boundary conditions at $x = 0$, and $x = L$, for all times.

The complete form of St. Venant equations is solved in the dynamic wave method and thus produces the most theoretically accurate results. The method accounts for conduit storage, backwater effects, entrance/exit losses, flow reversal, and pressurized flow. The time step should be so small to maintain numerical stability. Numerical instability is characterized by oscillations in the flow and water surface elevation that do not dampen out over time (Rossman, 2016). Stable explicit solutions of the St. Venant equations require smaller time steps than those it takes for a dynamic wave to travel down the length of the conduit (Cunge et al., 1980). In the kinematic wave method, the friction and gravity terms are only considered. SWMM also offers a steady flow analysis option which assumes that within each computational time step, the flow is uniform and steady. It simply translates inflow hydrographs with no delay or change in shape. The Manning equation is used to relate flow rate to flow area (or depth). Because it ignores the dynamics of free surface wave propagation it is only appropriate for rough preliminary analysis of long-term continuous simulations (Rossman, 2016).

Many studies have been done that confirm the accuracy of the dynamic wave model method, such as (Helmio, 2005), (Zhang, 2005), (Anderson et al, 2006), (Kuiry et al 2010), (Venutell, 2011) and (Akbari and Barati, 2012).

2.3.The rational method in the SWMM model

One of the most commonly used methods for calculating peak flows from small catchments is the rational method. This method is based on the fact that in a catchment, the maximum flow is obtained from rainfall with duration equal to the time of concentration. This method is sometimes used in preliminary screening-level models to generate runoff flows from long-term rainfall records or rainfall probability distributions with a minimum of site-specific data required

[see STORM (Corps of Engineers, 1977); NetSTORM (Heineman, 2004); Adams and Papa, 2000)]. The maximum flood discharge in a catchment is obtained based on the rational method using the following equation:

$$Q = \frac{1}{\alpha} CIA \quad (6)$$

In the above relation, Q is the peak discharge, I is the rainfall intensity, C is the runoff coefficient, A is the catchment or sub-catchment area, and α is a constant coefficient depends on the selected units of the equation.

To employ this method in SWMM the followings are implemented (Rossman, 2016):

1. Set the sub-catchment's percent imperviousness to 100C and its percent of imperviousness with no depression storage to 0.
2. Assign the same depression storage depth to both the pervious and impervious areas.
3. Use any values for slope and width, and 0 for both the pervious and impervious Manning's n.
4. Use the Horton infiltration option and let its maximum and minimum infiltration rates be the same.

Setting up a model in this form will produce the same results as if Equation (6) were implemented directly. When the Manning roughness coefficient, n, is 0, SWMM bypasses Equation 3 and simply converts all rainfall excess at each time step into instantaneous runoff (Rossman, 2016).

A hydrograph is calculated based on the rational method and transferred through the network conduits without considering the flow travel time and summed with the other hydrographs

obtained for the following sub-catchments. The Manning equation is employed for the conduit hydraulic analysis in the SWMM model.

3. The classical rational method

In the classical rational method, the peak flow is only determined considering the flow travel time through the conduits to determine the time of concentration. This leads to greater time of concentration and consequently lower rainfall intensities and peak flows compared to the rational method used in SWMM software. This method is widely used. In 1975, a survey among 37 Canadian municipalities revealed that more than 97% of them used the rational method for the design of the storm sewer systems (Badea and Bacotiu, 2002). Many types of research have been done which proves the ability of the rational method such as (Singh and Cruise, 1992), (Hue et al, and 2003), (Mega et al, 2019) and (Shad and Hoveida, 2015). The Manning equation is used for the hydraulic analysis in network conduits.

4. Optimization model

In this research, the genetic algorithm (GA) is used to minimize the total design cost. GA starts to optimize the problem with an initial population of chromosomes which are randomly generated at the beginning. The chromosomes evolve through successive iterations namely generations in GA (Gen and Cheng 1997). For more details about the genetic algorithm in water and wastewater engineering, readers are referred to the optimization of water distribution networks of different researchers (Dandy et al, 1996; Yaghi et al., 1999; Simpson et al., 1994; Wu and Simpson, 2001).

The objective function used in the optimization model is:

$$\text{minimize Cost}(D, S) = \sum_{i=1}^{NP} CP_i + \sum_{j=1}^{NM} CM_j \quad (7)$$

Where CP and CM are the costs of construction of sewers pipes and manholes, respectively. NP and NM are the number of the pipes and manholes, respectively.

To estimate the construction costs of the network, the aggregate official price list of the storm sewer networks of 2020 was used. The following equations are obtained based on real expenses data by using the curve fitting technique:

$$CP = 199.3D - 121E - 192.7D^2 + 161.2DE - 0.4781E^2 + 24.05 \quad (8)$$

$$CM = 55.872H^2 + 242.99H + 411.58 \quad (9)$$

In the above equations, D represents the pipe diameter (meters), E is the depth of excavation (meters), and H is the depth of the manhole (meters).

The objective function is subjected to the following constraints:

1- Flow velocity constraint:

$$V_{min} \leq V_i \leq V_{max} \quad (9)$$

$$i = 1, \dots, NM$$

The flow velocity in pipes should be kept between a minimum and maximum bounds to ensure self-cleaning and to prevent scouring. Minimum and maximum flow velocities are chosen as 0.9 and 4.5 m / s, respectively.

2- Flow depth constraint:

$$y_i \leq y_{max} \quad (10)$$

$$i = 1, \dots NM$$

y_i and y_{max} represent the relative flow depth and maximum relative flow depth in pipes, respectively. y_{max} is selected as 0.9 in this research.

3- Excavation depth constraint:

$$E_{min} \leq E_i \leq E_{max} \quad (11)$$

$$i = 1, \dots NM$$

E_i is the excavation depth for pipe i , E_{min} and E_{max} are the minimum and maximum excavation depths and they are selected as 1 and 6 meters, respectively.

4- The telescopic pattern of pipe diameter constraint:

$$D_i \geq [DU] \quad (12)$$

Where $[DU]$ is the vector of pipe diameters connected to the upstream end of pipe i .

5- Pipe slope constraint:

$$S_i \geq S_{min} \quad (13)$$

Where S_i is the slope of pipe i , and S_{min} is the minimum pipe slope recommended as 0.0005.

6- Outlet elevation from each manhole constraint:

The invert of the outlet pipe from each manhole should be equal to or lower than inlet pipes.

The initial population in the GA is randomly produced and then by random-based operators next generations are created. This mechanism often results in infeasible chromosomes which are not accepted for cost evaluation. For handling the aforementioned sewer constraints and avoiding infeasible solutions, several techniques have been proposed for metaheuristics so far. These techniques can be simply classified as the rejecting, repairing, penalizing, and modifying strategies (Gen and Cheng, 1997).

The first 3 above-mentioned constraints are added to the objective function by the penalty function method and the next 3 constraints are applied by decoding. Therefore, constraints 1 to 3 can be presented as the following penalty functions and added to the objective function:

$$P_{vmax} = \lambda_v \max\left(\left(\frac{V_i}{V_{max}} - 1\right), 0\right) \quad (14)$$

$$P_{vmin} = \lambda_v \max\left(\left(1 - \frac{V_i}{V_{min}}\right), 0\right) \quad (15)$$

$$P_{emax} = \lambda_e \max\left(\left(\frac{E_i}{E_{max}} - 1\right), 0\right) \quad (16)$$

$$P_{emin} = \lambda_e \max\left(\left(1 - \frac{E_i}{E_{min}}\right), 0\right) \quad (17)$$

$$P_y = \lambda_y \max\left[\left(1 - \frac{y_i}{y_{max}}\right)^2, 0\right] \quad (18)$$

P_{vmax} , P_{vmin} are the penalties of the constraints related to the maximum and minimum velocity, respectively, P_{emax} , P_{emin} , represent the penalties of the constraints related to the maximum and minimum allowable depths of excavation respectively, and P_y , is the penalty function for violating the relative depth of the flow. If the values of velocity, excavation depth, and relative depth of the flow are not within the allowable range, heavy penalties will be added to the

objective function which in its turn deviates the corresponding objective function from being minimum. The penalty function is zero if the values of the penalty functions are not violated. λ_V , λ_y and λ_E are coefficients correspond to velocity, relative flow depth, and excavation depth, respectively. The values of these parameters are usually chosen as large numbers. The final objective function is rewritten by applying the penalty functions as follows.

$$\begin{aligned} \text{minimize } Cost(D, S) = & \sum_{i=1}^{NP} CP_i + \sum_{j=1}^{NM} CM_j + \sum_{i=1}^{NP} P_V + \\ & P_y + P_E \end{aligned} \quad (19)$$

The diameter and slope of the pipes are considered as the decision variables that should be determined by the optimization process.

5. Methodology

The simulation model (SWMM software) is linked to the GA optimization model. The goal is to determine the storm return period with the optimal corresponding design alternative for which the total design cost plus the probable damage cost is minimum. In this regard, at the first stage, the optimal storm sewer network design for a certain selected rainfall return period is obtained. The required hydrologic data corresponding to the selected rainfall return period and hydraulic data are given to the developed model. The optimization data such as the list of the commercial pipes, the objective function, and constraints should be given to the program. Besides, the GA parameters including the initial population of chromosomes, and the mutation factor. The decision variables (pipe diameters and slopes) are initially selected by the GA. The simulation is then performed by the SWMM program. GA program calculates the objective based on the results of the simulation. The decision variables are changed by the optimization program until a minimum objective function in which all constraints are satisfied is achieved.

The risk analysis is done to find the optimal storm return period. This is performed by repeating the same procedure explained above for other storm return periods. The return period is sought in which the summation of the annual costs of the corresponding optimal design and the risk damage is minimum.

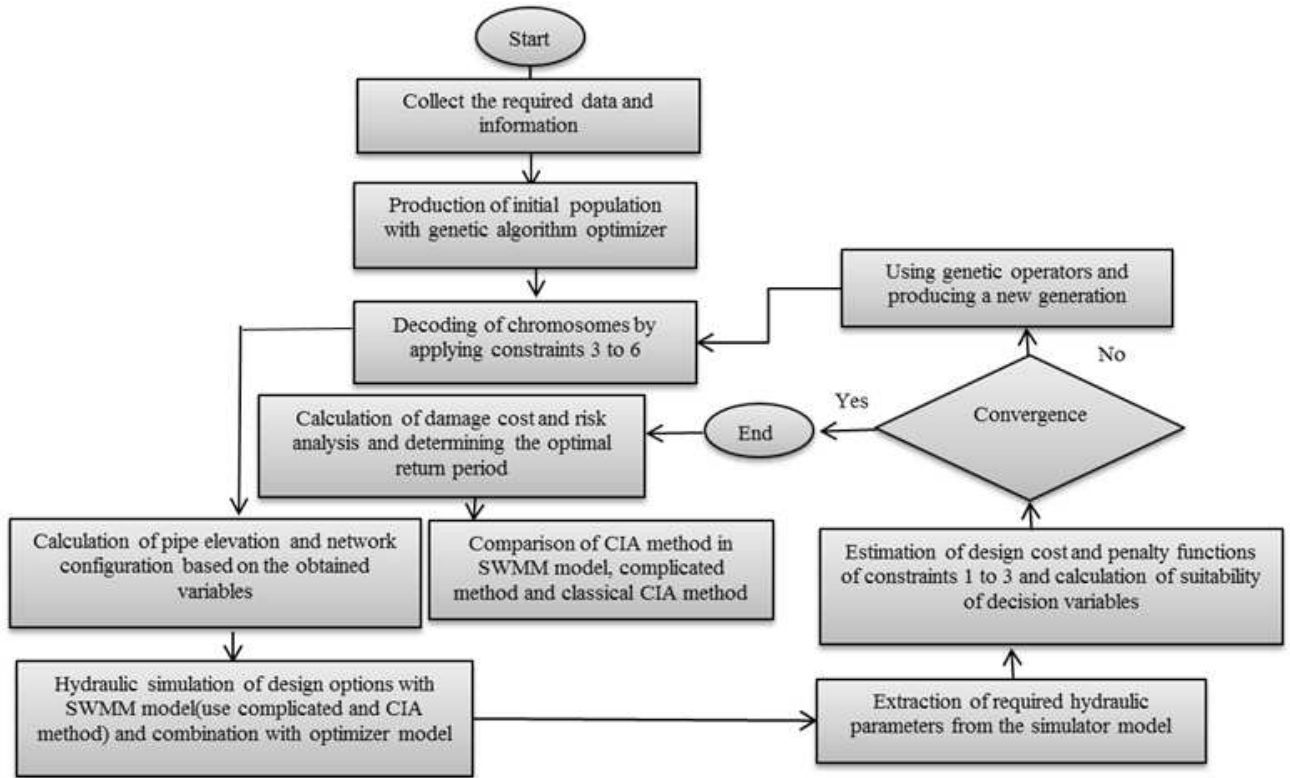


Fig. 2 Storm sewer network simulation-optimization flow chart

6. GA optimization parameters

The configuration of the genetic algorithm was determined using several test implementations with random initial generations on the proposed model to achieve the fastest convergence seeking the optimal solutions. For this purpose, the optimization model was implemented with

different values of parameters to find the most appropriate ones. These parameters are listed in Table 1.

Table 1 Parameters set in GA

Parameter	Population size	Mutation rates	λ_V	λ_Y	λ_E
Amount	200	0.02	1.0E+9	5.0E+10	5.0E+11

6.1. Performance of the optimization model

To evaluate the accuracy of the model in this research, a benchmark storm sewer network optimized by many other investigators and depicted in Figure 3 was employed. This network was initially designed and presented by Mays and Wenzel (1976). The network consists of 20 branches and 20 nodes (manholes) and one output.

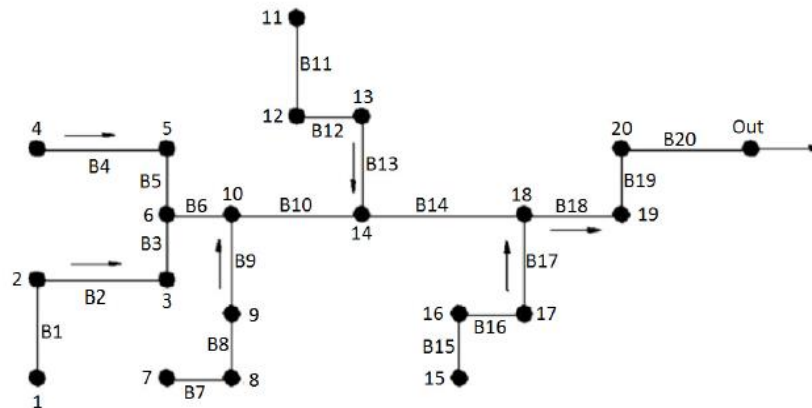


Fig. 3 Benchmark storm sewer network (Mays and Wenzel, 1976)

The developed optimization model is applied to the benchmark network and the results are compared with those of other researchers to ensure the validity of the model.

Figure 4 indicates that the optimal design is achieved within 50 generations and the converged minimal objective function is \$ 241763. The results show the good performance of the optimization method used in this study.

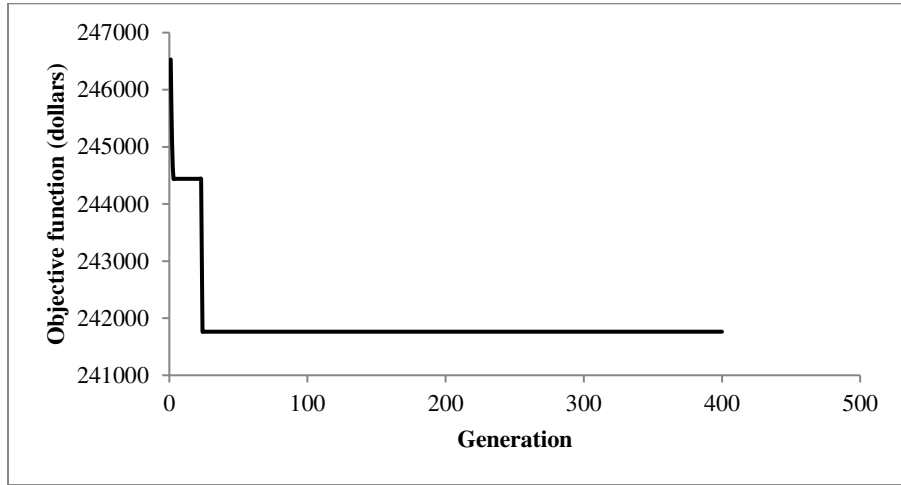


Fig. 4 Diagram of benchmark network optimization achievement

Comparison of the developed optimization model with others is presented in Table 2. The optimal cost of the present study is lower than all other results except that of Afshar who used the ant colony optimization method. This proves that the developed model has a good performance in terms of speed and accuracy.

Table 2 Benchmark problem result comparison of the present study with those of others

Researcher	Optimization method	Optimal cost (dollars)	The optimal cost of the present study (dollars)	Comparison with previous research
Mays and Wenzel(1976)	Differential discrete programming	265775	241763	-24012
Miles and Heaney(1988)	Differential discrete programming	245874	241763	-4111

Afshar et al(2006)	genetic algorithm	244747	241763	-2984
Afshar(2010)	Ant colony	241496	241763	267
Afshar(2008)	PSO	242162	241763	-399
Afshar and Rouhani(2012)	Cellular machine	247412	241763	-5649
Mousavi(2016)	genetic algorithm	241956	241763	-193

7. Case study

The study area is region 2, district 3 of Tehran with residential land use, roads, highways, and lawn. The total area of this region is 230 hectares. The region was divided into 17 sub-catchments. The layout of the storm sewer network is shown in Figure 5. The simulation is done by the SWMM software using the nonlinear reservoir model and the rational method to estimate the surface runoff. The rainfall intensity-duration-return period curves provided by Tehran Surface Water Management Comprehensive Plan, (2012) and shown in Figure 6 are used in the analysis to obtain the runoff.

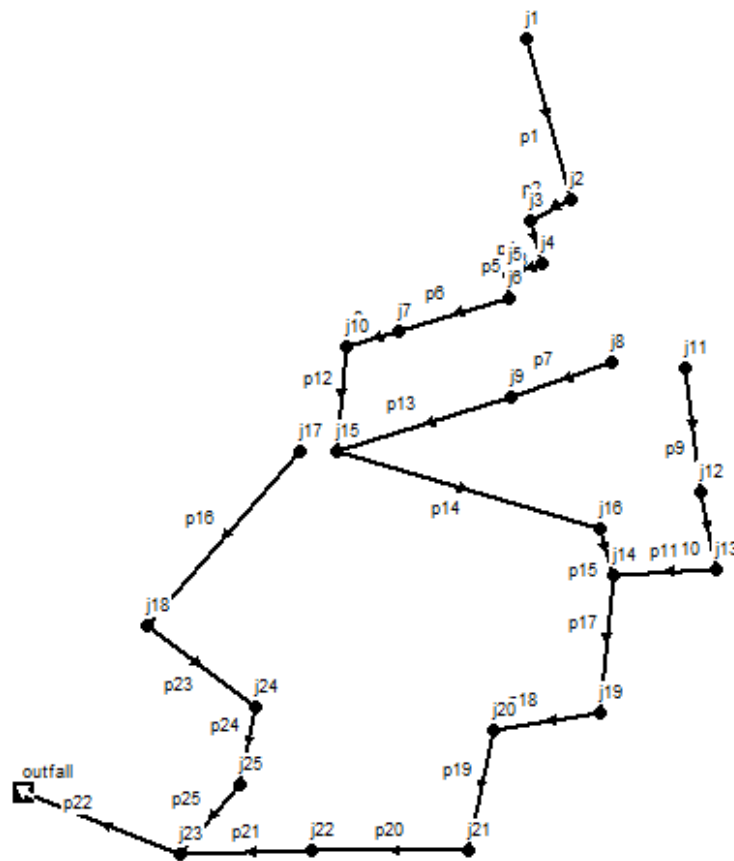


Fig. 5 The Layout of the storm sewer network of region 2 in Tehran city

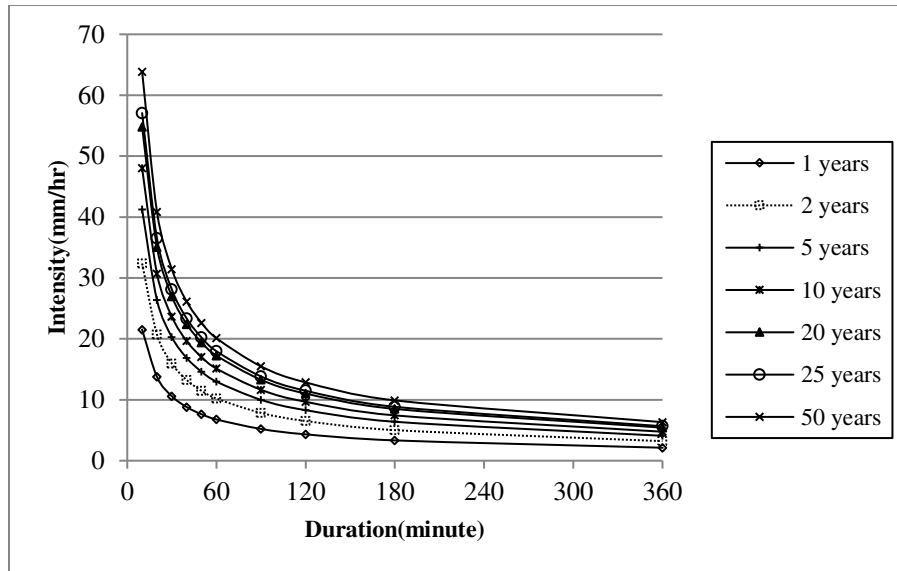


Fig. 6 The rainfall-intensity-duration-return period of the station covers the case study region

8. Damage functions

The damage costs are required for the risk analysis to determine the optimal storm return period. The damage cost for the case study is assumed as a function of the flood in the area out of the sewer system when a runoff greater than the storm sewer runoff design takes place. In this regard, the depth-damage percent curves used in previous studies have been used because reliable data in this respect are not available. The depth-damage percent curves of the residential and commercial buildings presented by Jaf (2015) were employed (Figure 7).

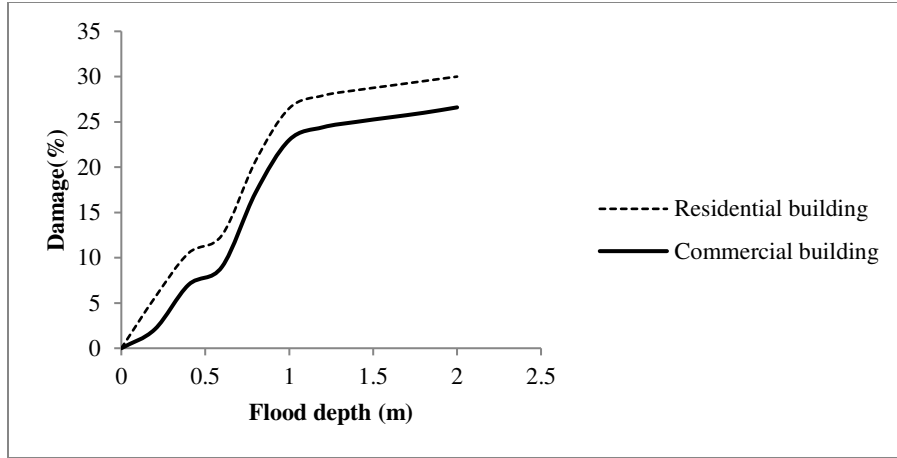


Fig. 7 Depth-damage percent curve of residential and commercial buildings provided by Jaf, 2015

The following functions of the damage percent related to the residential and commercial buildings in terms of the runoff depth have been obtained by using the curve fitting technique based on the data extracted from figure 7:

$$D_{\text{residential}} = -1.8071y^4 + 8.4036y^3 - 13.412y^2 + 99.376y + 0.0431 \quad (20)$$

$$D_{\text{Commercial}} = -0.644y^4 + 2.67y^3 - 3.265y^2 + 3.0443y - 0.029 \quad (21)$$

The road damage percent obtained by Hekmatifar, 2005, based on real data given in equation (22) is used in this study:

$$D_{\text{road}} = 0.03024D_{\text{residential}} - 0.072 \quad (22)$$

The following equation has been obtained by using real data provided by the lawn areas Organization of Tehran in terms of the road damages:

$$D_{\text{Lawn}} = 0.09D_{\text{road}} + 0.00004 \quad (23)$$

According to the statistics of the Deputy of Municipal Transportation and the statistics of the Scientific Association of Urban Economics of Iran, a traffic delay of 0.012 million hours per day

is produced due to street flooding when a runoff greater than the design runoff occurs. This delay has been converted to cost considering the probable rainy days. Using real data by Hekmatifar (2006), the damage percent due to traffic in terms of the road damage percent was obtained as follows:

$$D_{Traffic} = 0.0116D_{road} + 0.0005 \quad (24)$$

Multiplying the percentage of damages by the value of the construction of each above-mentioned damage yields the damage costs for every flooding event.

9. Risk analysis

The damage cost occurs when a runoff greater than the design discharge takes place. Since the rainfall and the resulted runoff are stochastic events, the damage due to flooding should be stochastic too. The probability of a flood occurring at least once greater than the design flood during the life of the design is called risk. The optimal return period is the one in which the total annual cost of the project and the risk of damage are minimum. Damage is considered cumulative. In other words, the probability of damages for all floods larger than the design flood should be summed as indicated mathematically by the equation:

$$D_T = \int_x^{\infty} D(x)dx \quad (25)$$

In the above relationship, D_T is the risk cost and $D(x)$ is the cost of flood damage with frequency x .

To calculate the annual design cost, the following equation is used (Swamee and Sharma, 2008):

$$A = C_0 \left(\frac{r(1+r)^T}{(1+r)^T - 1} + \frac{r}{(1+r)^T - 1} + \beta \right) \quad (26)$$

Where A is the annual design cost, C_0 is the design cost, r is the discount rate, T is the pipe design life, and β is the annual rate of repairs and maintenance.

In the present study, r , T , and β are considered equal to 0.08, 30 years, and 0.005, respectively.

10. Optimal design results using the complicated method

In this section, the results of the optimal design of the storm sewer network are presented using the complicated method. In this method, the nonlinear reservoir model to estimate the surface runoff is used and the dynamic wave method was used for flow routing in the conduits. The simulation-optimization developed model is applied for each storm return period to produce the optimal design of each one. The optimal design costs are introduced in column (5) of Table 3. These costs are converted to annual costs given in column (6) using equation (26). The total damage costs depicted in column (3) are obtained by using equations (20) through (24). The annual damage costs given in column (4) are calculated according to equation (25). The total annual cost introduced in column (7) shows that the 10 year is the optimal return period in which the total annual design and damage costs are minimum.

Table 3 Optimal design results using the complicated method

Return period	Frequency	Damage costs (dollars)	Annual damage risk cost (dollars)	Optimal Design cost (dollars)	Annual optimal design cost (dollars)	Total annual cost (dollars)
1	1	40000	1942080	36000000	3088720	5030800
2	0.5	2092000	1409080	41564000	3566120	4975200
5	0.2	2824000	671680	42080000	3610360	4282040
10	0.1	3772000	341880	44000000	3775120	4117000
20	0.05	4328000	139380	64000000	5491080	5630440
25	0.04	4500000	95240	84000000	7207040	7302280
50	0.02	5024000	0	1.2E+08	10295760	10295760

11. Optimal design using the rational method available in the SWMM model

The rational method available in SWMM in which the flow travel times in the conduits are not considered is used in the simulation-optimization developed model. The optimal designs for various storm return periods are shown in Table 4.

Table 4 Optimal design results using the rational method available in the SWMM model

Return period	Frequency	Damage costs (dollars)	Annual damage risk (dollars)	Optimal Design cost (dollars)	Annual Optimal design cost (dollars)	Total Annual cost (dollars)
1	1	52000	2311500	42840000	3611600	5923080
2	0.5	2488000	1672500	49460000	4163560	5836080

5	0.2	3360000	795300	50076000	4216400	5011720
10	0.1	4488000	402900	52360000	4400400	4803280
20	0.05	5152000	161900	76160000	6414360	6576280
25	0.04	5356000	109360	99960000	8520360	8689720
50	0.02	5980000	0	1.43E+08	12243960	12243960

Table 4 indicates that the 10 year return period is the optimal one. The optimal return periods determined by using the complicated and rational methods available in the SWMM software are similar. But, the optimal design cost and the total annual cost of the rational method are higher by 19 and 16.6 percent, respectively.

Figure 8 shows the optimization convergence's good performance in the developed model using complicated and rational methods for the 10 year return period.

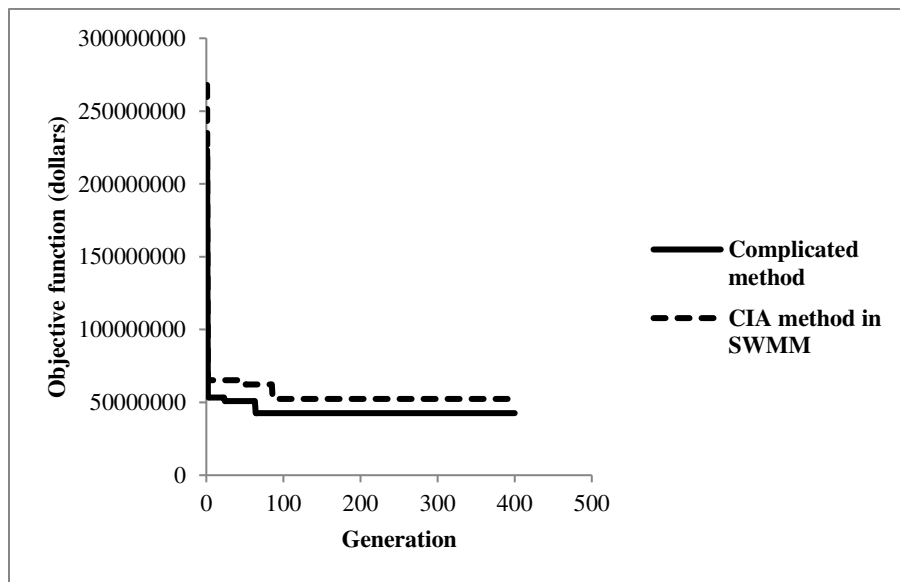


Fig.8 Optimization convergence performance for the 10-year optimal return period

Pipe diameters and slopes of the final optimal design with a 10-year storm return period of the complicated and rational methods resulted from the developed model using SWMM are given in Table 5.

Table 5 Pipe diameters and slopes of the final optimal design with a 10-year return period of the complicated and rational method using SWMM

The complicated method with dynamic wave routing										
Pipe	1	2	3	4	5	6	7	8	9	10
Diameter(mm)	450	450	450	525	525	525	300	525	450	450
Slope (%)	0.497	1.281	1.045	0.359	1.188	0.681	0.82	1.176	0.732	0.171
Pipe	11	12	13	14	15	16	17	18	19	20
Diameter(mm)	525	675	375	750	750	375	900	900	900	900
Slope (%)	0.258	1.007	0.797	1.494	0.288	0.694	0.942	0.312	1.461	1.068
Pipe	21	22	23	24	25					
Diameter(mm)	1050	1050	450	450	450					
Slope (%)	1.463	1.064	0.171	0.609	0.312					
rational method in SWMM										
Pipe	1	2	3	4	5	6	7	8	9	10
Diameter(mm)	675	675	675	675	675	675	450	525	675	675
Slope (%)	0.697	1.481	1.245	0.559	1.388	0.881	1.02	1.376	0.932	0.371
Pipe	11	12	13	14	15	16	17	18	19	20
Diameter(mm)	675	750	600	1050	1050	675	1200	1200	1200	1050
Slope (%)	0.458	1.207	0.997	1.694	0.488	0.894	1.142	0.512	1.661	1.268
Pipe	21	22	23	24	25					
Diameter(mm)	1050	1050	750	750	750					
Slope (%)	1.663	1.264	0.371	0.809	0.512					

12. Checking and adjusting the optimal design by the classical rational method

The main difference between the rational method used in SWMM software and the classic one is that the travel time in the conduits is considered in the latter one. The rainfall intensity is calculated based on the time of concentration. The time of concentration for each conduit is

considered equal to the inlet time plus the flow travel time through upstream conduits. Thus, as the flow is transferred downstream, the time of concentration will be increased and consequently, the rainfall intensity will be decreased. This reduces the flow discharge remarkably compared to the rational method in which the travel time is not considered and makes the design much more economic.

A computer program is developed for the design using the classic rational method taking into account the flow travel time in conduits. The same pipe diameters and slopes of the optimal design obtained by the complicated method introduced in Table 5 are given to this program. The flow depths in the pipes are calculated by the computer program. The results showed that the depth ratio constraint given in equation 4 in some of the pipes is violated. The diameters of these pipes were changed by trial and error so that the flow depth ratio becomes close to $y_{max}=0.9$ which was used in the optimization analysis of the complicated method. Then, based on the new diameters obtained from the classical rational method, the design cost of the storm sewer network is calculated. The details of the calculations are shown in Table 6. T_c is the time of concentration and T_f is the flow travel time. The results compared to the complicated method which is supposed to be the most precise one indicates that diameters of pipes (5, 6, 8, 13, 19, 20, 21, and 16) are increased and the design cost is increased by 4.7 percent.

Table 6 Results of the design by the classical rational method

Pipe	Length(m)	Increment (ha)	Cumulative area(ha)	T_c (min)	T_f (min) (2÷9)	Cumulative flow(m3/s)	Pipe diam (mm)	Velocity (m/s)	Design cost (dollars)
1	2	3	4	5	6	7	8	9	10
p1	200	8.35	8.35	20	2.19	0.5	450	1.52	46100176
p2	95	3.07	11.42	22.19	1.04	0.59	450	1.52	
p3	100	4.5	15.92	23.23	0.74	0.76	450	2.26	
p4	95	6.05	21.97	23.97	0.9	1.01	525	1.75	

p5	80	7.05	29.02	24.88	0.79	1.05	600	1.68
p6	122	6.22	35.24	25.67	1.08	1.29	600	1.89
p8	96	4.08	39.32	26.75	0.76	1.46	600	2.1
p12	97	5.03	44.35	27.51	0.89	1.58	675	1.82
p7	100	4.08	4.08	20	1.19	0.12	300	1.4
p13	90	5.8	50.15	21.19	0.97	2.09	450	1.55
p14	150	20.88	115.38	50.55	0.77	2.62	750	3.24
p15	86	21	136.38	51.33	0.51	2.96	750	2.83
p9	110	5.7	5.7	20	1.8	0.22	450	1.02
p10	85	4.5	10.2	21.8	1.42	0.47	450	1
p11	56	4	14.2	23.21	0.83	0.61	525	1.13
p17	87	10.5	166.48	75.87	0.57	2.89	900	2.54
p18	99	5.78	172.26	76.44	0.66	3.02	900	2.5
p19	100	4.91	177.17	77.1	0.6	3.15	1050	2.78
p20	102	5.54	182.71	77.7	0.61	3.3	1050	2.77
p21	95	2.41	185.12	78.32	0.74	3.37	1200	2.14
p16	86	3.77	3.77	20	1.59	0.23	450	0.9
p23	52	5	8.77	21.59	0.96	0.5	450	0.9
p24	46	2.06	10.83	22.56	0.68	0.55	450	1.12
p25	52	2.05	12.88	23.24	0.96	0.6	450	0.9
p22	63	6.51	227.88	103.26	0.33	3.67	1050	3.2

13. Checking and adjusting the optimal design by the kinematic wave method

The optimal design obtained by the complicated method is also checked and adjusted using the kinematic wave method available in SWMM for flow routing in the same manner as was done for the classical rational method. The results are shown in Table 7. Comparison of the results with the complicated method depicts that the diameters of pipes (8, 10, 11, 12, and 13) are increased and the design cost is increased by 2 percent.

Table 7 Design by kinematic wave method

kinematic wave method										
Pipe	1	2	3	4	5	6	7	8	9	10
Diameter(mm)	450	450	450	525	525	525	300	600	375	600
Pipe	11	12	13	14	15	16	17	18	19	20
Diameter(mm)	750	900	450	750	750	375	900	900	900	900

Pipe	21	22	23	24	25
Diameter(mm)	1050	1050	450	450	450
Design cost(dollar)	44871372				

14. Summary and concluding remarks

In this study, a simulation–optimization program for finding the storm sewer network optimal design has been developed. The simulation requires hydrologic and hydraulic calculations. The hydrologic calculations involve the rainfall-runoff relationships and determination of surface runoff in the sub-catchments. The hydraulic calculations deal with flow routing in the storm sewer network. Several methods are available for hydrologic and hydraulic simulations. The non-linear reservoir model to estimate the surface runoff produced by rainfall over a sub-catchment is the most complicated and precise one. The rational method is the simplest and has less accuracy compared to the non-linear reservoir model. The hydraulics of the flow in the sewers can be dealt with using various methods. The dynamic wave method is the most complicated and precise one. The kinematic wave method is the following one in terms of complexity and accuracy. The simplest method is the Manning equation which assumes a uniform flow in each conduit. The SWMM software includes all mentioned hydrologic and hydraulic simulation methods except that the rational method can be utilized in this software without considering the flow travel time. This leads to much higher flows and large-size conduits.

In this research, the SWMM program is linked to an optimization program that uses the genetic algorithm to develop a program that can do the simulation-optimization for the design of storm sewer networks. The performance of the developed program in terms of speed and accuracy was

checked by solving a benchmark problem. The results indicated good and successful performance.

The developed program was applied for a case study design problem in a district located in Tehran city. The optimal designs for various storm return periods were obtained. To determine the optimal storm return period, risk analysis was used. In this analysis, the risk of damages results when a flood larger than the design flood occurs in the region. Equations to calculate mathematically different damages were developed for the district in the case study. The return period of which the summation of the annual costs of the optimal design and the risk damage is minimum was determined. Different simulation methods were utilized but in all of them, the GA was used as an optimization solver.

The dynamic wave hydraulic simulation method is used in the simulation-optimization developed program in addition to the risk analysis to produce the optimal design of the case study. This method is regarded as a complicated method and its results are the most precise compared to the others. On the other hand, the rational method without consideration of the travel time in the pipes and using the Manning equation for the hydraulic analysis available in the SWMM program were employed to obtain the optimal design for various storm return periods. The risk analysis was utilized to determine the storm return period in which the design cost plus the risk damage cost is minimum.

Comparison of the complicated approach and the rational method without considering the flow travel time indicates that the storm return period is the same (10 year return period), but the cost of the optimal design using the rational method of the SWMM is higher by 16.6 percent. This

leads to the conclusion that using the rational method without considering the flow travel time is not an economic approach to design storm sewer networks.

A computer program has been prepared in which the classic rational method that considers the flow travel time is used. The optimal design obtained by the complicated method was checked by this computer program to produce the flow depth in all pipes. The results indicated that the flow depth constraint was violated in some pipes. The diameters of these pipes were adjusted by trial and error until the flow depth become close to the maximum allowable depth constraint. The cost of the resulted design was only 4.7 percent higher than that of the complicated method. This leads to the conclusion that using the rational method considering the flow travel time despite its simplicity it introduces practical acceptable economic designs which are on the safe side.

The same checking and adjusting of the pipe diameters were done on the optimal design obtained by the complicated approach by using the kinematic wave for flow routing in the sewer network. The results produced 2 percent higher costs. This shows the significance of considering the flow travel time in storm sewer design.

Declarations

Authors Contributions:

- 1- Sonia Sadeghi: PhD student of Water Structures, Tarbiat Modares University, Tehran, Iran. Researcher and author of the article.
- 2- Jamal Mohammad Vali Samani, Professor of Tarbiat Modares University, Tehran, Iran. Corresponding Author. First Supervisor of author.

- 3- Hossein Mohammad Vali Samani, Professor of Shahid Chamran University, Ahvaz, Iran.
Second Supervisor of author.

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Competing Interests: Not applicable.

Availability of data and materials: All data in the article is available.

Code availability: The software used in this research is open source. And it can be easily downloaded.

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Figures

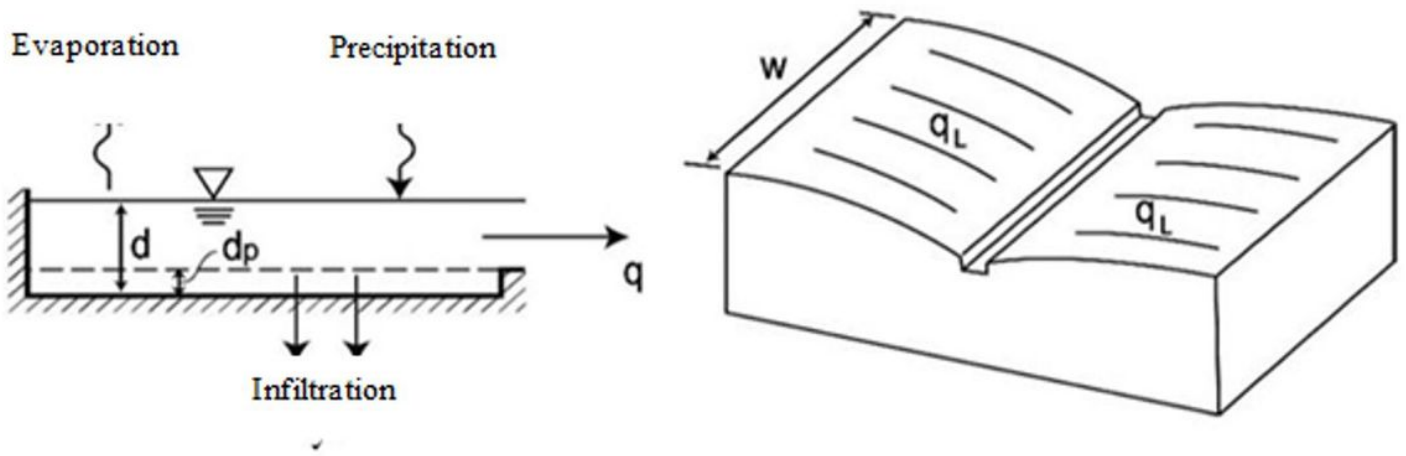


Figure 1

Nonlinear reservoir model of a sub-catchment

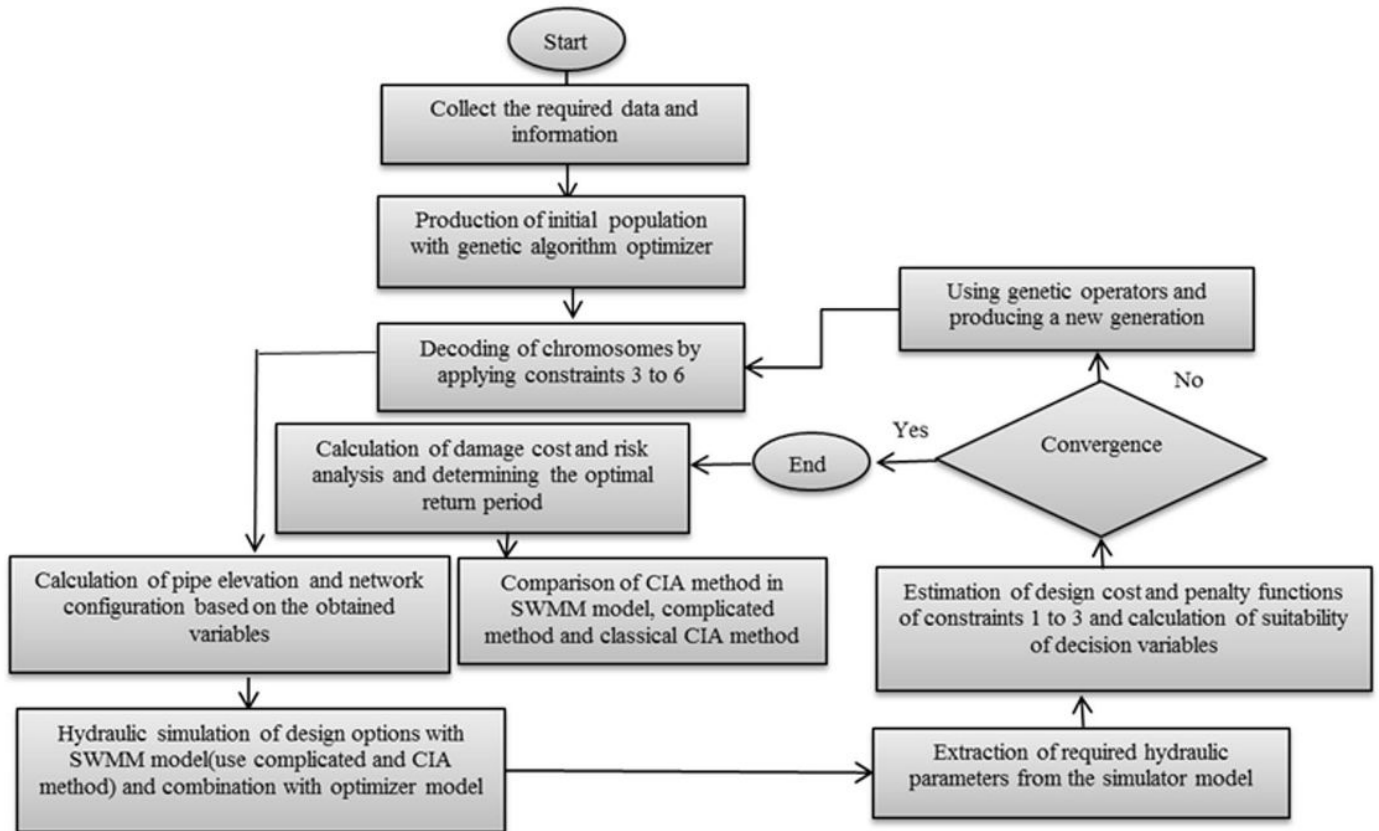


Figure 2

Storm sewer network simulation-optimization flow chart

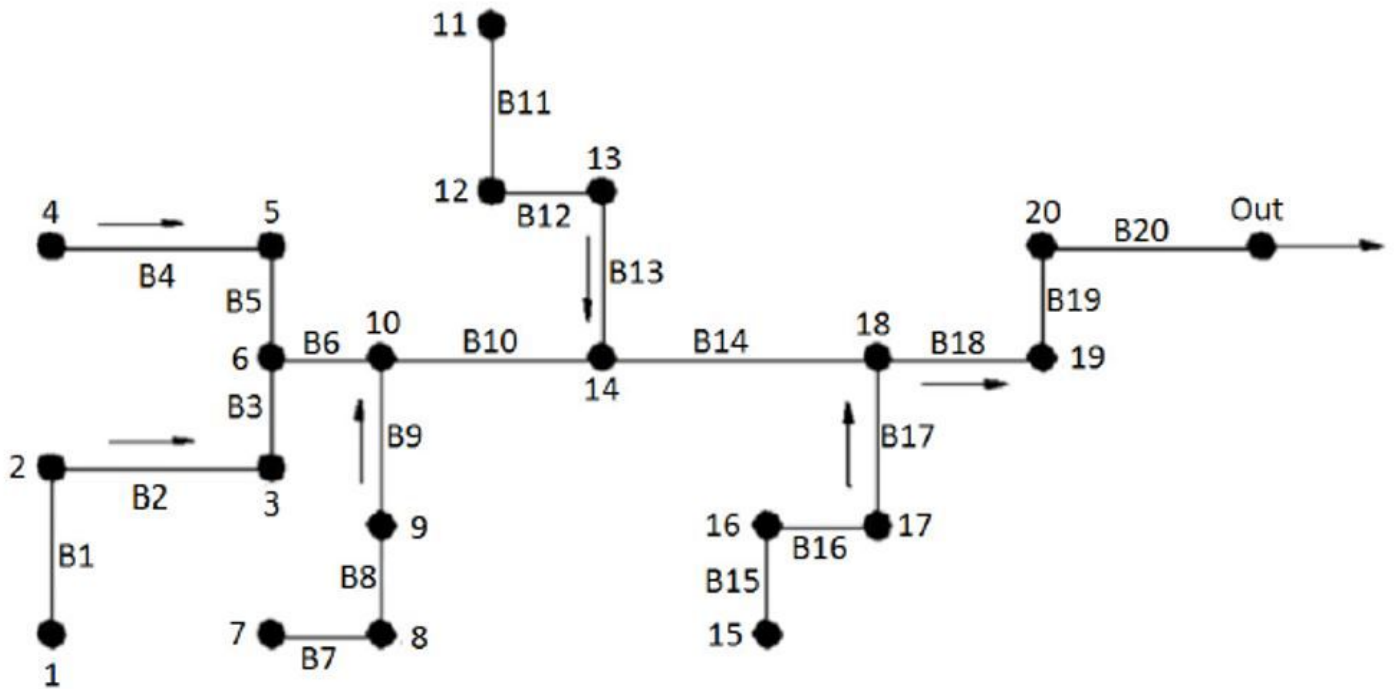


Figure 3
 Benchmark storm sewer network (Mays and Wenzel, 1976)

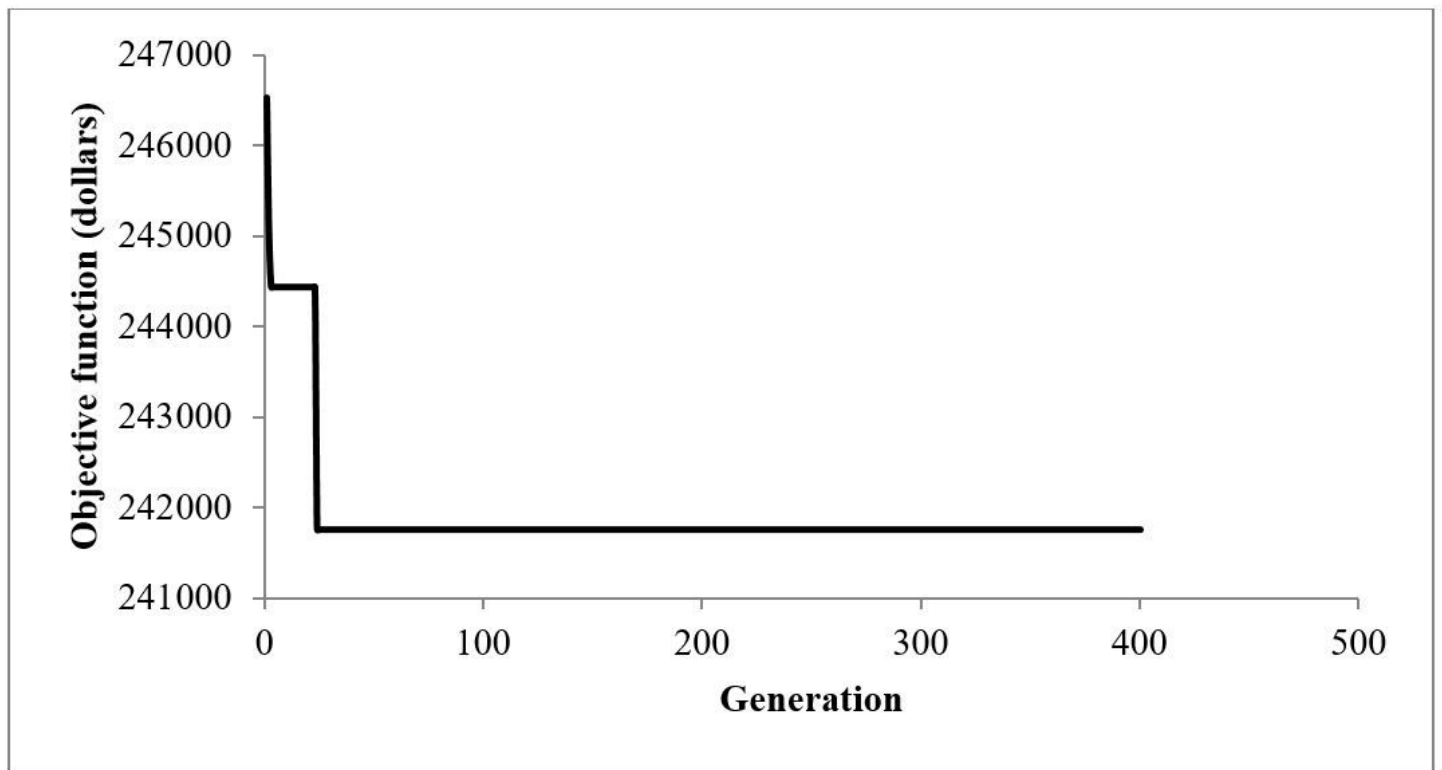


Figure 4

Diagram of benchmark network optimization achievement

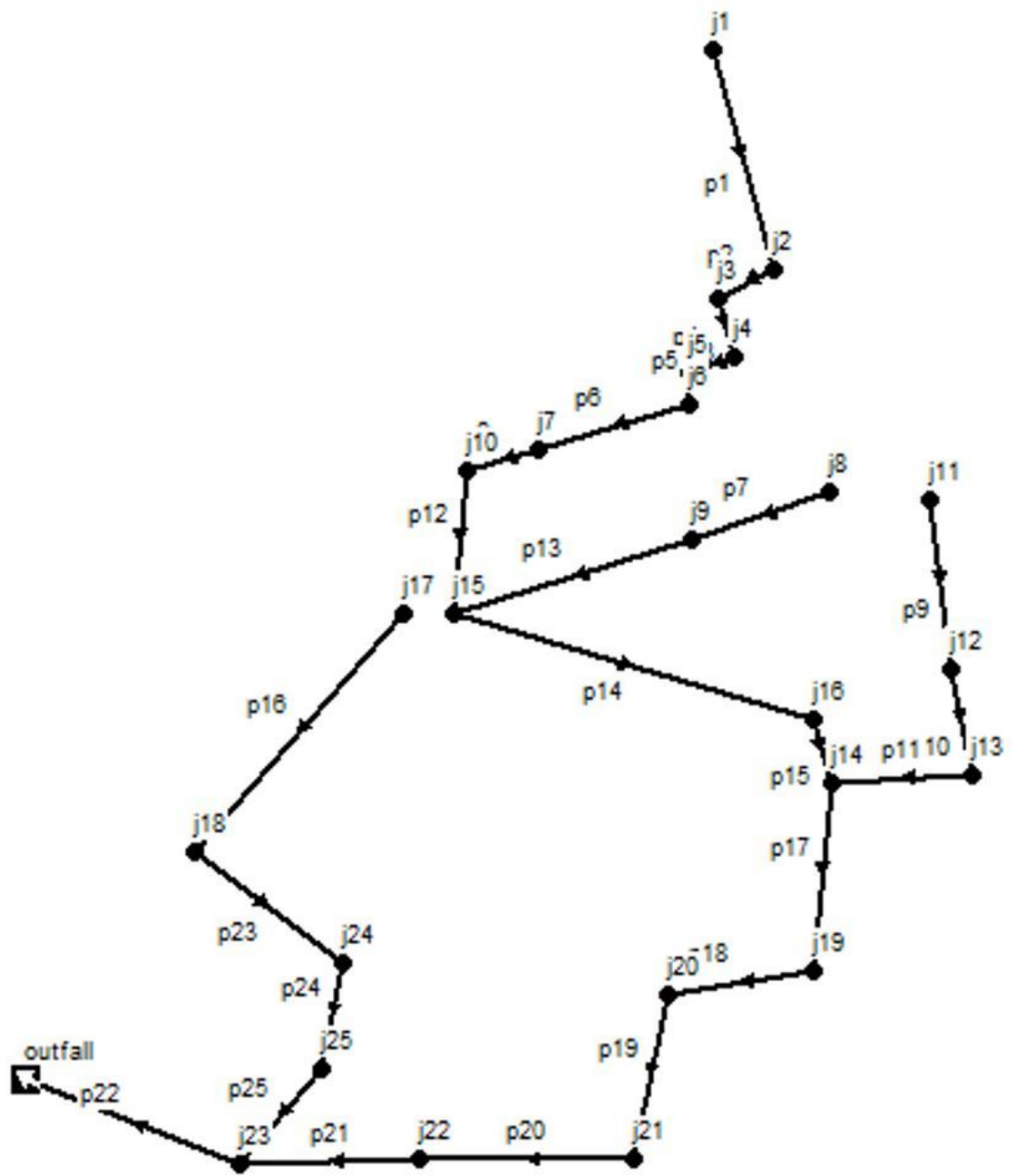


Figure 5

The Layout of the storm sewer network of region 2 in Tehran city

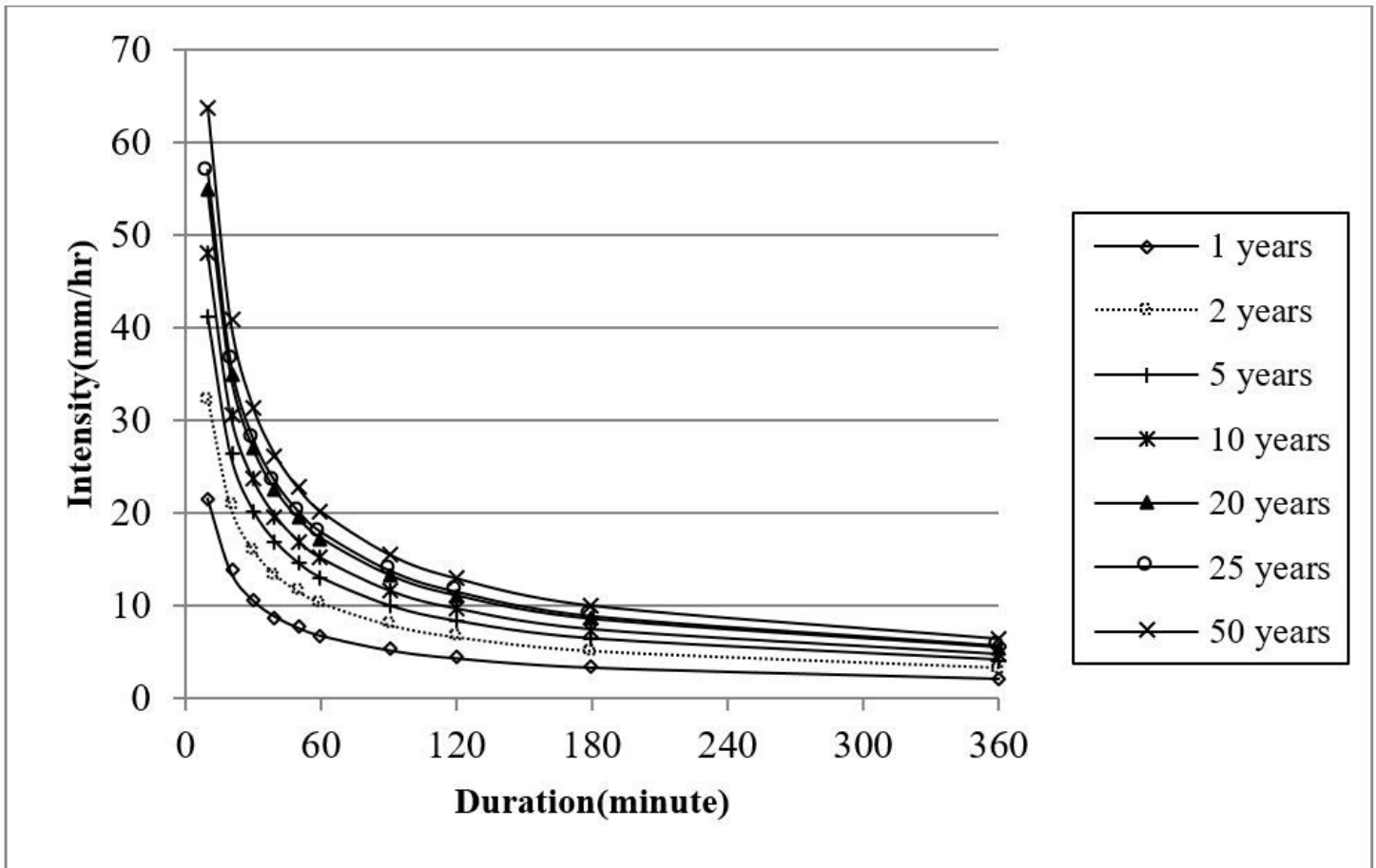


Figure 6

The rainfall-intensity-duration-return period of the station covers the case study region

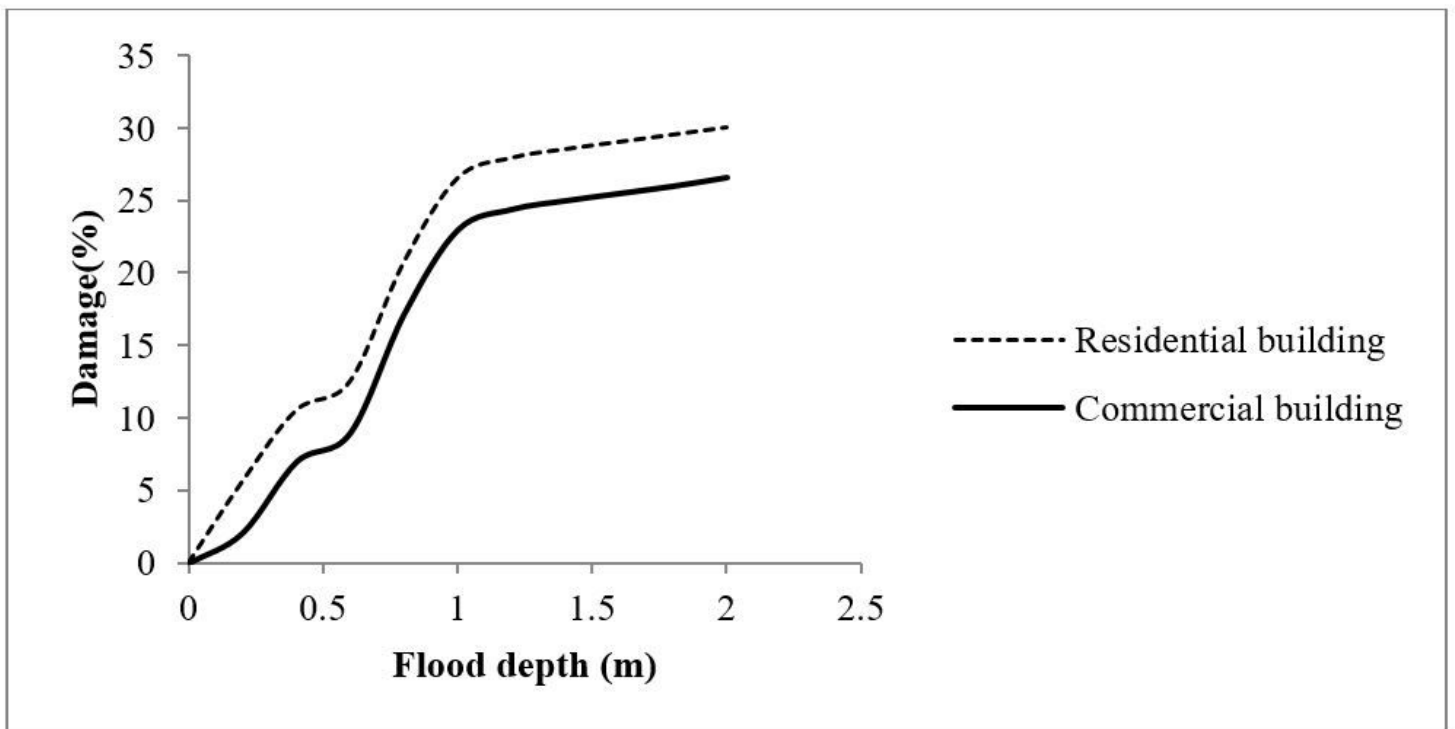


Figure 7

Depth-damage percent curve of residential and commercial buildings provided by Jaf, 2015

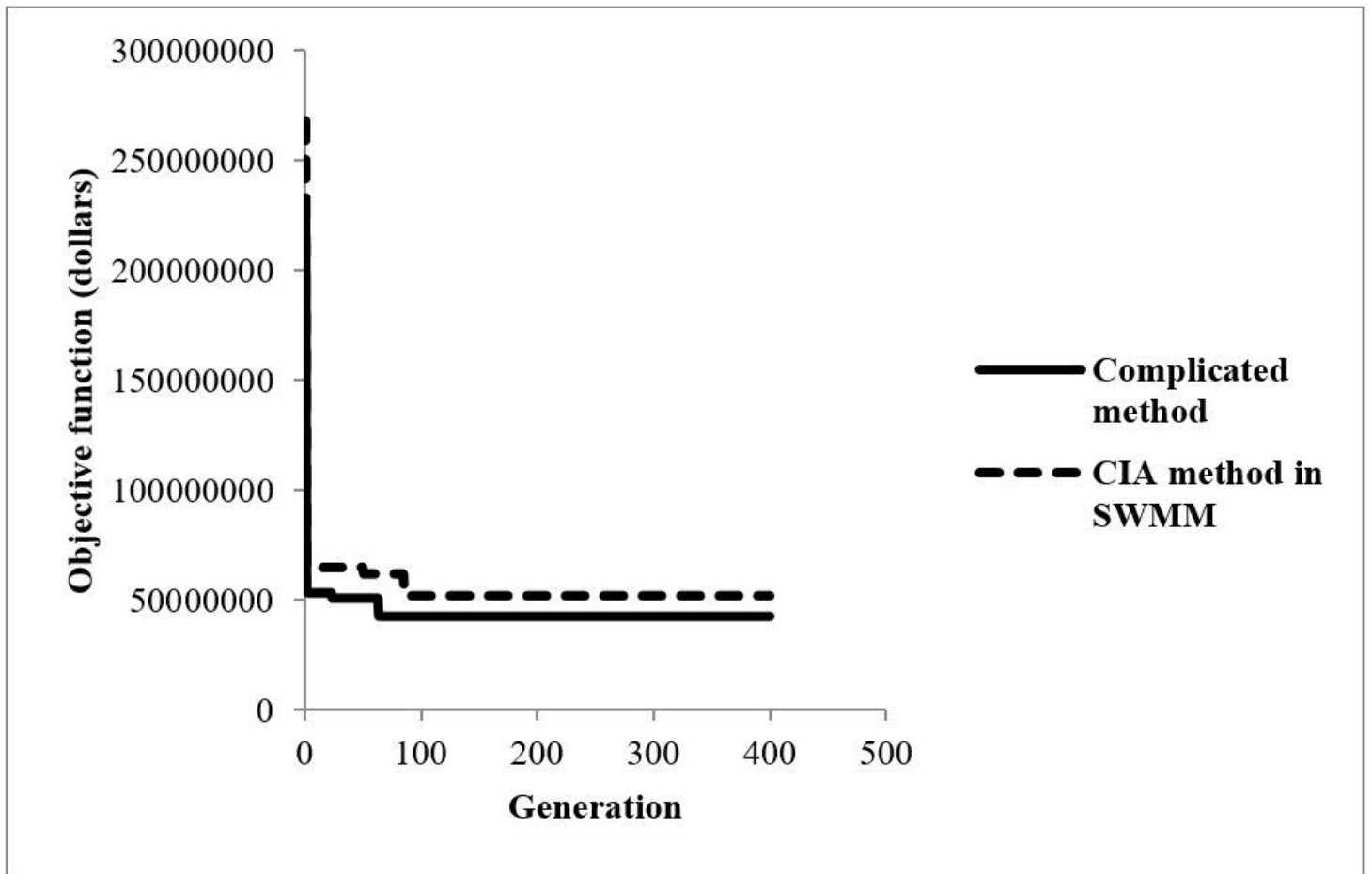


Figure 8

Optimization convergence performance for the 10-year optimal return period