Application of One-Dimensional Hydraulic Model to Irrigation Systems with Complex Inner Boundary Conditions: Case Study on Fayoum Governorate

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Abstract

Realizing the need for improving water resources management in Fayoum Governorate, Egypt, the SOBEK hydrodynamic model was applied to Bahr El-Gharq, Bahr El-Bashawat and Bahr El- Gargaba canals using the data measured during the hydrographic survey. In this paper, SOBEK was employed to simulate the flow in three conveyance canals with complex inner boundary conditions. The developed model incorporated different inner boundary conditions, including regulators and weirs. The study aims at evaluating the influence of illegal irrigation practices along the three canals on the system operation through examining different hydraulic scenarios, taken into consideration the futility of present weirs and the use of pumping machines along the streams.

The model was calibrated for a 5 days period and the calibrated parameter (Manning Coefficient) was found to be 0.023. Observed and simulated water levels were in close agreement. The model was operated using a time step of 10 min for a simulation period of 5 days. Five different scenarios were investigated. The model results revealed that the impact of lowering weirs' crest levels become insignificant when reduced by more than 50 cm. Therefore, it is recommended to lower the weirs' crest levels of the three canals by 50 cm. This solution will help in lowering the water levels by about 40 cm less than the design water levels, and hence the irrigation water will be allocated to fields via pumps rather than by gravity. Moreover, this solution will help in avoiding the risk of having the water levels in the three canals close to the bank levels. Also, the three canals could easily accommodate the design discharges with the possibility of increasing these discharges in case of future expansions.

***Key words**:* SOBEK; Fayoum Irrigation System; Weirs; and Carrying Capacity

1. INTRODUCTION

In order to ensure safe operation and to meet water requirements in Fayoum Governorate, a number of hydraulic structures were built for large-scale water distribution canals. The Fayoum basin has a special system of irrigation due to its topography. The land level varies from (+ 24.00) above M.S.L at Fayoum City to (- 44.00) under M.S.L at Lake Qarun (Veer et al., 1993). The average slope is considerably steep with small field slopes. Thus, a system of weirs along the canals was used to regulate the slope of the canals according to the slope of the land. Consequently, operation of these structures results in complex hydraulic responses in canals. In-depth understanding of those responses is prerequisite for successfully regulating those hydraulic structures.

In Fayoum, all command areas are irrigated by surface irrigation with continuous flow in all canals. This method of irrigation secures an even distribution of irrigation water along the canals. However, there are sometimes farmers' practices that cause lack of water at the tailends. This paper addresses the problem of lack of irrigation water at the downstream ends of Bahr El-Gharq, Bahr El-Bashawat, and Bahr El-Gargaba canals that are located within the Hassan Wasef Canal system.

The paper tries to put some recommendations that aim at improving the carrying capacity of the abovementioned canals, taking into consideration the change in their hydraulic behavior due to the change in the water distribution method. The Hassan Wasef Canal system comprises different branch canals and hydraulic structures. At the downstream end, the Hassan Wasef Canal is divided into two branches, namely, Bahr El-Gharq and Bahr El-Nazla. Bahr El-Gharq Canal has a length of 27 km, a width of 10 m, and an average water depth of 2.5 m. It is further separated at its downstream end into two other canals, namely, Bahr El-Bashawat that extends for 25 km and Bahr El- Gargaba that has a length of 23 km. Both Bahr El-Bashawat and Bahr El- Gargaba canals have an average water depth of 1 m and a width of 5 m. Irrigation water is distributed from the three branch canals to agricultural fields through gravity with the aid of several weirs that are located along the canal system. Bahr El-Gharq has 7 weirs along its length. Whereas, Bahr El-Gargaba and Bahr el-Bashawat encompass 6 and 3 weirs respectively. Bahr El-Gharq is also characterized by high water levels and low bank levels, particularly the left bank with a level that exceeds the water level by only 20 cm causing a severe risk. Moreover, there are some illegal cultivated areas on the two banks of Bahr El-Gharq Canal with the existence of large areas cultivated with rice, and hence farmers started to use pumps to lift water for irrigating these areas causing tangible decrease in water levels, and hence weirs became insignificant. This caused variability of discharges along the canal, which affected the equity of water distribution and adequacy of water use at tertiary level.

The present study aims at using the hydrodynamic numerical model “SOBEK” in evaluating the influence of illegal irrigation practices along the three canals on the system operation through examining different hydraulic scenarios. The hydrodynamic simulation of the canals will help in reaching solutions to resolve the problems of water shortage in these waterways, bearing in mind the futility of present weirs and the use of pumping machines along the streams.

2. MODEL DESCRIPTION

Use of hydrodynamic simulation models seems to be one of the most important tools for understanding the hydraulic behavior of main irrigation canals. Over the years, several mathematical models have been developed to study the flow dynamics of the main canal system. Finite-difference methods were developed in the 1970s and applied to a branching and looped network (Clemmens et al., 2005). The unsteady-flow simulation models subsequently developed are either upstream-control or downstream-control oriented and employ either local or central criteria to handle scheduled, arranged, or on-demand methods of water delivery (Clemmens et al., 2005). Some of the available canal hydraulic models are CANALMAN, DUFLOW, CARIMA, MODIS, USM, and Branch Canal Network Model.

SOBEK is a powerful 1D and 2D instrument developed by WL Delft Hydraulics, the Netherlands for flood forecasting, drainage systems, irrigation systems, sewer overflow, ground-water level control, river morphology, salt intrusion and water quality (Delft Hydraulics, 2005). SOBEK is equipped with a very robust numerical scheme that handles drying and flooding and sub- and super critical flows efficiently. It has been selected for application in irrigation systems of Fayoum Governorate due to its several distinct advantages. It can accommodate the irregular shape of canal cross sections and simulate flow for long canal systems with several lateral offtakes and in-line control structures such as weirs in case of Fayoum governorate. SOBEK also allows for the integration of user-defined modules through the use of specific data exchange formats. SOBEK's unique integrated format means that the effectiveness of measures taken can be checked to keep your system running at peak efficiency. The manual or automatic operation of pumps, sluice gates, weirs, storage tanks and other structures can all be incorporated into the model, giving you a realistic picture of how a system behaves in extreme scenarios. SOBEK is based on high-performance computer technology. That means it can handle water networks of any size (big or small) and complexity.

3. SOLUTION ALGORITHM

The dynamic behaviour of these systems can be well described by a set of equations known as the Saint-Venant equations [1]:

$$\frac{\partial Q}{\partial x} + B\frac{\partial H}{\partial t} = 0 \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial(\frac{Q^2}{A})}{\partial x} + gA(Sf - S_0) = 0 \tag{2}$$

where Q = discharge (m^3/s); x = longitudinal distance along the channel in the direction of flow (m); t = time (s); B = water surface width (m); H= water depth (m); S_0 = slope of channel bottom; S_f = slope of energy grade line; g = acceleration due to gravity (9.81 m/s^2). In order to calculate S_f, the Manning Equation is used.

$$V = \frac{1}{n} R^{2/3} S_f^{\ 1/2} \tag{3}$$

where R = hydraulic radius (m); V = mean velocity (m/s); n = Manning's resistance coefficient (m$^{1/2}$/s). There are many methods to solve numerically the Saint-Venant equations in order to better study real systems. In the present case, the referred equations were in the first place made linear, to overcome their non-linearity and in order to become possible the use of linear controllers. The equations are linearized assuming conditions near some steady state. This ensures that, for small deviations from a considered setpoint, the resulting equations will still describe the behavior of the system. The linearization of the Saint-Venant equations leads to:

$$\frac{\partial q}{\partial x} + B_0 \frac{\partial H}{\partial t} = 0 \tag{4}$$

$$\frac{\partial q}{\partial t} + 2V_0 \frac{\partial q}{\partial x} + B_0 \frac{\partial h}{\partial x}(c_0^{\ 2} - v_0^{\ 2}) - \xi_0 q - \gamma_0 q = 0 \tag{5}$$

where q and h are the variations of discharge and water depth, respectively, from a considered steady state, c the wave celerity ($c = \sqrt{\frac{gA}{B}}$), V flow velocity ($V = Q/A$) and 0 subscript stands for steady state values; ξ_0 and γ_0 are factors due to linearization defined by:

$$\xi_0 = -\frac{2gQ_0}{n^2 A_0 R_0^{\ 4/3}} + \frac{2Q_0}{A_0^{\ 2}} B_0 (\frac{dH}{dX})_0 \tag{6}$$

$$\gamma_0 = gS_0 B[1 + Co - (1 + Co + (co - 2)F_r^{\ 2})\alpha] \tag{7}$$

$$\alpha = \frac{\left(\frac{dH}{dX}\right)_0}{S_0} \tag{8}$$

$$Co = 1 + \frac{4}{3}\frac{P_0}{B_0}\left(\frac{dR}{dH}\right)_0 \tag{9}$$

Where Fr is the Froude number defined by $c\ Fr = V$. In spite of having now all the necessary equations linearized, there are still many steady state parameters left to determine, exception made to steady state discharge that one can choose freely as it is the condition for linearization.

Steady state parameters can be calculated by taking again the Saint-Venant equations and assuming no variations in discharge and flow depth in time. So, solving those leads to:

$$\frac{dH}{dX} = \frac{S_f - S_0}{1 - F_r^{\ 2}} \tag{10}$$

This equation represents the spatial gradually varied flow and will be used to determine the flow profile in steady state flow conditions.

4. DATA COLLECTION AND REQUIREMENTS

The study comprised field investigations, data collection, and hydrographic survey using high technology instruments, such as DGPS, Complete Total Station, Echo-sounding, and Range Finder. The hydrographic survey covered the whole length of the 3 canals. The hydrographic survey involved investigating cross-sectional details each 100 m and measurement of water surface slope each 1 km. Also, status of the banks was monitored and registered. On the other hand, water velocities were measured at 17 different locations including all weirs, of which 8 were measured along Bahr El-Gharq, 6 along Bahr El- Gargaba, and 3 along Bahr El-Bashawat. Using the collected data, longitudinal profiles of the three canals were drawn and compared with the design profiles. Moreover, the cross sections of the three canals were induced, and hence the actual discharges of the three canals were calculated at the upstream and downstream ends. The hydrographic survey and hydraulic measurements revealed that the observed longitudinal profile of Bahr El-Gharq has a bed level of 40 cm less than that of the designed profile, while the observed longitudinal profile of Bahr El-Bashawat has a bed level of 50 cm less than that of the designed profile. On the other hand, the measured longitudinal profile of Bahr El-Gargaba is in close agreement with the designed profile. Moreover, the bed widths of the measured cross sections of the three canals are in close agreement with the designed cross sections.

Moreover, the field measurements revealed that current water levels of Bahr El-Gharq canal at the discharge of 24 m^3/s are lower than the design water levels by about 35 cm. Whereas, the present water levels of Bahr El-Gargaba at the discharge of 3 m^3/s are less than that of the design cross section by about 50 cm. In the same regard, hydraulic measurements revealed that existing water levels of Bahr El-Bashawat are less than the design ones by about 30 cm in the reach from the intake to Km (8.000) and by about 1.10 m in the reach between (km 8.000) and km (11.000). In addition to all above, the data collected was used to make a comparison between design water levels and the current bank levels (Tables 1, 2, 3).

Table 1: Comparison between design water levels and current bank levels of Bahr El-Gharq

Reach	Left Bank	Right Bank
Intake – km (4.800)	Bank level is higher than water level by 60 cm	Bank level is higher than water level by 70 cm
km (4.800) – km (17.000)	Bank level is higher than water level by 60 cm	Water level is very close to bank level
km (17.000) – km (20.000)	Water level is very close to bank level	Bank level is higher than water level by 70 cm
km (20.000) – km (25.00)	Bank level is higher than water level by 60 cm	Water level is very close to bank level

Table 2: Comparison between design water levels and current bank levels of Bahr El-Gargaba

Reach	Left Bank	Right Bank
Intake – km (4.700)	Bank level is higher than water level by 50 cm	Water level is very close to bank level
km (4.700) – km (5.900)	Water level is very close to bank level	Water level is very close to bank level
km (5.900) – km (9.000)	Bank level is higher than water level by 50 cm	Water level is very close to bank level
km (9.000) – km (9.600)	Water level is very close to bank level	Water level is very close to bank level
km (9.600) – km (12.800)	Bank level is higher than water level by 50 cm	Water level is very close to bank level
Km (12.800) – km (15.350)	Water level is very close to bank level	Water level is very close to bank level

Table 3: Comparison between design water levels and current bank levels of Bahr El-Bashawat

Reach	Left Bank	Right Bank
Intake – km (7.900)	Bank level is higher than water level by 1.5 m	Bank level is higher than water level by 1.3 m
km (7.900) – km (11.200)	Water level is very close to bank level	Water level is very close to bank level

5. MODEL SETUP

The setup preparation for the hydraulic model involves specifications of canal cross sections, layout of the canal network, head and cross regulators, weirs, and upstream and downstream initial and boundary conditions.

5.1 Canal Network Definition

Both the upstream and downstream ends of the three canals are identified. The maximum selected distance between two neighboring points is also required for the network definition. The topographical identification is extracted from the index map of the Nile River System that is stored in SOBEK.

5.2 Cross Sections Data Definition

In SOBEK, the cross section database can be regarded as a library storing data for a large number of cross sections, organized in such a way that every cross section is identified by canal name and topographical identification. The cross section data are available at discrete points along the canal system. In this study, a space interval of (*dx*max) 100 m is selected; thus, cross sections are defined in the model setup at every 100 m. This brought the total number of modeled cross sections to 53 along Bahr El-Gharq, 39 along Bahr El-Gargaba, and 24 along Bahr El-Bashawat. At some places, where the structures exist, cross sections are specified both upstream and downstream of these features.

5.3 Initial Boundary Conditions

The initial conditions can be specified as global values of water levels and discharges for the entire canal network or as local values at different distances of a particular canal. These initial conditions are specified in the supplementary data file. The boundary conditions may be internal or external conditions. The internal boundary condition includes the specifications at nodal points and structures, whereas the external boundary condition includes the specification of constant values for h or Q or time varying values for Q or h at the starting point and endpoint. The daily discharges at the system source during the survey and the water level at the downstream end are specified in the time series database file to serve as boundary conditions.

5.4 Simulation Parameters

Before running the model simulation, control parameters such as simulation period, simulation time step, data to be stored, and storage time have to be specified. The simulation period is specified by start and end dates defined by year, month, day, hour, and minute. SOBEK checks the actual time and reads all data given as a time series during the simulation. The Courant number is used for selecting the time step, which controls the simulation process, and is calculated as follows:

$$\text{Courant number} = \frac{\Delta t(V + \sqrt{gy})}{\Delta x} \tag{11}$$

Where Δt = time step (s); V = mean flow velocity (m/s); y = water level (m); and Δx = dx-max.
In this study the time step was 10 min and the simulation period extends for 5 days.

6. MODEL CALIBRATION

In the present study, the resistance number is considered as the model calibration parameter. The resistance number used in this model is defined as the reciprocal of the Manning roughness coefficient. After setting up the model, an initial run is made with the measured value of Bahr El Gharq (resistance number of 40) using the measured discharges and water levels. For the simulation period of 5 days, a simulation time step of 10 min is selected. This period is selected for the calibration because measured data are available throughout the canal length. Thereafter, comparison is made between the observed and simulated water levels along the length of the canal. Selection of locations for calibration is based on the availability of observed flow data. Based on the comparison, the model parameter is adjusted. This process is continued until the observed and simulated values are in close agreement. As shown in Figure (1), the simulated water levels are in close agreement with the observed water levels. Considering the overall acceptability of the model results, the model setup is taken as calibrated.

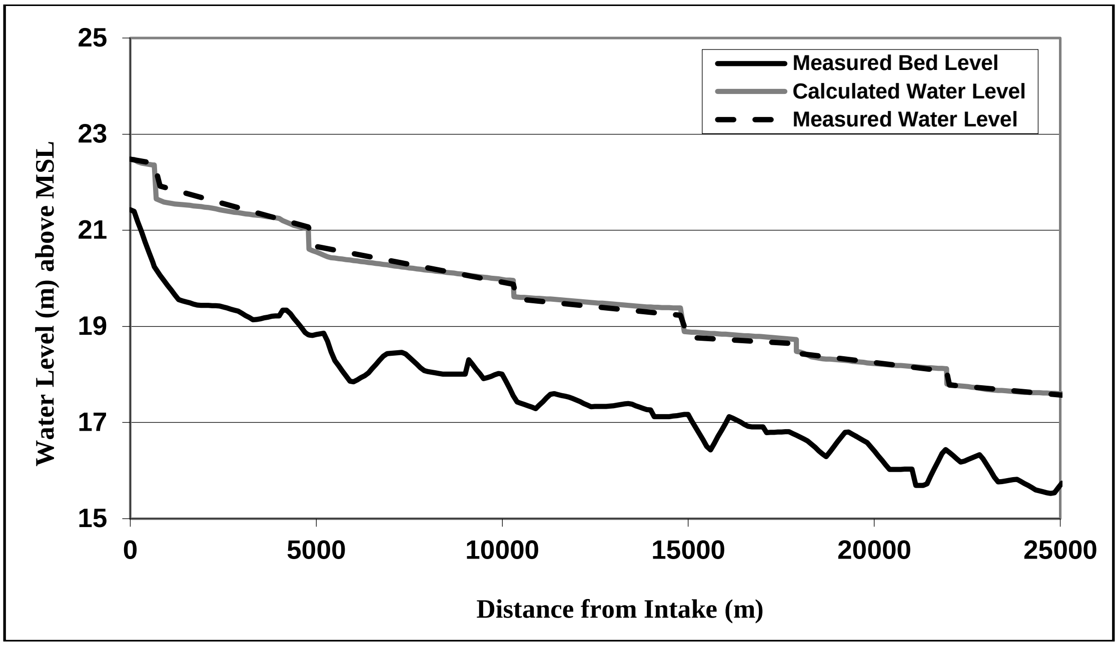

Figure 1: Model calibration

7. MODEL OPERATION AND RESULTS

The hydraulic performance of the canal was tested via five different scenarios (Table 4). The scenarios were investigated to figure out the best solution to improve the carrying capacity of the three canals.

Table 4: Operation scenarios

Scenario	Model Inputs	Conditions
1	Design discharges, current CSs, and current weirs' crest levels	- Max. discharge at Bahr El-Gharq intake = 24.11 m^3/s - Max. discharge at Bahr El-Gargaba intake = 3.80 m^3/s - Max. discharge at Bahr El-Bashawat intake = 5.06 m^3/s
2	Design water levels, current CSs, and current weirs' crest levels	- Max. water level at Bahr El-Gharq intake = 22.80 m - Max. discharge at Bahr El-Gargaba intake = 3.80 m^3/s - Max. discharge at Bahr El-Bashawat intake = 5.06 m^3/s
3	Design discharges, current CSs, and 25 cm reduction in weirs' crest levels	- Max. discharge at Bahr El-Gharq intake = 24.11 m^3/s - Max. discharge at Bahr El-Gargaba intake = 3.80 m^3/s - Max. discharge at Bahr El-Bashawat intake = 5.06 m^3/s
4	Design discharges, current CSs, and 50 cm reduction in weirs' crest levels	- Max. discharge at Bahr El-Gharq intake = 24.11 m^3/s - Max. discharge at Bahr El-Gargaba intake = 3.8 m^3/s - Max. discharge at Bahr El-Bashawat intake = 5.06 m^3/s
5	Design discharges, current CS, and 75 cm reduction in weirs' crest levels	- Max. discharge at Bahr El-Gharq intake = 24.11 m^3/s - Max. discharge at Bahr El-Gargaba intake = 3.80 m^3/s - Max. discharge at Bahr El-Bashawat intake = 5.06 m^3/s

Scenario (1) investigation of the current status

The model results of this scenario revealed that the maximum water level upstream the head regulator of Bahr El-Gharq is 22.60 m, whereas the maximum design water level equals 22.80 m (Figure 2). The model showed that the average water velocity of Bahr El-Gharq is 0.65 m/s. On the other hand, the simulation results of both Bahr El-Gargaba and Bahr El-Bashawat demonstrated that the maximum water level upstream the intakes of the two canals is 17.77 m, while the maximum design water levels are 18.31 m and 17.28 m respectively (Figures 3 and 4). Likewise, the average computed water velocities are 0.51 m/s and 0.42 m/s. Moreover, the results showed that computed water levels in some parts along Bahr El-Gharq and Bahr El-Gargaba are still close to the right bank levels. Whereas, bank levels of Bahr El-Bashawat are higher than computed water levels by an average of 1.20 – 1.50 m.

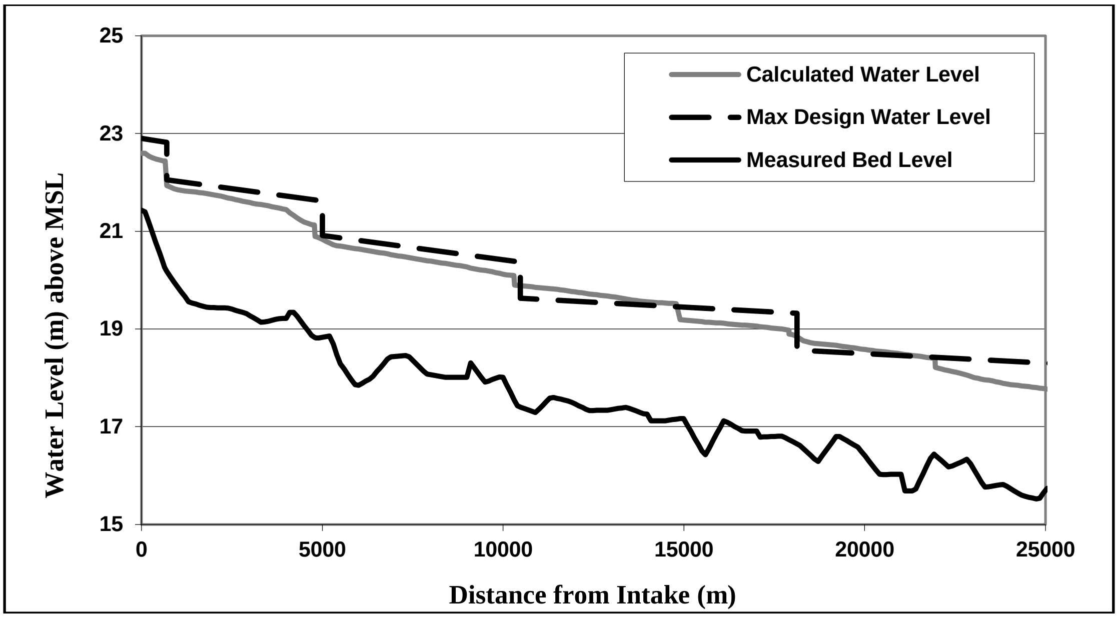

Figure 2: Computed water levels in scenario 1 versus design water levels of Bahr El-Gharq

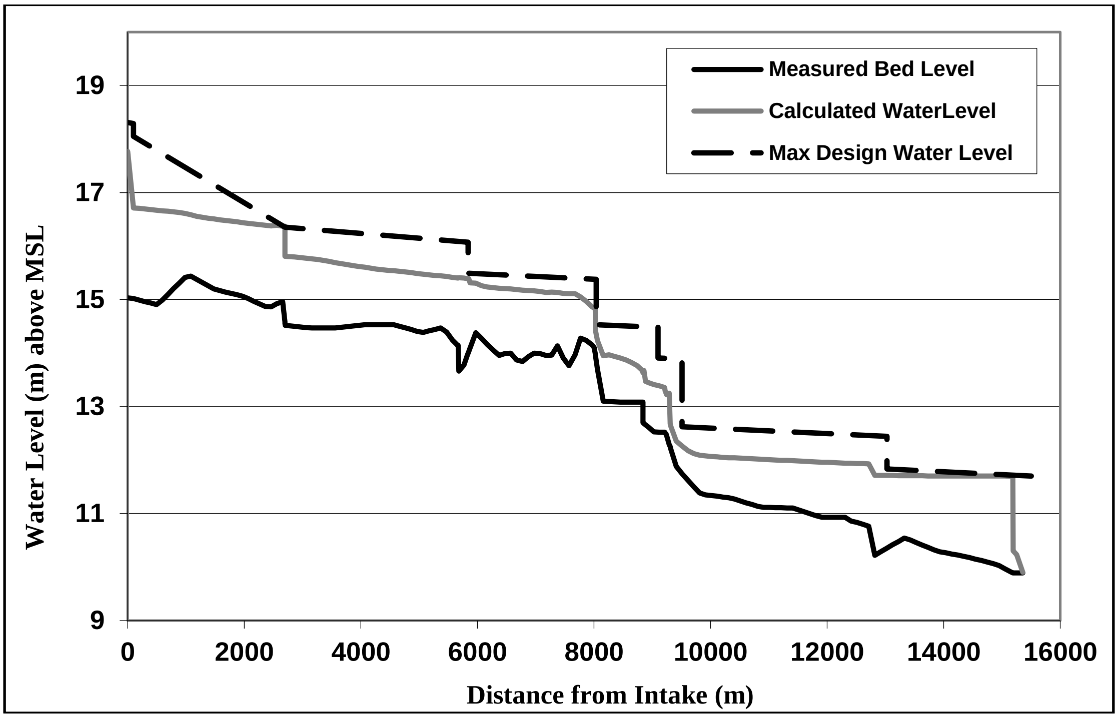

Figure 3: Computed water levels in scenario 1 versus design water levels of Bahr El-Gargaba

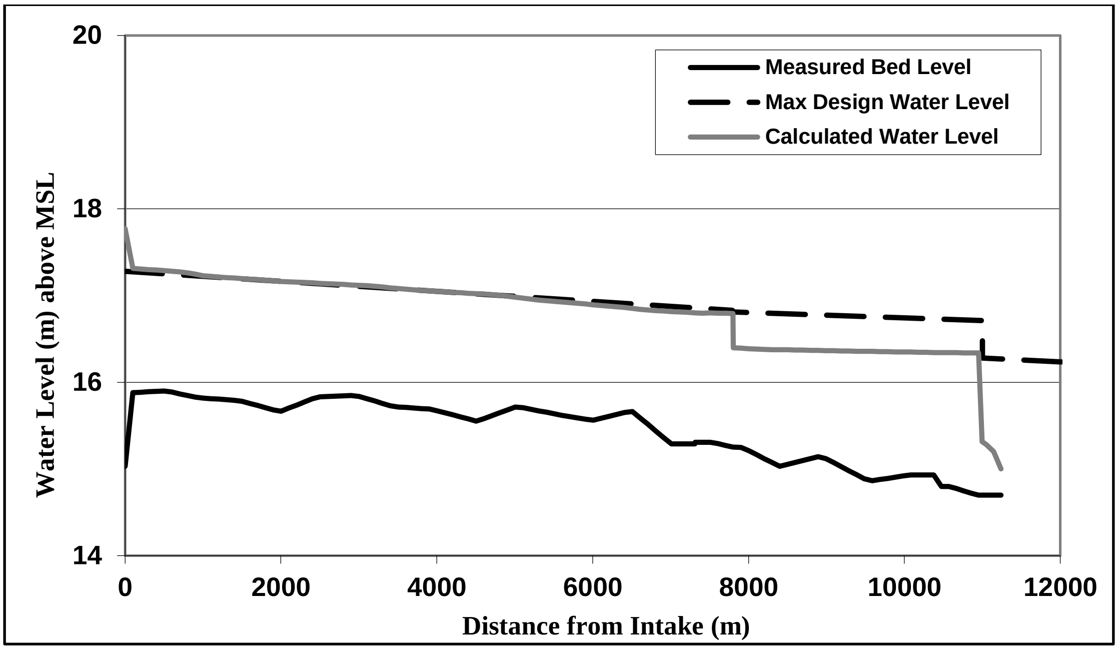

Figure 4: Computed water levels in scenario 1 versus design water levels of Bahr El-Bashawat

Scenario (2) investigation of the three canals hydraulic efficiency

The model results of this scenario revealed that the maximum discharge downstream the head regulator of Bahr El-Gharq is 33 m^3/s at the design water level of 22.80 m. Whereas, the required design discharge is 24.11 m^3/s. Figure 5 illustrates a comparison between computed water levels in Scenario 2 and maximum design water levels of Bahr El-Gharq. The model showed that the average water velocity of Bahr El-Gharq in this case is 0.77 m/s. On the other hand, the simulation results of both Bahr El-Gargaba and Bahr El-Bashawat showed that the maximum water level upstream the intakes of the two canals is 17.90 m, while the maximum design water levels are 18.31 m for Bahr El-Gargaba and 17.28 m for Bahr El-Bashawat (Figures 6 and 7). Correspondingly, the average computed water velocities are 0.74 m/s and 0.60 m/s. Moreover, the results showed that computed water levels in some parts along Bahr El-Gharq, Bahr El-Gargaba, and Bahr El-Bashawat are still close to the right bank levels. Whereas, left bank levels of Bahr El-Gharq, Bahr El-Gargaba, and Bahr El-Bashawat are higher than computed water levels by an average of 0.70, 0.45, and 1.20 m respectively.

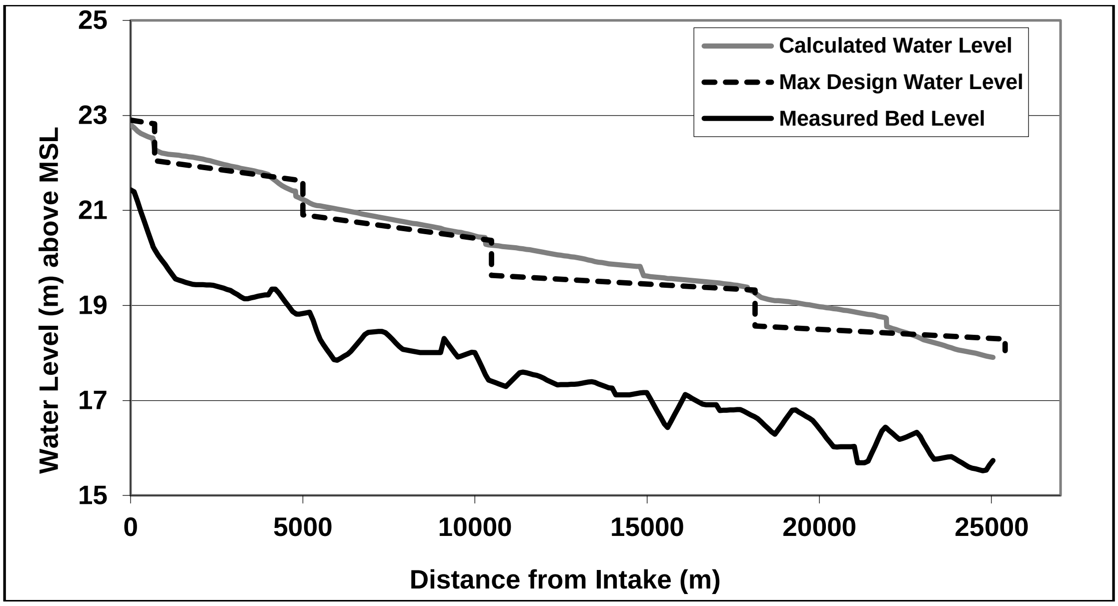

Figure 5: Computed water levels in scenario 2 versus design water levels of Bahr El-Gharq

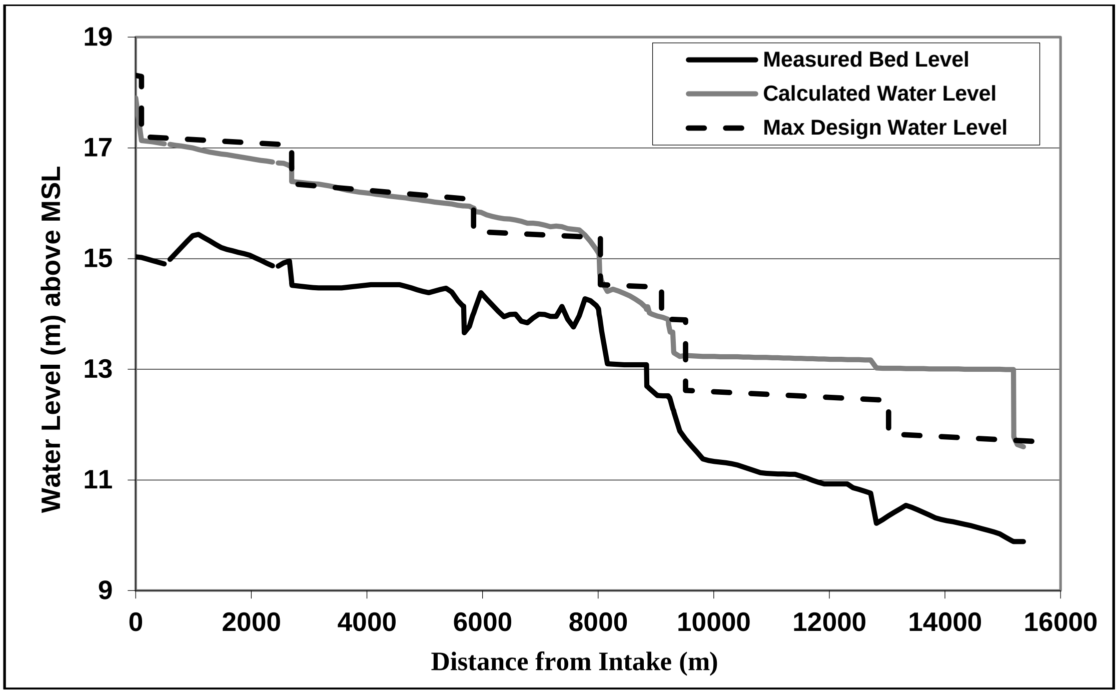

Figure 6: Computed water levels in scenario 2 versus design water levels of Bahr El-Gargaba

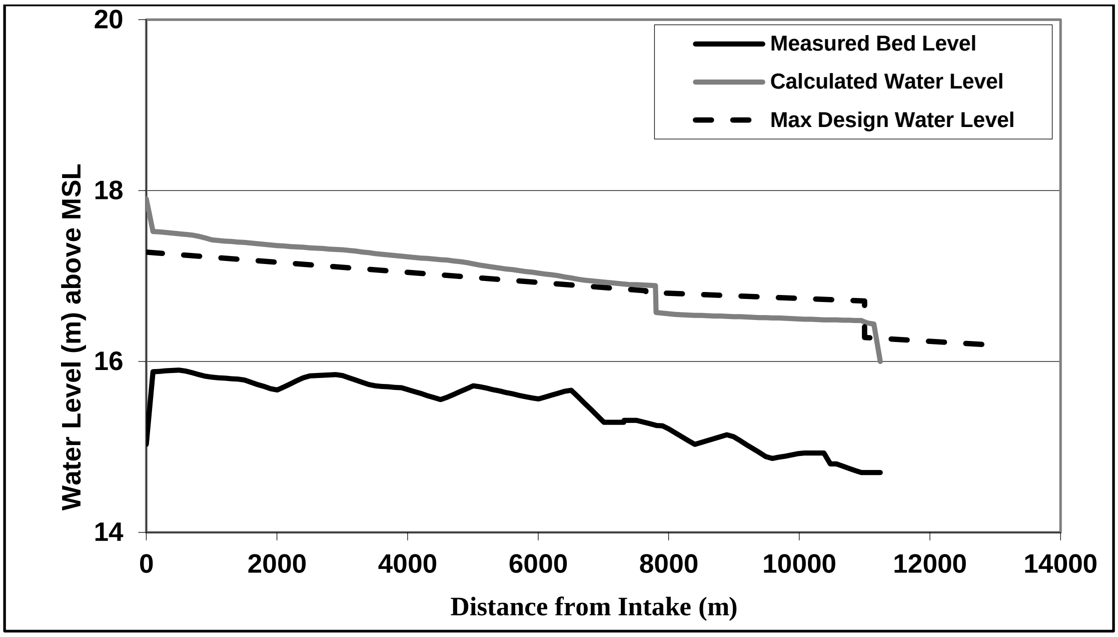

Figure 7: Computed water levels in scenario 2 versus design water levels of Bahr El-Bashawat

Scenario (3) weirs' crest levels are lowered by 25 cm

The model results of this scenario revealed that the maximum water level upstream the head regulator of Bahr El-Gharq is 22.45 m, whereas the maximum design water level equals 22.80 m (Figure 8). The model showed that the average water velocity of Bahr El-Gharq is 0.70 m/s. On the other hand, the simulation results of both Bahr El-Gargaba and Bahr El-Bashawat demonstrated that the maximum water level upstream the intakes of the two canals is 17.52 m, while the maximum design water levels are 18.31 m and 17.28 m respectively (Figures 9 and 10). The results showed that left bank levels for the three canals are higher than water levels by sufficient distance. However, water levels in some parts along both Bahr El-Gargaba and Bahr El-Bashawat are still close to the right bank levels. Likewise, the average computed water velocities are 0.57 m/s and 0.53 m/s.

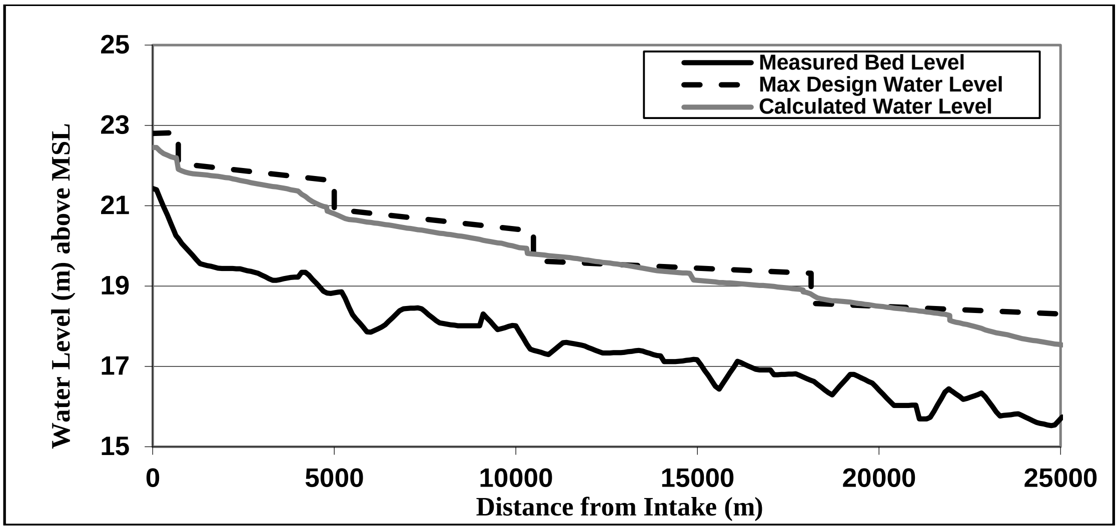

Figure 8: Computed water levels in scenario 3 versus design water levels of Bahr El-Gharq

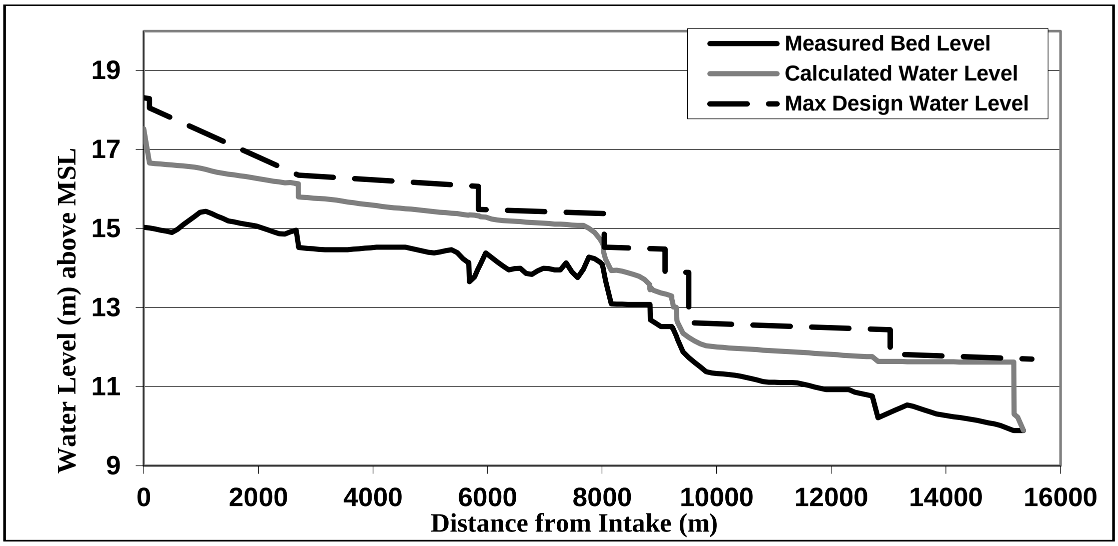

Figure 9: Computed water levels in scenario 3 versus design water levels of Bahr El-Gargaba

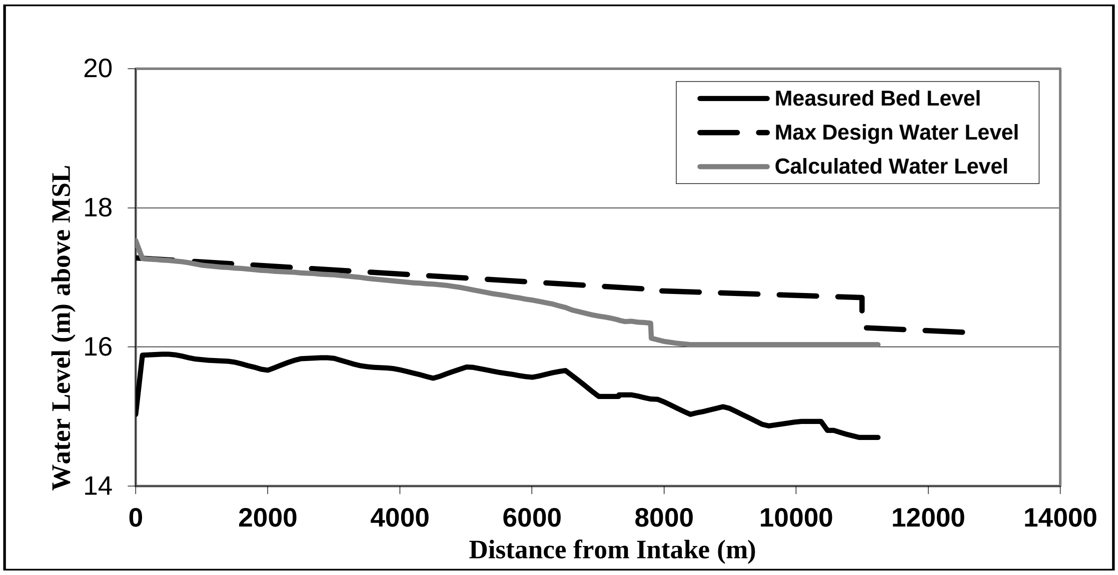

Figure 10: Computed water levels in scenario 3 versus design water levels of Bahr El-Bashawat

Scenario (4) weirs' crest levels are lowered by 50 cm

The model results of this scenario revealed that the maximum water level upstream the head regulator of Bahr El-Gharq is 22.37 m, whereas the maximum design water level equals 22.80 m (Figure 11). The model showed that the average water velocity of Bahr El-Gharq is 0.73 m/s. On the other hand, the simulation results of both Bahr El-Gargaba and Bahr El-Bashawat demonstrated that the maximum water level upstream the intakes of the two canals is 17.38 m, while the maximum design water levels are 18.31 m and 17.28 m respectively (Figures 12 and 13). Correspondingly, the average computed water velocities are 0.63 m/s and 0.54 m/s. The output of this solution showed that the levels of both right and left banks for the three canals are higher enough than water levels. Right bank levels of Bahr El-Gharq, Bahr El-Gargaba, and Bahr El-Bashawat are higher than computed water levels by an average of 0.35, 0.65, and 0.70 m respectively.

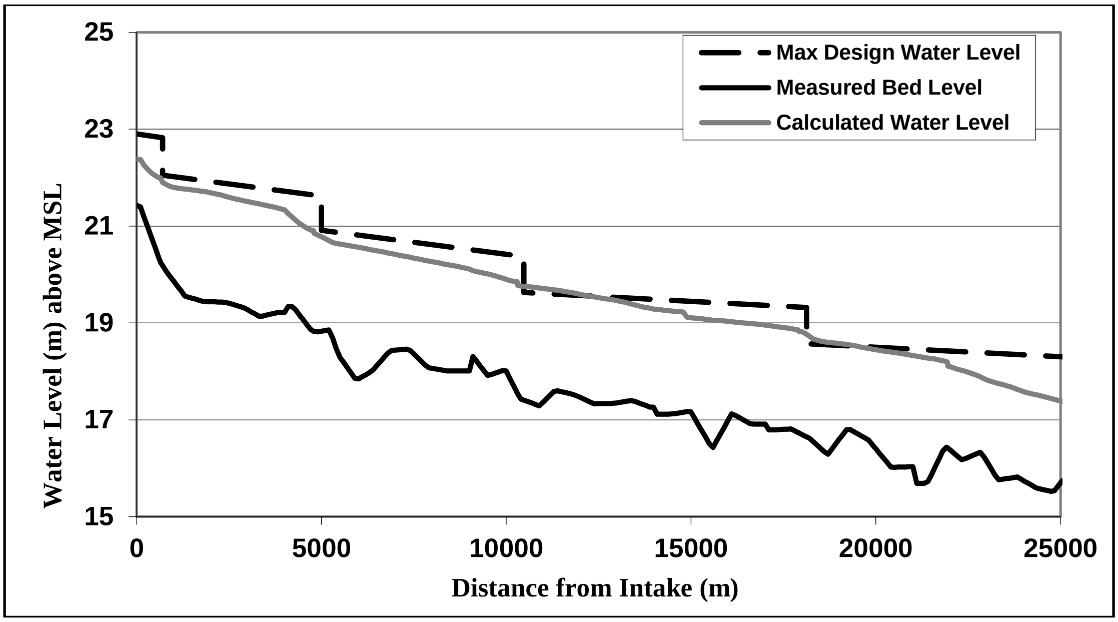

Figure 11: Computed water levels in scenario 4 versus design water levels of Bahr El-Gharq

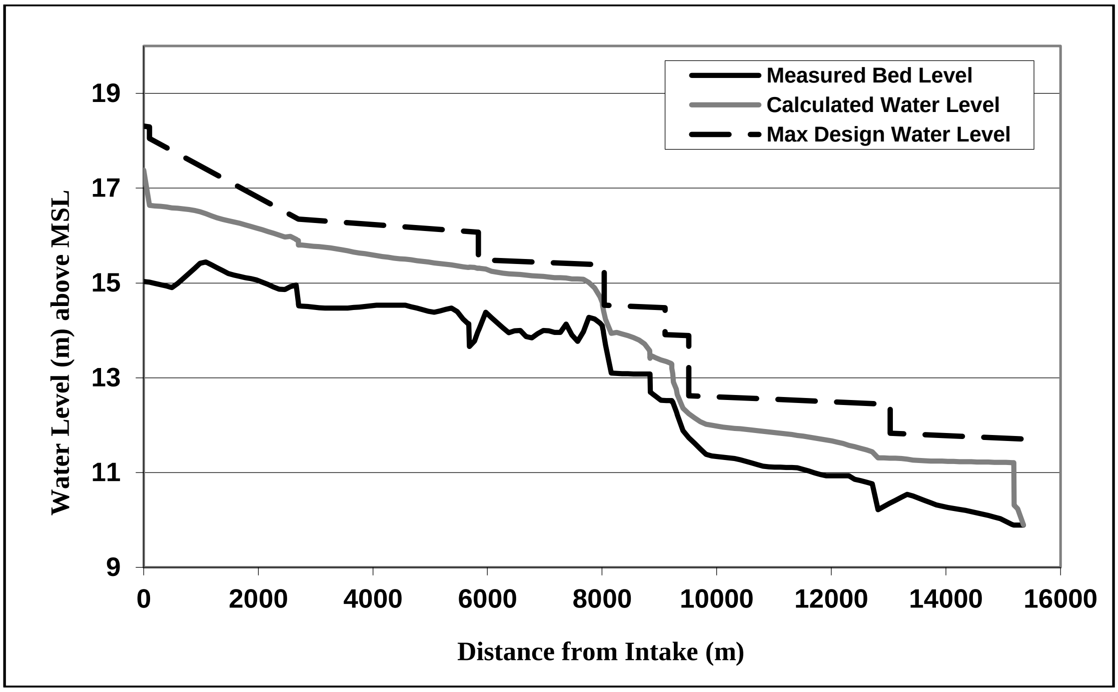

Figure 12: Computed water levels in scenario 4 versus design water levels of Bahr El-Gargaba

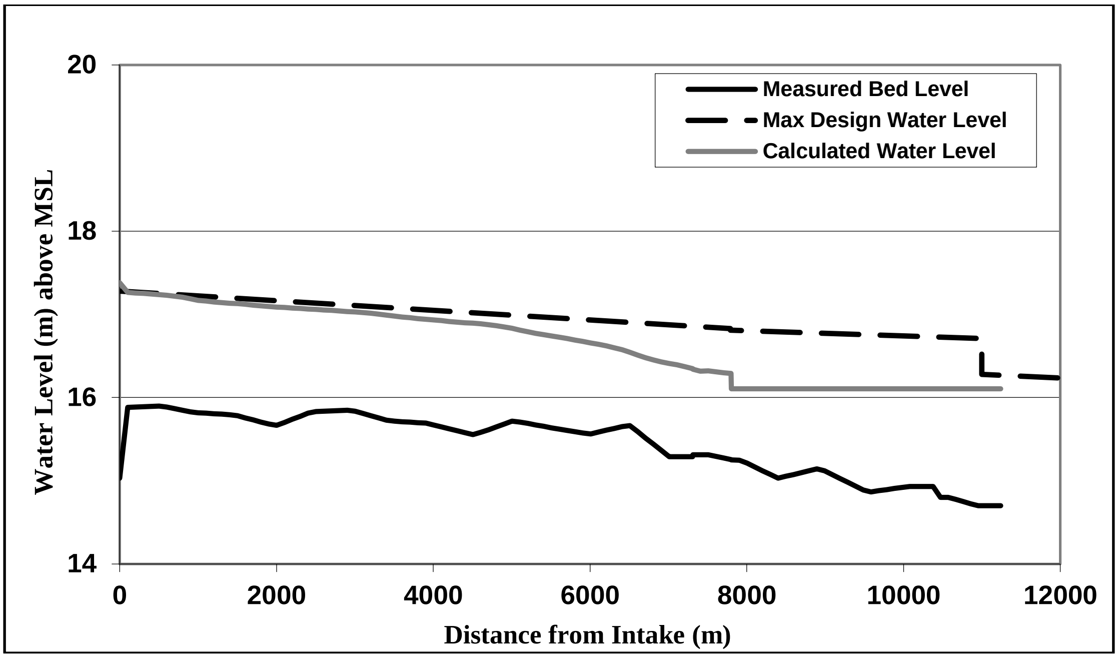

Figure 13: Computed water levels in scenario 4 versus design water levels of Bahr El-Bashawat

Scenario (5) weirs' crest levels are lowered by 75 cm

The model results of this scenario revealed that the maximum water level upstream the head regulator of Bahr El-Gharq is 22.35 m, whereas the maximum design water level equals 22.80 m (Figure 14). The model showed that the average water velocity of Bahr El-Gharq is 0.76 m/s. On the other hand, the simulation results of both Bahr El-Gargaba and Bahr El-Bashawat demonstrated that the maximum water level upstream the intakes of the two canals is 17.31 m, while the maximum design water levels are 18.31 m and 17.28 m respectively (Figures 15 and 16). Correspondingly, the average computed water velocities are 0.65 m/s and 0.60 m/s. The yield of this scenario proved that the levels of both right and left banks for the three canals are higher enough than water levels. Moreover, the results showed that lowering the weirs' crest levels by 75 cm does not have significant effect on water levels in the three canals.

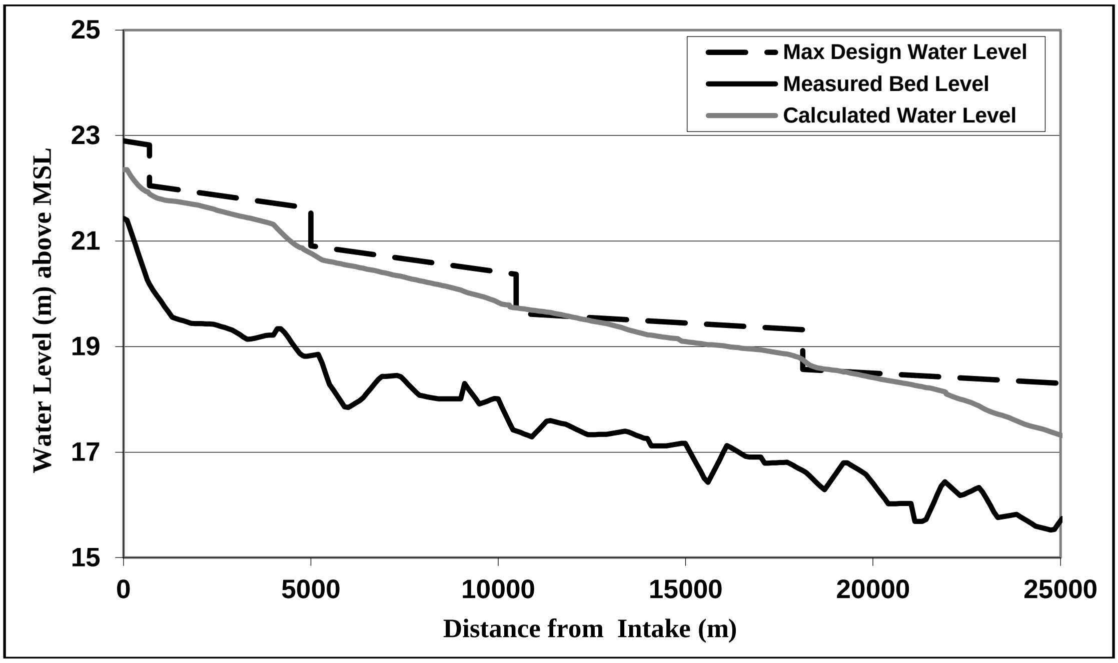

Figure 14: Computed water levels in scenario 5 versus design water levels of Bahr El-Gharq

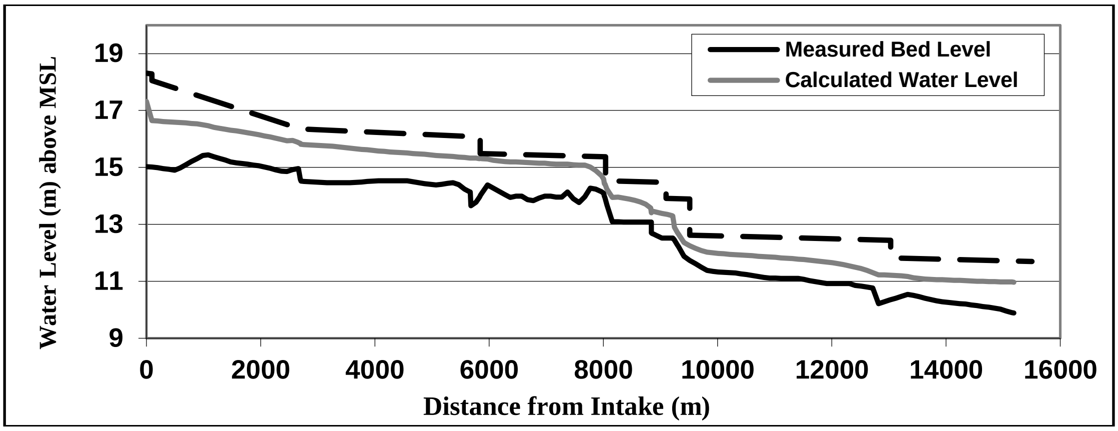

Figure 15: Computed water levels in scenario 5 versus design water levels of Bahr El-Gargaba

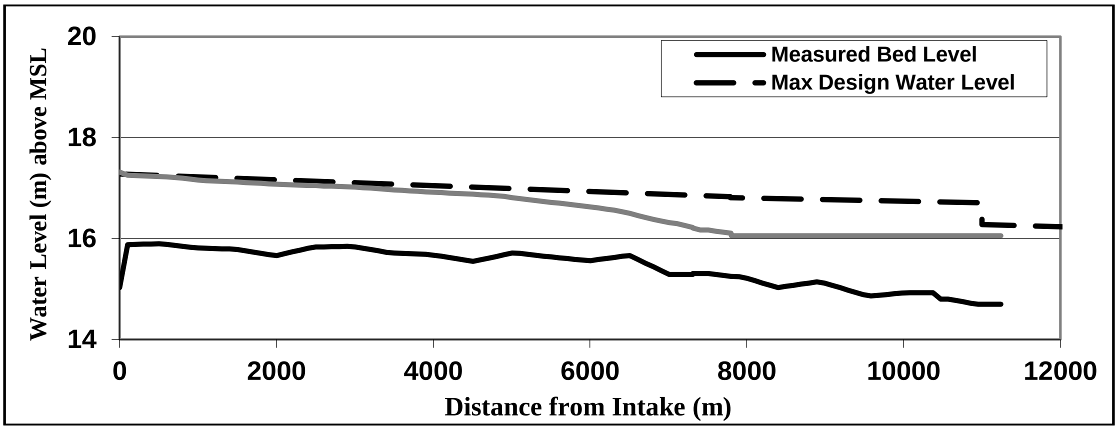

Figure 16: Computed water levels in scenario 5 versus design water levels of Bahr El-Bashawat

8. CONCLUSIONS AND RECOMMENDATIONS

In this study, the 1-D module of SOBEK was applied to Bahr El-Gharq, Bahr El-Gargaba, and Bahr El-Bashawat canals using the data measured in the hydrographic survey. The main goal was to figure out the appropriate solution to resolve the problems of water shortage in these waterways, bearing in mind the ineffectiveness of present weirs and the use of pumping machines along the streams as well as to avoid the risk of having water levels very close to bank levels. The calibrated resistance number was found to be 0.023. The model was operated using a time step of 10 min for a simulation period of 5 days. Five different scenarios were investigated. The model results revealed that maximum current water levels at Bahr El-Gharq are less than that of the design by an average of 35 cm at a discharge of 24 m^3/s. Whereas, maximum current water levels at Bahr El-Gargaba and Bahr El-Bashawat are less than the design water levels by 50 cm and 30 cm respectively.

It is clear from scenarios (4), (5) that the impact of lowering weirs' crest levels become insignificant when reduced by more than 50 cm. Therefore, it is recommended to lower the weirs' crest levels of the three canals, except for the first weir located at 760 m downstream Bahr El-Gharq intake by 50 cm. correspondingly, the water level downstream Bahr El-Gharq intake will reach 22.37 m at the design discharge of 24.11 m^3/s. This solution will help in lowering the water levels by about 40 cm less than the design water levels, and hence the irrigation water will be allocated to fields via pumps rather than by gravity. Also, this solution will help in avoiding the risk of having the water levels in the three canals close to the bank levels. Water levels will be 40 cm apart of the bank levels. Provided that the crest levels are lowered by 50 cm, the three canals could easily accommodate the design discharges with the possibility of increasing these discharges in case of illegal irrigation practices. For example, the water level downstream Bahr El-Gharq intake will reach 22.50 m at a discharge of 26 m^3/s to satisfy 13 cm increase in water level due to illegal rice cultivation requirements. Whereas, the canals can accommodate a discharge of 29 m^3/s to meet 23 cm increase in the water level downstream Bahr El-Gharq intake.

9. LIST OF ABBREVIATIONS

A: flow area (m^2);
g: ratio of weight to mass/acceleration due to gravity (9.81 $m\ s^{-2}$);
h: stage above datum/water level (m);
n: Manning's coefficient ($s\ m^{-1/3}$)
Q: discharge ($m^3\ s^{-1}$);
q: lateral inflow ($m^2\ s^{-1}$);
R: hydraulic or resistance radius (m);
Sf: Slope of energy grade line;
S0: Slope of channel bottom;
t: Elapsed time (s);
V: Mean flow velocity ($m\ s^{-1}$);
x: Longitudinal distance in direction of flow (m);
y: Water level (m);
Δt : Time step (s); and
Δx : dx-max =maximum distance between two neighboring h points in computational grid

10. ACKNOWLEDGMENTS

The authors are very grateful to their colleagues at the Survey Department of the Hydraulics Research Institute for providing the hydrographic and bathymetric data related to Bahr El-Gharq, Bahr El-Gargaba, and Bahr El-Bashawat canals.

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