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Improving irrigation system management: A case study: Bahr Sanhoor Canal, Fayoum, Egypt

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Abstract: Realising the need for improving irrigation system management, the Bahr Sanhoor Canal (BSC) was selected as a case study, and the effect of covering a reach with a concrete box culvert where the irrigation canal passes through a crowded rural area on the efficiency of the canal system was examined. The field inspection of the canal system revealed multiple problems. Two alternatives for improving irrigation management were introduced. A local alternative was offered by applying a suitable numerical model to enhance the efficiency of the current status of the canal system, the water level upstream of the covered part reached 13.54 m, this solution will lower the water levels by approximately 2 m, which is below the bank levels at an affordable cost. Additionally, it will help to avoid the risk of inundating the adjacent residential area. A sustainable and environmental alternative was considered to offer a new path in which the covered reach passed outside the residential area. This alternative is permanent and sustainable. Although the proposed second path to the right of the residential area is the long-term recommendation and is sustainable, any path of flow to either the left or right of the BSC will be associated with initial high costs. The two proposed alternatives may help decision makers improve the performance of irrigation systems.

Keywords: canal efficiency, covering, GIS, irrigation system management, numerical simulation

INTRODUCTION

Efforts have been dedicated to increase the efficiency of canal systems, particularly in developing countries. In these countries, deficiencies in finance and improper infrastructure are the major factors hindering improvements in the efficiency of irrigation systems. The best improvement in the operation and management of irrigation systems is economically feasible. Numerous studies have addressed this topic. AL-JAYYOUSI [1999] presented a methodology to improve the efficiency of the irrigation distribution system in Jericho City. MOLDEN *et al.* [2007] provided a basic conceptual framework for the use of performance assessments in support of irrigation service. FROEHLICH [2008] introduced an optimal design for canal cross-sections and analysed the solutions to obtain the generalised trapezoidal section within the form of sharp acute-angled cornered trapezoids, rectangles, triangles, and semicircles. ADHIKARI *et al.* [2009] studied primary canals of three ancient irrigation systems in the southern plains of Nepal. The authors introduced a scientific interpretation of the autochthonal technology applied to the systems that facilitated the employment of the identical channel network for irrigation, drainage and flood management.

Over the years, many numerical models have been developed to examine the flow dynamics of the canal system. The simulation of the unsteady flow equation began with the tactic of characteristics, because the solution method was applied to the one-lined canal. Finite-difference techniques were developed within the 1970s and applied to a branching and coiled network [CUNGE *et al.* 1980]. The unsteady-flow simulation models that were developed afterward are either upstream-controlled or downstream-controlled adjusted, and they utilise either limited or central criteria to handle scheduled, arranged, or on-demand strategies of water conveyance [MERKLEY, WALKER 1991; REDDY 1999]. A number of accessible canal models, such as CANALMAN, DUFLOW, CARIMA, MODIS, USM, the Branch Canal Network Model, and Sobek, were examined and assessed according to their practical merits and modelling abilities [CLEMMENS *et al.* 1993; HOLLY, PARRISH 1993; MERKLEY, ROGERS

1993; Rogers, Merkley 1993; Schuurmans 1993]. The utilisation and pitfalls of canal models were conjointly explored by Contractor and Schuurmans [1993]. The authors believed that three factors that influenced errors within the model output were a distance, time-step, and a weighting (coefficient) parameter. Occasionally, some of these models should be modified to feature specific operational settings or structures of an irrigation system not enclosed within the model. Many times, these models were applied in case studies (e.g., Wahl *et al.* [2011], Ibraheem *et al.* [2011], Abdelhaleem *et al.* [2013], Shawky *et al.* [2013], Abdelhaleem [2017], Elgamal *et al.* [2017], Elhamrawy [2018], Abdelhaleem *et al.* [2020]).

Egypt suffers from water shortages due to the incremental rate of population growth, as well as that of municipal and industrial needs [Abdelhaleem 2016; Allam, Allam 2007]. Accordingly, as a result of the growing demand for irrigation water with limited water resources, irrigation systems will have to become more efficient and reliable in the future. This fact attracts the attention of decision makers, who wish to improve the performance of irrigation systems. In Egypt, some irrigation canals cross urbanised or rural areas. Recently, the new social trend of the Egyptian lifestyle initiated unwelcome traditions and behaviours that cause open channels to pass by residential regions. These canals receive the unauthorised discharge of domestic wastewater, resulting in surface water pollution and spreading of pathogens. Solid waste is also disposed of in these open channels. To avoid blockages and the unauthorised discharge of wastewater, the Ministry of Water Resources and Irrigation (MWRI) developed a national strategy to transition the reaches passing by residential regions into covered conduits or pipes, which would permit using the surface ground for community service activities and improve the public health of the residents of those areas. Once the canal is covered, a vital issue is raised regarding the hydraulic efficiency of the entire canal system, particularly that of the covered reach.

In this paper, the Bahr Sanhoor Canal (BSC), a major waterway in the irrigation network of Fayoum Governorate, Egypt was selected as a case study to explore the effect of covering a reach of an irrigation canal with a concrete culvert on the hydraulic efficiency of this canal, considering the misuse and illegal practices by some residents of the neighbourhood of the command area's residents. The BSC suffers from multiple problems, such as excessive deposition, encroachment of banks, higher agricultural lands around the BSC, and higher upstream water levels due to culvert blockage. The BSC is unable to carry the designed level of discharge, and the adjacent inhabited areas are always inundated. This study aimed to test two alternative solutions for improving BSC system management. The first proposed alternative introduced a new flow path to the right or left of the residential area as a sustainable solution. This choice may be associated with high initial costs, requiring expropriation of lands and a number of weirs and pumping stations to be installed. The second alternate is considered to be a temporary and economically feasible one that would enhance the current efficiency of the canal system by developing a suitable hydrodynamic model at the prototype scale. The Sobek model was applied to the BSC and different scenarios were tested to introduce recommendations that aim to increase the carrying capacity and improve the hydraulic efficiency of the BSC, particularly at the covered section, to accommodate the maximum design discharge.

MATERIALS AND METHODS

STUDY AREA

The Bahr Sanhoor Canal (BSC) lies within the northeastern area of Fayoum Governorate between 29°10' N and 29°20' N and 30°50' E and 31°00' E (Fig. 1). The general climate of this area is characterised by hot, long and dry summers and warm, short winters with scarce precipitation. Additionally, great temperature variations occur between summer and winter and between day and night in this belt. The temperature ranges between 35.1 and 46.7°C in summer months and from 12 to 21.5°C during winter months. The common average annual rain amount is approximately 10.3 mm, which is relatively low. The overall annual evaporation intensity reaches 2296 $mm \cdot y^{-1}$, and the average humidity ratio is 51.6% [El Abd *et al.* 2014]. The Fayoum depression forms an oasis, and it directly joins the Nile River through the Bahr Youssef Canal, which derives its waters from the Al-Ibrahimiya Canal. The Fayoum basin encompasses a special irrigation system due to its topography. The land level varies from +26.00 m above mean sea level (MSL) at Fayoum town to –43.00 m below MSL at Qarun Lake [Veer *et al.* 1993]. The common slope is considerably steep (1.0 $m \cdot km^{-1}$). A system of weirs along the canals was applied to regulate the slope of the canals.

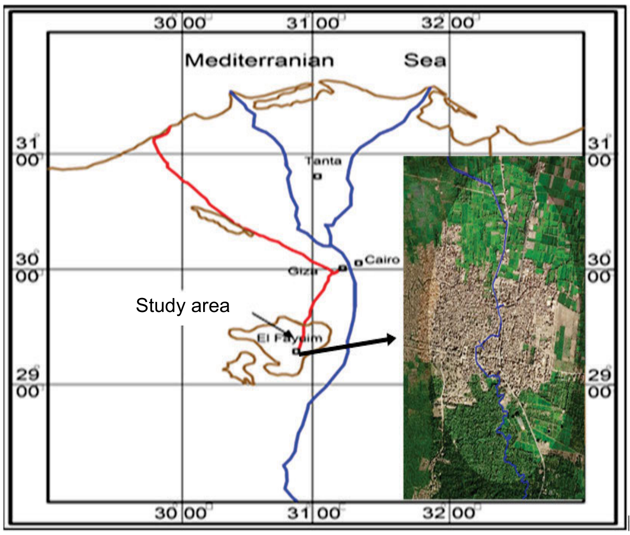

Fig. 1. Layout of the study area; source: own elaboration based on Google Earth

Field inspections and surveys revealed that the BSC spans a length of 25 km and serves 6,890 ha (16,409 feddans[1]). The BSC runs as a water carrier (branch canal) for 19.90 km, and then the canal is divided into 3 distributary canals, each 6 km long, that distribute irrigation water to the command areas. The BSC withdraws its water from the Bahr Youssef Canal via a head regulator with two openings (Fig. 2). The reach of the BSC from the head regulator to km 19.900 is characterised by higher adjacent irrigated lands. The BSC is considered as a drain for those lands because of its lower levels relative to the agricultural lands beside it. At km 7.000, a part of the canal (150 m) is covered

[1] 1 ha = 2.40 feddans.

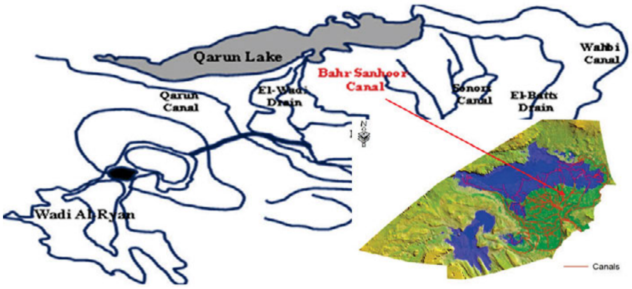

Fig. 2. Schematic map of the Bahr Sanhoor Canal within the Fayoum irrigation network; source: own elaboration based on Google Earth

with a single-opening reinforced concrete box culvert, which has a width of 3 m and a height of 2 m (Fig. 3). The canal system contains a number of off-take points, with 7 weirs, 4 of which are located upstream of the covered reach. The bed width of the canal varies between 3 and 6 m, while the water surface slope ranges from 5 to 10 cm·km^{-1}.

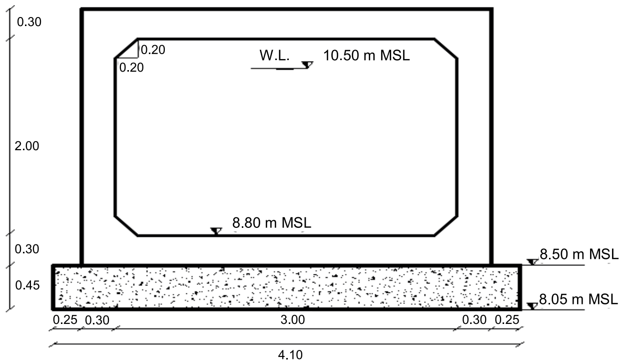

Fig. 3. Cross section of the box culvert at km 7.000 of the Bahr Sanhoor Canal (all dimensions in metres); source: own elaboration

DATA COLLECTION AND COMPLETENESS

Several site visits were carried out to collect a complete data picture of the study area. During these visits, measurements were collected, and photos were captured. Field investigations were taken, and the required data were assembled. Bathymetric and hydrographic surveys using high technology and calibrated instruments, such as a differential global positioning system (DGPS), complete total station, echo-sounder, velocity current-meter, and range finder, were utilised. The bathymetric survey was performed to cover the entire length of the canal system, including the 3 branches. The involved bathymetric survey investigated the cross-sectional details of each 100-m segment and measured the water surface slope every 1 km. Additionally, the status of the banks was monitored and registered. Furthermore, the flow velocities were measured at 22 different locations, including every weir (Fig. 5). The cross sections of the BSC were induced, and hence, the actual discharges were calculated at the upstream and downstream ends. The field survey and hydraulic measurements revealed that the observed longitudinal profile of the BSC was in close agreement with the designed profile except for the reach just upstream of the culvert (from km 3.00 to km 7.00) and the reach immediately downstream of the covered part (from km 7.150 to km 12.500). The average deposition in these two reaches was approximately 2 m. Figure 6 shows a longitudinal cross-section along the BSC. Moreover, the bed widths of the measured cross sections varied from 4 to 9 m compared to the designed range of 3 to 6 m. In addition, the water levels were very close to the bank levels within the vicinity of the residential areas just downstream of the culvert (from km 7.150 to km 8.450). After this distance, the bank levels were higher than the water levels by 2 to 3 m.

Fig. 4. The Bahr Sanhoor Canal at the covered reach: a) upstream view; b) downstream view, source: own elaboration (camera's photo)

Bank encroachment and misuse by some residents along the reaches just upstream and downstream of the covered part, lead to reduce the flow capacity in the BSC. Reconnaissance visits to the canal system demonstrated that the box culvert opening was almost blocked with garbage and solid waste. Therefore, the water levels upstream of the culvert were above the bank levels. Additionally, an average deposition of 2 m was realised at the cross sections just upstream and downstream of the covered area. Thus, the water flooded some of the houses located in the vicinity of the covered area. The field visit also indicated that a pipe of 2 m diameter was constructed parallel to the box culvert to allow some of the flow to pass by the pipe. Figure 4 depicts the current situation of the BSC at the covered section.

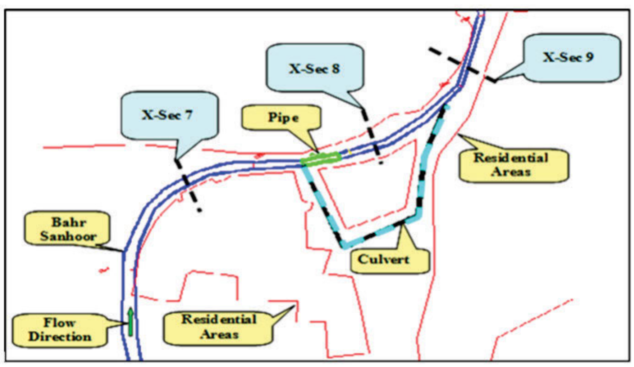

Fig. 5. Planned view of the study area with locations of discharge measurements, source: own elaboration (GIS map); source: own elaboration

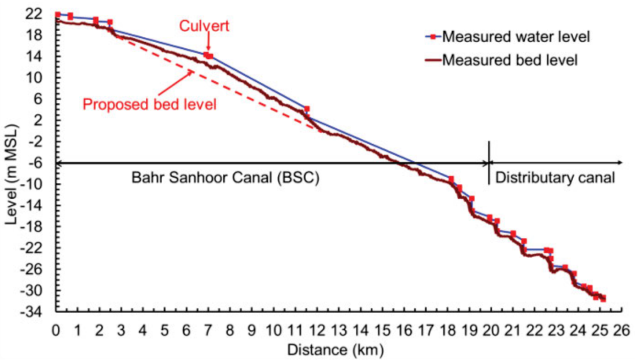

Fig. 6. Measured water and bed levels along the Bahr Sanhoor Canal, source: own elaboration

The field observations also revealed that the measured discharge varied between 2.16 $m^3 \cdot s^{-1}$ downstream of the head regulator of the BSC and 0.95 $m^3 \cdot s^{-1}$ at the downstream end of the canal. However, the discharge just upstream of the culvert was 2.03 $m^3 \cdot s^{-1}$. Additionally, the discharge passing through the culvert was 0.44 $m^3 \cdot s^{-1}$, which accounted for 21% of the flow just upstream of the culvert. The remaining 79% of the flow was discharged via the 2-m diameter pipe that was installed parallel to the culvert. Additionally, the top widths of the cross-sections upstream of the culvert ranged from 5.50 to 6.50 m, while the average top width of 3.50 to 5.50 m was recorded along the reaches downstream of the culvert. The average water depths along the reaches upstream of the culvert were in the range from 0.62 to 2.00 m. However, the values ranged from 0.80 to 1.52 m downstream of the culvert. Additionally, the average flow velocity varied from 0.49 $m \cdot s^{-1}$ downstream of the culvert to a maximum value of 0.77 $m \cdot s^{-1}$ upstream of the covered reach.

MODELLING THE PROBLEM IN HAND

Geographic Information System (GIS) and a digital elevation model (DEM) were used in combination with recent satellite images to establish a three-dimensional view of the study area at the covered reach and its land use. The GIS model was developed based on the control points, derivation of DEM, bathometric survey, topographic survey and analysis of recent satellite images.

DGPS is the primary survey reference for all surveying activities. This technique can produce real-time positions of a moving vessel. The accuracy of the collected data is directly based on the accuracy of the control points (BMS). The horizontal coordinates (x, y) of four points were calculated accurately using the single-point positioning technique of GPS, which was applied for several hours of observation spanning three consecutive days. The vertical coordinate of these points, (z) was determined using a levelling technique that used a precise levelling instrument between them and an irrigation gauge located at the front of the culvert. Digital elevation data from the NASA Shuttle Radar Topographic Mission (SRTM) were used. The SRTM data are available as 3 arc seconds. The output of this DEM was enhanced by using the field topographic and bathymetric survey. The model was applied to select the lowest path in the level to the right or left of the covered reach. The model was also used to define the required expropriation lands for each path.

A range of hydrodynamic models exist for flood statement and irrigation systems, and these models are developed by local institutions and international organisations. A key objective is to develop a model that can be employed for planning and as a tool for managing an existing hydraulic system. Sobek-1D2D was applied throughout this study because of its ability to consider the various local-scale features. Sobek could be a powerful package developed and modified by WL| Delft Hydraulics, a company in the Netherlands, for flood forecasting, drainage systems, irrigation systems, sewer overflow, groundwater level control, river morphology, salt intrusion and water quality [Deltares 2019]. Sobek has a very robust numerical scheme that handles drying, flooding and subcritical and supercritical flows efficiently. It has been applied to the irrigation systems of Fayoum Governorate because of its several distinct advantages. The model can accommodate the irregular shape of the canal cross-sections and simulate the flow of long canal systems with many lateral offtakes and in-line heading up structures, such as weirs and culverts, as in the case of the BSC. Sobek conjointly permits the integration of user-defined modules through the use of specific data exchange formats. Sobek has a unique integrated format in which the effectiveness of the implemented measures can be examined to keep the system running at high efficiency. The manual or automatic operation of pumps, sluice gates, weirs, and other structures can all be incorporated into the model, giving a dependable image of how a system behaves in extreme conditions. Thus, Sobek was selected to simulate the flow within the BSC, including its covered reach, due to its high flexibility in manipulating the Manning's coefficient value.

The dynamic behaviour of irrigation canal systems is often well delineated by a set of equations referred to as the Saint-Venant equations [Abbott 1979]:

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

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

where: Q = the discharge ($m^3 \cdot s^{-1}$); x = the longitudinal distance along the channel in the direction of flow (m); W = the water surface width (m); H = the water depth (m); t = the time (s); A = flow area (m^2); g = the gravitational acceleration (9.81 $m \cdot s^{-2}$); S_f = the slope of the energy grade line; S_0 = the bed slope of the channel.

To calculate S_f, the Manning equation was used.

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

where: R = the hydraulic radius (m); v = the mean velocity ($m \cdot s^{-1}$); n = Manning's roughness coefficient ($s \cdot m^{-1/3}$).

There are many approaches that can be used to numerically solve the Saint-Venant equations to improve real system investigations [Abbott 1979]. In this case, the equations were within the initial place and were made linear; thus, their non-linearity was addressed, making it possible to employ the linear controllers. The equations were linearised assuming conditions close to steady state. This method ensures that, for small deviations from a considered setpoint, the ensuing equations can still describe the behaviour of the system. The linearisation of the Saint-Venant equations may be expressed in the following forms:

$$\frac{\partial q}{\partial x} + W_0 \frac{\partial H}{\partial t} = 0 \tag{4}$$

$$\frac{\partial q}{\partial t} + 2v_0 \frac{\partial q}{\partial x} + W_0 \frac{\partial h}{\partial x} \left({c_0}^2 - {v_0}^2\right) - \xi_0 q - \gamma_0 q = 0 \tag{5}$$

where: q, h = the changes in discharge and water depth, respectively, from a considered steady state, c = the wave celerity ($c = \sqrt{\frac{gA}{W}}$), v = the flow velocity ($v = Q/A$), 0 = the subscript representing the steady state values; ξ_0, γ_0 = factors that due to linearisation are defined as:

$$\xi_0 = -\frac{2gQ_0}{n^2 A_0 {R_0}^{4/3}} + \frac{2Q_0}{{A_0}^2} W_0 \left(\frac{dH}{dX}\right)_0 \tag{6}$$

$$\gamma_0 = gS_0 W\left\{1 + C_0 + \left[1 + C_0 + (c_0 - 1)\mathrm{Fr}^2\right]\alpha\right\} \tag{7}$$

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

$$C_0 = 1 + \frac{4}{3}\frac{P_0}{W_0}\left(\frac{dH}{dX}\right)_0 \tag{9}$$

where: Fr = the Froude number, P_0 = initial pressure, C_0 = constant.

Despite having all the required equations linearised, there are still several steady state parameters left to determine, with the exception of the steady state discharge that one can opt for freely because it is a condition for linearisation. The steady state parameters are calculated by using the Saint-Venant equations and assuming no variations in discharge and flow depth over time. Thus, the following is obtained:

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

This equation represents the spatially gradually varied flow and is used to determine the flow profile in steady-state flow conditions.

NUMERICAL SIMULATION

The second alternative was to enhance the efficiency of the current situation of the BSC employing the Sobek model. To test the hydraulic potency of the canal system to convey the designed daily flows, a hydrodynamic model was developed for the canal system using the numerical scheme of Sobek [Deltares 2019]. The set-up of the hydraulic model concerned specifications of canal cross-sections along a total length of 20 km, the layout of the canal network, the head and offtakes, the weirs, the box culvert, the parallel pipe, and the upstream and downstream boundary conditions. Figure 7 illustrates the model domain.

The upstream and downstream ends of the canal were identified. The data identification was extracted from the index map of the Nile System that is kept in Sobek. The maximum chosen distance between two adjacent points was also required for the network delineation. In Sobek, the cross-section database is considered a library that stores data for a large number of cross-sections, and it is organised in such a way that every cross-section is defined by its name and topographical data [Contractor, Schuurmans 1993].

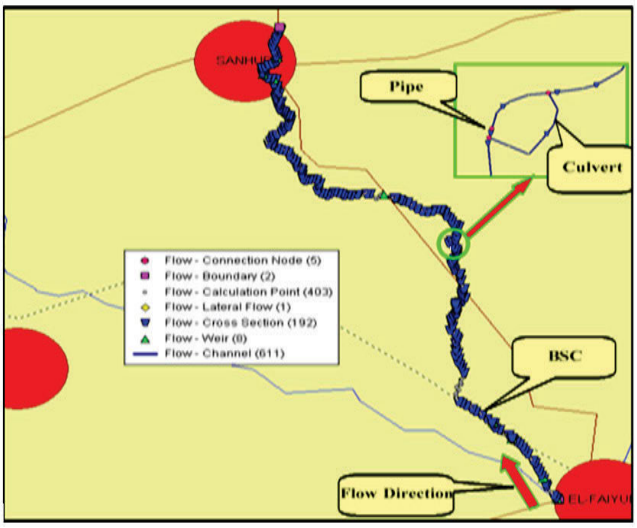

Fig. 7. Layout of the model network; BSC = The Bahr Sanhoor Canal; source: own elaboration

The cross-section data were available for separate points of the canal system. Throughout this study, an interval of 100 m was chosen; cross-sections were thus outlined within the model setup at every 100 m. Therefore, the total number of modelled cross-sections was 220. At some places, wherever the structures existed, cross-sections were specified for both upstream and downstream of them.

The initial conditions are defined as global values of water levels and flow discharges for the whole canal network or as local values at different distances of a selected canal. These initial conditions are identified 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/or structures, while the external boundary conditions include the specification of constant values for water level (h) or discharge (Q) or time series of varying values for Q or h at the boundary conditions. The daily discharges that were measured through the survey mission were defined as the upstream boundary in the time series database file, and the water level of −16.17 MSL was used at the downstream boundary condition.

Before running the simulation, the control factors, namely, simulation period, time step, stored data, and storage time, were defined. The simulation period was specified using start and end dates. Sobek checks the particular time and reads all given data as a time series during the simulation. The Courant number (C_n) was employed to determine the time step, which controls the simulation process and is computed as:

$$C_n = \frac{\Delta t\left(v + \sqrt{gH}\right)}{\Delta x} \tag{11}$$

where: Δt = the time step (s); v = the mean flow velocity ($\mathrm{m \cdot s^{-1}}$); H = the water depth (m); Δx = the dx-max.

In this study, the time step was 10 min, and the simulation period extended for 30 days.

In this analysis, the hydraulic resistance number was considered as the model calibration parameter. The Manning's roughness coefficient was selected as the resistance number used in this model. The Sobek model was calibrated against Manning's coefficient (n) by employing the measured water levels. For the simulation period of 30 days, a simulation time step of 10 min was used. The defined calibration period of was selected due to the availability of measured data along the length of the canal. After adapting the model, the BSC was divided into reaches and an initial n-value of 0.025 was assumed for all uncovered cross-sections; additionally, a value of 0.015 was specified for the covered reach based on the authors' experience in modelling. Employing this as an initial point, the n-value was gradually adjusted using the measured discharges and water levels until the associated errors between the measured and computed water levels were decreased. This process continued until the measured and computed water levels agreed on acceptable level. After many iterations, the errors were less than 1%, with an average calibrated n-value of 0.030 for the uncovered reaches and 0.014 for the covered reach. According to Figure 8, the simulated water levels are very close to the observed water levels. Respecting the overall adequacy of the model results, the model setup was considered calibrated.

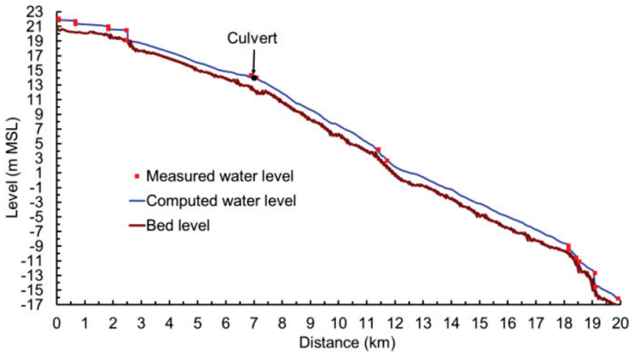

Fig. 8. Comparison between computed and measured water levels along the Bahr Sanhoor Canal; source: own elaboration

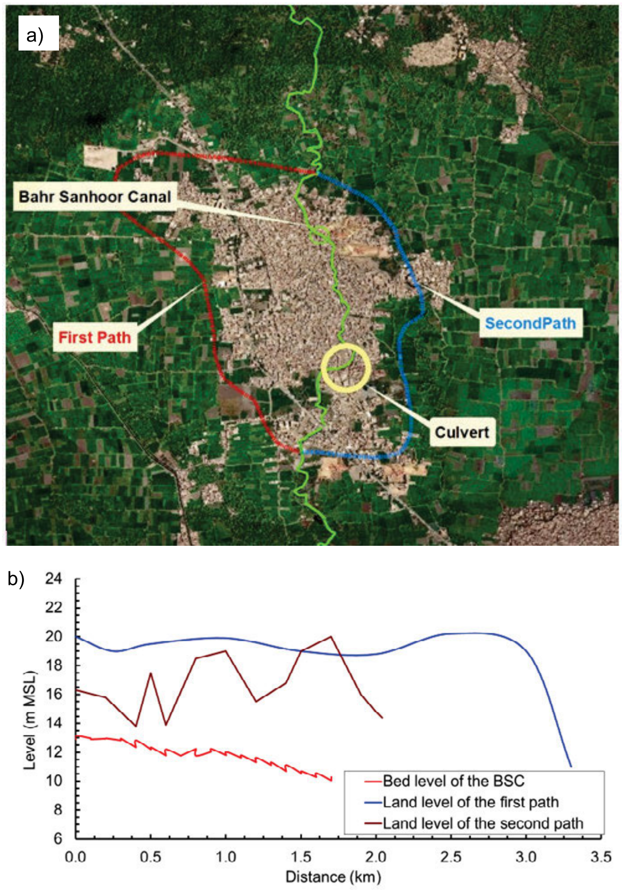

Fig. 9. Two proposed paths: a) plan view of the alternative paths, b) land levels of the two paths and bed levels of the Bahr Sanhoor Canal (BSC); source: own elaboration based on Google Earth

RESULTS AND DISCUSSION

GENERAL INFORMATION

The hydraulic design of the culvert was carried out, and it showed that the culvert could convey a discharge of 7.65 $m^3 \cdot s^{-1}$ with its current dimensions. The measured discharge through the culvert was 0.44 $m^3 \cdot s^{-1}$. This result confirmed that the culvert was blocked by solid waste due to its location in the residential area.

PROPOSED PATHS

To avoid culvert blockage and dredging difficulties in residential areas, a sustainable solution was proposed. The concept of this alternative for improving irrigation system management is to transport the BSC irrigation water away from the inhibited area. GIS and DEM, in combination with recent satellite images, were employed to establish a three-dimensional view of the study area and its land use. Several paths were examined on the right and left sides of the village. The most appropriate two paths are depicted in Figure 9.

Many paths were explored on the left side of the village, and the path with the least difference in the levels between the land level of the paths and the bed level of the BSC was selected. The proposed first path (on the left side) had a length of 3.30 km. The differences in the levels of this path were approximately 7.0 and 9.0 m at the start and end of the path, respectively. The required expropriation of land for this path was 6.55 ha. This alternative will be associated with high initial costs, requiring expropriation of lands and the installation of a number of weirs and pumping stations.

Several paths were also assessed on the right side of the residential area, and the path of the least differences in terms of the level with the Bahr Sanhoor Canal (BSC) was selected and is shown in Figure 9. The required length of the second path was approximately 2.040 km, which was less than the left path by approximately 1.260 km. The land level at the beginning of this path was 16.30 m MSL, and the corresponding bed level of the BSC was 13 m MSL. The average land level of the path was higher than the BSC's bed level by approximately 3.07 m, and the required expropriation land area is 4.05 hectares. The second path (on the right side) was economically recommended. The second path was hydraulically designed considering the typical cross-sections of the BSC, and it could freely convey the required designed discharge.

This alternative is permanent, sustainable and reasonable, but the decision makers should consider the financial hardships that the country faces before applying the alternative. Efforts are also needed to enforce the laws affecting irrigation networks, such as the transgressions observed in the study area, especially in the inhabited? region.

MODEL SCENARIOS

To operate the model after calibration, the maximum design discharge of 4.16 $m^3 \cdot s^{-1}$, which is equivalent to a crop water requirement of 52.8 m^3 per day per hectare, was used to predict the water levels along the BSC. The hydraulic performance of the canal was tested via five different scenarios (Tab. 1). The scenarios were investigated to determine the best solution to improve the carrying capacity of the canal system, especially at the reaches just upstream and downstream of the covered part. The scenarios also examined the appropriateness of the current dimensions of the box culvert to accommodate the designed discharge. Scenario 1 involved the present situation of the BSC, with a progressive build-up of sediments at some locations. Scenario 2 entailed the removal of solid waste and garbage by creating openings as manholes at the top of the box culvert. Another alternative to enhance the carrying capacity of the BSC, scenario 3, applied the same methods as those in scenario 2, in addition to dredging the canal reach just upstream of the covered part in the reach between km 5.000 and km 7.000 as well as dredging a 2-km reach located approximately 1 m downstream of the culvert. On the other hand, dredging cross-sections along a reach of 4 km just downstream of the culvert, in addition to the same measures taken in scenario 3, represented scenario 4. In contrast, scenario 5 was similar to scenario 3, except that the entire reach upstream of the covered part, for a distance of 7 km, was cleared.

Table 1. Scenarios designed for the BSC

Scenario	Description	Model inputs
1	current status	design discharges CSs
2	cleaning the opening of the box culvert	design discharges and current CSs
3	scenario 2 plus dredging 2 km just downstream and upstream the culvert	design discharges, dredged CSs by 1 m along the reaches 2 km upstream and downstream the culvert, and current CSs along the other reaches
4	scenario 3 plus dredging 2 extra km downstream the culvert	same as scenario 3 with two extra km downstream the culvert dredged to a bed level of 1 m less than current CSs
5	scenario 3 plus dredging the whole reach between the head regulator and the culvert	similar to scenario 3 with the whole reach upstream the culvert set to CSs dredged to a bed level of 1 m less than current CSs

Explanations: CSs = cross sections.
Source: source: own study.

- **Scenario 1. Investigation of the current status**

The model results of this scenario revealed that the maximum water level downstream of the head regulator is 22.30 m, corresponding to a design discharge of 4.16 $m^3 \cdot s^{-1}$. Figure 10 shows a comparison between the water levels at a discharge of 2.16 $m^3 \cdot s^{-1}$ and the computed water levels at the designed discharge. The figure indicates that the observed water levels are the maximum allowable levels that could be reached upstream of the culvert in the current situation. If these water levels are exceeded, the water will overflow the bank into the adjacent residential areas. Moreover, the computed water levels were higher than those observed during the survey by an average of 0.20 to 0.50 m along the whole reach upstream of the culvert. At km 7.000, just upstream of the culvert, the difference between the observed and computed water levels reached 1.10 m. The water level just upstream of the culvert was 15.45 m, which was higher than the bank level at this location. However, the water level reached 14.75 m just downstream of the covered reach. On the other hand, the designed water levels recorded higher values than the observed water levels along the reach just downstream of the culvert. The water level difference ranged from 0.74 m at a distance of 1.5 km downstream of the covered reach, which was very close to the bank level, to 0.3 m at the downstream end of the canal. The discharge flowing through the culvert was found to be 1.02 $m^3 \cdot s^{-1}$.

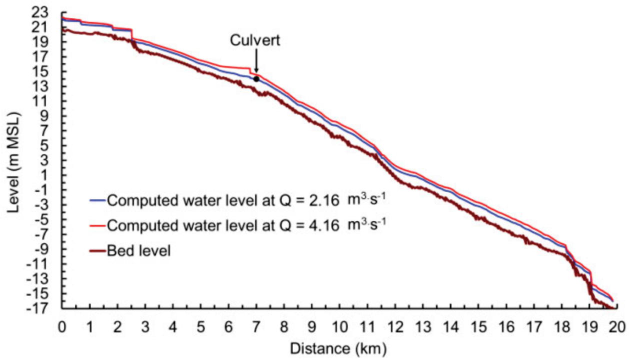

Fig. 10. Computed water levels of scenario 1 for observed and designed discharges; source: own elaboration

- **Scenario 2. Removal of waste from the culvert**

In this scenario, the water level just upstream of the culvert decreased by 0.65 m to reach a level of 14.80 after the cleaning process (Fig. 11). Nevertheless, this water level is still higher than the bank level. The water levels downstream of the culvert were similar to those computed in scenario 1.

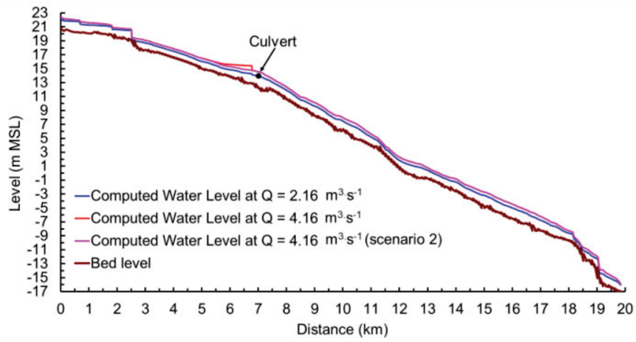

Fig. 11. Comparison between computed water levels before and after cleaning the culvert; source: own elaboration

- **Scenario 3. Removal of waste from the culvert and dredging a 2-km reach just upstream and downstream the culvert**

A comparison between this scenario and scenario 2 showed that the computed water levels decreased by 0.96 m to reach values of 13.84 m just upstream of the culvert and 13.76 m just downstream of the culvert (Fig. 12). This process changed the discharge in the culvert to a value of 2.84 $m^3 \cdot s^{-1}$. This value implies a 75% increase in the carrying capacity at the covered reach as a result of the dredging process compared to the impact of only clearing the culvert opening.

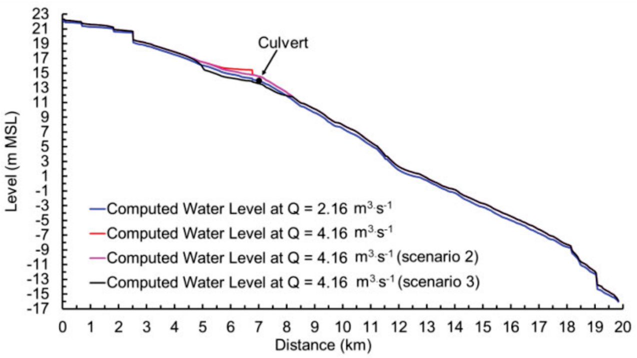

Fig. 12. Comparison between computed water levels of scenario 3 and scenarios 1 and 2; source: own study

- **Scenario 4. Same as scenario 3 plus dredging 2 extra km downstream of the culvert**

The model results of this scenario revealed that the upstream water levels were similar to those computed in scenario 3 (Fig. 13). This result implies that dredging more cross-sections downstream of the culvert had no effect on the water levels upstream of the culvert.

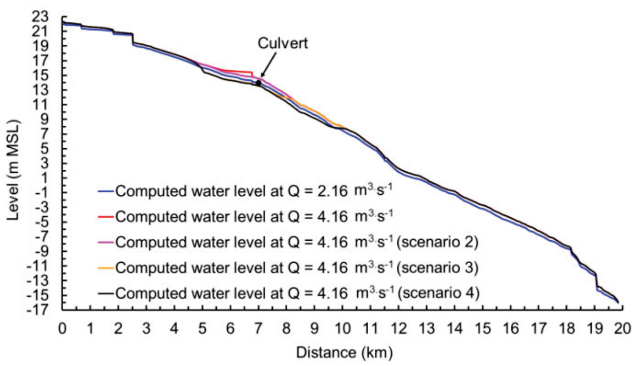

Fig. 13. Comparison between computed water levels of scenario 4 and scenarios from 1 to 3; source: own study

- **Scenario 5. Same as scenario 3 plus dredging the whole reach upstream of the culvert**

The output of this scenario showed that the water levels decreased by a range of 0.07 to 0.40 m in the distance between the head regulator and km 4.500 compared to those in scenario 4. However, the water levels along the reaches upstream of the culvert decreased by an average value of 0.30 m compared to those in scenario 3, reaching a level of 13.54 m just upstream of the culvert (Fig. 14). The water level just downstream of the culvert was 13.47 m. In this case, the culvert, with its current dimensions, was able to accommodate a discharge of 3.24 $m^3 \cdot s^{-1}$, which accounted for 78% of the designed discharge. Table 2 summarises the model results and costing execution for the 5 scenarios.

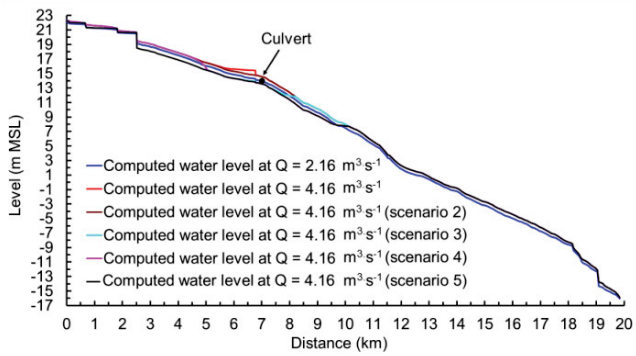

Fig. 14. Comparison between computed water levels of scenario 5 and scenarios 1–4; source: own study

CONCLUSIONS

In this paper, field measurements were employed to determine the appropriate solution to resolve the problem of reduced carrying capacity at the covered reach of the BSC and to avoid the risk of having water levels exceeding or very close to the bank levels due to blockages of this section. The main problem faced by the Bahr Sanhoor Canal (BSC) is that it passes through an urbanised area where residents' behaviour directly damages open canals crossing the rural regions. Two alternatives for improving irrigation system management were introduced. A local alternative was offered by applying a suitable numerical model to enhance the efficiency of the current status of the canal system. This temporary alternative recommended dredging the entire reach upstream of the culvert, starting from the head regulator and continuing for a distance of 7 km by 1 m, in addition to clearing waste from the culvert and dredging a distance of 2 km just downstream of the culvert (scenario 5). Correspondingly, the water level upstream of the covered part reached 13.54 m, though the designed discharge was 4.16 $m^3 \cdot s^{-1}$. This solution will lower the water levels by approximately 2 m, which is below the bank levels at an affordable cost. Additionally, it will help to avoid the risk of inundating the adjacent residential area.

Table 2. Summary of model results

Scenario	Water level just upstream the culvert (m)	Water level just downstream the culvert (m)	$Q_{culvert}$ ($m^3 \cdot s^{-1}$)	$Q_{culvert}$ / Q_{design} (%)	Situation of residential areas	Total cost (LE)
1	15.45	14.75	1.02	25	inundated	0.000
2	14.80	14.75	1.62	39	inundated	13,800
3	13.84	13.76	2.84	68	safe	346,600
4	13.84	13.76	2.84	68	safe	409,000
5	13.54	13.47	3.24	78	safe	736,600

Explanations: LE = Egyptian pound (LE100 ≈ USD6.4), *Q* = flow discharge.
Source: own study.

The second option, a sustainable and environmental alternative was considered to offer a new path in which the covered reach passed outside the residential area. This alternative is permanent and sustainable. Although the proposed second path to the right of the residential area is the long-term recommendation and is sustainable, any path of flow to either the left or right of the BSC will will require initial high costs, including expropriation of lands and the installation of a number of weirs and pumping stations. Thus, the decision makers should consider the financial hardships that the country faces; to justify the first alternative, it is essential to analyse the capital cost of providing a new path for the BSC and evaluate the sustainable and environmental benefits. Constructing adjacent embankments and/or berms of the irrigation networks should be prohibited, and efforts are also needed to enforce laws, such as transgressions, related to irrigation networks.

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REFERENCES

Abbott M. 1979. Computational hydraulics: Elements of the theory of free surface flows. London. Pitman Publishing Ltd. ISBN 9780273011408 pp. 324.

Abdelhaleem F. 2016. Discharge estimation for submerged parallel radial gates. Flow Measurement and Instrumentation. Vol. 52 p. 240–245. DOI 10.1016/j.flowmeasinst.2016.11.001.

Abdelhaleem F. 2017. Hydraulics of submerged radial gates with a sill. ISH Journal of Hydraulic Engineering. Vol. 23(2) p. 177–186. DOI 10.1080/09715010.2016.1273798.

Abdelhaleem F., Amin A., Basiouny M., Ibraheem H. 2020. Adaption of a formula for simulating bedload transport in the Nile River, Egypt. Journal of Soils and Sediments. Vol. 20(3) p. 1742–1753. DOI 10.1007/s11368-019-02528-8.

Abdelhaleem F., El-Ghoreb E., El-Belasy A. 2013. Managing water and salt balance of Wadi El-Rayan Lakes, El-Fayoum, Egypt. MEJ Mansoura Engineering Journal. Vol. 38(1) p. 45–63. DOI 10.21608/bfemu.2020.107891.

Adhikari B., Verhoeven R., Troch P. 2009. Appropriate rehabilitation strategy for a traditional irrigation supply system: A case from the Babai area in Nepal. Water Science and Technology. Vol. 60(11) p. 2819–2828. DOI 10.2166/wst.2009.721.

Al-Jayyousi R. 1999. Rehabilitation of irrigation distribution systems: The case of Jericho City. Water Resources Management. Vol. 13 (2) p. 117–132. DOI 10.1023/A:1008008517186.

Allam M., Allam G. 2007. Water resources in Egypt: Future challenges and opportunities. Water International. Vol. 32(2) p. 205–218. DOI 10.1080/02508060708692201.

Clemmens A., Holly F., Schuurmans W. 1993. Description and evaluation of program: Duflow. Journal of Irrigation and Drainage Engineering. Vol. 119(3) p. 724–734. DOI 10.1061/(ASCE)0733-9437(1993)119:4(724).

Contractor D., Schuurmans W. 1993. Informed use and potential pitfalls of canal models. Journal of Irrigation and Drainage Engineering. Vol. 119(4) p. 663–672. DOI 10.1061/(ASCE)0733-9437(1993)119:4(663).

Cunge J., Holly F., Verwey A. 1980. Practical aspects of computational river hydraulics. London. Pitman Publishing Ltd. ISBN 9780273084426 pp. 420.

Deltares 2019. Sobek. Hydrodynamics. Rainfall runoff and real time control. User Manual. Ver. 1 [online]. Delft. Deltares pp. 900. [Access 15.06.2020]. Available at: https://content.oss.deltares.nl/delft3d/manuals/SOBEK_User_Manual.pdf

El-Abd E., El-Osta M. 2014. Water logging in the new reclaimed areas Northeast El Fayoum, Western Desert, Egypt, Reasons and Solutions. Journal of Water Resource and Protection. Vol. 6(18) p. 1631–1645. DOI 10.4236/jwarp.2014.618147.

Elhamrawy A. 2018. Water and salt balance of Qarun and Wadi El Rayyan Lakes, Fayoum Governorate. PhD Thesis. Mansoura. Mansoura University, Faculty of Engineering pp. 185.

Elgamal M., El-Alfy K., Abdullah M., Abdelhaleem F., Elhamray A. 2017. Restoring water and salt balance of Qarun Lake, Fayoum, Egypt. MEJ Mansoura Engineering Journal. Vol. 42(4) p. 1–14. DOI 10.21608/BFEMU.2020.97674.

Froehlich D. 2008. Most hydraulically efficient standard lined canal sections. Journal of Irrigation and Drainage Engineering. Vol. 134 (4) p. 462–470. DOI 10.1061/(ASCE)0733-9437(2008)134:4(462).

Holly F., Parrish J. 1993. Description and evaluation of program: CARIMA. Journal of Irrigation and Drainage Engineering. Vol. 119(3) p. 703–713. DOI 10.1061/(ASCE)0733-9437(1993)119:4(703).

Ibrahim S., El-Belasy A., Abdelhaleem F. 2011. Prediction of breach formation through the Aswan High Dam and subsequent flooding downstream. Nile Water Science and Engineering Journal. Vol. 4(1) p. 99–111.

Islam A., Raghuwanshi N., Singh R. 2008. Development and application of hydraulic simulation model for irrigation canal network. Journal of Irrigation and Drainage Engineering. Vol. 134(1) p. 49–59. DOI 10.1061/(ASCE)0733-9437(2008)134:1(49).

Merkley G., Rogers D. 1993. Description and evaluation of program CANAL. Journal of Irrigation and Drainage Engineering. Vol. 119 (3) p. 714–723. DOI 10.1061/(ASCE)0733-9437(1993)119:4(714).

Merkley G., Walker W. 1991. Centralized scheduling logic for canal operation. Journal of Irrigation and Drainage Engineering. Vol. 117(3) p. 377–393. DOI 10.1061/(ASCE)0733-9437(1991)117:3(377).

Molden D., Burton M., Bos M. 2007. Performance assessment, irrigation service delivery and poverty reduction: Benefits of improved system management. Journal of Irrigation and Drainage. Vol. 56(2–3) p. 307–320. DOI 10.1002/ird.313.

Reddy J. 1990. Local optimal control of irrigation canals. Journal of Irrigation and Drainage Engineering. Vol. 116(5) p. 616–631. DOI 10.1061/(ASCE)0733-9437(1990)116:5(616).

Rogers D., Merkley G. 1993. Description and evaluation of program USM. Journal of Irrigation and Drainage Engineering. Vol. 119 (3) p. 693–702. DOI 10.1061/(ASCE)0733-9437(1993)119:4(693).

Schuurmans W. 1993. Description and evaluation of program MODIS. Journal of Irrigation and Drainage Engineering. Vol. 119(3) p. 735–741. DOI 10.1061/(ASCE)0733-9437(1993)119:4(735).

Shawky Y., Nada A., Abdelhaleem F. 2013. Environmental and hydraulic design of thermal power plants outfalls case study: Banha thermal power plant, Egypt. Ain Shams Engineering Journal. Vol. 4(3) p. 333–342. DOI 10.1016/j.asej.2012.10.008.

Veer M., Wormgoor J., Girgs R., Wolters W. 1993. Water management in tertiary units in Fayoum Irrigation and Drainage Systems. Vol. 7 p. 69–82. DOI 10.1007/BF00880909

Wahl T., Lentz D. 2011. Physical hydraulic modelling of canal breaches. Hydraulic Laboratory Report HL-2011-09. Denver, Colorado. U.S. Department of the Interior Bureau of Reclamation pp. 56.