



TWO-DIMENSIONAL FINITE-VOLUME RIVER DYNAMICS MODEL

VERIFICATION AND VALIDATION TESTS

April 2015

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Last document modification date: April, 2015.

Technical Support: support@hydronia.com

Web site: www.hydronia.com

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1 INTRODUCTION TO RiverFlow2D SUITE OF MODELS

RiverFlow2D, previously known as RiverFLO-2D, is a two-dimensional flexible-mesh depth-averaged hydrodynamic and mobile bed and pollutant transport model for rivers that belongs to Hydronia LLC suite of advanced computer models. RiverFlow2D use a fourth-generation finite-volume numerical method to solve the shallow water equations in 2D ensuring almost zero volume conservation error regardless of the complexity of the terrain. The model provides robust handling of wet and dry areas and is able to accurately account for subcritical and superclritical flows. RiverFlow2D includes tools to account for hydraulic structures such as culverts, weirs, gates, and bridges.

This document describes RiverFlow2D verification and validation test cases. The tests follow the verification guidelines recommended by the ASCE 3D Flow Model Verification and Validation Committee (ASCE, 2008). This process involves testing the model with analytical solutions, simplified cases, and tests with laboratory experiments and finally comparisons with documented real cases where field data is available. Numerical tests show that RiverFlow2D model results compare well, with both simplified solutions, benchmark experimental measurements and real-world flooding events.

1 ANALYTICAL SOLUTIONS AND SIMPLE TEST CASES

This section presents verification test where the RiverFlow2D model results are compared with analytical solutions for simple cases. These comparisons allow assessing that the numerical method of the shallow water equations used in RiverFlow2D is correct.

1.1 Uniform Subcritical Flow

The purpose of this test is to verify the RiverFlow2D capability to reach uniform subcritical flow conditions in a long rectangular channel. The problem consists of a 100 m long and 60 m wide rectangular channel with a bed slope of 0.01. The triangular-cell mesh has 7104 nodes and 13886 elements as shown in Figure 1.

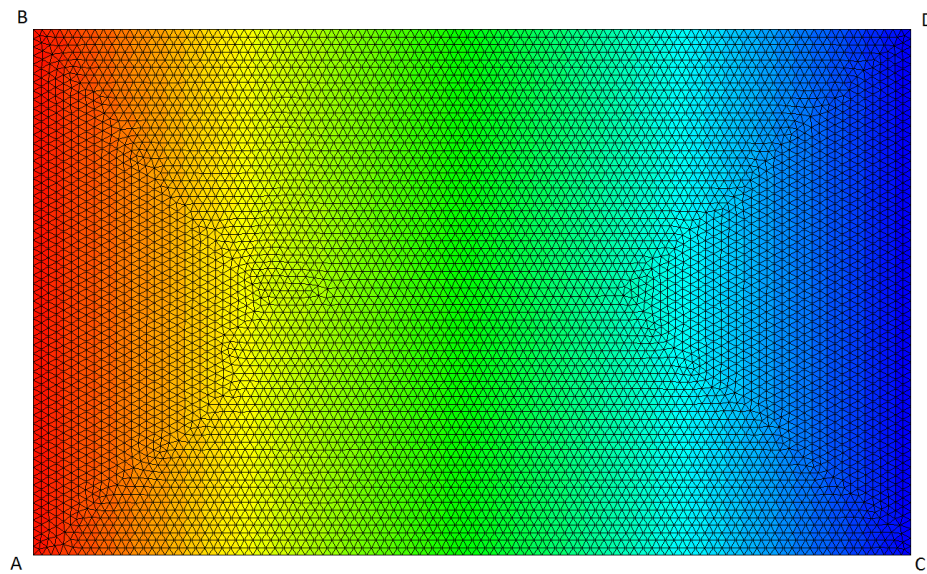


Figure 1 Triangular-cell mesh used in uniform subcritical flow test.

The channel was designed relatively wide, to avoid having to impose semi-slip condition as would be necessary on a narrow channel to account for wall friction.

The water flows from left to right. Boundary conditions include a constant water flow rate along boundary A-B of 146.76 m³/s and a uniform flow condition downstream at C-D, where only the bed slope = 0.01 is provided. On the closed boundaries A-C and B-D slip (no-flux) conditions are imposed. Manning's n was set to 0.04 for all elements.

1.1.1 Comparison of RiverFlow2D Results with Exact Solution

The exact solution for this test is given by the Manning's equation in metric units:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

where Q is the water discharge in m³/s, R is the hydraulic radius = A/P , P is the wetted perimeter and S is the bed slope.

For a rectangular channel $A = h B$, where h is the flow depth and B is the channel width and $P = B + 2h$.

For this test we have this data:

$$\begin{aligned} B &= 60 \text{ m} \\ S &= 0.01 \\ n &= 0.04 \\ Q &= 146.76 \text{ m}^3/\text{s} \end{aligned}$$

It can be easily checked that introducing $h = 1 \text{ m}$ we get:

$$\begin{aligned} A &= 60 \text{ m}^2 \\ P &= 63 \text{ m} \\ R &= 0.9677 \text{ m} \end{aligned}$$

And introducing these values in Manning's equation we obtain

$$Q = 146.76 \text{ m}^3/\text{s}$$

In addition we can compute the Froude number Fr to check that the flow is indeed subcritical:

$$Fr = \frac{Q/A}{\sqrt{g h}} = 0.78$$

Therefore, with the data and geometry provided the flow should converge to a water depth of 1 m and velocity 2.45 m/s and Froude = 0.78.

The RiverFlow2D model was run until convergence ($\sim 6 \text{ min.}$) using variable time-step option. The following table compares exact and numerical results obtained with RiverFlow2D.

Table 1 Numerical and exact result comparison

VARIABLE	EXACT SOLUTION	NUMERICAL RESULTS
Depth h (m)	1	0.99-1
Velocity U (m/s)	2.45	2.45-2.48
Froude number Fr	0.78	0.78-0.80
Outflow discharge (m^3/s)	146.76	146.76

1.2 Uniform Supercritical Flow

The purpose of this test is to verify the RiverFlow2D capability to reach uniform flow under supercritical conditions in a long rectangular channel. As in the previous test, this problem consists of a 100 m long and 60 m wide rectangular channel with a bed slope of 0.01, but to achieve supercritical flow, Manning's n is lowered to 0.02. The triangular-cell mesh has 7104 nodes and 13886 elements as shown in Figure 2.

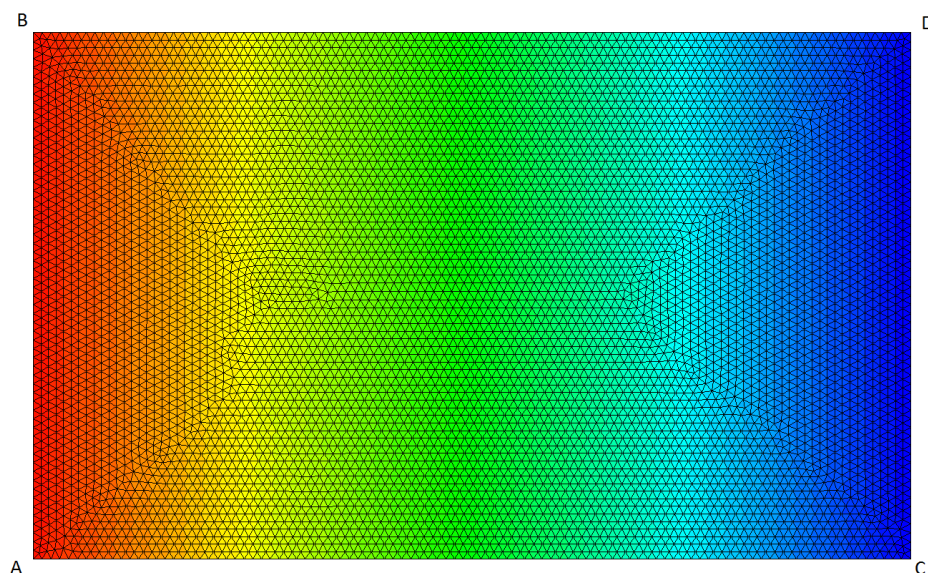


Figure 2 Triangular-cell mesh used in uniform supercritical flow test.

The channel was designed relatively wide, to avoid having to impose semi-slip condition as would be necessary on a narrow channel to account for wall friction.

The water flows from left to right. Boundary conditions include a constant water flow rate and depth along boundary A-B of 293.513 m³/s and 1 m respectively as required by supercritical flow conditions at inflow boundaries. Downstream at C-D, free condition (BCType = 11) was specified. On the closed boundaries A-C and B-D slip (no-flux) conditions are imposed. Manning's n was set to 0.02 for all elements.

1.2.1 Comparison of RiverFlow2D Results with Exact Solution

The exact solution for this test is given by the Manning's equation in metric units:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

where Q is the water discharge in m³/s, R is the hydraulic radius = A/P , P is the wetted perimeter and S is the bed slope.

For a rectangular channel $A = h B$, where h is the flow depth and B is the channel width and $P = B + 2h$.

For this test we have this data:

$B = 60$ m
 $S = 0.01$
 $n = 0.02$
 $Q = 293.513$ m³/s

It can be easily checked that introducing $h = 1$ m we get:

$A = 60$ m²
 $P = 63$ m
 $R = 0.9677$ m

And introducing these values in Manning's equation we obtain

$$Q = 293.513 \text{ m}^3/\text{s}$$

In addition we can compute the Froude number Fr to check that the flow is indeed subcritical:

$$Fr = \frac{Q/A}{\sqrt{g h}} = 1.56$$

Therefore, with the data and geometry provided the flow should converge to a water depth of 1 m and velocity 4.89 m/s and Froude = 1.56.

The RiverFlow2D model was run until convergence (~6 min.) using variable time-step option. The following table compares exact and numerical results obtained with RiverFlow2D.

Table 2 Numerical and exact result comparison

VARIABLE	EXACT SOLUTION	NUMERICAL RESULTS
Depth h (m)	1	0.99-1
Velocity U (m/s)	4.89	4.88-4.93
Froude number Fr	1.56	1.56-1.58
Outflow discharge (m ³ /s)	293.51	293.51

1.3 Dam-break Problem

This problem represents the flooding occurring after the instantaneous removal of a dam over a frictionless horizontal terrain. The test is particularly important because it allows evaluating the model capability to deal with high depth and velocity gradients and significant advective accelerations. The analytical solution of this problem can be obtained using the method of characteristics as proposed by Stoker (1957). The data is the water elevation at the reservoir and the initial depth of water on the river downstream as shown on the following figure:

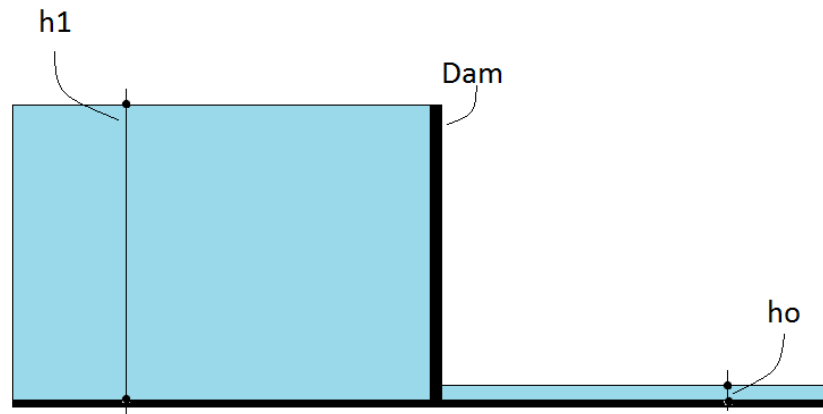


Figure 3 Initial conditions for dam break problem.

After the dam is removed at time = 0, a complex wave pattern develops, including a negative wave that moves upstream from the dam site, and a discontinuous wave that travels downstream along the river.

Table 3 Data for dam-break problem

VARIABLE	VALUE
Reservoir depth h_1 (m)	100
River depth h_0 (m)	10
Initial velocity U (m/s)	0

1.3.1 Comparison of RiverFlow2D Results with Exact Solution

The exact solution for this test provided by Stoker (1957) is given by analytical expressions for the regions indicated in the following figure:

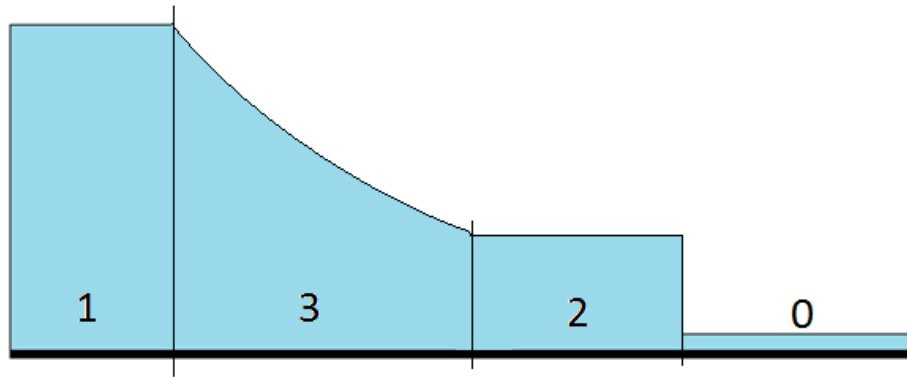


Figure 4 Dam-break problem regions for the analytical solution. Coordinate origin ($x = 0$) is located at the dam site between regions 2 and 3.

The exact expressions are summarized in the table below:

Table 4 Analytical solution for the dam-break problem, Stoker, 1957, pp. 334-338.

VARIABLE	REGION 0	REGION 1	REGION 2	REGION 3
Depth	h_0	h_1	Stoker Fig. 10.8.6	$c_3 = \sqrt{gh_3} = \frac{1}{9} \left(2c_1 - \frac{x}{t} \right)^2$
Velocity	0	0	$u_2 = 2 (g h_1)^{1/2} - 2 (g h_2)^{1/2}$	$u_3 = \frac{1}{9} \left(c_1 + \frac{x}{t} \right)$
Front speed			$\xi = c_0 \left\{ \frac{1}{8} \left[2 \frac{c_2^2}{c_0^2} + 1 \right]^2 - 1 \right\}^{1/2}$	

Table 5 Analytical solution for the dam-break problem, Stoker, 1957, pp. 334-338.

VARIABLE AT INITIAL DAM LOCATION	EXACT SOLUTION FOR ALL TIMES
Depth h	Stoker, 1957 figure 10.8.6
Velocity U	$\frac{2}{3} c_1 = \frac{2}{3} (g h_1)^{1/2}$

The dam-break problem was discretized with a 15025 node, 28998 element mesh spanning 500 m upstream and 500 m downstream from the initial dam site as depicted in the following figure:

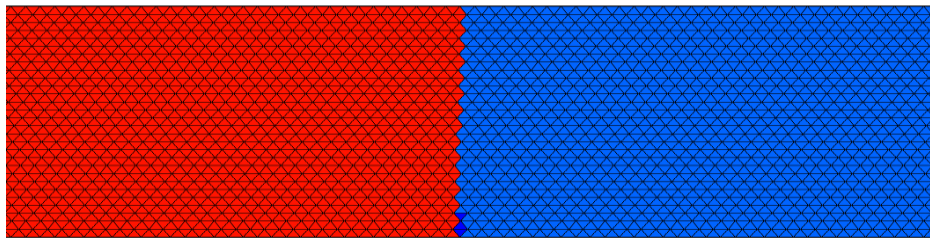


Figure 5 Detail of the triangular-cell mesh around the dam site. Red area indicates the reservoir at 100m depth and blue is 10 m depth downstream of the dam.

The following table summarized other data used for this test:

Table 6 Summary of data used for dam-break problem

VARIABLE	RiverFlow2D value
Reservoir depth h_1 (m)	100
River depth h_0 (m)	10
Initial velocity U (m/s)	0
Time step	Variable

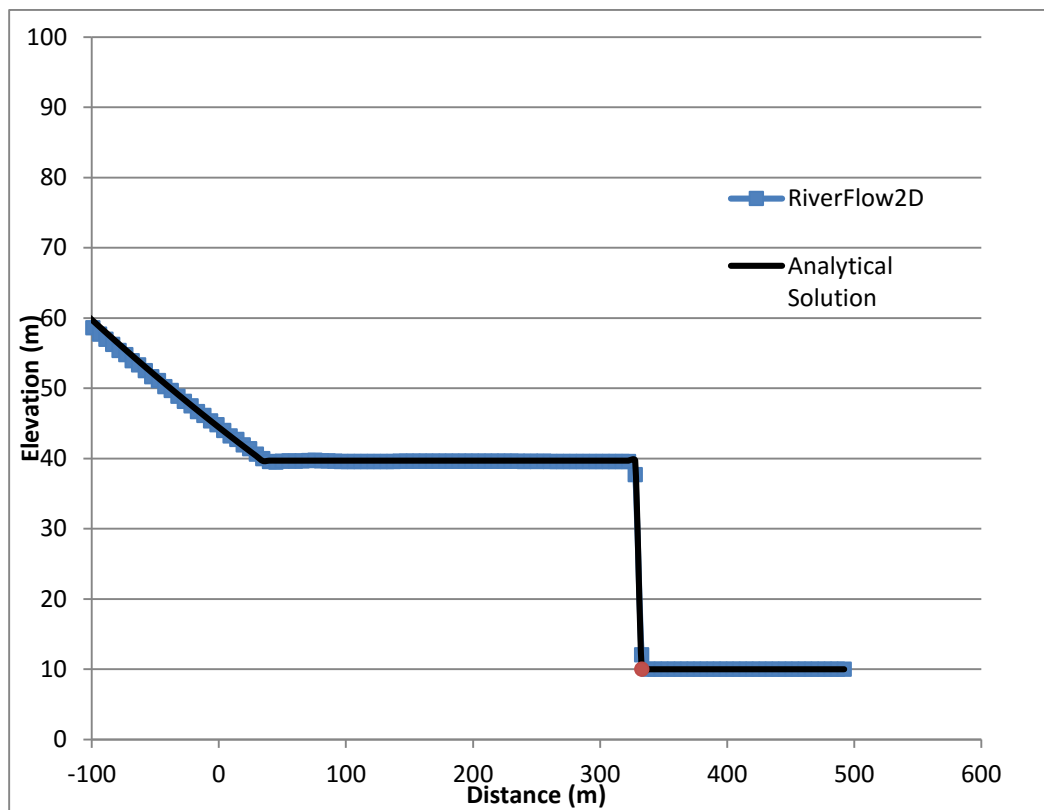


Figure 6 Water surface profile at 11 seconds after the dam-break.

Table 7 Exact and RiverFlow2D result comparison on region 2.

VARIABLE	EXACT SOLUTION	RiverFlow2D RESULT S
Depth (m)	39.7	39.66
Velocity (m/s)	23.17	23.19

1.4 Tidal channel (1D)

This test consist in the propagation of a sinusoidal wave in a frictionless rectangular channel 10 m deep, 3000 m long, and 200 wide shown in Figure 7. At the channel entrance on the left side of the channel, a sinusoidal wave is imposed of the form $\eta = a \sin \varpi t$. At channel exit the velocity is imposed as $u\eta\sqrt{g/h}$, where $a = 0.5 \text{ m}$, $\varpi = 0.021$. The initial conditions are zero velocities and flat $\eta = 0$ water surface. The downstream condition is an open outlet boundary and slip conditions are imposed on channel wall. The triangular-cell mesh has 313 nodes and 496 elements and frictionless bed.

The total simulation time is 1 h. and the time step is 1 s. A wet and dry minimum water depth threshold of 0.01 m. is assigned for the entire mesh. The output data is written every 10 s.

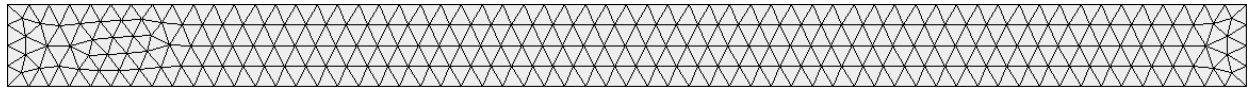


Figure 7 Mesh of One-dimensional Channel Test.

1.4.1 Comparison of RiverFlow2D Results with Exact Solution

Figure 8 compares water elevation for the analytical solution and for the numerical results for η at the channel exit.

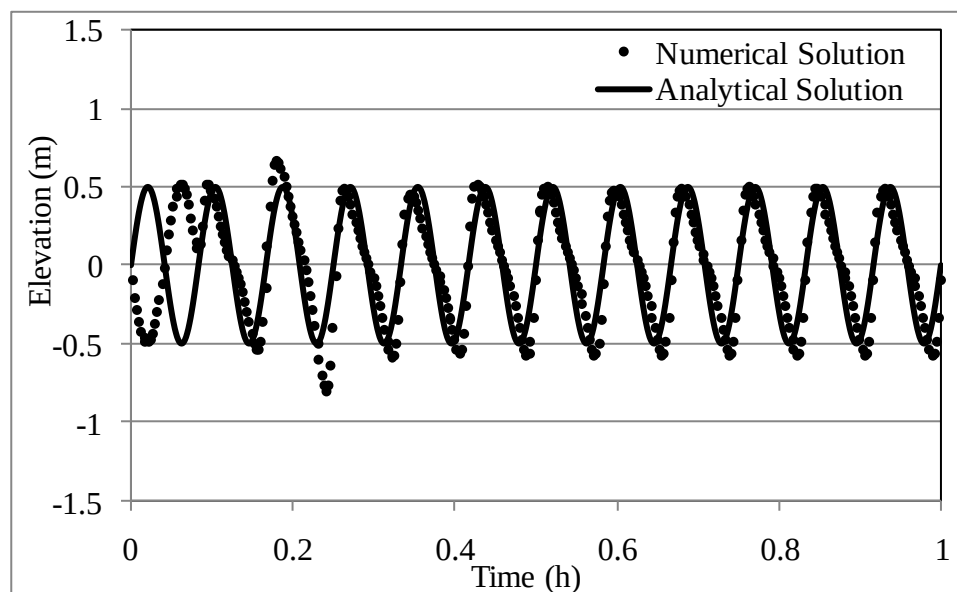


Figure 8 Elevations at exit of One-dimensional Channel Test.

1.5 Hydrostatic Conditions (2D)

The goal of this test is to evaluate the model performance under static conditions, where one part of the triangular-cell mesh is dry and has irregular bed elevations and relatively steep slopes. Dry steep slope elements may pose a severe numerical problem because if handled incorrectly, the 2D gravity term in the equations can generate erratic flow. Starting with a static initial water depth that leaves part of bed dry, a properly working dry bed algorithm should preserve the initial static conditions, minimize mass conservation errors and be free from erroneous velocities.

It appears that this numerical experiment was first proposed by Brufau et al., 2004. The problem consist of a square pool $1\text{ m} \times 1\text{ m}$ with a central bump on the bottom defined by the following formula:

$$z(x,y) = \max\left\{0, 0.25 - 5\left[\left(x - \frac{1}{2}\right)^2 + \left(y - \frac{1}{2}\right)^2\right]\right\}$$

The maximum bed elevation is 0.25 m. The initial condition is a water surface elevation of 0.1 m, and zero velocities everywhere. The triangular-cell mesh has 3052 nodes and 5899 elements and frictionless bed.

The total simulation time is 180 s. and the time step variable. The output data is reported every 36 s.

Figure 9 shows the bed elevation contour lines.

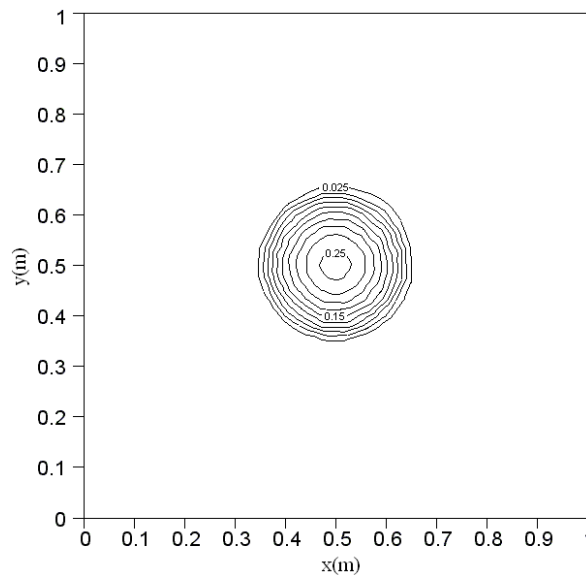


Figure 9 Bed contour lines for static dry-wet test.

1.5.1 RiverFlow2D Results

Figure 10 shows the model calculated water surface elevations on a profile across the tank mid section and Figure 11 present the velocity field.

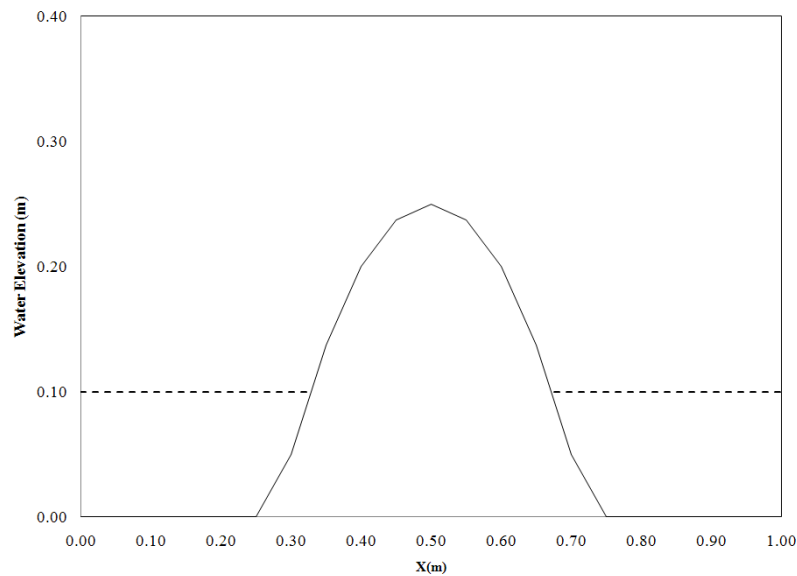


Figure 10 Profile across mid section for static dry-wet test.

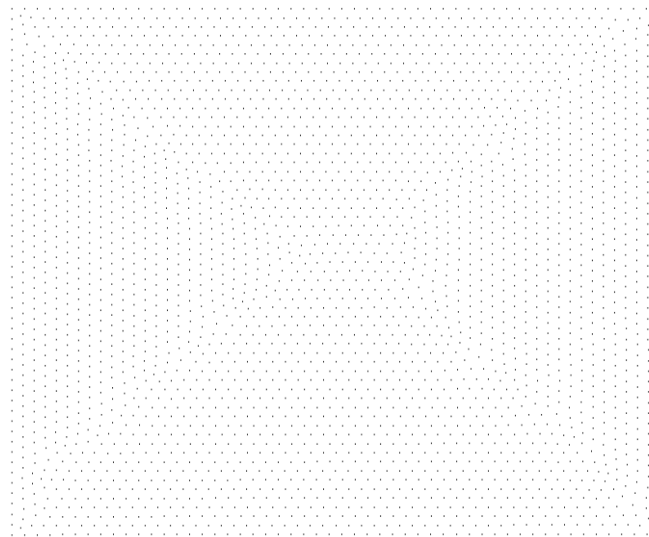


Figure 11 Velocity field for static dry-wet test. The points represent zero velocity for all the vectors in the field.

1.6 Three islands (2D)

The purpose of this test is to verify the model performance of dynamic flooding under drying and wetting bed. The test was first proposed by Kawahara and Umetsu, 1986. It consists of a rectangular channel with three bumps on its bottom. Figure 12 shows the bed elevation contour lines. The channel length is 75 m and width is 30 m. Two symmetrical bumps at the left have maximum elevation of 1 m and the central bump elevation is 3 m. The triangular-cell mesh has 1269 nodes and 2392 elements. The water flows from section AA–BB to section CC–DD. Boundary conditions include water elevation along boundary AA–BB as follows: $h = 0.5$ m for times 0 s to 300 s; $h = 1.0$ m, for times 300 s to 350 s; $h = 2.5$ m., for times greater than 350 s. Boundaries AA–DD and BB–CC are vertical walls, where a slip velocity condition is imposed. At boundary AA–B velocity is 1 m/s. The downstream

condition is an open outlet boundary and slip conditions are imposed on channel walls. Initial dry bed conditions ($h = 0.01$) are imposed everywhere.

The total simulation time is 900 s. and the time step is 0.01 s. A wet and dry minimum water depth threshold of 0.01 m. is assigned for the entire grid. The output data is reported every 20 s.

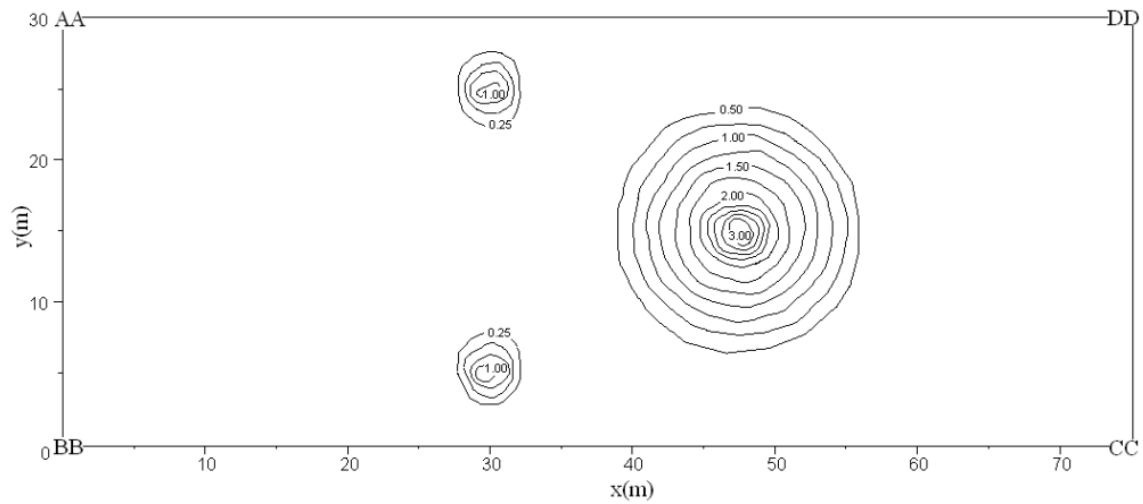


Figure 12 Bed contour lines for dynamic drying-wetting test

1.6.1 RiverFlow2D Results

Figure 13 and Figure 14 present velocity fields and water surface profiles for times 20 s and 900 s respectively.

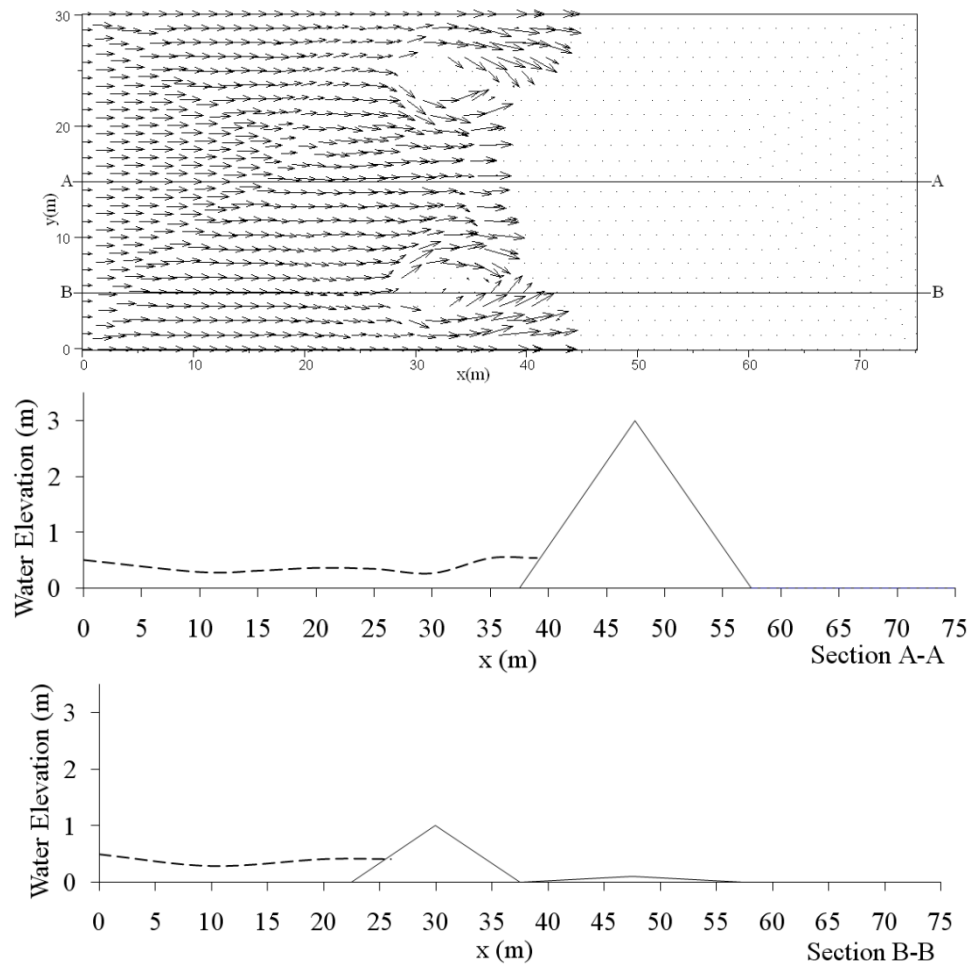


Figure 13 Dynamic drying-wetting test. Results at 20 s. Velocities and water elevations through indicated profiles.

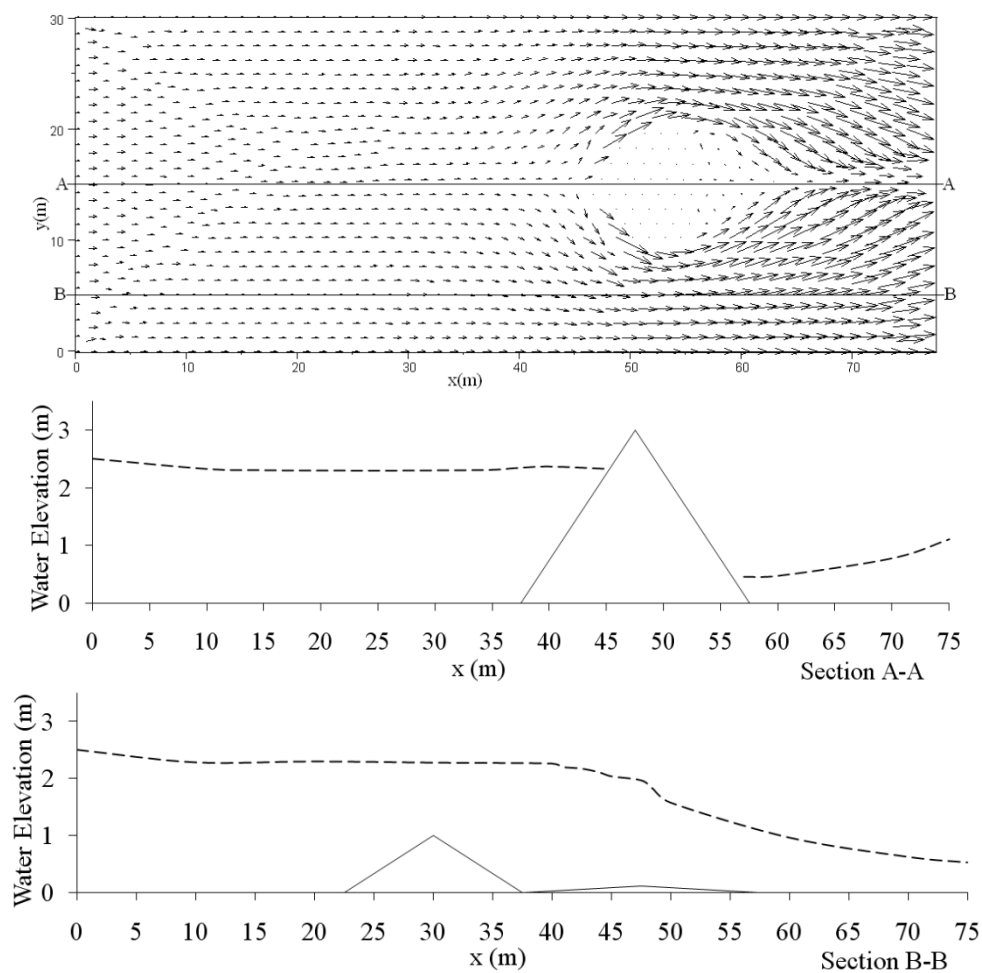


Figure 14 Dynamic drying-wetting test. Results at 900 s. Velocities and water elevations through indicated profiles.

2 Comparison with Laboratory Experiments

2.1 Dam-Break with Obstacle

In this test, RiverFlow2D model results are compared with experimental dam-break flood measurements from a laboratory study performed by Recherches Hydrauliques Laboratory Chatelet and University of Bruxelles (Brufau et al. 2002). The test involves the advance of a dam-break flood wave over a triangular obstacle located on a horizontal concrete bed channel as indicated in Figure 15. Initial reservoir depth is 0.75 m. The test starts by instantaneously removing the dam-gate. Depth evolution is measured at gauging points G1, G2, G3 and G4 shown in Figure 15.

The triangular-cell mesh used in the numerical computations has 4770 triangular elements. Manning's coefficient was set to 0.011 on the boundary elements and to 0.0125 on the rest of the domain. The downstream condition is an open outlet boundary. The total simulation time is 40 s. and the time step variable. The output data is reported every 1 s.

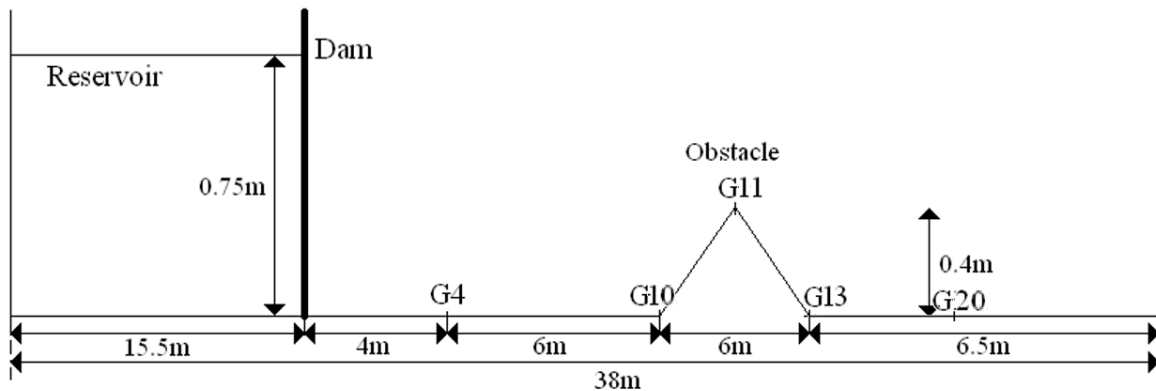


Figure 15 Experimental model for dam-break flood over a triangular obstacle. G4, G10, G11, G13 and G20 indicate location of gauging points.

1.1.1 Comparison of RiverFlow2D Results with Experimental Data

Numerical results were compared with the experimental measurements for the 5 gauging points or stations. Figure 16 shows the measured and numerical evolution of water depth at each station. The concordance between the experimental and numerical results is satisfactory. At the gauging points located before the obstacle, the prediction of the arrival time of the wave as well as the water depth is good. Point G13 is located at the vertex of the obstacle and therefore it is a critical point. It can be observed that the prediction of the transitions from wet to dry is correct.

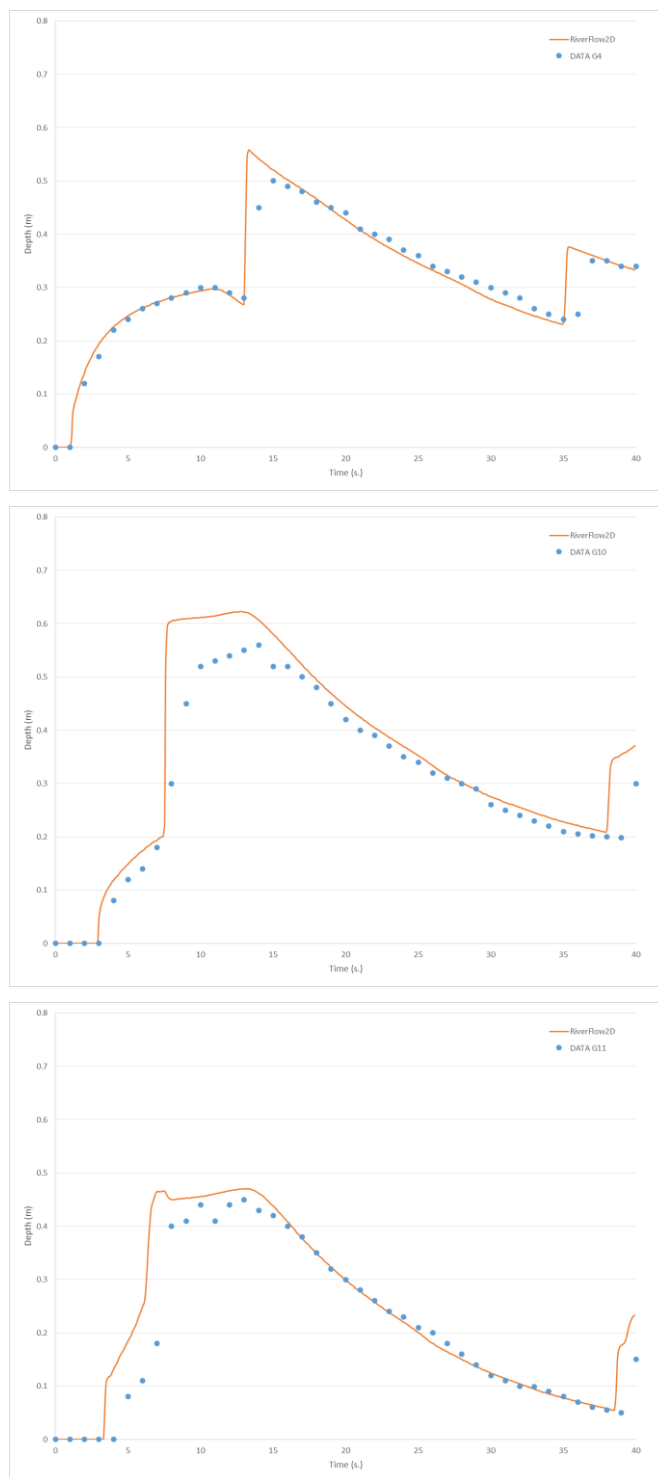


Figure 16 Depth evolution during 40 s at gauging points: G4, G10, and G11 in the simulation of a dam break flood over a triangular obstacle. Points indicate for measured data and solid line RiverFlow2D model results.

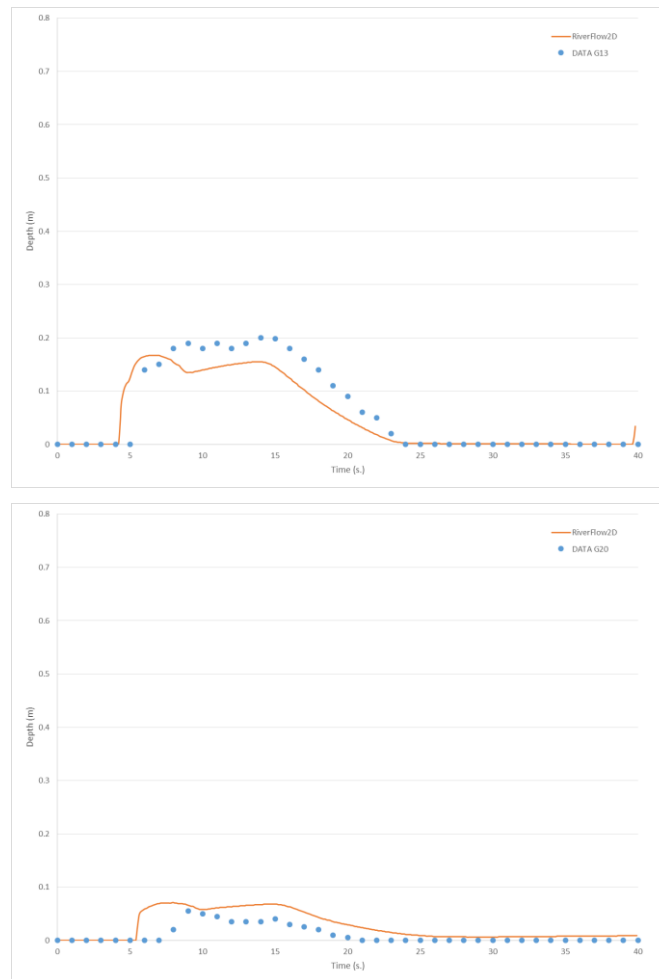


Figure 17 Depth evolution during 40 s at gauging points G13 and G20 in the simulation of a dam break flood over a triangular obstacle. Points indicate for measured data and solid line RiverFlow2D model results.

1.2 Tsunami wave

This test is suggested by NOAA Center for Tsunami Research to check the validity of computational tools for predicting the coastal effect of tsunamis. Here we compare RiverFlow2D with the experimental measurements for a tsunami wave. The experiment was based on a 1/400 reduced scale model of Monai village in Okushiri island (Japan) in order to reproduce a tsunami that took place in 1993 involving coastal waves that reached a height of 32 m (LeVeque and George, 2007). The setup was built and the experiment carried out at the “Central Research Institute for Electric Power Industry” (CRIEPI) laboratory, Abiko, Japan. A 205 m long, 6 m deep and 3.4 m wide tank was used for that purpose. The experimental data obtained at the scaled problem will be used as validation test case. Figure 18, shows a plant view of the coast elevation in the island scaled model.

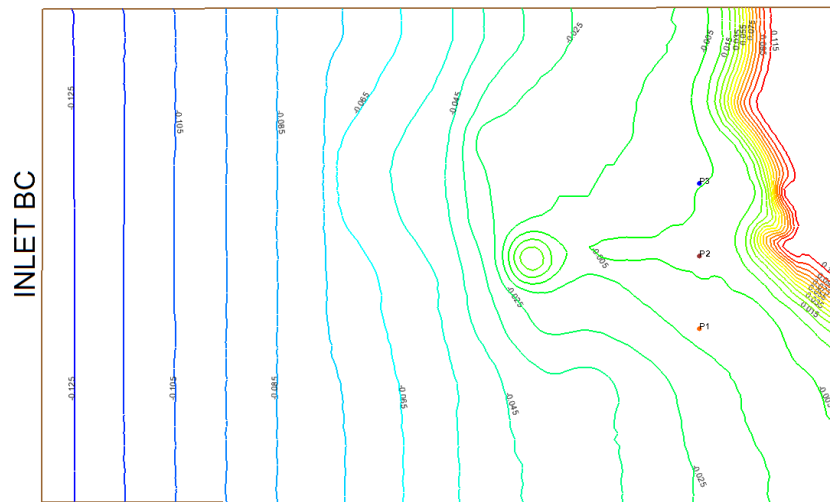


Figure 18 Topography for experimental model of Monai's tsunami.

Figure 18 also displays the location of the three water level gauging stations used (P1, P2 and P3), as well as the wave inlet boundary.

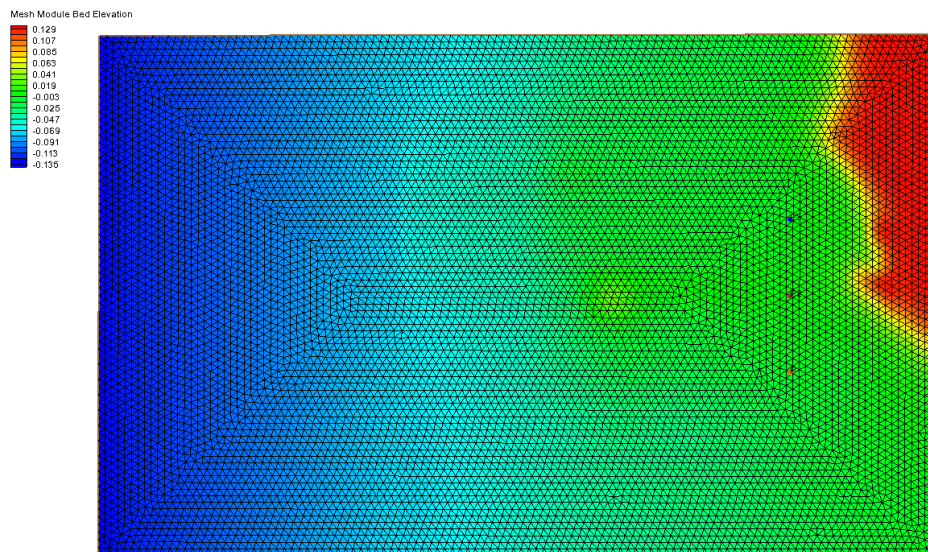


Figure 19 Mesh of 17314 triangular cells.

A water level oscillation was generated at that side of the tank, so that a water level time function is imposed as a

boundary condition to the numerical model. The curve used is plotted in Figure 20 where it can be seen that there is a constant water level initial value, labelled as zero level, that first increases with time, reaching the maximum value in about 13 s and then decreases.

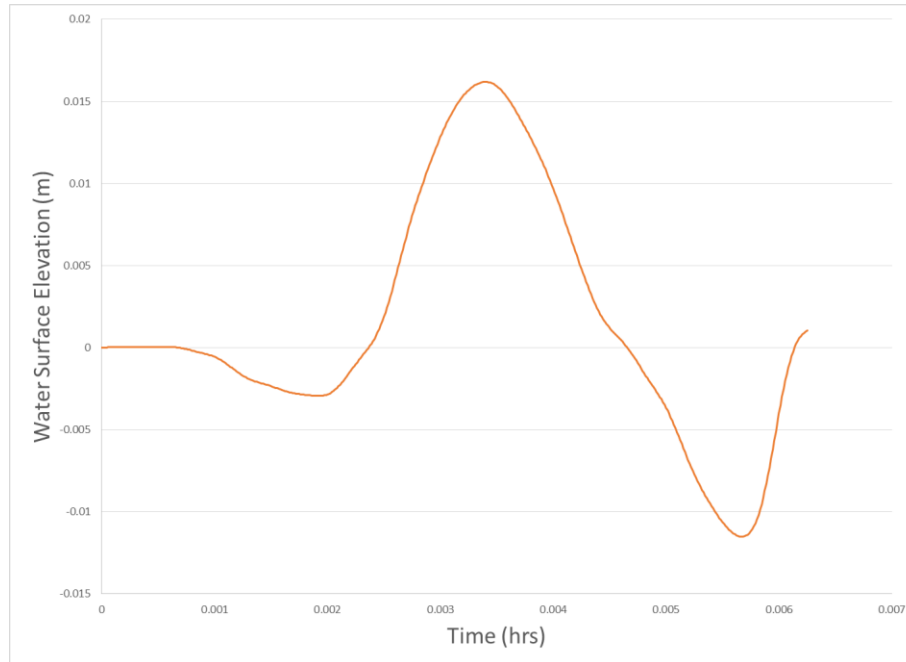


Figure 20 Boundary condition for tsunami experiment.

Additionally, the water is bounded by two impervious vertical side walls and the initially dry beach zone, where the wave runup and wetting/drying processes take place.

1.2.1 Comparison of RiverFlow2D Results with Experimental Data

Figure 21 shows the comparison of model results with experimental measurements corresponding to the 17314-triangular-cell mesh. Model results show a good agreement with the experimental data both in the wave amplitude and the arrival time.

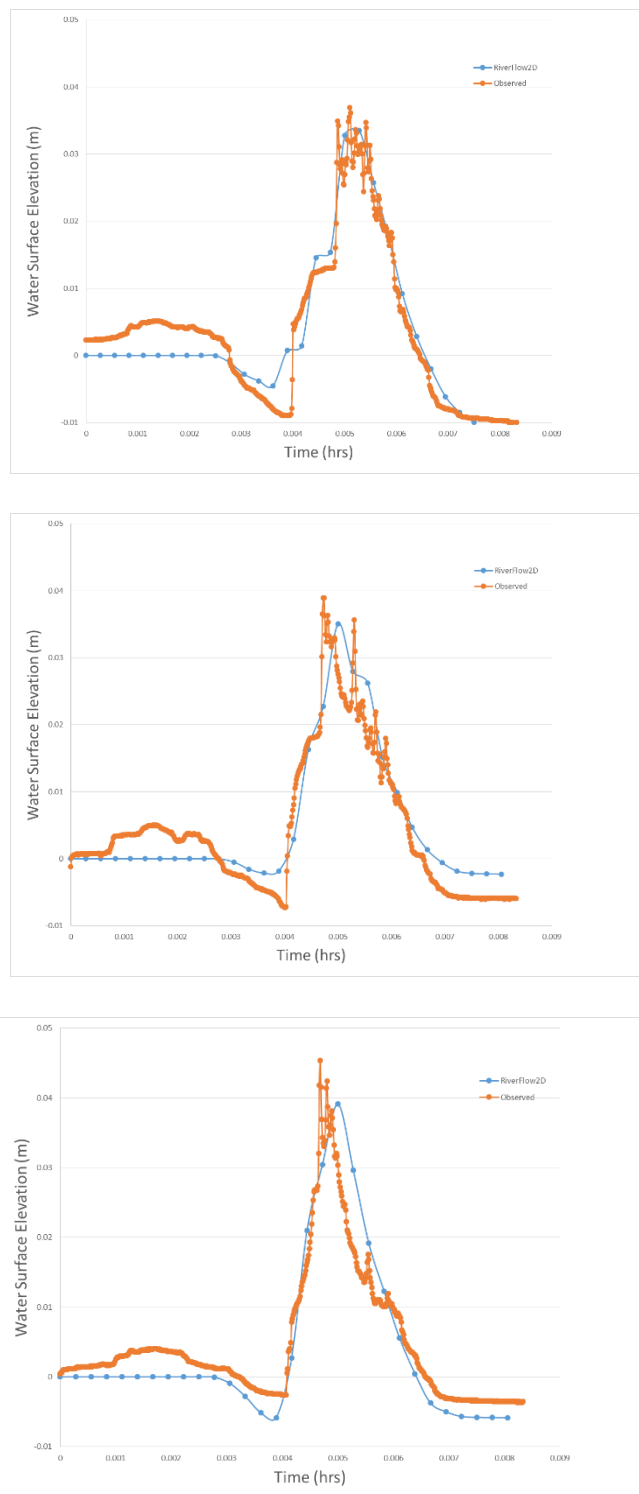


Figure 21 Results for the tsunami's experiment at points P1, P2 and P3 (from top to bottom).

3 Comparison with Field Data

3.1 *Tous dam-break*

3.1.1 *Description*

The failure of Tous Dam in Spain and the flooding of Sumacarcel, a small town located 5km downstream is considered as a case of study in this document. The Tous Dam overtopping failure occurred on October 20, 1982 due to extremely heavy rainfall in the area. The complete set of data used in this case of study was put together during an European project named Investigation of Extreme Flood Processes and Uncertainty (IMPACT) (Alcrudo and Mulet, 2007). The collected field data and information include estimates of the water depth marks that were located along the Sumacarcel City. The field data used in this document have been collected and reported by Alcrudo and Mulet.

The Tous outflow hydrograph was imposed as an inflow boundary condition at the downstream section of the dam. According the suggestion of IMPACT Project researches, a Manning coefficient of 0.030 was used in the simulation for the complete area with the exception of two dense orange tree orchards with a Manning coefficient of 0.075. Maximum water elevations were recorded at 21 locations.

A 7347 node/13869 element mesh was generated for the Jucar River Basin (see Figure 22 and Figure 23). The total time of simulation is 30 hour. The initial conditions are dry bed downstream from the dam location. Manning's n is set to 0.03, constant for all elements. Two large areas are set with a Manning's n of 0.075 that corresponds with two dense orange tree orchards polygons adjacent to Sumacarcel City (see Table 9)

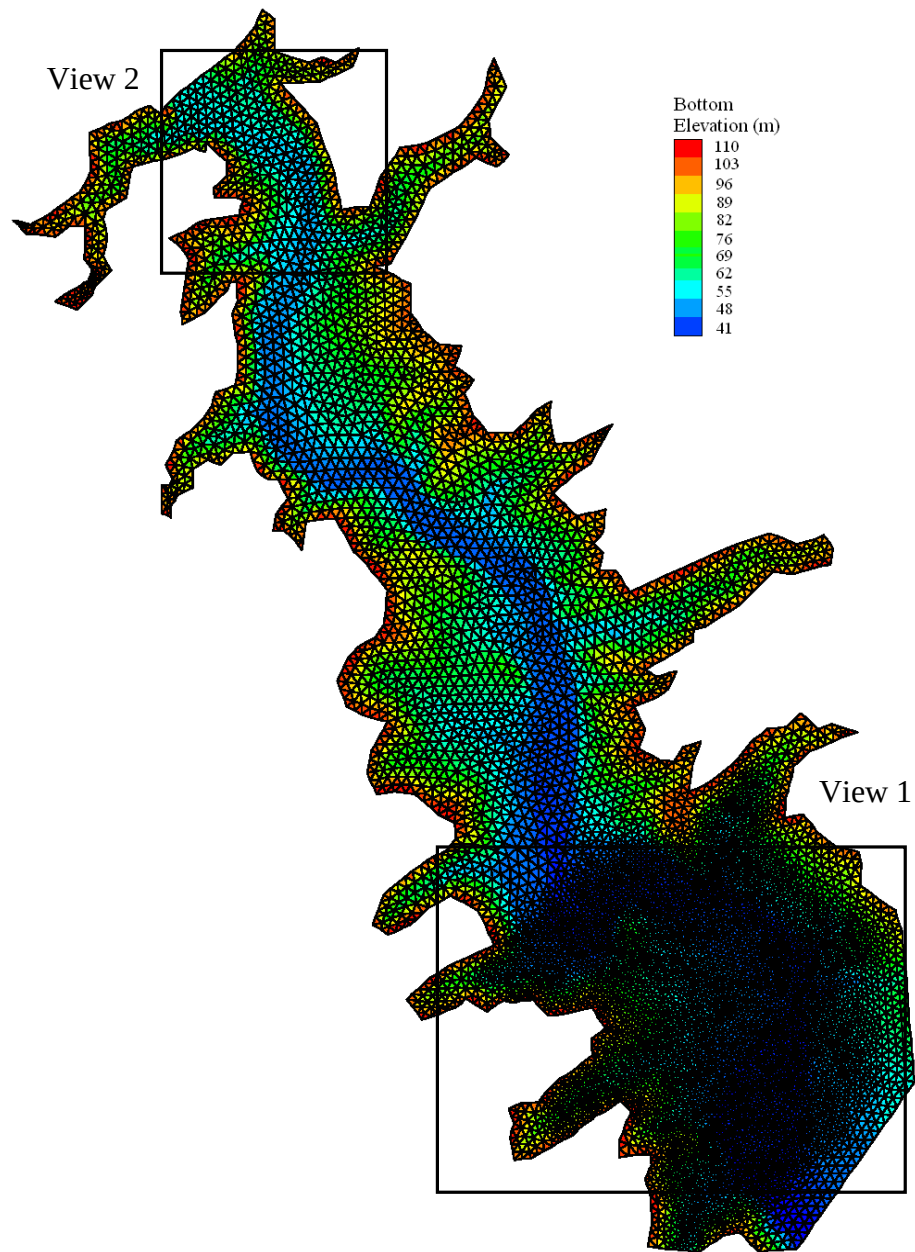


Figure 22 Tous dam-break test. Bottom elevations and mesh extent.

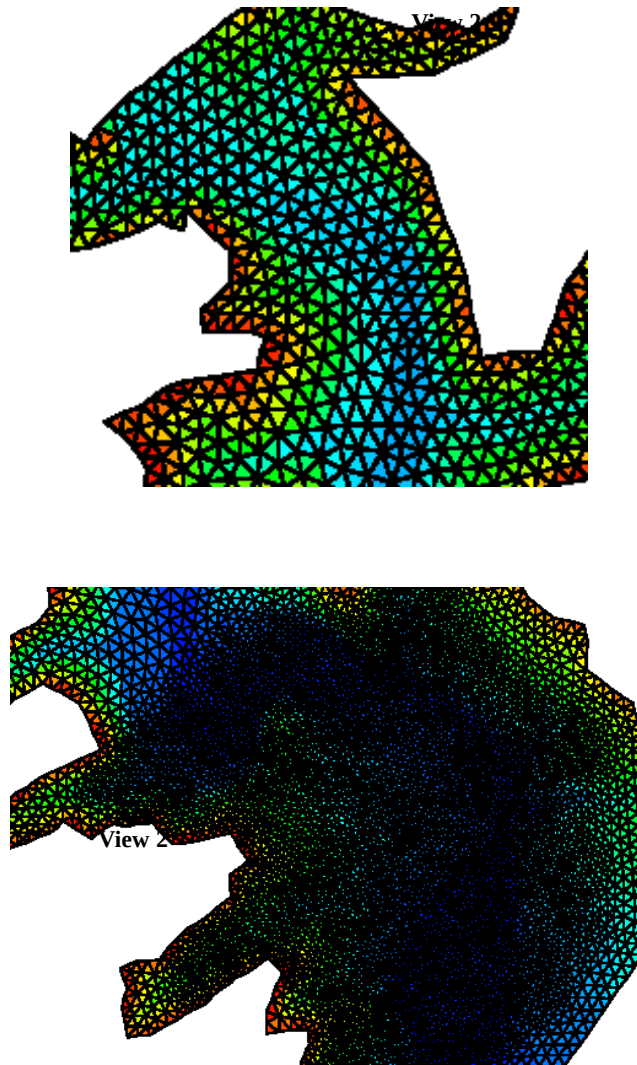


Figure 23 Tous dam-break test. Mesh (View 1) and Mesh detail (View 2).

Table 8 Polygon information covered by Orange Trees according Alcrudo and Mulet estimates.

Vertex Points	Zone 1		Zone 2	
	x(m)	y(m)	x(m)	y(m)
P1	2025	4040	2430	3250
P2	2500	3500	2730	2810
P3	2675	3800	3030	3120
P4	2130	4200	2770	3725

3.1.2 Comparison of RiverFlow2D Results with Field Data

Based on the interview of several eye witnesses of the flood, including a local police member, 21 maximum water elevation marks were collected. The 21 gauge estimates are compared with River Flo-2D results. The information collected by Alcrudo et al., 2007 is enumerated in Table 9

Table 9 Data of the surveyed research points.

			Observations	RiverFlow2D Results
Gauge	X (m)	Y (m)	Water Depth (m)	Water Depth (m)
1	2410	3290	18.25	17.81
2	2400	3335	8.50	10.96
3	2355	3315	7.50	9.42
4	2345	3380	7.00	8.63
5	2335	3175	0.20	0.04
6	2335	3420	5.50	5.59
7	2330	3365	6.00	5.47
8	2315	3450	5.00	5.96
9	2310	3590	0.00	0.00
10	2303	3255	4.00	3.44
11	2285	3425	2.00	2.11
12	2285	3500	5.50	6.07
13	2280	3280	2.75	3.28
14	2266	3550	2.00	1.93
15	2265	3400	0.00	0.00
16	2259	3530	3.50	3.46
17	2250	3440	0.00	0.00
18	2230	3525	0.00	0.00
19	2205	3445	2.50	3.12
20	2195	3440	2.00	3.12
21	2190	3485	0.00	0.37

Figure 24 compares surveyed water elevation marks and model results.

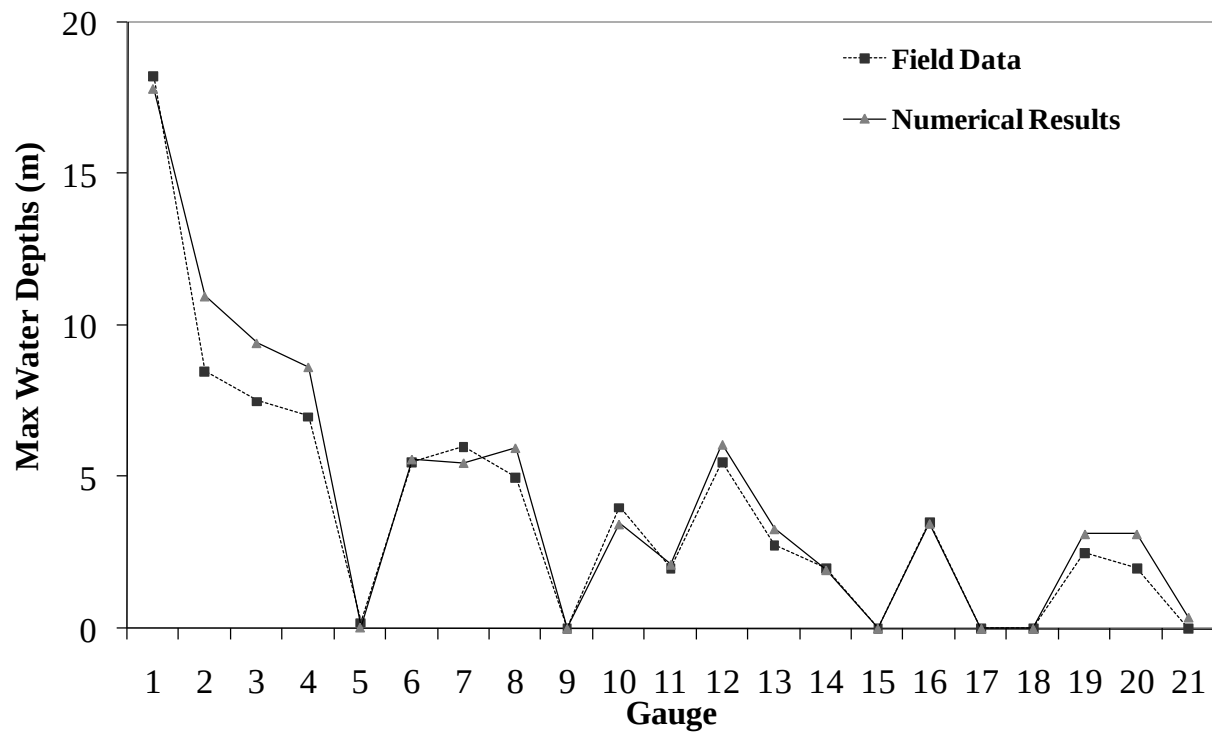


Figure 24 Tous dam-break test. Comparison of water elevation marks with numerical water surface depths for 21 locations along the Sumacarcel City.

Figure 25 show the velocity fields at two locations for time 11 h.

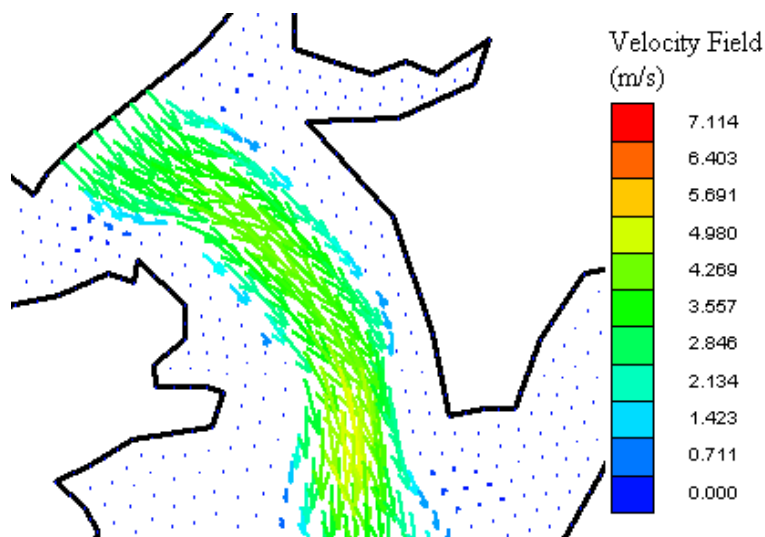


Figure 25 Tous dam-break test. Velocity field detail for view 1 T=11 h. (View 1).

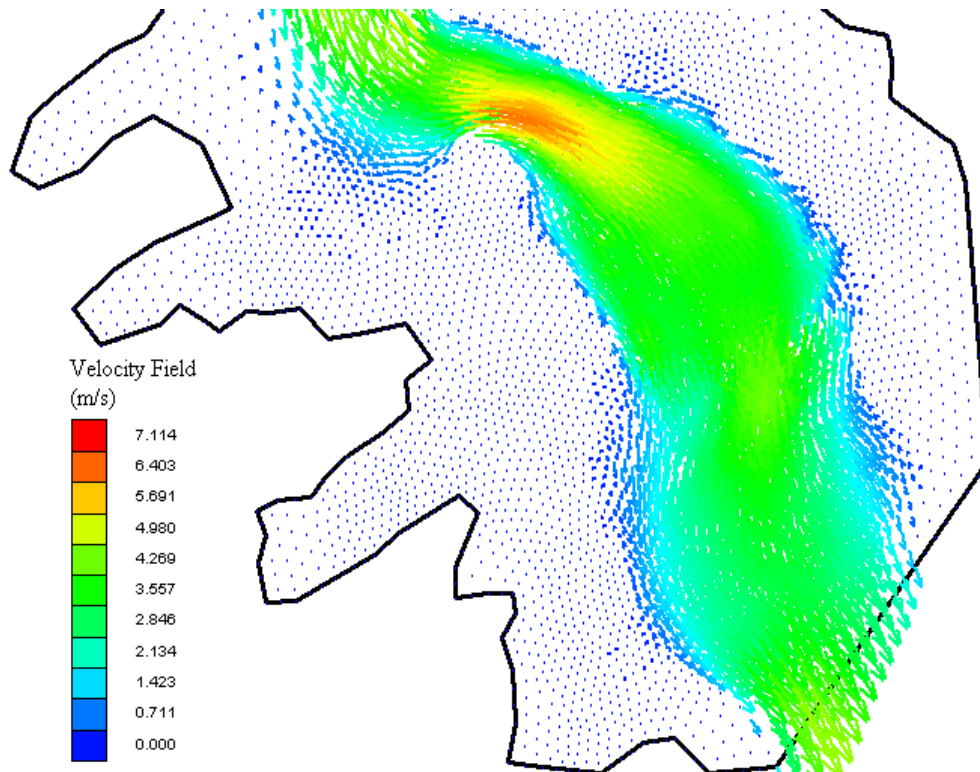


Figure 26 Tous dam-break test. Velocity field detail for view 2 at T=11 h (View 2).

Figure 28 shows the results of RiverFlow2D model compares with field data for Tous Dam-Break.

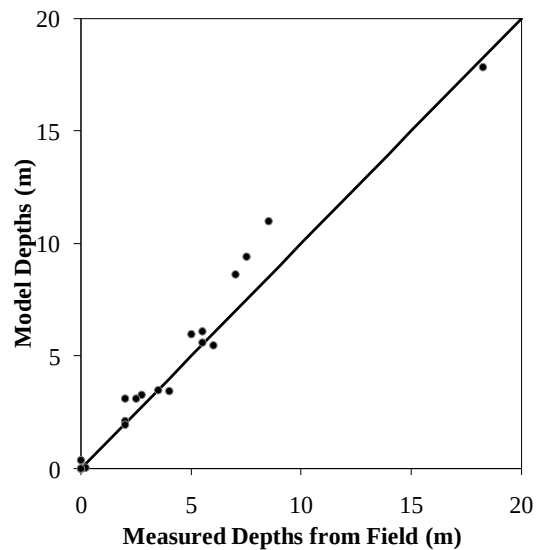


Figure 27 Comparison of the Numerical Results and Field Data for the gauging points.

3.2 Malpasset Dam-Break

An example of a Dam-break simulation is provided for the Malpasset dam. Malpasset dam was located in a narrow gorge of the Reyran River Valley, in France. Shortly after finalizing construction, the dam failed on December 2nd, 1959. The collected field data and information include estimates of the frontal wave pass inferred from the shutdown times of three electrical transformers that were located along the river valley, and a post-event survey that recorded 100 water elevation marks (Hervouet, 1999; Brufau et al. 2004).

In order to test RiverFlow2D for Malpasset dam-break event, we generated a 16150 node/30935 element mesh that extends for approximately 17.5 km (see figures 1 and 2). The total time of simulation was 0.7 hour with a time step of 0.5 s. Element sizes range from 20 m to 100 m approximately. The initial conditions were 100 m water surface elevation on the reservoir and dry bed downstream from the dam location. Manning's n was set to 0.03, constant for all elements.

3.2.1 Comparison of RiverFlow2D Results with Experimental and Field Data

Table 10 presents the electrical transformers shutdown times and the model frontal wave arrival times.

Table 10 Malpasset dam-break test. Data of electric transformers shutdown and RiverFlow2D frontal wave arrival times.

Transformer	X (m)	Y (m)	Measured shutdown Time (s)	RiverFlow2D Shutdown Time (s)
A	5550	6400	100	108
B	11900	5250	1240	1260
C	13000	4700	1420	1300

A survey was done for the local police showing nearly 100 points of water marks on the banks, the remarkable points surveyed are enumerated in Table 11.

Table 11 Data of the surveyed police points in the field.

Points	X (m)	Y (m)	Data Surveyed Water Level (m)	RiverFlow2D Results Water Level (m)
P1	4913.11	6244.01	79.15	84.69
P2	5159.75	6369.62	87.20	87.38
P3	5790.63	6177.76	54.90	57.18
P4	5886.54	6503.97	64.70	60.60
P5	6763.05	5429.6	51.10	50.47
P6	6929.97	5591.87	43.75	47.88
P7	7326.02	4948.78	44.35	38.81
P8	7441.01	5232.12	38.60	36.80
P9	8735.94	5264.61	31.90	31.91
P10	8628.60	5604.63	40.75	37.73
P11	9761.13	5480.36	24.15	25.41
P12	9800.00	4414.79	24.90	26.69
P13	10957.00	4651.94	17.25	25.55
P14	11156.99	5800.72	20.70	21.03
P15	11689.05	4592.36	18.60	18.94
P16	11626.05	5406.80	17.25	19.78
P17	12333.72	4269.74	14.00	16.44

Also in 1964 a non-distorted 1/400 scale model was constructed at National Hydraulic and Environment Laboratory (EDF-LNH, Chatou). The results for the gauges located in the physical model are showed in Table 12. Figure 28 compares surveyed water elevation marks and model results.

Table 12 Data of the gauges located in the physical model (EDF-LNH, Chatou)

Gauge	X (m)	Y (m)	Data Surveyed		RiverFlow2D Results	
			Water Level (m)	Time (s)	Water Level (m)	Time (s)
S6	4947.46	6289.71	84.2	10.2	86.16	10
S7	5717.30	6407.61	49.1	102.0	56.65	108
S8	6775.14	5869.23	54.0	182.0	54.79	216
S9	7128.20	51620	40.2	263.0	48.20	252
S10	8585.30	5443.08	34.9	404.0	36.82	396
S11	9674.97	5085.89	27.4	600.0	26.49	612
S12	10939.15	5044.78	21.5	845.0	20.08	864
S13	11724.37	4810.41	16.1	972.0	18.15	936
S14	12723.70	4485.08	12.9	1139.0	13.48	1260

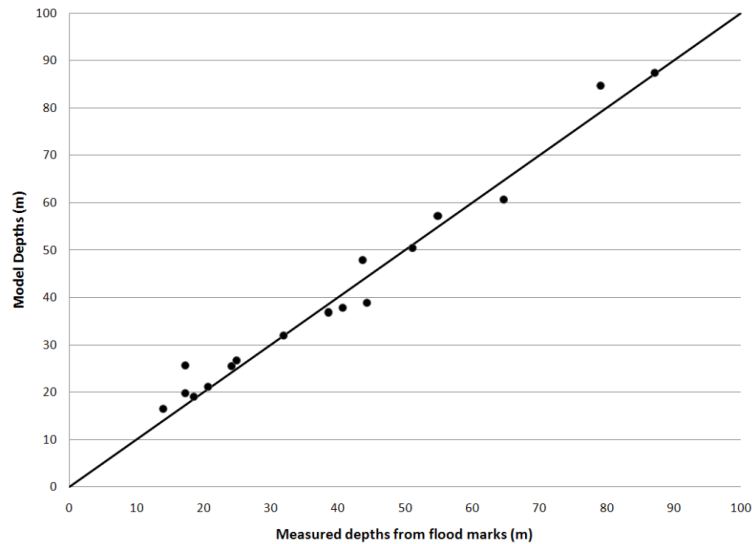


Figure 28 Malpasset dam-break test. Comparison of water elevation marks with numerical water surface elevations for 17 locations along the river valley.

Figure 29 and Figure 30 show the velocity fields at two locations for times 350 s, and 2400 s respectively.

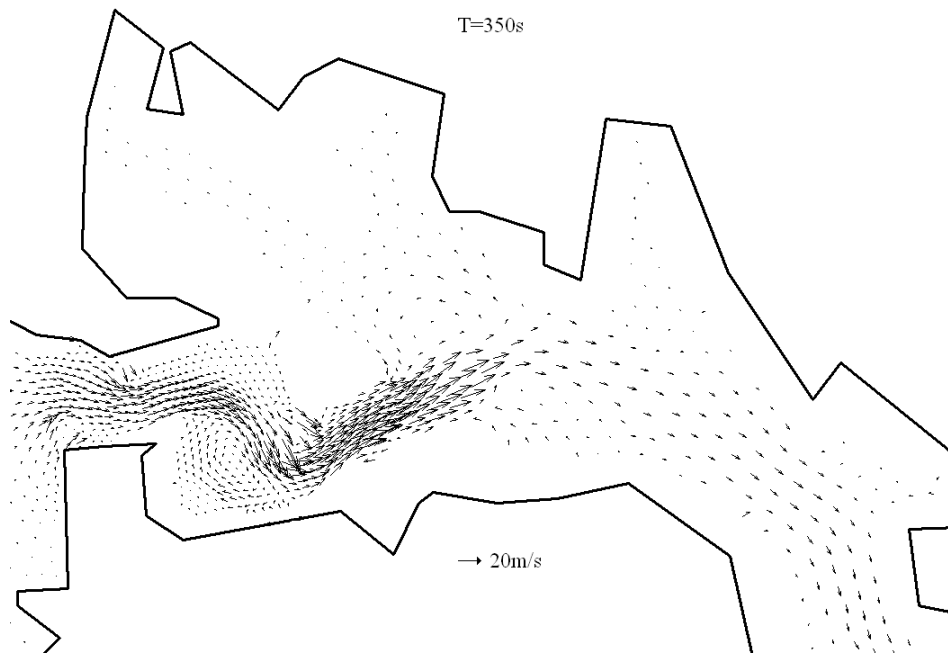


Figure 29 Malpasset dam-break test. Velocity field detail for view 1 $T=350$ s. after dam-break (View 1).

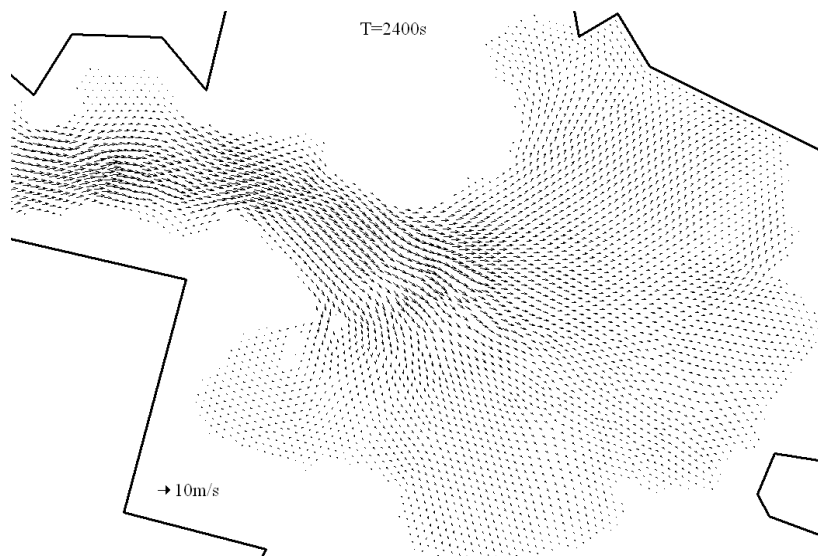


Figure 30 Malpasset dam-break test. Velocity field detail for view 2 at $T=2400$ s after dam-break (View 2).

Figure 31 shows the contour lines for depths at 350s, 900s, 1850s and 2400s.

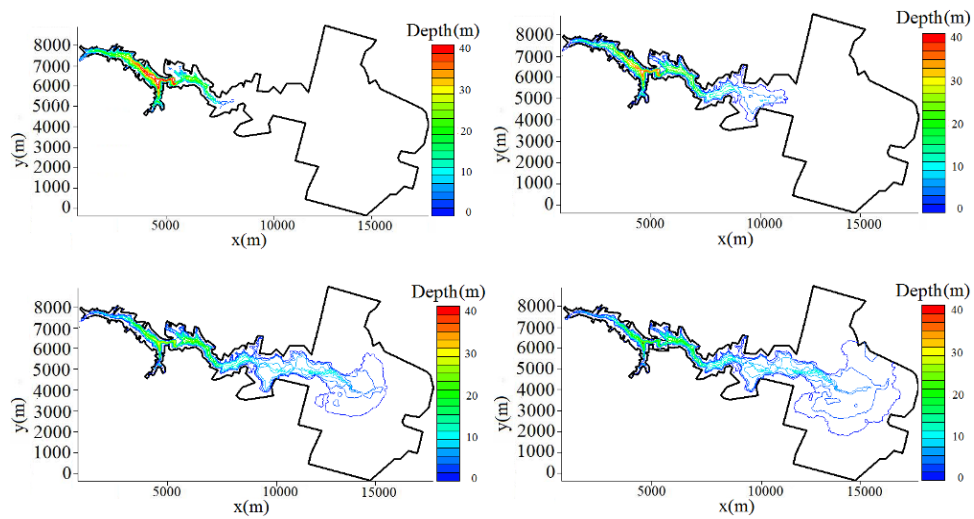
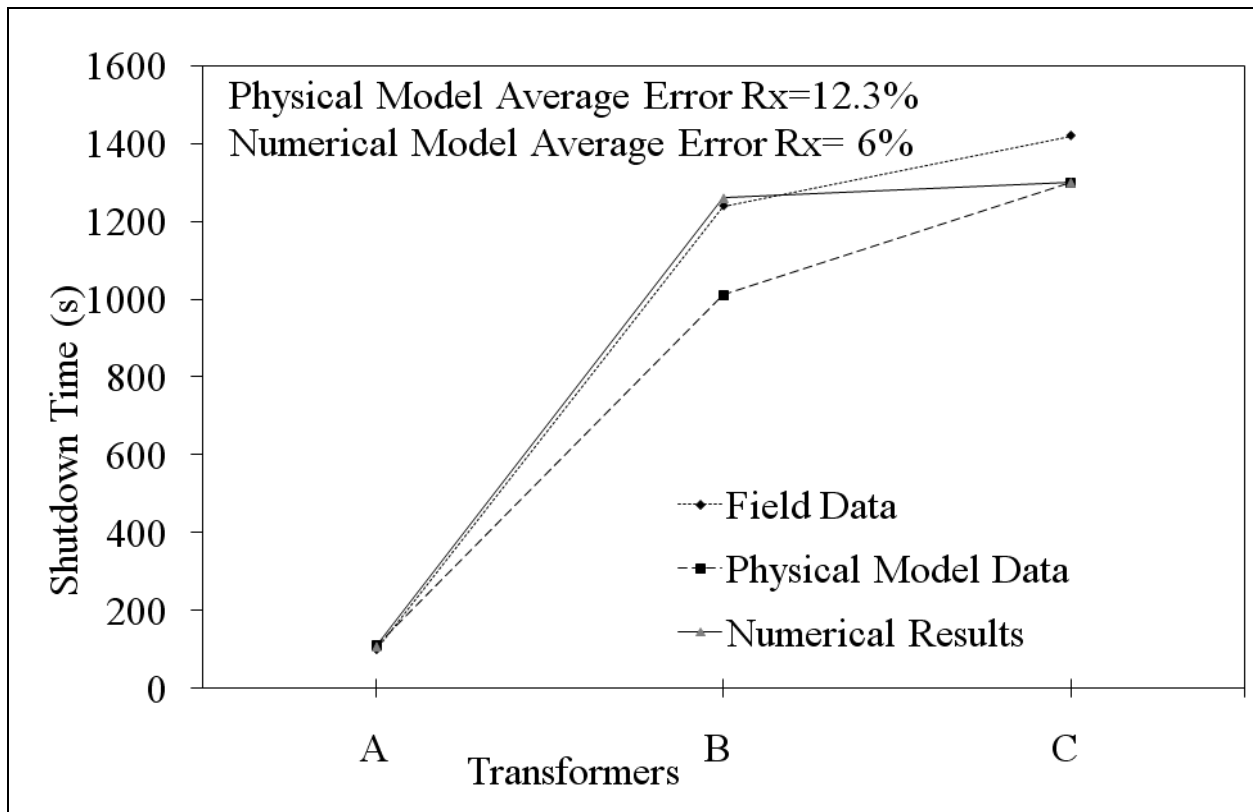
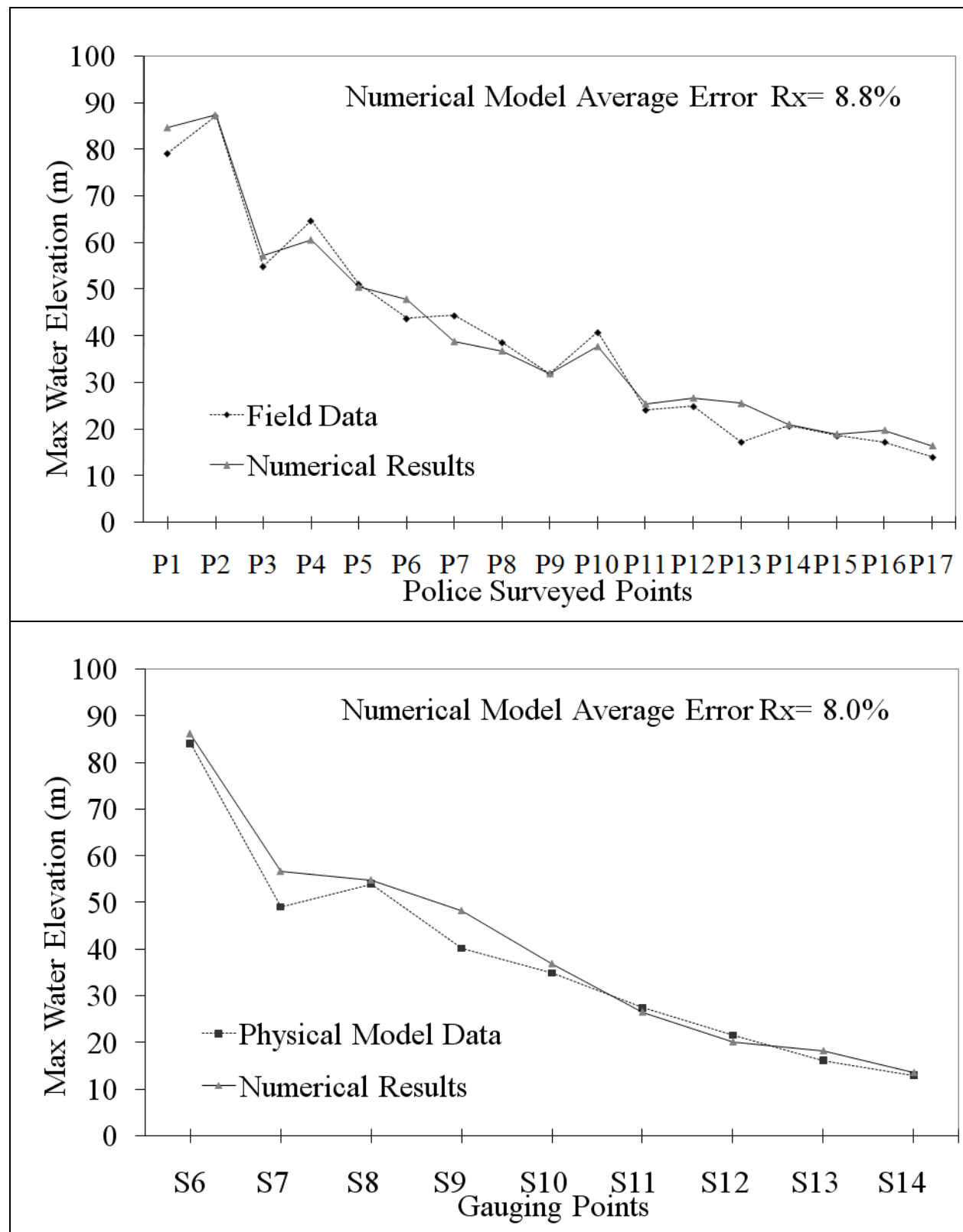


Figure 31 Contour lines for Depths of Malpasset Dambreak at T= 350s (a), 900s (b), 1850s (c) and 2400s (d).

Figure 32 shows the results of RiverFlow2D model compares with physical data and field data for Malpasset Dam-Break.





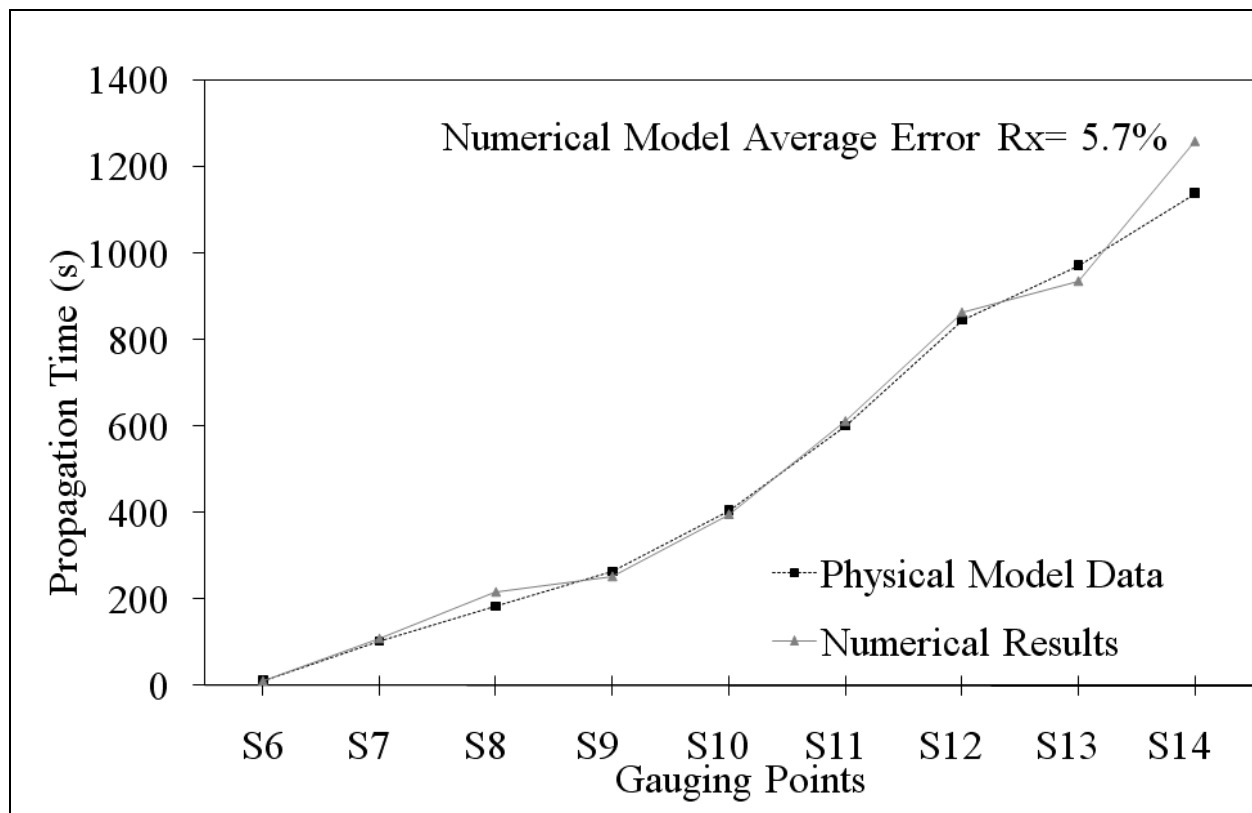


Figure 32 Comparison of the Numerical Results, Physical Data and Field Measures for the shutdown time of the Electrical Transformers along the river (a), the police surveyed points (b) and the gauging point (c and d).

3.3 Puyallup River

The Puyallup River drains the west and northwest sides of Mount Rainier in western Washington, USA. Almost the entire lower 28 miles of the Puyallup River is contained between levees and revetments that were constructed throughout the early half of the 20th century. While the levees are beneficial for flood control, their detrimental ecological effects are increasingly recognized. For the last several years Pierce County has prioritized providing improved flood protection and open space adjacent to the Puyallup River by removal of existing levees providing less than 100-year protection and building setback levees. Before reconfiguring the levees, the impact of the project is evaluated from multiple viewpoints. In some cases, understanding how the hydraulics of the river will change is best evaluated using a two-dimensional model.

In 2010, Pierce County tasked the team of Herrera Environmental Consultants and Northwest Hydraulic Consultants to design and evaluate a levee deconstruction / floodplain reconnection project. The project objectives included improving habitat diversity, reactivation of geomorphic processes that had been dormant and avoiding negative impacts to nearby homeowners such as increasing erosion rates or flooding risk. The team decided to use both a one-dimensional model and two-dimensional hydraulic model to assist in project design and impacts evaluation.

3.3.1 Hydraulic Analysis

Two hydraulic models were developed for this study; HEC-RAS which is a one-dimensional standard-step based model and RiverFlow2D which is a two-dimensional hydrodynamic and mobile bed finite element model. The HEC-RAS model was selected for its ability to quickly test multiple project designs and because it is the standard model used by federal agencies to demonstrate impacts to the water surface of a proposed project. RiverFlow2D was necessary to evaluate the detailed impacts of proposed logjams, hydraulic splits into low flow channels, and sediment transport potential. RiverFlow2D was selected over other 2D models because it is numerically robust; is relatively simple to create and refine the model mesh; and is well supported.

3.3.2 HEC-RAS Model

The HEC-RAS model encompasses 1.5 miles of the Puyallup River. Forty-seven cross-sections were cut from an existing conditions topographic surface. The cross-sections are spaced approximately 200-feet apart and extend beyond the extent of the mapped flood hazard area. The cross-sections are oriented perpendicular to the anticipated streamlines at flood conditions. Areas of similar hydraulic roughness were delineated during field investigations of the project site. Roughness values for these areas were assigned based on field observations and engineering judgment.

3.3.3 Implementation of the RiverFlow2D Model

RiverFlow2D requires a continuous topographic bed surface that represents both channel bathymetry and overbank areas. A high resolution existing condition surface was built and applied to the model. Aerial photography was used to delineate areas of similar bed roughness and the roughness values for these areas were assigned based on field observations and engineering judgment.

The model's computational mesh was developed using the topographic surface, bed roughness, and boundary conditions. The mesh was developed by first delineating model zones, including the river corridor and overbanks, and then breaklines along the top of the left and right bank levees. These breaklines ensured that the crests of levees were well represented in the model. Mesh density values were then assigned to the modeling zones and the breaklines. The overbank floodplain was meshed at a higher density than the uplands areas. An even higher mesh density was used along the river corridor and the levee breaklines.

3.3.4 Model Calibration

Two sets of observed water surface elevations were available to calibrate the hydraulic models. High water marks (HWMs) were recorded by the USGS after the historic February 1996 flood. In addition, the observed water surface during the bathymetric survey was used for calibrating the model. The estimated flows in the Puyallup River for the calibration events are 19,940 cfs and 450 cfs, respectively.

Figure 33 shows the comparison of the February 1996 HWMs to the RiverFlow2D, HEC-RAS, and FEMA FIS water surface profiles.

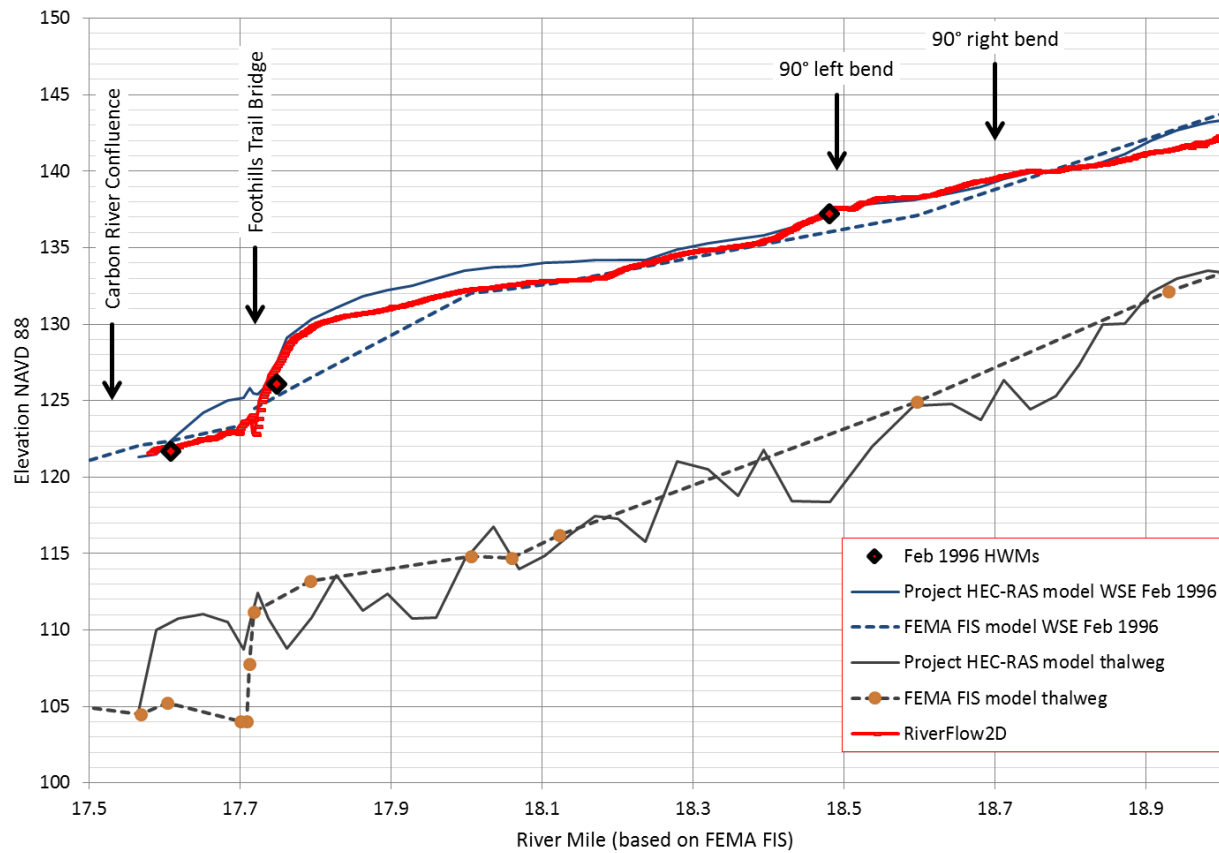


Figure 33 Comparison of Project RiverFlow2D with HEC-RAS Model, FEMA FIS model and High Water Marks.

Comparison of the RiverFlow2D model with HEC-RAS and observed water surfaces during bathymetric survey is shown in Figure 34.

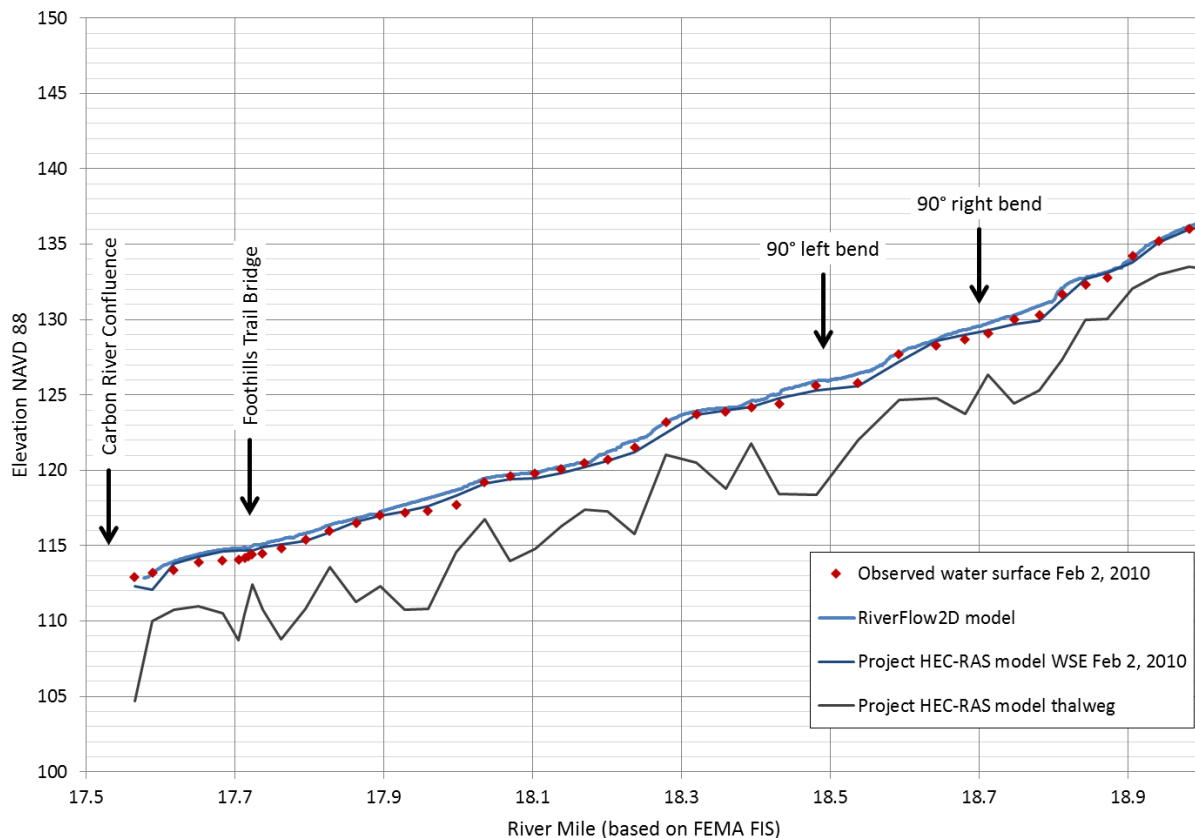


Figure 34 Comparison of RiverFlow2D with HEC-RAS and Measured Water Elevations.

Calibration of the RiverFlow2D model concentrated on replicating the simulated water surface profiles from the HEC-RAS model. Steady-state simulations were made and compared to the one-dimensional model. Calibration was achieved by slightly adjusting the model roughness parameters and increasing the number of mesh elements. A comparison of the 2D and 1D model water surface profiles is shown in Figure 34.

Profiles generally match in slope and gradient changes. The maximum difference is approximately 1.5 feet for the 100-year event, but most differences are significantly less. The match in profiles is impressive considering each model calculates water surface elevations using completely different methods.

3.3.5 RiverFlow2D Model Results for Proposed Conditions

The RiverFlow2D model was used to examine flow velocities and patterns, water surface elevations, bank and levee overtopping, and overbank flow in the vicinity of the project site for a wide range of flows under existing conditions. Proposed 30%, 60%, and 90% designs were modeled and evaluated for achieving the project objectives. The RiverFlow2D model was used to evaluate the designs as it could better replicate the intricacies of each alternative which included multiple low flow channels, numerous logjams, and revetments of various design.

In general, the alternatives were simulated by modifying the left bank floodplain topographic surface, changing overbank roughness coefficients, and by approximating logjams and revetments. Logjams and revetments were simulated by raising mesh nodes in the vicinity of each feature to their design height. Logjams were typically represented by 8-15 nodes. The 90% design achieved goals of maintaining flow in the excavated side channel during low discharges while having negligible impact on the surrounding floodplain. Figure 36 illustrates that the proposed 90% design only causes a rise in 100-year water elevations immediately upstream of certain engineered

logjams. This localized response is limited to approximately 50 feet upstream of each logjam. There is no increase in base flood elevation within the main channel or outside the project limits.

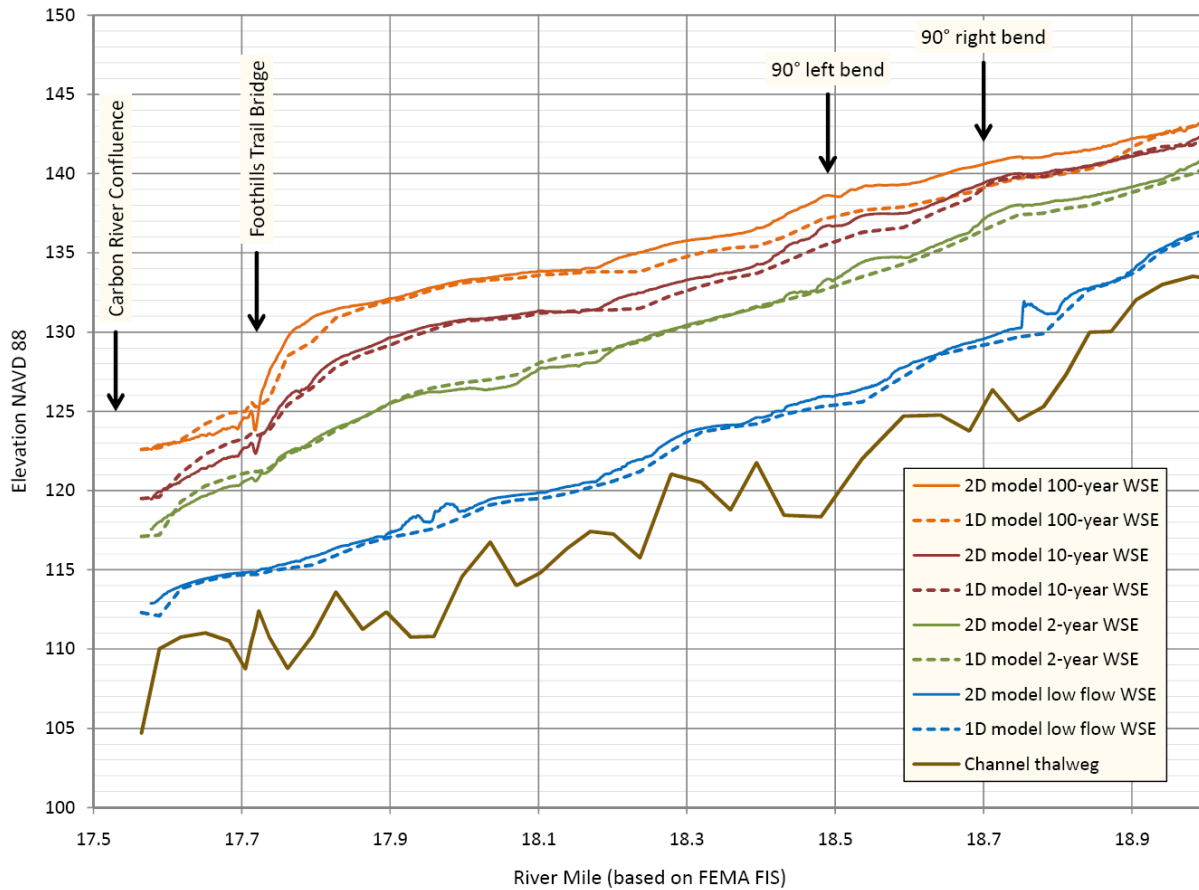


Figure 35 Comparison of HEC-RAS RiverFlow2D.

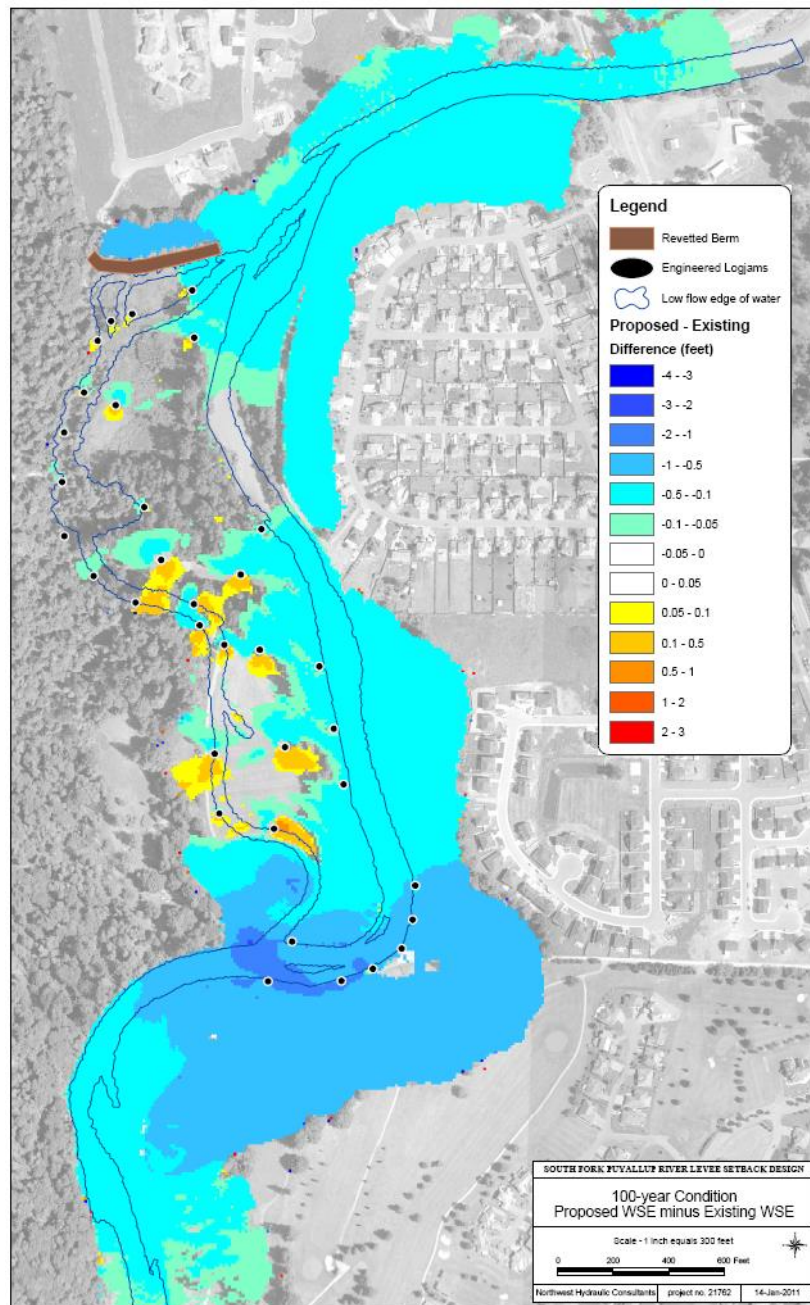


Figure 36 Water Elevation Differences Between Proposed and Existing Condition Scenarios.

3.3.6 Conclusions Regarding RiverFlow2D

The RiverFlow2D model proved numerical robust and was well calibrated to observed data and the HEC-RAS results and therefore well suited to evaluate the existing conditions and proposed floodplain reconnection designs.

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