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ORIGINAL ARTICLE

Hydraulic-based optimization algorithm for the design of stormwater drainage networks

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Abstract

Stormwater drainage networks are designed to reduce the risk of rainwater damage to the served area. The purpose of optimizing a stormwater drainage system is to reduce overall construction costs and to meet hydraulic design requirements. Currently, designs that rely on software or manual calculations are limited by the available time and the designer's capabilities. In fact, manual optimization for large networks consumes a lot of time and effort, and there is no guarantee that the optimal design is reached, also it is subject to human errors. In recent years, several researchers have focused on creating optimization design algorithms specifically for sewer and storm networks, such as genetic algorithm (GA), linear programming (LP), heuristic programming (HP),…etc. However, these studies were limited to covering one or two design parameters and constraints. Additionally, in some studies, the hydraulic performance of the designed network was not treated in a proper way, especially the water surface profile effects. So, the main objective of the study is to develop an effective hydraulic-based optimization algorithm (HBOA) that can dynamically get the optimal design with minimum total cost for a given storm network layout and meet all hydraulic requirements. To achieve this, a MATLAB code is created and coupled with SewerGEMS software that automatically simulates all expected optimization scenarios based on network hydraulic performance. The HBOA is validated economically and hydraulically using two benchmark examples from the literature. According to the economic validation, the total network cost generated by HBOA was the lowest when compared to the optimization methods found in the literature. During the hydraulic evaluation, it was observed that the optimization algorithm (GA-HP) used in the literature for the benchmark examples does not meet the hydraulic requirements where the networks are flooded, whereas HBOA meets the hydraulic requirements with minimal overall network cost. Also, the HBOA is applied to four real stormwater drainage networks that were already designed, constructed, and optimized manually. The four redesigned real cases using HBOA revealed a cost reduction of about 15% compared to the original designs, while consuming a few hours for the design and optimization processes. Finally, the developed HBOA is a robust, time-efficient, and cost-effective optimization and hydraulic design tool which could be used in the design of stormwater drainage networks with different design constraints with minimal human interference.

Keywords Stormwater drainage · Algorithmic optimization · SewerGEMS · Hydraulic-based algorithm · Computational hydrology · Minimum cost · Optimal design · Sustainable development

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Introduction

Flooding and other severe damage have resulted from the heavy rain, endangering human life. The stormwater network is a crucial component of the infrastructure for effectively draining local direct rainfall. The construction and maintenance of large-scale networks require huge costs. So, it is crucial to design an optimal stormwater drainage network to reduce the total construction cost without violating the network's functionality and safety. Nowadays, the design of stormwater networks that rely on software such as Storm

CAD, SewerGEMS, Storm Water Management Model (SWMM), and others has a hydraulic design idea linked with manual optimization of the system to reach the optimal design. The results of manual optimization are typically constrained by the designer's skills and time constraints. In reality, manual optimization is usually inefficient for large-scale networks as it depends on the trials conducted by the designer, which may be limited to some parts of the network, especially in large and complex networks. Additionally, manual optimization consumes a lot of time to conduct a limited number of trials. As a result, manual optimization does not necessarily reveal an optimum design.

So, in the previous studies, researchers tried to solve the main problem of reaching the optimum design dynamically using algorithms instead of manual optimization process that consumes a lot of time and effort. Unfortunately, most of these algorithms were developed for sewer networks using several optimization techniques such as genetic algorithm (GA), linear programming (LP), and heuristic programming (HP),…etc. The previous studies may be divided into three main groups based on the function of the designed gravity-flowing network. The first group is devoted to sewer networks, while the second group is for sewer-storm systems, and the last group is for storm systems.

Over the past decades, several researchers have been focused on the sewer network design optimization problem and proposed different methods, from traditional optimization techniques to modern heuristic search methods. For a portion of the Kerman sewerage system in Iran, Mansuri and Khanjani used the nonlinear programming technique to obtain the optimal design (Mansuri and Khanjani 1999). Then, Sotoodeh used Fletcher–Reeves method and achieved the lowest overall cost for this portion of Iran's sewage system (Sotoodeh 2004). Other optimization techniques are used for the optimization of sewer networks, such as genetic algorithm (GA), hybrid techniques based on cellular automata, tabu search (TS) and simulated annealing (SA), ant colony optimization algorithm (ACOA) and tree-growing algorithm (TGA), spanning tree and modified particle swam optimization (PSO), mixed-integer linear programming (MILP) and others (Haghighi and Bakhshipour 2012, 2015; Afshar and Rohani 2012; Afshar et al. 2016; Yeh et al. 2013; Emmerich et al. 2013; Duque et al. 2016; Navin and Mathur 2016; Safavi and Geranmehr 2017; Moeini 2017; Moeini and Afshar 2017, 2018; Hassan et al. 2020; Saldarriaga et al. 2021; Atiyah and Hassan 2021). When comparing the outcomes achieved through the utilization of genetic algorithm (GA) and TS to those obtained through alternative methods, it becomes evident that GA and TS yield optimal results with minimal network cost. However, it should be noted that GA is an unconstrained technique and is most effective when applied to a single design variable. If employed for multiple variables, GA requires a longer runtime. To overcome this problem, the genetic algorithm is coupled with a heuristic programming (GA-HP) technique to optimize the design of sewer networks (Hassan et al. 2018). The results prove that the GA-HP is more optimized and effective in designing large sewerage networks compared with the results of the previous studies. Other optimization methods such as cellular automata (CA), iterative mathematical optimization technique, and decomposition–dynamic programming aggregation technique are used to get the minimum total network sewerage network (Zaheri et al. 2020; Duque et al. 2020; Tian and He 2020). The sewer-storm network design problem was addressed in the previous studies.

Several researchers were applying fixed loads in sewer-storm systems and using steady-state simplified hydraulic equations like what is usually applied in separate sewage networks. On the other hand, different researchers applied the dynamic programming (DP) methods, which are the mostly commonly used method for the optimum design of storm-sewers. Robinson and Labadie (1981), Yen et al. (1984), Kulkarni and Khanna (1985), and Li et al. (1990) employed DP to optimally design sewer-stormwater networks. Dynamic programming methods, which are theoretically capable of finding the global optimum solution, suffer from the so-called curse of dimensionality; therefore, it does not apply to real-world sewer networks. Other researchers used linear programming methods to solve the problem of sewer-stormwater design, such as Swamee and Sharma (2013), Safavi and Geranmehr (2017), and Gupta et al. (2017). In a different approach, researchers combined linear programming (LP) with a heuristic approach (HA). Elimam et al. (1989) employed this combination to develop a sewer-stormwater network on a large scale, utilizing linear programming (LP) alongside a heuristic approach. Afshar and Zamani (2002) have used heuristic approaches on spreadsheet templates to get near-optimal solutions for the problem. Afshar (2006) developed a genetic algorithm (GA) application specifically for storm and storm-sewer networks in order to achieve optimal design. The decision variable in this case was the depth of the manholes within the gravity-flowing network.

To analyze the trial solutions obtained through the GA optimizer, a steady-state simulation was employed. The methodology was tested on both large-scale and small-scale examples. The results of this model outperformed other methods, providing a more cost-effective solution for the large-scale network. However, for the smaller network, the methodology did not yield significant improvements, likely due to the simplicity of the network itself. Afshar et al. (2006) developed more enhanced GA to get an optimized storm-sewer design. The proposed methodology depends on using GA and the TRANSPORT-SWMM module as search engines and hydraulic simulators, respectively. The pipe diameter and manhole depth were selected to be the decision variables. However, Haghighi and Bakhshipour (2012) found that GA was not computationally efficient compared to mathematical methods due to GA's slow progress in a random-based framework. So, the speed of GA becomes more serious when the number of variables and constraints increases. To obtain the optimal design for a storm-sewer network, various techniques such as cellular automata (CA), heuristic models, heuristic harmony search optimization algorithm, and large-system secondary decomposition–dynamic programming aggregation methods are employed (Guo et al. 2007; Steele et al. 2016; Tan et al. 2019; Tian and He 2020).

Over the past decade, researchers' majority focused on sewage networks only or combined with storm networks. Dynamic programming (DP) technique was used to get the optimum design for a given storm network and tested by discrete differential dynamic programming (DDDP) model (Meredith 1972; Mays and Yen 1975). Then, Afshar applied an adaptive refinement with ant colony optimization algorithms (ACOA) (Afshar 2006) and a re-birthing particle swarm optimization algorithm (RPSO) (Afshar 2008) to solve the same previous problem. Recently, several techniques were used to get the optimal design of the previous network, such as the single-stage CA method (Afshar et al. 2011), the two-stage hybrid cellular automata (HCA) (Afshar and Rohani 2012), and the two-phase simulation–optimization cellular automata (Zaheri et al. 2020). Also, the same previous network was solved using genetic algorithm coupled with a heuristic programming (GA-HP) technique (Hassan et al. 2018). The last methodology (GA-HP) gave the minimum total network cost among all other techniques.

Several studies have developed evolutionary algorithms to achieve the optimum design, taking into consideration three objectives, including reducing the capital cost, flood volume, and total suspended solids (Ghodsi et al. 2016; Eckart et al. 2018; Macro et al. 2019; Xu et al. 2020). In addition, a new methodology to achieve the optimum layout and design of storm-sewer systems is developed (Alfaisal and Mays 2021). The Storm Water Management Model (SWMM) is one of the most influential software for hydraulic simulations, outlining water depths and flow rates, and is used for both design and manual optimization (Cely-Calixto et al. 2020). So, several researchers relied on this software (SWMM) in studies by coupling it with different optimization techniques to achieve the optimal design (Seyedashraf et al. 2021; Fiorillo et al. 2023).

According to all prior studies, it was ultimately determined that there are three primary issues. First, only one or at most two of three design parameters—namely, pipe diameter, pipe slope, and nodal cover depth—were optimized in each study. The second issue relates to the uncertainty of achieving the minimum total network cost. Finally, none of the earlier works examined the hydraulic performance of the optimized network, as all of the previous studies were hydraulically dealing with the network as separate pipes instead of studying the influence of the water surface profile on the overall hydraulic performance of the network. Accordingly, the main objective of this study is to develop an effective hydraulic-based optimization algorithm (HBOA) that can be used to obtain the optimal design for a given layout of stormwater network dynamically with minimal network cost, efficient hydraulic performance, minimal human interference, and minimal run time, while taking all design parameters into account.

Materials and methods

Potential impacts of optimization process

Stormwater networks must be designed in a way that is both optimal and safe. The goal of optimal design is to reduce the total network cost to a minimum while maximizing hydraulic efficiency. An optimized stormwater network meets the sustainable development objectives adopted by all United Nations (UN) member states (Weiland et al. 2021), which focus on the interdependent environmental, social, and economic dimensions of sustainable development. Environmental aspects of optimizing the storm-sewer network can be summarized as follows: Inadequate drainage network design will result in increased surface runoff (due to flooding) on impervious surfaces, roads, and compacted soil, resulting in a high discharge of pollutants from storm-sewers to surface waters. In some cases, the majority of contaminated surface water sinks underground and contributes to groundwater recharge. In addition to increasing the amount of pollutants released from the urban basin, stormwater runoff can also contribute to the erosion of streams, weed growth, and changes in natural flow patterns. Flooding caused by inadequate stormwater design poses a risk of flooding surrounding waterways and their surrounding communities, particularly given the projected rise in greenhouse gases concentrations as a result of climate change.

The social aspects of the optimal storm-sewer network include; urban flooding due to improper design of stormwater network, resulting in traffic gridlock and loss of life and property, particularly in high storm events. In addition to that, urban flooding can lead to sinkhole collapses, resulting in a sudden fall of the road beneath the vehicles, resulting in significant repair costs and additional time. Under-engineered storm network design leads to increased maintenance costs. However, the optimum design of stormwater network reduces the total network cost to a minimum (economic aspects) with the best hydraulic efficiency to prevent urban flooding.

Table 1 Comparison between available software design packages

Item	Available software design package		
	Storm Cad	EPA SWMM	SewerGEMS
Numerical solver	GVF-rational method	SWMM solvers	-SWMM solvers -GVF-Rational Method -GVF-Convex (Sewer Cad)
Calculation type	Used for analysis and design	Used for analysis only	Used for analysis + design or analysis only
Routing method	–	Several methods (uniform, kinetic, dynamic wave)	Several methods (uniform, kinetic, dynamic wave)
Flow	Deal with peak discharge	Deals with flow as actual calculated flow	Deals with flow as actual culated flow or peak discharge (GVF-Rational Method)
Getting optimum design	Takes less time and effort, but does not get the optimum design	Takes more time and effort, to get the optimum design	Takes more time and effort, to get the optimum design

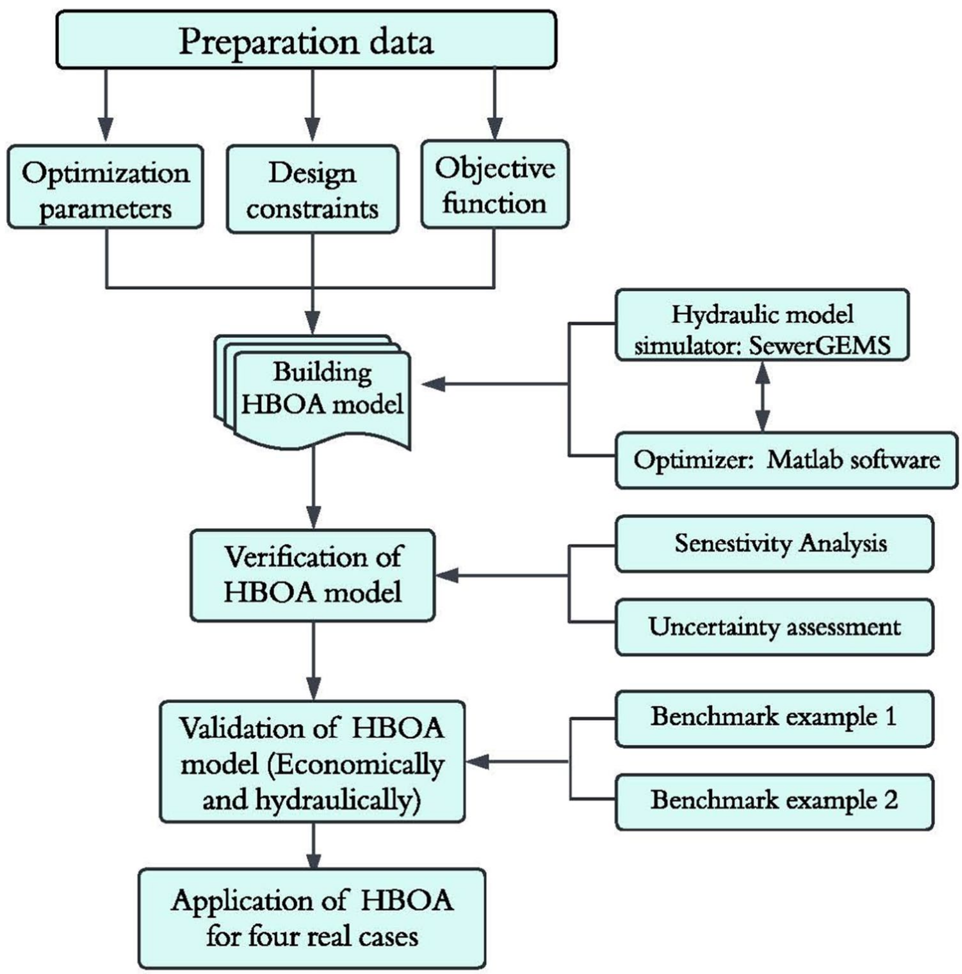


Fig. 1 Research methodology and approach

Optimization parameters

By reviewing the design procedures and standards related to stormwater drainage networks, the selected design parameters for a given layout are pipe diameter (D), pipe slope (S), and pipe material. Meanwhile, the other design parameters (i.e., soil cover, percentage full, etc.) are either related to the selected design parameters or considered design constraints.

Design constraints

The importance of the design constraints is raised due to their direct effect on the built model's performance. So, to build an optimization algorithm to be used in the dynamic design of a storm drainage system (the main objective of this study), two groups of design constraints are considered. The first group of design constraints is associated with hydraulic performance, while the second group is associated with design parameters.

Constraints related to hydraulic performance

Velocity constraint The pipe velocity should be greater than the minimum permissible velocity (which ranges from 0.3 to 0.6 m/s and varies based on the project area) for sediment cleaning. Also, the maximum velocity should be less than the maximum permissible velocity to prevent pipe abrasion, which leads to a shorter life span, which depends on the pipe material. Usually, the acceptable range of velocity is assigned based on the applied design standards in the project area.

Pipe slope constraint The slope of each pipe should be within a minimum and a maximum permissible value according to the applied design standards in the project area.

Flooding constraint The total flood volume in the storm drainage system should be less than the permissible value. The permissible value is determined according to the applied design standards in the project area. Some design standards do not allow the water depth inside manholes to be raised higher than a specific value. Also, other design standards specify the maximum fullness percentage for all pipes.

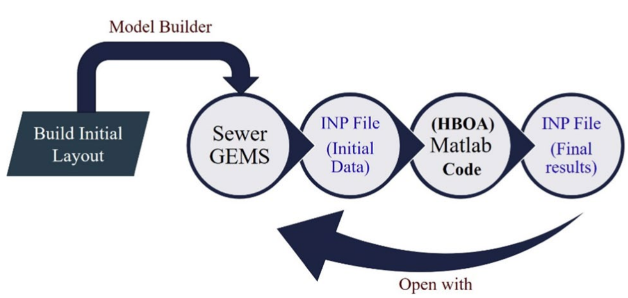


Fig. 2 Interrelation between SewerGEMS and the HBOA code

Constraints related to design parameters

Pipe diameter constraint The diameter of any downstream pipe should be equal to or greater than the diameter of the upstream pipe along the flow direction, based on the available commercial pipe diameters.

Pipe cover constraint It is necessary to provide adequate cover depth to avoid pipe damage due to loads. The cover depth should be greater than the minimum allowable cover depth, depending on local factors and specifically on the pipe material used.

Connection of pipes constraint The pipes in the stormwater network should be linked crown to crown at the manholes.

Objective function

The total construction cost (objective function) of the storm drainage system is mainly depending on the costs of pipes, earthwork, and manholes including purchasing, transporting, and laying the pipes in the excavated trenches, …etc. So, the total cost of the network is calculated as follows:

$$\text{Total Cost} = \text{Pipe Cost} + \text{Manhole Cost} + \text{Earthwork Cost} \quad (1)$$

Hydraulic model simulator

There are several available software design packages used for the design of stormwater networks, such as Storm Cad, EPA SWMM, SewerGEMS, …etc. Table 1 illustrates the comparison between the available design software packages based on their user manuals. SewerGEMS Bentley software is a fully-dynamic precipitation modeling software and surface runoff simulation. It is used to perform hydraulic modeling for drainage networks (stormwater and sewer networks) for different return periods. SewerGEMS software will be used as the hydraulic model simulator for this study to design the internal stormwater drainage system.

Optimizer

The optimizer can be built using different software, but in this study, the optimizer code is built using MATLAB due to the availability of a huge library of predefined functions, its ease of coding, and its graphical user interface.

Research methodology

The research methodology is illustrated in Fig. 1, and it will be described in the following sections.

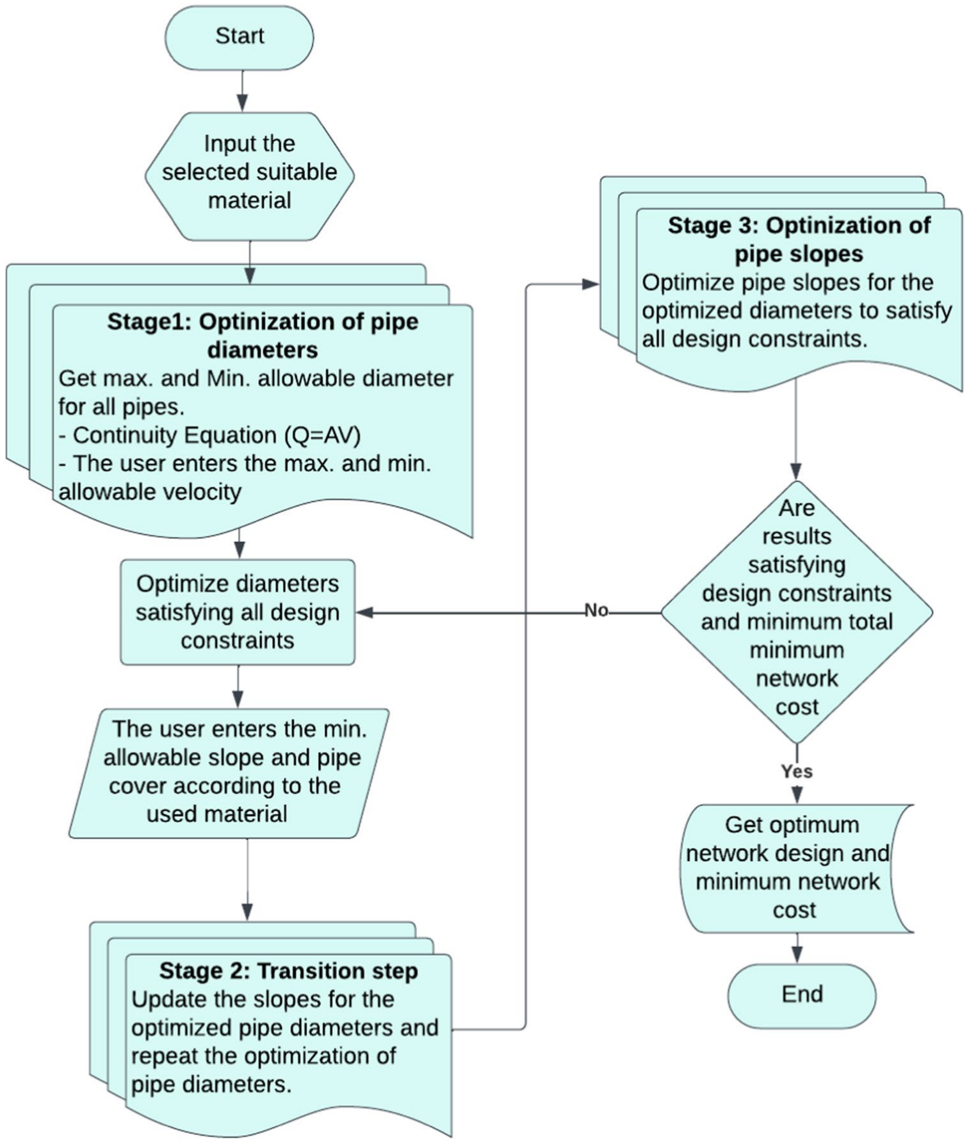


Fig. 3 Schematic flowchart for the optimization process of HBOA

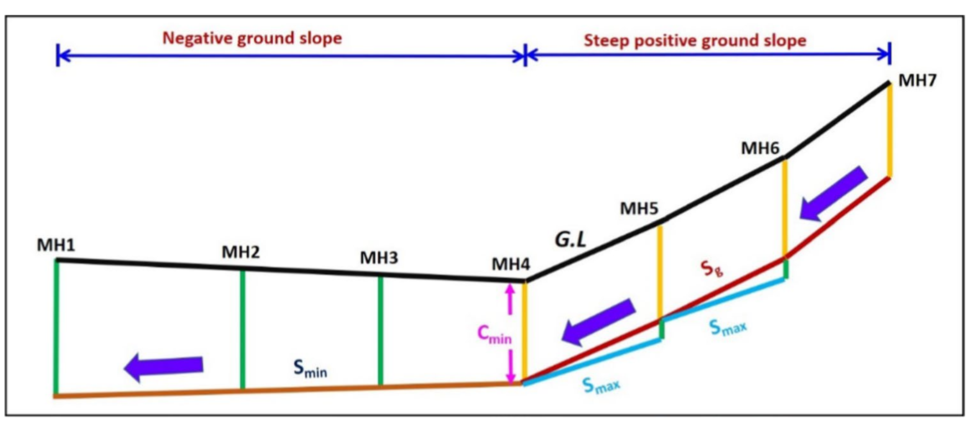


Fig. 4 Longitudinal profile of the technique used for the optimal slope

Building HBOA model

After the identification of the optimization parameters and the objective function, building of the new technique will be presented. The new technique is called the hydraulic-based optimization algorithm (HBOA). HBOA is built using MATLAB code linked with SewerGEMS software. SewerGEMS is used as a hydraulic model simulator, and MATLAB code is used as an optimizer. The technique used in building HBOA model can be summarized in the following steps.

Step 1: interrelation between SewerGEMS and the HBOA model To define a layout for a stormwater network, there is a need to determine certain information, such as topography, the plan of the study area, and the location of the outlet of the drainage system. All previous information will be used in building the proposed storm layout by using SewerGEMS. Pipes, nodes, and catchments with their initial characteristics will be built through a model builder in SewerGEMS. Also, rainfall data should be entered as time-depth, time-intensity, intensity duration frequency (IDF) curve, …etc.

For a given network layout, SewerGEMS will construct an INP file (which contains all of the initial data). At this time, the MATLAB code (HBOA) will be able to read this file, connect to it, and attempt to make the necessary changes for the optimization process in this file (the INP file). After that, SewerGEMS will receive the amended INP file and run a hydraulic simulation to test the system's hydraulic performance with the newly modified, optimized parameters. The INP file will be returned to the MATLAB code to make yet another modification if there is a hydraulic issue with the SewerGEMS simulation. This process will continue for the given layout until the final optimized parameters are obtained in order to get the lowest construction cost with the best hydraulic performance. The relationship between SewerGEMS and the HBOA code is shown in Fig. 2. The optimization process is divided into three sub-processes with different decision variables, which are solved iteratively using HBOA as described in the next sections. Figure 3 presents a schematic flowchart for the optimization process of HBOA.

Step 2: optimization to pipe diameter—stage 1 In the first optimization stage, the pipe diameters are considered decision variables of the optimization problem, and the pipe nodal elevations are fixed. The network starts with a large diameter and minimum pipe slope as initial values for all pipes, and all other parameters are fixed. First, the model optimizes all branch diameters, then optimizes the main pipe diameter network, and checks all design constraints until reaching an optimum design that satisfies all constraints with minimum cost.

If the designer is trying to optimize the pipe diameters for a particular pipe slope using the manual optimization process, this process will take a lot of time and effort, especially for a large network, depending on the design engineer's expertise. In manual optimization, there is a need to do several manual iterations in order to be close to the optimum pipe diameter because if you decrease the pipe diameter at the downstream end of the network, it can cause flooding at the upstream of the network, and so on, and at the end of this process, there is no guarantee that the optimal design is reached. HBOA will perform these iterations automatically, taking the design constraints into account, until it gets the optimal pipe diameters with minimal effort and time without any human intervention.

Step 3: transition step—stage 2 The outcomes from the previous stage are the optimum pipe diameters for a fixed pipe slope value (minimum pipe slope value). So before going to the third step, there is a need for this transition step. In this step, all the pipe slopes will be adjusted based on the last optimal diameters achieved from stage 1, based on the input data that tie the pipe slope to the pipe diameter. At this point, two checks must be made: first, that all design constraints are met; second, that the pipe diameters are still optimal with the modified pipe slopes; otherwise, stage 1 must be repeated until the adjusted optimum pipe diameters are reached. In this stage, if a designer has decided to perform the previous tasks manually, the designer will have to perform more and more iterations, which will take more time and effort, as mentioned in the previous stage. However, the HBOA model will do all these tasks automatically with minimal time and effort.

Step 4: optimization to pipe slope—stage 3 Meanwhile, in the third stage, the slope of the pipe is considered a decision variable of the optimization problem, while the pipe's diameter is obtained from the second stage. The user enters the allowable minimum slope for each used diameter according to the standards of the location of the study area and the allowable minimum cover according to the used material. The technique used to reach the optimal slope is that if the slope of the ground level (S_g) is negative, the slope of the pipe will be used as the minimum allowable pipe slope according to the diameter used. If the slope of the ground level (S_g) is positive, the slope of the pipe will be parallel to the slope of the ground level (S_g), except in cases where the slope of the ground level (S_g) exceeds the maximum allowable slope (S_{max}), so the drops will be done to satisfy the maximum allowable slope, as shown in Fig. 4. Pipe slope and pipe diameter updates are performed on a step-by-step basis (using HBOA instead of manual processes to reduce time and effort) until convergence is achieved, and the optimal design has been identified that meets all design requirements.

Verification of HBOA model

Verification is the process of determining if the software is designed and developed as per the specified requirements and its results are correct and stable or not. So, the developed HBOA model is verified using two different analyses: The first involves assessing the sensitivity of the model's output to its input parameters, and the second involves evaluating the model's final output based on various initial input data values under an uncertainty assessment process.

Validation of HBOA model

Validation is the process of checking if the software (end product) has met the client's true needs and expectations. So, both an economic and a hydraulic perspective are used to validate the HBOA model. The hydraulic point of view examines the hydraulic efficiency of the storm network for the entire layout, whereas the economic point of view examines getting the least overall network cost. The effectiveness of the suggested HBOA model is validated using two benchmark examples from the literature.

Application of HBOA model

Four actual storm networks from various three countries that have already been planned, built, and optimized manually are used to test the HBOA concept. The outcomes of HBOA for the real cases are compared with those of the actual designs in order to test the applicability of using the HBOA model.

Results and discussion

Verification of HBOA model

The verification of the developed HBOA model is conducted through two different analyses (sensitivity analysis and uncertainty assessment).

Sensitivity analysis

The sensitivity analysis is conducted to test the sensitivity of the model results to the input parameters. Pipe diameter is the only input parameter in the initial layout, while other parameters are calculated automatically within the HBOA environment. So, the pipe diameter is changed several times to start with several large and small values. Based on that, the initial pipe diameter is assumed to be in a range between 50 and 2000 mm. So, the initial depth is randomly selected several times (i.e., 1000 times) to be within the specified wide range. Then, the HBOA was run to find the optimum diameter based on each initial diameter. The results show that the final diameter has the same value (550 mm) regardless of the initial diameter value, as shown in Fig. 5.

Additionally, to confirm the results of sensitivity analysis, the root-mean-squared error (RMSE) equation is applied using Eq. (2).

$$\mathrm{RMSE} = \left[\frac{1}{ns}\sum_{i=1}^{ns}\left(X_\mathrm{r} - X_\mathrm{s}\right)^2\right]^{0.5} \tag{2}$$

where ns is the number of trials (1000 trial), X_r is the reference parameter (diameter) value, and X_s is the calculated parameter (diameter) value.

The root-mean-squared error equation is applied using 1000 initial diameter values. The calculated RMSE value is found to be equal to ZERO, which means that the HBOA model is insensitive to the initial diameters.

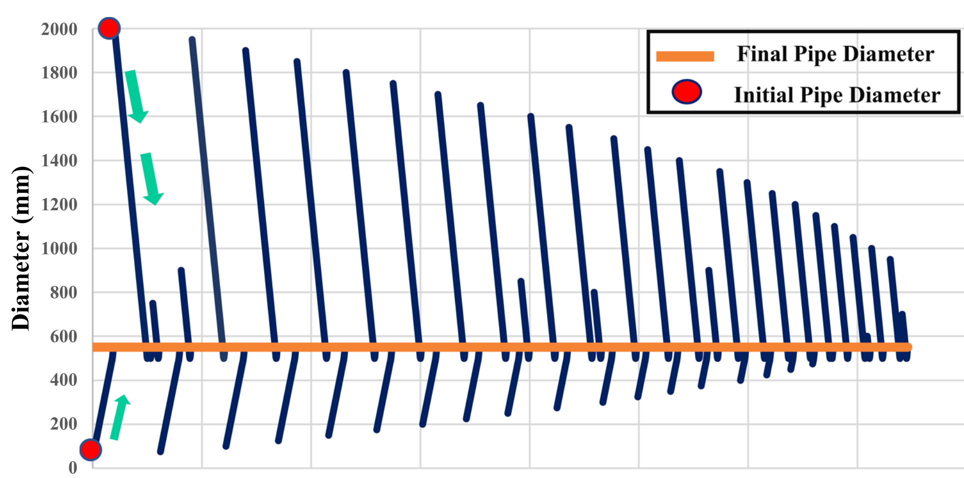


Fig. 5 Sensitivity analysis results

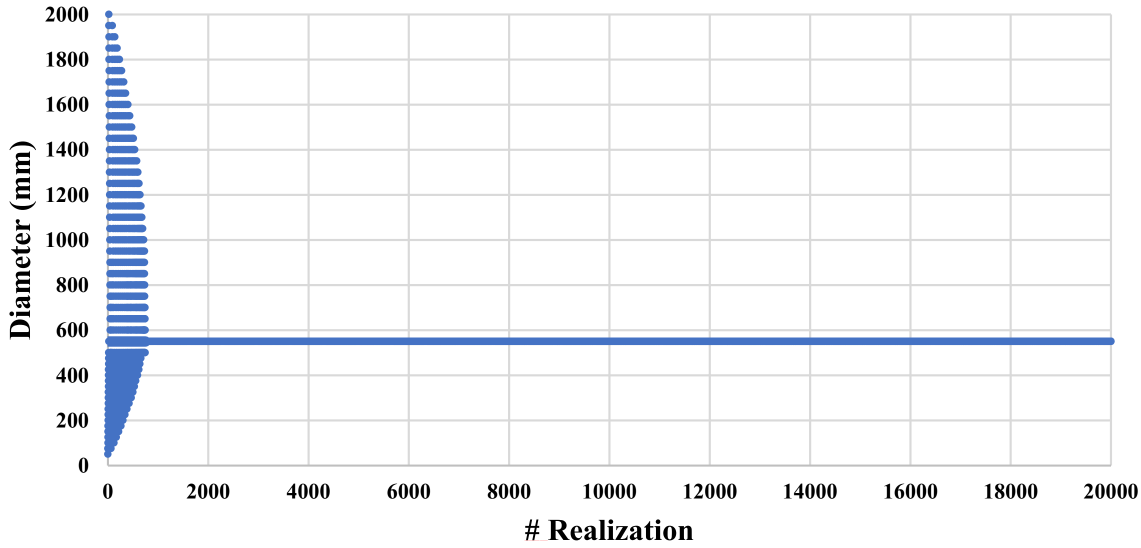


Fig. 6 Results of uncertainty assessment

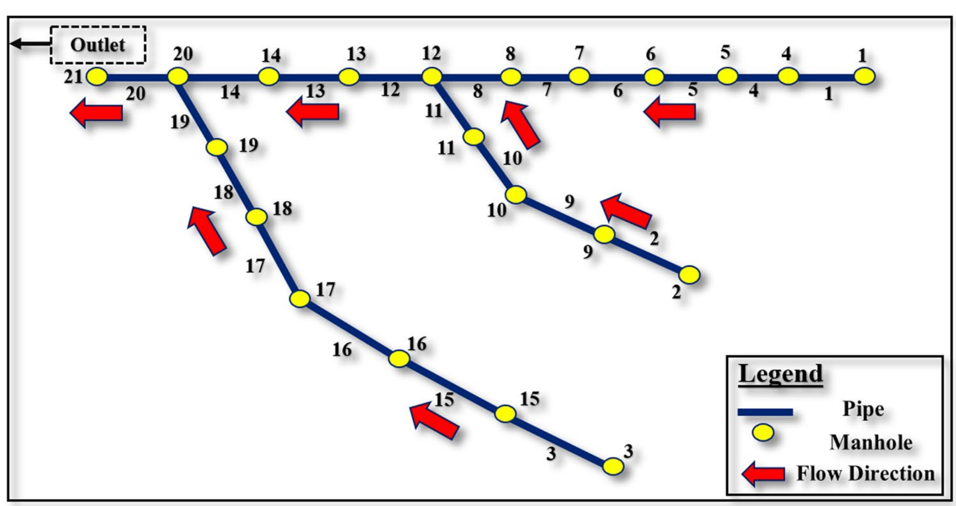
Uncertainty assessment

The uncertainty assessment of the HBOA model is conducted using the bootstrapping technique. The bootstrapping strategy is a technique for generating a large number of random samples with replacement from a single dataset to measure uncertainty (Zhang et al. 2017). In this study, this approach is utilized to create 20,000 random samples (realizations) or combinations for the storm network with various pipe diameters. In each realization, different pipe diameters are assigned to the pipe network. So, each pipe in the network has a different initial pipe diameter in the same realization. Then, the HBOA was run to find the optimum diameter for each realization. The model shows certain results as it gives the same final result regardless of the initial dataset values (refer to Fig. 6). Based on that, the newly proposed model (HBOA) is certain and stable.

Validation of HBOA model

Two benchmark examples from the literature are used to validate the performance of the proposed HBOA model. HBOA model was validated from an economic and hydraulic point of view. The first benchmark example is part of Kerman sewerage system in Iran, which was originally designed by Mansuri and Khanjani (1999) using mathematical programming and genetic algorithm (GA). The second benchmark example is part of the storm-sewer network, originally designed by Mays and Yen (1975), and the same example was also solved by several researchers. Each network consists of 21 nodes with 20 links; refer to Fig. 7 for the first benchmark

Fig. 7 Layout of the first benchmark example



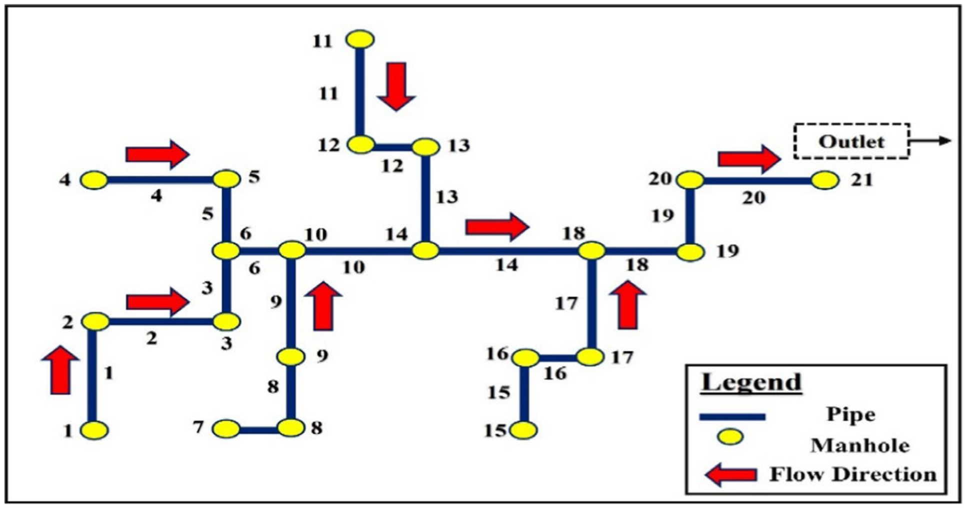


Fig. 8 Layout of the second benchmark example

Table 2 The network design constraints for the two Benchmark examples (Afshar et al. 2016; Hassan et al. 2018)

Design constraint	Benchmark examples	
	First example	Second example
Minimum velocity (m/s)	0.3	0.61
Maximum velocity (m/s)	3.0	3.66
Maximum fullness percentage for all pipes ($d/d_{max.}$)	0.82	0.82
Manning roughness coefficient (n)	0.013	0.013
Minimum soil cover (m)	2.45	2.4
Commercial pipe diameters (mm)	200, 250, 300, 350, 400, 450, 500, and 600	304.8, 381, 457.2, 533.4, 609.6, 762, 838.2, 914.4, 990.6, 1066.8, and 1219.2

Table 3 Results of HBOA compared to previous optimization methods—first benchmark example (Hassan et al. 2018)

Model	Optimization method	Total system cost ($)
Mansuri and Khanjani (1999)	NLP (nonlinear programming)	83,116
Sotoodeh (2004)	BFGS (Broyden–Fletcher–Goldfarb–Shanno algorithm)	82,732
	Fletcher–Reeves	81,553
Hassan et al. (2018)	GA-HP	81,265
Present model	HBOA	81,212

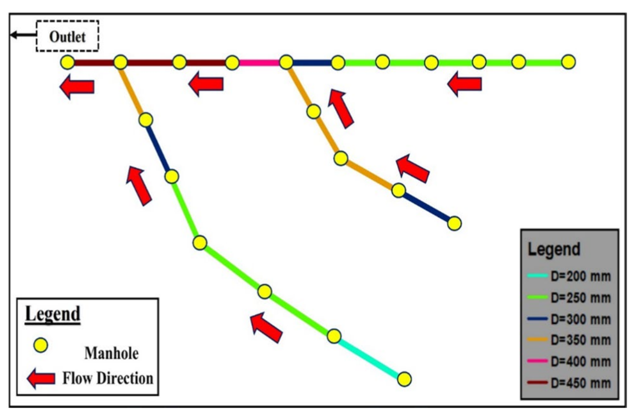


Fig. 9 The optimal pipe diameter (mm) of the drainage network for the first benchmark example using the GA-HP method and the HBOA method

and Fig. 8 for the second benchmark. Table 2 illustrates the design constraints for the two benchmark examples.

Economic validation (cost comparison) of HBOA model

First benchmark example The proposed HBOA method is used to solve this example, and the results are compared with the existing design as shown in Table 3. The cost function for excavation, manhole, and pipe installation was assigned as per Mansuri and Khanjani (1999). As depicted in Table 3, the HBOA model has the lowest construction cost and opti-

Table 4 Results of HBOA compared to previous optimization methods—second benchmark example (Hassan et al. 2018; Tan et al. 2019)

Model	Optimization method	Total system cost ($)
Mays and Yen (1975)	DDDP	265,775
Robinson and Labadie (1981)	Version of DP	275,218
Miles and Heaney (1988)	Spreadsheet	245,874
Afshar (2006)	ACO	241,496
Afshar et al. (2011)	CA	253,483
Afshar (2012)	Rebirthing GA	241,988
Roheni and Afshar (2012)	Hybrid CA	247,412
Yeh et al. (2013)	TS	244,571
Yeh et al. (2013)	SA	241,770
Zaheri et al. (2020)	Two-phase CA	240,084
Afshar et al. (2016)	Adaptive CA	239,757
Hassan et al. (2018)	GA-HP	239,672
Tan et al. (2019)	Harmony search	240,981
Present model	HBOA	238,030

mal design in comparison with other methods. Details of the optimal design attained by the proposed model (HBOA) are shown in Table 10 in Appendix A. There was no difference in the obtained pipe diameters, either in the results obtained using GA-HP (Hassan et al. 2018) or using HBOA. But the difference in the cost between the two models comes from the manholes cost due to the reduction of the total manhole depths using HBOA model. Figure 9 illustrates the optimal pipe diameters obtained from the literature review and from the HBOA method.

Second benchmark example The proposed HBOA method is used to solve this example, and the results are compared with the existing design as shown in Table 4. The cost function for excavation, manhole, and pipe installation was assigned as per Meredith (1972). The HBOA method has the lowest cost and optimal design in comparison with other methods. Details of the optimal design attained by the proposed model (HBOA) are shown in Table 11 in Appendix A. Figure 10 illustrates the difference in the designed optimal pipe diameters obtained from the literature review and from using HBOA method for this example.

Based on the results of the two benchmark examples, the HBOA optimization technique gives better results with a minimum total network cost and a minimum consumed time compared to the currently available optimization techniques (proposed by other researchers).

Hydraulic validation of HBOA model

The SewerGEMS program is used to simulate water flow within the network and evaluate the hydraulic performance of the two previous benchmark examples. The network surcharge and the maximum fullness percentage for all pipes (d/d_{max}) are the two key factors that must be examined in order to validate the hydraulic system.

First benchmark example During the evaluation of the results of the first benchmark example, there was a flood in the system equal to 3 m^3 (≈ 0.02% inflow), as shown in Fig. 11. Although this value is very small, it means that the given design constraints are not satisfied. The constraints were: No flooding occurs in the system, and the maximum fullness ratio of the pipe should not exceed 82% (see Table 2).

Figure 12 illustrates the pipe fullness inside the network for the first example as designed by the previous researchers and by the HBOA model. It is found that some pipes obtained using GA-HP method exceed the maximum fullness ratio (82%), but all pipes designed by HBOA do not exceed 82%.

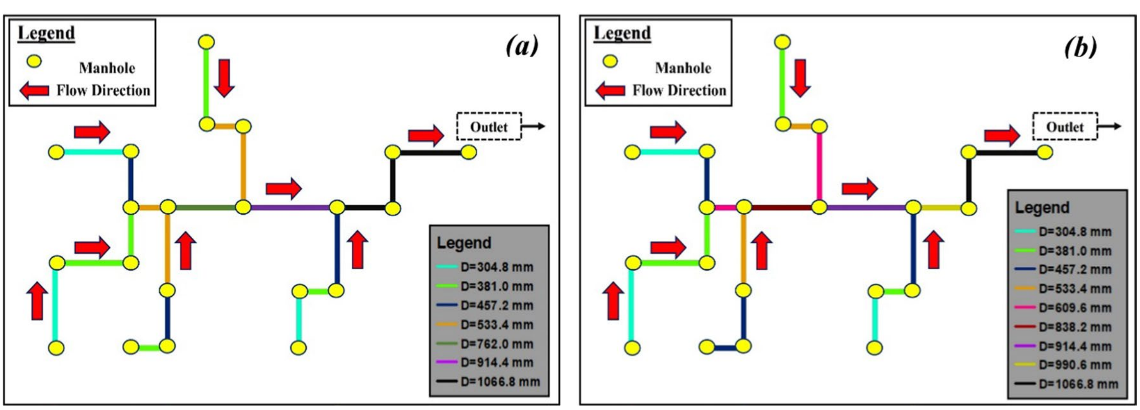
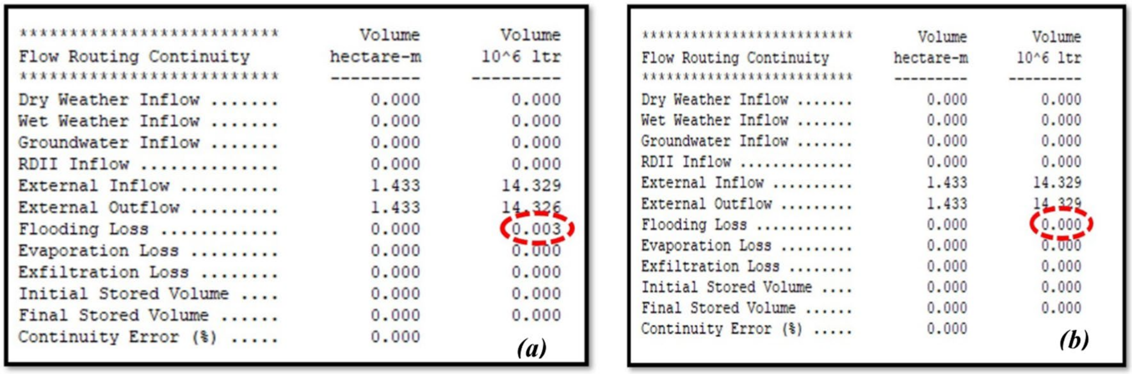


Fig. 10 The optimal pipe diameter (mm) for the second benchmark example (**a**) using the GA-HP method and (**b**) using the HBOA method



(a)

Flow Routing Continuity	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	1.433	14.329
External Outflow	1.433	14.326
Flooding Loss	0.000	0.003
Evaporation Loss	0.000	0.000
Exfiltration Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.000	

(b)

Flow Routing Continuity	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.000	0.000
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	1.433	14.329
External Outflow	1.433	14.329
Flooding Loss	0.000	0.000
Evaporation Loss	0.000	0.000
Exfiltration Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.000	

Fig. 11 The surcharge results for the first benchmark example **(a)** using the GA-HP method and **(b)** using the HBOA method

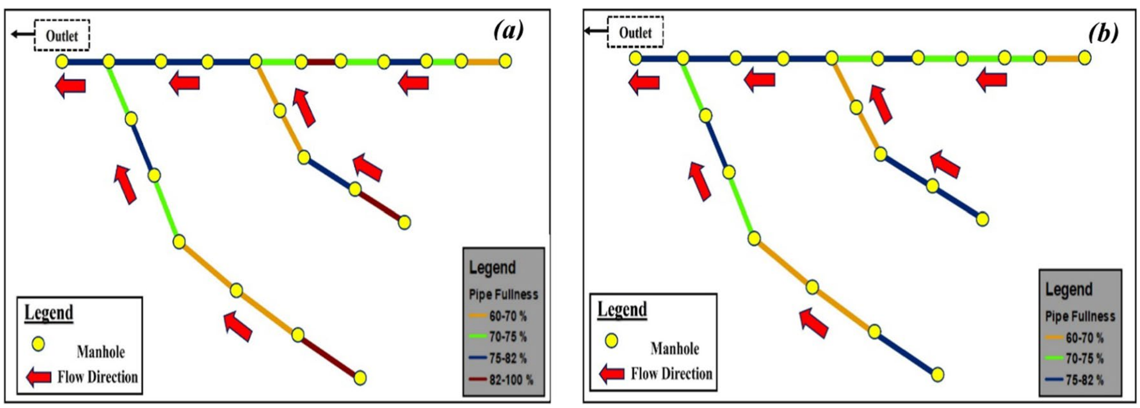


Fig. 12 The fullness percentage for all pipes for the first benchmark example **(a)** using the GA-HP method and **(b)** using the HBOA method

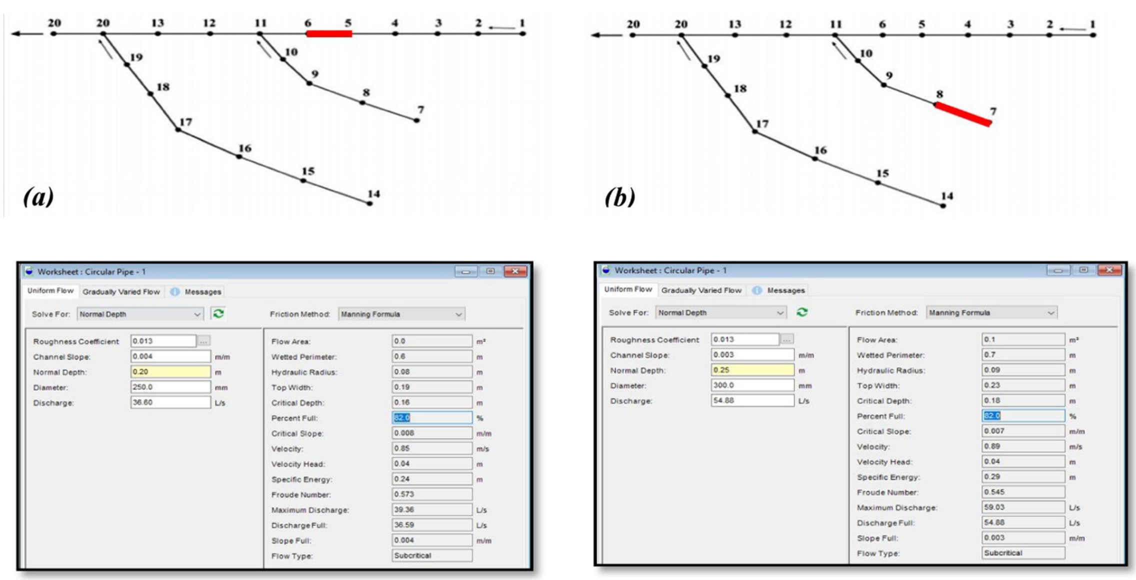


Fig. 13 The hydraulic calculation for the first benchmark example using the GA-HP method **(a)** for pipe number 7 and **(b)** for pipe number 2

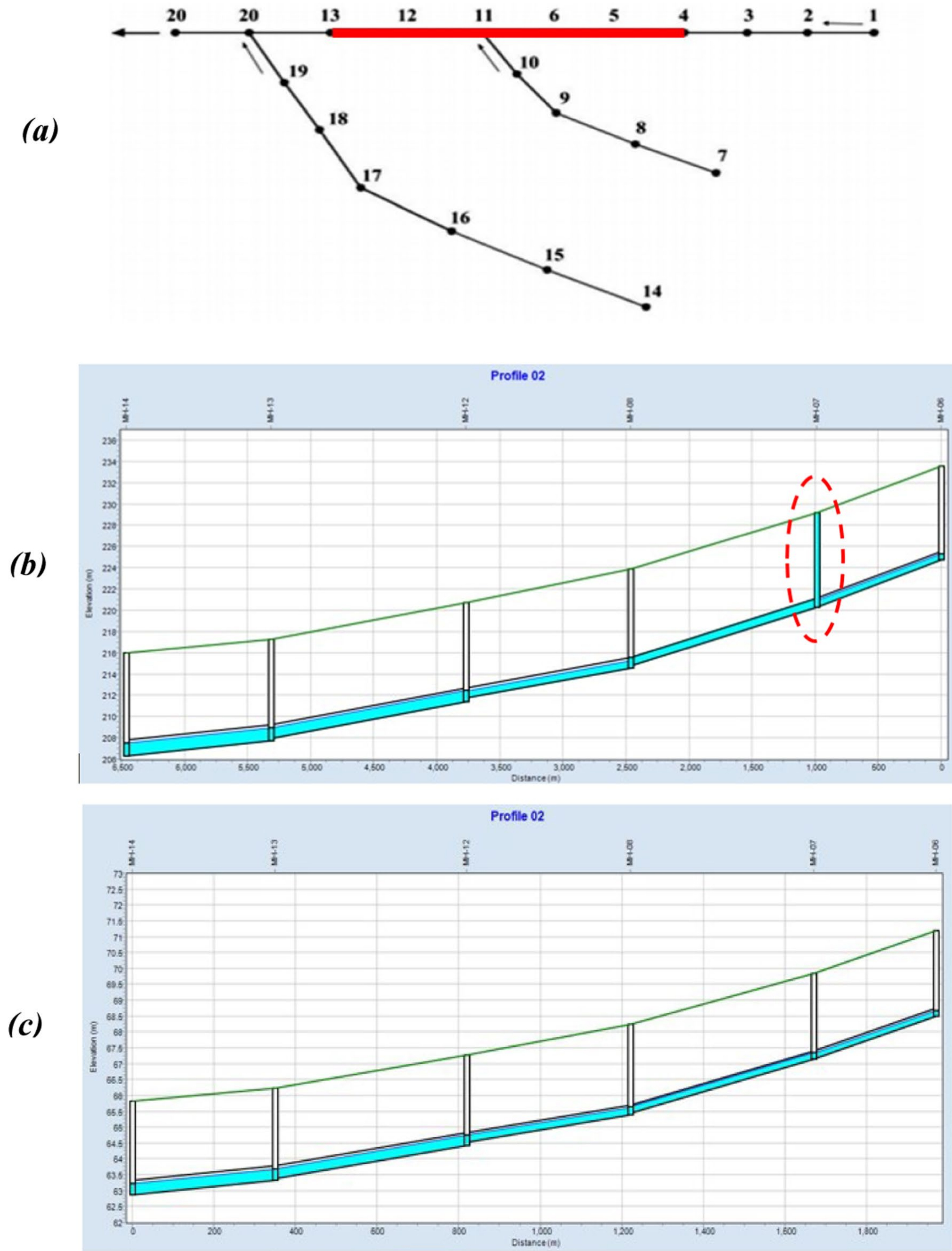


Fig. 14 Longitudinal profile for the first benchmark example **(a)** key plan, **(b)** GA-HP method, and **(c)** HBOA method

Figure 12a shows that pipes 2, 3, and 7 (as shown in Figs. 7 and 12, respectively) have higher pipe fullness ratios than 82% for the design according to GA-HP (see Hassan et al. 2018). However, when we tested pipe fullness for these pipes individually (using FlowMaster software and Manning equation), we found that pipes 2, 3, and 7 satisfy pipe fullness ratios of 82% according to the design constraints (see Fig. 13). On the other hand, if the HBOA model is used, the hydraulic results satisfy the design constraints and pipe fullness ratio, so there is a difference between the hydraulic results obtained by GA-HP and HBOA for the simulated network.

The explanation of this difference confirms that the design obtained using the GA-HP method is based on the design of each pipe separately within the network, as per the FlowMaster software, without checking hydraulic gradient across the entire network or considering upstream and downstream pipes. This is the reason for surcharged manholes numbers 7, 2, and 3 when the whole network was simulated with SewerGEMS (Fig. 14). Therefore, it is clear that the network optimization through the GA-HP method was done based on designing each pipe individually, knowing its flow, slope, and material type, and applying the Manning equation without considering the water surface profile and its effect on the entire network.

Second benchmark example In a similar manner to the first example, the network of the second benchmark has a flood in the system equivalent to 75 m^3 (0.03% inflow), as shown in Fig. 15. So, the previous researchers did not satisfy the design constraints. However, the HBOA-designed network does not have a surcharge, as shown in Fig. 16. The surcharged pipes are pipe numbers 4, 12, 13, 15, and 19 (see Figs. 8 and 16) which have fullness percentages higher than 82%. The main reason for having surcharged pipes for this example from the previous researchers is, as previously mentioned in the first example, due to the separate design of each pipe in the network as shown in Fig. 17. The longitudinal profiles along some of the surcharged pipes in the prior design are shown in Fig. 18, and the HBOA model solved this issue.

Finally, although it is unfair to compare the results of HBOA with the two benchmark networks due to the significant difference between the applied hydraulic principals in HBOA procedures and the procedures of the two benchmarks, HBOA gives better results not only from the hydraulic performance point of view but also from the construction cost point of view. To ensure a fair cost estimate comparison, for the first benchmark example, if the HBOA method permits all pipes to be filled to the same fullness percentage as in the hydraulic evaluation results from the literature, the optimum cost of the network using the HBOA method will be 81,180 $ instead of 81,212 $ (mentioned in Table 3), representing a cost

```
**************************        Volume         Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.000         0.000
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........        22.997       229.972
External Outflow .........        22.989       229.897
Flooding Loss ............         0.008         0.075
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.000         0.000
Continuity Error (%) .....         0.000
                                                   (a)
```

```
**************************        Volume         Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.000         0.000
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........        22.997       229.972
External Outflow .........        22.997       229.972
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.000         0.000
Continuity Error (%) .....         0.000
                                                   (b)
```

Fig. 15 The surcharge results for the first benchmark example **(a)** using the GA-HP method and **(b)** using the HBOA method

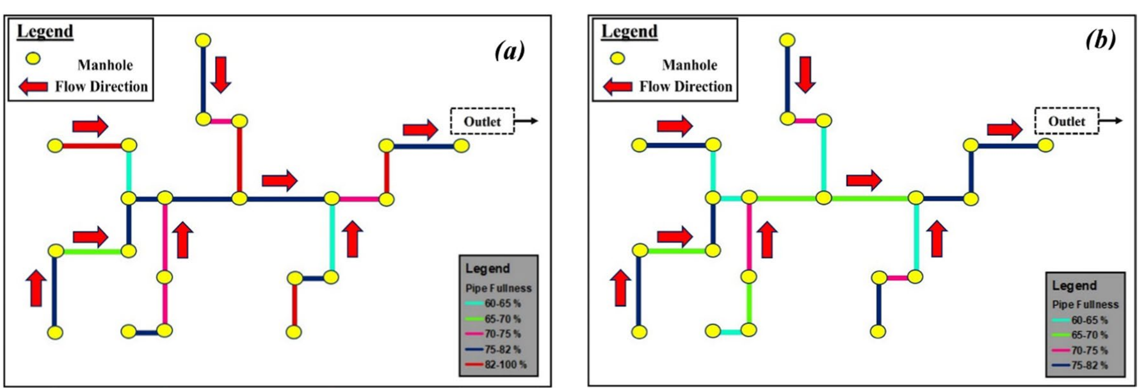


Fig. 16 The fullness percentage for all pipes for the second benchmark example **(a)** using the GA-HP method and **(b)** using the HBOA method

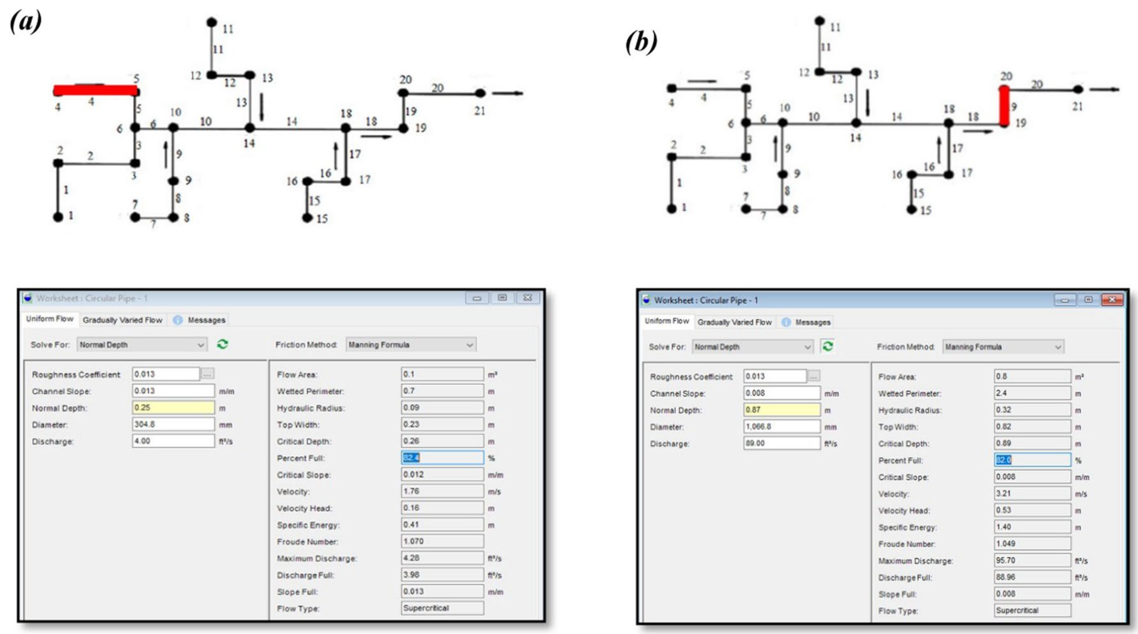


Fig. 17 The hydraulic calculation for the second benchmark example using the GA-HP method **(a)** for pipe number 4 and **(b)** for pipe number 9

reduction of approximately 0.1% compared to the findings of the literature review. In the same way, for the second benchmark example, if it is permitted to have the same fullness percentages for all pipes, the optimum cost of the network using the HBOA method will be 236,150 $ instead of 238,030 $ (mentioned in Table 4), representing a cost reduction of approximately 1.5% compared to the findings of the literature review.

Application of HBOA model

In addition to the validation process conducted using two benchmarks' examples, the performance of the proposed HBOA model is tested in this section using four real cases. The four real cases are selected from three different countries to present different standards and requirements. Furthermore, the four real cases present different project scales, and all of them are already constructed or under construction. The main characteristics of the four projects are presented in Table 5. All cases have different design constraints according to the standards applied in the served area of each network. The general alignments of the four real cases of the storm network are shown in Figs. 19, 20, 21, and 22.

Design constraints

Table 6 shows the design constraints for these four real cases: The cost objective function used in these four real cases is generalized to include the following four components: excavation cost, pipe cost, manhole cost, and fill cost, as shown in the following equation:

$$\begin{aligned}\text{Total Cost} = {} & (K * \text{Unit price of excavation}) \\ & + (L * \text{Unit price of each pipe diameter}) \\ & + (N * \text{Unit price of manhole}) \\ & + (Z * \text{Unit price of fill})\end{aligned} \tag{3}$$

where K and Z are the total cost function's parameters, it depends on the project location. The term L refers to the pipe length (m) associated with each pipe diameter, and N refers to the number of manholes. Table 8 provides an overview of the total cost function's parameters K and Z. On the other hand, in case (1), the Z parameter value will be set to zero as the backfilling cost is already covered by the unit prices of pipes and manholes. Meanwhile, in case (2), the unit prices of pipes and manholes include the excavation and backfilling costs, so the K and Z parameter values will be set to zero.

Where W is the width of the pipe trench (m), D is the pipe diameter (m), Y is the average excavation depth until the invert of the pipe (pipe cover plus pipe diameter) (m), and L is the length of the pipe (m). Unit prices of the excavation, pipes, manholes, and backfills are illustrated in Tables 12, 13, 14, and 15 in Appendix A.

Outputs from HBOA

Based on the given layouts for the four real cases, the HBOA model is used to reach the optimal design for each network

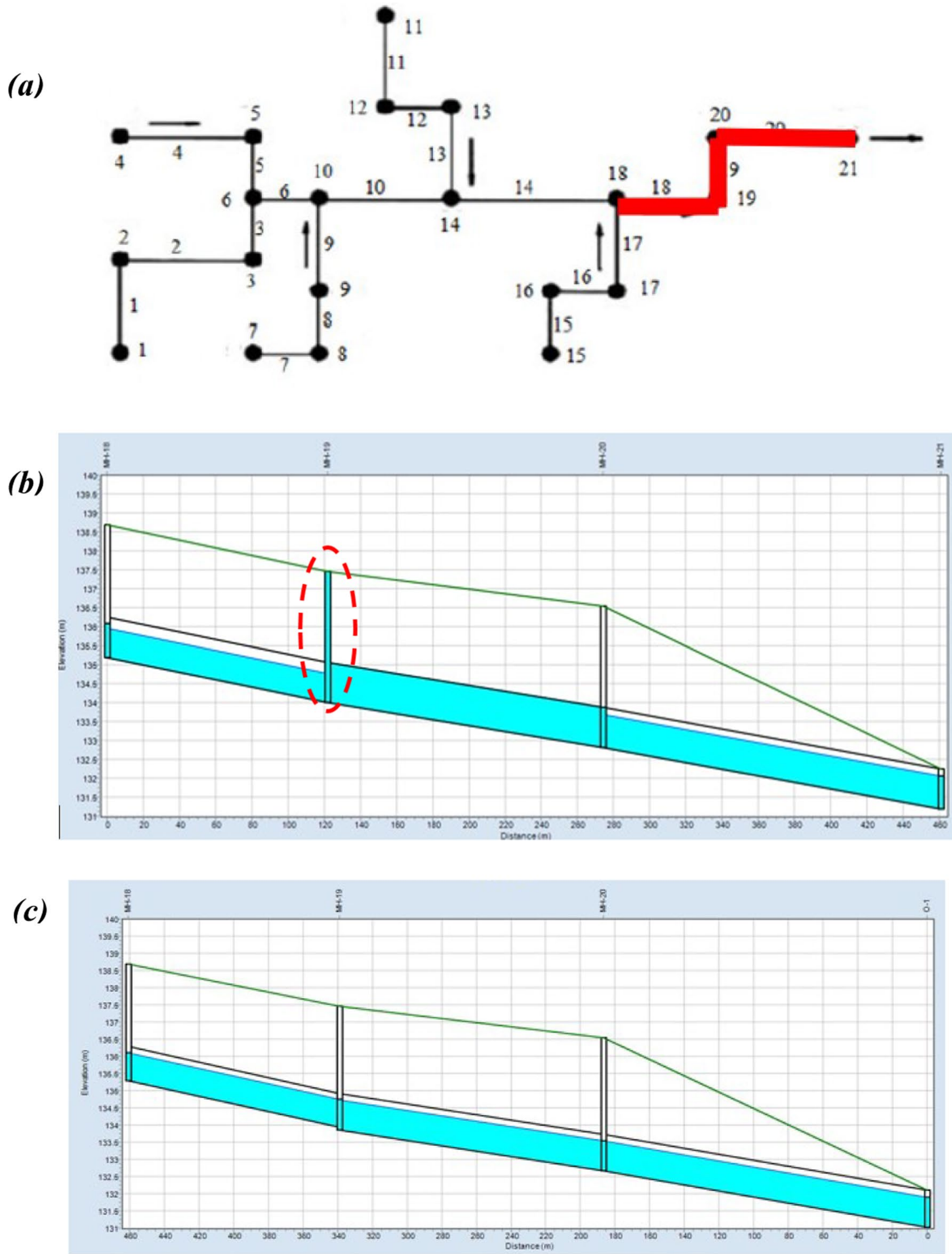


Fig. 18 Longitudinal profile for the second benchmark example **(a)** key plan, **(b)** GA-HP method, and **c** HBOA method

Table 5 Characteristics of four real projects

Case number	Project area	Project location	Total pipe length (m)	Number of pipes
1	Dalkhoot	Sultanate of Oman	680	24
2	Basra	Republic of Iraq	1190	27
3	Al Qassim	Kingdom of Saudi Arabia (KSA)	2165	31
4	Al Naq		12,350	174

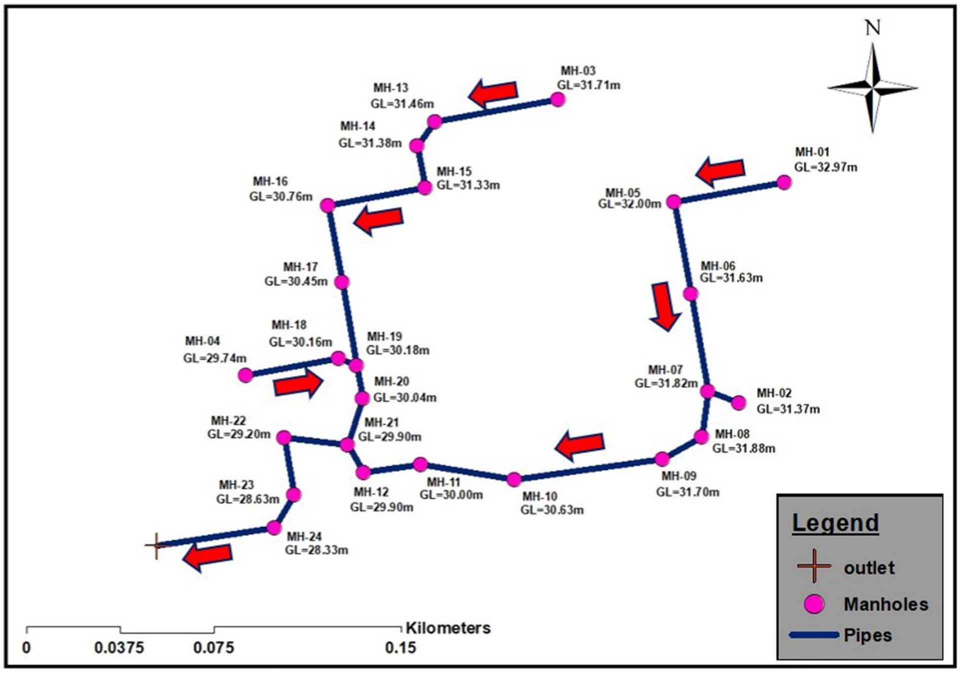


Fig. 19 General layout for the proposed storm network in Dalkhoot area (Case 1)—Oman

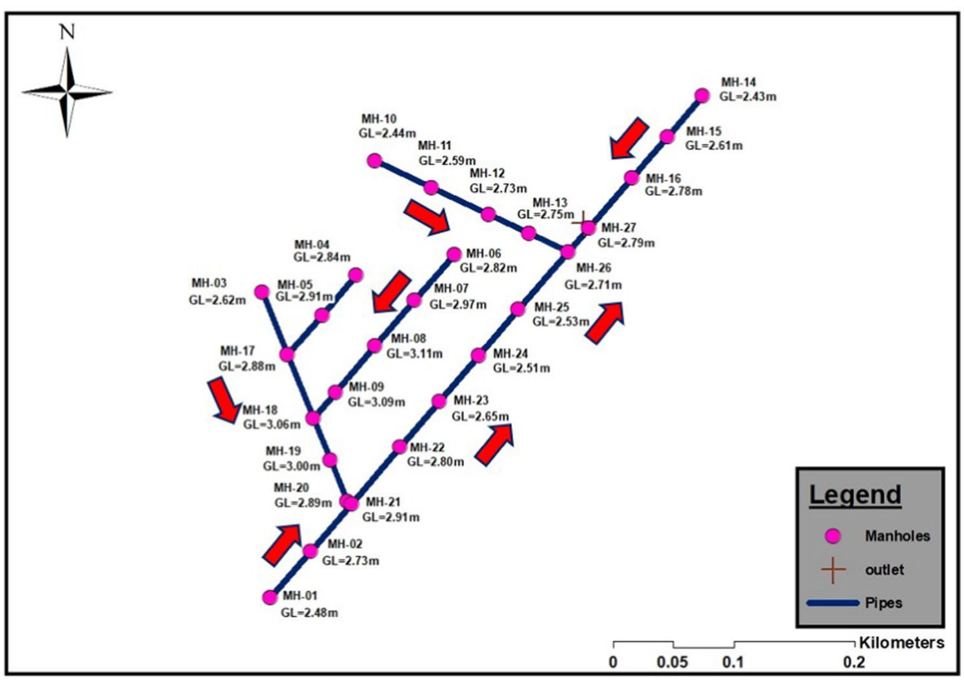


Fig. 20 General layout for the proposed storm network in Basra area (Case 2)—Iraq

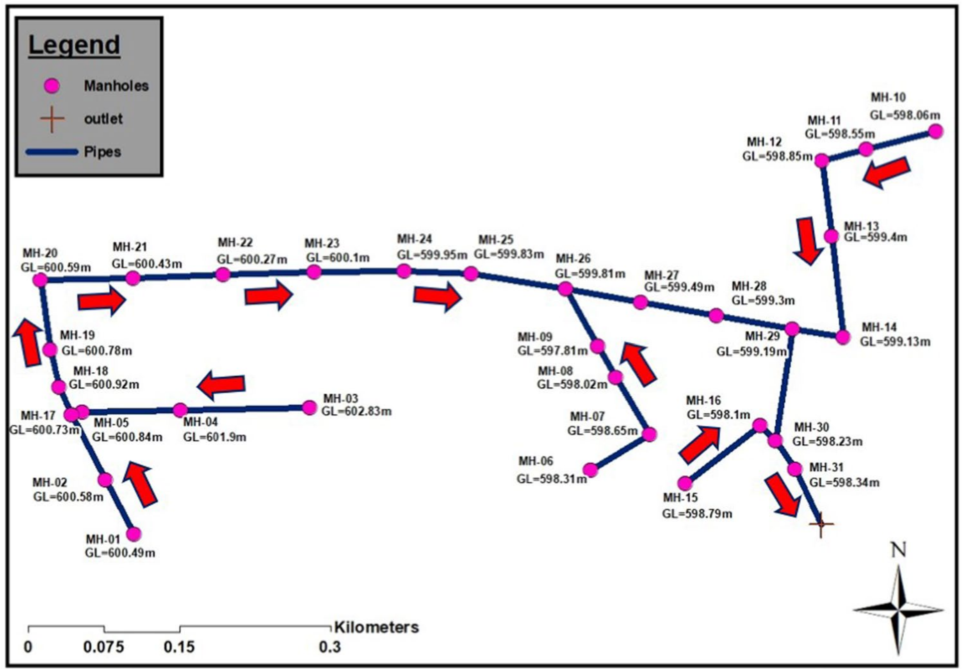


Fig. 21 General layout for the proposed storm network in Qassim area (Case 3)—KSA

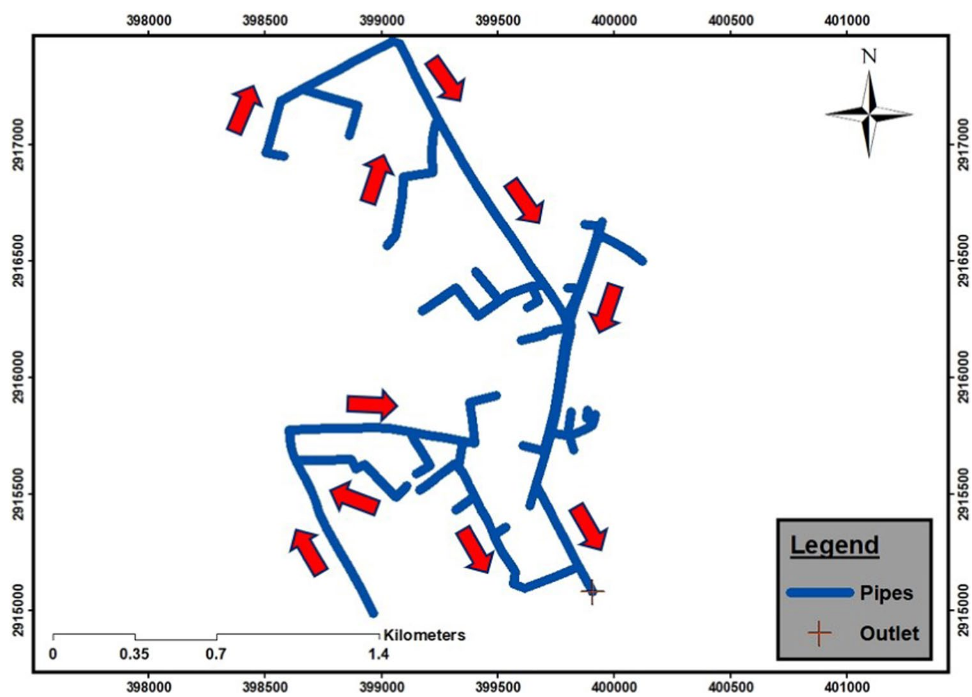


Fig. 22 General layout for the proposed storm network in Al Naq area (Case 4)—KSA

and compare the results with the final optimized design, as shown in Figs. 23, 24, 25, and 26. The detailed comparisons are presented in Tables 16, 17, 18, and 19 in Appendix A. Table 9 illustrates the summary of the comparison conducted between the four real cases.

The results from HBOA method for the four real cases showed that HBOA provided about 15% (on average) lower cost while consuming only few hours.

The main limitation of HBOA is that it can only be used if the drainage network is pipe-based, whereas it cannot be used if the network is box-based or open channel-based (any other section instead of pipes), where the code can be expanded in the future to include other cross-sections.

Table 6 Design constraints for the four real cases

Design constraints	Case (1)	Case (2)	Cases 3 &4
Minimum velocity (m/s)	0.30	0.60	0.30
Maximum velocity (m/s)	3.00	2.50	3.00
Pipe fullness ratio (d/d_{max})	1.00	1.00	1.00
Manning coefficient (n)	0.013	0.013	0.013
Minimum pipe slope	0.004	See Table 7	See Table 7
Minimum pipe cover (m)	1.00	1.60	1.00
Total flood volume (m^3)	0	0	0
Pipe commercial diameter (mm)	[300, 400, 500, 600, and 700]	[315, 400, 500, 630, and 700]	[500, 600, 700, 800, 900, 1000, 1100, 1200, 1300, 1400, 1500, 1600, 1700, 1800, 1900, 2000, and 2100]
Rainfall-distribution	SCS-Type II	SCS-Type II	SCS-Type II
Runoff method	EPA-SWMM	Unit hydrograph	EPA-SWMM
Loss method	SCS CN	SCS unit hydrograph	SCS CN
SCS CN	85		85
Design return period (year)	5	25	25
Maximum rainfall depth at the design return period (mm)	40.5	42.9	54

Table 7 The minimum allowable pipe slope for cases 2, 3, and 4

Case number	Diameter (mm)	Min. slope (%)	Diameter (mm)	Min. slope (%)
2	315	0. 24	700	0. 09
	400	0. 18	800	0.07
	500	0. 15	900	0.06
	630	0. 10	1000	0.05
3 &4	500	0. 12		
	600	0. 10		
	700	0.08		
	800	0.06		
	> 800	0.05		

Table 8 Total cost function's parameters K and Z

Case number	Parameter	
	K	Z
1	$K = \begin{cases} K_1 = WLY, & \text{if} Y \leq 2, \ (\text{Depth} = 0-2), \\ K_1 = 2WL, & \text{if} Y > 2, \ (Depth = 0-2) \\ K_2 = WLY - K_1, & \text{if} Y \leq 3, \ (\text{Depth} = 2-3) \\ K_2 = 3WL - K_1, & \text{if} Y > 3, \ \text{Depth} = 2-3 \end{cases}$ $W = 2D$	Zero
2	Zero	Zero
3 & 4	$K = (D + Y)YL$	$((D + Y)YL) - \left(\frac{\pi}{4}D^2\right)$

Summary and conclusions

A new optimization technique, HBOA (hydraulic-based optimization algorithm), is proposed and verified with two benchmark examples from the literature and provides the lowest total network cost with a cost reduction range of 0.1–1.5%. During the hydraulic evaluation of the HBOA model through the verification process using the two benchmarks' examples from the literature, some pipes in the original two networks in the literature were allowed to

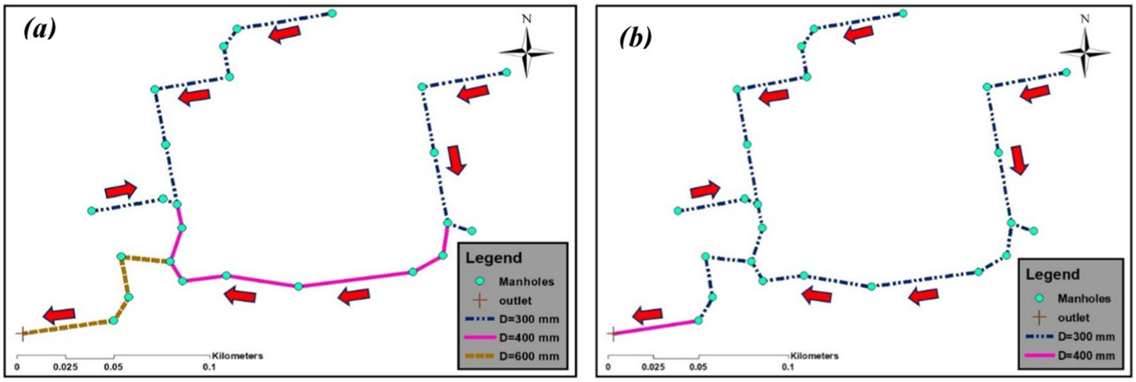


Fig. 23 The optimized pipe diameters for Case (1) **(a)** by the design engineer and **(b)** by the HBOA model

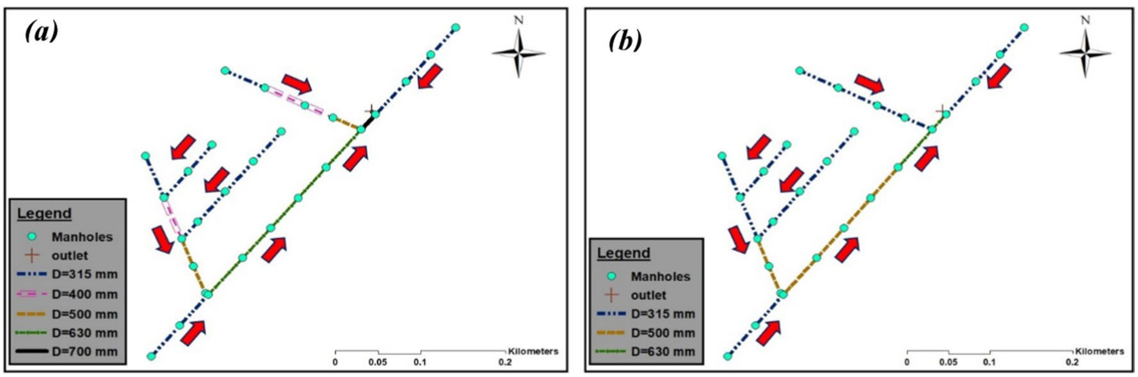


Fig. 24 The optimized pipe diameters for Case (2) **(a)** by the design engineer and **(b)** by the HBOA model

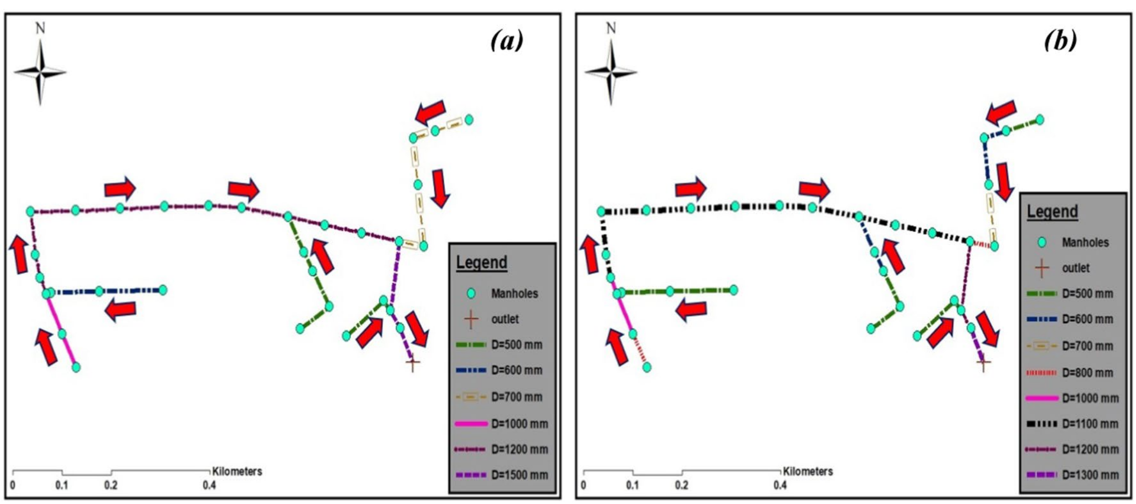


Fig. 25 The optimized pipe diameters for Case (3) **(a)** by the design engineer and **(b)** by the HBOA model

flood, which is against the hydraulic design requirement mentioned in the literature. The reason for the flooded network is that the methods used in the literature (GA-HP method) depend on designing each pipe in the network separately without studying the overall hydraulic performance of the whole network. In other words, Manning's formula was applied to each pipe individually, neglecting the effect of water surface profile (hydraulic gradient)

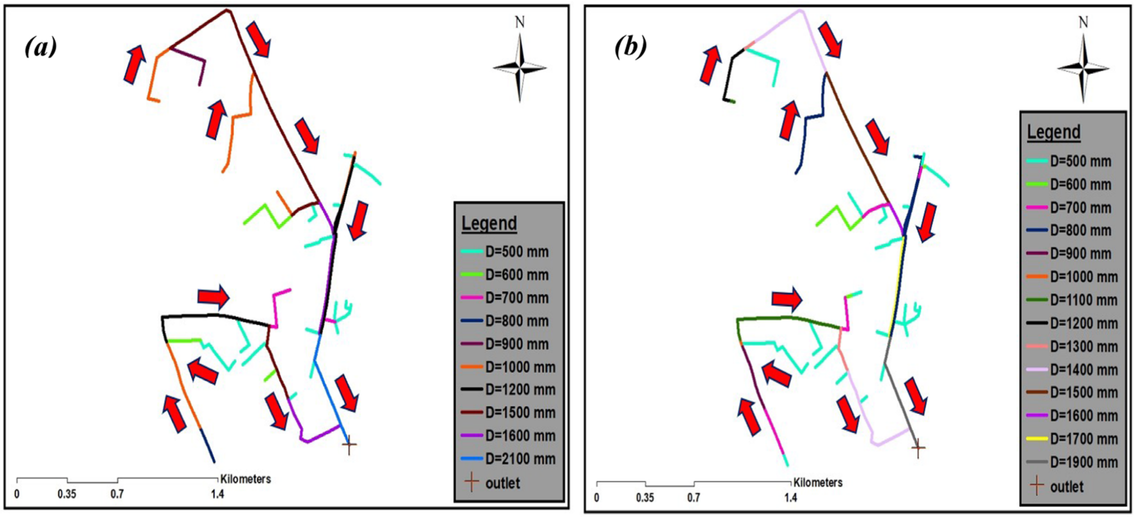


Fig. 26 The optimized pipe diameters for Case (4) **(a)** by the design engineer and **(b)** by the HBOA model

Table 9 Comparison between actual and HBOA results for the four real cases

Case no.	Project area	Project location	Total pipe length (m)	Optimized cost by		Cost reduction (%)
				The design engineer	HBOA model	
1	Dalkhoot	Oman	680	79,661 RO	66,202 RO	16.9
2	BASRA	Iraq	1190	381,881,015 IQD	342,650,000 IQD	10.3
3	Al Qassim	KSA	2165	3,987,222 SAR	3,355,823 SAR	15.8
4	Al Naq’		12,350	39,848,190 SAR	34,078,885 SAR	14.5

of the connected pipes on the flow characteristics of the designed pipe.

The HBOA is applied to four real storm networks from three different countries (representing different design constraints) that have already been designed, constructed, or under construction, and optimized by the design engineers. The results from HBOA for the four real cases are compared with the final optimized results. The results showed that the HBOA provided about 15% lower cost while consuming only few hours to reach the optimum design of each network.

Finally, the hydraulic-based optimization algorithm (HBOA) is a more robust and efficient tool that can be used by all infrastructure designers to achieve the optimal design of stormwater drainage networks in a dynamic process, efficient hydraulic performance, in addition to minimizing the consumed design time and total network cost.

Appendix

See Tables 10, 11, 12, 13, 14, 15, 16, 17, 18, and 19.

Table 10 Detailed results obtained using the HBOA model for the first benchmark example

Pipe	Invert elevation (m)		D (mm)	V (m/s)	d/d_{max}
	Upstream	Downstream			
1	71.89	70.96	250.0	0.80	0.67
2	67.95	66.98	300.0	0.89	0.82
3	70.35	68.69	200.0	0.77	0.82
4	70.96	69.40	250.0	0.80	0.73
5	69.40	68.49		0.81	0.76
6	68.49	67.15		0.91	0.71
7	67.15	65.44		0.85	0.82
8	65.44	64.53	300.0	0.73	0.70
9	66.98	66.50	350.0	0.72	0.75
10	66.50	65.60		0.89	0.64
11	65.60	64.48		0.85	0.67
12	64.43	63.37	400.0	0.90	0.80
13	63.32	62.88	450.0	0.73	0.82
14	62.88	62.42		0.75	0.82
15	68.69	67.40	250.0	0.76	0.67
16	67.40	65.90		0.83	0.69
17	65.90	64.10		0.82	0.74
18	64.05	63.35	300.0	0.65	0.82
19	63.30	62.62	350.0	0.58	0.74
20	62.42	61.34	450.0	1.19	0.82

Table 11 Detailed results obtained using the HBOA model for the second benchmark example

Pipe	Invert elevation (m)		D (mm)	V (m/s)	d/d_{max}
	Upstream	Downstream			
1	149.70	148.17	304.8	1.88	0.77
2	148.10	145.66	381.0	2.50	0.66
3	145.66	143.52	381.0	2.59	0.81
4	146.65	144.97	304.8	1.85	0.78
5	144.97	143.45	457.2	2.12	0.62
6	143.29	140.25	609.6	3.22	0.63
7	146.49	144.97	457.2	2.04	0.64
8	144.97	141.92	457.2	2.98	0.66
9	141.85	140.32	533.4	2.69	0.71
10	140.02	138.49	838.2	3.03	0.70
11	145.05	142.00	381.0	2.59	0.81
12	141.85	140.32	533.4	2.69	0.71
13	140.25	138.72	609.6	2.87	0.64
14	138.42	135.37	914.4	3.66	0.68
15	139.94	138.40	304.8	1.77	0.82
16	138.40	137.43	381.0	1.87	0.74
17	137.35	135.83	457.2	2.37	0.62
18	135.29	133.94	990.6	3.65	0.82
19	133.94	132.74	1066.8	3.22	0.82
20	132.74	131.12	1066.8	3.39	0.82

Table 12 Pipes, manholes, and earthwork unit prices for Dalkhoot area (case 1)

Pipe cost		Manhole cost		Excavation and backfilling	
Pipe diameter (mm)	Unit price (Rial Omani/m)	Manholes	Unit price (Rial Omani/no)	Cut/fill	Unit price (Rial Omani/m^3)
300	15	R.C. Manholes Chamber Ring Diam. 1200 mm (Type A)	2,000	(Depth 0 to 2 m)	13
400	24			(2 m < Depth ≤ 3 m)	15
600	86				

Table 13 Pipes unit prices for Basra area (case 2)

Pipe cost		Manhole cost	
Pipe diameter (mm)	Unit price (Dinar Iraqi/m)	Manholes	Unit price (Dinar Iraqi/no)
315	150,600	Manholes Diam. 1200 mm (Type A), $H < = 2.5$ m for $315 \le D \le 500$ mm	3,450,000
400	196,600	Manholes Diam. 1500 mm (Type B1), $H < = 2.5$ m for $630 \le D \le 900$ mm	3,910,000
630	271,350	Manholes Diam. 1500 mm (Type B2), 2.5 m $< H < = 6.0$ m for $315 \le D \le 900$ mm	4,600,000
700	357,600	Manholes Diam. 1200mm (Type A), $H < = 2.5$ m for $315 \le D \le 500$ mm	3,450,000

Where D is the pipe diameter (mm), and H is the depth of the manhole (m)

Table 14 Pipes unit prices for Al Qassim and Al Naq' areas (cases 3&4)

Pipe diameter (mm)	Unit price (SAR/m)
500	595
600	675
700	830
800	1050
900	1250
1000	1560
1100	1700
1200	1830
1300	1975
1400	2100
1500	2350

Table 15 Earthwork unit prices for Al Qassim and Al Naq' areas (cases 3&4)

Excavation and backfilling	Unit price (SAR/m^3)
Cut	20
Fill	15

Table 16 Comparison between original design and HBOA results for the Dalkhoot area (case 1)

Pipes			Manholes		
Diameter (mm)	Total length (m)		Depth to invert level (m)	Number of manholes	
	Original design	HBOA		Original design	HBOA
300	364	593	1.30–1.36	9	14
400	203	86	1.37–1.56	3	3
600	112	–	1.60–1.91	4	3
			1.92–2.17	5	4
			2.18–2.53	3	-

Table 17 Comparison between original design and HBOA results for the Basra area (case 2)

Pipes			Manholes		
Diameter (mm)	Total length (m)		Depth to invert level (m)	Number of manholes	
	Original design	HBOA		Original design	HBOA
315	622.0	802.8	1.91–1.92	–	6
400	144.6	–	1.93–2.36	5	5
500	114.5	290.0	2.37–2.72	5	6
630	274.8	94.6	2.73–3.25	9	8
700	31.5	–	3.26–3.60	8	2

Table 18 Comparison between original design and HBOA results for the Al Qassim area (case 3)

Pipes			Manholes		
Diameter (mm)	Total length (m)		Depth to invert level (m)	Number of manholes	
	Original design	HBOA		Original design	HBOA
500	350.9	557.9	1.50–1.80	4	10
600	236.8	223.3	1.81–2.34	3	10
700	344.8	100.3	2.35–2.62	1	3
800	–	111.7	2.63–3.35	12	4
1000	133.3	102.4	3.36–4.78	11	4
1100	–	860.5			
1200	890.3	146.8			
1300	–	60.9			
1500	207.7	–			

Table 19 Comparison between original design and HBOA results for the Al Naq area (case 4)

Pipes			Manholes		
Diameter (mm)	Total length (m)		Depth to invert level (m)	Number of manholes	
	Original design	HBOA		Original design	HBOA
500	2105	3207	1.50–2.18	35	56
600	783	518	2.19–3.40	47	44
700	425	782	3.41–5.03	57	40
800	214	2069	5.04–7.82	19	19
900	387	380	7.83–12.79	16	15
1000	2076	30			
1100	–	902			
1200	2065	372			
1300	–	348			
1400	–	1480			
1500	2335	842			
1600	1302	208			
1700	–	554			
1900	–	662			
2100	662	–			

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Declarations

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Ethical approval We declare herein that our paper is original and unpublished elsewhere, and that this manuscript complies to the Ethical Rules applicable for this journal.

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