COMPARISON OF NUMERICAL HYDRAULIC MODELS APPLIED TO THE REMOVAL OF SAVAGE RAPIDS DAM NEAR GRANTS PASS, OREGON

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INTRODUCTION

Savage Rapids Dam is located in southwestern Oregon, on the Rogue River, 5 miles upstream from the town of Grants Pass (Figure 1). The dam, owned by the Grants Pass Irrigation District, is 39 feet high and has been diverting irrigation flows since its construction in 1921. Fish ladders on the dam are old, do not meet current National Marine Fisheries Service (NMFS) fisheries criteria, and delay migrating fish. In addition, the fish screens on the north side of the dam do not comply with current NMFS fisheries criteria. The Bureau of Reclamation (Reclamation) is designing the dam removal construction and a pumping plant to replace the existing dam to alleviate these fish passage problems. As part of the alternative analysis and design process, numerical models have been utilized to help predict the flow hydraulics, as well as the timing and magnitude of sediment release from the dam.

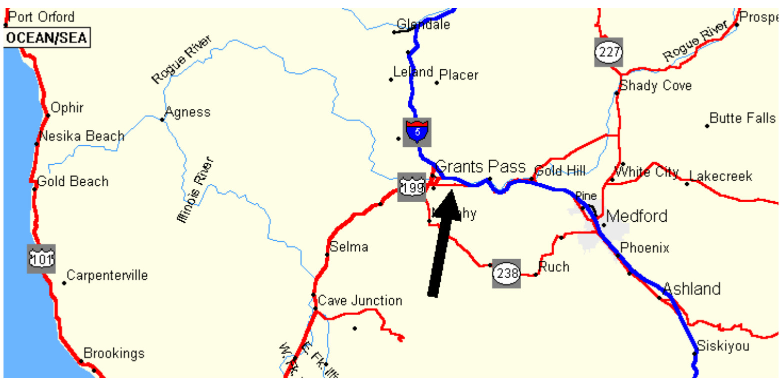


Figure 1 Location map.

OBJECTIVE

The objective of this paper is to present and compare the hydraulic results obtained with a number of numerical modeling tools utilized in the design of the Savage Rapids Dam removal project. The paper will focus the comparison to a calibration run of modeled data results to measured water surface elevation and velocity data. The paper will also discuss the critical input data and calibration parameters for each of the numerical models. Finally, the paper will discuss the modeling experience gained and what areas of modeling still need further research to answer dam removal project questions. It is hoped this case study will prove useful to future numerical model studies of dam removal projects and provide guidance to the appropriate use of numerical models.

METHODS

Numerical models utilized in the Savage Rapids Dam removal project are listed in Table 1. This paper will be limited to the hydraulic results only for water surface and velocity data. All models were run with flows based on discharges associated with the USGS gaging station located in Grants Pass, approximately five miles downstream. The simulation reach extends from the Savage Rapids Park, 0.5 mile upstream of the dam, to about 0.5 mile downstream of the dam for the two-dimensional models. The HEC-RAS and HEC-6t models were extended farther downstream to the Applegate River confluence, about 12 miles from the dam (USBR, 2001).

Table 1 List of numerical models applied to the Savage Rapids Dam removal project.

Model	Source	Description	Project Application
HEC-RAS	USACE Hydrologic Engineering Center (Brunner, 2002)	One-dimensional hydraulic model	Boundary conditions for two-dimensional models; fish passage velocity rating curves
HEC-6t	USACE Hydrologic Engineering Center (Thomas, 1996)	One-dimensional hydraulic and sediment routing model	Rate of reservoir sediment erosion and redistribution into the downstream river channel to evaluate potential impacts to flood stage, fish habitat areas, and downstream water users
MIKE-21	Danish Hydraulic Institute (DHI, 1996)	Two-dimensional finite-difference hydraulic model	Alternative assessment for 4 potential pumping plant locations based on potential for sediment deposition due to backwater eddies
GSTAR-W	Bureau of Reclamation (Lai, 2005)	Two-dimensional hydraulic model	Pumping plant intake and cofferdam design; dam removal alternative analysis

HEC-RAS was utilized to model one-dimensional sub-critical flow. Calibration parameters were limited to the Manning's roughness coefficient and it ranged between 0.035 and 0.040 which are typical for this type of river environment. The expansion coefficient was set at 0.3 and the contraction coefficient was set at 0.1, but not adjusted during calibration.

The MIKE-21 Flow Model is a two-dimensional finite-difference model that uses a square mesh of uniform size (DHI, 1996). Major input data and parameters are the representation of the channel bathymetry, boundary conditions at the upstream and downstream boundaries, channel roughness, eddy viscosity, and total simulation time. A uniform grid cell size of 2-by-2 meters (6.5-by-6.5 foot) was used to represent the channel and flood plain bathymetry. The final calibration resulted in a Manning's roughness parameter of 0.030. An eddy viscosity formulation is included in the 2D model computations to account for momentum fluxes due to turbulence, vertical integration, and sub-grid scale fluctuations. A final eddy viscosity of 1.0 was used.

GSTAR-W offers two-dimensional diffusive wave and dynamic wave solvers, as well as explicit and implicit solvers for solution efficiency and robustness. A detailed description of the mathematical formulation and the numerical methods has been reported by Lai and Yang (2004) and Lai (2005). Both diffusive wave and dynamic wave solutions were obtained for this study so that a comparison may be made between the two solvers. GSTAR-W uses flexible unstructured mesh and the mesh used for this project consists of 20,145 elements and 20,468 nodes with a typical element size of 5 by 12 feet. The major calibration parameter is the flow loss coefficient that was determined to be 0.05 for the diffusive wave model and 0.04 for the dynamic wave model.

For both two-dimensional models, the upstream boundary conditions were a flow discharge of 2,800 ft^3/s where a uniform flow is assumed with flow velocity orthogonal to the boundary. The downstream boundary was a water surface elevation from the calibrated HEC-RAS model. In this study, the downstream elevation was based on the HEC-RAS model as described by Bountry and Randle (2003) and it was determined to be 935.53 ft.

MEASURED DATA DESCRIPTION

A river and floodplain survey was conducted in April 2002 (average discharge of 2,800 ft^3/s) by Reclamation to document the existing channel bottom and topography both upstream and downstream of the dam during a reservoir drawdown period. Data in the channel was collected by boat equipped with survey equipment and a depth sounder. In addition, velocity profile and discharge measurements were made in the river channel both upstream and downstream of the existing dam site. Some additional survey data for floodplain areas was also utilized for topography data, but did not contain any water surface elevation or velocity data.

The relative vertical elevations of the survey data collected should be accurate to the nearest centimeter because the survey was tied to a NGS monument using both global positioning system (GPS) equipment and total station. However, due to turbulence along the water surface in riffle and rapid sections, measurements from the boat can vary by a few tenths of a foot at any given location. This is considered acceptable due to the non-uniform channel bed that typically varies in elevation at least one bed-material particle size (cobble size material ranges from 0.2 to 0.8 feet). Measurements in pool sections do not tend to fluctuate as much as in riffle sections because velocities are slower and it is easier to hold a position.

A RD Instruments 1200 kHz Rio Grande acoustic Doppler current profiler (ADCP) was used to measure velocity, depth, and discharge. The standard deviation for an average velocity measurement was computed to be ±0.3 ft/s (Bountry and Randle, 2003). The average ADCP-measured discharge was about 2 % higher than the USGS reported discharge at Grants Pass.

COMPARISON OF RESULTS

Water Surface Elevation: Computed water surface elevations from different models are compared with the measured elevation along the thalweg in Figure 2. It is seen that all model results agree with the measured elevations well. This indicates that any model used in this project is appropriate in predicting the water surface elevation. Some minor discrepancies do exist among models but they are mostly limited to an area near the radial gates where a hydraulic jump exists. As anticipated, the dynamic wave model predicts the existence of the jump, while the diffusive wave model is incapable of simulating the hydraulic jump.

For a more quantitative comparison, the computed water surface elevations from MIKE-21 was statistically compared to the measured water surface elevations at every point where a corresponding measured data point was available. Results were separated into three reaches: upstream of radial gate outlets, scour hole immediately downstream of the dam, and the downstream river channel from the scour hole to the end of the model (Table 2).

Table 2 Differences between computed (MIKE-21) and measured water surface elevation.

	Upstream of Dam (feet)	Scour Hole Just Downstream of Dam (feet)	River Channel Downstream of Scour Hole (feet)
Count	476	160	386
Mean	0.3	0.4	0.3
Maximum	1.2	0.8	1.0
Minimum	-1.5	0.0	-1.0
Range	2.7	0.7	2.0
Variance	0.0	0.0	0.0
Standard Deviation	0.4	0.1	0.4

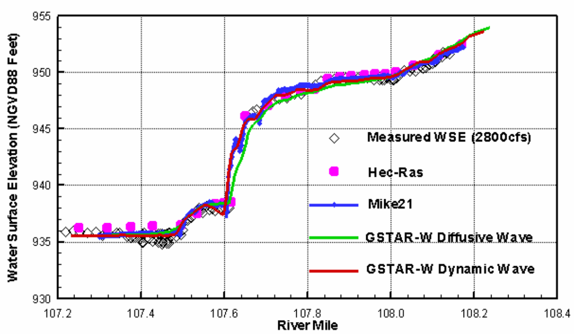


Figure 2 Comparison of predicted and measured water surface elevations for all models.

Figure 3 Velocity measurement points for the simulated river reach (points shown in red).

<u>Velocities and Flow Patterns:</u> The computed velocity vectors and flow patterns are compared with the measured data so that the flow hydraulics may be compared in greater detail. It is noted that a good prediction of the water surface elevation does not guarantee a good prediction of velocities and flow patterns.

The ADCP-measured and depth-averaged velocity data were compared for the measurement points displayed in Figure 3. Upstream of the dam, eight cross sections were compared along an

800-foot section of river. Downstream of the dam, two areas are compared: one is immediately downstream of the dam on the right side where a scour hole is present; another is downstream of the excavated channel used to bypass flow through the radial gates.

A comparison is first made between the GSTAR-W model results and measured velocity vectors at the eight cross sections upstream of the dam (Figures 4 and 5). Agreement is favorable for both diffusive and dynamic wave models except at a few locations. Overall, the difference between the two solutions is not appreciable. The dynamic wave model is capable of predicting the flow separation on the left bank of cross sections 3 and 4 while the diffusive wave model is not.

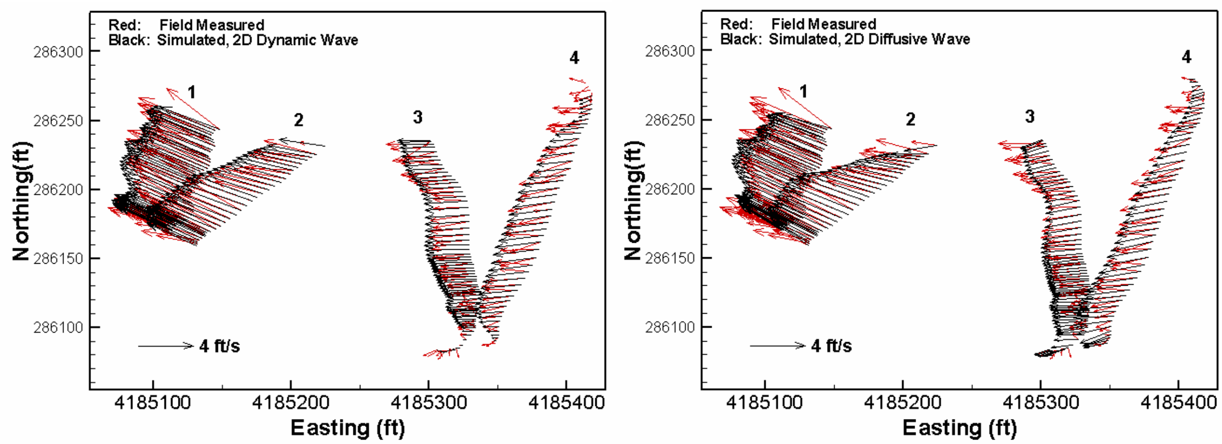


Figure 4 Comparison of predicted and measured velocity vectors at cross sections 1 to 4 upstream of the dam.

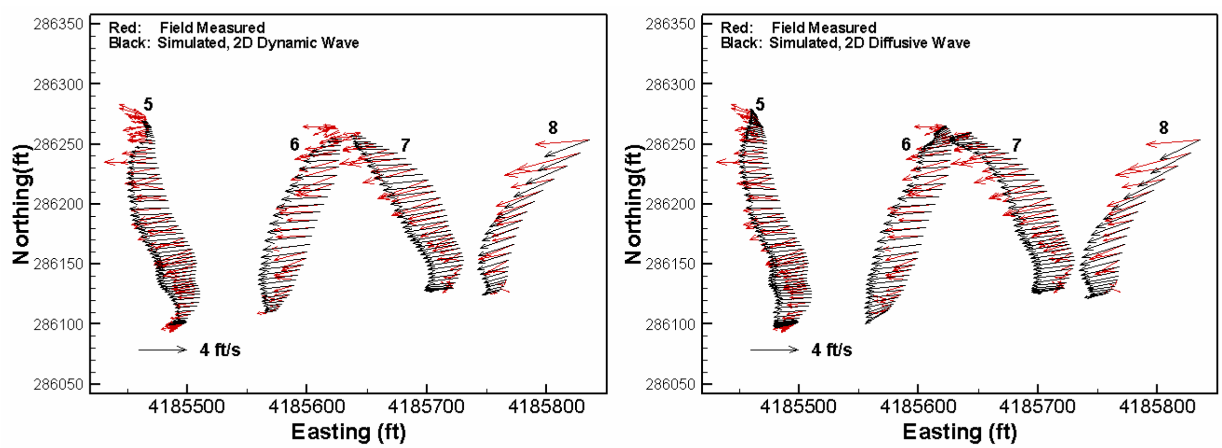
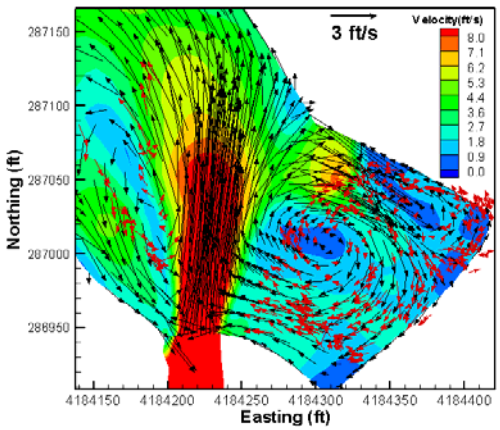
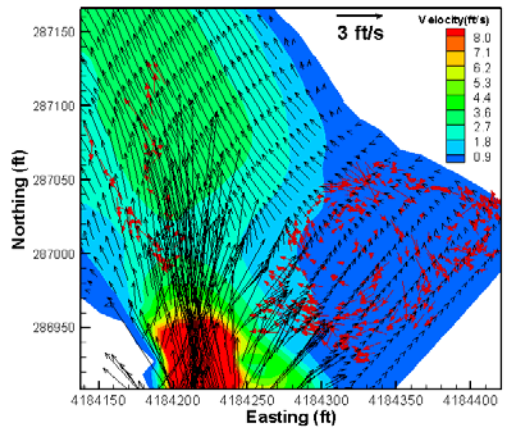


Figure 5 Comparison of predicted and measured velocity vectors at cross sections 5 to 8 upstream of the dam.

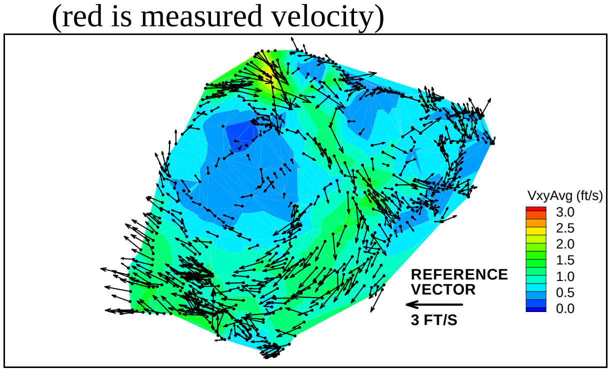
A comparison of velocities and flow patterns was done in a complex eddy area downstream of the dam where two different flow directions were measured (Figure 6c). It is clear that the diffusive model (Figure 6b) is incapable of predicting any eddies and therefore, the velocity results in such areas are in gross error. On the other hand, the dynamic wave model of GSTAR-W is quite good in predicting the eddy structures (Figure 6a). It is noted that the two-eddy structure on the right of the jet stream from the excavated channel is well predicted both in terms of size and location. In addition, the eddy on the left of the jet stream is also predicted. However, the dynamic wave model of MIKE-21 failed to predict the two-eddy structure. The exact reason is unknown but one possibility could be due to the use of a square mesh. These results indicate that the dynamic wave model did the best overall representation of the complex eddies and flow separation present in this case study.



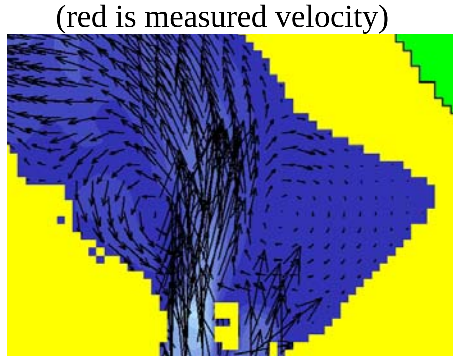
(a) GSTAR-W dynamic wave solution
(red is measured velocity)



(b) GSTAR-W diffusive wave solution
(red is measured velocity)



(c) Measured velocity vectors



(d) MIKE-21 solution (no scale)

Figure 6 Comparison of velocity vectors and flow patterns downstream of the dam.

More detailed comparisons were made between the MIKE-21 computed results and the measured results. The maximum computed velocity was compared to the maximum measured velocity for three reaches in the calibration model (Table 3). The range of computed velocities compared closely with measurements except for in the scour hole immediately downstream of the dam where measured velocities were higher than computed (see Figures 6c and 6d).

Statistics for a comparison of measured to computed velocities at all eight cross sections upstream of the dam are listed in Table 4. For a total of 418 measurements, the mean, minimum, and maximum computed velocities are very close to measured data and the average velocities are within the ±0.3 ft/s accuracy of ADCP velocity measurements. The "mean difference" between computed and measured values is 0.2 ft/s, and the standard deviation is 0.8 ft/s.

Table 3 Comparison of measured to computed (MIKE-21) maximum velocities.

Reach Upstream of Dam (ft/s)		Scour Hole Just Downstream of Dam (ft/s)		River Channel Downstream of Scour Hole (ft/s)	
Computed Values	Measured Values	Computed Values	Measured Values	Computed Values	Measured Values
8.0	8.7	1.5	2.5	6.1	6.1

Table 4 Statistics for measured versus computed velocities for eight cross sections upstream of dam for calibration flow of 2,800 ft^3/s.

	Measured Velocity (ft/s)	Computed Velocity (ft/s)
Count	418	418
Mean	2.5	2.7
Maximum	6.2	5.8
Minimum	0.1	0.3

DISCUSSION AND CONCLUDING REMARKS

Based on this study, the most important input data and calibration parameters are summarized as follows:

1. A good bathymetric survey is important to represent the topography of the study area. If detailed flow hydraulics such as velocity and flow patterns is needed, local topography for the interested area should be surveyed accurately as local features influence local flows significantly.

2. The most critical calibration parameter is the Manning's roughness coefficient. It varies among different models due to differences in model assumptions, approximations and formulations.

3. Based on GSTAR-W simulations, we do not find the selection of turbulence models important, at least for the present study. It is recommended that it should be regarded as a secondary calibration parameter at the most.

Based on the results, following findings and recommendations may be drawn:

1. If water surface elevation along the thalweg is the major hydraulic variable of interest, any of the models used in this project may be suitable as far as a good calibration study is carried out. Use of two-dimensional models is warranted only for: (a) cases that require detailed flow velocity and/or flow patterns; or (b) require topography or flow features of 2D nature.

2. The diffusive wave solver is suitable for many applications that require the water surface elevation, water depth and bulk velocities. We found the dynamic wave solver necessary when eddies and flow separations is the interested outputs.

3. For the diffusive wave solver, the Manning roughness coefficient should be interpreted as the energy loss coefficient as extra losses due to eddies, separations, and hydraulic jumps are lumped together with the coefficient. So the coefficient used for the diffusive wave solver is usually higher than that for dynamic wave solver. Hydraulic jump can only be simulated with the dynamic wave model and a smooth transition will be predicted by the diffusive wave solver. If details around a jump are not important, the diffusive wave solution may still be used even if there are hydraulic jumps.

The currently available hydraulic modeling tools were sufficient for most design questions associated with the removal of Savage Rapids Dam. We found using a range of modeling tools worked well to meet a range of project needs while also complementing each other to improve confidence in results. Although not discussed in detail here, sediment modeling tools are still limited to one-dimensional scenarios and could use further development to address modeling sediment through riffle pool systems and erosion of reservoir sediment during dam removal.

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