HOW MUCH TIME DO WE HAVE? THE VARIABILITY OF RIVERINE FLOOD WAVE ROUTING SPEED.

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Introduction

During the 2010/2011 summer the most significant flooding in over 30 years was experienced throughout Australia's eastern states. Localised flash flooding occurred in many regions and riverine flooding affected numerous cities and towns along a number of Australia's major rivers. The Murrumbidgee River downstream of Burrinjuck Dam experienced relatively significant flooding (approximately 10% AEP) during December 2010. Analysis of this event and comparison to the 1974 flood event (approximate 1.5% AEP) revealed that the speed at which the 2010 event propagated was approximately half that of the 1974 event. This observation highlights the commonly used axiom that "every flood is unique".

Hydrologists use flood routing techniques to predict flood wave characteristics as they propagate along a river channel. A range of methods are used but all generally have the same goal in mind, that is, to determine the amount of attenuation that occurs with wave propagation as well as the flood wave travel time. The SES use flood plans, prepared for flood liable areas, to layout how flood response should occur. A key part of these flood plans, particularly for those towns located on major rivers, is how long people have from the first warning until the arrival of flows which may isolate and then inundate the location in question. It is therefore of upmost importance that calculation of flood wave speed is accurate, although due to the complex nature of flooding this has historically proved to be complicated.

Given routing speed variability and its importance to flood response planning, it was of interest to compare routing techniques. In particular comparison of simple 1D modelling techniques to full 2D hydrodynamic models were assessed. Comparisons were made against observed data to check the relative effectiveness of the model systems applied and a range of events were modelled in order to examine how the different methods coped with varying event magnitudes.

Variability Of Wave Speed Propogation

For most natural river systems the velocity of the flood wave is approximately equivalent to wave celerity. In natural river channels wave celerity is a function of flow and channel characteristics with flood wave celerity able to be approximated by the following equation:

$$c = \sqrt{(g \times \frac{A}{B})} \qquad \text{(Equation 1)}$$

In Equation 1, *g* is the acceleration due to earth's gravity, *A* is the cross sectional flow area of the river reach and *B* is the top of cross section width at the cross section. Parameters *A and B* are displayed in Figure 1 along with *b* which represents the in-bank width and *y* the elevation at which overbank flooding occurs.

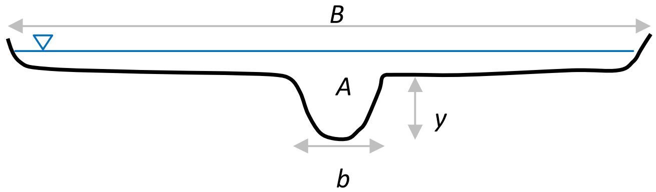

Figure 1: Typical River Cross Section

It must be noted that *A* is not only a function of the channel cross section but also a function of flow depth. This means that any increase in depth will lead to an increase in flow area.

Cunge, Holly and Verwey (1994) exemplify the behaviour of a flood wave in a composite river cross section (such as that displayed in Figure 1) with experiments conducted on the Tvertsa River in Russia. The results of this experiment led to a number of interesting observations:

- Wave celerity varies with the depth of water in the flooded valley;
- The maximum observed celerity occurred at in-bank full flow;
- Once flow exceeds the in-bank capacity and water begins to flow in the overbank, wave celerity decreases. Referring back to Equation 1 this is due to an increase in channel width (*B)*;
- Celerity decreases more rapidly from its in-bank maximum to its flooded valley minimum when *B* is large;
- Minimum value of celerity is obtained approximately at the same depth as the minimum value of the steady flow velocity in the flooded range, and;
- For depths exceeding this depth the celerity and flow velocities increase at the same rate.

Therefore it can be seen that the findings in the Tvertsa River study correlate with those expected from Equation 1. This demonstrates that wave celerity increases with an increase in flow depth only if the flow width remains relatively unchanged (i.e. in the case of in-bank flow). When flow exceeds the in-bank the flow width increases dramatically in contrast to flow area, thus reducing wave celerity. As flood waters continue rising in the overbank section wave celerity starts increasing again due to the increase in area/width ratio.

Wong (1984) also noticed a similar pattern in his analysis of six reaches in three separate Australian rivers. Hundreds of events were analysed with comparisons being made between wave speed and peak discharge. Wong's research also agreed with the Tvertsa River study in that wave celerity was not only dependant on discharge but also on channel characteristics. Importantly Wong's work highlighted the non-linearity of the system by fitting various power functions to each set of data.

These studies then highlights an issue which has the potential to create risk, i.e. in-bank flood events are generally of minimal interest for many people due their benign nature and the majority of people have little experience with large floods due to the rarity of such events. Thus impressions of flood travel time may naturally be informed by the most common "flood" events, those minor out of bank events such as the December 2010, which have the slowest travel times.

Flood Wave Speed Observations

To highlight the flow/celerity relationship the two most recent significant Murrumbidgee River flood events were investigated. Peak flows and travel times were compared for the December 2010 and August 1974 flood events. The table below shows that the August 1974 flood event was larger and moved more rapidly than the December 2010 flood event. With an approximate doubling of flow came a halving of travel time.

Table 1: Comparison of 1974 and 2010 Murrumbidgee River Flood Events

	Event	Time of Peak	Cumulative Time (hr)	Flow (GL/day)	Level
d/s Burrinjuck Dam	2010 1974	3/12/2010 12:00 29/8/1974 12:00	0 0	- 393	- 13.1
Gundagai	2010 1974	4/12/2010 13:00 30/08/1974 0:00	25 12	278 482	10.2 11.0
Wagga Wagga	2010 1974	6/12/2010 14:00 30/08/1974 20:00	74 32	203 493	9.7 10.7
Narrandera	2010 1974	12/12/2010 9:00 3/09/1974 0:00	213 108	98 266	8.0 9.0

To more firmly establish the indicative trend observed from the above results, the 13 largest flood events at Wagga with appropriate data for analysis were analysed. The travel time between Gundagai and Wagga on the Murrumbidgee River was calculated along with the average flow at these two gauges. The same trend was noticed with larger events travelling more rapidly than events with lower discharge. Two obvious outliers are present with a possible explanation being that these two events may have received a substantial portion of discharge from local tributary flow around Wagga (Tumut River and Tarcutta Creek). This could potentially increase the average flow without significantly decreasing the travel time.

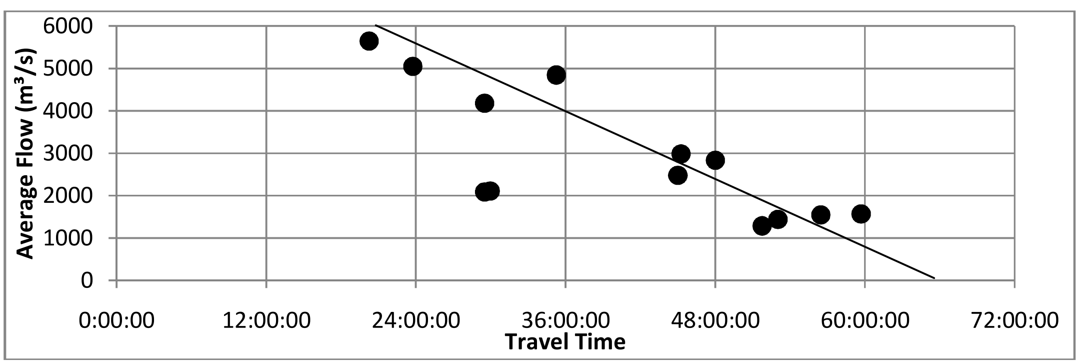

Figure 1: Gundagai to Wagga, Historical Floods– Flood Wave Travel Times

Routing Estimates from Alternative Methods

An approximate 120 km section of the Murrumbidgee River from Eringoarrah to near Collingullie was modelled using two significantly different modelling techniques. The modelled region is located at the convergence of two significantly different geographical types. The upper portions of the reach are significantly more mountainous with the lower regions flatter with a large floodplain. This kind of variation in land type is common in Australian rivers thus making this region a suitable area to perform such a study. Three different events were tested, the 2010 (10% AEP), 1974 (1.5% AEP) and an extreme event with an approximate 0.2% AEP.

It is important to note that this exercise was not about confirming the suitability of one modelling method over another as equal amounts of effort (including calibration) were not expended for each. Instead the concept was to see how little effort might be expended to get a decent result for flood wave travel time estimation.

Note it was also the paper's intent to trial the Muskingum Cunge routing method however this was found to be cumbersome and overly sensitive to parameters which were interpretative in nature and hence relied on a great deal of user knowledge/skill. Given that the intent of this paper is to compare various methods to see if simpler methods could produce robust flood wave speed estimates, and given that the Muskingum Cunge approach was anything but simple, it did not seem appropriate to continue on with it.

1D model – simplified approach

A 1D quasi 2D model was used in conjunction with a highly simplified approach. The approach was:

- Extract cross-sections at chainage spacing of ~ 5 km from a high resolution DTM grid utilised in the below mentioned 2D model build;
- Build model. Note there was no separation of the in-bank and floodplain flow paths then and this is far from the recommended approach when modelling extensive floodplain systems. Again the intent is simply to see how little effort might achieve a reasonable result for flood wave travel time estimation, not to confirm the suitability (or lack thereof) of a 1d approach generally (the suitability of which has been well and truly confirmed in the past); and
- Apply hydrograph and downstream tail water as per the 2D model build below.

No calibration was carried out, nor were results compared to the 1974 or 2010 December event observed peak level set at all. The model was however built using standard roughness values for the area that had previously been shown to achieve a reasonable calibration at Wagga Wagga.

2D model

A 2D model of the Murrumbidgee River and surrounding floodplain was built in a hydrodynamic modelling package. The model build work was carried out as part of a paid consultancy project and as such time was available for a far more comprehensive model build job than that undertaken in regards to the 1D model work described above.

The model grid size utilised in the model build process is a finite difference grid of 40 m by 40 m. The model grid size was adopted after considering the extent of the modelling area and the modelling run times involved. As the model was constructed purely in 2D some manipulation of the grid was required to provide a reasonable approximation of the River's in-bank hydraulic properties.

The model was calibrated to an event with an AEP of ~ 1.5%. Calibration utilised a range of observed data from the 1974 flood event including gauged water levels, approximately 100 surveyed peak flood levels and peak flood extents. The calibration focussed on the replication of peak flood levels and did not consider flood wave travel speed.

To verify that the model was correctly calculating celerity an additional event was modelled. The observed flood wave travel time between the Erringorrah gauge (situated at the upstream end of the model domain) and the Hampden Bridge gauge (at Wagga) was calculated for the second peak of the December 2010 event (approximate 5 year ARI). As only water levels were available at Erringorrah the flow was calculated via linear interpolation of flows at the Gundagai and Wagga gauges. This event was used as it was the only substantial event on record at both gauges. The results indicate that the model not only reproduced peak water level heights but also flood wave celerity.

Table 2: Comparison of Flood Wave Travel Times

	Travel Time (hr)	% Difference
Observed	28	-
2D Model	26	-7%
1D Model	56	+107%

Results

Table 3 displays the modelled flood wave travel time and percent attenuation of the three flood events mentioned above. The flood wave travels significantly slower in the 1D model than it does in the 2D model or the real river system. In addition to this the percent attenuated varied greatly between the 1D and 2D models. For the two larger events the 1D model attenuated flow significantly more that what would occur in reality.

Table 3: Comparison of 1D and 2D Model Results

Event	1D Model		2D Model	
	Travel Time (hrs)	Attenuation (%)	Travel Time (hrs)	Attenuation (%)
2010	60	13	48	13
1974	48	18	28	6
Extreme Event	36	14	20	3

Note that the 1D model, not being schematised to separate in-bank and floodplain flow, will calculate wave celerity using the average depth at each cross section. This will likely lead to exaggerated flood travel time as floodplain depth (and extent) will tend to produce average depths much less than main channel/river depth. More accurate results would be expected by separating over bank and in-bank flow components. The tendency of 1D models to lump together flow paths of different levels will often results in the overestimation of flood travel time, although the degree to which this occurs will depend on the degree of separation of various flow paths via model schematisation.

Further Analysis of the 2D model

As the 2D model can accurately reproduce both peak flood levels and flood wave celerity further analysis has been undertaken to determine the difference in wave celerity for events of various discharges. Nineteen events of various discharges ranging from 1,000 m^3/s to 10,000 m^3/s at 500 m^3/s intervals were modelled in the calibrated river reach mentioned above.

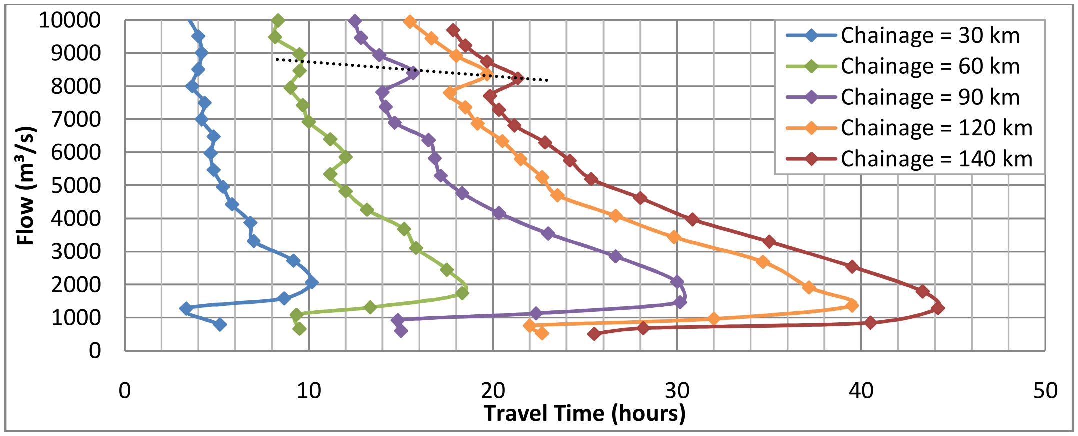

Figure 2: Modelled 2D Flood Wave Travel Times for Various Flows

The figure above displays the peak discharge versus travel time at five cross sections along the reach. It can be seen that events with a peak discharge of approximately 2,000 - 2,500 m³/s have the longest travel time and therefore the slowest wave celerity. Interestingly this is approximately the peak discharge of the most recent Murrumbidgee River Flood event in December 2010 which has an AEP of approximately 10%. It is these kinds of events that happen relatively frequently that are at risk of being used to determine flood wave travel time by on ground personnel. It is also interesting to note that at approximately 8,500 m³/s (note the black dotted line on Figure 2), travel times increase slightly before decreasing again. This is likely due to the flood wave overtopping the primary flood plain much the same way velocities decreased when the in-bank capacity was exceeded.

Conclusion

Reliable estimates of flood travel time are vital for the work the SES do in preparing to respond to a flood emergency. As shown herein flood travel time can vary greatly for the same reach of river depending on event magnitude. It is important to recognise that the flood most often observed, i.e. the out of bank flood, will tend to greatly exaggerate the flood travel time (by two to three times relative to faster larger floods).

SES flood plans prepared for flood liable areas located on major rivers should be sure to utilise either worst case estimates for flood travel time or provide specific travel times for events of varying magnitude. In utilising models to derive flood travel time, the SES should be mindful that:

- A calibrated flood model under the NSW Floodplain Risk Management Program will be calibrated with a focus on flow and flood level, not necessarily travel time;
- A 1D model, unless schematised to separate out flow paths (in-bank and floodplain at least), will tend to overestimate flood travel time due to the use of mean depth in celerity calculations; and
- A calibrated 2D model can be used to achieve accurate estimates of flood travel time that scale well (i.e. events of various magnitude can be modelled within the one system).

References

Cunge, J, Holly, F, Verwey, A 1980, 'Practical Aspects of Computational River Hydraulics', Pitman Publishing Limited, London

Wong, T 1984, 'Improved Parameters and Procedures for Flood Routing in Rivers', Monash University, Melbourne