A modified design of el-Mahrousa canal under continuous flow system using SOBEK

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Simulation models for unsteady open channel flows have been commercially available for more than 2 decades. One of the famous commercially available unsteady-flow simulation software package is: SOBEK from Delft Hydraulics. In this paper, the main features of this unsteady-flow simulation package to control open channel flows are described. The proposed Software package is used to assess the impact of water allocation on El-Mahrousa branch canal. The actual cross sections of the canal under continuous flow after implementation of Irrigation Improvement Project (IIP) are checked as an initial scenario. Two proposals are investigated to indicate the use of the model. The proposed solutions are compared from technical and economical point of views. Also, the suitable design of the control structures which achieve water saving are proposed. Finally, the study approaches canal operation under maximum water requirement.

لاشك أن الطفرة التي يشهدها مجال الذكاء الاصطناعي في الآونة الأخيرة قد أثرّت تأثيراً ايجابياً في مجال التطبيقات الهندسية، حيث أن البرامج المتاحة حالياً ومنذ حوالي عقدين من الزمان لمحاكاة السريان في القنوات الترابية المكشوفة قد أصبحت متاحة لجميع الباحثين والمهتمين بهذا المجال، ومن اشهر البرامج المستخدمة لمحاكاة السريان داخل القنوات الترابية برنامج السوبيك، وقد تم استخدام الإصدار رقم ٢،١١ من هذا البرنامج لمحاكاة السريان داخل ترعة المحروسة وهى احد الترع التي يتم تغذيتها من ترعة الكانوبية والتي يجرى حالياً تطويرها بمعرفة قطاع تطوير الري ضمن مشروع تطوير الرى الثاني (IIP2)، حيث تم دراسة القطاعات الفعلية الموجودة لمعرفة هل تستطيع تحمل التصرفات المطلوبة تحت نظام التيار المستمر، كما تم وضع تصورين لتطوير الترعة تحت تأثير نفس النظام، وقد تمت مقارنة التصورات من عدة أوجه واقترح الباحث التصور الأفضل كما تم شرح سريع لخصائص البرنامج المستخدم وعرض أهم المعادلات المستخدمة في عملية المحاكاة.

Keywords: Unsteady flow, Open channels, Control systems, Simulation, SOBEK, IIP; El-Mahrousa canal and simulation.

1. Introduction

It is generally recognized that agriculture in Egypt consumes about 84 percent of the water used in the country [1]. This means that saving irrigation water has a great impact on water strategy in Egypt. After High Aswan Dam (HAD) construction, rotational irrigation system was applied in Egypt's old lands. This system has many defects such as; water shortage at canal tail, increasing of seepage, and overtopping especially at low level banks. Irrigation Improvement Project (IIP) is the promising solution to face irrigation water problems and old irrigation system disadvantages.

The program of irrigation improvement being carried out by the Ministry of Water Resources and Irrigation (MWRI) through its Irrigation Improvement Sector (IIS) aims to modernize Egypt's irrigation infrastructure by restructuring the tertiary system and changing the operation of the secondary system from rotational to continuous flow [7]. After a pilot phase supported by USAID, the first large-scale application of irrigation improvement started in 1996 under the World Bank/KfW-financed Irrigation Improvement Project, covering 248 000.0 feddan divided between three sub-project areas in the northern part of the Nile Delta: Mahmoudia (131 000 feddan) in Beheira Governorate; and Manaifa (42 000 feddan) and El Wasat (75 000 feddan) which are adjacent to each other in Kafr El Sheikh Governorate. The project components can be listed as: improvement of the delivery system through, the installation of new regulating structures on branch canals, the rehabilitation of canals, the provision of a telemetry system; and the installation of 11.0 drainage re-use pumping stations at selected sites.

The most existing problems which were faced after implementing irrigation improvement project are that pumps were established according to personal inclinations and the lack of an appropriate schedule between mesqas, in addition to the existing water shortage problem due to rice cultivation. These problems have a side effect in main canal, branch canal, and improved mesqas. It needs an effective tool to model the field situation and proposes the necessary decisions.

Simulation models for unsteady open channel flows have been commercially available for more than 2 decades. Most of these models are now available for personal computers and can be used to study the control of irrigation canals. Under IIP, hydraulic design of improved branch canals has been carried out using a version of Mott MacDonald's in-house simulation model, HYDRO, which was specially customized at the start of the project in 1996/97 to meet the specific requirements of IIP. This version of the model runs under DOS, which is not now available on most computers. Another limitation of HYDRO is that it does not have the facility to simulate the control algorithms which would be applied in the case of using automated gates with local controllers. To meet future modeling requirements, especially involving automatic control, it was decided to acquire the SOBEK software package developed by Delft Hydraulics of the Netherlands [9].

In the late 1980s, an ASCE task committee was formed to evaluate the various unsteady flow simulation models that were available for studying canal control methods. At that time, nearly all of these models were run on mainframe and minicomputers. By the time the task committee finished their work and published their results, many of these models were available for personal computers [3]. The committee was interested primarily in the models' ability to simulate water level and flow variations in canal systems with many gates and weirs. All of the available models adequately simulated water level response in these canals. CanalCAD [4] was the first unsteady flow simulation software that was developed primarily to test automatic canal-control algorithms. To date, only a few research groups have made use of this model for example, Parrish and Khalsa [8]; Burt et al. [2]; Wahlin and Clemmens [10].

In 1993, Delft Hydraulics introduced a new unsteady-flow simulation software package, SOBEK. It includes a link to MATLAB so that control decisions can be made within that framework. Water levels are passed to MATLAB and gate position changes are passed back to SOBEK. The control routines are written as MATLAB files. Recently, canal control studies have been conducted with the SOBEK–MATLAB combination by Delft Hydraulics [5].

The main objectives of this paper are to assess the impact of water allocation on El-Mahrousa branch canal using the unsteady-flow simulation software package (i.e. SOBEK). Also, the suitable design of the canal which achieve water saving was proposed. Additionally, this paper aims to check the actual cross sections of the canal under continuous flow after implementation of irrigation improvement project. This study approaches canal operation under maximum water requirement.

2. Site specifications and data

In order to assess the impact of water allocation on El-Mahrousa branch canal after implementing IIP2, the following paragraphs give full description of El-Mahrousa canal in terms of location, control structures, irrigation practices, crop pattern, and drainage system. It also includes the irrigation situation after applying IIP2 such as improved mesqas, pumps discharges, and new control structures.

2.1. Location

The command area of El-Mahmoudia canal is located near the northern edge of the west Delta. This canal runs for a distance of 77 km from the Rashid branch of the Nile down to the Mediterranean Sea at Alexandria. The total area irrigated from the canal is about 280 000 feddan. El-Mahmoudia canal receives water from three different sources. The main source is the El-Atf pumping station on the

Nile, which provides about 80% of the total annual supply to the canal. The secondary water sources are: Edko drainage reuse pumping station and the tail escape of the Khandak El-Sharqi canal.

El-Kanobia canal receives water from El-Mahmoudia canal at km 38.98 right side. The total area irrigated from El-Kanobia is about 29732 feddan. Its discharge and length are 1.05 Mm^3/day and 14.85km, respectively.

El-Mahrousa canal is fed from El-Kanobia canal at km 11.47 left side. The total area irrigated from El-Mahrousa is about 1520 feddan. Its length is 5.15 km. The command area of El-Mahrousa, which will be improved under the IIP2, is about 1262 feddan, see fig. 1. The data were collected by Ministry of Irrigation and Water Resources (WMRI)[6]. El-Mahrousa canal consists of main flow path and two Low Levels Mesqas (LLM). The first low levels mesqa is fed from El-Mahrousa at km 1.60 R-side (i.e. Abdel-Wikiel LLM), while the second Low level mesqa is fed at km 2.10 R-side (i.e. El-Hood LLM), see fig. 2.

2.2. Control structures

A head regulator controls the canal discharge. The Regulator is a single vent RC slab regulator, it is equipped with vertical lifting gates operated manually by a gatekeeper. The maximum and minimum design discharges are 0.07 Mm^3/day and 0.035 Mm^3/day, respectively. There are two RC slab bridges at km 0.625 and 1.53. In addition, the canal has two big low level mesqas. The two low level mesqas are fed from El-Mahrousa branch canal through steel pipe culverts. The collected data are as follows in table 1.

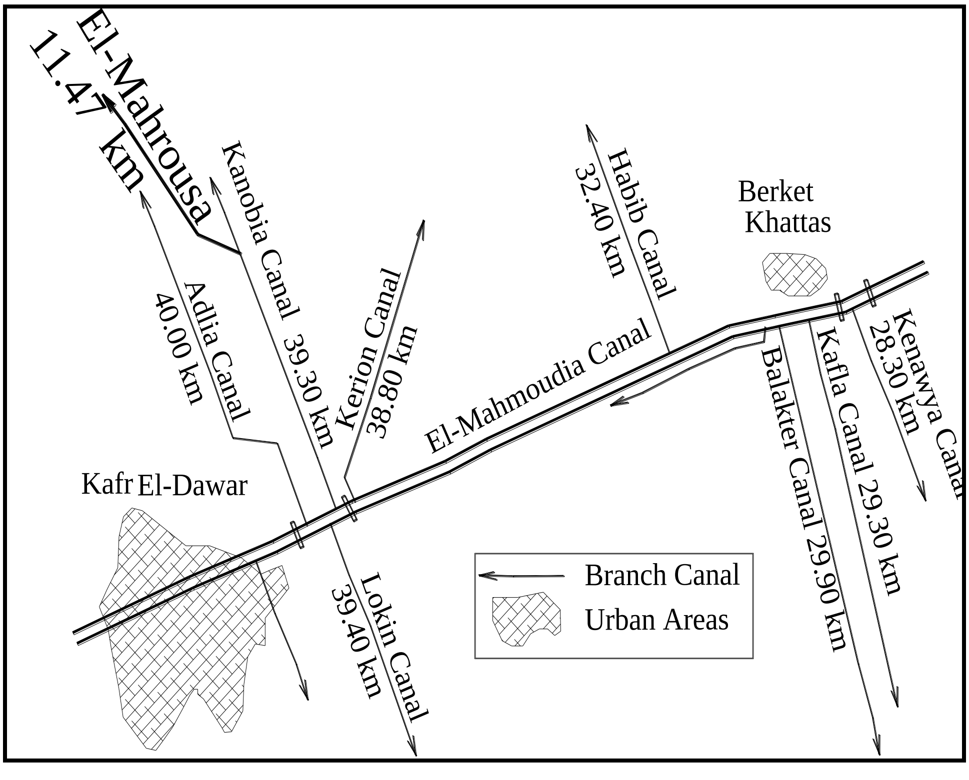

Fig. 1. Layout of El-Mahrousa canal.

Table 1
Control structures on El-Mahrousa

Structure	Location (km)	Bed level	Width (m)	Height (m)	Length (m)	Dia. (m)
H. Reg.	0.000	-0.80	2.0	1.60	20.0	×
Bridge (1)	0.625	-1.11	2.6	1.60	20.0	×
Bridge (2)	1.530	-1.54	1.0	2.0	12.0	×
Culvert (1)	1.600	-1.60	×	×	10.0	0.630
Culvert (2)	2.100	-1.55	×	×	10.0	0.630

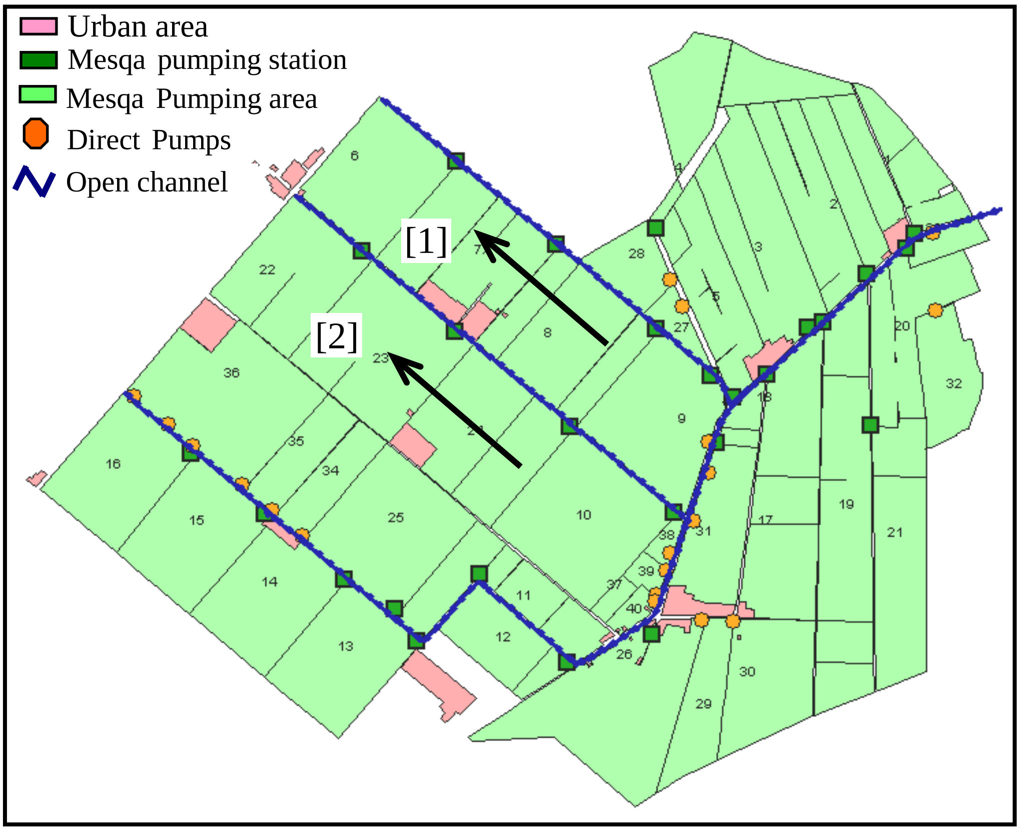

Fig. 2. A schematic view of the different flow paths [1] Abdel-Wikiel LLM [2] El-Hood LLM.

2.3. Crop pattern

The main summer crops in the command area of El-Mahrousa canal are rice, cotton, and maize. On the other hand, winter crops are clover, wheat, and sugar beets. A limited area is planted with summer and winter vegetables. Farmers are used to cultivate cash crops. The crop pattern is used for determining the actual needs for water. Really, the cultivated rice area is 32.23 % of the total area.

2.4. Irrigation practices

For earthen mesqas, every farmer had his legal point of lifting. He runs his pump when he wants to irrigate. But after the implementation of irrigation improvement project, a group of farmers cooperates with each other with single point lifting water from small pumping station and irrigate their fields by virtue schedule. Water is stored in the main canal during the time when no water is pumped. This encourages the farmers to irrigate at night.

2.5. Drainage system

On the right bank of the Mahmoudia canal, secondary drains discharge into the Moheet Edko drain which forms the north-east boundary of the project area, bordering Lake Edko. The Moheet Edko drain in turn discharges into the Edko drain and then via Edko pumping station into Abu Qir bay. Drainage water from the upper part of the Moheet Edko catchment is lifted by the Barseeq pumping station situated mid-way along the drain. Sub-surface drainage was already been installed over most of the sub-project area, apart from a very small area where it is considered that no sub-surface drainage is required.

3. Applied model

The unsteady-flow simulation software package, SOBEK by Delft Hydraulics 1993 is used for simulating the flow field. SOBEK is an integrated software package for river, urban or rural management. Seven program modules work together to give a comprehensive overview of waterway systems. Its integrated framework also means that SOBEK can link river, canal and sewer systems for a total water management solution. This program is the product line designed for simple and complex river systems and estuaries. It simulates the water flows, the water quality and morphological changes in river systems, estuaries and other types of alluvial channel networks. The networks can be branched or looped. SOBEK-River is able to work with complex cross-sectional profiles consisting of various sub-sections.

3.1. Fundamental equations

The flow in one dimension is described by two equations: the momentum equation and the continuity equation. The continuity equation reads as in eq. (1), while the momentum equation is presented by eq. (2).

$$\frac{\partial A_f}{\partial t} + \frac{\partial Q}{\partial x} = q_{lat}\,. \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A_f}\right) + gA_f\frac{\partial h}{\partial x} + \frac{gQ|Q|}{c^2RA_f} - W_f\frac{\tau_{ui}}{\rho_w} = 0.0. \tag{2}$$

where: A_f is the wetted area; q_{lat} is the lateral discharge per unit length [m^2/s]; Q is the discharge [m^3/s]; t is the time [s]; x is the distance [m]; g is the gravity acceleration [m/s^2] (g=9.81 m/s^2); h is the water level [m] (with respect to the reference level); C is the Chézy coefficient [$m^{1/2}/s$]; R is the hydraulic radius [m]; W_f is the flow width [m]; τ_{wi} is the wind shear stress [N/m^2]; ρ_w is the water density [kg/m^3].

The SOBEK-Flow-module uses the Chézy bed friction value in solving the water flow equations. After the actual value of the Chézy coefficient has been computed for the main channel and floodplains, the representative Chézy coefficient for the entire cross-section will be computed. The following equation is used for that purpose:

$$c = \frac{1}{A_f\sqrt{R}}\int_0^{W_L} c_y d_y \sqrt{R_y}\, d_y\,. \tag{3}$$

3.2. Sediment transport

A sediment transport formula can be selected, for instance Engelund-Hansen or Meyer-Peter-Müller. Often sediment transport formulas can be rewritten as a power law of the flow velocity:

$$s = au^b\,, \tag{4}$$

in which: s is the sediment transport per unit width, excluding pores [m^2/s], a is a coefficient [m^{2-b}/s^{1-b}], b is an exponent representing the degree of non-linearity [-] and u is the current velocity [m/s].

3.3. Bed level

For the bed level, the sediment transport balance in SOBEK is used for the total cross section:

$$\frac{1}{1-\varepsilon_p}\frac{\partial A}{\partial t}+\frac{\partial S}{\partial x}=0, \qquad (5)$$

in which: A is the area of cross section [m^2], S is the sediment transport through a cross section [m^3/s], p is the fraction of pores [-].

3.4. Flow chart of solution procedure

The overall procedure for solving the complete simulation SOBEK model of the problem may be introduced as a flow-chart, see fig. 3.

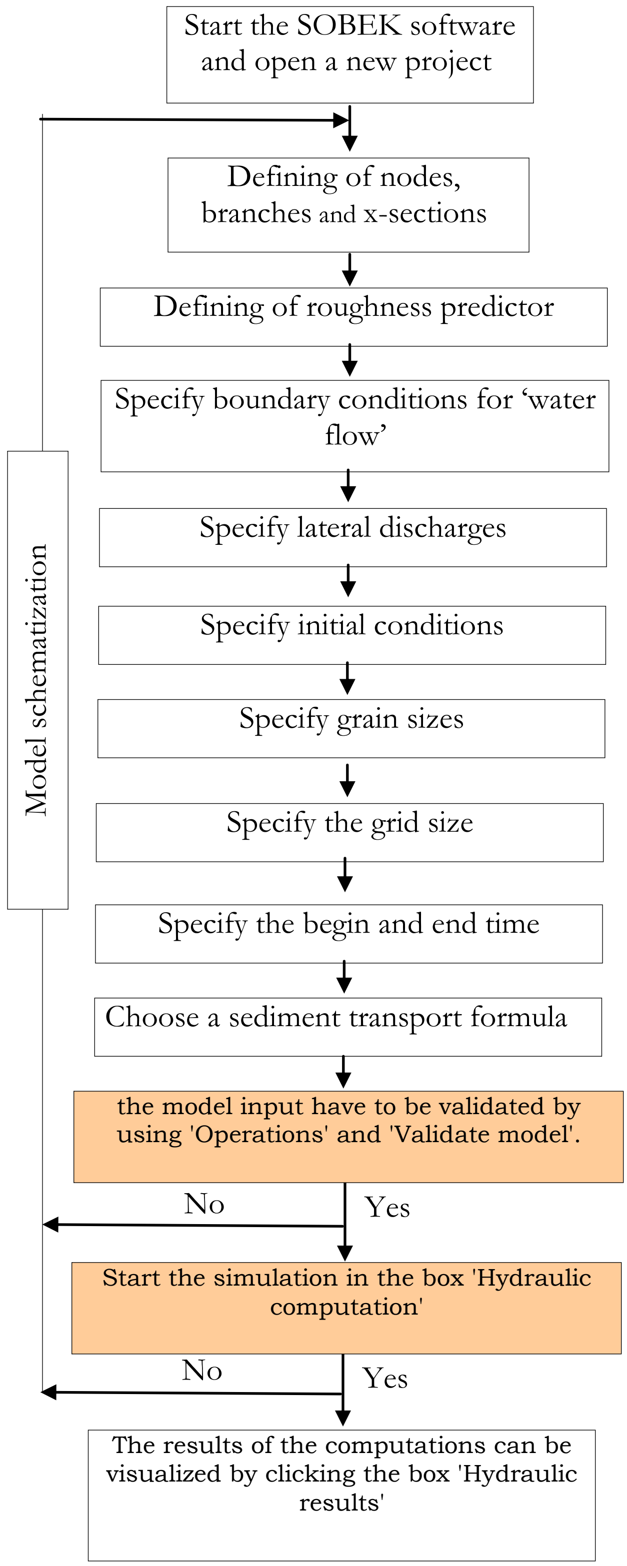

Fig. 3. The simulation flowchart.

4. Initial operation scenario

El-Mahrousa canal has been schematized based on an underlying GIS map of the area. This gives a better overview of the results and shows which low level mesqas are schematized in the model as side branches. These paragraphs give a description of how the schematization of El-Mahrousa was made. Firstly, in GIS a shape file was made in which all the areas, belonging to one mesqa pumping station or a direct extraction are joined. These new areas have a new number, which can be recognized also in the name of the lateral extractions in the model, see fig. 4. Structural data were collected about all structures through the studied canal.

4.1. Modeling of pumping stations

The maximum discharge of El-Mahrousa canal is about 0.07 Mm^3/day. It means that, maximum water duty in the command area equals 46.05 m^3/day/feddan. Actually, this value may be decreased to reach (35-40) m^3/day/feddan. The mesqa pumping stations are modeled in SOBEK as lateral abstractions. In the modeling file of El-Mahrousa canal, the water duty for each pump has been calculated. It is necessary to mention that, at some locations a few pumps must be added as one abstraction at a certain location in the model. Therefore, these areas are collected to a total water duty for this location. For each abstraction, the water duty per day is distributed over the day following the graph in fig. 5. In SOBEK the tables for each extraction point in this file are added to a lateral node.

4.2. Effect of initial water depth

A few runs are done with different initial water depths for El-Mahrousa canal. As soon as the initial water depth is lower than 0.3, the model will not run anymore, because there is not enough water available for all the pumping station. The minimum and maximum water levels for an initial water depth of 0.3 and 1.2 m are compared as side views along the canal and the low level mesqas.

The water surface profiles for the different proposed initial water depths are extracting from the model, these are made by exporting the resulting water levels for all the nodes for a certain day as "*.csv" file. In a developed spread sheet the Min. and Max. Values are calculated and compared to a file in which the side view is copied (in side view window of SOBEK: select copy data). The formulas in the excel file can also be used to make other side views. Fig. 6 shows the comparison between the water surface profiles for different proposed initial water depths in case of 100% abstraction for lateral points (i.e. all lifting points are working at the same time).

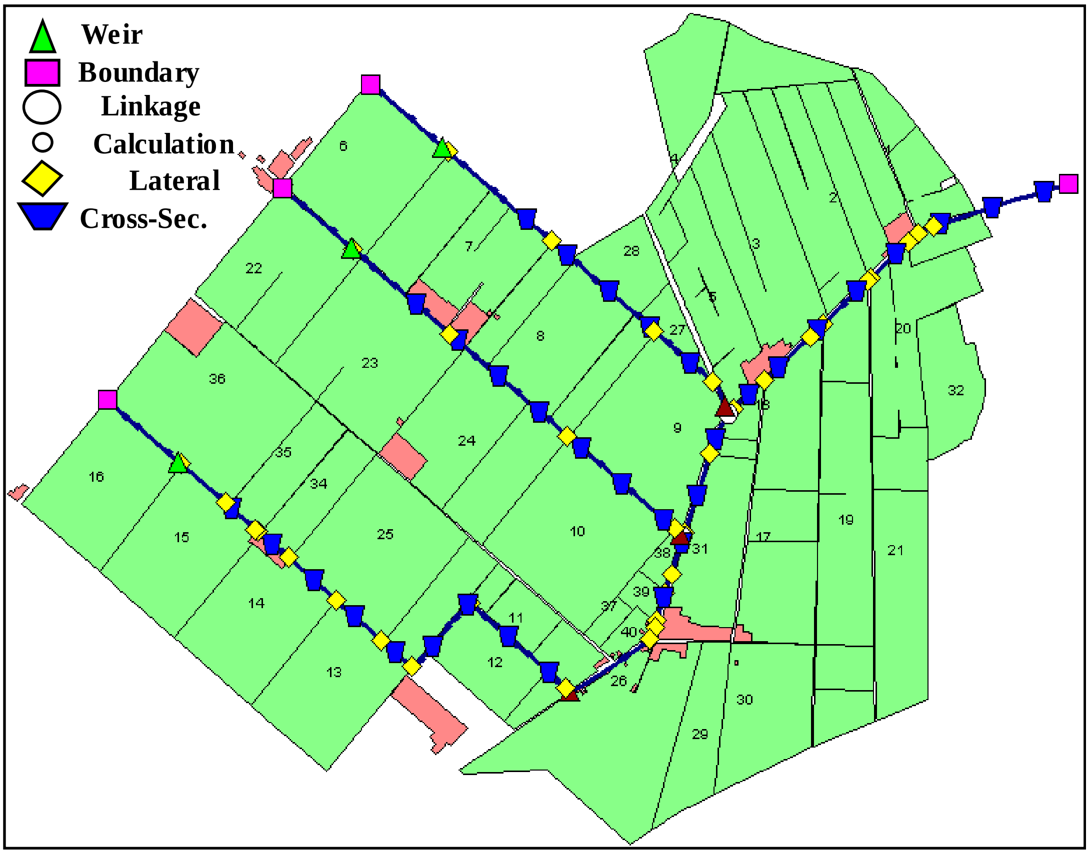

Fig. 4. Schematic map for the study area including digital nodes in SOBEK.

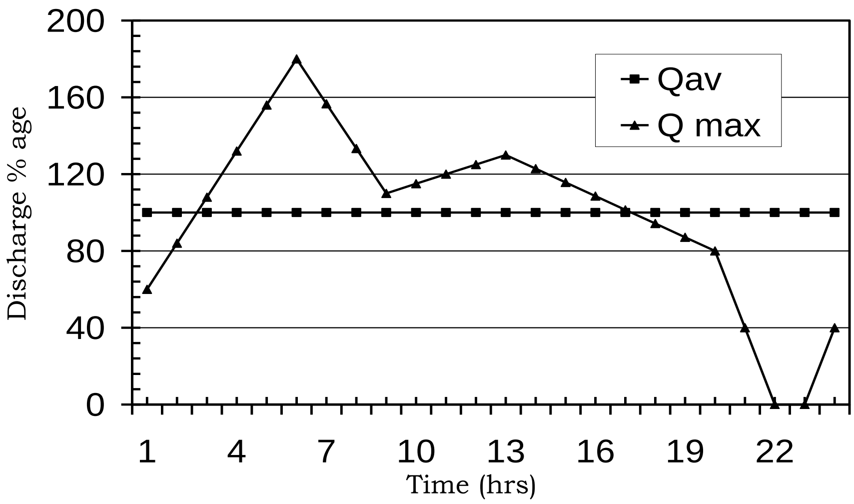

Fig. 5. Distribution of the abstraction over a day.

It can be seen that, there are unacceptable water levels, especially at the two low level mesqas. Also, it is found that, the difference between water levels for different initial water depths in low level mesqas is very small, see fig. 6-b and 60c. This is because the water levels are nearly the same upstream the intakes of low level mesqas for different initial water depths. The water surface is overtopping the bank in some cases. It can be said that, the overtopping may occur especially for low level mesqas.

It can be concluded that, applying the continues flow through El-Mahrousa canal under the existing situations will cause many problems. These problems include: overtopping of water in many reaches especially throughout the low level mesqas and increasing of the irrigation lifting height especially for the first reach of the canal itself.

4.3. Effect of abstraction percentage

It is important to make a simulation for some rare cases, in which the abstraction at lateral points is less than 100%. Fig. 7 shows the comparison between the water surface profiles in case of 90% and 75% abstraction at the different lateral points for the case of 1.2m initial water depth. It can be clearly seen that, there are unacceptable water levels in all reaches. The water surface exceeds the bank levels in all cases. It can be said that, the overtopping may occur.

5. Proposed solutions

As recognized from results of the initial scenario, the actual sections of El-Mahrousa canal are not sufficient to carry the required discharges under continuous flow. It means that, the area of IIP2 project in El- Mahrousa canal will suffer from a big problem. It can be said that, the escapes at the end of the side branches will escape big quantities of water.

The optimal scientific solution for the problem has depended on the investigation of the problem reasons. In addition, it has investigated the problem effects on area of IIP2 project and the necessary steps for the problem elimination. Proper solutions may be suggested and easily studied for the present problem to define the optimum proposal.

5.1. First proposal

In this proposal different cross sections are modified at different locations of both two low level mesqas and main flow path. It should be filled and/or cut depending on the required water surface level. Generally, the purpose of cutting is increasing the cross-section area to match the storage needs during the low abstraction of pump stations. In addition, the filling process is required to increase the bank levels.

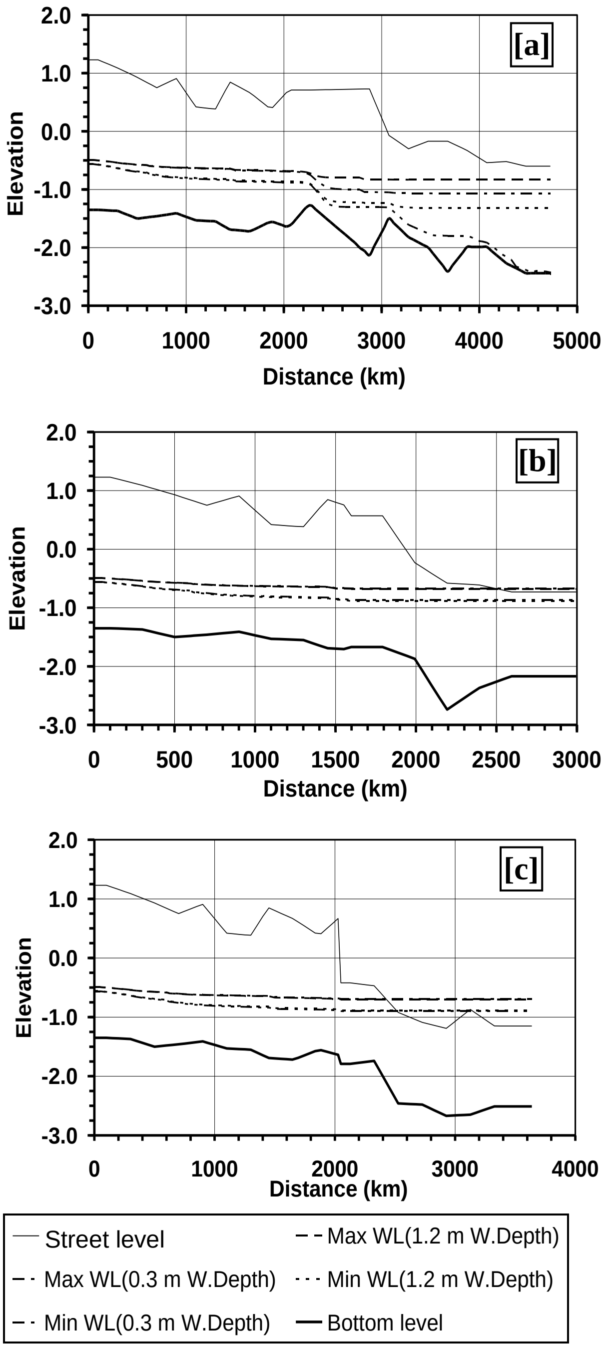

Fig. 6. Street, Bed, Max. and Min water levels for different initial water depths and 100% abstraction ratio
a- Main flow path, b- Abdel-Wikiel LLM, c- El-Hood LLM.

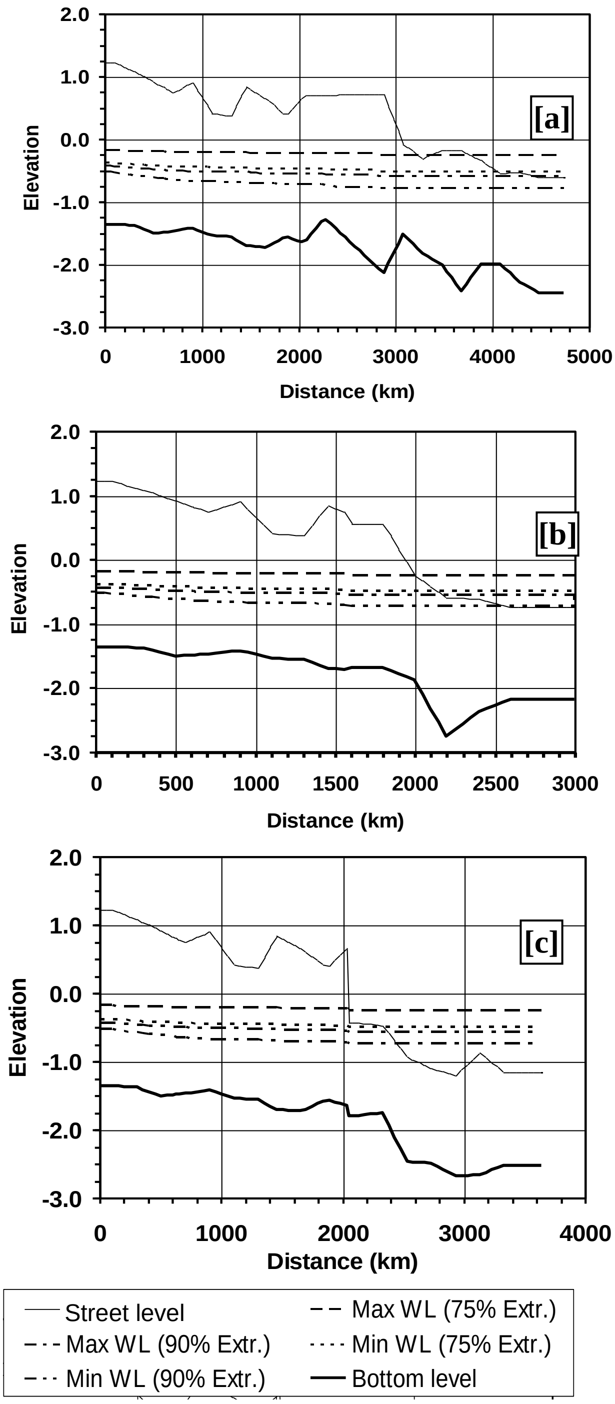

Fig. 7. Street, Bed, Max. and Min water levels in case of 1.2m initial water depth for different abstraction ratios
a- Main flow path, b- Abdel-Wikiel LLM, c- El-Hood LLM.

As a result, the storage capacity of the cross section is increased. On the other hand, pitching is urgently required after the cross sections modification. It can be said that, the pitching is necessary to keep the stability of the new cross sections.

5.2. Second proposal

The second proposal has suggested keeping all cross sections as it is. Also, it proposed new control structures to satisfy the optimal water levels during the operation of the main flow path and branches. It is differ to the first proposal in the technique, which was discussed. The main differences between the first and the second proposals are listed as: The period, which has been required for the application of each proposal, efficiency of each one to convert the canal to a canal suitable to carry the required discharges, efficiency of each one to increase the time before losing water by the end escapes, keeping the steady state of the cross sections and reducing the difference between the maximum and minimum water levels.

6. Simulation for first proposal

In this proposal, it is assumed to deal with the worst case of the initial simulation. The initial water depth is assumed to be 1.2 m, while the abstraction at lateral points were assumed to be 100 and 75% of the maximum extraction value. In addition, the cross sections are modified at different locations, see fig. 8. All modified cross sections are described in details as presented in table 2. Moreover, the volume of cut and fill for different cross sections are presented in table 3. In case of the main flow path, it is suggested that the bed erasing up to level (-2.45) and banks rising up to level (+0.50) starting from km 2.80, see fig. 9-a. For Abdel-Wikiel LLM, it has suggested the bed erasing up to level (-2.45) and banks rising up to level (-0.45) starting from km 1.70, see fig. 9-b. Finally, the bed will be erased up to level (-2.70) and banks rising up to level (-0.45) starting from km 2.10 for El-Hood LLM, see fig. 9-c. Some selected samples for the old and new cross sections are presented as shown in fig. 10.

Table 2
Description of modified x-sections

X-Sec No.	Location (km)	X-Sec No.	Location (km)	X-Sec No.	Location (km)
1	0.3/AW	8	0.7/HO	15	3.3/MA
2	0.5/AW	9	0.9/HO	16	3.5/MA
3	0.7/AW	10	1.1/HO	17	3.7/MA
4	0.9/AW	11	1.3/HO	18	3.9/MA
5	0.1/HO	12	2.5/MA	19	4.1/MA
6	0.3/HO	13	2.7/MA	20	4.3/MA
7	0.5/HO	14	3.1/MA	×	×

(NB: AW= Abdel-Wikiel LLM, HO = El-Hood LLM and MA = the main flow path)

Table 3
Cut and Fill volumes for different modified x-sections

X-Sec No.	Cut m^3	Fill m^3	X-Sec No.	Cut m^3	Fill m^3	X-Sec No.	Cut m^3	Fill m^3
1	60	128	8	0	847	15	616	0
2	268	357	9	0	440	16	666	33
3	284	456	10	0	375	17	642	238
4	429	541	11	130	122	18	151	0
5	0	338	12	450	0	19	168	363
6	330	217	13	430	61	20	168	363
7	0	421	14	430	61	×	×	×

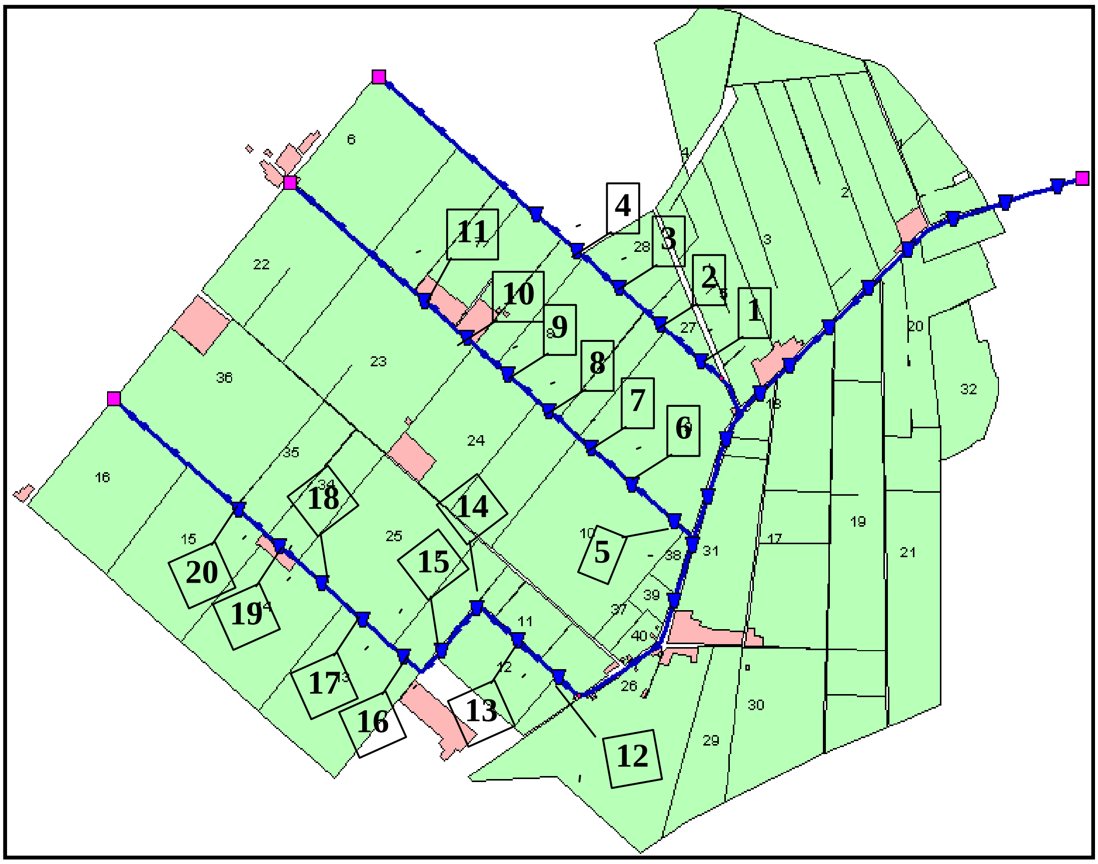

Fig. 8. Schematic map for the study area including the numbers of modified cross-sections.

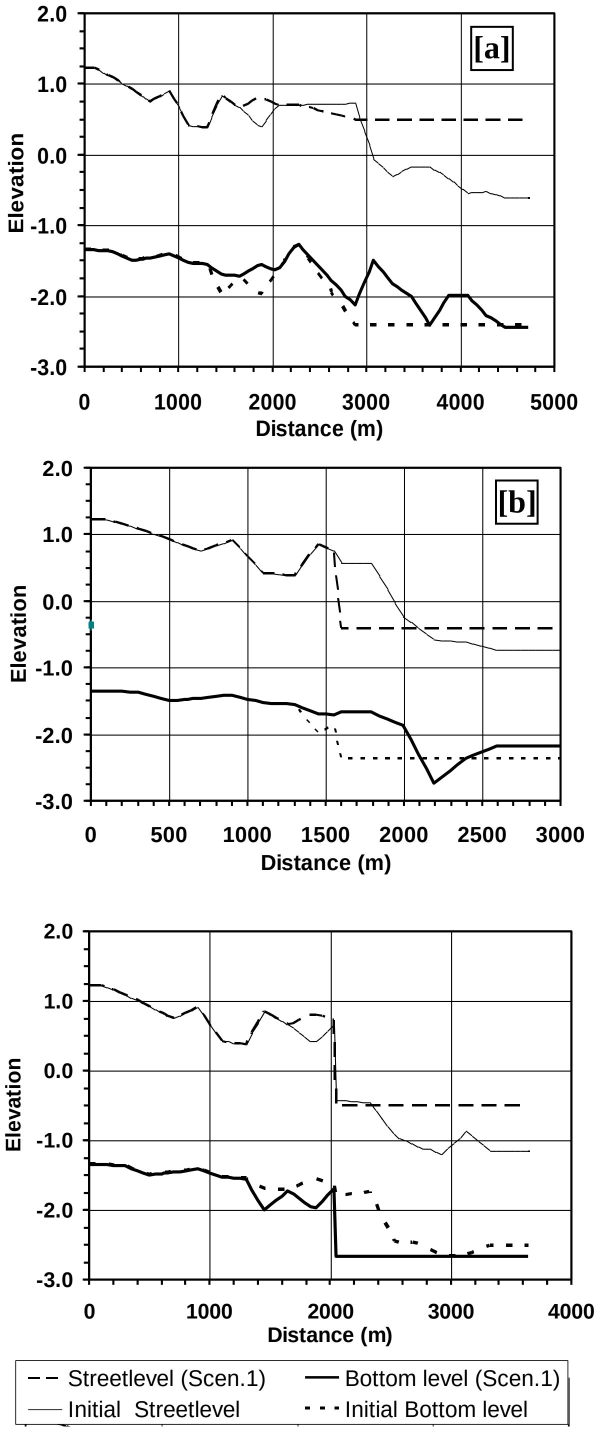

Fig. 9. Street and bed levels for both initial case and first proposal a- Main flow path, b- Abdel-Wikiel LLM, c- El-Hood LLM.

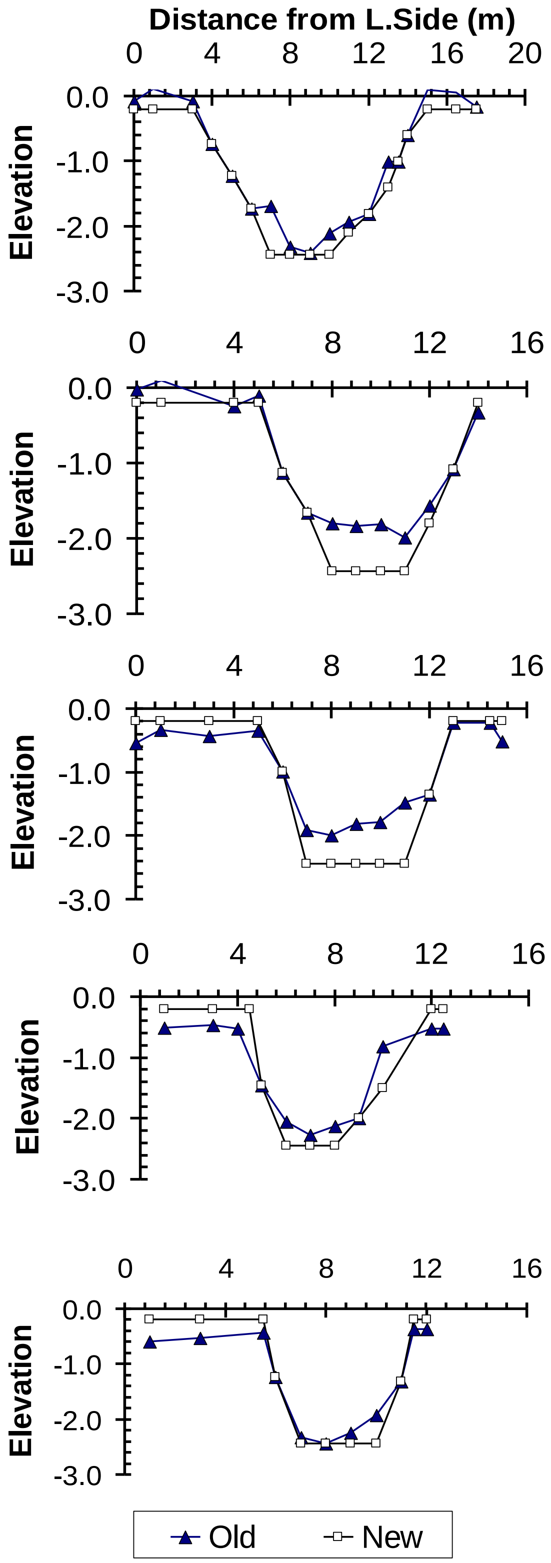

Fig. 10. Comparison between old and new cross-sections a- X-Sec.16, b- X-Sec.17, c- X-Sec.18, d- X-Sec.19, e- X-Sec.20.

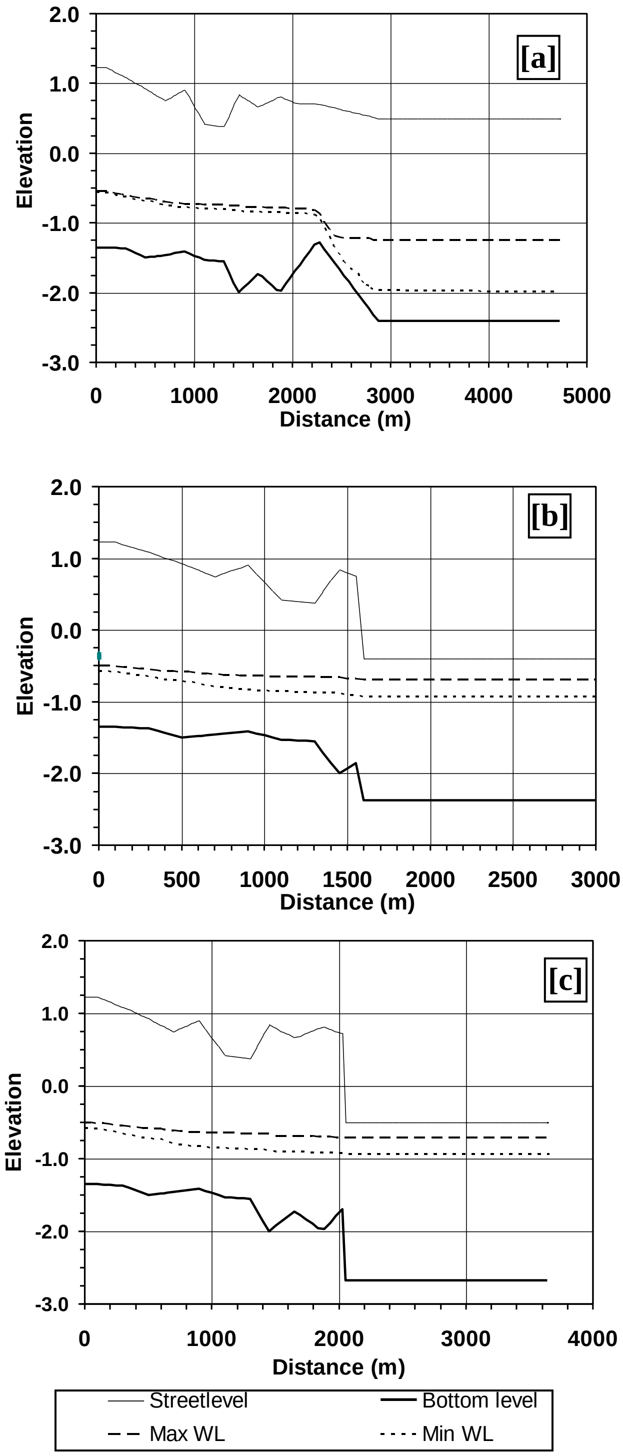

Fig. 11. Street, Bed, Max. and Min water levels 100% abstracting at lateral points
a- Main flow path, b- Abdel-Wikiel LLM, c-El-Hood LLM.

In case of 100% abstracting at lateral points, the water surface profiles for the different flow paths are extracting from the model. In a developed spread sheet the Min. and Max. values are calculated and compared, see fig. 11. For the main flow path, it can be noticed that the distance between water surface profile and bank levels ranges between 1.7m to 2.5m, see fig. 11-a. Generally, the water depth in the channel is shallow through the main flow path within the modified reach. It may cause an increasing of the running cost.

For Abdel-Wikiel LLM, the distance between the water surface profile and bank levels ranges between 0.3 to 0.6m see fig.11-b. For El-Hood LLM, It can be said that the water surface ranges in acceptable range, see fig. 11-c. It ranges between 0.2 to 0.5m from average bank level.

It is important to make a simulation for another case, in which the abstraction at lateral points is less than 100%. Fig. 12 shows the water surface profiles in case of 75% abstraction at lateral points. In case of main flow path, it can be clearly seen that the modified cross sections can bear the water for more than 103hrs without any flooding or water escaping, see fig. 12-a. The other low level mesqas can bear water for more than 27hrs. Figs. 12-b and 12-c presents the max. and min. water levels for Abdel-Wikiel LLM and El-Hood LLM, respectively. It is clear that Abdel-Wikiel LLM can transport water without any losses for more than 27hrs with 75% abstraction rate, see fig. 12-b. In addition, El-Hood LLM can transport the water without any problems for more than 40hrs.

Fig. 13 presents the relationship between the water depth and time for the selected three different places on the network. In case of the 100% abstraction at lateral points it is clear that, the water depth varies periodically with time. It means that the water depth increases at low abstraction during the day and vice versa, see fig. 13-a. This figure proves that, the model reaches the steady state conditions as the water depth varies in same ranges during the study period. On the other hand, in case of 75% abstraction it is clear that the water depths increase as the time pass as the difference between the outflow and inflow are

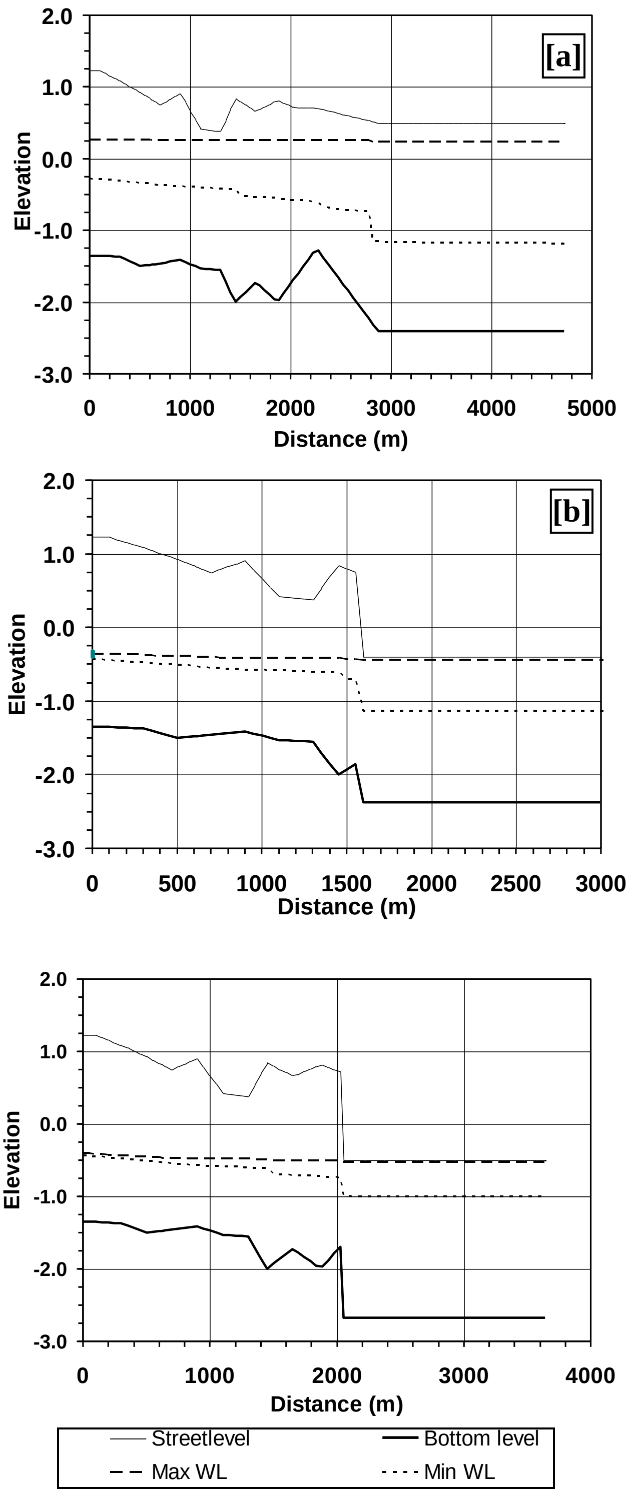

Fig. 12. Street, Bed, Max. and Min water levels for 75% abstracting at lateral points, a- Main flow path, b- Abdel-Wikiel LLM, c- El-Hood LLM.

storing in the network itself, see fig. 13-b. As a result, the network can bear the water without any loss for about 27hrs. as presented before.

7. Simulation for second proposal

In the second proposal, the priority is to keep all cross sections unchanged. The main purpose of this concept is to keep all hydraulic, biological, and cultivation conditions the same. As a result, it is necessary to find another method to control the flow through network. The new method has to achieve the saving of water and keeping water surface within optimal ranges. It is proposed three rectangular sharp crested weirs at three locations, see fig. 14. All design criteria of the three proposed weirs are listed in table 4.

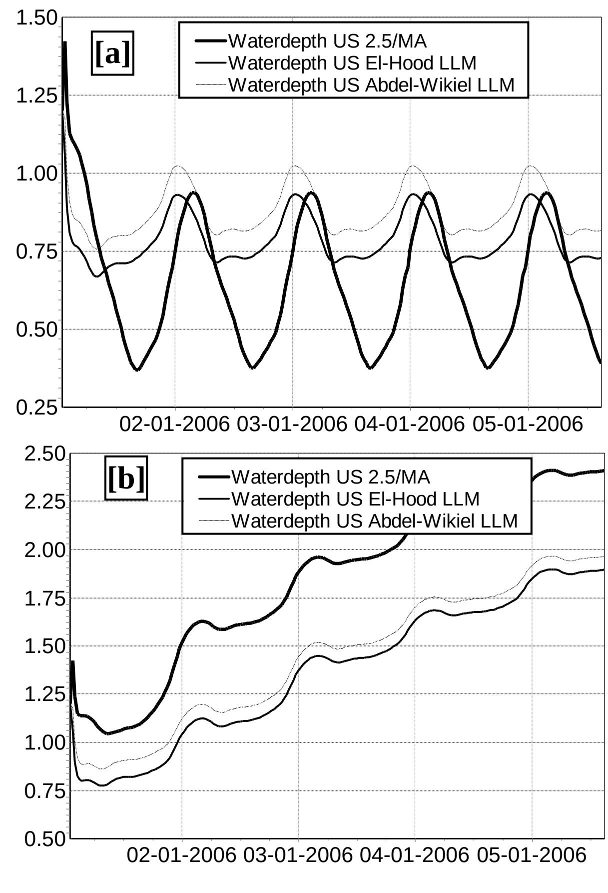

Fig. 13. Relationship between water depth and time [a] 100% abstraction ratio [b] 75% abstraction ratio.

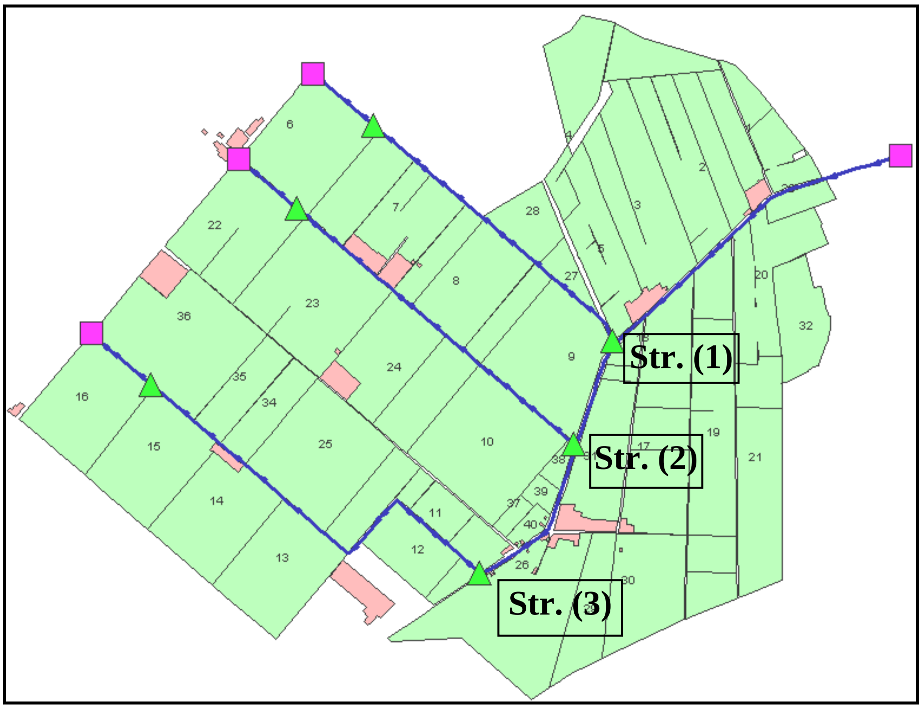

Fig. 14. Schematic map for the study area under the second proposal.

Table 4
Details for the proposed control structures

Proposed structure	Description	Location (km)	Width (m)	Crest level
Str. (1)	Rec. Sharp crested weir (1)	0.0/AW	2.0	-0.465
Str. (2)	Rec. Sharp crested weir (2)	0.0/HO	2.0	-0.465
Str. (3)	Rec. Sharp crested weir (3)	2.45/MA	2.0	-0.520

(NB: AW= Abdel-Wikiel LLM, HO = El-Hood LLM and MA = the main flow path)

Table 5
Cost comparison between the two proposals

Item		1st proposal		2nd proposal	
Description	Rate (L.E.)	Quantity	Value	Quantity	Value
Filling (m^3)	20	5 361	107 220	-	-
Cut (m^3)	10	5 222	52 220	-	-
Pitching (m^3)	200	2 000	400 000	-	-
P.C. (m^3)	800	-	-	300	240 000
R.C. (m^3)	2 000	-	-	60	120 000
			559 440		360 000

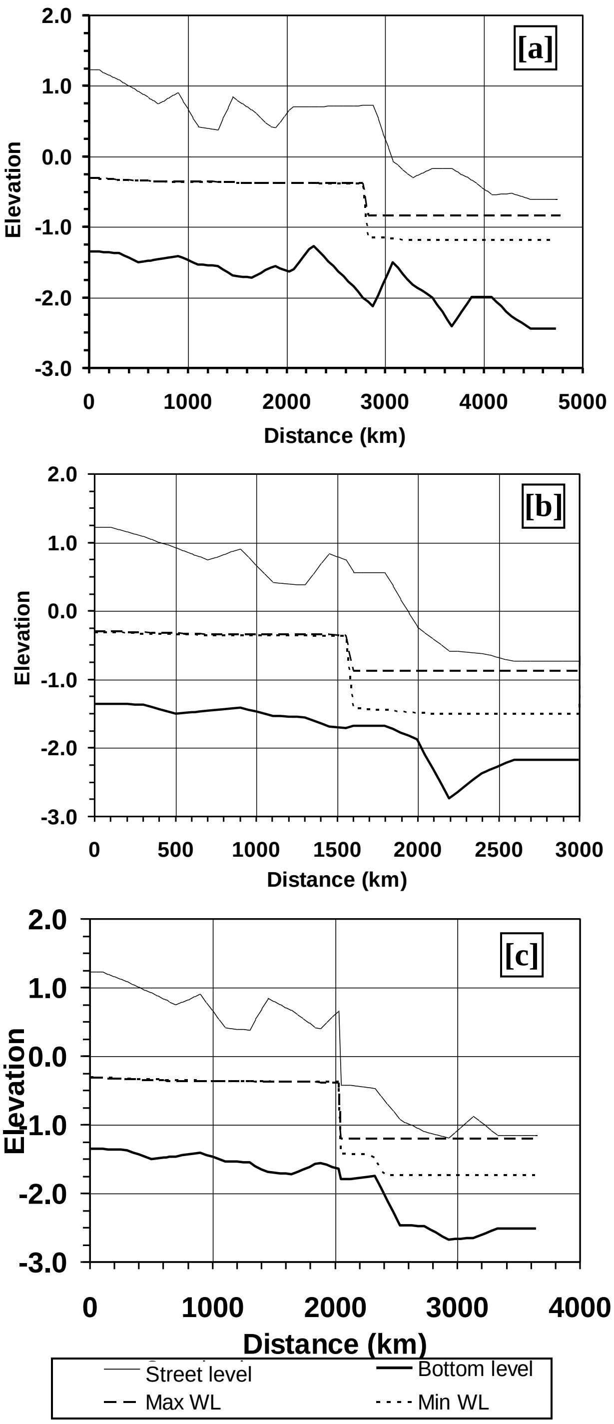

Fig. 15. Street, Bed, Max. and Min water levels 100% abstracting at lateral points a- Main flow path, b- Abdel-Wikiel LLM, c- El-Hood LLM.

In case of 100% abstracting at lateral points, the water surface profiles for the different flow paths are extracting from the model. In the developed spread sheet the Min. and Max. values are calculated and compared, see fig. 15. For the main flow path, it can be noticed that the distance between water surface levels and bank levels ranges between 0.3m to 0.7m, see fig. 15-a. For Abdel-Wikiel LLM and El-Hood LLM, it is clear that the water surface profile can be considered in the optimum range, see fig. 15-b and 15-c. It ranges between 0.1 to 0.8m from bank level. Generally, the water surface profile can be considered better than the first proposal especially for the main flow path.

Fig. 16 shows the water surface profiles in case of 75% abstraction at lateral points. In case of main flow path, it can be clearly seen that the modified cross sections can bear the water for more than 22hrs without any flooding or water escaping, see fig. 16-a. Figs. 16-b and 16-c present the max. and min. water levels for Abdel-Wikiel LLM and El-Hood LLM, respectively. It is clear that Abdel-Wikiel LLM can transport water without any losses for more than 18hrs with 75% abstraction rate, see fig. 16-b. In addition, El-Hood LLM can transport the water without any problems for more that 10hrs, see fig. 16-c.

Fig. 17 presents the relationship between the water depth and time for selected three different places on the network. In case of the 100% abstraction at lateral points it is clear that, the water depth varies periodically with time. It means that the water depth increase at low abstraction during the day and vice versa, see fig. 17-a. On the other hand, in case of 75% abstraction it is clear that the water depths increase as the time pass as the difference between the outflow and inflow are storing in the network itself, see fig. 17-b.

8. Comparison and discussion

The comparison between the two proposals has been investigated depending on many factors. Mainly, the comparison between them depends keeping the existing situations of cross sections, their efficiency to reduce distance between water surface level and bank levels, their efficiency to reduce cost to the minimal values and increasing the time before water losing.

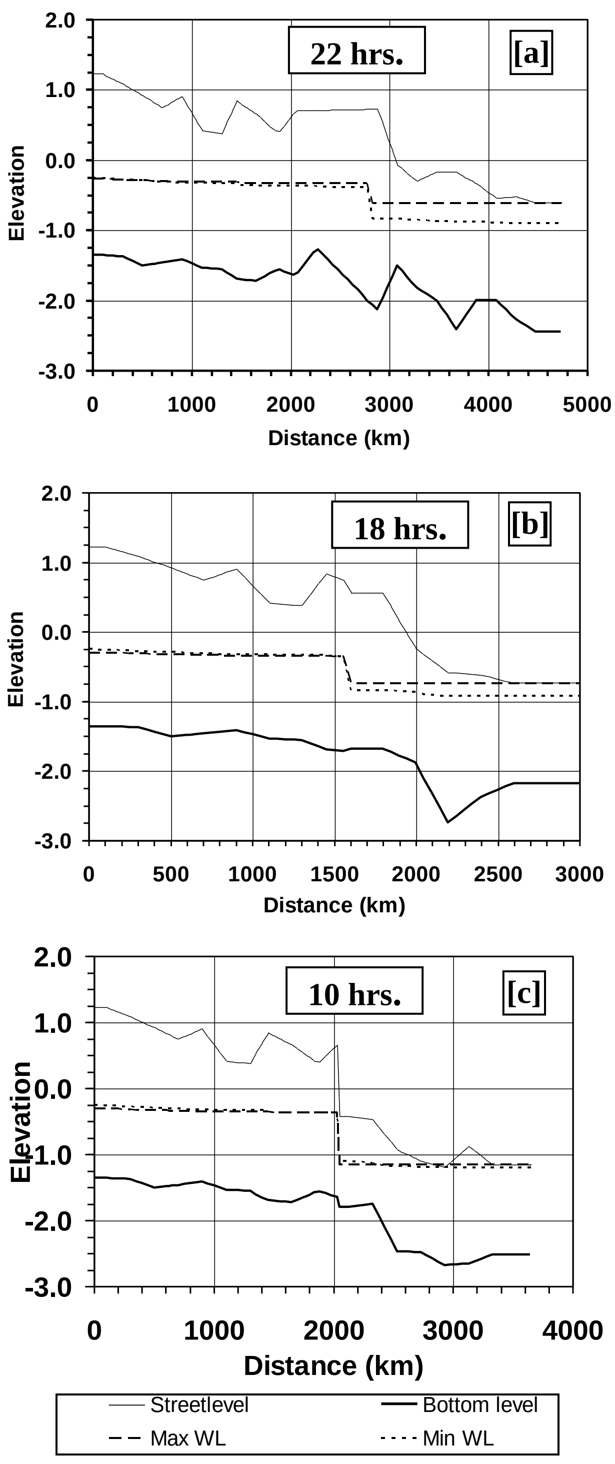

Fig. 16. Street, Bed, Max. and Min water levels for 75% abstracting at lateral points, a- Main flow path, b- Abdel-Wikiel LLM, c- El-Hood LLM.

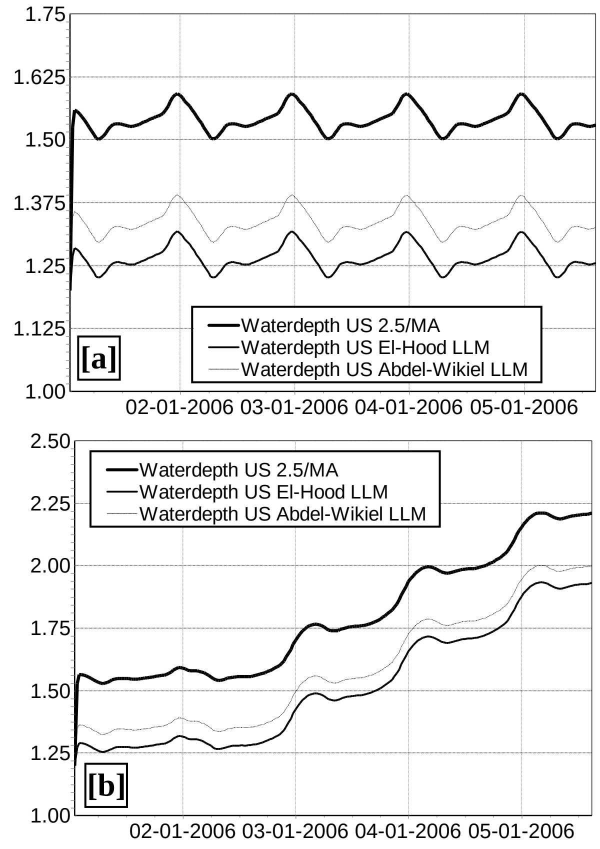

Fig. 17. Relationship between water depth and time [a] 100% abstraction ratio [b] 75% abstraction ratio.

The efficiency to reduce distance between water surface level and bank levels (i.e. free board) plays an important role to choose the best proposal. In case of the second proposal, the average distance between the water levels and bank levels are less than 0.5m, while it is about 2.2m in case of the first proposal, see fig. 18. It can be said that, the second proposal may be better than the first proposal.

The cost is very important factor affecting the choosing of the optimum proposal. It includes two types of costs: initial cost and running cost. From running cost point of view, it is clear that the first proposal needs high cost as the depth between the water level and bank levels are higher than the second proposal.

No doubt, keeping of the existing cross sections may be the best from the cost point of view. In addition, it may play better role to keep the stability of the biological life and more safe for on farm irrigation system. Table

5 presents an estimation for the initial direct cost of the main items for the two proposals. It clears that the second proposal may be considered as better than the first proposal.

Fig. 19 represents the comparison between the alternative proposals as the time before water losing. It is clear that, the first proposal may be better than the second one as it increases the time. The average time before water losing for the first proposal is about 55hrs, while it is about 18hrs for the second proposal.

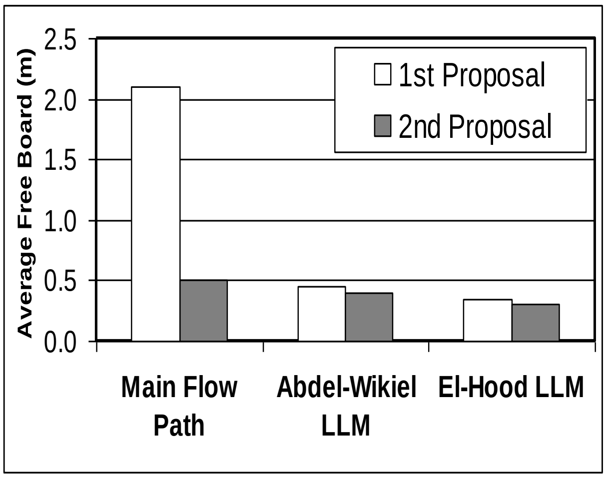

Fig. 18. Comparison between the two proposals for average free board at the different reaches.

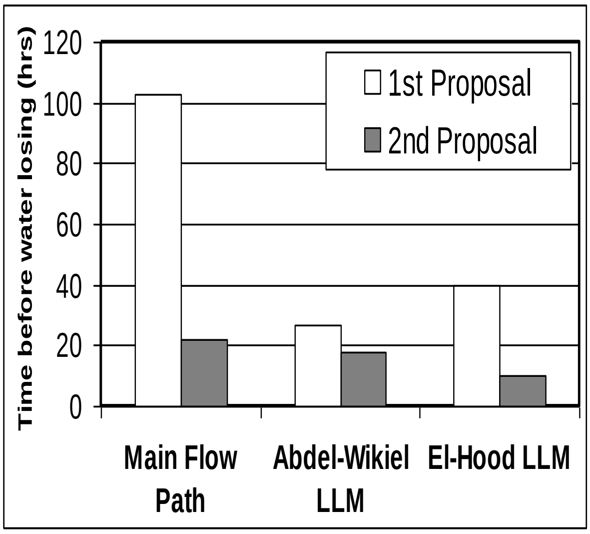

Fig. 19. Comparison between the two proposals for the time before water losing at the different reaches.

Finally, it can be said that the second proposal is more effective to keep the free board in the optimum range than the first proposal. Also, it can be noticed that the first proposal requires a large amount of money for different civil works and hence the cost of the second proposal is small compared to the first proposal.

9. Conclusions

One of the famous commercially available unsteady-flow simulation software package (SOBEK from Delft Hydraulics) was used to assess the impact of water allocation on El-Mahrousa branch canal. The actual cross sections of the canal under continuous flow after irrigation improvement project implementation are checked as an initial scenario. The results of the initial scenario indicated that the actual sections of El-Mahrousa canal are not sufficient to carry the required discharges under continues flow. Two proposals were investigated to indicate the use of the model. In the first proposal, different cross sections are modified at different locations of both two low level mesqas and main flow path. The second proposal has suggested keeping all cross sections as it is. Also, it proposed new control structures to satisfy the optimal water levels during the operation of the main flow path and branches. The suitable design of the control structures which achieve water saving are proposed. The different proposed solutions are compared from technical and economical point of views. Comparing the results, it can be concluded that the second proposal is more safe and economic than the first proposal.

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Notations

A_f wetted area,[L^2],
A area of cross section, [L^2],
b An exponent representing the degree of

non-linearity, [-],
C Chézy coefficient, $[L^{1/2}/T]$,
g Gravity acceleration, $[L/T^2]$,
h Water level, [L],
p Fraction of pores, [-],
q_{lat} Lateral discharge per unit length, $[L^2T^{-1}]$,
Q Discharge, $[L^3T^{-1}]$,
R Hydraulic radius, [L],
s Sediment transport per unit width, excluding pores, $[m^2/T]$,
S Sediment transport through a cross section, $[L^3/T]$,
T Time, [T],
u Current velocity, [L/ T] ;
x Distance, [L],
W_f Flow width, [L],
τ_{wi} Wind shear stress, $[F/L^2]$, *and*
ρ_w Water density, $[M/L^3]$,

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