# Development of the main system of bisintaway canal under the irrigation improvement project "case study"

Hossam El-Din M. Moghazy [a], Osama K. Saleh [b],
Adel A. S. Salem [a] and Noha F. Abd El-Azim [a]

[a] *Irrigation and Hydraulics Engg., Dept., Faculty of Eng., Alexandria University, Alexandria, Egypt*
[b] *Water and Water Structures Engg. Dept., Faculty of Engg., Zagazig University, Zagazig, Egypt*

In order to overcome the present rotation irrigation system disadvantages, the Ministry of Water Resources and Irrigation in Egypt has initiated the Irrigation Improvement Project (IIP) which is considered a national irrigation project. The main objective of this study is to develop the existing improvements applied to the main irrigation system of Bisintaway branch canal which is considered one of the pilot command areas under improvement by the IIP in El-Beheira governorate. To achieve the goal of the study, different operation scenarios are introduced to investigate the effect of different factors on the performance of the existing IIP applied to Bisintaway canal. HYDRO-IIP canal simulation model, which is currently used by the Ministry of Water Resources and Irrigation, is introduced in this study. It is used to design the waterline along Bisintaway canal, to calculate the maximum discharges and velocities and to simulate the different operating scenarios. Among these scenarios was introducing the effect of the storage capacities of the sub-branches of the canal, and the introduction of different abstraction patterns, where, seven different abstraction patterns were studied. Field measurements of the water levels along Bisintaway canal were also introduced to verify the simulation model and check the actual performance of the canal under the IIP improvements.

تشير الدراسات أن الأمن المائى المصرى فى تناقص مستمر وذلك نتيجة لثبات حصة مصر من ايراد نهر النيل والتى تبلغ 55.5 مليار متر مكعب سنويا مع وجود زيادة مضطردة فى تعداد السكان بالاضافة الى مشروعات قومية وعملاقة لزيادة الرقعة الزراعية، مما أدى الى التفكير الجاد فى الاستغلال الأمثل لمياه نهر النيل. وبناء عليه جارى تنفيذ مشروع تطوير الرى الذى يهدف الى تحقيق عدالة توزيع المياه ورفع كفاءة الرى على كافة المستويات. والغرض الأساسى من هذا البحث هو تقديم الاقتراحات لتحسين أداء مشروع تطوير الرى الذى تم تنفيذه بترعة بسنتواى الواقعة بمحافظة البحيرة. ولتحقيق هذا الهدف تم دراسة عدة اقتراحات تشغيل على مستوى شبكة الرى الرئيسية وقد تم خلال الدراسة استخدام برنامج Hydro – IIP حيث أنه البرنامج المستخدم من قبل وزارة الموارد المائية والرى، والمستخدم فى التصميم الهيدروليكى وحساب مناسيب المياه والتصرفات والسرعات على أساس محاكاة السريان المستقر والغير مستقر فى القنوات المائية المكشوفة. وقد تم خلال الدراسة تطبيق برنامج Hydro – IIP على ترعة بسنتواى وفروعها وذلك لدراسة بعض مقترحات التشغيل مثل تأثيرالتخزين بفروع الترعة وتأثير معدلات السحب المختلفة من الترعة مع عقد المقارنات بين المقترحات المختلفة. هذا بالاضافة الى التحقق من كفاءة أداء منشآت التحكم على الترعة عن طريق دراسة مناسيب المياه المقاسة فعليا على طول ترعة بسنتواى، والتى تمت بواسطة المركز القومى لبحوث المياه.

**Keywords:** Irrigation improvement, Continuous flow, Downstream control, Canal simulation, Water surface profile

## 1. Introduction

Water resources in Egypt are becoming scarce. Surface water resources originally from the Nile are now fully exploited. Satisfying future demands in Egypt depends on better utilization and efficient use of present water resources. Optimal water management is an essential pre-requisite for sustainable development of Egypt. Hvidt [1] stated that the future looks bleak if Egypt does not succeed in formulating and implementing a water policy which can match the limited freshwater supply to the increasing demand. The per capita water resources which are currently below 1000 cubic meters/capita/year are expected to drop to about 500 cubic meters/capita/year in 2025. And if the present management practices and cropping patterns prevail, this could mean that up to 60 % of the agricultural land will not be irrigated. Therefore, the Ministry of

Water Resources and Irrigation started developing new projects in order to save water and increase crop production. Among these projects is the Irrigation Improvement Project (IIP), the reuse of drainage or treated wastewater, and using modern irrigation systems such as drip, sprinkler and pivot irrigation.

The IIP was developed with the major objective of increasing irrigation water use efficiency and agricultural production. Metawie [2] and Kotb [3] summarized the broad objectives of the IIP as follows: Increasing agricultural production and farm incomes by improving the irrigation infrastructure, facilitating a more equitable distribution of water and improving on-farm water management. The main improvements have been introduced to the IIP through the improvements of the main and the tertiary irrigation systems. Bekheit [4] stated that the IIP proposed some improvements for the main system which includes introducing of continuous flow and downstream control systems, installing discharge regulators at the head of the delivery canals, providing the maximum feasible degree of overnight storage, and establishment of Water Boards. Whereas, El-Kashef [5] stated that the IIP proposed three alternatives to improve the existing situation at the tertiary level. They are; replacing the existing mesqas by low pressure pipelines, replacing the existing mesqas by raised-lined mesqas, and improving the existing low level mesqas.

## 2. Description of the IIP at the main system

### 2.1. Continuous flow system

The main key element of improving irrigation system strategy is to switch the operation of the distributary canals from rotational system to continuous flow. Easter et al. [6] stated that continuous flow means that the main canals are operated continuously with slow variations in flows. While according to Zaki and El-Quosy [7], with continuous flow water can be made available on 24 hours per day and during 30/31 days a month. The discharge in the canal may vary or change from season to season according to the crop water requirements. In principle, irrigation water flows also in mesqas continuously 24 hours per day. However in most cases farmers irrigate only 12 to 18 hours a day. Consequently, there must be a possibility to store water during the remaining hours of the day.

### 2.2. Downstream control

The downstream control system as stated by Bekheit [4] is a control system, in which the water level downstream of each water level regulator is maintained at a constant level. In downstream control, the adjustment goes from the downstream to the upstream end. In case the water level downstream of the cross regulator drops due to an increase in downstream demand, the gate should open for more discharge (automatic control gates are used). Due to the positive storage wedge, this increase in discharge happens immediately. This control system is generally found in main and secondary canals, but not in tertiary canals. Generally, downstream control requires horizontal embankments between the cross regulators to prevent overtopping in case of zero flow conditions.

### 2.3. Discharge regulators

Khalil [8] stated that the installation of discharge regulators on branch canals operating on a concept of water level sensitivity, control the amount of water released from the branch canals to smaller canals or ditches. Thereby, the exact amount of water is released, eliminating losses and avoiding the crop damage that results from excess water.

### 2.4. Night storage

Halcrow [9] concluded that under the IIP, the night portion of the released water which was usually lost to the drains via the tail escape due to the inclination of farmers to irrigate at night is now stored in the branch canals.

## 3. Description of Bisintaway command area

Bisintaway command area is located in Damanhour district in El-Beheira governorate. It runs about 8.915 km and serves a gross area of approximately 5500 feddans. It takes its water from El-Mahmoudia canal at km 16.330. Bisintaway branch canal is the first improved canal under the IIP, which is financed by different international agencies. Bisintaway command area is nearly flat with a gentle slope from south to north. Its levels range from about (3.00 m) above the mean sea level at the south of the command area near El-Mahmoudia canal, and the slope gradually decreases until it reaches a little below (1.00 m) at the north. The water duty for Bisintaway command area is based on the crop evapotranspiration (ET). The average water duty was estimated by the IIP directorate of El-Beheira governorate to be 35 $m^3$/fed/day and the average discharge entering Bisintaway branch canal is 2.19 $m^3$/sec.

The whole Bisintaway command area is served by eleven sub-branches on both right and left banks of Bisintaway branch canal, and 115 mesqas. There are also 17 pump sumps, which are considered direct irrigation offtakes. Bisintaway canal is a typical earthen canal. Its longitudinal slope varies from 5 cm/km starting from the intake of the canal up to km 5.000, and increases to 10 cm/km from km 5.000 till the end of the canal. Existing cross-sections have irregular shapes. Therefore, the side slope for the existing cross-sections cannot be determined exactly. The bed width of Bisintaway canal is very wide especially at the beginning of the canal. It reaches a width of about 7.0 m, then, it narrows gradually until it reaches a width of 1.5 m at the end of the canal. The sub-branches of Bisintaway canal have longitudinal bed slopes ranging from 29 cm/km in some sub-branches to 10 cm/km in others. Their cross-sections have irregular shapes and the bed widths about 1.0 m.

## 4. Description of the existing IIP at the main system of Bisintaway command area

For Bisintaway canal, the rotational flow is replaced by continuous flow system. This change in the operational procedure led to changing the ordinary sliding gates to new automatic gates to control and regulate the water levels and the flow through the canal. At the intake of Bisintaway canal, Avio 285/190 gate is installed downstream of the old head regulator at km 0.050, the old gate is considered now as an emergency gate. A discharge regulator 4300 l/s double baffle distributor is installed downstream of this Avio gate in order to control the passing discharge. At km 5.000 Avis 135/90 gate is installed downstream of the existing cross regulator. These Avio and Avis gates will react automatically to the demand. Discharge regulators (baffle distributors) are also installed at the sub-branches' offtakes. The cross-section of Bisintaway canal and its sub-branches are reshaped to contain a storage wedge. The bank levels are also redesigned for extra water storage. Accordingly, the water level is allowed to rise in the branch canal in case of no withdrawal or in case of maximum storage. Pumps are installed at all mesqa offtakes, from Bisintaway branch canal or its sub-branches. Also, direct irrigation offtakes are combined in 17 pump sumps.

## 5. Canal simulation model: HYDRO-IIP

HYDRO-IIP is a customized version of Mott MacDonald's general HYDRO-ID hydraulic simulation program, MacDonald [10]. It has been developed specifically for canal design and operation analysis under the Irrigation Improvement Project for the Egyptian Ministry of Water Resources and Irrigation, and is considered the main applicable program till now. Nour El-Din et al. [11] stated that the HYDRO-IIP program considers irrigation networks as nodes linked to each other by reaches. The continuity and the momentum equations are applied to each node and each reach. The equations are transformed to a nonlinear scheme using Preissman's procedure, which is a four-point implicit finite difference solution of the St. Venant equations. Then, a linear system of equations is driven which is solved simultaneously using a sparse matrix. The program simulates water levels, discharges and provides accurate and timely information for a variety of steady and unsteady flow conditions in canals and other open channel systems. It is a one dimensional flow simulation model coupled with a graphically oriented data management and visual display system, which guide the user through all stages required for setting up the mathematical model of a proposed or existing irrigation canal.

The basic assumptions associated with the use of HYDRO-IIP are:

- Flow is considered one dimensional.
- Depth and velocity vary only in the longitudinal direction.
- Wind and evaporation has a negligible effect on the water surface.
- The density of water remains constant and fluid is incompressible.
- The topography remains constant throughout the simulation.
- Manning's equation can be used to describe resistance effects.
- Water is free of sediments.

Meanwhile, the limitations of using this program are that it cannot be used in circumstances where wind effects are significant, the coarse cross-section spacing may cause instability, and the model is accurate for gradually varied flow only.

The model is built up by dividing the channel into a series of nodes and reaches. Data requirements comprise the channel cross-sections, inflow and outflow patterns at the nodal points, structure characteristics, dimensions and the boundary conditions. A range of irrigation structures are incorporated including various types of control gates, culverts, weirs and bridges. Moreover, the behaviour of the system is particularly sensitive to the outflow patterns which are derived from the farmers' irrigation practices in terms of duration, timing and application rate.

The model is used either to design a new canal system or to improve an existing one. Before running the model, it is essential to collect the basic information that describes the canal system under consideration. This information is prepared in the form of data files which are used to run the program. The model is firstly run in the steady state in order to create a backwater file (starting water levels) and, then, it should converge easily to transient mode which gives results consistent with the boundary conditions and abstractions specified. If problems arise, a warning message appears and data files are checked. The main output from the program is the longitudinal water level profile or waterline, superimposed on bed levels and banks for a selective series of consecutive nodes. It is possible to display the profile either in a step by step mode or to animate changes over a series of specified time steps. The different waterlines corresponding to different time steps are examined and analyzed carefully to specify two important waterlines which are the maximum and minimum waterlines. The maximum waterline corresponding to the maximum levels allowed in each reach, taking into consideration the levels of existing banks and surrounding land, is called the "bonded level" which represents the level of the gate pivot. The minimum waterline represents the minimum water levels required to command the offtakes to mesqa pump sumps and sub-branch canals corresponding to maximum water requirements. The hydraulic model is also used to select the optimum location, size and type of control structures in conjunction with allowable remodeling of cross-sections, to keep daily water level variations within this waterline envelope of levels. Fig. 1 summarizes the main components of the HYDRO-IIP.

## 6. Development of the IIP at the main system of Bisintaway command area

After applying continuous flow system, irrigation water flows continuously at all canal levels with great variation in water levels with time and distance. Since the flow is considered unsteady and non-uniform, the mathematical model, HYDRO-IIP model is used in this study to simulate Bisintaway branch canal. It is used to design the minimum and maximum water levels, bed levels, and to estimate the passing discharges and velocities along the canal starting from its intake till its tail escape.

Before running the simulation, basic information and data about the studied

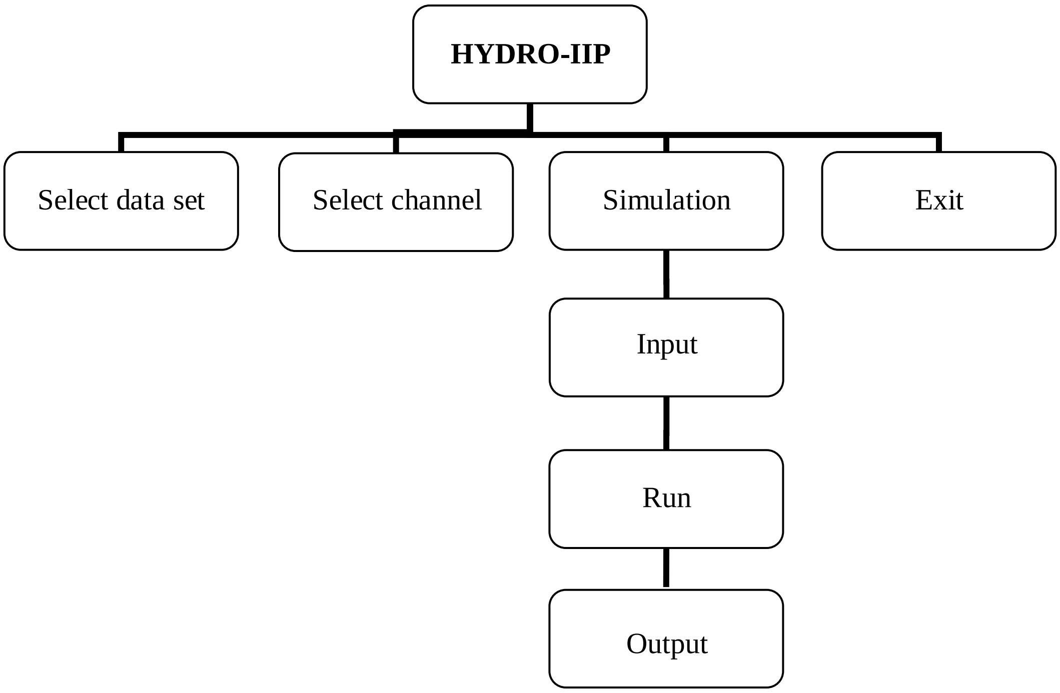


Fig. 1. Main components of HYDRO-IIP.

command area were collected accompanied by a location map and a layout map for the canal and its sub-branches and mesqas. Other information including cross-sections at different locations, longitudinal sections, as well as maximum and minimum water levels at all offtakes and structures along the canal were also collected. Moreover, data of all structures along the canal and their relevant hydraulic parameters, location of all sub-branches and mesqas with their levels, lengths and served areas and the water duty of the area were included. These information were obtained from the IIP directorate of Damanhour. The input data were organized in several files. The simulation starts by a steady state run in order to fix the water levels in the canal and to obtain an initial condition (canal full) for the unsteady state. Downstream control structures are, then, introduced to control the downstream water levels and discharges. Where, a downstream Avio gate has been introduced downstream of the old head regulator at km 0.050, and a discharge regulator 4300 l/sec double baffle distributor has been installed downstream of this Avio gate to control the passing discharge. At Km 5.000, an Avis gate has been introduced downstream of the existing cross regulator. These downstream control structures are introduced to the model by matrices representing the passing discharge according to the upstream and downstream water levels. A transient run is, then, carried out to find the outputs of the program. The outputs include the maximum water level (bonded level) based on the maximum allowable levels in each reach taking into consideration the levels of existing banks. It also includes the minimum water level representing the maximum demand requirements of the canal. Moreover, the maximum discharges and velocities along the canal at different time intervals are also included. The outputs from the program are analyzed and different operating scenarios are introduced.

### *6.1. Existing water surface profile*

The simulation is the one performed by the Improvement Sector to obtain the maximum and minimum water lines for Bisintaway canal and the discharges and velocities along the canal. Bisintaway branch canal, which is 8.915 Km long, has been divided into four reaches according to the designed bed widths, as shown in fig. 2 Three sections are considered in each reach. They are taken at the beginning, middle, and end of each reach. The minimum water depth ($y_{min}$), the maximum velocity ($v_{max}$) and the maximum discharge ($Q_{max}$) are estimated at each section.

Bisintaway canal is introduced to the model as a network of nodes and reaches as shown in Fig. 3, the sub-branches along the canal are considered as outflow points and their storage effects are not taken into consideration and, thus, neither nodes nor reaches are taken along them. Fig. 4 shows the obtained water surface profile for this simulation, where, the upper line represents the maximum waterline (bonded level) at Q = 0.0, whereas, the lower one represents the minimum waterline at Q = $Q_{max,}$ and in between is the storage wedge. Table 1 summarizes the results of this simulation, which are the minimum water depths, the maximum velocities and the maximum discharges at the different studied sections along the canal.

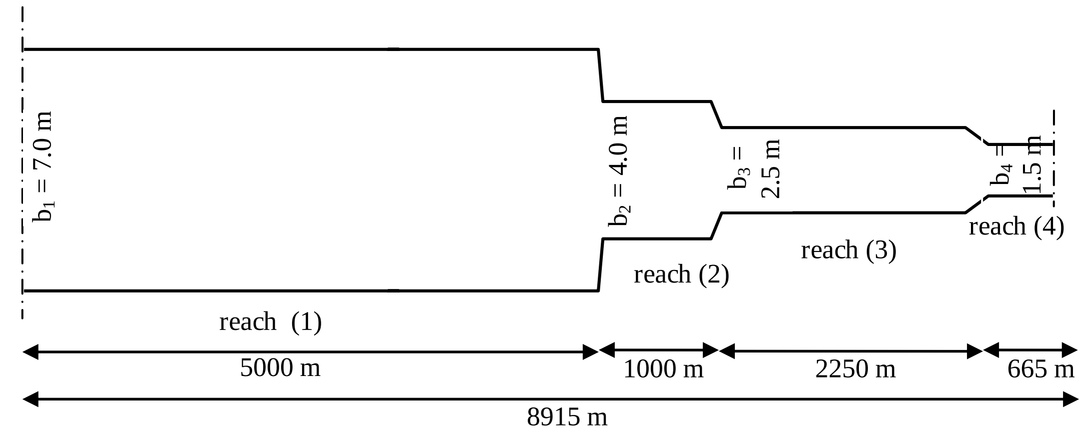


Fig. 2. Studied reaches along Bisintaway canal.

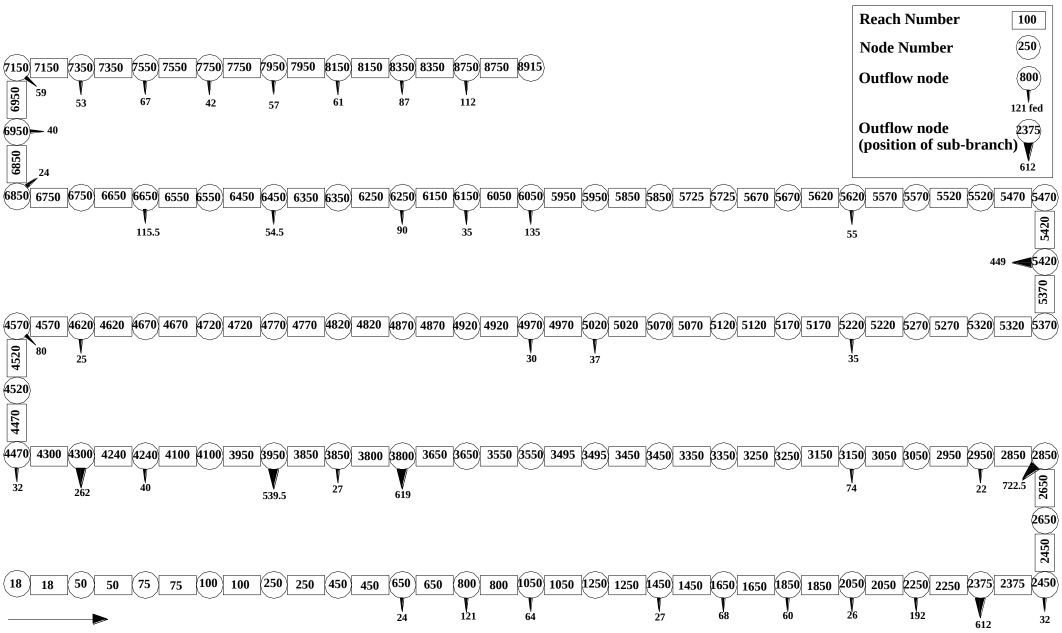


Fig. 3. Network for the nodes and reaches taken along Bisintaway canal.

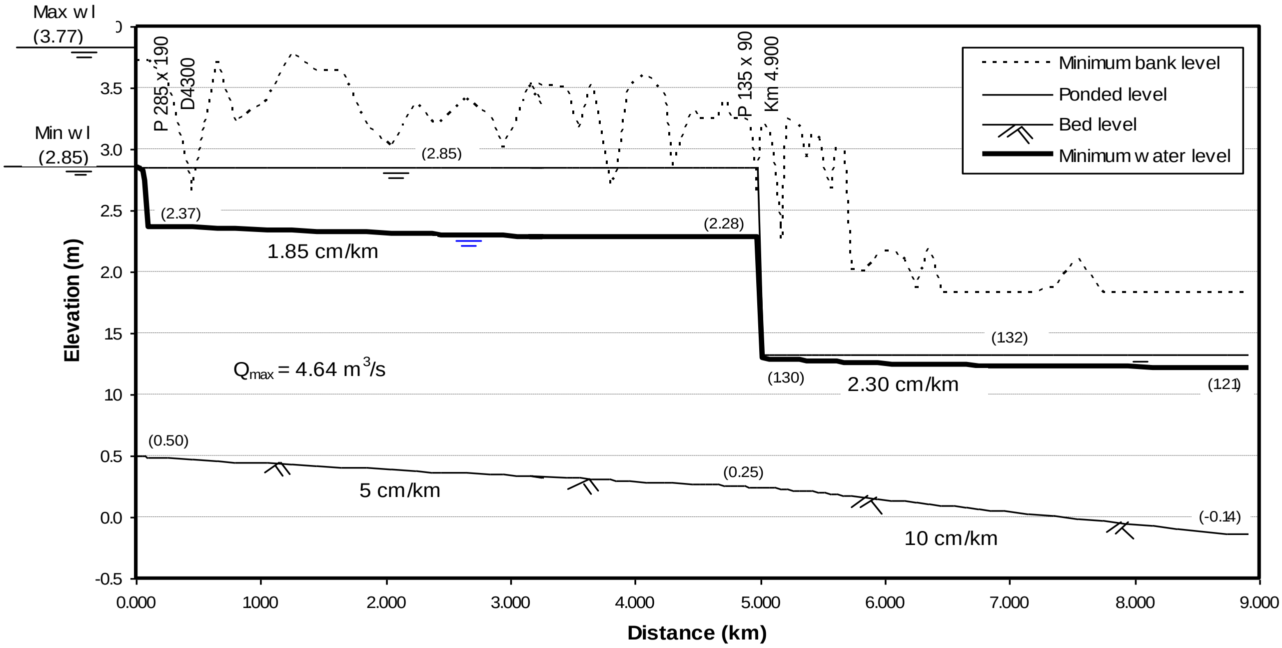


Fig. 4. Existing waterline of Bisintaway canal.

Table 1
Results of the simulation of the existing waterline

| Section | Reach (1) | | | Reach (2) | | | Reach (3) | | | Reach (4) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) |
| No.(1) | 1.88 | 0.65 | 4.64 | 1.05 | 0.27 | 1.44 | 1.11 | 0.20 | 0.82 | 1.31 | 0.03 | 0.10 |
| No.(2) | 1.96 | 0.16 | 2.90 | 1.06 | 0.18 | 1.00 | 1.16 | 0.13 | 0.55 | 1.35 | 0.00 | 0.00 |
| No.(3) | 2.03 | 0.08 | 1.47 | 1.11 | 0.17 | 0.95 | 1.29 | 0.04 | 0.18 | 1.35 | 0.00 | 0.00 |

### *6.2. Effect of sub-branches storages on the water surface profile*

In this simulation, the six governorate sub-branches of Bisintaway canal are taken into consideration to study their storage effect on the waterline. Each sub-branch is divided into a number of nodes and reaches. Outflow points are taken along the relevant nodes. The simulation is carried out and the effect of the sub-branches storage capacities on the minimum waterline is presented in fig. 5. It is noticed that the waterline is quite higher than in the existing case, where, the effect of the storage in the sub-branches is neglected. Hence, it can be concluded that the storage capacities of the sub-branches increases the over-all storage capacity of the canal, and, this leads to a higher waterline. Table 2 summarizes the results of this simulation. Taking the effect of sub-branches into consideration leads to increasing the minimum water depth by a maximum of 3.00%, It is also noticed that this simulation decreases the estimated velocities along the canal, where, the velocities decreased by a maximum 6.15%. Moreover, the passing discharges through the studied sections have, also, decreased, where, the maximum design discharge decreased by approximately 7.3% because the water stored in the sub-branches during the non-irrigation periods is used during the irrigation periods, which, in turn decreases the passing discharges through Bisintaway canal itself.

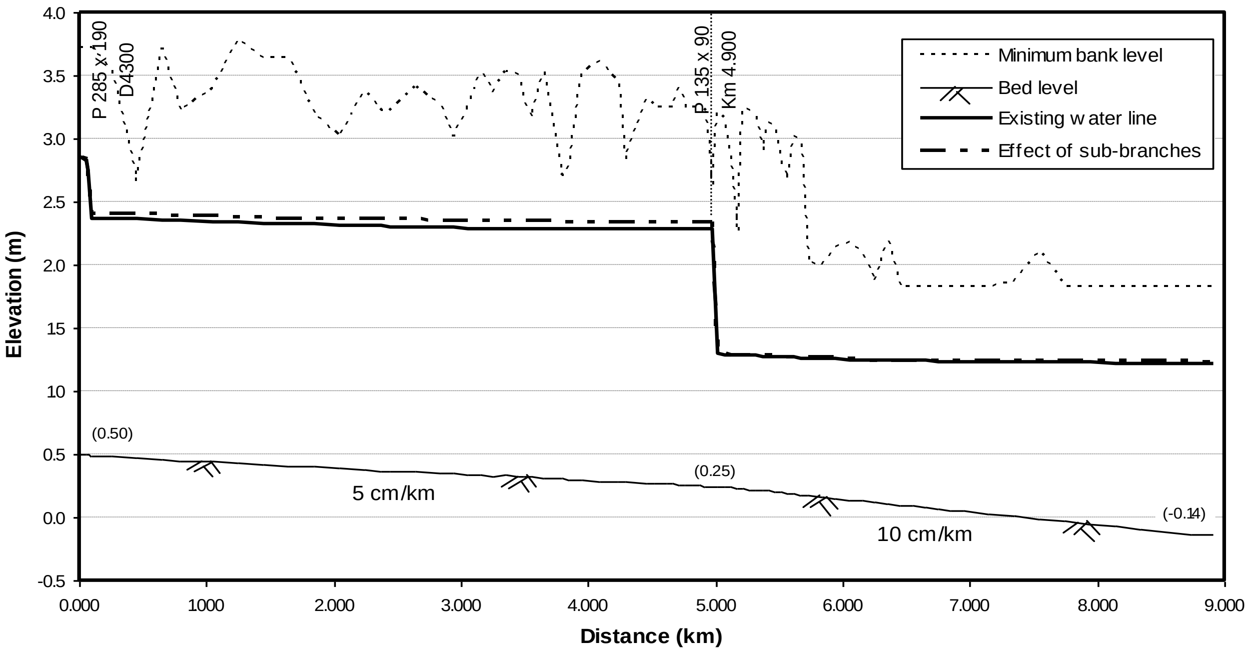


Fig. 5. Effect of storages in sub-branches on the waterline.

Table 2
Comparison between the existing simulation and the effect of storages in sub-branches

| Cases | Section | Reach (1) | | | Reach (2) | | | Reach (3) | | | Reach (4) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) |
| Existing | No.(1) | 1.88 | 0.65 | 4.64 | 1.05 | 0.27 | 1.44 | 1.11 | 0.20 | 0.82 | 1.31 | 0.03 | 0.10 |
| | No.(2) | 1.96 | 0.16 | 2.90 | 1.06 | 0.18 | 1.00 | 1.16 | 0.13 | 0.55 | 1.35 | 0.00 | 0.00 |
| | No.(3) | 2.03 | 0.08 | 1.47 | 1.11 | 0.17 | 0.95 | 1.29 | 0.04 | 0.18 | 1.35 | 0.00 | 0.00 |
| Effect of sub-branches | No.(1) | 1.92 | 0.61 | 4.30 | 1.05 | 0.27 | 1.44 | 1.12 | 0.20 | 0.82 | 1.33 | 0.03 | 0.10 |
| | No.(2) | 2.02 | 0.15 | 2.72 | 1.07 | 0.18 | 1.0 | 1.18 | 0.13 | 0.55 | 1.37 | 0.00 | 0.00 |
| | No.(3) | 2.09 | 0.08 | 1.47 | 1.12 | 0.17 | 0.95 | 1.31 | 0.04 | 0.18 | 1.37 | 0.00 | 0.00 |

### *6.3. Effect of using different abstraction patterns on the water surface profile*

The HYDRO-IIP model allows daily abstraction patterns to be entered from which nodal outflow files are automatically set up according to specified command area and water duty. A standard set of pattern files are available. These abstraction files are based on an assumed distribution of 16 hours "on" and 8 hours "off". Where farmers start to irrigate at 8.00 am and continue till 12.00 pm. The peak hour is noticed at 4.00 pm as shown in fig. 6. This pattern takes a sine curve shape. In this study, seven scenarios are presented to observe the effect of changing the abstraction patterns on the water surface profile. In these simulations, the patterns are considered to take a uniform shape and in each case the number of "on" hours is changed starting from 8 hours "on" till 20 hours "on".

Fig. 7 shows the effect of using different uniform patterns on the waterline. Only four patterns are shown and compared to the existing case. These patterns are: 8, 10, 12, and 20 hours "on", respectively. Whereas, the effect of the other studied patterns lied within the chosen patterns. It is clear from the figure that decreasing the number of the "on" hours (10 hours "on" and below) leads to lowering the waterline, since the same amount of water is withdrawn from the canal but in a shorter period of time.

Whereas, increasing the number of "on" hours (12 hours "on" and more) leads to increasing the waterline than the existing case. This means that the number of "on" hours can be decreased and give similar waterline under the condition that farmers withdraw water on uniform basis. It is also noticed that the waterline due to the existing case lies within the waterlines due to uniform 10 hours "on" and uniform 12 hours "on" patterns. Table 3 summarizes the results of the four studied uniform patterns simulations. Great variations in the water depths, velocities and discharges are noticed along the different studied sections. Where, The minimum water depths decreased by a maximum of 25.12% and 4.08%, the velocities increased by a maximum of 87.5% and 18.75%, and the maximum design discharges increased by 26.72% and 10.13% for the cases of 8 and 10 hours "on" , respectively. Meanwhile, The minimum water depths increased by a maximum of 3.57% and 10.34%, the velocities decreased by a maximum of 12.5% and 50%, and the maximum design discharges decrease by 7.97% and 43.1% for the cases of 12 and 20 hours "on", respectively.

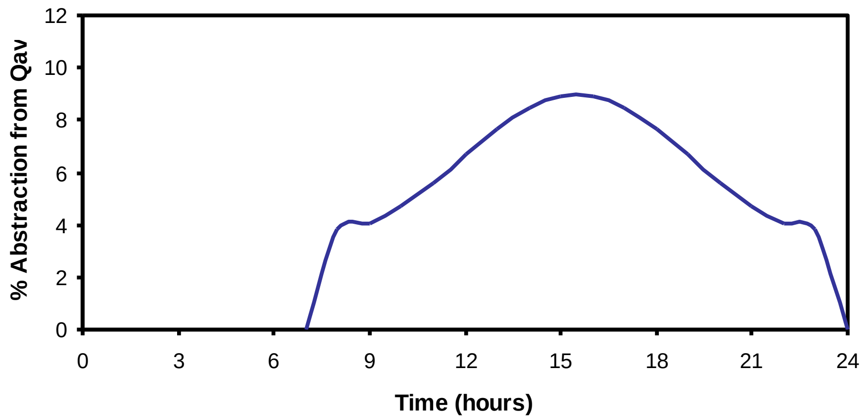


Fig. 6. Sine curve flow abstraction pattern.

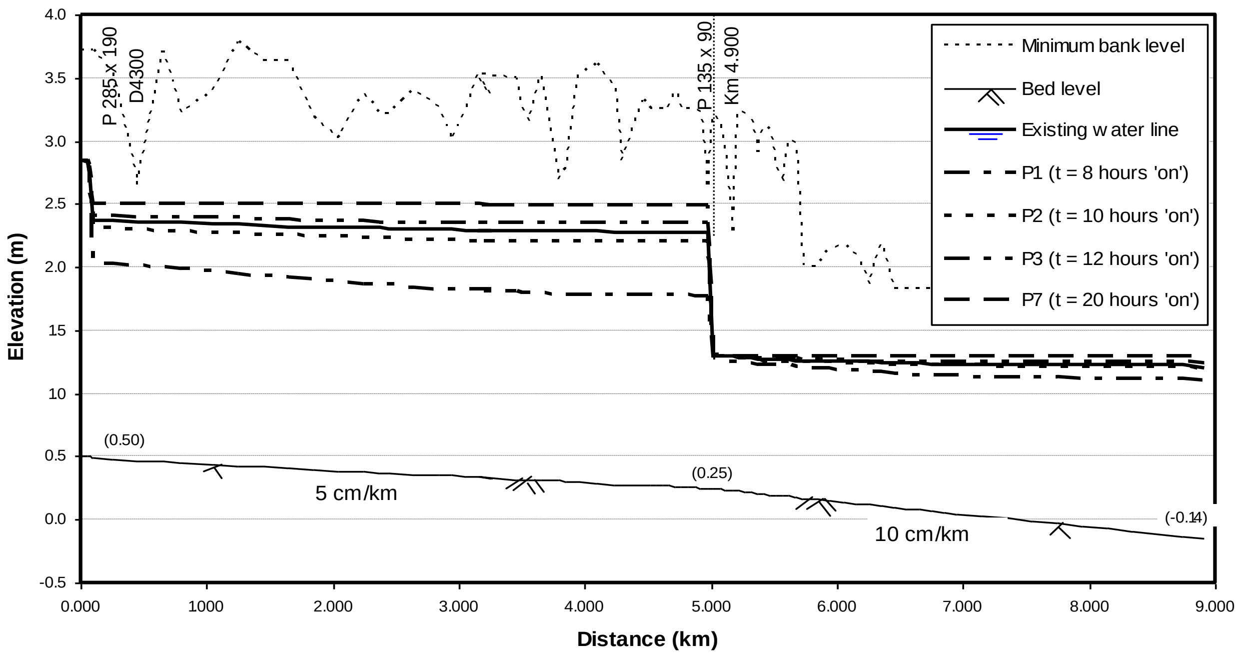


Fig. 7. Effect of using different uniform patterns on the waterline.

Table 3
Comparison between the existing simulation and the effect of different uniform abstraction patterns

| Cases | Section | Reach (1) | | | Reach (2) | | | Reach (3) | | | Reach (4) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) | $y_{min}$ (m) | $v_{max}$ (m/s) | $Q_{max}$ (m³/s) |
| Existing | No.(1) | 1.88 | 0.65 | 4.64 | 1.05 | 0.27 | 1.44 | 1.11 | 0.20 | 0.82 | 1.31 | 0.03 | 0.10 |
| | No.(2) | 1.96 | 0.16 | 2.90 | 1.06 | 0.18 | 1.00 | 1.16 | 0.13 | 0.55 | 1.35 | 0.00 | 0.00 |
| | No.(3) | 2.03 | 0.08 | 1.47 | 1.11 | 0.17 | 0.95 | 1.29 | 0.04 | 0.18 | 1.35 | 0.00 | 0.00 |
| 8 hours "on" | No.(1) | 1.55 | 0.83 | 5.88 | 1.03 | 0.37 | 1.91 | 1.05 | 0.29 | 1.09 | 1.21 | 0.04 | 0.14 |
| | No.(2) | 1.49 | 0.26 | 3.72 | 1.02 | 0.25 | 1.32 | 1.08 | 0.19 | 0.74 | 1.25 | 0.00 | 0.00 |
| | No.(3) | 1.52 | 0.15 | 1.96 | 1.05 | 0.23 | 1.25 | 1.19 | 0.06 | 0.25 | 1.25 | 0.00 | 0.00 |
| 10 hours "on" | No.(1) | 1.83 | 0.72 | 5.11 | 1.04 | 0.29 | 1.55 | 1.10 | 0.23 | 0.92 | 1.30 | 0.03 | 0.13 |
| | No.(2) | 1.88 | 0.19 | 3.14 | 1.05 | 0.20 | 1.08 | 1.15 | 0.15 | 0.64 | 1.34 | 0.00 | 0.00 |
| | No.(3) | 1.95 | 0.09 | 1.59 | 1.10 | 0.19 | 1.05 | 1.28 | 0.05 | 0.23 | 1.34 | 0.00 | 0.00 |
| 12 hours "on" | No.(1) | 1.92 | 0.60 | 4.27 | 1.05 | 0.25 | 1.39 | 1.13 | 0.19 | 0.76 | 1.34 | 0.03 | 0.10 |
| | No.(2) | 2.03 | 0.14 | 2.64 | 1.07 | 0.17 | 0.93 | 1.18 | 0.12 | 0.53 | 1.38 | 0.00 | 0.00 |
| | No.(3) | 2.10 | 0.07 | 1.44 | 1.12 | 0.15 | 0.87 | 1.32 | 0.04 | 0.18 | 1.38 | 0.00 | 0.00 |
| 20 hours "on" | No.(1) | 2.02 | 0.36 | 2.64 | 1.06 | 0.17 | 0.89 | 1.16 | 0.12 | 0.66 | 1.38 | 0.02 | 0.09 |
| | No.(2) | 2.16 | 0.08 | 1.95 | 1.09 | 0.11 | 0.73 | 1.22 | 0.08 | 0.50 | 1.42 | 0.00 | 0.00 |
| | No.(3) | 2.24 | 0.04 | 0.91 | 1.15 | 0.10 | 0.70 | 1.36 | 0.02 | 0.15 | 1.42 | 0.00 | 0.00 |

## 7. Performance of Bisintaway canal

In order to calibrate the hydraulic simulation model (HYDRO-IIP) and check the actual performance of the canal, field data are necessary to compare between the measured and the simulated design parameters. Field data representing the measured water levels along Bisintaway canal were monitored at eight locations by the National Water Research Center during the months June, July, August and September of summer 2005. These locations are; upstream the old head regulator (at km 16.330 of El-Mahmoudia canal), upstream and down stream the Avio gate at km 0.050, downstream the distributor, at km 2.050, upstream and downstream the cross Avis gate at Km 5.000, and at the tail escape. The measured water levels were recorded every hour during the 24 hours of the day of each month.

### *7.1. Status of the old head regulator*

Fig. 8 presents the water levels measured upstream and downstream the old head regulator during summer 2005. From this figure, it can be noticed that the measured water levels are within the maximum and minimum recorded levels of El-Mahmoudia canal at km 16.330 during the last five years.

### *7. 2. Status of the Avio downstream water level control gate*

Fig. 9 presents the measured water levels upstream and downstream the Avio 285/190 control gate at km 0.050. It is noticed that the Avio gate is controlling water during the period of the first till the third of June, where, the water levels are below the designed bonded water level, which is (2.85 m). During the period from the fourth of June till the twenty-fourth of August, the Avio gate is fully opened and the measured water levels, upstream and downstream the gate, are almost identical. Whereas, after the twenty-fourth of August till the end of September, the gate is controlling water but with the downstream water levels being higher than the designed bonded water level by 13 to 42 cm.

### *7.3. Status of the double baffle distributor*

Fig. 10 presents the water levels upstream and downstream the double baffle distributor. It can be observed that the difference between the water levels upstream and downstream the distributor has increased with great fluctuation from time to time during the period when the Avio gate is fully opened and not controlling the water levels (fourth of June till the twenty-fourth of August). The MWRI staff reported that during this period not all the vents of the distributor were opened and water was passing over the distributor.

Whereas, during the periods when the Avio gate is controlling the water levels, the difference between the water levels upstream and downstream the distributor has ranged from 23 to 65 cm and from 28 cm to 1.0 m during the period from the first till the third of June and from the twenty-fifth of August till the end of September, respectively. These values indicate the losses through the distributor, where, no water is overtopping the vents.

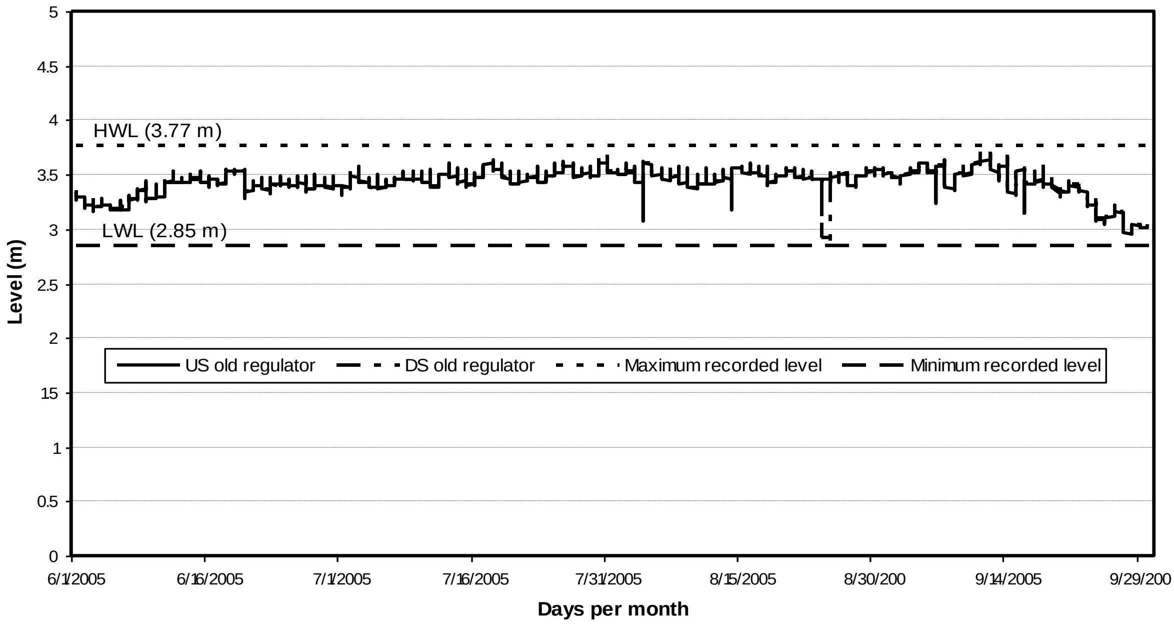


Fig. 8. Measured water levels upstream and downstream the old head regulator.

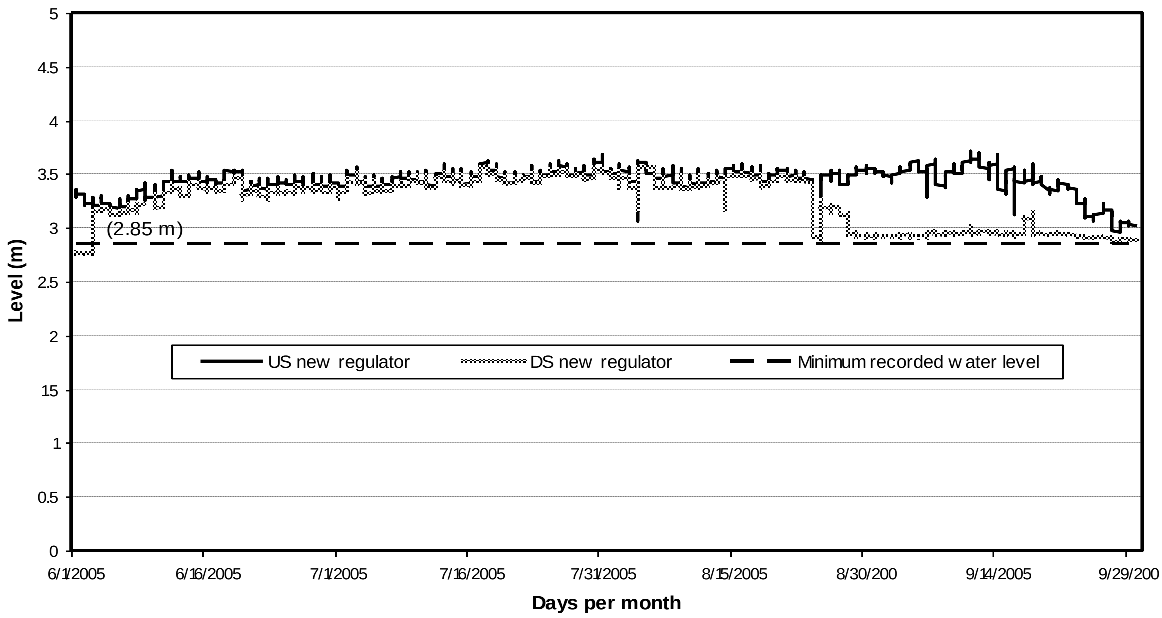


Fig. 9. Measured water levels upstream and downstream the Avio gate at Km 0.050.

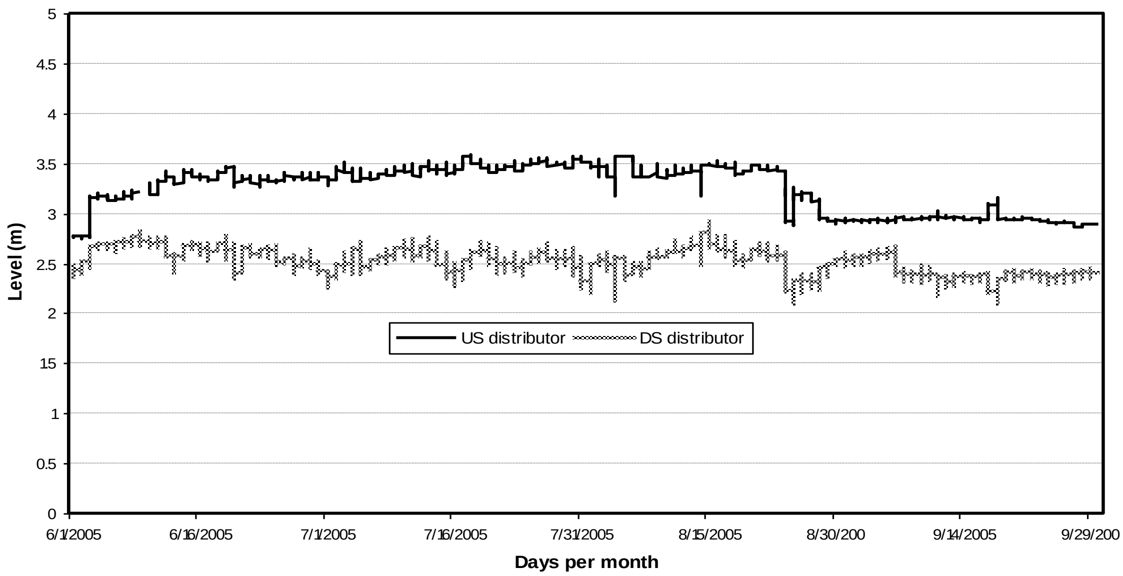


Fig. 10. Measured water levels upstream and downstream the double baffle distributor.

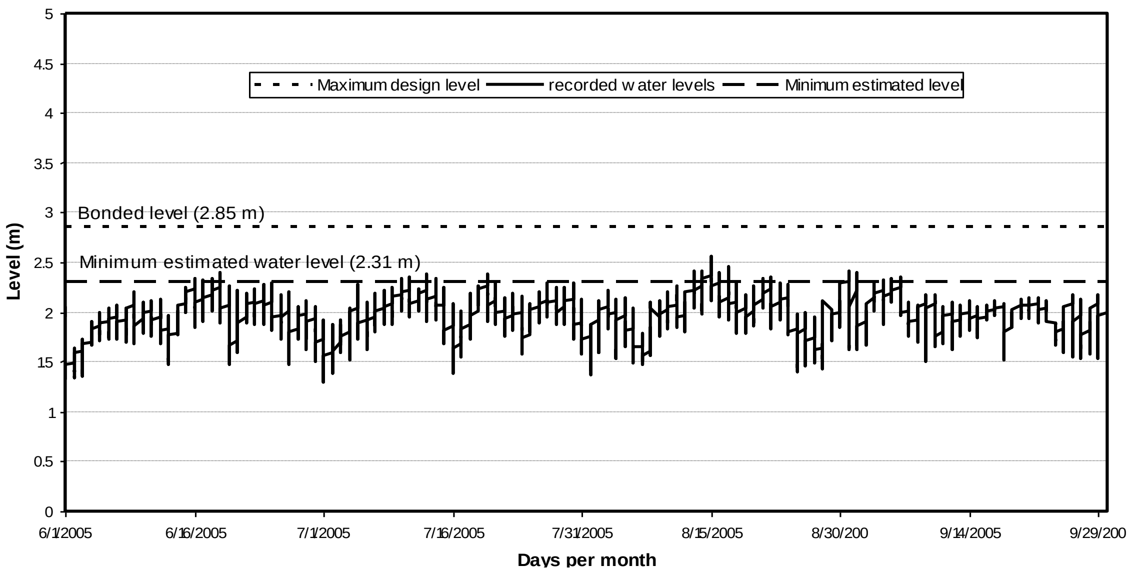


Fig. 11. Measured water levels at km 2.050.

### *7.4. Status of the point at km 2.050*

Fig. 11 presents the measured water levels at km 2.050. It can be observed that all the water levels lie below the maximum designed bonded water level (2.85 m). However, most of the measured levels, except for few exceptions, lie below the minimum water level estimated by the HYDRO-IIP simulation model. This can be due to the fact that the actual abstraction patterns are different from the pattern used in the program (sine curve).

### *7.5. Status of the Avis downstream water level control gate*

Fig. 12 presents the water levels upstream and downstream the Avis 135/90 control gate at km 5.000. It can be noticed that the downstream water levels are within the design bonded water level (1.32 m) for most of the time, except for some exceptions when the water levels exceeds the maximum design value. This increase in water levels is due to the fact that the gate was never being fully closed, due to the presence of big amount of trash upstream the gate.

*7.6. Status of the tail escape*

Fig. 13 presents the water levels measured at the tail escape. It can be observed that all the measured water levels are below the sill level design value (1.40 m) for approximately all of the time. No water is spilled in the drain. This is because; the measured water levels are recorded during the summer, the season of maximum water demand requirements.

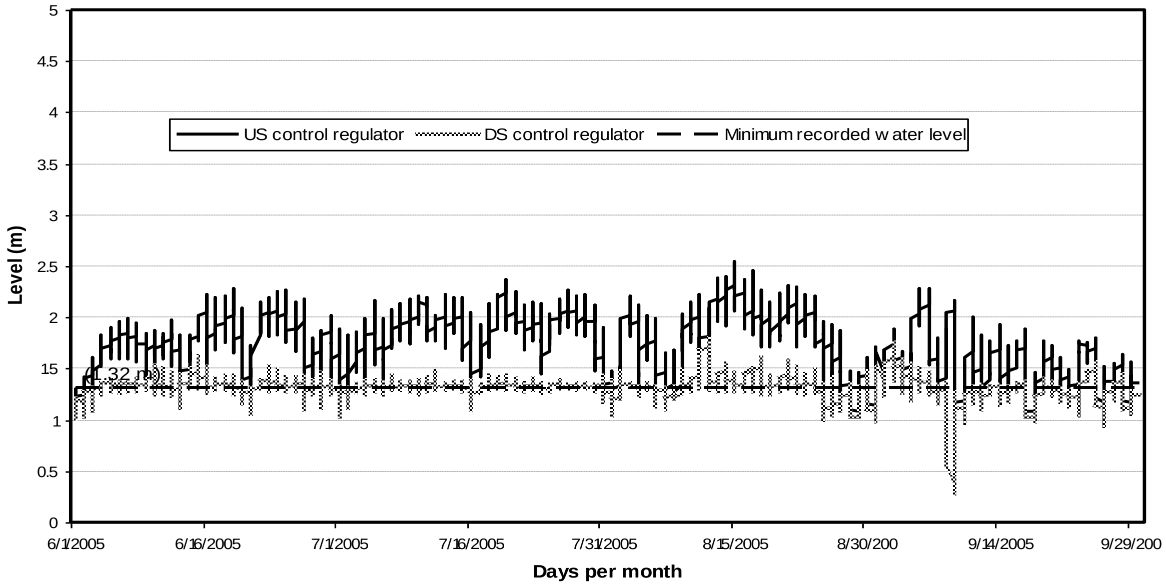


Fig. 12. Measured water levels upstream and downstream the Avis gate at km 5.000.

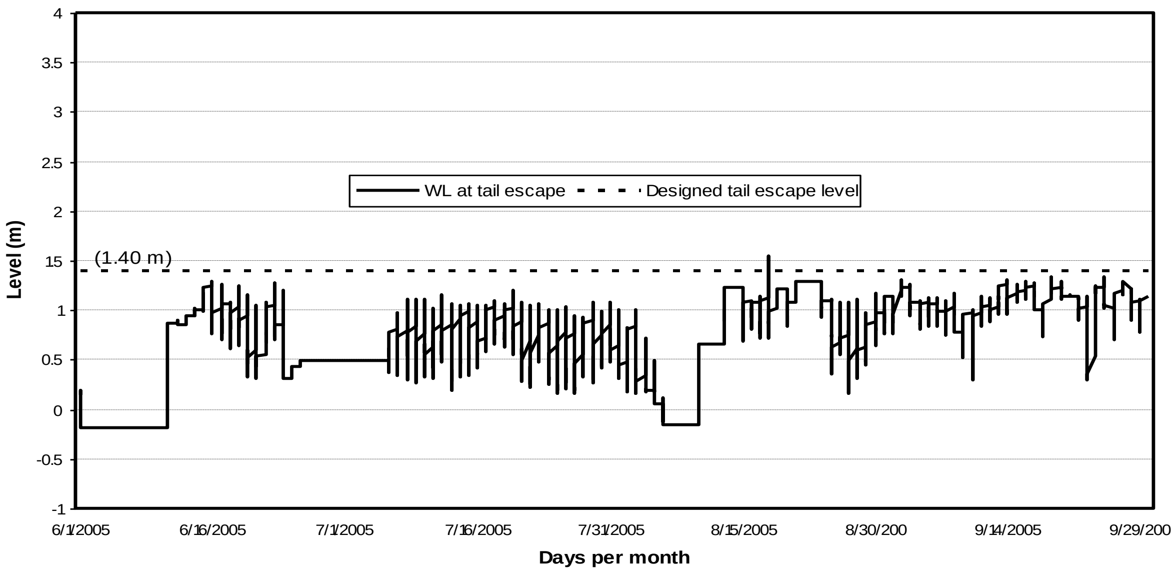


Fig. 13. Measured water levels at the tail escape.

## 8. Summary and conclusions

The main objective of this study is to focus on developing the IIP at the main system of Bisintaway canal, which in turn will improve the performance of the IIP in the future stages. Bisintaway command area, which is one of the pilot areas under improvement by the IIP in El-Beheira governorate, was selected as the case study to collect the required data, to carry out the analysis and to investigate the suggested operation scenarios and design criteria. The HYDRO-IIP canal simulation model, currently used by the MWRI in the design and simulation of the branch canals, was introduced in this study to simulate the operation of Bisintaway canal under different proposed scenarios.

According to the different operation scenarios introduced to the main irrigation system of the case study, the following conclusions can be introduced:

1. Introducing the storage capacities in the sub-branches of Bisintaway canal leads to higher water levels in the canal and decreases the passing velocities and discharges. The sub-branches storages increase the over-all storage capacity of the canal.
2. Introducing uniform abstraction patterns based on assumed distribution of 10 hours “on” and below leads to lowering the water levels and increasing the passing velocities and discharges than the existing case (16 hours "on"). Meanwhile, introducing uniform abstraction patterns of 12 hours "on" and above leads to higher water levels and decreases the passing velocities and discharge. Moreover, the existing waterline lies within 10 and 12 hours “on” uniform patterns
3. The behaviour of farmers and their withdrawal of water from the canal have a great effect on the waterline, passing velocities and discharges through the canal. Questionnaires should be held to study the behavior of farmers and their habits of irrigation period and time before applying the model to any canal.
4. The Avio gate at the head of Bisintaway canal is operating well and efficient at the first and last periods of the summer. The sudden change in its status is probably due to human interference. The distributor downstream the Avio gate is not operating well for most of the studied period with water passing over its vents. The Avis gate at km 5.000 is working well for most of the summer with some exceptions. Also, the measured water levels at the tail escape are below the designed value of the sill level and no water is spilled to the drain.

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