7 DESIGN FLOOD MODELLING

7.1 Design Rainfall

7.1.1 Approach

The design flood flows used to provide inflows to the hydraulic 2D / 1D model of the Williams River were established using the calibrated hydrological model with AR&R (1987) recommended design rainfall parameters. The hydrological model was used to provide total inflow hydrographs to the Williams River, Myall Creek and Carowiry Creek. The hydrologic model was also used to produce local inflow hydrographs at various locations along the 2D / 1D hydraulic model. This is the same process that was used in the calibration of the hydraulic model.

The design rainfall depths and temporal patterns for the Williams River catchment as recommended in AR&R (1987) were input to the calibrated hydrological model. As previously discussed, rainfall across the catchment can be highly variable. To represent the variability in rainfall within the hydrological model, the catchment was split in to four IFD parameter regions. In each of the four regions representative AR&R parameters were determined and used to calculate rainfall depths (temporal patterns remain the same).

The four areas within the Williams River catchment are presented in Figure 7-1. The resulting design intensities are presented for each of the four regions in Table 7-1.

The rainfall depths in Table 7-1 are point rainfall intensities. To use these rainfall depths over a catchment, areal reduction factors from AR&R (1987) are used to account for the variability of rainfall over the catchment. For the critical duration of 36hours (see Section 7.1.2) an areal reduction factor of 0.92 was used.

Table 7-1 Average Rainfall Intensities (AR&R 1987)

Upper North Area

Duration (Hours)	Average Rainfall Intensity (mm/hr)				
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
12	10.1	11.6	13.5	16.1	18.2
24	6.8	7.7	9.0	10.6	12.0
36	5.4	6.1	7.0	8.3	9.3
72	3.4	3.8	4.4	5.2	5.8

Upper South Area

Duration (Hours)	Average Rainfall Intensity (mm/hr)				
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
12	10.6	12.6	15.1	18.8	21.7
24	7.3	8.6	10.2	12.5	14.4
36	5.8	6.8	8.1	9.8	11.2
72	3.9	4.4	5.2	6.2	7.0

Central Area

Duration (Hours)	Average Rainfall Intensity (mm/hr)				
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
12	9.8	11.1	12.9	15.3	17.1
24	6.5	7.4	8.6	10.2	11.4
36	5.1	5.8	6.7	8.0	8.9
72	3.2	3.6	4.2	5.0	5.6

Lower Area

Duration (Hours)	Average Rainfall Intensity (mm/hr)				
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
12	9.0	10.2	11.8	13.9	15.6
24	6.0	6.9	8.0	9.6	10.8
36	4.7	5.4	6.4	7.7	8.7
72	2.9	3.5	4.1	5.0	5.7

K:\B16030.k.gjr_WilliamsR_FS\RAFTS\Design_Hydrology\[ARR_intensities_for_RAFTS.xls]Report_Tables

7.1.2 Rainfall Losses

Rainfall losses for the design hydrological model were 0mm initial loss and 2mm continuing loss. The continuing loss remains the same as the calibrated hydrological model. Initial loss of 0mm is a conservative loss reflecting a catchment with rainfall prior to design event.

7.1.3 Critical Durations

Several durations for the 1% AEP event were initially simulated with the hydrological model to determine the duration that results in the highest peak inflow. The peak flow at Dungog was used to assess peak inflows to floodplain. The 36 hour event results in the highest peak flows at Dungog in the hydrologic model with a peak flow at Dungog of 4,010m^3/s.

To ensure that the event with the greatest inflows resulted in the highest simulated water levels over the Williams River catchment, three different durations were simulated using the hydraulic model.

The 24, 36 and 72 hour duration 1% AEP Williams River events were simulated with the hydraulic model. Critical duration simulations had zero inflow from the Hunter River and a constant ocean boundary of 0mAHD.

Results show that the 36 hour event is critical over the majority of the floodplain. In the upper section of Myall and Carowiry Creeks the 24 hour simulation resulted in higher levels than the 36 hour simulation. Critical duration results from the hydraulic modelling are presented in Figure 7-2.

The Hunter River catchment is larger in size than the Williams River, therefore a slightly longer duration Williams River event was used for the Hunter River dominated events (see Section 7.1.8). The 48 hour duration Williams River was combined with the Hunter River events for the Hunter River dominated events only.

7.1.4 Comparison of Design Flows to Flood Frequency Analysis

There is considerable difference between flows calculated using the hydrologic models with AR&R (1987) rainfall depths and the flood frequency analysis. The peak 1% AEP flow from the hydrological model at Dungog was 4,010m^3/s (using an areal reduction factor of 0.92). This value is significantly higher than the value of 2,253m^3/s obtained from the flood frequency analysis.

The highest value for 1% AEP flow at Glen Martin produced by the flood frequency analysis was from the FLIKE software. The results produced by George Kuczera's FLIKE flood frequency analysis indicate the Glen Martin Log Pearson III 1% AEP flood peak is 2,230m^3/s.

The flood frequency analysis for both Dungog and Glen Martin indicate the AR&R isopleths may be overestimating the rainfall. However, there is sufficient uncertainty in the flood frequency analysis process not to adopt these flows over the AR&R derived flows. The technical committee adopted a conservative estimate of flow at Dungog that is an average of the Flood Frequency Analysis (2,253m^3/s) and the AR&R derived flows (4,010m^3/s). The adopted 1% AEP peak flow at Dungog was 3,130m^3/s.

7.1.5 Revised Rainfall Estimates

In order to achieve a 1% AEP flow of 3,130m^3/s, iterations of the following method were used:

- Rainfall depths were factored in hydrologic model
- Inflows from hydrological model were inputted to hydraulic model
- Peak flow was assessed by summing flows at the road bridge

A factor of 0.77 was required. This factor was applied to AR&R rainfall depths for all design events.

7.1.6 Probable Maximum Flood Estimates

Probable Maximum Precipitation (PMP) estimates have been derived using the empirical equations presented in the latest edition of Australian Rainfall and Runoff (2001, Book VI). The Williams River catchment is in transition zone between the Generalised Southeast Australia Method and the Generalised Tropical Storm Method (GTSM). As per Section 3.10.2 of Book VI (AR&R 2001) estimates of PMP depth for both GTSM and GSAM are compared and the higher value taken.

For the GSAM zone estimates of PMP rainfall depth are based on catchment area (km^2) and 1 in 50 AEP, 72 hour rainfall intensity. Equations listed allow direct calculation of 36 hour PMP depth estimate. This method resulted in an estimation of PMP depth of 997mm.

Predictions for PMP depth in the GTSM zone are a function of a number of catchment properties:

- Catchment area (km2)
- Distance of catchment centroid to coast (km)
- Latitude of catchment centroid (degrees south)
- Height of intervening barriers between the catchment centroid and the coast (m)

Equations in AR&R (2001) do not allow direct estimation of the 36 hour PMP depth. PMP depth estimations were calculated for 24 and 48 hour events. PMP depths for the 24 and 48 hour events were interpolated to obtain an estimation for the 36 hour event. This process resulted in an estimation of PMP depth of 1152mm. This rainfall depth is factored as a result of the flood frequency analysis (see Section 7.1.4). The factored rainfall depth estimate for the PMP is 887mm.

The GTSM estimate of PMP depth is the higher estimate and is, therefore, adopted. Inflows to the hydraulic model are generated using the hydrologic model using a PMP depth of 887mm and the 100 year temporal pattern.

7.1.7 0.5% AEP Rainfall Estimates

Australian Rainfall and Runoff standard methods were used interpolate 0.5% AEP (1 in 200 year ARI) rainfall depths. The procedure is document in Section 3.6.3 of Book VI (AR&R, 2001). Rainfall depth is factored as a result of the flood frequency analysis (see Section 5.6). Factored and un-factored rainfall depths for the four IFD regions (See Figure 7-1) are presented in Table 7-2.

Table 7-2 Rainfall Depths 0.5% AEP Event

	IFD Parameter Region			
	Lower	Central	Upper South	Upper North
Un-factored Rainfall Depth (mm)	357	364	457	380
Factored Rainfall Depth (mm)	275	280	352	292

K:\B16030.k.gjr_WilliamsR_FS\RAFTS\Design_Hydrology\1_in_200yr_Calculations\[1_in_200yr_Calcs.xls]1_in_200yr_Rainfalls

7.1.8 Combinations of Hunter River Inflows

Due to the size of the Hunter River catchment water levels in the lower Williams River are strongly influenced by the flows in the Hunter River. The linked Williams River – Lower Hunter River model was used to simulate three Hunter River magnitude events coinciding with a 1% AEP Williams River event. Peak flows were timed to coincide at the confluence. The 20%, 5% and 1% AEP Hunter River event magnitudes were simulated.

A generic spring tide was used as the ocean boundary condition. A synthetic tide was created from mean high water spring and mean low water spring joined with a sine curve with period of 12.5 hours.

Results of the Hunter River event magnitude sensitivity testing are presented in long profile format in Figure 7-3. The levels in the lower Williams River are strongly influenced by the size of the event in the Hunter River. The Hunter River Catchment above the Williams River confluence is approximately 20,500km^2.

The size and response times of the Williams River and Hunter River catchments means that it is unlikely for flood peaks to coincide at the confluence of the rivers. Therefore, the likelihood of a 1% AEP Williams River event and a 1% AEP Hunter River event will be significantly less than 1% AEP. It is more likely that a Williams River flood would have peaked and be receding as the larger Hunter River peaks.

For the rarer events (0.5%, 1%, 2% and 5% AEPs) two combinations are simulated for each AEP. Rarer events in one river are combined with a smaller event the other river, e.g. a 1% Williams River event combined with a 5% Hunter River Event. The second simulation involves the same AEPs but in opposite rivers, e.g. a 5% Williams River event combined with a 1% Hunter River event. A maximum of the two events is created, this is an envelope of the maximum values that have occurred in either events.

A matrix of design events was agreed upon with the flood study technical committee. This matrix of design events is presented in Table 7-3. See Section 7.2 for discussion of downstream boundary conditions.

Table 7-3 Matrix of Design Events

Event Number	Name	Williams River	Hunter River	Ocean
1a	0.5% AEP WR	0.5% AEP	5% AEP	0.8m peak tide (from L+T Study) plus 0.91m CC
1b	0.5% AEP HR	5% AEP (48 hour)	0.5% AEP	0.8m peak tide (from L+T Study), 0.91m CC + Storm Surge
2a	1% AEP WR	1% AEP	5% AEP	0.8m peak tide (from L+T Study) plus 0.91m CC
2b	1% AEP HR	5% AEP (48 hour)	1% AEP	0.8m peak tide (from L+T Study), 0.91m CC + Storm Surge
3a	2% AEP WR	2% AEP	10% AEP	0.8m peak tide (from L+T Study) plus 0.91m CC
3b	2% AEP HR	10% AEP (48 hour)	2% AEP	0.8m peak tide (from L+T Study), 0.91m CC + Storm Surge
4a	5% AEP WR	5% AEP	10% AEP	0.8m peak tide (from L+T Study) plus 0.91m CC
4b	5% AEP HR	10% AEP (48 hour)	5% AEP	0.8m peak tide (from L+T Study), 0.91m CC + Storm Surge
5	10% AEP WR	10% AEP	No Inflow	Mean Spring Tide plus 0.91m CC
6	20% AEP WR	20% AEP	No Inflow	Mean Spring Tide plus 0.91m CC
PMF	PMF WRHR	PMF	PMF	0.8m peak tide (from L+T Study) plus 0.91m CC

7.2 Downstream Boundary Conditions

This section of the report details the downstream boundary conditions used for design simulations. The sensitivity of the hydraulic model results on the downstream condition used is documented.

7.2.1 Linkage to Hunter River Model

The confluence of the Williams River and Hunter River occurs at Raymond Terrace. Historically, there has been interaction between the rivers during flood events. Due to the complex interaction between the two rivers, the Williams River 2D / 1D hydraulic model was combined with an existing Hunter River 2D / 1D TUFLOW hydraulic model (developed for the RTA).

This configuration is the same as that used in the 1990 calibration event. A description of the linkage between the Williams River model and the Hunter River model is provided in Section 6.1.3.

7.2.2 Elevated Ocean Levels

Newcastle City Council commissioned Lawson and Treloar (Lawson and Treloar, 1999) to conduct a joint probability study of design water levels in Newcastle Harbour. This report concludes that jointly occurring rainfall and water levels are virtually independent. It recommended a water level of 0.8mAHD in the harbour if a 99% confidence limit is required.

DHI Water & Environment are currently upgrading of the Lower Hunter flood model for Newcastle Council (in progress). To be consistent with this project, downstream boundaries used in this study were provided to BMT WBM by DHI. This is a tidally varying boundary with a peak water level of 0.8mAHD, based on the joint probability study (Lawson and Treloar, 1999). This tidal varying boundary is used for Williams River dominated flood events. The boundary conditions for the design simulations are shown in Figure 7-4.

The Lower Hunter Flood Study (PWD, 1994) analysed 24 years of water level data for the Newcastle Tide gauge and found a 1% AEP water level of 1.34mAHD. In order to be conservative the Hunter River dominated events (events 1B, 2B, 3B and 4B – See Table 7-3) have a synthetic storm surge component added to match the 1.34mAHD level used in the Lower Hunter Flood Study. This boundary condition is presented in Figure 7-4.

7.2.3 Generic Mean Spring Tide

A generic spring tide was synthesised by fitting a sinusoidal curve to the Mean High Water Spring (MHWS) and Mean Low Water Spring (MLWS). A period of 12.5 hours was assumed. The synthetic spring tide is presented in Figure 7-4.

7.2.4 Sensitivity to Ocean Levels

Three model runs were simulated in order to test the sensitivity of the model to ocean levels. The sensitivity testing used a 1% AEP Williams River and a 1% AEP Hunter River event (peak flows coincident). The four ocean boundaries were:

- Generic Spring Ocean Tide
- 0.8m peak tide (from L+T Study), provided by DHI
- 0.8m peak tide (from L+T Study), provided by DHI + 0.91m
- 0.8m peak tide (from L+T Study), provided by DHI + 0.91m and storm surge component.

Results from the modelling indicate that the model is relatively insensitive to ocean levels. The difference in water levels at Raymond Terrace between spring tide and time varying 99% exceedance level (L+T Study) plus 0.8m is 30mm. Ocean level sensitivity results are presented in long profile format (Seaham-Raymond Terrace) in Figure 7-5.

7.2.5 Enhanced Greenhouse Effect

Much research is currently being undertaken into the impacts enhanced greenhouse effect (climate change) on weather patterns and ocean levels. Predicted impacts encompass a wide spectrum of changes.

To be conservative all design simulations included an allowance for impact of enhanced greenhouse effect on ocean levels. The allowance is towards the high end range of predictions and was agreed upon with the technical committee. The allowance for enhanced greenhouse effect is 0.91 metres.

The hydraulic modelling results are relatively insensitive to downstream ocean level (see Section 7.2.2).

7.3 Other Inflows

Inflow to Grahamstown Dam from the Campvale Swamp pumps is assumed to be constant at the maximum combined pump rate of 19.8m^3/s.

Local inflows to the lower Hunter River were provided by DHI Water and Environment. Due to the small volume of flows and questionable hydrograph shape, it was agreed with the technical committee to omit these inflows for the Williams River dominated events. The local inflows were included for the Hunter River dominated events (events 1B, 2B, 3B and 4B – See Table 7-3) based upon an area weighted multiplier of a lower Williams River catchment.

7.4 Design Model Geometry

7.4.1 Model Extent

The hydraulic model used for calibration had a reduced extent, with the model commencing at Dungog. This allowed flows, derived from historic records, to be inputted directly into the hydraulic model. This configuration avoids the uncertainties in the hydrologic model, primarily due to the shortage of pluviograph records.

The area around the Dungog bridges has complex flow behaviour. A finer scale model was developed for the re-assessment of the Dungog rating curve (see Section 5.3.2). This model extends from the edge of the study area (5km upstream from Dungog) to a point approximately 2.6km downstream of Dungog. This finer scale (10m cell size) model was combined with the 40m model to provide a more accurate representation of flows in the upper Williams River.

The design model consists of 1D elements and three 2D domains. The models are dynamically linked via 1D – 2D connections and 2D – 2D connections. The three 2D domains are:

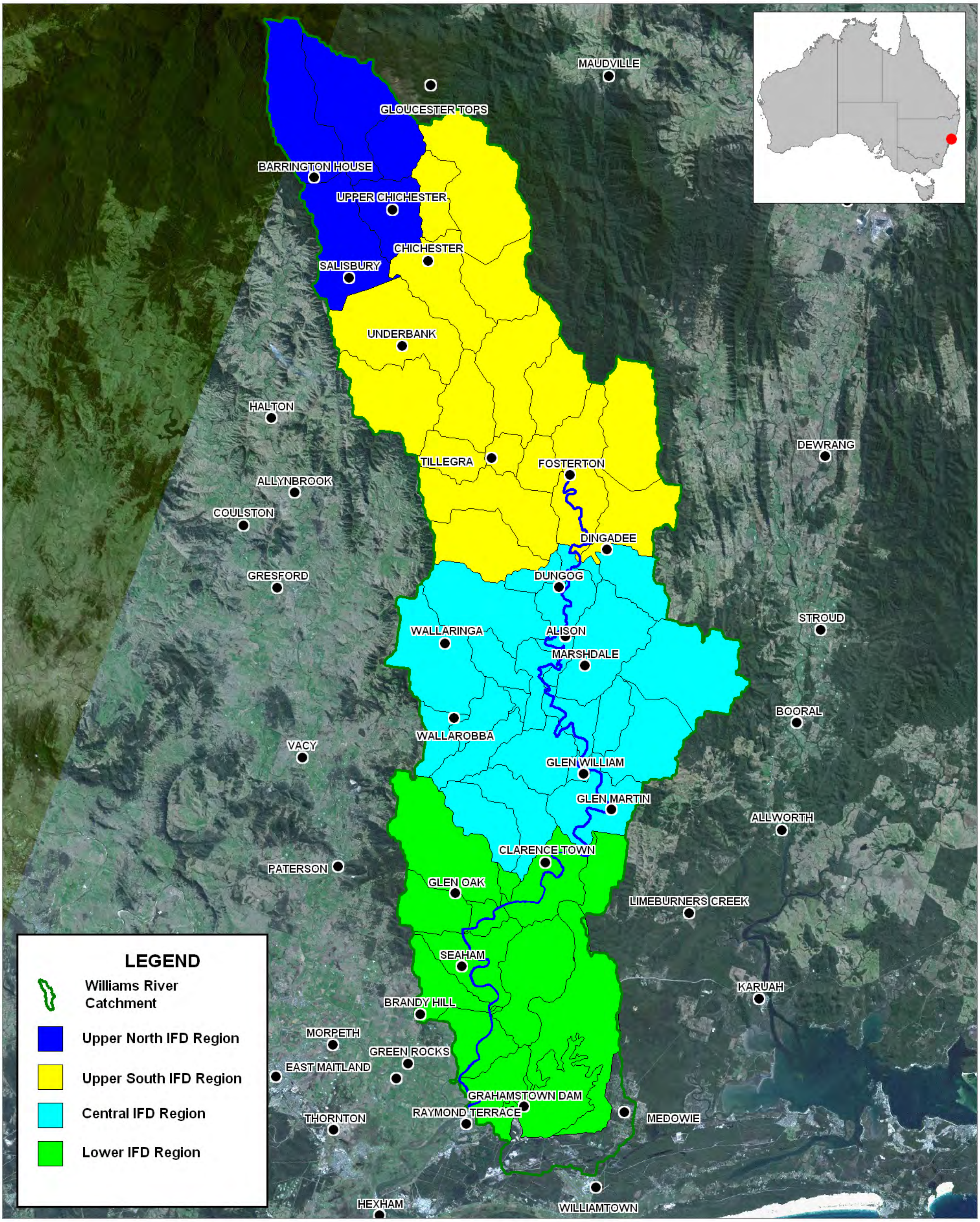
- Upper Williams River - 10m cell size

- Lower Williams River - 40m cell size
- Lower Hunter River - 40 cell size

The locations of each of these and the linkages are presented in Figure 7-6

7.4.2 Irrawang Spillway Upgrade

As part of the Stage 2 augmentation works undertaken by Hunter Water Corporation upgrades were made to the Irrawang spillway. The works involved the constructions of a larger spillway at Irrawang and discharge channel under the Pacific Highway. These works were completed in December 2005. Design hydraulic modelling was updated to reflect the stage-discharge relationship for the upgraded Irrawang Spillway provided by HWC.



Title:

Subcatchment IFD Regions

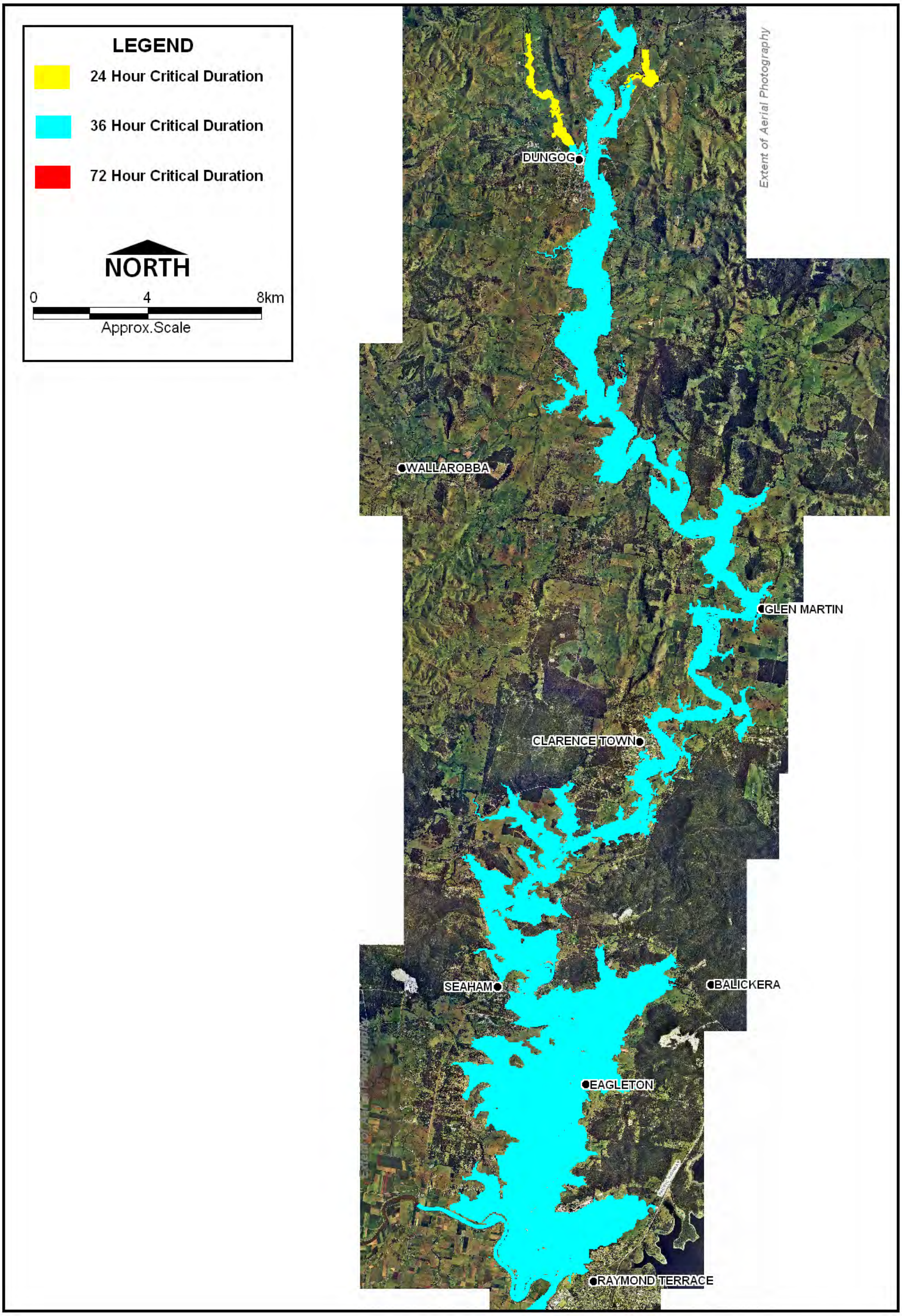
Figure: 7-1

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

BMT WBM
www.wbmpl.com.au

Filepath : I:\B16030_I_brh Williams River PEV\DRG\Design_Report\FLD_008_080704_IFD_Regions.WOR



Critical Duration 1% AEP

Figure 7-2

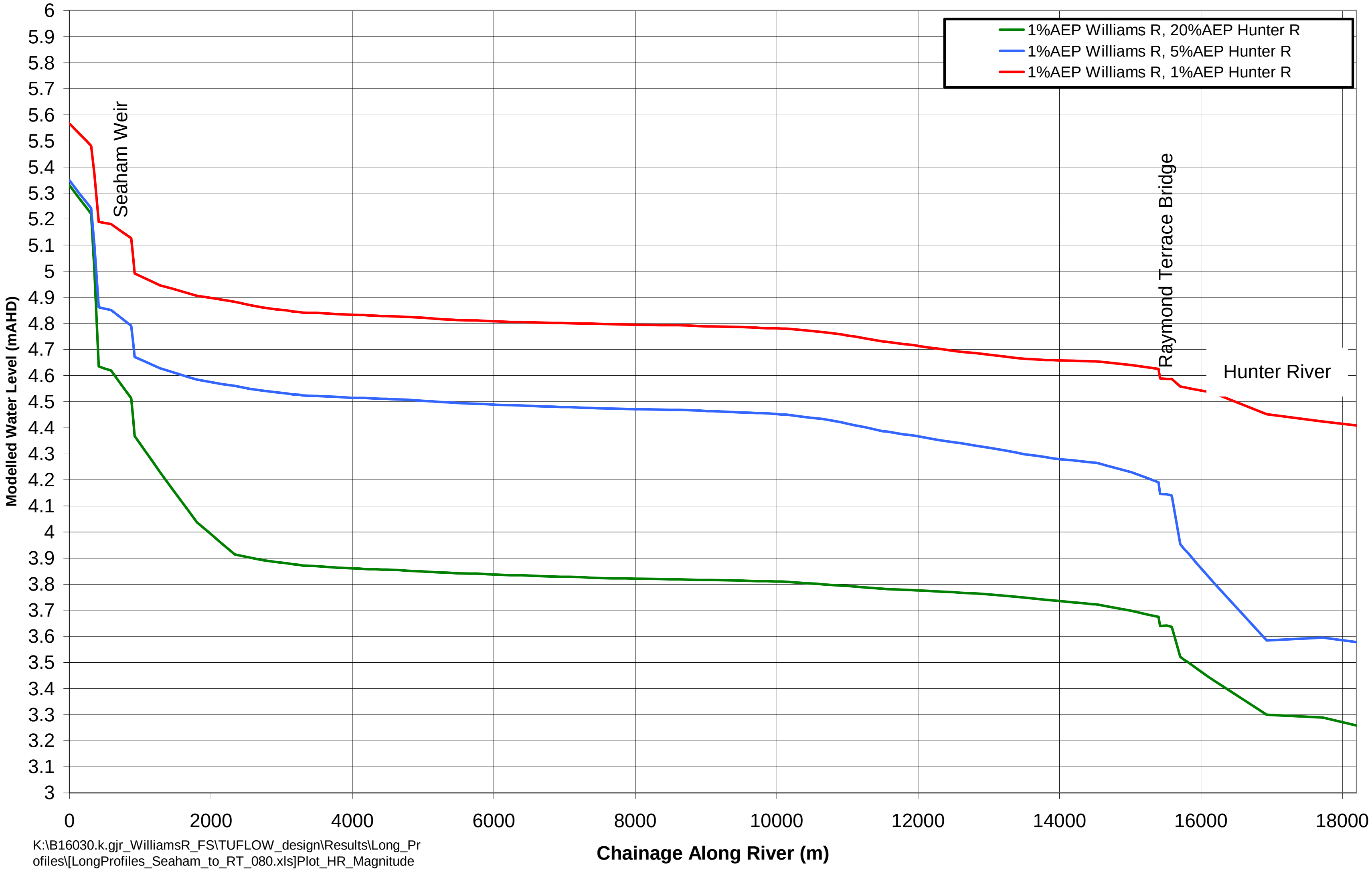


Figure 7-3 Long Profiles – Sensitivity to Hunter Event Magnitude

BMT WBM

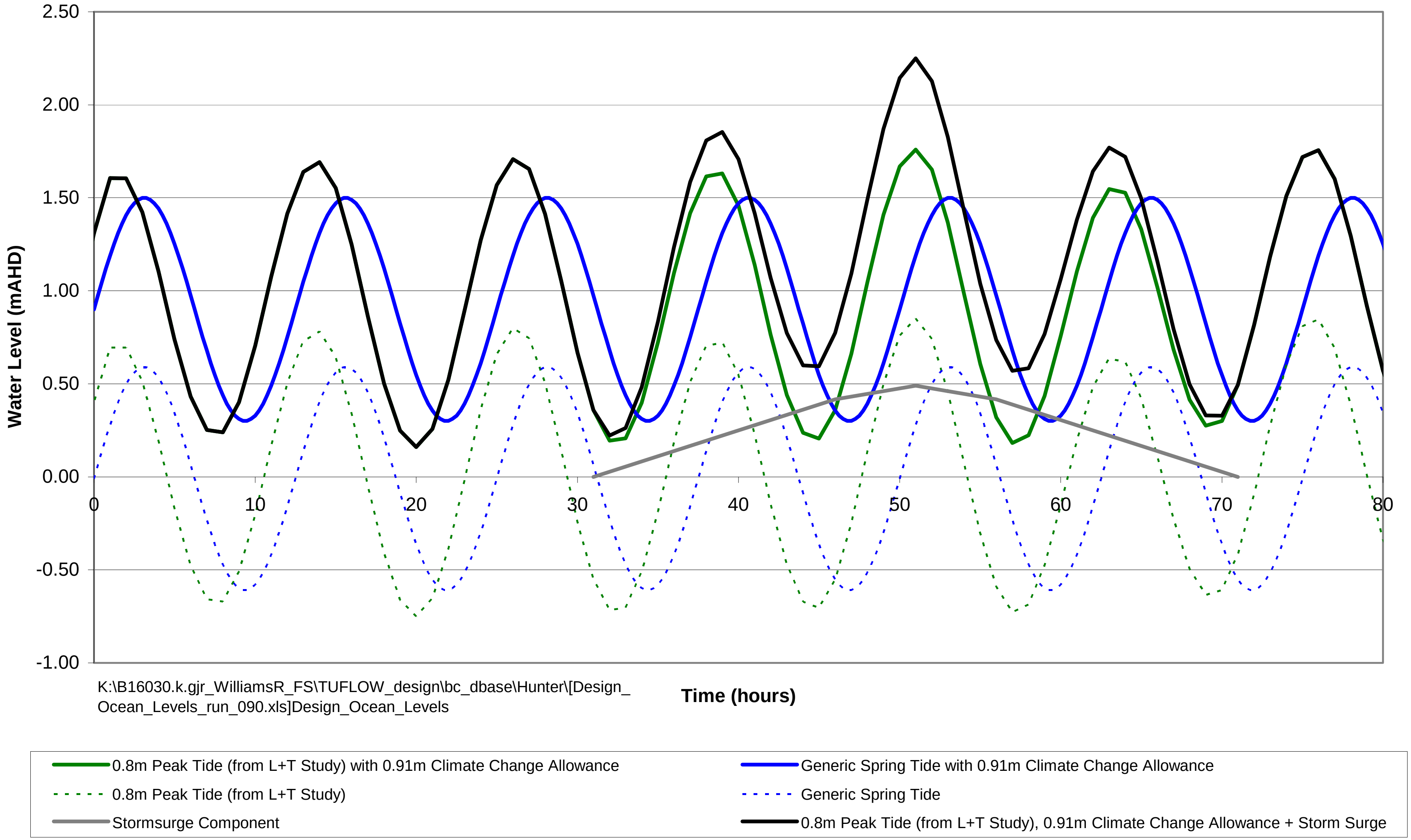


Figure 7-4 Downstream Ocean Levels

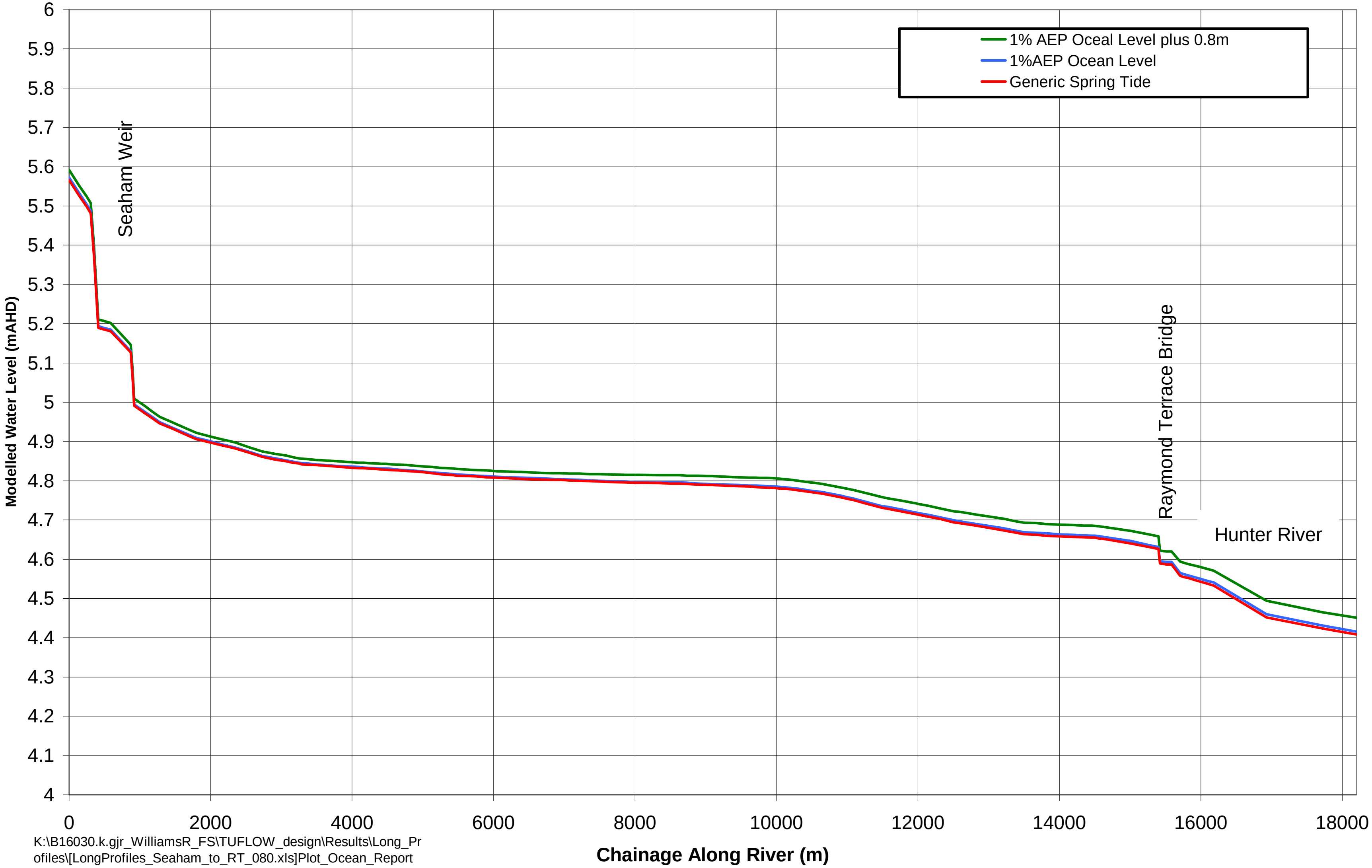
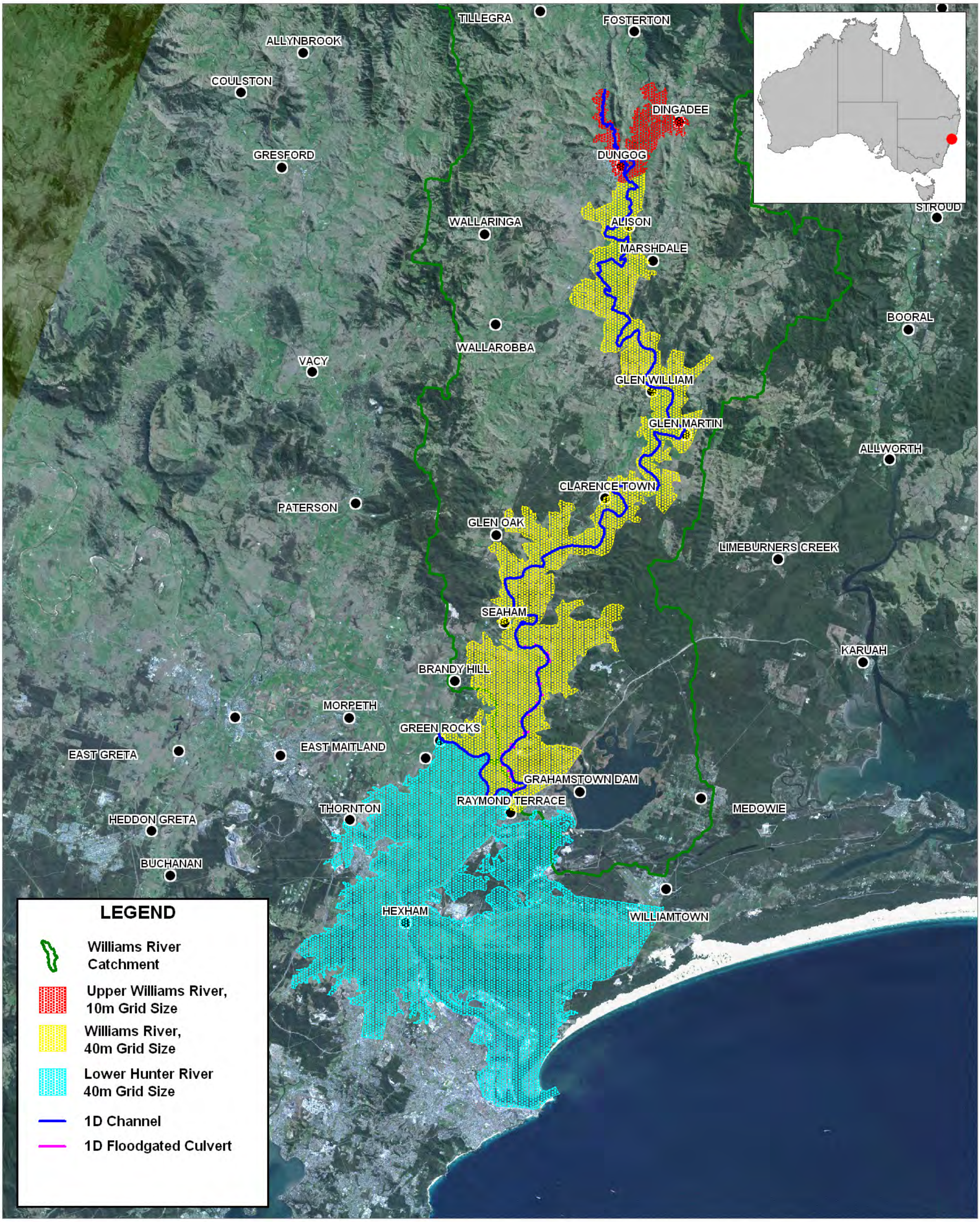


Figure 7-5 Long Profiles – Sensitivity to Ocean Levels



Title:

Design Model Configuration

Figure: 7-6

Rev: A

BMT WBM
www.wbmpl.com.au

Filepath : \\wbmfs2\drafting\B16030_I_brh Williams River PEV\DRG\Design_Report\FLD_014_080822_Design_Model.WOR

8 SENSITIVITY ANALYSES

During the course of the model calibration and design modelling, a number of sensitivity analyses were undertaken to determine if the model results are sensitive to assumptions in the modelling. These analyses are discussed further below.

8.1 Sensitivity to Hydrological Model Parameters

Flows to the hydraulic model from the hydrologic model determine flood levels and behaviour. The sensitivity of modelled flows in the hydrological model at Dungog to a number of hydrological parameters has been assessed.

The base case for the sensitivity runs was the 1% AEP, 36 hour event. This model has a factor applied to rainfall depths as detailed in Section 7.1.4. Rainfall depths are factored to obtain a flow at Dungog of 3,130m^3/s based on the flood frequency analysis. This assessment of flow at Dungog for the purpose of factoring rainfall depth was done using the hydraulic model. Flows at Dungog in the hydrological model base case are slightly higher (3,140m^3/s) in the hydrologic model.

The difference is due to the representation of the river and floodplain in the hydrologic and hydraulic models. The hydraulic model is a more accurate representation of the peak flow.

8.1.1 Initial and Continuing Losses

The sensitivity of the modelled flow at Dungog on the initial and continuing losses was assessed. Two the hydrological model simulations were completed in order to assess sensitivity of hydrology results to adopted rainfall losses. Rainfall losses in the base case are 0mm initial loss and 2mm/hr continuing loss (See Section 7.1.2). Losses used in the sensitivity analyses and modelled peak flows at Dungog are presented in Table 8-1. Flow hydrographs at Dungog for the rainfall parameter sensitivity analyses are presented in Figure 8-1.

Table 8-1 Sensitivity Runs – Rainfall Losses

Name of Sensitivity Simulation	Initial Loss (mm)	Continuing Loss (mm/hr)	Peak Flow at Dungog (m^3/s)
Base Case (1% AEP, 36 hour)	0	2.0	3140
Rainfall Loss Increase	40	4.0	2677
Rainfall Loss Decrease	0	0.0	3567

K:\B16030.k.gjr_WilliamsR_FS\RAFTS\Design_Results\Sensitivity_Runs\[Dungog_Q_Sensitivty_022.xls]Tables

Hydrological modelling results are relatively sensitive to the rainfall loss parameters used. Increasing the continuing loss from 2.0mm/hr to 4.0mm/hr and introducing an initial loss of 40mm results in a decrease in predicted flow of 462m^3/s or 14.7%. Decreasing the continuing losses from 2.0mm/hr to 0.0mm/hr results in a flow increase of 427m^3/s or 13.6%.

8.1.2 Non-Linearity Exponent

The RAFTS software package uses the following equation to model storage in each sub area:

$$S = Bq^{(n+1)}$$

Where:

- S = Storage (hours x m^3/s)
- B = Storage delay time coefficient
- q = discharge (m^3/s)
- n = storage non-linearity exponent

The storage non-linearity exponent in the calibrated hydrologic model was -0.285 (default). Two RAFTS simulations were completed to test the sensitivity of the model to the non-linearity exponent. The sensitivity simulations used values of non-linearity exponent of -0.1 and -0.4. Peak flows at Dungog are presented in Table 8-2. Modelled flow hydrographs at Dungog are presented in Figure 8-2.

Table 8-2 Sensitivity Runs – Non-Linearity Exponent

Name	RAFTS Non-Linearity Exponent	Peak Flow at Dungog (m^3/s)
Base Case (1% AEP, 36 hour)	-0.285	3,140
Non-Linearity Increase	-0.10	2,748
Non-Linearity Decrease	-0.40	3,232

K:\B16030.k.gjr_WilliamsR_FS\RAFTS\Design_Results\Sensitivity_Runs\[Dungog_Q_Sensitivty_022.xls]Tables

The hydrological modelling results are less sensitive to the non-linearity exponent than to rainfall losses. Increasing the storage non-linearity exponent (more linear relationship) from -0.285 to -0.10 results in a decrease in flow of -392m^3/s, this represents a decrease of 12.5%. Decreasing the storage non-linearity exponent (less linear relationship) from -0.285 to -0.40 results in an increase of flow of 92m^3/s or 2.9%.

8.1.3 Storage Delay Time Coefficient (B) Multiplication Factor

The storage equation used in RAFTS is discussed in 8.1.2.

The storage delay time coefficient (B) is calculated in RAFTS (based on sub-catchment area, fraction urbanised and slope of channel). The B multiplication factor is used as calibration factor. The hydrological model was calibrated with a B multiplication factor of 0.8. Two sensitivity simulations were conducted in order to assess the sensitivity of the hydrological modelling results to the B multiplication factor. The sensitivity simulations used B multiplication values of 0.6 and 1.2.

Peak flows at Dungog are presented in Table 8-3. Modelled flow hydrographs at Dungog are presented in Figure 8-3.

Table 8-3 Sensitivity Runs – B Multiplication Factor

Name	RAFTS B Factor	Peak Flow at Dungog (m3/s)
Base Case (1% AEP, 36 hour)	0.8	3,140
B Factor Increase	1.2	3,010
B Factor Decrease	0.6	3,200

K:\B16030.k.gjr_WilliamsR_FS\RAFTS\Design_Results\Sensitivity_Runs\[Dungog_Q_Sensitivty_022.xls]Tables

The hydrological model is less sensitive to B multiplication factor than both rainfall losses and storage non-linearity exponent. Increasing the B factor from 0.8 to 1.2 (a 50% increase) results in a decrease to peak flow of 130m^3/s or 4.1%. Decreasing the B multiplication factor from 0.8 to 0.6 (a 25% decrease) results in an increase to peak flow of 60m^3/s or 1.9%.

8.2 Sensitivity to Hydraulic Model Parameters

Two sensitivity analyses were simulated with the hydraulic model. The base case event that these were based on had the following combination of events;

- 0.5% AEP Williams River inflow;
- 5% AEP Hunter River inflow; and
- 1% AEP level plus 0.91m allowance for sea level rise.

8.2.1 River and Floodplain Manning's n

The majority of the simulations for the calibration exercise were focussed on varying the values of Manning's n throughout the model. To test the sensitivity of the modelling results to the Manning's n used a sensitivity simulation was carried out. The following changes were applied to Manning's n values adopted in the design modelling:

- 20% increase to all values in the rivers and major creeks.
- 50% increase to all values in the floodplain

Results of the Manning's n sensitivity simulation are presented in Figure 8-4. Results for the sensitivity run are significantly higher than the base case. Peak water levels in the Williams River are between 0.11m and 0.91m higher than the base case.

8.2.2 Structure Losses

Bridge structures provide considerable constriction to flow in the Williams River. These structures con convey a large percentage of the flow. A sensitivity analysis was conducted to determine the sensitivity of the results to the structure losses calculated for each structure. Structure losses were multiplied by a factor of two for each structure in the hydraulic model.

Results of the structure loss sensitivity simulation presented in Figure 8-4. Peak water levels in the Williams River are between 0.01m lower and 0.10m higher than the base case.

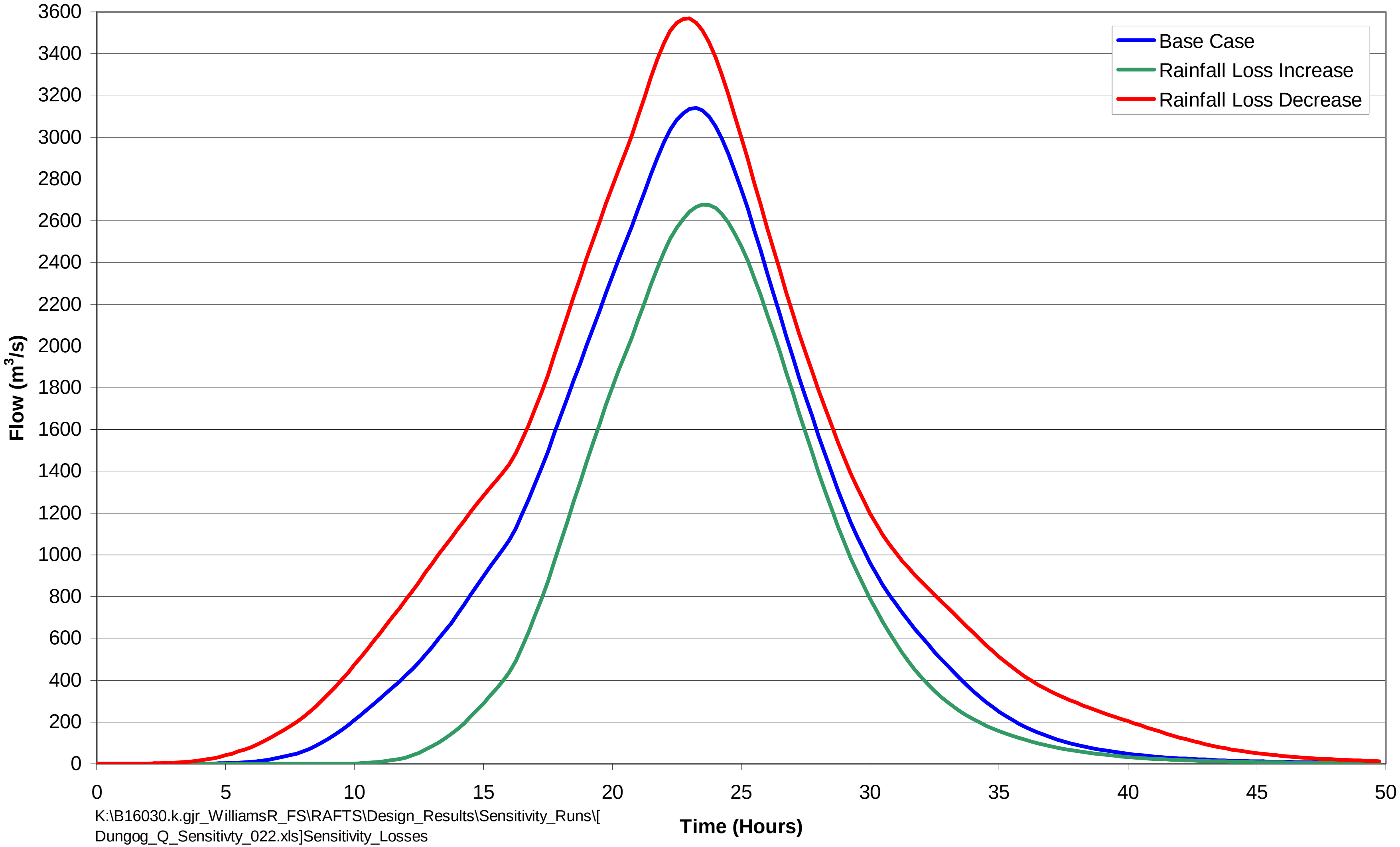


Figure 8-1 Sensitivity of Flows at Dungog to Rainfall Loss Parameters

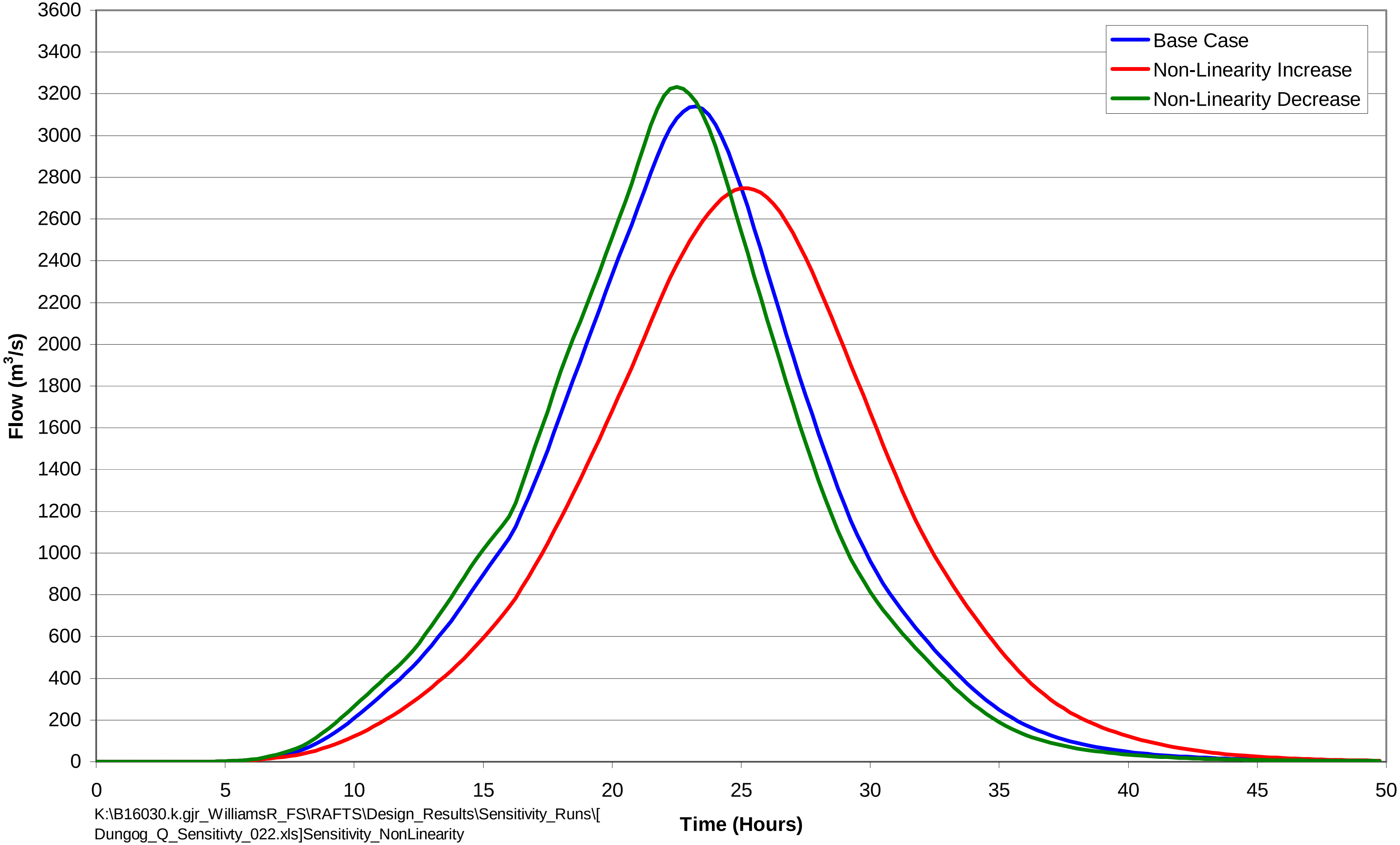


Figure 8-2 Sensitivity of Flows at Dungog to Non-Linearity Exponent

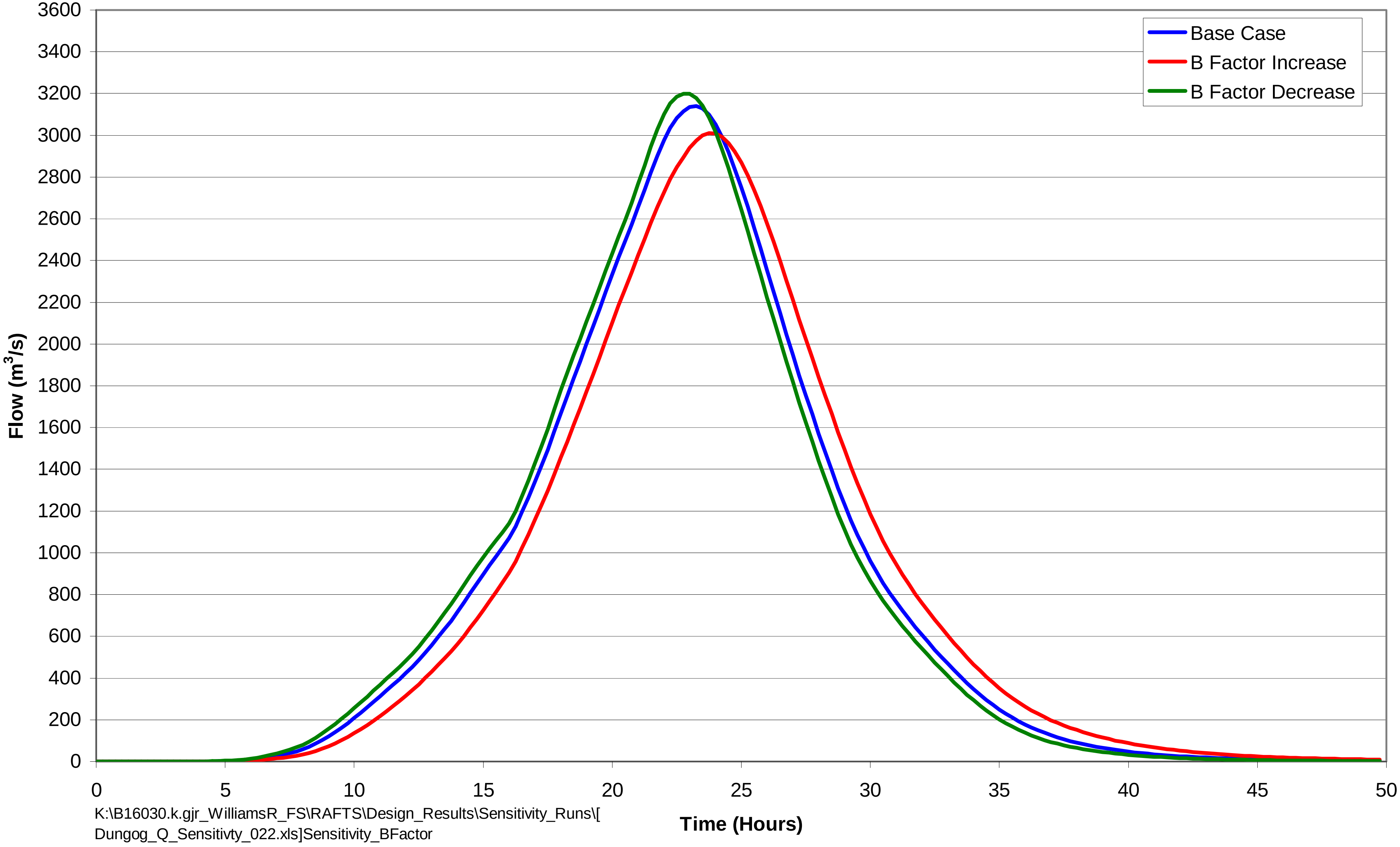


Figure 8-3 Sensitivity of Flows at Dungog to B Factor

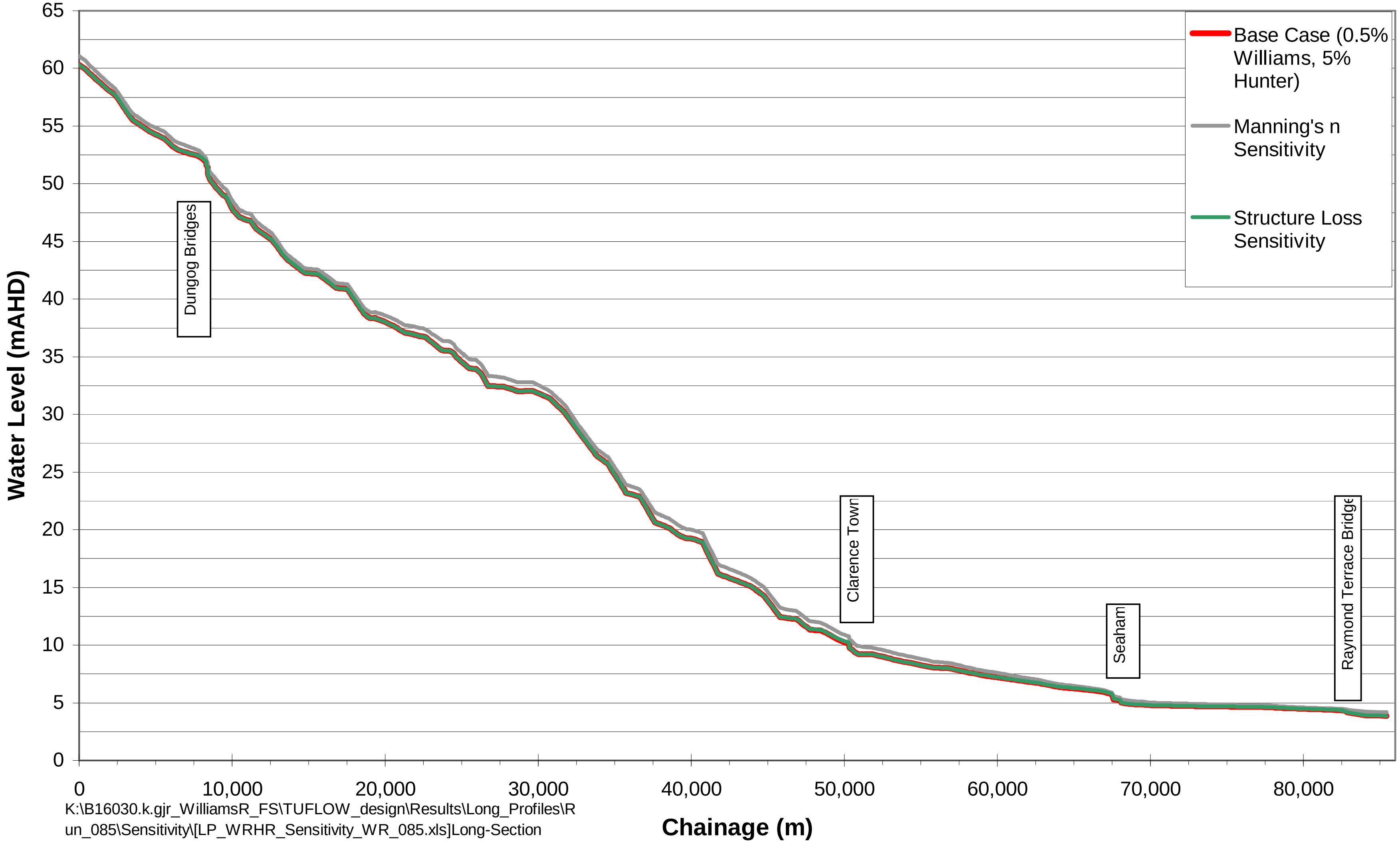


Figure 8-4 Long Section Hydraulic Parameter Sensitivity

9 DESIGN FLOOD BEHAVIOUR

9.1 Presentation of Results

Design flood levels, depths, velocities and velocity-depth product are presented for seven design events. Design flood mapping is presented in Drawing 2 to 71 and an index is presented in Table 9-1. Long sections for all design events are presented in Drawing 72 and 73 for the Williams River and Hunter Rivers respectively.

The results for the 0.5%, 1%, 2% and 5% AEP are a maximum envelope of Williams River and Hunter River events (see Section 7.1.8).

Peak water level does not occur everywhere at the same time. Therefore, values presented are based on the maximum that occurred at each computational point in the model during the entire flood. Hence, results presented do not represent an instantaneous point in time, but rather an envelope of the maximum values that have occurred.

Unless otherwise stated, presentations in this report are based on peak values. Flood level and depth are peak value to occur at any computational timestep. Peak velocity and peak velocity depth product presented are calculated at the peak water level. See Section 9.7 for discussion on the interpretation and accuracy of the results presented.

Levels, flows and velocities for the left and right banks and main channel are presented in tabular format in Table 9-4 to Table 9-14. The geographical location of the tabular outputs is presented in Figure 9-5. To prevent data from different events being mixed (e.g. presenting maximum of the 1% AEP events), results for all 11 simulations are presented.

Table 9-1 Drawing Numbers for Design Maps

Event	Levels	Depth	Velocity	Velocity-Depth	Hazard
0.50%	2 and 3	4 and 5	6 and 7	8 and 9	10 and 11
1%	12 and 13	14 and 15	16 and 17	18 and 19	20 and 21
2%	22 and 23	24 and 25	26 and 27	28 and 29	30 and 31
5%	32 and 33	34 and 35	36 and 37	38 and 39	40 and 41
10%	42 and 43	44 and 45	46 and 47	48 and 49	50 and 51
20%	52 and 53	54 and 55	56 and 57	58 and 59	60 and 61
PMF	62 and 63	64 and 65	66 and 67	68 and 69	70 and 71

I:\B16030_I_brh Williams River PEV\DRG\A3_Drawing_Addendum\[Drawing_Addendum_Index.xls]Table

9.2 Results at River Gauging Locations

The following tables summarise the level-stage-AEP-flow relationships for the river gauging stations. The relationship is derived from the hydraulic modelling results.

The relationship for Dungog gauge is presented in Table 9-2. Water Level in metres AHD (mAHD) is provided as well as the gauge level for the stage staff and the automated DNR water level recorder. Figure 9-6 shows the relationship between stage (on the Dungog gauge) and probability of flood event.

Table 9-2 Stage-AEP-Flow Relationship at Dungog Gauge

Level (mAHD)	Gauge Staff Height (m)	DNR Gauge Level (m)	Annual Exceedance Probability	Flow (m3/s)
49.91	8.81	8.69	20%	1,485
50.35	9.25	9.13	10%	1,842
50.87	9.77	9.65	5%	2,301
51.35	10.25	10.13	2%	2,721
51.88	10.78	10.66	1%	3,150
52.40	11.30	11.18	0.50%	3,838
56.63	15.53	15.41	PMF	11,361

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Stage_Flow_ARI\[Dungog_Gauge.xls]Stage-AEP-Flow

The relationship for Glen Martin gauging station is presented in Table 9-3. Figure 9-7 shows the relationship between stage (on the Glen Martin gauge) and probability of flood event.

Table 9-3 Stage-AEP-Flow Relationship for Glen Martin Gauge

Level (mAHD)	Gauge Height (m)	Annual Exceedance Probability	Flow (m3/s)
10.57	10.68	20%	1,679
11.72	11.83	10%	2,165
12.90	13.02	5%	2,781
13.81	13.92	2%	3,434
14.43	14.55	1%	4,080
15.08	15.20	0.50%	4,830
20.71	20.82	PMF	13,239

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Stage_Flow_ARI\[Glen_Martin_Gauge_090108.xls]Stage-AEP-Flow_Design_Results

9.3 Design Flood Levels, Depths and Extents

Design flood levels and depths are presented in the A3 drawing addendum for all seven design events (see Table 9-1 for index).

Levels in the lower Williams River are dictated by the Hunter River flooding. For the events where a maximum envelope of two events (Williams and Hunter River floods) the Hunter River flood produces higher water levels in the lower Williams River. The extent of the Hunter River dominance varies with the event but is in typically within 1km of Seaham Weir. The extent of the Hunter River influence is shown on the water level maps for these events.

In relation to design model results for flood level and depths, the following points are made:

- High depths and flows are predicted for sections of the floodplain north of Seaham even in smaller events (20% AEP);
- Large areas of the floodplain experience high depths of greater than 4m in rarer events;
- There is considerable head gradient in the vicinity of Seaham Weir. This is due to a number of factors; constriction of flow, the sharp bend in river upstream of weir and losses across weir;
- Flood gradients in the lower Williams River are relatively flat.

The flood extents for the 0.5% and 1% AEP flood events were refined from the model results to produce smoother edges of the extent. This process was carried out by intersecting and extended flood surface with the 5m resolution Digital Elevation Model. The resulting smoothed flood extents for these two events are presented in Figure 9-8 and Figure 9-9.

9.4 Design Flood Flows and Velocities

Design flood flows and velocities are presented for all seven design events in the A3 drawing addendum (see Table 9-1 for an index).

In relation to design model results for flood flows and velocities, the following points are made:

- Large portions of the floodplain have high flows;
- Breakout over the levee system south of Seaham is predicted in 20% and 10% AEP events. Flow over the weir is shallow and peak water level behind weir is much less than the level in the river;
- In the 20% AEP event there are river bends where approximately 50% of the flow is conveyed in the floodplain.

9.5 Flood Behaviour Dynamics

9.5.1 Lower Floodplain

The levees are predicted to first overtop (in a Williams River flood event) in the section immediately south of Seaham. In larger events overtopping occurs along virtually the entire length of the levee.

Peak flows in the Hunter and Williams Rivers are timed to coincide, this leads to overtopping of the levees on the Hunter and Williams Rivers occurring at a similar time. Overtopping from the Hunter River fills up the lower portion of the floodplain. Levels and velocity vectors at selected times are presented for a 1% AEP Williams event and 1% AEP Hunter Event in Figure 9-1and Figure 9-2 respectively.

Levels and velocities at selected times for a 10% AEP Williams River event (no Hunter River inflows) are presented in Figure 9-3.

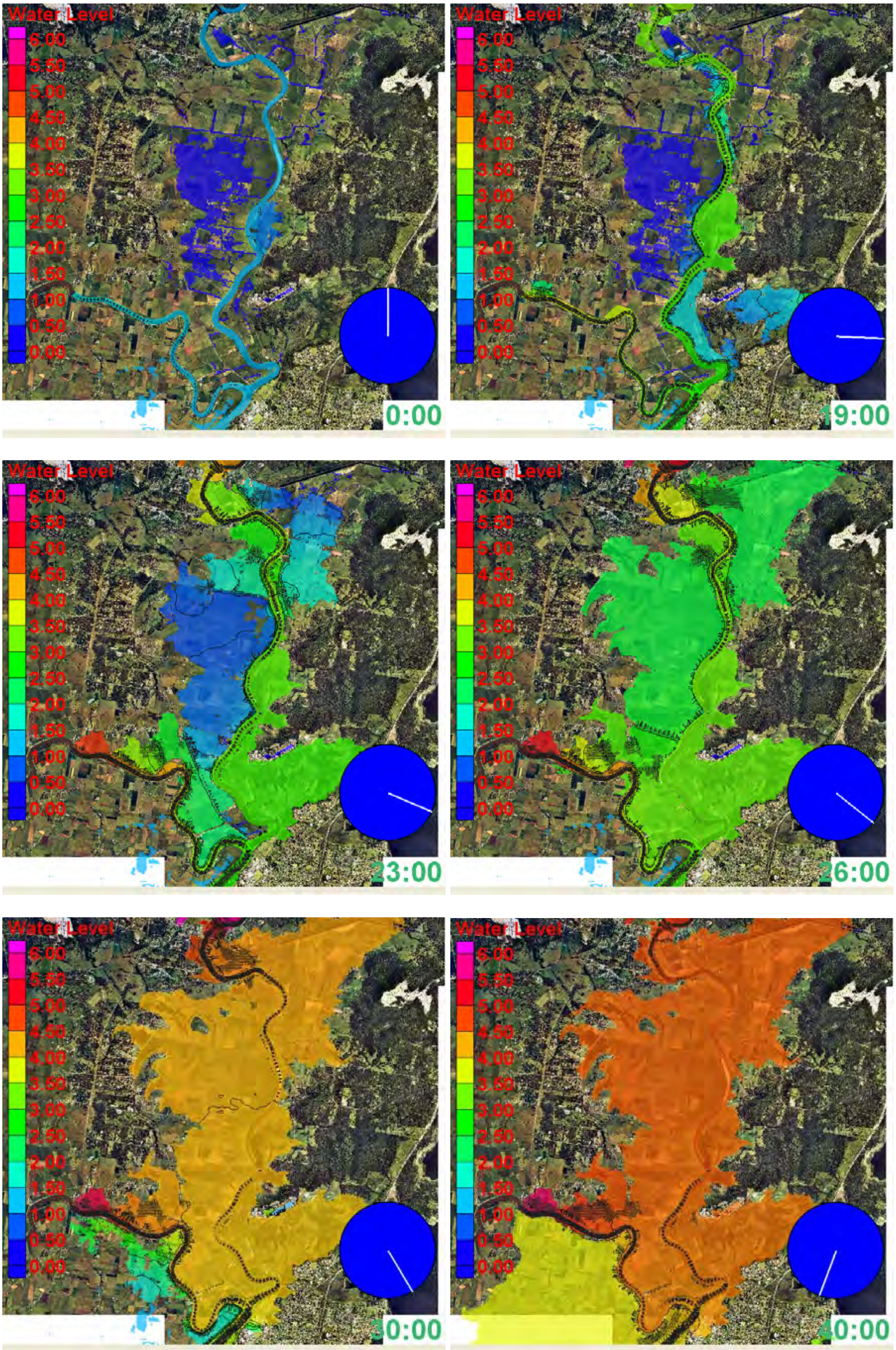


Figure 9-1 Lower Floodplain 1% AEP Williams River Event

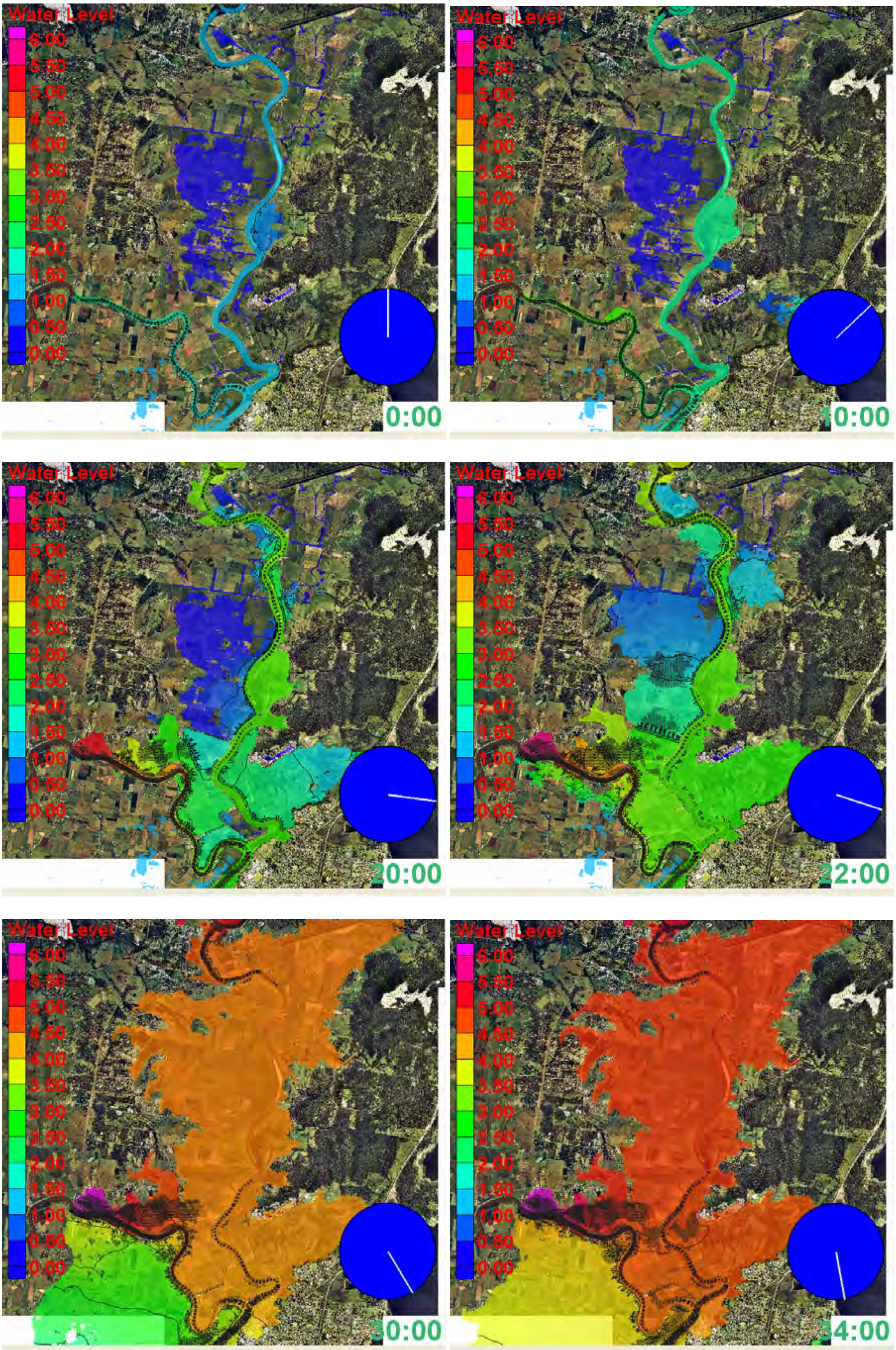


Figure 9-2 Lower Floodplain 1% AEP Hunter River Event

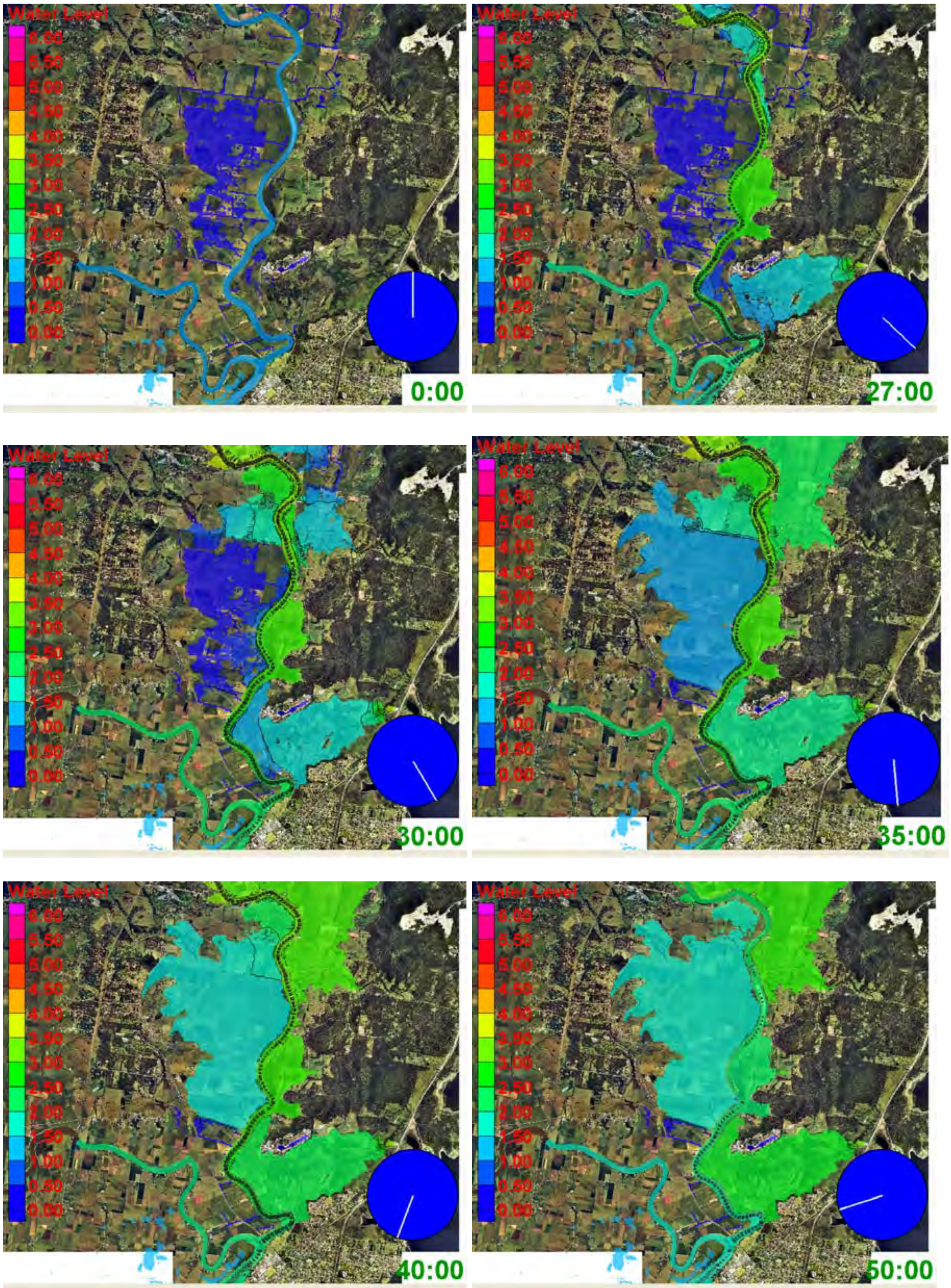


Figure 9-3 Lower Floodplain 10% AEP Williams River Event

9.5.2 Dungog

With no structures to protect Dungog, the inundation gets progressively worse as the events get larger. Water back up from Myall creek causes flooding to properties in the northern sections of town. Streets most affected are Hooke Street and Lord St. Flooding from the Williams River affects properties in Chapman St.

9.5.3 Clarence Town

Flooding in Clarence Town is backwater from the river, with flows and velocities typically low. The main areas of Clarence Town affected by flooding are the land between Grey Street and Rifle Street as well as the Southern end of Durham St. In larger events flooding backs up from Rifle Street to Queen Street.

9.5.4 Seaham

Flooding is predicted in small events (20% AEP) in both the low-lying area east of Warren Street and the rear of the properties east of Still Street. East Seaham Road is also overtopped in smaller events. In larger events inundation across Warren Street (near Nelson Street) occurs. Inundation is predicted to occur in the vicinity of Dixon Street and Brandon Street for larger events.

9.5.5 Raymond Terrace

There is no flooding from the rivers in Raymond Terrace for the 20% and 10% AEP events. The levees protecting Raymond Terrace are overtopped in 5% AEP and rarer events. Water level gradient across the levee flattens out, with water levels behind the levee similar to those predicted in the river. In larger events the predicted flood extents increase with no significant change in flood behaviour. The major areas affected are Hunter Street, King Street, Port Stephens Street and Carmichael St. Water levels are higher in Hunter River events compared to Williams River events.

9.6 Provisional Flood Hazard Mapping

The flood hazard level is often determined on the basis of the predicted flood depth and velocity. A high flood depth will cause a hazardous situation at low or no velocity. High velocities are dangerous at shallower depths and may cause structural damage (e.g. scour).

The NSW Government Floodplain Management Manual (2005) defines flood hazard categories as follows.

- **High hazard** possible danger to personal safety; evacuation by trucks difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings.
- **Low hazard** should it be necessary, truck could evacuate people and their possessions; able-bodied adults would have little difficulty in wading to safety.

Figure L2 of the floodplain management manual (NSW, 2005) are used to determine hazard categorisation. This figure is reproduced in this report as Figure 9-4.

Maps of provisional flood hazard are presented in the A3 drawing addendum (see Table 9-1 for an index).

9.7 Hydraulic Categories

Three hydraulic categories are defined in the NSW Floodplain Development Manual (2005): floodways, flood storage and flood fringe. The definition of the three hydraulic categories is based on qualitative assessments rather than quantitative thresholds.

Floodways are determined first. Floodways are the areas where a significant volume of water flows during floods according to the NSW Floodplain Development Manual.

For the purpose of studying the flow distribution, the total flow can be divided in unit flows (flow per meter width) across the floodplain. The integration of the peak unit flows along lines perpendicular to the main flow provides similar total flow values. Lines perpendicular to the main flow were digitised at close spacing down the catchment. The average unit flow of this line was determined. Unit flow at points spaced regularly along each of these lines was compared to the average unit flow, points of greater than average were defined as being within the floodway.

Sections with high but uniform flow across floodplain were originally defined as non-floodway in this process (with the exception of the main channel). To overcome this, floodway extents from the average unit flow process were combined with areas of high velocity-depth product (greater than $1.0m^2/s$).

Once the floodways were determined, the remainder of the floodplain is a combination of flood storage and flood fringe areas. The floodplain areas outside of the floodways are essentially characterised as flood storage. The flood fringe areas are those areas within the flood storage that contains a volume of water of small significance for the flood behaviour. Filling of these areas would have a minimal impact on flood behaviour.

The flood fringe was calculated using the following process.

1. The floodplain was divided into smaller regions. 21 regions were used over the floodplain.
2. In each region the volume of water required to raise the floodway area by a depth of 0.1m was determined.
3. The flood fringe areas were those of lowest depth required with equivalent volume to that calculated in step 2.
4. The resulting areas were then smoothed to remove small islands and irregular areas.

The provisional hydraulic categories for the 0.5% and 1% AEP are presented in the A3 drawing addendum as Drawings 78 to 81.

9.8 Interpretation of Results

The interpretation of the drawings, maps and other data presented in this report should include an appreciation of the limitations of the accuracy. While the points below highlight these limitations, it is important to note that results presented provide an up-to-date, conservative prediction of design flood behaviour using the best flood modelling techniques currently available. Points to remember are:

- Recognition that no two floods behave in exactly the same manner;
- Design floods are a best estimate of an “average” flood for their probability of occurrence;
- The photogrammetry used to generate the DEM has uncertainties associated. The photogrammetry used for this study has a quoted accuracy of ±0.5m. Flood depths and flood extents, which are determined using this DEM, should be interpreted accordingly.

All design floods are based on statistical analyses of **recorded** data such as rainfall and flood levels. Statistical analysis is used in the creation of design isopleths (AR&R, 1987) as well as the flood frequency analysis (WBM and George Kuczera). The longer the period of recordings that the statistical analysis is based on, the greater the certainty. For example, derivation of the 1% AEP rainfall from 20 years of recordings would have a much greater error margin than from 100 years of recordings.

Similarly, the accuracy of the hydrologic and hydraulic computer models is dependent on the amount and range of reliable rainfall and flood level recording for model calibration.

The error margin in this study is regarded as better than moderate due to:

- A reasonable amount of rainfall and flood level data;
- Calibration and verification of the hydrologic and hydraulic models to three historical events;
- The model parameters being generally typical of those used nearby.

Data that would have significantly reduced the error margin are:

- Continued long-rainfall (particularly pluviograph data) and river flood levels;
- Peak flood levels on the floodplain (installation of peak flood recorders is relatively inexpensive);
- Peak flood levels upstream and downstream of major structures and along major river bends.
- Stream flow gauging during flood time.

9.9 Effect of Coincident Hunter River Flood

The results in the lower part of the study area (i.e. downstream of Seaham) are also somewhat dependant of the assumptions made regarding the coincident flood events in the Hunter River. As discussed in Section 7.1.8, smaller Hunter River flood events were assumed to coincide with large Williams River events (e.g. 1% AEP Williams River flood with 5% AEP Hunter River flood).

However, it is possible that large Williams River flood events could occur with much smaller Hunter River flood events (e.g. 1% AEP Williams River flood with 20% AEP Hunter River flood). In these cases, the flood gradients downstream of Seaham would be steeper than those assumed in this study. This type of flood event would have lower flood levels and higher velocities downstream of Seaham.

As the flood mapping presented in this report is an envelope of two flood events (for the larger, rarer flood events), the assumptions regarding the coincident Hunter River flood have little bearing on the envelopes presented in this report. However, it should be noted that Williams River flood events with small or no flood in the Hunter River would produce higher velocities than presented in this report. Further, these flood events are likely to show the highest impacts for floodplain changes (e.g. residential filling, road embankments etc). In future studies and impact assessments, these Williams River flood events with small or no flood in the Hunter River will also require appropriate consideration.

9.10 Effect of Chichester Dam

Chichester Dam is located in the upper Williams River catchment and is above the floodplain study area. Chichester Dam is modelled in the hydrologic model, which is used to derive flows to the hydraulic model.

To test the sensitivity of the design modelling on the initial water level in Chichester Dam, the hydrology model was simulated with the dam initially empty. Inflows generated using the hydrologic model were then inputted to the hydraulic model to predict water levels.

The following event combination was in the Chichester Dam sensitivity analysis:

- 1% Annual Exceedance Probability (AEP) Williams River flood;
- 5% AEP Hunter River flood, peaks flows coincide at Raymond;
- The downstream boundary water level (Hunter River) is a generic spring tide.

The inflows to the upstream of the hydraulic model for the Chichester Dam sensitivity run are presented in Figure 9-10. The hydrological model results indicate the peak inflow (to hydraulic model) decreases from 2,730m^3/s for the base case design run to 2,440m^3/s for the Chichester sensitivity run. Chichester Dam being initially empty results in a reduction of peak flow into the hydraulic model of 290m^3/s or slightly greater than 10%.

Modelling results predict that Chichester Dam being initially empty will cause the following reductions in peak water levels at key locations:

- 420mm at Dungog
- 580mm at Glen Martin
- 230mm above Seaham weir
- 100mm below Seaham weir
- 35mm at the Raymond Terrace Bridge.

The maximum decrease in water level between the base case and the Chichester sensitivity run are 730mm and occur approximately 2km upstream of Glen Martin. Long profiles of the modelled water surface from the upper extent of hydraulic model to Glen Martin are presented in Figure 9-11.

Chichester Dam has flood storage above the full supply level of 156.2mAHD. This results in the dam having a lagging effect on flood propagation. Thus, even when dam is initially full (as assumed in all design runs) the dam has an effect on flows below dam. Results from hydrological model for the 1% AEP event indicate that peak inflow to dam is 1,303m^3/s and peak outflow is 1,265m^3/s. Chichester Dam results in a decrease to peak flows of 38m^3/s or 3%. Chichester Dam inflows and outflows for the 1% AEP event are presented in Figure 9-12.

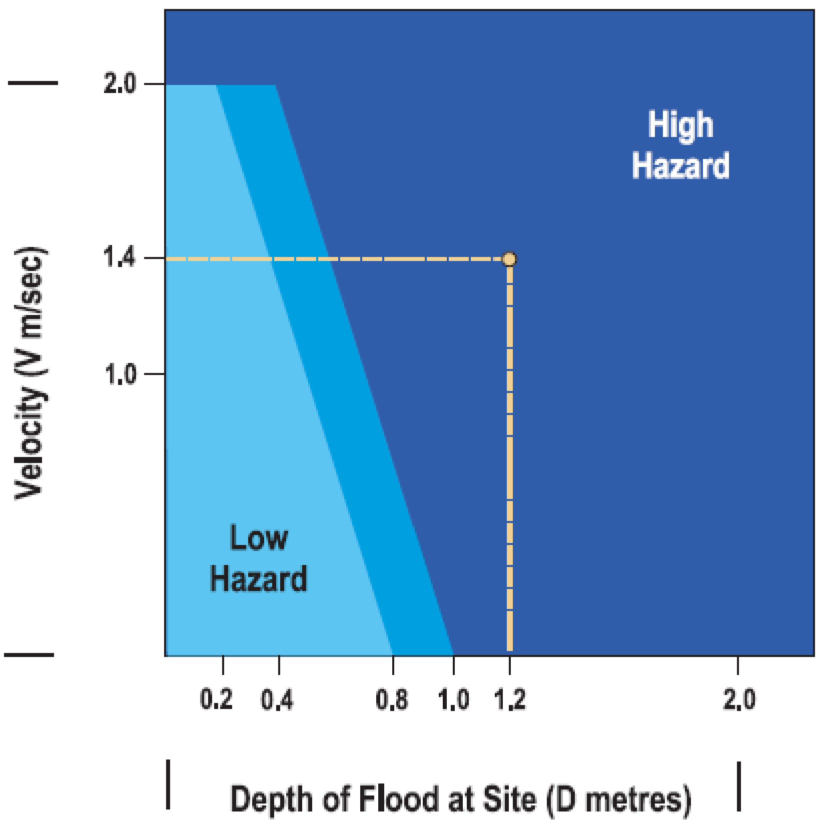
The following comments are made in regard to the assumption in the design events that Chichester Dam is full at the start of the flood event:

- The size of the dam is relatively small in relation to the size of its catchment. Hence, it is full or near full for a significant proportion of the time;
- It is likely that large flood events (eg 1% AEP) will occur in the wet season and in a wet year.

Hence, it is concluded that there is a strong likelihood that the dam would be full at the start of the flood event.

9.11 Effect of Grahamstown Dam

Inflows and outflows from Grahamstown Dam are presented in Figure 9-13 for a 1% AEP 36 hour design event. Peak inflow to the dam is 162m^3/s and peak outflow in 130m^3/s. This represents a decrease in flow of 20%. Flows associated with the dam are relatively small in comparison to flows in the Williams and Hunter Rivers. Sensitivity testing found there was little change in predicted peak water levels by varying the initial dam level from full to empty.



Notes

The degree of hazard may be either -

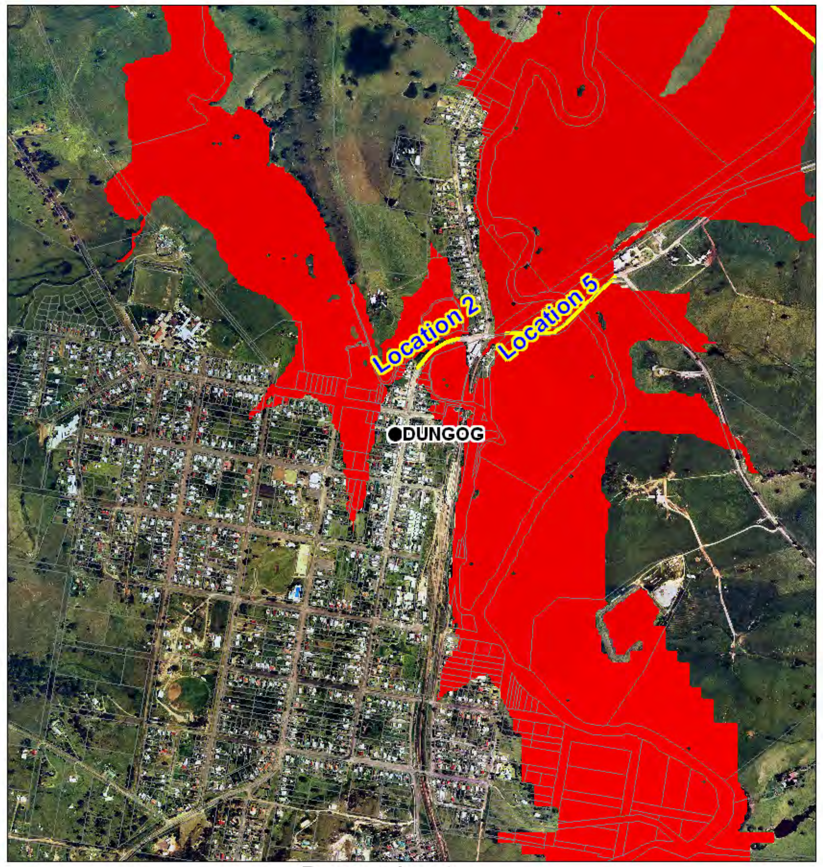
- reduced by establishment of an effective flood evacuation procedure.
- increased if evacuation difficulties exist.

In the transition zone highlight by the median colour, the degree of hazard is dependant on site conditions and the nature of the proposed development.

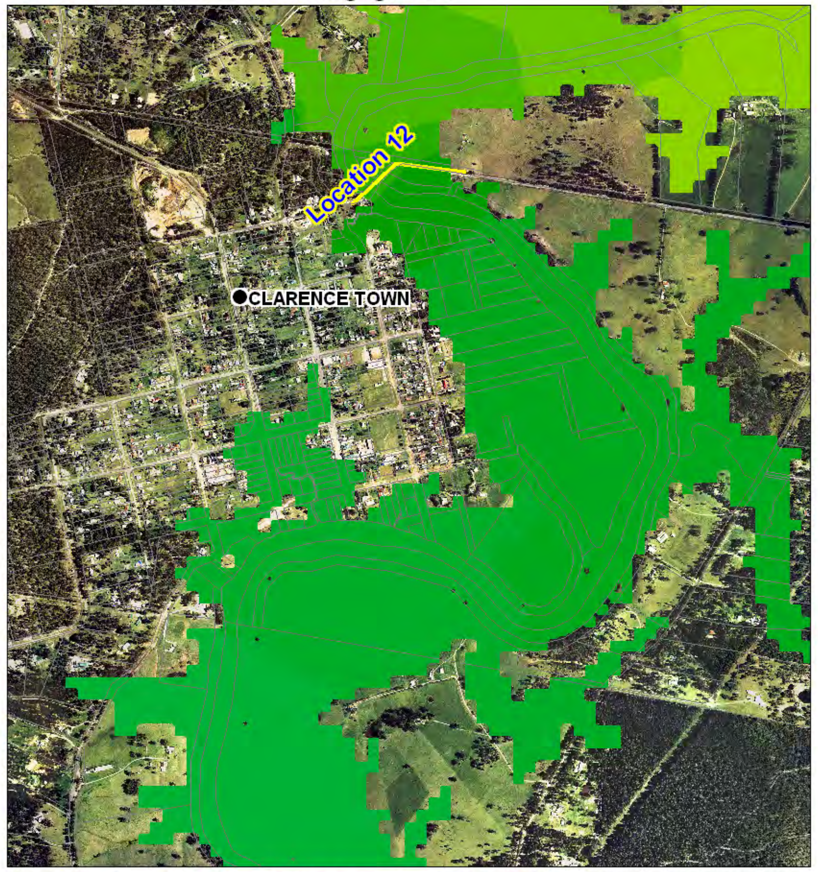
Example:

If the depth of flood water is **1.2 m**
and the velocity of floodwater is **1.4 m/sec**
then the provisional hazard is **high**

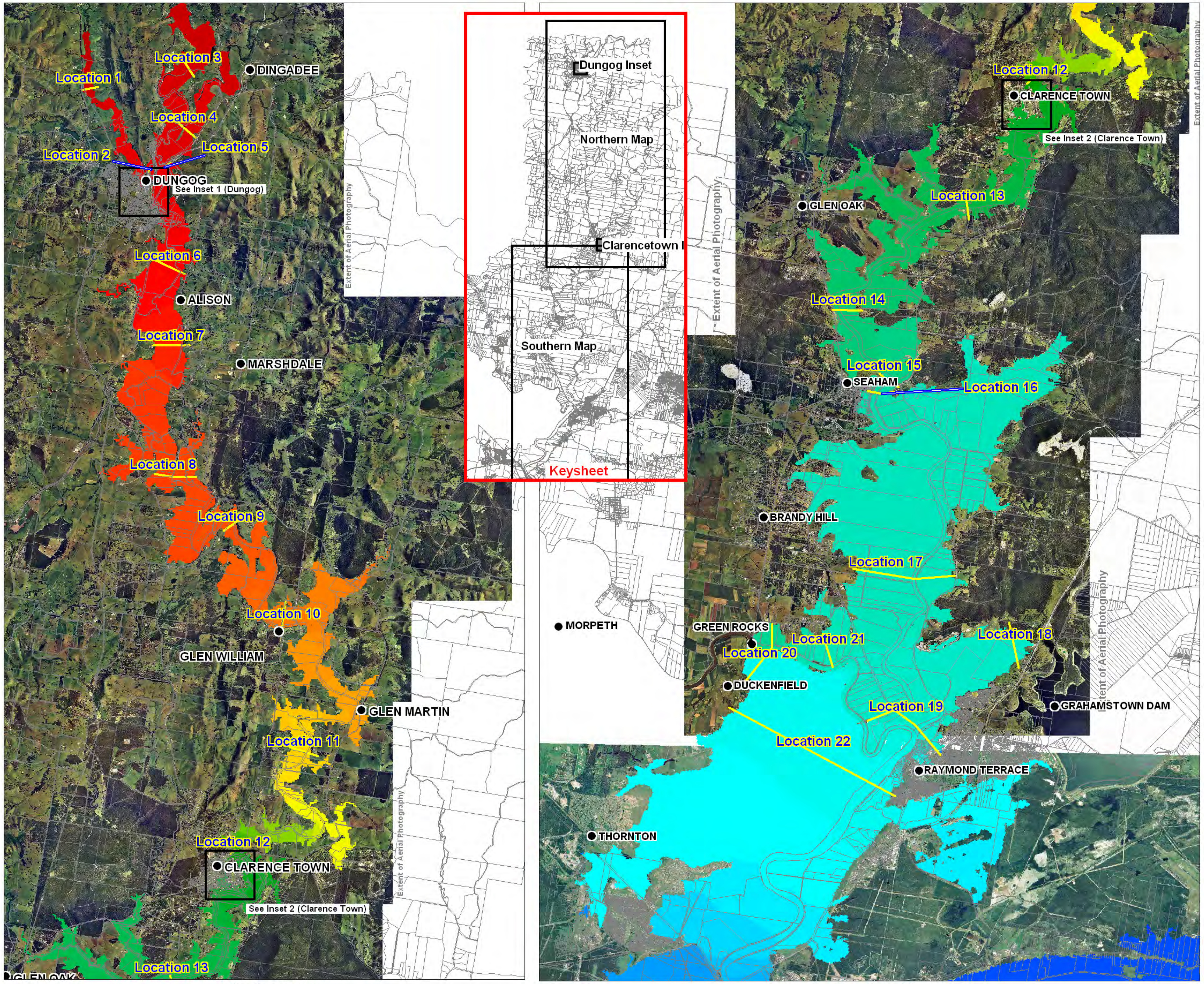
Figure 9-4 Provisional Hydraulic Hazard Categories (NSW, 2005)



Dungog Inset



Clarencetown Inset



Northern Map

Southern Map

Modelled Water Level (mAHD)

LEGEND

- Towns
- Cadastral Boundaries
- Location of Output

Title:

Location of Tabular Output

Figure: 9-5

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

N

Scale - Maps 1 and 2: 0, 2.5, 5km

Scale - Insets 1 and 2: 0, 0.5, 1km

BMT WBM

www.wbmpl.com.au

Filepath : I:\B16030_I_brh Williams River PEV\DRG\Design_Report\FLD_001_080601_Base_A3_workspace.WOR

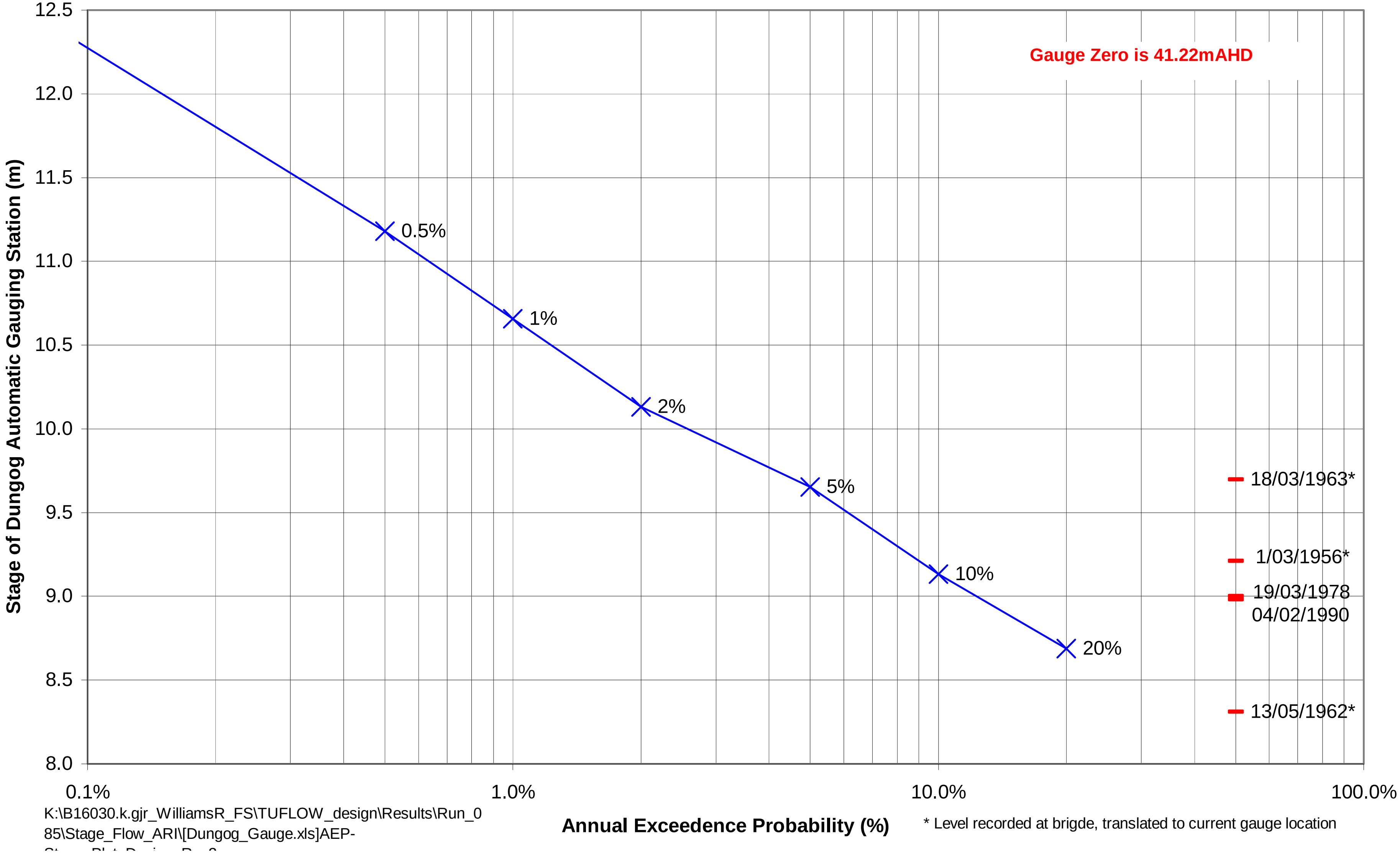


Figure 9-6 Stage vs Probability Relationship for Dungog Gauge

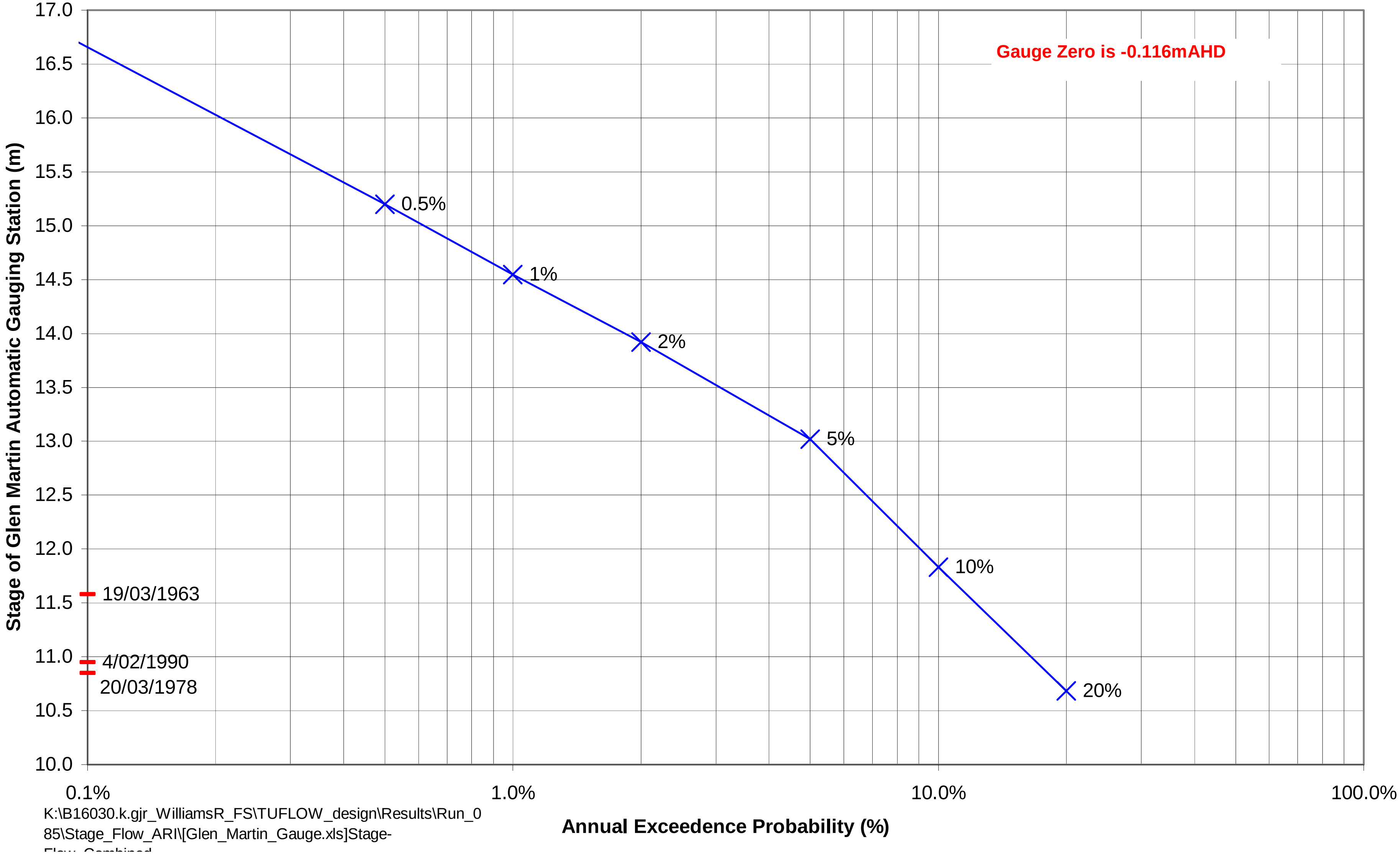
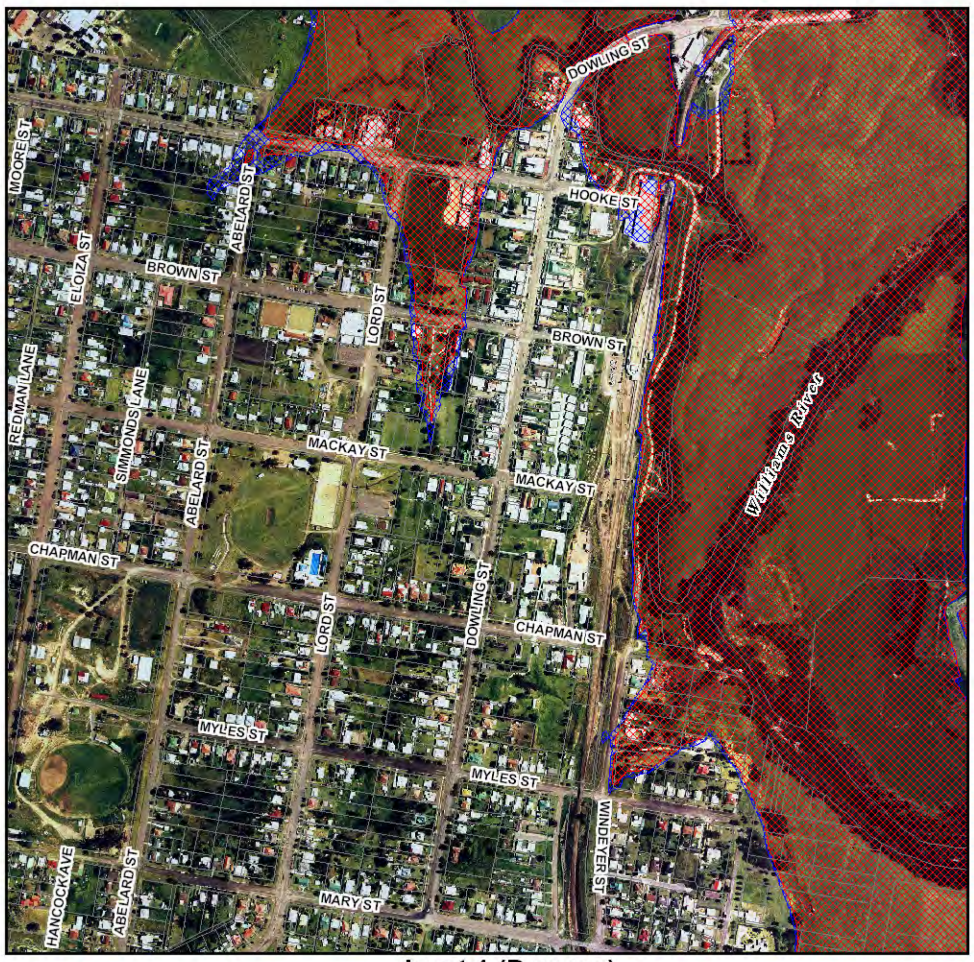
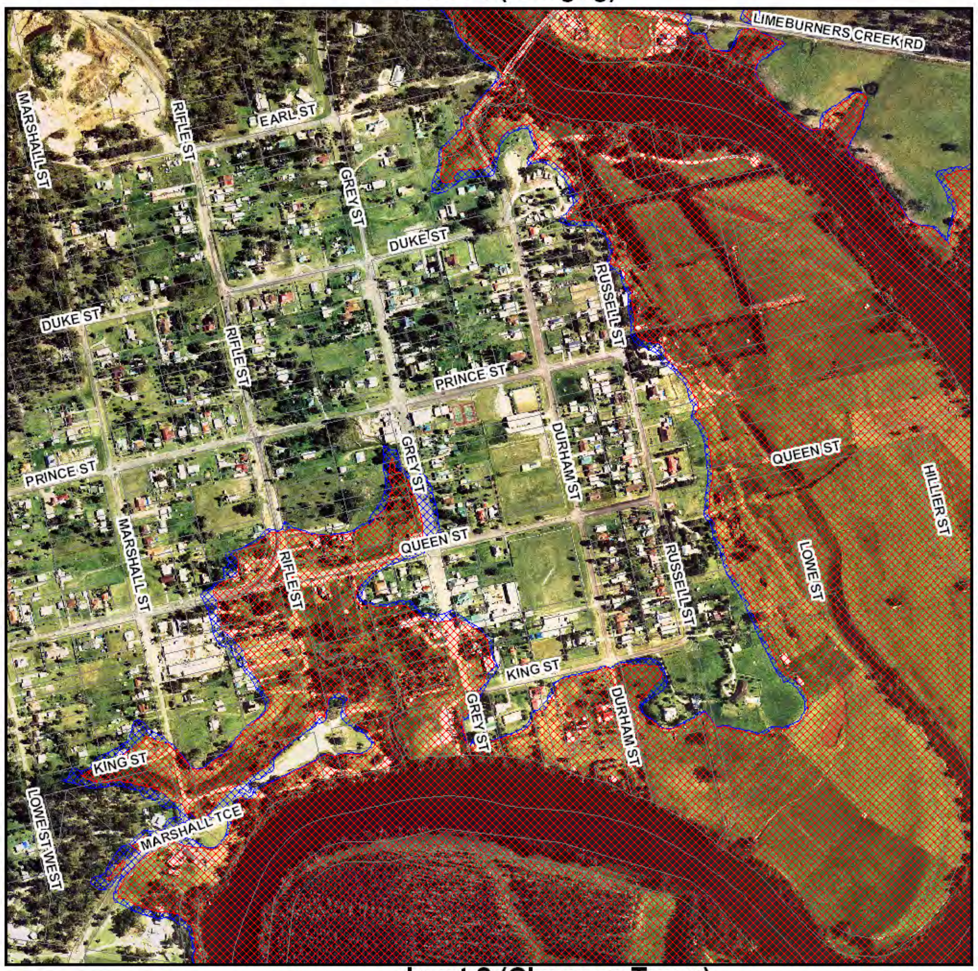


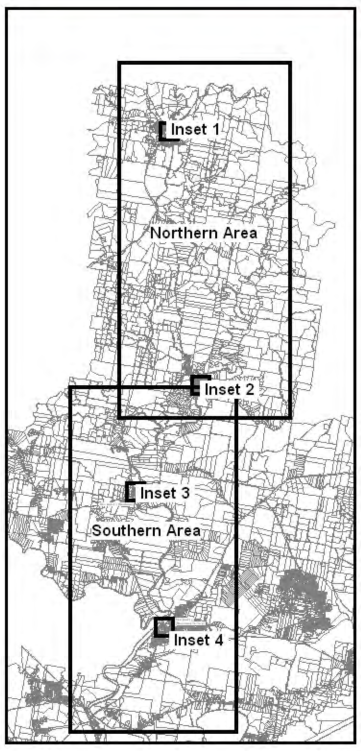
Figure 9-7 Stage vs Probability Relationship for Glen Martin Gauge



Inset 1 (Dungog)



Inset 2 (Clarence Town)

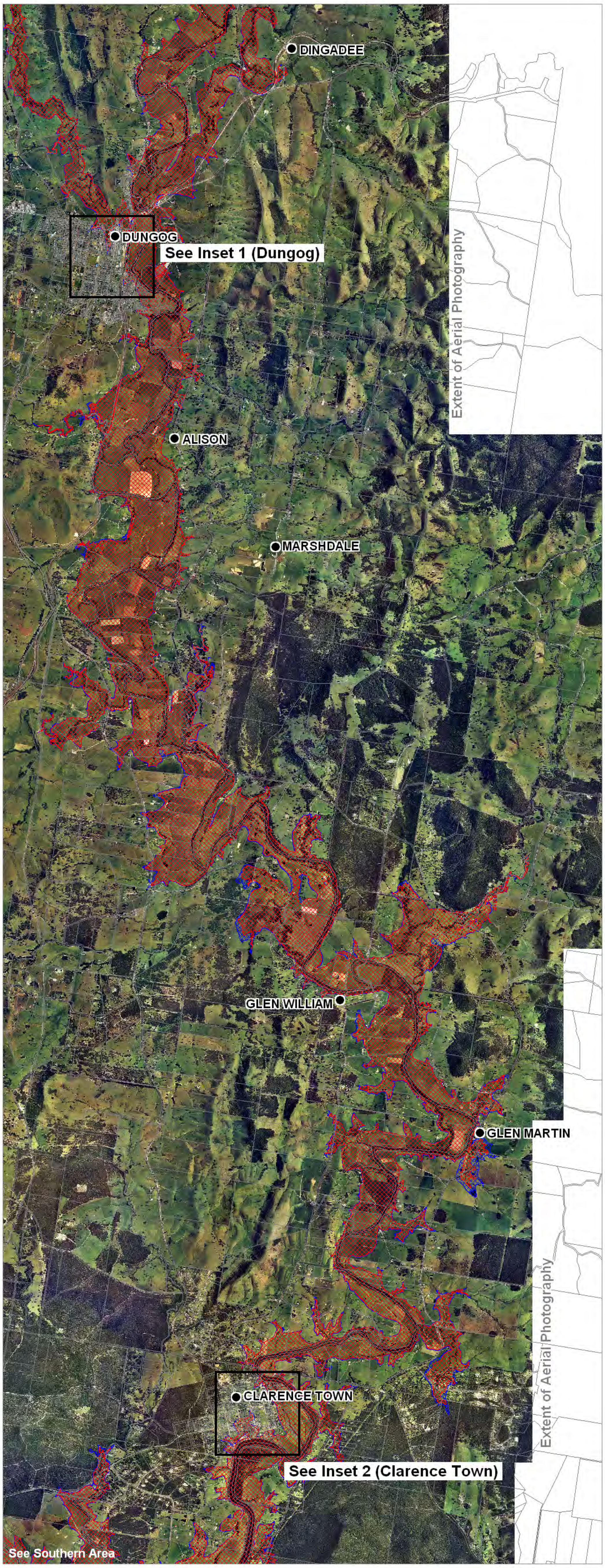


LEGEND

- Towns
- Cadastral Boundaries

Extent of Flooding

- 1% AEP Event (Maximum Envelope)
- 0.5% AEP Event (Maximum Envelope)



Title:

0.5% AEP and 1% AEP Flood Extents: Northern Area

Figure:

Figure 9-8

Rev:

A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

N

0 1 2km
Scale - Main Map

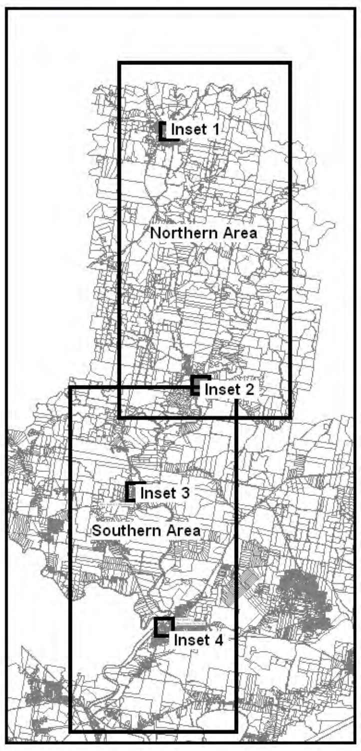
0 200 400m
Scale - Inset

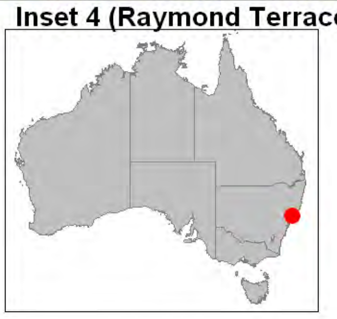
BMT WBM
www.wbmpl.com.au

Filepath : \\wbmfs2\drafting\B16030_I_brh Williams River PEV\DRG\Design_Report\FLD_015_080902_Q100_Q200_Extents_A3.WOR

Inset 3 (Seaham)

Inset 4 (Raymond Terrace)



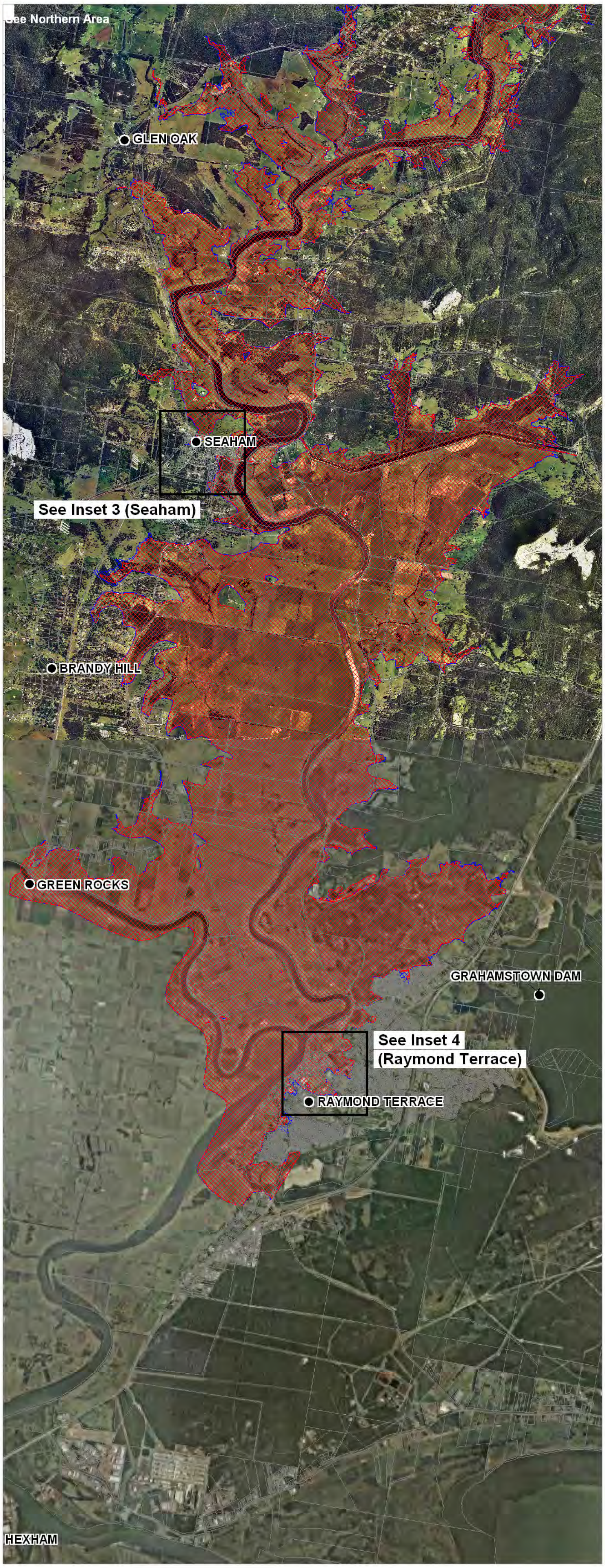


LEGEND

- Towns
- Cadastral Boundaries

Extent of Flooding

- 1% AEP Event (Maximum Envelope)
- 0.5% AEP Event (Maximum Envelope)



Title:

0.5% AEP and 1% AEP Flood Extents: Southern Area

Figure:

Figure 9-9

Rev:

A

N

0 1 2km

Scale - Main Map

0 200 400m

Scale - Inset

BMT WBM

www.wbmpl.com.au

Filepath : \\wbmfs2\drafting\B16030_I_brh Williams River PEV\DRG\Design_Report\FLD_015_080902_Q100_Q200_Extents_A3.WOR

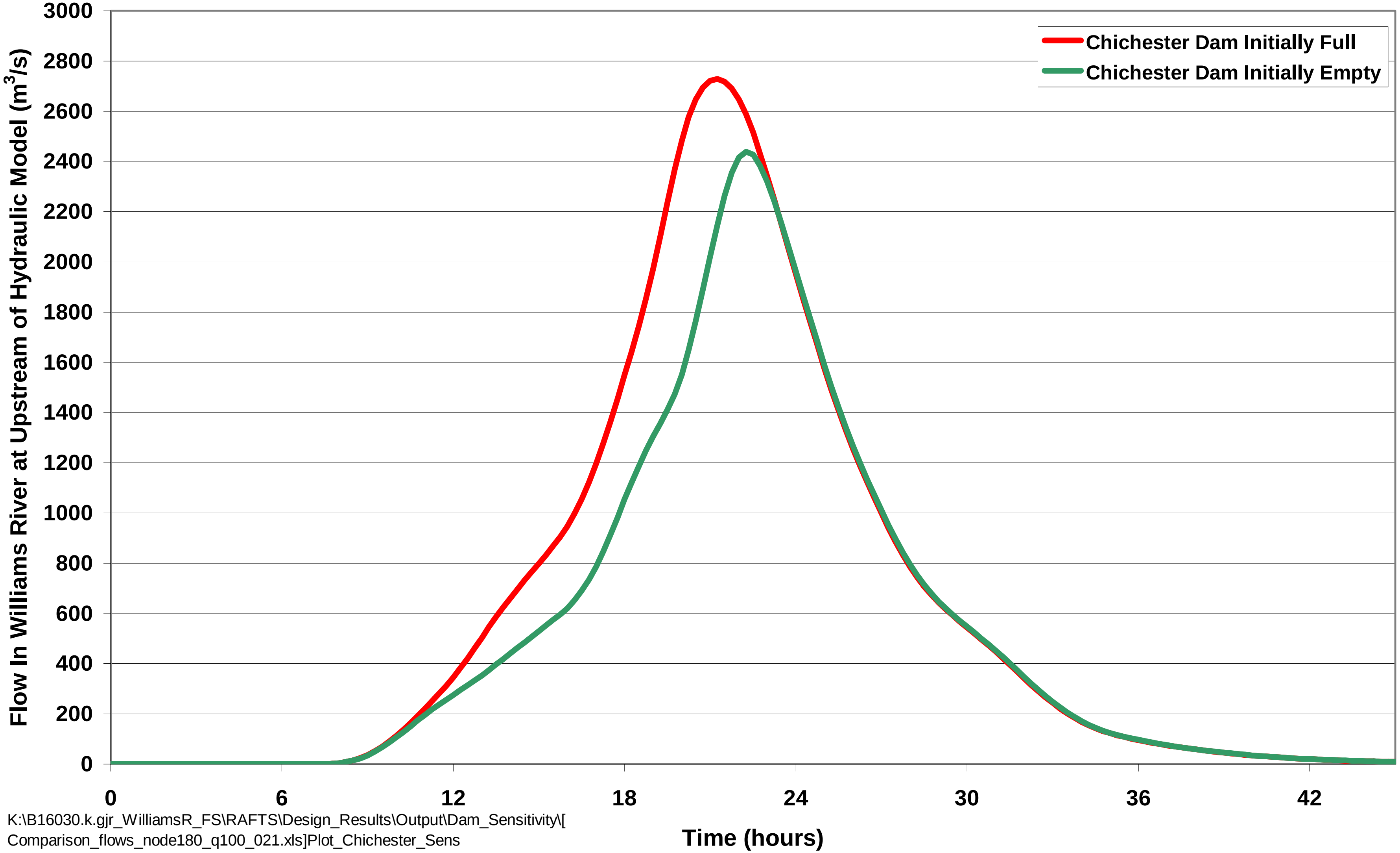


Figure 9-10 Sensitivity of Inflows to Hydraulic Model to Initial Chichester Dam Level

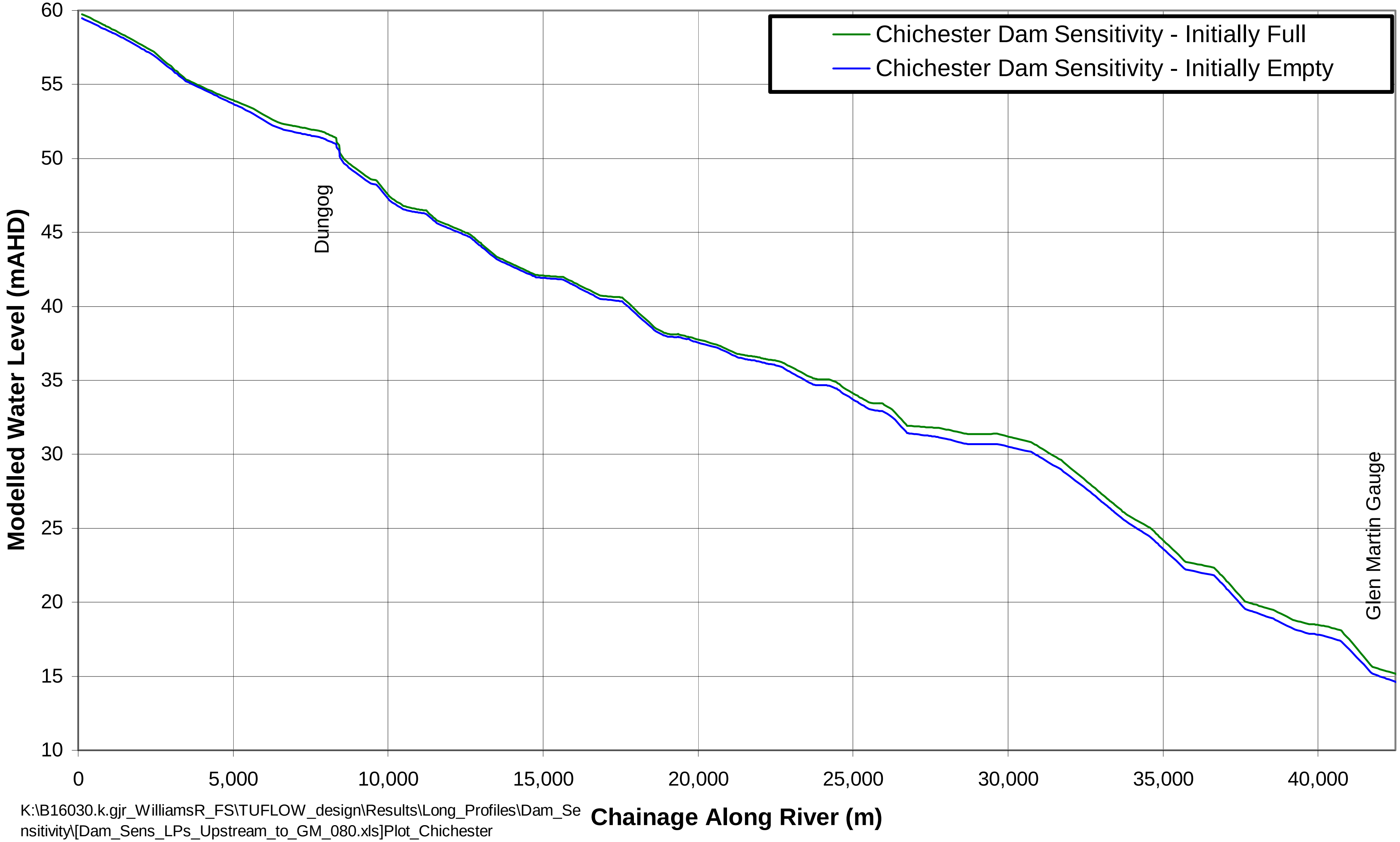


Figure 9-11 Sensitivity Peak Water Level to Chichester Dam Initial Level

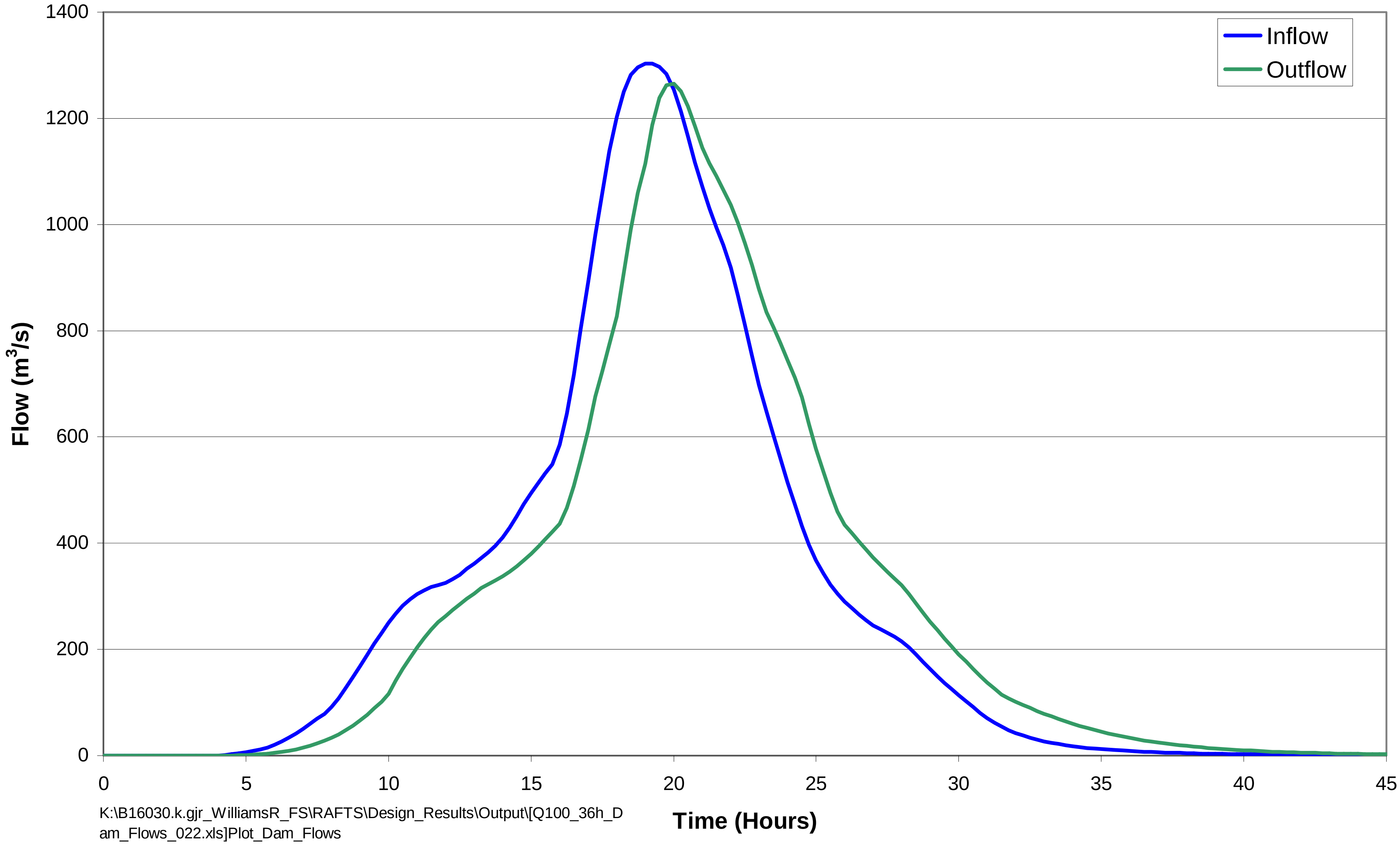


Figure 9-12 Chichester Dam Inflows and Outflows – 1% AEP 36 hour Event

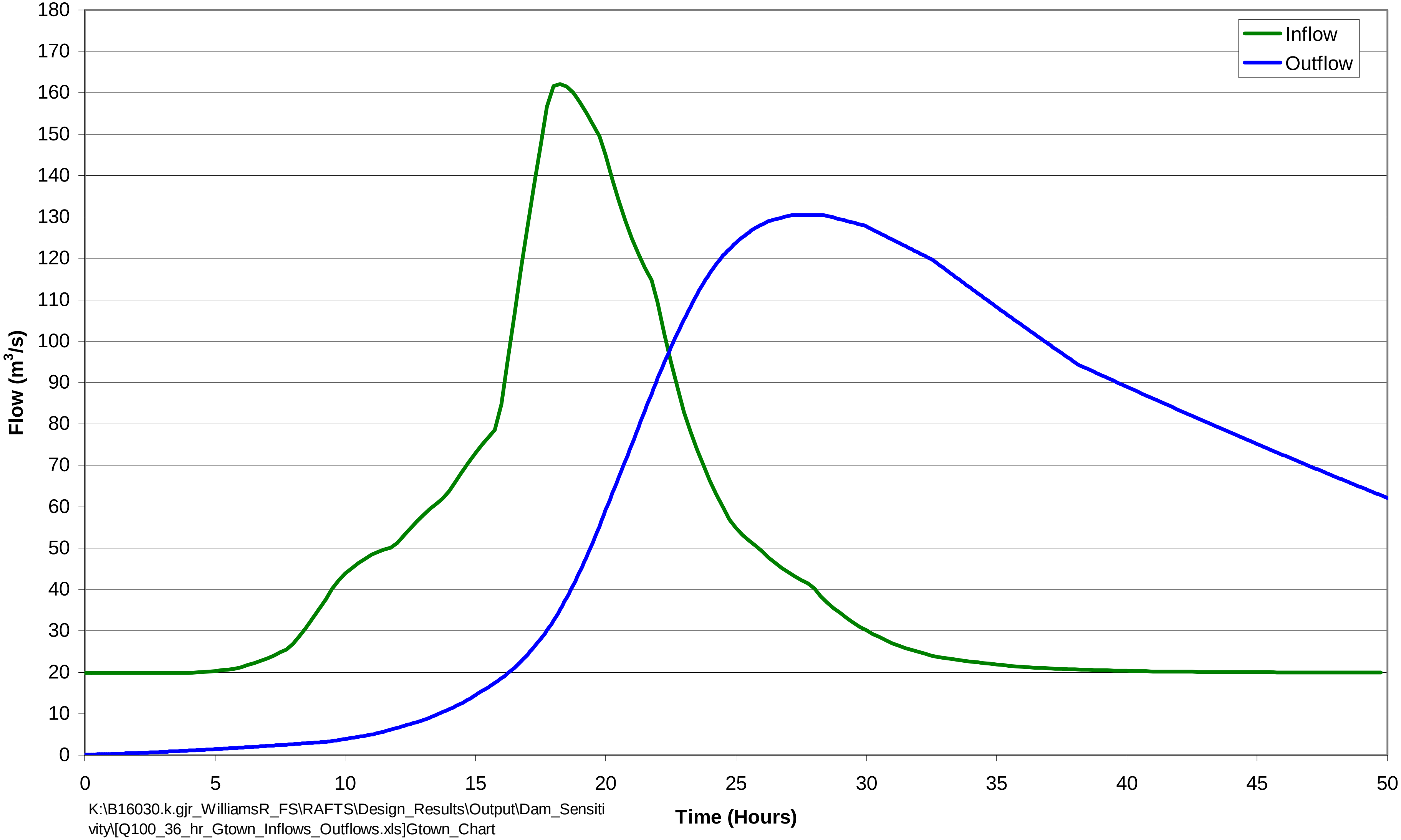


Figure 9-13 Inflows and Outflows Grahamstown Dam – 1% AEP Event

Table 9-4 0.5% AEP Williams River Event, 5% AEP Hunter River Event

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	59.28	59.16	0.00	1.73	0.93	0	53	295	348
Location 2 (Myall Creek) Dungog Bridge	Not Wet	49.88	Not Wet	0.00	2.03	0.00	0	484	0	484
Location 3	56.60	56.61	56.58	2.07	3.46	2.21	428	1,431	1,512	3,371
Location 4	52.82	52.99	52.87	1.52	2.41	1.44	1,241	1,089	1,575	3,905
Location 5 Dungog Road, Bridge	51.67	51.42	51.26	2.48	5.18	3.15	578	1,698	1,562	3,838
Location 6	Not Wet	44.36	44.44	0.00	3.51	2.06	0	1,398	2,639	4,037
Location 7	Not Wet	39.74	39.75	0.00	3.46	1.85	0	1,738	2,405	4,143
Location 8	33.80	34.05	33.96	1.45	1.96	0.83	3,099	719	1,365	5,184
Location 9	Not Wet	30.82	30.65	0.00	3.86	2.03	0	3,052	1,551	4,604
Location 10 Pinebrush Road Bridge	23.16	23.28	Not Wet	1.33	3.64	0.00	2,120	2,018	0	4,138
Location 11	15.39	15.34	15.29	1.79	2.56	1.17	393	1,907	2,530	4,830
Location 12 Clarencetown, Rd Bridge	9.88	10.13	Not Wet	2.69	4.29	0.00	1,700	3,126	0	4,827
Location 13	Not Wet	7.83	7.84	0.00	2.35	1.10	0	3,988	683	4,671
Location 14	6.51	6.53	Not Wet	0.69	2.02	0.00	1,458	3,105	0	4,563
Location 15 Seaham Weir	5.73	5.75	Not Wet	2.23	3.06	0.00	915	3,546	0	4,461
Location 16 Seaham, Road Bridge	5.05	5.17	Not Wet	1.33	3.60	0.00	841	3,739	0	4,580
Location 17	4.67	4.66	4.66	0.18	0.63	0.30	742	714	2,053	3,508
Location 18 Irrawang Swamp	4.48	4.48	4.48	0.08	0.08	0.03	56	88	21	165
Location 19 Raymond Terrace, Road Bridge	4.34	4.34	4.39	0.16	1.09	0.69	236	1,937	2,509	4,682
Location 20 (Hunter R)	5.47	5.39	5.03	0.53	2.18	0.01	922	2,326	8	3,256
Location 21 (Hunter R)	4.72	4.69	Not Wet	0.68	1.86	N/A	1,355	1,916	N/A	3,270
Location 22 (Hunter R)	3.89	3.88	3.83	0.28	1.36	0.40	2,687	3,295	599	6,581

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_1A_Q200_085.xls]Design_Table

Table 9-5 5% AEP Williams River Event, 0.5% AEP Hunter River Event (Event 1B)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	58.94	58.79	0.00	1.69	0.78	0	54	168	222
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.51	Not Wet	0.00	1.49	0.00	0	291	0	291
Location 3	55.70	55.70	55.65	1.25	2.64	1.32	144	969	577	1,691
Location 4	50.59	50.33	50.32	0.25	2.26	1.65	300	774	508	1,582
Location 5 Dungog Road, Bridge	49.88	49.67	49.69	2.66	3.79	1.69	300	1,243	368	1,911
Location 6	Not Wet	43.52	43.59	0.00	3.15	1.29	0	1,136	855	1,991
Location 7	Not Wet	38.75	38.87	0.00	3.02	1.10	0	1,332	697	2,029
Location 8	31.80	32.16	32.10	1.03	1.88	0.55	1,205	710	498	2,413
Location 9	Not Wet	28.20	28.18	0.00	3.22	1.14	0	2,029	250	2,278
Location 10 Pinebrush Road Bridge	20.94	21.25	Not Wet	0.26	3.72	0.00	161	1,932	0	2,093
Location 11	12.64	12.58	12.59	1.21	2.50	0.98	145	1,403	842	2,391
Location 12 Clarencetown, Rd Bridge	7.67	7.68	Not Wet	1.04	3.00	0.00	210	2,180	0	2,390
Location 13	Not Wet	6.34	6.35	0.00	1.38	0.63	0	2,067	279	2,347
Location 14	5.73	5.73	Not Wet	0.29	1.26	0.00	506	1,804	0	2,310
Location 15 Seaham Weir	5.42	5.43	Not Wet	1.13	1.41	0.00	413	1,846	0	2,260
Location 16 Seaham, Road Bridge	5.24	5.31	Not Wet	0.88	1.55	0.00	630	1,666	0	2,295
Location 17	5.15	5.15	5.15	0.10	0.35	0.16	480	423	1,178	2,082
Location 18 Irrawang Swamp	5.09	5.09	5.09	0.04	0.04	0.03	32	53	24	109
Location 19 Raymond Terrace, Road Bridge	5.05	5.05	5.05	0.11	0.87	0.57	239	1,538	2,652	4,429
Location 20 (Hunter R)	6.58	6.30	5.15	1.03	2.82	0.39	2,446	3,392	1,046	6,885
Location 21 (Hunter R)	5.29	5.27	Not Wet	0.97	2.30	N/A	2,382	2,514	N/A	4,896
Location 22 (Hunter R)	4.95	4.94	4.95	0.41	0.94	0.52	6,001	2,741	1,037	9,779

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_090\Design_Table\[WRHR_Des_1B_Q200_090.xls]Design_Table

Table 9-6 1% AEP Williams River Event, 5% AEP Hunter River Event (Event 2A)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	59.17	59.04	0.00	1.71	0.89	0	51	251	302
Location 2 (Myall Creek) Dungog Bridge	Not Wet	49.49	Not Wet	0.00	1.92	0.00	0	427	0	427
Location 3	56.32	56.32	56.30	1.86	3.26	1.99	328	1,303	1,214	2,846
Location 4	52.03	51.78	51.78	0.27	2.36	2.01	533	1,000	869	2,402
Location 5 Dungog Road, Bridge	51.21	50.95	50.84	2.36	4.86	2.71	496	1,594	1,061	3,150
Location 6	Not Wet	44.13	44.21	0.00	3.42	1.86	0	1,329	2,071	3,400
Location 7	Not Wet	39.49	39.52	0.00	3.38	1.63	0	1,642	1,847	3,489
Location 8	33.27	33.53	33.45	1.37	1.95	0.76	2,533	722	1,084	4,340
Location 9	Not Wet	30.25	30.16	0.00	3.72	1.78	0	2,818	1,074	3,892
Location 10 Pinebrush Road Bridge	22.68	22.84	Not Wet	1.06	3.77	0.00	1,412	2,009	0	3,421
Location 11	14.74	14.69	14.64	1.66	2.57	1.07	325	1,749	2,005	4,080
Location 12 Clarencetown, Rd Bridge	9.20	9.38	Not Wet	2.27	4.06	0.00	1,129	2,950	0	4,079
Location 13	Not Wet	7.27	7.28	0.00	2.10	0.96	0	3,392	535	3,926
Location 14	6.08	6.10	Not Wet	0.55	1.87	0.00	1,053	2,757	0	3,810
Location 15 Seaham Weir	5.39	5.38	Not Wet	1.88	2.89	0.00	678	3,064	0	3,741
Location 16 Seaham, Road Bridge	4.83	4.93	Not Wet	0.94	3.25	0.00	537	3,289	0	3,825
Location 17	4.55	4.54	4.54	0.15	0.53	0.24	586	571	1,595	2,752
Location 18 Irrawang Swamp	4.39	4.39	4.39	0.07	0.07	0.02	47	73	13	133
Location 19 Raymond Terrace, Road Bridge	4.26	4.24	4.30	0.14	1.04	0.64	207	1,839	2,245	4,291
Location 20 (Hunter R)	5.45	5.36	5.02	0.53	2.20	0.01	912	2,340	7	3,259
Location 21 (Hunter R)	4.64	4.62	Not Wet	0.71	1.88	N/A	1,355	1,940	N/A	3,295
Location 22 (Hunter R)	3.73	3.73	3.64	0.26	1.35	0.33	2,260	3,158	473	5,890

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_2A_Q100_085.xls]Design_Table

Table 9-7 5% AEP Williams River Event, 1% AEP Hunter River Event (Event 2B)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	58.94	58.79	0.00	1.69	0.79	0	54	169	223
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.51	Not Wet	0.00	1.63	0.00	0	292	0	292
Location 3	55.70	55.70	55.65	1.25	2.64	1.32	144	969	577	1,691
Location 4	50.59	50.33	50.32	0.25	2.26	1.65	300	774	508	1,582
Location 5 Dungog Road, Bridge	49.88	49.67	49.69	2.66	3.79	1.69	300	1,243	368	1,911
Location 6	Not Wet	43.52	43.59	0.00	3.14	1.29	0	1,132	858	1,990
Location 7	Not Wet	38.75	38.87	0.00	3.02	1.10	0	1,332	696	2,027
Location 8	31.80	32.16	32.10	1.03	1.88	0.55	1,205	710	499	2,414
Location 9	Not Wet	28.20	28.18	0.00	3.21	1.14	0	2,028	249	2,278
Location 10 Pinebrush Road Bridge	20.94	21.25	Not Wet	0.25	3.73	0.00	158	1,931	0	2,088
Location 11	12.60	12.54	12.56	1.21	2.55	0.98	145	1,408	838	2,391
Location 12 Clarencetown, Rd Bridge	7.47	7.46	Not Wet	0.90	3.08	0.00	148	2,241	0	2,389
Location 13	Not Wet	5.97	5.98	0.00	1.45	0.63	0	2,051	252	2,304
Location 14	5.32	5.33	Not Wet	0.21	1.45	0.00	326	1,889	0	2,215
Location 15 Seaham Weir	5.00	4.98	Not Wet	1.15	1.86	0.00	348	1,755	0	2,103
Location 16 Seaham, Road Bridge	4.78	4.84	Not Wet	0.59	1.83	0.00	335	1,776	0	2,111
Location 17	4.68	4.68	4.68	0.10	0.45	0.15	419	385	989	1,793
Location 18 Irrawang Swamp	4.59	4.59	4.59	0.04	0.04	0.01	29	46	10	85
Location 19 Raymond Terrace, Road Bridge	4.53	4.53	4.54	0.12	1.04	0.63	220	1,834	2,426	4,480
Location 20 (Hunter R)	6.32	6.07	5.30	0.93	2.78	0.32	2,072	3,255	630	5,957
Location 21 (Hunter R)	4.93	4.89	Not Wet	1.04	2.26	N/A	2,247	2,438	N/A	4,685
Location 22 (Hunter R)	4.32	4.32	4.32	0.36	1.21	0.49	4,265	3,182	839	8,285

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_090\Design_Table\[WRHR_Des_2B_Q100_090.xls]Design_Table

Table 9-8 2% AEP Williams River Event, 10% AEP Hunter River Event (Event 3A)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	59.07	58.95	0.00	1.65	0.84	0	51	213	264
Location 2 (Myall Creek) Dungog Bridge	Not Wet	49.14	Not Wet	0.00	1.74	0.00	0	369	0	369
Location 3	56.05	56.06	56.02	1.60	3.02	1.75	240	1,166	935	2,341
Location 4	51.51	51.25	51.25	0.25	2.29	1.88	427	913	729	2,069
Location 5 Dungog Road, Bridge	50.75	50.50	50.48	2.27	4.46	2.41	440	1,461	820	2,721
Location 6	Not Wet	43.93	44.01	0.00	3.33	1.68	0	1,262	1,613	2,875
Location 7	Not Wet	39.24	39.30	0.00	3.26	1.44	0	1,537	1,407	2,944
Location 8	32.75	33.04	32.97	1.26	1.94	0.70	2,032	717	865	3,614
Location 9	Not Wet	29.61	29.49	0.00	3.55	1.65	0	2,547	723	3,270
Location 10 Pinebrush Road Bridge	22.14	22.34	Not Wet	0.76	3.74	0.00	822	2,020	0	2,841
Location 11	14.12	14.07	14.02	1.50	2.56	0.97	260	1,628	1,546	3,434
Location 12 Clarencetown, Rd Bridge	8.55	8.68	Not Wet	1.85	3.76	0.00	687	2,736	0	3,422
Location 13	Not Wet	6.76	6.77	0.00	1.86	0.83	0	2,883	411	3,294
Location 14	5.70	5.71	Not Wet	0.40	1.76	0.00	692	2,501	0	3,193
Location 15 Seaham Weir	5.10	5.06	Not Wet	1.78	2.60	0.00	566	2,604	0	3,170
Location 16 Seaham, Road Bridge	4.54	4.63	Not Wet	0.68	2.86	0.00	327	2,833	0	3,160
Location 17	4.24	4.24	4.24	0.13	0.48	0.23	480	498	1,377	2,355
Location 18 Irrawang Swamp	4.13	4.13	4.13	0.06	0.07	0.02	39	60	9	109
Location 19 Raymond Terrace, Road Bridge	4.02	4.00	4.06	0.10	0.88	0.51	135	1,543	1,595	3,273
Location 20 (Hunter R)	4.65	4.62	4.62	0.24	1.73	0.01	316	1,620	3	1,939
Location 21 (Hunter R)	4.26	4.27	Not Wet	0.35	1.61	N/A	561	1,621	N/A	2,182
Location 22 (Hunter R)	3.48	3.42	2.72	0.19	1.22	0.27	944	2,716	353	4,012

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_3A_Q050_085.xls]Design_Table

Table 9-9 10% AEP Williams River Event, 2% AEP Hunter River Event (Event 3B)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	58.81	58.62	0.00	1.71	0.72	0	54	127	180
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.11	Not Wet	0.00	1.57	0.00	0	245	0	245
Location 3	55.40	55.42	55.34	1.08	2.51	0.99	94	861	354	1,309
Location 4	50.05	49.81	49.79	0.24	2.24	1.50	217	694	394	1,305
Location 5 Dungog Road, Bridge	49.60	49.20	49.29	2.20	3.22	1.34	220	1,054	214	1,488
Location 6	Not Wet	43.24	43.33	0.00	2.96	1.08	0	1,029	535	1,564
Location 7	Not Wet	38.41	38.59	0.00	2.85	0.89	0	1,192	410	1,601
Location 8	31.38	31.72	31.67	0.88	1.90	0.42	842	703	314	1,858
Location 9	Not Wet	27.16	27.25	0.00	3.05	0.75	0	1,730	70	1,800
Location 10 Pinebrush Road Bridge	20.11	20.49	Not Wet	0.11	3.72	0.00	40	1,681	0	1,722
Location 11	11.50	11.43	11.46	0.82	2.57	0.86	65	1,255	541	1,861
Location 12 Clarencetown, Rd Bridge	6.75	6.67	Not Wet	0.23	2.52	0.00	14	1,836	0	1,850
Location 13	Not Wet	5.29	5.30	0.00	1.22	0.53	0	1,628	170	1,798
Location 14	4.81	4.82	Not Wet	0.07	1.30	0.00	92	1,647	0	1,739
Location 15 Seaham Weir	4.61	4.60	Not Wet	0.85	1.78	0.00	221	1,472	0	1,693
Location 16 Seaham, Road Bridge	4.47	4.51	Not Wet	0.32	1.69	0.00	151	1,589	0	1,740
Location 17	4.39	4.38	4.39	0.08	0.44	0.09	289	378	589	1,256
Location 18 Irrawang Swamp	4.28	4.28	4.28	0.03	0.04	0.01	23	35	6	63
Location 19 Raymond Terrace, Road Bridge	4.17	4.15	4.20	0.13	0.95	0.58	179	1,685	1,944	3,807
Location 20 (Hunter R)	5.82	5.67	5.18	0.70	2.56	0.14	1,372	2,825	167	4,364
Location 21 (Hunter R)	4.64	4.63	Not Wet	0.95	2.12	N/A	1,812	2,242	N/A	4,054
Location 22 (Hunter R)	3.71	3.70	3.62	0.23	1.26	0.29	2,035	2,937	420	5,392

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_090\Design_Table\[WRHR_Des_3B_Q050_090.xls]Design_Table

Table 9-10 5% AEP Williams River Event, 10% AEP Hunter River Event (Event 4A)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	58.95	58.81	0.00	1.64	0.79	0	52	171	224
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.80	Not Wet	0.00	1.62	0.00	0	319	0	319
Location 3	55.84	55.85	55.80	1.44	2.88	1.57	185	1,077	745	2,007
Location 4	51.03	50.77	50.76	0.26	2.33	1.78	363	842	615	1,819
Location 5 Dungog Road, Bridge	50.30	50.07	50.07	3.00	4.09	2.12	380	1,339	582	2,301
Location 6	Not Wet	43.73	43.81	0.00	3.25	1.49	0	1,202	1,218	2,420
Location 7	Not Wet	39.00	39.09	0.00	3.15	1.23	0	1,437	1,025	2,462
Location 8	32.26	32.58	32.51	1.15	1.97	0.63	1,571	720	667	2,958
Location 9	Not Wet	28.90	28.82	0.00	3.38	1.34	0	2,281	430	2,711
Location 10 Pinebrush Road Bridge	21.50	21.77	Not Wet	0.43	3.74	0.00	353	2,027	0	2,380
Location 11	13.27	13.22	13.15	1.28	2.57	0.98	183	1,529	1,069	2,781
Location 12 Clarencetown, Rd Bridge	7.86	7.90	Not Wet	1.25	3.40	0.00	295	2,475	0	2,770
Location 13	Not Wet	6.16	6.17	0.00	1.62	0.72	0	2,359	305	2,665
Location 14	5.23	5.24	Not Wet	0.24	1.61	0.00	361	2,195	0	2,555
Location 15 Seaham Weir	4.68	4.66	Not Wet	1.54	2.38	0.00	409	2,149	0	2,558
Location 16 Seaham, Road Bridge	4.22	4.34	Not Wet	0.39	2.46	0.00	157	2,380	0	2,537
Location 17	4.04	4.04	4.04	0.10	0.46	0.16	332	395	941	1,668
Location 18 Irrawang Swamp	3.97	3.97	3.97	0.06	0.06	0.01	32	49	8	88
Location 19 Raymond Terrace, Road Bridge	3.87	3.86	3.91	0.08	0.79	0.43	96	1,348	1,240	2,683
Location 20 (Hunter R)	4.57	4.54	4.55	0.24	1.73	0.00	303	1,626	0	1,929
Location 21 (Hunter R)	4.07	4.15	Not Wet	0.32	1.62	N/A	465	1,629	N/A	2,094
Location 22 (Hunter R)	3.41	3.32	2.26	0.13	1.13	0.27	403	2,472	348	3,223

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_4A_Q020_085.xls]Design_Table

Table 9-11 10% AEP Williams River Event, 5% AEP Hunter River Event (Event 4B)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	Left Bank	Channel	Right Bank	Left Bank	Channel	Right Bank	Left Bank	Channel	Right Bank	Total
Location 1 (Myall Creek)	Not Wet	58.81	58.62	0.00	1.71	0.72	0	54	127	180
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.10	Not Wet	0.00	1.57	0.00	0	245	0	245
Location 3	55.40	55.42	55.34	1.08	2.51	0.99	94	861	354	1,309
Location 4	50.05	49.81	49.79	0.24	2.24	1.50	217	694	394	1,305
Location 5 Dungog Road, Bridge	49.60	49.20	49.29	2.20	3.22	1.34	220	1,054	214	1,488
Location 6	Not Wet	43.24	43.33	0.00	2.96	1.08	0	1,029	536	1,564
Location 7	Not Wet	38.41	38.59	0.00	2.86	0.89	0	1,194	409	1,603
Location 8	31.38	31.71	31.67	0.88	1.90	0.42	839	703	312	1,854
Location 9	Not Wet	27.16	27.25	0.00	3.05	0.68	0	1,729	63	1,792
Location 10 Pinebrush Road Bridge	20.11	20.48	Not Wet	0.11	3.71	0.00	40	1,681	0	1,722
Location 11	11.50	11.43	11.45	0.81	2.57	0.86	64	1,255	540	1,860
Location 12 Clarencetown, Rd Bridge	6.74	6.66	Not Wet	0.22	2.52	0.00	13	1,835	0	1,848
Location 13	Not Wet	5.24	5.25	0.00	1.23	0.54	0	1,633	169	1,802
Location 14	4.59	4.61	Not Wet	0.05	1.32	0.00	59	1,669	0	1,728
Location 15 Seaham Weir	4.37	4.36	Not Wet	0.91	1.93	0.00	210	1,528	0	1,738
Location 16 Seaham, Road Bridge	4.24	4.29	Not Wet	0.21	1.78	0.00	89	1,672	0	1,761
Location 17	4.18	4.18	4.18	0.07	0.46	0.08	255	389	454	1,098
Location 18 Irrawang Swamp	4.10	4.10	4.10	0.04	0.04	0.01	23	36	6	65
Location 19 Raymond Terrace, Road Bridge	4.00	3.99	4.04	0.10	0.86	0.49	128	1,502	1,520	3,151
Location 20 (Hunter R)	5.36	5.28	5.02	0.52	2.34	0.01	874	2,434	3	3,311
Location 21 (Hunter R)	4.37	4.42	Not Wet	0.81	1.97	N/A	1,374	2,069	N/A	3,443
Location 22 (Hunter R)	3.47	3.42	2.91	0.17	1.21	0.27	977	2,679	354	4,010

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_090\Design_Table\[WRHR_Des_4B_Q020_090.xls]Design_Table

Table 9-12 10% AEP Williams River event, No Hunter Inflows (Event 5)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	58.82	58.63	0.00	1.65	0.72	0	53	130	182
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.38	Not Wet	0.00	1.65	0.00	0	255	0	255
Location 3	55.65	55.65	55.59	1.20	2.60	1.26	133	948	532	1,613
Location 4	50.51	50.25	50.24	0.25	2.36	1.63	285	757	489	1,531
Location 5 Dungog Road, Bridge	49.81	49.60	49.62	2.59	3.73	1.60	286	1,221	335	1,842
Location 6	Not Wet	43.48	43.56	0.00	3.12	1.25	0	1,120	807	1,927
Location 7	Not Wet	38.69	38.82	0.00	2.99	1.06	0	1,308	641	1,949
Location 8	31.68	32.03	31.98	0.98	1.99	0.51	1,098	716	446	2,259
Location 9	Not Wet	27.91	27.90	0.00	3.18	1.06	0	1,943	190	2,133
Location 10 Pinebrush Road Bridge	20.66	20.98	Not Wet	0.19	3.73	0.00	102	1,879	0	1,981
Location 11	12.13	12.07	12.10	1.15	2.60	0.93	118	1,344	703	2,165
Location 12 Clarencetown, Rd Bridge	7.09	7.01	Not Wet	0.50	2.87	0.00	53	2,088	0	2,140
Location 13	Not Wet	5.46	5.47	0.00	1.37	0.59	0	1,854	201	2,055
Location 14	4.70	4.72	Not Wet	0.07	1.43	0.00	95	1,836	0	1,930
Location 15 Seaham Weir	4.24	4.22	Not Wet	1.13	2.14	0.00	245	1,706	0	1,950
Location 16 Seaham, Road Bridge	3.52	3.83	Not Wet	0.13	2.01	0.00	26	1,881	0	1,907
Location 17	2.76	2.74	1.44	0.17	0.70	0.07	357	576	126	1,059
Location 18 Irrawang Swamp	2.33	2.33	2.33	0.14	0.17	0.01	26	42	1	69
Location 19 Raymond Terrace, Road Bridge	0.64	2.03	0.78	0.00	0.80	0.00	0	878	0	878
Location 20 (Hunter R)	1.51	2.00	2.01	0.00	0.02	0.00	0	10	0	10
Location 21 (Hunter R)	1.69	2.00	Not Wet	0.00	0.02	N/A	0	13	N/A	13
Location 22 (Hunter R)	1.92	1.92	0.91	0.00	0.52	0.13	0	793	86	878

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_5_Q010_085.xls]Design_Table

Table 9-13 20% AEP Williams River event, No Hunter Inflows (Event 6)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	58.71	58.48	0.00	1.68	0.66	0	55	95	150
Location 2 (Myall Creek) Dungog Bridge	Not Wet	48.02	Not Wet	0.00	1.67	0.00	0	208	0	208
Location 3	55.40	55.42	55.34	1.08	2.55	0.99	94	860	352	1,307
Location 4	50.05	49.80	49.79	0.24	2.35	1.50	215	706	394	1,315
Location 5 Dungog Road, Bridge	49.60	49.20	49.28	2.24	3.21	1.33	220	1,052	212	1,485
Location 6	Not Wet	43.24	43.33	0.00	2.96	1.08	0	1,029	528	1,557
Location 7	Not Wet	38.35	38.55	0.00	2.85	0.86	0	1,186	378	1,564
Location 8	31.27	31.60	31.56	0.83	2.01	0.39	751	704	274	1,728
Location 9	Not Wet	26.86	26.92	0.00	3.00	0.80	0	1,651	31	1,683
Location 10 Pinebrush Road Bridge	19.79	20.21	Not Wet	0.09	3.72	0.00	22	1,598	0	1,620
Location 11	11.07	10.99	11.02	0.67	2.59	0.80	43	1,203	434	1,679
Location 12 Clarencetown, Rd Bridge	5.93	6.28	Not Wet	0.00	2.28	0.00	0	1,661	0	1,661
Location 13	Not Wet	4.86	4.88	0.00	1.15	0.47	0	1,468	128	1,597
Location 14	4.26	4.29	Not Wet	0.02	1.19	0.00	20	1,464	0	1,484
Location 15 Seaham Weir	3.92	3.90	Not Wet	0.68	1.90	0.00	123	1,363	0	1,486
Location 16 Seaham, Road Bridge	3.44	3.58	Not Wet	0.01	1.61	0.00	1	1,464	0	1,464
Location 17	2.73	2.72	0.79	0.16	0.70	0.08	344	565	66	975
Location 18 Irrawang Swamp	2.04	2.05	2.03	0.15	0.26	0.00	21	35	0	56
Location 19 Raymond Terrace, Road Bridge	0.64	2.01	0.78	0.00	0.76	0.00	0	855	0	855
Location 20 (Hunter R)	1.51	1.97	2.01	0.00	0.02	0.00	0	10	0	10
Location 21 (Hunter R)	1.69	1.97	Not Wet	0.00	0.02	N/A	0	14	N/A	14
Location 22 (Hunter R)	1.90	1.89	0.91	0.00	0.53	0.13	0	790	84	874

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_6_Q005_085.xls]Design_Table

Table 9-14 PMF Williams River Event, PMF Hunter River Event (PMF)

	Flood Level (mAHD)			Velocity (m/s)			Flow (m3/s)			
Location	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Left Bank**	**Channel**	**Right Bank**	**Total**
Location 1 (Myall Creek)	Not Wet	60.29	60.28	0.00	1.80	1.31	0	65	845	910
Location 2 (Myall Creek) Dungog Bridge	52.96	53.10	Not Wet	2.07	2.88	0.00	127	1,209	0	1,336
Location 3	59.29	59.33	59.33	3.63	4.76	4.29	1,946	2,633	6,356	10,936
Location 4	57.20	57.20	57.20	1.88		1.75	6,881	2,603	1,554	11,038
Location 5 Dungog Road, Bridge	54.68	54.92	55.13	2.14	6.47	6.56	1,110	3,136	7,113	11,361
Location 6	46.39	46.46	46.36	0.40	4.17	3.64	112	2,066	10,117	12,295
Location 7	Not Wet	42.23	41.99	0.00	3.73	3.58	0	2,412	10,561	12,973
Location 8	37.58	37.90	38.02	0.84	2.09	2.68	3,832	765	9,383	13,980
Location 9	Not Wet	34.88	34.33	0.00	4.47	4.56	0	4,650	9,347	13,997
Location 10 Pinebrush Road Bridge	27.75	27.80	27.86	1.87	2.87	0.47	8,428	2,049	65	10,542
Location 11	20.86	20.95	20.97	0.88	3.13	1.67	1,112	3,584	8,544	13,239
Location 12 Clarencetown, Rd Bridge	15.15	15.53	15.40	3.36	3.18	1.96	6,356	4445.55	433	11,234
Location 13	12.80	12.83	12.90	1.88	3.82	1.77	1,484	9,280	2,755	13,519
Location 14	11.18	11.14	11.00	1.63	3.10	0.28	7,309	6,780	24	14,113
Location 15 Seaham Weir	10.26	10.32	10.20	2.92	3.23	2.76	4,180	8,463	789	13,432
Location 16 Seaham, Road Bridge	9.70	9.96	9.87	4.67	1.50	1.09	12,393	1,617	425.26	14,435
Location 17	9.59	9.60	9.61	0.36	1.12	0.57	3,428	2,074	8,593	14,095
Location 18 Irrawang Swamp	9.58	9.55	9.58	0.11	0.23	0.13	253	661	367	1,281
Location 19 Raymond Terrace, Road Bridge	9.55	9.63	9.54	0.14	0.50	0.80	804	1,726	9,844	12,373
Location 20 (Hunter R)	10.29	9.80	10.05	2.56	4.29	1.66	11,442	6,781	13,114	31,337
Location 21 (Hunter R)	9.65	9.65	N/A	0.75	1.59	N/A	4,539	1,959	N/A	6,498
Location 22 (Hunter R)	9.53	9.49	9.51	0.79	1.19	0.82	3,327	6,127	27,838	37,291

K:\B16030.k.gjr_WilliamsR_FS\TUFLOW_design\Results\Run_085\Design_Table\[WRHR_Des_PMF_085.xls]Design_Table

10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

The following points summarise the findings for the Williams River Flood Study:

- A two-dimensional (2D) model of the Williams River from 5km above Dungog to the confluence with the Hunter River. This model accurately simulates flooding behaviour of three historical events.
- The Williams River model was linked to an existing model of the Hunter River from Green Rocks to Newcastle Harbour.
- The model has successfully been used to derive a detailed representation of the river and floodplain for the 20%, 10%, 5, 2%, 1%, 0.5% AEP design flood events as well as the probable maximum flood,
- The model consists of a the following model elements:
  - 10m x 10m grid over the upper Williams River
  - 40m x 40m grid over the lower Williams River
  - 40m x 40m grid over the over Hunter River
  - One-dimensional elements, these represent the bank-to-bank areas of the main river and creeks as well as culverts and weirs
- These different sections of the model are dynamically linked
- The inflows to the 2D / 1D hydraulic model were derived from a calibrated hydrological model of the catchment
- The size of flood flows as derived from the hydrological model using Australian Rainfall and Runoff methods and data appear to be larger than flows derived from flood frequency analysis

10.2 Recommendations

The 2D / 1D flood model of the Williams River floodplain should form the basis of all future floodplain risk management investigations for the study area.

11 REFERENCES

AUSTROADS (1994) *Waterway Design A Guide to the Hydraulic Design of Bridges, Culverts and Floodways* AUSTROADS

CameronMcNamara (1985) *New Bridge Over The Williams River at Dungog* 84/5262A

Floodplain Development Manual (2005) *Floodplain Development Manual - The Management of Flood Liable Land* Department of Infrastructure, Planning and Natural Resources

Kinhill Engineers Pty Ltd (1993) *Grahamstown Dam Augmentation Hydrologic and Hydraulic Report* SE2094/BB

Lawson and Treloar (1999) *Design Water Levels in Newcastle Harbour – Joint Probability Study* March 1999

PWD (1994) *Lower Hunter River Flood Study (Green Rocks to Newcastle)* PWD 91077 August 1994

APPENDIX A: TECHNICAL SPECIFICATION

Specification

TECHNICAL BRIEF

TABLE OF CONTENTS

TECHNICAL BRIEF

1. INTRODUCTION

The confluence of the Williams and Hunter Rivers is located at Raymond Terrace approximately 20 km north-west of Newcastle. The Williams River catchment extends some 100 km north of Raymond Terrace to the Barrington Tops. A major tributary of the Williams is the Chichester River. The Chichester dam, original constructed in 1926, exists at the confluence of the Chichester and Wangat Rivers. Both catchments cover an area of approximately 1100 square kilometres. Floods at the junction of the Williams River with the Hunter River are affected by flooding from the Paterson River, which joins the Hunter 15 kilometres upstream. Grahamstown spillway discharges into Irrawang swamp and may impact on flooding at Raymond Terrace.

Port Stephens and Dungog Councils, through a joint Floodplain Management Committee propose to prepare a comprehensive floodplain management strategy for the Williams River from Dungog to Raymond Terrace in accordance with the process outlined in NSW Government's Floodplain Development Manual (2005). The process requires a flood study to be undertaken to define the flood behaviour in the Williams River and at its junction with the Hunter River. The study will produce information on flood levels, velocities and flows for a full range of flood events, under existing catchment and floodplain conditions. The study catchment area is shown in Figure 1.

A previous Flood Study and Mike 11 model (Lawson and Treloar 1994) was undertaken of the Hunter River from Green Rocks (west of Raymond Terrace) to Newcastle. The Paterson River Floodplain Management Study vol 3 (WBM Oceanics for Bewsher Consulting Pty Ltd 2001) updated the Hunter River study and Mike 11 model from Hinton to Green Rocks. This study will incorporate and if necessary convert those outcomes, plus run the 0.5%AEP additional flood event in the Hunter from Green Rocks to Windeyer's Creek, south of Raymond Terrace. The study will also investigate the probability analysis of combined floods in both the Hunter and Williams rivers. Concurrent adjacent studies currently in progress include Pacific Highway Upgrade - F3 to Raymond Terrace Project Hunter River modelling (WBM Oceanics) for Maunsell Australia on behalf of RTA; Upgrading of Lower Hunter Flood Model at Hexham (DHI Water & Environment) for Newcastle Council and upgrade of Maitland (Hunter River) Flood Model to MIKE 11 (Webb McKeown & Associates) for Maitland Council.

Councils have engaged and completed Aerial Photogrammetry of the Study area up to and above the PMF. The data consists of digital spot levels, break lines and contours on a 1:16000 digital orthophoto background. The vertical accuracy is +/- 0.5 metre. Photography to produce a vertical accuracy of +/- 0.25 metre, if required, has been flown but not developed for the study area at 1:8000.

Both Councils are interested in future planning issues, flood warnings, evacuations and access issues, as well as the normal existing flood issues. Councils have adopted the 0.5% AEP flood as the Flood Planning Level for planning purposes and wish to continue this principal on this and future flood studies.

2. OBJECTIVES

The primary objective of the study is to define the riverine flood behaviour in the Williams River from Raymond Terrace to 5 km above Dungog, to incorporate the Hunter River Flood Model at Raymond Terrace and analyse the combined effect of flooding from both rivers and any affect from the Paterson River. The study will produce information on flood flows, velocities, levels and extents for a full range of flood events under existing catchment and floodplain conditions.

This study will also form the basis for a subsequent floodplain management study where detailed assessment of flood mitigation options and floodplain management measures will be undertaken.

The study and model should:

- Provide broad flood information for future development anywhere on the floodplain to PMF as shown on figure 2[#1];
- Provide detailed flood information for roads and rail overtopping[#2].
- Be readily adaptable to provide Councils with the ability to forecast flooding;
- Provide Council with flood data tied to gauging stations.
- Provide the hydraulic categories and undertake provisional hazard mapping.

[#1] Aerial Photogrammetry Survey data is available to +/- 0.5 metre vertical accuracy.
[#2] Aerial Photogrammetry Survey data available to +/- 0.5 metre vertical accuracy. Additional survey may be required as determined during the course of the study using either the 1:8000 Aerial Photogrammetry Survey or land surveying techniques. Hydrosurvey, crossings, and culvert details will be required.

It is expected that hydrologic and hydraulic modelling will be required to satisfy the study objectives.

3. STUDY AREA

The study area comprises the Williams River from the Hunter River at Raymond Terrace to 5 km above Dungog, and the Hunter River from Green Rocks (5km west of Raymond Terrace) to Windeyers Ck (1km south of Raymond Terrace) and the corresponding catchment area. The extent of the study area for which hydraulic analysis is required is shown in Figure 2 and the catchment area is shown in Figure 1.

The extent of the upstream and downstream boundaries for the hydraulic modelling, including tributaries, will be agreed between the Technical Committee and the consultant prior to the commencement of the study following a joint site reconnaissance of the study area.

4. SCOPE OF WORK

The Consultant shall provide all services required to satisfy the objectives of the flood study. The services shall include but not necessarily be limited to the following major tasks.

4.1. Compilation And Review Of Available Information

The Consultant shall compile and review all information that is pertinent to the flood study. A preliminary list of available data and previous reports is provided in Section 6. This list may not be exhaustive. The Consultant shall contact all relevant authorities and other sources for the purposes of data compilation (for example: Council, Water Board, RTA, SRA, Dept Planning, Dept Natural Resources, Bureau of Meteorology, local SES, local newspapers and historical societies).

Site reconnaissance shall be undertaken to obtain an appreciation of all significant factors and works that may affect flood behaviour.

The Consultant shall comment on the adequacy/accuracy of the existing information for the purposes of this study including the current 1:16,000 and the need for the undeveloped (1:8000) Photogrammetric Survey.

4.2. Acquisition of Additional Data

Flood Data
The consultant shall conduct a resident survey to determine historical flood behaviour, obtain flood levels, photographs and other relevant information of past events to assist in setting up the hydraulic model and calibration. This may include preparation and distribution of a suitable questionnaire to residents and interview of residents.

Survey Data
Should acquisition of additional data be necessary, the Consultant shall submit to Council a brief outlining details of the data required, together with a firm quotation for the cost and timing of the work. Following receipt

of written approval by Council, the Consultant shall undertake the additional data collection. The Consultant shall be responsible for engagement and supervision of an approved sub-consultant, where necessary to complete the work.

This work will include:

- Hydrosurvey to obtain river cross-sections;
- Topographic/Photogrammetric survey of hydraulic structures, including roads, bridges, culverts etc

All data obtained should be compatible with Council's GIS and in digital format (Port Stephens GIS is CadCorp while Dungog Council's GIS is Mapinfo). Data should be incorporated in a suitable format where the raw data or processed data can be readily retrieved and used for other purposes by Council.

4.3. Set up of Hydrologic and Hydraulic Models

The Consultant shall set up appropriate computer-based hydrologic and hydraulic models for the purposes of the study. The models shall have the capability to represent all features of the study area, which are likely to have a significant effect on flood behaviour.

The extent of the models shall be sufficient to establish reliable boundary conditions for the defined study area. The design and layout of the models shall also be suitable for testing of development proposals, flood mitigation options and floodplain management measures in a future floodplain management study. The models should be capable of achieving the objectives outlined in section 2.

The modelling is to be undertaken in the most cost effective way taking into consideration not only the cost of the modelling but also the cost of obtaining survey data and the availability of existing data.

The concurrence of the Committee shall be obtained in relation to the selection and design of the models.

Discharge-frequency analysis including Flow and Peak Flood Height rankings and Stage Height frequency curves shall be undertaken at existing river gauging stations. The results shall be checked and compared against the hydrological model results

4.4. Calibration and Verification of Models

The Consultant shall calibrate and verify the hydrologic and hydraulic models using the available data from a minimum of three (3) historical flood events. All relevant factors such as catchment changes shall be considered in the calibration and verification of the model and appropriate modifications made in setting up any digital terrain models (DTM's).

The Consultant shall undertake sensitivity analyses to test the effect of different combinations of model parameters on the calibration for the hydrological and hydraulic models. The principal parameters are those simulating friction, infiltration losses, energy losses and flows at structures. The concurrence of Council shall be obtained in relation to the adoption of model parameter values.

A detailed calibration/verification progress report on the hydrological and hydraulic modelling shall be prepared for Council's review, including an assessment of the accuracy of the hydraulic model *prior* to modelling the design events.

4.5. Modelling of Design Events for Existing Conditions

The Consultant shall undertake modelling for seven (7) design flood events. The design flood events shall be the 0.5% AEP, 1% AEP, 2% AEP, 5% AEP, 10% AEP, 20% AEP together with the Probable Maximum Flood.

The Consultant shall modify the calibrated hydrologic and hydraulic models and any DTM's as necessary to represent accurately the existing catchment and floodplain conditions before modelling the design events.

The Consultant shall establish the inputs to the models of design events, including:

- Design rainfalls according to the current version of Australian Rainfall & Runoff;
- Downstream boundary conditions;
- Catchment rainfall losses;

- Hydraulic roughness of channel beds and floodplain areas;
- Starting water levels for storages at Chichester Dam and Grahamstown Lake.

The modelling of design events shall be undertaken for a range of storm duration's to ensure that the critical events are identified. Sensitivity analyses shall be carried out to assess the effects of changing model parameter values and design inputs on the results, including the sensitivity of the design flood levels to downstream boundaries and starting water levels for Chichester Dam and Grahamstown Lake.

For the purpose of submitting a proposal it shall be assumed that the design flood events shall be modelled with an elevated Hunter River tailwater level to be nominated by the Committee. Two additional model runs shall be undertaken to assess the sensitivity of Hunter River boundary conditions on for the 0.5% AEP Williams River design flood. A total of 12 model runs to be nominated by the Committee shall be undertaken to assess the sensitivity of the starting water levels in Chichester Dam and Grahamstown Lake on Williams River design flood levels.

Where feasible, the modelling results shall be checked using alternative methods. For example, checking of peak flow estimates by an alternative hydrologic method and checking of head losses at bridges and culverts by an alternative hydraulic method. Also, where feasible, the design results shall be checked against flood flow and level estimates obtained from frequency analysis of historical records where available.

The Consultant shall liaise with Council in relation to the definition of existing conditions, the selection of design inputs to the models, and the adoption of design results.

4.6. Greenhouse Induced Sea Level Rise

The Consultant shall assess the potential impact of a greenhouse induced sea level rise for 1 (one) mid range sea level rise scenario on the 0.5% AEP Williams River design flood.

4.7. 4.6 Flood Hazard Assessment

The Consultant shall define the hydraulic categories (flood fringe, flood storage, floodways) for 2 (two) design flood events and the provisional hazard categories (low, high) for 4 (four) design flood events in accordance with ***Appendix L*** of the Floodplain Development Manual (2005). The Committee shall nominate the design flood events.

The Consultant shall prepare hydraulic and flood hazard mapping for the nominated design flood events.

4.8. Public Participation and Community Consultation

Community Consultation Program

An effective community consultation program is necessary to identify local flooding concerns and to collect information on flooding and flood behaviour.

The Consultant shall prepare a preliminary community consultation program for the inception meeting of the floodplain risk management committee that demonstrates how they intend to meet these goals. The program shall run concurrently with all stages of the study and shall include:

- Brochures and questionnaires to advertise the study and collect input from residents;
- Public notices in local newspapers to seek public participation;
- Community consultation to obtain both input and feedback from the public;
- Direct contact with local community groups to promote flood awareness and encourage community involvement in the study;
- A presentation on the draft Flood Study to a meeting of each of the Dungog and Port Stephens Full Council.

Public Exhibition of Draft Reports

The Consultant shall prepare an appropriately worded press release announcing the proposed public exhibition of the draft flood study report, explaining the objectives and purpose of the Flood Study. Council will place the advertisement in local newspapers and the Consultant's responsibility for the public exhibition will be to:

- Prepare suitable material for the exhibition;
- Prepare information for council's web site in collaboration with council;
- Undertake four information sessions/public meeting/workshops at Raymond Terrace, Seaham, Clarencetown and Dungog;
- Collate and assess comments and responses;
- Report the outcomes of the exhibition.

4.9 Meetings and Progress Reporting

The Consultant shall attend meetings with Technical Committee representatives and the Floodplain Risk Management Committee to discuss the progress reports and details on progress of the study. It is envisaged that such meetings will be required every 4 to 6 weeks throughout the study period. The Consultant shall allow for the costs of 8 meetings in the fee for the study.

The Consultant shall provide a work program and timetable of major tasks for completion of the study in a form suitable for updating to show the status of the technical work, timing and expenditure during the course of the study.

Monthly progress reports shall be submitted to Council outlining progress on the technical work, together with an updated work program. Any issues that may affect the timely and efficient completion of the study shall be identified in the progress reports.

Sufficient information shall be provided to enable Council to check that progress on the study is acceptable and to decide on the future direction of the study. The progress reports may include draft versions of the relevant sections of the final report, where appropriate.

4.10 Printing of Reports

Four hard and electronic (CD/DVD) copies of the 'preliminary draft report' shall be submitted to Council for review. After review of the preliminary draft report by the Floodplain Management Technical Sub-Committee, the Consultant shall undertake any additional work necessary to achieve the 'Committee draft' for approval or amendment by the Floodplain Management Committee. Ten hard and four electronic (CD/DVD) copies of the Committee draft report are to be provided. After review of the draft report by the Committee, the consultant shall undertake any additional work necessary to achieve the 'final draft' to be placed on public exhibition. Ten hard and four electronic (CD/DVD) copies of the exhibition report will be required.

After the public exhibition, the Consultant shall undertake any additional work deemed necessary by the Committee to achieve the final report.

Twenty hard copies of the final report will be required as well as five electronic copies on CD/DVD. Five electronic copies on CD/DVD shall be provided containing allsurvey data including DTM models, all modelling data files, modelling result files, survey data files and plans. The CD shall be structured and include documentation showing a clear description of the nature of and relationship between the data files. All graphical and mapping information generated for the study shall be provided in 'Mapinfo' and 'Cadcorp' format suitable for importing to both Dungog and Port Stephens GIS systems.

Printing of the final report shall not commence without the written direction of Council.

The cost of all work associated with preparing the approved draft exhibition, final reports, CD's and printing of the reports shall be included in the consultant's fee estimate.

4.11 Presentation of Results/Reporting

The methodology and findings of the study shall be presented in sufficient detail to enable checking of the validity of the conclusions.

5. THE REPORT

On completion of the study, the Consultant shall present a Final report. Whilst the format is not rigid, the report shall generally incorporate the following:

Foreword - Explain the function of a flood study in the series of activities associated with implementation of the NSW Government's Flood Prone Land Policy.

Summary - Outline the aims, methodology and findings of the study.

Introduction - Set the scene for the reader regarding the nature of the study, the need for it and the elements comprising the study.

Background - Detail the parties' involved, previous studies and any databases.

Data Collection - Provide a description of the data collected, including topographic and hydrographic survey, photogrammetry, digital terrain models (DTM's) and historical flood information etc.

Historical Flood behaviour and Flooding Mechanisms – Provide a description of the observed flood events, the nature of flooding and all flooding mechanisms affecting the study area including the Hunter River flooding in the lower reaches of the study area. This will include the effects of Chichester Dam and Grahamstown storage and spillway.

Hydrology and Flood Frequency Analysis - Include a review of available techniques and justifications for adoption of the selected methodology. All the databases used or generated should be summarised or referenced in this section and detailed in an appendix or compendium of data, as appropriate. Calibration and sensitivity analysis should be discussed.

Hydraulics - Include discussion of available techniques and justification for adoption of the selected methodology. Shortcomings, the expected order of accuracy, and any assumptions necessarily associated with selected modelling procedures should be discussed. Calibration and sensitivity analysis should be discussed.

Modelling - The modelling procedure and results should be discussed in detail. All relevant information and data associated with running the model should be referenced in this section and detailed in the compendium of data or appendix.

Chichester Dam – Discuss the impact of the dam on the design flood events.

Greenhouse Assessment – Discuss the implication of a sea level rise on the design flood levels including the landward extent of these impacts.

Description of Flood Behaviour & Hazards – Discuss the flood behaviour based on the results of the modelling and how the hazards and flood behaviour change over the full range of flooding events.

Hydraulic Categorisation and Provisional Hazard Mapping – To be undertaken generally in accordance with Appendix L of the Manual. Discuss the methodology used in undertaking the hydraulic categorisation and hazard mapping.

Presentation of Results and Findings – The Report shall include the following information for the(0.5% AEP, 1% AEP, 2% AEP, 5% AEP, 10% AEP, 20% AEP and PMF) design flood events and the calibration and verification flood events:

- Animated representation of modelled flood events using dot AVI files with suitable resolution
- Flood profiles along all watercourses and tributaries
- Flood polygons plans of the flood surface and the extent of inundation for each design event in gradations of 0.1 metres
- Plans showing flow distribution and velocities
- Flood levels, velocities and flows in tabular form

- Isohyetal diagrams and pluviometer records for a representative calibration event and a representative design event
- Results of the sensitivity analyses of hydrologic and hydraulic parameters
- Drawings similar to Fig B2 in the Manual showing a typical dwelling in relation to various flood events at different locations
- The hydraulic categorisation and flood hazard mapping for 0.5%, 1%, 5% and AEP design flood events shown on separate plans of suitable scale
- DTM of surveys used for data input to models
- Stage Height frequency curves at existing river gauging stations.

All mapping and plans noted above shall also be available in digital GIS format that is suitable for import to CadCorp and Mapinfo.

References - As appropriate

Appendices – As appropriate

6. AVAILABLE INFORMATION

Council is aware of the following information that is available for the purposes of the study. However Council has not compiled or reviewed this information. It shall be the responsibility of the Consultant to undertake this task, including any additional information that may be relevant.

6.1 Data

Rainfall and stream gauging data, Tidal records, and Historical flooding data (Manly Hydraulics Laboratory, and Department Natural Resources).

1:25,000 topographic maps.

Councils have engaged and completed Aerial Photogrammetry of the Study area up to and above the PMF. The data consists of digital spot levels, break lines and contours on a 1:16000 digital orthophoto background. The vertical accuracy is +/- 0.5 metre.

1:8000 Aerial Photography has been flown for the same area but not developed.

Hydrosurvey will be the responsibility of the consultant.

Cadastral data will be provided in digital format from Council's GIS software (Port Stephens – Cadcorp, Dungog – Map Info)

6.2 Reports and Investigations

- Lower Hunter River Flood Study (Green Rocks to Newcastle) (Lawson & Treloar 1994)
- Grahamstown Dam Augmentation, Hydrologic and Hydraulic Report (Kinhill Engineers 1993)
- Grahamstown Stage 2 Augmentation, Hold Point 2 Report Irrawang Swamp (GHD 2000)
- Fourth major source of water supply environmental impact report (HDWB 1995)
- Investigation of the persistence in climate for source modelling of Hunter Water Corporations water supply system (Urban Water Cycle Solutions 2001)
- Major Source Review (Hunter District Water Board 1990)
- Drought Report 1990-92: vol 2 Appendix A meteorological climatic situation, B rainfall characteristics and C source behaviour (Hunter Water Corporation 1992)
- Paterson River Flood Study (WBM 1997) and Floodplain Management Study and Plan (including extension Flood Study) (Bewsher Consulting 2001)
- Pacific Highway Upgrade - F3 to Raymond Terrace Project Hunter River modelling (WBM Oceanics) for Maunsell Australia on behalf of RTA (in progress)

- Upgrading of Lower Hunter Flood Model at Hexham (DHI Water & Environment) for Newcastle Council (in progress)
- Upgrade of Maitland (Hunter River) Flood Model to MIKE 11 (Webb McKeown & Associates) for Maitland Council (in progress)

7. ADMINISTRATION OF THE STUDY

7.1 Council's Authorised Representative

The study will be administered by Port Stephens Council. Personnel authorised to issue instructions in regard to this study are:

Mr Wal Mills (Strategic Engineer)

Council may request the advice and support of the Department of Infrastructure, Planning and Natural Resources in any aspect of the study.

7.2 Consultant's Project Manager

The Consultant shall nominate a Project Manager who will be responsible for day-to-day liaison with Council's authorised representative. No change of personnel for this role will be permitted without the approval of Council.

7.3 Progress Payments

The study will be carried out on a time and expense basis to an approved Upper Limiting Fee. Payments will be based upon receipt of an itemised monthly account together with a progress report, which outlines the work undertaken. The approved Upper Limiting Fee is not to be exceeded without the formal approval of Council.

In the event of any circumstance which may result in over expenditure being likely, the Consultant shall immediately notify Council in writing. If the Consultant considers that at any time, the scope of work under this brief has been varied, the Consultant shall advise Council in writing of the additional cost associated with such variations, at the time they arise.

7.4 Ownership and Copyright

At the completion of the project the Consultant is required to hand-over all model data files, survey data files and provide details of the hardware and software requirements to run the models.

Ownership of the computer data files and copyright of the study report shall rest with Port Stephens and Dungog Council and the Department of Natural Resources.

8. DURATION OF STUDY

It is expected that the final draft report for public exhibition will be completed within a period of twelve months. This duration should include five weeks for review of the draft report by the technical committee, six weeks for public exhibition of the report, six weeks for consideration of the Consultant's recommendation following receipt of public submissions and a further six weeks for Council adoption. The final report shall be provided within four weeks of Council's written approval of printing.

9. QUALITY ASSURANCE

A Quality Assurance system shall be maintained throughout the duration of the project.

The full version of the Consultant's "Plan for the Work" shall be submitted within four weeks from the date of the letter of engagement. Documentary evidence of the quality control measures used to ensure that the main activities in the Plan are satisfactorily completed shall be required from time to time.

10. INSURANCE

The Consultant is responsible for taking out insurance giving cover to their firm, their employees and any agent engaged by the Consultant.

Professional Indemnity and Public Liability shall have a minimum cover of $2 million and $10 million respectively, for each and every agent for the currency of the commission.

The Consultant shall be expected to produce documentary evidence of the insurance policies.

11. OCCUPATIONAL HEALTH AND SAFETY (OH&S)

All works carried out shall be done in accordance with the requirements under the "*Occupational Health & Safety Act 2000*" and associated legislation, Codes of Practice, Australian Standards and relevant Council policies.

The Consultant will be required to demonstrate to Council that they have a suitable OH&S Policy in place and that their staff, involved in the study, are adequately trained and responsible in relation to the requirements of the Act.

12. CONDITIONS OF ENGAGEMENT

The study shall be carried out in accordance with the "*Consultancy Agreement*".

13. ACCEPTANCE

A signed letter of acceptance is mandatory before any work can commence on the study.

14. SELECTION OF CONSULTANTS

The Consultants' proposals for the study will be assessed using the Value Selection Process developed by the Association of Consulting Engineers Australia (ACEA) so that both price and non-price attributes can be taken into account. The method assesses ability and merit to provide a clear indication of the most appropriate Consultant by balancing the 'value for money' constraints against the required standards and scope of the work.

The factors/attributes that will be used to assess the proposals are

- Relevant Experience on similar projects
- Technical Skills of Personnel working on project
- Project resources
- Methodology
- Capacity to undertake the project
- Fee

15. ADDITIONAL INFORMATION TO BE SUBMITTED IN PROPOSAL

All consultants are familiar to the Technical Assessment Committee and proposals for the study shall be restricted to the following information:

15.1 Methodology

- An outline of the proposed study methodology
- Details of the proposed hydrologic and hydraulic models with reasons for their use
- Details of the additional data collection proposed to be undertaken
- Outline how it is proposed to undertake a community consultation/involvement program
- Examples of results presentation for input to Council's GIS systems.

15.2 Staffing (refer Schedule 7)

- Structure of the proposed study team, together with roles and responsibilities of team members. No variation will be permitted in the defined structure, roles and responsibilities of the study team without prior approval of Council
- Curriculum Vitae of team members only, including details of experience in similar projects and experience with the proposed models
- Details of sub-consultants to be used

15.3 Quality Assurance (refer Schedule 12)

Council will require details of the Consultant's Quality Assurance system and how it will be applied to the study.

15.4 Insurance/Occupational Health and Safety (OH&S) (Refer Schedule 13)

Council will require documents satisfying the Consultant's insurance requirements and OH&S commitment.

15.5 Fees (Refer Schedule 2)

- The study will be carried out on a time and expense basis to an approved Upper Limiting Fee
- The Consultant's fee proposal should be submitted in accordance with the itemised phases given in the Table of Fees
- Hourly charge-out rates for team members shall also apply for any additional work
- Breakdown of the Upper Limiting Fee to show:
  1. Professional fees and the number of hours to be worked by each team member for each major task
  2. Disbursements
  3. Sub-consultant fees
     - Estimate for collection of additional data
     - Estimate for additional work not included in this brief

15.6 Program (Refer Schedule 6)

A program showing the timing, duration and completion dates of major tasks of the study, together with estimated monthly expenditures.

Figure 1 – Williams River Catchment

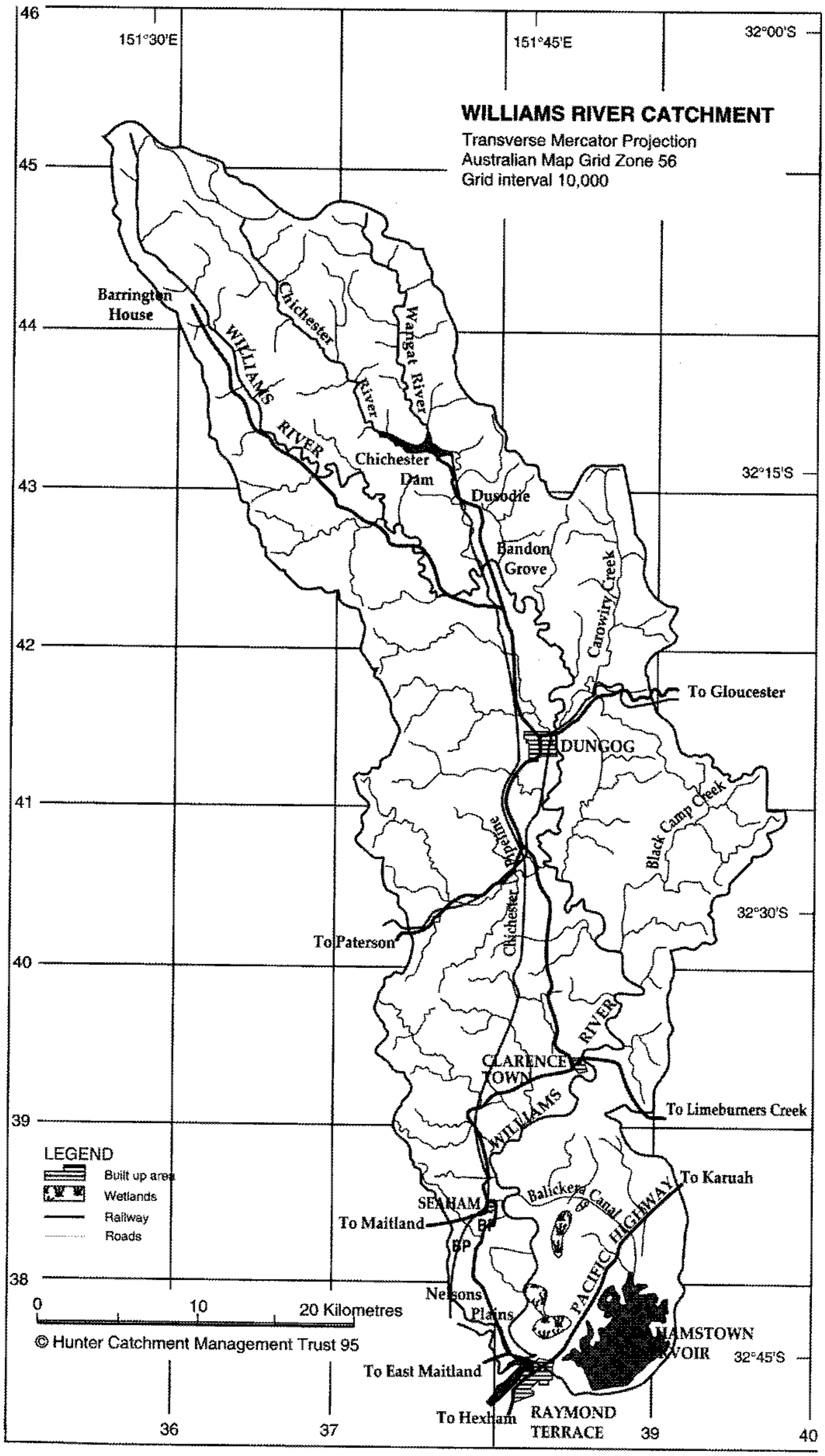
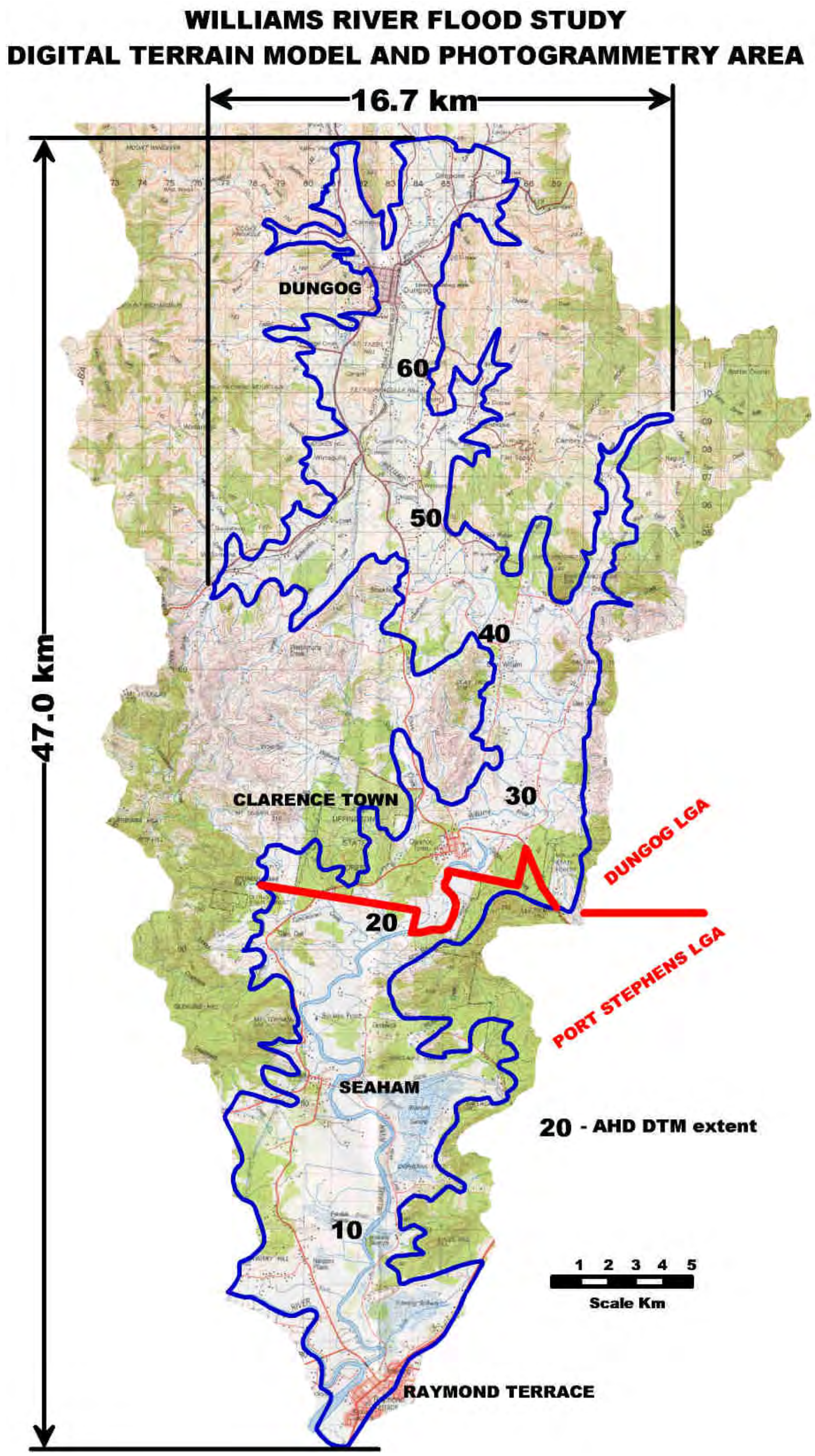
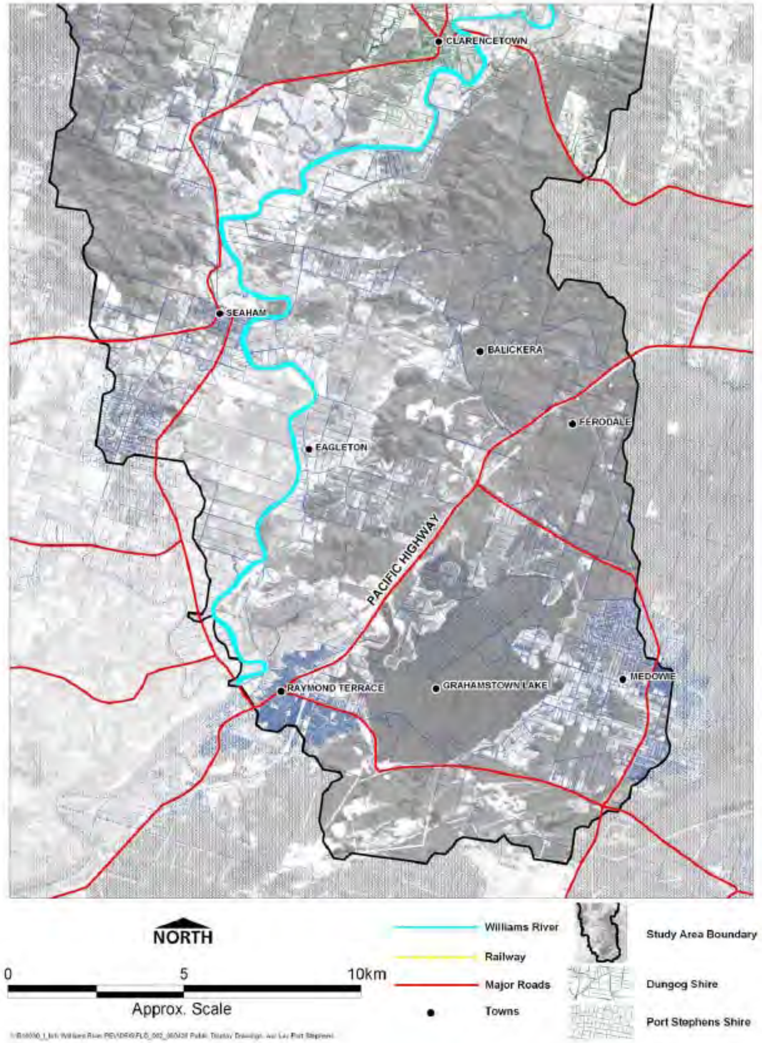


Figure 2 – Williams River Flood Model Area



APPENDIX B: RESIDENT SURVEY QUESTIONNAIRE



Williams River Flood Study Questionnaire

Port Stephens COUNCIL ...a community partnership

WBM

Port Stephens Council and Dungog Shire Council have engaged WBM Oceanics Australia to carry out a Flood Study for the Williams River catchment and floodplain.

This is the first of a three-step process towards best practice management of flood-liable land as outlined in the NSW Government Floodplain Development Manual (2005):

1. Stage 1 - The *Flood Study* determines the nature and extent of the flood problem;
2. Stage 2 - The *Floodplain Risk Management Assessment* evaluates management options for the floodplain in respect of both existing and proposed developments; and
3. The final *Floodplain Risk Management Plan* outlines the preferred management strategy for exhibition and public review, before implementation of the Plan.

We need your help! Please tell us about any floods that you have experienced in the past, or any recent changes (i.e. last 10 years) to the land, which you think may affect flooding in the future.

We would appreciate if you could take the time to answer the following questions and tell us about any additional information you think is relevant to this study. Please use the map on the back of this questionnaire to draw the approximate location of your residence as well as any flood marks you may be aware of. We would also appreciate if you could permit us to make copies of any photos or videos related to flooding in the Williams River catchment and floodplain that may be in your possession.

All information will be kept confidential and will only be used for the purposes of the flood studies listed above.

Your Details

Name:______________________________.

Property Address (on floodplain):________________________.

Lot and DP (if available):_________________________.

Telephone:_____________________________.

A. How many years have you lived in the Williams River area ?

❑ 0-5 years ❑ 5-10 years ❑ 10-20 years ❑ more than 20 years

B. To help the State Emergency Services in developing evacuation plans, could you please provide us with the following demographic information ?

Do you own or rent the house you reside in ? ❑ Owned ❑ Rented

How many people reside in the house? __________________________

Please indicate how many people fall into each age bracket: ❑ 0-15 ❑ 16-25 ❑ 26-50 ❑ Over 50

BMT WBM

C. Do you recall any significant floods in your area ? If so, please provide dates (even if it is just the year) from the most severe event to the least severe and a small description (please see second sheet).

D. For those floods listed in Question C, do you have any records of the height the flood waters reached ? For example, you may have made a mark on a shed or fence post. If yes, please provide a short description of the location:

❑ Yes ____________________
❑ No

E. Have you noticed any changes to the floodplain or river in recent years that may affect flooding ? This could include raising roads, construction of levees, general filling etc. Please indicate such changes on the map provided overleaf. Also, please indicate in words how flooding was / may be altered by this change.

____

____

F. In your opinion, what causes flooding in your area, and are there any features/structures that make flooding worse ?

____

G. If you/were the owner of a business that was affected by flooding in the Williams River catchment, would you please provide information on the following items (the information you give will be confidential and will only be compiled into statistic data for the flood studies):

Type of building: ____________________

Building material: ____________________

Number of car parks provided and location: ____________________

Damages in previous flood events: ____________________

Loss of business due to flooding, turnover loss: ____________________

Closure time during flood: ____________________

Clean-up time: ____________________

Please use the reply paid envelope to return your completed questionnaire before 2nd June, 2006.

As part of the study, the consultants may want to clarify the information provided by you, or may want to survey any flood marks that you have. A member of the study team will contact you if further information is required.

For more information regarding this study, please contact WBM (flooding consultants) on Free Call 1800 613 133.

Thank you for the time and effort involved in this response,

Greg Rogencamp
WBM
Williams River Flood Study Manager

Most severe flood experienced (year and approximate month):____________________

Parts of the house / property that flooded (depth): ____________________

Damage in the house (including loss of electricity, phone, water) and garden and approximate cost:
____

Did access roads get flooded ? ❑ Yes ❑ No
If yes, which ones (duration and depth) ? ____________________

Did you require evacuation? ❑ Yes ❑ No

What were the flood characteristics (duration, flow velocities, flow directions)? ____________________

Did you get any sort of flood warning ? ❑ Yes ❑ No
If yes, when and by what means? ____________________

What emotions did you feel during the flooding experience? ❑ Panic ❑ Anxiety ❑ Frustration
❑ Inconvenience ❑ Excitement ❑ None ❑ Other: ____________________

Additional personal comment/opinion about flooding in the Williams River catchment, either on the flood aspects or the social consequences on yourself, your family and the community, and what could be done to reduce the mental and physical stress related to flooding experiences ?
____

Second largest flood experienced (year and approximate month):____________________

Parts of the house / property that flooded (depth): ____________________

Damage in the house (including loss of electricity, phone, water) and garden and approximate cost:
____

Did access roads get flooded ? ❑ Yes ❑ No
If yes, which ones (duration and depth) ? ____________________

Did you require evacuation? ❑ Yes ❑ No

What were the flood characteristics (duration, flow velocities, flow directions) ? ____________________

Did you get any sort of flood warning ? ❑ Yes ❑ No
If yes, when and by what means ? ____________________

What emotions did you feel during the flooding experience ? ❑ Panic ❑ Anxiety ❑ Frustration
❑ Inconvenience ❑ Excitement ❑ None ❑ Other: ____________________

Additional personal comment/opinion about flooding in the Williams River catchment, either on the flood aspects or the social consequences on yourself, your family and the community, and what could be done to reduce the mental and physical stress related to flooding experiences ?
____

APPENDIX C: RESIDENT SURVEY RESULTS

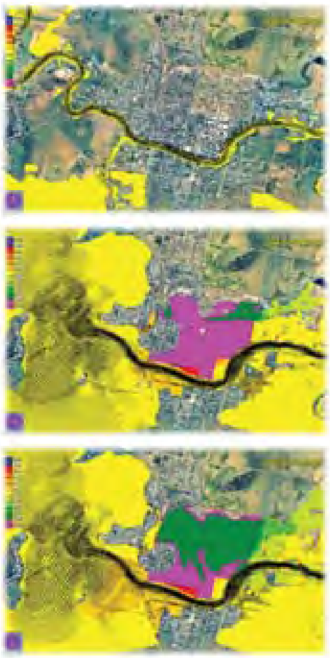
Respondent	Flood Marks	Flood Data 1990	Flood Data 2000	Flood Data 2001	Flood Data 1955	Flood Data 1974	WBM Survey Number
J Aspinal	Yes	No	No	No	No	No	1
John Fiarragher	No	No	No	No	No	No	2
Ken Mitchell	Yes	No	No	No	No	No	4
Brian Hazell	Yes	No	No	No	No	No	20
R &L Palmer	Yes	No	Yes	No	No	No	10
Trevor Foot	Yes	No	No	No	No	No	12
Max Maddock	Yes	No	No	No	No	No	16
Harold Lance Kennedy	Yes	No	Yes	Yes	No	No	18
Ralph Campbell	Yes	No	No	No	Yes	No	23
Jennifer Musicka	Yes	Yes	No	No	No	No	19
Kevin McDonald	Yes	No	No	No	No	No	24
Ross & mary Duncan	Yes	No	No	No	No	No	26
Peter & Rosalee Clark	Yes	No	Yes	Yes	No	No	29
Brian Lonsdale	Yes	No	No	No	No	No	31
Paul Hughes	Yes	No	No	No	No	No	32
J.M. Gleeson	Yes	No	No	No	No	No	34
Neville & Irene James	Yes	No	No	No	No	No	36
Len Robert	Yes	No	No	No	No	No	37
Virginia Carol Anderson	Yes	Yes	No	No	No	No	25
Chris & Brenda Low	Yes	No	No	No	No	No	3
Shortland Family Practice	Yes	No	No	No	No	No	38
John & Christine Green	Yes	No	No	No	No	No	40
Mr L T Dillon	No	No	No	No	No	No	14
Mr L T & Estate Late M A Dillon	Yes	No	No	No	No	No	14
Ms K A Nickerson	Yes	Yes	No	No	No	No	28
Ms J & Mr W A Coe& Ms N M Hepplewhite	Yes	No	No	No	No	No	27
Mr D J & Mrs T M Cowan	Yes	Yes	No	No	No	No	5
Mr G H & Mrs A Robards	Yes	No	No	No	No	No	6
Mr R & Mrs F A Griffiths	Yes	Yes	No	No	No	No	30
Mr W W & Mrs W A Brown	Yes	Yes	No	No	No	No	7
Mr J S Kun	Yes	Yes	No	No	No	No	33
Mr W A & Mrs B A McKinnon-Matthews	Yes	No	Yes	No	No	No	13
Mr J & Mrs L Storm	Yes	Yes	No	No	No	No	15
Mr C & Mrs J M Creal	Yes	No	No	Yes	No	No	17
Ms J H Dircks	Yes	Yes	Yes	Yes	Yes	Yes	39
Mr D P & Mrs M A James	Yes	No	Yes	No	No	No	21
Mrs C J Jones	Yes	No	No	No	No	No	8
Mr F G Prentice & Ms R C Meincke	Yes	No	No	No	No	No	35
Mr I G & Mrs J L Crawford	Yes	No	No	No	No	No	9
Mr T H Boorer	Yes	No	No	No	No	No	22
Mr N & Mrs D I Thompson	Yes	Yes	No	No	No	No	11

G:\Admin\B16030.g.pev Williams\Questionnaire\[Summary_Responses.xls]Summary_Table

APPENDIX D: DESCRIPTION OF TUFLOW

Flood and Tide Simulation Software

TUFLOW



Computer animation stills showing the effect of a proposed levee for Casino, Richmond River, NSW. The three stills are at the start, before the peak and at the peak of the flood.

Yellow indicates <5cm change in flood level, red/orange shades indicate an increase and green shades a decrease. Pink areas were previously flooded, but are now flood-free if the levee is built.

TUFLOW
The Basics

Floods and storm tides cause extensive damage, stress, loss of life-and-limb and dislocate communities. To understand and manage these risks requires modelling software that takes on the challenge of accurately predicting inundation patterns from floods and storm tides.

TUFLOW meets this challenge effectively, reliably and within an economical cost structure.

TUFLOW models: flooding in major rivers through to complex overland and piped urban flows; estuarine and coastal tide hydraulics; and storm tide inundation.

TUFLOW is one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software. It simulates the complex hydrodynamics of floods and tides using the full 1D St Venant equations and full 2D free-surface shallow water equations.

Capabilities – An Overview

- robust and rapid wetting and drying
- superior 1D and 2D linking options
- multiple 2D domains of any orientations and cell sizes
- 2D representation of hydraulic structures
- automatic flow regime switching over levees and embankments
- 1D and 2D supercritical flow and weir flow
- flexible and effective data management
- constructs models from layers of GIS data
- quality control outputs.

A proven and reliable solution for modelling

- flooding in major rivers
- complex overland and piped urban flows
- storm tide inundation of coastal plains
- estuarine and coastal tidal hydraulics.

Links to other software

- Dynamic links to the ISIS and XP-SWMM 1D schemes offers unparalleled performance in 1D/2D hydraulic modelling.
- Construct models using the SMS or XP-SWMM Graphical User Interfaces or alternatively use a GIS for data management, manipulation and presentation.

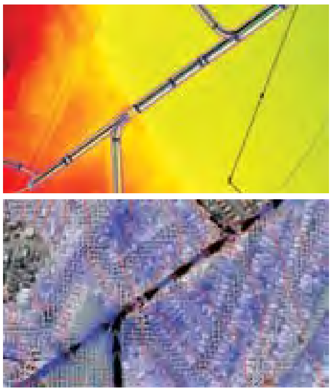
GIS presentation of TUFLOW results, Goulburn River, Victoria

TUFLOW BMT WBM

Web **www.tuflow.com**

TUFLOW

Flood and Tide Simulation Software



The top image shows GIS layers for a 1D domain that is carved through a 2D domain.

The lower image shows the 1D and 2D velocities as arrows, depth of inundation as blue shades and water levels as red contours.

Dynamic Linking

The dynamic linking capability between 2D and 1D domains is a major strength of TUFLOW. With the adaptation of TUFLOW to floodplain modelling, more flexible and complex linking was developed.

The advanced linking functions have been extensively applied to a wide range of models varying from major river systems to fine-scale urban flood models to coastal storm tide inundation.

Linking of 1D and 2D Domains

1D and 2D domains can be linked anywhere along the perimeter of the active 2D cells. The links can be at any orientation to the 2D grid, start completely dry, and wet and dry during a simulation.

1D Elements Inside 2D Domains

Internal links are used to model flowpaths within or under the 2D domains that are better represented using a 1D solution. This may be a culvert through an embankment, or a complex underground pipe network.

Multiple 2D Domains (Optional Module)

The study area can be divided into any number of 2D domains, with each domain having its own orientation and cell resolution. These domains can be linked to form one overall model.

1D Waterway in a 2D Floodplain

More advanced linking allows the modelling of a waterway in 1D and overbank areas in 2D. This is useful where the drain, creek or river is too coarsely represented by the 2D resolution and is better represented by 1D cross-sections and structures.

Background

TUFLOW was originally the product of a joint project between WBM Pty Ltd and The University of Queensland in 1989/1990 to develop a 2D modelling system with dynamic links to a 1D system. TUFLOW stood for Two-dimensional Unsteady FLOW.

The project was successful and the software widely applied by WBM within the Australian industry through the 1990s. Since 1997 there have been considerable improvements to the software, especially in the areas of flood modelling and GIS linkages.

Solution Scheme

TUFLOW's 2D solution is based on the Stelling finite difference, alternating direction implicit (ADI) scheme that solves the full 2D free surface shallow water flow equations over a regular grid. The 1D scheme is a finite difference, second-order, Runge-Kutta solution.

The schemes have been improved to handle upstream controlled flow regimes (eg. supercritical and weir flow), bridge decks, box culverts, robust wetting and drying, and other key features.

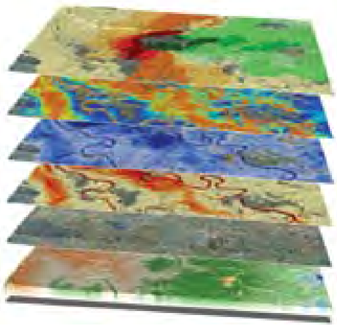
1990 flood, Throsby Creek, Newcastle, Australia. Reproduced by TUFLOW.

Results Presentation

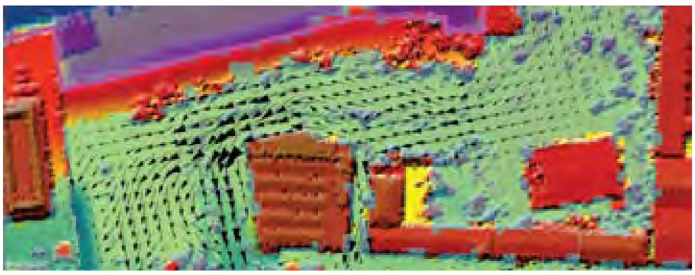
TUFLOW outputs SMS and GIS formatted files containing a variety of data, such as water levels, velocity vectors, depths, unit flow vectors, energy level, flood hazard categories, Froude number and other data types.

The user can readily:

- Display DTMs, aerial photos and other GIS data in the background.
- Create computer animations showing the rise and fall of the flood using SMS, WaterRIDE or XP-SWMM.
- Interactively select and graph time-series results from the 1D and 2D domains.
- Produce high quality maps for reports, plans and public displays using GIS.



Flood risk mapping from TUFLOW results. Herbert River, Queensland, Australia



TUFLOW modelling of storm tide inundation from a hypothetical breach along the River Thames, London (courtesy Halcrow, HR Wallingford, UK Environment Agency)

Web:
www.tuflow.com

Information and sales:
sales@tuflow.com

APPENDIX E: MODELLING LOGS

Run ID	Modeller	Changes
		Runs Pre 011 are model development
WR_cal_2001_011.tcf	PAR	2d timestep 20sec, 1d 10sec
WR_cal_2001_011_alpha.tcf	PAR	Mannings n for channels above seahm increased from 0.5 to 0.6 and 0.6 to 0.7
WR_cal_2001_012.tcf	PAR	Code polygon changed, looking at previous results to make sure extent is efficient. nwk for FG5.660 changed as inverts were set to -74 not -0.74, 1D depth limit factor set to 10, seaham, rock wall was a problem otherwise
WR_cal_2001_012_ESTRY.tcf	PAR	Estry only version of model
WR_cal_2001_80m_013.tcf	PAR	Converted to an 80m model for faster simulation. Some drainage lines removed where the are not representiative in an 80m model. X channels used to bring culverts further from levee so as not avoid HX and SX connections being applied to the same cell.
WR_cal_2001_80m_013_alpha.tcf	PAR	testiing if changing a drainage line at cell 322 184 changed provides better stability
WR_cal_2001_80m_013_beta.tcf	PAR	Sensitivity test removing all drainage lines form model
WR_cal_2001_1dUS_80mDS_014.tcf	PAR	Upstream of channel MGA_90 (north of seahm weir) at pinch point in the model, 1d only downstream of this model is 80m grid
WR_cal_2001_80m_014.tcf	PAR	014 runs are a comparison of the 80m model, 40m model and 1d US of seaham/80m model
WR_cal_2001_014.tcf	PAR	All 014 runs have levee sections and roads updated from DEM interrogated zlines. Plot output locatiosn added to 014 runs
WR_cal_2001_1dUS_80mDS_014_alpha.tc	PAR	As above but with 30 second 2d timestep
WR_cal_2001_80m_014_alpha_tuflow.tcf	PAR	Testing aplha version, so TS files are modified to import with appropriate number of data outputs
WR_cal_2001_QTUS_80m_015.tcf	PAR	QT boundary added above dungog, recorded flows are used from level data. Rating curve from PINEENA DVD is located at: K:\B16030.k.gjr_WilliamsR_FS\DAT_RIVER_DATA\PINNEENA\Dungog_rating_curve.xls
	PAR	Material layer updated
	PAR	stability polygons used used to raise ZC values in 2 locations
WR_cal_2001_QTUS_80m_016.tcf	PAR	Upstream QT boundary fixed
WR_cal_2001_QTUS_80m_017.tcf	PAR	Areas that are bouncing due to topography have zpts region inspected from DEM (region size 80/3) to obtain average value in region.
WR_cal_2001_QTUS_80m_n0pt05_018.tcf	PAR	1d channels US of MGA_78 (~Clarencetown) varying manning n 0.05
WR_cal_2001_QTUS_80m_n0pt06_018.tcf	PAR	As above but n = 0.06
WR_cal_2001_QTUS_80m_n0pt07_018.tcf	PAR	As above but n = 0.07
WR_cal_2001_QTUS_80m_n0pt08_018.tcf	PAR	As above but n = 0.08
WR_cal_2001_QTUS_80m_no_raftsQ_019.	PAR	using WR_cal_2001_QTUS_80m_n0pt07_018.tcf, with no Rafts inflows, only DS boundary and US QT boundary determine if hydrology local flows are to early
WR_cal_2001_QTUS_80m_020.tcf	PAR	Using flows from RAFTS 2001 hydrology version 006
WR_cal_2001_RAFTS_80m_030.tcf	JJ	RAFTS inflows in upper catchments restored (changed 1d_nwk_WIL_019, 1d_bc_WIL_Rafts_002)
WR_cal_2001_RAFTS_80m_031.tcf	JJ	as above but channel configuration around Dungog improved [more cross sections and improved specification of bridge crossing] (changed 1d_nwk_WIL_020
WR_cal_1978_RAFTS_80m_032.tcf	JJ	as above but with embankments and bridge abutments around Dungog specified (added 2d_zln_embank_DUNGOG_001)
WR_cal_2001_RAFTS_80m_033.tcf	JJ	as above for 2001 event
WR_cal_1978_RAFTS_80m_033.tcf	JJ	as above but with reduced mannings n in channel of WIL (changed 1d_nwk_WIL_022)
WR_cal_1990_RAFTS_80m_033.tcf	JJ	as above for 1990 event
WR_cal_2001_RAFTS_40m_033.tcf	JJ	as "WR_cal_2001_RAFTS_80m_033.tcf" but with 40 m grid
WR_cal_1978_RAFTS_40m_033.tcf	JJ	as "WR_cal_1978_RAFTS_80m_033.tcf" but with 40 m grid
WR_cal_1990_RAFTS_40m_033.tcf	JJ	as "WR_cal_1990_RAFTS_80m_033.tcf" but with 40 m grid
WR_cal_1990_RAFTS_40m_034.tcf	JJ	as above but with 1D timestep of 1s (from 2s)
WR_cal_1990_RAFTS_40m_035.tcf **to** WR	JJ	investigation of sensitivity of 2D timestep
WR_cal_1990_RAFTS_40m_037.tcf	JJ	increased mannings n on the floodplain (from 0.037 to 0.07) [changed WIL_Mat_016.tmf]
WR_cal_1978_RAFTS_40m_039.tcf	PAR	Based on run 038, but with hydrology revised RAFTS run 010 and longer simulation (9 days) to get second flood peak
WR_cal_1978_QTUS_40m_040.tcf	GJR	Using Dungog inflows (estimated where no record)
WR_cal_1990_QTUS_40m_040.tcf	GJR	Using Dungog inflows (estimated where no record) - multiple 2D domain run
WR_cal_1978_QTUS_40m_041.tcf	GJR	Double gauged inflows at Dungog
WR_cal_1990_QTUS_40m_041.tcf	GJR	Double gauged inflows at Dungog
WR_cal_1990_QTUS_40m_042_1d.tcf	GJR	ESTRY only with n for MGA channels doubled
WR_cal_1990_QTUS_40m_043_1d.tcf	GJR	ESTRY only with n for MGA channels as for previous runs (ie about 0.055)
WR_cal_1990_QTUS_40m_044_1d.tcf	GJR	ESTRY only - same as 43 but wihtout any RAFTS inflows - just dungog - checking lag of dungog flows to glen martin- seems bout right
WR_cal_1990_HTUS_40m_045.tcf	GJR	Used HT boundary at Dungog gauge - this increased flows for out of bank time
WR_cal_1990_HTUS_40m_046.tcf	GJR	Used new RAFTS model (ver 018) which has link channels for routing of flows - improved calib
UWR_cal_1990_RAFTS_10m_051.tcf	PAR	Smaller 10m model of Upper Williams River, created in order to re-rate the Dungog Gauge.
		Downstream Boundary conditions is a HT boundary taken from H_MGA_34.2 from run WR_cal_1990_RAFTS_40m_048.
		1d network shortened to extend only down to MGA_34
UWR_cal_1990_RAFTS_10m_052.tcf	PAR	Area Us of dungog (near gaugiing station) modelled in 2d - bridges at dungog modelled in 2d using flow constrictions.
		Additional PO output near dungog
		Code polygon US of Myall creek shifted to null areas US of 1D model
		Culvert added (1d_nwk_Dungog_Culvert_052) underneath road on the dungog side of bridge.
		1d XS updated, XS generator used to convert Surveyed points in to csv cross-sections was ignoring left hand point. some zero values found in survey points were removed.
UWR_cal_1990_RAFTS_10m_053.tcf	PAR	

Run ID	Modeller	Changes
UWR_cal_1990_RAFTS_10m_054.tcf	PAR	In channel mannings n = 0.055
UWR_cal_1990_RAFTS_10m_055.tcf	PAR	In channel mannings n = 0.065
UWR_cal_1990_RAFTS_10m_056.tcf	PAR	In channel mannings n = 0.07
UWR_cal_1990_RAFTS_10m_057.tcf	PAR	FC losses recalculated, separating the losses into left bank, channel and righ bank. In channel mannings n = 0.07 PO !Moved (from version 048) to ensure that it is halway along 1d-2d connection to avoid miscalculation
UWR_cal_1990_RAFTS_10m_058.tcf	PAR	In channel mannings n = 0.08
UWR_cal_1990_RAFTS_10m_059.tcf	PAR	Changes to PO only, used a different version to avoid overwriting results
WR_cal_1990_QTUS_40m_062	PAR	
WR_cal_1990_QTUS_40m_063	PAR	QT boundary removed from US of Hunter 1d, BC returns to 1d network setup by PEV and recorded bydrographs at morpeth and hinton
	PAR	Reads Code for Hunter River model as MID (created from WR_cal_1990_QTUS_40m_062 grid check file - has all other columns removed)
WR_cal_1990_QTUS_40m_064	PAR	Return to inflow at upstream end of Hunter River 1d, z lines for levee read in as THICK as opposed to RIDGE THICK, because DEM is higher in some locations.
	PAR	Hunter river 1d netowrk slightly changed at confluence, increase in the formloss (from 0.2 to 0.5) at confluence
	PAR	Reads code in MID format for Hunter 2D
	PAR	Start time at 3hrs (0300 on the 01/02/1990) tive give the model more time to warm up
	PAR	Bridge Channel added to hunter 1D. Form loss of 0.2 used (from 15540)
WR_cal_1978_QTUS_40m_065	PAR	Cross section near Dungog Brigdes moved slightly as it was not snapped to 1D Channel. Fixes to WLL to ensure not gaps in mapping
	PAR	Mannings n from Seaham to Clarence town decreased from 0.045 to 0.04
WR_cal_1990_QTUS_40m_065	PAR	As above plus Ocean Boundary from MIKE11 extended backward to 1/02/1990 by copying tidal cycle
WR_cal_1990_QTUS_40m_066	PAR	Mannings n from Seaham to Clarence town increased to 0.05
WR_cal_1990_QTUS_40m_067	PAR	Mannings n increased to 0.055 (seham to clarencetowm) XS downstream of Seaham Spillway set as centre cross-section so, because downstream XS unrepresentative of conveyance
WR_cal_1990_QTUS_40m_068	PAR	Small changes to code polygon, areas where water almost to extent of model. Mannings n reduced to 0.05 (seaham to clarencetown)
WR_cal_1978_QTUS_40m_069	PAR	Model Rerun after calibration of weir using 1d only model to calibrate to 1978 and 2001 weir levels. Breakline for rail network added from QASCO photogrammetry
WR_cal_1978_QTUS_40m_070	PAR	Mannings n from Seaham to Clarence town increased to 0.045
UWR_cal_1990_RAFTS_10m_060	PAR	Upper Williams River only model,. 1990 RAFTS inflows scaled by a factor of 1.7
UWR_cal_1990_RAFTS_10m_060FD	PAR	As Above. FD stands for Factored Depth - Depth at the downstream node is increased by a factor of 10%.
WR_cal_1963_DS_of_DG_40m_071	PAR	Williams River only model. Inflows generated at Dungog Road bridge (model extends only this far). Inflows for Alan Nash's (SES) record book
WR_cal_2700_Cumec_Inflow_40m_071	PAR	1963 hydrograph scaled up to 2700 cumecs
UWR_4500cumec_10m_061	PAR	A simple hydrograph which increase linearly to 4500m3/s into Williams River at time 25hours. Perctengae flow into Main and Myall Creek are factored using ratio's from 1990 RAFTS inflows. Downstream boundary levels are estimated based on inflows and levels from previous runs.
DESIGN RUNS		
WR_Q100_36hr_40m_071	PAR	Based on Calibration Model version 71
WR_Q100_36hr_40m_072	PAR	RAFTS inflows have a factor of 0.72 applied so as to match 3130 cumecs at Dungog. Z line for Railway added. Changes made to network at Dungog Bridges (modelled in 1d)
WR_Q100_36hr_40m_073	PAR	RAFTS inflows factor increased to 0.73. Stability Polygon Added
WR_Q100_36hr_40m_074	PAR	Changes to network near dungog. WLL's Changec
WR_Q100_36hr_40m_075	PAR	Changes to Materials near Grahamstown Dam Spillway. Stability polygons changed.
WRHR_Q100_36hr_40m_075	PAR	Hunter River model added. Grahamstown dam added, campvale pumps added to gtown dam inflows
WRHR_Q100_36hr_40m_076	PAR	Changes to Spillway at grahamstown dam. Changes to water level lines
WRHR_Q100_36hr_40m_077	PAR	Grahamstown Dam modelled as per HWC grahamstown dam augmentation stage 2 report (feb 2004)
WRHR_Q100_36hr_40m_078	PAR	Upper Williams River (10m) model added to williams/hunter model. Slope added to culvert at dungog to provide better stability. Change to zlg to let flow into culvert SX connections
WRHR_Q100_40m_079	PAR	2d_2d connection between UWR and WR model has a vertices moved for stability reasons.
WRHR_Q100_40m_080	PAR	Factor for RAFTS flows reduced to 0.71. Event combinations changed
WRHR_Q100_40m_080_Sens1	PAR	Gtown Dam Empty
WRHR_Q100_40m_080_Sens2	PAR	Chichester Dam Empty
WRHR_Q100_E05_081	PAR	RAFTS (RAFTS run 022) has factor on rainfall not in bc_dbase
WRHR_Q100_E05_082	PAR	SX connections are used for the weir over the rail bridge at dungog, in an effort to get the model stable. Rail weir is modelled as 1d w channels
WRHR_Q100_E05_083	PAR	Weir over bridges at dungog attached to 1d nodes at bridge not to 2d. 1 weir for each bridge, (not left bank right bank).
WRHR_Q100_Ev_084	PAR	Etxra channel MGA_09B added for stability in large events. Changes at Pinebrush road Bridge. Climate change allowance of 0.91 added.
WRHR_Des_Ev1B_085	PAR	Changes to hxe to ensure all wet cells are included. Extra PO added for tabular output. Z1 output type.
WRHR_Q100_Ev2A_085	PAR	This is a manning's n sensitivity run. Manning's n values in river are increased by 20%, floodplain values increased by 50%. Changes to 1d_nwk files and changes to tmf.

BMT WBM Brisbane	**Level 11, 490 Upper Edward Street Brisbane 4000 PO Box 203 Spring Hill QLD 4004 Tel +61 7 3831 6744 Fax +61 7 3832 3627 Email wbm@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Denver	**14 Inverness Drive East, #B132 Englewood Denver Colorado 80112 USA Tel +1 303 792 9814 Fax +1 303 792 9742 Email wbmdenver@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Melbourne	**Level 5, 99 King Street Melbourne 3000 PO Box 604 Collins Street West VIC 8007 Tel +61 3 8620 6100 Fax +61 3 8620 6105 Email wbmmelbourne@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Morwell	**Cnr Hazelwood Drive & Miners Way Morwell 3840 PO Box 888 Morwell VIC 3840 Tel +61 3 5135 3400 Fax +61 3 5135 3444 Email wbmmorwell@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Newcastle	**126 Belford Street Broadmeadow 2292 PO Box 266 Broadmeadow NSW 2292 Tel +61 2 4940 8882 Fax +61 2 4940 8887 Email wbmnewcastle@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Perth	**1 Brodie Hall Drive Technology Park Bentley 6102 Tel +61 8 9328 2029 Fax +61 8 9486 7588 Email wbmperth@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Sydney	**Suite 206, 118 Great North Road Five Dock 2046 PO Box 129 Five Dock NSW 2046 Tel +61 2 9713 4836 Fax +61 2 9713 4890 Email wbmsydney@wbmpl.com.au Web www.wbmpl.com.au**
BMT WBM Vancouver	**1190 Melville Street #700 Vancouver British Columbia V6E 3W1 Canada Tel +1 604 683 5777 Fax +1 604 608 3232 Email wbmvancouver@wbmpl.com.au Web www.wbmpl.com.au**