Hunter River Flood Study (Muswellbrook to Denman)

Model Revisions Report

For: Muswellbrook Shire Council

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PROJECT INFORMATION

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PREPARATION, REVIEW AND AUTHORISATION

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ABBREVIATIONS

AHD	*Australian Height Datum*
AEP	*Annual Exceedance Probability*
ARF	*Areal Reduction Factor*
ARR 1987	*Australian Rainfall and Runoff (1987)*
ARR 2016	*Australian Rainfall and Runoff (2016)*
DEM	*Digital Elevation Model*
FFA	*Flood Frequency Analysis*
FRMS	*Flood Risk Management Study*
IEAust	*Institution of Engineers Australia*
IFD	*Intensity Frequency Distribution*
LiDAR	*Light Detection and Ranging*
RHDHV	*Royal HaskoningDHV*
TUFLOW	*A hydraulic model that is used to simulate flood events.*
XP-RAFTS	*A hydrologic model that is used to estimate runoff hydrographs.*
1D	*One-dimensional*
2D	*Two-dimensional*

1 INTRODUCTION

Muswellbrook Shire Council (Council) commissioned Royal HaskoningDHV (RHDHV) to produce the Hunter River (Muswellbrook to Denman) Floodplain Risk Management Study (FRMS) on behalf of Council and The NSW Office of Environment and Heritage (OEH). The FRMS builds on the Hunter River Flood Study (Muswellbrook to Denman) that was prepared by Worley Parsons in 2014.

One of the initial tasks of the FRMS was to undertake a technical adequacy review of the 2014 flood study. This review was prepared by RHDHV in March 2016 and identified a number of issues regarding the reliability of the Hunter River Flood Models that were developed as part of the 2014 study.

Subsequent to this review being completed, OEH were made aware that rating curves for many of the Upper Hunter stream gauges had been recently revised by WaterNSW. The revised rating curves substantially reduce the assumed flows for a given gauge stage. The revisions are due to the increase in vegetation densities both within the channel and on the channel banks over the last two decades.

A meeting was held on 29 October 2016 to discuss the need to recalibrate and verify the Hunter River Flood Models that were developed by Worley Parsons in 2014 as part of the Flood Study. It was decided that the Hunter River model calibration and design event verification needed to be revisited to ensure confidence in the outcomes of the FRMS and potential future uses of the model.

The model revision process also provided an opportunity to update the models to be consistent with the recently formalised Australian Rainfall and Runoff 2016 (Commonwealth of Australia) guidelines. The 2014 flood study applied the methods documented in the Australian Rainfall and Runoff 1987 (IEAust) guideline.

The following scope for the model revision process was established by RHDHV in consultation with OEH and Council:

- Review and analyse recent changes to stream gauge rating curves.
- Modify the Hunter River hydraulic model to more reliably represent the current floodplain characteristics.
- Recalibration of the Hunter River hydrologic and hydraulic models using data from flood events that occurred in 1988 and 2000.
- Undertake flood frequency analysis using data from the Muswellbrook stream gauge.
- Apply the outcomes from the model calibration and verification process and the Australian Rainfall and Runoff 2016 methods to establish revised design event conditions for a full range of Annual Exceedance Probability (AEP) flood events.
- Verify the revised design model outcomes using available data from the 1955 and 1971 events.

This report documents the methodologies, assumptions and results from the above assessment.

1.1 Terminology

To improve readability, the following terminology is used in this report:

- **Flood Study Models** refers to the hydrologic and hydraulic models developed as part of the Hunter River (Muswellbrook to Denman) Flood Study (Worley Parsons, 2014)
- **Current Study or Revised Models** refers to the hydrologic and hydraulic models revised as part of the model revision process that is documented in this report. These models are expected to be used in the FRMS.

1.2 Report Structure

This report documents the model revisions and is structured as follows:

- **Section 2** reviews information that was used in this report.
- **Section 3** describes changes to the Hunter River Catchment and Floodplain.
- **Section 4** describes revisions that have been made to the Flood Study Models.
- **Section 5** documents flood frequency analysis that has been undertaken using data from the Muswellbrook Bridge Stream Gauge.
- **Section 6** documents the revised design event simulation assumptions and results.
- **Section 7** documents model verification analysis that was undertaken using flood levels from the 1971 and 1955 events.
- **Section 8** documents sensitivity analysis that was undertaken to gain an understanding of the flood mitigation benefit afforded by Glenbawn Dam and to assess the model's sensitivity to changes in the channel roughness assumptions.

2 REVIEW OF AVAILABLE INFORMATION

This section reviews information that was used to inform model revisions.

2.1 Relevant Reports

The following reports were reviewed as part of the model revision process.

Hunter River Flood Study (Muswellbrook to Denman): (Worley Parsons, 2014)

The Hunter River Flood Study (Muswellbrook to Denman) was produced by Worley Parsons in 2014 as part of the NSW Government's Floodplain Management Program. The study is informed by an integrated hydrologic and hydraulic model of the Upper Hunter River Floodplain Catchment. The model encompasses the entire extent of the Hunter River Floodplain that is located within the Muswellbrook Council Local Government Area (LGA). The upstream portion of the model (from the upstream LGA boundary to the Goulburn River) was developed in TUFLOW as a two-dimensional (2D) hydraulic model, while the lower portion of the model (from the Goulburn River to the downstream LGA boundary) was developed in TUFLOW as a one-dimensional (1D) hydraulic model dynamically linked to the upstream 2D model.

Surface elevations within the hydraulic model are informed by Light Detection and Ranging (LiDAR) data that was acquired by State Water in 2010. The integrated hydrologic and hydraulic models were calibrated using available information from flood events that occurred in 1998, 2000 and 2007. The study did not attempt to use available information from the 1955 or 1971 events or the extensive Muswellbrook Stream Gauge record to verify the model results.

The hydrologic and hydraulic models developed as part of this study were provided to RHDHV for use in the FRMS. RHDHV have modified some aspects of the models. All modifications are noted in **Section 4** of this report.

Muswellbrook Flood Study (WRC, 1986)

The Muswellbrook Flood Study was prepared by the Water Resources Commission in 1986. The study includes a review of stream gauge data between 1907 and 1984 and information from the 1955, 1971 and 1977 floods.

Flood Frequency Analysis (FFA) was undertaken using data from the Muswellbrook (210022) and Muswellbrook Weir (210008) stream gauges. In 1987, during the completion of this study, Glenbawn Dam was upgraded. Since the flood mitigation capacity of the dam did not materially change (i.e. the flood storage volume remained effectively the same), the study concluded that the dam upgrade would not alter the flood mitigating benefit afforded by the dam. Accordingly, flood frequency curves were established for pre and post Glenbawn Dam scenarios. This analysis concluded that Glenbawn Dam would reduce flood levels at the Muswellbrook gauge location by 220 mm in a 1% AEP event.

The study did not include any hydraulic modelling but established an indicative 1% AEP flood extent using anecdotal evidence, surveyed flood levels from the 1955 event and topographic survey.

The following information from this study has been used in this assessment:

- Surveyed flood levels from the 1955 and 1971 events that are provided in Appendix A of the WRC report have been used in this model revision exercise to estimate the extent and nature of flooding within Muswellbrook for these events.

- The Muswellbrook Stream Gauge (210002) did not operate between 1926 and 1961. During this period flows were gauged at Muswellbrook Weir (210008), which was located on the downstream side of Muswellbrook. **Table 4.2** from the WRC report provides a summary of the peak annual flows recorded at Muswellbrook Weir and correlates these flows to the Muswellbrook (210022) location. This information was used in the flood frequency analysis that is documented in **Section 5** of this report.

The WRC (1986) study also included some information on major flood events that occurred prior to the implementation of the Muswellbrook (210022) Stream Gauge in 1907. This data was also was used in the flood frequency analysis that is documented in **Section 5** of this report.

Denman Flood Study (WRC, 1986)

The Denman Flood Study was prepared by the Water Resources Commission in 1986. The study includes a review of stream gauge data, local flood characteristics and information from the 1955, 1971 and 1977 floods. The study did not include any hydraulic modelling but established an indicative 1% AEP flood extent using anecdotal evidence, surveyed flood levels from the 1955 event and topographic survey. The study also discussed the flood mitigation benefits of the Denman Levee, which was in an advanced stage of planning and design at the time of writing.

Surveyed flood levels from the 1955 and 1971 events that are provided in Appendix A of the WRC study have been used in this model revision exercise to estimate the extent and nature of flooding within Denman for these events.

1955 Flood Reports

OEH provided the following two 1955 flood reports:

- Flood Report: Hunter River 1955 (Bernard, 1968)
- A Report on the Flood of February 1955 (Hunter Valley Conservation Trust).

Both of these reports provide information on the 1955 flood in the Upper Hunter Valley.

2.2 Model and GIS Files

The hydrologic and hydraulic models and some GIS files that were developed as part of the Flood Study (Worley Parsons, 2014) were provided to RHDHV for use in the FRMS. RHDHV have modified some aspects of the models. All modifications are noted in **Section 4** of this report.

2.3 Information provided by WaterNSW

WaterNSW provided the following information that was used in this study:

- All rating curves that were used to calculate stream flows at the Muswellbrook (210022) and Denman (210055) stream gauges between 1990 and the present time.
- Photographs that depict the change in riparian vegetation at the Muswellbrook (210022) and Denman (210055) stream gauge locations.

Data from the following stream gauges was analysed:

Muswellbrook Bridge Hunter River (210002)

This gauge is located at the Kayuga Road Bridge, immediately upstream of the Muswellbrook Levee. The gauge operated between 1907 and 1927 before being temporarily decommissioned. The gauge was reinstated in 1961 and is still in operation.

Flood heights, rating curves and photos of riparian vegetation at the gauge were provided by WaterNSW. This data formed the basis of the FFA which was used to derive design flows.

Muswellbrook Weir Hunter River (210008)

The Muswellbrook Weir gauge was located downstream of Muswellbrook and operated between 1918 and 1963. As previously mentioned, the Muswellbrook Flood Study (WRC, 1986) derived a relationship between the Muswellbrook Bridge and Muswellbrook Weir gauges to form a continuous flow series at the Muswellbrook Gauage (210002). This data formed the basis of the FFA which was used to derive design flows.

Denman Hunter River (210055)

This gauge is located at the Golden Highway Bridge, to the east of Denman. The gauge commenced operation in 1959 and is still in operation. Due to the limited gauge record, no FFA has been undertaken using the gauge record. However, data from this gauge has been used to inform the model calibration and revisions that are discussed in **Section 4**.

3 CHANGES TO THE CATCHMENT AND FLOODPLAIN

The model calibration and verification process that is documented in this report seeks to demonstrate the reliability of the design flood models using data from floods that occurred in 1955, 1971, 1998 and 2000. It is therefore necessary to form an understanding of changes to both the contributing catchment area and floodplain between the 1950s and the present time. This section documents key changes to the catchment and floodplain that are relevant to this study.

3.1 Floodplain Changes

The following changes to the floodplain have occurred since 1955:

- **Vegetation Changes**: The density of vegetation on the channel banks and within the channel has increased significantly since the early 1990s. This is understood to be the result of Landcare initiatives such as fencing riparian zones and the establishment of self-propagating willows in the channel. **Appendix A** provides a series of pictures provided by WaterNSW that were taken at the Muswellbrook and Denman gauge locations in 1980 and 2011. These photographs clearly show the significant increase in vegetation densities at both locations.

- **Physical Changes**: key physical changes to the floodplain in the Denman and Muswellbrook areas include:

  - Construction of the Muswellbrook and Denman levees in the early 1990s.

  - Channel straitening and sand and gravel exraction on the floodplain near Denman.

  Key physical changes are diagrammatically presented in **Appendix A**.

3.2 Rating Curve Changes

WaterNSW have recently revised the rating curves at the Muswellbrook (210002) and Denman (210005) gauges. The revisions are due to the increase in vegetation densities within the channel and channel banks over the last two decades. **Figures 1** and **2** show the rating curves that were provide by WaterNSW for 1990, 1998-2000 and 2010-current conditions for the Muswellbrook and Denman gauges respectively. It noted that these rating curves were not available when the Flood Study (Worley Parsons, 2014) was undertaken.

Figure 1 shows that the estimated capacity of the Hunter River Channel at bank full (approximately gauge height 10m) has reduced from 2,200 m^3/s to 900 m^3/s over the last 20 years. This implies an increase in the frequency of overbank events and magnitude of floodplain conveyance and generally higher flood levels.

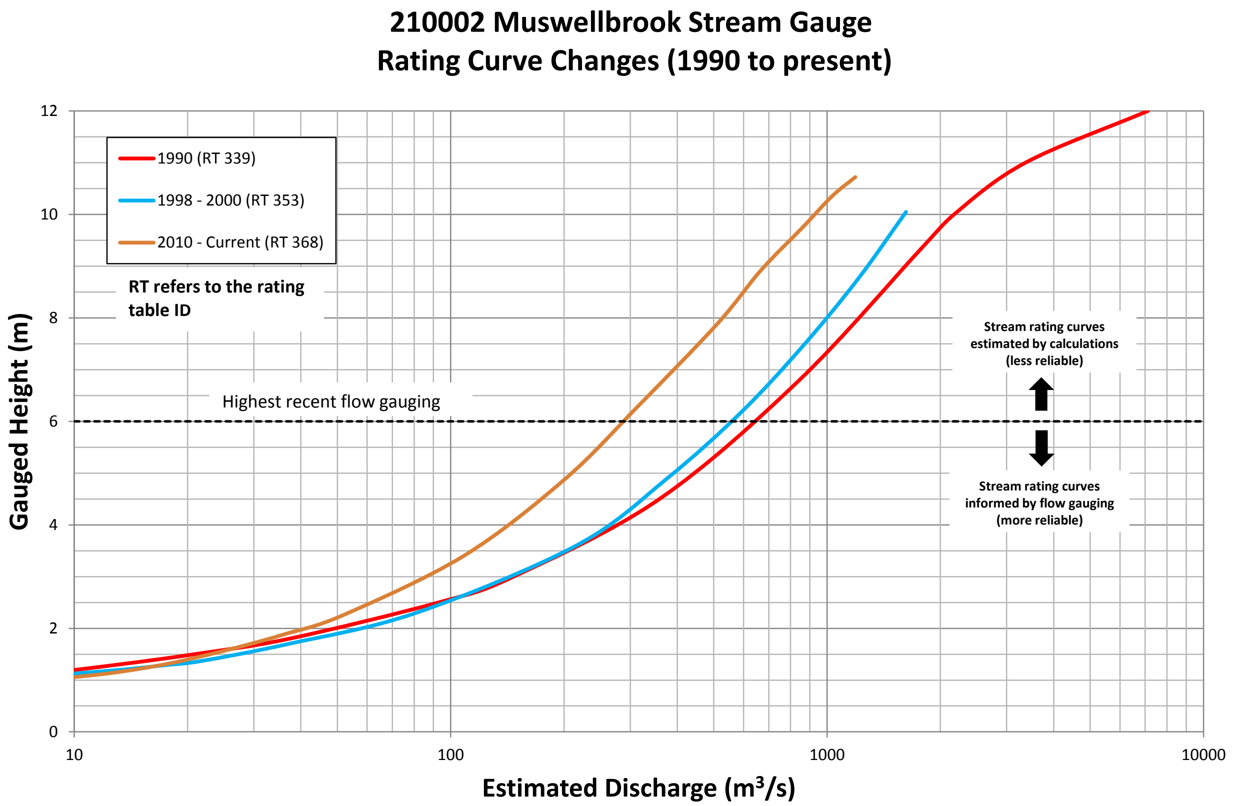


Figure 1 – *Muswellbrook Gauge (210002): Rating Curve Changes*

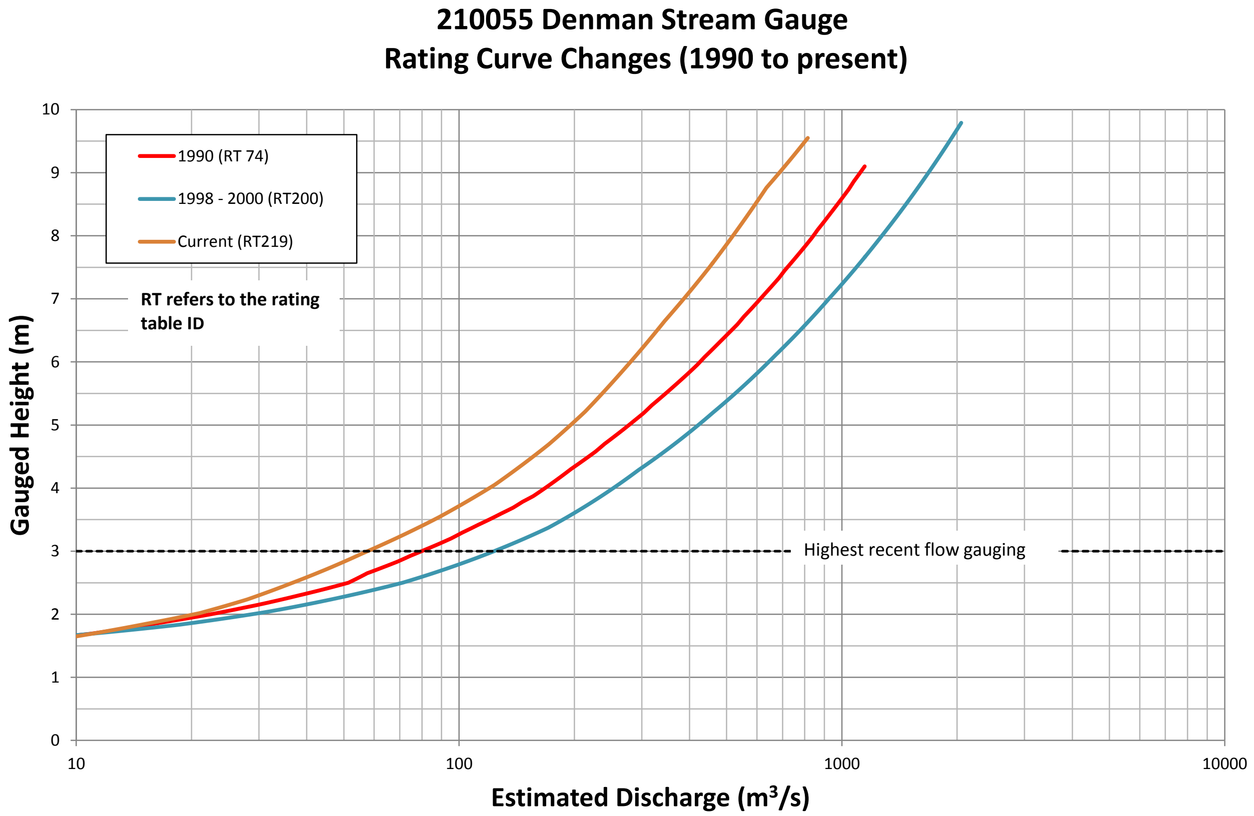


Figure 2 – *Denman Gauge (210055): Rating Curve Changes*

3.3 Glenbawn Dam

Glenbawn Dam is located on the Hunter River, approximately 35 km upstream of Muswellbrook. The dam's catchment accounts for approximately 33% of the Hunter River Catchment upstream of Muswellbrook. Construction of the dam commenced in late 1947 and was completed in late 1957. According to the Aberdeen Flood Study (2013, WMAwater), the dam wall was only partially constructed during 1955 and the 1955 flood event passed through the dam relatively un-attenuated. Glenbawn Dam was constructed with a dam wall height of 78 m, a storage capacity of 300,000 ML and a flood mitigation capacity of 133,000 ML.

An upgrade of Glenbawn Dam was undertaken in 1986 / 1987. The upgrade comprised raising the dam wall height to 100 m and reconfiguring the outlet controls. The upgrades increased the dam's storage capacity to 750,000 ML. However, the flood mitigation capacity was reduced from 133,000 ML to 120,000ML. The Muswellbrook Flood Study (1986) references a study by Hayes (1982) which found that the flood storage capacities of 133,000 ML and 120,000 ML would *"effectively have the same mitigating effect"*.

The adequacy of the flood mitigation function of this dam has not been reviewed as part of the model revision process. However, the Aberdeen Flood Study (2013, WMAwater) concluded that no outflow from the dam's spillway is expected for the 0.2% AEP and lower magnitude flood events.

The dam and contributing catchment areas are included in the hydrologic model. The dam storage captures all runoff from all design events (except the PMF). The model is configured to release water from the dam at a constant rate of 128 m^3/s, which is the estimated capacity of the twin outflow values. This is the same approach that was applied in the Aberdeen (2013, WMAwater) and Muswellbrook (WorleyParsons, 2014) flood studies.

4 MODEL RECALIBRATION AND REVISION

A number of revisions were made to the hydrologic and hydraulic models that were developed as part of the Muswellbrook Flood Study (Worley Parsons, 2014). The revised models were recalibrated using available information from the 1998 and 2000 events. The recalibrated models were applied to assess design flood events.

This section describes the revisions and associated rationale and is structured as follows:

- Model revisions are discussed in **Section 4.1**.
- The model recalibration outcomes are described in **Section 4.2.**
- **Section 4.3** summarises the key outcomes from the model revision and recalibration process.

4.1 Model Revisions

This section describes revisions that were made to the hydrologic and hydraulic models.

4.1.1 Hydrologic Model Revisions

One of the initial tasks of the FRMS was to undertake a technical adequacy review of the Muswellbrook Flood Study (Worley Parsons, 2014). This review was prepared by RHDHV in March 2016 and identified that the hydrologic models developed to simulate design events were parameterised with substantially longer lag times than the calibration models. Refer to the Calibration Review Memo (RHDHV, 2016) for detailed information on this discrepancy. No explanation for this discrepancy in lag times between the calibration and design models was provided in Worley Parsons (2014).

As a result of this discrepancy, the design event hydrologic models were not used and the calibration models were adopted as the ‘working model’ to be used in the FRMS&P study. Some parameters were adjusted during the model recalibration process that are described in **Section 4.2**.

4.1.2 Hydraulic Model Revisions

The following changes were made to the hydraulic model that was prepared as part of the Muswellbrook Flood Study (Worley Parsons, 2014):

- The hydraulic roughness categories were more reliably defined to improve the spatial definition of the various land-uses and vegetation densities within the floodplain and channel zone.
- The channel invert was lowered in pool sections. This was required as the model’s Digital Elevation Model (DEM) was informed by LiDAR which measured the standing pool level at the time of survey as the channel invert.

Once model revisions were made, a number of rating curve runs were simulated with various roughness coefficients to identify Manning’s roughness values that would recreate the rating curves provided by WaterNSW.

The model revisions are discussed further below.

Improved Definition of Hydraulic Roughness Categories

Hydraulic roughness is a key parameter in any hydraulic model. Typically, a floodplain is divided into a number of roughness categories based on land use and vegetation densities. A review of the definition of hydraulic roughness categories in the TUFLOW model developed for the Muswellbrook Flood Study (Worley Parsons, 2014) concluded that:

- The channel zone was generally defined as the base of the channel only, with the channel banks assumed to be low roughness floodplain category.
- Numerous areas of dense floodplain vegetation (i.e. olive groves) were not defined.

Hydraulic roughness definition was revised in the entire 2D model domain. Key changes included:

- More reliable definition of the channel bank / floodplain interface.
- The channel zone was divided into a vegetated and un-vegetated category.
- Areas of dense floodplain vegetation (i.e. olive groves) were included.

Figure 3 shows an example of the changes made to the roughness category definition around Muswellbrook, a key area of the hydraulic model.

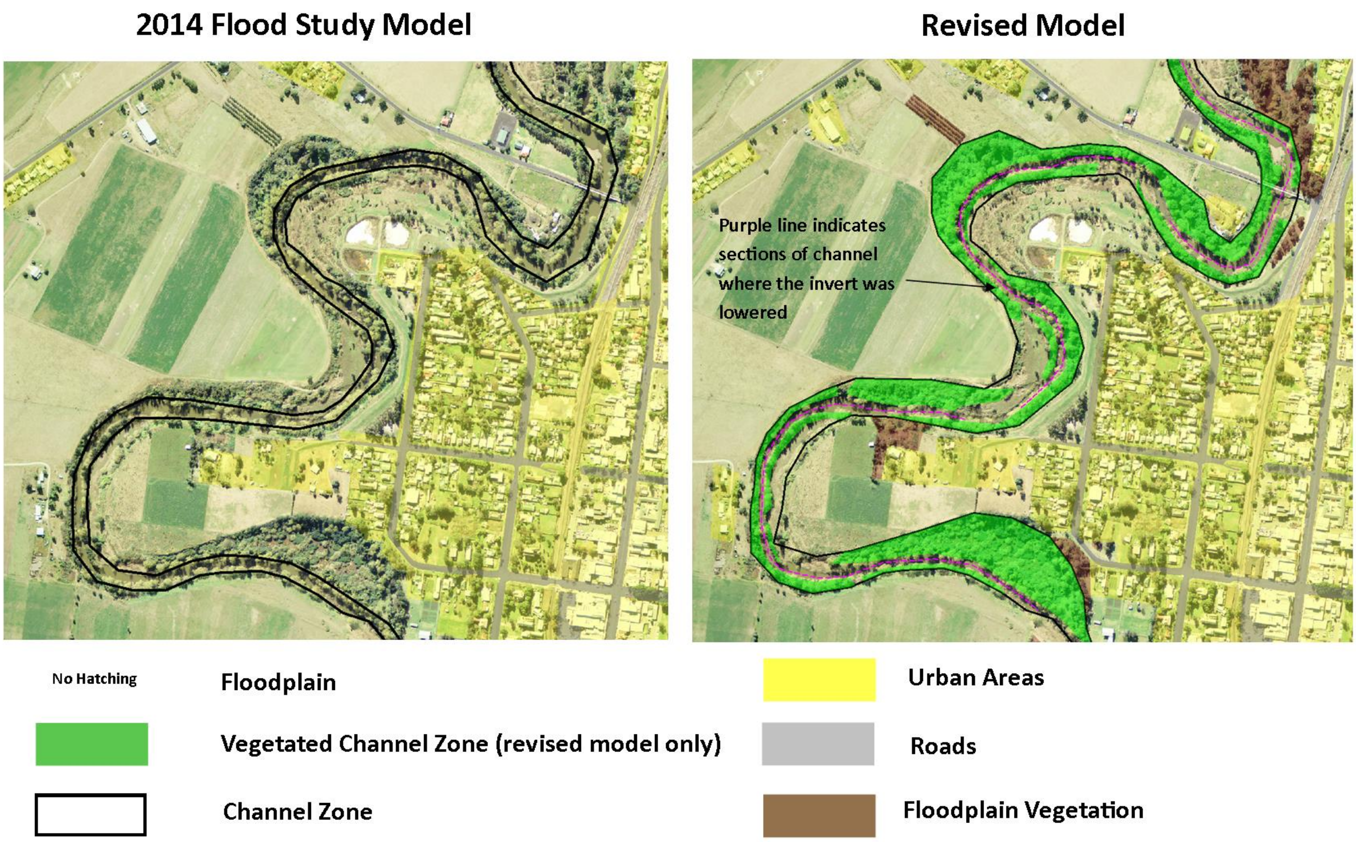


Figure 3: *Changes to roughness categories*

Improved Definition of Channel Invert

The hydraulic model's DEM represents the surface levels of the floodplain and channel. The DEM is based on the LiDAR survey data. The LiDAR survey measured the level of standing water in the channel at the time of survey, rather than the channel invert. This has resulted in the cross-sectional area of the channel being understated. There is no survey information available that reliably defines the channel bathymetry. In the absence of any definitive data, it was decided to lower the channel invert of pool zones by 2 m to improve the channel conveyance.

Figure 4 shows a channel section at the Muswellbrook Gauge (210002) that was provided by WaterNSW. The LiDAR levels and the adopted channel deepening approach are shown diagrammatically.

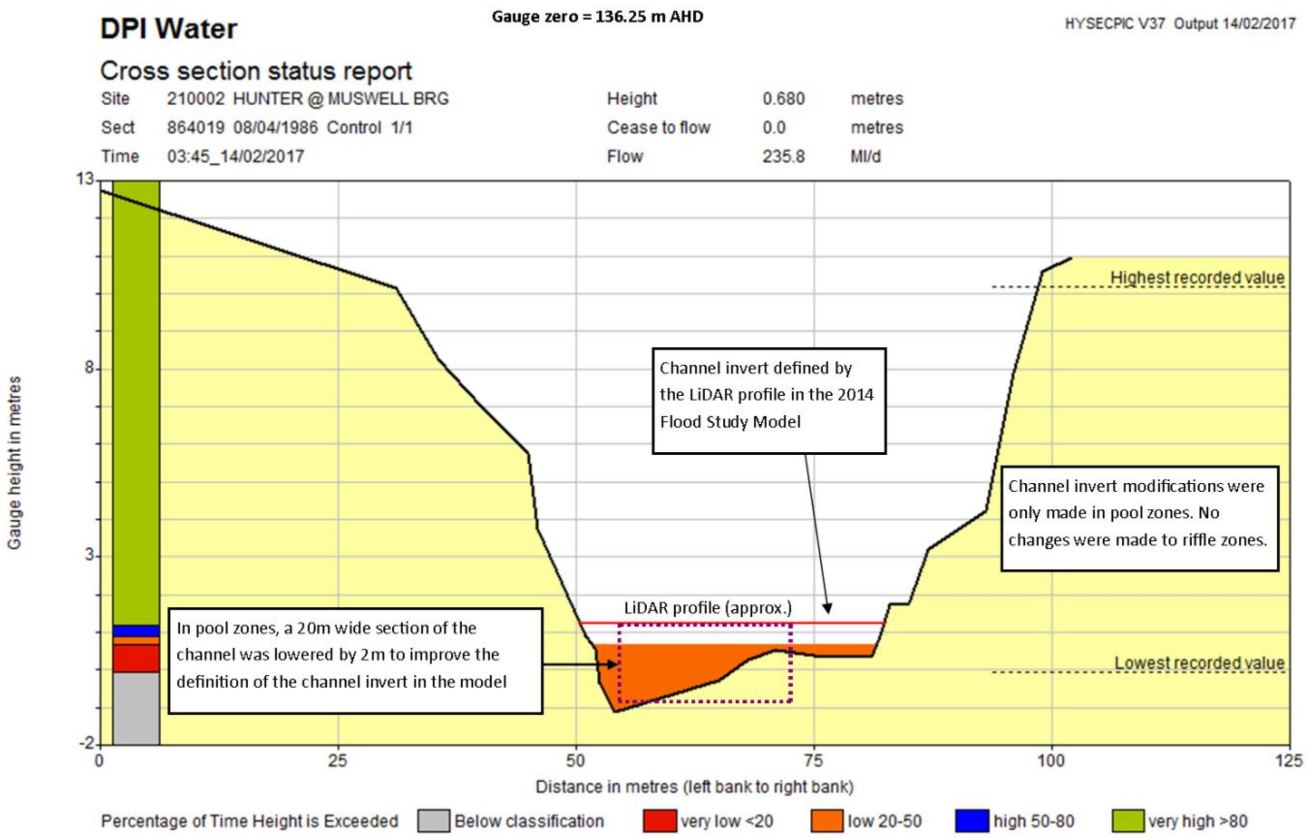


Figure 4: *Channel deepening approach*

Changes to Roughness Coefficients

Once the above-mentioned revisions were made to the hydraulic model, a number of 'rating curve simulations' were run with various roughness coefficients. The objective of this analysis was to identify Manning's roughness values that would recreate the rating curves provided by WaterNSW. A 'rating curve simulation' was undertaken by slowly increasing the flows through the model and extracting a water level / discharge profile at the gauge location from results files.

This process was applied to assess five roughness value configurations, which are referred to as R1 to R5. Values for R1 and R2 are not discussed as they were too low. Values for R3 to R5 are provided in **Table 1**. The values adopted in the Muswellbrook Flood Study (Worley Parsons, 2014) are also shown for context.

***Table 1**: Roughness Value Configurations*

Scenario	Channel Invert	Channel Zone Roughness		Floodplain Roughness		Urban Area Roughness	
		Un-Vegetated Areas	Vegetated Areas	Grassed Areas	Tree Areas	Roads	Lots
2014 Flood Study	LiDAR Levels	0.035		0.035	0.06	0.02	0.08
R3	Lowered in pool zones by 2m	0.030	0.060	0.035	0.06	0.02	0.08
R4		0.035	0.100	0.035	0.06	0.02	0.08
R5		0.035	0.150	0.035	0.06	0.02	0.08

Note: LiDAR survey does not define the channel invert when standing water is present.

Hydraulic model results are presented for the Muswellbrook and Denman gauges in **Figures 5** and **6**. By way of explanation:

- The black lines represent the WaterNSW rating curves for each gauge.
- The thick dashed line represents the simulated rating curve for the total floodplain (i.e. channel plus floodplain).
- The thin dashed line represents the simulated rating curve for the channel zone.

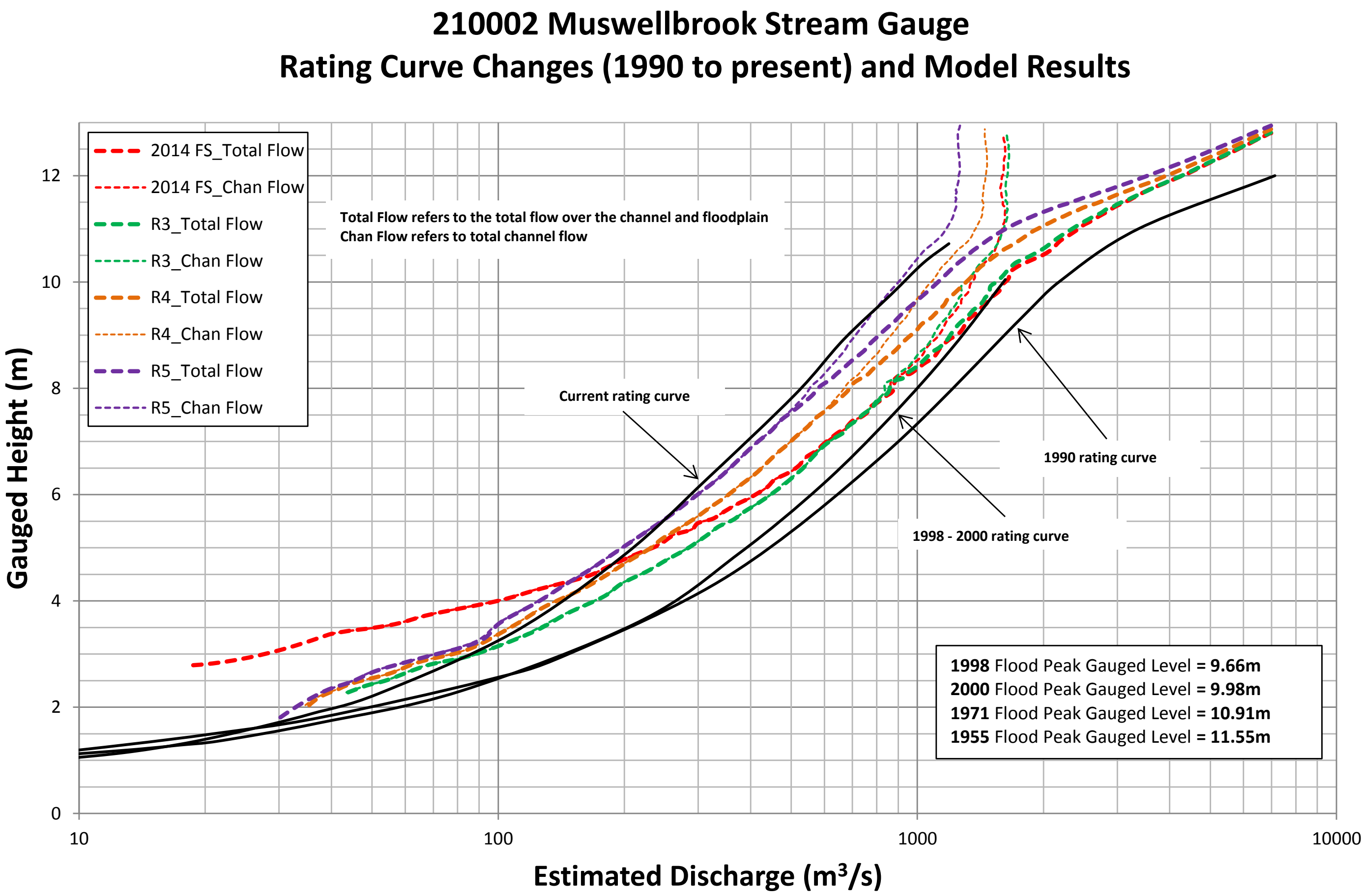


Figure 5: *Rating Curve Model Results (Muswellbrook Gauge: 21002)*

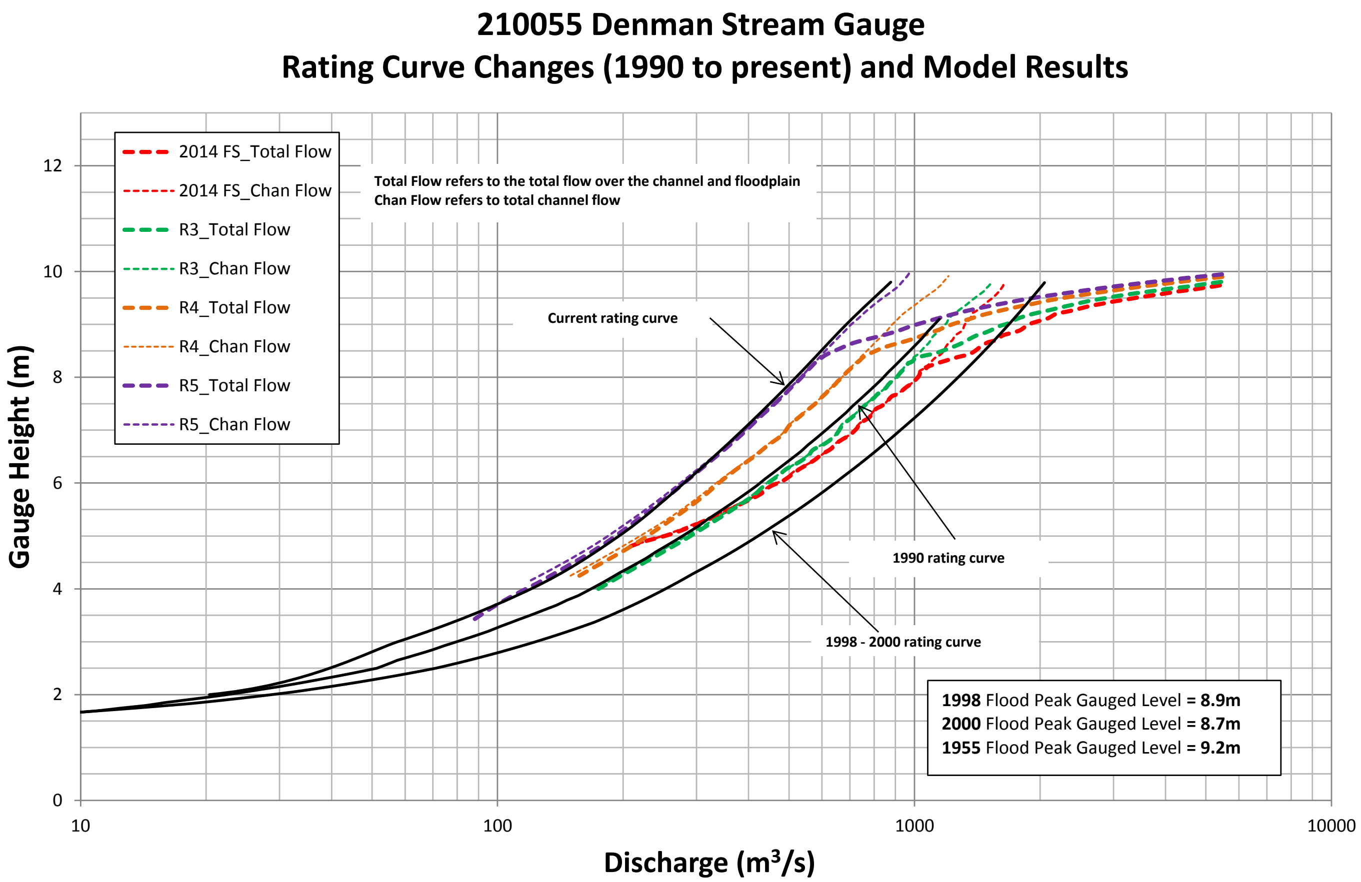


Figure 6: *Rating Curve Model Results (Denman Gauge: 21002)*

The R5 roughness value configurations produced results that were similar to the current rating curves at both the Muswellbrook and Denman gauges and the R3 produced similar curves to the 1998 – 2000 rating curve: Accordingly, the

- R3 configuration was applied to the Calibration Model, which was used to simulate floods that occurred in 1998 and 2000; and
- R5 configuration was applied to the Design Models.

4.2 Model Recalibration

The hydrologic and hydraulic models were recalibrated using available data from flood events that occurred in 1998 and 2000. These floods are estimated to have been less than 5% AEP events. The objective of the recalibration exercise was to improve confidence in the hydrologic model parametrisation and verify the hydraulic model. This section describes the recalibration approach and outcomes.

4.2.1 Available Data

A review of available stream gauge and rainfall data was undertaken. The following data was considered in the recalibration process:

- Data from 7 steam gauges.
- Data from 7 pluvio rainfall gauges.

- Data from 20 daily read rainfall gauges.

Figure 7 locates the above stream and rainfall gauges. It is noted that no material volumes of water were released from Glenbawn Dam during either event.

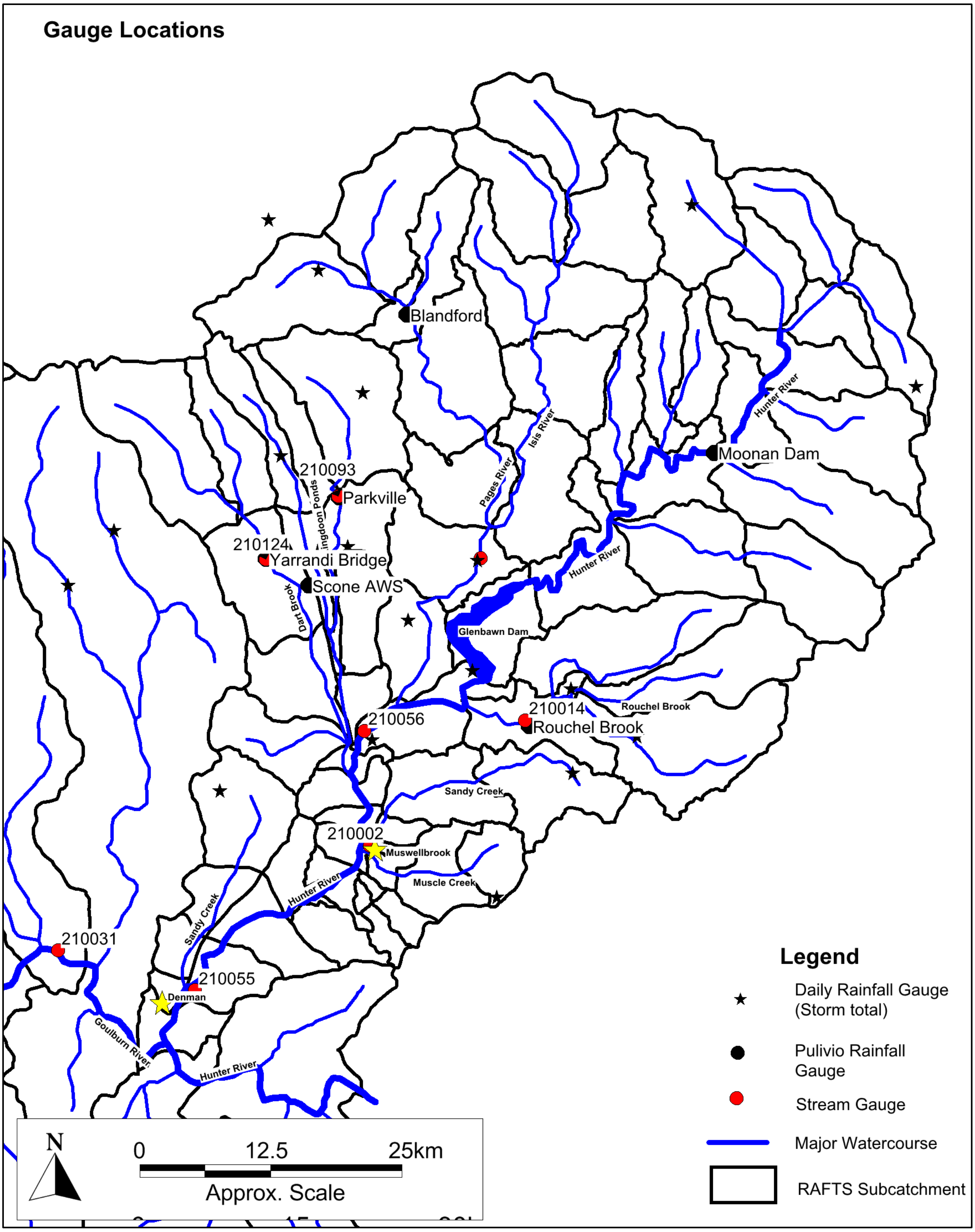


Figure 7: *Rainfall and Stream Gauge Locations*

4.2.2 November 2000 Event Calibration Approach

The November 2000 event comprised intense rainfall in the headwater catchments of the Pages River, with 220mm recorded over a 48 hour period. Moderate amounts of rainfall (70 to 100mm over 48 hours) occurred elsewhere in the Upper Hunter. **Figure B1** in **Appendix B** shows the rainfall recorded at the seven pluvio gauges. The resulting flood inundated low lying portions of the Hunter River Floodplain.

The recent revisions to the rating curves made by WaterNSW reduced the peak gauged flows at the Muswellbrook Gauge (210002) from 1,877 to 1,600 m^3/s. Accordingly, the revised calibration was targeting a lower peak flow than the calibration undertaken as part of the Flood Study (Worley Parsons, 2014). The calibration approach applied by Worley Parsons (2014) was reviewed. This review concluded that:

- The assumed distribution of the intense rainfall recorded at the Blandford pluvio gauge is not supported by the daily read data. **Figure B3** in **Appendix B** shows the distribution of the pluvio rainfall data that was applied by Worley Parsons (2014).
- The simulated flows in the Pages River were significantly overstated (by 100%).
- The simulated flows at the Muswellbrook Gauge (210002) were moderately overstated (by 15%).

On balance of evidence, it was decided to reduce the assumed application area of the high intensity rainfall recorded at the Blandford gauge to catchments closer to the gauge. **Figure B4** in **Appendix B** shows the revised distribution of the pluvio rainfall data, with some explanatory notes.

Calibration results are discussed separately in **Section 4.2.4 .**

4.2.3 1998 Event Calibration Approach

The August 1998 event comprised three runoff events that occurred over a three week period. The third event produced the highest gauged levels and was the focus of the calibration. For the third event, the highest rainfall occurred in the Rouchel Brook Catchment (115mm over three days). Moderate amounts of rainfall (65 to 80 mm over three days) occurred elsewhere in the Upper Hunter. **Figure B2** in **Appendix B** shows the rainfall recorded at the seven pluvio gauges. The resulting flood inundated low lying portions of the Hunter River Floodplain.

The recent revisions to the rating curves made by WaterNSW reduced the peak gauged flows at the Muswellbrook Gauge (210002) from 1,958 to 1,630 m^3/s. Accordingly, the revised calibration was targeting a lower peak flow than the calibration undertaken as part of the Flood Study (Worley Parsons, 2014). The calibration approach applied by Worley Parsons (2014) was reviewed. This review concluded that:

- The simulated flows in the Rouchel Brook Catchment were significantly understated (by 100%).
- The simulated flows at the Muswellbrook Gauge (210002) were understated (by 15%).

On balance of evidence, it was decided to moderately reduce the assumed application area of the high intensity rainfall recorded at the Rouchel Brook gauge to catchments closer to the gauge. **Figure B6** in **Appendix B** shows the revised distribution of the pluvio rainfall data, with some explanatory notes.

Calibration results are discussed separately in **Section 4.2.4 .**

4.2.4 Changes to the Hydrologic Model Parameters

The hydrologic models prepared by Worley Parsons for the 1998 and 2000 calibration events were modified to incorporate the revised rainfall assumptions that are discussed above. The modified models were applied to simulate both events and all model parameters were reviewed. The following adjustments were made to model parameters to improve the overall calibration outcome:

- The Storage Coefficient Multiplication Factor (Bx) was adjusted from 1.0 to 1.2. This moderately increases the attenuation of runoff hydrographs from the model's sub catchments, reducing peak flows.

- Initial and continuing loss (IL & CL) rates were simplified. The 2014 model calibration included six different IL and CL zones which ranged from IL 5mm and CL 1 mm/hr to IL 15 mm and CL 2.5 mm/hr. The following loss rates were adopted for all Upper Hunter River Catchments in the revised calibration:

    - Initial Loss Rate: 15 mm (1998 event, i.e. wetter antecedent conditions) and 30 mm (2000 event, i.e. drier antecedent conditions); and

    - Continuing Loss Rate: 1.5 mm/hr (both events).

It is noted that the calibration revision process did not establish justification to increase catchment lag times, as was done by Worley Parsons in the design event simulations.

4.2.5 Calibration Results

This section discusses the calibration results for the 2000 and 1998 events.

2000 Event Results

Calibration results for the 2000 event are provided in the following figures:

- **Figure 8** compares the simulated and gauged hydrographs at the Muswellbrook Bridge gauge location. Similar charts are provided for other key gauges in **Appendix B** (Figures B7 to B10).
- **Figure 9** and **Figure 10** show the hydraulic model results in the Muswellbrook and Denman areas respectively. Peak gauged and simulation levels are noted at gauge locations.

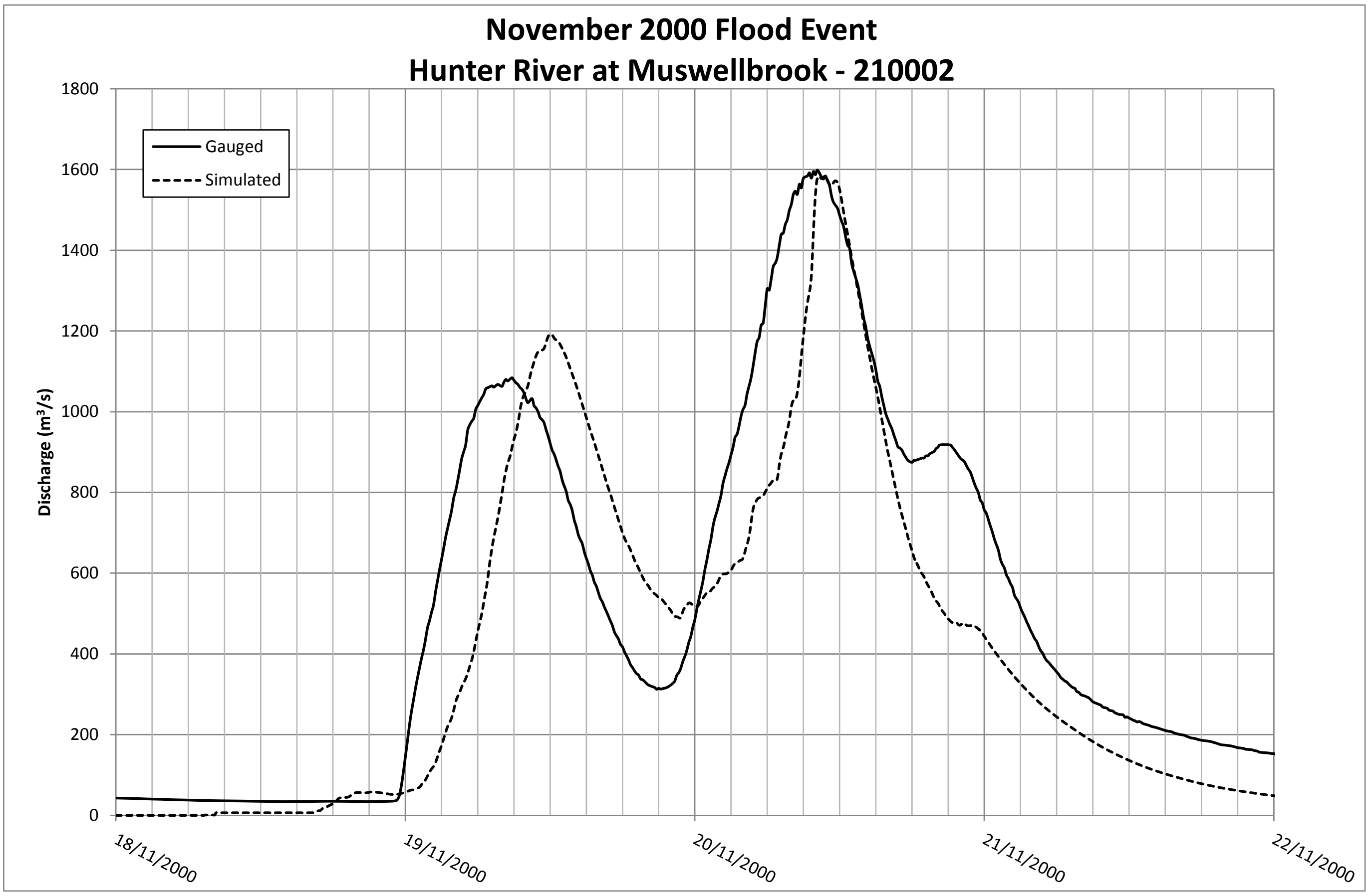


Figure 8 – *2000 Event Calibration Results: Muswellbrook Bridge Gauge*

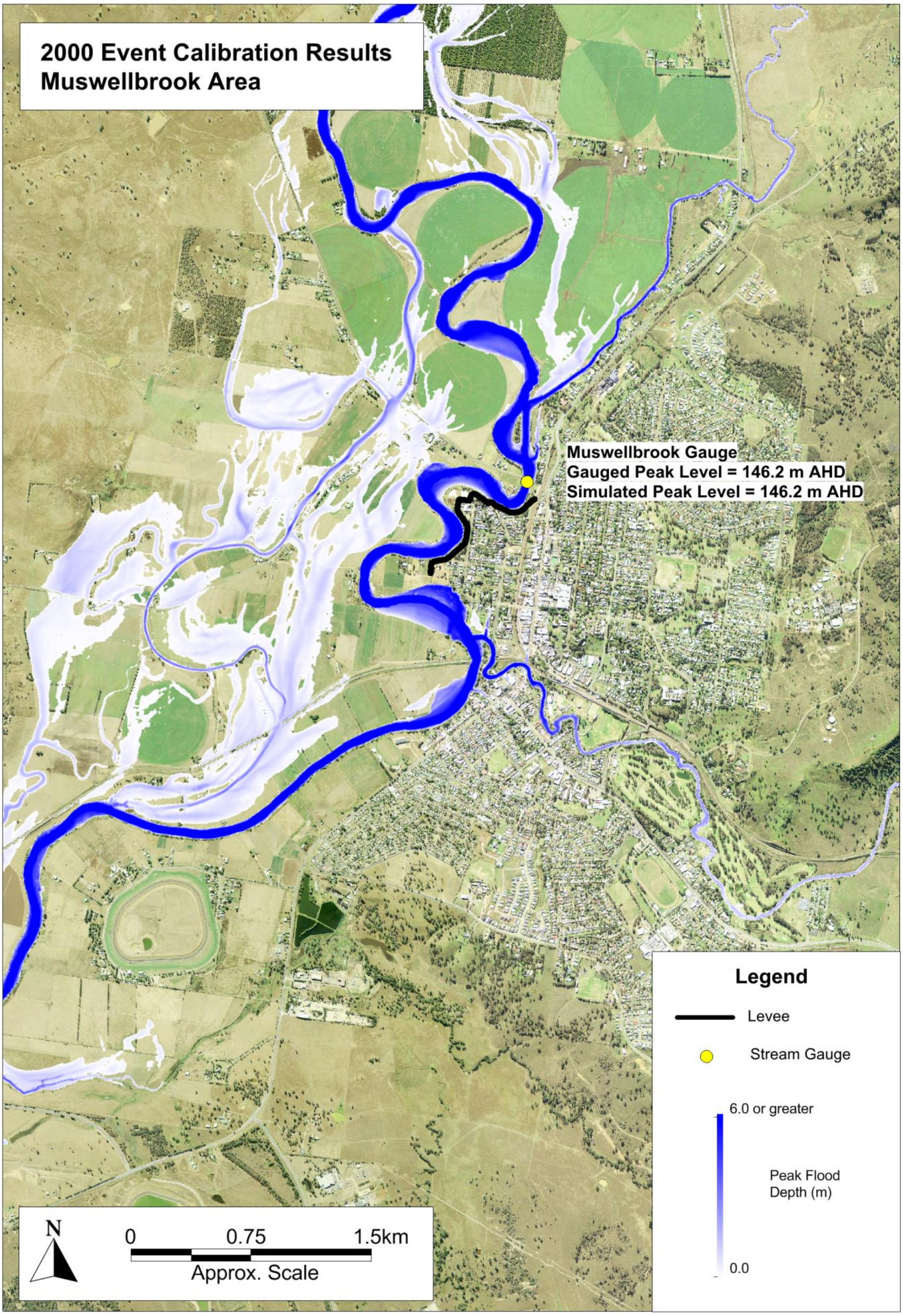


Figure 9 – *2000 Event Calibration Results: Muswellbrook Area*

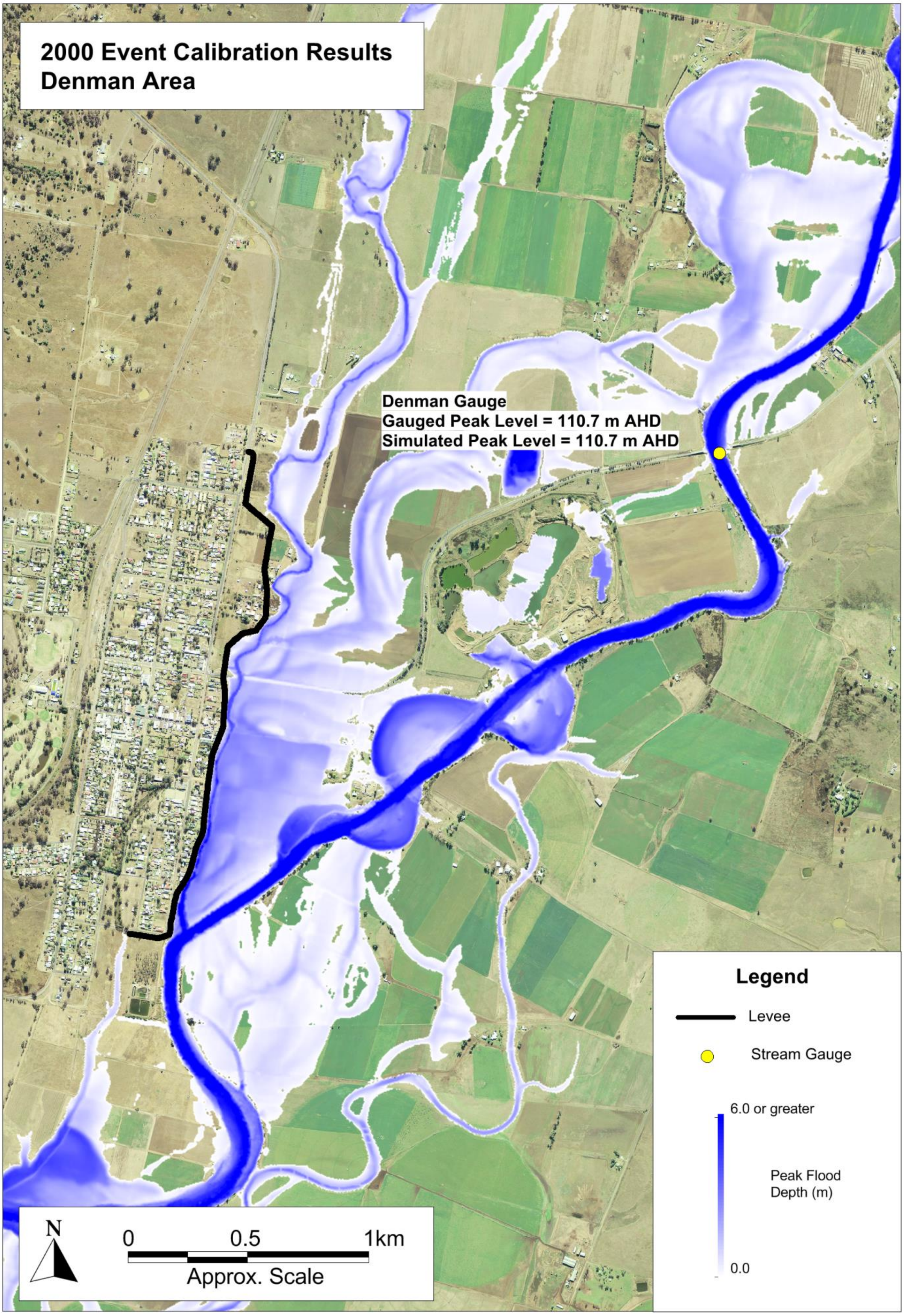


Figure 10 – *2000 Event Calibration Results: Denman Area*

1998 Event Results

Calibration results for the 1998 event are provided in the following figures:

- **Figure 11** compares the simulated and gauged hydrographs at the Muswellbrook Bridge gauge location. Similar charts are provided for other key gauges in **Appendix B** (Figures B11 to B15).

- **Figure 12** and **Figure 13** show the hydraulic model results in the Muswellbrook and Denman areas respectively. Peak gauged and simulation levels are noted at gauge locations.

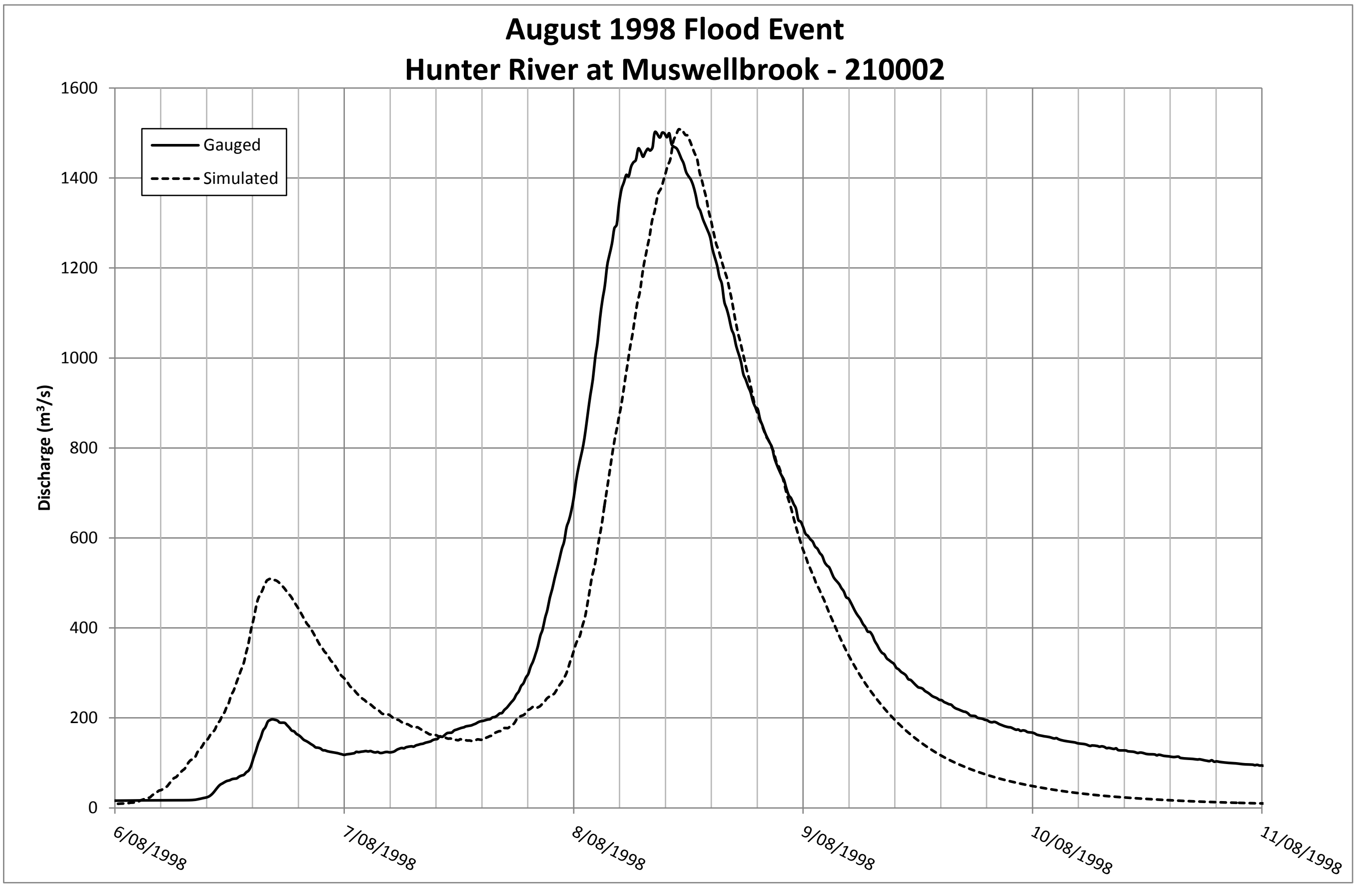


Figure 11 – *1998 Event Calibration Results: Muswellbrook Bridge Gauge*

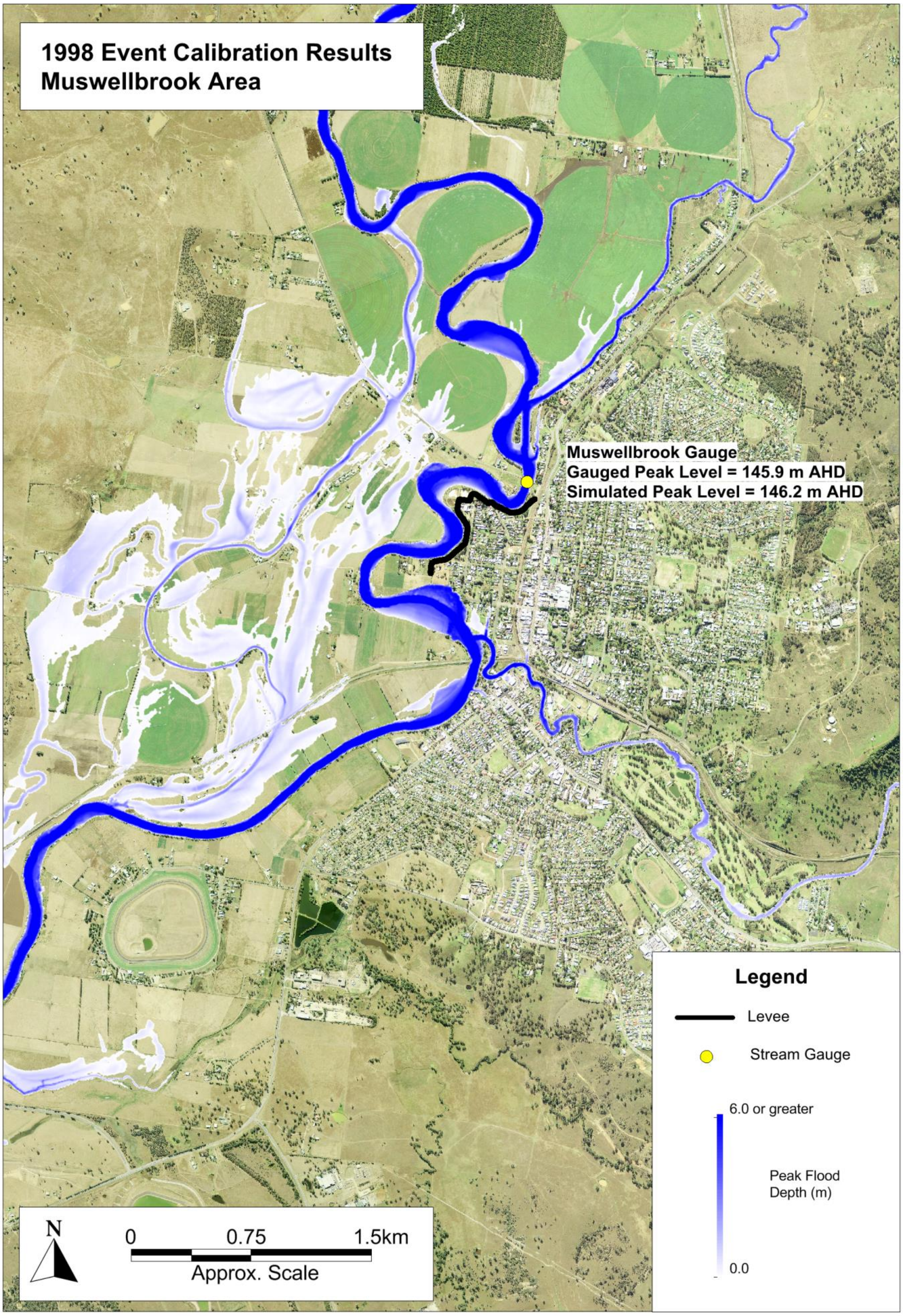


Figure 12 – *1998 Event Calibration Results: Muswellbrook Area*

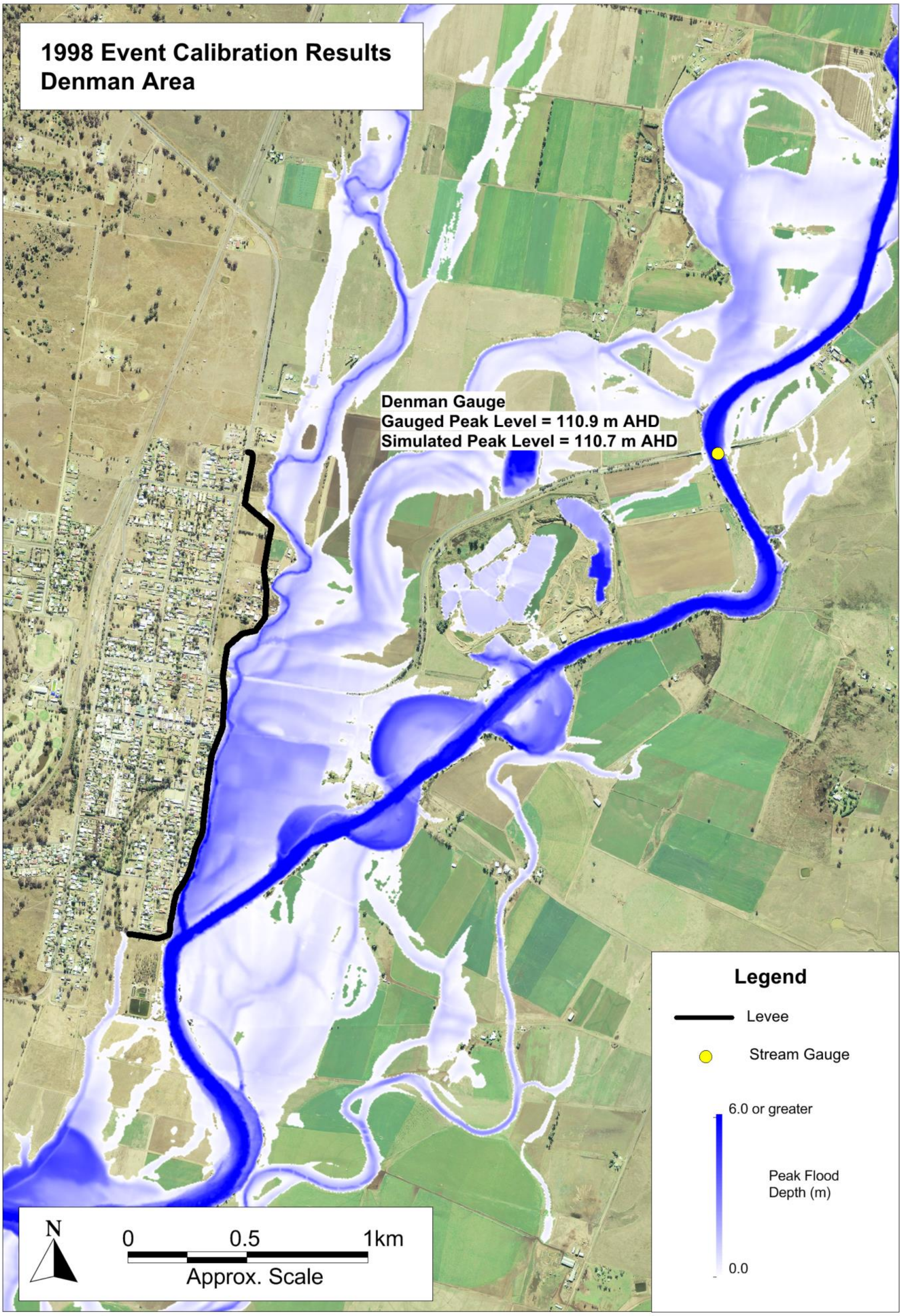


Figure 13 – 1998 Event Calibration Results: Denman Area

Results Discussion

The following conclusions can be made from the calibration results presented in this section:

- For both events, the hydrologic model reproduced runoff hydrographs that have similar shape, peak and timing to the gauged hydrographs at all Hunter River gauges. This indicates that the hydrologic model is reliably parametrised.

- With reference to **Figure 9** and **Figure 10**, the hydraulic model reproduced the gauged peak flood levels at both the Muswellbrook and Denman gauges.

- With reference to **Figure 12** and **Figure 13**, the hydraulic model simulated a peak flood level that was 300mm higher at the Muswellbrook Gauge and 200mm lower at the Denman Gauge for the 1990 event. It is noted that the total peak flow simulated by the hydrologic model at Muswellbrook (**Figure 11**) and Denman (**Figure B15**) were close to the gauged estimates. It is possible that the discrepancy in simulated and gauged levels in the hydraulic model is due to the runoff from Sandy Creek (the tributary that joins the Hunter River upstream of the Muswellbrook Gauge) being understated. This catchment is next to the Rouchel Brook Catchment which received the most intense rainfall in the 1998 event. Higher runoff from this tributary would have locally increased channel flows as floodplain flows break out from the channel approximately 3km upstream of the Sandy Creek confluence (as shown in **Figure 12**). However, there is no data to support this hypothesis so no changes to the rainfall assumptions were made.

4.3 Model Revision and Re-Calibration Outcomes

The model revision and re-calibration process has achieved the following outcomes:

- Verified the changes made to the hydraulic model.

- Demonstrated that the hydrologic modelling approach can simulate runoff hydrographs that are similar to gauged hydrographs.

The calibration revision process did not establish justification to increase catchment lags, as was adopted by Worley Parsons in the Flood Study design event simulations.

The calibrated model was applied to simulate design storm events. This is discussed further in **Section 6**.

5 FLOOD FREQUENCY ANALYSIS

Flood Frequency Analysis (FFA) applies observed annual peak discharge data to calculate the AEP of a given discharge. This analysis assumes that previous floods will occur at the same frequency in the future and that the flood record is an accurate representation of the catchment's flood behaviour.

This section documents the FFA that has been undertaken using available stream gauge data at Muswellbrook and is structured as follows:

- **Section 5.1** presents annual series data that has been extracted from the stream gauge data.
- **Section 5.2** discusses the FFA approach.
- **Section 5.3** discusses the FFA methodology and results.

5.1 Annual Series Data

Section 2.1 established that by merging the stream flow data from the Muswellbrook Bridge (210002) and Muswellbrook Weir (210008) gauges a continuous flow record for the 1907 – 2016 period can be established. Flow data over this 109 year period is not homogenous as there have been numerous changes in the catchment. The most significant has been the construction and upgrade of Glenbawn Dam, although changes in land use and vegetation density are also likely to be important factors.

The flow data has been separated into the following three distinct periods for use in FFA calculations:

- Pre-Glenbawn Dam (1907 to 1955)
- Post Glenbawn Dam (1956 to 1986)
- Post Glenbawn Dam Upgrade (1986 to 2016)

These datasets are presented in the following sections.

5.1.1 Pre-Gauging Flood Events

The Muswellbrook Flood Study (1986) analysed anecdotal data from floods that occurred prior to the commencement of the gauge record in 1907. This data included local flood marks and flood levels marked on bridge plans. These flood levels were converted to flows based on a rating curve derived by WRC during the study (see **Section 2.1.2**).

***Table 2**: Historic Flood Events Data set (Source WRC, 1986)*

Year	Flow (m^3/s)
1864	3962
1867	1912
1870	**5912**
1893	3110

The largest flood event to occur at Muswellbrook was the 1870 event with an estimated peak flow of 5,912 m^3/s. Other notable floods occurred in 1864 and 1893.

5.1.2 Pre-Glenbawn Dam Data

Flow data is available for the period between 1907 and 1955, prior to the construction of Glenbawn Dam. This data is made up from:

- Gauged data from the Muswellbrook gauge (210002) for the 1913 to 1927 period.
- Data from the Muswellbrook Weir gauge for the 1928 to 1955 period. As discussed in **Section 2.1 ,** WRC (1986) correlated the peak annual flows from this gauge to the Muswellbrook gauge (210002) location.

Table 3 presents annual maximum series of peak flood flows for the Pre-Dam series.

Table 3: *Pre-Glenbawn Dam Data Set*

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1907	231	**1917**	479	**1927**	340	**1937**	141	**1947**	150
1908	755	**1918**	68	**1928**	1284	**1938**	88	**1948**	162
1909	81	**1919**	49	**1929**	1490	**1939**	73	**1949**	1373
1910	1465	**1920**	495	**1930**	1431	**1940**	309	**1950**	1693
1911	505	**1921**	1537	**1931**	1619	**1941**	122	**1951**	851
1912	197	**1922**	308	**1932**	494	**1942**	1051	**1952**	1534
1913	2156	**1923**	8	**1933**	589	**1943**	139	**1953**	373
1914	130	**1924**	125	**1934**	764	**1944**	315	**1954**	1278
1915	310	**1925**	34	**1935**	448	**1945**	1080	**1955**	*5013*
1916	440	**1926**	305	**1936**	84	**1946**	764		

The 1955 flood event was the largest flood to have occurred since the commencement of stream gauging at Muswellbrook.

5.1.3 Post-Glenbawn Dam Data

Flow data is available for the period between 1956 and 1986, after the construction of Glenbawn Dam and before the dam's upgrade in 1987. The Annual Post-Dam series was made up of gauge data available from:

- Data from the Muswellbrook Weir gauge for the 1956 to 1960 period. As discussed in **Section 2.1 ,** WRC (1986) correlated the peak annual flows from this gauge to the Muswellbrook gauge (210002) location.
- Gauged data from the Muswellbrook gauge (210002) for the 1961 to 1986 period.

Table 4 presents annual maximum series of peak flood flows for the Post - Dam series.

Table 4: *Post-Glenbawn Dam Data Set*

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1956	704	**1967**	394	**1978**	865
1957	508	**1968**	701	**1979**	255
1958	494	**1969**	383	**1980**	8
1959	146	**1970**	313	**1981**	86
1960	300	***1971***	*3207*	**1982**	77
1961	93	**1972**	232	**1983**	165
1962	874	**1973**	117	**1984**	1153
1963	385	**1974**	327	**1985**	237
1964	542	**1975**	136	**1986**	57
1965	28	**1976**	2109		
1966	28	**1977**	679		

5.1.4 Post-Glenbawn Dam Upgrade Data

Data available for the 1987 – 2016 period implicitly incorporates the effects of the Glenbawn Dam upgrade. This data was obtained from the Muswellbrook Gauge (no. 210002). **Table 5** presents annual maximum series of peak flood flows for the Post - Dam series.

Table 5: *Post-Glenbawn Dam Upgrade Data Set*

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1987	183	**1997**	120	**2007**	256
1988	139	**1998**	1502	**2008**	245
1989	546	**1999**	227	**2009**	77
1990	808	**2000**	1598	**2010**	197
1991	107	**2001**	237	**2011**	424
1992	2144	**2002**	87	**2012**	195
1993	217	**2003**	117	**2013**	259
1994	72	**2004**	182	**2014**	20
1995	321	**2005**	52	**2015**	82
1996	999	**2006**	12	**2016**	182

5.2 Flood Frequency Analysis Approach

The following six step process was applied to complete the FFA:

- **Step 1** - Assess the hypothesis from the 1986 Flood Study that the Post-Glenbawn Dam and Post-Glenbawn Dam Upgrade series are homogenous;
- **Step 2** - Undertake FFA on the Post Glenbawn Dam data set (1956 to 2016);
- **Step 3** - Undertake FFA on the Pre-Glenbawn Dam data set (1907 to 1955);
- **Step 4** - Compare Pre and Post Glenbawn Dam FFA results;
- **Step 5** – Apply Bayesian Methods to incorporate Pre-Glenbawn Dam data and historical flood events into the post dam FFA; and
- **Step 6** – Undertake the final FFA.

5.3 Flood Frequency Analysis Methodology and Results

5.3.1 Methodology

The FFA was undertaken using the Flike software package (version 5.0.251.0), using the annual maximum method. This method applies the highest recorded discharge for each year of record to the FFA. This method prevents the inclusion of successive dependent peaks. A Bayesian maximum likelihood approach was used to fit a specified probability distribution for each data set. This analysis used a Log-Pearson III (LP3) distribution.

5.3.2 Step 1 - Proof of Homogeny of Post-Glenbawn Dam and Post-Glenbawn Dam Upgrade Annual Series

The Muswellbrook Flood Study (1986) examined a study performed by Hayes (1982) which analysed the impact of Glenbawn Dam on floods at Muswellbrook. The study found that the original and upgraded dams have effectively the same mitigation effect. The upgraded dam was increased in capacity; however the available flood mitigation storage was reduced leading to a negligible net difference in flood mitigation properties. The current study sought to investigate this hypothesis via statistical analysis.

Statistical analysis using the t-test and the Mann-Whitney U-test was undertaken on the post-dam and post upgrade data sets. The t-test and the Mann-Whitney U-test analyse the mean and median of each of these data sets. The results of these tests showed that the impact of the dam on the two data sets is not statistically significant ($p>0.05$).

This analysis verified that the Post Glenbawn Dam and Post Glenbawn Dam Upgrades were statistically similar. Accordingly, it was considered appropriate to merge the two data sets to form a single post dam annual series for the 1956 to 2016 period.

5.3.3 Step 2 - Post Glenbawn Dam FFA (1956 to 2016)

Based on the results from the above analysis, FFA was conducted on the merged Post-Glenbawn Dam series (1956 to 2016). The results are shown on **Figure 14.** It is noted that the 1971 event is calculated as being a 1% AEP event.

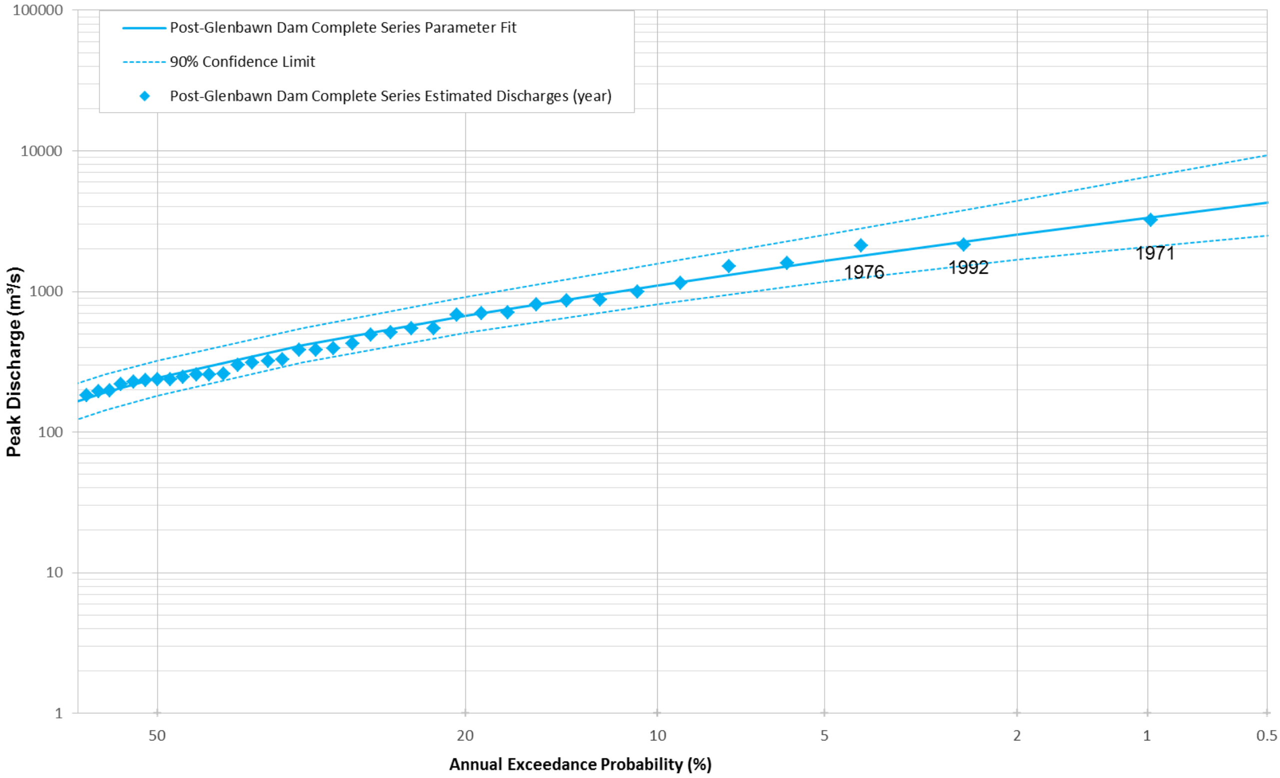


Figure 14: *Post-Glenbawn Dam complete annual series Flood Frequency Analysis*

5.3.4 Step 3 - Pre-Glenbawn Dam FFA (1907 to 1955)

FFA was undertaken on the Pre-Glenbawn Dam annual series data set. These results are shown in **Figure 15**. It is noted that the 1955 event is calculated as being a 1.5% AEP (or 1 in 75 year) event.

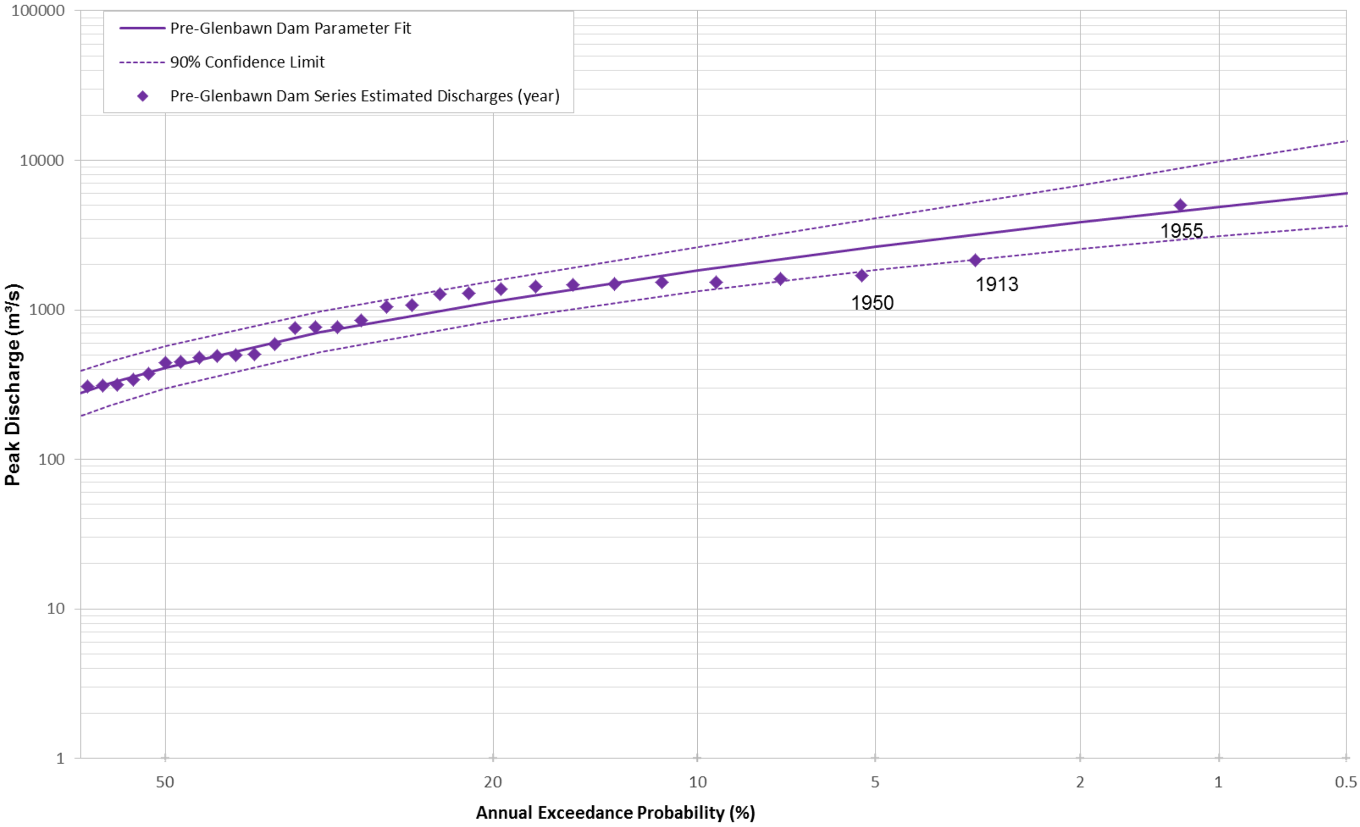


Figure 15: *Pre-Glenbawn Dam Flood Frequency Analysis (1907 to 1955)*

5.3.5 Step 4 - Comparison between Pre and Post Glenbawn Dam FFA

Figure 16 compares the Post-Glenbawn Dam and Pre-Glenbawn Dam FFAs. The attenuation of peak discharges provided by Glenbawn Dam result in lower peak discharge estimates for the Post-Glenbawn Dam complete series (as expected).

The Pre-dam annual discharges were 'transformed' to equivalent post-dam flows so that they could be incorporated using Bayesian Methods (see Section 5.3.6). Comparison of the results in **Figure 16** indicate that the 1955 flood event would have exceeded the peak discharge in the 1971 flood event (3207 m^3/s) had Glenbawn Dam been constructed at the time of the event. This is shown using the black arrows in **Figure 16**.

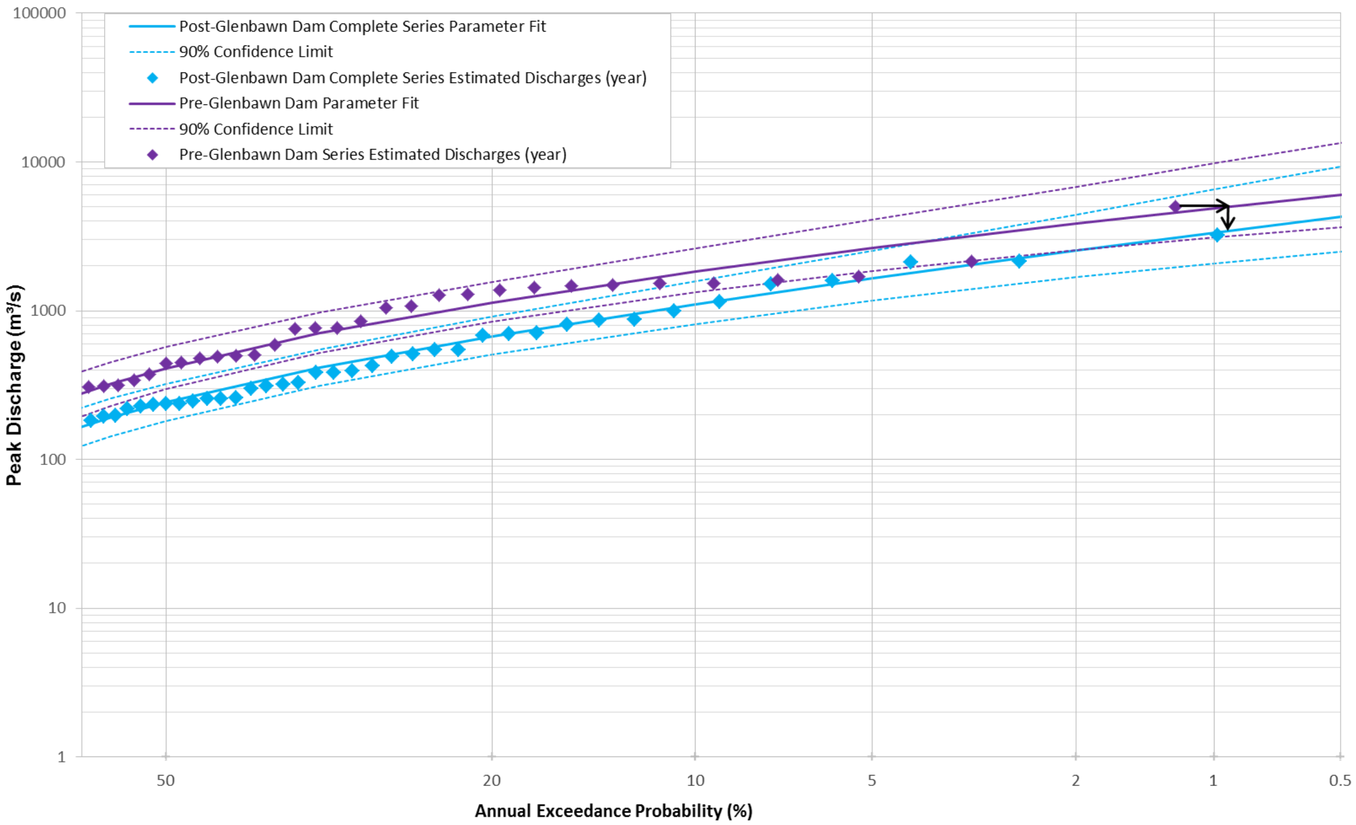


Figure 16: *Post-Glenbawn Dam and Pre-Glenbawn Dam Flood Frequency Analysis*

5.3.6 Step 5 - Use of Bayesian Methods

Historic flood events of an unknown magnitude can be incorporated into the FFA using Bayesian Methods. This process allows flood events of an unknown magnitude to be included above and below a certain threshold. Since the 1971 event was the largest flood event in the Post Glenbawn Dam record, this event was selected as the threshold (3207 m^3/s).

It was assumed that the following floods would have exceeded the peak discharge in the 1971 flood event (3207 m^3/s) if they were to occur under current catchment and floodplain conditions (i.e. with Glenbawn Dam constructed):

- **1870 flood event** - A review of the historical flood information provided from previous reports (see Section 2.1) indicates that the 1870 flood event was the largest flood event to have occurred since European settlement at Muswellbrook in the 1820s.

- **1955 flood event –** is generally accepted to have been a larger flood than the 1971 event.

Bayesian Methods were applied to incorporate the years between European settlement and the dam construction in 1956 into the final FFA. The significant flood events of 1955 and 1870 were incorporated above the 1971 event flow threshold (3207 m^3/s) and the remaining years were assumed to be below this threshold.

5.3.7 Step 6 - Final FFA Results

The Final FFA was used to derive design flow estimates at Muswellbrook and was calculated as follows:

- The homogenous Post-Glenbawn Dam data series (1956 to 2016) was incorporated into the FFA (see Section 5.3.3); and

- The years prior to the Glenbawn Dam construction were included using Bayesian methods (see Section 5.3.6).

The design FFA plot at Muswellbrook is displayed in **Figure 17** and the design flows tabulated in **Table 6**. This analysis produced a 1% AEP flow of 3,583 m^3/s which is slightly larger than the 1971 flood event (3,207 m^3/s). **Table 6** and **Figure 17** also present the flows derived in the hydrologic model, which are discussed in **Section 6**.

Table 6*: Flood Frequency Analysis: Design Flows at the Muswellbrook Gauge*

Event (AEP)	Flow (m^3/s)	90% Confidence Limits		Hydrologic Model Flows (m^3/s)
		Lower Flow (m^3/s)	Upper Flow (m^3/s)	
0.2 EY	**680**	524	888	**640* (20% AEP)**
10%	**1137**	877	1479	**1080**
5%	**1714**	1297	2295	**1650**
2%	**2682**	1954	3861	**2900**
1%	**3583**	2493	5571	**3510**
0.5%	**4643**	3056	7884	**4070**
0.2%	**6308**	3825	12106	**4860**

*Note: 0.2 EY has a slightly different probability of occurrence to the 20% AEP, equivalent to 18.13% AEP

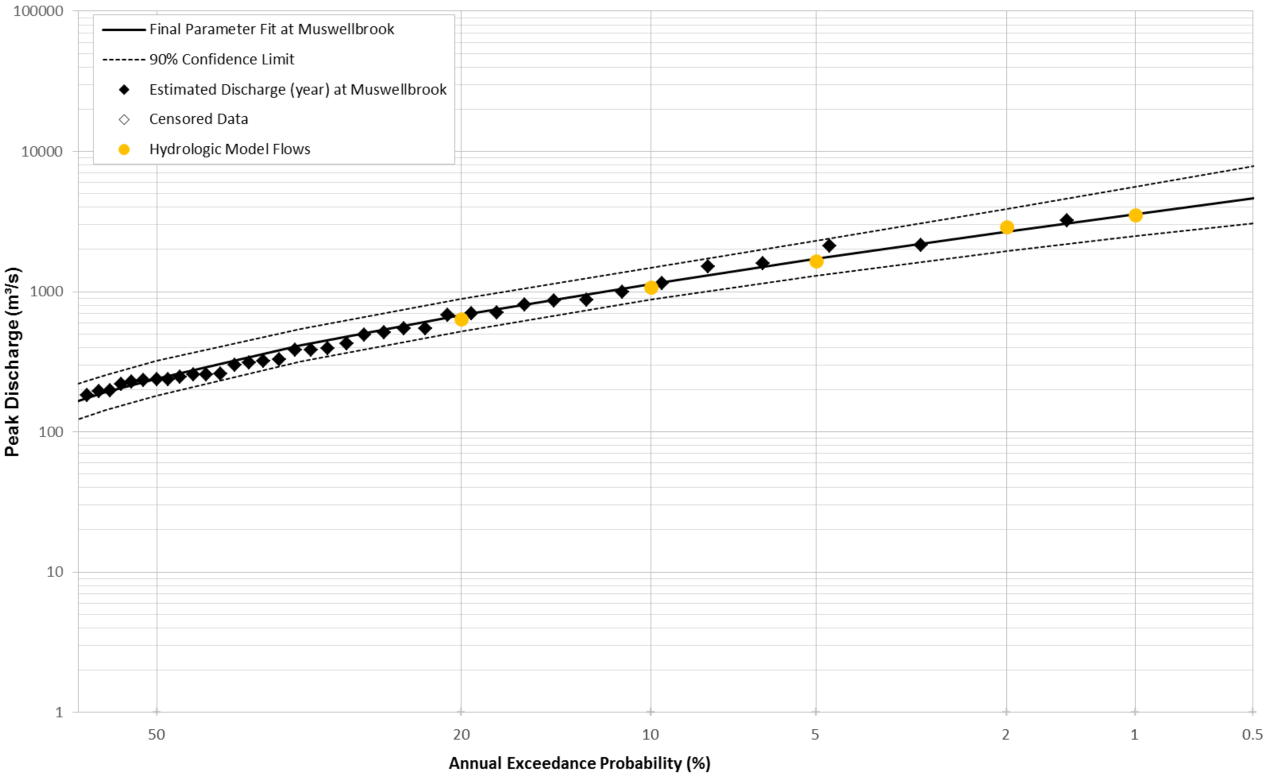


Figure 17: *Final Muswellbrook Flood Frequency Analysis with Hydrologic Model Flows*

6 REVISED DESIGN EVENT RESULTS

The revised hydraulic and hydrologic models that are described in **Section 4** were applied to simulate design storm events using the methods recommended in the Australian Rainfall and Runoff Guidelines (Commonwealth of Australia, 2016). This guideline is referred to as ARR 2016 in the remainder of this section.

This section documents the revised design event simulation methodologies, assumptions and results and is structured as follows:

- **Section 6.1** documents the revised hydrologic modelling that was undertaking using ARR 2016 methods.
- **Section 6.2** compares the hydrologic results produced using the ARR 1987 and ARR 2016 methods.
- **Section 6.3** documents the revised hydraulic model results.
- **Section 6.4** discusses changes to the Flood Study (Worley Parsons, 2014) results.

6.1 Hydrologic Modelling of ARR16 Design Events

Hydrologic modelling has been undertaken in accordance with ARR 2016 using the XP-RAFTS model of the Hunter River catchment which was revised during the model calibration process that is described in **Section 4**. A range of design events between the 20% and 0.2% AEP were simulated.

This section describes the methodologies and assumptions applied to simulating the design events. Results are also discussed.

6.1.1 ARR 2016 - Design Rainfall

The Bureau of Meteorology (BoM) revised Intensity-Frequency Duration (IFD) rainfall depths as part of the ARR 2016 program. Design rainfall data provided by the BoM is an important input into a hydrologic model to determine flows for design storm events. This data was obtained for a range of storm events (both AEP and duration) in gridded format.

In large catchments with great changes in elevation, such as in the Hunter River Catchment, it is common for IFD depths to vary significantly across the catchment. As such, the average design rainfall depth for each sub-catchment was extracted from the gridded data provided by the BoM and input into the hydrologic model on a sub-catchment by sub-catchment basis.

6.1.2 Design Temporal Patterns

ARR 2016 recommends undertaking hydrologic modelling using an “ensemble” of ten storm temporal patterns. These ensembles account for the variability of temporal patterns that can occur in events of similar magnitudes. In the analysis of the resulting flows, ARR 2016 recommends selecting the temporal pattern that produces the peak flow just above the mean peak flow (i.e. the 6th highest peak flow). For the Hunter River catchment, an ensemble of “East Coast South” areal temporal patterns were applied to all design rainfall simulations.

6.1.3 Design Loss Parameters

ARR 2016 recommends using catchment specific loss parameters from calibrated hydrologic models if they are available. Otherwise, ARR 2016 provides recommended initial, continuing and pre-burst losses for ungauged catchments. For the Hunter River Catchment, ARR 2016 recommends an initial loss of 44 mm and continuing loss of 3.1 mm/hr.

The current study selected design continuing loss parameters based on the model calibration process. As discussed in **Section 4**, the calibration process applied a continuing loss of 1.5 mm/hr for both the 1998 and 2000 event simulations.

The initial losses were determined based on the design flow estimates from the FFA and the recommended initial and pre-burst losses from ARR 2016. For frequent flood events, it was found that an initial loss of 55 mm produces a hydrologic model flow that matches the flows derived in the FFA. For more rare events, the initial and pre-burst losses recommended in ARR 2016 were found to match the FFA flows.

Table 7 summarises the loss parameters adopted in the hydrologic model.

Table 7: *Hydrologic Model Losses*

Event (AEP)	Continuing Loss (mm/hr)	Pre-Burst Loss (mm)	Initial Loss adopted (mm)
20%	1.5	-	55
10%	1.5	-	55
5%	1.5	-	55
2%	1.5	*7.8*	36.2*
1%	1.5	*10.6*	33.4*
0.2%	1.5	*10.6^*	33.4*
0.5%	1.5	*10.6^*	33.4*

* Note: ARR 2016 Initial loss equals recommended initial loss (44 mm) minus pre-burst loss
^ Note: Preburst losses are not provided for events greater than the 1% AEP

6.1.4 Areal Reduction Factors

Areal Reduction Factors (ARF) are used to account for the spatial variation of design rainfall data which relates to a specific point in a catchment rather than to the entire catchment area. ARR 2016 recommends using the following equations for the South East Coast Region, where the Hunter River catchment is located.

Equation 1: Short duration ARF equation (less than and equal to 12 hours)

$$ARF = Min[1, 1 - 0.287(Area^{0.265} - 0.439 log_{10}(Duration)).Duration^{-0.36} + 2.26 \times 10^{-3} \times Area^{0.226}.Duration^{0.125}(0.3 + log_{10}(AEP)) + 0.0141 \times Area^{0.213} \times 10^{-0.021\frac{(Duration-180)^2}{1140}}(0.3 + log_{10}(AEP))]$$

Equation 2: Equation for durations between 12 hours and 24 hours

$$ARF = ARF_{12\,hour} + (ARF_{24\,hour} - ARF_{12\,hour})\frac{(Duration - 720)}{720}$$

Equation 3: Long duration ARF equation (greater than 24 hours to 168 hours)

$$ARF = Min\{1, [1 - a(Area^{b} - c\log_{10}Duration)Duration^{-d} + eArea^{f}Duration^{g}(0.3 + \log_{10}AEP) + h10^{iArea\frac{Duration}{1440}}(0.3 + \log_{10}AEP)]\}$$

Where:
Duration = storm duration (minutes)
Area = area of interest (km²)
AEP = Annual exceedance probability as a fraction (between 0.5 and 0.0005).

Table 8: *Parameters for ARF long duration equation (Equation 3)*

Region	a	b	c	d	e	f	g	h	i
South – East Coast	0.06	0.361	0	0.317	8.11E-05	0.651	0	0	0

The equations above were used to calculate the ARF for the hydrologic modelling undertaken in the current study. By way of example, an ARF of 0.85 was calculated for the 1% AEP design event.

6.1.5 Critical Duration Assessment

For all AEP event simulations, a critical duration assessment was carried out for flows at the Muswellbrook Gauge to determine which storm duration produces the highest flows in the Muswellbrook area. The flow hydrographs for the 1% AEP event of varying durations at the Muswellbrook gauge are shown in **Figure 18**. The 24 hour duration event was found to be critical along the Hunter River at the Muswellbrook Gauge.

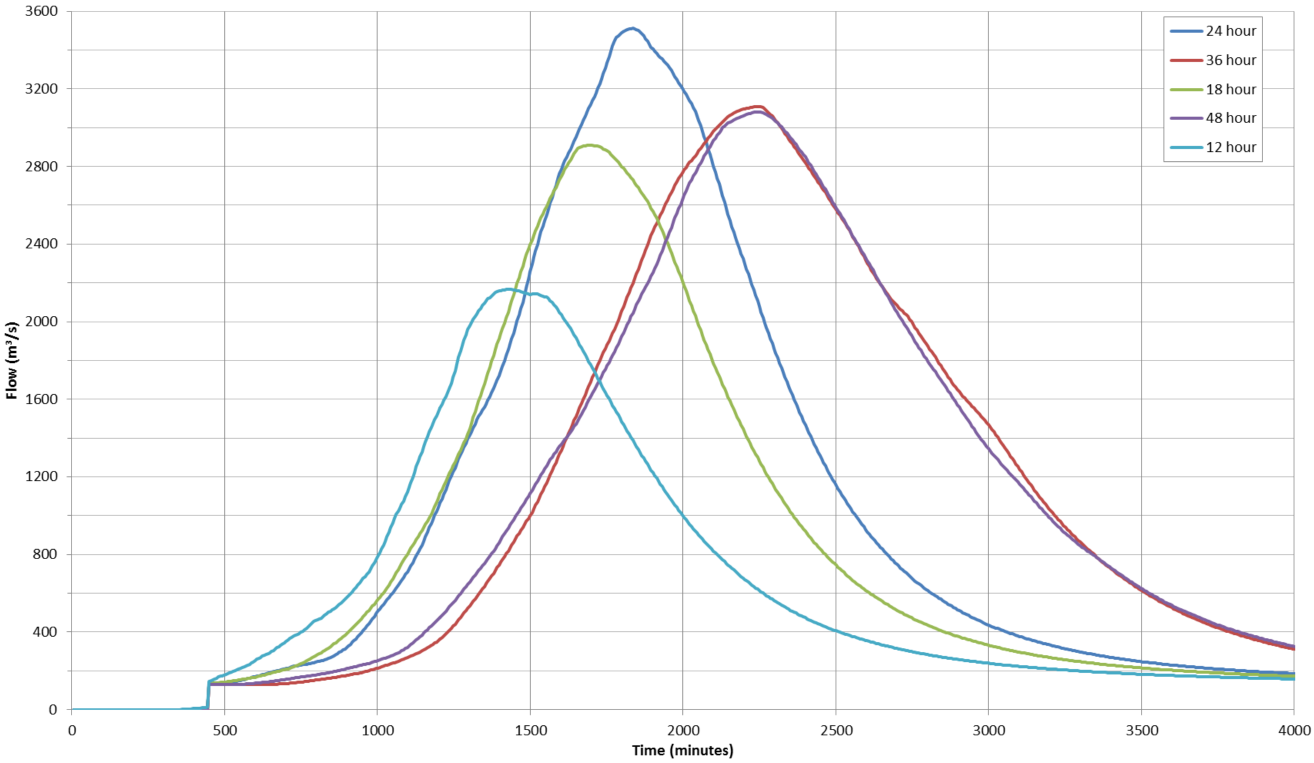


Figure 18: *Muswellbrook Gauge, Hunter River – Critical Duration -1% AEP Flow Hydrographs*

In contrast, the 2014 Flood Study found that the 48 hour and 36 hour durations were critical using the techniques recommended in ARR 1987.

6.1.6 Ensemble Storm Analysis

ARR 2016 recommends undertaking hydrologic modelling using an ensemble of ten storm temporal patterns. These ensembles account for the variability of temporal patterns that can occur in events of similar magnitudes. In the analysis of the resulting flows, ARR 2016 recommends selecting the temporal pattern that produces the peak flow just above the mean peak flow (i.e. the 6th highest peak flow). **Figure 19** shows the ten ensemble storm hydrographs at the Hunter River inflow boundary. It is noted that Hunter River inflow boundary is located upstream of the Muswellbrook Gauge. Hence, the peak flows are slightly lower than results reported at the Muswellbrook Gauge location. Storm 9 produced the 6th highest flow and was adopted for design event simulations.

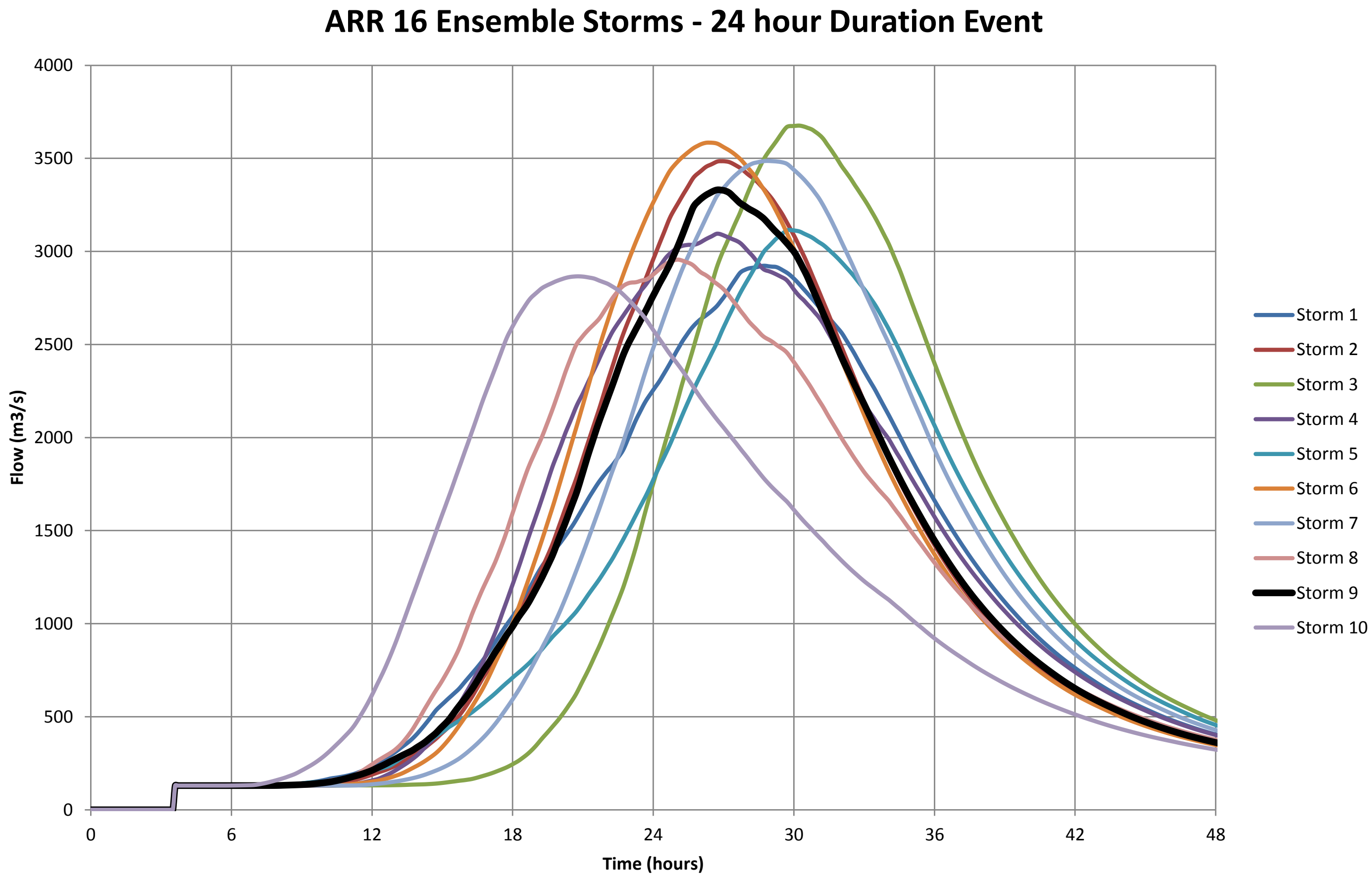


Figure 19: *1% AEP: Ensemble Storm hydrographs (Hunter River Inflow Boundary)*

6.1.7 Hydrologic Model Design Flow Results

Design flow results derived in the hydrologic model are presented in **Table 9** below. The 1971 and 1976 events are also presented to provide historical context to these revised design flow results. It is noted that the revised design peak flows are similar to the flows calculated using the FFA that are documented in **Table 6**.

Table 9: *Design and Historic Event Flows (Muswellbrook Gauge)*

	Post Glenbawn Dam Construction								
Event (AEP)	**20%**	**10%**	**5%**	**1976**	**2%**	**1971**	**1%**	**0.5%**	**0.2%**
Flow at Muswellbrook Gauge (m^3/s)	637	1076	1653	**2109**	2895	**3207**	3512	4072	4857

6.2 ARR 1987 vs ARR 2016

The revised hydrologic model (as described in **Section 4**) was applied to simulate the governing duration 1% AEP event applying both the ARR 1987 and ARR 2016 methods. For the ARR 1987 method simulation, the initial and continue losses adopted in the Flood Study (Worley Parsons, 2014) were applied. For the ARR 2016 method, the revised loss assumptions that are documented in **Section 6.1** were applied. The resulting hydrographs are provided in **Figure 20**.

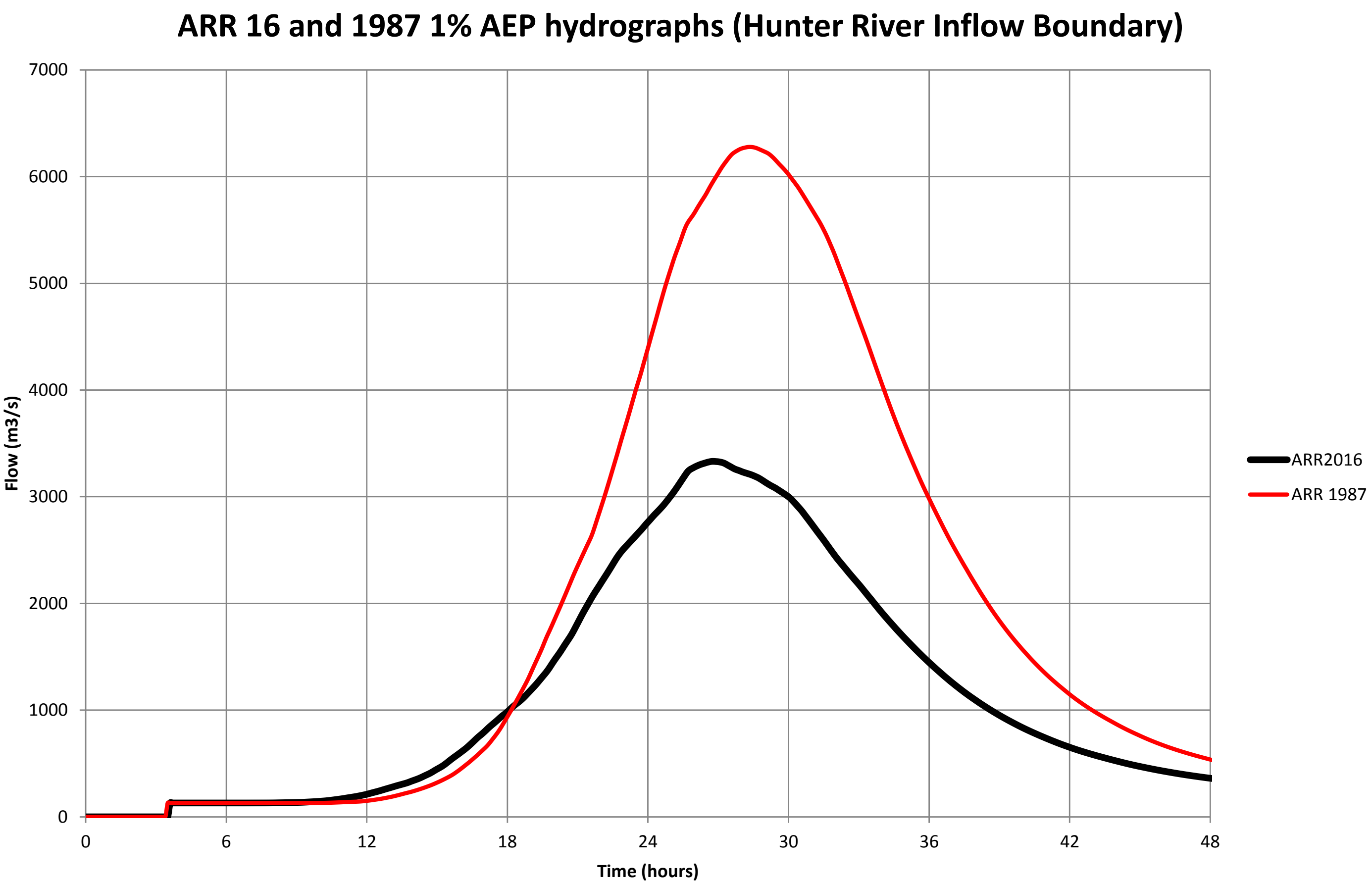


Figure 20: *1% AEP Hunter River Inflow Hydrographs: ARR1987 and ARR 2016 Methods*

The hydrographs provided in **Figure 20** show that the ARR 2016 method produces a peak flow that is substantially lower than the flow calculated using the ARR 1987 method. This is due to the ARR 2016 method producing substantially lower rainfall excess (i.e. the portion of the IFD rainfall that is converted to runoff in the model). **Table 10** provides a break-down of some of the key contributing factors at two locations within the simulated catchment area.

Table 10 – *Comparison of ARR 1987 and ARR 2016 Methods*

Catchment	**Hunter River Catchment @ Muswellbrook (HUNTER I)**						
1% AEP	Critical Duration	IFD Rainfall (mm)	Areal Reduction Factor	Initial Loss (mm)	Continuing Loss (mm/hr)	Rainfall Excess (mm)	Resulting Flow (m^3/s)
ARR 1987	36 hour	206	0.92	20.0	2.5	117	6,280
ARR 2016	24 hour	155	0.85	33.4	1.5	80	3,330
Catchment	**Pages River Headwater Catchment (P RIVER F)**						
ARR 1987	36 hour	207	0.92	20.0	2.0	132	3,140
ARR 2016	24 hour	152	0.85	33.4	1.5	82	1290

The information in **Table 10** indicates that the rainfall excess calculated using the ARR 1987 method is approximately 50% higher than the depth calculated using the ARR 2016 method. The key contributing factors to this are:

- Lower IFD depths (reduced from 206 mm to 155 mm). This is partially due to the ARR 1987 method having a longer critical duration event; and
- Lower ARF (reduced from 0.92 to 0.85).

It is noted that the higher initial losses applied to the ARR 2016 method simulation is approximately offset by lower continuing losses.

6.3 Revised Hydraulic Model Results

The outflows from revised design hydrologic model were applied to the revised hydraulic model that is described in **Section 4.** It is noted that the R5 roughness assumptions (refer **Table 1**) were applied to the design simulations. **Figure 21** and **Figure 22** show the revised 5 and 1% AEP design flood extents in the Muswellbrook area, while **Figure 23** and **Figure 24** show the same information for the Denman Area. Changes to the Flood Study (Worley Parsons, 2014) flood levels are discussed in **Section 6.4**.

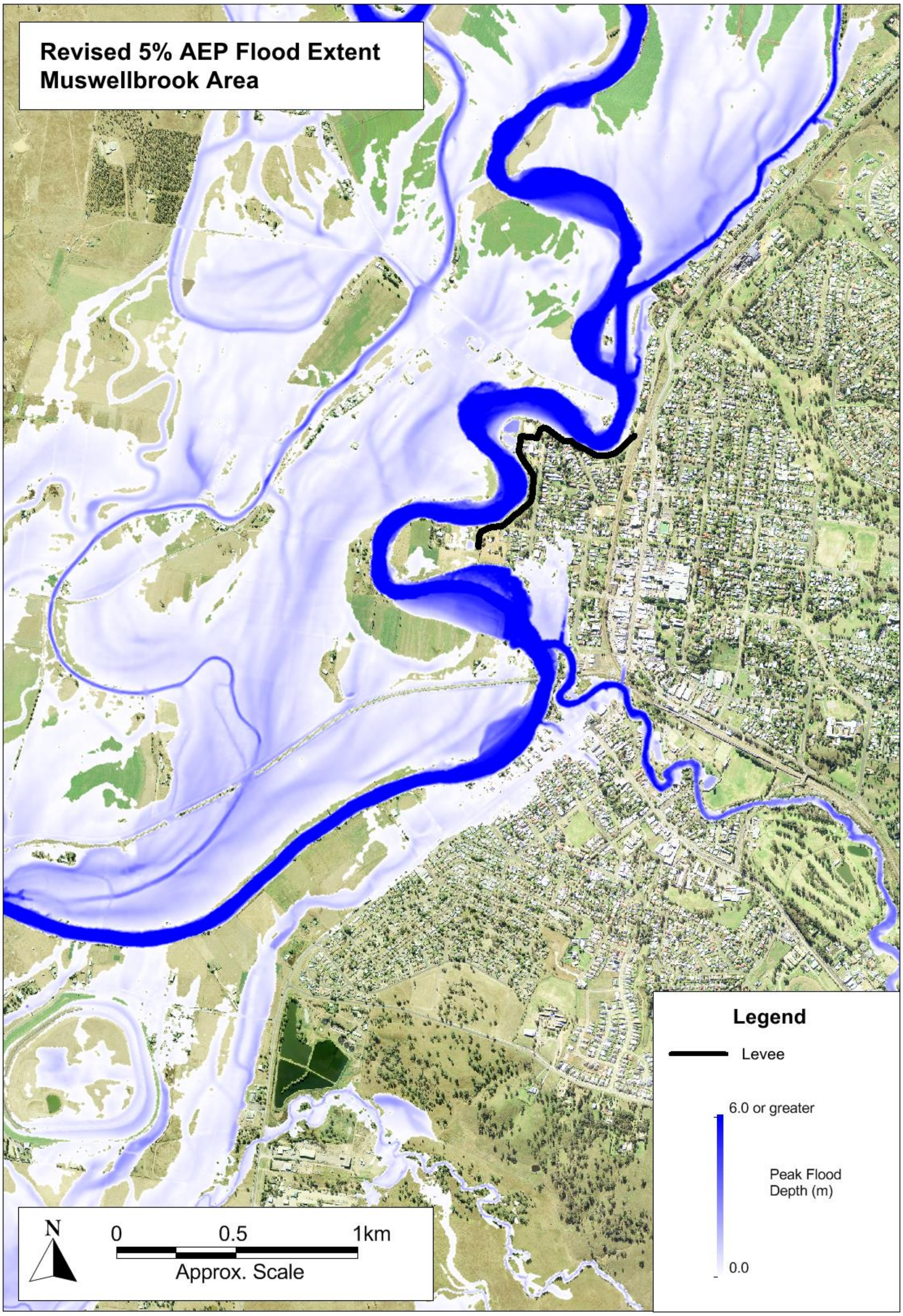


Figure 21*: Revised 5% AEP Flood Extent: Muswellbrook Area*

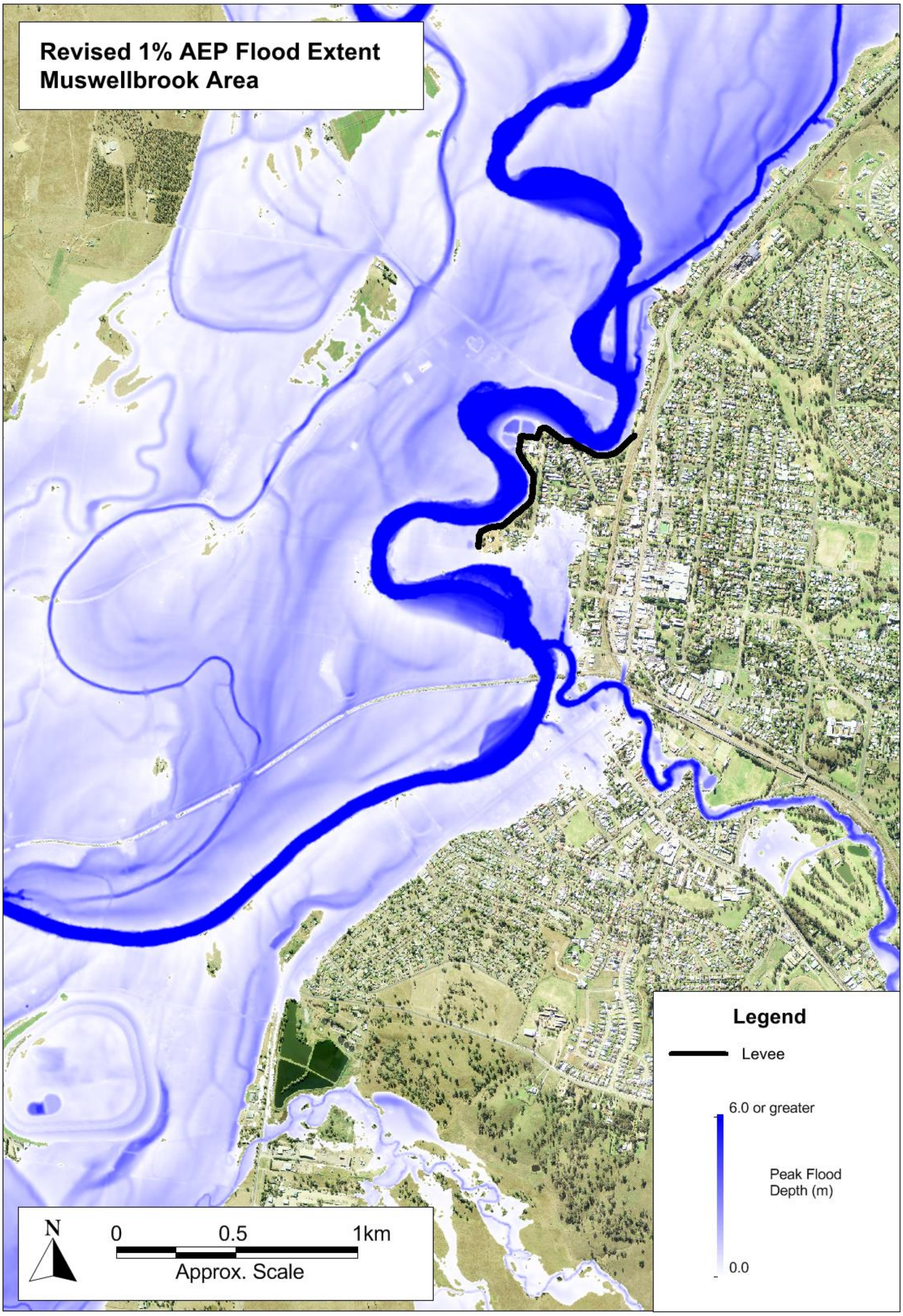


Figure 22: *Revised 1% AEP Flood Extent: Muswellbrook Area*

Figure 23: *Revised 5% AEP Flood Extent: Denman Area*

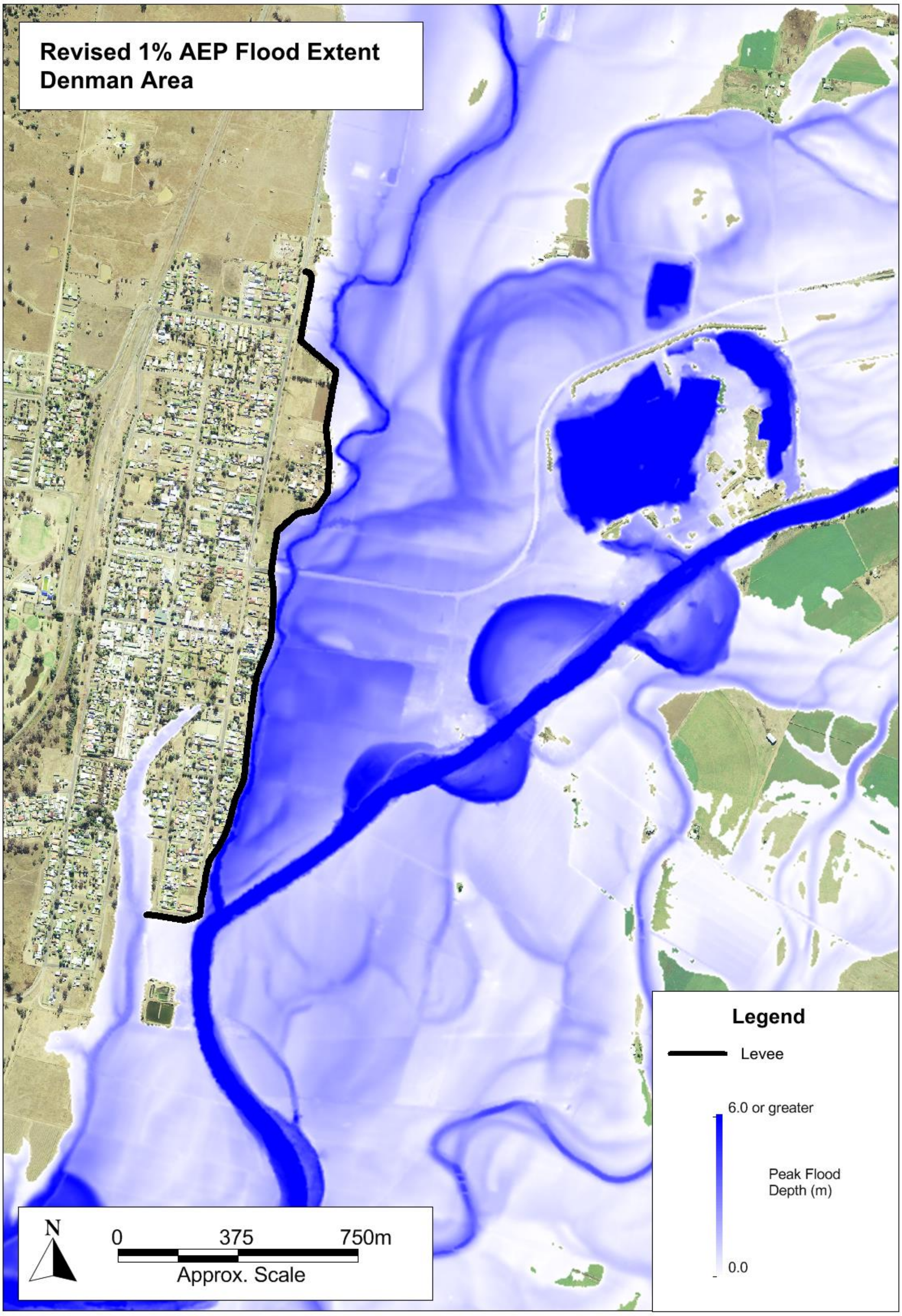


Figure 24: *Revised 1% AEP Flood Extent: Denman Area*

6.4 Changes to Flood Study Model Results

Flood level difference maps have been prepared to show the changes in peak Flood Study (Worley Parsons, 2014) design flood levels due to the various model changes that are documented in this report. **Figure 25** and Figure 26 present flood level difference maps for the Muswellbrook and Denman area respectively.

With reference to **Figure 25** and Figure 26, the various model changes that are documented in this report will result in 1% AEP flood level reductions ranging from 90 to 360 mm. Reductions in the flood affected areas of Muswellbrook are typically in the 90 to 340 mm range. Flood level reductions adjacent to the Muswellbrook Levee are in the 230 to 270 mm range, while moderately higher (140 to 350 mm) reductions are predicted adjacent to the Denman Levee.

The reduction in flood levels is due to the significant reduction in the assumed peak flows, partially offset by higher channel roughness assumptions.

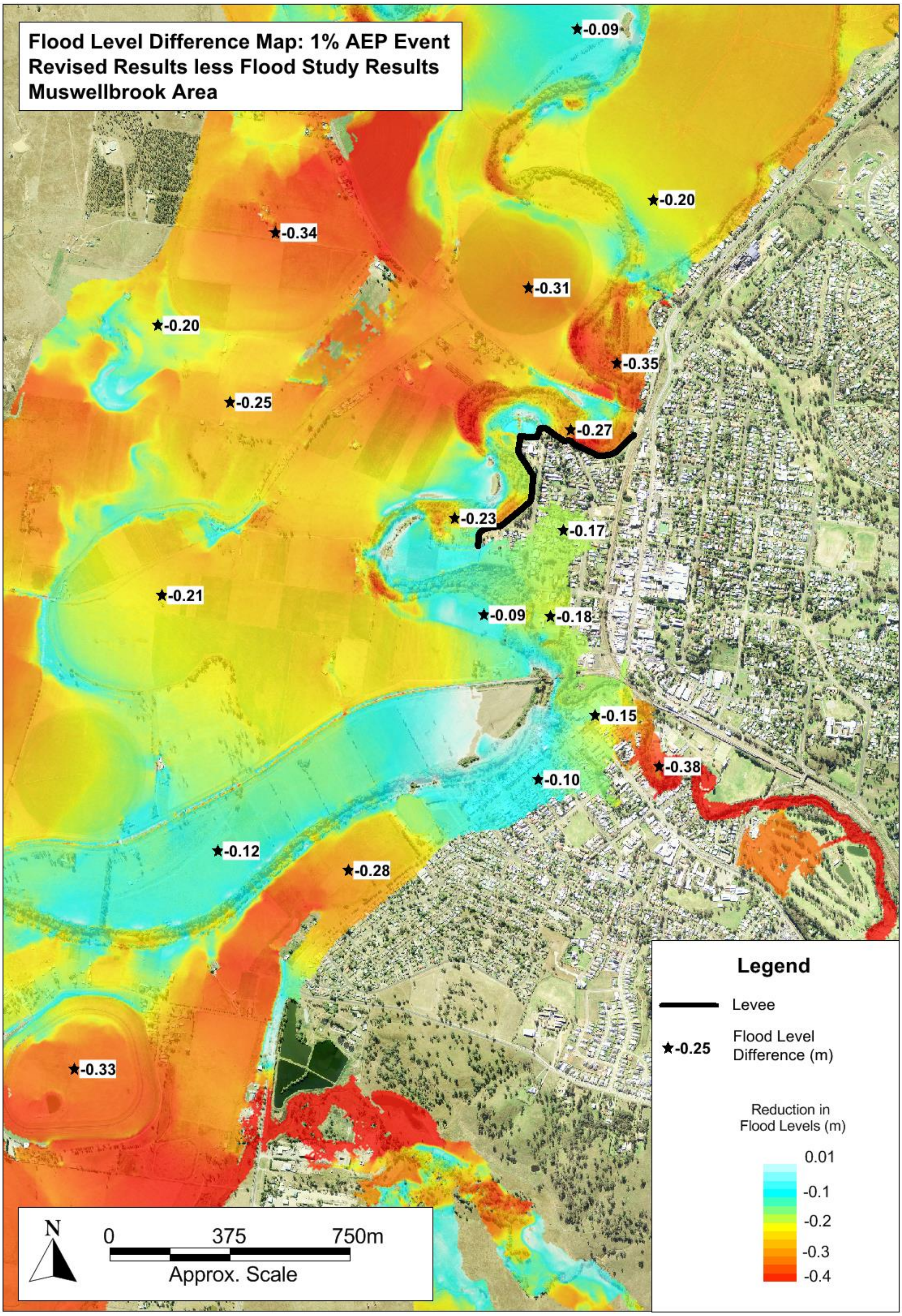


Figure 25: *Changes to 1% AEP Levels: Muswellbrook Area*

Note: a negative number represents a reduction in flood levels for the current study compared to the 2014 Flood Study.

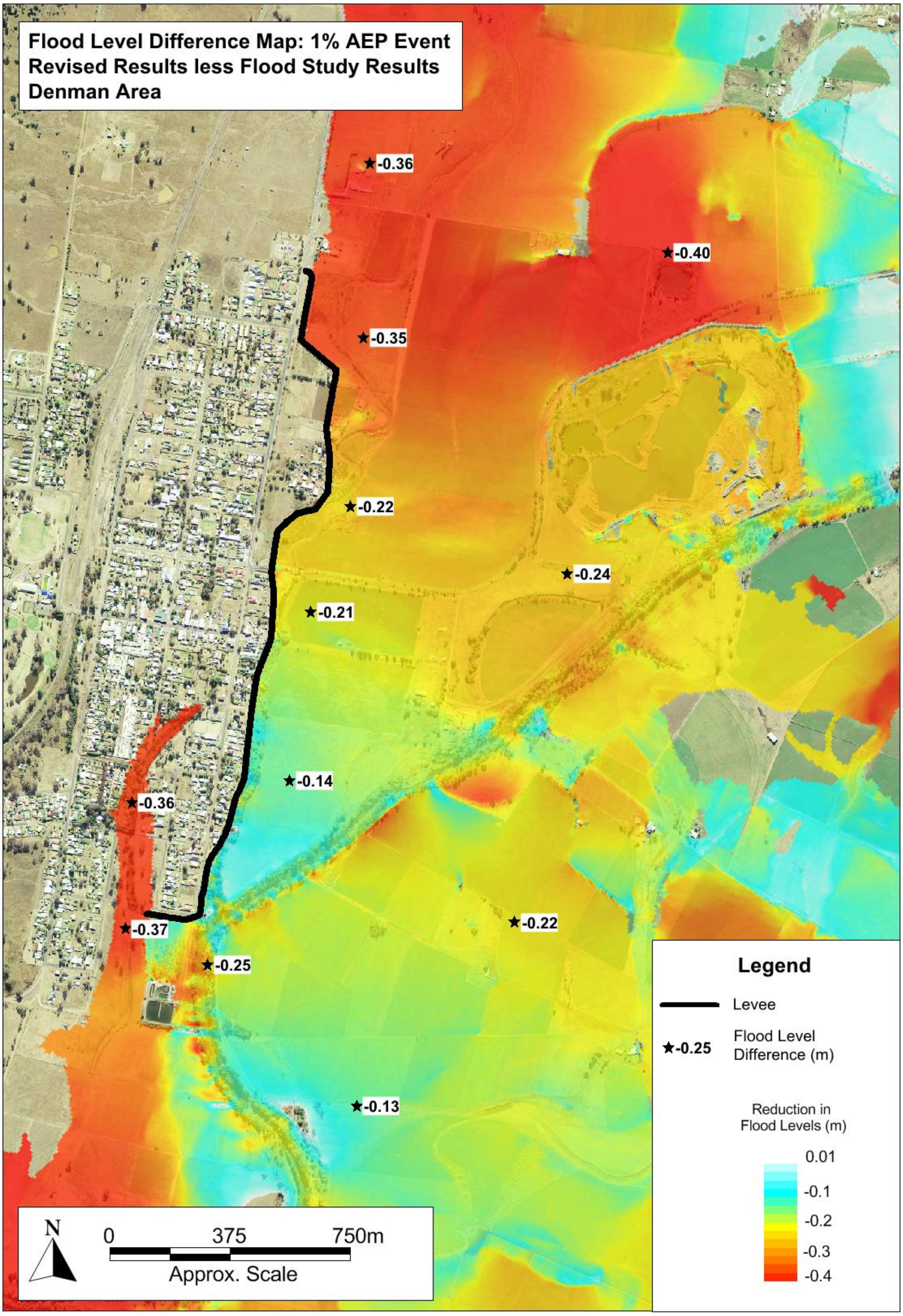


Figure 26: Changes to 1% AEP Levels: Denman Area

Note: a negative number represents a reduction in flood levels for the current study compared to the 2014 Flood Study.

7 MODEL VERIFICATION

The revised design model results were verified using surveyed flood levels from the 1971 and 1955 events in Muswellbrook and Denman that are documented in the WRC Flood Studies. The location of the flood levels are shown in **Figure C1** in **Appendix C**.

The objective of the verification process was to compare the revised design flood levels to known flood levels from historic events of similar magnitude. To facilitate this analysis, the hydrologic and hydraulic models were revised to represent the catchment and floodplain conditions in 1971 and 1955 that are discussed in **Section 3.1** and **Appendix A**. A summary of the key changes required includes:

- **1971 Conditions Model** – Lower channel roughness values and the physical changes noted in **Appendix A**.

- **1955 Conditions Model** - Lower channel roughness values, the removal of Glenbawn Dam from the hydrologic model, and the physical changes noted in **Appendix A.**

The 1971 and 1955 Conditions Models were simulated for the 1% AEP event. The resulting flood levels were compared to the surveyed flood levels that are documented in the WRC Flood Studies. Tabulated results are provided in **Appendix C**, while **Table 11** provides a summary of the calculated 20th, 50th (median) and 80th Percentile differentials (calculated as the simulated level less surveyed level).

Table 11 – *Verification Results Summary*

	Muswellbrook	**Denman**
1971 Event (1 in 70 year flood)		
20th Percentile Differential	0.04	0.18
Median Differential	**0.24**	**0.41**
80th Percentile Differential	0.64	0.60
1955 Event (likely to have been a 1% AEP or greater flood)		
20th Percentile Differential	-0.36	-0.33
Median Differential	**-0.05**	**-0.03**
80th Percentile Differential	0.19	0.05

The verification results presented in **Table 11** indicate that the 1% AEP design model (updated to represent conditions in 1955 and 1971) produced flood levels that were:

- Moderately (median differentials were 250 to 410 mm) above 1971 levels, which is in line with expectations given the 1971 event has an estimated 1 in 70 year AEP.
- Similar (median differentials were -30 to -50 mm) to 1955 levels, which is in line with expectations given the 1955 event is likely to have been a 1% or greater flood at Muswellbrook.

8 SENSITIVITY ANALYSIS

Sensitivity analysis was undertaken to gain an understanding of the flood mitigation benefit afforded by Glenbawn Dam and to assess the model's sensitivity to changes in the channel roughness assumptions. The following scenarios were simulated for the 1% AEP design event:

- **Channel Roughness Sensitivity Scenario** – For this Scenario, the design model was simulated with R3 roughness assumptions (as described in **Section 4**).

- **No Dam Scenario** - For this scenario Glenbawn Dam was removed from the hydrologic model.

For each scenario, flood level difference maps have been prepared to show the changes in peak design flood levels in the Muswellbrook Area. Figure 27 and **Figure 28** present flood level difference maps for the Channel Roughness and No Dam scenarios respectively.

With reference to Figure 27, the Channel Roughness results indicate that the higher channel roughness in the R5 assumptions would increase 1% AEP levels in the Muswellbrook Area by between 110 to 370 mm relative to the R3 assumptions. Flood level increases in the flood affected areas of Muswellbrook range from 110 to 340 mm, while flood levels adjacent to the Muswellbrook Levee are increased by 440 mm. These results confirm that the model is sensitive to the assumed channel roughness.

With reference to **Figure 28**, the No Dam results indicate that Glenbawn Dam would reduce 1% AEP levels in the Muswellbrook Area by between 110 to 340 mm. Flood level reductions in the flood affected areas of Muswellbrook range from 120 to 340 mm, while flood levels adjacent to the Muswellbrook Levee are reduced by 370mm.

For both scenarios, the higher magnitude reductions occur in areas of the floodplain where significant flow conveyance occurs or areas immediately upstream of a constriction in the floodplain or a major hydraulic control (such as a rail embankment across the floodplain).

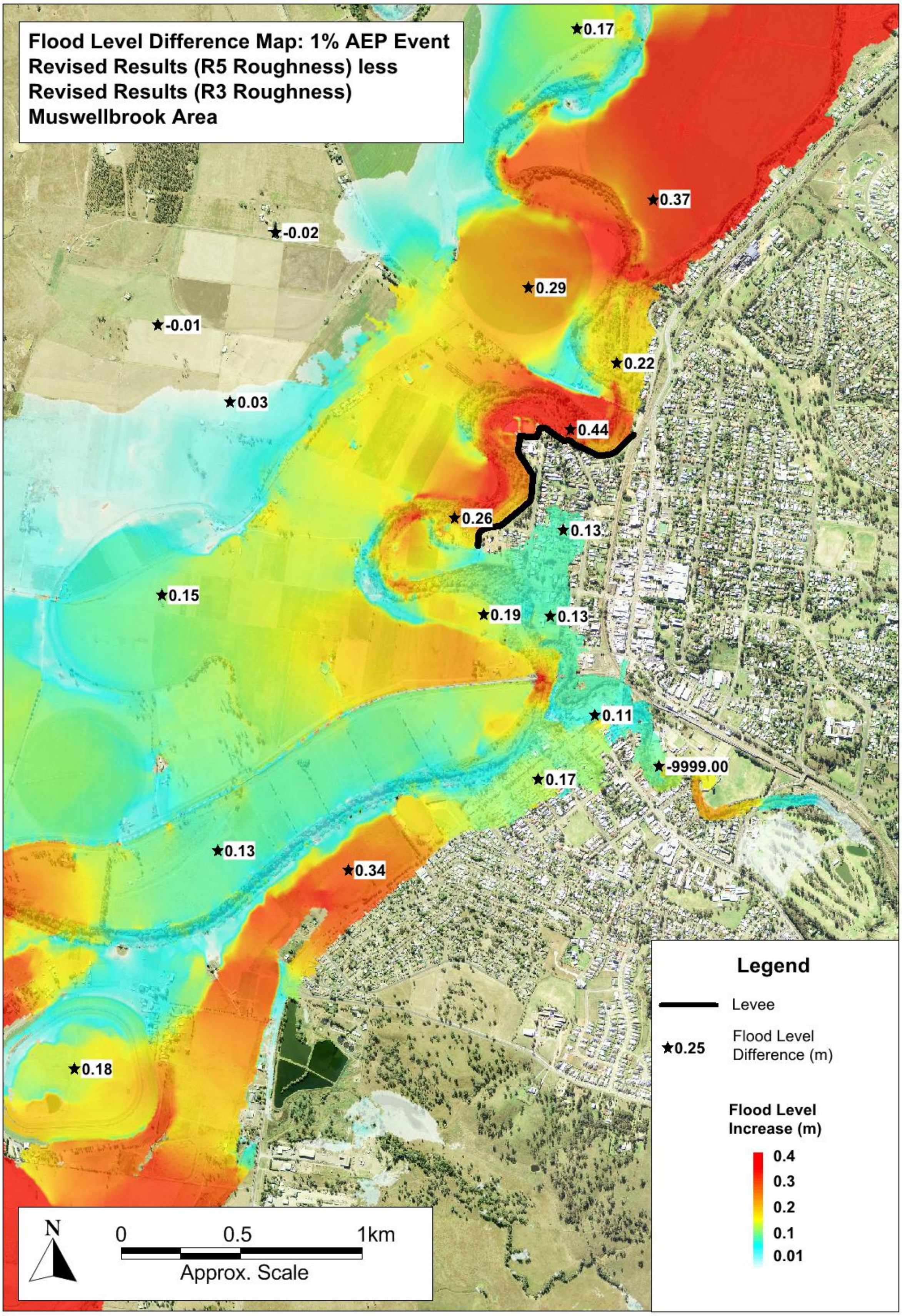


Figure 27: Sensitivity Analysis: Channel Roughness: 1%AEP Event

Note: a positive number represents an increase flood levels due to the use of higher roughness values.

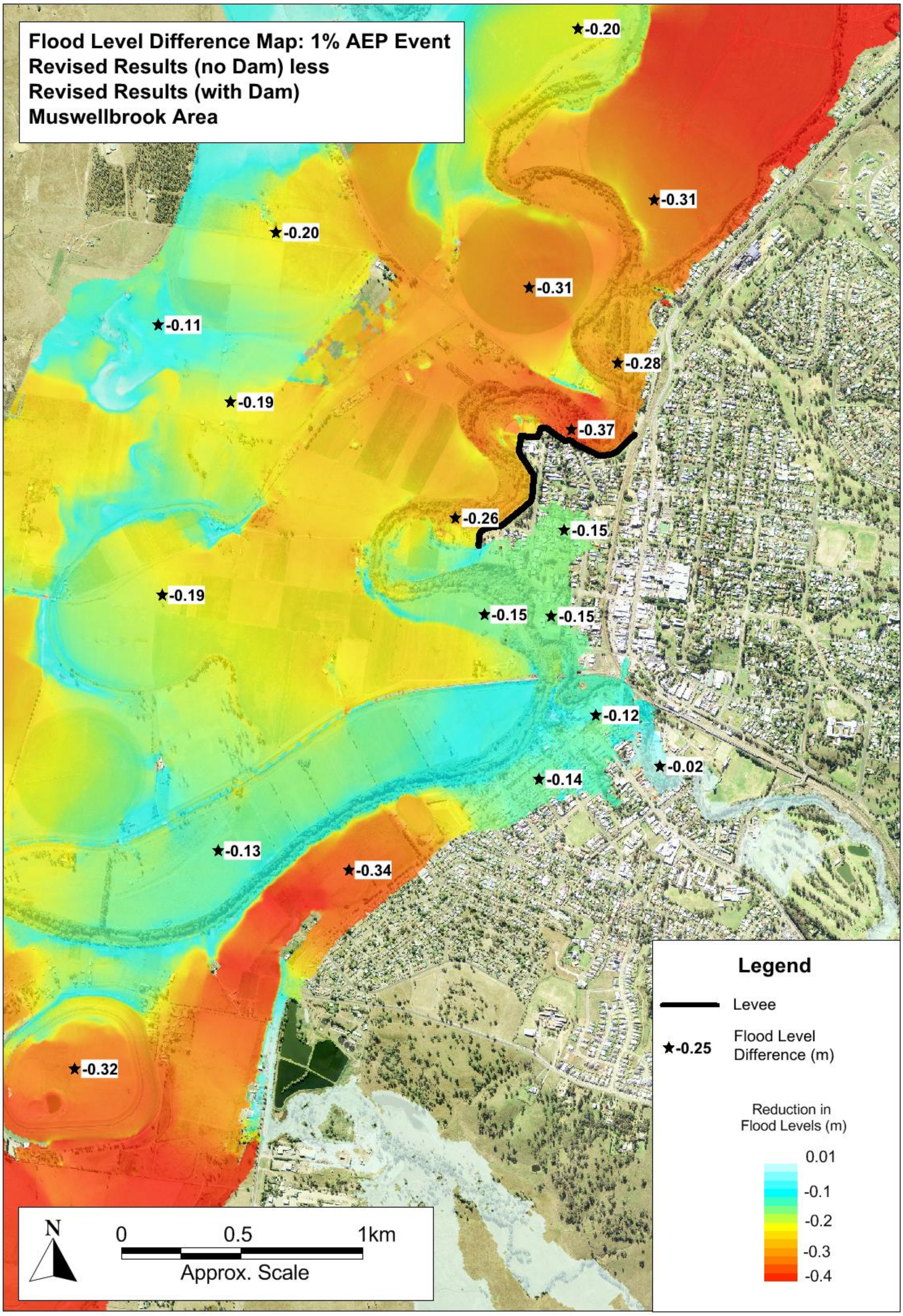


Figure 28: *Sensitivity Analysis: Glenbawn Dam: 1%AEP Event*

Note: a negative number represents a reduction in flood levels due to the flood storage of Glenbawn Dam

9 SUMMARY

This report documents proposed revisions to the hydrologic and hydraulic models that were developed as part of the Hunter River Flood Study (Muswellbrook to Denman) that was prepared by Worley Parsons in 2014. The model revisions were undertaken for the following reasons:

- A technical adequacy review of the 2014 flood study identified a number of issues regarding the reliability of the models that were developed as part of this study.

- Rating curves for many of the Upper Hunter stream gauges have been recently revised by WaterNSW. The revised rating curves substantially reduce the assumed flows for a given gauge stage. The revisions are due to the increase in vegetation densities both within the channel and on the channel banks over the last two decades.

- To update the hydrologic modelling approach to be consistent with the methods recommended in the Australian Rainfall and Runoff (2016) guideline.

The following key model revisions are recommended in this report:

- The hydraulic model was updated to improve spatial definition of the various roughness categories – Refer to **Section 4.1.2** for further details.

- Hydraulic roughness coefficients for the channel zone were revised (to be significantly rougher) to account for increased channel vegetation densities. The adjusted roughness coefficients produced simulated rating curves that were similar to the revised rating curves provided by WaterNSW – Refer to **Section 4.1.2** for further details.

- The hydrologic model was recalibrated using available stream gauge and rainfall data. Some changes to model parameters were made during the recalibration process – Refer to **Section 4.2** for further details.

- The revised models were applied to simulate design flood events using the Australian Rainfall and Runoff (2016) methods.

The various model changes that are documented in this report will result in 1% AEP flood level reductions (relative to the 2014 Flood Study levels) ranging from 90 to 360mm. Reductions in the flood affected areas of Muswellbrook are typically in the 90 to 340 mm range. Flood level reductions adjacent to the Muswellbrook Levee are in the 230 to 270 mm range, while moderately larger (140 to 350 mm) reductions are predicted adjacent to the Denman Levee.

The revised design event results were verified by Flood Frequency Analysis (**Section 5**) and available flood level data from the 1971 and 1955 events (**Section 7**). It is noted that both of these verification methods use independent data.

Sensitivity analysis was undertaken to gain an understanding of the flood mitigation benefit afforded by Glenbawn dam and to assess the model's sensitivity to changes in the channel roughness assumptions (**Section 8**).

10 REFERENCES

1) Commonwealth of Australia (Geoscience Australia) (2016), *‘Australian Rainfall and Runoff: A Guide to Flood Estimation’*

2) Hunter Valley Conservative Trust (no date provided), *‘A Report on the Flood of February 1955 in the Hunter Valley of NSW’*

3) Institution of Engineers Australia (1987), ‘*Australian Rainfall and Runoff – A Guide to Flood Estimation’*

4) New South Wales Government (2005), *‘Floodplain Development Manual – The Management of Flood Liable Land’*

5) R. L. Bernard (1968), ‘*Flood Report: Hunter River 1955’*

6) *Water Resource Commission (1986), ‘Muswellbrook Flood Study’*

7) *Water Resource Commission (1986), ‘Denman Flood Study’*

8) *Worley Parson (2013), ‘Hunter River Flood Study (Muswellbrook to Denman)’*

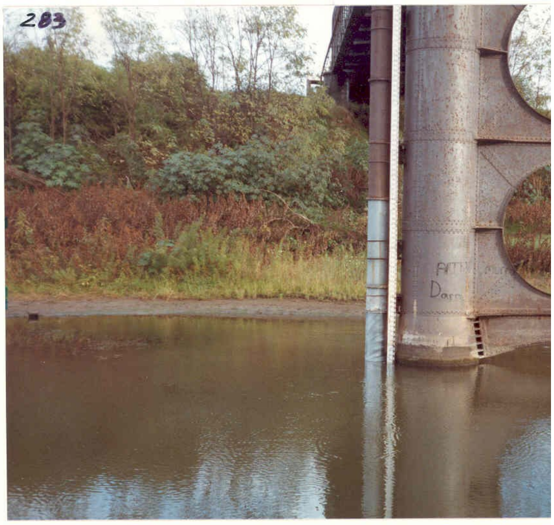
Appendix A – *Floodplain Changes*

Kayuga Road Bridge: Looking Upstream

1980

2011

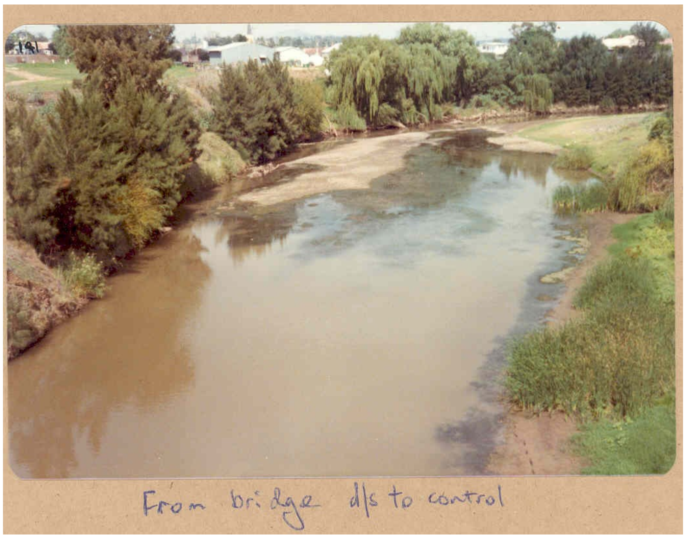
Kayuga Road Bridge: Western Bank



1980

2011

Kayuga Road Bridge: Looking Downstream



1980

2011

Denman Gauge: Upstream

1980

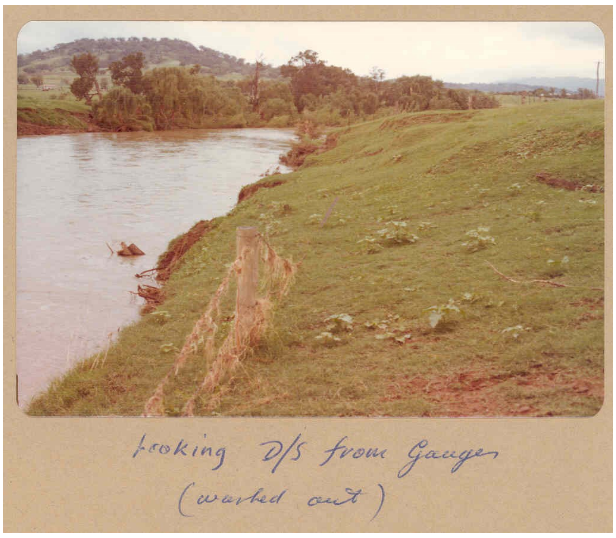
2011

Denman Gauge: Channel Bank near the gauge

1980

2011

Denman Gauge: Downstream



1980

2011

Floodplain Changes: Muswellbrook Area

1950s

1970s

2013

Key Floodplain Changes (1950s to 1970s)

- Minor increase in vegetation within the channel zone.
- No material change to floodplain vegetation.

The images show the Hunter River Floodplain immediately to the west of Muswellbrook. Note: note the 1950s image was provided at a slightly different orientation to the 1970s and 2013 image.

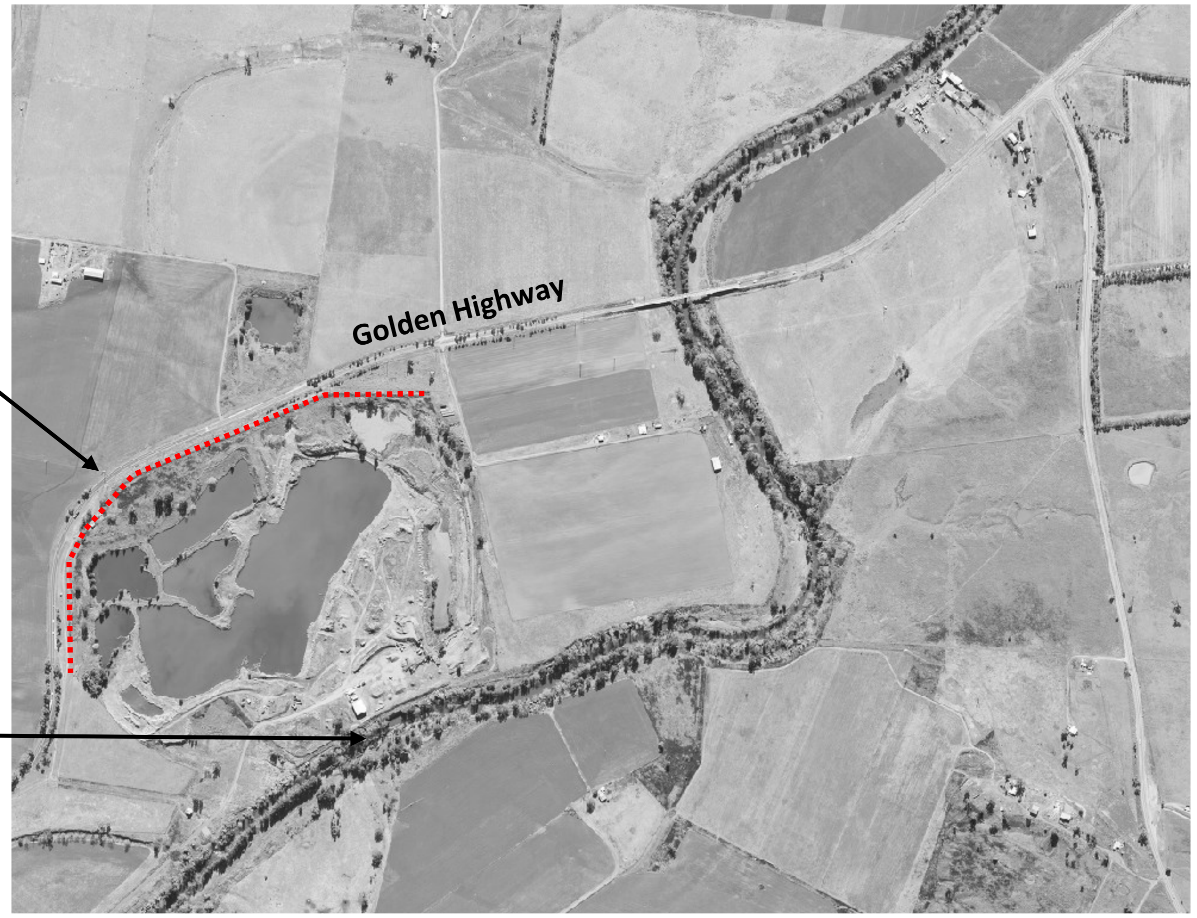
Key Floodplain Changes (1970s to 2013)

- Significant increase in vegetation within the channel zone.
- No material change to floodplain vegetation.
- Muswellbrook levee was constructed in the 1990s.

Floodplain Changes: Denman Area

1950s

2013



2m high visual berm

Channel straitening occurred sometime between when the 1950s and 1970s images were taken.

1970s

Key Floodplain Changes (1950s to 1970s)

- No significant increase in vegetation within the channel zone .
- No material change to floodplain vegetation.
- Some sections of the Hunter River Channel were straitened.

The images show the Hunter River Floodplain immediately to the east of Denman.

Key Floodplain Changes (1970s to 2013)

- Significant increase in vegetation within the channel zone .
- No material change to floodplain vegetation.
- Denman levee was constructed in the 1990s.
- A quarry was established between the Hunter River Channel and the Golden Highway. A 2m high visual berm has been established along the northern boundary.

Appendix B – *Calibration Charts*

Pluvio Rainfall Data

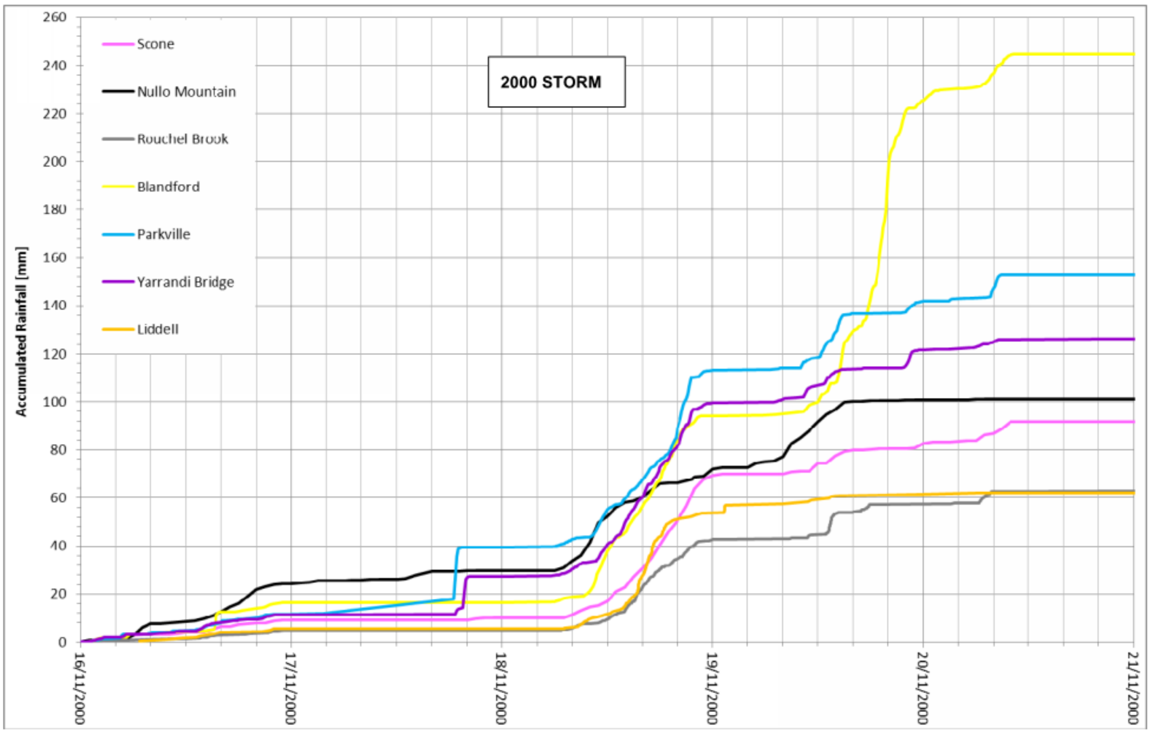


Figure B1: Pluvio Rainfall Data for November 2000 event (Worley Parsons, 2014)

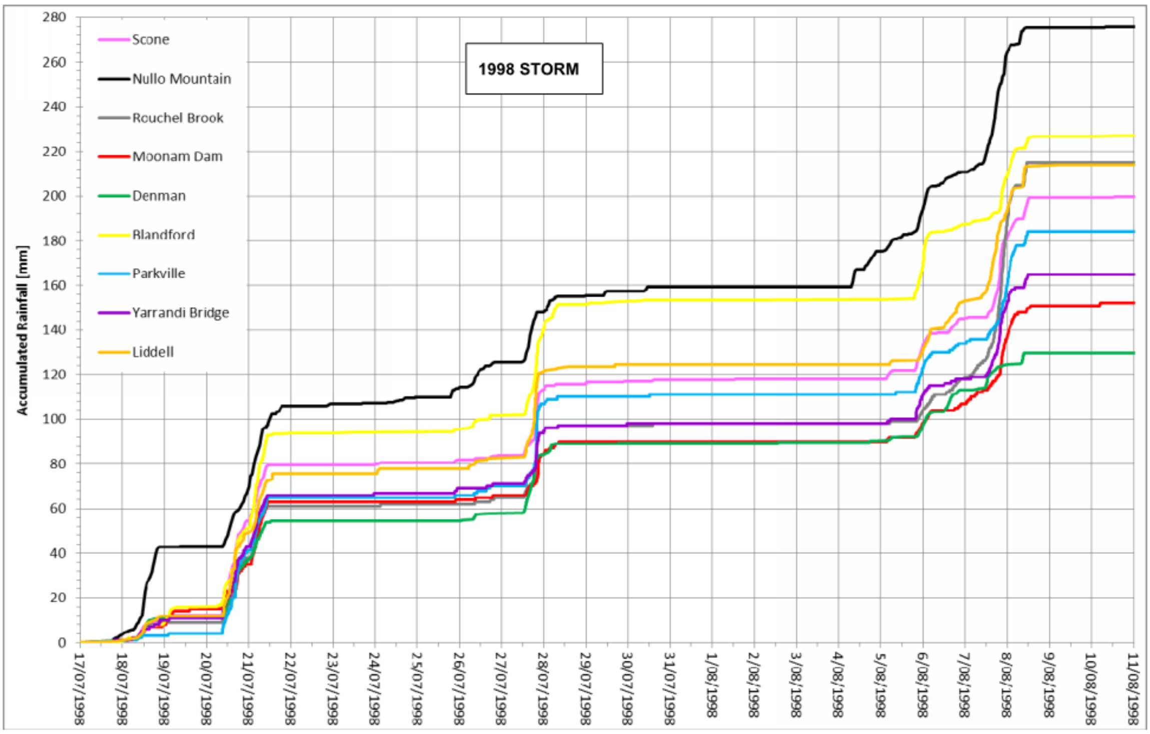


Figure B2: Pluvio Rainfall Data for August 1998 event (Worley Parsons, 2014)

Rainfall Distribution

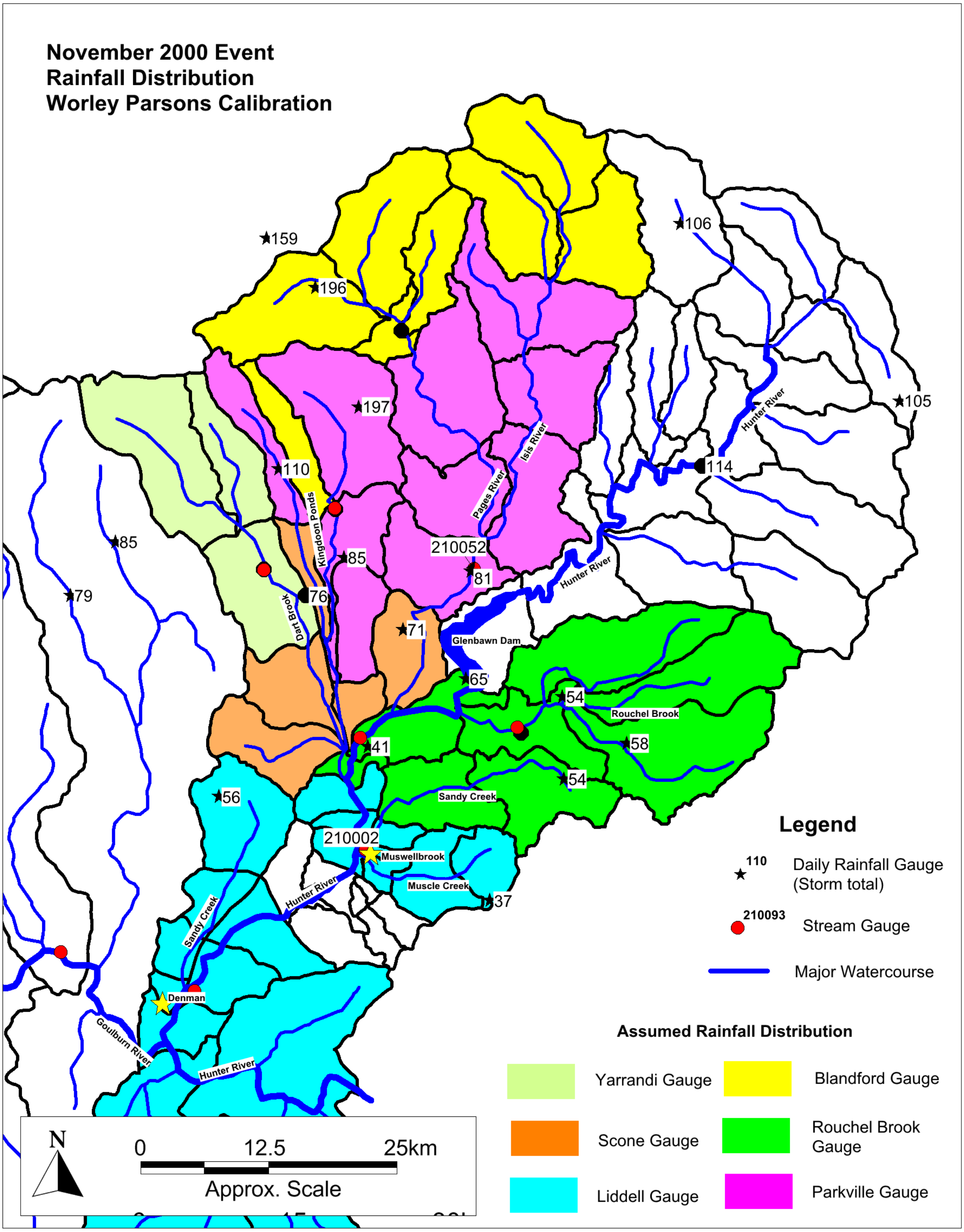


Figure B3: Rainfall distribution November 2000 Event: Flood Study

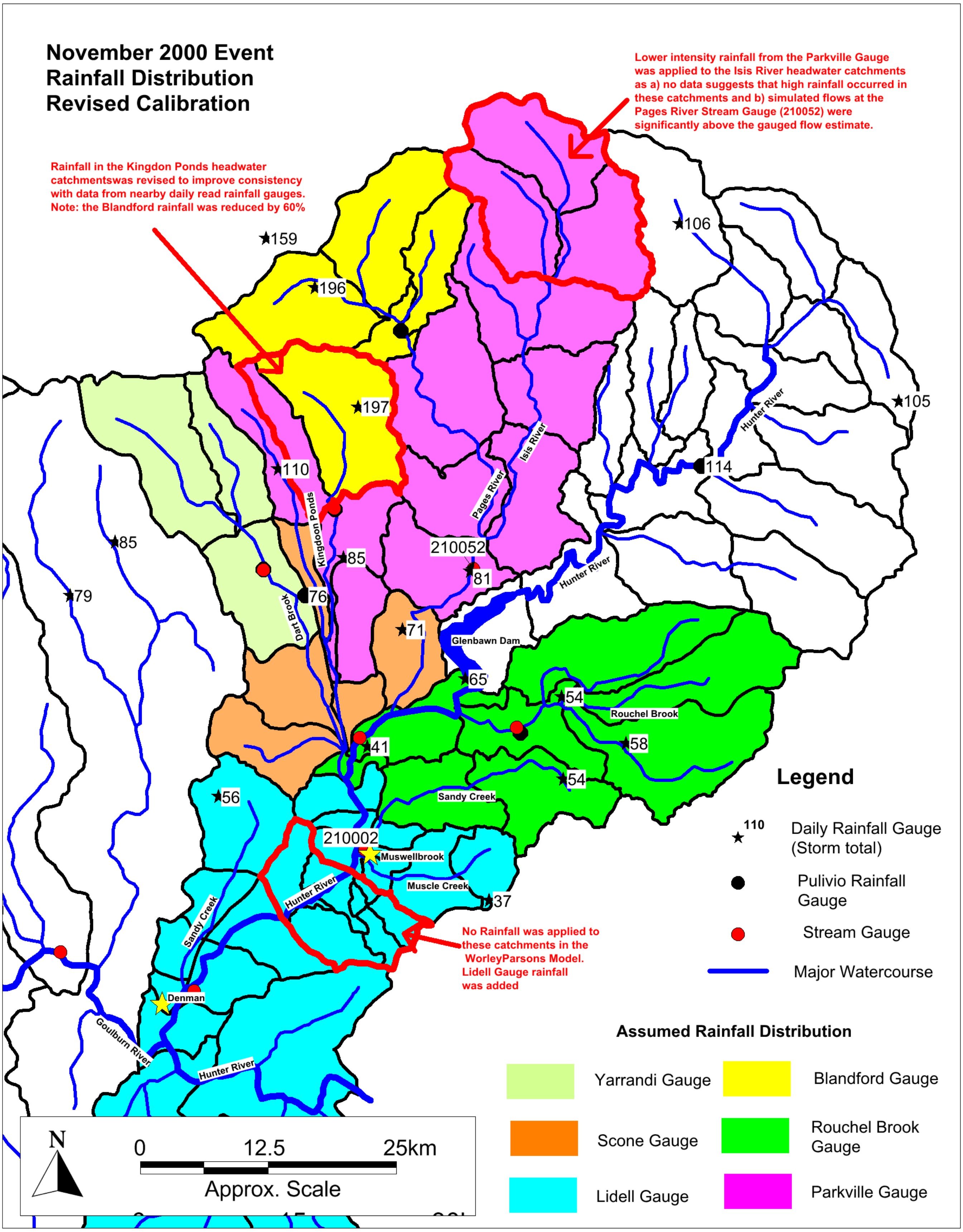


Figure B4: Rainfall distribution November 2000 Event: Revised

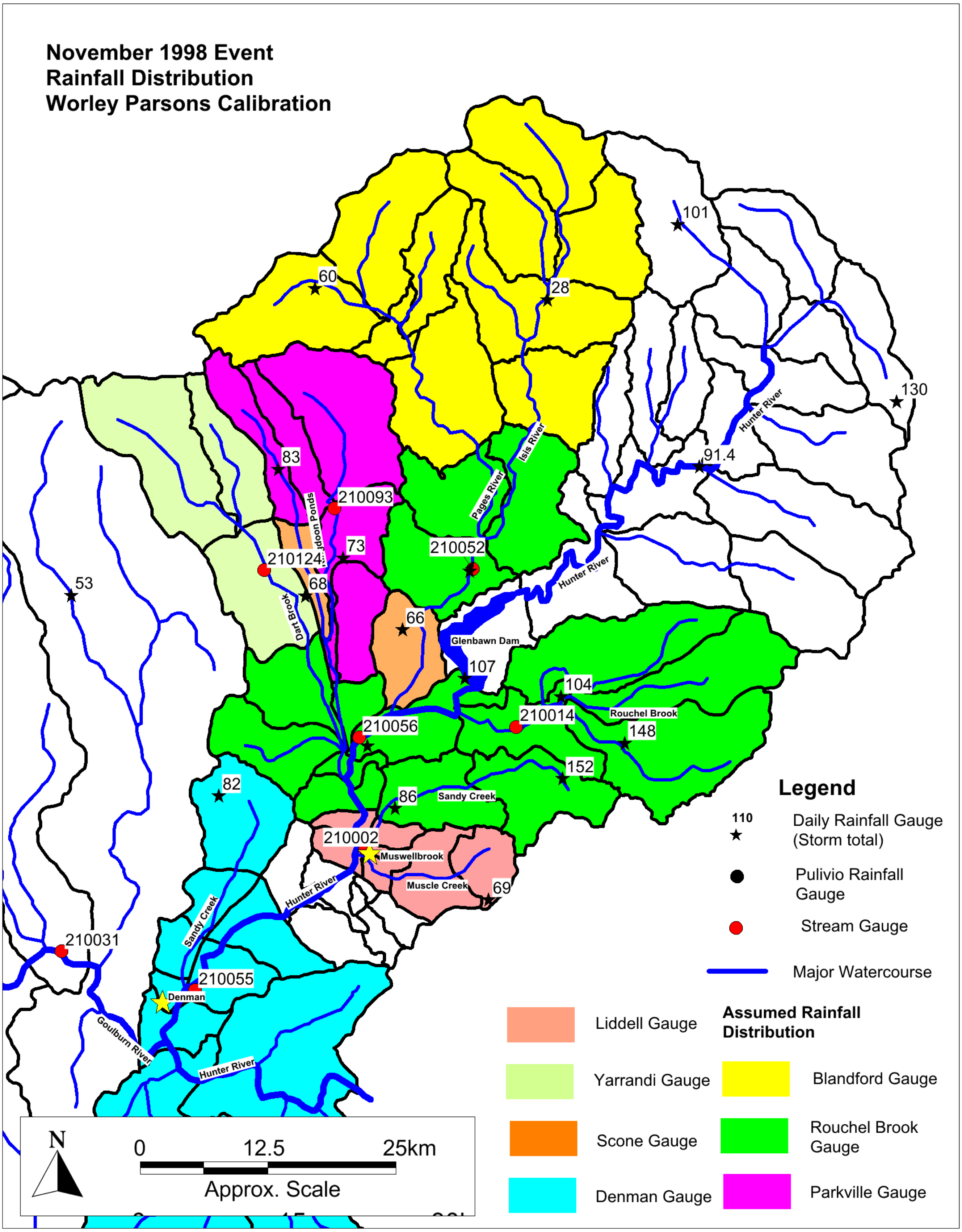


Figure B5: Rainfall distribution August 1998 Event: Flood Study

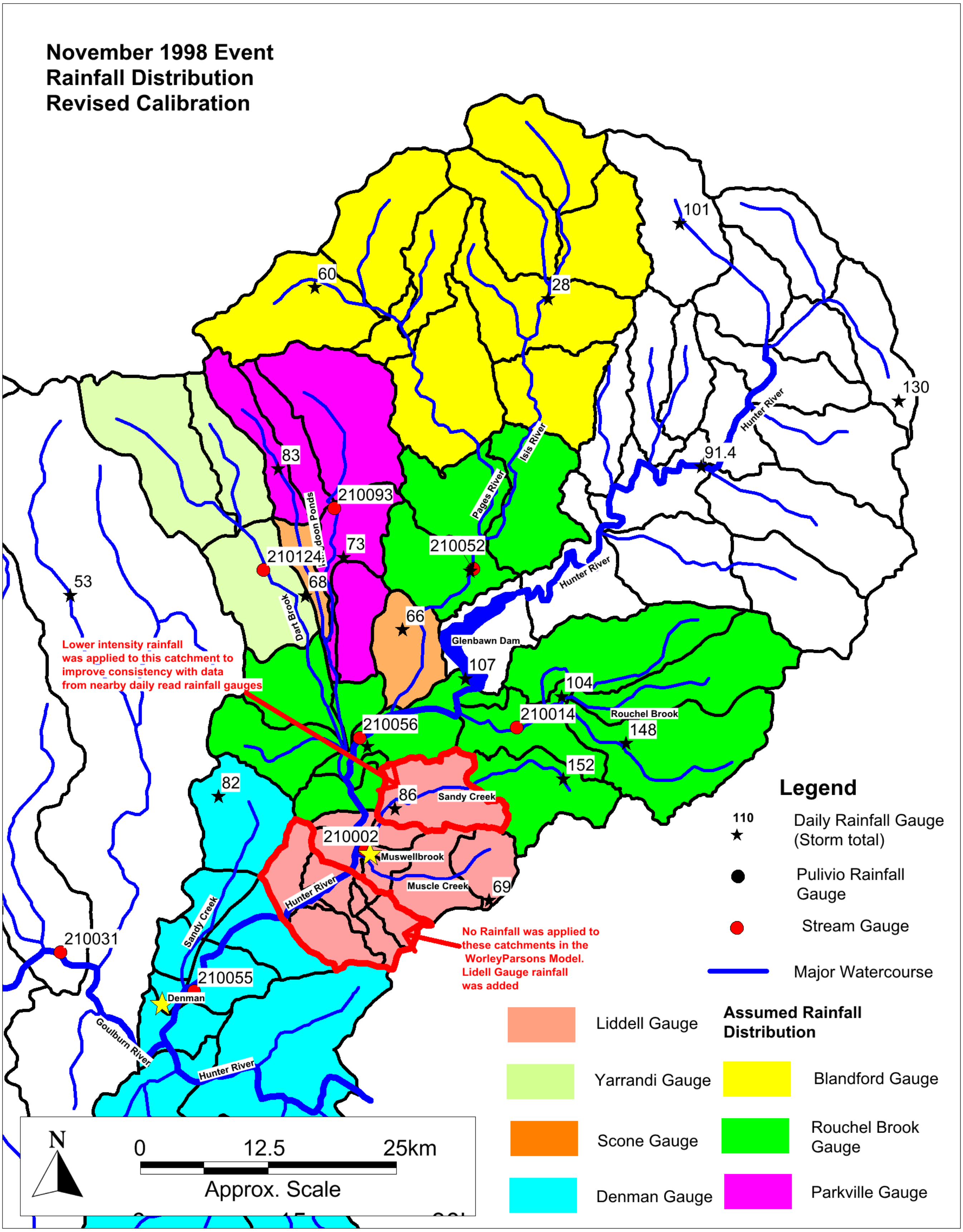


Figure B6: Rainfall distribution August 1998 Event: Flood Study

Calibration Hydrographs – November 2000 Event

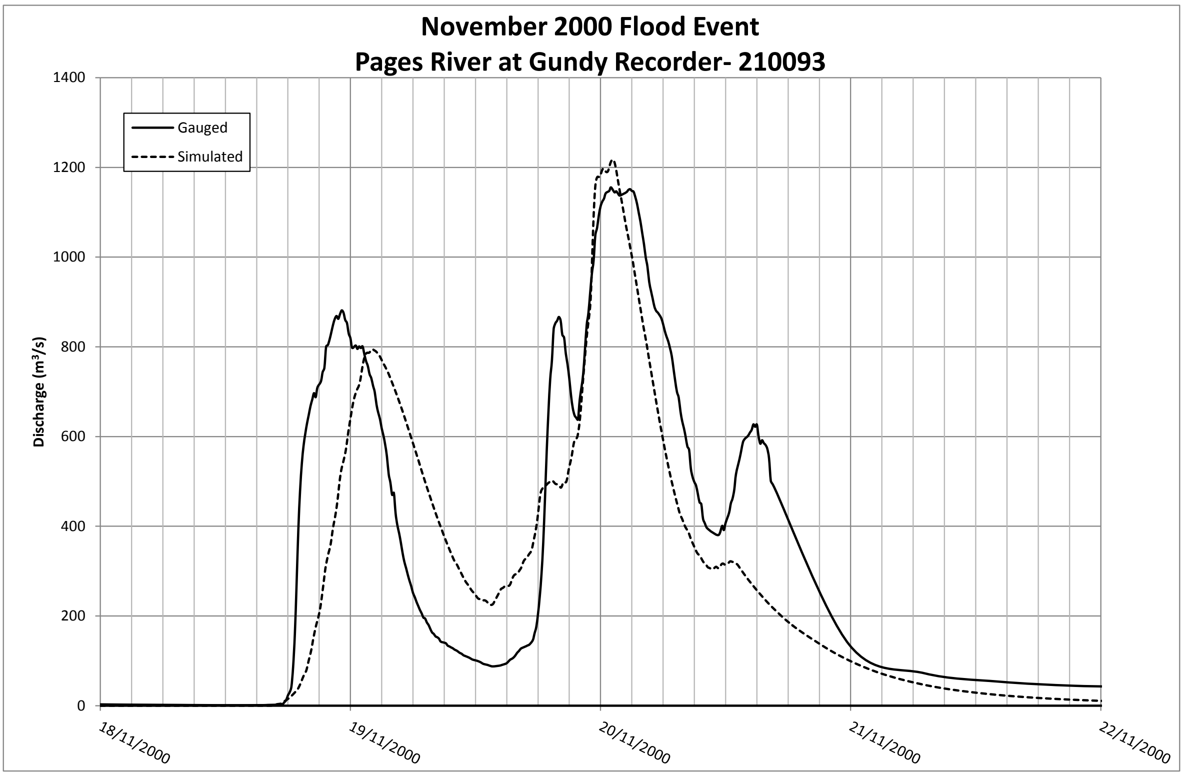


Figure B7: Pages River at Gundy: November 2000 Event

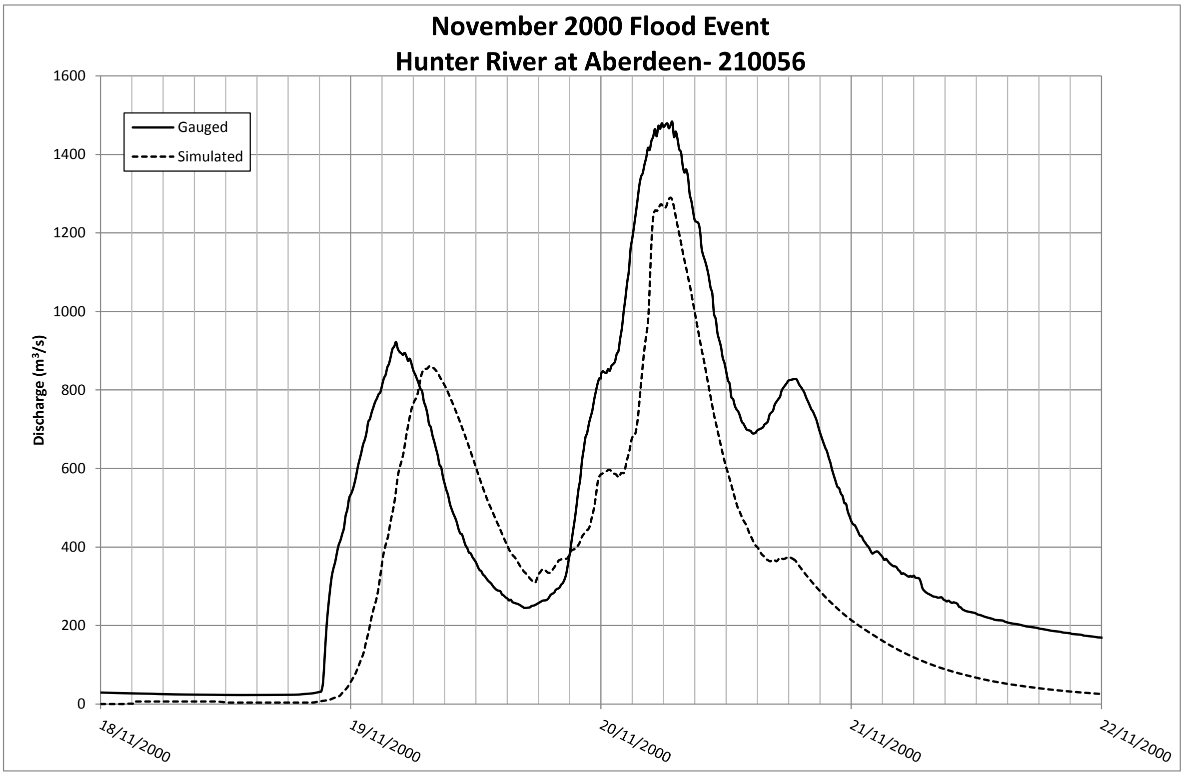


Figure B8: Hunter River at Aberdeen: November 2000 Event

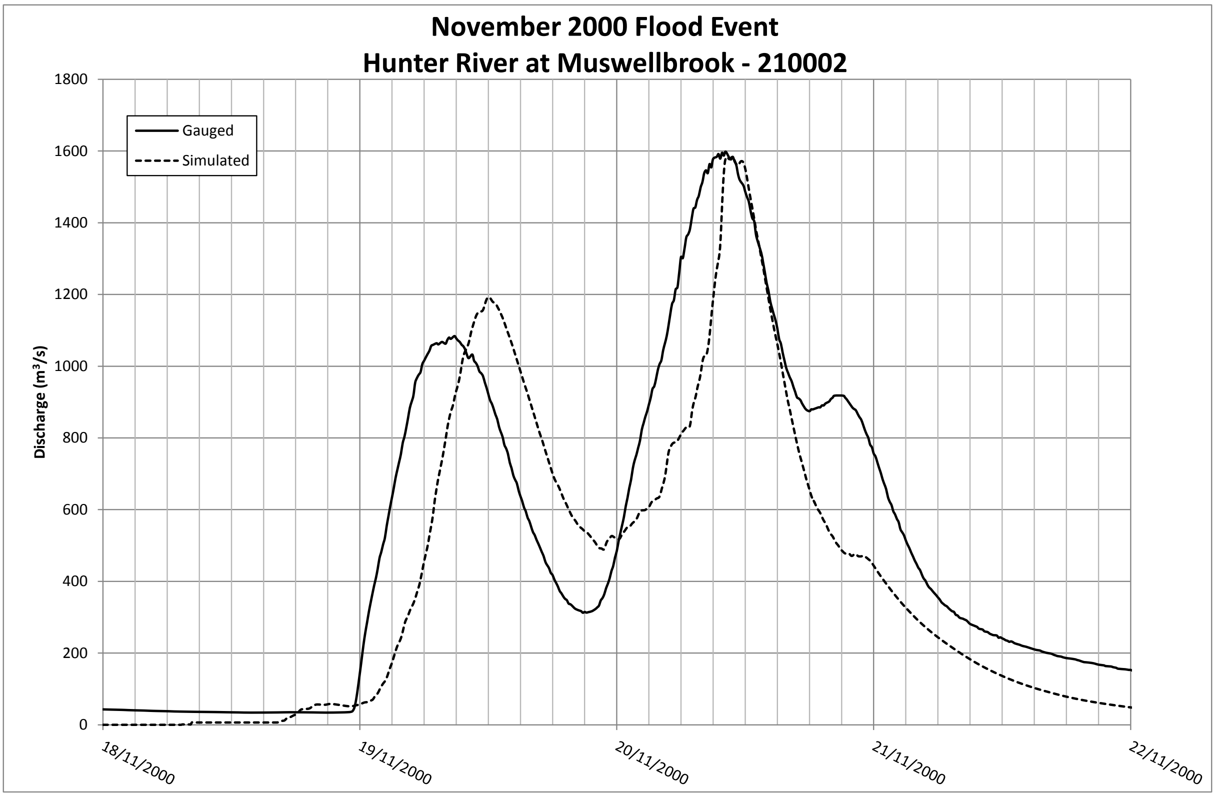


Figure B9: Hunter River at Muswellbrook: November 2000 Event

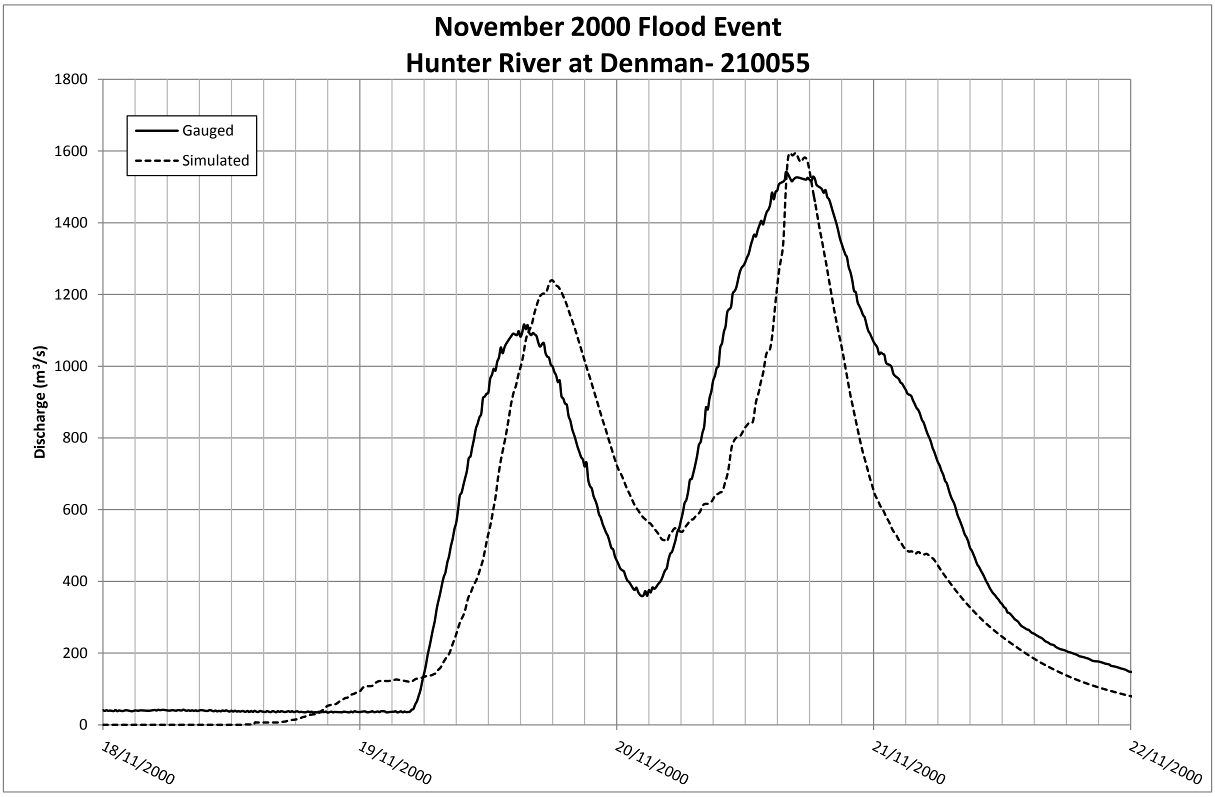


Figure B10: Hunter River at Denman: November 2000 Event

Calibration Hydrographs – August 1998 Event

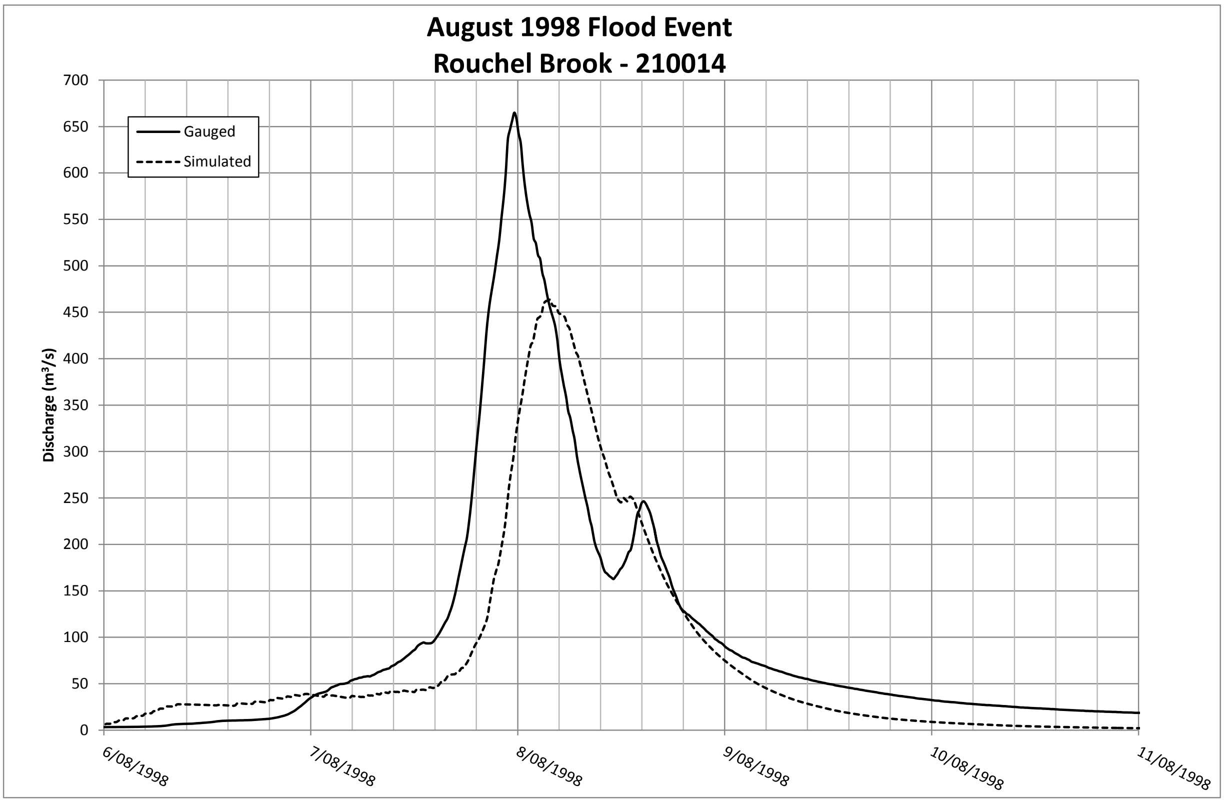


Figure B11: Rouchel Brook: August 1998 Event

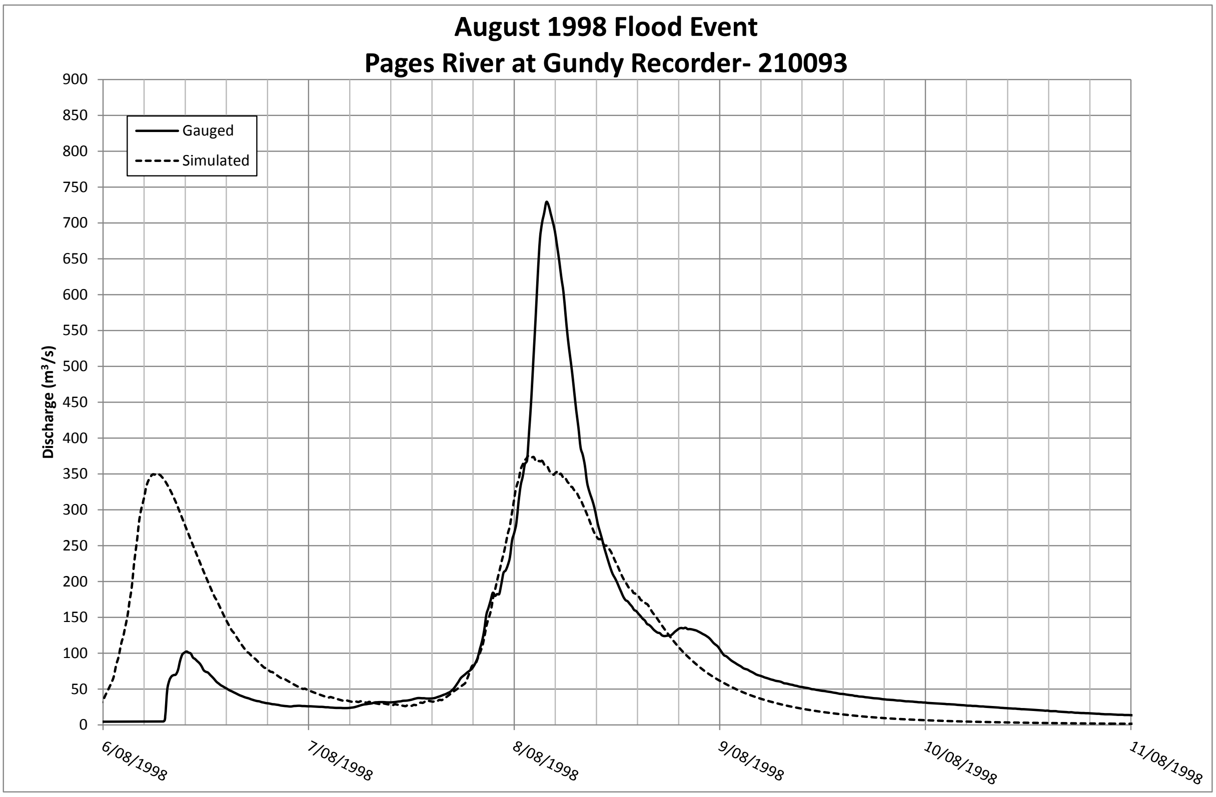


Figure B12: Pages River at Gundy: August 1998 Event

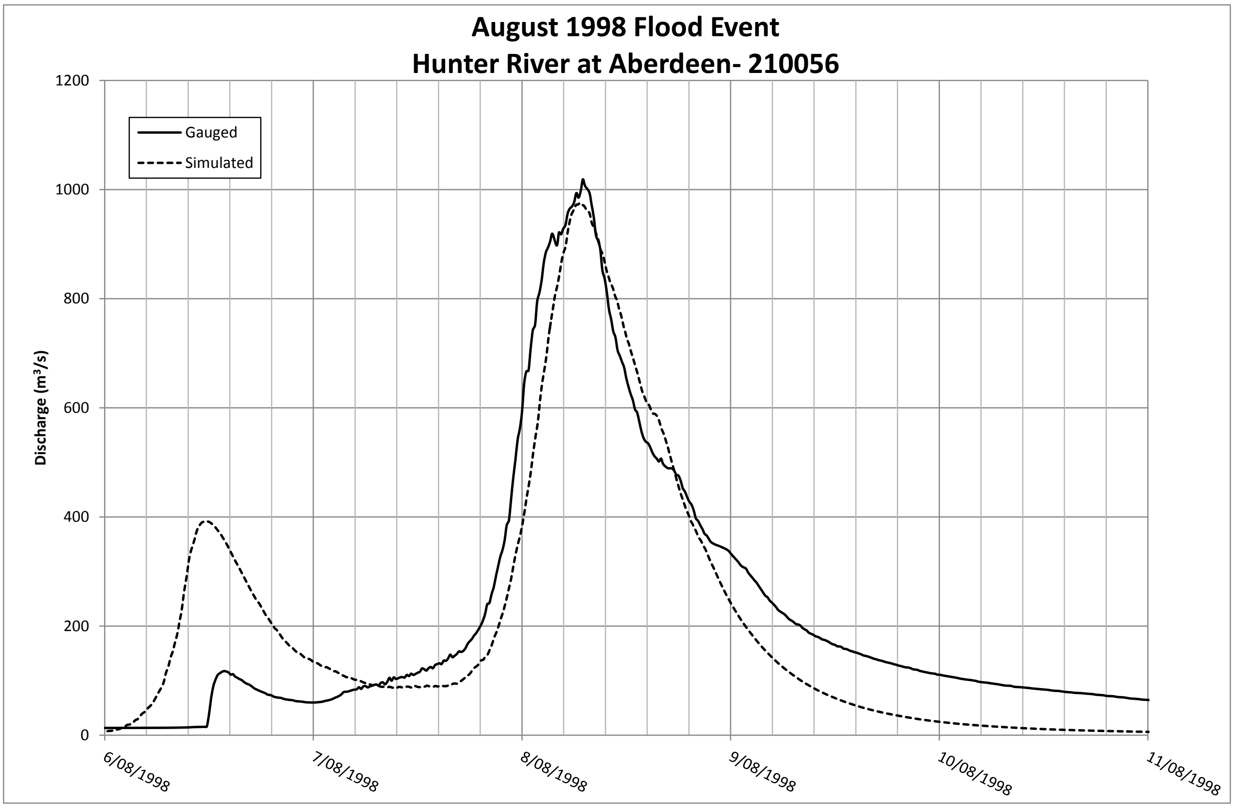


Figure B13: Hunter River at Aberdeen: August 1998 Event

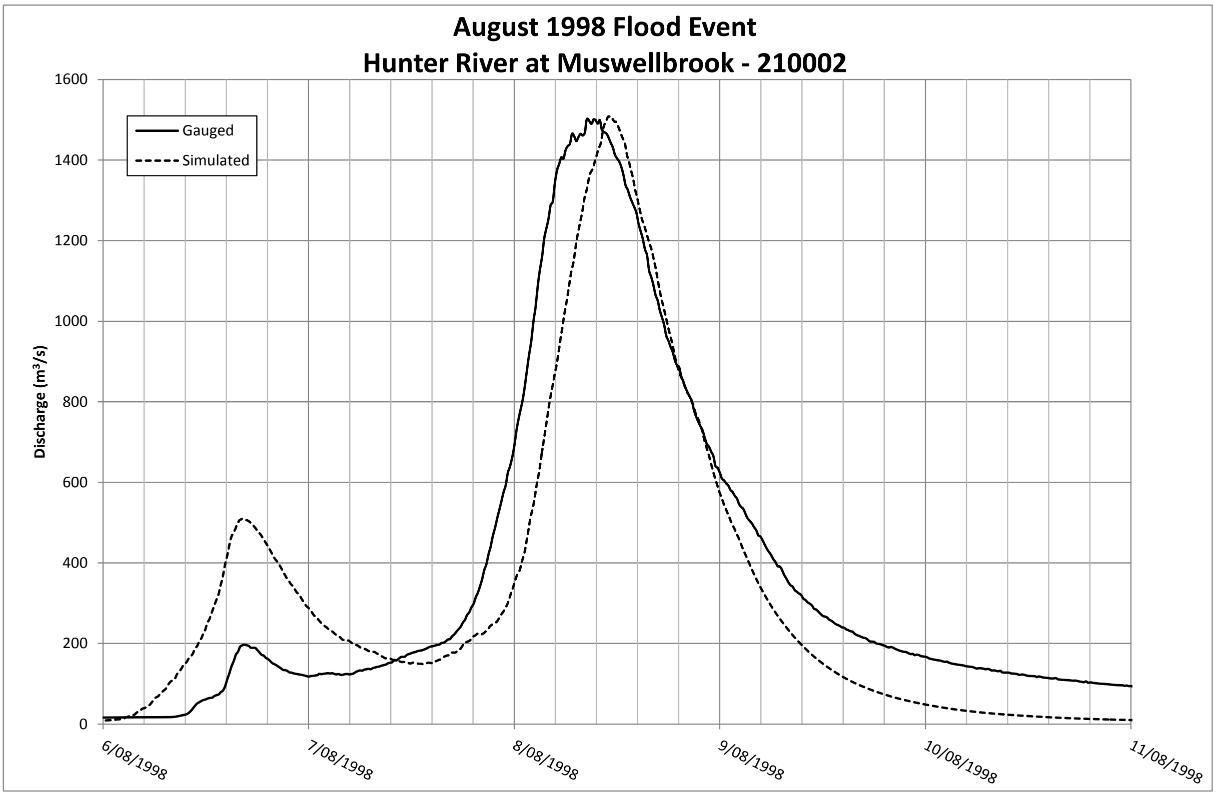


Figure B14: Hunter River at Muswellbrook: August 1998 Event

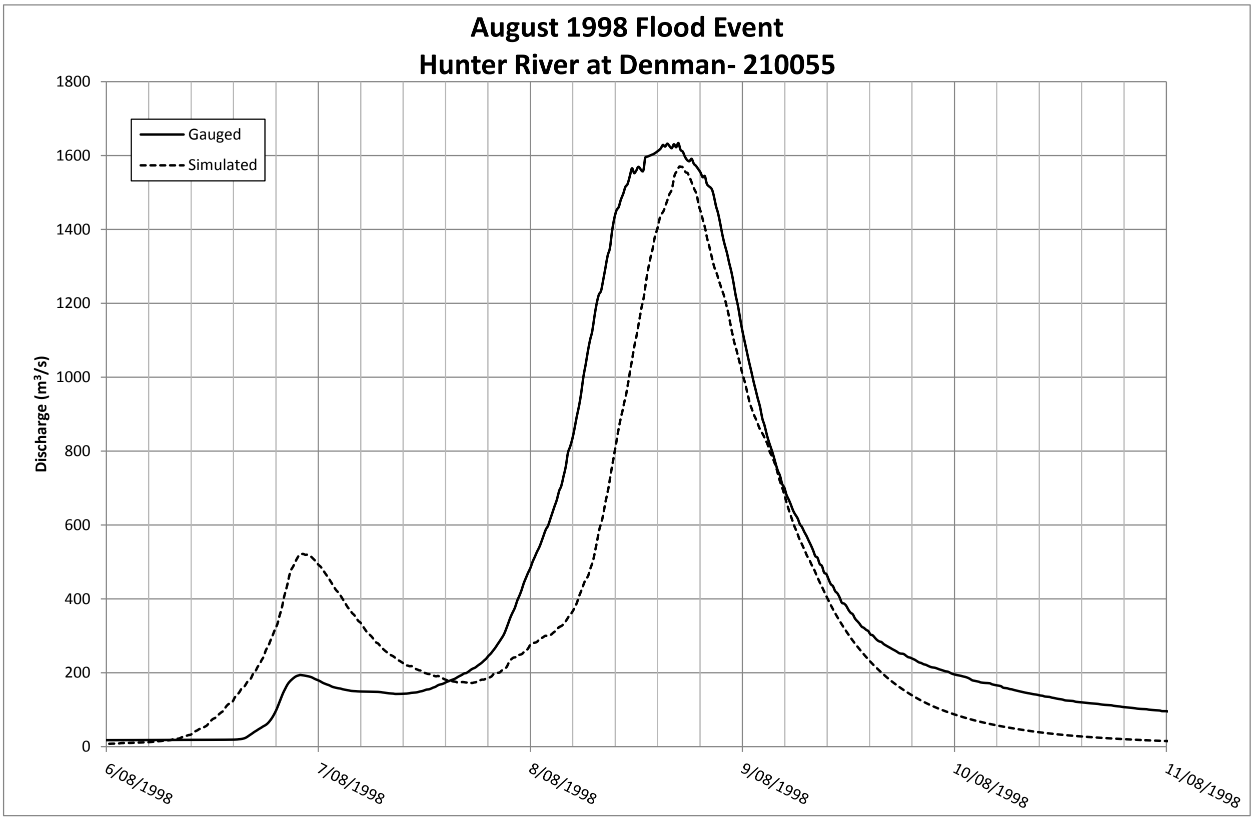


Figure B15: Hunter River at Denman: August 1998 Event

Appendix C – *Model Verification Results*

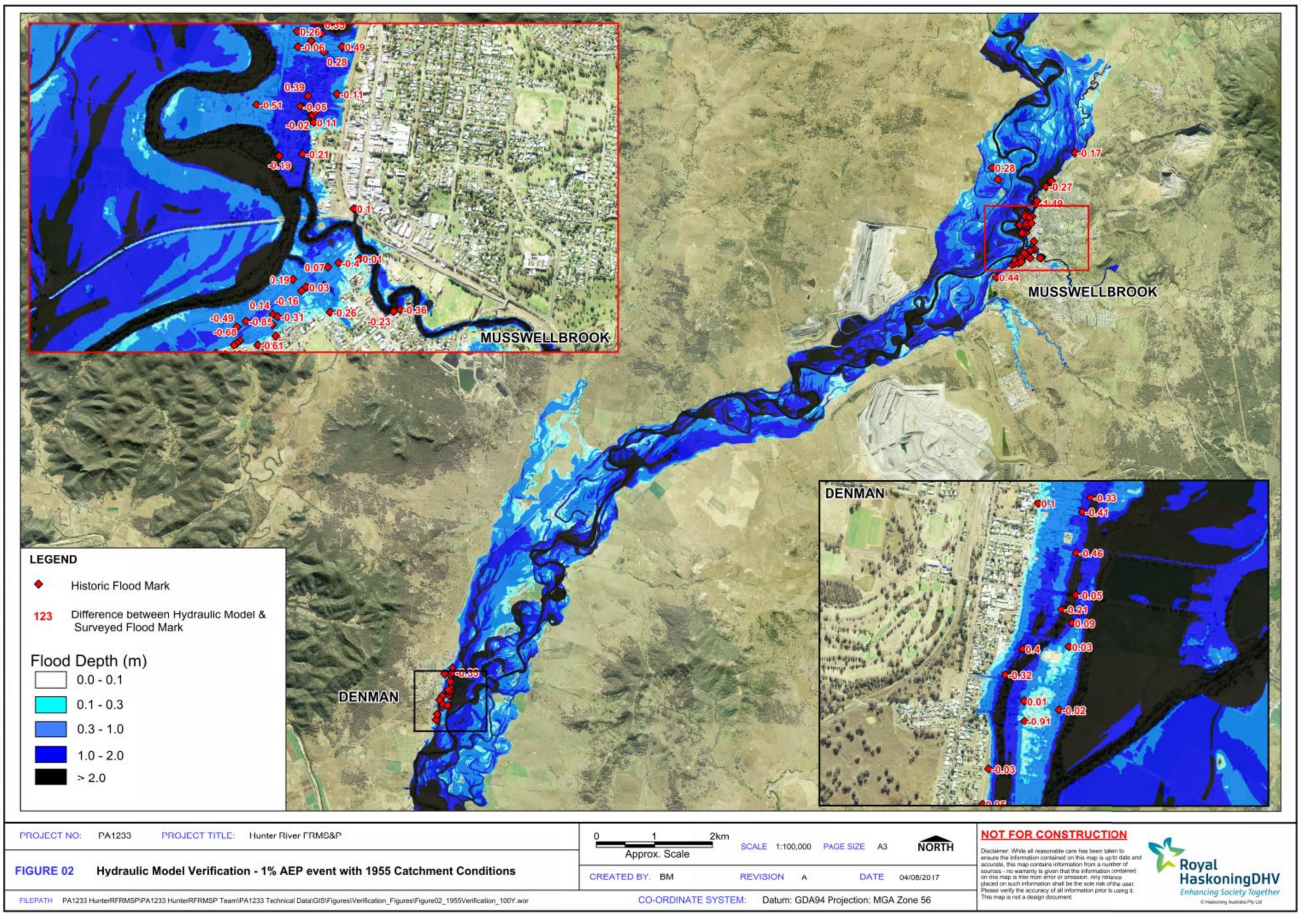


Figure C1: Historic Flood Marks

Denman Verification Results

				Verification Model Runs			
	Flood Level Data			1%_1955 Conditions		1%_1971 Conditions	
Property	**1955**	**1971**	**1976**	**Level**	**Diff_1955**	**Level**	**Diff_1971**
Babbington Street_1	106.69	0	0	106.7	0.0		
2 Babbington St 01	106.31	0	0	106.4	0.1		
2 Hyde Street	107.9	106.27	0	107.0	-0.9	107.0	0.7
10 Hyde Street	107.21	106.42	0	107.2	0.0	106.8	0.4
11 Palace Street	108.01	0	0	108.0	0.0		
Palace Street (Old Chu	108.15	0	0	108.2	0.0		
43 Palace Street	108.14	0	0	108.2	0.1		
47 Palace Street	108.64	108	107.31	108.3	-0.3	108.0	0.0
Cnr. Palace Street and	108.43	107.91	107.4	108.4	0.0	108.1	0.2
60 Palace Street	108.48	0	0	108.3	-0.2		
76 Palace Street	109.15	107.73	0	108.7	-0.5	108.3	0.6
84 Palace Street	109.26	108.22	0	108.8	-0.4	108.5	0.2
92 Palace Street	109.25	0	0	108.9	-0.3		
102 Palace Street	109.25	0	0	109.2	0.0		
1 Paxton Street	107.33	106.83	106.66	107.7	0.4	107.4	0.6
2 Paxton Street	107.79	0	0	107.5	-0.3		
52 Paxton Street	108.78	0	0	108.9	0.1		
		20th Percentile			**-0.33**		**0.18**
		Median			**-0.03**		**0.41**
		80th Percentile			**0.05**		**0.60**

Muswellbrook Verification Results

				Verification Model Runs			
	Flood Level Data			1%_1955 Conditions		1%_1971 Conditions	
Property	1955	1971	1976	Level	Diff_1955	Level	Diff_1971
36 Sydney Prince of Wales	145.47	144.66	144.64	145.1	-0.4	144.8	0.2
1 Barrett	144.79	144.07	143.68	144.5	-0.3	144.3	0.2
1 Forbes	144.17	0	0	143.7	-0.4		
113 Sydney	145.14	143.99	0	144.3	-0.9	144.1	0.1
114 Sydney Telecom Depc	144.84	143.37	143.37	144.2	-0.7	144.0	0.6
116 Sydney	144.85	143.38	143.17	144.1	-0.7	143.9	0.5
118 Sydney	144.02	143.45	143.29	144.0	0.0	143.9	0.4
119 Sydney	144.66	143.82	143.5	144.2	-0.5	144.0	0.2
15 Brook.	145.65	145.07	144.07	145.4	-0.2	145.2	0.1
16 Wilder St.	145.72	145.2	0	145.4	-0.4	145.3	0.1
184 Sydney	142.63	141.93	0	143.1	0.4	142.6	0.7
19 Jordon	145.15	144.74	0	144.9	-0.3	144.7	-0.1
26 Aberdeen	149.48	0	0	149.2	-0.3		
3 Wilkins St.	147.06	0	0	147.6	0.5		
30 Hunter Terrace	146.15	144.96	0	146.3	0.1	145.9	0.9
34 Hunter Terrace	146.35	145.27	144.86	146.3	0.0	146.0	0.7
35 Aberdeen	149.29	0	0	149.4	0.1		
36 Hunter Terrace	146.32	145.44	145.44	146.4	0.1	146.0	0.6
37 Aberdeen	149.5	0	0	149.4	-0.1		
4 Skellatar	144	143.63	143.56	144.4	0.4	144.3	0.6
41 Scott	146.93	146.58	0	146.9	-0.1	146.6	0.0
5 Aberdeen	150.03	149.25	147.83	148.5	-1.5	148.3	-1.0
5 Brook	145.63	145.18	144.77	145.4	-0.2	145.2	0.0
5 Collins Lane	148.55	148.83	148.41	148.8	0.3	148.8	-0.1
50 Sydney	144.95	144.31	141.13	145.0	0.1	144.8	0.5
51 Scott	146.87	0	0	147.1	0.3		
6 Flemming	144.89	0	0	144.3	-0.6		
68 Ford	146.87	146.36	0	147.1	0.3	146.8	0.5
70 Sydney	144.75	144.56	144.16	144.8	0.0	144.6	0.0
74 Sydney	144.83	144.62	144.62	144.7	-0.2	144.5	-0.1
75 Ford	147.46	146.95	0	147.8	0.4	147.5	0.5
77 Sydney	144.47	0	0	144.7	0.2		
78 Ford	147.3	0	0	147.3	0.0		
80 Aberdeen	150.41	0	0	150.2	-0.2		
94 Sydney	144.33	143.92	143.69	144.5	0.1	144.3	0.4
8 Aberdeen	148.13	147.36	147.36	148.5	0.4	148.3	0.9
8 Bridge	145.21	144.9	144.9	145.3	0.1	145.1	0.2
		20th Percentile			**-0.36**		**0.04**
		Median			**-0.05**		**0.24**
		80th Percentile			**0.19**		**0.63**