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Solving the problem of sedimentation at water intake of Rowd El-Farag pump station using 2D model

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KEYWORDS

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Dredging;
CCHE2D;
Sedimentation;
Water intake

Abstract This research presents and analyzes the problem of sedimentation at water intake of Rowd El-Farag pump station using a 2 dimensional computational model, CCHE2D. A study reach with a length of 1.53 km from 7.785 to 9.31 km from El-Roda gauging station was selected. The pump station is located at 8.63 km from El-Roda gauging station. The study shows there is a significant morphological change in this reach due to the long study period and two hydraulic Structures (Imbaba and Rowd El-Farag Bridges). Moreover, the different alternatives for sediment control are discussed such as: dikes on the western side of the river at different locations or dredging the study area at different levels 14 and 12.5 (1 and 2.5 m respectively) below minimum water level. Finally, the research recommended the using of dredging as a sustainable solution for sediment control at Water Intake of Rowd El-Farag Pump Station although this solution is an expensive solution.

1. Introduction

Nile River is subjected to different floods with a relatively wide variations ranging from low to high floods. The Nile flood can be as high as 150 billion cubic meters per year (1878) as natural inflow at Dongla gauging station and as low as 43 billion cubic meters per year (1913). In the last two decades the discharge through Reach Four (which starts from Asyut Barrage at 544.75 km to the Delta Barrage at 953.00 km with a length of about 408.25 km) of the Nile is between 436.34 and 2094.91 m^3/s. Even though high floods have their side effects on riverbanks, hydraulic structures and riverbed. Low floods have many other critical outcomes on the availability of water resources and low water levels. The side effects of low flows are many. The examples of these side effects are the water supply deficiency, navigation problems, and some local sedimentation problems.

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In practice, there are many different purposes for water use such as irrigation, drinking, cleaning, and cooling. This water is extracted from different intakes from dynamic fresh water sources in the rivers or streams. The inlets of these pipes are usually submerged under water surface to ensure a continuous water supply. If such submergence is not deep enough or the water surface is fluctuating causing the pipe submergence depth to become smaller, problems of vortexing and cavitation are always expected.

This research presents and analyzes the problem of sedimentation at water intake of Rowd El-Farag pump station using a 2 dimensional computational model, CCHE2D. The different alternatives for sediment control are submerged dikes to control flow directions and dredging are discussed.

2. Objective

The objective of this research is to analyze the problem of sedimentation at the water intake of Rowd El-Farag pump station and the different alternatives for sediment control using 2D model as follows:

1. Analyze and discuss the factors causing the problem of sedimentation at the intake of Rowd El-Farag pumping station. Also, different alternatives for sediment control to solve the problem of sedimentation at the intake of the pumping station.
2. Recommending a solution of the problem.

3. The study reach

A Nile reach with a length of 1.53 km was selected where the pump station is at the middle. The selected reach covers the area from 7.78 km to 9.31 km downstream El-Roda gauging station (934.785 km to the 936.31 km from Aswan Dam). The intake of Rowd El-Farag pump station is located at 8.63 km downstream El-Roda gauging station as shown in Fig. 1. The pump station is located in between two bridges. Imbaba Bridge is located at the upstream where Rowd El-Farag Bridge is located at the downstream of the pump station [1,2].

Imbaba Bridge is a steel structure, with two levels one for railway and the second one for roadway. It has seven piers, the second pier from the left bank is circular with 10.6 m diameter and the other six piers are rectangular with 15 m long and 3.6 m wide having rounded noses. The deep scour hole lies 97 m from the left bank and 32 m downstream of the centerline of the bridge with a bottom elevation of −8.3 m [3]. It was found that the mean velocity is not larger than 0.6 m/s.

The reach is nearly straight. There are some low lands (shallow areas) in the eastern bank of the river, at the intake. The low lands extend from downstream Imbaba Bridge until the axis of the pump station. The top width of the cross-sections ranges between 316 m at cross-section (1) just downstream of Imbaba Bridge and 470 m at cross-section (20) downstream Rowd El-Farag Bridge during the low water level, and on the other hand, the top width ranges between 410 m at cross-section (1) and 688 m at cross-section (20) during the high water level.

Figure 1 Plan at Rowd El-Farag pump station and part of Reach Four.

4. Rowd El-Farag pump station

The pump station was constructed in 1903 (one of the oldest pump stations in Egypt) with three groups of intakes:

1. The newest intake consists of 4 lines; each one has a diameter of 1000 mm and of 100 m long with a total capacity of 216,000 m^3/day.
2. The old intake consists of 4 lines; two lines with a diameter of 1100 mm, one line with a diameter of 1400 mm and the last one with a diameter of 1000 mm and of 75 m long with a total capacity of 480,000 m^3/day in peak time.
3. The third intake (American intake) consists of 2 lines each one has a diameter of 1800 mm and of 150 m long with a total capacity of 384,000 m^3/day in peak time.

5. Problem identification

Fig. 1 it shows that the area upstream the pump station is characteristic by:

1. There is an island (Gizert El-Zamalek) upstream the study reach causing the river width narrower and resulting in scour, and as the river width is winding in the study reach, this causes sediment deposition (sedimentation).
2. Also, there is a bend located upstream the pump station intake by 1200 m. The outer curve on the western side causing scour at that place and the inner curve at the eastern side causing deposition at that place (the pumps station is in the east side).
3. On the other hand, Imbaba Bridge is located upstream the pump station intake by 950 m. this bridge causing scour around the bridge piers and sediment deposition downstream at the intake of the pump station.
4. The dismantled Abu Al-Ela metallic Bridge which is stored just downstream intake of the pump station resulting in a dead zone area in its place and causing sediment deposition at the intake.
5. Finally, as shown in Table 1. the study reach at the cross-section number 13 just upstream the intake of the pump, the river width is narrower compared by the cross-section number 14. The river width increase which makes sediment deposition.

6. Morphology of the study area

Figs. 2–7 and Table 1 show the morphological changes in the study reach during the period from 1982 to 2007. Generally, the cross-sectional area and the hydraulic depth became smaller and shallower in the year 2004 compared with the year 1982. This means that there is an aggradation during this period especially in the right bank of the cross-sections. On the other hand, during the period from the year 2004 to the year 2007 generally, the cross-sectional areas and the hydraulic depths became larger and deeper in the year 2007 compared to the year 2004, due to the dredging in the cross-sections as a result of the navigation project.

7. Methodology

Different techniques were used in identifying the sedimentation problem within the study reach during the period from 1982 to 2007.

7.1. Data collection

Two sets of data were collected:

Table 1 Cross-sections properties for the years 1982, 2004 and 2007 at water level 14.

X-Sec	Km D.S El-Roda	1982			2004			2007		
		A	D	T	A	D	T	A	D	T
1	7.78	2285.20	7.10	321.89	1964.59	5.66	347.34	2218.56	7.23	307.02
2	7.89	1851.87	5.65	327.86	1564.48	4.70	332.92	1846.39	5.75	320.87
3	8.00	1856.74	5.26	353.22	1403.94	4.81	291.98	1800.76	5.31	339.33
4	8.11	1751.35	4.78	366.70	1372.60	4.96	276.77	1739.30	4.83	359.84
5	8.15	1630.88	4.64	351.84	1343.29	5.17	259.89	1642.98	5.55	296.28
6	8.21	1561.29	4.49	347.71	1365.05	5.20	262.59	1644.34	5.74	286.34
7	8.26	1572.77	4.55	345.64	1348.21	4.95	272.44	1686.39	5.67	297.27
8	8.30	1575.24	4.57	344.83	1282.92	4.43	289.87	1613.06	5.68	284.20
9	8.36	1553.29	4.51	344.66	1326.27	4.38	302.46	1454.92	5.38	270.37
10	8.41	1600.47	4.50	355.69	1287.50	4.40	292.53	1605.72	5.30	303.01
11	8.45	1616.37	4.49	360.14	1314.82	4.67	281.46	1597.00	5.14	310.44
12	8.49	1614.75	4.45	362.98	1362.48	4.51	301.87	1616.51	5.31	304.18
13	8.55	1572.89	4.23	371.51	1290.98	3.99	323.70	1704.13	5.00	340.96
14	8.65	1687.78	3.61	467.12	1274.00	3.70	343.95	1683.64	4.01	420.06
15	8.76	1472.84	3.18	462.91	1140.10	3.64	313.39	1529.98	3.54	432.06
16	8.88	1449.86	3.23	448.31	1438.71	3.51	410.02	1425.66	3.50	406.90
17	8.98	1470.47	3.21	458.17	1451.47	3.51	413.22	1371.27	3.47	395.60
18	9.09	1467.60	3.10	473.85	1381.45	3.41	405.63	1383.42	3.25	425.08
19	9.19	1640.66	3.14	522.88	1172.89	2.85	411.85	1143.25	2.90	394.90
20	9.31	1449.39	2.85	507.67	1116.13	2.72	409.89	1071.61	2.43	440.83

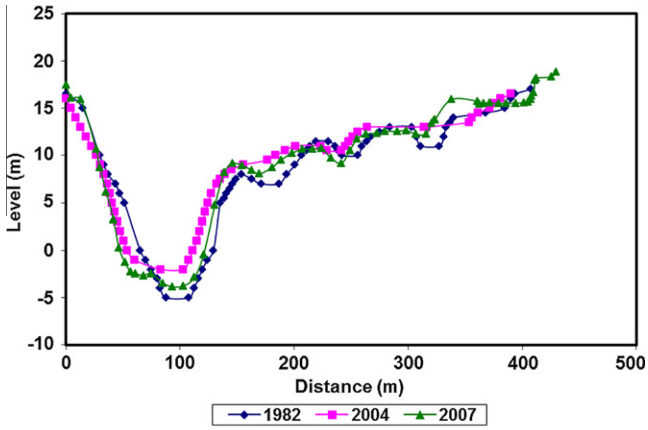

Figure 2 Cross-section comparison for the years 1982, 2004 and 2007 at sec (1) at 7.79 km downstream El-Roda gauging station.

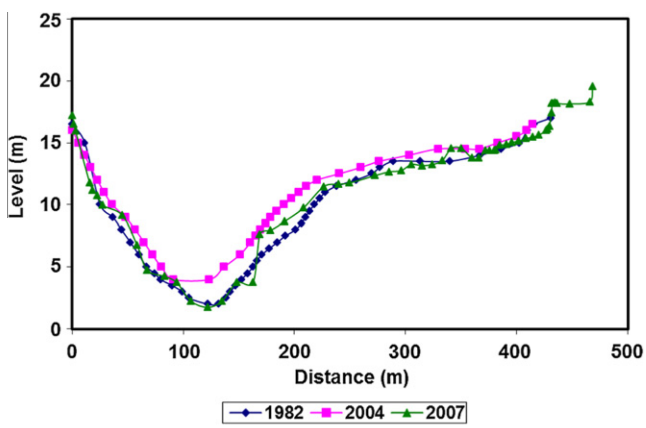

Figure 3 Cross-section comparison for the years 1982, 2004 and 2007 at sec (3) at 8.01 km downstream El-Roda gauging station.

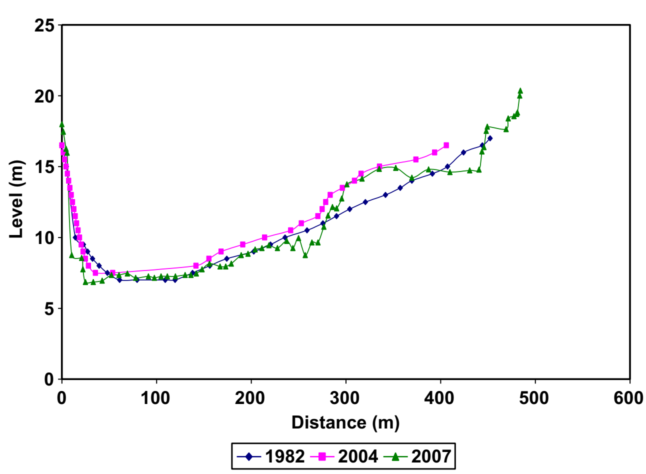

Figure 4 Cross-section comparison for the years 1982, 2004 and 2007 at sec (12) at 8.49 km downstream El-Roda gauging station.

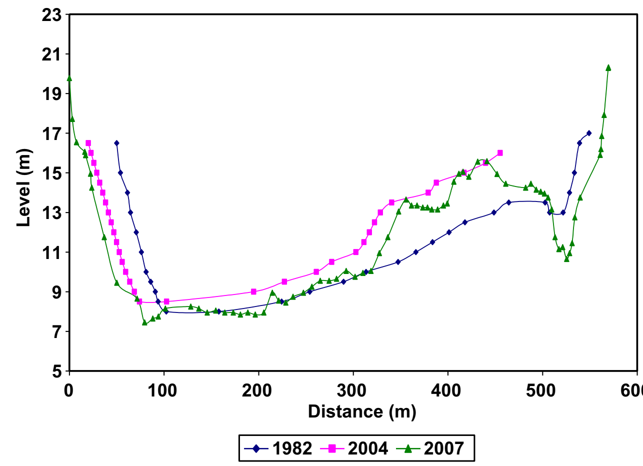

Figure 5 Cross-section comparison for the years 1982, 2004 and 2007 at sec (14) at 8.67 km downstream El-Roda gauging station.

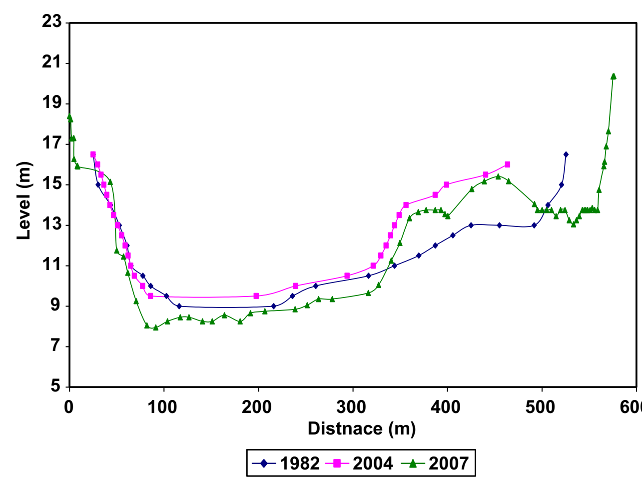

Figure 6 Cross-section comparison for the years 1982, 2004 and 2007 at sec (15) at 8.77 km downstream El-Roda gauging station.

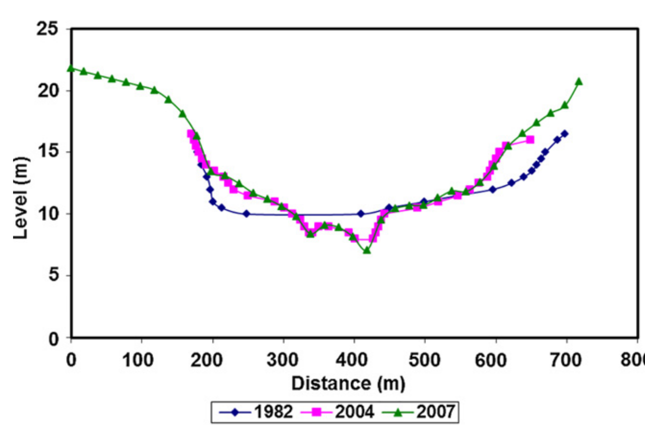

Figure 7 Cross-section comparison for the years 1982, 2004 and 2007 at sec (18) at 9.09 km downstream El-Roda gauging station.

7.1.1. The geometric data

The length of the study reach is about 1.53 km; this length was divided into 20 cross-sections. The distance between each two successive cross-sections ranges between 40 and 110 m.

Three different sets of cross-sections are used:

a) The first sets are the cross-sections extracted from the Nile contour maps developed by (Nile Research Institute) NRI 1982 [4].
b) The second sets are the cross-sections extracted from the Nile hydrographic maps developed by NRI 2004 [5].
c) The third sets are the cross-sections which were surveyed in 2007.

7.1.2. The hydrologic data

a) The water levels were recorded at 12 gauging stations along Reach Four: which begins at downstream Assuit Barrage at 382.22 km upstream Roda Gauge Station and the last station at Delta Barrage at 26.25 km downstream Roda Gauge Station.
b) Daily discharges downstream Assuit Barrage was collected during the period from 1982 to 2009. Fig. 8 shows the frequency of the discharge during this period.

7.1.3. Bed material sample

The samples of the bed material along with the study reach were collected by (NRI). The bed material of the study area consists of sand and silt. The fine sand is ranged between 60.61% and 88.06% of the bed material sample on the other hand; the percentage of silt is ranged between 0.82% and 14.45% of the bed material sample.

7.2. Numerical model: Center for Computational Hydrosince and Engineering (CCHE-2D)

CCHE-2D model is a free model developed by National Center for Computational Hydrosince and Engineering (NCCHE), The University of Mississippi as one as a group of many models in hydrosince area. The 2-D hydrodynamic model (CCHE-2D) used for simulating the flow field is based on the solution of Navier–Stokes equations for turbulent flow [6,7]. The governing equations of (CCHE-2D) used for simulating the flow field are the momentum equations in x and y directions, in addition to the continuity equation and sediment transport equation.

The family of CCHE-2D model is an integrated package for simulation and analysis of free surface flows, sediment transport and morphological processes. In addition to the numerical model, this family includes two more members: a mesh generator (CCHE-2D Mesh Generator) and a Graphical Users Interface (CCHE-2D-GUI) [8–10]. The first module concerned with discretization of the study area, while the second one can be considered as visual interface, see Fig. 9.

7.2.1. Mesh generation

The first step to use (CCHE-2D) model is to generate a mesh which is used to represent the computational domain and discretize the governing equations. A mesh composed of 27 by 15 lines (J&I) is representing the study reach of 1.53 km. The mesh lines are dense around the intake of the pump station as shown in Fig. 10.

7.2.2. Model calibration for water levels

The discharge released through Reach Four during the period from 1982 to 2009, Fig. 8 ranged from 224.54 to 2116.67 m^3/s with an average amount 1187.5 m^3/s. The model calibration includes three scenarios (low, average and high flows are released through reach four) during the period from 1990 to 2010 as follow 436.34, 1331.02 and 2094.91 m^3/s. The results of (CCHE-2D) model were compared by the actual data.

Figs. 11–13 show that the results of the observed data are close with the results of (CCHE-2D) Model and that of the actual data.

7.2.3. Sediment transport and bed change simulation

The cross-section prediction is very important for any future studies. The first step for the model prediction will be to run the model for the three different scenarios (436.34, 2094.91 and 1337.73 m^3/s) for one year.

The second steps, three trials with one or two dikes with different positions were tried. The first trail was construct a dike at the western part of cross-section number 5, the second trail was construct two dikes at the western part of cross-sections

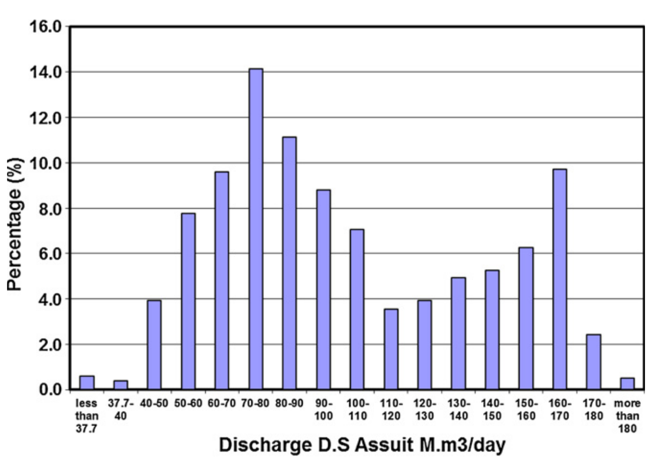

Figure 8 Frequencies of water discharges down stream Assuit Barrage (1982–2009).

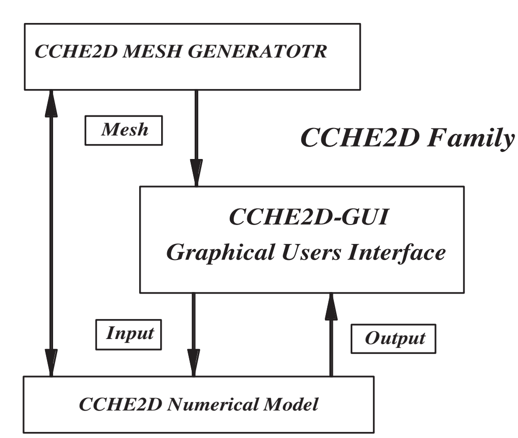

Figure 9 Components of numerical model (CCHE-2D).

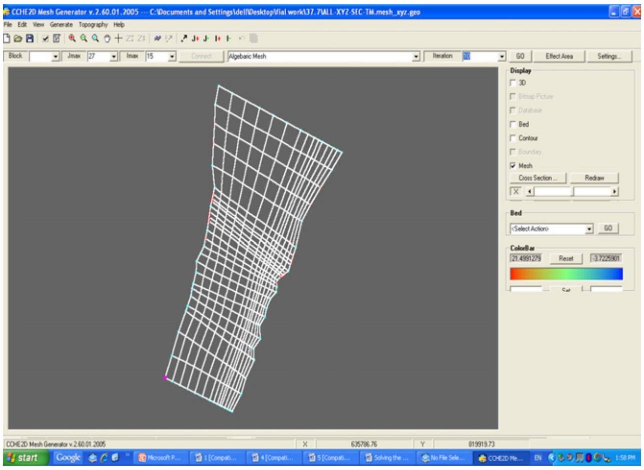

Figure 10 Mesh representing the study area.

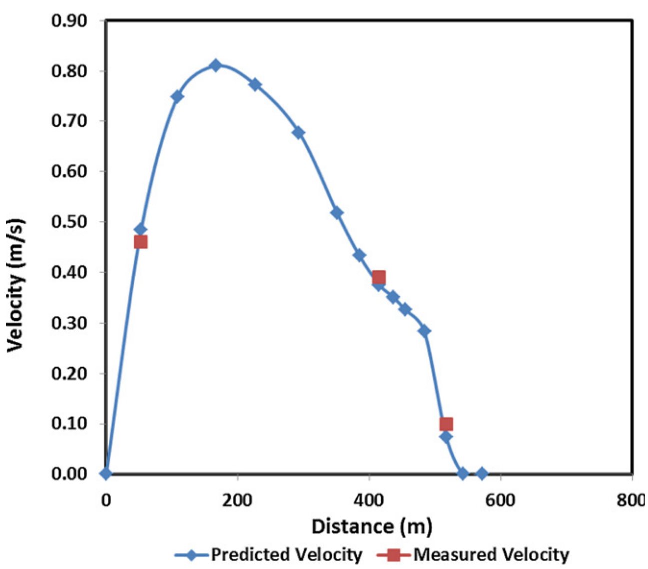

Figure 11 Comparison between predicated and measured velocity at cross-section (14) at the intake.

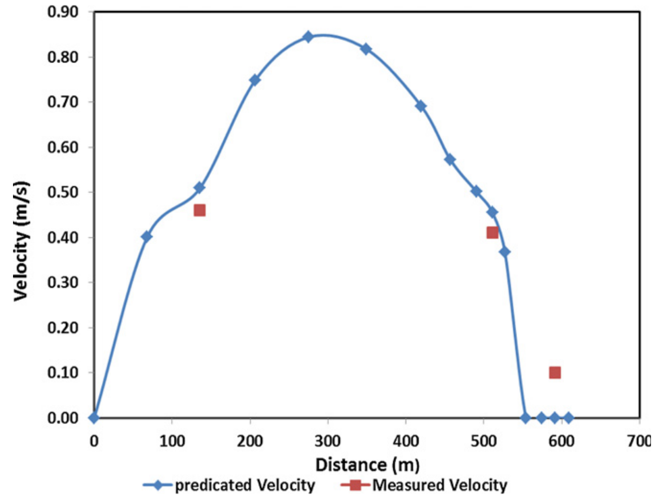

Figure 12 Comparison between predicated and measured velocity at cross-section (18) downstream the intake.

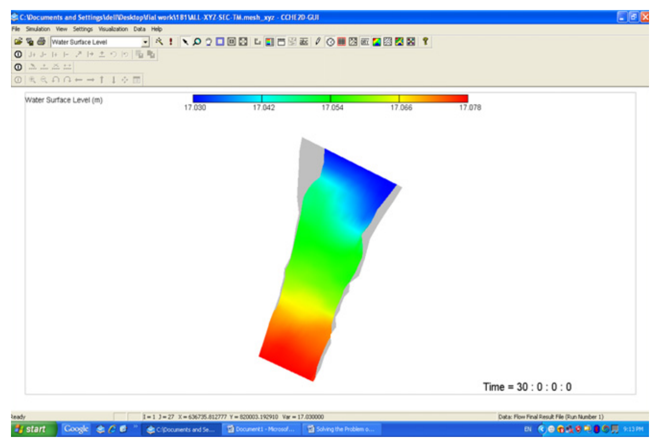

Figure 13 Water level calibration for a flow of 2094.91 m^3/s.

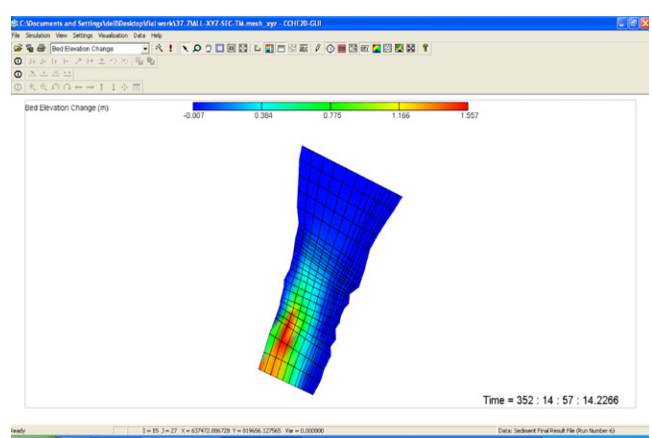

Figure 14 Bed level changes due to passing a flow of 436.34 m^3/s for one year.

number 5 and 7 and the last trail was construct two dikes at the western part of cross-sections number 2 and 5. Finally, running the model with 1337.73 m^3/s for each trails.

The last step, two bed levels were presented according to dredging the bed. First; bed tested was at a level 14 (one meter under the minimum water level in the study reach) where second; bed tested was at a level 12.5 (2.5 m under the minimum water level in the study area). Finally, running the model with 1337.73 m^3/s for each bed level.

8. Analysis

Several scenarios were presented:

First scenario was by running the model with 436.34 m^3/s (minimum flow) for one year. Figs. 14 and 30 show that there is a very minor change in the bed level (deposition up to 0.05 m) at the area of the intake of the pump station except the western part of the study area upstream of the intake and a deposition up to 1.57 m. Fig. 15 shows the velocity at the eastern side of the study area is slow velocity especially at the intake of the pump station up to 0.14 m/s and a high

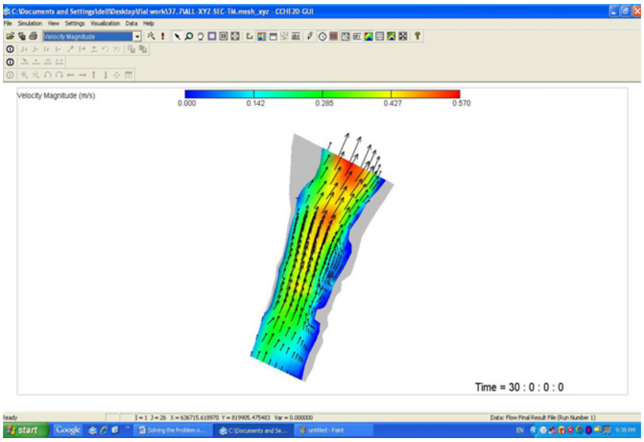

Figure 15 Velocity magnitude due to passing a flow of 436.34 m^3/s for one month.

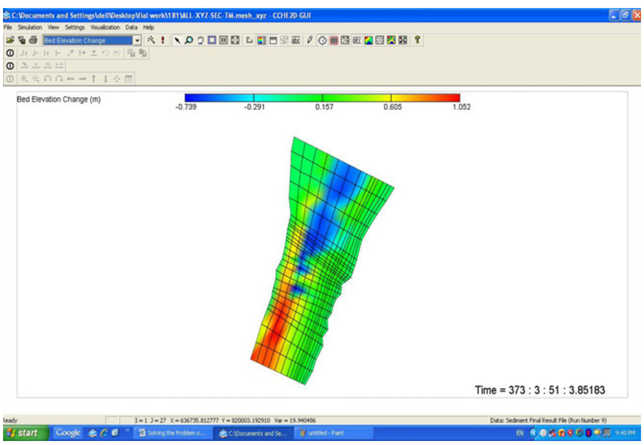

Figure 16 Bed level changes due to passing a flow of 2094.91 m^3/s for one year.

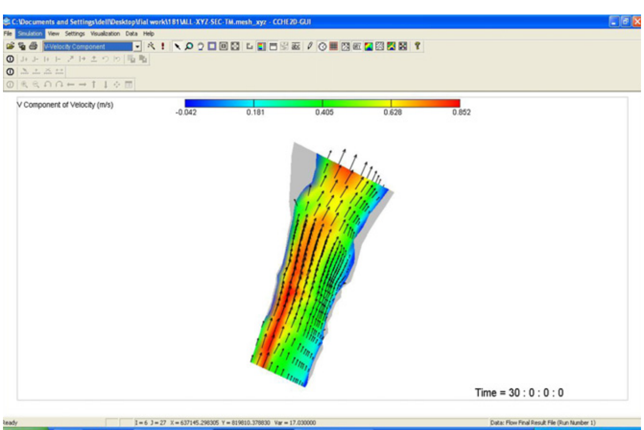

Figure 17 Velocity magnitude due to passing a flow of 2094.91 m^3/s for one month.

velocity at the middle part of the cross-section at the intake up to 0.57 m/s and downstream the intake.

Second scenario was by running the model with 2094.91 m^3/s (maximum flow) for one year. Figs. 16 and 31 show that there is a minor change in the bed level at the area of the intake of the pump station (deposition up to 0.15 m) expect the middle part of the cross-section at the intake has a degradation up to 0.64 m and downstream the intake. On the other hand the western part of the study area has a deposition up to 1.05 and downstream Imbaba Bridge. Fig. 17 shows that the velocity at the eastern side of the study area has a slow velocity especially at the intake of the pump station up to 0.22 m/s and a high velocity at the middle part of the cross-section up to 0.92 m/s and downstream the intake.

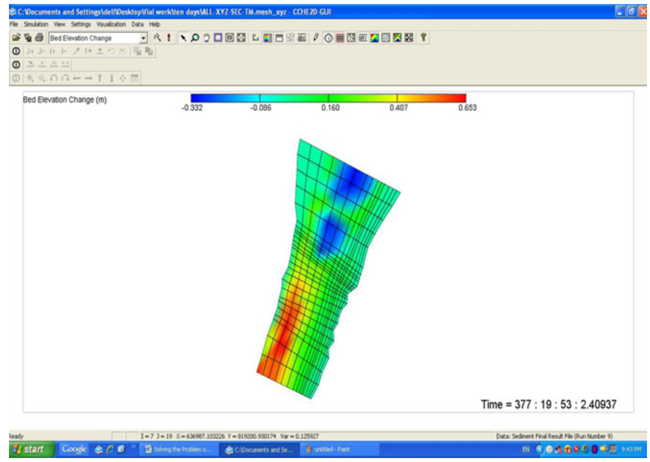

Figure 18 Bed level changes due to passing a flow of 1337.73 m^3/s for one year.

Third scenario was by running the model with 1337.73 m^3/s (average flow) for one year. Figs. 18 and 32 show there is a some change in the bed level at the area of the intake of the pump station (deposition up to 0.11 m) except the middle part of the cross-section has a degradation up to 0.22 m and downstream of the intake. On the other hand the western part of the study area downstream Imbaba Bridge has a deposition up to 0.65 m. Fig. 19 shows that the velocity at the eastern side of the study area has a slow velocity especially at the intake of the pump station up to 0.16 m/s and a high velocity at the middle

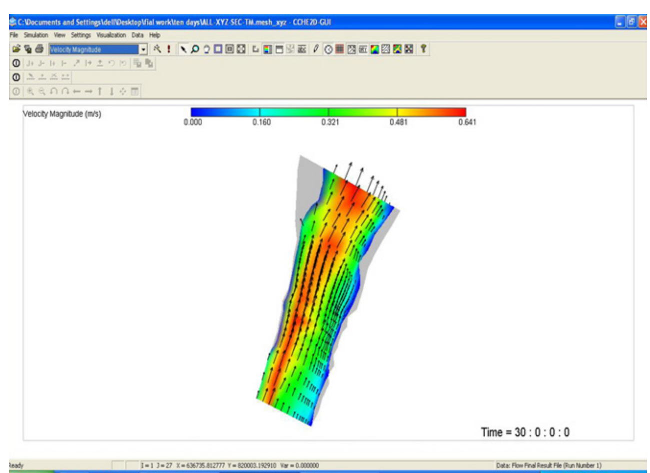

Figure 19 Velocity magnitude due to passing a flow of 1337.73 m^3/s for one month.

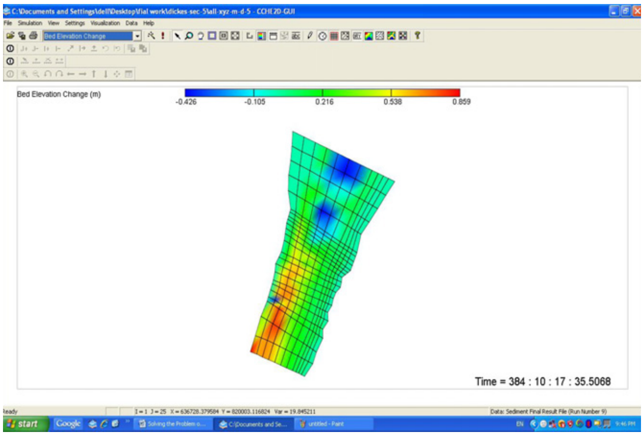

Figure 20 Bed level changes due to passing a flow of 1337.73 m^3/s for one year with a dike at cross-section number 5.

part of the cross-section up to 0.64 m/s and downstream the intake.

9. Alternative for sediment control

The flow-field features described above were considered by means of several sediment-control modifications used to modify the local flow field and to adjust the local riverbed bathymetry. In concept, the modifications would greatly decrease the amount of sediment accumulating at the intake, and eliminate the substantial buildup of sediment within the intake structure [11].

Complete prevention of sediment entry in. the intake was considered infeasible. Some sediment conveyed suspended in the flow will still enter the intake. The study, therefore, aimed to prevent all but a trace of sediment (i.e., only a little medium/fine-size sand and silt) from entering the intake. This fine sediment would remain in suspension and pass through the station's circulating water system.

Two alternative sets of sediment-control modifications were considered:

1. A submerged guide wall (dike or dikes) extending partially across the river sections at different locations [11,12].

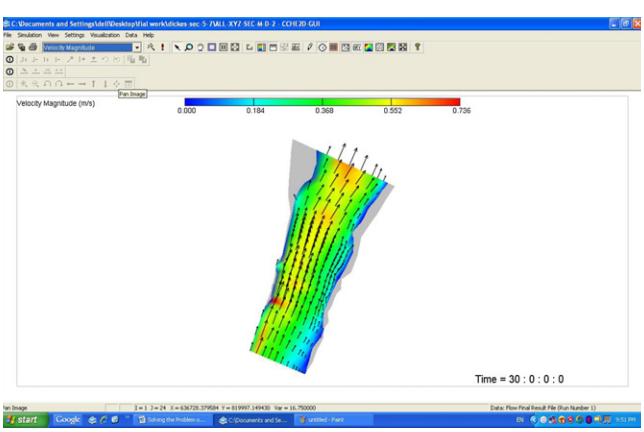

Figure 21 Velocity magnitude due to passing a flow of 1337.73 m^3/s for one month with a dike at cross-section number 5.

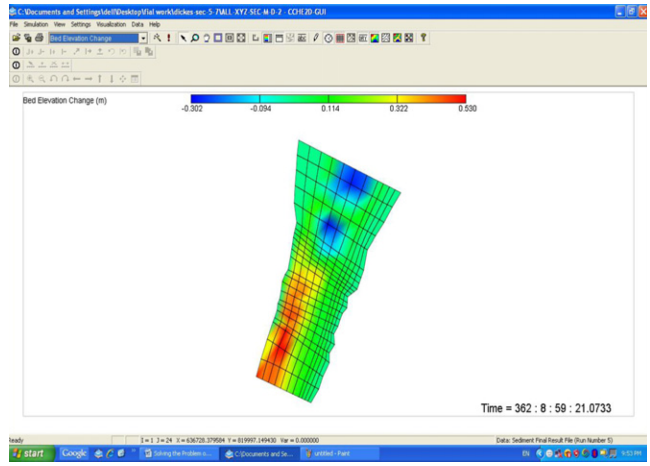

Figure 22 Bed level changes due to passing a flow of 436.34 m^3/s for one year with two dikes at cross-sections numbers 5 and 7.

2. Dredging the study area at different levels under the minimum water level in the study area.

9.1. First alternative [submerged guide wall (dike)]

First scenario was by running the model with 1337.73 m^3/s for one year after constructing a dike at Section 5 at 500 m upstream the intake. Figs. 20 and 33 show that there is a change in the bed level (deposition up to 0.14 m) at the area of the intake of the pump station and there is a degradation downstream the intake up to 0.43 m. On the other hand there is a deposition up to 0.86 m at the western part of the study area downstream Imbaba Bridge. Fig. 21 shows that the velocity at the eastern side of the study area has a velocity at the intake of the pump station up to 0.21 m/s and a high velocity around the dikes up to 0.85 m/s (this is normal because the submerged dike makes the cross-section narrower and the velocity becomes higher).

Second scenario was by running the model with 1337.73 m^3/s for one year after constructing two dikes at Sections 5 and 7 at 500 and 390 m upstream the intake

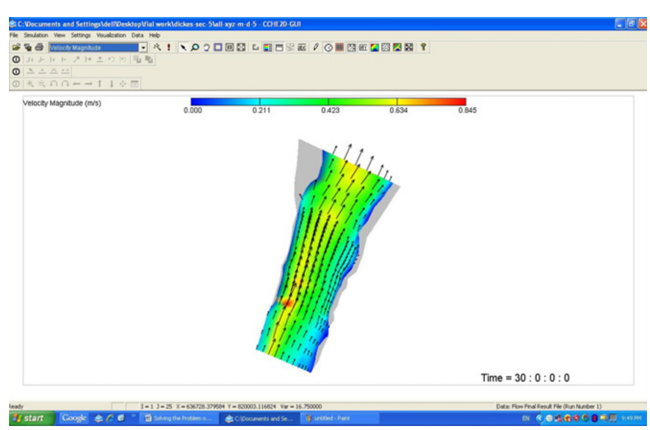

Figure 23 Velocity magnitude due to passing a flow of 1337.73 m^3/s for one month with two dikes at cross-sections number 5 and 7.

respectively. Figs. 22 and 34 show there is a change in the bed level (deposition up to 0.08 m) at the area of the intake of the pump station and there is degradation up to 0.3 m downstream the intake. On the other hand there is a deposition up to 0.53 m at the western part of the study area downstream Imbaba Bridge. Fig. 23 shows that the velocity at the eastern side of the study area has a velocity up to 0.18 m/s at the intake of the pump station and a high velocity between 0.55 and 0.74 m/s around the dikes ranges at Sections 5 and 7 respectively (which is normal because the submerged dike makes the cross-section narrower so the velocity become higher).

Third scenario was by running the model with 1337.73 m^3/s for one year after constructing two dikes at Sections 2 and 5 at 752 and 500 m upstream the intake respectively. Figs. 24 and 35 show that there is a change in the bed level (deposition up to 0.03 m) at the area of the intake of the pump station and there is degradation up to 0.08 m downstream the intake. On the other hand there is a deposition up to 0.13 m at the western part of the study area downstream Imbaba Bridge. Fig. 25 shows that the velocity at the eastern side of the study area has a slow velocity especially at the intake of the pump station up to 0.18 m/s and a high velocity around the dikes up to 0.74 m/s (this is normal because the submerged dike makes the cross-section narrower so the velocity become higher).

9.2. Second alternative [dredging]

The depth of water under minimum water level (15.0) and at the cross-section representing the first group of intake is ranged between 0 on the eastern side to 3.2 m on the western side. On the other hand; the depth of water under minimum water level at the cross-section representing the second group of intake is ranged between 0.75 and 6.75 m. Finally, the depth of water under minimum water level at the cross-section representing the third group of intake is ranged between 0. 5 and 3.2 m.

First scenario was by running the model with 1337.73 m^3/s for three and half years after dredging the study area at level 14 (one meter under the minimum water level in the study area).

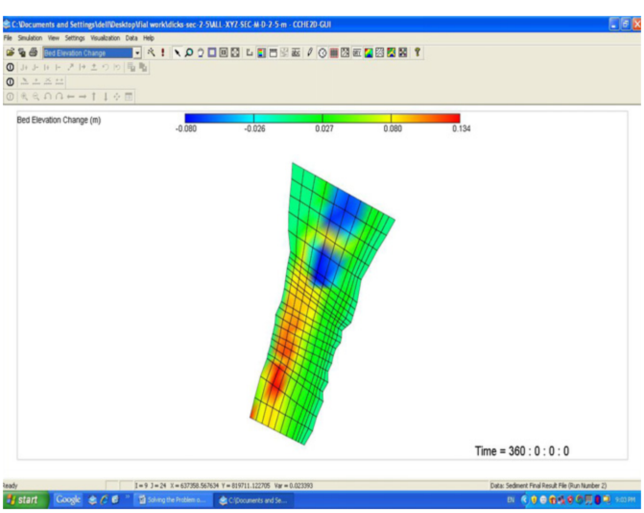

Figure 24 Bed level changes due to passing a flow of 436.34 m^3/s for one year with two dikes at cross-sections numbers 2 and 5.

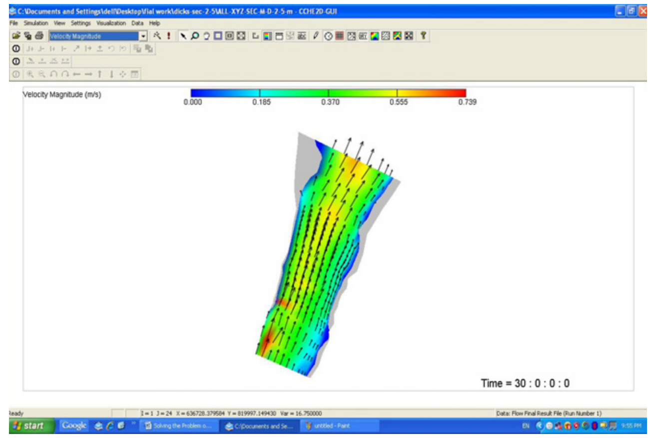

Figure 25 Velocity magnitude due to passing a flow of 1337.73 m^3/s for one month with two dikes at cross-sections numbers 2 and 5.

The volume of dredging is calculated using GIS technique as shown in Fig. 38 this volume of dredging is 261,000 m^3 at level (14 m). Figs. 26 and 36 show that there is a change in the bed level (deposition up to 0.25 m) at the area of the intake of the pump station and there is degradation up to 0.5 m downstream the intake. On the other hand there is a deposition up to 0.91 m at the western part of the study area downstream Imbaba Bridge. Fig. 27 shows that the velocity at the eastern side of the study area has a velocity up to 0.25 m/s at the intake of the pump station and a high velocity up to 0.63 m/s downstream at the intake.

Second scenario was by running the model with 1337.73 m^3/s for four years after dredging the study area at level 12.5 (2.5 m under the minimum water level in the study area). The volume of dredging is calculated using GIS technique as shown in Fig. 39 this volume of dredging is 521,000 m^3 at level (12.5 m). Figs. 28–37 show that there is a change in the bed level (deposition up to 0.2 m) at the area of the intake of the pump station and there is degradation up to 0.5 m downstream at the intake. On the other hand there is a deposition up to 0.91 m at the western part of the study area downstream Imbaba Bridge. Fig. 29 shows that the

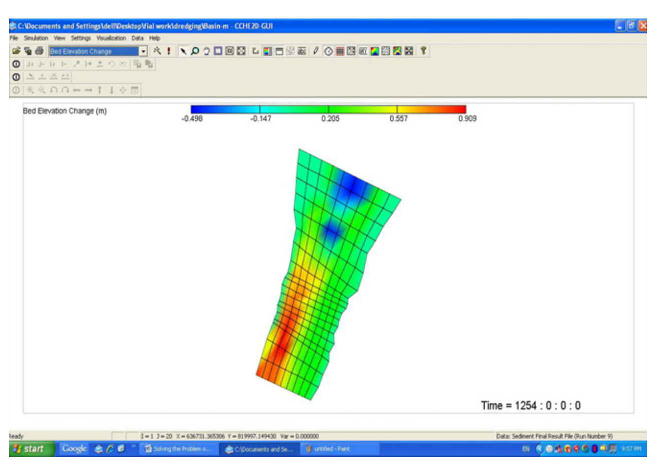

Figure 26 Bed level changes due to passing a flow of 1337.73 m^3/s for three years and half with a dredging at level 14.

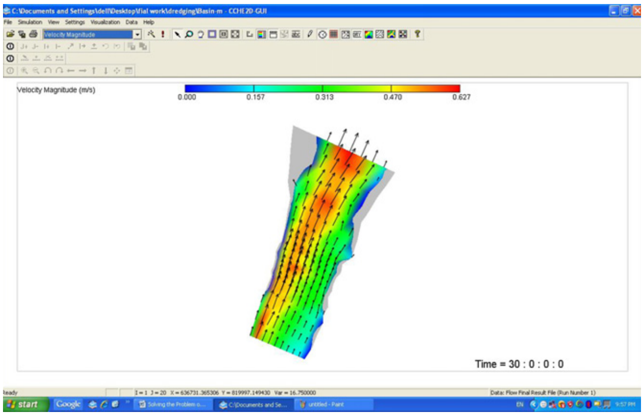

Figure 27 Velocity magnitude due to passing a flow of 1337.73 m^3/s for one month with a dredging at level 14.

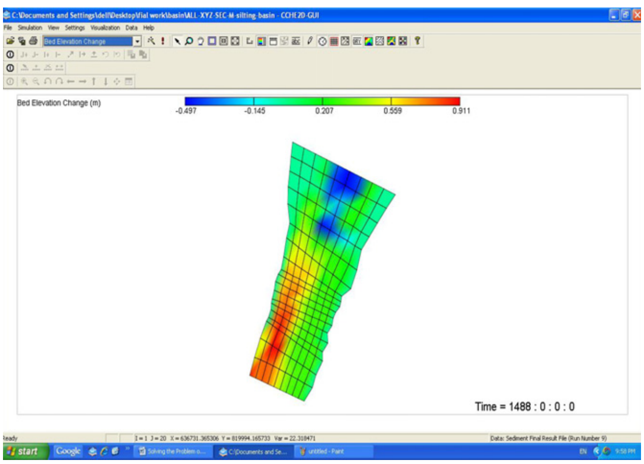

Figure 28 Bed level changes due to passing a flow of 1337.73 m^3/s for four years with a dredging at level 12.5.

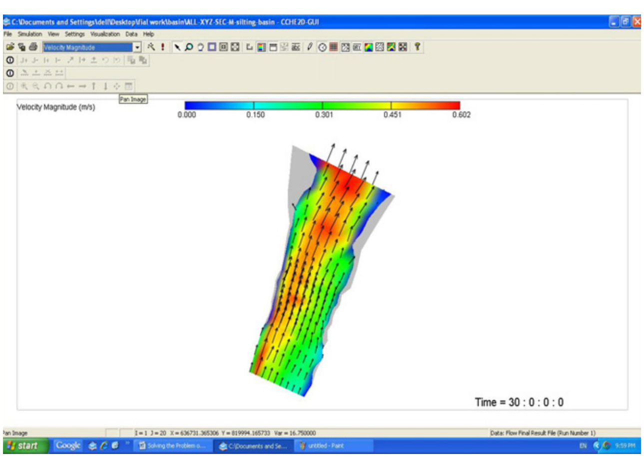

Figure 29 Velocity magnitude due to passing a flow of 1337.73 m^3/s for one month with a dredging at level 12.5.

velocity at the eastern side of the study area has a slow velocity especially at the intake of the pump station up to 0.2 m/s and a high velocity downstream at the intake up to 0.6 m/s.

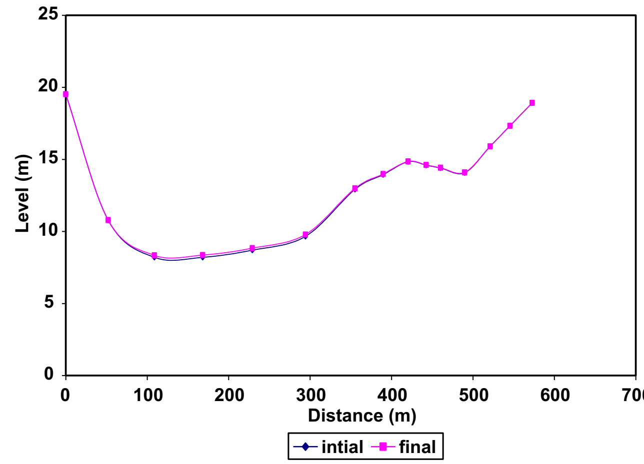

Figure 30 Cross-section comparison in the year 2007 and after passing flow 436.34 m^3/s for one year at intake at 8.67 km downstream El-Roda gauging station.

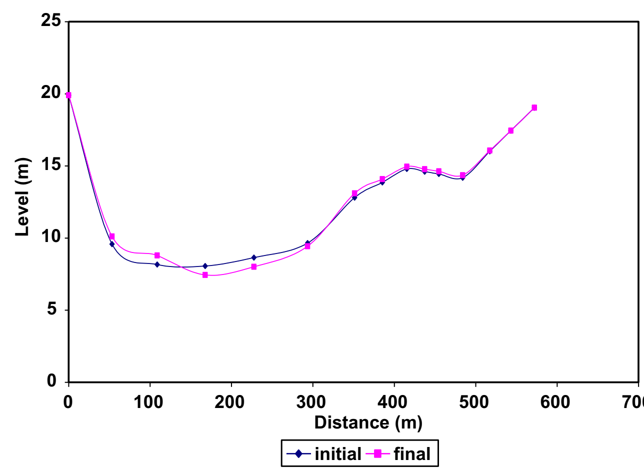

Figure 31 Cross-section comparison in the year 2007 and after passing flow 2094.91 m^3/s for one year at the intake at 8.67 km downstream El-Roda gauging station.

10. Discussion

Discussion of the problem of sedimentation at water intake of Rowd El-Farag pump station during the period from 1982 to 2007 using three sets of cross-sections in 1982, 2004 and 2007 covering a study area with a length of 1.53 km which is located between 7.785 and 9.31 km from El-Roda gauging station. The intake of the station is located at 8.63 km from El-Roda gauging station. The study showed that there is a significant morphological change in this reach due to the long study period and two hydraulic Structures (Imbaba Bridge and Rowd El-Farag Bridge just upstream and downstream the study area) respectively. Dredging the navigation path in the western side of the study area during this period (through the Nile navigation project) was the result due to placing the dismantled Abu Al-Ela metallic Bridge store just downstream at the intake.

CCHE-2D model is used for simulating both flow and sediment for different flows scenarios (436.34, 1337.73 and 2094.91 m^3/s) for one year the results showed that there are

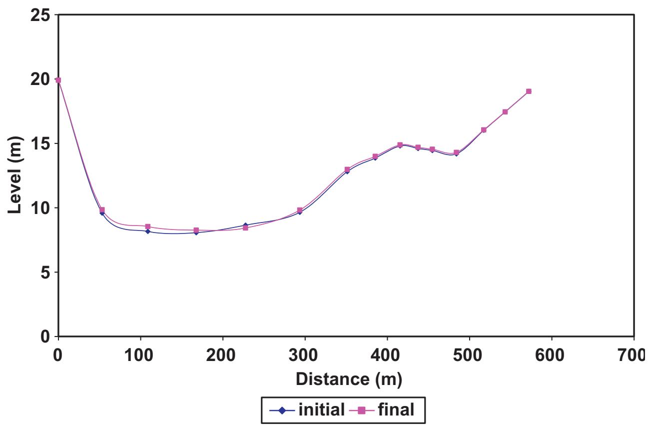

Figure 32 Cross-section comparison in the year 2007 and after passing flow 1337.73 m^3/s for one year at the intake at 8.67 km downstream El-Roda gauging station.

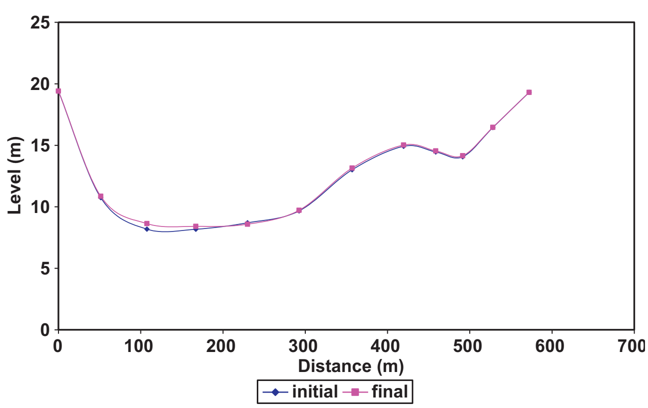

Figure 33 Cross-section comparison in the year 2007 and after passing flow 1337.73 m^3/s for one year with a dike at sec (5) at the intake at 8.67 km downstream El-Roda gauging station.

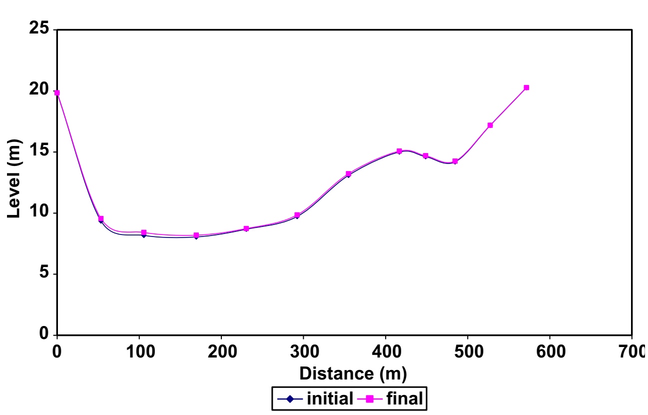

Figure 34 Cross-section comparison in the year 2007 and after passing flow 1337.73 m^3/s for one year with dikes at sec (5 and 7) at the intake at 8.67 km downstream El-Roda gauging station.

a small amount of deposition ranging from (0.05 to 0.15 m) at the intake and a significant deposition (0.65 to 1.57 m) down-

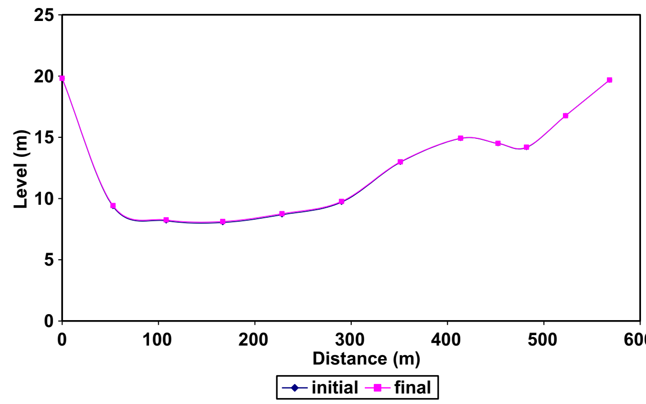

Figure 35 Cross-section comparison in the year 2007 and after passing flow 1337.73 m^3/s for one year with dikes at sec (2 and 5) at intake at 8.67 km downstream El-Roda gauging station.

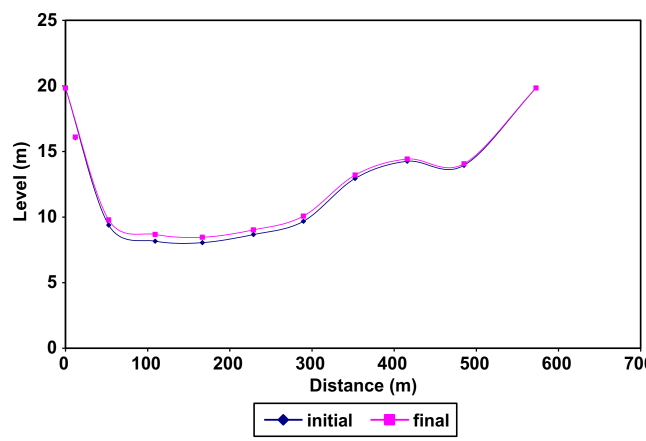

Figure 36 Cross-section comparison in the year 2007 and after passing flow 1337.73 m^3/s for three years and half after dredging at level 14 m at intake at km 8.67 downstream El-Roda gauging station.

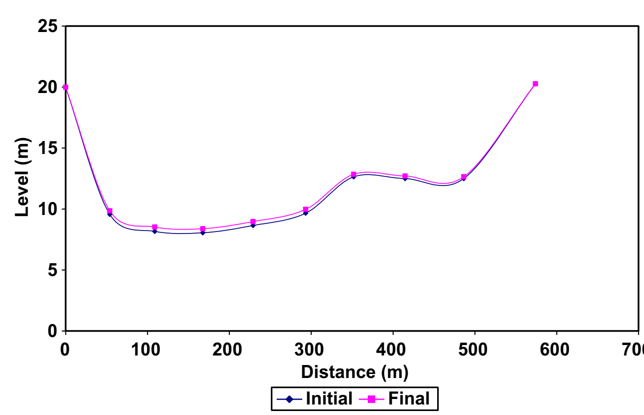

Figure 37 Cross-section comparison in the year 2007 and after passing flow 1337.73 m^3/s for four years after dredging at level 12.5 m at intake at km 8.67 downstream El-Roda gauging station.

stream Imbaba Bridge on the other hand there is a scour in the middle and western part of the stream especially downstream at the intake.

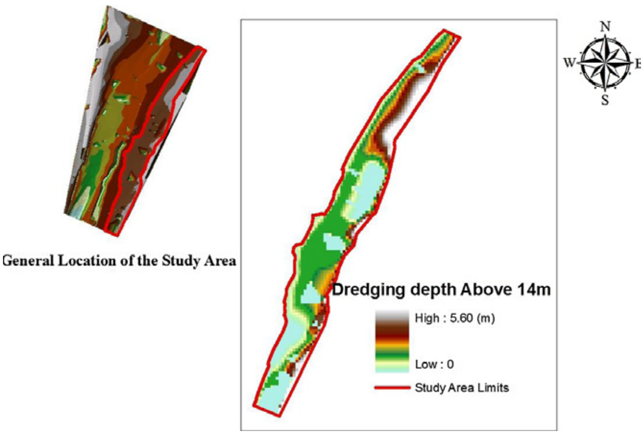

Figure 38 Dredging area at level above 14 m.

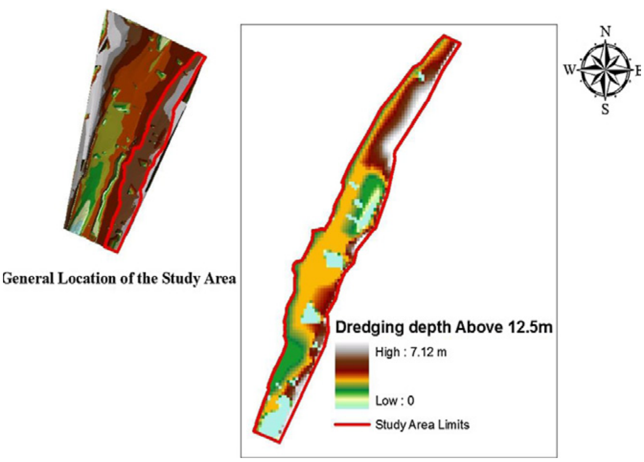

Figure 39 Dredging area at level above 12.5 m.

Two alternative sets of sediment-control modifications were considered:

1. A submerged guide wall (dike or dikes) extending partially across the river at different locations. The results show that this solution does not solve the problem of sedimentation at the intake of the pump station.
2. Dredging the study area at level 14 (one meter under the minimum water level in the study area) or dredging the study area at level 12.5 (2.5 m under the minimum water level in the study area) will solve the problem of sedimentation at the intake of the pump station. For the dredging at level 14 m one meter under the minimum water level in the study area was recommended by the NRI after three years and half with the same series of flows passed during the period from 2007 to 2010 the model result showed that there is a only deposition of 0.25 m at the intake. On the other hand the second alternative after dredging the study area at level 12.5 (2.5 m under the minimum water level in the study area) after four years with the same series of flood passed in the period from 2007 to 2010 the model result showed that there is only deposition of 0.2 m at the intake. So this is a good solution although the dredging is an expensive solution. This solution will cost 2.6 Million Egyptian pounds if the dredging at level 14 m and will cost 5.2 Million Egyptian pounds if the dredging at level 12.5 m (the dredging of 1 m^3 will cost 10 Egyptian pounds).

11. Conclusions

As a sustainable solution for the control of the sediment depostion around the pump station intake, it is advisable to dredge the pump intake area to the level 14 m (one meter under the minimum water level) as the solution costs less (L.E 2.6 Million) than dredging to the level 12.5 m which costs (L.E 5.2 Million).

References

[1] NRI, Study the problem of sedimentation at Rowd El-Farag water station intake, NRI report, NWRC, Egypt, 2007.
[2] NRI, Study the problem of sedimentation at Rowd El-Farag water station intake, NRI report, NWRC, Egypt, 2002.
[3] Hydraulic Research Institute, Study of Nile River Bed at the Cairo Bridges, Cairo, Egypt, 1981.
[4] NRI, Nile Research Institute contour maps, Nile Research Institute, National Water research Center, El-Qanater, Egypt, 1982.
[5] NRI, Nile Research Institute recent topographic and hydrographic digital maps, Nile Research Institute, National Water research Center, El-Qanater, Egypt, 2007.
[6] Jia Y, Wang SSY. Numerical model for channel flow and morphological change studies. J Hydraul Eng, ASCE 1999;125(9):924–33.
[7] Jia Y, Wang SSY, Yichen Xu. Validation and application of a 2D model to channels with complex geometry. Int J Comput Eng Sci 2002;3(1):57–71.
[8] Zhang Yaoxin. CCHE2D – GUI-Graphical user interface for the CCHE2D model Users' Manual-Version 2.2. NCCHE Technical Report. NCCHE-TR-2005-03, 2005.
[9] Zhang Yaoxin, Yafei Jia. CCHE2D Mesh Generator Users' Manual – Version 2.6. NCCHE Technical Report. NCCHE-TR-2005-05, 2005.
[10] Weiming Wu. CCHE2D Sediment Transport Model version 2.1 NCCHE Technical Report. NCCHE-TR-2001-03, 2001.
[11] Michell F, Ettema R, Muste M. Case study: sediment control at water intake for large thermal-power station on a small river. ASCE 2006.
[12] Rajatnam N, Nwachukwu BA. Flow near groin-like structures. J Hydraul Eng, ASCE 1983;109(3):480–634.

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