Original Research Article

SCOUR EVALUATION FOR THE NILE RIVER BENDS ON ROSETTA BRANCH

ABSTRACT

The objectives of this research were to analyze and evaluate the effect of releasing flow discharges on river meandering in addition to estimating the local scour at 13 bridge piers distributed on 3 bridges located on Rosetta branch. The meandering length was 3.5Km (from km 145.00 to km 148.50) D.S of El-Roda Gauge at Kafr El-Zayat City. Several sorts of data were collected including site maps, velocity measurements, bed samples, hydrographic survey data, water levels and discharges at several years and seasons, as well as visual inspection photos to be used in the current study. The configurations in bed level, the thalwege line, and the scour holes were determined by comparing the surveyed entire reach at years 1982, 1998, 2003 and 2006.

Study area was simulated four times by 2-D mathematical model "SMS" using a survey reach at years 1982, 1998, 2003 and 2006. This was done to estimate the velocities and the water levels for different discharges at the entire reach. The flow was used as upstream boundary condition and the water level was used as downstream boundary condition. The model was calibrated and verified using the field measured velocities.

Two proposed alternatives were suggested and numerically simulated separately. The first alternative, the outer bends were filled with layers of filter and riprap up to level -5.00 m MSL. In additional to the first alternative, the inner sides of the bends were dredged to level -3.00 m MSL as second alternative. The two alternatives were numerically tested under maximum and emergence flows. The results analysis proved that flow conditions were enhanced and improved under the second alternative when compared to the first one. Based on the results, layers of filter and riprap were designed to fill the scour holes.

Keywords: *River Meandering; Fill; Dredging; Numerical Modeling, Nile River, Nile Delta, Scour and River Bends.*

1. INTRODUCTION

During high floods, higher discharges than the annual maximum were released. These peak discharges cause local scour in the vicinity of bridges, harbors and other structures, also inundate to former flood plains that are currently in use. The Nile River is relatively straight with some sinuous reaches over short distances that are related to steeper slopes. The increase in sinuosity in turn increases the bed slope. Steeper portions become more active and bank erosive. Consequently, scouring action was expected to continue in these areas. The average bed slope along the Damietta and Rosetta Branches of the Nile Delta (240km from Delta Barrage, Fig. (1) was 5.6cm/km. The suspended bed material loads for the Nile downstream Aswan has changed substantially as a result of the creation of Lake Nasser, [1].

The meander wavelengths of the River Nile are varied from 2500m to 4500m. The meander pattern was subsequent to the construction of the High Aswan Dam (H.A.D.) as a result of a reduction in discharge and sediment load. After constructing H.A.D, the Nile was considered as a very low energy river with low water surface gradients. From the Aswan Dam to the head of the Nile Delta, the river distance is about 950km, and the river bed drops ranging from +79m to +11m, giving rise to an average slope of 7.2cm/km.

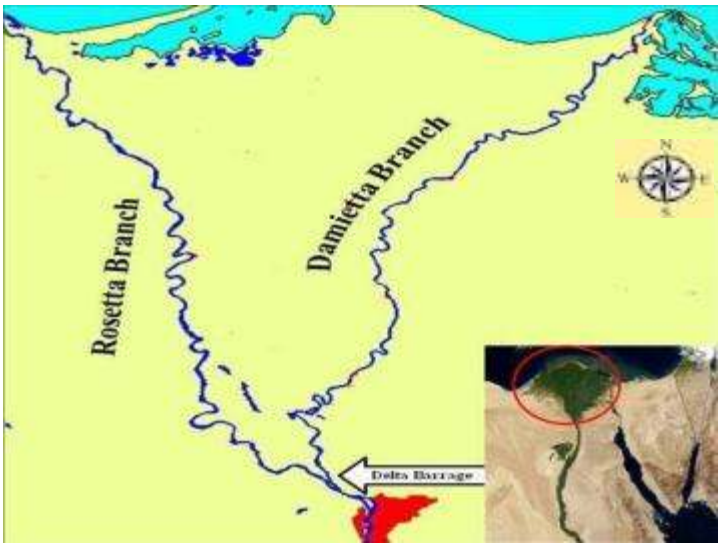

Fig. 1. Schematic Sketch for Rosetta and Damietta Branches

The released water from Aswan Dam was kept as far as possible equal to the water demand, leaving no surplus water to be wasted into the sea except during the winter closure period and in emergency cases belonging to the decided regulations of the HAD. High discharges released from HAD were determined according to the regulation guidelines for operating the HAD. These peak discharges cause damages to the water control structures along the Nile and its branches. Relatively high discharges cause local scour near bridges, harbors and other structures. Also, relatively high discharges cause inundation to former flood plains currently in use. Such inundation in turn ruins agricultural properties, urban areas, and roads and may expose human lives to danger. The emergency floods were reported in [2] and [3].

Many researches were involved in studying and simulating the flow characteristics around bridge piers; [4] proposed a nonlinear analysis procedure for the performance of bridges with pile foundations under flow-induced loads to evaluate the flood-resistant capacity of scoured bridges. [5] Studied the scour in the wake region of piers, they demonstrated that the scouring action and the geometry of scour holes developed in cohesive sediment was not the same as cohesion less sediments and proportion of clay fraction is the most significant variables controlling the depth of scour. [6] Investigated the final scour depth around cylindrical piers under flood waves, considered only clear-water conditions, proposed an approach to estimate the final scour depth under a flood wave, based on the local scour depth calculated with steady flow equations under peak flow conditions.

Focusing on previous studies related to the flow in meander channels, [7] defined meander migrations a process in which water flow erodes soil on one bank and deposits it on the opposite bank. Therefore, a gradual shift of bank line occurs over the long term. Bank erosion undermines bridge piers and abutments, scours the foundations of parallel highways, and causes loss of useful land. [8] explored the features of migrating sediments generated behind piers by local scour processes, and found the main dimensionless factors governing the dune processes, deduced an equation describing the dune evolution and the downstream propagation as a function of the key parameters. [9] Presented an analysis of evolution of fluvial dynamics along the meandering profile of the central sector through the cartographic study of a long segment of the Ebro River. [10] Discussed dynamic processes of meanders and avulsions with particular attention to factors that may trigger avulsions. A preliminary evaluation scheme is presented for meanders with avulsions.

2. PROBLEM DESCRIPTION AND SITE LOCATION

Kafr El-Zayat City is located at the outer curve of a very sharp bend at Km 123 of Rosetta Branch. The study area was 3.5km long, located downstream of Delta Barrage from km 145.00 to km 148.50 downstream of El-Roda Gauge Station. The study area is a bend consisting two highway bridges and one railway bridge were located. Fig. (2) and Table (1) show the geometry and location of the 13 bridge piers and their distance from the left bank. It should be denoted that the coordinates of the study area were (289181.00 m E, 3410603.00 m N), and (287432.08 m E, 3413467.01 m N) UTM

Fig.2. Schematic Sketch for the Study Reach

Table 1: Location and Dimensions of the Bridge Piers

Bridge No.	Bridge 1			Bridge 2						Bridge 3			
Location (km)	146.00			146.239						147.682			
Pier No.	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8	Pier 9	Pier 10	Pier 11	Pier 12	Pier 13
Pier Shape	Rec	Rec	Rec	Rec	Rec	Cir	Rec	Rec	Rec	Rec	Rec	Cir	Rec
Diameter (m)	------	------	------	------	------	14.00	------	------	------	------	------	11.00	------
Width (m)	16.00	16.00	16.00	4.00	4.00	------	4.00	4.00	4.00	4.00	4.00	------	4.00
Length (m)	26.50	26.50	26.50	15.00	15.00	------	15.00	15.00	13.00	13.00	13.00	------	13.00
Dist. (m)	41.25	182.1	300.9	77.64	147.3	170.0	206.1	276.8	347.7	58.11	132.04	157.61	195.90

Where: Location = downstream of El-Roda Gauge Station, Rec = rectangular, Cir = circular, Dist. = distance from left bank.

3. METHODOLOGY

The "SMS" 2-D mathematical model would be employed, at first, to simulate the morphological and hydrological characteristics in the reach of Rosetta branch. The present study would be carried out applying the following:

1. Collecting the available data of the reach under the study related to hydrographic and hydraulics.

2. Reviewing the available scour hole information in the available literature.
3. Reviewing the previous available studies related to this subject, also determine the different flows at several years passing in the Rosetta Branch from the HAD.
4. To study the development of the morphology on the bend, the reach available bed level data at several years were compared.
5. The reach was numerically simulated for four times using the surveyed data at different years aiming to model calibration and verification.
6. Simulating different proposed alternatives using 2-D model to predict and evaluate the expected scour bed for reach under study including scour around the bridge piers.

The previous steps could be followed when a 2-D model used to simulate a meandering reach, [11], [12], [13], [14], and [15]

4. MODEL CONSTRUCTION

The simulated length was about 3.5km, including 13 piers under 3 bridges. The mesh was generated for the studied area, and the bed elevations were determined using the bathymetric survey of the river. The mesh was designed by dividing the studied reach into different regions. Each region was divided into elements called quadrilateral elements and triangular elements. It should be mentioned that the designed mesh was condensed at the locations of the bridge piers to simulate the dimensions of piers with high accuracy Fig. (3).The depth file was created based on the hydrographic survey data collected in 2006. The discharge and the water level were used as upstream and downstream boundary conditions respectively. The hydraulic roughness coefficient was defined at each grid point and ranged from 0.02 to 0.05.

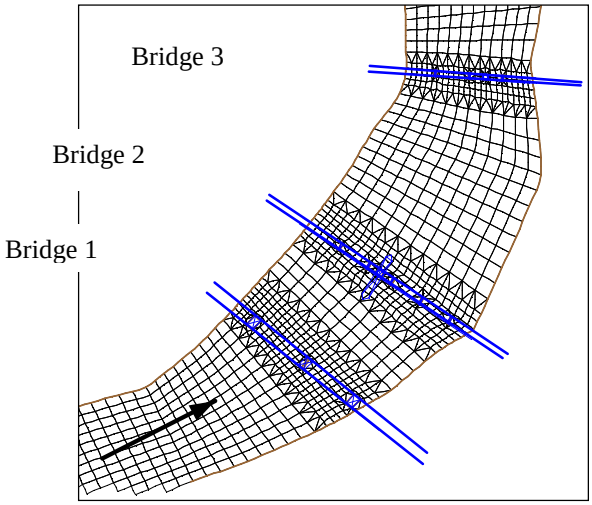

Fig.3. Mesh Generation

4.1. Model Calibration

The model was run using the field hydraulic measurements in 2006. The discharge of 222.8m^3/sec and the corresponding water level of 2m MSL were used as upstream and downstream boundary conditions respectively. In calibration process, the velocity distributions were located at 3 different cross sections, Fig.(4). The water surface slope was adjusted in the model by changing the roughness coefficient up to a good agreement between the prototype, and model water surface slope was obtained after which the roughness coefficient was fixed. Figs. (5-7) showed good agreement between the velocities obtained from the used model and field measurements at different cross sections. The presented velocity distribution was agreed with [16].

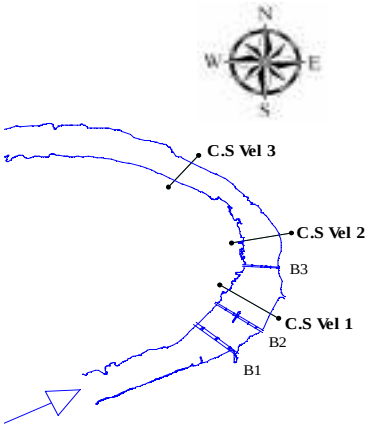

Fig. 4. Location of the Calibration Cross Sections

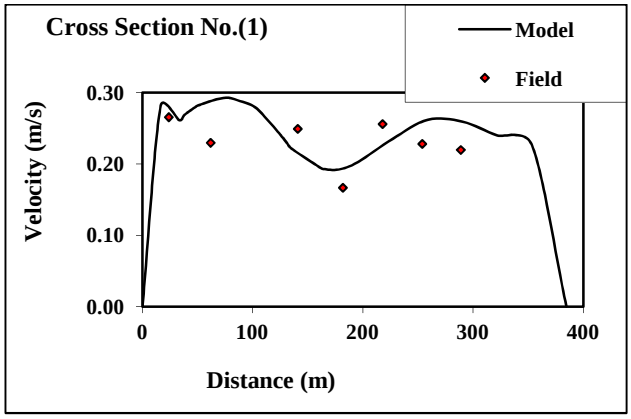

Fig. 5. Flow Velocity Calibration at Cross Section (1)

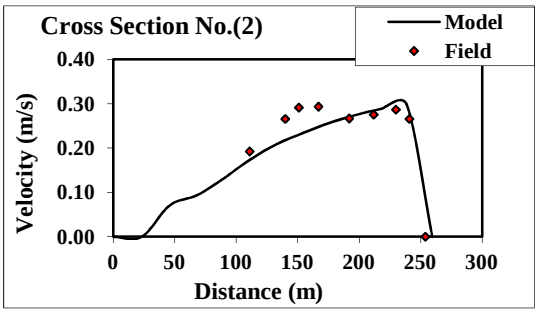

Fig. 6. Flow Velocity Calibration at Cross Section (2)

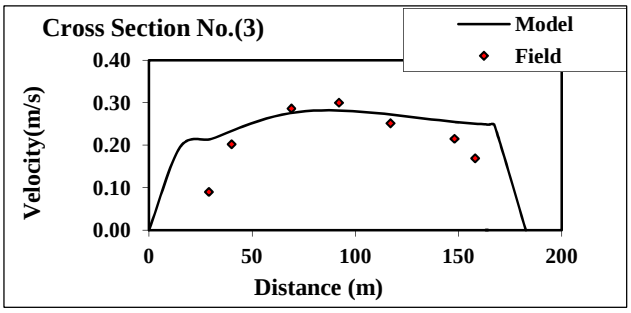

Fig. 7. Flow Velocity Calibration at Cross Section (3)

5. BED ELEVATION CONTOUR MAP AT YEARS 1982, 1998, 2003 AND 2006

Fig. (8) showed a comparison in bed profiles along the study reach during years 1982, 1998, 2003 and 2006. The figure presented that most of the studied area was exposed to the scouring action. Moreover, the maximum scour occurred at the outer bank on the contrary the deposition region.

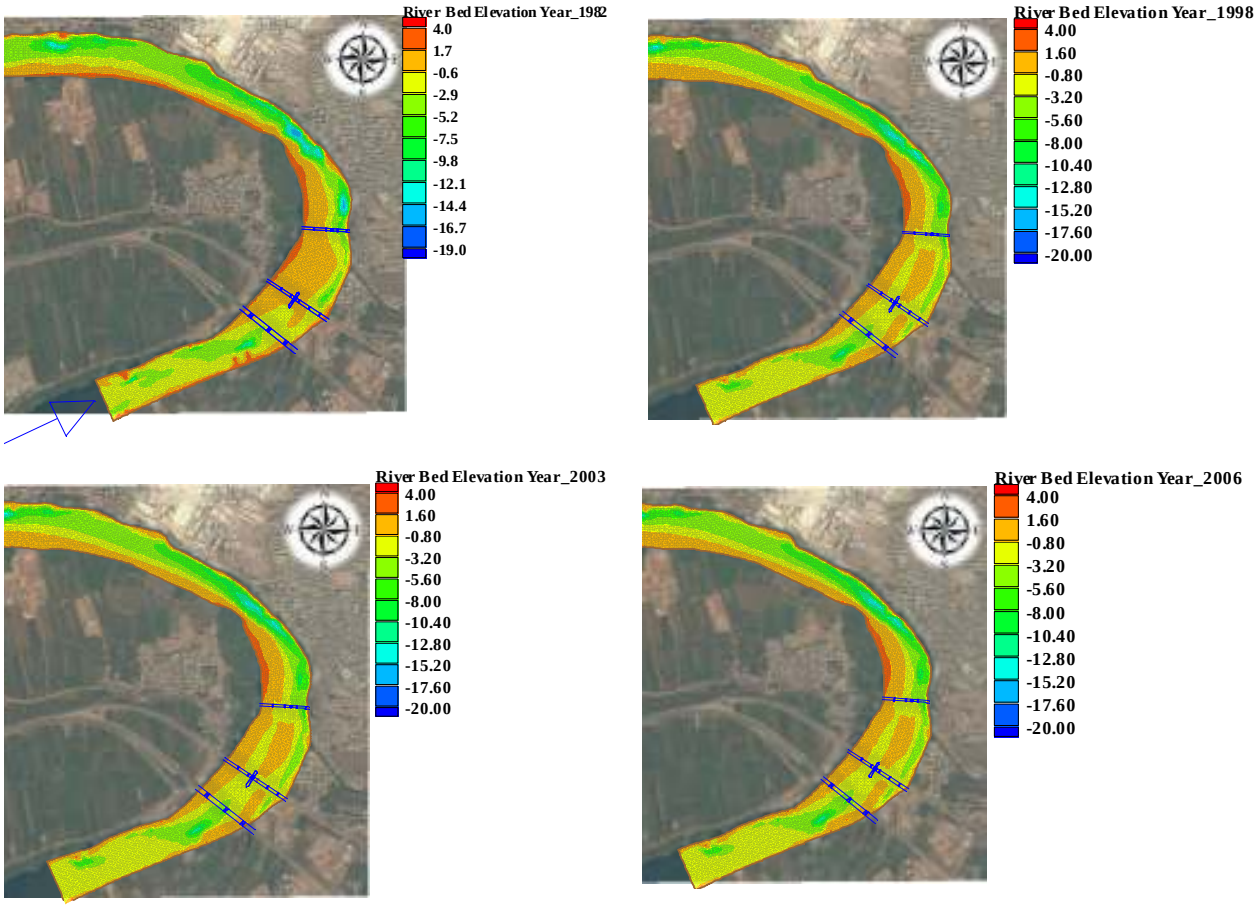

Fig.8. River Bed Elevation for Years 1982, 1998, 2003 and 2006

6. SCOUR HOLES IN THE AREA OF STUDY

Fig. (9) showed the location and the geometry development of the scour holes in the studied area from years 1982 to 2006. In the outer curve, the velocity was higher than inner curve. Consequently, the scour holes were located at the outer curve of the meander which in turn exposes it to the risk of failure. From figure investigations it's dedicated that the area of scour holes No.1were wider in year 2006 than year 1982, however the scour holes No.2 &3 were deposited in the same time period. The output findings were found to be match with the results obtained by [17].

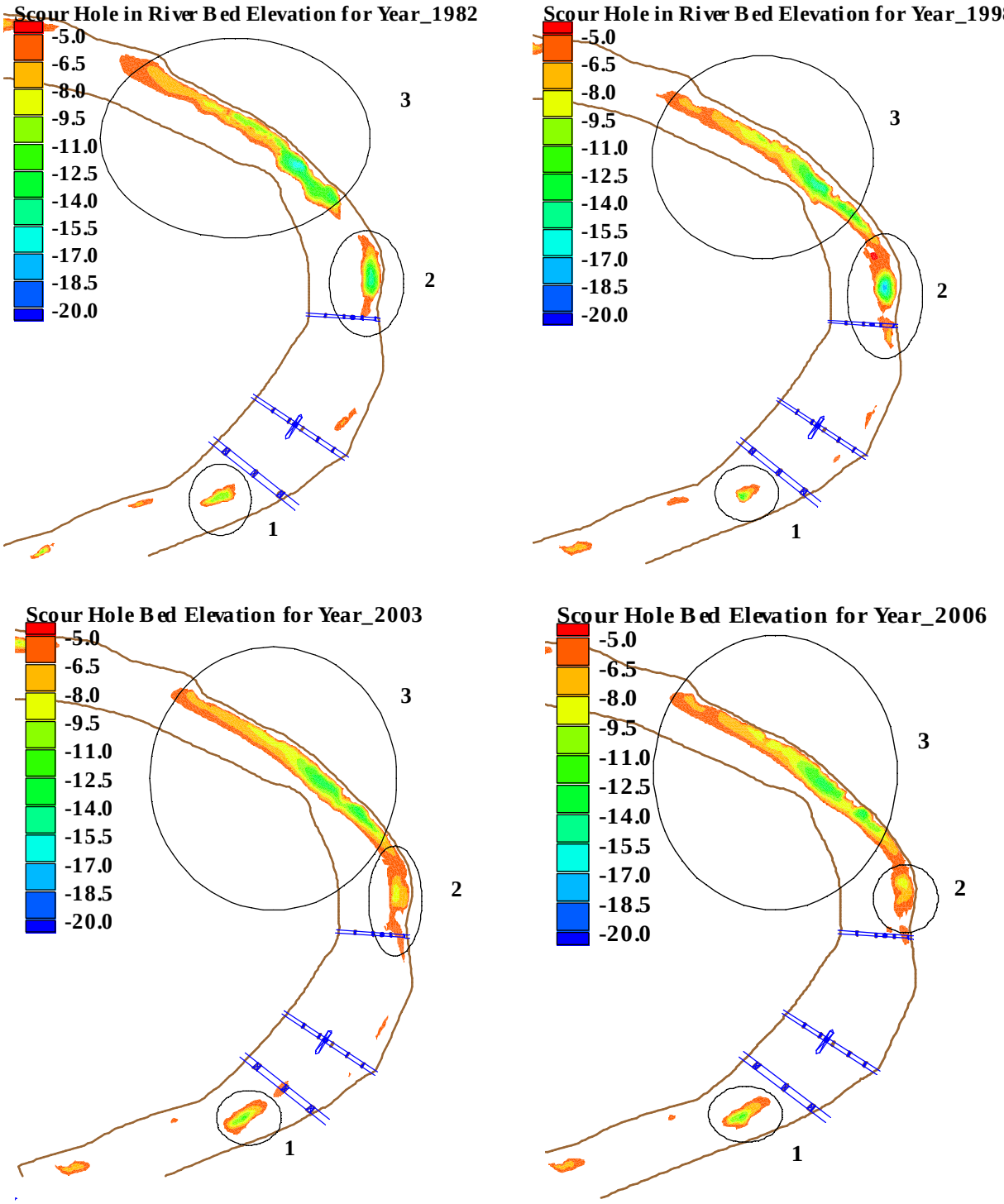

Fig.9. Scour Holes Location in Study Area at Years 1982, 1998, 2003 & 2006

Fig. (10) presented comparison between the geometry of scour holes at different cross sections at years 1982, 1998, 2003 and 2006. It was noticed that for the tested cross sections the scour holes became the deeper and shifted to the left side on 1998 when compared to other years. This owned to the high flood occurred in this year. Focusing on cross section 3, the geometry of scour hole was on a large scale due to its location; just downstream the bridge piers and narrow width.

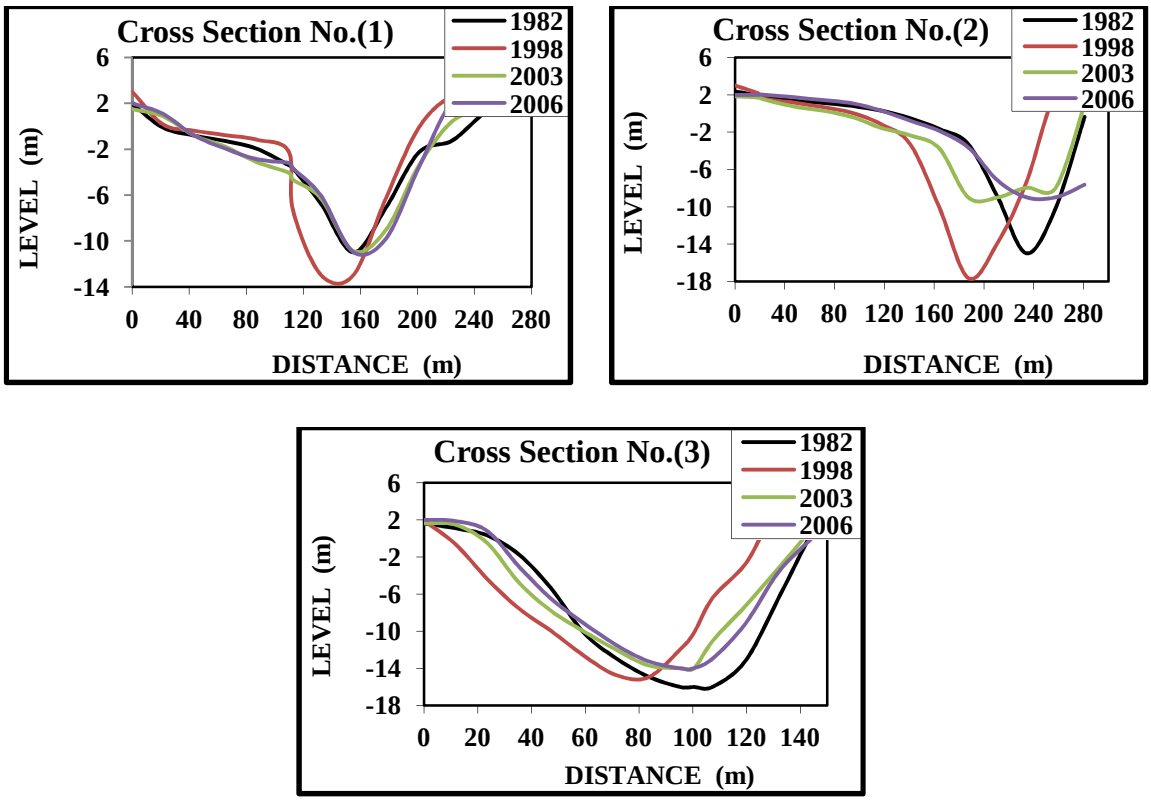

Fig.10. Scour Holes Cross Sections for Years 1982, 1998, 2003 and 2006

7. THE MODELED REACH

The survey data on year 2006 were used in the simulation as the most updated measurements for the present conditions. Within this reach, (2) bridges were presented, one for railway and two for highways. Also, the river bank in front of Kafr El-Zayat were included**.** The calibration of the hydrodynamic model was carried out by comparing the predicted velocities obtained from the numerical model and the corresponding field measurements at the3 cross sections presented in Fig. (4).

7.1. Simulation of the Proposed Solutions and Results

Two proposed alternatives to improve the morphology of the bend were suggested and simulated separately by the SMS model. In the first alternative, the scour hole of the outer bend was filled by layers of filter and riprap up to level -5.00m MSL. In additional to alternative 1, dredging the inner sides to level -3.00m MSL was proposed as second alternative. The model was run for the two alternatives at maximum and emergence flow with its corresponding water levels which were809.03m^3/s, 2546.30m^3/sec, +2.60m MSL and +5.90m MSL respectively. The flow discharge was used as upstream boundary condition and the water level was used as downstream boundary condition.

7.2. The First Alternative Simulation

Fig. (11) presented the bed levels of the reach after filled with filling materials to level -5.00 MSL to simulate the first alternative. It was noticed that the most of the filling areas were concentrated at the outer curves where the scour regions were highlighted. These also were illustrated in Fig. (12) that presented (3) cross sections distributed along the reach. The locations of these sections were presented in Fig. (11) and were selected after carefully study for the entire reach to present the maximum bed morphological changes. The maximum depth of the scour holes at cross sections (1), (2), and (3) were -11, -9 and -14 MSL, respectively. Consequently, the depth of filling layers was more than 12m for some holes.

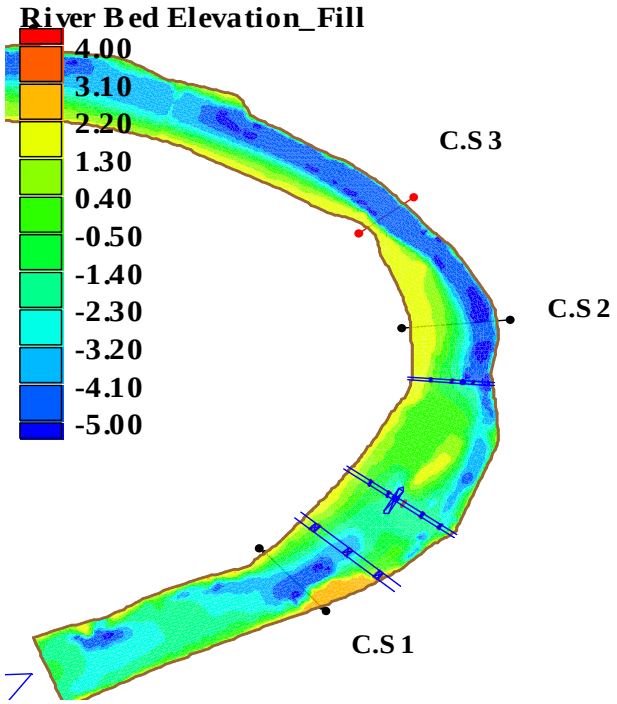

Fig. 11. River Bed Elevation in Case of Alternative (1)

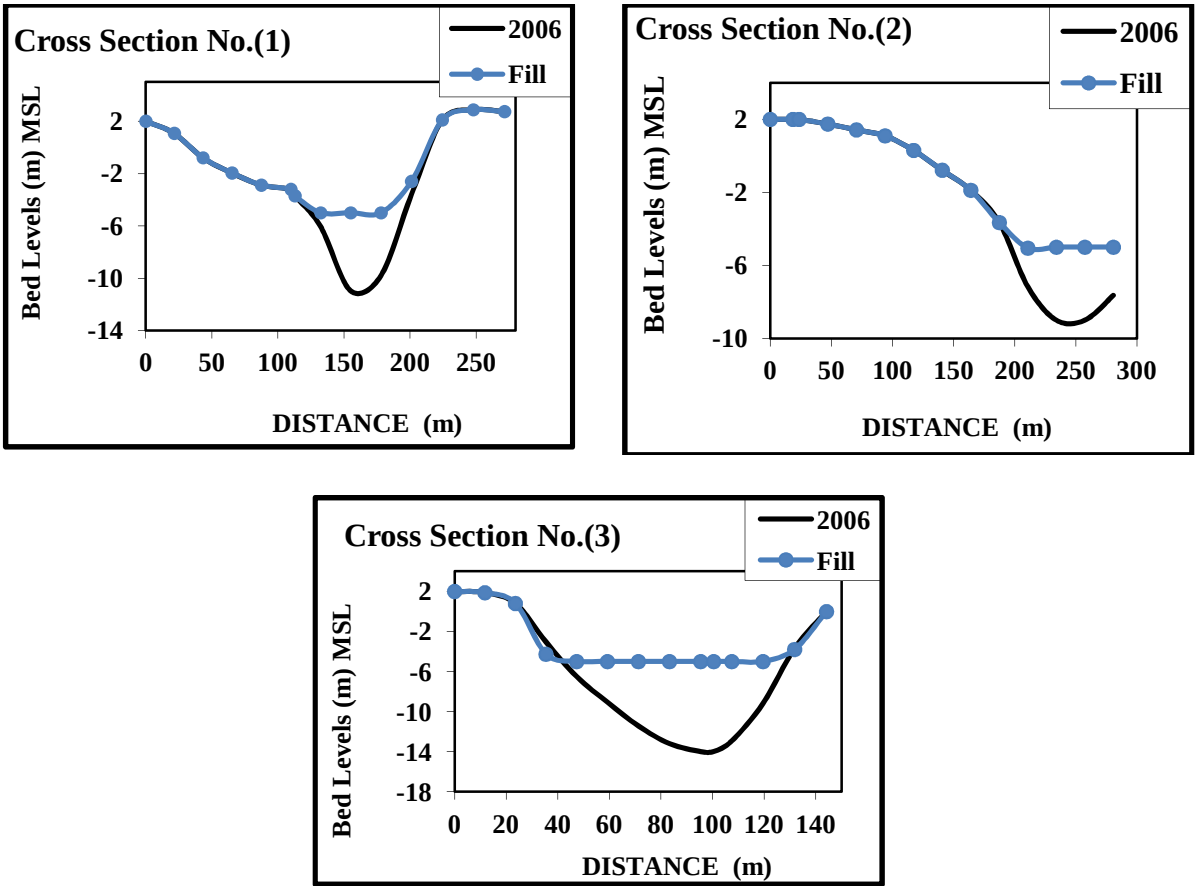

Fig.12. Bed Profiles at Different Cross Sections after Simulating Alternative (1)

7.2.1. First Alternative Model Run Results

❖ **Maximum Flow Run**

In case of Maximum flow, the discharge was 809.03m^3/s and its corresponding water level was +2.60m MSL. The predicted velocities were ranged from 0.45 to 1.05m/sec in the outer curve for different cross sections. However, the average velocities of the reach were around 0.70 m/s.

Fig. (13) showed the velocity profiles of alternative (1) compared to the basic case at different cross sections. The same curves trend was found. Moreover, it was declared that the velocity values were higher than the basic case resulted in decreasing the cross sectional area after filling.

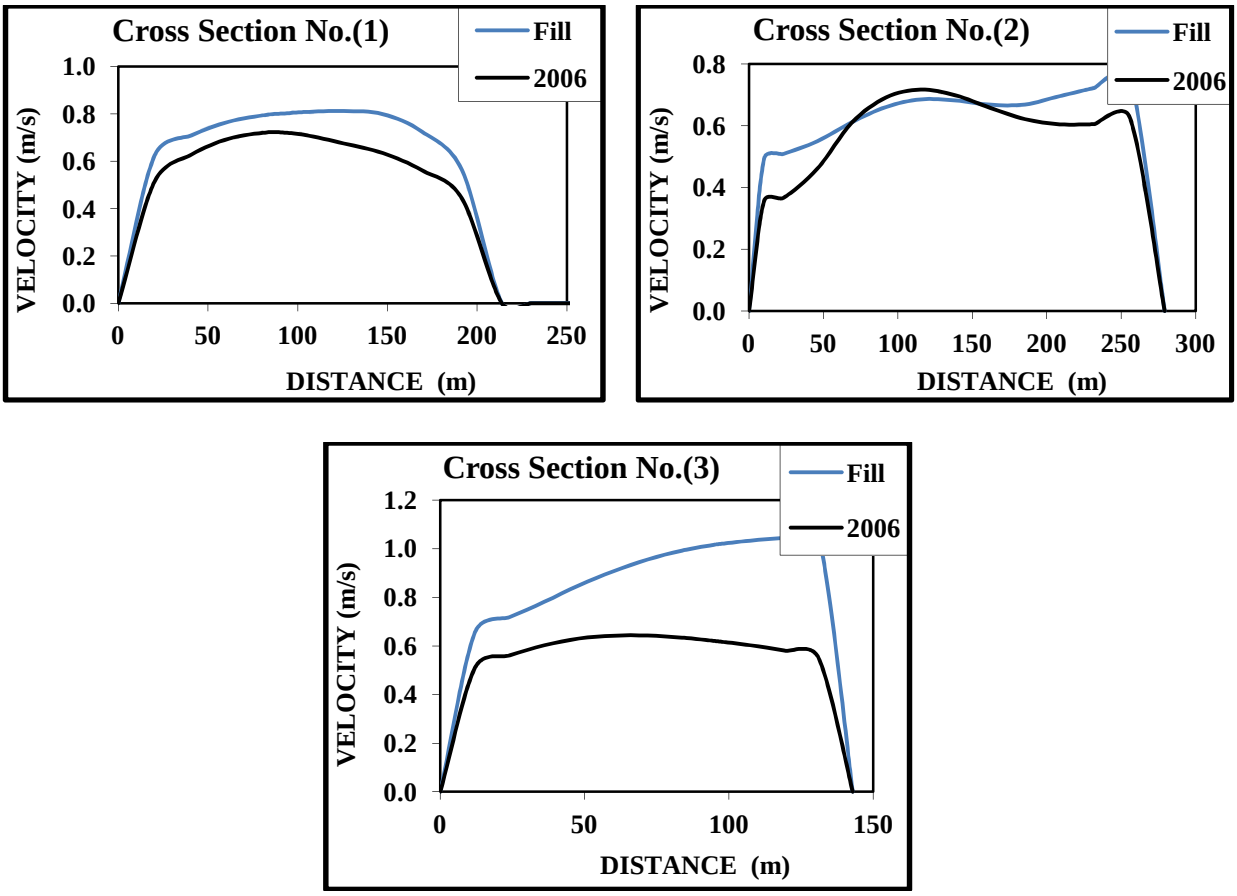

Fig.13. Velocity Profiles for the Basic Case and Alternative (1)for the Max Flow

❖ **Emergency Flow Run**

The model was run at emergency discharge with its corresponding water levels. The discharge was 2546.30m^3/s and its corresponding water level was +5.90m MSL. The resulted velocities were ranged from 1.00 to 2.20m/sec in the outer curve. While the average velocities of the reach werearound1.50m/s.

Fig. (14) shows the velocity profiles of alternative (1) in case of emergency flow comparing to the original results at cross sections No (1) to (3). The Fig. shows that the results of velocity profiles in case of alternate (1) were similar to the profiles as the original case. It is clear that the values of the velocities at cross sections 1&3 increased than in case of original case because of considerable part of those sections were filled.

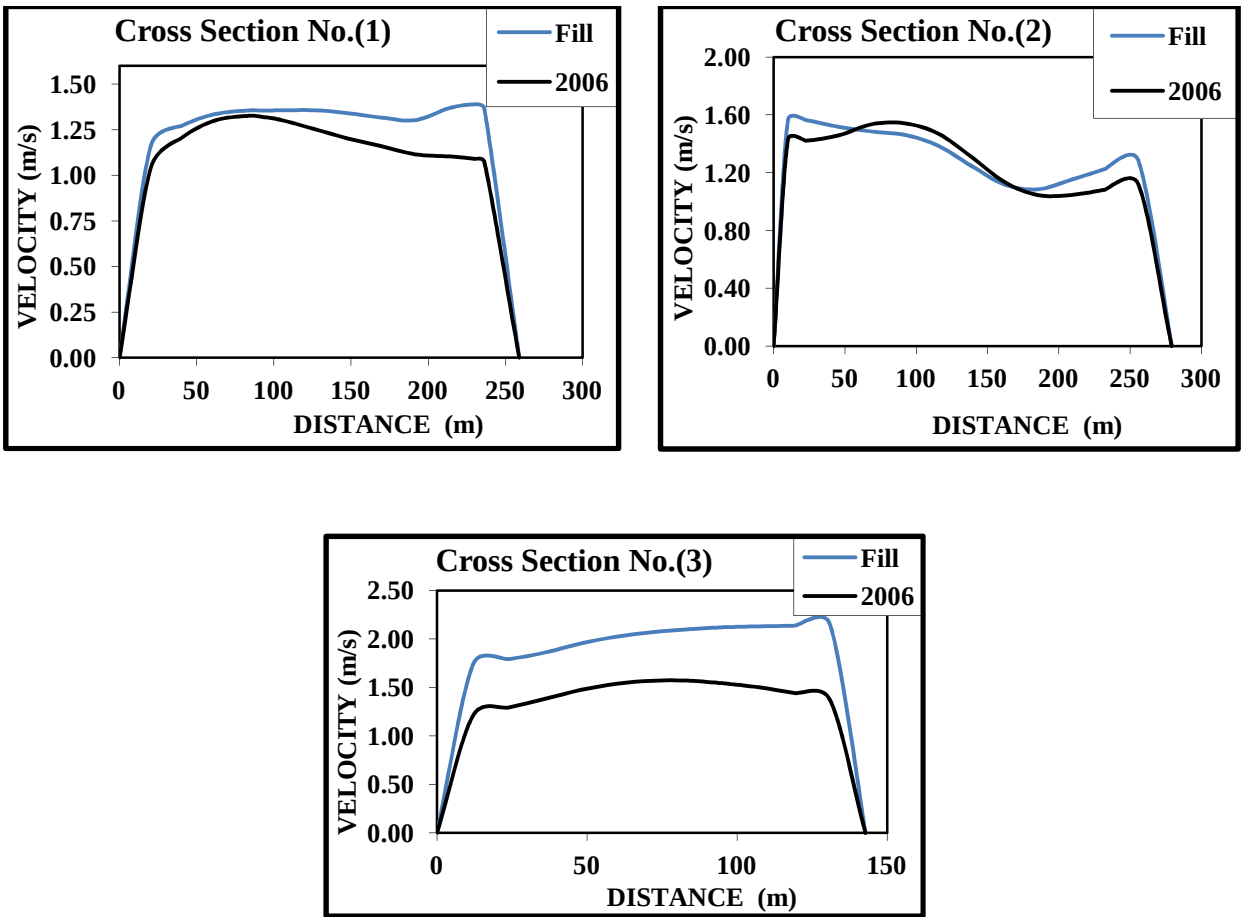

Fig.14. Cross Sections Velocity Profile of the Original & Alternative (1) at Future Flow

7.3. The Second Alternative Simulation

The bed levels of the reach are filled to level -5.00 MSL and the other part are dredged to level +3.00 MSL to represent the second alternative. Fig. (15) shows the entire reach bed elevation in case of alternative 2. It is clear that the most of the filling areas are concentrated at the outer curves and the dredging area in the inner curve. These also are shown at Fig. (16), which represents three cross sections distributed along the reach. The location of these sections is shown in Fig. 15. The level of deepest point of the scour holes at cross sections from 1 to 3 are 2, 2 and 2 above MSL, respectively. This means that the filling layers of some holes are more than 12m and the dredging layers at some locations within 5m.

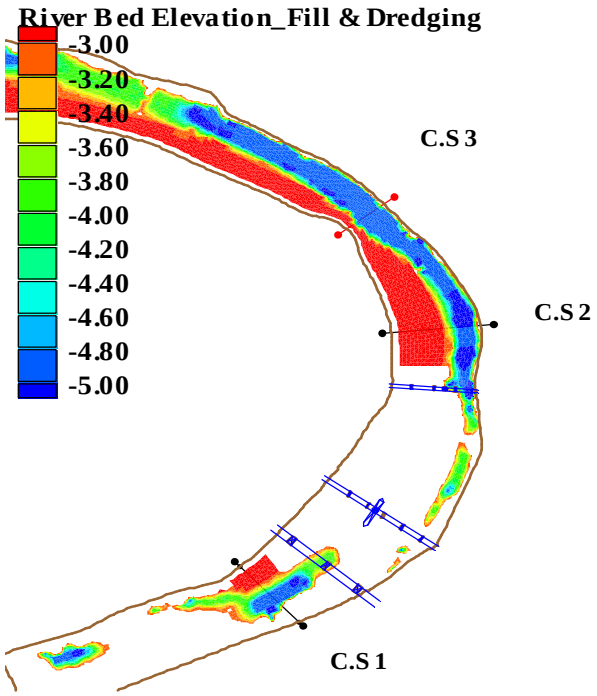

Fig.15. River Bed Elevation in Case of Alternative (2)

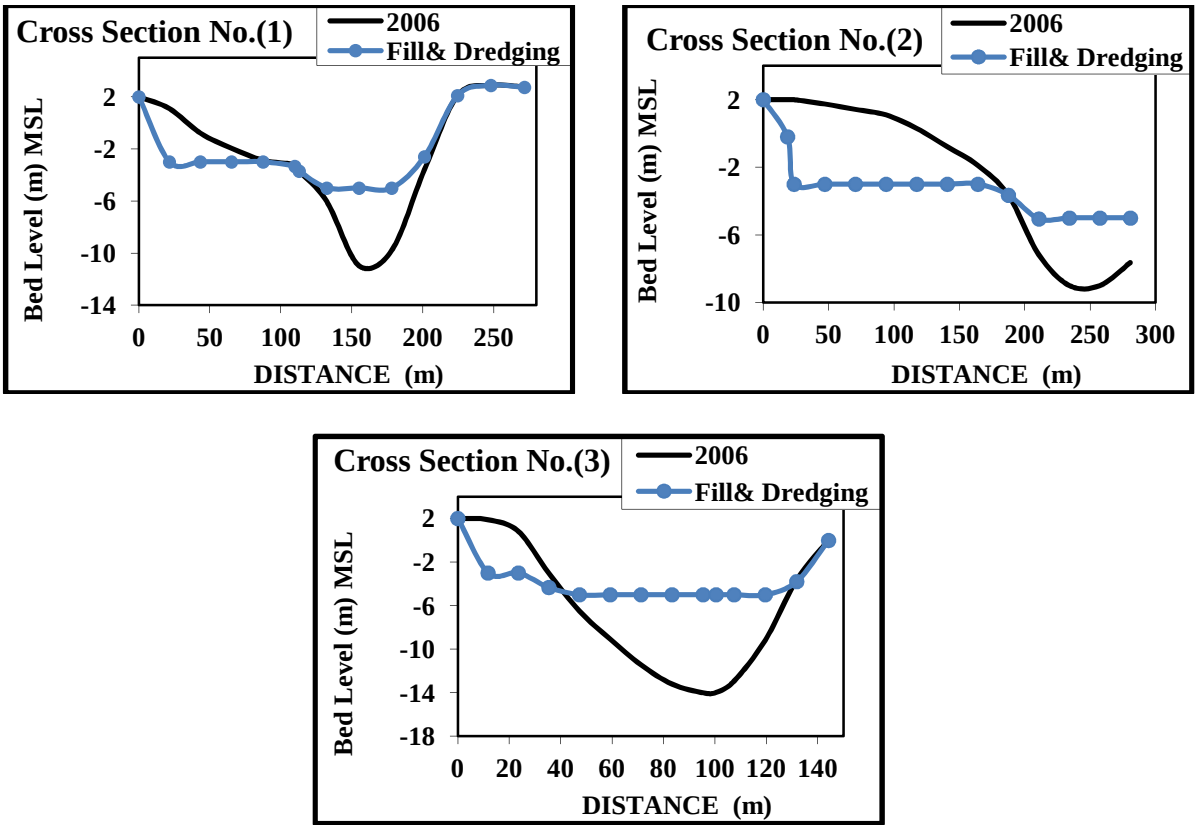

Fig.16. Cross Sections in Case of Original Year and Alternative (2)

7.3.1. Second Alternative Model Run Results

❖ **Maximum Flow Run**

In case of Maximum flow, the discharge was 809.03m^3/s and its corresponding water level was +2.60m MSL. The flow velocities along the reach show that the maximum value of velocities was occurred at the outer curves. The resulted velocities are ranged between 0.28 and 0.93 m/sec at the concerned section. Fig. (17) shows the velocity profiles of alternative 2 comparing to the original results at cross sections No 1 to 3. The figure shows that the results of velocity profiles in case of alternative 2 were redistributed along the sections to be more regular than in case of the original at cross sections No 1& 3. It is clear that the values of the velocities increased at cross section3 and decreased at cross sections No 2 comparing with the original case because of considerable part of those sections were filled and dredged respectively.

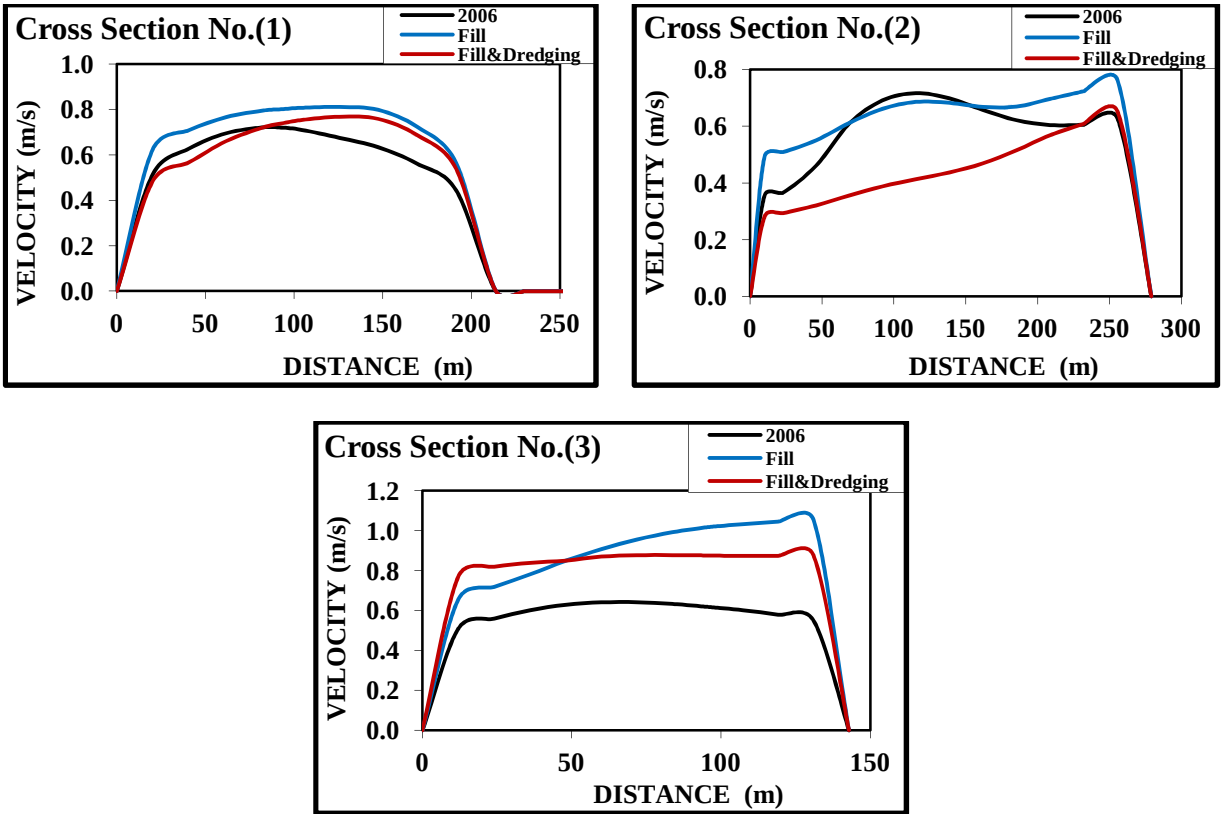

Fig.17. Cross Sections Velocity Profile of the Original, Alternative 1 and Alternative 2 at Maximum Flow

❖ **Emergency Flow Run**

The model was run at emergency discharge with its corresponding water levels. The discharge was 2546.30m^3/s and its corresponding water level was +5.90 m MSL. The resulted velocities recorded in this case are ranged between 0.80 and 2.00 m/sec in the outer curve. While the average velocities along the reach were around 1.40 m/s.

Fig. (18) showed the velocity profiles at three cross sections along the reach for alternative 2 in the case of emergency flow compared to the basic case. The figure showed that the results of velocity profiles were redistributed along the cross sections to be more regular than the basic case at cross section No.1. It was declared that the velocities values were increased at cross sectionNo.3 and decreased at cross section No. 2 compared to the basic case. That was resulted in the effective part of these sections were filled and dredged respectively.

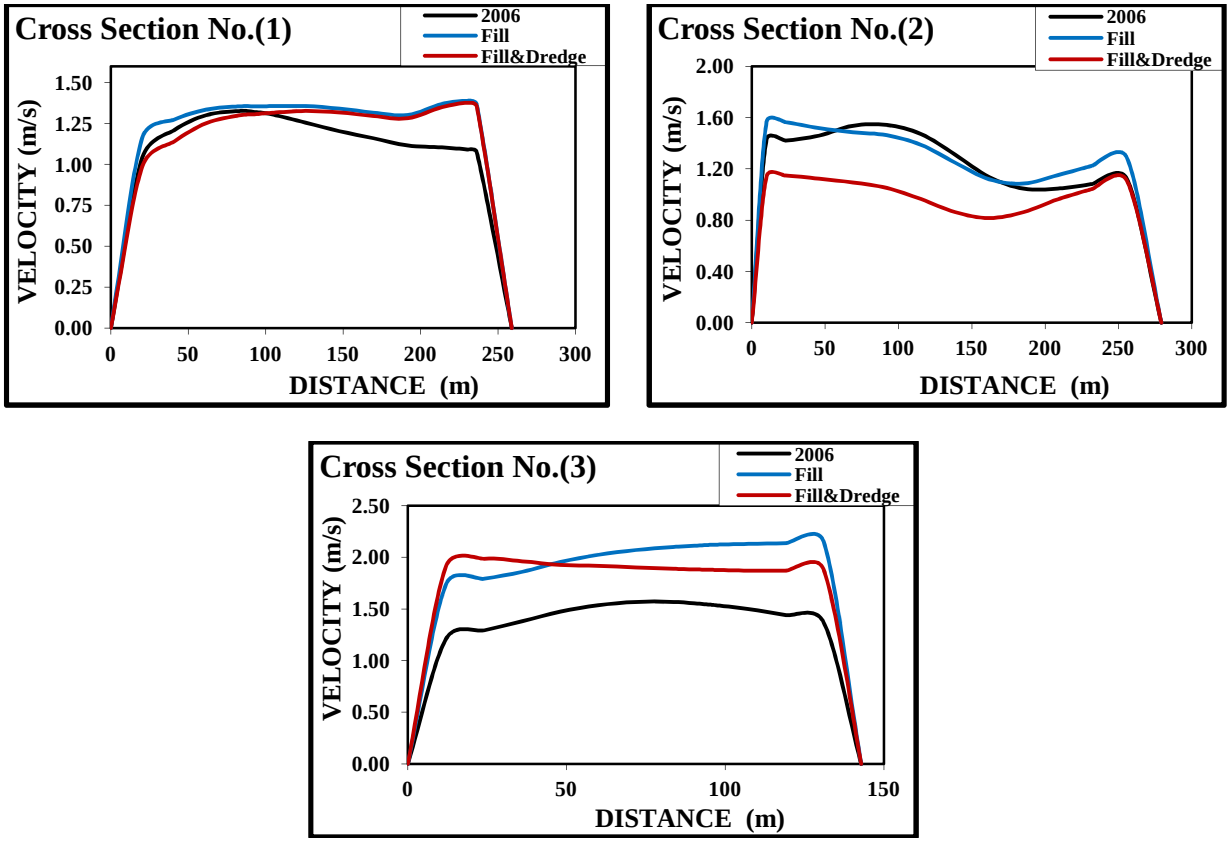

Fig.18. Cross Sections Velocity Profile of the Original, Alternatives (1) and (2)at Emergency Flow

8. CONCLUSION:

Based on the results of comparing the two purposed solutions by surveying of year 2006, the following was obtained:

1. Unexpected velocity profiles resulted in the human interference were found at some locations.
2. The maximum scour depth was found at the piers located in the middle of the cross section.
3. The maximum scour depth was directly proportional to discharge.
4. The increase of the scour hole around the piers of the first bridge (upstream) was higher than the increase of the scour hole around the piers of the second and third bridges (downstream).
5. When the scour holes (at the outer curve) were filled up to level of -5 MSL (Alternative 1), the velocity values along the tested cross sections were increased. Consequently, the probability of the expected scour was increased.
6. When the scour holes were filled up to level of -5m MSL and the other side dredged to -3m MSL (Alternative 2), slight differences were found in velocity profile compared to the basic case. Consequently, the probability of the expected scour was reduced.
7. In the case of alternative 2 the velocity profiles along the tested cross sections were redistributed and turned more regular compared to the alternative 1 and the basic case.
8. In the case of maximum and emergency flows, the obtained velocity profiles gave the similar trend, with differences in values.

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