Design and assessment of water source alternatives for Mariout 2 water treatment plant extension

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ABSTRACT

Factors such as population increase and industrialization, coupled with the establishment of touristic villages, have necessitated an upgradation of water treatment plants (WTPs) in Egypt. In this study, three different water source alternatives were designed and compared with a simple decision matrix to select the most appropriate one for upgrading and extending the Mariout 2 WTP. The first two alternatives are located on the k-40 Alex-Cairo desert road and k-77 EL Nasr canal, respectively, where the water source is obtained from the Nile River. The third alternative is located at the k-51 Alex-Matrouh coastal road and a non-conventional seawater source is used. The design results showed that the required energy power of the first, second, and third alternatives were 0.31, 0.066, and 0.72 kw/purified m³, respectively. The operational costs of the first, second, and third alternatives were 0.665, 0.426, and 6.621 EGP/m³, respectively. The cost of the intake pipes was found to be the lowest for the third alternative, whereas it was found to be the highest for the first one. Based on the results obtained from the decision matrix, the third alternative was found to be the most appropriate alternative followed by the second one. This study may assist in making decisions regarding the water source selection and treatment methods for the extension of the fourth stage of the Mariout 2 WTP.

Key words: alternatives comparison, operational cost, seawater desalination, water sources, water treatment unit design

HIGHLIGHTS

- Designing the infrastructure of water treatment plants.
- Estimating the cost of intake pipes.
- Designing non-sustainable water treatment plants.
- Decision-making.
- Selecting the most appropriate water source based on the cost estimation.

1. INTRODUCTION

Increasing water requirements caused by population growth, urbanization, and economic development necessitates an upgradation of water treatment plants (WTPs). Such upgradation requires an approximate prediction of future populations using various methods (Aziz & Mustafa 2019). In addition, the design of each typical unit of the water treatment plant involves intake, coagulation, flocculation, sedimentation, filtration, disinfection, storage, and pumping (Warren Viessman & Hammer 1985; Hammer & Hammer 1996; Lomus et al. 2017; Aziz & Mustafa 2019). Some factors such as WTPs’ lifetime, maintenance, economic and political situation, technical problems, and water demand have a great impact on the design of WTP units. Furthermore, the availability of water sources and their quality impact WTP design (Emelko et al. 2011, Hou et al. 2020).

The main sources of water are conventional and non-conventional. Conventional water sources include surface water (such as rivers and lakes) and groundwater. Treating surface water to cover water demand is considered, in some cases, as a time-consuming process and a non-sustainable option (Liu et al. 2012) and may also require the acquisition of valuable land for the construction of treatment units. As far as groundwater is concerned, its availability is often limited with disadvantages such as poor quality and expensive treatment methods, and, thus, it is not sufficient to meet increased water demands (White et al. 2020).
2007). Consequently, shifting to non-conventional water sources is an urgent necessity. Non-conventional water sources that have been considered in water supply systems are rainwater (collected in separate wastewater networks), wastewater (Kalavrouziotis & Apostolopoulos 2007; Xu & Yuan 2021), and seawater (Nápoles-Rivera et al. 2013). Although potable water can be extracted or produced from wastewater (Law 2003), it may be unfit for human consumption and, therefore, may be commonly used for non-potable purposes (Aly et al. 2018). On the contrary, desalinated water is of potable quality and may be used for both potable and non-potable purposes (Liu et al. 2012). Previous studies have demonstrated that the costs of desalinated and reclaimed water mainly depend on plant capacity (Gikas & Tchobanoglous 2009; Gude 2016; Herrera-León et al. 2018). Moreover, designing desalinated seawater supply systems located at long distances and/or at altitudes very diverse from the coastline will require long pipelines and huge pumping stations (Herrera-León et al. 2018), which, in turn, will cause additional costs. Li et al. (2019) reported that consumers living far from water sources, for example, at distances ranging from 0.5 to 14.4 km, experience difficulties in obtaining water. Carravetta et al. (2016) reported that the cost of one cubic meter of treated desalinated water by reverse osmosis (RO) plants is still high and, thus, more efforts are required to reduce the energy consumption of the process without impacting the efficiency of the membranes. Hence, changing the water source from a conventional to a non-conventional one for an existing treatment plant with an existing distribution network may help reduce the total cost of cubic meter of purified water and at the same time meet the demand for freshwater.

As various alternatives can be used to upgrade the existing WTPs, a strategy to estimate the price of the produced water and compare different alternatives is essential. Cost items may include capital cost, which is related to the establishment and upgradation of a plant and the price of membrane modules, and the operating cost that is required for plant operations such as energy, chemicals, labor, and maintenance (Kumar et al. 2015; Abbasi et al. 2021). Various research studies have been conducted to model the cost of water production (Zhu et al. 2009; Park et al. 2010; Franceschini & Turina 2012; Cabrera et al. 2013; Ghaffour et al. 2013). However, calculating costs in approximate values may be more useful for evaluating alternatives and making decisions.

There are different ways and strategies to evaluate the alternatives related to water projects. In evaluating such strategies, all possible alternate solutions should be first considered and then the best among them that achieves the proposed targets selected (Latinopoulos et al. 1997). The decision-making process consists of several steps such as determining the problem, finding the relative factors, searching the potential alternatives, analyzing the alternatives, selecting the best one, and taking the final decision (Zhu & Fei 2019). The decision-making process can be initiated by several ways such as mathematical programming (Andreu et al. 1996; Deng et al. 2010; Li et al. 2015; Dong et al. 2016), fuzzy comprehensive evaluation (Gong & Jin 2009; Maurya & Singh 2021), multiple criteria decision-making (Eriyeti et al. 2019; Chiu et al. 2020; Herndon et al. 2020), and weighted decision matrix (Eriyeti et al. 2019) The weighted decision matrix is a simple way that can provide quick assistance to identify the best alternative (Chiu et al. 2020). All criteria that influence a decision should be considered (Vrekos et al. 2019), and these criteria may include economic, environmental, socio-political, and technical (Eriyeti et al. 2019; Vrekos et al. 2019; Chiu et al. 2020; Khan et al. 2021).

Most previous studies were concerned only with the designing required for water treatment units (Aziz & Mustafa 2019) and reclaimed wastewater units (Arif et al. 2018) and/or estimating the required costs (Eli Oklejas & Nielsen 1995; Kumar et al. 2015; Al-lami et al. 2019) without taking into account the water sources. The aim of this study is to bridge this gap by selecting three alternative water sources to upgrade the Mariout 2 WTP. The first two alternatives are conventional water sources in the form of surface water from the Nile River and located at a distance from the distribution network. The third alternative is a non-conventional water source represented by the Mediterranean Sea. The study includes designing the treatment units of the three alternatives and estimating the cost of intake pipes and operating costs required to produce 1 m³ of treated water. Finally, a simple decision matrix is designed to compare and evaluate the three alternatives to select the most appropriate one.

2. MATERIALS AND METHODS

2.1. Case study description

The Mariout 2 WTP, also called the K-40 WTP, lies on the K-40 Alex-Cairo desert road (Egypt) and it was constructed to feed the south and the west of Alexandria and some regions of Matrouh with drinking water since 1988 with a total design capacity of 566,000 m³/d. Based on data obtained from Central Agency for Public Mobilization and Statistics (CAPMAS)
at 2006 and 2017, the Mariout 2 WTP serves 11% of the Alexandria government population and 16.5% of the Matrouh government population. As shown in Figure 1, the Mariout 2 WTP consists of three stages. The first stage, known as the Degermon stage, was constructed in 1988 with a capacity of 86,000 m³/d. The second stage, known as the Etabla stage, was constructed in 1995 with a capacity of 240,000 m³/d. The third stage, known as the Hoarse stage, was constructed in 2007 with a capacity of 240,000 m³/d. The components and treatment units’ dimensions of each stage are provided in Table 1. Also, the design criteria of each treatment unit are provided in Table 1. These values (treatment units’ dimensions and design criteria of each treatment unit) are used to estimate the current design and maximum flow.

2.2. Experimental plans

Figure 2 illustrates the conceptual framework of the current study’s experimental plan that consists of three main phases. The initial phase is the design of elements included in each alternative and it consists of three steps. First, estimate the recent maximum and design flow rate based on the existing treatment units. Second, estimate the future population and future flow rate of the target year (2047) and hence determine the required design flow rate that is required for the fourth extension of the treatment plant. The population forecast was calculated using data obtained from the Central Agency for Public Mobilization and Statistics for the year 2017 and it was carried out according to the standard methods (arithmetic and geometric methods as shown in Equations (1) and (2)) mentioned in the Egyptian Code of Design Principles and Conditions of Implementation for Drinking Water and Drainage Plants (Al-Hattab et al. 2016b). Third, design the intakes and treatment units of the three water source alternatives based on the determined flow rate. The second phase of this study is cost estimation that includes intake pipes (intake pipes) and its accessories, the required energy to operate the treatment plant, and the value of the operating cost. The third phase of this study is designing a decision matrix to evaluate and compare the three alternatives and select the most appropriate one.

Geometric method

\[ P_n = [P_o (1 + r)]^n \]  

Arithmetic method

\[ P_n = P_o + A \times n \]  

Figure 1 | Description of the stages of the Mariout 2 water treatment plant.
where $P_n$ is the target year population (capita) \((P2047)\); \(P_o\) is the last population (capita) \((P2017)\); \(r\) is the geometrical increase rate; \(n\) is the number of years to the target year; \(A\) is the arithmetic increase rate.

2.3. Design requirement and criteria

The current design and maximum flow rate of the Mariout 2 WTP (at 2018) was calculated based on the dimensions and design criteria of treatment units (shown in Table 1). Based on the determined future population, the future flow rate required to design the fourth stage of the Mariout 2 WTP was estimated at 2047. The WTP alternatives were designed according to the Egyptian Code Design Principles and Condition of Implementation for Drinking Water and Drainage Purification Plants (Al-Hattab et al. 2016). The seawater treatment plant was designed according to Voutchkov (2017), as shown in Table 2, and 8-inch Filmtec membranes were suggested for use.

2.4. Cost analysis program

The operation costs of the different alternatives, such as fees, alum, chlorine, and tools, were determined according to the statistical standards of the Alexandria Water Company (AWCO). The cost of electricity can be determined by computing the total power of the required pumps for each alternative and multiplying it with the price of kW of electricity \((0.9\) Egyptian pounds (EGP))\). The total power of the required pumps includes the total power of low-lift and high-lift pumps that can be determined according to the Egyptian Code Design Principles and Condition of Implementation for Drinking Water and Drainage Purification Plants (Al-Hattab et al. 2016). The electricity cost of 1 m$^3$ of purified water can be estimated by multiplying the power required for each cubic meter (total power divided by the total flow rate) with the price of kW of electricity \((0.9\) EGP). The same method can be used to determine the cost of the chemicals (e.g. the cost of alum required to purify one cubic meter = amount of alum required × price of ton). The amount of alum required per cubic meter can be estimated based on the required dosage of alum. The Lewaplus2 RO program was used for RO membrane calculation. For intake
pipe cost calculation, a website (www.concastpipe.com) was referred to know the type of pipe material used, its diameter, and its length.

### 2.5. Design of decision matrix

The evaluation of the three proposed alternatives was conducted using a simple decision matrix. The values of operation cost, intake pipe cost, total dissolved solids (TDS), total suspended solids (TSS), and land requirement were the factors considered in the simple decision matrix. The weights of each considered factor were justified, and these weights were estimated based on the preferences of the decision maker (Vrekos et al. 2019). Given the weight of the different parameters, the weighted score of each alternative was determined by multiplying the importance degree (weight) of a specific criterion (Nia et al. 2019), which was derived as shown in the following equation.

\[
\text{Score (weighted percentage)} = W_i \times R_{ij} = W_i \frac{f_{ij} \min}{T_{ij}}
\]  

### Table 1 | Unit description of the three stages of the Mariout 2 WTP

| Units               | First stage (Degermon stage)                          | Second stage (Etabla stage)                             | Third stage (Hoarse stage)                                 | Design criteria          |
|---------------------|------------------------------------------------------|-------------------------------------------------------|----------------------------------------------------------|--------------------------|
| Intake pipes        | 2 pipes × 1,000 mm + 2 pipes × 1,500 mm              | 14 pumps Q = 2,000 m³/h, h = 14 m                      | 4 pumps Q = 2,000 m³/h, h = 16 m                         | Velocity (1–1.5) m/s     |
| Low-lift pumps      | 4 pumps discharge (Q) = 990 m³/h, head (h) = 12.9 m, 2 pumps Q = 2,000 m³/h, head = 14 m | 16 m, 8 pumps Q = 2,400 m³/h, h = 20 m                 |                                                          |                          |
| Flash mixing        | One circular tank Diameter = 4 m, depth = 6 m         | One rectangular tank Length = 4.5 m, m, depth = 5 m    | One rectangular tank Length = 4.5 m, m, depth = 5 m     | Denition time (DT) minimum = 20 s |
| Gentle mixing       | Four circular tanks Diameter = 11 m, depth = 6 m     | Four rectangular tanks length = 12.5 m, m, depth = 5 m | Four rectangular tanks length = 12.5 m, m, depth = 5 m  | Denition time (DT) minimum = 20 min |
| Sedimentation basins | Four circular tanks Diameter = 30 m, depth = 6.5 m   | Four rectangular tanks length = 70 m, m, depth = 5.5 m | Four rectangular tanks length = 74 m, m, depth = 5.5 m | DT minimum = 2 h, Overflow rate (OFR) maximum = 40 m³/m²/d |
| Filters             | 12 filters length = 10 m, width = 8 m                | 12 filters length = 17 m, width = 6 m                 | 24 filter length = 9, width = 4 m                       | Rate of filtration (ROF) maximum = 180 m³/m²/d |

### Table 2 | Design guidelines for an 8-inch Filmtec membrane (Voutchkov 2017)

| Items                        | Membrane characteristics |
|------------------------------|--------------------------|
| SDI (Silt Density Index)     | <1                       |
| Average flux (L/m² h)        | 36–43                    |
| Maximum element recovery %   | 30                       |
| Active area (ft²)            | 400                      |
| Maximum permeate flow rate (m³/d) | 42                   |
| Minimum concentrate flow rate (m³/d) | 2.3                |
where \( W_i \) is the weight of any alternative \((i)\); \( R_{ij} \) is the weighted ratio of factor \((j)\) for an alternative \((i)\); \( f_{\text{min}} \) is the minimum value of factor \((j)\); and \( f_{ij} \) is the factor \((j)\) for a specific alternative \((i)\).

### 3. RESULTS AND DISCUSSION

#### 3.1. Future required flow rate

To design various water source alternatives that are available to expand the Mariout 2 WTP, it is important to predict the future required flow rate. Hence, the value of the current flow rate and served population by the WTP was estimated.

The design value of the current flow rate of the Mariout 2 WTP was determined by computing the maximum discharge of all existing units (intake, low-lift pump, flash mixing, coagulation, sedimentation, filters, reservoir, and high-lift pump) and taking the minimum discharge (Q design). Table 3 presents the value of the maximum discharge of each unit using the unit dimensions and criteria provided in Table 1 according to the Egyptian Code Design Principles and Condition of Implementation for Drinking Water and Drainage Purification Plants (Al-Hattab et al. 2016). For example, the maximum discharge of the existing intake pipes (2 pipes \( \times \) 1,000 mm + 2 pipes \( \times \) 1,500 mm) equals 661,284 m\(^3\)/d, which was determined by multiplying the maximum velocity (1.5 m/s) and the pipes’ area (\( A = 2 (\pi \times 0.5^2 + \pi \times 0.75^2) \)). We followed the same procedure to determine the maximum discharge of all existing units as shown in Table 3. The minimum value of the estimated discharges in Table 3 is supposed to be the design discharge of the WTP and it equals 470,160 m\(^3\)/d (approximately 470,000 m\(^3\)/d).

The total required flow rate per day in 2047 equals 684,965 m\(^3\) and it was estimated based on population forecasting (data are not given). Hence, the value of the future daily flow rate required for the fourth extension of the Mariout 2 WTP equals 214,965 m\(^3\) (flow rate required = total flow rate at 2047 (684,965) – current capacity of WTP (470,000)). According to the Egyptian Code (Al-Hattab et al. 2016), an additional value of water demand should be added to the total calculated value to meet firefighting demands. So, the total future flow rate required to design the fourth stage of the Mariout 2 WTP is 237,397 m\(^3\)/d.

#### 3.2. Preliminary design of the Mariout 2 WTP fourth stage

For extending the Mariout 2 WTP, a new source of water should be made available. Three alternatives of water sources have been suggested in this study for the WTP extension as shown in Figure 3. The first alternative suggested is located at the k-40 Alex-Cairo desert road (the same place as the previous three stages) where the water intake is from the Mariout canal branch from the Noubria canal, and it is located at a distance of around 28 km from the New Burg el Arab reservoir (distribution area). The second alternative suggested lies on k-77 on the EL Nasr canal, which is a branch of the Noubria canal, and it is around 17 km from the New Burg el Arab booster (it is the nearest raw water conventional source). The third alternative lies on the k-51 Alex – Matrouh coastal road where the water intake is from the Mediterranean Sea, and it is only 3 km away from the Burg el Arab coastal booster. The reasons for choosing the third alternative are as follows: first, the long distance and high pollution of the conventional water sources closest to the treatment plant, and second, the intake will be close to the distribution system, which, in turn, will reduce the electrical energy consumption required to deliver water. Table 4 provides a summary design of the intake pipe and treatment units required for the fourth stage of the Mariout 2 WTP for the first and second alternatives. The design steps and methods followed those of the previously published studies (Hammer & Hammer 1996; Al-Hattab et al. 2016; Aziz & Mustafa 2019).

#### Table 3 | Maximum discharge of the treatment units of the Mariout 2 WTP

| Unit         | \( Q_{\text{maximum}} \) (m\(^3\)/d) |
|--------------|--------------------------------------|
| Intake       | 661,284                              |
| Low-lift pump| 954,720                              |
| Flash mixing | 1,491,955.2                           |
| Coagulation  | 767,134                              |
| Sedimentation| 827,824.6                             |
| Filters      | 470,160                              |
As mentioned above, the water source of the third alternative is the non-conventional seawater source. So, the treatment processes for this alternative are different than those used for the first and second alternatives where the water source is conventional. The main seawater treatment process is RO membrane, which should undergo pretreatment to reduce membrane fouling and extend its lifetime (Aly et al. 2018). In this study, the pretreatment methods that were suggested were the pressure filter method, followed by a cartridge filter method, and the RO unit suggested was an 8-inch Filmtec membrane (Samir El-

Table 4 | Summary design of intake and treatment units for the fourth stage of the Mariout 2 WTP for the first and second alternatives

| Units                  | First alternative                        | Second alternative                        | Design criteria                          |
|------------------------|------------------------------------------|-------------------------------------------|------------------------------------------|
| Intake pipes           | 2 pipes × 1,500 mm                       | 2 pipes × 1,500 mm                       | Velocity (1–1.5) m/s                     |
| Low-lift pumps         | 11 pump (132 KW) discharge (Q) = 2,400 m³/h, head (H) = 12.0 m | 7 pump (132 KW) discharge (Q) = 2,120 m³/h, head (H) = 14.0 m |                                            |
| Flash mixing           | One circular tank                        | One circular tank                        | DT minimum = 20 s                        |
|                        | Diameter = 4.9 m                         | Diameter = 4.9 m                         | DT minimum = 20 min                      |
|                        | Depth = 5.7 m                            | Depth = 5.7 m                            | DT minimum = 20 min                      |
| Gentle mixing          | 12 circular tanks (diameter = 18.5 m, depth = 2 m) | 12 circular tanks (diameter = 18.5 m, depth = 2 m) |                                            |
| Sedimentation basins   | 12 circular tanks (diameter = 39 m, depth = 3 m) | 12 circular tanks (diameter = 39 m, depth = 3 m) | DT minimum = 2 h, OFR maximum = 40 m³/m²/d |
| Filters                | 42 filters length = 8.5 m, 8 m           | 42 filters length = 8.5 m, 8 m           | ROF maximum = 180 m³/m²/d                |
| Reservoir (storage tanks) | 8 reservoirs length = 49 m, m depth = 5 m | 8 reservoirs length = 49 m, m depth = 5 m |                                            |
| High-lift pumps        | 14 pump (533 KW), discharge (Q) = 1,978.3 m³/h, head (H) = 54 m | No need for using high-lift pump due to the lower level of the storage tank |                                            |
| Intake pipes           | One pipe 2,000 mm with 28 km length      | One pipe 2,000 mm with 17 km length      | Notes:                                   |

- First alternatives (12 air valves, 12 Butterfly valves, 12 non-return valves, and 12 washing valves are used.)
- Second alternatives (7 air valves, 7 butterfly valves, 7 non-return valves, and 7 washing valves are used.)

Figure 3 | Location of the three suggested WTP alternatives.
The value of the design flow rate was also different and equaled 791,323.3 m$^3$/d (assuming SDI = 1 with a 30% recovery rate). The design steps of the membrane unit and pretreatment were in accordance with those of Voutchkov (1968). Table 5 presents a summary design of the intake and treatment units required for the fourth stage extension of the Mariout 2 WTP.

### Table 5 | Summary design of the intake and treatment units for the fourth stage of the Mariout 2 WTP for the third alternative

| Units                         | Third alternative                                                                 | Design criteria                                                                 |
|-------------------------------|----------------------------------------------------------------------------------|---------------------------------------------------------------------------------|
| Intake pipes                  | 4 pipes 2,000 mm                                                                 | Velocity (1–1.5) m/s                                                            |
| Low-lift pumps                | 32 pumps (549 KW)                                                                | Discharge ($Q$) = 2,355 m$^3$/h, head ($h$) = 12.0 m                          |
| Pressured filters             | 327 filters                                                                      | ROF (170–480) m$^3$/m$^2$/d                                                   |
| Cartridge filters             | 10,304 cartridge filters                                                         | Diameter (0.5–3.6) m, Height = (1–7.5) m                                       |
| Membrane                      | 2,849 vessels with six membrane modules in each vessel                           | Flux = (13–18) l/h/m$^2$                                                      |
| Reservoir (storage tanks)     | Eight reservoirs                                                                | Feed vessel 8-inch = (8–14) m$^3$/h and maximum = 17 m$^3$/h                  |
| High-lift pump                | 14 pump (533 KW) discharge                                                       | Concentrate water ≥3 m$^3$/h                                                   |
| Intake pipes                  | One pipe 2,000 mm with 3 km length                                               | Maximum operating pressure = 1,200 psi = 83 bars                              |
|                              |                                                                                 | TDS for the Mediterranean Sea = 38,600 ppm$^a$                                 |
|                              |                                                                                 | Osmotic pressure feed = 395.2 psi                                              |
|                              |                                                                                 | Recovery = 30%                                                                 |
| Notes:                       |                                                                                 | Two air valves used (maximum distance between valves 2,500 m and ball valve $D$ = 250 mm) |
|                              |                                                                                 | Two butterfly valves (maximum distance between valves 2.5 km)                  |
|                              |                                                                                 | Two washing valves (maximum distance between valves 2.5 km)                    |

$^a$According to Bashitialshaaer et al. (2011).

Manharawy 2001; Voutchkov 2017). The value of the design flow rate was also different and equaled 791,323.3 m$^3$/d (assuming SDI = 1 with a 30% recovery rate). The design steps of the membrane unit and pretreatment were in accordance with those of Voutchkov (1968). Table 5 presents a summary design of the intake and treatment units required for the fourth stage extension of the Mariout 2 WTP.

### Table 6 | Operating cost estimation of the three alternatives of water sources for the fourth extension of the Mariout 2 WTP

| Item                                          | Value (EGP/m$^3$) |
|-----------------------------------------------|-------------------|
| First alternative                             | Second alternative| Third alternative|
| Electricity                                   | 0.298             | 0.059             | 0.65           |
| Alum                                          | 0.029             | 0.029             | 0.029          |
| Chlorine                                      | 0.014             | 0.014             | 0.014          |
| Fees                                          | 0.102             | 0.102             | 0.102          |
| Others (tools, transport, maintenance, benzene, etc.) | 0.22              | 0.22              | 0.22           |
| Operating cost of membrane unit$^a$           | –                 | –                 | 5.604          |
| Total                                         | 0.665             | 0.426             | 6.621          |

$^a$Detailed calculations are given in Table 7.
3.3. Cost estimation and assessment

The estimated cost includes the cost of electricity and chemicals used (alum, chlorine, other), and for the purposes of this study, the estimated cost is the operating cost, besides the intake pipe cost. The cost of electricity can be determined by computing the total power of the required pumps and multiplying it with the price of kW of electricity (0.9 EGP). The electricity cost of 1 m³ purified water can be estimated by multiplying the power required for each cubic meter (total power divided by total flow rate) with the price of kW of electricity (0.9 EGP). The same method can be used to determine the cost of the chemicals (e.g. the cost of alum = amount of alum required × price of ton). Table 6 provides a cost estimation of the electricity and chemicals (operating costs) used for the three alternatives in accordance with the standards of AWCO. It can be noticed from the data presented in Table 6 that the costs of the chemicals required and other items (fees, tools, transport, maintenance, and benzene) are the same in the three alternatives where these costs do not depend on the treatment methods. Also, it can be noticed that the cost of a cubic meter of purified water in the third alternative (6.621 EGP) is higher than that of the estimated costs of the first and second alternatives (0.665 and 0.426 EGP, respectively). This difference in cost results from the higher power required for the third alternative besides the operating cost (detailed values of the operating costs for the third alternative are shown in Table 7) of membrane units and membrane modules (Eli Oklejas & Nielsen 1995; Zhu et al. 2009). The lowest estimated operating cost is obtained from the second alternative where no high-lift pumps are required.

### Table 7 | Operating cost variables of the membrane system (third alternative)\(^a\)

| Item                                      | Total cost (LE)/m³ | %   |
|-------------------------------------------|--------------------|-----|
| Power                                     | 2.474              | 44.1|
| Cartridge replacement                     | 0.116              | 2.1 |
| Membrane replacement                      | 0.781              | 13.9|
| Pretreatment chemical                     | 0.256              | 4.6 |
| Membrane cleanings                        | 0.062              | 1   |
| Maintenance and parts                     | 0.561              | 10  |
| Labor                                     | 1.087              | 19.4|
| Contingency                               | 0.264              | 4.8 |
| Total                                     | 5.604 LE/m³        |     |

\(^a\)Costs have been estimated based on the design of the third alternative.

### Table 8 | Detailed estimation of intake pipes and their fitting for the three alternatives of water sources

| Item                  | Intake pipes | Price EGP | Total KEGP | Valves                                      | Total EGP | Total cost KEGP |
|-----------------------|--------------|-----------|------------|---------------------------------------------|-----------|-----------------|
|                       | L (km)       | Material  |            | Type                          | N | D (mm)        |          |                |              |
| First alternative     | 28           | Reinforced concrete | 31,325 | 877,100 | Air valve                          | 12 | 250          | 9,500      | 114,000        | 54,000     | 877,904        |
|                       |              |           |            | Wash valve                      | 12 | 300          | 4,500      |                     |            |                |
|                       |              |           |            | Elbow                          | 4  | 2,000        | 159,131    | 636,524        |            |                |
| Second alternative    | 17           | Reinforced concrete | 31,325 | 532,525 | Air valve                          | 7  | 250          | 9,500      | 66,500         | 31,500     | 533,259        |
|                       |              |           |            | Wash valve                      | 7  | 300          | 4,500      |                     |            |                |
|                       |              |           |            | Elbow                          | 4  | 2,000        | 159,131    | 636,524        |            |                |
| Third alternative     | 3            | Reinforced concrete | 31,325 | 93,975  | Air valve                          | 2  | 250          | 9,500      | 19,000         | 9,000      | 94,639         |
|                       |              |           |            | Wash valve                      | 2  | 300          | 4,500      |                     |            |                |
|                       |              |           |            | Elbow                          | 4  | 2,000        | 159,131    | 636,524        |            |                |
The cost of intake pipes can be determined based on the type of pipe materials and the price of its fittings (e.g. valves). Table 8 shows the detailed estimation of costs required for the intake pipes and valves of the three alternatives. As shown in the table, the estimated cost is mainly dependent on the pipe length and number and type of valves used. Figure 4 presents the cost required of intake pipes, valves, and the total cost of the three alternatives (the values of intake pipes and total cost were divided by 1,000 EGP to clarify the value of valve costs). It can be noticed from Figure 4 that the cost of the third alternative intake pipes equals 93,975 million EGP (KEGP), which is lower than the intake pipes' cost of the first and second alternatives (877,100 and 532,525 KEGP, respectively). This is due to the greater length of the intake pipes for the first and second alternatives (28 and 17 km) compared with the length of the third alternative intake pipe (3 km). It is important to mention that the total price is highly impacted by the price of the intake pipes since the price of the valves is almost the same for the three alternatives.

### 3.4. Present worth

Table 9 presents the summary of the cost analysis for the three alternatives. To estimate the annualized project cost, the project cost was multiplied by the capital recovery factor, which is a factor used to convert the present project cost to a series of equal payments at an interest rate (assume $i = 6\%$) over the planning period ($n = 30$ years (study period from 2017 to 2027)) as shown in Equation (4) (Arif et al. 2020). The total operation and maintenance costs per year have been estimated by multiplying total operation and maintenance costs for cubic meter and flow rate per year ($237,397 \div 365 \text{ m}^3$). Based on the data presented in Table 9, it is found that the second alternative is the most cost-effective one, while the third alternative is the highest cost alternative.

\[
\text{CRF} = \frac{i (1 + i)^n}{((1 + i)^n - 1)} = \frac{0.06 (1 + 0.06)^{30}}{(1 + 0.06)^{30} - 1} = 0.0619
\]

### Table 9 | Present cost summary of the three alternatives

| Cost item                                      | Alternative 1 | Alternative 2 | Alternative 3 | Unit  |
|-----------------------------------------------|---------------|---------------|---------------|-------|
| Total intake pipes and their fitting cost     | 877,904       | 533,259       | 94,639        | KEGP  |
| Total operation and maintenance costs         | 0.665         | 0.426         | 6.621         | EGP/m³ |
| Total operation and maintenance costs per year| 57,622        | 36,912        | 573,709       | KEGP/year |
| Annualized project cost\(^a\)                 | 54,342        | 33,008        | 5,858         | KEGP/year |
| Annualized project cost + annual operation and maintenance (O & M) cost | 111,964 | 69,920 | 579,567 | KEGP/year |
| Cost (m³)\(^b\)                               | 1.29          | 0.81          | 6.67          | EGP/m³ |

\(^a\)Annualized project cost = Total intake pipes and its fitting cost × CRF for \(i = 6\%\), planning period = 30 years, CRF = 0.0619.

\(^b\)Cost (m³) = \frac{\text{annualized project cost } + \text{ annual O & M costs}}{\text{average design flow } \times 365 \text{ m}^3/\text{year}}.
3.5. Design of decision matrix

To select the most appropriate alternative from the three suggested alternatives, a simple decision matrix has been designed. The considered factors in the decision matrix are those related to the cost and the effluent quality. Table 10 shows the values of different factors in the decision matrix and the suggested weight percentage (Eriyeti et al. 2019; Vrekos et al. 2019; Zhu & Fei 2019). The values of the operation cost and intake pipe costs are those determined in the previous step. For determining the effluent quality factor, the only parameters that are considered are TDS and TSS due to the lack of sample analyses. In Table 10, the land required to construct each alternative is also one of the factors or criteria included in the design matrix. The required land was estimated based on the treatment units’ dimensions, and an additional value was added as the space between the units (Kumar et al. 2015). Although the number and dimensions of the treatment units for the first and second alternatives are identical, the required construction areas are quite dissimilar. The reason for this is the number of low-lift and high-lift pumps is unequal as shown in Table 4. As expected, the lowest construction area is required for the third alternative because the membrane process requires a lower footprint (Herrera-León et al. 2018). The percentage of factor weight was suggested based on its significance for decision-makers (e.g. AWCO authorities). For example, the percentage weight of the required area for construction is 15% of the area available (AWCO will not pay for it). On the contrary, the percentage weight of the operating cost is the highest as it is the most significant factor (AWCO pays heavy charges every year) in decision matrix. So, the alternative with the lowest operating cost is considered to have the highest percentage.

To determine the weight percentage of the considered factors (score) for each alternative, it has been assumed that the alternative with the highest impact gets the highest score (Eriyeti et al. 2019; Salisbury et al. 2019; Vrekos et al. 2019). Then, the two other alternatives get a ratio of the weight percentage (Eriyeti et al. 2019; Salisbury et al. 2019; Vrekos et al. 2019). For example, the lowest operating cost is achieved from the second alternative. So, the second alternative will receive the highest weight of the operating cost factor (40%) and the first alternative will receive just a ratio of the weight (the value of the lowest operating cost (0.42) divided by the value of the first alternative operating cost (0.66) multiplied with 40% = 25.5%). The same method can be followed to determine the score of the considered factors for each alternative.

### Table 10 | Criteria considered in the decision matrix and its suggested weight

| Criteria | Suggested weight (%) | Value of each alternative |
|----------|----------------------|---------------------------|
|          |                      | First alternative | Second alternative | Third alternative |
| Operating cost (EGP) | 40 | 0.66 | 0.42 | 6.621 |
| Intake pipe cost (KEGP) | 20 | 877,904 | 533,259 | 94,639 |
| Effluent quality TDS (PPM) | 13 | 450 | 400 | 188.9 |
| Effluent quality TSS (PPM) | 12 | 10 | 12 | 1 |
| Land required (m²) | 15 | 469,047 | 46,859 | 30,077 |

**Figure 5** | Weight percentages (scores) of the considered factors for the three alternatives.
Figure 5 presents a comparison of the considered factor scores for the three alternatives. It can be seen from the figure that the second alternative receives the highest score of the operating cost (40.0%) and the lowest score of TSS effluent quality (1.0%). In contrast, the third alternative gets the lowest score of the operating cost (2.5%) and the highest score of TSS effluent quality (12.0%). Also, the third alternative gets the highest score of both TDS effluent quality and required land due to the fact that membranes produce effluents with higher quality (Liu et al. 2012) and lower footprints (Kumar et al. 2015). It is essential to mention that the first alternative receives lower scores than the second and the third alternatives for all considered factors. This may be attributed to the long distance between the water source and the distribution system. From the results obtained from the decision matrix, it can be demonstrated that the suggested water source of the third alternative is the most appropriate option since it gets the highest total score (62.5%) of the considered factors, followed by the second alternative, which gets a 60% score. From a practical point of view, the second alternative may be the most appropriate alternative because it requires the lowest operating cost and no high-lifting pumps. So, more research should be conducted to address this point in depth and to study various ways to reduce the energy consumed in membrane systems.

4. CONCLUSION

The Mariout 2 WTP, which lies on the k-40 Alex-Cairo desert road, is one of the biggest AWCO’s WTPs that supplies the south and west of Alexandria and regions of Matrouh with drinking water since 1988. Owing to an increase in population, industrial zones, and touristic villages and also an increase in consumption, the Mariout 2 WTP cannot meet the present-day water demands, and so, it is necessary to design a fourth stage of the Mariout 2 WTP. In this study, three water source alternatives were suggested and designed to meet the water requirements for the target year 2047. The first two alternatives are located on the k-40 Alex-Cairo desert road and k-77 EL Nasr canal, respectively, where the water source is obtained from the Nile River. The third alternative is located at the k-51 Alex-Matrouh coastal road and it is a non-conventional sea water source. A decision matrix was produced to compare the three alternatives and select the best one. It was found from the forecast calculation that to design the fourth stage of the Mariout 2 WTP, 237,397 m³/d of water was required. The intake pipes and treatment units of the three alternatives were designed based on the calculated water demand. Cost estimation showed that the operating cost to treat a cubic meter of water acquired from the third alternative was the highest (6.62 EGP) and the operating cost obtained from the second alternative was the lowest (0.42 EGP). On the contrary, the cost of the intake pipes obtained from the third alternative was the lowest since it was the nearest source from the distribution system (3 km). Also, the cost of the intake pipes obtained from the first alternative (28 km) was the highest. The present worth of the three alternatives was calculated based on the operating costs and force main costs. It should be noted that the present worth of the third alternative obtained the highest value (6.67 EGP/m³) due to its higher operating cost. However, since no pumps were required, the second alternative provided the lowest present worth that equaled 0.81 EGP/m³.

To compare the three alternatives and select the most appropriate one, a simple decision matrix was designed. Some factors were considered in the decision matrix, such as the operating cost per cubic meter of produced water, cost of intake pipes, quality of effluent water (includes only TSS and TDS), and required land for the construction of the treatment units of each alternative. However, the third alternative produced the highest operating cost value. It was the appropriate option since it secured the highest total score (62.5%) of all the factors considered at the decision matrix. The second alternative secured the next highest score (60.0%) based on the results obtained from the decision matrix. Subsequently, the third alternative may be considered as the best one.

This is a case study and the results were obtained based on a simple decision matrix, where inserted weights were suggested by the managers of the AWCO keeping in mind the applications of functions to reflect local reality. For future use, the definitions about weights must be adjusted to the local reality for each study and/or the water company. Some factors should be addressed in future research, such as ways to reduce energy consumption by membranes and ways to treat brine. The approaches monitored in this study should be included in future comparisons of water detection strategies in Egypt.

CONFLICT OF INTEREST

The authors declare that they have no conflict of interest.

DATA AVAILABILITY STATEMENT

All relevant data are included in the paper or its Supplementary Information.
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