TWO-DIMENSIONAL FLOOD PLAIN FLOW. II: MODEL VALIDATION

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ABSTRACT: Information from two floods on the Waihao River rural flood plain in New Zealand was used to validate a two-dimensional flood plain flow model, Hydro2de. An aerial photogrammetric survey digitally described the terrain. Measurements during and after the floods, and a global positioning system survey of positions from photographic records and other information recalled years later by flood plain residents, provided flood level, depth, and extent data to test the model. Uncalibrated, with inputs from the river and levee overtopping or breaches, the model underestimated levels, depths, and the area covered by floodwaters. The estimates were sufficiently close to be useful for flood plains without previous flood measurements. Calibrated to reproduce the measured flood extents, the model gave levels and depths closer to reality that were good at the edges of the flood plain subareas, but underestimated depths in the center of the flow. The underestimates occurred because the model did not include details of houses, hedges, and fences, nor any wave action of the water flow. A comparison is made to one-dimensional modeling using the MIKE11 model. Two-dimensional flood plain flow modeling promises benefits over one-dimensional modeling, in particular because the former does not require operator choice of a network of ''channels'' to represent the flood plain. The two-dimensional modeling reported here shows that improving the accuracy of the digital terrain model would provide the most improvement to accuracy of the results. Model functionality could also usefully be improved.

INTRODUCTION

Human settlement on flood plains is common in New Zealand. Major cities, smaller towns, and rural farm properties located on flood plains have been protected against flooding up to a ''design'' level by levees (''stopbanks'' in New Zealand) and other river control works. Consequences of failure due to breaching and overtopping of these works are in some places mitigated by controls on floor level of buildings, flood warning systems, and insurance. Such failures, while infrequent on rivers with significant riparian development, are serious hazards, with severe human safety and economic consequences. There is a large potential benefit for flood event contingency planning. This includes building development and floor level controls, information on potential flood-prone areas during times of flooding, and information for potential purchasers of property. Being able to accurately predict the extent, level, and velocity of floodwaters from river breakout flows assist immensely in this task.

A model for prediction on a particular flood plain can make use of data from previous floods for its development and calibration. If there are no suitable data for the flood plain, it is desirable to be able to transpose the model calibration factors, such as Manning's *n*, from another flood plain. This problem, involving complex river and flood plain geometry and flows over initially ''dry'' land, has been treated using one-dimensional unsteady flow models that are readily available from major software developers specializing in hydraulic software (e.g., NWS 1999; DHI 1999). With these programs, considerable operator skill is required to estimate hydraulic boundaries of the water flow, flow directions, and appropriate terrain roughness values for the imposed ''network'' of one-dimensional channels. In many cases, the flow directions on the flood plain, and therefore the network of model channels, are very difficult to assess and can change during a flood event.

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Sometimes complex river topography makes it difficult to correctly model increases in water level, even using a network of channels in the channel system. An example is shown in Fig. 1. This shows the effects of non-uniformity at the confluence of two rivers. At points upstream and downstream during a flood event, there was 1 m or more of freeboard, while near the confluence the levees overtopped and breached. These levees were designed using a steady-state water surface profile analysis (one-dimensionally), with effects of river bends added to the levee heights where the afflux was significant. It is very difficult to correctly model the increase in water level at this location, as the streamlines of the water flow are not known to correctly place model subchannels in the river system. The contour plan of the Waihao River flood plain shown in Fig. 2 also illustrates complex river topography. Flood plain and river channel models are needed, which handle the terrain using a grid of points, i.e., two-dimensionally.

Flood plain models also need to handle the initially ''dry'' flow boundaries and transitional flow regimes (e.g., subcritical to supercritical when overtopping). They need to be sufficiently robust computationally for operational use (Cunge et al. 1980; Ligget 1987; Zhao et al. 1994). Two-dimensional models are presently under development. These models find flow directions and hydraulic boundaries from the underlying terrain structure, starting from an initially dry flood plain, us-

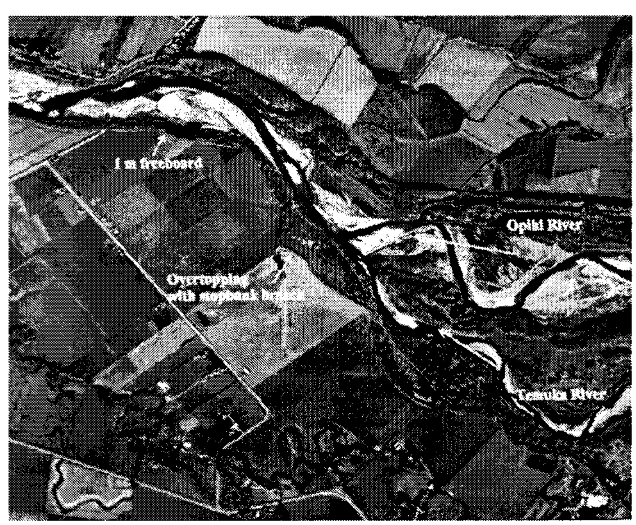

FIG. 1. Opihi and Temuka River Confluence

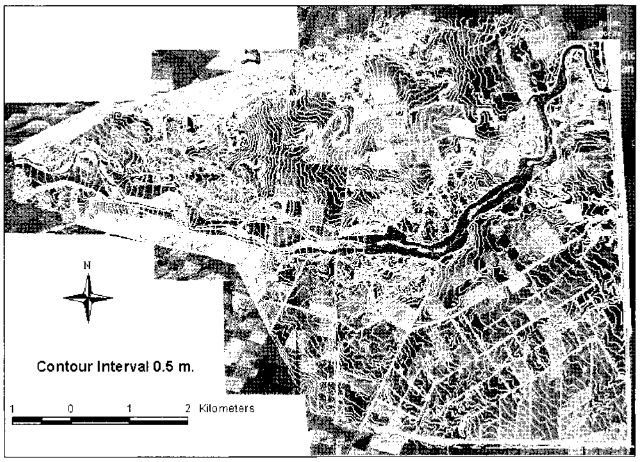

FIG. 2. Contour Plan of Waihao River Flood Plain

ing the two-dimensional shallow-water equations of motion. The models need to start from an ''initially dry'' state to ensure the water flow directions are correct and are calculated from the terrain shape and the inflow point and discharge. It is necessary to investigate the ability of these models to cope with real terrain situations and to model sufficiently accurately without previous floods for calibration.

This paper describes a study in which one of these models under development, the Hydro2de model (Beffa 1994, 1996; Beffa and Connell 2001), was used with purpose-measured terrain and flood data from two recent events on a 50 km^2 rural flood plain in the South Island of New Zealand. A digital terrain model (DTM) was constructed using data from an aerial photogrammetric survey (APS), ground-truthed by a global positioning system (GPS) survey. The flood data came from aerial photographs, video photographic records, real-time flood observations, and detailed interviews of flood plain residents (Connell et al. 1998). The GPS survey also gave ground levels at the positions where flood levels were recorded, allowing a check of the accuracy of the DTM. A gauging station (calibrated water level measurement site) on the river upstream of any flood breakouts provided overall river discharge records. Although there was a variety of terrain and surface cover, including small forested areas, farm houses, buildings, fences, bridges, roads, and railway lines, there were no large urban areas in the flood plain. Thus, most of the requirements for straightforward model verification (that it functioned as intended) and validation (that it reproduced real behavior sufficiently well) were satisfied. This made the study economically feasible and provided data that could be transposed to other flood plains.

HYDROLOGY AND FLOOD PLAIN

The Waihao River flows to the South Pacific Ocean on the East Coast of the South Island of New Zealand (Fig. 3). The drainage basin area is 550 km^2, with 484 km^2 upstream of the gauging station at McCulloughs Bridge, 10 km upstream of the study area (Fig. 3). Maximum elevation is 1,500 m. Mean annual rainfall is approximately 1,000 mm, and mean discharge is 3.7 m^3/s. Predominantly yellow-brown and yellow-grey earths and rendzina soils (Cutler 1968) overlie moderately indurated greywacke (quartzite) and nonfoliated schist and tertiary rocks (limestones, siltstones, and sandstones or mudstones) (Mutch 1963). Mean annual and estimated 1% annual exceedance probability (''100-year'') floods are 250 and 1,250 m^3/s, respectively. These have been derived from 15 years of continuously recorded river levels and 40 years of historical records of river flood heights.

The river flows in one, and sometimes two, thalweg channels in a bed of greywacke gravels. It has a steep gradient that flattens to about 0.25–0.3% over the final 10 km to the coast. Over the last 2 km, the river bed changes from gravel to silt, with a very low slope. The coastline has a gravel beach dune that is between 5 and 6 m above mean sea level at its apex. Various grasses and shrubby plants grow in the overall river bed when low flows leave exposed gravel banks. In places, larger trees such as willows (*Salix* species) have been planted to assist bank stabilization.

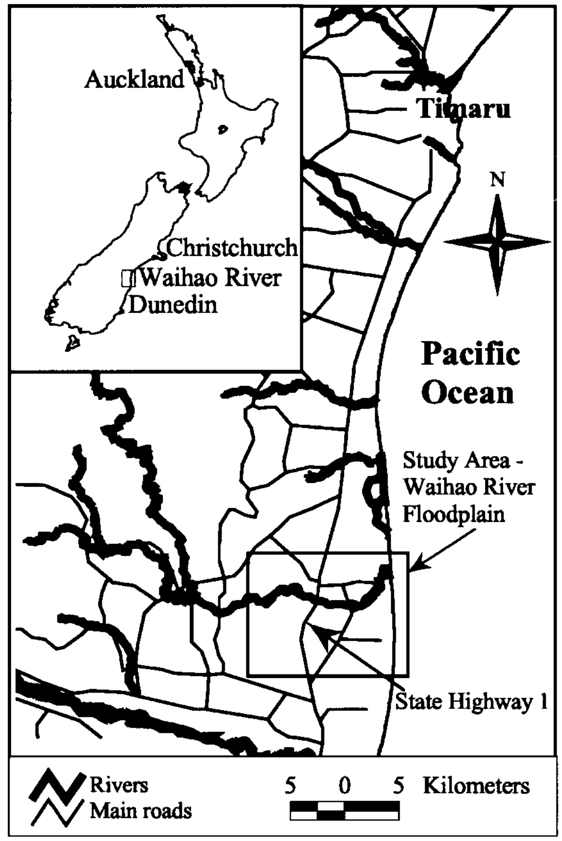

FIG. 3. Locality Plan of Waihao River

Historically, most flooding occurs in the study area (Fig. 3). The lower part of this area below State Highway 1 is confined between levees. The predominant land use in this lower flood plain is pastoral and arable farming. Approximately 50% of the flood plain is in grazed pasture, 50% is in arable crops, and a small portion of the area is in woodlots and areas of scrub. There are approximately 40 farm properties and 150 inhabitants.

The South Island first-ranked highway (State Highway 1) and the main trunk railway line both cross the river (seen most clearly in Fig. 6). There are other minor waterways and constructed drains in the flood plain. A contour map of the area is shown in Fig. 2. A geographic information system (GIS) was used to manipulate terrain information for modeling and presentation [ARC/INFO; ESRI (1999)]. The upstream gauging station has continuous level recording calibrated to discharge by current meter ratings. Flood hydrographs were routed to the downstream river reaches using the one-dimensional unsteady flow model, MIKE11 (DHI 1999).

TERRAIN AND CHANNEL INFORMATION

The DTM (Fig. 2) was photogrammetrically derived from the purpose-flown, commercial aerial photography survey. It has over 90,000 points and more than 6,000 breaklines (defining ridges and valleys). The specified standard error to be achieved was ± 0.3 m for surface level, and ± 0.3 m for horizontal location. A check using 300 GPS-measured levels over 30 km^2 indicated a standard error for this subset of ± 0.264 m, thus inside the specification. Some partial areas of the complete DTM were found to be consistently high or low, as groups of points in these areas exhibited significant (at 99%) biases.

The GPS survey measured ground level at 10 ‘‘control’’ points for the aerial photogrammetry, other known survey points in the area, and at the points where flood levels were determined—182 for the 1986 flood, 108 for the 1994 flood. Claimed accuracy was ± 0.03 m, including an allowance for uncertainty in the local survey datum as compared with the satellite datum. The bed level information for the riverbed was as existing conventionally surveyed cross sections.

HYDRO2DE MODEL

The Hydro2de model (Beffa 1994, 1996; Beffa and Connell 2001) was chosen as the two-dimensional flow model for the study. It was capable of handling the complex topography and necessary transitional (supercritical and subcritical) flows, could incorporate an initially dry terrain, had good input/output features for use alongside the GIS, and was known to be computationally robust. It was still under development when the study began. This became an advantage, as some desirable changes made obvious by the present study were incorporated into the model.

The model solves the depth-averaged shallow water flow equations in conservation form (Abbot 1979), in which depth and specific flow are related to spatial coordinates $[x,y]$ using conservation of volume and momentum. The current implementation requires a uniform rectangular grid over the subareas. Explicit time integration by a finite volume method balances fluxes entering and leaving ‘‘cells’’ of any desired size required by the model. Output from Hydro2de is primarily in graphic form. Spatial distributions of water velocity (speed and direction), x and y velocity components, depth, water level, Froude number, and energy slope are available. The input spatial distributions of ground level and surface roughness coefficients (Mannings n) are also available.

For Hydro2de purposes, the flood plain section to be modeled is described by two layers in the GIS: a uniform grid of ground levels extracted from the surveyed DTM, and the values on the grid of the local surface resistance to flow. The ground level grid was at 20×20 m for the flood plain subareas. Individual breakout flows were modeled using 5×5 m grids. To represent surface roughness, aerial photographs were scanned into the GIS and used to digitize estimated Manning’s n coefficients (Table 1). Finer grids would have allowed better definition of physical features and might have led to more accurate results, but the mesh size specified generated grids of 20,000 to 52,000 cells for the four subareas (two areas for each bank or flood plain). This was approaching an upper limit with the computing equipment available (SUN or DEC workstations and a Pentium-based PC) to allow reasonable computation times for multiple reruns. One run for the four subareas of the model typically took 24 hours on the 166 MHz Pentium.

To ensure that no significant features were omitted from the model, a check of the grid against a contour model of the DTM was undertaken. This revealed several places where the grid omitted features. The notable ones were the railway line embankment and some openings for local drainage in an irrigation channel. The ground levels at these points on the grid (or the closest point to the feature) were changed to ensure the feature was modeled in the grid. The Manning’s n was varied from the figures given to provide correct hydraulic conveyance for open drains of width less than the mesh size (20 m). No other parameters are needed for the Hydro2de model.

The calculation domains were rectangular in shape and therefore inputs were from their boundaries. Details of the procedures for this study are given below.

TABLE 1. Manning’s *n* Values Used for Flood Plain

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

MIKE11 MODEL

The MIKE11 model (DHI 1999) is widely used in Europe, Asia, and Australasia. It is a commercial finite difference model based on the shallow water equations and was chosen to do the one-dimensional analyses for this study. This model requires the flood plain to be broken up into a network of channels and cross channels. At the time of this study, the model did not have a routine to take out cross sections of the bed levels, or the resistance coefficients of the flood plain, from a digital terrain model. Software was developed to enable this to be undertaken.

The flood plain was divided into 108 channels with over 900 cross sections using over 50,000 data points. Channels were determined using the following criteria:

1. The fall of the land using a contour plan of the flood plain
2. The expected width and alignment of water that would be approximately level
3. Channels over 0.5 m deep, i.e., the depth between either side and the lowest point in the center was over 0.5 m
4. The positions of the overflow or breakout points from the river or levees
5. The number of channels to make the analysis reasonable to undertake
6. The areas that were flooded in both the 1986 and 1994 flood

ESTIMATION OF FLOOD DISCHARGES ONTO FLOOD PLAIN

Area with No Levees

The upper area of the model, upstream of State Highway 1, did not have any levees; therefore, a digital terrain model of the river constructed from the cross-section information was inserted into the digital terrain model of the flood plains on both banks of the river.

The hydrograph from the gauging station at McCullough’s Bridge (10 km upstream of the study area) was routed down to the study area using MIKE11. This hydrograph was used as the input for both models. The discharge hydrographs for the 1986 flood, after routing to the study area, is shown in Fig. 4. The 1986 upstream hydrograph is a reconstruction using slope-area methods (Mosley and McKerchar 1992), as the recorder was destroyed during the event.

Area with Levees

After the flood events in 1986 and 1994, all the overtopping lengths and levee breaches were surveyed. The locations of breakouts downstream of the State Highway for the 1986 flood are shown in Fig. 5.

Neither model, as used on the flood plain, could incorporate the levee overtopping and breaching (including their times) to

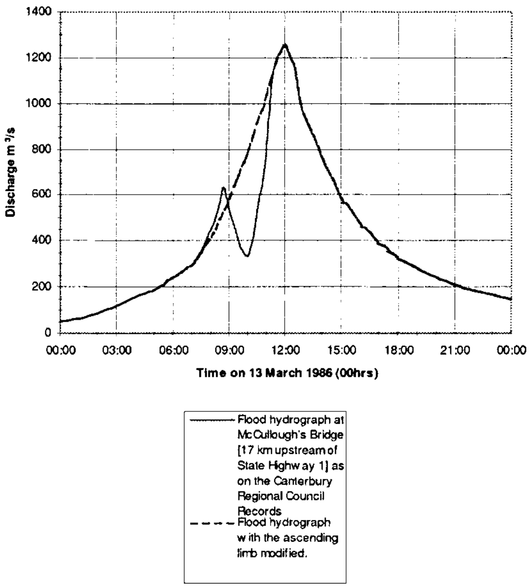

FIG. 4. Hydrograph of 1986 Flood

the required level of detail to give good results. So each breach and overtopping reach had to be analyzed in detail first and then hydrographs developed using this information and other information available from the time of the floods. The overtopping and breaching were modeled using Hydro2de, as MIKE11 could not handle the transition from subcritical to supercritical flow.

Overtopping Analyses

The overtopping was modeled using a typical cross section of the levee. The field survey after both flood events detailed all the lengths of levee and flow depth where overtopping occurred. Several scenarios were modeled to obtain a discharge per unit width for different overtopping heights. Fig. 6 shows the results of the analysis for 0.2 m overtopping. Fig. 7 shows the effect of Manning's *n* on the overtopping discharges

Breach Analyses

The breaches were also modeled using Hydro2de. This was because the water flow was critical through the breach and MIKE11 could not be used for transition from subcritical to supercritical flow. These models were run until the water levels in the river were calibrated to the observed flood level at the breach, so that the best estimate of the flood flow onto the flood plain would be obtained. Enough of the flood plain was included to ensure that downstream boundary conditions would not affect the results. Figs. 8 and 9 illustrate an example of a breach analysis.

Final Overflow Hydrographs

The final hydrographs of the breakout flows were constructed using (Connell et al. 1998):

1. The overflow and breach analyses. These provided peak outflows from the river.
2. The river hydrograph routed to the study area (using MIKE11). This provided the time that the flood wave reached the area.
3. Observations during the flood of the times that overtopping started and breaches occurred.
4. A water surface profile analysis of the peak flood levels in the river. This analysis determined the peak discharge at each cross section on the river so that the time that the breach occurred (either before or after the peak of the flood) could be determined using 1–3 above.

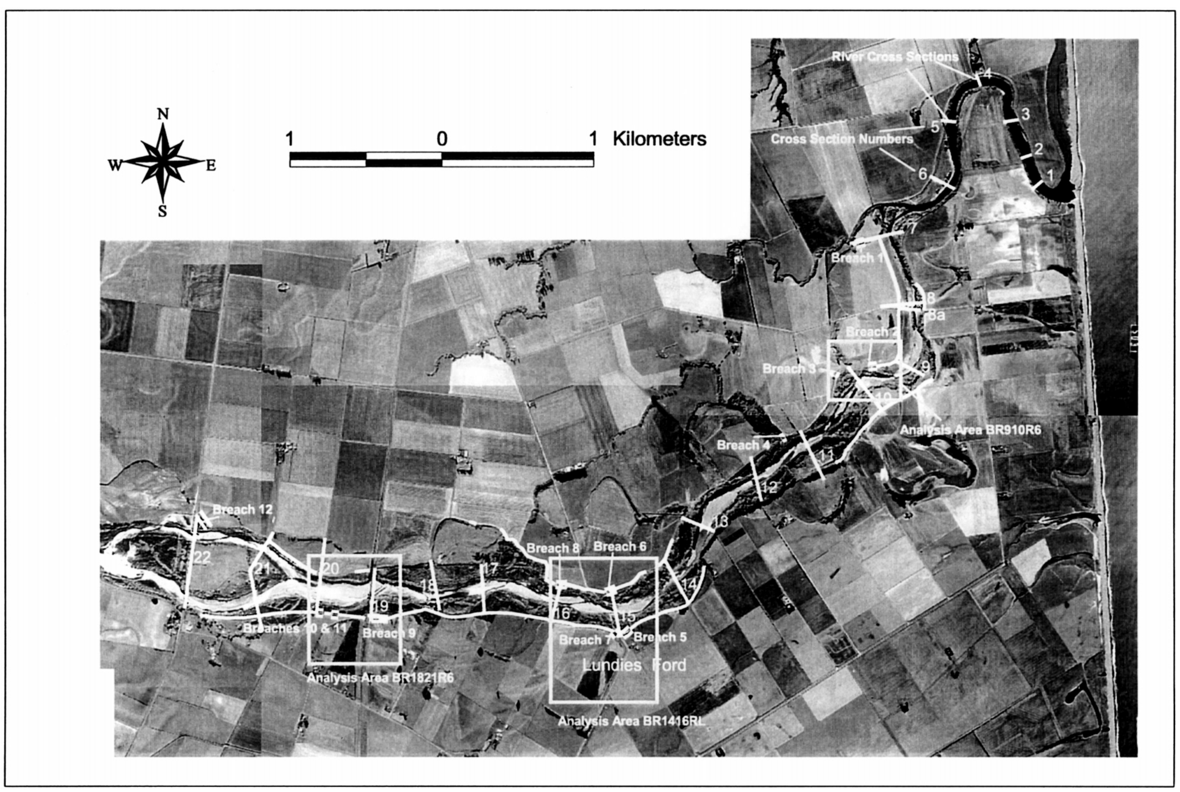

FIG. 5. Breakouts and Overtopping Areas March 1986 Flood

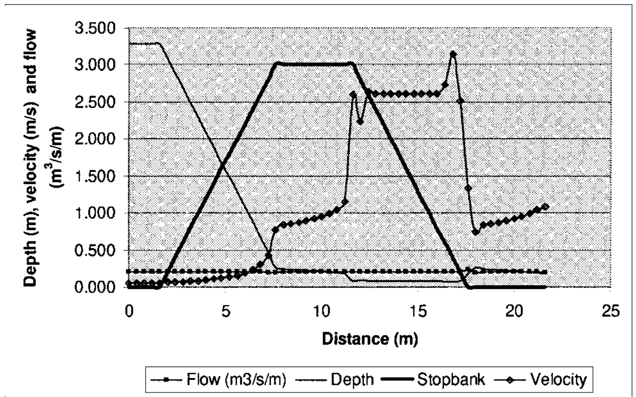

FIG. 6. Levee Overtopping Profiles—0.2 $m^3/s/m$

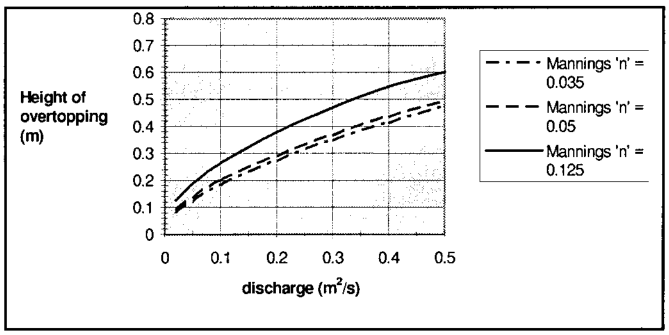

FIG. 7. Manning's *n* Effect on Overtopping Discharges

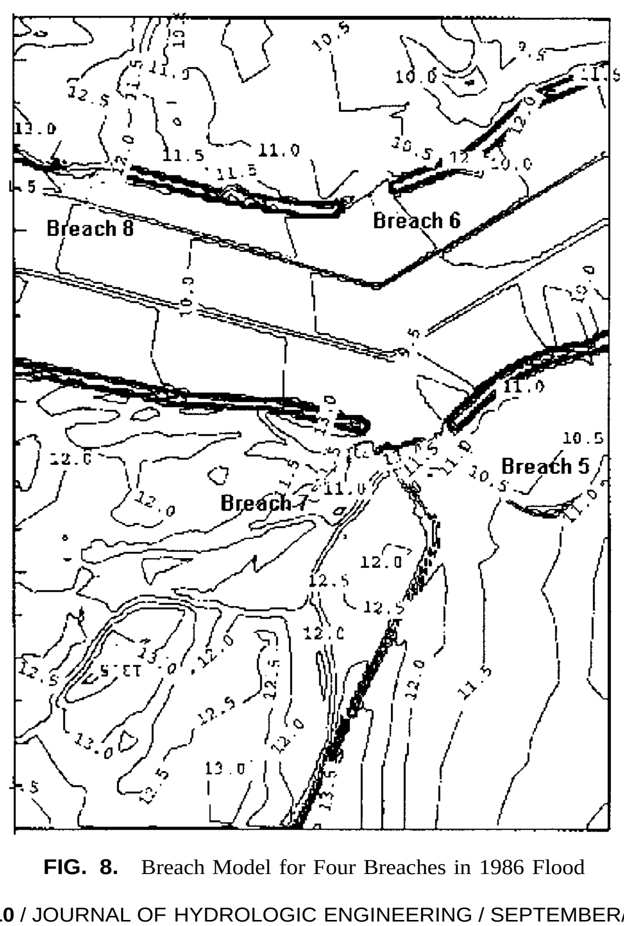

FIG. 8. Breach Model for Four Breaches in 1986 Flood

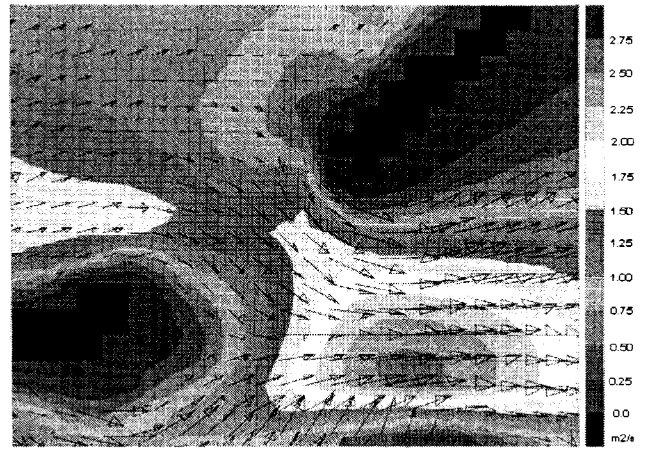

FIG. 9. Velocity Vectors and Unit Discharge Rates Used to Estimate Outflow for Breach 5 of Fig. 8

TABLE 2. Initial and Final Calibrated Peak Discharges for Breach and Overtopping Sites

Breach or overtopping	Peak initial discharge (m^3/s)	Peak final discharge (m^3/s)
(a) 1986 Flood		
Upstream end of model	1,250	1,150
North side of river		
Overtopping and breach 8 cross section 16–17	108	No change
Overtopping and breach 6 cross sections 15–16	123	No change
Overtopping cross sections 14–15	35	No change
Breach 2 cross sections 9–10	182	No change
Overtopping 8–9	20	No change
South side of river		
Overtopping cross section 20–21	15	40
Breach 11 cross section 19–20t	75	85
Breach 10 cross section 19–20b	52	60
Breach 5 and overtopping cross sections 14–15	126	130
Overtopping cross sections 10–11	25	50
Overtopping cross sections 9–10	15	40
Overtopping cross sections 7–8	10	30
(b) 1994 Flood		
Upstream end of model	1,070	700
North side of river		
Overtopping and breach 6 cross sections 16–17	101	No change
Overtopping and breach 5 cross sections 12–13	137	No change
Overtopping cross sections 8–10	20	No change
South side of river		
Overtopping and breach 7 cross sections 18–20	92	No change
Overtopping cross sections 17–18	6	No change
Overtopping and breach 3 cross sections 9–11	184	204
Overtopping and breach 1 cross sections 7–8	150	No change

The magnitudes of the breakout flows are listed in Table 2.

FLOOD PLAIN MODELS

The channel geometry upstream of State Highway 1 was built into the DTM using conventionally surveyed (EDM) river and levee cross sections and locations. This model had either the river channel or flood plain at the east and west boundaries of the model, so the flood hydrograph was entered on the western boundary and water flowed out of the east boundary.

In the downstream areas, only the flood plain was modeled. The river was not included in these models. As Hydro2de calculation grids are rectangular and these flood plains were irregular in shape at the river boundary, the flood plains could not reach the edge of the calculation area. Therefore, inflows were entered from the edge of the calculation area. The inflows from levee breaches to flood plain subareas were modeled as channels of width equal to the actual breach (to the nearest 20 m) entering the subarea at the actual location and at ground level.

Overtopping was modeled as a channel from the edge of a calculation area to the edge of the flood plain that turned parallel to, and was the same length as, the actual overtopped section of levee. On the length parallel to the flood plain, a ''crest'' one cell wide of constant level, at the actual levee level, was located between the channel and the subarea so that constant flow per unit width occurred. The discharges calculated with the individual breach areas or overtopping areas were put into these channels. Breach and overtopping discharge hydrographs were obtained by individually modeling each location, as previously described. The river hydrograph (taking account of upstream breakout flows) was applied to the breach or overtopping site geometry as estimated from measurements made during and after the floods.

The coastal dune was not included in the DTM. Actual flood waters reached the dune, ponded upstream of it, and then flowed to the ocean when the dune breached by piping failure. Modeling was carried out, with a dune of approximately the correct location and crest height (about 5.5 m above MSL datum) inserted into the DTM, until flood waters reached their maximum levels. Then part of the model dune was removed, to represent the piping failure that actually occurred, and to allow the flood water to flow to the ocean.

''High'' ground, irrelevant to flow computations, was inserted on the GIS DTM layer to match the actual flood plain to the overall rectangle required by the Hydro2de grid. The only significant effect was a need to correct for the volume of water in the model ''channels,'' through the ''high'' ground, which delivered breakout flows from the edge of the computational grid to their actual locations on the DTM of the subareas.

Fences were not explicitly modeled, nor were buildings—but areas of higher resistance were inserted into the model to represent the area of a building, including the hedges and other structures in the building area. Hedges were inserted into the roughness layer, but the 20 m grid was not fine enough to model these well. The roughness value for ''general pastoral farm land'' (Table 1) was intended to include the effects of fences (but see the discussion section).

1986 AND 1994 FLOOD INFORMATION

Some rainfall, flood, and flood damage data existed for floods in March 1986 and March 1994, particularly that gathered by the regional government authority (Canterbury Regional Council or CRC). These formed the basis of an augmented dataset gathered specifically for this study. March is late in the Austral summer, or early in the fall.

In the 1986 event, there was about 200 mm of rainfall in 10 hours in the catchment. The resulting river hydrograph at the water level recorder gauge, had an instantaneous peak of 1,250 m^3/s (0.01% AEP) at about 10:30 a.m. on March 13, 1986. The first breakouts from the river occurred about 10:00 a.m. and continued until the river dropped below the bottom of the levees. The eventual pattern of breakouts and observed extent of flooding are indicated in Fig. 10. Floodwaters entered four homes and several farm buildings, closed roads for 12–18 hours, and necessitated shifting livestock to high ground.

In the 1994 event, there was about 150 mm of rainfall in 12 hours in the catchment. The resulting river hydrograph had an instantaneous peak of 1,000 m^3/s (0.02% AEP) at about 12:00 midday on March 19. The first breakouts from the river occurred about 11:30 a.m. and continued until about midnight. Floodwaters did not enter any dwellings, but did enter several farm buildings, closed roads temporarily, and necessitated shifting livestock to high ground.

CRC staff carried out postflood analyses of the flood extents, mapping both events soon after they occurred. They in-

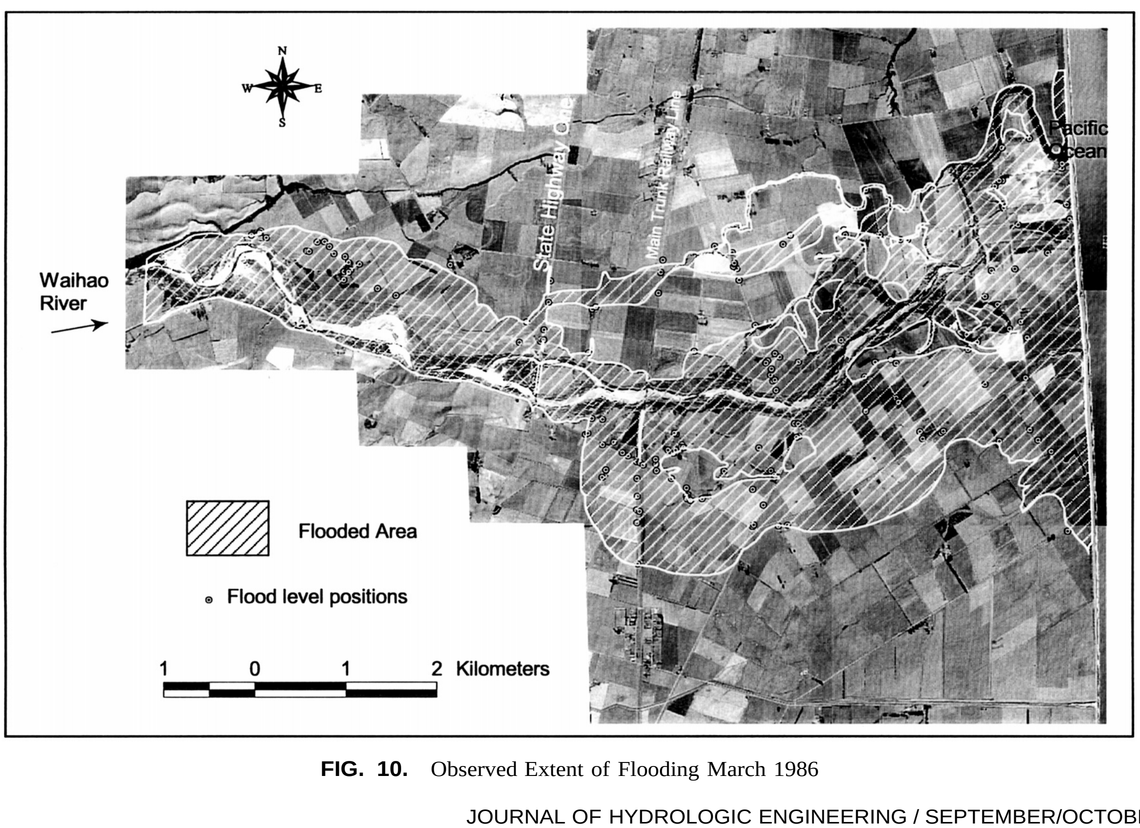

FIG. 10. Observed Extent of Flooding March 1986

terviewed flood plain residents and undertook aerial observation and photography. This information was supplemented for the present study by extensive interviews of 40 flood plain residents, carried out in 1995, nine years and one year after the events (Connell et al. 1998). In almost all cases, the residents could point to positions on buildings, fences, or other structures where the maximum flood levels reached. Many were able to provide still photographs and several had video records, whose time and location could be established to verify the flood levels given by the residents and provide many further levels to use.

Of the 182 flood levels recorded for the 1986 flood, 83 came from resident-supplied information, 95 from photographs or video, and 4 from CRC information. The CRC data points were discarded due to difficulty (after a 10-year interval) of locating them accurately in the field. Of the 108 flood levels recorded for the 1994 flood, 65 came from resident-supplied information and 43 from photographs or video. The ground levels at the points whose flood levels were used were measured in the 1996 GPS survey.

FLOOD SIMULATIONS

Initial (uncalibrated) runs were carried out using both models. These gave the results in Table 3 for the differences between observed and modeled flood levels. The ''uncalibrated'' model runs represent the situation when using the models for prediction, with measured terrain data (corrected for features not in the grid) and channel geometry and the flood hydrographs from:

1. The overtopping and breach analysis for the section of the river with levees
2. The discharge at the recorder routed to the study area for the reach upstream of State Highway One without levees

TABLE 3. Initial, Uncalibrated Differences in Flood Levels between Observed and Modeled

Flood	1986	1986	1994	1994
Program	2de	MIKE11	2de	MIKE11
Mean difference	−0.252	−0.356	−0.098	−0.165
Standard deviation	0.275	0.411	0.314	0.324
Standard deviation of mean	0.023	0.036	0.032	0.032

The extents of flooding from the ''uncalibrated'' Hydro2de runs were a little less than the recorded extents (Figs. 4 and 5). Considerable confidence could be placed in the GPS levels and the standard error of the DTM (±0.3 m; see the next paragraph). Apart from model error, the input values for discharge and roughness could be in error. The exact shape of the discharge hydrograph for the 1986 event (Fig. 4) was unknown (recorder destroyed by event) and there was no independent check on the shape of the downstream hydrographs. The roughness values (Table 1) were reasonable estimates based on a wealth of published data for uniformly rough channels (Hicks and Mason 1991). But there is much less roughness information for broadscale terrain, and this study relied on one writer's experience in calibrating flood flows for previous flood plain studies.

Further runs were carried out increasing some of the breakout discharges, within the range considered ''reasonable.'' Runs that gave the ''best fit'' to measured extents for the 1986 flood are shown in Fig. 11. The changes in discharges are as in Table 2. For these runs, the calculated average flood levels were 0.205 and 0.082 m below the measured average flood level at the measured points, for the 1986 and 1994 events, respectively. It is worth noting that the standard errors of these differences at ±0.271 m for the 1986 flood and ±0.303 m for the 1994 flood, are less than or equal to the expected error of ±0.300 m from terrain elevations generated from the DTM and those surveyed. This included the combined error from each of: DTM error ± 0.264 m (using the data point ground levels), the estimated error in estimating the flood level

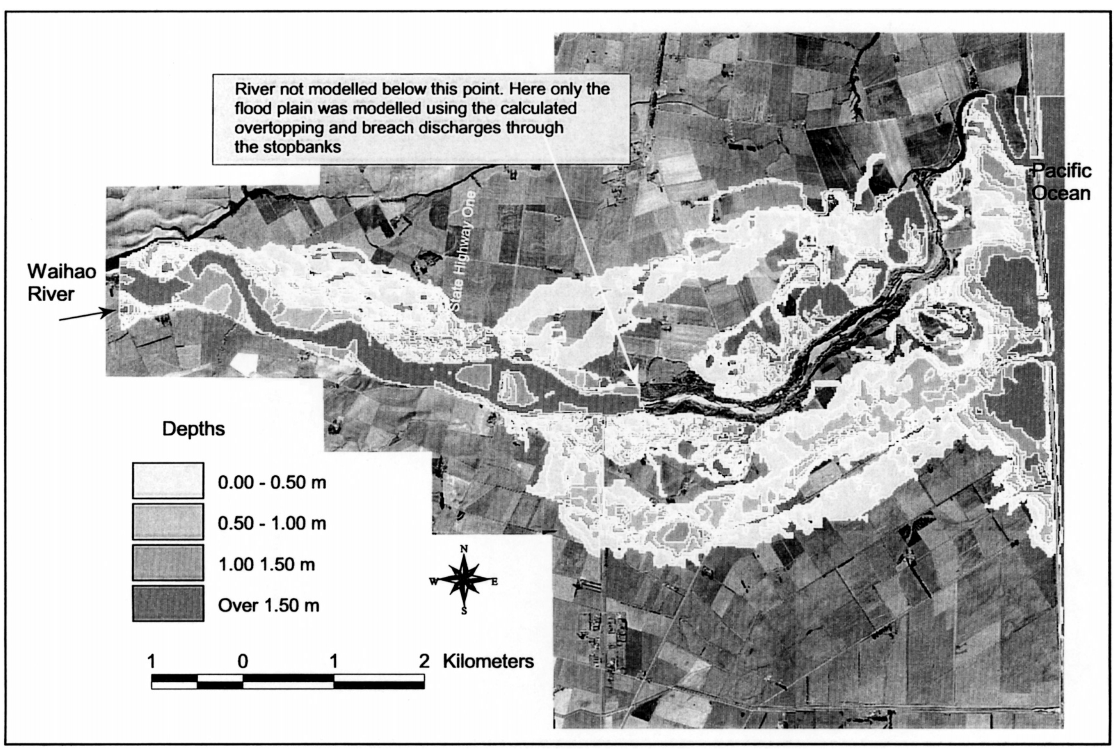

FIG. 11. Modeled Extent of Flooding March 1986 Flood

±0.140 m; and surveyed flood level error ±0.030 m at the measured points.

SEPARATION OF EDGE LEVELS FROM CENTER FLOOD LEVELS

It was suspected that points near the boundaries of the flooded areas were being modeled more successfully than points distant from them. An analysis separating the points in this way showed that the 65 edge level differences were −0.062 ± 0.258 m for the 1986 event, and the 50 edge level differences were −0.054 ± 0.288 m for the 1994 event. The negative figure means that modeled flood levels were on average less than measured flood levels. The remaining center level differences were −0.210 ± 0.261 m for the 1986 event (80 points) and −0.294 ± 0.276 m for the 1994 event (47 points). For both together the figures are −0.060 ± 0.273 m for the edge level differences and −0.237 ± 0.267 m for the center level differences. [These figures differ from an earlier analysis in Connell et al. (1998), as more edge points have been used here, giving a more accurate result. The difference in the edge and center level differences is now 0.177 m, as compared with 0.3 m in Connell et al. (1998).]

This analysis corrects for the average ground levels of the DTM (a result of DTM specification requirement of one standard deviation being less than ±0.3 m) as the dataset for the DTM edge levels was found to be significantly low. The calculated flood levels in the center would have been higher had the DTM been correct. The observed to calculated differences in center levels reduce more if all the edge level difference (i.e., the −0.060 m) is taken out. If the uncorrected values are used, this gives a value −0.107 ± 0.290 m for both events for the edge levels and −0.230 ± 0.291 m for the center levels, i.e., 0.123 m difference in the edge and center levels.

FLOOD DEPTHS

The depth differences after increasing the breakout discharges to calibrate the model to give the best fit to the area flooded were analyzed in the same manner as the flood level differences. This gave the results in Table 4. Figs. 12(a and b) compare calculated and observed depths for the 1986 and 1994 floods, respectively. The standard errors were less than those of the level difference runs; the averages for both floods are ±0.291 m for the flood levels and ±0.259 m for the flood depths. This reduction can be attributed to the DTM high and low areas. When the level data were reworked to correct the suspected ''high'' and ''low'' areas in the DTM (mentioned in the section titled Terrain and Channel Information), they gave errors similar to the depth errors.

FLOOD VELOCITIES

Although the model provides information on velocities, no field-measured velocity data were available to show any comparisons. However, a comparison was undertaken of the calculated velocities and the observed flow depths to see if there was any correlation. It was thought the larger errors could occur at the higher velocities. The results (Fig. 13) showed that there was some correlation, with the r^2 value being 0.55. This was a reasonable result, as errors would arise from areas of high resistance with a low local velocity and higher surrounding velocities, as well as in the deeper areas adjacent to the beach dune with lower velocities.

TABLE 4. Final, Calibrated Differences in Flood Depths between Observed and Modeled

Flood	1986	1986	1994	1994
Program	2de	MIKE11	2de	MIKE11
Mean difference	−0.158	—	−0.192	—
Standard deviation	0.229	—	0.350	—
Standard deviation of mean	0.031	—	0.064	—

ANALYSIS OF ERRORS

There are many potential sources of error in this study: model systematic and parametric errors, measurement errors, and computational errors. As many as possible have been considered, where possible estimated, and where possible taken into account in drawing conclusions.

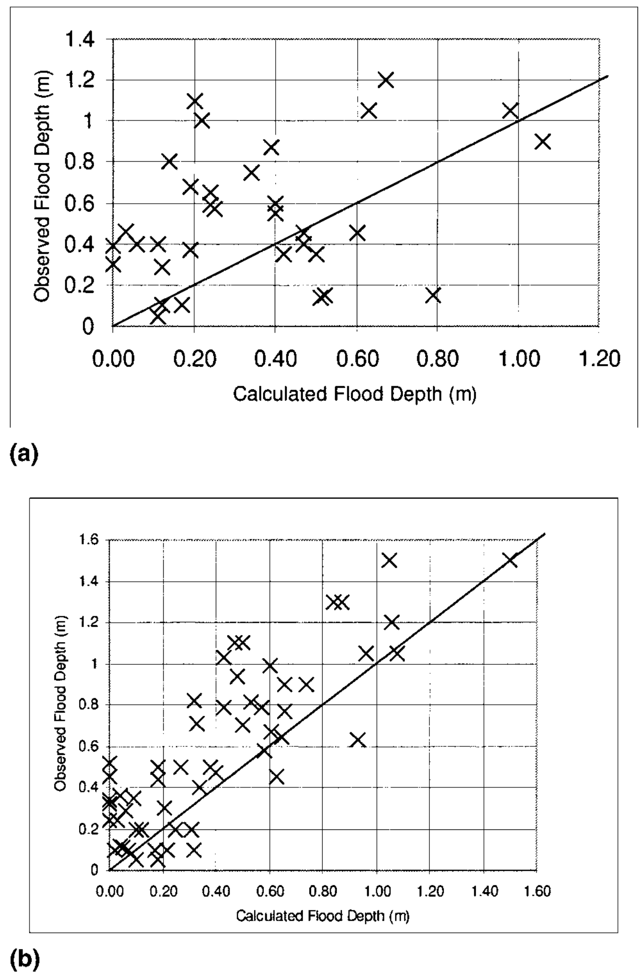

FIG. 12. Observed and Calculated Peak Flood Depths for (a) 1986 Flood; (b) 1994 Flood

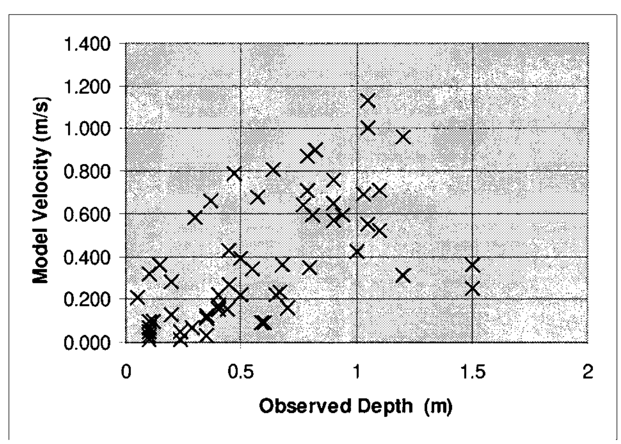

FIG. 13. Difference in Observed and Calculated Peak Depths Compared with Calculated Peak Velocity

The Hydro2de model performance in this study gives no reason to believe the model contains unidentified conceptual or theoretical errors. There are two identified errors arising from the model. It does not completely dry out; therefore, small flood waves that pass over large areas die out and do not pass down the whole area. The second is due to the rectangular grid. This means that small features that are one cell wide, e.g., a channel that flows diagonally across the flood plain, ''zig-zags'' across it in the model, with a length 1.41 times the actual channel length, and a (tangent) slope 0.71 times the actual channel slope. This factor was checked on one of the flood plain subareas using a 10 m grid. This made very little difference to the results, -0.219 ± 0.277 m with a 20 m grid to -0.212 ± 0.273 m with a 10 m grid.

There has been no basis available for independent evaluation of the roughness coefficient estimates given the unavailability of velocity data. All that could be checked was whether the product of resistance and velocity gave the correct depths.

Considerable care was taken to minimize measurement errors and to estimate their magnitude. The GPS survey of 300 ground level/flood level pairs has a claimed ''instrumental'' accuracy of ± 0.02 m. Allowing for the uncertainty introduced by relating the local survey datum to the satellite datum increases this to ± 0.03 m. The relatively high accuracy of the GPS survey of ground levels at the measured positions allowed an independent check of the accuracy of the APS, which was used to generate the DTM. Of the 90,000 points in the DTM, 300 corresponding to the GPS-measured points were checked for level, giving a standard error of ± 0.264 m. This was inside the specification of ± 0.3 m for the DTM.

There is little basis for estimating the reliability of the measurement of (horizontal) positions other than to use the results of the study. This error would have little effect on the results, as the flood plain has about a 1:300 gradient.

Examining the vertical errors using the results of the comparison of the observed and calculated data showed that the standard errors are ± 0.291 m, about the same as the expected errors from the DTM error, the survey error, and the estimated error in the observed levels of ± 0.300 m. It would be expected that the standard errors of the observed and calculated data should be larger than the DTM errors and data and survey errors. A probable reason that these errors are of the same order is that each level was calculated using an area of the DTM, i.e., from the non-uniform parts of the equations (Henderson 1996). This area would have a standard error less than the point standard error of ± 0.264.

Comparing the differences of the observed and calculated depths reduces the error to ± 0.259 m. The reason for the reduction is that one of the errors of the DTM, the high and low areas mentioned above, is eliminated from this analysis. This gives a good validation of the data's integrity.

The higher errors for the MIKE11 analysis as compared with the Hydro2de analysis for the 1986 flood (the 1994 errors are very similar) can be attributed to the modeler having to determine water flow directions and, also, the uniform water level across a cross section. Improved results could be obtained using MIKE11-GIS, which is now available (DHI 1999). This would interpolate flood levels based on the heights at the center of the cross sections. This would not actually be correct according to the analysis undertaken here, as the cross sections were chosen so that the water level would be quite close to level in each channel. Interpolating from adjacent channels would be incorrect in this situation.

DISCUSSION

The model chosen, and the data gathered, provided an appropriate test of two-dimensional flood plain flow modeling in this predominantly rural context. Known limitations of the model, and identified deficiencies in the data, did not prevent useful evaluation being carried out. Use of the uncalibrated model has been shown to be feasible, with data that could be made available in other catchments and flood plains, and parameters that can be estimated from published information. Although the uncalibrated model underestimated flood levels and extents for both field-measured events in this study, the magnitude of underestimation (average about ± 0.050 m between the uncalibrated and the calibrated data) was not large.

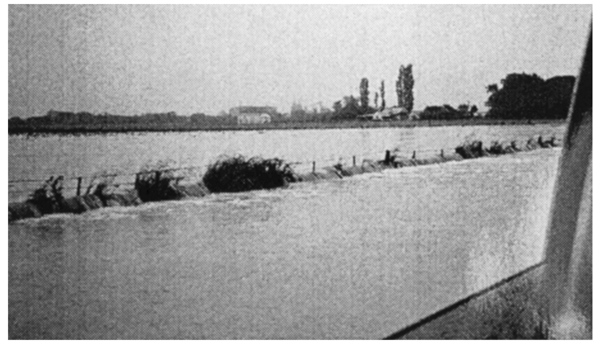

FIG. 14. Water Buildup behind Fenceline

Obtaining a best fit to the extent of the flooded area showed that the model representation would be considerably improved if above-ground buildings, fences and hedges, and in-water hydraulic structures, could be included in the model detail. This would allow the model to determine the higher water levels in the centers of the flood plain subareas. Higher water levels are expected here as fences were observed to gather floating debris during flood plain flow and increase flood levels on their upstream side by 0.1–0.3 m. Fig. 14 shows the difference in downstream and upstream water levels on a fenceline.

The flood level and timing data gathered from flood plain residents were an integral part of the study and a very useful complement to other data gathered. The results used were mostly peak level data, as points that were on the rising limb did not give good comparisons, probably due to the fact that the time of the photograph was not accurately known. The only points used that were not peak data and yet gave good results were those in an area where overtopping had occurred. Just before a breach occurred, these were effectively like a peak level. The mean and standard deviation of the levels from photographic data were not significantly different from those obtained verbally from the residents.

A comparison of the one-dimensional and two-dimensional models was undertaken only on the uncalibrated models. Cunge (1998) points out that calibration of a model destroys its predictive capability. These models need to be set up so that they can model different sized flood events, especially larger events. Even these models have a weakness in this case, as they are based on the particular breaches and overtopping that occurred during these flood events. Therefore, using these models intact for large events would not be advisable without consideration of different breach and overtopping scenarios.

Calibrating the one-dimensional model would not obtain any further information than what is known already from the study. The calibration of the two-dimensional models improved the mean of the flood levels calculated, which would also happen with the one-dimensional analysis. However, it did not improve the standard error of the flood levels as these were below the levels of the field data. Calibration with the one-dimensional model could improve the standard errors, as they are larger than those of the field data, but not a great deal would be achieved in terms of information gained. At best this standard error could be as low as the field data, but this is unlikely, considering the error for the 1986 flood data was nearly 50% over the field data error.

The study is sufficiently encouraging to warrant further development of two-dimensional flood plain flow modeling as an operational tool for regional government authorities and others. But it has also drawn attention to required improvements in terrain and channel data, river discharge and stage estimation, model parameter estimation, and model functionality.

CONCLUSIONS

1. The Hydro2de model, used with published parameter values and premeasured terrain and channel geometry, predicted flood plain flows, flood levels, and flood extents to an operationally useful degree on this 50 km^2 rural flood plain.
2. The results from two-dimensional modeling using Hydro2de were better than those for one-dimensional modeling using MIKE11 with the same common parameters for the 1986 flood. Results of the two approaches were similar for the 1994 flood.
3. Field measurements from two flood events allowed the Hydro2de model to be calibrated to predict the flood levels and flood extents with about the same degree of error as the known DTM errors.
4. The model underpredicted flood levels in the center of the flow. The reason for this was that the model did not contain details of buildings, fences, and hedges. Better representation of these would improve the accuracy of the results in these areas.
5. Flood level data gathered from flood plain residents, where it was on their land, was a valid and useful complement to flood extent data gathered by regional government staff.
6. Better estimation of river stage hydrographs would improve the accuracy of model results.
7. Improved accuracy of digital terrain specification (e.g., to ± 0.15 m in rural areas) from aerial photogrammetric survey or another technique is desirable.
8. Better representation of the DTM surface in the model with a non-uniform grid would improve the accuracy of the results.

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