Application of GA-HP model for the optimal design of large sewer networks

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Abstract. This research paper describes the application of a GA-HP model to the design of large gravity sewer networks. The main problem associated with the optimum design of sewer networks is achieving applicable designs that offer the lowest possible construction costs. The GA-HP model is a new technique that combines a Genetic Algorithm (GA) with Heuristic Programming (HP). One of the reasons for adopting this model is that determining the optimal design of sewer networks requires two stages: the GA can be used to obtain the optimal diameter, and then HP can be applied to obtain the optimal slope and other hydraulic characteristics. The GA was run using the tournament selection method with one-point crossover and a population size of 200. To ensure the efficiency of the GA-HP model for the design of large networks, the model was examined in two case studies, these being the medium and large networks located in the holy city of Karbala in Iraq, which contains 90 pipes and 91 manholes, and 354 pipes and 355 manholes, respectively. The cost for the applied manual designs was then compared with that for the designs obtained from this model for these networks, which indicated potential savings of 23.7% and 26.6% for the medium and large networks, respectively.

Keywords: GA-HP model; Optimisation models; Genetic Algorithm; Large sewer networks.

Introduction

Population growth in cities, which is generally accompanied by urban, industrial and commercial expansion requires additional thought about the best methods for removing used water, both from human
activity and from rain and flood-water. This must be carried out healthily and safely, with costs kept as low as possible [1]. Various sewage projects to achieve this have been promoted as vital projects, necessary to urban development, health, and cultural progress, making the study of this subject of tangible significance, particularly where it expands research in this area.

Sewer networks perform a particularly vital function in the winter season due to the increased need for wastewater management. Without effective sewerage, urban flooding, with its concomitant problems, could occur due to stormwater, leading to environmental damage, public inconvenience, economic disarray, and even threats to public safety and an increased risk of infectious disease. This makes it vital to maintain the reliable performance of storm sewer systems [2]. However, faced with tight budgets and stringent regulations, sewer engineers are confronted with significant challenges that require them to pursue new cost-effective strategies for the design, operation, and management of compliant sewers. This study focuses on the first step of this process, the cost-effective design of sewer networks.

The choice of sewer pipe layout in such a network is an important factor in the design, which in turn depends on several external factors such the topography of the area, which is the most important factor; the location of treatment plants for wastewater; and the location of natural drainage for rainwater, which generally defines the outlets of the network. An expert designer engineer can choose the layout of the pipes for such sewer networks relatively effectively in many cases; however, experience alone is not sufficient to ensure the optimal selection of the diameters and slopes of pipes in networks designed based on a specific layout. Moreover, previous design methods were forced to be simple and relies on hydraulic simplifications that often caused the resulting designs to be far from optimal, with excessive dimensions increasing costs or causing inefficiency, leading to bottlenecks and material damage in the city.

Many previous studies have examined the problem of the optimal design of sewerage networks using various optimisation methods, with researchers such as [3-5] using Linear Programming methods; [6 and 7] using Non-Linear Programming methods and [8-10] using dynamic programming methods. Recently, still other optimisation methods have been used to solve sewage networks optimisation problems, including cellular automata [11-14] and meta-heuristic techniques. In particular, meta-heuristic methods have been carefully improved and used extensively to address complicated hydraulic problems such as water networks. Meta-heuristic methods that have been identified as effective include Tabu Search [15-19] and genetic algorithms (GAs).

Genetic algorithms (GAs) are adaptive or constructed random search techniques that can be utilised to solve problems of optimisation by creating a population of individuals, each of which represents a potential answer to a particular issue. These are then refined using natural selection on a
“survival of the fittest” basis [20]. [21-26] have all applied GA in pipeline network systems for water distribution, and [27] presented a model for optimising pipe networks by employing the GA and compared the results with those from other optimisation techniques. They found that the GA technique was very effective in obtaining global optimal solutions with relatively few evaluations with respect to the domain of the space of the search.

[28] later used Genetic algorithms and Tabu Search (TS) techniques to optimise the design of wastewater collection networks. They developed a strategy of dynamic search, utilising an adaptive rule to assist the search procedures in finding better designs. They found that conventional designs result in deeper elevations at outlets as compared with both GA and TS designs. In addition, GA designs led to larger diameters compared with TS design for many pipes, causing TS to attain greater cost savings than GA. [29] evolved an AGA to find the optimum solution for a sewer network design including pipe diameters, slopes, and pump indicators. They focused on handling the non-linear and discrete constraints of the problem, and the proposed method systematically satisfied all the constraints of the sewer system, removing the need to discard or repair infeasible portions or to apply penalty factors to the cost function. They found that adding adaptive constraints as part of the computational handling method made the optimisation more efficient in terms of speed and reliability.

[30] used a GA connected with a uniform flow hydraulic module to determine finding the optimum design of drainage networks in rural areas. The study employed GA for the selection of channels’ geometric features, minimising the cost of construction, whilst a uniform flow phase flux function was employed to evaluate performance in the channels in hydraulic terms and with regard to satisfying the required level of constraints.

[31] developed a new model using both a Genetic Algorithm and Heuristic Programming, known as the GA-HP method, to obtained optimum designs for sewer networks. The submitted model was examined in conjunction with examples of sewer networks from the literature, with results suggesting that the GA-HP model was superior to all previous techniques. [32-34] then proposed a hybrid Genetic Algorithm (GA-TGA) method to compute the optimum layout for sewer networks. In the current study, the GA-HP method proposed by [31] is tested to ensure the efficiency of this model with regard to the design of relatively large networks based on its performance in two case studies in the holy city of Kerbala, Iraq.

1. Case studies
The two case studies used in this research are located in Karbala, Iraq. The centre of Karbala province is in the Middle Euphrates region, 105 km southwest of Baghdad, on the edge of the desert in the Western Euphrates and the left side of Al Husseiniya creek. More precisely, the city is located between longitudes 43° 15' 0" E and 44° 15' 0" E, and latitudes 32° 7' 30" N and 32° 46' 5" N, as shown in Figure 1. It is bordered to the north and west by Anbar province, to the south by Najaf province, and to the east and northeast by Babil province. The ground level in the city is about 34 to 42 meters above sea level [35 and 36]. The city of Karbala was chosen for this study for several reasons. Two of the most important were that the city's semi-flat slope of about 0.005 was likely to result in a challenge by limiting the slopes available to the optimisation program, and the fact that it a major destination for visitors due to the location of the shrine of Imam Hussein [37 and 38].

The selected sewer network has a limitation of a minimum ground depth cover close to 1.0 m, and a maximum and minimum velocity of 3.0 m/s and 0.6 m/s, respectively. Pipe sizes were selected from the set of available pipe diameters (200 mm, 250 mm, 315 mm, 400 mm, and 600 mm). The value of the Manning coefficient was taken to be 0.013 in all cases [39]. Figure 1 shows the locations of the two case studies used in this research. The first case study is the Shohdaa AlMudhafen district, while the second case study is the Al-Amil district.

Materials and Methodology

3.1 Formulation of the GA-HP Optimisation Problem

The objective function for a sewer networks problem with minimum construction costs (Z_{min}) is generally formulated as follows:

\[ Z_{min} = \sum_{i=1}^{n} f_i(d_i, Z_i, C_i) L_i \quad \ldots (1) \]

where \( Z_{min} \) is the total cost of the sewer network, \( d_i \) is the diameter of the pipe in the \( i \)th link, \( Z_i \) is the excavation depth (average) for the \( i \)th link, \( C_i \) is the unit cost of excavation and pipe for the \( i \)th link, and \( L_i \) is the pipe length for the \( i \)th link.

GAs are, essentially, unconstrained optimisation methods. When applied to determine the optimum solutions for constrained problems, such as sewer networks, these thus require constrained problems to be converted to unconstrained ones. A penalty technique is often utilised to achieve this, which involves including constraints within the problem objective function by identifying a cost or penalty in the following form (penalised objective function):

\[ C_{total} = \sum_{i=1}^{n} f_i(d_i, Z_i, C_i) L_i + \delta \cdot f(G) \quad \ldots (2) \]
where \( \delta \) is the penalty value, \( f \) is some function of the violation of the constraint matrix \( G \), and \( g(ij) \) is the \( j \)th violation of the constraint in pipe \( i \).

This objective function is subject to several constraints that may be formulated as

\[
\begin{align*}
D_i &\geq D_{i-1} \quad \ldots \ (3) \\
V_{\text{max}} &\geq V_i \geq V_{\text{min}} \quad \ldots \ (4) \\
G L_i - C L_i &\geq C_{\text{min.}} \quad \ldots \ (5) \\
G L_i - I L_i &\geq Z_{\text{max.}} \quad \ldots \ (6) \\
S_{\text{max.}} &\geq S_i \geq S_{\text{min.}} \quad \ldots \ (7)
\end{align*}
\]

where \( D_i \) is the pipe diameter \( i \); \( D_{i-1} \) is the pipe diameter \((i-1)\); \( V_i \) is the velocity of pipe \( i \); \( V_{\text{max.}} \) and \( V_{\text{min.}} \) are the maximum and minimum permissible peak flow velocities in the pipes, respectively; \( G L_i \) is the ground level upstream of link \( i \); \( C L_i \) is the crown elevation upstream of link \( i \); \( C_{\text{min.}} \) is the minimum allowable ground cover; \( I L_i \) is the invert elevation upstream of link \( i \); \( Z_{\text{max.}} \) is the maximum allowable depth of pipe; \( S_i \) is the slope of pipe \( i \); and \( S_{\text{max.}} \) and \( S_{\text{min.}} \) are the maximum and minimum allowable pipe slopes, respectively. More details for the formulation of the GA-HP optimisation sewer networks problem used in this study can also be found in [31].

**Cost functions**

In this research, Iraqi cost functions were used in the design process; these functions have not been used previously, and were thus determined by collecting information for the cost of buying and installing pipes, the cost of earthworks, and the cost of manhole construction in different depths and sizes for the area, as laid out in Tables 1 to 3, from the Department of Networks Design in the Directorate of the Karbala Sewers [40 - 42]. The cost functions were then derived from this information using a regression analysis method in IBM SPSS. The use of specific materials determines the price of a unit of length for a given diameter of pipe in commercial markets. However, the use of these pipes in sewer works makes them subject to cost functions that also incorporate excavation depth and country conditions as additional factors determining the construction price per unit length. As excavation depths and the prices of pipes may vary widely in the same network, it is thus difficult to obtain a single function that covers the movement of prices for all diameters and excavation depths with the required accuracy. This made it preferable to determine the cost functions in three segments for a range of different diameters, as follows:

1. \( 0.2 \text{ m} \leq D_i \leq 0.315 \text{ m} \) \& \( 1 \text{ m} \leq Z_i \leq 6 \text{ m} \)

\[
C_p = -5490 \, D_i^2 + 3182.6 \, D_i - 5.03 \, Z_i + 71.43 \, Z_i \, D_i - 378
\]
\ldots \ (8)

2. \( 0.4 \text{ m} \leq D_i \leq 0.9 \text{ m} \) \& \( 1 \text{ m} \leq Z_i \leq 8 \text{ m} \)

\[
C_p = 482.3 \, D_i^2 + 2.565 \, D_i + 14.7 \, Z_i + 19.06 \, Z_i \, D_i + 46.35
\]
\ldots \ (9)
3. $1 \leq D_i \leq 1.8 \text{ m} \quad \& \quad 4 \leq Z_i \leq 8 \text{ m}$

$$C_p = 1792.14 D_i^2 - 3639.2 D_i - 30.5 Z_i + 226.03 Z_i D_i + 2268.03 \quad \ldots (10)$$

where $C_p$ is the unit pipe installation cost ($1,000 \text{ ID/m}$) and $Z_i$ is the pipe excavation depth (average) in m.

**Figure 1**: Location of the two case studies
Table 1: Cost of dirt works for pipes (ID/meter length) [40].

| Excavation depth (m) | Commercial pipe diameters (mm) | Cost (ID/meter length) × 1000 |
|----------------------|--------------------------------|-------------------------------|
| 2-1                  | 200 – 250                      | 2-1                           |
| 3-2                  | 315 – 400                      | 3-2                           |
| 4-3                  | 400 – 500                      | 4-3                           |
| 5-4                  | 500 – 600                      | 5-4                           |
| 6-5                  | 600 – 700                      | 6-5                           |
| 7-6                  | 700 – 800                      | 7-6                           |

Table 2: Cost of buying pipes (ID/meter length) [40].

| Pipe diameters (mm) | Cost (1000 ID / m length) |
|--------------------|---------------------------|
| 200                | 12.5                      |
| 25                 | 20                        |
| 30                 | 30                        |
| 40                 | 65                        |
| 50                 | 90                        |
| 60                 | 14                        |
| 70                 | 16                        |
| 80                 | 21                        |
| 90                 | 27                        |
| 100                | 450                       |
| 110                | 500                       |
| 120                | 600                       |
| 130                | 660                       |
| 140                | 750                       |
| 150                | 800                       |
| 160                | 850                       |
| 170                | 100                       |
| 180                | 125                       |

Table 3: Cost of constructing manholes (ID/each manhole) [40].

| Type of manhole | Dimension (cm) | Depth (m) | Cost (1000 ID) |
|-----------------|----------------|-----------|----------------|
| AS              | 90 x 60        | 1 - 1.69  | 1000 – 1350    |
| BS              | Φ 110          | 1.7 - 2.99| 1500 – 2000    |
| BD              | Φ 110          | > 2.99    | 1750 – 2500    |
| CS              | Φ 150          | 1 – 3.24  | 2000 – 2450    |
| CD              | Φ 150          | > 3.24    | 2700 – 3000    |
| CD1             |                |           | 10,000 – 25,000|

A single equation is was, however, derived representing the construction cost of manholes for all depths and sizes:

\[ C_m = -24.3 Z_m^2 + 1411.7 Z_m + 22.9 DM - 6.74 Z_m DM - 1830.2 \]  

where \( C_m \) is the cost of manhole construction, \( Z_m \) is the manhole depth (m), and DM is the equivalent diameter of the manhole (83 mm for type AS, 110 mm for types BS and BD, and 150 mm for types CS and CD). The equations derived have high R-squared values, at 0.98, 0.99, 0.99 and 0.967, for Eqs. 8, 9, 10 and 11, respectively. These high values indicate that the equations are acceptably close to generating realistic values for these costs.
Sewer networks in the cases studies

The first case study is located in the city centre, close to the Shohdaa Al-Mudhafen quarter. It is located between latitudes 32° 36' 5" N and 32° 36' 24" N, and longitudes 43° 59' 19" E and 44° 0' 5" E, covering about 0.195 km², as shown in Figure 1. It includes 91 nodes (manholes) and 90 links (pipes), and the total length of the network is 3,605 km. The layout of the network is presented in Figure 2, and the total cost of the manual design for the network as-built was 529.7 million ID.

Figure 2: Layout of the first case study (Shohdaa Al-Mudhafen district) [40].

The second case study is located in Al-Hur subdistrict, in the third sector along from the Al-Amil quarter. It is located between latitudes 32° 37' 24" N and 32° 38' 2" N, and longitudes 43° 59' 2" E and 43° 59' 37" E, covering about 0.66 km², as displayed in Figure 1. The network of this case may be considered a large sewer network, including 355 nodes (manholes) and 354 links (pipes). The total length of the
network is 13,506 km, and the layout of the network is presented in Figure 3. The total cost of the manual design for the network as built equals 2,190 million ID.

2. Results and Discussion

In this study, the performance of the proposed GA-HP model was tested for both relatively small and large real networks with a flat slope as found in Karbala Governorate, Iraq, with the aim being to obtain the optimum design of these sewer networks and to compare these with the manual designs (as built) in terms of the total cost. The applied GA-HP model utilised the Tournament selection method and the one-point crossover technique to offer the best efficiency with regard to identifying the optimum design [31], and these criteria were thus used in this study. The existing sewer networks of cases study areas in selected residential quarters of Karbala city, as shown in figures 2 and 3, were thus redesigned optimally using the same criteria and constraints used in the manual designs. The same cost functions were then applied to both design types, and the final costings compared.

Figures 4 illustrates the optimum convergence curve for the optimum cost solution verses the generation number to demonstrate the progress of operations for the first sewer network of the case study, with a minimum diameter of 200 mm. This design was found with a TOS selection method, using a one-point crossover with a mutation of one gene per chromosome, a probability of crossover (Pc) = 1, a mutation probability (Pm) = 0.05, and a population size equal to 200 chromosomes. The total construction cost of the sewer network was thus lowered from 529.7 million ID in the manual design (as built) to 380.8 million ID in the optimum design with minimum diameter = 200 mm, a reduction of about 28.1%, as shown in table 4. This comparison includes all costs for excavations, pipes, and manholes.

The second case study was for a relatively large network including 355 manholes and 354 pipes, with a total length of network of 13,506 m. Figure 5 illustrates the convergence curve for the optimum cost solution for this case with generation numbers showing the evolution operation for the second sewer network. The total cost of the network was lowered from 2,190.6 million ID in the manual design (as built) to 1,567.43 million ID in the optimum design, resulting in a reduction of about 28.45%, as shown in table 4. The comparison included the costs of all excavations, pipes, and manholes.

It is important to note that the network layout of the case studies remained fixed and without change during the optimal design process, and that the number and locations of the manholes, as well as the lengths of pipes used, were kept as per the real situation. This means that all reductions in cost were the result of reducing the cost of excavations and pipe diameters. The proposed GA-HP model is thus expected to provide higher savings percentages if used to design main trunk lines as these are very
expensive, due to the need for them to contain high discharges and to extend for long distances. Table 5 illustrates a summary comparison between the amount of earth excavation and length of pipes with different diameters required for the real and optimum designs of sewer networks for both cases investigated.

**Conclusions**

In this study, the GA-HP model was applied to obtain optimum solutions (minimum cost) for two cases study sewer networks in the Karbala governorate, Iraq. The first case study, of a relatively small sewer network, included 91 manholes and 90 pipes, with a total length of a network of 3,605 km. The second case study was a relatively large sewer network, consisting of 355 manholes and 354 pipes, with a total length of network of 13,506 km. The layout in both cases was fixed as in the real situation, so that the optimisation model included only the cost of soil excavation and pipe diameters. The cost function used with both actual and optimum models was developed for this study based on the real prices of sewer networks in the study region as determined by a nonlinear regression model.

The optimal designs were determined using the TOS selection method with a one-point crossover, mutation of one-gene per chromosome, a probability of crossover (Pc) = 1, a mutation probability (Pm) = 0.05, and a population size equal to 200 chromosomes. The results indicated that the total construction cost of the first network was reduced 529.7 million ID in the manual design (as built) to 380.8 million ID in the optimum design, a reduction of 28.1%, while for the large sewer network in the second case, the total cost was reduced from 2,190.6 million ID in the manual design (as built)) to 1,567.43 million ID in the optimum design, a reduction very close to 28.45%. The high costs of sewer networks projects make such percentages of reduction very significant, and optimising sewer design is clearly important in the process of reducing the costs of related infrastructure projects.
Figure 3: Layout of the second case study (third sector from Al-Amil district) [43].
Figure 4: The iterations of optimum total cost solution by used the GA-HP model for the first case study.

Figure 5: The optimum cost solution of iterations by the proposed GA-HP model for the second case study.
Table 4: The percentage savings percentage for each case study

| Case study       | Type of design | Cost (million ID) | Saving percentage |
|------------------|----------------|-------------------|-------------------|
| First case study | Manual design  | 529.7             | 28.1 %            |
|                  | min. D = 200   | 380.8             |                   |
| Second case study| Manual design  | 2,190.67          | 28.45%            |
|                  | min. D = 200   | 1,567.43          |                   |

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