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Proceedings of the 36th Hydrology and Water Resources Symposium: The art and science of water, 2015 / pp.96-103

Developing water level maps in the joint probability zone influenced by extreme tide and rainfall events

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Flood risk in estuarine regions can be difficult to estimate due to the relationship between extreme ocean levels and catchment generated streamflow. One method for estimating flood risk from two extremes is the design variable method, but this method is typically applied at an individual location along the river reach, so that, it is unclear how feasibly the method can be implemented with output from two dimensional hydraulic models. Adapting the design variable method to a spatial setting presents interesting challenges in terms of computational demand, how to incorporate regions outside the main channel and whether or not the method can be applied independently across grid cell locations. To test these issues, a case study is presented for the Nambucca catchment in the Mid North Coast region, NSW. At this location 144 water level maps were used, based on TUFLOW model output, corresponding to combinations of 12 storm tide return-levels and 12 rainfall event return-levels, from 63%AEP to 0.05%AEP. The design variable method was modified to accommodate partially wet grid cells and applied repeatedly and independently to each grid cell location. The two-dimensional design variable method is computationally intensive but yielded good results for this case study, showing considerable potential for wider application.

1. INTRODUCTION

Estuarine regions upstream from the mouth of the river can be tidally influenced and subject to freshwater streamflow from the catchment. Flood events can be generated due to extreme ocean levels (including storm surge), extreme streamflow or a combination of the two. Furthermore, the coincidence of extreme events typically occurs at a rate much higher than independent random chance because the two events are often associated with a common weather system (Svensson and Jones, 2002; 2004; Hawkes and Svensson, 2006; Zheng et al., 2013). For example, a storm-front may simultaneously cause significant amounts of rainfall on an upstream catchment as well as increased ocean levels due to an inverse barometric effect and wind setup.

The region that is affected by two or more extremes is referred to as the joint probability zone, and the task of flood risk estimation in this zone is complicated due to the dependence of extreme events. The design variable method (Zheng et al., 2014) is a bivariate flood frequency method that is suitable for application in the joint probability zone because it can accommodate extremal dependence. For a region of interest, the hydraulic domain can be considered as having an upstream boundary (streamflow) and a downstream boundary (the ocean). The design variable method requires specification of extreme value probability distributions for rainfall events and storm tide events that characterise each boundary and a parameter controlling their extremal dependence. The method also requires a response table, which is based on output from a hydraulic model and relates the water levels at a location of interest to conditions specified at the boundary events. This method has been successfully applied in numerous case studies (e.g. Zheng et al., 2014), where the hydraulic response table is generated for a set of locations along the main channel and the flood risk is estimated at each location.

Two-dimensional hydraulic models are extensively used for estimating flood levels, because correct specification of the spatial domain is important for correctly determining hydraulic behaviour. Whereas the design variable method has only been applied at specific locations along a main channel, it has never been applied in a two-dimensional setting across a grid of grid cell locations. It is especially important to test if it can be applied at locations in the overbank region, as these grid cells are only inundated once the water reaches their elevation threshold. The simplest method for achieving this is to repeatedly and independently apply the method at each grid cell and subsequently combine the results into a map of water levels. There are three issues that challenge the feasibility of this approach:

1. **Hydraulic consistency** – the method is applied independently at each grid cell. It is important to check that water levels are physically realistic without discontinuities or the presence of disconnected bodies of water.
2. **Partial inundation** – some grid cell locations are not ‘wet’ for all combinations of boundary events. That is, they are only inundated under more extreme boundary conditions. This issue will be addressed by ensuring that there are a sufficient number of boundary cases causing inundation, with which the probability estimate is then constructed.
3. **Computational time** – while the method is applied to a significantly larger number of locations than previously (5 orders of magnitude), efficient design of the algorithm will make sure that the computational burden is manageable.

The issues of spatial consistency, partial inundation and computational time are tested using a case study located in the Nambucca catchment of the Mid North Coast region, NSW.

2. CASE STUDY

The Nambucca River catchment is located in northern New South Wales. The flood levels in the lower reaches of the Nambucca River are influenced by extreme rainfall over the catchment and extreme storm tides at the ocean boundary. Based on work prepared for the Nambucca Shire Council, modelled flood levels for combinations of boundary conditions were available from a Tuflow 1D-2D hydraulic model (WMA, 2013). The model is of the Nambucca River, Warrell Creek and tributaries, and covers a catchment area of 1315 km^2. The model was calibrated to peak flood survey levels (1890-2011) and large historical events (1972, 1977, 2009) recorded at gauges located at Bowraville, Macksville, Stuarts Island and Utungan. Design rainfall data for several locations in the catchment was sourced from Manly Hydraulics Laboratory and the Bureau of Meteorology. A comparison of the design flood levels obtained from the joint probability method with levels obtained from streamflow gaugings was made at the Macksville site due to the long historical record (121 years, 1890-2011) and an additional continuous recording gauge covering the period 1997-2010 (MHL, upstream of Pacific Highway).

Figure 1 shows the domain of the hydraulic model covering the Nambucca River from Bowraville to Nambucca Heads, Taylors Arm to the west of Macksville and Warrell Creek to the South of the Nambucca River. For this domain, 144 water-level maps were generated (WMA, 2013), based on TUFLOW model output, corresponding to combinations of 12 storm tide return-levels and 12 rainfall event return-levels respectively, from 63%AEP to 0.05%AEP, the additional case of 100%AEP represents the lower boundary tide and ‘no rainfall’ cases respectively. Table 1 shows water levels corresponding to the Pacific Highway Bridge at Macksville for these combinations of AEPs.

Figure 1 Nambucca River and Warrell Creek, domain of hydraulic model, Macksville, NSW. Figure obtained from WMA (2013).

Table 1 Flood levels for various combinations of rainfall and tide levels at Macksville (Pacific Highway Bridge) of Nambucca River catchment

		Tide levels (AEPs)											
		100%	**63.1%**	**39.3%**	**18.1%**	**9.5%**	**4.9%**	**2%**	**1%**	**0.5%**	**0.2%**	**0.1%**	**0.05%**
Rainfall levels (AEPs)	**100%**	0.60	1.35	1.38	1.42	1.45	1.48	1.52	1.55	1.58	1.62	1.65	1.68
	63.1%	*1.29*	*1.70*	*1.73*	*1.75*	*1.77*	*1.80*	*1.82*	*1.84*	*1.87*	*1.90*	*1.92*	*1.94*
	39.3%	*1.61*	*1.92*	*1.94*	*1.96*	*1.98*	*2.00*	*2.02*	*2.04*	*2.06*	*2.08*	*2.10*	*2.12*
	18.1%	*1.83*	*2.08*	*2.09*	*2.11*	*2.12*	*2.14*	*2.16*	*2.18*	*2.19*	*2.21*	*2.23*	*2.25*
	9.5%	*2.26*	*2.43*	*2.44*	*2.46*	*2.47*	*2.49*	*2.51*	*2.52*	*2.54*	*2.56*	*2.58*	*2.59*
	4.9%	*2.82*	*2.96*	*2.96*	*2.98*	*2.98*	*2.99*	*3.00*	*3.01*	*3.02*	*3.04*	*3.05*	*3.06*
	2%	*3.32*	*3.42*	*3.42*	*3.43*	*3.44*	*3.45*	*3.46*	*3.46*	*3.47*	*3.48*	*3.49*	*3.50*
	1%	*3.68*	*3.76*	*3.76*	*3.77*	*3.78*	*3.78*	*3.79*	*3.80*	*3.81*	*3.82*	*3.82*	*3.83*

	0.5%	*4.20*	*4.27*	*4.27*	*4.28*	*4.28*	*4.29*	*4.29*	*4.30*	*4.30*	*4.31*	*4.32*	*4.32*
	0.2%	*4.95*	*4.99*	*4.99*	*4.99*	*5.00*	*5.00*	*5.00*	*5.01*	*5.01*	*5.02*	*5.02*	*5.03*
	0.1%	*5.48*	*5.51*	*5.51*	*5.51*	*5.52*	*5.52*	*5.52*	*5.52*	*5.53*	*5.53*	*5.53*	*5.53*
	0.05%	*5.91*	*5.93*	*5.93*	*5.93*	*5.93*	*5.94*	*5.94*	*5.94*	*5.94*	*5.95*	*5.95*	*5.95*

3. THE DESIGN VARIABLE METHOD FOR A SINGLE LOCATION

Application of the design variable method is outlined in Zheng et al. (2014) and will be included in the latest version of Australian Rainfall and Runoff guidelines. Standard application of this method is specified for an individual location where a table of hydraulic response, such as Table 1, has already been determined from a hydraulic model. An additional piece of information is required, which is a parameter controlling the dependence between the rainfall and storm surge events. A map of dependence along the Australian coastline was developed by Zheng et al. (2014) and depends on the locations and duration of a storm event. The critical duration of the Nambucca River catchment was determined to be between 36-48 hours based on standard application of the design storm approach. Based on these storm durations, a dependence parameter, α=0.90, was taken from the map to represent the dependence between extreme rainfall and storm tide.

Algorithms to apply the design variable method are outlined in Zheng et al. (2015). Based on Table 1 and the parameter α=0.90, Figure 2 shows the output water levels from the design variable method at Macksville (Pacific Highway Bridge) for various AEPs. The difference between the water levels for the complete dependence (red dotted line) and complete independence (blue dotted line) is not very large for this location. This indicates that one of the flood-producing mechanisms dominates the final estimates of flood levels. Further investigation of Table 1 shows the dominant mechanism to be the extreme rainfall because at less frequent AEPs there is a larger variation with changes in rainfall than with changes in tide.

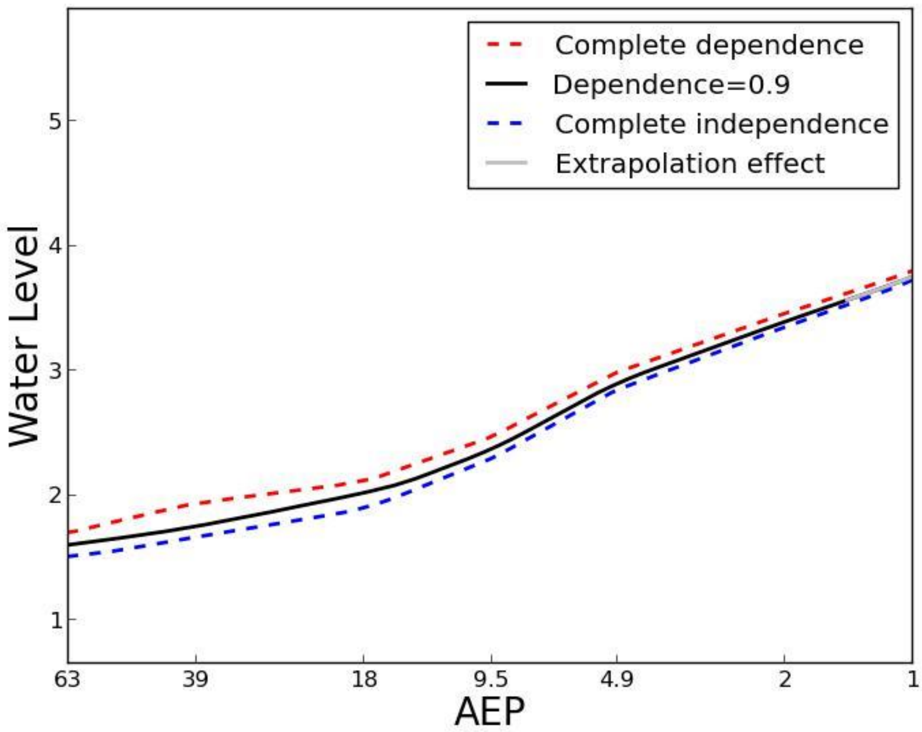

Figure 2 Water levels at Macksville (Pacific Highway Bridge) in the Nambucca river catchment, against the annual exceedance probability (AEP). The dependence parameters with 0 (red dot line) and 1 (blue dot line) represent complete dependence and independence, respectively.

4. THE DESIGN VARIABLE METHOD FOR MULTIPLE LOCATIONS

The design variable method outlined in Section 3 was applied at a single location in the Nambucca river. It can be easily repeated at multiple locations along a river reach, but the main interest of this study was to determine if it could be applied repeatedly to all gridded output locations from the 2-D hydraulic model.

Computational time: The model domain is 915 x 1126 = 1,030,290 grid cells. Of these, there are 101,844 "main channel" grid cells (yellow, Figure 3) and an additional 94,288 "intermittently wetted" grid cells in the overbank region (blue, Figure 3). For each grid cell there are 12 tide cases and 12 flow cases which is a total of 144 runs that form the hydraulic response table for each grid cell location. In other words, a table similar to Table 1 is constructed for each of the ~200,000 inundated locations. The method required 6 hours to calculate results for the main channel grid cells and 12 hours for the "intermittently wetted" grid cells. There is scope to improve the performance of the method by selecting interpolation parameters other than the default values but this may lead to loss of precision without testing (currently interpolated for 100 water levels using 100 increment intervals). The 144 files were located on identical grids to provide ease of construction of the hydraulic response table. It is important that the individual model runs are checked for consistency because any numerical issues in the initial files are transferred to the design variable method. The method was implemented on a desktop, so the total size of all 144 files, at 1.5Gb, was not an issue, but would be a practical consideration if the method were implemented via a web portal.

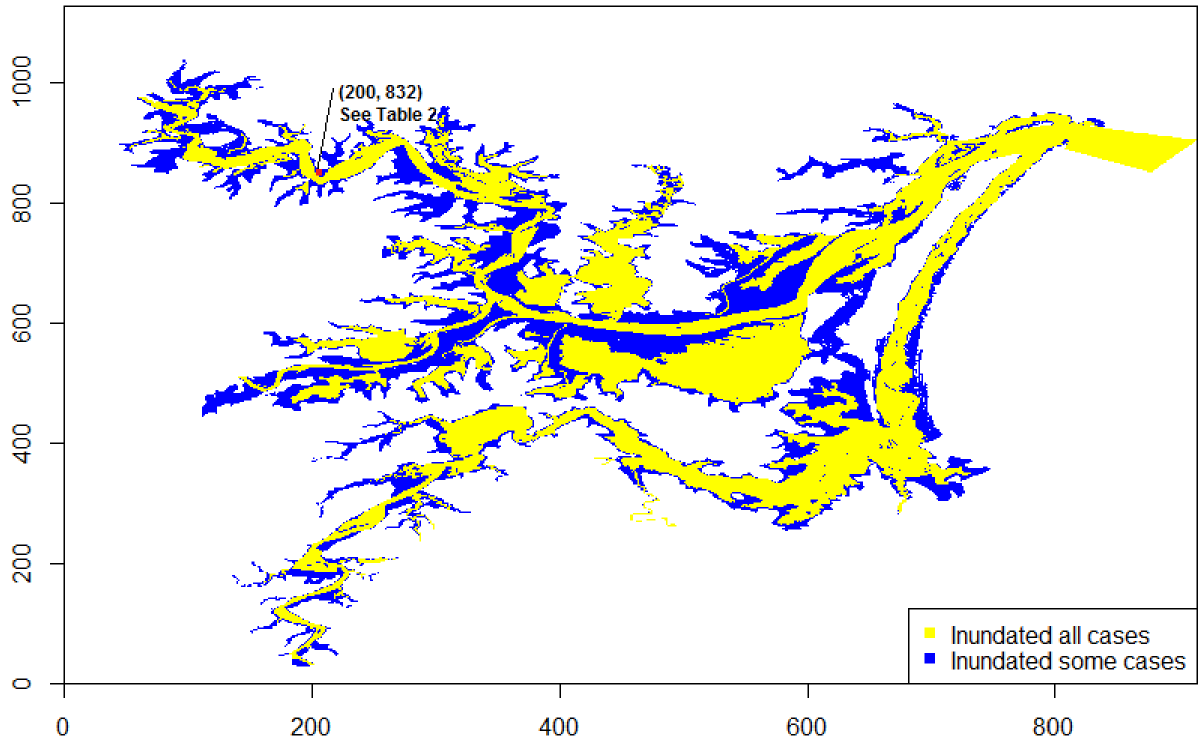

Figure 3 Plot of Nambucca River at Macksville showing locations that are part of the main channel (yellow – inundated all cases – corresponding to no-rain, neap tide case) and showing case of maximum inundation (blue – 0.05% AEP tide = 0.05% AEP flow).

Partial inundation: For "main channel" grid cells, the method is identical to standard application of the design variable method, as in Section 3. For the "intermittently wetted grid cells" the hydraulic response table is only partially complete and this is a difference from the one-dimensional implementation. Table 2 shows a hydraulic response table with *NA* values that indicate combinations when the grid cell was dry. When the NA values occur in a block like this, the algorithm can work exactly the same as for a full table (i.e. Table 1) by simply ignoring those combinations. However, there are a number of cases where partial cross sections can be problematic:

- Cases where the table has only 1 column of values and the rest is NAs. The contour routine cannot accommodate these cases. Even though there are combinations of events that cause the grid cell to be "wet", an answer cannot be determined at these locations.
- The missing values cause an "island effect" in that they surround a value in the hydraulic response table. This suggests that the hydraulic model which created the grid files may have numerical issues. Nonetheless, the design variable method cannot accommodate this case and so it must be handled with algorithms that check for consistency of the input.
- Cases where the NA values diagonally traverse the table. These can be accommodated by the interpolation routine, but care is needed.
- If users are missing one or more model runs from the hydraulic response table, for example, the (0.05%, 0.05%) run. This NA value cannot be accommodated because it does not represent a case where the grid cell was dry, but a case where the data is truly missing.

Hydraulic consistency: The issue of spatial consistency of the output is deferred until the discussion of results (Section 5). There is an additional consistency issue, which is that the code assumes that a hydraulic response table is monotonic increasing. In theory, more extreme inputs should lead to higher water levels. In practice, it is possible for this relationship to be violated (see grey shaded values in Table 2). One way for this to occur is if the method for calculating a boundary condition has altered, e.g. the design rainfall peak may have increased, but the temporal-pattern/routing/loss-parameters or other assumptions may have caused a lower flood peak that leads to a lower water level. The issues of non-monotonic increasing values was detected for the Macksville case study. The issue is related to how the upper boundary was derived – having two tributaries it is not straightforward to assign a single AEP to the boundary condition. Also, different durations have different temporal patterns. Usually, the discrepancy in water levels is minor, as in Table 2. The mechanism for handling these values is to enforce monotonicity on the hydraulic response table (by propagating the higher water level) and to flag locations to the end-user where this has occurred.

Table 2 Hydraulic response table for grid cell (200,832) (see Figure 3 for location). NA values show cases when a grid cell is dry. Highlighted cases of rain=0.05% with tide = (5% or 0.05%) are not monotonic increasing when compared to the values at lower AEPs.

		Storm Tide (% AEP)											
		Lower Bound	63%	39%	18%	10%	5%	2%	1%	0.5%	0.2%	0.1%	0.05%
Rainfall levels (AEPs)	No Rain	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	63%	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	39%	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	18%	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
	10%	5.68	5.73	5.73	5.73	5.73	5.74	5.74	5.74	5.75	5.75	5.75	5.76
	5%	7.04	7.04	7.04	7.04	7.04	7.04	7.04	7.04	7.05	7.05	7.05	7.05
	2%	7.63	7.63	7.63	7.63	7.63	7.63	7.63	7.63	7.63	7.63	7.64	7.64
	1%	8.31	8.31	8.31	8.31	8.31	8.31	8.31	8.31	8.31	8.31	8.31	8.31
	0.5%	8.79	8.79	8.79	8.79	8.79	8.79	8.79	8.79	8.79	8.79	8.79	8.79
	0.2%	9.43	9.44	9.44	9.44	9.44	9.44	9.44	9.44	9.44	9.44	9.44	9.44
	0.1%	9.98	9.99	9.99	9.99	9.99	9.99	9.99	9.99	9.99	9.99	9.99	9.99
	0.05%	10.44	10.44	10.44	10.44	10.44	10.43	10.44	10.44	10.44	10.44	10.44	10.43

5. RESULTS

The issue of hydraulic consistency was the main concern for two-dimensional application of the method and the main reason for this pilot study. This critical question is whether the spatial consistency of the hydraulic model, provided as input to the design variable method, is retained even though the method is applied independently at each grid cell. Figure 4 shows output from the design variable method. At a coarse scale these figures appear smooth. The smoothness of the results was also confirmed with closer inspection of the values at higher resolutions (not shown). At this stage, more work is required to validate the approach. For example a longitudinal profile of output water levels could be compared to water levels from a 'standard' design event (e.g. 1% AEP profile).

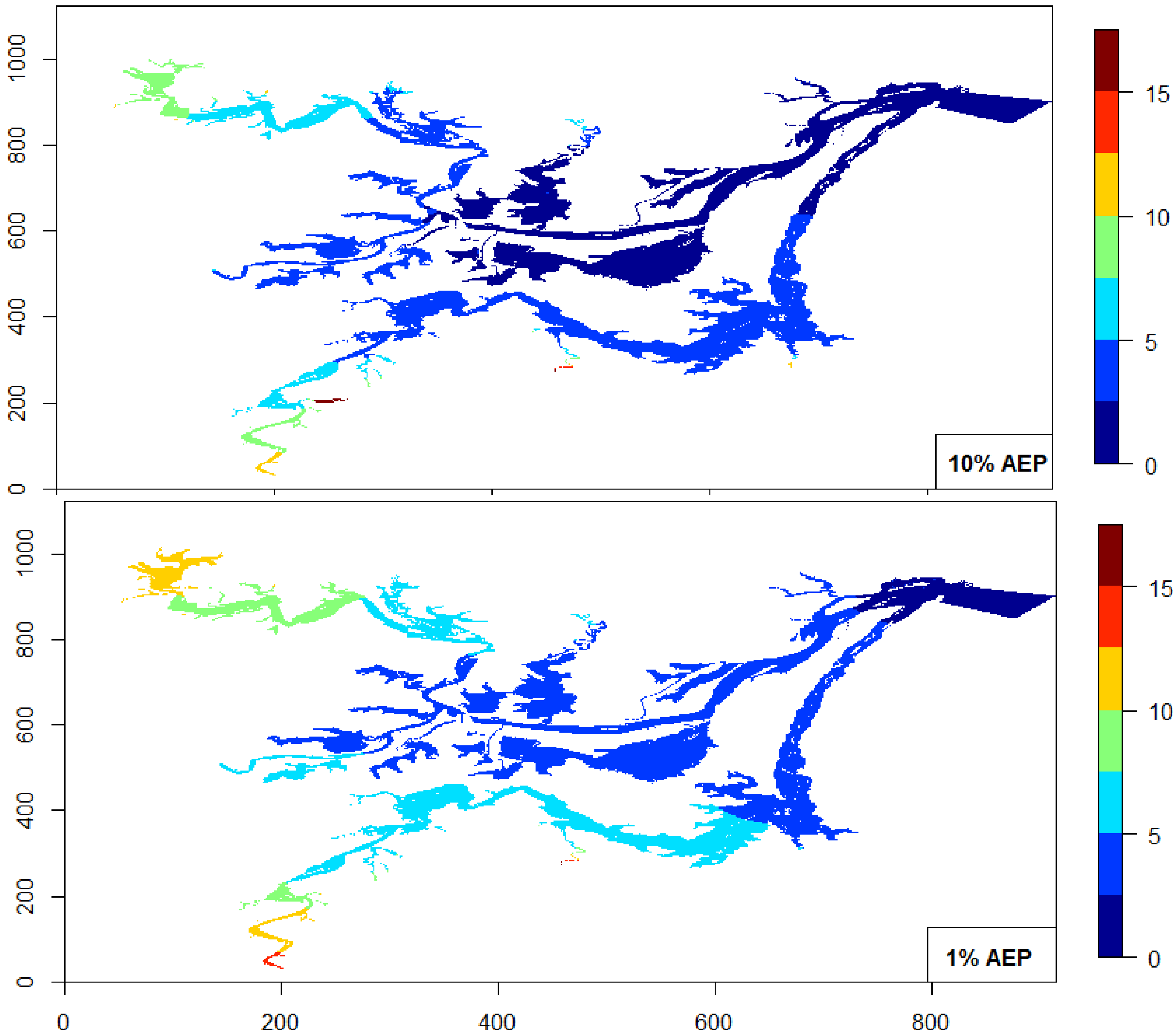

Figure 4 Water level contours (in m) for 10% AEP and 1% AEP cases. The output from design variable method shows good spatial consistency

6. CONCLUSIONS

This paper demonstrated application of the design-variable method to two-dimensional water levels produced by the hydraulic model TUFLOW at Macksville in the Mid North Coast region, NSW. Macksville is situated below two tributary catchments and is also tidally influenced, so that determining flood water levels in this region is a joint probability problem. A number of issues were considered in applying the method:

1. **Hydraulic consistency**: despite the method being applied independently to grid cell locations, the results (Figure 4) show spatial consistency at a coarse scale. This suggests that the design variable method does not cause the output to diverge between grid cell

locations. An additional issue of hydraulic consistency arises where combinations of more extreme floods producing less extreme water levels. This issue is common with the one-dimensional implementation and occurs due to changes in the boundary conditions (e.g. changes in temporal pattern). This issue was addressed by artificially raising the water level at affected locations to enforce monotonicity in the hydraulic response table. Additional output should be provided to give feedback to the user where this has occurred and to flag any other issues with the input needing to be checked.

2. **Partial inundation**: a key difference from the single-site implementation was the inclusion of locations that were not strictly located within the main channel. These grid cells are only made wet for less frequent events, leading to some cases where the hydraulic response table had NA values. These cases were managed by requiring the NA values to be in a contiguous block and to place restrictions on the total proportion of NA values.
3. **Computational efficiency**: Whereas the design variable method has previously been applied to single locations along a main river reach, this study applied the method to approximately 200,000 locations. The computational time was manageable (18 hours), due to the efficiency of the underlying one-dimensional implementation, and with further room for improvement by adjusting numerical parameters controlling the integration. The one-dimensional version of the method can be easily deployed on a web-server given the low overhead requirements for calculation and data transfer. However, with 1.5Gb input files and 18 hours computation, further consideration needs to be given to how end-users might access or implement the two-dimensional version.

Overall, the results from this pilot study are promising, but more work is needed to validate the spatial consistency of the method and apply it to more case studies . They indicate that the method would be feasible for two-dimensional application, which is appealing given the popularity of two-dimensional flood models.

7. ACKNOWLEDGMENTS

This work was a collaboration between WMA Water who provided guidance on the Macksville case study and University of Adelaide who applied the design variable method. We are grateful to the two anonymous reviewers who provided many helpful comments. We acknowledge Geoscience Australia who provided funding towards the revision of Australian Rainfall and Runoff, which sponsored this work.

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