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Investigating Flow Patterns in Rantambe Reservoir through Two-Dimensional Hydrodynamic Modelling

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A R T I C L E I N F O

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A B S T R A C T

The reservoir storage capacity is being depleted due to sedimentation, making sediment removal vital for maintaining the required capacity. Sediment deposition is a common issue in the Rantambe reservoir in Sri Lanka, and flushing is the most effective technique to remove sediments. Therefore, the objective of this research is to study the two-dimensional (2D) flow patterns in the Rantambe reservoir under different combinations of inflows and releases through power intake. The International River Interface Cooperative (iRIC), 2D flow computational model was used to develop the 2-D models for the Rantambe reservoir to generate flow patterns. The model was calibrated and validated against the field measurements carried out at selected sections of the reservoir. The results show higher accuracy; for calibration, Percentage Bias (PBIAS) is 5.2% and Mean Relative Absolute Error (MRAE) is 0.18 whereas, for validation, PBIAS and MRAE are 11.3% and 0.22, respectively. The model predictions of velocity and depth show satisfactory agreement with field observations. Therefore, the developed iRIC model of the Rantambe reservoir can be used to obtain sufficiently accurate flow patterns, velocity and water surface elevation for the effective flushing, and maintenance of the Rantambe reservoir.

1. INTRODUCTION

Rantambe Reservoir constructed across the Mahaweli River had a capacity of 11.2 MCM in 1990, however, currently, it is reduced badly, due to sediment accumulation (Ratnayesuraj *et al.* 2015). The reservoir is fed by the release from the Randenigala reservoir and the Uma Oya. To provide the water to the hydropower station, a spillway and sediment flushing sluices are provided at the Intake of the hydropower station. Sedimentations are deposited on the riverbed during the high inflows to the reservoir. Many techniques, namely flushing, sluicing, dredging, and water & soil conservation in the catchment are used to reduce reservoir sedimentation and remove sediment. Among these techniques, flushing is considered an economical approach to rapidly restore the storage capacity of the reservoir (Ratnayesuraj *et al.* 2015).

Natural reservoirs typically have highly complex flow patterns (Moussa, 2012) Hence, there is a necessity to develop numerical models to study the flow, flow velocity, flow patterns, sediment movement and water surface elevation of Rantambe reservoir to carry out flushing effectively. Even though a physical model of the Rantambe reservoir was developed in the past to study the flushing (Ratnayesuraj *et al.* 2015), no numerical models were developed, which is a sophisticated method to study the flow patterns and sediment movement accurately by incorporating the actual river bed bathymetry.

One-dimensional (1D) modelling was extensively used in the past due to the computation simplicity and lesser computational time even though it has many shortcomings, namely the incapability to represent the whole topography of river channels and floodplains and the incapability to model lateral flow (Horritt & Bates, 2001; Pinos & Timbe, 2019;). These shortcomings of 1D models can be overcome by applying two-dimensional (2D) modelling and they are capable of modelling lateral flow movement explicitly with the continuous representation of topography (Pinos & Timbe, 2019; Kourgialas & Karatzas, 2013). However, the main drawback of 2D models when compared with the 1D model is the larger computational time required for the simulation (Bates & De Roo, 2000; Suja & Rajapakse, 2020). Therefore, the objective of this research is to develop 2D models to study the 2D flow patterns in the Rantambe reservoir under different combinations of inflows and release through power intake.

2. MATERIALS AND METHOD

2.1. Study Area

The Rantambe reservoir is located downstream of Randenigala reservoir, central province of Sri Lanka, and its storage is affected by sedimentation. The bathymetry data obtained from the Mahaweli Authority of Sri Lanka was used to construct the reservoir bed, which is depicted in Figure 1. Moreover, the colour palate shows the bed elevation variation with respect to mean sea level, and the deepest area of Rantambe reservoir was identified near the spillway which is 137 m. One assumption was made that the bed elevation was not changed. However, in real situations, the bed is frequently changing due to sedimentation and erosion. The best possible way to use this model accurately is to modify the bed elevation from time to time by using newly updated bathymetry data.

2.2. Hydraulic Modelling

2.2.1. 2D Model

There are several commercial and public domain software packages available to develop 2D models, and each software package is developed based on a variety of numerical schemes and offers a range of graphical pre and post-processor modules. The River2D solver developed by the International River Interface Cooperative (iRIC) (Hokkaido University, Japan) was used in this study. River2D is a computational model for simulating horizontal 2D flow, and it is intended for use on natural streams and rivers and has special features for accommodating supercritical or subcritical flow transitions and variable wetted areas. River2D solver is a public-domain software package that solves shallow water computations using the finite difference scheme (Nelson *et al.* 2016). It is also a 2D depth-averaged hydrodynamic model. Velocity distributions in the vertical are assumed to be uniform and pressure distributions are assumed to be hydrostatic.

2.2.2. Governing equations

The flow computation module is based on the Saint-Venant Equations of channel flow which is given in equations 1 and 2 (Rutschman & Hager, 1996). In describing the resistance to flow in an unsteady, non-uniform flow model, the vertical acceleration of water was assumed to be negligible compared with the gravitational acceleration and that yields hydrostatic pressure distribution.

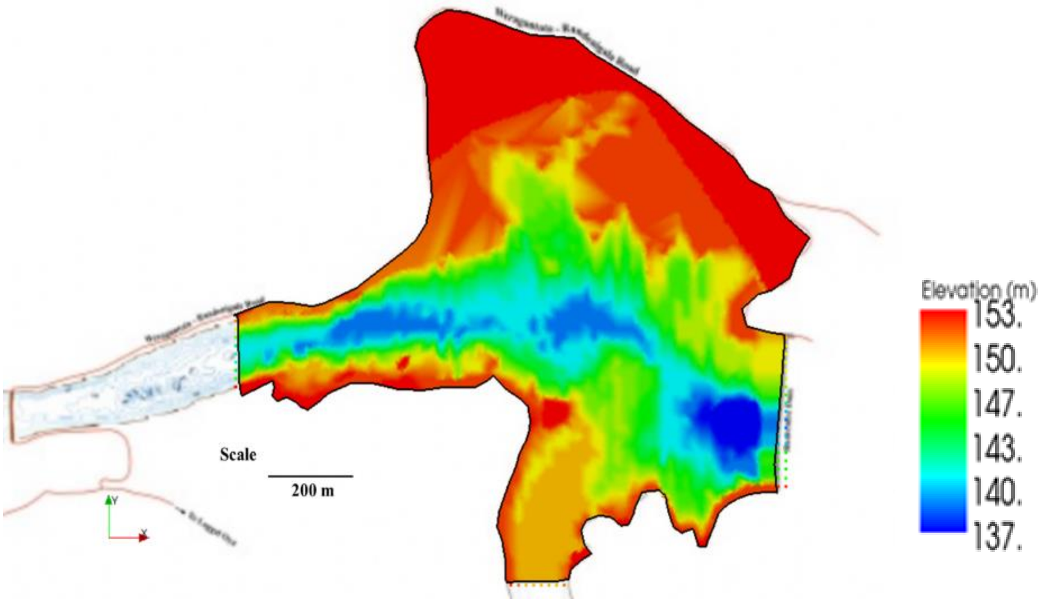

Figure 1: Reservoir Bathymetry

St- Venant equation:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial \varkappa} + g\frac{\partial h}{\partial x} + g(S - S_f) = 0 \qquad (1)$$

$$\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial y} + g\frac{\partial h}{\partial y} + g(S - S_f) = 0 \qquad (2)$$

where

u is the velocity in the x direction, or zonal velocity; v is the velocity in the y direction or meridional velocity; g is the acceleration due to gravity; h is the height deviation of the horizontal pressure surface from its mean height H; Sf and S are friction slope and slope of the channel respectively.

The St- Venant equations are derived from the depth-integrating Navier–Stokes equation, in the case where the horizontal length scale is much greater than the vertical length scale. Under this condition, conservation of mass implies that the vertical velocity of the fluid is small. It can be shown from the momentum equations that vertical pressure gradients are nearly hydrostatic, and that horizontal pressure gradients are due to the displacement of the pressure surface, implying that the horizontal velocity field is constant throughout the depth of the fluid. The set of continuity and momentum equations of 2D unsteady flow in the Cartesian coordinate system are given in equations (3-5);

Continuity equation:

$$\frac{\partial \mathrm{h}}{\partial \mathrm{t}} + \frac{\partial(\mathrm{hu})}{\partial \mathrm{x}} + \frac{\partial(\mathrm{hv})}{\partial \mathrm{y}} = 0 \qquad (3)$$

Momentum equations:

$$\frac{\partial hu}{\partial t} + \frac{\partial(hu^2)}{\partial x} + \frac{\partial(huv)}{\partial y} = -gh\frac{\partial H}{\partial x} - \frac{\tau_{bx}}{\rho} + + \frac{\partial}{\partial x}\left(V\frac{\partial(hu)}{\partial x}\right) + + \frac{\partial}{\partial y}\left(V\frac{\partial(hu)}{\partial y}\right) \qquad (4)$$

$$\frac{\partial hv}{\partial t} + \frac{\partial(huv)}{\partial x} + \frac{\partial(hv^2)}{\partial y} = -gh\frac{\partial H}{\partial y} - \frac{\tau_{by}}{\rho} + \frac{\partial}{\partial x}\left(V\frac{\partial(hv)}{\partial x}\right) + \frac{\partial}{\partial y}\left(V\frac{\partial(hv)}{\partial y}\right) \qquad (5)$$

where h = water depth; u, v = depth-averaged velocity components; τ_{bx} = riverbed shear stress in the x-direction; τ_{by} = riverbed shear stress in the y direction; ρ = the water density H = stage height ($H = h + z_b$); z_b= bed elevation; V = eddy viscosity; t = time; and x, y = spatial coordinates in the Cartesian system. Bed shear stress components are given in equations (6-8);

$$\tau_{bx} = \rho \mathrm{C_f u}\sqrt{\mathrm{u}^2 + \mathrm{v}^2} \qquad (6)$$

$$\tau_{by} = \rho \mathrm{C_f v}\sqrt{\mathrm{u}^2 + \mathrm{v}^2} \qquad (7)$$

$$\mathrm{v} = \frac{k}{6}\mathrm{uh} \qquad (8)$$

where $\mathrm{C_f}$ = riverbed friction coefficient; k = Karman constant; and u = shear velocity.

The high-order Godunov scheme known as the Cubic Interpolation Psuedoparticle (CIP) method was used for the application of the equations of water flow (Jang & Shimizu, 2005; Wongsa, 2014). River2D uses a non-dimensional Chezy coefficient to close the stress terms. The Chezy coefficient (Cs) is related to the effective roughness height, K_s is given in equation 9.

$$C_s = 5.75\, log\left[12 + \left({}^{H}/_{K_s}\right)\right] \qquad (9)$$

where C_s is the Chezy coefficient; H is the effective roughness height; K_s is the grain roughness scale. Where flow resistance is due primarily to bed material roughness, a good starting point for K_s is 1-3 times the largest grain diameter. Final values of roughness can be calibrated with measured water surface elevations.

2.3. Grid generation

The triangular grid was generated as depicted in Figure 2 for the model and the elevation was mapped to the grid using a rectangular regular network of the provided bathymetry. Finer grids were used near the wall to improve the accuracy of the results.

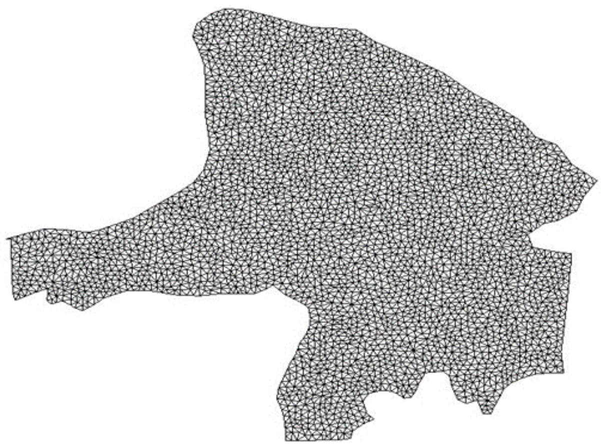

Figure 2: Grid of computational model

2.4. Model calibration and validation

Field measurements, namely water surface elevation, depth and velocity were taken at some selected sections in the Rantembe reservoir which is depicted in Figure 3 to calibrate and validate the model results. Global positioning system (GPS) coordinates of the respected location were obtained directly from the GPS receiver and are tabulated in Table 1. Water surface elevation downstream was 149.2 m during the field visit, which was directly read from the scale installed in the dam. Uma Oya discharge was calculated based on the velocity area method.

Figure 3: Locations used for model calibration and validation

Table 1: Coordinates of Locations used for model calibration and validation

Points	N	E
01	7°12'14.38"	80°56'40.35"
02	7°12'5.32"	80°56'38.56"
08	7°12'9.34"	80°56'47.43"
09	7°12'3.37"	80°56'45.64"
10	7°11'59.53"	80°56'54.67"
11	7°12'3.31"	80°56'53.77"
12	7°11'58.38"	80°56'45.64"

An appropriate cross section was selected for the field measurement & velocities were measured with respect to the velocity area method. The narrowest cross-section of Uma Oya was selected to obtain sufficient depth for the velocity measurements by using the propeller-type current meter. There were no flow diversions along the cross-section used for the field measurements. A long rope was used to align the cross-section. Field measurement of depth and velocity at a few selected sections in the reservoir was carried out from a boat. Flow pattern was computed using upstream discharges and downstream water levels.

The goodness of fit between simulated and measured velocity was numerically analysed using objective functions, namely Percentage Bias (PBIAS) (Gupta *et al.* 1999; WMO, 1975) and Mean Relative Absolute Error (MRAE) (WMO, 1975) and those are given in equations 10 and 11;

$$PBIAS = \frac{\sum_{i=1}^{n}(O_i - S_i)\times 100}{\sum_{i=1}^{n} O_i} \qquad (10)$$

$$MRAE = \frac{1}{n}\sum_{i=1}^{n}\frac{|S_i - O_i|}{O_i} \qquad (11)$$

where O_i and S_i are the observed /measured and simulated velocity in i^{th} hour; $\bar{O}$ is the mean of measured velocity; n is the total number of hours. The higher rating of model performance is attained when values of PBIAS and MRAE approach 'zero'.

3. RESULTS AND DISCUSSION

3.1. Results of Model Calibration

Velocities for known discharges were used to calibrate and validate the model. The flow models were calibrated by varying the flow resistance or roughness of the bed, which essentially controls the slope of the water-surface elevation until the simulated velocities resulted in the best match to the measured values. Altogether, eighteen locations along a chainage as shown in Figure 3 were selected to measure the velocity to calibrate the model. The following boundary conditions were used in the model when the model was used for calibration and validation.

Boundary conditions:

Randenigala discharge = 80 m^3/s
Uma Oya discharge = 1.9 m^3/s
Water surface elevation = 149.2 m

River2D uses a non-dimensional Chezy coefficient to close the stress terms, and Flow resistance is due to bed material roughness. Chezy coefficient (Cs) is related to the effective roughness height Ks [Eq 09].

It is recommended to use a value as a good starting point for k is 1-3 times the largest grain diameter. The largest grain diameter of the reservoir is 0.002 m (Ratnayesuraj *et al.* 2015). Final values of roughness were calibrated to the measured value of parameters until the simulated velocities resulted in the best match to the measured values. The measured and modelled velocity variation at calibration is depicted in Figure 4.

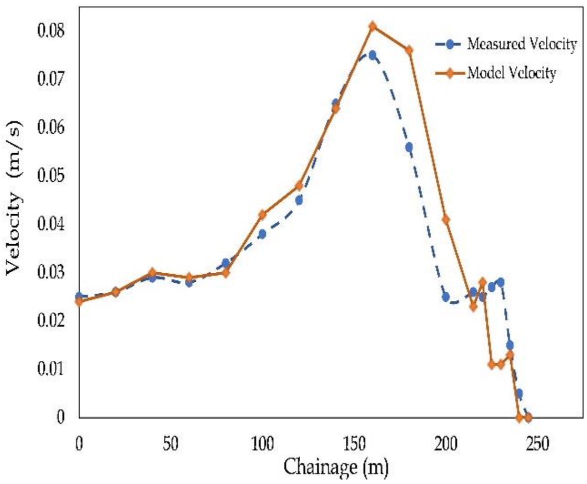

Figure 4: Velocity variation at Calibration

Model performance was assessed at the calibration in terms of two objective functions and those values are tabulated in Table 2. The values of the objective function show good agreement between simulated and measured values as they are within the accepted ranges specified in the literature (Moriasi *et al.* 2007; Waseem *et al.* 2017). Therefore, the Final value of Ks used in this model is 0.0325.

Table 2: Values of Objective Functions at Calibration

Objective Functions	Values at Calibration	Accepted Criteria
PBIAS	5.2%	<25%
MRAE	0.18	<0.25

3.2. Validation of the model

The model which was run with the model parameter used in calibration was validated using the velocity measurements taken at five locations denoted as 8-12 in Figure 3. The measured and modelled velocity variation at validations is depicted in Figure 5.

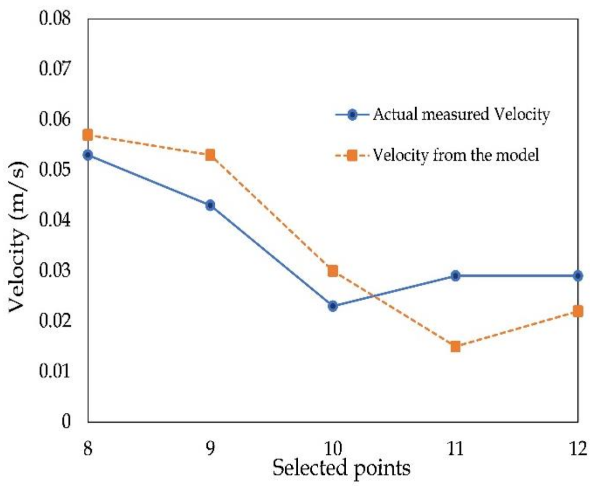

Figure 5: Velocity variation at Validation

Model performance was evaluated at the validation in terms of two objective functions and those values are tabulated in Table 3. They show good agreement between simulated and measured values as they are within the accepted ranges (Moriasi *et al.* 2007; Waseem *et al.* 2017). Therefore, the developed model can be used to study the flow, water surface and velocity variation for effective flushing.

Table 3: Values of Objective Functions at Validation

Objective Functions	Values at Validation	Accepted Criteria
PBIAS	11.3%	<25%
MRAE	0.22	<0.25

3.3. Velocity variation at Rantambe reservoir

The velocity variation at the Rantambe reservoir is depicted in Figure 6. Using the colour palate, the velocities of each node of the reservoir can be identified. Three types of regions can be seen in Figure 6: high velocity region; moderate velocity region and low velocity region. A maximum velocity of 0.95 m/sec was observed near the power intake in the developed model using the River2D solver of the iRIC model. A minimum velocity of 0 m/sec and an averaged moderate velocity of 0.15 m/sec were also observed in the developed model. Figure 7 shows the water velocity vector variation at Rantambe reservoir and the velocity magnitude is directly proportional to the length of the arrow on a particular place. The flow pattern varies from place to place at the reservoir.

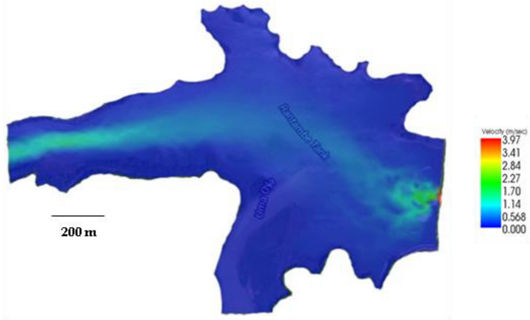

Figure 6: Velocity variation inside the Rantambe

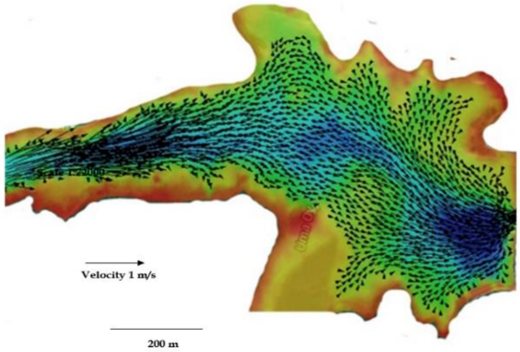

Figure 7: Velocity vector variation

4. CONCLUSIONS

The depth-averaged computational iRIC model of the Rantambe reservoir was developed to investigate the two-dimensional flow patterns in the Rantambe reservoir. The model was calibrated and validated against the field measurements of the velocity at a few selected sections of the reservoir. The developed model shows higher accuracy at calibration and validation. The iRIC model was shown to be stable and accurate in the application for depth-averaged flow computations over irregular bed topography as found in natural reservoirs. The depth-averaged iRIC model of the Rantambe reservoir is capable of providing the flow pattern, flow velocities and water surface elevation satisfactorily. The developed model is a useful tool for the operation plans and maintenance of the Rantambe reservoir, especially to estimate discharges and water surface elevations to create velocity patterns conducive to the flushing of the reservoir.

5. LIMITATIONS

The calibration and validation were carried out using the same boundary conditions but in different locations due to the unavailability of data to validate the results with a different set of data and boundary conditions. Therefore, it is highly recommended to carry out validation with a new set of data and boundary conditions to enhance the robustness of model validation.

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7. REFERENCES

Gupta, H. V., Sorooshian, S., Yapo, P. O. 1999, Status of automatic calibration for hydrologic models: comparison with multilevel expert calibration. *Journal of Hydrological Engineering*, 4(2), 135–143.

Jang C L & Shimizu Y 2005, Numerical simulation of relatively wide, shallow channels with erodible banks. *Journal of Hydraulic Engineering*, 131(7) 565–575.

Kourgialas, N. N., & Karatzas, G. P. 2013, A hydro-economic modelling framework for flood damage estimation and the role of riparian vegetation. *Hydrological Processes*, 27(4), 515–531.

Moriasi, D. N, Arnold, J. G., Lewis, M. W. V., Bingner, R. L., Harmel, R. D., & Veith, T. L. 2007, Model Evaluation Guidelines for Systematic Quantification of Accuracy in Watershed Simulations. *American Society of Agricultural and Biological Engineers*, 50(3), 885-900.

Moussa, M 2012, 'Predicting the deposition in the Aswan High Dam Reservoir using a 2-D (CCHE-2D)', Publication of Nile Research Institute (NRI), *National Water Research Center (NWRC)*, Egypt, pp.468-493.

Nelson et al. 2016, The international river interface cooperative: Public domain flow and morpho dynamics software for education and applications, *Advances in Water Resources* 93 62-74

Pinos, J. & Timbe, L 2019, Performance assessment of two-dimensional hydraulic models for generation of flood inundation maps in mountain river basins, *Water Science Engineering* 12(1) 11-18.

Ratnayesuraj C, Rifas K, Risly S. & Weerakoon, S.B. 2015, Model study of sediment flushing in Rantemebe reservoir, *ACEPS 2015*, pp.300-307.

Rutschmann, P., & Hager, W. 1996, Diffusion of flood waves. *Journal of Hydrology*, 178, 19–32.

Suja, A. C. A. & Rajapakse R. L. H. L. 2020, Evaluation of topographic data sources for 2D flood modelling: a case study of Kelani basin, Sri Lanka. *IOP Conference Series: Earth Environmental Science* 612 012043.

User's manual, International River Interface Cooperative, 2019.

Waseem, M., Mani, N., Andiego, G., & Usman, M. 2017, A review of criteria of fit for hydrological models. *International Research Journal of Engineering and Technology*, 04 (11), 1765-1772.

Wongsa, S. 2014, Simulation of Thailand Flood 2011, *International Journal of Engineering and Technology*, 6(6), 452–458.

World Meteorological Organization. 1975, Inter-comparison of conceptual models used in operational hydrological forecasting. (Operational hydrology report no.7/WMO- No 429). Geneva, Switzerland.

Weerakoon. S.B. 2003, 'Depth – Averaged flow computation at a river confluence', *Water and Maritime Engineering,* ice, vol 156, pp 73-83.