Flood Hazards to Buildings: Importation of Substandard Design Methods

At the suggestion of the Chief Executive of IPENZ, Andrew Cleland, I wish to report to the Department of Building and Housing my concerns at the increasing use of imported design methods which demonstrably fail to meet Surface Water standards set in our Building Codes.

I understand that the people using these substandard methods have been given the impression that the imported techniques are new, making the Building Code obsolete so that checking for compliance with that code is irrelevant and unnecessary.

1. The St Venant Solution

In the 1970s and 1980s the Ministry of Works and Development conducted an exhaustive examination of flow design techniques as part of the quality assurance for the design of the hydropower canal networks then under construction. Because of the scale of these canals, software development and full-scale field commissioning tests took some 100 man-years! As an example, Figure 1 shows part of the arrangements for dambreak testing, involving fully opening and closing a two-tonne irrigation canal gate in under 10 seconds.

Figure 1. Dambreak Wave Testing on the Rangitata Diversion Race: 1977

My responsibility in these tests was the development of the MWD Rivers package and supervision of numerical model calibration for the various waterways.

By 1980 the Rivers package had implemented solutions by both energy (Bernoulli) analysis and momentum (St Venant) analysis.

It was found that the two solutions gave perceptibly different results, and after possibly the most exhaustive field verification in the world, the St Venant solution was rejected for all but specialist applications involving surges and hydraulic jumps.

2. The 2D Shallow Water Solution

In the mid-1980s, I was commissioned to undertake modelling investigations for the new Tauranga Harbour Bridge. These were undertaken using two-dimensional solutions of the Shallow Water equations – see Figure 2.

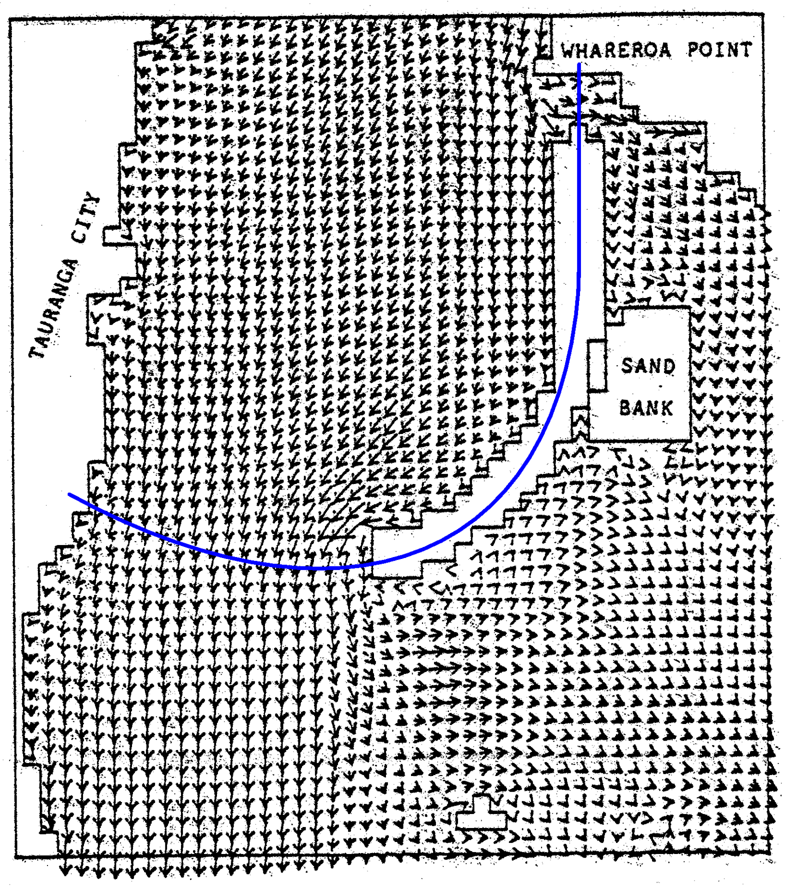

Figure 2. 2D Model of Tidal Flows through Proposed Tauranga Harbour Bridge Waterway

Again it was found that the shallow water solution differed significantly from the Bernoulli solution specified by current New Zealand bridge standards, so considerable adjustment to the 2D solution was required to make it compliant through the waterway under the bridge.

3. Building Code Updates

This preference for the Bernoulli solution over both the St Venant and Shallow Water solution was reflected in the last revision of Approved Document E1 of the Building Code some ten years later, as I had a review role in that revision.

While I would have preferred some extension to examples of application of the Bernoulli solution in unsteady flow conditions, I certainly endorsed the methods applied in Verification Method E1/VM1 with full confidence that these were superior to St Venant and Shallow Water solutions for the chosen examples.

If asked for the basis of this preference at the time, I would have drawn attention to the differences between the alternative solutions, presenting as an example a test problem such as that attached in Appendix A. This has recently been widely circulated around the New Zealand hydraulic modelling community.

As well as the authoritative New Zealand tests at large scale, the Bernoulli solution has been widely endorsed by laboratory and field tests in many countries. In contrast, I am not aware that anyone (including the software importers) has offered any experimental verification for claims of superiority of the two alternative solutions in situations such as the examples given in Approved Document E1.

Therefore if solutions found for the typical drainage problem in Appendix A differ from that found using Verification Method E1/VM1, the software analysis will be clearly substandard, often giving an overestimate of channel flow capacity to the order of 15-60%.

Widespread adoption of these imported solutions would return us to the early 1980s in terms of New Zealand technology, but lacking the safeguards then applied of routine checks for false gains in energy. Of course, new methods of presentation have since evolved (for example, the use of Email for distribution of reports was unheard of then), but core solutions to the three equations were certainly then available for critical comparison and assessment.

To put this in context, universal adoption of these "new" methods would more than negate the effect of the increases in rainfall intensity recently specified in "Preparing for Climate Change" by the Ministry for the Environment. Design drain sizes would then if anything undergo a net *reduction* compared with those designed for the same catchments under the rules applying at the time of the last update to Approved Document E1.

4. Conclusion

Our standards rest on a long history of expert institutional memory, and should not lightly be ignored. If there is a Commission of Enquiry resulting from the next major flood disaster, questions are bound to be asked why nothing was done to stop the use of design methods which are demonstrably substandard. I do not wish to be accused of failing to alert the authorities when I am aware that such methods are increasingly widely used in many centres.

Declaration of possible conflict of interest: *Alastair Barnett is Director of HYDRA Software Ltd, a company offering both packaged Bernoulli and St Venant solutions for the past 20 years. As a New Zealand-based company, our interest in the NZ regulatory environment is stronger than that of direct competitors based in other jurisdictions.*

Appendix A

Discrepancies Between Equations: Demonstration Problem

The Bernoulli equation is the basis for calculations in standards such as Verification Method E1/VM1 included with Approved Document for New Zealand Building Code Clause E1 “Surface Water”.

Two more equations have recently been proposed as alternatives for urban drainage modelling: the St Venant Equation and the 2D Shallow Water Equation. However it is not widely publicised that solutions to these two equations are incompatible with the standard Bernoulli equation, and indeed with each other. The following demonstration problem has been designed to expose these discrepancies. It is simple enough to be hand calculated by the method of Verification Method E1/VM1, and should therefore take less than an hour to set up and run in any good computational modelling package.

If the resulting discrepancies are significant, consideration should be given to limiting the use of the defective solutions, or at least to making compensating corrections to the models used.

The Problem

An existing open drain is rectangular in cross-section and 9m wide. Between 10m and 110m from the coast the drain runs through higher ground, so a narrower channel reach has been cut through, based on a 1.8m diameter semicircular bottom shape with vertical walls above the open semicircle. Downstream of the cutting, the channel invert is at datum level, and upstream of the cutting the channel invert is at 0.100m above datum, giving a constant bed slope of 0.1% to the connecting semicircular channel reach.

The channel lining is concrete throughout, with a Manning n of 0.013. The semicircular channel reach is square ended in plan at both ends.

At the coast the static water level applying for flood design calculations has been assessed as 0.9m above datum, and upstream of the higher ground a road runs beside the drain, with a minimum road crown level of 1.155m above datum measured at 120m from the coast. The Council is concerned about possible flooding of low-lying properties behind the road, and has commissioned a study to recalculate the flood flows for various return periods.

They now require a flow capacity check of the last 120m of the drain so they can determine if the revised 100 year ARI flood flow will overtop the road crown.

Approach

The intention is for modellers to test their own modelling techniques by setting up and solving the Problem as defined above. As a demonstration this has already been done on the *AULOS* package marketed by HYDRA Software Ltd, with results as presented in the next page. Those results should be read in conjunction with the accompanying Notes.

(Dr) A.G. Barnett 11 November 2010

Solutions

1. Bernoulli
   Method E1/VM1 is simple to apply, because the set Problem has been deliberately designed to give a uniform profile exactly filling the semicircle. The semicircular reach can then be regarded as the bottom half of a circular pipe culvert. The corresponding flow for a half full 1.8m diameter pipe of slope 0.001 and Manning n 0.013 is 1.817 cumecs. Applying Paragraph 4.1.8 with T_w = 0.9m and k_e = 0.5 for square ends then gives the upstream depth as H_w = 1.056m, or level 1.156m which just overtops the road crown.

   Therefore the drain capacity estimate by the method provided by the New Zealand standard can be taken as **1.817 cumecs.**

   The *AULOS* test model gave (see note 1) a flow capacity of 1.816 cumecs for a semicircular pipe and 1.807 cumecs for a segmented open channel.

2. St Venant
   The *AULOS* test model gave (see note 2) a flow capacity of 2.489 cumecs, an increase of 36.9% on the standard method.

3. 2D Shallow Water
   The *AULOS* test model gave (see note 3) a flow capacity of 2.919 cumecs, an increase of 60.6% on the standard method.

Significance

The larger the flow capacity estimated, the less conservative the design calculations. Suppose the Council found that the 100 year ARI flood was in fact 2.9 cumecs. Then if the 2D Shallow Water result was accepted, the Council would find the road would actually be overtopped roughly (see Note 4) every 10 years according to the standard model.

If the Council found that the 100 year ARI flood was in fact 2.5 cumecs, then if the St Venant result was accepted, the Council would find the road would actually be overtopped roughly (see Note 4) every 20 years according to the standard model.

Paragraph 1.0.1 of Verification Method E1/VM1 states "This Verification Method shall be used only if the *territorial authority* does not have more accurate data available from sophisticated hydrological modelling of the catchment undertaken as part of its flood management plans." This implies a requirement on the Council to apply an accuracy test before setting aside E1/VM1.

If the Council did approve setting aside the Bernoulli method, and as a result flooding became far more frequent than expected, it would be necessary to abandon that decision, and to reject the less conservative St Venant and 2D Shallow Water design solutions.

Notes

1. Verification Method E1/VM1 does not distinguish between water level and total energy head (or static head) in the wider parts of the channel where velocity head is small. In this Problem the velocity head is only about 2mm in the 9m wide rectangular channel sections, but this small difference does still register in the last decimal place of most computational models. E1/VM1 also neglects small wide channel slope terms. In consequence, it was necessary to adjust the water level at the coast down by 3mm to 0.897m and upstream at chainage 120m, down by 2mm to 1.154m. Assuming the transitions between the wider and narrower channels occurred between 9m and 10m from the coast, and between 110m and 111m from the coast, the correct depth of 0.900m was then maintained (to the nearest mm) throughout the semicircular reach of the computational Bernoulli solution, and the analytical solution was matched to the accuracy stated using a semicircular pipe cross-section.

   These same adjusted boundary levels and transition geometries were re-used in all the other solutions to maintain strict comparability.

   In a computational model, it may not be easy to specify an open topped semicircular shape as an exact geometrical option, so the *AULOS* Bernoulli test model also used an alternative segmented description based on points every 9 degrees of arc – that is, 21 points around the semicircle. This gave a cross-section fractionally smaller than the true semicircle, but as noted the flow difference from the standard was still negligible.

2. The St Venant (momentum) solution used by *AULOS* conforms to textbook analysis descriptions such as that in “Practical Aspects of Computational River Hydraulics” by Cunge, Holly and Verwey [Pitman, London, 1980], in that it makes a first order correction for width changes. However no corrections are permitted for poorly documented non-standard flow reduction devices such as section “effective area”.

3. The 2D Shallow Water solution for laterally uniform flow which is everywhere aligned with one of the solution axes will be exactly the same as the St Venant solution, except that the hydraulic radius is replaced with the flow depth in the Manning formula. If the semicircular reach cross-section is represented by a single transverse mesh 1.8m wide in a 2D modelling package such as Mike 21, the resulting solution can be represented by a St Venant solution on a rectangular channel, but with hydraulic radius replaced by flow depth. This is the basis of the reported *AULOS* solution, which produces increased flow through the narrow reach because the average flow depth of $\pi d/8$ (d = 1.8m) from a rectangular section of equal area is significantly greater than the semicircular hydraulic radius of $d/4$.

   A 1.8m transverse mesh also allows the 9m wide sections to be represented as five mesh widths, so it is the coarsest grid resolution which can be expected to give a reasonable solution for the channel capacity. Finer transverse meshes would allow the introduction of no-slip wall conditions, in which case the solution would become dependent on the lateral shear assumptions made. There would also be a refinement of the modelling of flow through the channel transitions. However from extensive 2D modelling experience these effects are not thought likely to make more than say 10% difference to the channel capacity solution without the use of extreme settings of lateral resistance. Also 2D grids significantly finer than 1.8m are likely to require unacceptably long run times for production drainage models, even with currently available processing speeds.

Other adjustments can be made to decrease the 2D modelled flow capacity, such as increasing the Manning n values, but these can only be set objectively by matching a standard solution such as the Bernoulli solution. If the Bernoulli solution is to be adopted as superior in this way, then why run the 2D solution at all?

4. The ratio between floods of different return periods, for example Q_{100}/Q_{10}, is not constant from one catchment to another, as strictly the individual flow curves have to be calibrated using long term records from the relevant catchment. An example of the process is given in Pilgrim (ed.) “Australian Rainfall and Runoff” [IE Aust 1987] p.225. In this example, (South Creek at Elizabeth Drive) the corrected 1 in 10 flood was given as 221 cumecs, the corrected 1 in 20 flood was 281 cumecs and the 1 in 100 flood was 383 cumecs, giving Q_{100}/Q_{10} = 1.73 and Q_{100}/Q_{20} = 1.36.

   Of course this is a far bigger catchment than that of the specified Problem, and in a different climatic regime, but local rainfall intensity ratios, for example I_{100}/I_{10}, are not too far different. For example, the depth-duration-frequency table from the NIWA HIRDS V2 package for Albert Park is remarkably consistent in intensity ratios over all durations. The variation in I_{100}/I_{10} from 10 minutes to 72 hour duration is always between 1.42 and 1.48, and that for I_{100}/I_{20} between 1.27 and 1.31. Albert Park is in the centre of Auckland, and one of the longest continuously maintained rainfall observation sites in New Zealand, so the figures from it can reasonably be cited as typical.

   Rainfall intensities are not the same as flood flows, but are linearly related in simple rainfall-runoff models such as the Rational Method. Therefore the reasonable correspondence between the flood ratios from South Creek and the intensity ratios from Albert Park suggest that average values of these ratios are adequately accurate to represent typical urban catchments where accumulated flow records from long periods of observation are rarely available.

   This justifies the ratios of Q_{100}/Q_{10} = 1.61 and Q_{100}/Q_{20} = 1.37 used for the discussion of significance.