

2-D MODELING OF FLOW DYNAMICS DOWNSTREAM NEW ASSUIT BARRAGE

M. B. Ezzat, R. A. Rady, F. S. Fahmy

Researcher, National Water Research Center (EGYPT)
E-mails: mohbahgat@gmail.com, redam@mwri.gov.eg, fahmi@hri-egypt.org

ABSTRACT

To make optimum use of the River Nile water, a decision to construct a new barrage downstream the old Assuit Barrage was taken based on a detailed feasibility study. The impacts of constructing the new Barrage were assessed using the 2-D capabilities of a three-dimensional hydrodynamic model "Delft-3D". The model was calibrated against measured data, precisely velocity distributions from field survey at the Barrage site. The impact of constructing the Barrage on the flow distribution and velocities was then investigated using this model. Model predicted hydrodynamic parameters, without and with the Barrage, were analyzed in detail. The model revealed that the best flow distribution between the left and right branches of Bani-Murr Island is 60% and 40% respectively. In the same context, the velocity comparison between baseline tests and the Barrage's different operating schemes indicates that the proposed alternatives has almost no effect on the velocity magnitudes or flow distribution at the downstream reach, except for the case of passing 1000 m³/s through the powerhouse with equal flow distribution between Bani-Murr Island branches.

Key words: Barrage; 2-D Modeling; Velocity Distribution

1. INTRODUCTION

Water resources management in Egypt depends on a complex set of infrastructure along the entire length of the river. The key element of this infrastructure is the High Aswan Dam (HAD) that forms Lake Nasser. Downstream of the HAD, there are seven barrages to increase the river's water level so that it can flow into first-level irrigation canals. One of them is the 350 km long Ibrahimiya Canal completed in 1873, the largest artificial canal in the world. It branches off the left bank of the Nile in Assiut and then runs parallel to the river. Its discharge is increased by the Assiut Barrage completed in 1903. Other large barrages exist at Esna and Naga Hammadi on the main Nile, as well as the Delta Barrage, the Zifta Barrage and the Damietta Barrage on the Damietta branch and the Edfina Barrage on the Rosetta branch of the Nile.

Given that new sources of water are limited, the Nile undoubtedly remains the main source of water in Egypt; with 90 % of the population living on 5 % of the national territory along the river banks. Under such conditions, competition for water for agriculture, electricity, domestic and industrial use prompts for efficient water management. Moreover, the water management system, comprising 200 control structures (dams and regulators), which was established in early 1900s requires to be updated to adequately respond to the increasing and competing need for water.

In response to these challenges, the Government of Egypt has embarked on an extensive program in the Water Resource and Irrigation Sector, captured in the Integrated Water Resource Management Strategy (IWRMS) up to year 2017. The major expected outcomes of the IWRMS include the development of a master plan of the grand barrages and regulators, assessment of the conditions of these infrastructures and the proposal of an action plan with the view to meeting water demand through optimal management. Most of the infrastructures concerned in the master plan are at risk as their reconstructions or rehabilitations are overdue, affecting the stability of these structures. In this context, the Government of Egypt has made the decision to review and take actions for specific infrastructures, including the Assuit Barrage. The Assuit Barrage serves for electricity as well as domestic and industrial use. Like most of the old water infrastructures on the Nile and branches, the Assuit Barrage shows signs of deterioration dating back some years now, and hence preventing its optimal operation.

Visible cracks on major segments of the Barrage are noticed. In addition, the equipment for water control such as the vents and associated mechanic system are also in a bad condition, thereby generating high operating and maintenance costs. In order to remedy the situation, the Government undertook a pre-feasibility study in 2005 with the assistance of Mott MacDonald in association with CES Salzgitter, Fichtner and Inros Lackner, all of Germany, and Hamza Associates of Egypt, to investigate the present structural and operational conditions at the Barrage and to establish the need for either rehabilitation of the current Barrage or reconstruction of a new structure. The results of the said study were in favor of a reconstruction of a new Barrage. This new Barrage would provide an increase in the head, allowing for more water discharges into the diversion canal. The new Barrage will also include a low head hydropower plant providing about 40 Megawatts.

The aim of the current study is to investigate the impact of constructing the new Assuit Barrage on the hydrodynamic processes at the downstream side using 2-D capabilities of a 3-D numerical model "Delft-3D". Combinations of different operation modes and flow distributions were tested to figure out the most appropriate operation strategy for the new Barrage. This implies examining the effect of constructing the new Barrage on the River regime downstream the new Barrage, which will be obviously clear through predicting the discharge distribution on both sides of Bani-Murr Island, the discharge distribution on both sides of Walidia Island, and to explore how far the

flow conditions will meet the need for sufficient water discharge into Walidia Power Plant. Moreover, the natural River flow velocity distributions at certain locations, especially on both sides of Walidia and Bani-Murr islands for different operating modes were predicted.

Study area

The study area is located in Assiut city adjacent to an intensive populated area called "Walidia". Assiut city lies just to the upstream side of the project. About 600 m downstream of the existing Barrage, Bani-Murr Island begins and extends for about 2 km. The island splits the downstream channel into two branches with the western branch wider and shallower. The intake for a 650 MW thermal power plant is located on that branch, about 2.8 km downstream of the exiting Barrage. The existing navigation lock is located on the western side of the existing Barrage. Assiut Barrage, 400 km upstream of Cairo, is the last barrage downstream of the High Aswan Dam (HAD) before the Nile reaches Cairo (Fig. 1). It was built between 1898 and 1902 in order to divert Nile river flows to the Ibrahimiya canal. The Barrage was remodeled extensively between 1934 and 1938, increasing the annual discharge to the Ibrahimiya canal which, in its present form, has a length of about 350 km and irrigates an area of 690,000 ha. The Barrage was designed as an arched viaduct founded on a mass concrete floor, with a 16 m wide lock positioned on the extreme left bank (Fig. 2). The overall length of the structure is 820 m with a water-way capable of discharging 14,000 m³/s provided by 110 individual openings of 5 m width. Each opening contains a double leaf vertical lift roller gate designed for a maximum head difference of 4.2 m. The Barrage's primary purpose was to ensure adequate irrigation supplies to Middle Egypt during the early summer; as such it was not intended to be used during the annual Nile flood. The original 1902 design assumed that all gates would be fully raised to allow the flood to pass unheeded.

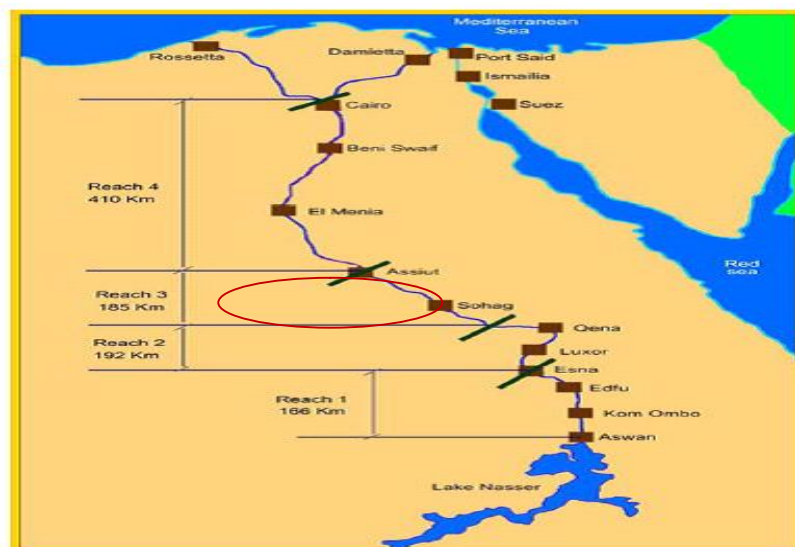


Fig. 1. Location of Assiut Barrage



Fig. 2. Old Assiut Barrage

A feasibility study was carried out by Mott MacDonald in association with CES Salzgitter, Fichtner and Inros Lackner, all of Germany, and Hamza Associates of Egypt, to investigate the present structural and operational conditions at the Barrage and to outline options for the future. The principal conclusion of this feasibility phase, completed in December 2005, was a recommendation to construct a new barrage downstream of the existing one rather than rehabilitating the existing Barrage. The design of the new Barrage at Assiut commenced with an extensive qualitative assessment of thirteen potential sites, ranging from a location some 3.5 km upstream of the existing Barrage, to a position some 2.5 km downstream. The limiting factors on the location were the need to construct a separate link canal once the new Barrage position extended upstream of the Ibrahimiya Canal, and the extensive remedial works to lower groundwater levels in a dense urban environment once the barrage location is downstream of the existing structure. The finally accepted position was between 200 m and 300 m downstream of the existing Barrage. The proposed Barrage components are:

- Sluiceway: 8 radial gates, 17 m wide;
- Hydropower plant: 4 turbines x 8 MW;
- Additional navigation lock: 160 x 17 m chamber;
- Closure dam: embankment type;
- Rehabilitation of the existing navigation lock; and
- Rehabilitation of the existing Ibrahimiya head regulator.

Numerical modeling of barrages

This section demonstrates some of the previous researches and studies that encompassed the use of numerical hydrodynamic models to explore the effect of constructing barrages on the flow characteristics. For example, the effects of constructing a barrage across the Tawe Estuary in South Wales were studied in (1). The proposed barrage was modeled to simulate its impact on the environment, including the pollutant behavior upstream of the barrage and sedimentation effects downstream. In addition, the ISIS numerical model was employed to optimize the hydraulic performance of the new Naga Hammadi Barrage stilling basin in Egypt (2). The model was utilized to simulate the sluiceway behavior of the new barrage and it showed very good results in simulating the water level drop and the flow velocity downstream the sill. Also, the two dimensional Tidal Flow Development numerical model was used to investigate the feasibility of constructing a new barrage for the Severn Estuary in UK with more emphasis on predicting the effect of such barrage on the tidal resonance in the channel (3). In the same context, both 2-D and 3-D numerical models were utilized to predict the close-to-surface flow patterns downstream the Mersey Barrage in UK (4). The 3-D model accurately predicted the velocity profiles. Moreover, the 3-D numerical computational fluid dynamics program "SSIIM" was used to predict the flow field downstream the Kuffa Barrage on the Euphrates River in Iraq (5).

2. METHODS

A (large-scale) numerical model, 2DH, hydrostatic assumption using Delft-3D software package was applied to investigate and satisfy the above mentioned objectives. Delft3D is an integrated, powerful and flexible software, which was developed by Deltares, the Netherlands. The Delft3D package is mostly used for the modeling of coastal, river and estuarine areas. It encompasses a number of well-tested and validated modules, which are linked to one-another.

Delft3D-FLOW is a commercial software program available from WL|Delft Hydraulics (now Deltares) and was used in this study. Delft3D-flow is the hydrodynamic module of Delft3D, which is WL|Delft Hydraulics' fully integrated program for modeling water flows, waves, water quality, particle tracking, ecology, sediment and chemical transport, and morphology. Its capability for modeling subcritical free surface flows, handling different types of boundary conditions, and availability of different turbulence models made the model suitable for use in this study. Mesh generation capabilities and bathymetry or topography generation are other powerful features available in this software.

Delft3D-FLOW is a multi-dimensional (2D or 3D) hydraulic (and transport) simulation model; it essentially solves the Navier-Stokes equations for incompressible fluid under shallow water and Boussinesq assumptions. In the vertical momentum equation, the vertical acceleration is neglected, which leads to the hydrostatic pressure distribution. In 3D models, the vertical velocities are computed from the continuity equation. The set of partial differential equations in combination with an appropriate set of initial and boundary conditions is solved on a finite difference staggered grid. In the horizontal direction, Delft3D-FLOW uses structured orthogonal direction; Delft3D-FLOW uses structured orthogonal curvilinear coordinates. Both Cartesian and spherical coordinates are supported. In the vertical direction, Delft3D-Flow offers two different vertical grid system, the σ coordinate system (σ - grid) and the Cartesian coordinate system (Z -grid). In the σ grid, the vertical grid consists of layers bounded by two sigma planes; these are not strictly horizontal but follow the bottom topography and free surface. Because the σ grid is boundary-fitted both to the bottom and to the moving free surface, a smooth representation of the topography is obtained.

Solution algorithm

The continuity and momentum equations for turbulent flow were solved to obtain the water velocity.

The depth – averaged continuity equation is given by:

$$\frac{\partial \zeta}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} + \frac{\partial[(d+\zeta)u\sqrt{G_{\eta\eta}}]}{\partial \xi} + \frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} + \frac{\partial[(d+\zeta)v\sqrt{G_{\xi\xi}}]}{\partial \eta} = Q \quad (1)$$

With Q representing the contributions per unit area due to the discharge or withdrawal of water precipitation and evaporation:

$$Q = H \int_{-1}^0 (q_{in} - q_{out}) d\sigma + P - E \quad (2)$$

With q_{in} and q_{out} the local sources and sinks of water per unit of volume [1/s], respectively, P the non- local sources term of precipitation and E non-local sink term due to evaporation.

The Momentum equations in horizontal direction are given by:

$$\begin{aligned} & \frac{\partial u}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}} \frac{\partial u}{\partial \xi} + \frac{v}{\sqrt{G_{\eta\eta}}} \frac{\partial u}{\partial \eta} + \frac{\omega}{d+\zeta} \frac{\partial u}{\partial \sigma} - \frac{v^2}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \frac{\partial \sqrt{G_{\eta\eta}}}{\partial \xi} + \frac{uv}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \frac{\partial \sqrt{G_{\xi\xi}}}{\partial \eta} - \mathcal{F}v = \\ & = - \frac{1}{\rho_0 \sqrt{G_{\xi\xi}}} P_\xi + F_\xi + \frac{1}{(d+\zeta)^2} \frac{\partial}{\partial \sigma} \left(v_V \frac{\partial v}{\partial \sigma} \right) + M_\xi \end{aligned} \quad (3)$$

and

$$\begin{aligned} & \frac{\partial v}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}} \frac{\partial v}{\partial \xi} + \frac{v}{\sqrt{G_{\eta\eta}}} \frac{\partial v}{\partial \eta} + \frac{\omega}{d+\zeta} \frac{\partial v}{\partial \sigma} - \frac{uv}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \frac{\partial \sqrt{G_{\eta\eta}}}{\partial \xi} + \frac{u^2}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \frac{\partial \sqrt{G_{\xi\xi}}}{\partial \eta} - \mathcal{F}u = \\ & = - \frac{1}{\rho_0 \sqrt{G_{\eta\eta}}} P_\eta + F_\eta + \frac{1}{(d+\zeta)^2} \frac{\partial}{\partial \sigma} \left(v_V \frac{\partial u}{\partial \sigma} \right) + M_\eta \end{aligned} \quad (4)$$

The vertical eddy viscosity coefficient ν_V . Density variations are neglected, except in the baroclinic pressure terms, P_ξ and P_η represent the pressure gradients. The forces F_ξ and F_η in the momentum equations represent the unbalance of horizontal Reynold's stresses. M_ξ and M_η represent the contributions due to external sources or sinks of momentum (external forces by hydraulic structures, discharge or withdrawal of water, wave stresses, etc). The vertical velocity ω in the adapting σ - co-ordinate system is computed from the continuity equation:

$$\frac{\partial \zeta}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \frac{\partial[(d+\zeta)u\sqrt{G_{\eta\eta}}]}{\partial \xi} + \frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \frac{\partial[(d+\zeta)v\sqrt{G_{\xi\xi}}]}{\partial \eta} + \frac{\partial \omega}{\partial \sigma} = H(q_{in} - q_{out}) \quad (5)$$

At the surface the effect of precipitation and evaporation is taken into account. The vertical velocity ω is defined at the iso σ - surface. ω is the vertical velocity relative to the moving σ - plane. It may be interpreted as the velocity associated with up- or downwelling motions. The "physical" vertical velocities \mathcal{W} in the Cartesian co-ordinate system are not involved in the model equations. Computation of the physical vertical velocities is only required for post-processing purposes. These velocities can be expressed in the horizontal velocities, water depth, water levels and vertical ω velocity according to:

$$\mathcal{W} = \omega \frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\eta\eta}}} \left[u \sqrt{G_{\eta\eta}} \left(\sigma \frac{\partial H}{\partial \xi} + \frac{\partial \zeta}{\partial \xi} \right) + v \sqrt{G_{\xi\xi}} \left(\sigma \frac{\partial H}{\partial \eta} + \frac{\partial \zeta}{\partial \eta} \right) \right] + \left(\sigma \frac{\partial H}{\partial t} + \frac{\partial \zeta}{\partial t} \right) \quad (6)$$

Model domain

The study comprised field investigations, data collection, and hydrographic survey using high technology instruments, such as DGPS, Complete Total Station, Echo-sounding, and Range Finder. The hydrographic survey covered 10.6 km length of the River within the vicinity of the Barrage. The hydrographic survey involved investigating cross-sectional details each 100 m. Also, status of the banks was monitored and registered. On the other hand, water velocities were measured at 8 different locations. The reliable data regarding to bathymetry and River characteristics, boundary conditions, roughness coefficients, and calibration data were collected to represent the reality sufficiently well. The numerical model was constructed for a length of approximately 10.6 km (4.6 km downstream and 6 km upstream of the old existing Barrage), Fig. 3. Boundary conditions that were used for the numerical model consist of a discharge at the upstream side of the model and a water level at the downstream side of the model, in addition to one boundary discharge at the intake of Ibrahimiya canal. Both the intake and outfall of the Walidia Power Plant were simulated in the model through point discharge.

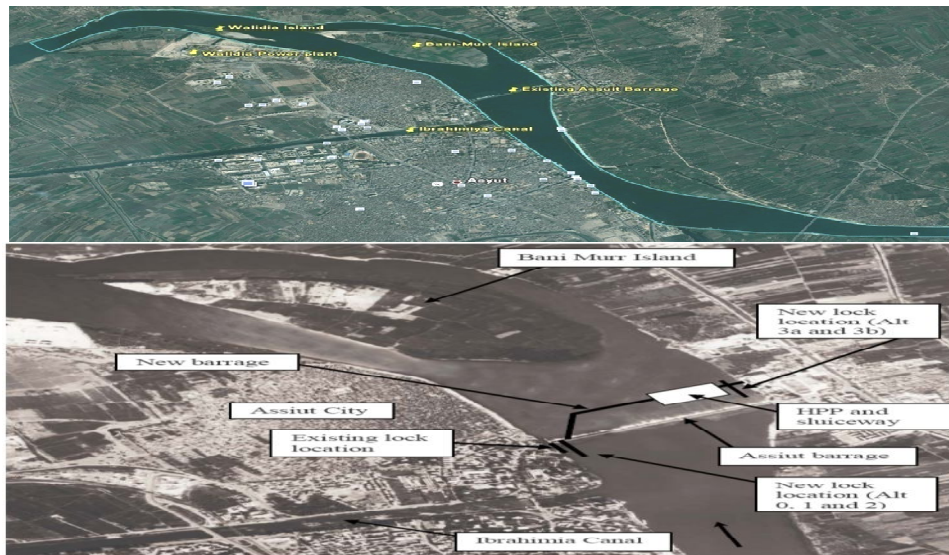


Fig. 3. Modeled Area (image source: Google Earth™)

Computational grid

The computational grid for the hydrodynamic model is curvilinear and covers the area in the vicinity of the Barrage. The model grid was generated using the software tool DELFT-RGFGRID and its general characteristics are described in Table 1.

Table 1. Model grid characteristics

Model dimensions	stream length 10 km
	distance in E-W direction = 8350 m
	distance in N-S direction = 5690 m
Grid size	$m \times n = 771 \times 147 = 113,337$ nodes
Cell length	min = 6 m
	max = 17 m
	average = 11 m
Cell width	min = 5 m
	max = 11 m
	average = 7 m

Modeling of the Barrage

The coordinates of the Barrage's vents were deduced from the field measurements. The size and locations of the modeled vents were restricted to the model grid, where each vent located in one grid cell and separated by thin-dam that represents the Barrage's piers. Porous plate option was applied to adequate the width of vents in the model with the actual Barrage's vents. The gates were introduced in the model through barrier option in order to have correct representation of the total blockage to the flow. Fig. 4 illustrates the actual location of the Barrage's vents.

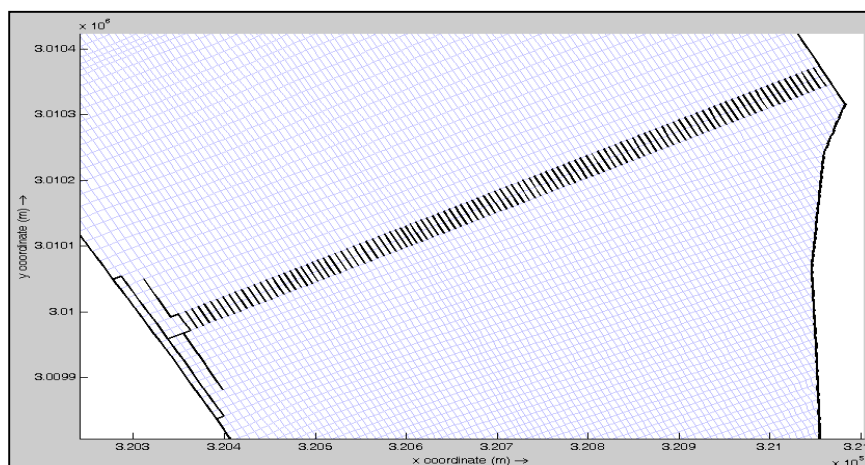


Fig. 4. Model grid with indication to the location of the Barrage

The barrier structure option in Delft3D-FLOW was employed to allow for the temporal variation of the gate opening. The model depth at the barrier point was decreased to the sill depth for appropriate modeling. Upstream of a barrier the flow is accelerated due to contraction, whilst the flow in the downstream is decelerated due to expansion. The expansion introduces a hydraulic jump between the upstream and downstream water levels, which is independent of the grid size. The energy loss for a barrier was taken into account by adding an extra quadratic friction term to the momentum equations. The quadratic friction was added to the momentum equations for all layers, which are open. The discharge relation presently available at barrier points in Delft3D-FLOW assumes subcritical free surface flow. The depth-averaged flow-rate through a barrier is given by:

$$Q = \mu A \sqrt{2g|\zeta_u - \zeta_d|} \quad (7)$$

With μ the barrier contraction coefficient ($0 < \mu \leq 1$). The contraction coefficient is used to determine the friction coefficient and depends on the direction of the barrier. In 2D, the energy loss coefficient perpendicular to the flow is specified according to:

$$C_{loss} = \frac{1}{2\mu^2} \quad (8)$$

One of the main features in modeling a barrage in a construction phase is the porous plate function that is used to account for partial blockage. Porous plate is a partially transparent structure that extends into the flow along one of the grid directions, that covers some or all layers in the vertical direction. The effect of this partial blockage was implemented through reducing the porosity. Delft3D-FLOW was extended with functionality that allows the user to partially close cell interfaces (by changing the porosity of the cell interfaces). This effect is included in the relation between the discharge q and the velocity U at the interface of two cells: $q = pAU$, where p is the porosity and A is the total interface area. The porosity p is interpreted here as the ratio of the opening when the gate is partially closed and the local water depth. The bottom line is that the computed velocity U represents the flow through the opening is significantly higher than the effective (depth-averaged) velocity at the interface, which equals $u = pU$.

Model calibration

In order to perceive a certain level of confidence in the model results, calibration was executed. The initial conditions can be specified as global values of water levels and discharges for the entire covered area of the River. The boundary conditions may be internal or external conditions. The internal boundary condition includes the specifications at nodal points and structures, whereas the external boundary condition includes the specification of constant values for h or Q or time varying values for Q or h at the starting point and endpoint. The daily discharges at the system source during the survey and the water level at the downstream end are specified to serve as boundary conditions. Before running the model simulation, control parameters such as simulation period, simulation time step, data to be stored, and storage time were specified. The Courant number is used for selecting the time step, which controls the simulation process.

The calibration was carried out using an upstream boundary condition of a total River discharge of $Q = 1800 \text{ m}^3/\text{s}$ and a downstream boundary condition of a water level of 45.98 m. Water velocities were measured at 8 different locations (Fig. 5) for the purpose of calibrating the model. The results of this simulation compared to the field measurements are given in Figs. 6a-6b. The calibration results revealed that the computed velocities coincide with those measured during the survey.

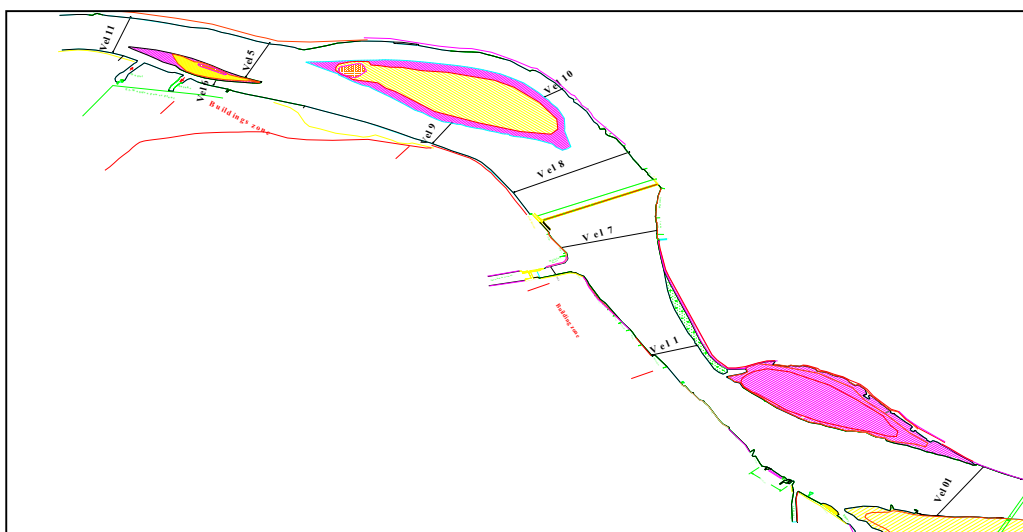


Fig. 5. Locations of velocity cross sections

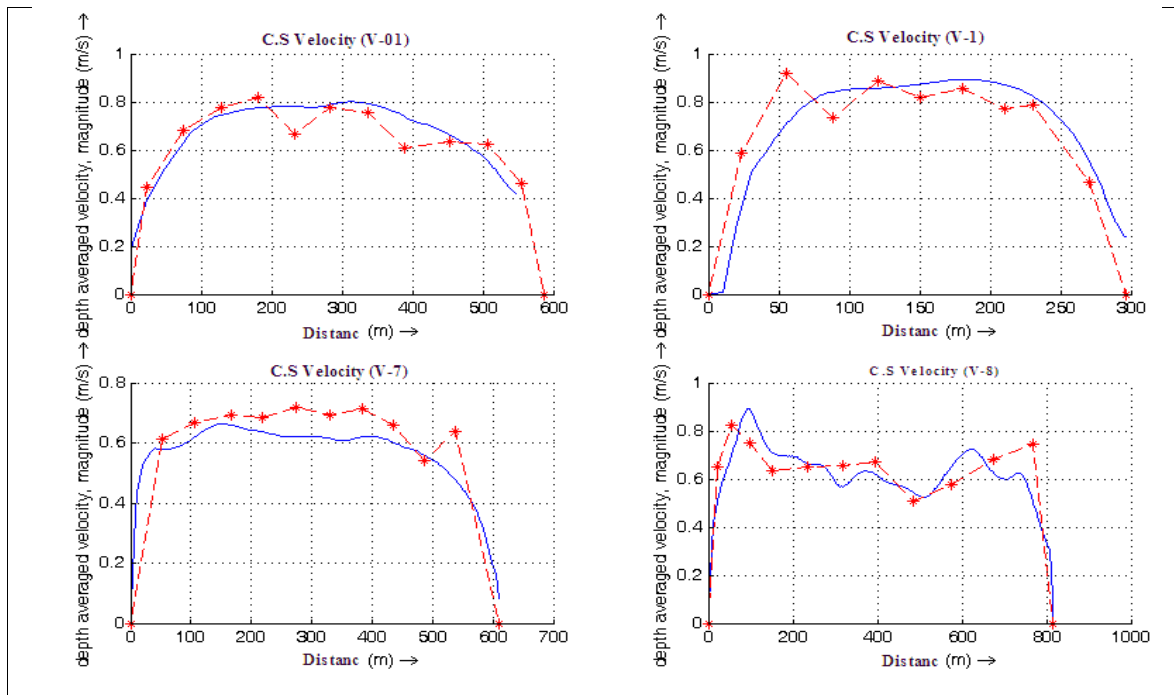


Fig. 6a. Calibration results at cross sections V-01, V-1, V-7, V-8

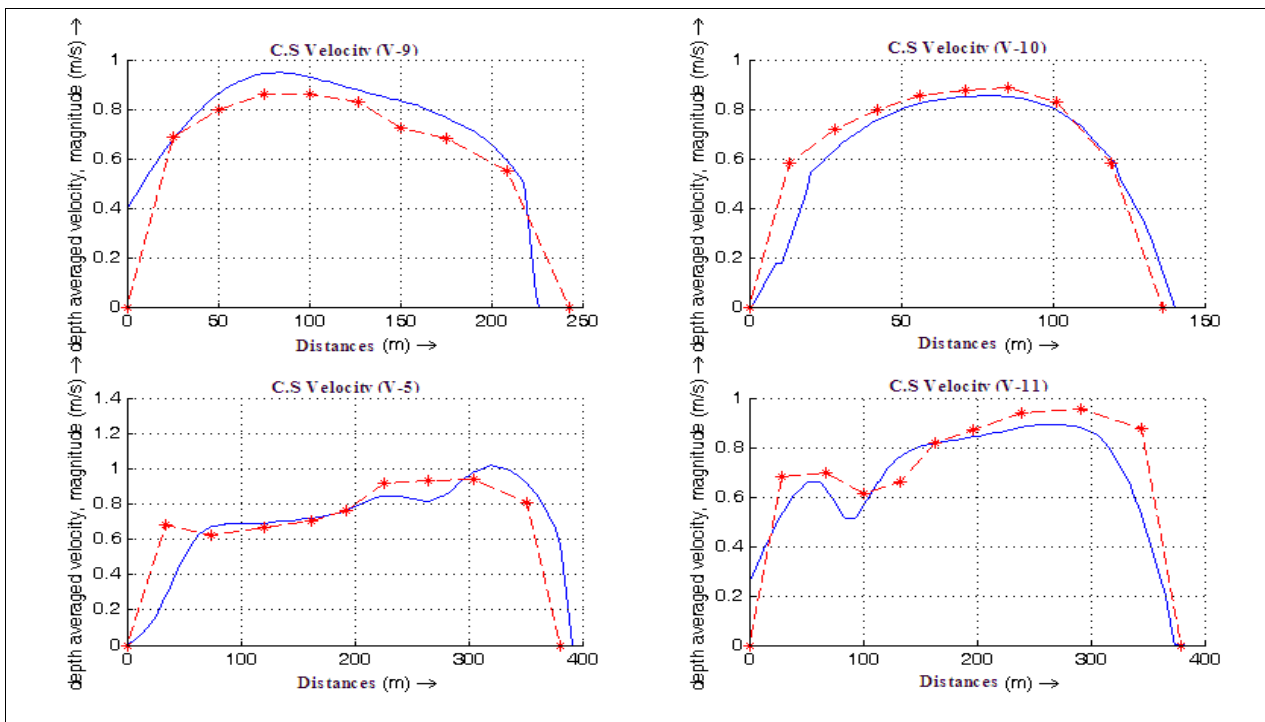


Fig. 6b. Calibration results at cross sections V-9, V-10, V-5, and V-11

3. RESULTS AND DISCUSSION

The new Barrage as well as the closure dam were inserted into the model at the exact locations proposed in the feasibility study. The model grid was designed to correctly simulate the Barrage piers in order to have a correct representation for the total blockage of the flow. Fig. 7 demonstrates the actual location of the Barrage and its components.

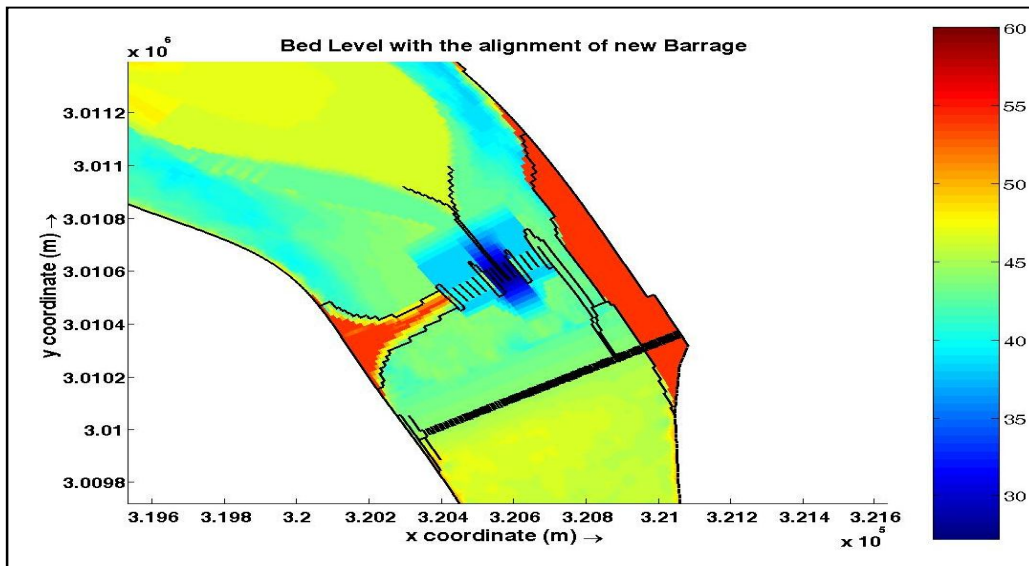


Fig. 7. Actual location of the new Barrage and its components in the model

The downstream water levels were calculated for each River discharge according to the downstream reach hydrograph shown in Fig. 8 (6). Discharges that passed through the powerhouse opening were set at equal values. Also, it was considered to have the same River flow distribution for the two branches of Bani-Murr Island as observed during field measurements. This implied a flow division of 40% and 60% through the right and left branches respectively.

With a River discharge of 1000 m³/s, three tests were carried out. The first test represented the case of no new Barrage. The second test entailed a 100% discharge flowing through the spillway of the new Barrage with a flow distribution of 40% and 60% passing through the eastern and western sides of Banni-Murr Island respectively. The second operating mode of the new Barrage involved a 100% discharge passing through the powerhouse accompanied with equal flow distribution through the eastern and western sides of Banni-Murr Island. The results of these simulations showed that with regards to the baseline test (case of no new Barrage) the flow distribution was found to be 43% at the eastern side and 57% at the western side of Bani-Murr Island, whilst for Walidia Island it was found to be 90% and 10% at the eastern and the western branches respectively. On the other hand, the first operating mode case of the new Barrage showed a flow distribution of 92% and 8% at the eastern and western sides of Walidia Island respectively. Whereas, for the second operating mode case the flow distribution was found to be 91.4% at the eastern branch and 8.6% at the western branch of Walidia Island. Results of the three simulations are presented in Figs. 9a-9c. In addition, velocity distributions for the three cases are illustrated in Figs. 10a-10b.

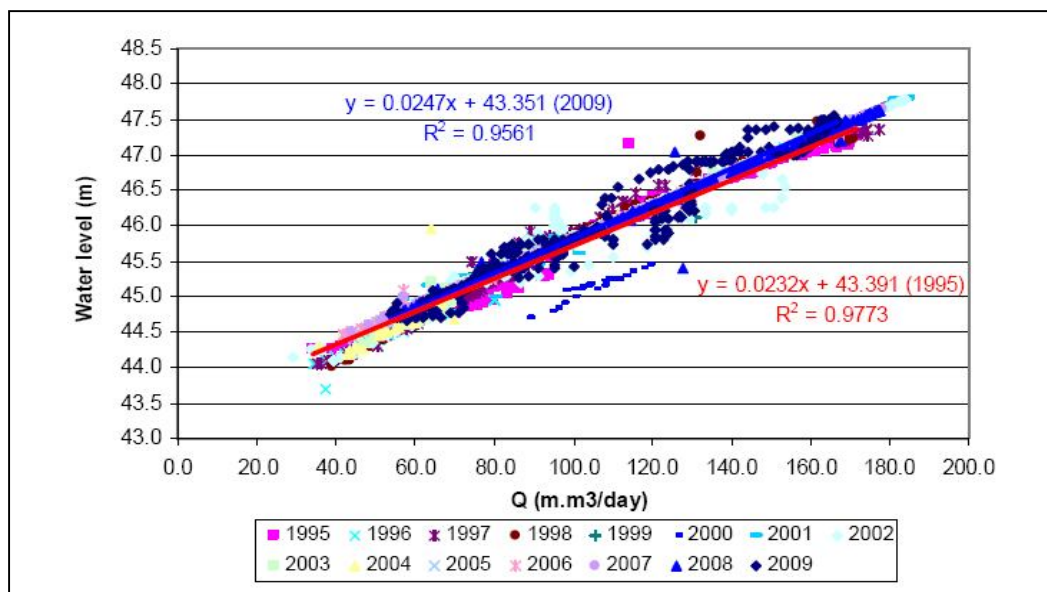


Fig. 8. Downstream Assuit Barrage rating curve

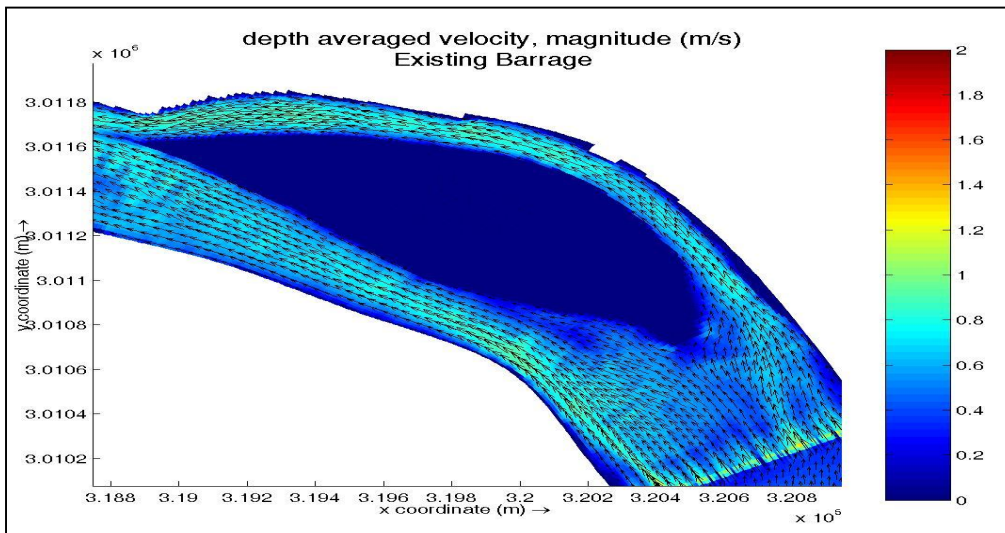


Fig. 9a. Flow distribution at both sides of Bani-Murr Island, case of old Barrage only

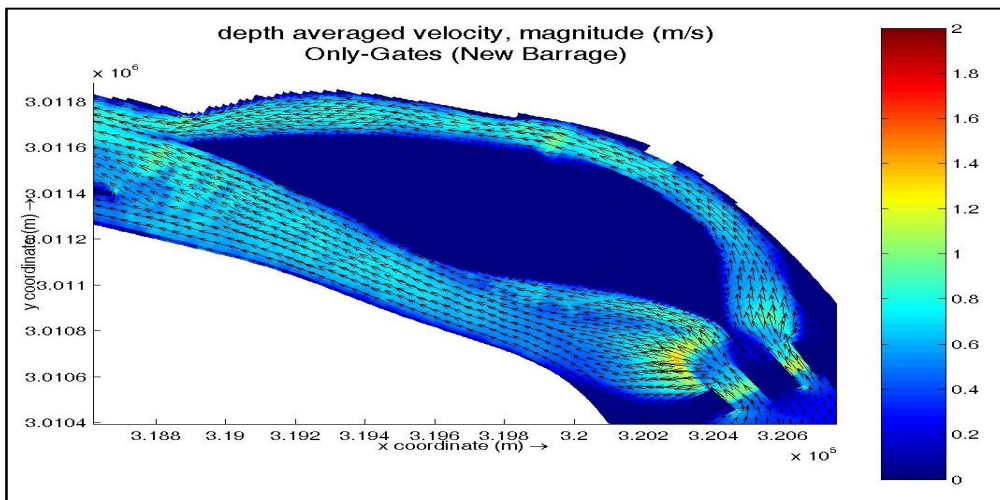


Fig. 9b. Flow distribution at both sides of Bani-Murr Island, case of 100% flow through spillway

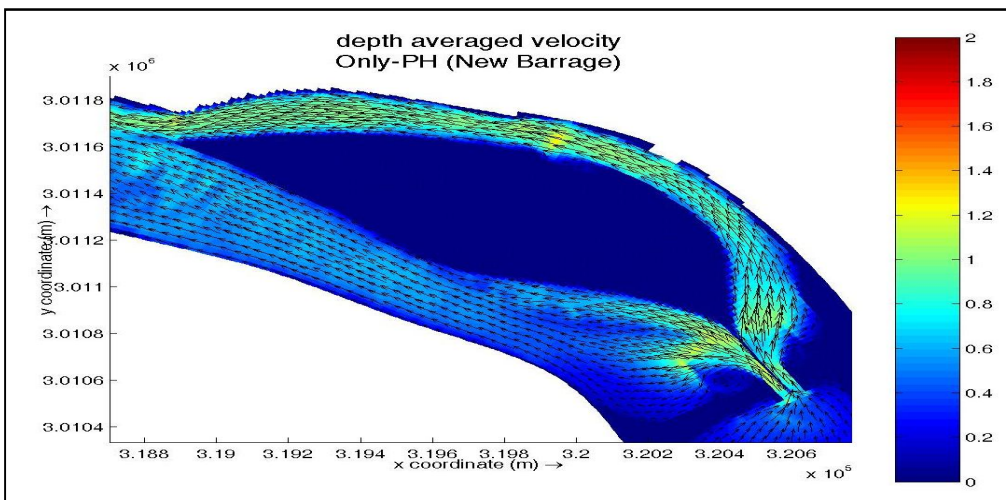


Fig. 9c. Flow distribution at both sides of Bani-Murr Island, case of 100% flow through powerhouse

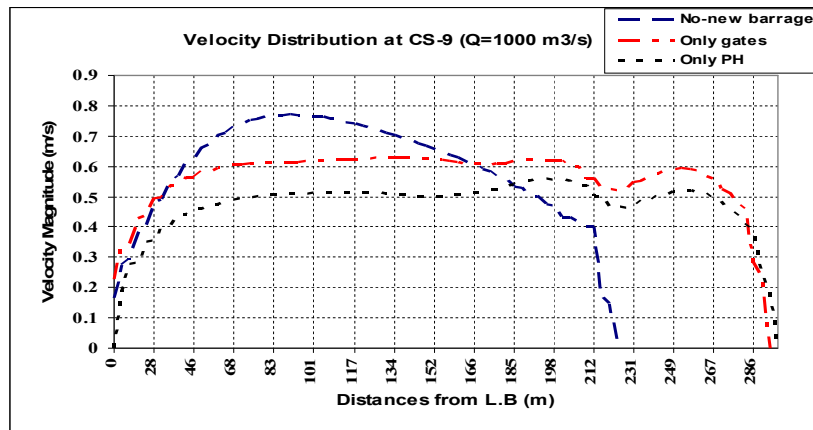


Fig. 10a. Velocity distribution at the left side of Bani-Murr Island

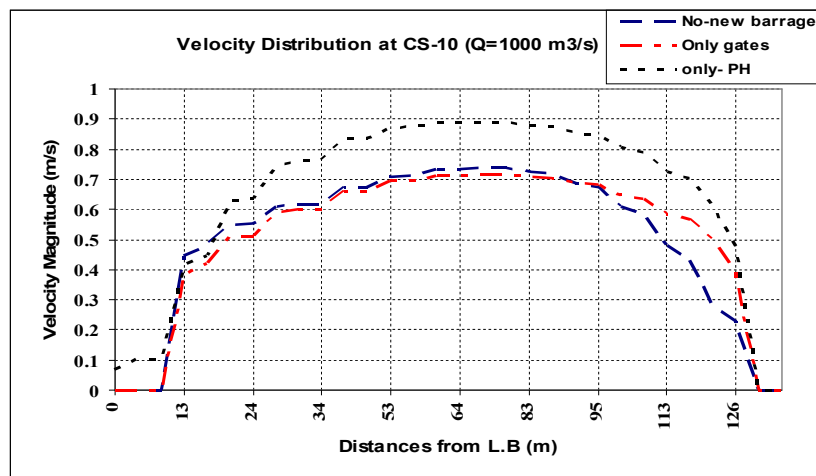


Fig. 10b. Velocity distribution at the right side of Bani-Murr Island

With a River discharge of 1600 m³/s, three tests were carried out. The first test corresponded to the case of no new Barrage. The second test entailed a 50% discharge flowing through the spillway of the new Barrage, while the remaining 50% was released via the powerhouse. The second operating mode of the new Barrage involved a 100% discharge passing through the spillway. In both operating modes the flow distribution was set at 40% and 60% passing through the eastern and western sides of Banni-Murr Island respectively. The results of these runs revealed that with regards to the baseline test (case of no new Barrage) the flow distribution was found to be 37.7% at the eastern side and 62.37% at the western side of Bani-Murr Island, whilst for Walidia Island it was found to be 88.7% and 11.3% at the eastern and the western branches respectively. On the other hand, the first operating mode case of the new barrage showed a flow distribution of 89% and 11% at the eastern and western sides of Walidia Island respectively. Whereas, for the second operating mode case the flow distribution was found to be 89% at the eastern branch and 11% at the western branch of Walidia Island. Results of the three simulations are presented in Figs, 11a-11c. In addition, velocity distributions for the three cases are illustrated in Figs. 12a-12b.

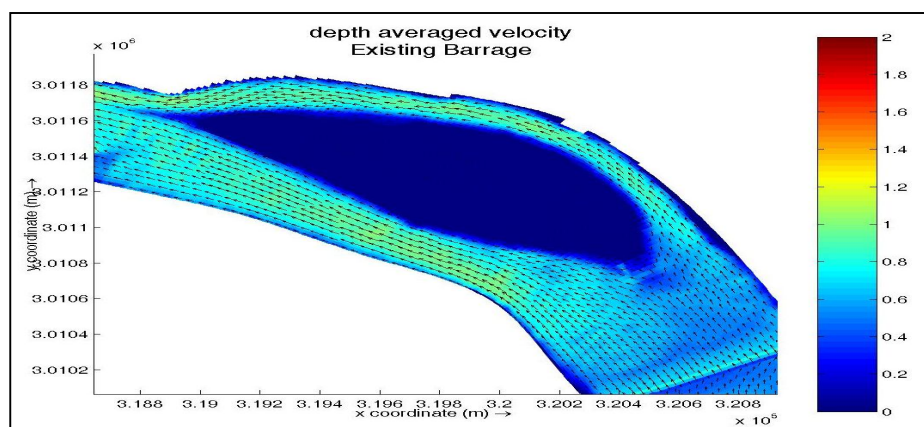


Fig. 11a. Flow distribution at both sides of Bani-Murr Island, case of old Barrage only

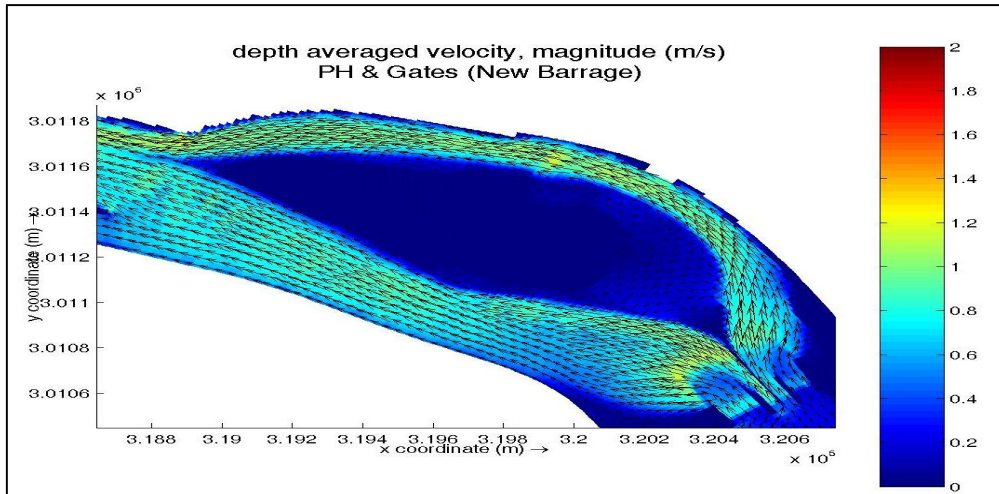


Fig. 11b. Flow distribution at both sides of Bani-Murr Island, case of new Barrage with 50% flow passing through spillway and 50% via powerhouse

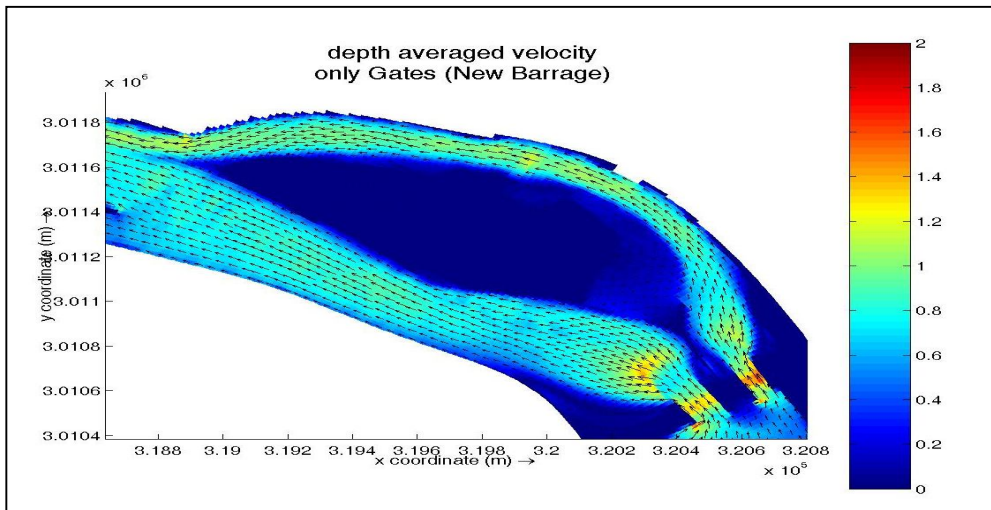


Fig. 11c. Flow distribution at both sides of Bani-Murr Island, case of new Barrage with 100% flow passing through spillway

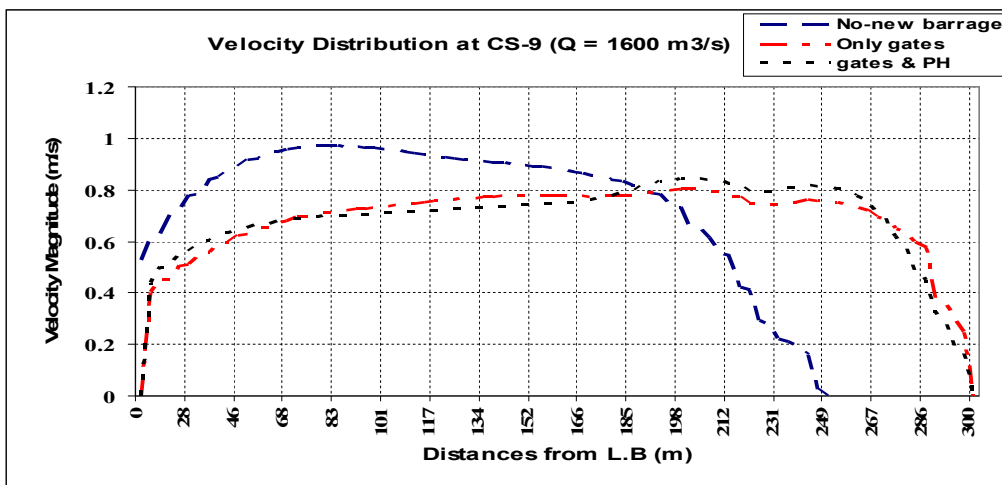


Fig. 12a. Velocity distribution at the left side of Bani-Murr Island

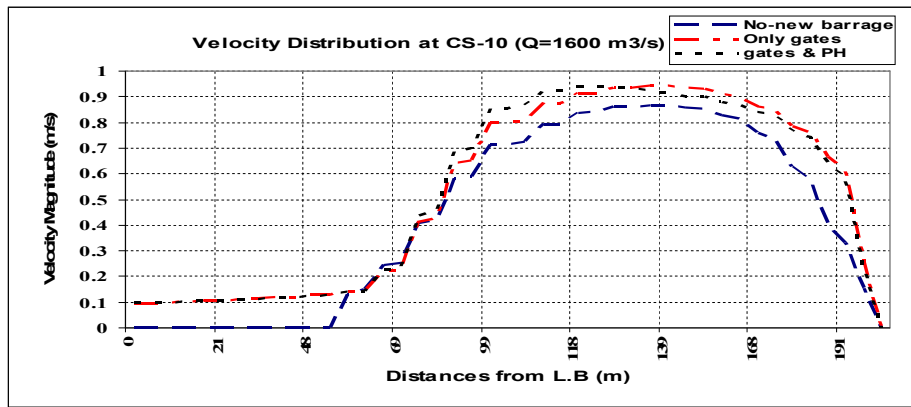


Fig. 12b. Velocity distribution at the right side of Bani-Murr Island

Model modification

The model was modified just downstream the new Barrage through some dredging works to reach a bed level of 43 m in a certain area on Bani-Murr Island. The guide wall that separates the eastern and western sides of Bani-Murr Island was removed. Fig. 13 shows the model setup after being modified.

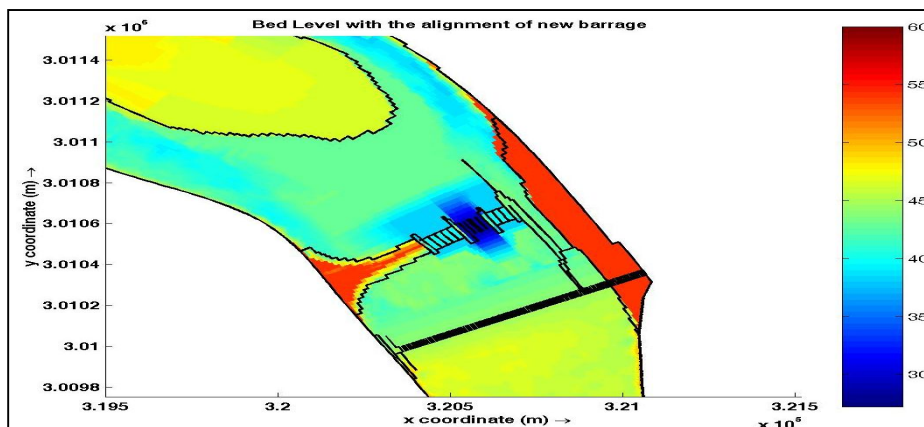


Fig. 13. Actual location of the new Barrage and its components after model modification

The simulation for this case was carried out by using River discharge of 1000 m³/s with 100% discharge flowing via the powerhouse. The model results showed that the flow distribution was 40% at the eastern side and 60% at the western side of Bani-Murr Island, whilst for Walidia Island it was found to be 91% and 9% at the eastern and the western branches respectively. Moreover, no difference of the water level downstream of the new Barrage was realized (Fig. 14). In addition, velocity distributions for this case are presented in Figs. 15a-15b.

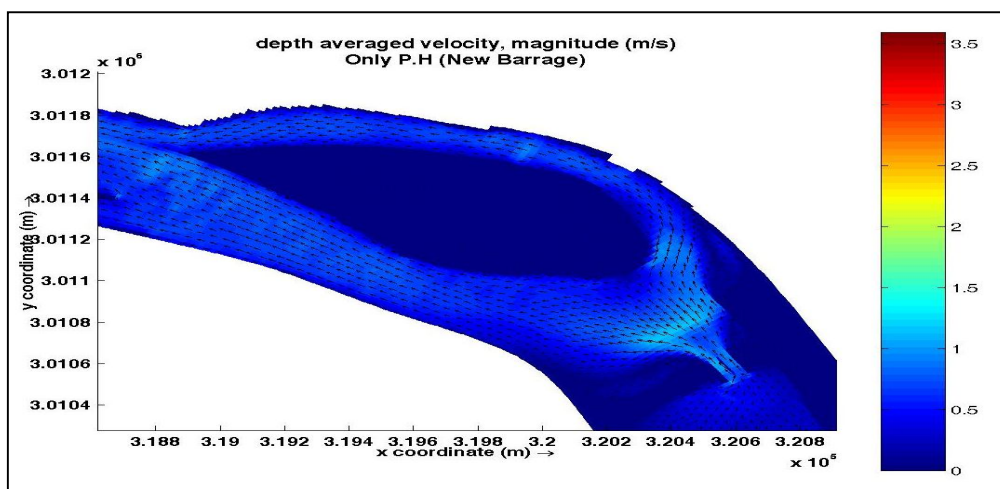


Fig. 14. Flow distribution at both sides of Bani-Murr Island, case of P.H only

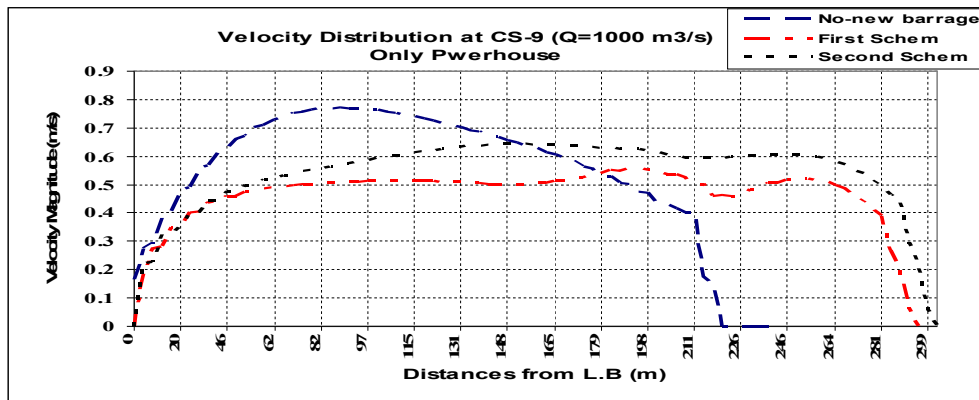


Fig. 15a. Velocity distribution at the left side of Bani-Murr Island

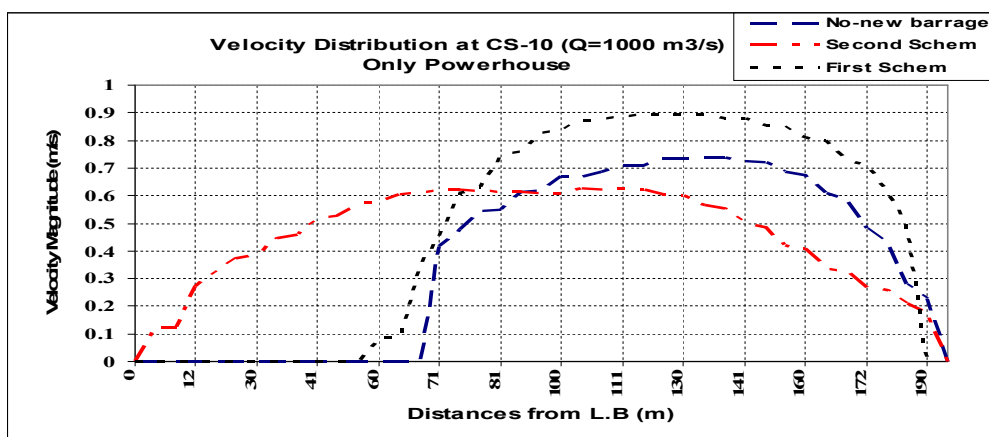


Fig. 15b. Velocity distribution at the right side of Bani-Murr Island

4. CONCLUSIONS

This study presents an example for the use of numerical models in investigating the effect of constructing a new barrage on a river regime. The model has been calibrated by comparing model predictions with observed velocity measurements at different cross sections. The study examined the model results in order to determine whether the numerical model is able to predict velocity distributions and flow characteristics in the study reach. According to the results obtained by this research, the following points are concluded:

- The flow approaches the new Barrage with inclined angle, especially at the west gates and powerhouse, which will affect the hydraulic efficiency of the spillway and the powerhouse as well.
- The best flow distribution between the left and right branches of Bani-Murr Island is 60% and 40% respectively, which is to be considered in preparing the operational scheme of the new Barrage.
- Modifications are needed for the upstream nose of Bani-Murr Island due to high flow velocity. The velocity in that area exceeds 1.2 m/s, which could cause bed erosion and bank instability.
- Passing 50% of the total discharge (1000 m³/s) through the right branch causes very high velocity. Moreover, water level difference between the two branches exceeds 10 cm downstream the new Barrage, and hence will affect the efficiency of the powerhouse and lead to high bed erosion and bank failure.
- The velocity comparison between base line tests and the Barrage's different operating schemes indicates that the proposed alternatives has almost no effect on the velocity magnitudes or flow distribution at the downstream reach, except for the case of passing 1000 m³/s through the powerhouse with equal flow distribution between Bani-Murr Island branches.
- The modification proposed via removing the guide wall of Bani-Murr Island gives very good results with regards to the velocities magnitude and the flow distribution between Bani-Murr Island branches.

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