American Journal of Engineering and Technology Management
2016; 1(4): 65-77
http://www.sciencepublishinggroup.com/j/ajetm
doi: 10.11648/j.ajetm.20160104.13

A Study on Scour for Irrigation Canals in Egypt, "Case Study: The First Reach of El-ibrahimeya Canal"

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To cite this article:
Mohammed Gamal Abdalla. A Study on Scour for Irrigation Canals in Egypt, "Case Study: The First Reach of El-ibrahimeya Canal". *American Journal of Engineering and Technology Management.* Vol. 1, No. 4, 2017, pp. 65-77. doi: 10.11648/j.ajetm.20160104.13

Received: October 22, 2016; **Accepted**: November 5, 2016; **Published**: January 20, 2017

Abstract: A general scour was observed in the first reach of El-Ibrahimeya canal from its intake to km. 60. Also, a local scour was observed around the bridge piles across the canal at km. 23 which may lead to a failureto the bridge because the condition of interred pile length that equals one and half of the free pile length was not satisfied. Moreover, the designed level of the bridge equals (42.15) while the current bed level equals (40.40) that represent a seriousness for the bridge safety. The main objective of this research paper is to study the scour phenomenon generally along the reach under study and for the bridge, especially. A hydrographic survey for canal under study has been carried out. Also, the required data was obtained and analyzed, including discharge values, soil samples, and water levels at different sites along the canal. A one-dimensional mathematical model, namely (SOBEK-1D) was used to simulate the reach under study based on different Scenarios to determine the possible different solutions for the existing problem. A complete design process was carried out for the suggested protection layer related to km. 23 bridge against scour.

Keywords: Scour, El-ibrahimeya Canal, Mathematical Model, and SOBEK-1D

1. Introduction

Scour has been known as a severe hazard to the performance of flowing water in any stream. It can be defined as a natural phenomenon caused by the flowing water in rivers and streams. It leads to the lowering of the stream bed level by the erosion of water. It can either be caused by normal or flood flow. In other words, it can occur under any flow conditions but the scour effect is higher in case of larger flow. The characteristics of scour and fill in the riverbed during a flood have significant relation to the riverbed stability [1]. It is one of the main causes of bridge failures. Flood flow in natural rivers scours the river bed and creates large holes around bridge piers that gradually extend beneath them, eventually destroying them. Scour depth can be measured by the amount of reduction below the designed dimensions of the channel while the depression or the void caused when the sediment is washed away is called the scour hole, figure (1). The total scour for any stream can be classified into three components: general scour, contraction scour, and local scour which can be divided into sub-divisions as shown in figure (2) [2].

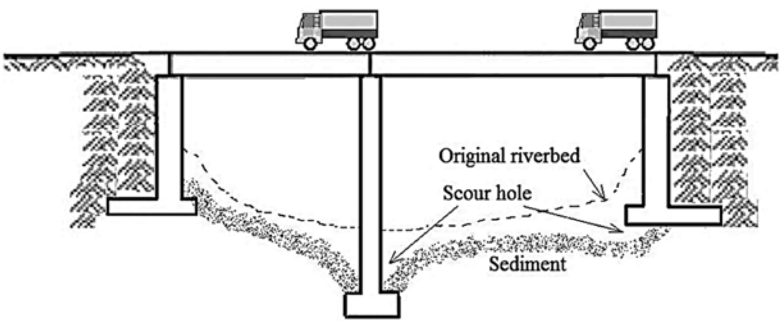


Figure 1. *A detailed sketch for a scour hole.*

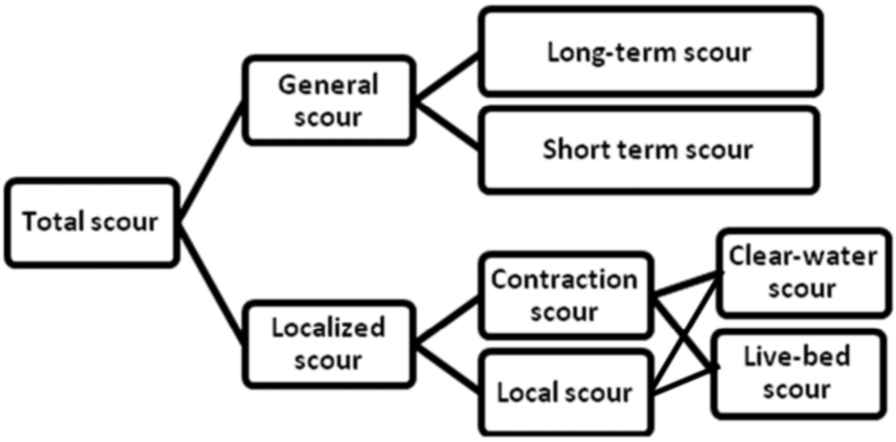


Figure 2. *A schematic illustration of scour types.*

On one hand, General scour results from the change in a channel regime through the degradation of the bed level due to natural or human causes that lead to overall lowering of the longitudinal profile of the river channel. It is classified into long-term and short-term scour that are differentiated by the period of composing scour. On the other hand, localized scour is attributed to the existence of riverine structures, such as bridges, which can be definitely divided into contraction and local scour. Contraction scour is caused by reduction of the flow area due to natural or human means as demonstrated in figure (3) that leads to higher values of average flow velocity, and consequently the increase of the erosive forces on the channel bed. These forces finally cause the channel boundary to be lowered.

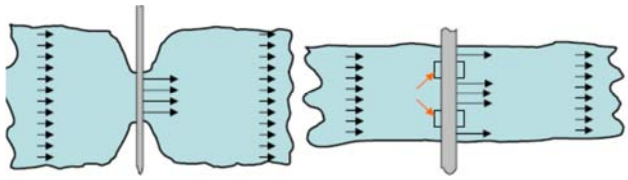


Figure 3. *Mechanism of contraction in open channel.*

Local scour refers to the sediment removal around bridge foundation, namely abutments, or piers, or piles. It occurs because of the flow acceleration at the location of the bridge foundation due to the interaction between water and bridge foundation. This interaction results in vortices that cause a scour hole around bridge foundation as illustrated in figure (4).

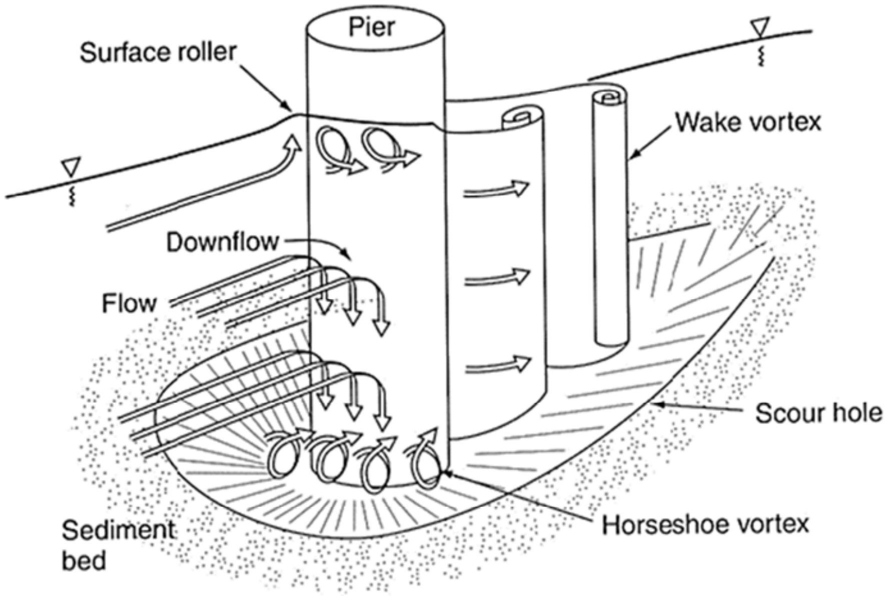


Figure 4. *A scour hole composed due to flow vortices [3].*

The overall scour at any position in the channel is the summation of all the previous types of scour if they already existed. When considering scour, it is essential to distinguish between cohesive and non-cohesive materials that form the channel boundary. Many research studies by several investigators have been carried out including field and laboratory works to accurately study the scour process and especially estimating the bridge scour using either empirical equations or neural networks.

2. Literature Review

Ettema et al. presents a review on the scour conditions at the bridge abutments and the difficulties that are faced during the accurate estimation of scour [4]. The previous studies related to abutments scour did not recognize the different forms of scour at the bridge abutments and consequently the estimation of scour length was inaccurate, so the design process of bridges was poor. The review presents some substantial resolved scaling issues associated with laboratory experiments. These issues concern parameters that can be neglected and these parameters are not included in the relationships used for predicting scour depth.

Ghorbani made a field study of scour at bridge piers in case of flood plain rivers [5]. It was based on the field evaluation of the pier failures related to scour in cohesion less-bed rivers. The different hydraulic effects, including flow velocity, flow depth, bridge pier geometry, and sediment characteristics such as specific gravity, particle size, angle of internal friction, and particle size distribution were considered and the scour depth was related to these variables in order to estimate it.

Lu et al. perform both field measurements and simulation of scour depth of bridges constructed in fluvial rivers during floods based on a case study, namely the Si-Lo Bridge in the lower Cho-Shui River, the longest river in Taiwan [6]. This study is very important for cost-effective bridge foundation design. The collected field data was used to validate the applicability of the proposed model. The results showed that the local scour formulae may lead to overestimate of the local scour depth.

Borghei and Sahebari predicted the scour properties at river junctions [7]. The discharge ratio of the tributary to the main branches, the junction angle, the ratio of mean downstream velocity threshold velocity are the important non-dimensional variables. To estimate the scour depth, a relationship between it and the non-dimensional variables was derived. It is recommended to study this problem on the basis of laboratory experiments in further studies.

Tulimilli et al. used a 3D software, namely Componential Fluid dynamics (CFD) to simulate the scour pattern in experimental flumes [8]. The software displayed the bed shear stress distribution to accurately estimate the bed levels displacements. The procedure provides a good foundation to use the software for natural channels.

Lu et al. predicted the maximum general scour depth during a flood for intermittent rivers [9]. A high efficient numbered brick column laying method was used based on successfully measured short-term general scour data during typhoon-induced floods for both gravel-bed and sand-bed reaches in the steep intermittent rivers in Taiwan. Based on the experimental results, the scoured flow depth formula was developed as follows which gave better predictions to estimate the scour depth compared to the one given by Blench [10]:

$$d_s = 1.26 * \left(\frac{q^{0.8} * S_O{}^{0.27} * \sigma_g{}^{0.74}}{d_{50}{}^{0.23}}\right) \quad \text{(Lu et al., 2012)} \qquad (1)$$

$$d_s = 1.23 * \left(\frac{q^{2/3}}{d_{50}{}^{1/12}}\right) \quad \text{(Blench, 1969)} \qquad (2)$$

where:

d_s = scour depth;

q = discharge per unit width;

S_o = longitudinal bed slope;

σ_g = geometric standard deviation of bed material; and

d_{50} = median sediment size of bed material.

Roca and Whitehouse presented both a framework and methods to develop a probabilistic scour risk assessment using fragility curves to account for uncertainty in input variables, prediction methods and performance of structures [11]. It is important to understand the risks related the possible riverbed movements in the lateral and vertical directions to define the different protection works against scour.

Singh and Maiti carried out series of laboratory experiments to study the scour properties around a circular obstruction [12]. The developed scour hole in the cohesive material took the shape of cone with a deeper depth close to the pier nose and extended in the downstream direction. The kinematic and dynamic boundary conditions that affect the exact scour depth make the process of field values estimation as a complex task so the laboratory simulation is of full control on all parameters. Relationships were derived to calculate the exact scour depth around circular piers depending on shape and size of the pier, soil properties of the channel bed, Froude number of flow, and vortex formation and nature around the piers.

El Barbary and El-Sersawy carried out a hydraulic analysis and scour evaluation for metro tunnel river crossing which was constructed to cross the hydraulically complex reach of the Nile River located downstream El-Malik El-Saleh and El-Giza Bridges [13]. The river reach at the location of the tunnel is divided into two channels, the eastern channel (secondary channel) is crossed by El-Malik El-Saleh Bridge and the western channel (main channel) is crossed by El-Giza Bridge. One and two dimensional models for simulation the flow pattern, evaluating the expected morphological changes at the tunnel location using the historical and recent data, and estimation of the scour depths due to release different scenarios of discharges were applied. It was concluded that the combination of one and two dimensional models provides a flexible method to evaluate the morphological changes and a good diagnostic tool to predict the design scour with reasonable confidence.

Luh and Liu showed that the scour monitoring system has demonstrated the capability to measure scouring depth [14]. The field results indicated that scour monitoring system using scouring sensor and antenna stand has the potential for real world application.

Prendergast and Gavin paved the way for low-maintenance non-intrusive structural health monitoring to detect and monitor scour development around structures [15]. Instrumentation related to traditional scour monitoring often requires expensive installation and maintenance. Also, data interpretation from these instruments is difficult and time consuming.

Capape and Martin-Vide made an evidence of transient scour and fill [16]. The transient scour and fill refers to the general scour and fill in a riverbed due to the passage of a flood. Large sand-bed Pilcomayo River data was used. The results showed that the velocity leaded to loops with large hysteresis which is inexplicable by the unsteady flow over a fixed bed.

3. Site Description

El-Ibrahimeya canal is considered as one of the main canals in Egypt. Its length is about 267 km with its branches. It is considered the greatest infrastructure that was constructed in the era of the Khedive Ismael under the rule of Ottomans. It was named after the Khedive Ibrahim, the father of the Khedive Ismael. Also, it is considered one of the best irrigation structures in the world in that time. It supplies Assuit, Beni-Suef and Minya Governorates with water. The reach under study is about 60 km length and takes its water just before Assuit barrage from The Nile River in Assuit Governorate and it ends just before Dairout barrage as shown in figure (5). Also, the figure showed the location of the bridge under study at km 23.

Figure 5. *A google earth photo for reach under study.*

4. Data Collection

The hydrographic survey of the reach under study was carried out by Hydraulics Research Institute "HRI" of the National Water Research Center, Ministry of water resources and Irrigation, Egypt. Using the provided echo-sounder light boat connected to digital global positing system (DGPS), to record each data set point consisting of X and Y positions as well as the flow depth at an interval of one second on the equipped data logger. The hydrographic survey was carried out along the length of the channel by making cross sections each 100 m. The bray-stoke type current meter was used for velocity measurements provided with counters and timers. The discharge values were measured at five cross sections of km 6, 26, 41, 47, and 53. The discharge value equals 565 m^3/s at El-Ibrahimeya intake during the measuring process while the discharge value equals 350 m^3/s just before Dairout regulator. The measured water surface slope equals 5.8 cm/km as the water level is (49.90) at El-Ibrahimeya intake while the water level is (46.42) at Dairout regulator. Bed material sampler was used to get three soil samples for each cross section and these samples were analyzed to plot grain size distribution curves and obtaining the properties of each sample.

5. Data Analysis

The obtained data was analyzed to get the best use of it. Table (1) illustrates the design data for El-Ibrahimeya canal which shows the bed width and level at different locations. Table (2) shows the maximum and minimum discharges for El-Ibrahimeya canal intake from 1955 to 1965. It is obvious from the table that the maximum discharges range from 428.24 m^3/s to 820.60 m^3/s while the minimum ones vary between 19.68 m^3/s and 57.87 m^3/s.

Table 1. *Design data for some cross sections along the length of El-Ibrahimeya canal.*

km.	Bed width (m)	Bed level (m)
0.00	55	+(43.50)
28.60	55	+(41.79)
59.40	60	+(40.20)
60.60	60	+(40.15)

Table 2. *Maximum and minimum discharges in different years for El-Ibrahimeya canal.*

Year	Maximum discharge (m^3/s)	Minimum discharge (m^3/s)
1955	802.08	34.72
1956	820.60	57.87
1957	811.34	57.87
1958	820.60	19.68
1961	820.60	—
1963	810.19	57.87
1964	798.61	34.72
1965	428.24	34.72

Figure (6) demonstrates a comparison between the measured longituduinal section of El-Ibrahimeya in 1960 and the design one. It is obvious from the figure that:

- There is a scour from km. 0 to km. 10 ranged from 1 m to 2.25 m. The actual level increases with a distance 0.3 m more than the design one from km. 10 to km. 13
- The actual level gradually decreases from km. 13 until the distance between the actual and design level equals 1.5 m at km. 16.
- The bed level reached the design level at km. 17
- The actual level increases more than the design level from km. 30.5 to km. 32.0 until it reaches a maximum value equals 0.6 m.
- The actual level decreases more than the design one from km. 32 to km. 40 until it reaches a maximum value of 0.5 m.
- From km. 40 to km. 46, the actual level increases more than the design one with a value ranged from 0.5 m to 1.4 m.
- The actual level decreases more than the design one from km. 46 to km. 55 until it reaches a maximum value equaled 0.9 m.
- From km. 55 to km. 60, the actual level increases more than the design one until it reached a maximum value of 1.5 m.

Figure (7) illustrates a comparison between the current and design longitudinal sections of El-Ibrahimeya canal. It is apparent from the figure that:

- There is a general scour in El-Ibrahimeya canal from km. 0 to km. 30 and the difference between the current and design bed levels reached a distance equaled 2.5 m.
- Also, the current scour increases more than that in 1960 with a distance ranged between 0.5 m and 1.5 m.
- The current scour decreases more than the design one with a distance of 1 m from km. 30 to km. 40 and it increases more than that in 1960 with an average distance equaled 0.6 m.
- From km. 40 to km. 47, the current bed level approaches the design one with an average distance equaled 0.3 m while this distance was 1.4 m. in 1960.
- From km. 47 to km. 55, the current scour reaches a distance of 0.5 m but this distance was 0.9 m in 1960.
- From km. 55 to km. 60, the current scour reaches a distance of 1.5 m while this distance was 0.7 m in 1960.
- The scour reaches a distance of 1.75 m at the bridge located at km. 23.

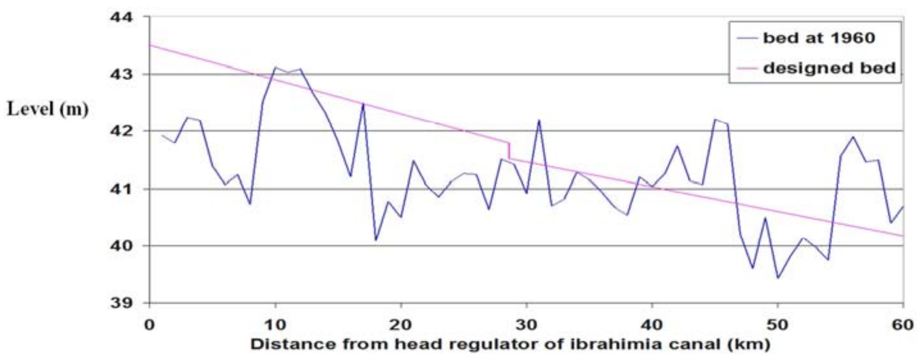


Figure 6. *Comparison between the measured longitudinal section in 1960 and the design one for El-Ibrahimeya canal.*

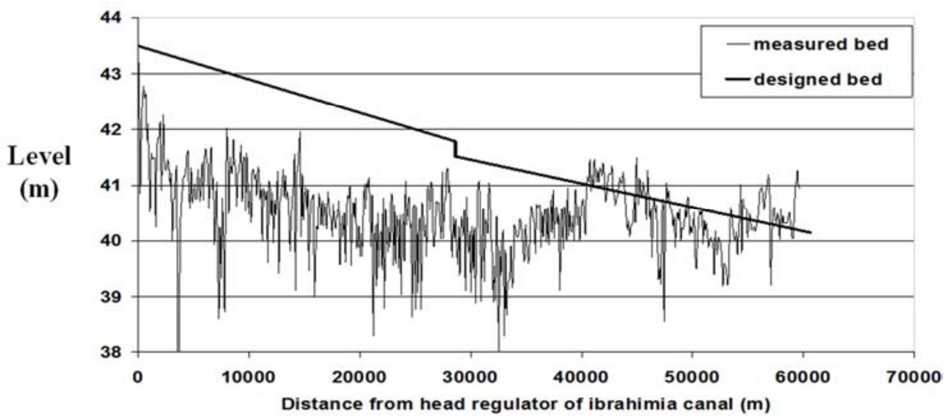


Figure 7. *Comparison between the current longitudinal cross section and the design one for El-Ibrahimeya canal.*

From the previous results, the scour occurred because of the flood discharges that pass through the cabal before 1960, before the construction of High Aswan Dam. From the previous results, it can be noted that the current scour increased more than that before 1960 because of the emergency discharges between 1960 and 1964 (During the construction of High Dam).

A comparison between the current and design cross sections of El-Ibrahimeya canal. Two examples of these comparisons at km. 1.00 and km. 23.00, the location of the bridge under study are shown in figures (8) and (9).

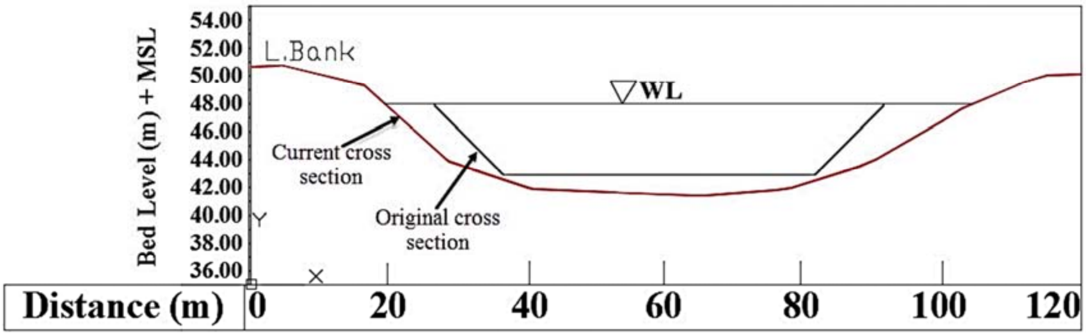


Figure 8. *Comparison between the current cross section and the design one of El-Ibrahimeya canal at km. 0.*

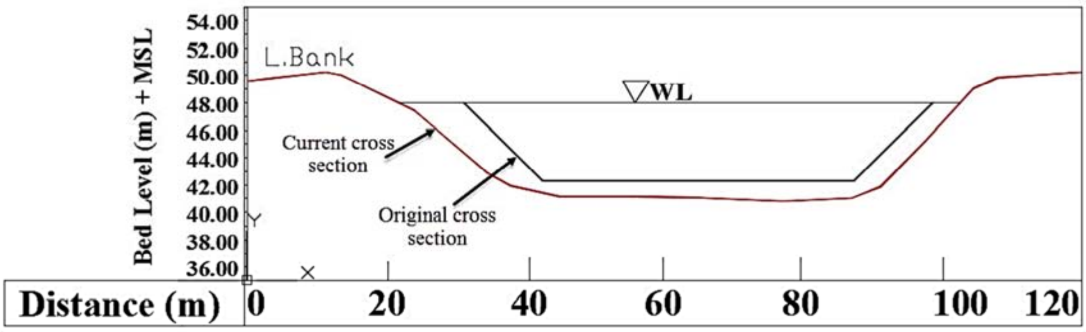


Figure 9. *Comparison between the current cross section and the design one of El-Ibrahimeya canal at km. 23.*

By comparing the maximum and minimum water levels of El-Ibrahimeya canal at different years, the water levels relatively approach the design ones. The maximum water level just after the intake of the canal equaled (50.14) while the maximum one just before Dairout regulator equaled (46.25). Tables (3) and (4) show the maximum and minimum water levels at El-Ibrahimeya canal intake and Dairout regulator at different years.

Table 3. *Maximum and minimum water levels for El-Ibrahimeya canal intake in different years.*

Year		2003		2004		2005		2006	
		Max.	**Min.**	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**	**Min.**
Water levels (m)	In front of the gates	(50.22)	(47.68)	(50.28)	(47.60)	(50.28)	(47.11)	(50.42)	(46.40)
	Behind the gates	(49.86)	(44.00)	(49.90)	(44.00)	(49.90)	(44.10)	(50.05)	(45.60)
Discharge (m^3/s)		454.86	23.15	457.18	81.02	457.18	23.15	474.54	17.36

Table 4. *Maximum and minimum water levels just before Dairout regulator of El-Ibrahimeya canal in different years.*

Year		2003		2004		2005		2006	
		Max.	**Min.**	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**	**Min.**
Water levels (m)	In front of the gates	(45.98)	(42.60)	(46.00)	(43.00)	(45.97)	(42.40)	(46.15)	(43.60)
	Behind the gates	(45.00)	(41.25)	(44.98)	(41.60)	(44.98)	(41.35)	(45.15)	(42.60)
Discharge (m^3/s)		168.52	23.15	167.15	34.72	167.15	37.72	178.78	49.90

Figures. (10) and (11) demonstrate comparisons between the maximum water levels and the left and right banks, respectively. From the comparison, it is clear that the water approaches the left bank with a distance equaled 10 cm from km. 10 to km. 30 while the water approaches the right bank with a distance of 25 cm from km. 10 to km. 25

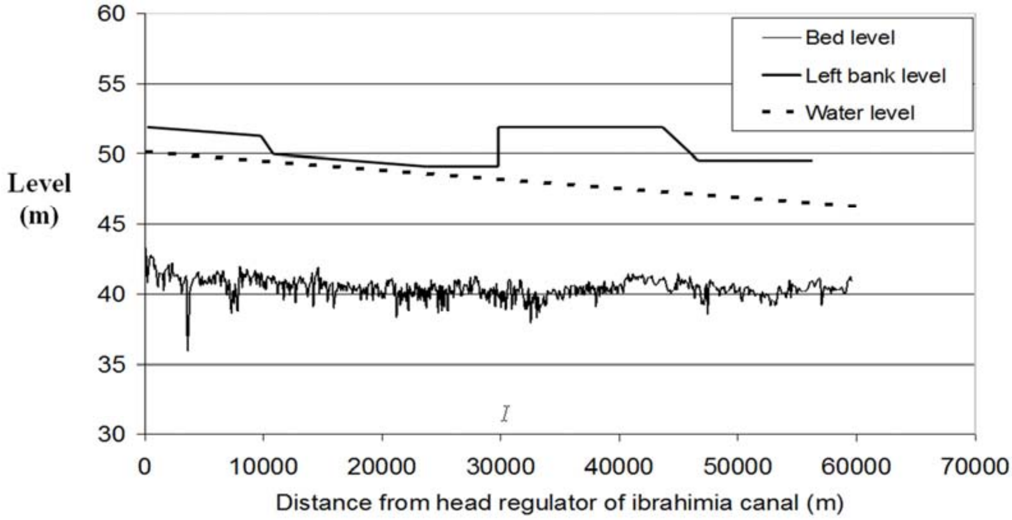


Figure 10. *Comparison between maximum water and left bank levels of El-Ibrahimeya canal.*

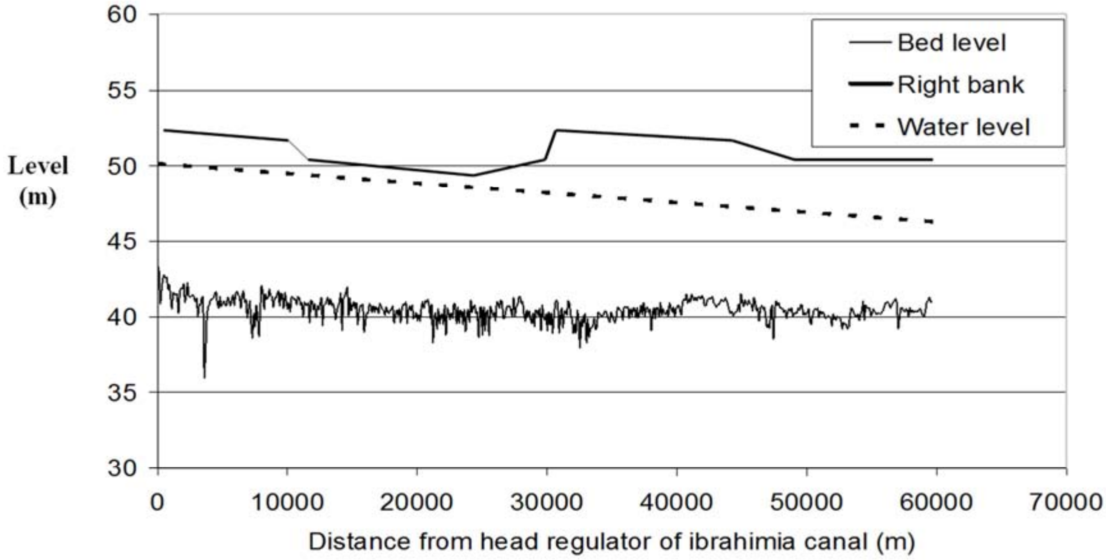


Figure 11. *Comparison between maximum and right bank levels of El-Ibrahimeya canal.*

6. Governing Equations and Model Set up

6.1. Basic Equations

The following equations are used:
Continuity Equation

$$\frac{\partial (h)}{\partial t} + \frac{\partial (vh)}{\partial y} + \frac{\partial (uh)}{\partial x} = 0.0 \quad (3)$$

- Momentum equation in X-direction

$$\frac{\partial (uh)}{\partial t} + \frac{\partial (u^2 h)}{\partial x} + \frac{\partial (uvh)}{\partial y} = -gh\frac{\partial H}{\partial x} - \frac{\tau_x}{\rho} + D_x \quad (4)$$

- Momentum equation in Y-direction

$$\frac{\partial (vh)}{\partial t} + \frac{\partial (v^2 h)}{\partial y} + \frac{\partial (uvh)}{\partial x} = -gh\frac{\partial H}{\partial y} - \frac{\tau_y}{\rho} + D_y \quad (5)$$

where:
τ_x= shear stress at X-direction;
τ_y= shear stress at Y-direction;
v = velocity in Y-direction;
u = velocity in X-direction;
ρ = water density;
g = gravitational acceleration.
t = time; and
h = water depth at any point.

6.2. Calibration Process

To calibrate the existing model of canal under study, the discharges and water levels measured in the field were used in order to determine the suitable Manning's roughness coefficient value. The results show that a relatively coincidence between the measured water levels and the calculated ones based on the model as depicted in figure (12), so the model can be perfectly used to give good results.

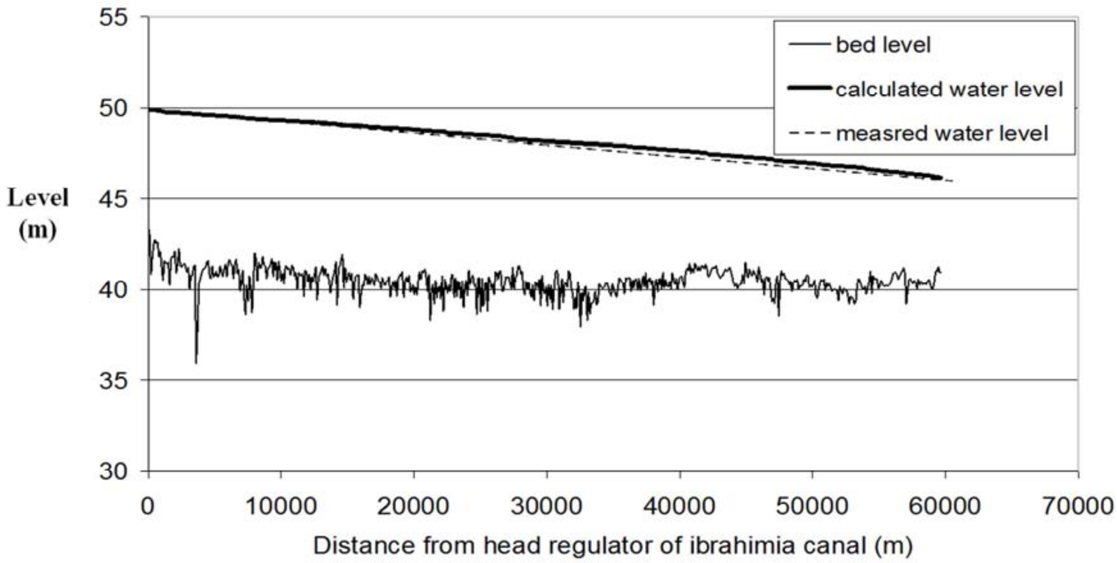


Figure 12. *Comparison between the calculated water levels from the model and the measured ones.*

7. Working Scenarios Using the Model

The model was operated after the calibration process to determine the different water levels and velocity values along the canal length for maximum or minimum discharge values before and after foundation protection for the bridge at km. 23, respectively. From the velocity results, the possible scour locations could be determined. The following scenarios could be given depending on the water levels just before and after the head and control regulator at the beginning and end of the reach as boundary conditions which would include flood discharges in the future as follow:

7.1. Scenario No. (1)

This scenario was used to test the canal efficiency at maximum designed water levels so that the expected discharge and velocity values could be expected. Table (5) represents the input and output values for this scenario. Figure (13) demonstrates the velocity distribution along the length of the reach under study.

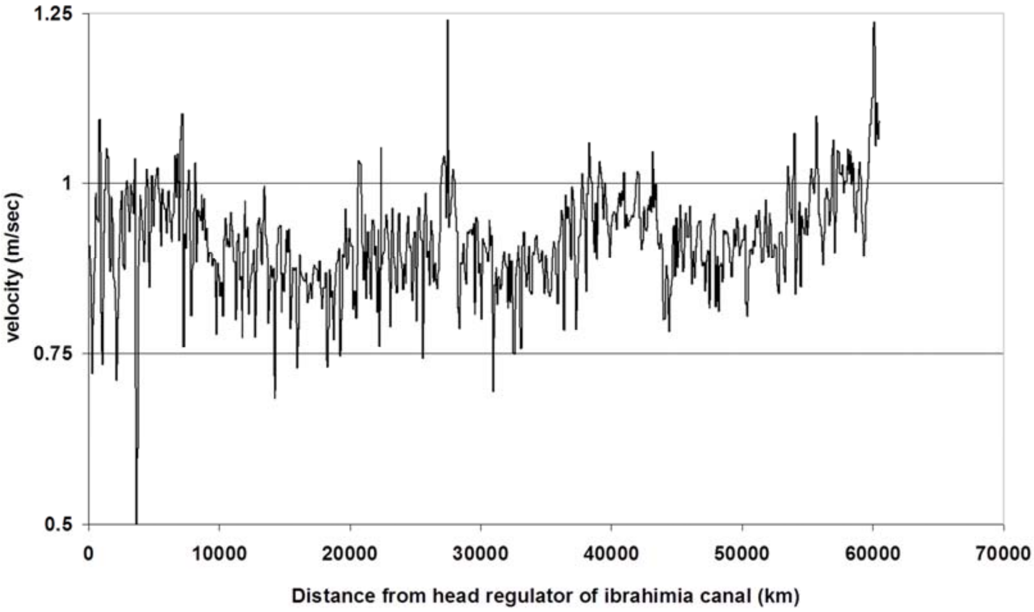


Figure 13. *Velocity distribution along the length of El-Ibrahimeya canal in case of maximum discharges.*

Table 5. *Input and output variables of the model for scenario No. (1).*

Boundary conditions		Model results		
Water level just after head regulator (m)	Water level just before control regulator (m)	Max discharge (m^3/s)	Velocity (m/s)	
			Velocity range	Prevailing velocity
(50.14)	(46.25)	490	(0.75 - 1.00)	0.85

From the table and figure results, it is obvious that the velocity values are accepted ones for this canal of sand bed but the maximum velocities along the reach under study are in a few places and definitely at the bridges locations. The maximum velocity for the bridge at km. 23.00 reached 1.5 m/s. Referring to figures (11), (10) and (13), it is noted that the maximum water levels approach the left bank with a distance equals 10 cm from km. 10 to km. 30 and approaches the right bank with a distance equals 25cm. from km. 10 to km. 25

7.2. Scenario No. (2)

In this scenario, minimum water design levels were represented in the model to determine the predicted discharge and velocity values along the reach under study as illustrated in table (6). Velocity distribution along the reach under study is shown in figure (14).

Table 6. *Input and output variables of the model for scenario No. (2).*

Boundary conditions		Model results		
Water level just after head regulator (m)	Water level just before control regulator (m)	Max discharge (m^3/s)	Velocity (m/s)	
			Velocity range	Prevailing velocity
(48.50)	(46.00)	306.6	(0.60 – 0.80)	0.70

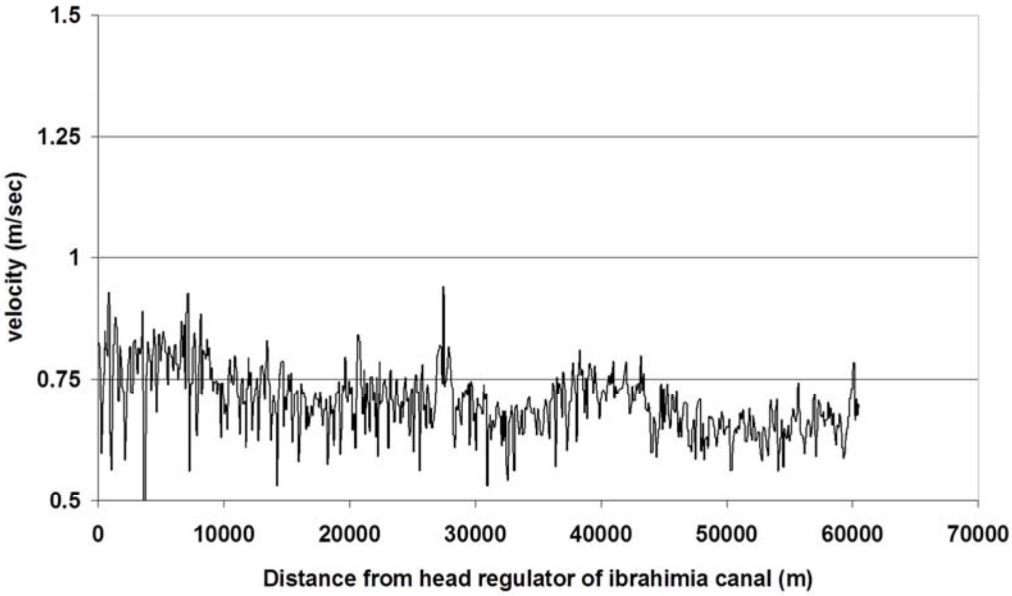


Figure 14. *Velocity distribution along the length of El-Ibrahimeya canal in case of minimum discharges.*

It is apparent from the table and figure results that the velocity values are agreeable for the reach under study with sand bed soil.

7.3. Scenario No. (3)

This scenario represents a test for the canal when the bridge foundation was protected with a well-graded rock filter to bed level equaled (42.15) which equals the design level. The boundary conditions and the results of the model are demonstrated in table (7) and the velocity distribution along the reach under study is depicted in figure (15).

Table 7. *Input and output variables of the model for scenario No. (3).*

Boundary conditions		Model results		
Water level just after head regulator (m)	Water level just before control regulator (m)	Max discharge (m^3/s)	Velocity (m/s)	
			Velocity range	Prevailing velocity
(50.14)	(46.25)	485	(0.75 - 1.30)	0.85

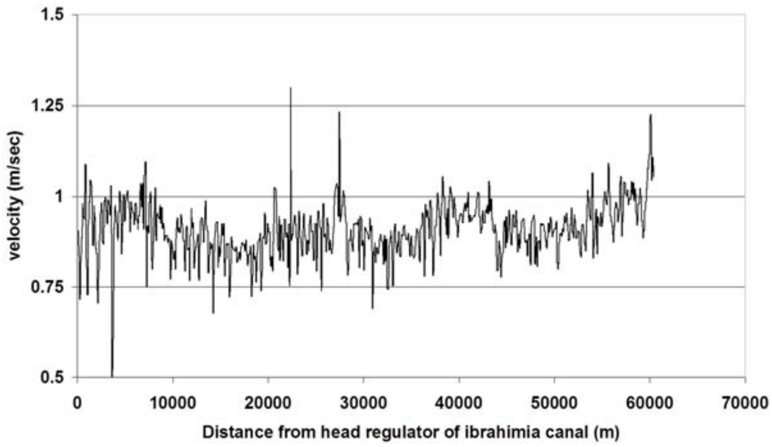


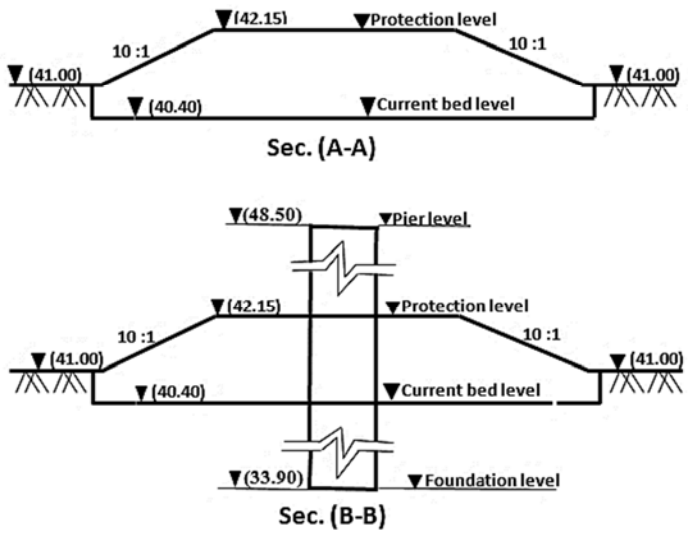
Figure 15. *Velocity distribution along the length of El-Ibrahimeya canal with the protection of bridge foundation in case of maximum discharges.*

Based on the previous table and figure, it is clear that the range of velocity is relatively allowable but the maximum velocity is recorded as 1.2 m/s at the location of the bridge at km. 23. Also, the distance between the maximum water levels and the left bank was given as 10 cm from km. 10 to km. 30 while the distance between the maximum water levels and the right bank was observed as 25 cm from km. 10 to km. 25. From the previous results of operating scenarios, the bridge located at km. 23 needs a protection layer against scour.

8. Protection of Bridge Foundations

In order to verify the condition of embedded length of bridge piles at km. 23, the foundation level at this length should be increased by a distance equaled 1.75 m above the current level. Also, the suggested protection should extend for a distance 15 m upstream and downstream the bridge. Then, longitudinal slopes are used to connect the suggested protection level to the current level. Figure (16) shows the location of the desired protection as well as a longitudinal section through this protection. As a result of the previous suggested protection, the model results show that the velocity at the bridge location increases from 1.05 m/s to 1.20 m/s so a suggested protection of graded rock is required at the bridge location to prevent the scour process.

In order to determine the suggested materials to protect the canal bed upstream and downstream the bridge located at km. 23, grain size distribution curves should be known. Also, the mean velocity at this location should be given to perfectly design the different layers of the protection. From the analysis of bed material samples at the bridge location, it is clear that the soil is fine sand with d_{50} equals 0.15 mm while the max predicted mean velocity equals 1.20 m/s from the mathematical model used in the study so the velocity used in the design process equals 1.8 m/s with a safety factor equals 1.5 and the design process is carried as follows:



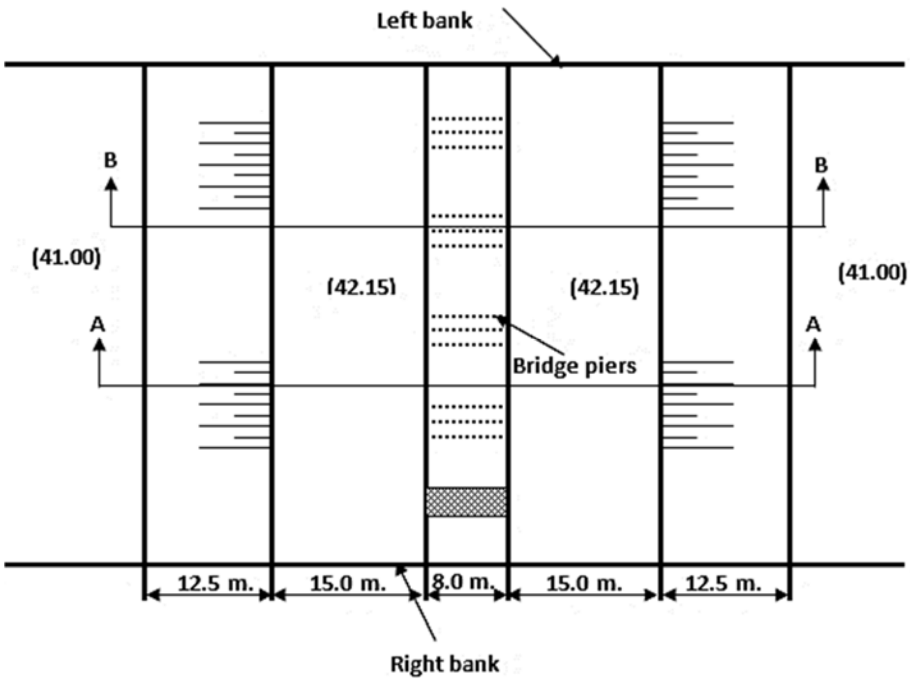


Figure 16. *Suggested protection layer against scour at the bridge location.*

9. Design of Protection Layers

9.1. Surface Protection Layer

In order to design this layer to resist the flow velocity and to calculate d_{50} for this layer, Scobey diagram and U. S. Army Corps of Engineers equation are used as follows [17]:

$$U = C\,[2g\,(S_s - 1)]^{1/2}\, d_{50}^{1/2} \quad (6)$$

where:

U = flow velocity;

S_s= specific gravity of the stone; and

C = Isbash's turbulent coefficient equaled 0.86 for high turbulent level flow and 1.2 for low turbulent level flow.

From Scobey diagram, it is obvious that the minimum value of d_{50} equals 7 cm to resist water velocity equaled 1.8 m/s. Also, d_{50} equals 8 cm from the previous equation using C = 0.86. To get a complete grain size distribution for this layer, the following equations given by Simons and Senturk (1992) [18] can be used:

$$d_o = 0.2\, d_{50} \quad (7)$$

$$d_{20} = 0.5\, d_{50} \quad (8)$$

$$d_{100} = 2.0\, d_{50} \quad (9)$$

where:

d_o= diameter of soil particle in which 0% of soil is finer by weight;

d_{20}= diameter of soil particle in which 20% of soil is finer by weight; and

d_{100}= diameter of soil particle in which 100% of soil is finer by weight.

From the previous equations, complete characteristics of grain size distribution curve could be defined for this layer with d_{50} = 80 mm. and thickness = 55 cm.

9.2. Filter Protection Layers

Because of the discrepancy between the d_{50} of the bed materials and the surface protection materials, a filter of well graded materials is put between the surface protection materials and the canal bed in one layer or several ones as follows, figure (17):

- The first layer: this layer is suggested to be with a thickness equaled 40 cm work as a replacement and filter layer above the canal bed directly. d_{50} for this layer equaled 0.2 mm which calculated from the following equation:

$$\frac{(d_{50})_f}{(d_{50})_b} < 40 \quad (10)$$

where:

$(d_{50})_f$ = mean diameter of the filter material; and

$(d_{50})_b$ = mean diameter of the bed material.

Grain size distribution curve for this layer can be plotted based on Eqs. No. (7), (8), and (9) suggested by Simons and Sentruk.

- The second layer: this layer is designed like the first layer with a thickness equaled 40 cm and d_{50} = 3mm and this layer is consisted of a mixture of sand and gravel and put directly above the first layer as the first layer is considered as a base for the second layer.
- The third layer: based on the design procedure of the first and second layer, this layer is put above the second layer and below the surface protection layer. Moreover, it consists of a well-graded gravel with a thickness of 40 cm and d_{50} = 20 mm.

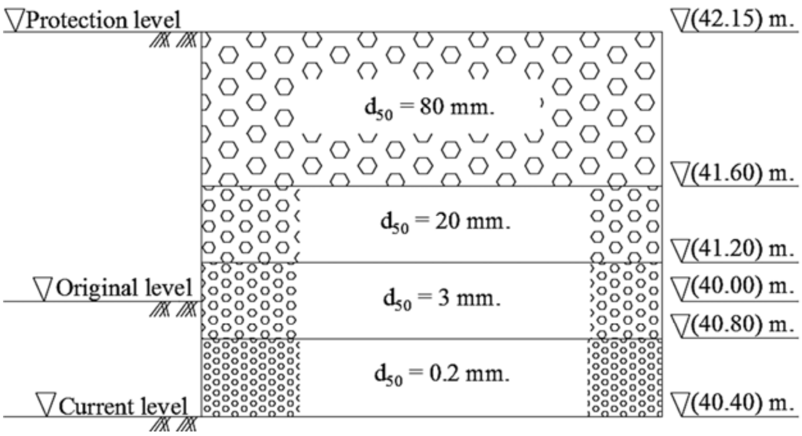


Figure 17. *Different dimensions for protection layers.*

10. Conclusions

A complete hydrographic survey for the first reach of El-Ibrahimeya canal from the channel intake to km. 60 at the location of Dairout control regulator to study the scour phenomena for the canal generally and design a protection for the bridge foundation at km. 23 against scour by representing all data in a one-dimensional model and the results show that:

- There is a general scour before 1960 ranges from 1 m to 2.25 m because of the flood effect before the construction of High Aswan Dam as the maximum discharge through the canal equaled 820.60 m^3/s in 1958 while the maximum discharge through the canal equaled 457.18 m^3/s in 2005.
- The current scour increases than that in 1960 with an average value of 0.7 m because of the passing of emergency floods through the canal from 1960 to 1964 as the maximum discharge through the canal equaled 810.19 m^3/s in 1963.
- The prevailing velocity values along the reach under study equals 0.85 m/s based on the mathematical model results and this value is allowable for this canal of sand bed material.
- The mean velocity of water at the bridge location equals 1.05 m/s before the protection and 1.2 m/s after the protection of the bridge foundation.
- The maximum discharge that passes through the canal at maximum water levels equals 490 m^3/s and decreases until it reaches 485 m^3/s so the discharge decreases by an accepted percent equaled 1% after the protection of the bridge foundation.
- For the canal cross sections with design width equaled 60 m, the scour results in a new width equal 100 m.
- Depending on the comparison between the maximum water levels and the right and left banks levels, it is apparent that the distance between the maximum water levels and the right bank levels equals 10 cm from km. 10 to km. 30 while the distance between the left bank levels and the maximum water levels equals 25 cm from km. 10 to km. 25.

Acknowledgments

Grateful thanks for all staff of Hydraulics Research Institute (HRI), National Water Research Center, for their help in obtaining data.

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