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Comparison of observations by flood plain residents with results from a two-dimensional flood plain model: Waihao River, New Zealand

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Abstract

The two-dimensional computer model 'Hydro2de' is used to analyse flood flows for the Waihao River flood plain, Canterbury Plains, South Island, New Zealand. In the computer model the ground topography, based on a Digital Terrain Model, is described by a set of nodes on a grid and the flow direction is determined as part of the solution of the two-dimensional shallow water equations. The model flood levels, calculated for floods in March 1986 and March 1994, are compared with field data based on information, photographs and videos obtained from flood plain residents some years after the events. The results are also compared with maps of the extent of flooding prepared by the local authority after the events. The modelled flood levels calculated for the edge of the flows were close to observed levels, but calculated levels in the centre of the water flow were low because the flood plain model did not include details such as buildings and fences.

Keywords: flood plain, finite volume, two-dimensional, Waihao River, computer modelling, flood level.

Introduction

General overview

The public need to know where flooding is likely to occur when they build or buy a dwelling or business on a flood plain. Well-documented past floods can give a great deal of information for a given site, however data may be available for only one, or a few, events. Better information can be provided by combining site information with:

a) hydrological information to determine frequencies of flooding,
b) geomorphic studies, discussions with river control staff and historical data to assess possible river breakout locations and flow paths and,
c) computer modelling techniques to model the breakout flows on the flood plain.

If there is no past flood information the above process is necessary to provide information. This paper describes a two-dimensional computer model, for modelling the flows of floodwater on a flood plain, and compares the results with observations by flood plain residents.

Computer modelling of flood plains

Present computer models in widespread use are one-dimensional and require the flood plain to be defined as a network of channels, each defined by a set of cross-sections along a channel. There are many areas on flood plains and in river systems with complex topography where it is difficult to define the channel network, because it is difficult to determine the direction of water flow. There may be a significant component of the flow in the direction perpendicular to the direction of flow chosen. This has been recognised by many authors (Cunge *et al.*, 1980; Labadie, 1994; Ligget, 1987; Zhao *et al.*, 1994).

Figure 1 shows examples of problem areas. The flow will diverge from a breakout at Area A. Here a channel cross-section needs to be curved or many channels need to be defined, so that the flow is at right angles to them. However it is difficult to define the degree of curvature and how the flow will spread out. At Area B the water can change direction. Initially a breakout from the river adjacent to Area B would result in water flow in the direction of the right arrow at Area B. Afterwards, overflows from breakouts upstream could result in the flow direction changing (left arrow).

Two-dimensional computer models, by solving the two-dimensional shallow water equations, calculate the water flow direction as part of the

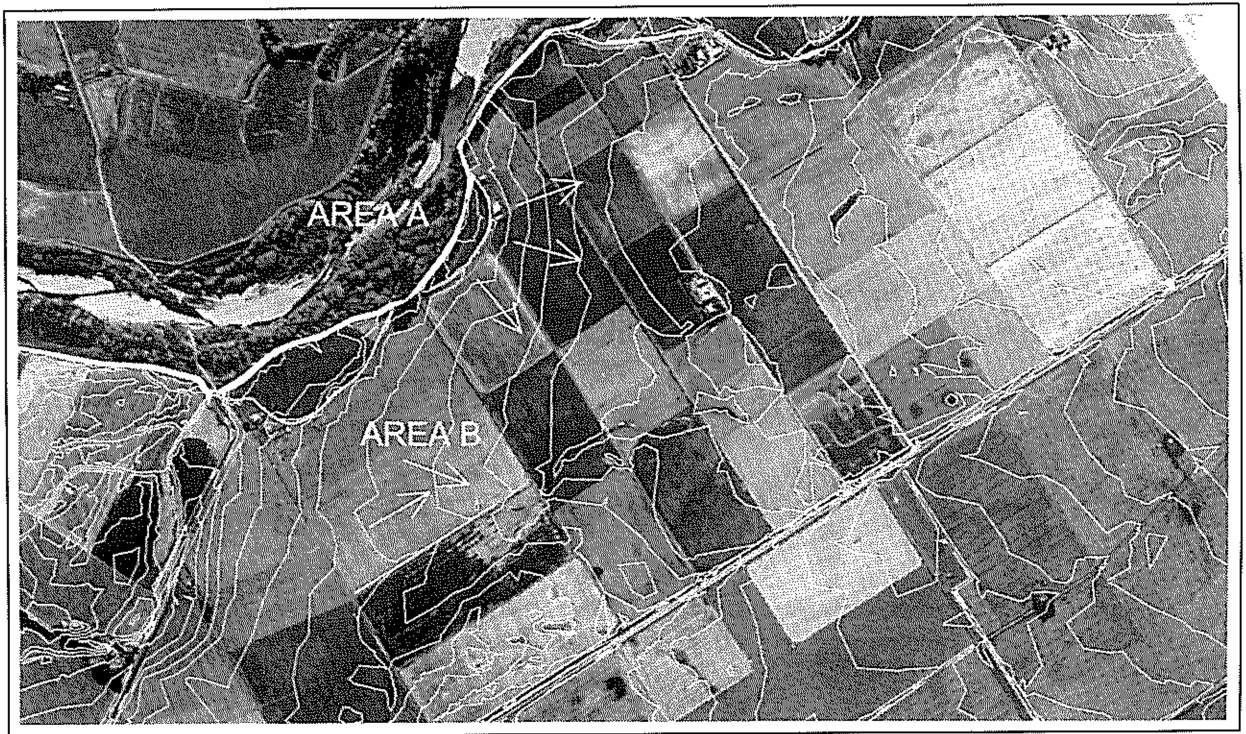


Figure 1 – Contour plan of part of the Waihao River flood plain at 0.5 m intervals, showing the difficulty of defining channels with one-dimensional models.

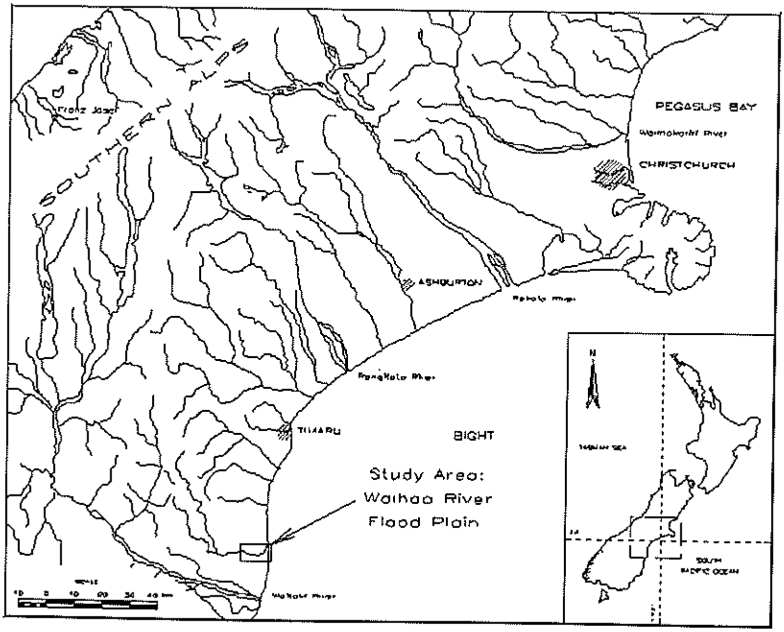
analysis. Operational two-dimensional computer models have recently been developed (Beffa, 1994). Personal computers are now sufficiently powerful to run these models, and computerised photogrammetry techniques available can now produce a Digital Terrain Model (DTM) or computer model of the ground surface of a flood plain at a realistic cost.

Case studies are needed to verify their accuracy and ability to model flood plain flows. Little work has been done in this area: analyses have used limited data points (Zhao *et al.*, 1994; Bechteler *et al.*, 1994) or compared percentages of the flood plain inundated (Bates *et al.*, 1994). A more comprehensive study using many data points from historical floods is necessary to validate this modelling technique.

Case study: Waihao River flood plain

The Waihao River is situated on the east coast of the South Island, New Zealand, about 200 km southwest of Christchurch (Fig. 2). The Waihao River flood plain has been chosen for study because a Digital Terrain Model was available for the Waihao River flood plain, and there have been two recent floods on the flood plain, in March 1986 and March 1994.

Figure 2 – Study area: Waihao River flood plain.



The model results can therefore be verified using flood level information for these events from local residents, from their videos and photographs, and from flood flow and level information from the Canterbury Regional Council. This approach to collecting field data has not been attempted before in New Zealand.

Theory

The computer model Hydro2de (Beffa 1994, 1996) solves the two-dimensional shallow water flow equations. These equations model water flow in the two horizontal directions of an area. The two-dimensional equations in Cartesian co-ordinates as described by Jansen *et al.* (1975) are:

(a) Continuity of mass

$$\frac{\partial z_w}{\partial t} + \frac{\partial}{\partial x}(h\bar{u}) + \frac{\partial}{\partial y}(h\bar{v}) = 0 \qquad (1)$$

where z_w is the water level, h is the depth of flow, $\bar{u}$ and $\bar{v}$ are the depth-averaged velocities in the x and y directions respectively and t is time.

In this equation the change in water level with time is balanced by the change in flow in the x-direction and the change in flow in the y-direction.

(b) Continuity of momentum in both flow directions

$$\frac{\partial}{\partial t}(h\bar{u}) + \frac{\partial}{\partial x}(\bar{u}^2 h) + gh\frac{\partial z_w}{\partial x} + \frac{\partial}{\partial y}uvh + \tau_{xb} + h\frac{\partial \tau_{xy}}{\partial y} + h\frac{\partial \tau_{xx}}{\partial x} = 0 \qquad (2a)$$

$$\frac{\partial}{\partial t}(\bar{v}h) + \frac{\partial}{\partial x}(\bar{u}\bar{v}h) + gh\frac{\partial z_w}{\partial x} + \frac{\partial}{\partial y}(\bar{v}^2 h) + \tau_{yb} + h\frac{\partial \tau_{yx}}{\partial y} + h\frac{\partial \tau_{yy}}{\partial x} = 0 \qquad (2b)$$

where τ_{xx} and τ_{yy} are the normal stresses due to turbulence, τ_{xy} and τ_{yx} are the shear stresses due to turbulence, and τ_{xb} and τ_{yb} are bed shear stresses that are estimated using Manning's friction law.

Numerical techniques

Numerical techniques are used to solve these equations, as analytical methods can not. Hydro2de uses the Finite Volume Method, a development of the finite difference technique (Hirsch, 1988). This method balances all the fluxes entering and leaving each cell, using explicit time integration.

For areas that are initially dry, the calculation can proceed at only one space step per time step, because the ground topography changes in slope between each node on the calculation grid. The program therefore uses an explicit scheme, as it requires fewer calculations per time step than an implicit scheme (Beffa, 1994). The numerical fluxes are estimated according to Roe (1981). Accuracy of a second order is attained using a variable extrapolation (MUSCL) approach (van Leer, 1977). The equations are solved in conservation form and thus are valid even where hydraulic jumps occur.

With numerical techniques, it is difficult to know if the answers are correct. For some simple cases (e.g. a dam-break wave on a frictionless plain) there are analytical solutions that can be compared with the numerical solution. For practical applications the only way to test a model of the kind used here is to compare the results with field data.

Gathering of flood plain information

Survey of flood plain residents

All 40 residents of the Waihao flood plain were visited on their properties by the first author. After he explained the project and its value to flood modelling of other flood plains that have not been flooded, and discussed flooding on their property, the residents described all the flood levels that they could recall. Generally they remembered levels in buildings and at other easily identifiable locations. They also provided videos and photographs that were used to verify, where possible, the information obtained from the residents' recollections. They were asked how accurate the levels were, and an independent assessment was also made of the data's accuracy. Most levels were estimated to be within ± 0.20 m. All levels indicated by the residents were pegged in the field.

Photographs provided by the residents were also a valuable source of flood levels. The main problem was attempting to establish the time of the photograph. The most useful photographs were those taken after the peak, as they often illustrated a peak debris level (Fig. 3). Series of

Figure 3 – Debris levels on a fence line – the photograph is blurred because it was taken off videotape.

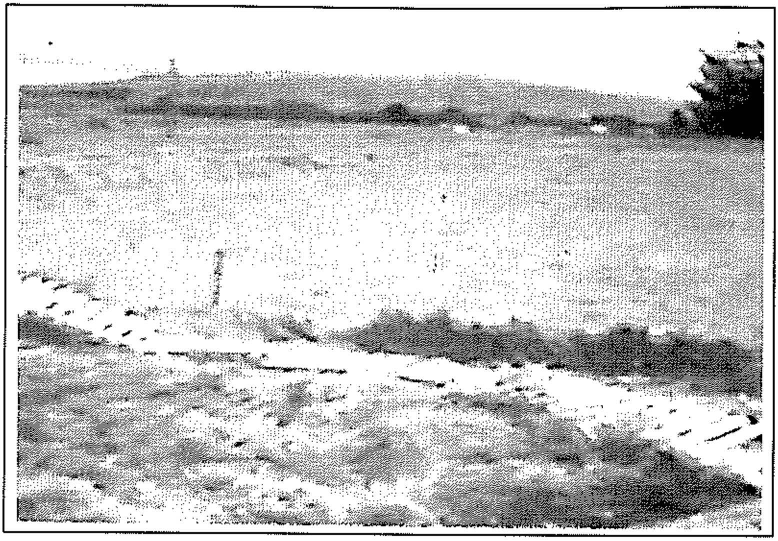
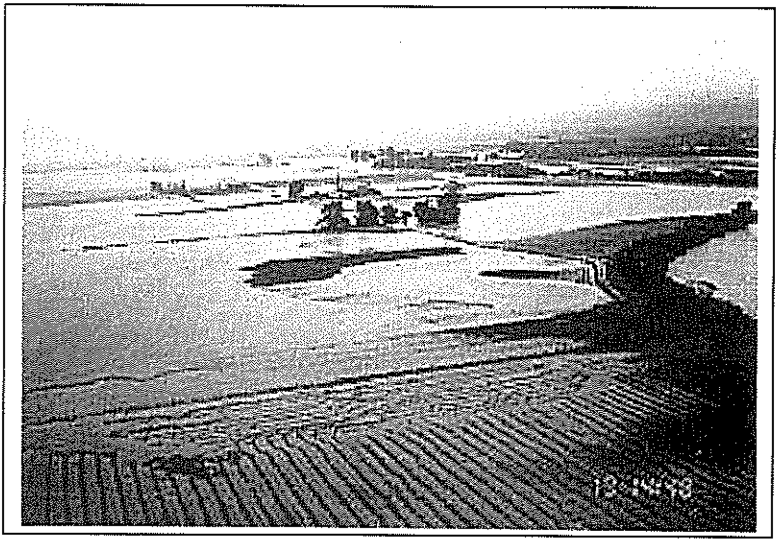


Figure 4 – Aerial photograph of 1994 flood.



photographs taken during the flood could be used if the resident knew the time they were taken, or if the level of one of the photographs could be related to the peak flood level. Figure 4 is an oblique aerial photograph of the flooding; the time can be seen in the bottom right corner. Flood levels at features could be determined to within ± 0.15 m. In some cases the position of flood levels on the photograph could be determined to within a few metres by lining up background features. Using these and similar techniques a series of flood levels was assembled from the photographs and videos.

An error was estimated for each flood level. The averages of these estimated errors were ± 0.133 m for the 1986 flood and ± 0.146 m for the 1994 flood.

Canterbury Regional Council also mapped the limits of the flooding after both the 1986 and 1994 floods. Figure 6 shows the flood extent map and the flood level positions for the 1986 flood. These were compared visually with the model analyses.

Field Global Position System survey

The field survey used a Trimble 4000SSE real-time kinematic Global Positioning System (GPS) to map the flood level pegs based on the residents' information. The photographs, and photographs of the video data points (Fig. 3), were taken into the field and used for the survey. As the flood level points were widely spaced, using a GPS made surveying the flood levels cost-effective. A conventional survey would have taken about two months, as it would have required many intermediate control points.

The survey team consisted of three people: two survey technicians, and the first author present to identify the pegs indicating the flood levels. The team took 7 days in the field to survey the 450 levels from both floods and necessary points for control over the 30 km^2 of the flood plain. The survey experienced some problems with the radio link to the base station. Without this problem, the fieldwork would have taken 5 days.

Levels were measured to ± 0.02 m. The uncertainties of ± 0.017 m between the plane of the survey datum of the DTM and the plane of the satellite datum, and in positioning the staff of the GPS to the height of the level of ± 0.01 m, increased this error to ± 0.03 m. The results of the survey were put into the ARC/INFO Geographic Information System [GIS] (Environmental Systems Research Institute). Table 1 shows examples of the levels obtained.

Table 1 – Observed flood levels (part of 1986 flood data)

Final list number	Mark in number	Photograph Number or Video (V)	Description	Reduced (m)	Estimate of accuracy (m)	Time	Time compared to peak	Accuracy of time	Source of level
1	2		Flood mark in pump house to the south west of Dickson house	36.262	± 0.025	Peak			M. Dickson
2	5		M.Dickson silo – 50mm under floor	35.91	± 0.05	Peak			M. Dickson
3	6		M.Dickson shed south of house – flood mark inside	36.217	± 0.10	Peak			M. Dickson
4	7		Wool press in woolshed 30m south of Dickson house – flood mark	36.095	± 0.025	Peak			M. Dickson
5	9	86-76	Edge of flooded area to centre right of photo on fence line	31.295	± 0.15	8.30am	4.5hrs before	within 1 hr	L.Paul - photograph
6	10	86-76	Edge of flooded area before hedge near left fence	31.884	± 0.20	8.30am	4.5hrs before	within 1 hr	L.Paul – photograph
7	11	86-77	Level on fence in centre of photo	32.254	± 0.15	8.30am	4.5hrs before	within 1 hr	L.Paul – photograph
8	12	86-78	Level at the edge of the dry piece in	32.727	± 0.05	8.30am	4.5hrs before	within 1hr	L.Paul – photograph

Flood hydrographs

The only hydrograph available was derived from data from a water level recorder (administered by the South Canterbury Catchment and Regional Water Board for the 1986 flood and the Canterbury Regional Council for the 1994 flood) 10 km upstream of the study area.

The recorder was washed out in the March 1986 flood and it was impossible to gauge the river. Therefore the hydrograph was constructed using a slope-area calculation to estimate the flood peak, estimates of the time of the flood peak from the local residents, and the hydrographs of other rivers in the district to estimate the correct shape.

The recorder failed again in the March 1994 flood. This time a Canterbury Regional Council officer observed the flood levels on a staff gauge on the bridge 200 m downstream of the recorder site. The peak discharge was again calculated using the slope-area technique.

Stopbank overtopping and breach information

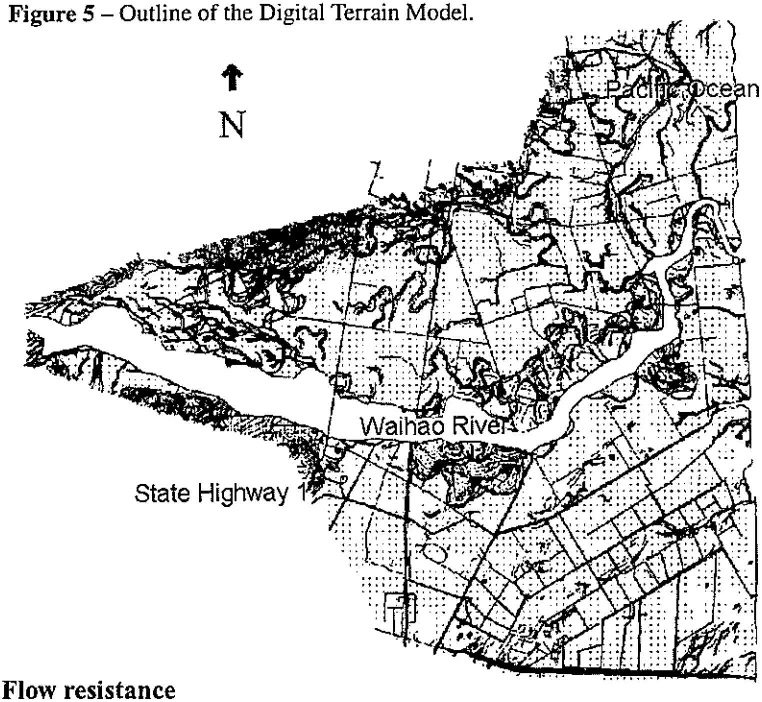
After the floods Catchment Board or Regional Council staff surveyed the river, including flood levels, the position and length of the stopbank breaches, and the lengths and heights of overtopping.

Flood plain description

Digital Terrain Model

A Digital Terrain Model (DTM) of the flood plain was developed using aerial photogrammetry by a commercial survey company, Australian Aerial Mapping. The DTM was formulated only for the flood plain, as either vegetation or water covered most of the river area inside the stopbanks. The specification for this model required that it describe the ground levels to within ± 0.3 m. It had over 90,000 spot height points, a maximum grid spacing of 100 m, and 6,000 breaklines that highlighted the ridges and valleys on the flood plain. Figure 5 shows an outline of the points and breaklines of the DTM. ARC/INFO (Environmental Systems Research Institute) was used to compare the DTM levels with the ground levels at the field data points. This showed the DTM to be within the specification, with a standard error of ± 0.264 m. This analysis also showed that groups of points (with 95 % significance) were either high or low, indicating that these areas of the DTM were either high or low. These areas occurred because the method used in DTM construction is based on control points to calibrate the photogrammetric analysis. The high and low areas tend to be those most distant from the control points.

Figure 5 – Outline of the Digital Terrain Model.



Flow resistance

The flood plain modelling used Manning's 'n' as the resistance coefficient for the water flow. A layer in ARC/INFO was used for the coefficient value over the flood plain. Aerial photographs were scanned into ARC/INFO and used as a background to digitise areas of different resistance values. The values initially chosen were based on literature (Henderson, 1966; Chow, 1959) and on one of the authors' previous work in calibrating floods, and are shown in Table 2.

Table 2 – Manning's 'n' values used for the flood plain

Description	**Mannings 'n'**
General pastoral farm land (grass and fences)	0.05
Areas of trees	0.125
Hedges	0.125
Crops	0.07
Roads	0.03

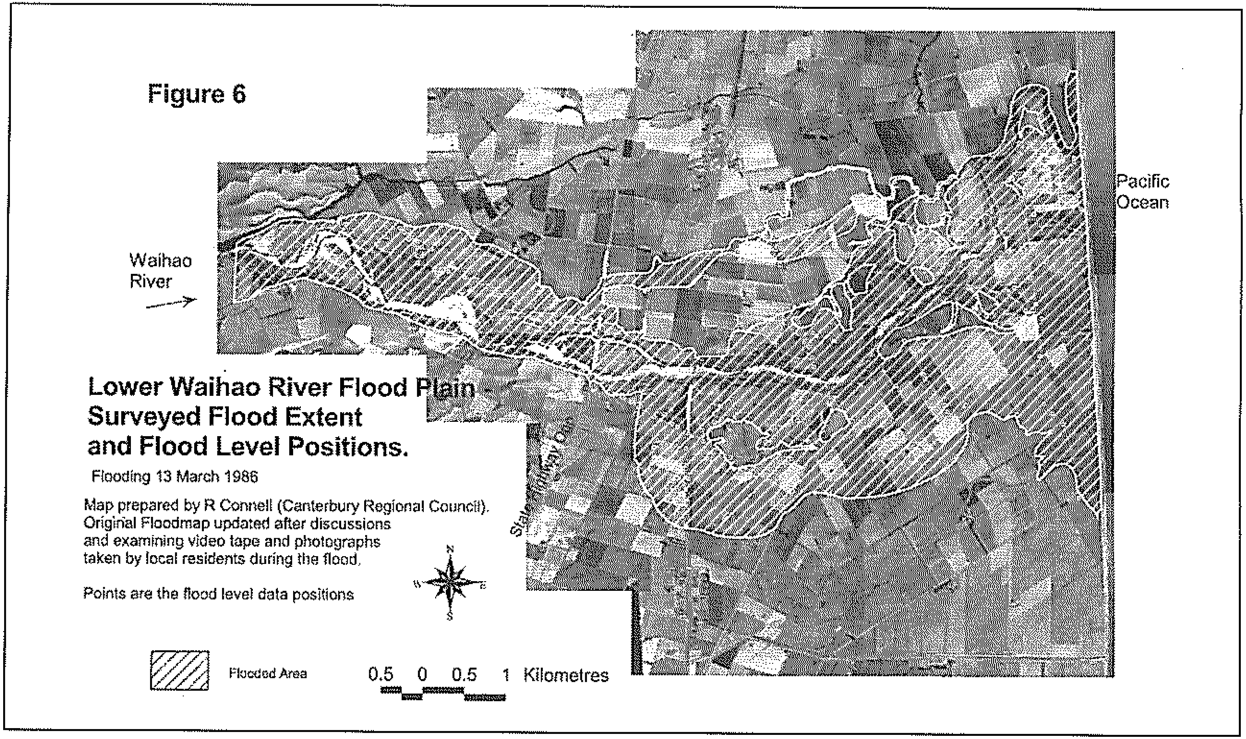


Figure 6 – 1986 flood: surveyed flood extent and flood data points.

The 'n' value for general pastoral land is high, to allow for the effect of fences on flood levels (many levels were along fence lines). A value of 'n' for open ground would be nearer to 0.03 – 0.035. The value for crops is an average and in most cases crops were harvested, except for potatoes; the resistance for this crop varies depending upon the depth of flow.

Two-dimensional flood plain analysis

The two-dimensional program Hydro2de (Beffa 1994, 1996) was used for the analysis. A two-dimensional computer model requires the flood plain topography to be represented as a set of nodes f(x,y) in either a uniform or non-uniform grid. Hydro2de uses a uniform rectangular grid. ARC/INFO was used to extract uniform grids of the topography, from the DTM 'TIN' (Triangular Irregular Network) or computer model of the ground surface, and resistance values from the 'TIN' of the Manning's 'n' layer.

Calculation of discharges onto the flood plain

Ideally, it would have been preferable to combine the flood plain and river, and run the model with the inflow determined from the river hydrograph at the upstream end of the study area. The model could have been run until the stopbanks were overtopped in the model. From R.Connell's experience, the stopbanks in this region breach when the overtopping reaches 150 mm. When the overtopping reached this level, a breach could have been inserted into the model. This procedure could have then been repeated until enough breaches were inserted into the model that no further overtopping occurred, allowing simulation of the whole flood.

This was possible for the reach of the study area above State Highway One. Here the river is not stopbanked, so water can flow out of the river onto the flood plain. In this area a DTM of the river was inserted into the DTM of the flood plain. The DTM of the river was constructed using the average parameters of the river cross-section data, and aerial photographs of the river.

However, for the stopbanked area below the river, a complete model of the river and flood plain was not possible, as it would have taken too long to run the model.

Selecting a grid size for the analysis

A 5 m grid size would have been required to obtain a reasonably accurate estimate of breach size, as the breaches varied from 30 m to 100 m in length. A model using a grid of this size however would have required 2560 hours for each run on the Pentium 166 computer used for most of this work. A 20 m grid size was finally chosen, allowing a set of runs to be completed in 40 hours and therefore several re-runs could be undertaken. Halving the grid size increases the calculations required 8 fold, as the time step is reduced by a half, as well as the space steps in both directions.

This meant that stopbank breaches had to be modelled individually.

Stopbank breach and overtopping models–transcritical flow

It was very likely that super-critical flow would occur through the stopbank breaches. Hydro2de can handle both sub-critical and super-critical flow and the transition between the two, transcritical flow (Beffa, 1996). The stopbank overtopping was also modelled with Hydro2de to obtain the expected flows for the range of overtopping heights. These discharges were then put into the models of the flood plain in this area. The final hydrographs of the outflows onto the flood plain were constructed using the results of:

i) The stopbank breaching and overtopping analyses.

ii) The hydrographs of the floods on the river recorded at the waterlevel recorder site 10 km upstream of the flood plain. These were routed to the study area using MIKE11, a one-dimensional unsteady-state numerical model (Danish Hydraulic Institute).

iii) The times of the breakouts and overflows indicated by the residents, photographs and videos.

iv) Calculations for the river flood backwater curve, carried out after the floods by the staff of the South Canterbury Catchment and Regional Water Board using CHANEL, a backwater curve program from the Otago Catchment and Regional Water Board.

Flood plain models

The analysis divided the flood plain into four calculation areas: one for the upstream area that was not stopbanked, one on the north side, and two on the south side of the stopbanked area.

Modelling overtopping and breach flows

Hydro2de uses a uniform grid that is rectangular in shape. The Waihao River flood plain is irregular in shape, so higher flood-free areas around the flood plain were used to fill out the rectangle. For the downstream end of an area, either a weir or water level condition was inserted into the model. At the ocean a lower area was placed into the model well below sea level with a water level condition for the sea level. The breaches were modelled by placing virtual channels from the edge of the calculation area to the DTM, i.e. through the higher areas. These channels were the same width as the breach, to the nearest 20 m, and entered the flood plain at its local level. The overtopping was modelled in a similar manner, except that a weir was placed at the edge of the flood plain over the length of the overtopping.

Flood plain features

Before the grids were adopted for Hydro2de, they were checked to ensure they did not omit significant surface features.

The checks were done in ARC/INFO using two layers (the contour map showing the features, and the position of the grid points). Where a feature was missed, the points were altered, one by one, using adjacent points closest to the feature. For features less than 20 m in width, the Manning's 'n' value of the point was adjusted to give the approximate conveyance (Hydro2de now has an option to do this correctly).

Because of the transcritical flow ability of Hydro2de, terraces and other positions where super-critical flow could occur did not need to be separately defined in the model.

Comparison with flood observations

After an initial run of the models, 34 of the surveyed flood level points were discarded. All of these points were from photographs. Some data points were not at or close to the peak of the event. These points could not be readily compared to the model, as the hydrograph shape was not known accurately because the water-level recorder had been destroyed during the flood. For some photographs the time was not known accurately.

Other data were discarded because the wrong position was surveyed, the data did not fit with other data in the same area (this sometimes occurred when the data was for a point off a resident's property), or the Regional Council staff could not locate the data points.

Results

Initial results

For both floods, the models were first run with the discharges calculated by the overtopping and breaching analyses. Table 3 gives the results of these initial runs.

Table 3 – Differences in the modelled and observed flood levels

Flood	**Differences in the modelled and observed flood levels (m) The latter figure is the standard deviation of the data.**
1986 flood	-0.252 ± 0.275
1994 flood	-0.082 ± 0.314

The average values of levels in the initial model runs were below the observed flood levels. Therefore further runs were undertaken to produce a better match between the average values for the computer model and the observed levels.

Adjusting the discharges: final results

As flow velocity data were not available for the two floods, velocity data could not be used to validate the Manning's 'n' values or discharges used in the analyses. A best fit to the levels will validate only the product of these two parameters. Therefore discharge only was used to calibrate the model with the observed flood data.

Model runs were continued until the breakout and overflow discharges could not be increased without increasing the extent of flooding well beyond that recorded. Figure 7 shows the final extent of flooding calculated.

At this stage the average calculated flood level was still 0.2 m below the surveyed flood level. It was not possible to obtain the average flood levels on the observed flooded area by increasing the discharges. The final runs were therefore carried out only to obtain the best fit to the flooded area. For the analysis the data were divided into two sections: points at the edge of the flooding, and points in the centre of the flow. Table 4 shows these results.

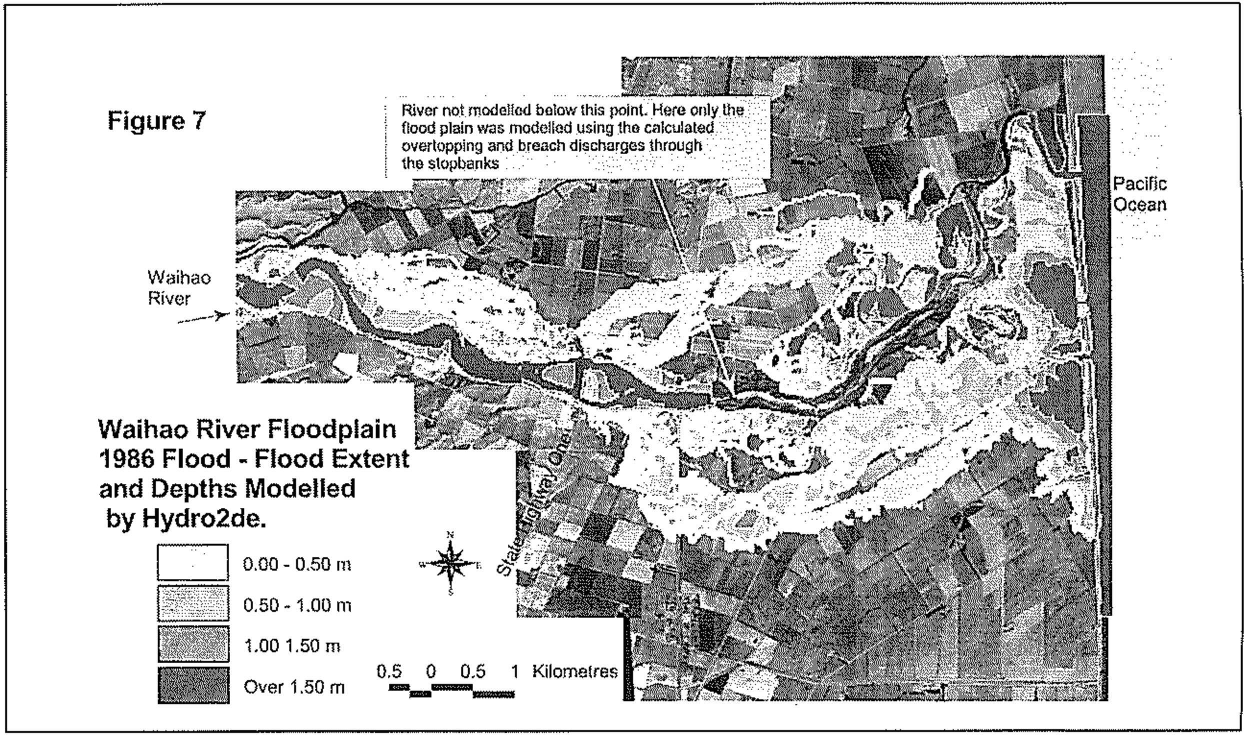


Figure 7 – 1986 flood: flood extent and depths modelled by 'Hydro2de'.

Table 4 – Differences in observed and calculated levels for edge and centre points (DTM correction).

Flood	Edge levels (m) (difference and standard deviation)	Centre levels (m) (difference and standard deviation)
1986	-0.010 ± 0.284	-0.300 ± 0.271
1994	-0.036 ± 0.300	-0.291 ± 0.323
Both floods (all data)	-0.022 ± 0.290	0.297 ± 0.291

These differences have also been corrected for average differences in surveyed ground levels and DTM spot heights for each group of data. The differences between surveyed ground levels and the DTM levels for the 80 edge levels were significant at the 99% confidence level.

The differences can be attributed to the high and low areas of the DTM that were described in the section on the DTM. The edge levels were in these areas.

Table 5 gives the raw results without this correction.

Table 5 – Differences in observed and calculated levels for edge and centre points (no DTM correction).

Flood	Edge levels (m) (difference and standard deviation)	Centre levels (m) (difference and standard deviation)
1986	-0.120 ± 0.284	-0.228 ± 0.271
1994	-0.091 ± 0.300	-0.262 ± 0.323
Both floods (all data)	-0.107 ± 0.290	-0.230 ± 0.291

However it is likely that neither the results in Table 4 nor those in Table 5 will be correct, as the flood levels had to be calculated from an area of the DTM, i.e. the area required for the backwater component of the equations (Henderson, 1966). This means that the average error of the DTM area used to calculate the flood levels will not be the same as that of the point. The actual difference probably lies in between the values in Tables 4 and 5, e.g. -0.01 m and -0.12 m for the 1986 flood, because the DTM error of the area used by the backwater component of the equations to calculate the flood levels would be in this range.

Flood depths of final results

Table 6 gives the results of the depth comparisons.

Table 6 – Differences in observed and calculated depths for points at the edge and centre.

Flood	Edge depths (m) (difference and standard deviation*)	Centre depths (m) (difference and standard deviation)
1986	-0.120 ± 0.207	-0.184 ± 0.217
1994	-0.005 ± 0.323	-0.252 ± 0.345
Both floods (all data)	-0.036 ± 0.291	-0.207 ± 0.259

* only 3 data points (the values of less than zero were omitted)

The standard deviation of the results from both floods was reduced to even further below that of the DTM error of ± 0.264 m. In the case of the 1986 event this was well below the expected standard deviation of the data without modelling errors of ± 0.3 m, while for the 1994 event it was a little larger.

The reduction in the standard error can also be attributed to the DTM having high and low areas. Analysis of the DTM levels using field data showed that these areas contained enough points to determine that they were either high or low at a significance of 95%. The depth analysis would therefore eliminate most of this error, with a much smaller error coming from the incorrect slope of the DTM into and from these areas.

Discussion

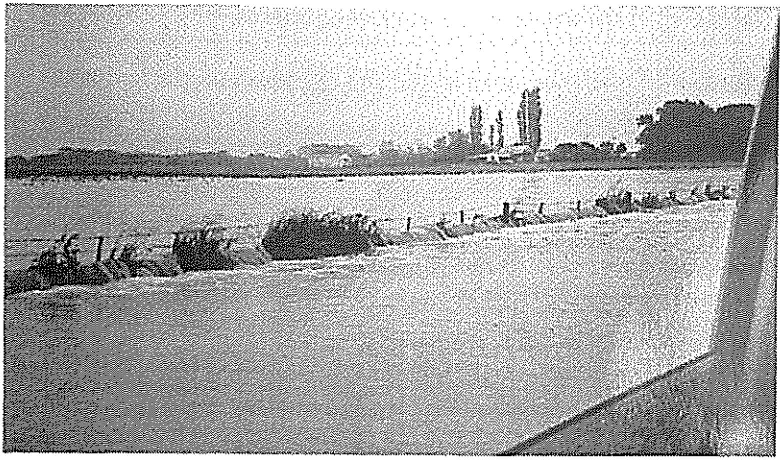
Modelled edge and centre flood levels

The results show that while the modelling accurately predicts water levels at the edges of the water flow, the levels (the average of Tables 4 and 5) and depth (Table 6) predicted in the centre of the flow are about 0.2 m too low.

The differences in the edge and centre levels can be explained by several factors not covered in the modelling.

i) Areas of buildings in the model were represented only by increases in flood plain resistance. A better representation would be obtained by putting the individual buildings into the model.

Figure 8 – Effect of a fence on water levels.



ii) The grid size was too coarse to include fences or hedges, which can cause localised increases in flood level. Figure 8 shows the effect of a fence, with about a 0.2 m difference in water levels upstream and downstream of the fence.

iii) The model does not take into account fluctuations of the water surface due to turbulence (Nezu and Nakagawa, 1993). Many of the photographs showed waves in the water surface, especially in areas of rapid flow.

iv) There are also eddy effects at the interface of deep and shallow water (Smart, 1992) that increase the overall resistance values in these areas.

Standard deviation of the modelling results

The standard deviation of the differences of the initial runs was of a similar order to the expected standard deviation of the observed data. The error of the observed data is the combined error of the DTM error ± 0.26 m, the error in estimating the flood levels of around ± 0.14 m, and surveying errors of ± 0.03 m.

The errors of the modelling include use of incorrect Manning's 'n' values, use of spot heights from the DTM for the centre of each 20 m cell that may not be the mean height of that cell, the errors in estimating the flows onto the flood plain, and numerical errors in the model. It is not possible to quantify the modelling errors using these results.

The total expected standard deviation, not including the modelling errors, was ± 0.30 m for the 1986 flood. The actual standard deviation of the differences was ± 0.275 m.

For the 1994 flood the total expected standard deviation, not including the modelling errors, was ± 0.30 m. The actual standard deviation of the differences was ± 0.314 m.

The standard deviation of the differences was less than the expected standard deviation because the backwater component of the equations used an area of the DTM to calculate each flood level and flood depth. This area would have had a lesser standard deviation error than the ± 0.264 m calculated using the point data.

The low standard deviation of the results also leads to the conclusion that the Digital Terrain Model error is the main source of error in the modelling, and improving the DTM accuracy will improve the results.

Diagonal flow over a rectangular two-dimensional grid

It was thought that diagonal flow across the uniform numerical grid would result in flood levels that would be too high. For flow at 45 degrees to the grid, the connection from the points on one line (say 7 points) would not be over the full width (it would only be 6 points) to the points on the next line of the grid. This is shown in Figure 9. Table 7 below gives the results for channels flowing diagonally across the calculation grid (for the second-order scheme).

Figure 9 – Cell alignment for a channel at 45 degrees to a uniform grid.

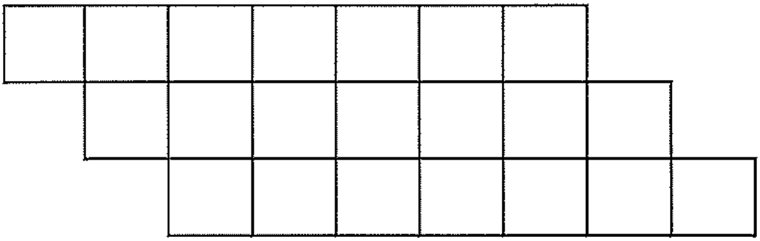


Table 7 – Results for flow in a uniform channel running diagonally over a rectangular grid

Channel width as a number (n) of 20m cells	Equivalent Diagonal width $=20n/\sqrt{2}$ (m)	Discharge put into Hydro2de (m^3/s)	Depth calculated by Hydro2de (m)	Equivalent depth calculated with Manning's formula using the Hydro2de discharge (m)
10	141.4	253.35	1.39	1.29
5	70.7	133.43	1.57	1.34
2	28.2	52.63	2.29	1.305

The results from Table 7 show that for diagonal flow across a uniform grid the grid causes over-estimation of the flood depths. The table shows that this effect increases as the number of cells in the channel carrying the flow decreases.

This effect was not significant in the flood plain analyses: another run, using a 10 m grid on one of the sub-areas, did not change the results significantly. The change was from – 0.219 ± 0.277 m using a 20 m grid to – 0.212 m ± 0.273 m using a 10 m grid.

Velocities

No velocity data were available for the floods modelled. The best fit to the data was obtained using the conveyance represented by the discharges put into the model and the resistances used. Neither of these variables is necessarily individually correct–only their product is correct.

Some surface velocity data could be estimated from videotapes. This would allow mean velocities for the point to be determined. Another possible method to check the velocities would be to show the local observers a series of maps indicating the progress of the floodwaters over the flood plain calculated by the models, with the estimated time on each map. They could then comment on the accuracy of these maps.

The photographs of the floods do give a few views of the progress of the flood wave down the flood plain. One of these was analysed; it indicated that the flood wave in the model had not travelled far enough, suggesting that the resistance coefficients were over-estimated. This would be expected as the section on resistance coefficients stated that the open pastoral land value was over-estimated to allow for levels on fences.

Validation of the integrity of the observed data

This method of data collection has not been tried in New Zealand before and may not regarded as 'scientific'. The study shows that the data were useful as:

i) Much of the data could be validated with photographs and/or videotape or was obtained from these sources.

ii) The standard deviation of the differences in the modelled and observed data was much less than expected. This improved when the depth data were analysed, as these data reduced the DTM error and took out the effect of the control point errors of the DTM. The data would not have been as useful if the standard deviation of the differences between the calculated flood levels and observed flood levels had turned out to be well above that of the expected differences.

In one sense, neither the modelled levels not the levels gathered from the resident's recollections are 'truth'. Most of the levels had photographic evidence and are similar to conventionally measured 'truth'. Because the standard deviation of the differences was less than expected it could be stated that each method verified the other. The extent of flooding measured by regional government staff is more conventional 'truth'. This compared well with the calibrated results model extent of flooding.

Conclusions

The two-dimensional model, Hydro2de, gave very good results, with errors less than those expected from the DTM error. Calibrating the model to obtain the best fit to the flooded area showed that Hydro2de correctly predicted levels at the edge of the flooding, but predicted flood levels in the centre of the flooding that were too low by about 0.2 m. This may be because the analysis did not include details of buildings and fences, or the effects of wave action. Improvements in the model, with better representation of buildings by using non-floodable cells and the additional losses due to obstacles (vegetation, fences etc.) have since been implemented (Beffa, in press).

The initial runs of the model, prior to calibration, gave a good representation of the flooding on the Waihao River flood plain. Therefore, the model should give a reliable answer for the flow paths of estimated outflows on flood plains, even where flood levels have not previously been surveyed and recorded.

The program over-estimated levels, and hence under-estimated the flood plain conveyance of water flowing diagonally across the uniform calculation grid. A non-uniform gridding program needs to be developed and the program improved to use this grid. The difficulty with this solution is that usually the direction of the flow is not known beforehand or may change during the flood.

Obtaining data on flood levels from past floods from verbal accounts given by flood plain residents is a useful method to obtain data. Photographs can confirm many of the residents' observations. The differences between the calculated flood levels and those observed by flood plain residents were less than those expected from the standard errors of the Digital Terrain Model and in the estimation of the observed flood levels.

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References

Bates, P.D.; Anderson, M.G.; Hervouet, J.M. 1994: Computation of a flood event using a two dimensional finite element model and its comparison to field data. In P. Molinaro and L.Natale (*eds.*) *Proceedings Speciality Conference Modelling of Flood Propagation Over Initially Dry Areas*, American Society Civil Engineers, New York: 243-256.

Bechteler, W.; Nujic, M.; Otto, A.J. 1994: Calculation of 2D flood propagation using the program package floodism. *In* P. Molinaro and L.Natale (*eds.*) *Proceedings Speciality Conference Modelling of Flood Propagation Over Initially Dry Areas*, American Society Civil Engineers, New York: 257-267.

Beffa, C. 1994: Praktische Lösung der tiefengemittelten Flachwassergleichungen. *Communication 133*, Laboratory of Hydraulics, Eidgenossische Technische Hochschule, Zurich.

Beffa, C. 1996: Application of a Shallow Water Model to Braided Flows. *In* Muller *(ed.) Proceedings of the Hydroinformatics '96 Conference*, Zurich, A.A.Balkema: 667-672.

Beffa, C. 1998 in press: Two-dimensional Modelling of Flood Hazards in Urban Areas, *Proceedings 3rd International Conference on Hydroscience and Engineering.* Cottbus/Berlin.

Chow, V.T. 1959: *Open Channel Hydraulics*, McGraw-Hill, New York.

Cunge, J.A.; Holly, F.M.; Verwey, A. 1980: *Practical Aspects of Computational Hydraulics*, Pitman, London.

Henderson, F.M. 1966: *Open Channel Flow*, MacMillan, New York.

Hirsch, C. 1988: Numerical Computation of Internal and External Flows. *In: Volume 1: Fundamentals of Numerical Discretization*, Wiley & Sons.

Jansen, P.; van Bendegom, L.; van den Berg, J.; de Vries, M.; Zanen, A. 1975: *Principles of River Engineering*, Pitman, London.

Labadie, G. 1994: Flood waves and flooding models, *In* Rossi, G.; Harmancioglu, N.; Yevjevich, V. *(eds.) Coping with Floods* 13-15 November 1992 NATO Advanced Study Institute Series, Kluwer Academic, Dordrecht, The Netherlands: 291-323.

Liggett, J.A. 1987: Forty Years of computational Hydraulics – 1960-2000, *Proceedings 1987 National Conference on Hydraulic Engineering*: 1124-1133.

Nezu I.; Nakagawa, H. 1993: *Turbulence in Open Channel Flows*, Balkema, Rotterdam.

Roe, P.L. 1981: Approximate Riemann solvers, parameters vectors and difference schemes. *Journal Computational Physics 50*: 447-481.

Smart, G.M. 1992: Stage-discharge discontinuity in composite flood channels. *Journal of Hydraulic Research 30:* 817-832.

Van Leer, B. 1977: Towards the ultimate conservative difference scheme. A new approach to numerical convection. *Journal Computational Physics 23*: 276-99.

Zhao, D.H.; Shen, H.W.; Tabios III, G.Q.; Lai, J.S.; Tan, W.Y. 1994: Finite-Volume Two Dimensional Unsteady Flow Model for River Basins. *Journal of Hydraulic Engineering, American Society Civil Engineers 120 (7)*: 863-883.

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