STORM LATERAL RUNOFF CALIBRATION BY DIRECT HYDRAULIC BALANCES

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ABSTRACT

Storm runoff is usually measured only at the catchment outlet, which restricts hydrological modelling to lumped empirical whole-catchment measures. This defeats physical analysis of the runoff process where the actual catchment has a diversity of permeabilities, slopes and storm intensities. A study of riparian resistance on a suburban reach of the Opanuku Stream in West Auckland demonstrates that stream roughness measurements are highly sensitive to lateral inflows. This means that, provided the mass balance modelling error is negligible, the lateral runoff from the local subcatchment into the reach can be calibrated at the same time as the stream roughness. This residual flow difference between reach inflow and outflow offers a new hydraulic calibration technique to support improved analysis of local storm runoff.

KEYWORDS

Residual flow, hydropower, mass balances, riparian resistance

PRESENTER PROFILE

Alastair Barnett has over forty years of experience applying hydraulic modelling in twenty countries. His award-winning *AULOS* design software has been proven through the last twenty years of analysis of linked surface and subsurface network problems such as hydropower surge dynamics and urban drainage failures, including tsunami inundation.

1 INTRODUCTION

The replanting of stream banks in native vegetation is encouraged by the Auckland Council (that section formerly Auckland Regional Council) as contributing strongly to improvements of freshwater habitats. Yet, this planting if it replaces bare or grassed banks, offers more hydraulic resistance and may therefore raise flood levels.

To ensure that a flooding hazard is not created by providing environmental enhancement, the effect of riparian planting on design peak flows must be quantified. An investigation was therefore commissioned to consider typical scenarios, to identify which ones, if any, require riparian planting to be avoided or controlled.

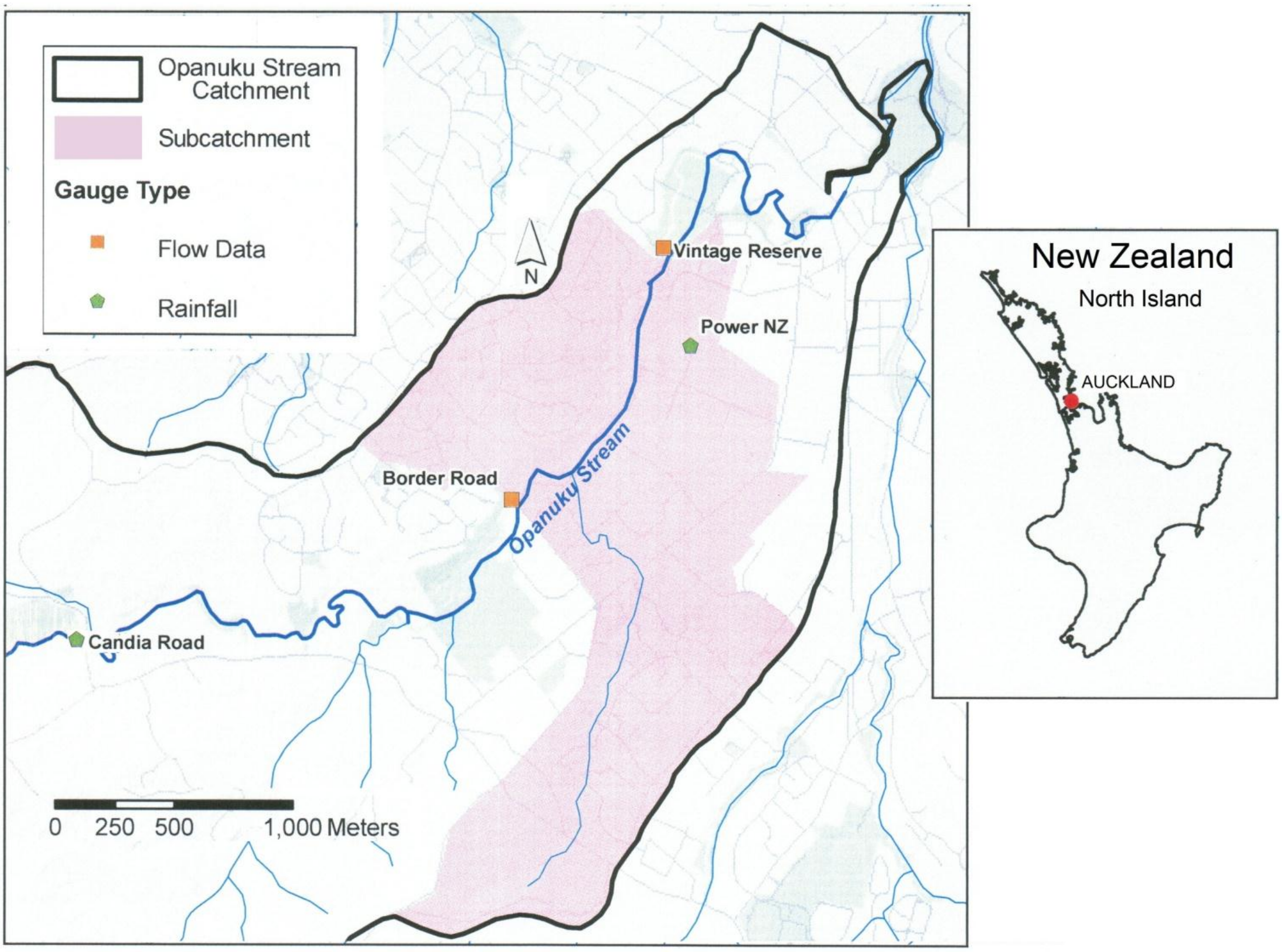
The purpose of the project was a contribution to the management of flooding risk in conjunction with freshwater habitat enhancement, for which an understanding of effects of riparian planting on flood levels was necessary.

Precise calibration of channel resistance cannot be expected if good quality field data is not available, regardless of the performance of the calibration technique. For resistance calibration, a field measurement of slope must be available, and the most direct way of obtaining this at sites where at least two level recorders operate along the same channel, providing continuous stream level measurements at both ends of a channel reach.

Accordingly the Council data archives were searched for suitable sites. Only one site met this requirement as well as other criteria (see Barnett & Shamseldin, 2012), and that was the Opanuku Stream between the level recorders at Border Road and Vintage Reserve in West Auckland (see Figure 1).

Once water levels are monitored upstream and downstream, reach volumes are continuously accessible as well as water level slopes. This paper deals with the practicalities of using this information in a mass balance equation to determine hydrographs of flows entering laterally between the two ends of the reach, thereby producing indirect hydraulic measurements of the local subcatchment runoff.

Figure 1. The Calibration Reach on Opanuku Stream, West Auckland.



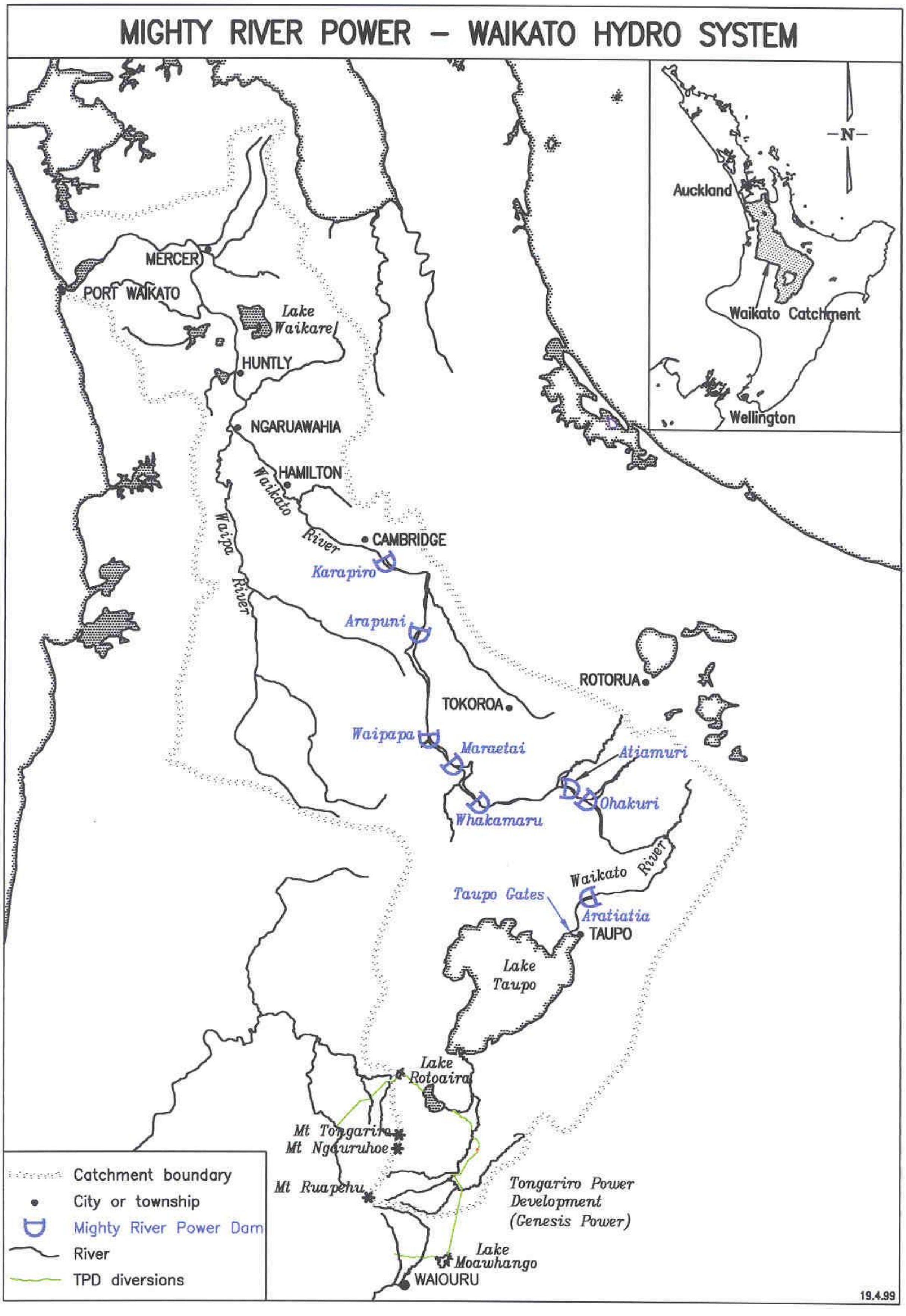
2 HYDROPOWER MODEL CALIBRATION

2.1 FUNDAMENTALS

Hydropower cascades such as the Waikato River (Barnett et al., 2000) are more intensively monitored than typical storm water catchments, because continuous data enables the efficiency of conversion of river energy to electrical energy to be tracked and optimized in real time.

This has high value to a hydropower generating company, as improved generating efficiency translates directly into higher profits.

Figure 2. Dams on the Waikato Hydro System



Typically flows are measured through all dams (see Figure 2) and water levels are monitored at least upstream and downstream of every dam.

This means the river can be divided into reaches, each with flow and water level records available at high time resolution at both the upstream and downstream ends.

An integral mass balance can be applied to the storage in each reach. Over a finite time interval, this can be expressed (e.g. Henderson, 1966) as

Inflow volume – outflow volume = increase in storage (1a)

Or

$$-\int_{t_1}^{t_2} \int_A \boldsymbol{n}.(\boldsymbol{v}-\boldsymbol{u})\,\rho\, dA Dt \quad = \quad \left[\int_R \rho\, dR\right]_{t_1}^{t_2} \qquad (1b)$$

Equation (1b) is the formal CELL Integral expression of (1a) (see Barnett et al., 1995), where t_1 and t_2 are the start and end times of the time interval, A is the area of the control surface enclosing the reach storage, $\boldsymbol{n}$ is the normal to the control surface (positive outwards, hence the negative sign in front of the integral), $\boldsymbol{v}$ is the fluid velocity in an inertial frame, $\boldsymbol{u}$ is the control surface velocity, ρ is the density, Dt is a time increment, written with a capital D as it is "following the storage" during its existence between t_1 and t_2, and the region R is the storage volume inside the surface.

Equations (1a) and (1b) are completely general expressions of conservation, applying even through flow discontinuities where differential equations do not exist. Equation (1a) is slightly less general than (1b) only in the assumption that water density is constant, so that a volume balance replaces a mass balance.

The CELL Integral expression is required for derivation of numerical solution schemes without the introduction of restrictive assumptions of continuity of derivatives, such as used for differential analysis. These assumptions are best avoided because it is well known that discontinuities are common in hydropower problems, for example with respect to sudden changes in flow through power stations. It follows that methods such as finite difference or finite volume analysis will introduce errors at all occurrences of such discontinuities.

2.2 LEVEL POOL ROUTING

Considering a river reach between selected dams, and taking the downstream end first, the measured water level is the headwater level above the downstream dam. At low flows the dam reservoir can be regarded as a level pool, and provided the surface area of that pool is known, the storage can be computed if the surface level is known.

The outflow is also known from downstream measurements, so the inflow to the pool can then be derived using Equation (1) (either (a) or (b)). Reference to the upstream end then provides the main channel inflow measurements, and these can be subtracted from the derived total inflow to leave a balance known as the residual flow. In a computational model, this flow can be derived as a time series by applying the reservoir levels to the end of a lateral channel with large enough cross-section and short enough length to have negligible surface slope. Such a schematic channel is known as a "stub" tributary, and the model solution should ensure the flows through this nominal tributary just balance the sum of storage changes and the difference between inflow at the upstream end and outflow at the downstream end.

Considering that this residual flow includes groundwater exchanges as well as surface water inflows and direct surface precipitation, it provides a measure of the responses of the immediate reach lateral catchment to the prevailing conditions. This is particularly useful during storm events, as the continuous catchment runoff can be derived by purely hydraulic computation for comparison with runoff derived by hydrological modeling.

2.3 BACKWATER ROUTING

In hydropower schemes, the water levels at the upstream end of the reservoirs is usually also available as tailwater levels below dams. In general these do not coincide exactly with the headwater levels measured at the downstream end of the reach, because of

(a) Flow resistance, which applies a slope to the reservoir surface, particularly towards the upstream end where the cross-section is much smaller.

(b) Dynamic waves, which are excited by flow changes, especially sudden ones as discussed in Section 2.1.

(c) Possible wind shear effects on the water surface, including short waves.

(d) Transducer level measurement errors at both gauges.

Assuming (c) and (d) both have only minor effects and (b) can be accounted for by the dynamic properties of the computational model, the remaining part of the difference is a backwater effect, in which a downstream obstruction creates ponding upstream. Backwater profiles are well studied, and typically feature a gradual transition from a level pool to a gradient consistent with the given flow through a sloping channel.

Computationally, such profiles are solved by the Standard Step method (Henderson, 1966) or more modern variations (Barnett, 2012), which may take into account unsteady flow fluctuations. The main parameter is the resistance coefficient (usually the Manning n in New Zealand practice) and a single value of this can usually be fitted to match the correct difference between upstream and downstream levels throughout the study period.

Interpolating along the whole ponded backwater reach using the corresponding profiles, and allowing for dynamic waves, a storage volume can be derived at each time step, again allowing the residual flow through a stub tributary to be found by application of Equation (1).

Where there is evidence that the reservoir water surface does in fact slope appreciably at the upstream end, this method is more accurate than the level pool routing discussed in the previous section, so the extra information from an upstream tailwater gauge supports an improved result.

In both level pool routing and backwater routing, the reliance on the mass balance Equation (1) places heavy emphasis on precise mass balance modelling. Inflow and outflow volumes through each and of the reach are typically quite similar, so the cumulative volumes measured through each end are often an order of magnitude greater than their differences and the measured changes in storage. Subtraction between these two smaller quantities gives in turn the residual volumes of flow through the sides of the reach, so if computational errors do creep in, they will have a particularly strong effect on the predicted residual flows.

3 STORMWATER CATCHMENTS WITH ONE GAUGED RECORDING SITE

The Opanuku Stream can be taken as an example of a catchment with only one gauged level recording site by considering only the record from the Vintage Reserve location marked in Figure 1. For this purpose other records are set aside in the interim, in particular that from the level recorder at Border Road. Since the Border Road recorder was installed only recently, this initial approach effectively represents the Opanuku catchment status prior to that installation, as well as that of most other catchments through the Auckland region.

The Opanuku has a mild slope in this area, corresponding with subcritical flow. Since in subcritical flow profiling, control is exercised from downstream (see Henderson, 1966) a numerical model of the gauging reach must start from downstream of the recorder, ending where the recorded level is available as an upstream boundary condition. Such a model should then produce a "loop rating" with flows on the rising limb of a hydrograph exceeding flows on the falling limb. This is well-known from hydraulic theory to be an improvement on a single rating curve. If the downstream boundary is far enough removed from the recorder, then an arbitrary level boundary condition can be used, because the backwater profiles will all converge to the same level for a given channel roughness as they tail out upstream, and it is this level which will determine the gradient through the gauging reach.

Cross-sections have been surveyed upstream and downstream of the gauging site at chainage Opanuku 4.798, but unfortunately only one of the adjacent downstream cross-sections (at Opanuku 4.839) passed quality control tests. As this is only 41m downstream, tailing out cannot be expected before the level at the recorder is affected. All that can then be done is to project a uniform cross-section far enough downstream from Opanuku 4.839 for tailing out to become effective to give uniform flow. The level of a uniform flow depends on the slope of the projected channel, and the best information on which to choose a slope was an assumption that the stream would continue downstream at the average gradient of 0.0024 measured upstream to Border Road.

3.1 LOOP RATING FROM MODEL

A complete model can now be constructed for the reach just downstream of the gauging site – see Figure 3. This takes as the upstream boundary condition the level hydrograph recorded at the gauging site, with a nominal fixed water level as the boundary downstream of the reach projected with uniform cross-section. To assist with the tailing out of the associated errors upstream, an overfall was added at the downstream end of the uniform reach, so that the effective downstream boundary was an overfall rating curve automatically derived by the *AULOS* package (HYDRA Software Ltd, 2013) which was used for the modelling.

Two significant floods were available for investigation, that of 1 October 2006 (here called the "calibration" flood) and that of 24 August 2008 (here called the "verification" flood). A model channel with levels specified continuously upstream and downstream responds by allowing flows to pass through, and these are dependent on the chosen channel resistance. Figure 3 shows the channel has two distinct regimes – a low flow channel and berms vegetated by heterogeneous grasses, shrubs and trees.

Accordingly, after some experiment, the decision was made to schematize the resistance by a base value for low flows with one relative resistance multiplier applied to all riparian berms. This applies compound channel analysis (Barnett, 2002).

Figure 3. Vintage Reserve Recorder, looking downstream

The resulting model loop rating is presented as a red line in Figure 4. This was produced with a Manning n=0.040, a relative resistance rr=10, and a downstream runout slope S=0.0024. The anomalous "bump" on the recession curve (the return red curve on the left) at just above 20 cumecs seems to coincide with an intense burst of rain recorded at the adjacent Power NZ rain gauge (see Figure 1, Figure 8).

The other information in the plot is the measured rating data. The green "original rating" line is the rating curve derived by Auckland Council (then Auckland Regional Council) up till July 2005. The black "composite" line is an adjusted rating prepared by the writer taking into account data up until November 2007, in particular the green crosses recorded between December 2005 and June 2006, which suggested a move to the left (increased resistance) around the time of the calibration flood. Full details are given in Barnett & Shamseldin, 2012. Most of the other gauging points resolved at this scale are the blue circles measured from the start of gauging in 1999 until May 2005.

3.2 SENSITIVITY ANALYSIS

Figure 5 shows the lower left-hand corner of Figure 4, with the red line marking the response of the fitted model as before. In this range a number of recent gauging points (blue diamonds) indicate a return to values comparable with the early part of the record, and the red lines (up and return) are seen to be a good fit through the scatter. The width of the loop (the distance between these red lines) is governed by the change in water surface gradient between the rising and recession limbs, which reflects the sharpness of the flood peak and hence the storm intensity.

Figure 4. Model Flow Rating against Levels Recorded at Vintage Reserve Gauge

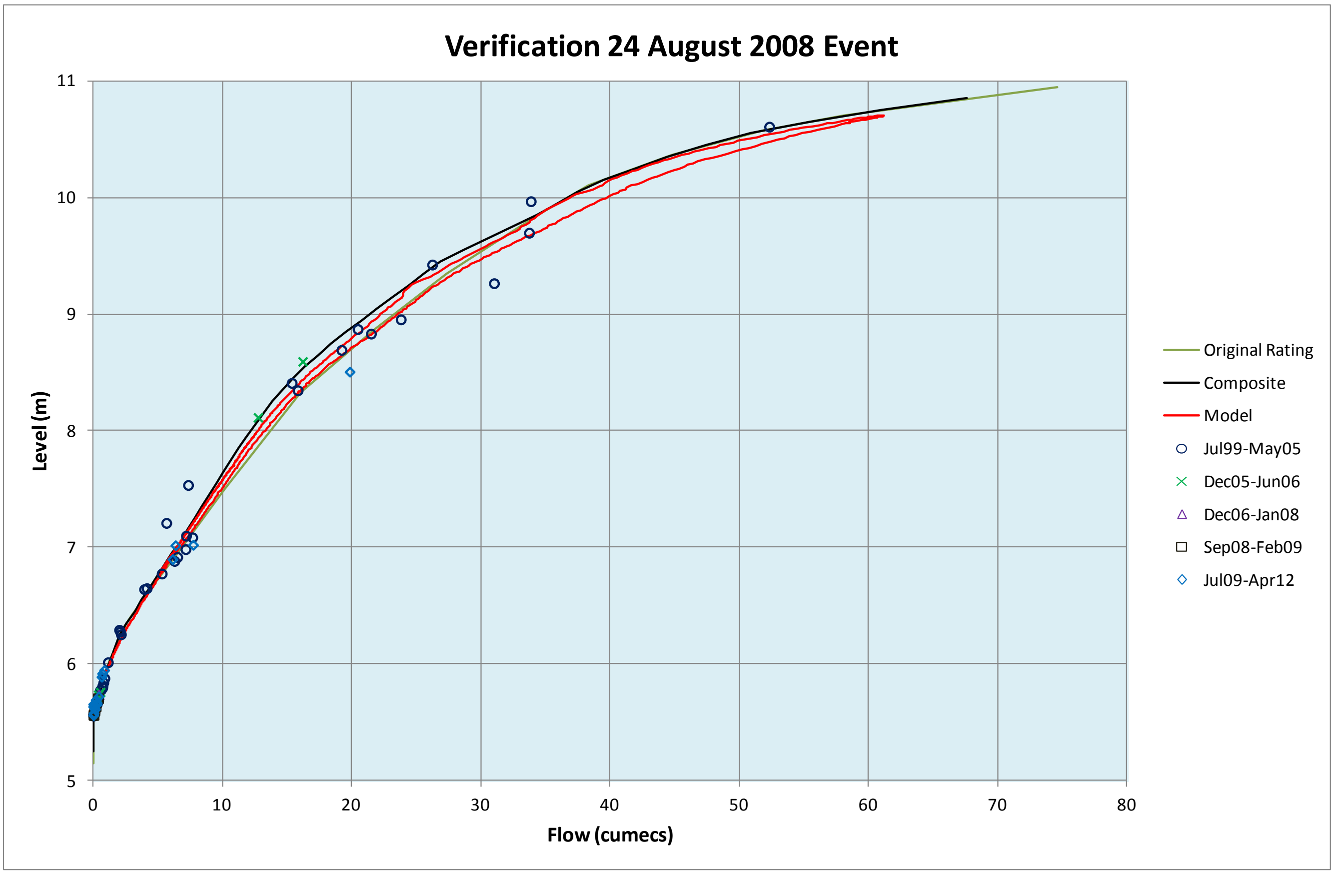
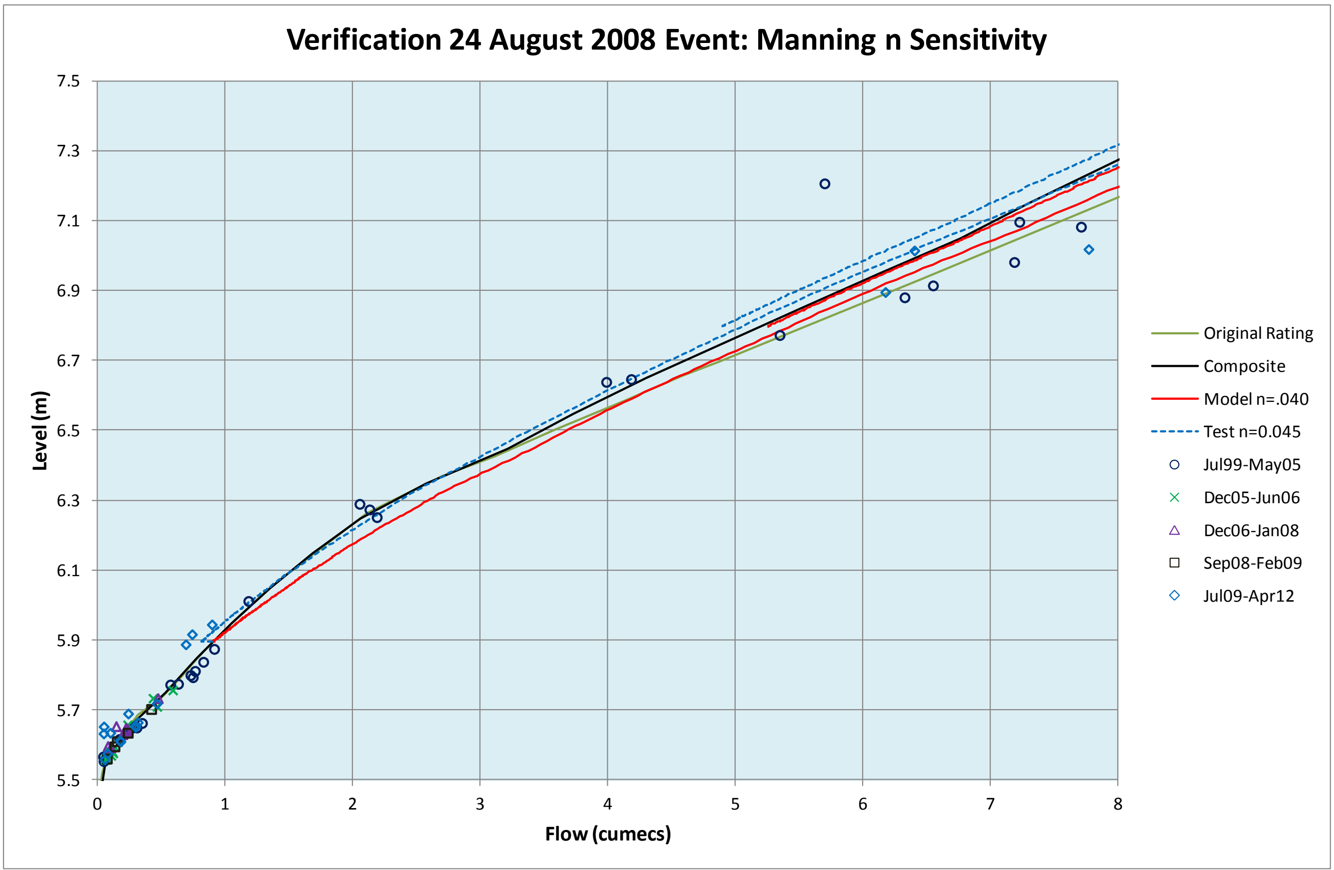


Figure 5. Model Response to Variation Of Manning n



To test the sensitivity of this model rating curve to the Manning n value, the model was re-run with Manning n=0.045 (the value giving the best fit for the calibration flood) and all other settings unchanged. The results are plotted (dashed blue curve) in Figure 5.

The initial conditions are taken to be the same, fitting the "composite" curve, but the increase in the base Manning n immediately reduces the model flow, shifting the curve substantially to the left even at low flows. Clearly the flows are too low, being mainly to the left of the composite curve even in rising flows.

Figure 6. Model Response to Variation of Riparian Resistance Setting

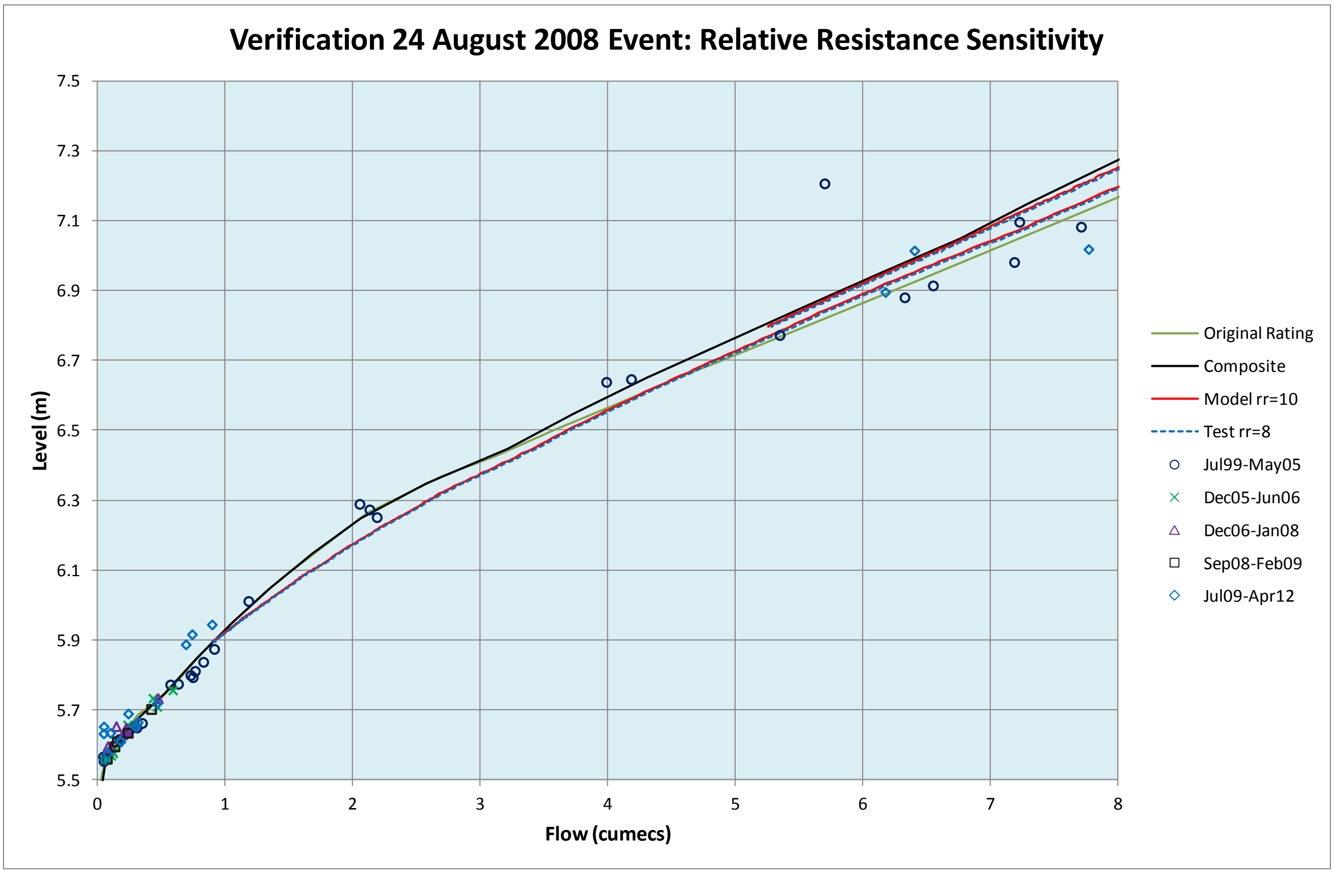
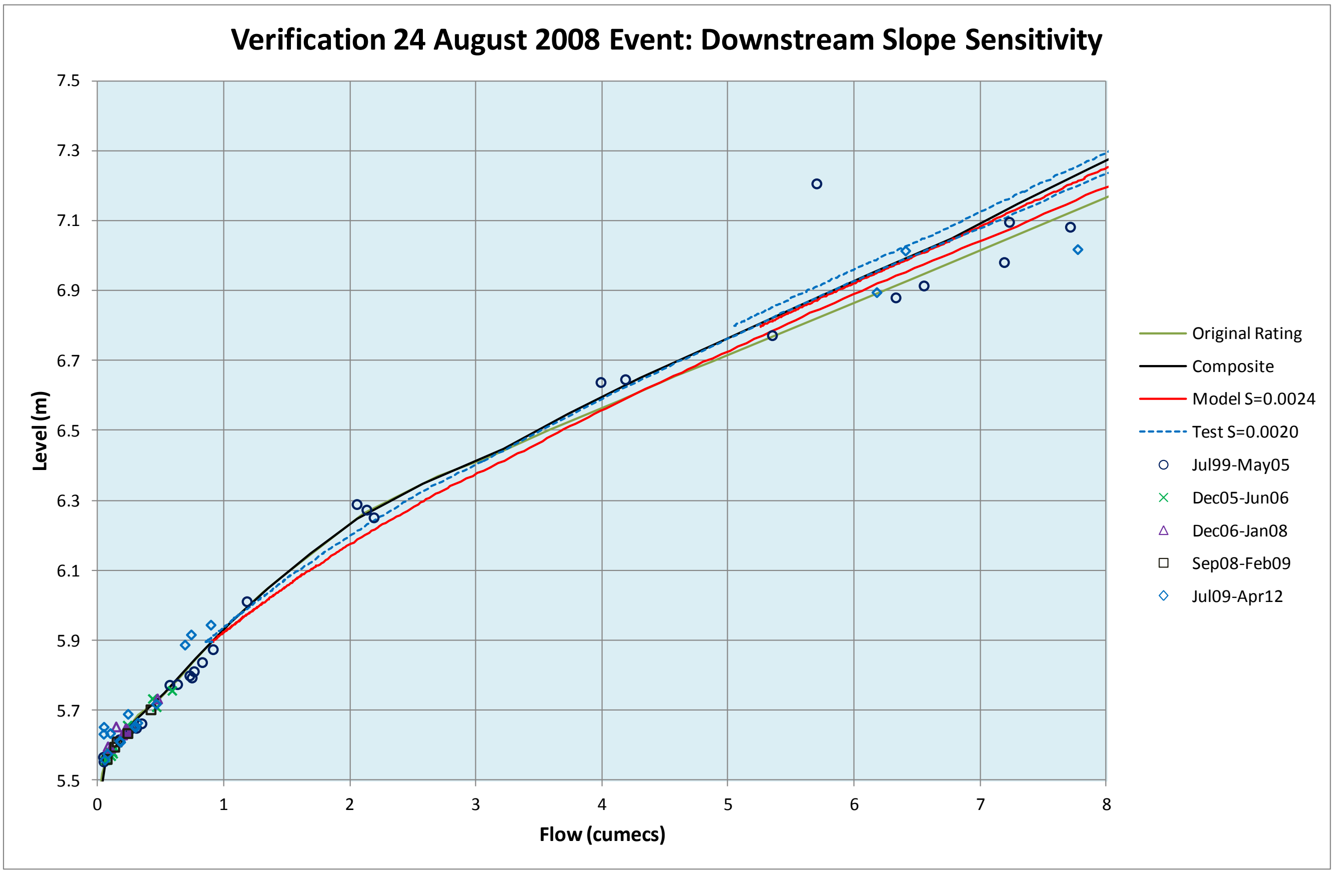


Figure 6 shows the effect of varying the relative (riparian) resistance value. The red curve is as in Figure 5, but this time the dashed blue curve shows the model response to a change to rr=8 instead of the calibrated rr=10. As would be expected, there is little difference at low flows as the water is not passing through the vegetation. As the water level rises, the riparian resistance has more effect, but clearly the model is not very sensitive to a 20% variation in this value.

The effect of varying the assumed slope to the downstream boundary is shown in Figure 7, this time with the dashed blue curve showing the model response to a decrease in the assumed runout slope below the calibrated value of S=0.0024. Instead, a slope of S=0.0020 was tried.

The effect is muted at low flows, because in this range the short reach to the last surveyed cross-section at opanuku 4.839 is still dominant in setting the backwater profile. This supports the choice of the low flow Manning n as demonstrated by Figure 5. As the flow rises, the runout slope starts to have some effect, with increasing slope producing more flow at a given level. The calibrated value gives a better fit.

Figure 7. Model Response to Variation of Downstream Runout Slope



The three figures 5, 6 and 7 together show that the base Manning n is the main factor controlling calibration, as the relative resistance formulation for the riparian vegetation implies that the riparian resistance also rises as the low flow Manning n is increased.

In summary, the availability of flow gauging data enables a water level recording site to be used in a way comparable with the downstream end of a hydropower model reach, adding a synthesized flow record to the level hydrograph measured at the same point.

This flow record is not directly measured as can be done at a hydropower station, so must be regarded in principle as of slightly lower accuracy. However this depends on the scatter of the gaugings, which may be highly repeatable in tightly controlled sites like a natural overfall or installed weir. Also some hydropower station layouts are not ideal for gauging purposes, particularly those with low heads. In the Vintage Reserve example, there is also unusually little downstream cross-section information, so in most cases the need for an assumed runout slope would not arise, improving calibration accuracy.

4 ADDING A SECOND WATER LEVEL RECORDER

Although the existence of a water level recording site plus flow gauging almost matches the downstream end of a hydropower reach, application of the mass balance Equation (1) to derive inflow is still not possible, except in the rare cases where the river reach approximates a level pool. To go further, a second water level recorder is required at the upstream end of the reach. In the example, this is now available at Border Road.

With a second recorder, backwater routing is now possible, giving a continuous estimate of the volume of storage within the reach. This in turn allows the total inflow to be derived, but if the upstream site is not gauged (as at Border Road), the parallel with a hydropower reach is not yet complete, because the inflow at the upstream boundary is not directly measured to allow the residual flows to be found by subtraction.

4.1 REQUIRED ASSUMPTIONS FOR UPSTREAM MANNING RESISTANCE

Resistance losses can be calibrated only in relation to slopes along the channel. Defining a "reach" as a length of the channel with reasonably homogeneous bed material, riparian vegetation, and flow conditions, the Manning n is therefore strictly a measurable property of a reach rather than of a cross-section. Provided the reach between the gauges appears to be reasonably homogeneous with that from below the downstream gauge, the downstream resistance characteristics can be extrapolated upstream for use at the upstream boundary. *If* the low flow Manning n and riparian resistance figures are comparable throughout the reach, this approach promises to deliver reasonably accurate upstream inflows. The test of this accuracy is the resulting residual flows, because hydrological analysis provides considerable guidance, both in terms of cumulative runoff from the lateral subcatchments and of likely runoff hydrograph peaks.

4.2 LIMITS ON CUMULATIVE LOCAL RUNOFF

In a situation like the Opanuku Stream example, the total cumulative runoff contributing to the reach would not be expected to exceed the total storm precipitation measured over the local subcatchment, and cumulative runoff from a storm would not be expected to be negative either. These are outer bounds, and more refined hydrological modeling can be expected to push these upper and lower limits closer together, as well as providing a first approximation to likely runoff hydrographs.

Figure 8. Comparison of Flow Hydrographs

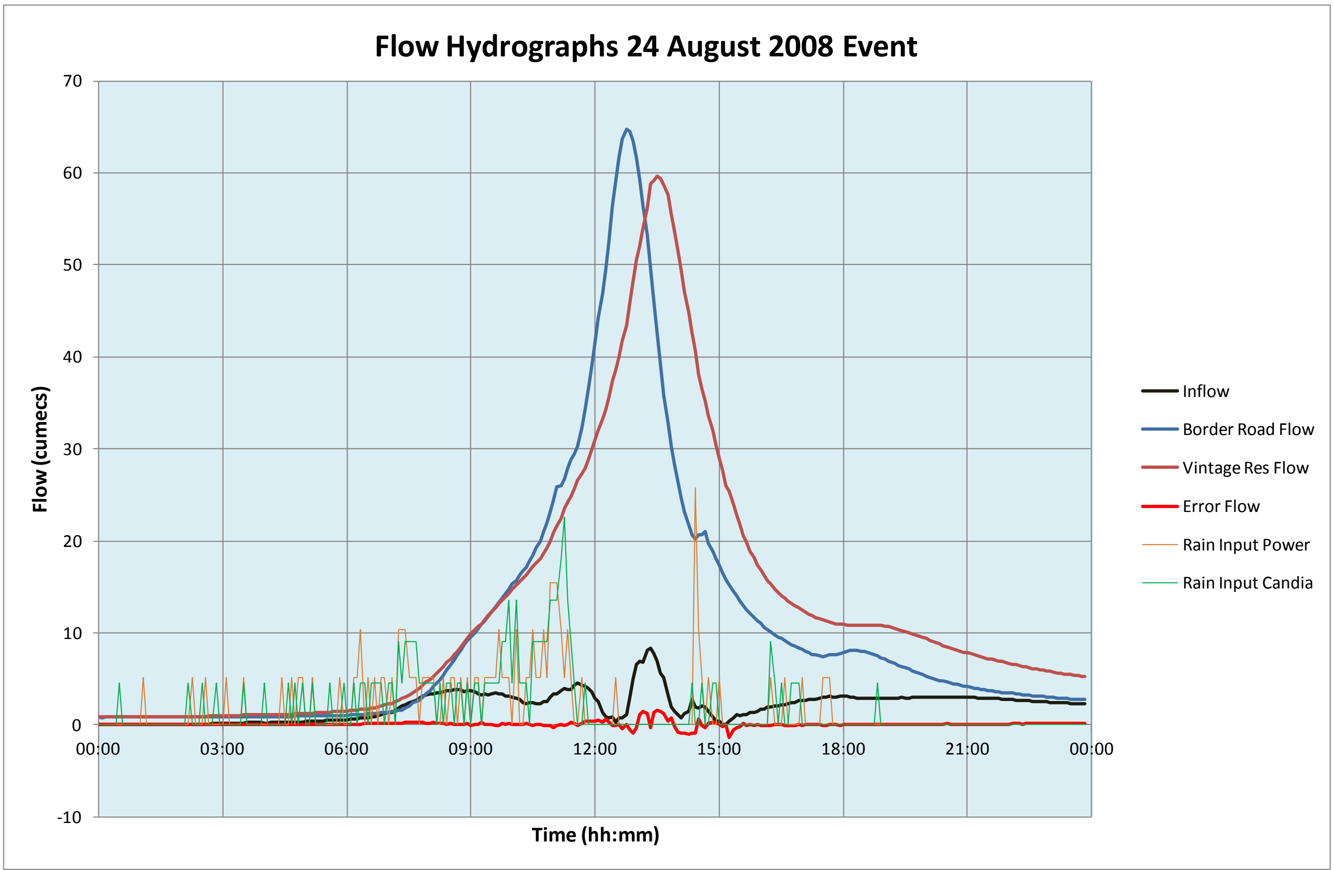


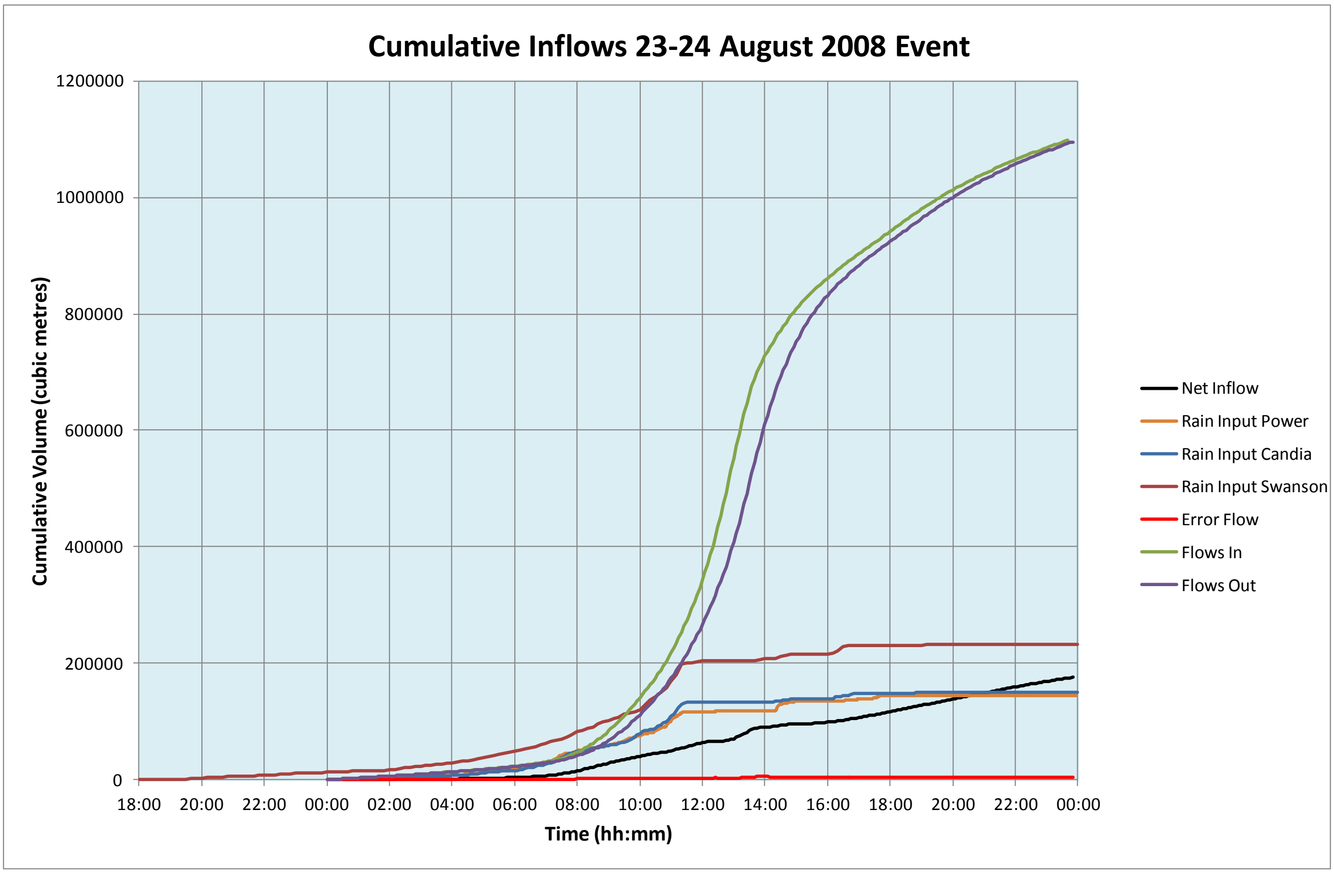
Figure 8 shows the hydrographs derived by applying the model calibration as described in Section 3, but with the Manning n=0.040 and relative resistance rr=10 extrapolated upstream to the Border Road gauge. The upstream boundary of the model is now supplied by the level hydrograph recorded at that gauge. Since the Vintage Reserve gauge is no longer at the upstream boundary, the level boundary must now be applied to a stub tributary joining the main stream at that point. The residual flow is then part of the solution, and is drawn in or pushed out according to the solution of Equation (1) in response to the measured level changes.

4.3 SENSITIVITY TO ASSUMED LOCATION OF SUBCATCHMENT OUTFLOWS

Because there is no physical tributary at that point, model representation of all the catchment runoff as entering at that point is not strictly accurate. To assess the error involved in not allowing for translation of inflows from the actual (presumably) distributed points of entry to a single downstream point, a further refinement was then applied. The residual flow hydrograph drawn through the downstream stub was moved upstream to a second stub tributary near the mid-point of the reach, and advanced by 10 minutes to allow for the approximate time of translation estimated from that point to the original stub tributary.

A second model run then determined a fresh residual flow through the stub tributary at the Vintage Reserve gauge, but this time with the first estimate of residual flow entering at mid-reach. The new residual flow can be regarded as a measure of the translation error produced by moving the lateral inflow to a point more representative of the actual subcatchment feeding the reach. Figure 8 shows the resulting flow hydrographs, with the residual flow plotted as the black "Inflow" line and the translation error plotted as the red line marked "Error flow".

Figure 9. Cumulative Inflows



Clearly the translation error is an order of magnitude smaller than the residual flow, with both positive and negative variations. The derived upstream inflow hydrograph at Border Road and downstream outflow hydrograph at Vintage Reserve are also plotted, showing that both are another order of magnitude larger than the residual flow.

Finally, the rainfall inputs measured at the Power NZ and Candia Road rain gauges (see Figure 1) are plotted. To provide for direct comparison with the residual flows, these rainfall intensities have been multiplied by the area of the local subcatchment to give representative precipitation inflows as measured at these gauges.

4.4 COMPARISON OF CUMULATIVE RUNOFF FLOWS

Figure 9 shows the corresponding cumulative flows, which offer a different perspective on the same data. First it is clear that the translation error flows are essentially noise, with mean flow approximately zero. This is especially in comparison with the Flows In (Border Road + catchment inflows) and Flows Out (Vintage Reserve), so the vertical difference between these two lines at any time indicates the current storage volume in the model.

The other three lines are the basis for a comparison of considerable importance. The spiky nature of the rain input records is now smoothed, showing similarities between the event as recorded at Power NZ and as recorded at Candia Road, both in timing and in intensity of bursts. However, the significantly different record from another adjacent gauge at Swanson (not shown in Figure 1) illustrates the difficulty of setting up rainfall/runoff models, even if the precipitation gauges are relatively close together as in this case.

Comparison of these three curves with the Net (residual) Inflow curve adds the necessary upper bound to the residual flow calibration, as it is clear that a Net Inflow accounting for more than 100% of the event precipitation is equally as unphysical as a negative Net Inflow. The result plotted shows residual flows accumulating at approximately 55%-65% of the cumulative precipitation in the earlier part of the event.

Late in the event the residual flows continue to accumulate some hours after rainfall has ceased and crossing the 100% precipitation line for Power NZ and Candia Road, though not for Swanson. This suggests either that Swanson is more representative and the catchment response includes long delay factors, or that this residual flow modelling approach is less successful late in the recession.

Figure 10 gives another view of the verification, replacing the direct precipitation curves with runoff curves produced by a rainfall/runoff hydrological model, and indicating the sensitivity of the model to the Manning n adopted for the whole reach.

This used the Hycemos-U hydrological modeling package originally developed by NIWA (Barnett et al., 1992), as this is a physically based model relying on kinematic wave propagation over an "open book" model, in which the two side sloping faces can each be calibrated separately from the central collecting "gutter".

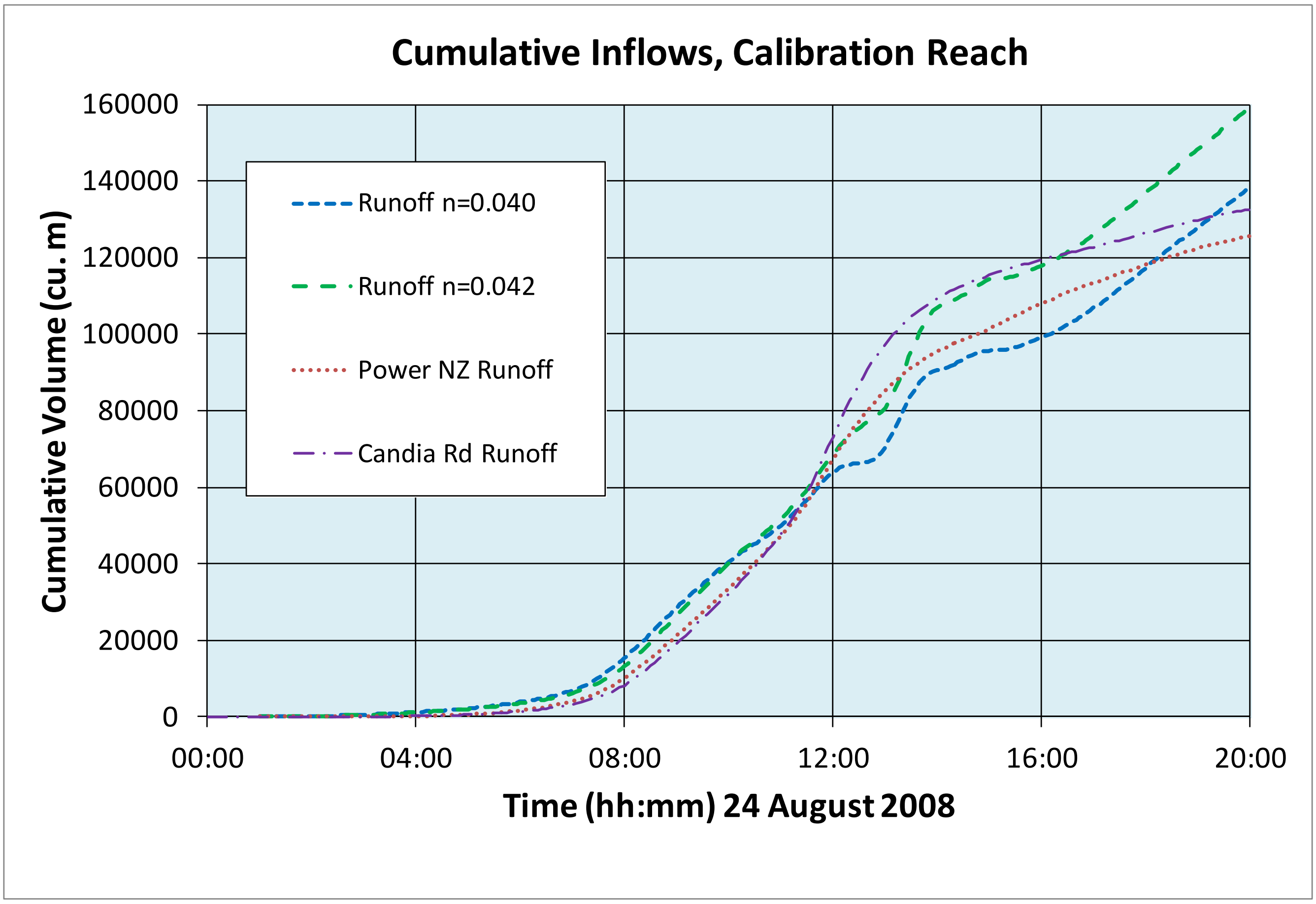
The runoff model had a catchment area of 2.71 km2 as measured by Council staff, and 65% was a "fast response" hillslope, which is typical of runoff coefficients recommended for mixed residential/industrial areas by authorities such as the New Zealand Building Code Clause E1. 30% of the area was a "slow response" hillslope, with the balance being the collecting channel and an allowance for losses. The curves marked "Power NZ Runoff" and "Candia Rd Runoff" are the runoff hydrographs produced by this hydrological model in response to the precipitation at the corresponding rain gauge sites.

The runoff inferred from residual flows is plotted for the two values of Manning n, and it is clear that a minor increase in resistance produces a major increase in residual flow, so that n=0.042 is obviously too high on this measure.

4.5 DISCUSSION

Even the curve with n=0.040 starts to over-run in the end, as would be expected because this is also the Net Inflow curve plotted in Figure 9. By comparison with the curve with n=0.042, this test suggests the resistance should be lowered even further, but this runs counter to the gauging evidence of Figure 5. Of course, the average resistance for the reach may be lower than that at the downstream gauging station, but the available measured database is not extensive enough for this possibility to be explored.

Figure 10. Resistance Calibration for Verification Event



However the sensitivity of residual flow to the reach resistance is clearly valuable in refining resistance measurements, because Figure 10 supports a more precise determination of the reach resistance than does Figure 5, which suffers from some scatter of gauging measurements as well as apparent slow timewise variations in the rating.

Such a sensitive measurement of resistance enables policies such as riparian planting to be evaluated with some confidence for effects on flood levels. This was the original purpose of the investigation, but it may be that the second product of the analysis becomes the more important over time. That product is a new hydraulic calibration technique to support improved analysis of local storm runoff.

5 DEPENDENCE ON MODELLING PLATFORM

While the above results are interesting, they would not be useful if they could not be repeated by other independent researchers using different modelling platforms.

The present results were obtained using the *AULOS* hydraulic modelling software developed by HYDRA Software Ltd. Specifically, the energy solution was applied, using the "Compound" hydraulic radius option at all sections with the "Floodplain" correction switched off. No eddy losses were applied at any point. The low flow Manning n was applied only to the low flow channel (effectively that below the terrain as surveyed by Lidar), and the stated relative resistance factors were applied at all higher points.

The energy solution is an unsteady Bernoulli-type formulation, and the "Compound" hydraulic radius invokes classic conveyance theory, modified as in Barnett (2002). The interpretation of the Manning n values therefore strictly relates only to this theory.

However, for flow residuals to be applied, the first requirement of the hydraulic analysis is to interpolate a water surface between the water levels measured at the upstream and downstream ends of the reach at consecutive time intervals. Given that the mean water surface gradient is continually defined in this way, the time variation of volumes within the reach should not be heavily dependent on the model used, whether this is energy, 1D momentum, 2D momentum, or even some simpler kind of spatial interpolation.

The second hydraulic modelling requirement is the relation between steady flow and resistance for the upstream and downstream control points, where the levels are recorded. Thirdly, at the downstream end the model is required to translate timewise rates of change at the fixed measuring point into water surface slopes and acceleration terms during the rising and falling limbs of the hydrograph, so that a looped rating can be related to the stage/discharge gauging points measured from time to time.

The key requirement of the chosen hydraulic model is the ability to maintain accurate mass balances, as residual flows are relatively small compared with main stream flows. This means that mass balance errors of the order of say 10% would produce false residuals totally dominating the actual lateral flow contributions, on which the whole method is based.

5.1 ON THE DIMENSIONALITY OF MODELS

References to the dimensionality of models as "1D" or "2D" continue to cause confusion. With respect to the surveyed terrain (including the perimeters of pipes and channels), both "1D" and "2D" models are fully three-dimensional. This can be demonstrated by the ability of both to compute a volume, for example that of a pond, which undoubtedly involves three spatial dimensions. In fact, a pond volume can even be computed using a "0D" model such as a depth contour map, because "1D" and "2D" conventionally refer only to horizontal dimensions, and a contour map uses a vertical projection of the three-dimensional terrain surface.

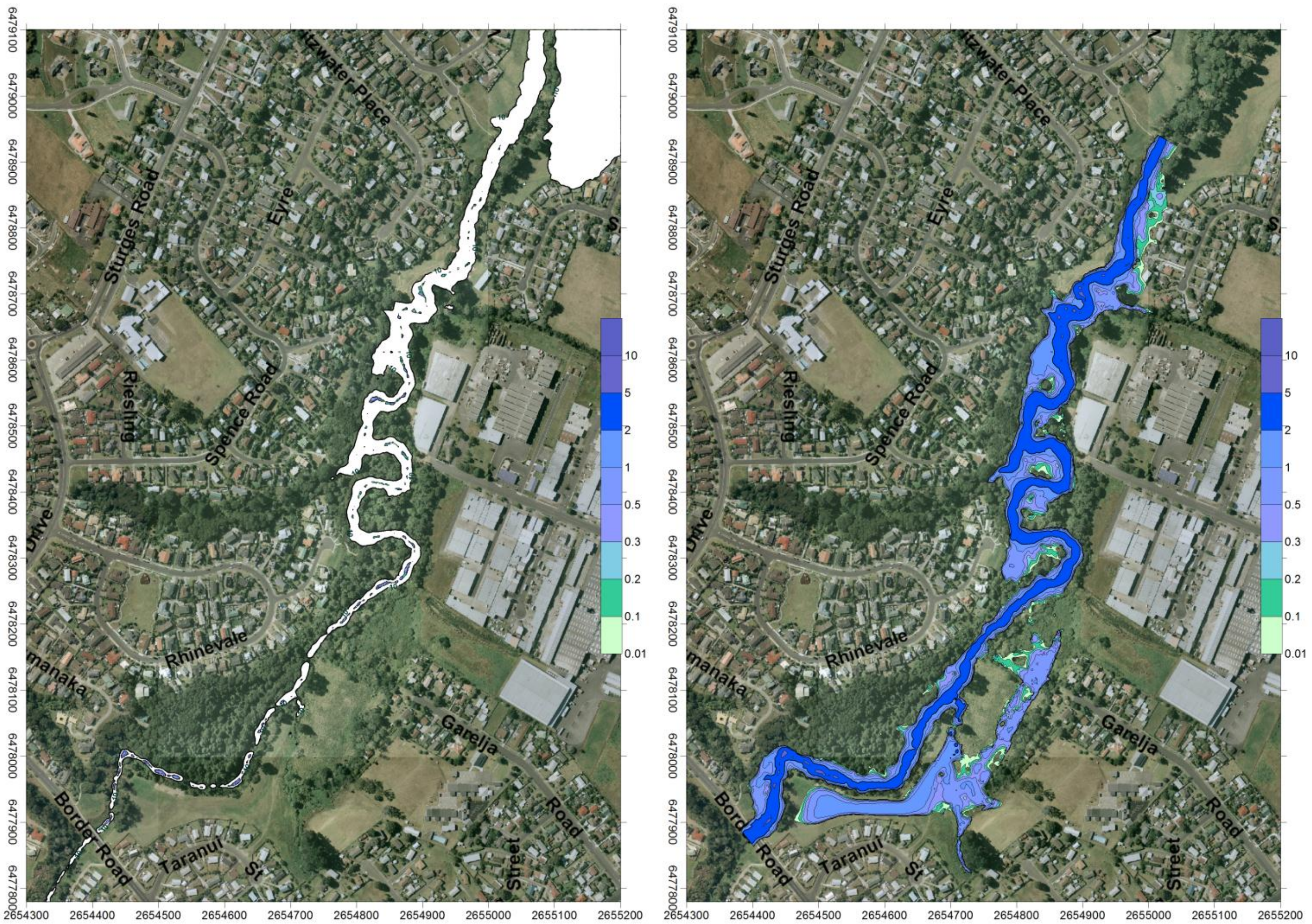
The conventional "1D" and "2D" refer specifically not to the model itself, but to the method of analysis of the model. "Analysis" is defined by the Concise Oxford Dictionary as "Resolution into simple elements", and 1D analysis uses a projection of the three-dimensional terrain surface similar to a contour map, but the projection is now horizontal instead of vertical, resolving the terrain into vertical slices instead of horizontal slices. Computation of the pond volume can now take into account longitudinal variations in water level between slices, but in the case where the pond surface is horizontal, the volumes computed by 0D and 1D analysis will be the same if the projections are both made to the same resolution. 2D analysis adds a second horizontal projection of the three-dimensional terrain surface, usually lateral (orthogonal to the first longitudinal projection) to enable a momentum vector equation to be resolved into two orthogonal scalar components. These two projections intersect to form slabs (seen vertically), so the pond volume computed by a 2D analysis can now take into account variations in water level between adjacent slabs laterally as well as longitudinally.

However in the case where the pond surface is horizontal, the volume computed by 2D analysis will match those computed by 0D and 1D analysis. If the pond surface is horizontal laterally, as would be expected for a hydropower reservoir even if there was longitudinal slope, there will be no difference between the volumes computed by 1D and 2D analysis.

5.2 AULOS MASS BALANCES

The mass balance accuracy of *AULOS* was investigated for the verification event of 24 August 2008. Figure 11 shows the depth contours for the test reach at 0000 and 1300 hours respectively on that date.

Figure 11. Flow Depth Contours of Verification Event at 0000 (Left) and 1300 (Right)



The depth contours are based on a 1mx1m grid, and are relative to a terrain model derived at the same resolution from the Lidar survey.

NZ Map Grid coordinates are provided for location.

Note that no georeferenced sections were available for the channel below the Lidar survey limits. Therefore because the low flow levels at 0000 hours (before the flood) are in many cases below the Lidar-based terrain, the aerial photograph background has been blanked below the 10m terrain contour in the left-hand contour plot, as otherwise the low flow contour remnants would be almost invisible.

The mass balance errors for the Verification flood are summarized in Table 1. As water density is assumed constant, the various masses are scaled into volumes in cubic metres. The tabulated volumes are, from left to right the "1D slices", computed by accumulating the volumes under the model water surface along the channel, the "Net Inflow" which is the difference between the inflows and outflows accumulated from 00:00 to the given time, the "Gross Inflow" which is the cumulative upstream inflow at Border Road, and the "2D Grid", which accumulates the volumes measured vertically on the 1mx1m grid used to prepare the flood maps in Figure 11.

The "Net inflow" volume is given an initial value equal to the "1D slices" volume, but (in the absence of georeferenced sections for the low flow channel) the initial 2D grid error indicated in the left-hand plot in Figure 11 also needs correction. As shown in Figure 8, the modelled flood gradually increased from a steady low flow between 00:00 and about 07:00, reaching a level which should cover the terrain-based version of the low flow channel by about 08:00. Therefore at that time the "2D Grid" model was corrected to a value equal to the "1D slices" volume, and the same correction carried through the remainder of the event.

The flood peaked at around 13:00 hours, so that time was chosen for the test mass balance report.

Table 1: Volume Balance Errors, Verification Run

*Note the asterisk * denotes values reset at the initial correction point*

24/8/2008	1D Slices	Net Inflow	Gross Inflow	1D Slices	Gross	2D Grid	Grid
Time (hrs)	Volume m^3	Volume m^3	Volume m^3	% Error	% Error	Volume m^3	% Error
00:00	4433	4433*	0	0	0	-	-
08:00	10312	10294	31622	0.18	0.06	10312*	0
13:00	149220	148807	482083	0.28	0.09	149651	-0.29

Table 1 shows that at peak flood, the mass balance error between "1D Slices" and "Net Inflow" was 0.28% in terms of the 1D slices, or 0.09% in terms of the Gross Inflow Volume. The mass error of gridding the solution into a 2D 1mx1m grid was -0.29% in terms of the 1D Slices.

Note this solution ran with 30 second time steps. The 24-hour simulation took 1.5 seconds to run, using an HP 8530p Elitebook with a 2 Core CPU T9600 @ 2.80GHz and 4.0 GB of RAM. The Operating System was Windows 7. If a 2D solution is used, run times will be orders of magnitude longer than this, for no discernable advantage if lateral water surface slopes are insignificant.

6 CONCLUSIONS

1. Residual flow methods have been found to produce high value improvements in generating efficiency in hydropower cascades such as the Waikato River.

2. These are based on integral mass balance equations relating changes in storage to differences between outflow and inflow.

3. Stormwater catchments with one gauging site can be modelled to produce a loop rating for each flood past the recorder.

4. This rating depends mainly on a base Manning n as the main calibration factor.

5. A second water level recorder on the same stream allows residual flows to be estimated, provided the resistance values at the rating point can be extrapolated over the intervening reach.

6. This turns out to give an extremely sensitive calibration of reach resistance as well as a direct hydraulic estimate of local storm runoff.

7. The method should not be dependent on the modelling platform used, provided accurate mass balances are maintainable. Mass errors should be well under 1%.

8. The conclusion that riparian resistance clearly decreased in the Opanuku test reach between 2006 and 2008 should not be dependent on the modelling platform.

DISCLAIMER

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