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Williams River Flood Study

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DOCUMENT CONTROL SHEET

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Title : ***Author :***	Williams River Flood Study – Final Report Greg Rogencamp, Phillip Ryan
Synopsis :	Report for the Williams River Flood Study. Report covers data collection, development and calibration of computer models, flood frequency analysis, establishment of design flood behaviour and flood mapping.

REVISION/CHECKING HISTORY

REVISION NUMBER	DATE OF ISSUE	CHECKED BY		ISSUED BY	
0	21 Feb 2007	WJS		GJR	
1	DRAFT				
2	11 July 2008	PAR		GJR	
3	22 August 2008	PAR		GJR	
4	5 Sept 2008	GJR		PAR	
5	18 June 2009	SAW		PAR	

DISTRIBUTION

DESTINATION	REVISION			
	2	3	4	5
Wal Mills	1	1	10	20
Peter Medi	1	1		
Greg McDonald	1	1		
Greg Bernard	1	1		
FPM Committee		6		
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FOREWORD

The NSW State Government's Flood Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the NSW State Government's Floodplain Management Manual (FMM, 2001).

Under the Policy the management of flood prone land remains the responsibility of Local Government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the State Government through the following four sequential stages:

Stages of Floodplain Management

	Stage	Description
1	Flood Study	Determines the nature and extent of the flood problem.
2	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed developments.
3	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain.
4	Implementation of the Plan	Construction of flood mitigation works to protect existing development. Use of environmental plans to ensure new development is compatible with the flood hazard.

This study represents the first of the four stages for the Williams River area. It has been prepared for Port Stephens Council and Dungog Shire Council to describe and define the existing flood behaviour and establish the basis for floodplain risk management activities in the future.

This study was funded by Commonwealth and NSW State Assisted Floodplain Management Program in conjunction with Dungog Shire and Port Stephens Council.

EXECUTIVE SUMMARY

Introduction

Dungog Shire Council and Port Stephens Councils have commissioned this study in order to define the riverine flood behaviour in the Williams River from Raymond Terrace to 5km above Dungog. The study encompasses the Lower Hunter River (from Green Rocks to Newcastle Harbour) and investigates the effect of combined flooding from both Hunter and Williams Rivers.

The study is aimed at producing information on flood behaviour for a wide range of flood events under existing floodplain conditions. The study uses the most up-to-date modelling techniques currently available. This study will form the basis of the subsequent floodplain management study.

Data Collection

The data collection phase involved the following major steps:

- Review of reports, studies and files on previous flood assessments in the study area;
- Collection of floodplain topography using aerial photogrammetry
- River cross-sections from three different sources (DOC, DNR and a survey carried out by HWA for this study);
- Data on river and floodplain structures including numerous bridges, levee data, Seaham Weir data and details of Chichester Dam and Grahamstown Dam;
- Rainfall data including daily rainfall stations and pluviographic rainfall data;
- Streamflow data sourced from Hunter Water Corporation, Manly Hydraulics Laboratory and DNR;
- Historical flood records sourced from DNR Williams River files, stream gauges within the catchment, Department of Main Roads reports and a resident survey commissioned for this study. There were 42 responses from the resident survey, which in turn yielded approximately 25 flood records.

Historical Flood Behaviour and Flooding Mechanisms

The Williams River catchment extends from Raymond Terrace, approximately 20km north-west of Newcastle, to the Barrington Tops. The Williams River flows into the Hunter River at Raymond Terrace. The Hunter River enters the Pacific Ocean at Newcastle Harbour. The Williams River catchment is approximately 1,100 km^2 in area.

The Chichester Dam is located at the confluence of the Chichester and Wangat Rivers, in the upper catchment. Chichester Dam has a capacity of 21,500 megalitres at the full supply level of 156.2mAHD.

Grahamstown Dam is located in the south-east of the Williams River catchment. The dam is primarily a water supply source for the region.

A network of levee banks has been constructed along the banks of the Williams River between Seaham and the confluence with the Hunter River.

A major weir on the Williams River has been constructed at Seaham that limits the tidal influence on the river, allowing water to be extracted from the Williams River.

The Williams River has a long history of flooding. Resident survey and interviews revealed recollection of numerous floods of various magnitudes. Twenty flood events with an estimated flow at Glen Martin of greater than 1,000m^3/s have occurred. The largest of these (at both Dungog and Glen Martin) was in March 1963.

In the lower Williams River flood levels are also affected by flooding in the Hunter River. In February 1955 there was a major flood in the Hunter River and lower Williams River. The February 1955 flood event resulted in 14 deaths and approximately 18,000 homes being flooded.

Hydrology and Flood Frequency Analysis

Hydrologic modelling calculates the quantity and rate of catchment runoff from rainfall during a flood event. A hydrological model (RAFTS-XP) was developed of the Williams River catchment using topographical data. The catchment was divided into 59 sub-areas.

It became apparent during the calibration process that the high flows derived using the DNR rating curve for the Dungog gauge did not match well with the modelled flows. As well, a comparison of the peak flows compared with those at Glen Martin during the same flood event highlighted some inconsistencies.

In order to assist in determining the relationship between flood levels recorded at the Dungog gauge and flows in the river and over the floodplain, a local 2D/1D flood model of the area was developed. The model was run for a synthetic flood event with a rising hydrograph. The levels calculated by this model at the Dungog gauge location were compared with the flows at the Dungog road bridge. Thus, a revised rating curve was derived.

The hydrological model was calibrated to the February 1990, March 1978 and May 2001 flood events. The 1955 flood was considered for the calibration of the Williams River flood model. However, this flood was relatively small in the Williams River catchment and was primarily a Hunter River flood event that resulted in significant back-up flooding in the lower Williams River floodplain. Furthermore, the calibration of Lower Hunter River flood models to this flood event have been well documented in other studies (eg. Lower Hunter River Flood Study).

The following conclusions can be drawn from the calibration of the hydrological model to these three flood events:

- The hydrological model is quite sensitive to the spatial distribution of the rainfall (and probably the temporal distribution). The quality of the model calibration exercise is compromised by a lack of pluviograph stations in the catchment;
- The hydrological model adequately represents the attenuation and flood wave propagation characteristics of the catchment (upstream of Dungog);

In summary, the hydrological model (upstream of Dungog) provides an adequate representation of dynamic flows from the catchment for the purposes of this study and subsequent floodplain management studies.

A flood frequency analysis was completed by BMT WBM using available data from both the Dungog and Glen Martin gauges. Professor George Kuczera (The University of Newcastle Research Associates) was commissioned to conduct an independent flood frequency analysis on the Williams River data. The results of these two flood frequency analysis are presented below.

Table ES-1 Flood Frequency Analysis Results

AEP	ARI (years)	Dungog Flow (m^3/s) GP: Peak Over Threshold	Glen Martin Flow (m^3/s) Annual Max GEV	Glen Martin Flow (m^3/s) Annual Max LPIII	Glen Martin Flow (m^3/s) FLIKE
20%	5	833	1036	1027	
10%	10	1105	1366	1333	
5%	20	1408	1629	1590	1665
2%	50	1863	1883	1864	2008
1%	100	2253	2017	2031	2230
0.5%	200	2688	2114	2169	2424

Hydraulic Modelling

The Williams River 2D/1D hydraulic model covers an area of 146 km^2 from approximately 5 km upstream of Dungog down to Raymond Terrace (at the junction with the Hunter River). The model is based on a 40m square grid, resulting in approximately 90,000 2D cells, with 163 1D sections representing the Williams River and tributaries. The TUFLOW software was used to develop and simulate the hydraulic model.

The model represents the topography of the river and floodplain, all major road and rail crossings, Seaham Weir, all major river levees (including flood-gates) and varying vegetation cover on the floodplain and river-banks.

There is considerable interaction between flooding in the lower parts of the Williams River and the Hunter River. Hence, the 2D1/D TUFLOW model of the Williams River was linked to a 2D/1D TUFLOW model of the Hunter River. This Hunter River model was developed as part of a project for the Roads and Traffic Authority (RTA) and is being used for investigations into a new Pacific Highway (F3) crossing of the Hunter River.

The hydraulic model was calibrated to the February 1990, March 1978 and May 2001 flood events and involved the following general approach:

- The February 1990 flood event calibration was used to calibrate the lower section of the Williams River model and the lower Hunter River model as it was the calibration event with the largest Hunter River flows (coincident with a Williams River flood).
- The March 1978 flood event calibration was used to calibrate the section of river from Clarence Town to Seaham as it contained good peak level data along this reach. This flood event also had relatively well defined flood peaks, enabling calibration of the timing of the flood peak movement down the river system.

- The May 2001 flood event was used as a general verification event following the above two steps. The May 2001 flood is the only significant flood event since the installation of the automated recorder at Dungog (even though it was not a major flood event).

The following conclusions can be drawn on the calibration of the hydraulic model to three flood events:

1 The Manning's n value for the lower reaches of the Williams River (upstream of Seaham) of 0.05 is derived from the calibration of the flood gradient in the March 1978 flood. The Manning's n value for the upper reaches of the Williams River (around Dungog) of 0.08 is derived from the calibration to the gauging station recorded flows and levels for the March 2000 flood event. The Manning's n values for the reaches in between are a linear variation of these values.

2 The shape of the hydrograph and the speed of propagation of the flood wave along the river are considered acceptable given the uncertainties in the input data. This is demonstrated in the calibration to the March 1978 and May 2001 flood events;

3 The combined Williams River and Hunter River 2D/1D model is acceptable at replicating floods in both river systems as demonstrated by the February 1990 flood.

In summary, the hydraulic 2D/1D model (linked to the Hunter River 2D/D model) provides an adequate representation of dynamic flood behaviour in the study area for the purposes of this study and subsequent floodplain management studies. However, it needs to be noted that the model is aimed at representing long duration flood events dominated by Williams River flows (and subsequent back-up in tributaries) and not the finer scale flood behaviour and steeper flood gradients of small tributary inflows.

Design Flood Modelling

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.

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 hydrological model at Dungog was 4,010m^3/s (using 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.

Hence, 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.

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.

Probable Maximum Precipitation (PMP) and 0.5% AEP rainfall intensity estimates have been derived using the techniques presented in the latest edition of Australian Rainfall and Runoff (2001, Book VI).

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.

A matrix of design events was agreed upon with the flood study technical committee and is presented below.

Table ES-2 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

The downstream boundary conditions (ie the assumed water levels in the ocean during the flood events) were derived from previous studies.

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.

The following conclusions were drawn from the testing of the sensitivity of the hydrological model to various model assumptions and parameters:

- The peak flows are moderately sensitive to assumptions of initial and continuing rainfall losses. The values of 0mm initial loss and 2mm/hr continuing loss derived from the calibrated hydrological model have been used in design flood modelling;
- The peak flows are moderately sensitive to assumptions regarding the non-linearity exponent of the model. The value of –0.285 derived from the calibrated hydrological model has been used in design flood modelling;
- The peak flows are relatively insensitive to assumptions regarding the storage delay time coefficient (B) of the model. The value of 0.8 derived from the calibrated hydrological model has been used in design flood modelling.

The following conclusions were drawn from the testing of the sensitivity of the hydraulic model to various model assumptions and parameters:

- The peak flood levels are relatively sensitive to assumptions regarding the Mannings n along the river and on the floodplain;
- The peak flood levels are relatively insensitive to assumptions regarding the hydraulic losses at the major bridge structures.

Design Flood Behaviour

Design levels flood levels, depths, velocities and velocity-depth product are presented for seven design events in the A3 Drawing Addendum (Drawings 2 to 71) and long sections for all design events are also presented.

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

The following comments are made on the results presented in this report.

- 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.

- 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.
- In relation to design model results for flooding dynamics, the following points are made:
  - The lower floodplain 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.
- In relation to design model results for four townships in the study area, the following points are made:
  - 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 Street.
  - 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 Street. In larger events flooding backs up from Rifle Street to Queen Street.
  - Flooding in Seaham 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 St. 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.
  - 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.

Provisional flood hazard mapping has been presented in the report for all seven design flood events. 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.

Three hydraulic categories are defined in the NSW Floodplain Development Manual (2005): floodways, flood storage and flood fringe. These have been mapped for two design flood events (the 0.5% AEP and 1% AEP flood events).

The effect of the assumed storage available in Chichester Dam prior to a flood event was assessed. It was found that peak flows and levels were relatively insensitive to the assumed water level in the dam. If the dam was assumed to be completely empty, the peak flows only decrease by approximately 10%.

A similar assessment was carried out for the assumed storage level in Grahamstown Dam. The assessment indicated that there was little change in predicted peak water levels by varying the initial dam level from full to empty.

Conclusions and Recommendations

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 and a 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.

In conclusion, it is recommended that 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.

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GLOSSARY

annual exceedance probability (AEP)	The chance of a flood of a given size (or larger) occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (i.e. a 1 in 20 chance) of a peak discharge of 500 m^3/s (or larger) occurring in any one year. (see also average recurrence interval)
Australian Height Datum (AHD)	National survey datum corresponding approximately to mean sea level.
average annual damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage. The annual average damage is the average damage per year that would occur in a designated area (e.g. the Casino area) from flooding over a very long period of time. In many years there may be no flood damage, in some years there will be minor damage (caused by small, relatively frequent floods) and, in a few years, there will be major flood damage (caused by large, rare flood events). Estimation of the average annual damage provides a basis for comparing the effectiveness of different floodplain management measures (i.e. the reduction in the annual average damage).
average recurrence interval (ARI)	The long-term average number of years between the occurrence of a flood as big as (or larger than) the selected event. For example, floods with a discharge as great as (or greater than) the 20yr ARI design flood will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event. (see also annual exceedance probability)
catchment	The catchment at a particular point is the area of land that drains to that point.
minimum floor level	The minimum (lowest) habitable floor level specified for a residential building. The minimum operational floor level specified for a commercial/community/industrial building
design flood	A hypothetical flood representing a specific likelihood of occurrence (for example the 100yr ARI or 1% AEP flood).
development	Existing or proposed works that may or may not impact upon flooding. Typical works are filling of land, and the construction of roads, floodways and buildings.
effective warning time	The available time that a community has from receiving a flood warning to when the flood reaches them.
flood	Relatively high river or creek flows, which overtop the natural or artificial banks, and inundate floodplains and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.

flood awareness	An appreciation of the likely threats and consequences of flooding and an understanding of any flood warning and evacuation procedures. Communities with a high degree of flood awareness respond to flood warnings promptly and efficiently, greatly reducing the potential for damage and loss of life and limb. Communities with a low degree of flood awareness may not fully appreciate the importance of flood warnings and flood preparedness and consequently suffer greater personal and economic losses.
flood damage	The tangible and intangible costs of flooding.
flood behaviour	The pattern / characteristics / nature of a flood.
flood fringe	Land that may be affected by flooding but is not designated as floodway or flood storage.
flood hazard	The potential risk to life and limb and potential damage to property resulting from flooding. The degree of flood hazard varies with circumstances across the full range of floods.
flood level	The height or elevation of floodwaters relative to a datum (typically the Australian Height Datum). Also referred to as "stage".
flood liable land	see flood prone land
floodplain	Land adjacent to a river or creek that is periodically inundated due to floods. The floodplain includes all land that is susceptible to inundation by the probable maximum flood (PMF) event.
floodplain management	The co-ordinated management of activities that occur on the floodplain.
floodplain management measures	A range of options that are aimed at reducing the impact of flooding. These can include flood, property and response modification measures. Preparation of a floodplain risk management plan requires a detailed evaluation of a range of floodplain management measures.
floodplain risk management plan	A document outlining a range of actions aimed at improving floodplain management. The plan is the principal means of managing the risks associated with the use of the floodplain. A floodplain risk management plan needs to be developed in accordance with the principles and guidelines contained in the NSW Floodplain Management Manual. The plan usually contains both written and diagrammatic information describing how particular areas of the floodplain are to be used and managed to achieve defined objectives.
floodplain management scheme	A floodplain management scheme comprises a combination of floodplain management measures. In general, one scheme is selected by the floodplain management committee and is incorporated into the plan.

Flood planning levels (FPL)	Flood planning levels selected for planning purposes are derived from a combination of the adopted flood level plus freeboard, as determined in floodplain management studies and incorporated in floodplain risk management plans. Selection should be based on an understanding of the full range of flood behaviour and the associated flood risk. It should also take into account the social, economic and ecological consequences associated with floods of different severities. Different FPLs may be appropriate for different categories of landuse and for different flood plans. The concept of FPLs supersedes the "standard flood event". As FPLs do not necessarily extend to the limits of flood prone land, floodplain risk management plans may apply to flood prone land beyond that defined by the FPLs.
flood prone land	Land susceptible to inundation by the probable maximum flood (PMF) event. Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Risk Management Plans should encompass all flood prone land (i.e. the entire floodplain).
flood proofing	Measures taken to improve or modify the design, construction and alteration of buildings to minimise or eliminate flood damages and threats to life and limb.
flood source	The source of the floodwaters. In this study, the Richmond River is the primary source of floodwaters.
flood storage	Floodplain area that is important for the temporary storage of floodwaters during a flood.
floodway	A flow path (sometimes artificial) that carries significant volumes of floodwaters during a flood.
freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determing the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
historical flood	A flood that has actually occurred.
hydraulic	The term given to the study of water flow in rivers, estuaries and coastal systems.
hydrograph	A graph showing how a river or creek's discharge changes with time.
hydrology	The term given to the study of the rainfall-runoff process in catchments.
intangible damages	Non-monetary damages that arise from the adverse social and environmental effects caused by flooding (e.g. personal injury, stress, anxiety)
peak flood level, flow or velocity	The maximum flood level, flow or velocity that occurs during a flood event.
probable maximum flood (PMF)	An extreme flood deemed to be the maximum flood likely to occur.

probability	A statistical measure of the likely frequency or occurrence of flooding.
runoff	The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek.
stage	See flood level.
stage hydrograph	A graph of water level over time.
tangible damages	Monetary losses that are directly attributable to flooding (e.g. damage to houses, loss of business)
velocity	The speed at which the floodwaters are moving. A flood velocity predicted by a 2D computer flood model is quoted as the depth averaged velocity, i.e. the average velocity throughout the depth of the water column. A flood velocity predicted by a 1D or quasi-2D computer flood model is quoted as the depth and width averaged velocity, i.e. the average velocity across the whole river or creek section.
voluntary house purchase	A floodplain risk management measure to purchase residential properties located in high floodway hazard areas.
voluntary house raising	A floodplain risk management measure to raise applicable residential buildings and reduce the risk of above floor flooding.
water level	See flood level.

LIST OF ABBREVIATIONS

AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
AR&R	Australian Rainfall and Runoff
cm	centimetre
cumecs	cubic metres per second
DEM	Digital Elevation Model
DIPNR	Department of Infrastructure, Planning and Natural Resources
DLWC	Department of Land and Water Conservation
DTM	Digital Terrain Model
GEV	Generalised Extreme Value
GIS	Geographic Information System
km	kilometre
LGA	Local Government Area
m	metre
m^3/s	cubic metres per second
m AHD	Elevation in metres relative to the Australian Height Datum
PMP	Probable Maximum Precipitation
PMF	Probable Maximum Flood
PW (or PWD)	NSW Public Works (or Public Works Department) (now Department of Public Works and Services)
TIN	Triangular Irregular Network
WRC	NSW Water Resource Commission (now Department of Infrastructure, Planning and Natural Resources)

1 INTRODUCTION

1.1 Purpose of Study

The purpose of the study is to define the riverine flood behaviour in the Williams River from Raymond Terrace to 5km above Dungog. The study encompasses the Lower Hunter River (from Green Rocks to Newcastle Harbour) and investigates the effect of combined flooding from both Hunter and Williams rivers.

The study is aimed at producing information on flood behaviour for a wide range of flood events under existing floodplain conditions. The study uses the most up-to-date modelling techniques currently available. This study will form the basis of the subsequent floodplain management study.

1.2 Elements of Study

The general approach and methodology employed to achieve the study objectives involved the following steps (as shown in Figure 1-1):

- Compilation and review of available information
- Acquisition of additional data, including resident survey to determine nature and extent of historical flooding
- Development of hydrological and hydraulic models
- Calibration and verification of models
- Modelling of design events under existing conditions
- Reporting and mapping

The basic methodology above was utilised to complete the study. Details of the full methodology are provided in the following sections.

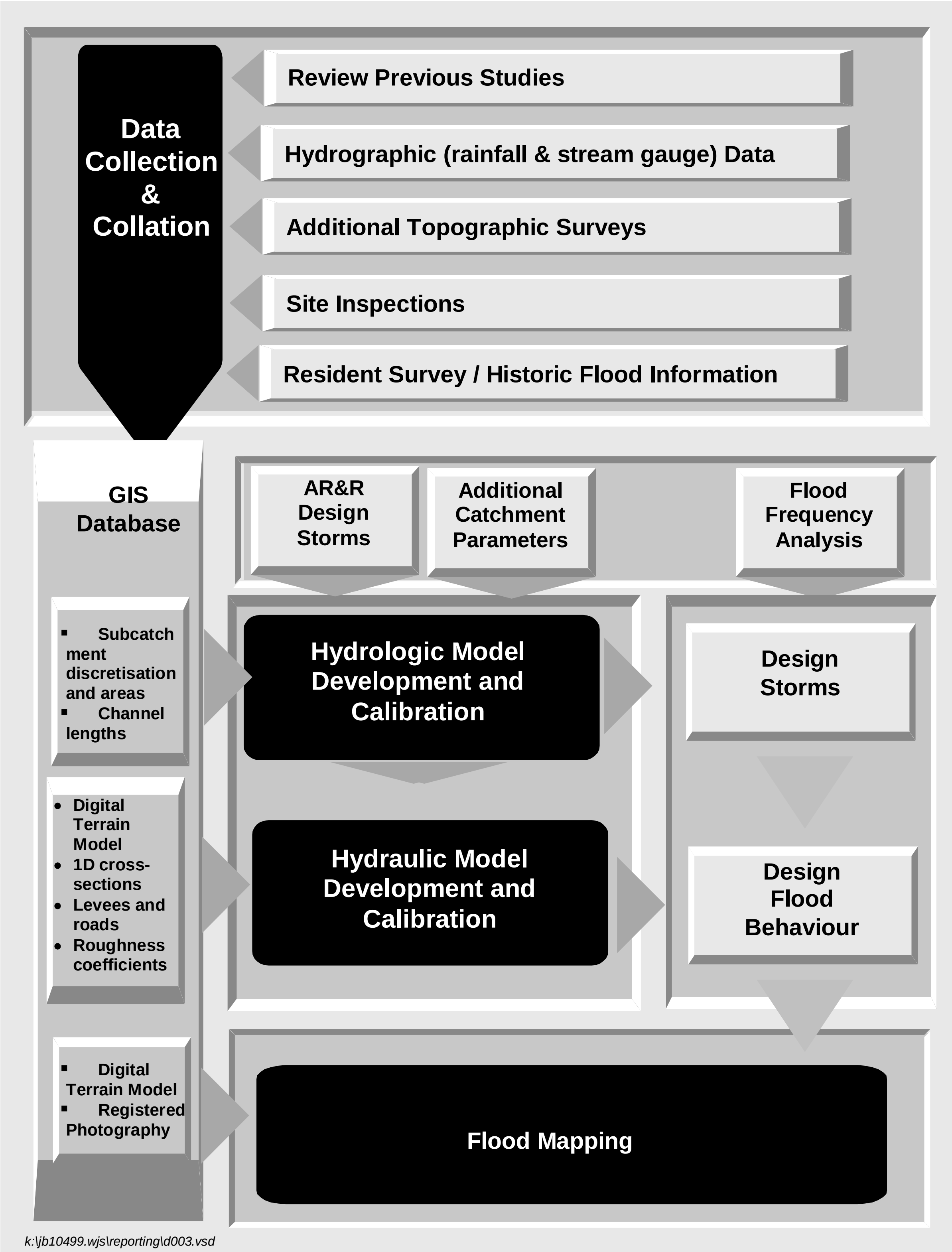

Figure 1-1 Study Approach

2 BACKGROUND

Flood investigations carried out in the past have addressed various aspects of flooding in the Williams and Hunter rivers. There have been numerous studies on flooding in the Williams River and lower Hunter Rivers.

Studies relevant to the current flood study are reviewed in Section 3.1. Relevant studies are:

- Cameron McNamara Consultants (1985) *New Bridge Over Williams River At Dungog*
- Kinhill Engineers Pty Ltd (1993) *Grahamstown Dam Augmentation Hydrologic And Hydrology Report* Hunter Water Corporation Ltd
- New South Wales Public Works Department (1994) *Lower Hunter River Flood Study (Green Rocks To Newcastle)* Newcastle City Council and Port Stephens Council
- Umwelt (Australia) Pty Limited (1996) *Seaham Weir Operation Study And Plan*
- Lawson and Treloar (1999) *Design Water Levels in Newcastle Harbour – Joint Probability Study*
- GHD (2000) *Grahamstown Dam Stage 2 Augmentation Mike11 Analysis – Irrawang Swamp Area* Hunter Water Corporation
- WBM Oceanics Australia (2001) *Paterson River Floodplain Management Study and Plan* Port Stephens Council and Dungog Council
- Hunter Water Corporation (2004) *Grahamstown Dam Augmentation Stage 2 Design and Calculation Report*

3 DATA COLLECTION

3.1 Review of Previous Reports and Studies

Previous studies relevant to the current flood study were acquired and reviewed. The most relevant aspects of these previous studies are described below.

3.1.1 New Bridge Over Williams River At Dungog

Cameron McNamara (1985) investigated a number of road alignments for the replacement of the road bridge at Dungog. This report details historical flooding at the site, design flood levels/discharges, modelling undertaken for bridge assessments and possible route alternatives. A number of historic flood levels detailed in this report were used in the flood frequency analysis conducted as part of this flood study.

3.1.2 Grahamstown Dam Augmentation Hydrologic And Hydrology Report

Kinhill Engineers Pty Ltd (1993) completed this study as part of the Grahamstown Dam Augmentation. This study involved the use of calibrated hydrologic and hydraulic models to assess impacts to magnitude and frequency of flooding from augmentation of Grahamstown Dam. The study utilised RORB and MIKE-11 models for hydrologic and hydraulic modelling. A design peak discharge at Glen Martin of 2,550m^3/s was adopted in the study. The study concluded that flooding at Raymond Terrace is caused by Hunter River derived flooding, not flows generated from the Grahamstown Dam or Williams River catchments.

3.1.3 Lower Hunter River Flood Study (Green Rocks To Newcastle)

The NSW Public Work Department (PWD, 1994) investigated flooding in the lower Hunter River. The study utilised computer modelling to determine flood behaviour in the Hunter River from Green Rocks to Port Newcastle. The study used the software packages RORB and MIKE-11 respectively for hydrologic and hydraulic modelling. Models where calibrated to recorded levels for the 1955 and 1971 flood events and verified with 1972, 1977, 1978 and 1985 events. Calibrated models were used to define water levels and velocities for a selection of design events.

To determine design flows in the Williams River to the Hunter River, the PWD (1994) Lower Hunter River Study involved a flood frequency analysis for flows in the Williams River at Glen Martin. An annual series Log-Pearson III frequency was completed on 65 years of flow data in the Williams River at Glen Martin. The flood frequency analysis estimated a 1% AEP discharge at Glen Martin of 2,680m^3/s.

3.1.4 Seaham Weir Operation Study And Plan

The Seaham Weir Operation Study And Plan (Umwelt, 1996) describes the weir and the operation of the weir. Various aspects of the weir are detailed including structure and operation, hydrological aspects, water quality and environmental consideration. The report outlines an operation plan for the weir. Sections of the report most relevant to the current flood study are the operation and structure of the weir.

3.1.5 Design Water Levels in Newcastle Harbour – Joint Probability Study

This study, completed by Lawson and Treloar, for Newcastle City Council quantified design water levels in Newcastle Harbour for a range of exceedance probabilities. The design levels presented in the joint probability study are used to define downstream boundary conditions of the design events presented in this study (with allowance for Enhanced Greenhouse Effect). See Section 7.2.2 for more details on downstream boundary conditions used in this study.

3.1.6 Grahamstown Dam Stage 2 Augmentation Mike11 Analysis – Irrawang Swamp Area

GHD completed an investigation of effects of Grahamstown Dam Stage 2 Augmentation on the flooding in Irrawang Swamp. The study utilised a Mike11 model. The study investigated flows and levels at the Pacific Highway Culvert and Newline Road Bund.

3.1.7 Paterson River Flood Study and Floodplain Management Studies

The Paterson River Floodplain Management Study was completed by Bewsher Consulting Pty Ltd. BMT WBM completed Volume 3 of the study encompassing the hydraulic investigations. This study builds upon the Paterson River Flood Study (WBM, 1997).

The flood study report details the development and calibration of hydrologic and hydraulic models and defines design flood behaviour for a number of annual exceedance probability events.

3.1.8 Grahamstown Dam Augmentation Stage 2

Hunter Water Corporation (HWC) provided sections of the “Grahamstown Dam Augmentation Stage 2 Design and Calculation Report” to BMT WBM. Sections provided by HWC detail the Stage-Storage relationship of Grahamstown Dam. The report details of the spillway rating curve derivation.

3.1.9 DNR Williams River Files

BMT WBM was given access to DNR records of flooding in the Williams River. The DNR files contained a number of documents that were useful either as background information or directly relevant to this study, these include:

- Recorded maximum flood levels for the March 1978 flood event, these were recorded on flood boards between Seaham and Glen Martin
- Annual and Partial Series flood frequency analysis for Glen Martin Gauge
- Peak recorded flood levels at Dungog for various events
- Long Profiles of flooding in the Williams River between Glen Martin and Seaham
- Recorded levels upstream and downstream of Seaham weir for various events

3.2 Topographical Data

Several sources of topographic data were required for hydrologic and hydraulic model development. These sources are detailed below. Location of the various data sources is presented in Drawing 1.

3.2.1 Floodplain Topography

Aerial photogrammetry of the study area was provided by QASCO. This data consists of break lines, contours and spot elevations on a 1:16000 digital orthophoto background. The vertical accuracy of the photogrammetry is +/- 0.5 metre.

A Digital Elevation Model (DEM) is a three-dimensional representation of the ground surface. A DEM is used to define the ground surface elevations over the floodplain. A DEM based on the aerial photogrammetry was created.

Road and railway break lines from the photogrammetry are inputted directly in to the hydraulic model to ensure correct representation of these critical structures. More details on modelling of ridges and gullies using break lines is given in 6.1.2

3.2.2 DOC Survey River Cross-sections

The Department of Commerce (DOC) provided 22 cross sections of the Williams River. These cross sections were surveyed in 1993. The location of the cross-sections is presented in the hydraulic model development section Drawing 1.

3.2.3 DNR Survey River Cross-sections

DNR provided 16 cross-sections of the Williams River as well as numerous cross-section of the Hunter River. The location of the cross-sections is presented in the hydraulic model development section Drawing 1. The cross-sections were surveyed in May 2005.

3.2.4 HWA Survey River Cross-sections

BMT WBM commissioned Hunter Water Australia (HWA) to conduct additional hydrosurvey sections of the Williams River. 93 additional sections were surveyed; these extend from Seaham to the limit of the study area. Cross-sections in the Williams River, Myall and Carowiry Creeks were provided. HWA were also commissioned to provide a long-section of the river bed between Clarence Town and Seaham.

The location of the cross-sections is presented in the hydraulic model development section Drawing 1.

3.3 Structure Data

3.3.1 Bridge Data

Dungog Road Bridge

There are a number of key bridges in the study area:

- Rail bridge over Williams River at Dungog
- Rail bridge over Myall Creek at Dungog
- Stroud Hill Road over Williams River at Dungog
- Stroud Hill Road over Myall Creek at Dungog
- Alison Road, between Dungog and Clarence Town
- Pine Brush Road at Glen William
- Durham Street/Limeburners Creek Road at Clarence Town
- East Seaham Road at Seaham
- Seaham Road/ William Bailey Street at Raymond Terrace
- Details of rail bridges were obtained from Australian Rail Track Corporation (ARTC). Major road crossings (Dungog, Clarence Town and Raymond Terrace) were obtained from Roads and Traffic Authority NSW (RTA). Drawings for local roads were provided by Dungog Shire Council and Port Stephens Councils. Details for the Stroud Hill Road Bridge over Williams River at Dungog were also obtained from Cameron McNamara Report (1985).

3.3.2 Levee Crest Data and Floodgate Data

Survey of the levee crest was provided by the former Department of Natural Resources. Survey included break lines along the length of the levee system. Details of floodgates through levee were provided including shape, size, number of barrels, and inverts levels.

3.3.3 Chichester Dam Details

Hunter Water Corporation provided the Stage-Storage-Discharge relationship for the Chichester Dam. Daily levels and spillway flows were provided for Chichester Dam for the 1990 and 2001 historic events. Hunter Water Corporation unable to provide levels and spillway flows for 1978 event.

Grahamstown Dam

3.3.4 Grahamstown Dam Details

Current Stage-Storage and Stage-Discharge relationships were provided by Hunter Water Corporation.

Daily levels were provided for Grahamstown Dam for 1978, 1990 and 2001 events. No flows over the spillway were recorded for the historic events used for calibration/verification.

The Grahamstown Dam spillway was upgraded in 2005 (completed December 2005). The works involved the construction of a larger spillway at Irrawang and a discharge channel under the Pacific Highway.

3.3.5 Seaham Weir Details

Hunter Water Corporation provided details of the Seaham weir layout and elevations. Summary details of the three main weir sections are:

- 2 x 20m vertical lift gates, invert of -0.5mAHD.
- 120m long concrete spillway at 1.178mAHD.
- Rock filled wall, approximate length 220m, elevation varies minimum elevation 1.922mAHD.

Ground survey along the crest of the rockwall was provided, this survey was undertaken in 2005.

Daily water levels (8am reading) in the Seaham weir pool were provided for February 1990 and May 2001 flood events. DNR records included peak level upstream of the weir for the March 1978 event.

The construction of the current floodgate and spillway structures was undertaken between 1977 and 1978. Hunter Water Corporation was able to provide a photo of the weir as at July 1978. No record of the status of the weir during the March 1978 flood could be found.

Seaham Weir (Photo: HWC)

3.4 Rainfall Data

3.4.1 Daily Rainfall Stations

There are a 27 daily rainfall stations that have data for at least one of the calibration events event. The stations are within or close to the Williams River catchment. A summary of the events recorded at each station is presented in Table 3-1. Stations listed with “Accumulation” for an event indicate that the station was operational but at least one of the daily readings was an accumulation of rainfall over multiple days. The location of the daily rainfall stations is presented in Figure 3-1.

Table 3-1 Summary Daily Rainfall Records

Station Number	Station Name	March 1978	February 1990	May 2001
60089	Moana	Accumulation	No Record	No Record
61010	Clarence Town (Grey St)	Recorded	Recorded	Recorded
61017	Dungog Post Office	Accumulation	Recorded	Accumulation
61024	Gresford Post Office	Recorded	Recorded	Recorded
61031	Raymond Terrace (Kinross)	Recorded	Recorded	Recorded
61046	Morpeth Post Office	Accumulation	Accumulation	Accumulation
61064	Raymond Terrace Post Office	Accumulation	Accumulation	No Record
61068	Salisbury Post Office	Accumulation	No Record	No Record
61071	Stroud Post Office	Accumulation	Recorded	Accumulation
61076	Wallaroo State Forest	Recorded	Recorded	No Record
61078	Williamtown RAAF	Recorded	Recorded	Recorded
61096	Paterson Post Office	Accumulation	Recorded	Accumulation
61122	Tillegra	Recorded	No Record	No Record
61129	Kinross	Recorded	No Record	No Record
61136	Barrington Guest House	Recorded	Recorded	Accumulation
61151	Chichester Dam	Recorded	Recorded	Recorded
61169	Durham Park	Recorded	No Record	No Record
61170	Dungog - Main Ck (Yeranda)	Recorded	Recorded	Recorded
61189	Shellbrook	Recorded	No Record	No Record
61238	Pokolbin (Somerset)	Recorded	Recorded	Accumulation
61250	Tocal AWS	Recorded	Recorded	Recorded
61258	Gostwyck House	Recorded	No Record	No Record
61292	Masseys Ck (Glengarvan)	Recorded	Recorded	Recorded
61311	Grahamstown (Hunter Water Board)	Recorded	Recorded	Recorded
61350	Upper Chichester (Simmonds)	No Record	Recorded	Recorded
61361	Wallaringa	No Record	Recorded	No Record
61364	Dungog (Leawood)	No Record	Recorded	Recorded

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3.4.2 Pluviograph Stations

There are four pluviograph stations within or close to the Williams River catchment. Chichester Dam and Williamtown stations recorded all three calibration/verification events. Grahamstown Dam and Upper Allyn (Bald Knob) stations recorded only the March 1978 event. A summary of the rainfall records for each of the calibration/verification events is presented in Table 3-2. The location of the pluviograph stations is presented in Figure 3-1.

Table 3-2 Summary Pluviograph Records

Station Number	Station Name	March 1978	February 1990	May 2001
61151	Chichester Dam	Recorded	Recorded	Recorded
61311	Grahamstown	Recorded	Not Recorded	Not Recorded
61078	Williamtown	Recorded	Recorded	Recorded
61325	Upper Allyn (Bald Knob)	Recorded	Not Recorded	Not Recorded

3.5 Streamflow Data

There are a number of stream gauging stations within the Williams River catchment. Some stations have been recently installed or upgraded and data was limited or not available for historic events. Data varies in frequency from peak record only to automated recorder. A summary of the available streamflow data is presented in Table 3-3. The location of the river level recording stations is presented in Figure 3-1.

Streamflow data was sourced from the following organisations:

- Hunter Water Corporation
- Manly Hydraulics Laboratory
- DNR
- Bureau Of Meteorology

Table 3-3 Summary Streamflow Data

Gauge Location	March 1978	February 1990	May 2001
Chichester River	No	No	Yes
Chichester Dam	Daily Levels	Daily Levels	Daily Levels
Wangat River	No	No	Yes
Williams River at Tillegra	Yes - Limited Points, SES data	Yes	Yes
Williams River at Dungog	Some data from SES	Some data from SES	Yes
Williams River at Glen Martin	Yes - Limited Points	Yes	Yes
Williams River at Seaham	Peak - time and level only	No	Yes
Williams River at Raymond Terrace	Peak - time and level only	Yes	Yes
Grahamtown Dam	Daily Levels	Daily Levels	Daily Levels

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3.6 Historical Flood Records

Information on flooding within the study area has been previously collected during surveys after floods and during previous studies on flooding. The following sources yielded useful data:

- DNR Williams River Files (See Section 3.1.9)
- Stream gauges within the catchment (See Section 3.5)
- Department of Main Roads Report (See Section 3.1.1)

To supplement these data an historical flood information survey was carried out as discussed in the next section.

3.6.1 Resident Survey

An extensive survey of residents within the study area was conducted in July 2006. The purpose of the resident survey was to gather historical from those who have experienced Williams River floods and to identify local concerns within the region.

The resident survey consisted of the following steps:

- Questionnaire mailed to residents
- Submitted questionnaires reviewed and prioritised
- Interviews of residents by WBM staff
- Survey of provided flood levels

Information requested in the questionnaire covered a range of issues including floods experienced, damages incurred, flood behaviour, isolation, flood warning and perceived cause of flooding. Two versions of the questionnaire were produced; one for residents in Dungog Shire Council and one for residents in Port Stephens Council. The questionnaire consisted of a map and three pages of questions. The questions remained the same over both council areas and the maps changed. The questionnaire is presented in Appendix B.

The knowledge of local residents about the flooding of the Williams and lower Hunter Rivers was found to be invaluable in developing an understanding of flooding in the Williams River and the interactions with the Hunter River. A number of flood heights were identified thanks to local knowledge and records of flood heights. These flood heights were surveyed and are presented in Section 3.6.2.

A summary of resident questionnaire results is presented in Appendix C.

Key observations on flood behaviour arising from resident knowledge:

- Water breaks the Williams River levees in the upper sections and progressively fills the sections on the western bank of the lower Williams River. These sections behind the main river levee fill till the height of the levees are overtopped causing the next section to fill. The western floodplain of the Williams River (between Seaham and Raymond Terrace) acts as a series of cascading pools.

- Turbulence at the confluence of the Williams and Hunter Rivers was noted
- Flow in the upstream direction over the Seaham weir was reported
- When the Hunter River is in flood at the same time this effectively leaves the Williams River flows with no where to go. This causes the most severe flooding, as the Williams River flows are unable to escape.

A summary of the number of responses with flood information for various years is presented in Table 3-4.

Table 3-4 Flood Information Responses for Various Years

Year	Number of Responses
1990	10
2000	6
2001	4
1955	2
1974	1

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

3.6.2 Survey of Flood Records

BMT WBM documented locations of peak flood levels for historic events provided by residents. Monteath and Powys surveyors to survey these flood marks using ground survey techniques. A total of 20 points were surveyed. Details of these flood marks are presented in Table 3-5.

Table 3-5 Locations Surveyed – BMT WBM Resident Interviews

Address	Flood Event	Surveyed Level (mAHD)
33 Hunter street, Raymond Terrace	May 2001	2.24
33 Hunter street, Raymond Terrace	May 2001	2.43
2 King street, Raymond Terrace	1955	6.04
2 King street, Raymond Terrace	1990	2.20
61 Sturgeon street, Raymond Terrace	1955	5.60
61 Sturgeon street, Raymond Terrace	1990	3.00
5 Still street, Seaham	2000	3.08
8 Holmwood road, Seaham	1990	3.77
1357 Clarencetown road, Seaham	1990	9.54
180 Fords road, Clarencetown	1990	8.81
15 Durham street, Clarencetown	1990	5.18
543 Glen Martin road, Glen Martin	1990	12.72
1770 Clarencetown road, Glen Oak	2001	3.92
Alison road, south of Pinebrush road intersection	2001	32.36
320 Marshdale road, Marshdale	1990	53.99
Mill Race Gauging station, Fosterton road	for Gauge Records	49.10
1134 East Seaham road, Clarencetown	NA	4.73
1134 East Seaham road, Clarencetown	1990	6.77
393 Italia road, Balickera	1990	2.86
393 Italia road, Balickera	2001	3.08

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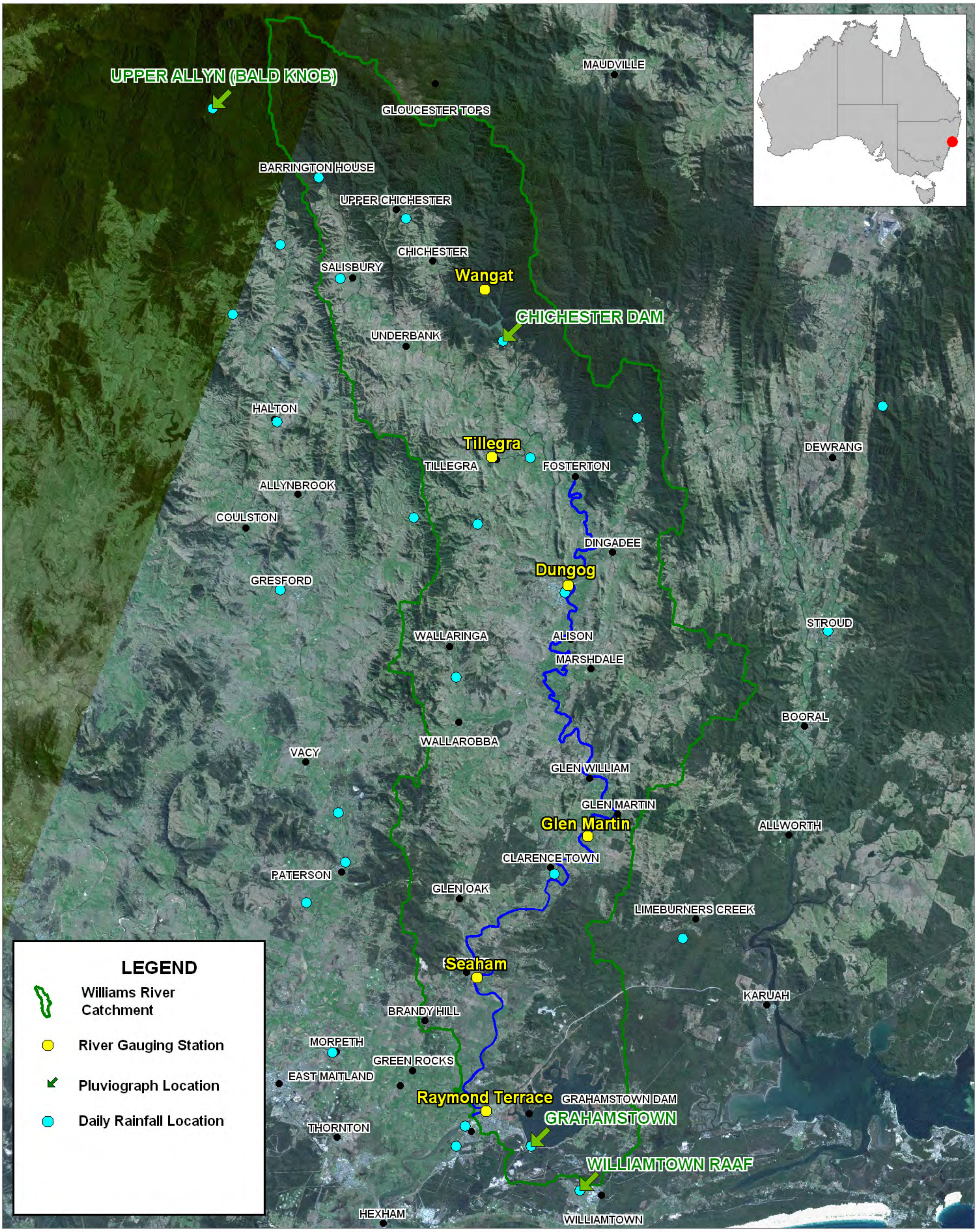

Title:

Rainfall and River Gauging Station Locations

Figure: 3-1

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.

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Approx. Scale

BMT WBM

www.wbmpl.com.au

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4 HISTORICAL FLOOD BEHAVIOUR AND FLOODING MECHANISMS

4.1 Catchment Description

The Williams River catchment extends from Raymond Terrace, approximately 20km north-west of Newcastle, to the Barrington Tops. The Williams River flows into the Hunter River at Raymond Terrace. The Hunter River enters the Pacific Ocean at Newcastle Harbour.

The Williams River catchment is approximately 1,100 km^2 in area. Elevations in the catchment range from greater than 1400mAHD in the Barrington Tops to sea level. The river is tidally influenced from the Hunter River to the Seaham weir. Slopes in the upper catchment vary between 5-25%. Slopes in the lower catchment are generally less than 1%.

The upper sections of the catchment lie within the Barrington Tops National Park and Chichester State Forest. These reserve areas are both densely forested. The majority of the lower catchment is cleared, primarily for agricultural use.

The Williams River is initially a series of steep mountain streams that combine to form a major flowpath. The major tributaries in the upper catchment are Chichester River, Wangat River, Myall Creek and Carowiry Creek.

Major towns in the catchment are Raymond Terrace, Seaham, Clarence Town and Dungog.

4.2 Storages

The Chichester Dam is located at the confluence of the Chichester and Wangat Rivers, in the upper catchment. Chichester Dam has a capacity of 21,500 Megalitres at the full supply level of 156.2mAHD. Chichester Dam was completed in 1926 but has undergone a number of modifications, the most relevant to this study being changes to spillway level in 1965 and 1985.

Grahamstown Dam is located in the south-east of the Williams River catchment. The dam is primarily a water supply source for the region. The dam was constructed between 1955 and 1965.

The dam has its own small catchment but also extracts water from the Williams River. Water is extracted from the Williams River upstream of the Seaham Weir (which limits saltwater intrusion), via the Balickera Canal. The conveyance system includes about 5km of canal including a pumping station partway along canal. (HWC 2008).

Capacity in Grahamstown Dam is 190,000 Megalitres at the full supply level of 12.8mAHD (HWC 2008). A number of upgrades and modifications have been made to Grahamstown Dam since construction was completed. The most relevant to the current study is the construction of the larger spillway at Irrawang completed in December 2005). For more details on the upgraded Irrawang spillway see Section 7.4.2.

4.3 Levees and Flood Gated Culverts

Levee banks have been constructed along the banks of the Williams River between Seaham and the confluence with the Hunter River. The Hunter has levee banks from Green Rocks to approximately 600m downstream of the Williams River confluence. A series of flood gated culverts allows drains behind the levee system to discharge to the Williams and Hunter Rivers

4.4 Bridges and Weirs

A major weir on the Williams River has been constructed at Seaham. The weir is operated by Hunter Water Corporation. The weir limits the tidal influence on the river, allowing water to be extracted from the Williams River. Water is transferred to Grahamstown Dam via the Balickera canal and pump station.

There are a number of road and rail bridges over the Williams River and associated tributaries. A list of major crossings relevant to the current study is detailed in 3.3.1.

4.5 History of Flooding on Williams River

The Williams River has a long history of flooding. Resident survey and interviews revealed recollection of numerous floods of various magnitudes.

20 flood events with an estimated flow at Glen Martin of greater than 1,000m^3/s have occurred. The largest of these (at both Dungog and Glen Martin was in March 1963. The five largest floods recorded at Glen Martin gauge (operational since 1927) were in descending order of magnitude:

- March 1963
- February 1990
- February 1929
- March 1978
- March 1956

In the lower Williams River flood levels are also affected by flooding in the Hunter River. In February 1955 there was a major flood in the Hunter River and lower Williams River. The February 1955 flood event resulted in 14 deaths and approximately 18,000 homes being flooded (Floodplain Development Manual, 2005). This event was a bigger event in the Hunter River than the Williams River, the event is the 17th biggest on record at the Glen Martin gauging station.

More recently in August 2007 a car was fatally swept off a flooded causeway into the Williams River near Bandon Grove north of Dungog.

4.6 General Flooding Mechanisms

Developing an appreciation of the flooding processes on the Williams River is an important step in defining the flood behaviour and developing appropriate computer models.

A general understanding of the different patterns of flooding, or flood behaviour, was obtained based on consultations with local residents and others, and an understanding in flood hydraulics.

For the Williams River, floods originate from one or more of the following sources:

- Heavy prolonged rainfall over the Williams River catchments;
- Flooding of the Hunter River causing a backflow and/or backwater effect in the Williams River. This would be more influential in the lower reaches, with the Seaham Weir spillway (elevation of 1.178mAHD) preventing back flow in smaller events.
- Localised rainfall not being able to drain because of high river levels and/or constrictions caused by the flood drainage structures.

5 HYDROLOGY AND FLOOD FREQUENCY ANALYSIS

5.1 Purpose of Hydrologic Model

Hydrologic modelling calculates the quantity and rate of catchment runoff from rainfall during a flood event. The model produces estimates of the discharges in the river and its tributaries during the course of a flood. The amount of runoff from the rainfall and the attenuation of the flood wave as it travels down the river are dependent on:

- Catchment slope, area, vegetation and other catchment characteristics;
- Variation in the distribution, intensity and amount of rainfall; and
- The antecedent conditions of the catchment.

These factors are represented in the model by:

- Sub-dividing the catchment into a network of sub-catchments inter-connected by channel reaches representing the creeks and rivers. The sub-catchments are delineated so that they each have a general uniformity in their slope, land-use, vegetation density, etc;
- The amount and intensity of rainfall is varied across the catchment based on available information. For the historical events chosen for calibration, a reasonable amount of rainfall information was available;
- The antecedent conditions are modelled by varying the amount of rainfall that is "lost" into the ground and "absorbed" by storages. This is represented in the model by initial and continuing loss values. For very dry antecedent conditions a higher initial rainfall loss typically results. The continuing loss rate is generally a function of ground coverage and soil type.

The output from the hydrologic model is a series of flow hydrographs at selected locations such as at the boundaries of the hydraulic model. These hydrographs are used by the hydraulic model to simulate the passage of the flood down the Williams River and over the floodplains (see Section 6).

5.2 Hydrological Model Development

The Williams River catchment was defined using topographical data. Specifically, a digital terrain model of the catchment area and surrounds was developed using digital topographical maps.

The catchment was divided into 59 sub-areas as shown in Figure 5-1. The delineation of the sub-areas was based on defining somewhat discrete sub-catchments. The overall aim was to make most sub-area approximately the same size.

The hydrological model was developed in RAFTS-XP and parameters of catchment area, mean catchment slope and 'roughness' parameter were input for each sub-area. The catchment area and mean catchment slope parameters were derived from interrogation of the catchment DTM. The 'roughness' parameter was derived from an assessment of the dominant land-use type in each sub-area. For forested areas, the n value used was 0.10 and for cleared land (e.g. pasture), the n value used was 0.05.

The linkages between the sub-areas were originally developed using the link-lag option in RAFTS-XP. This link is rather simplistic as it merely lags the hydrograph from one sub-area but a set time interval determined by the user. This lag time is usually a function of the length of travel to the outlet of the next downstream sub-area and assumed flood wave celerity (i.e. travel hydrograph speed).

It was found that this simplistic link-lag option was not providing sufficient attenuation of the hydrographs and resulted in a peaky hydrograph shape. Numerous trials of various values for the global B value were also tried. However, these trials did not significantly improve the issue.

In order to overcome this issue, the link lags were replaced by routing links. These more complex link types attempt to represent the conveyance capacity of the watercourses linking the sub-areas. A channel routing technique is used by RAFTS-XP, which results in varying lag times as a function of flow, channel shape/channel slope and roughness, which is a more realistic representation of nature.

Chichester Dam was represented in the hydrological model using the standard representation of detention basins provided in XP-RAFTS. The spillway details and rating curves provided by HWA were included in the model. Water levels provided by HWA for the 1990 and 2001 events were used as initial levels. HWA were unable to provide water level for the 1978 flood event, it was assumed to be full for this event.

5.3 Re-Assessment of Dungog Gauge Rating Curve

5.3.1 Background and Need for Re-Assessment

It became apparent during the calibration process that the high flows derived using the DNR rating curve for the Dungog gauge did not match well with the modelled flows. As well, a comparison of the peak flows compared with those at Glen Martin during the same flood event highlighted some inconsistencies.

For example, in the March 1978 flood event, the peak flow at Dungog using the DNR rating curve was 770m^3/s. The catchment above this gauge is 591km^2 and the majority of the intense rainfall for this event fell upstream of this gauge (see Figure 5-3 and Figure 5-4).

In comparison, at Glen Martin the peak recorded flow was 1,770m^3/s. The catchment above the Glen Martin gauge is 977km^2. Hence, for a 65% increase in catchment area, the calculated peak flow (using the DNR rating curve) increased by a factor of 2.3. This does not appear to be plausible given the rainfall distribution for this event and the likelihood of some attenuation of the flood peak between Dungog and Glen Martin.

The highest flow measurement at the Dungog gauge occurred during the March 2000 flood event. Discussions with the DNR officers associated with this measurement confirmed that a peak flow of 462m^3/s was measured at the road bridge when the gauge was recording 7.0m (41.1mAHD). Above this level, the rating curve was derived by linear extrapolation from the two highest rating points. Given that the flow patterns above 7.0m become rather more complex in nature, this approach is considered too simplistic.

Above 7.0m on the gauge, the component of the total flow that is over the floodplain increases. The floodplain at the gauge is around 300m wide (depending on the exact location of measurement).

Discussions with Mr Peter Ryan, who owns the floodplain land adjacent to the Dungog gauge, indicated that velocities across the deeper parts of this floodplain can be high enough to knock over fences. This implies that a significant proportion of the flow will be over the floodplain during large events.

5.3.2 Local Flood Model Development

In order to assist in determining the relationship between flood levels recorded at the Dungog gauge and flows in the river and over the floodplain, a local 2D/1D flood model of the area was developed. This model extended to the upper limits of the 40m 2D model area (i.e. about 5km upstream of Dungog) and extended to a point approximately 2.6 km downstream of Dungog.

Care was taken to represent local features such as the culvert under Stroud Road just west of the Williams River road bridge. Losses for the bridges in this model were also entered into the 1D and 2D model elements.

5.3.3 Rating Curve Development

The model was run for a synthetic flood event with a rising hydrograph. The levels calculated by this model at the Dungog gauge location were compared with the flows at the Dungog road bridge. Thus, a revised rating curve was derived.

The Manning's n values for the river and floodplain were changed until a good match was achieved with the DNR rating curve for recorded flow and level points. The Manning's n used for the main river section of this model was 0.08. This value would account for the roughness of the bed and banks as well as the bend losses due to the sinuosity of the river in this reach.

The resulting rating curve confirmed the initial suspicions that the DNR rating curve under-estimates the higher flows. Furthermore, the flow patterns of the 2D model area upstream of Dungog indicated relatively high floodplain velocities in accordance with the anecdotal evidence supplied by Peter Ryan. Figure 5-2 presents and compares the two rating curves.

The impact of this revised rating curve for gauge levels under 7m is not significant. However, above 7m the impact is quite significant. For a gauge level of 9m (approximately the peak level of the February 1990 and March 1978 flood events), the flow changes from 740m^3/s to 1510m^3/s. This is an increase of over 100% compared to the original rating curve ad more closely simulates actual events.

5.4 Choice of Calibration / Verification Events

5.4.1 Available Records of Peak Flood Level Data

Following the resident survey, it became apparent that there is a bias in the available peak flood level data to the more recent flood events. A disproportionate amount of data was available from a small recent flood such as the May 2001 flood compared to a relatively large flood such as the March 1978 flood. This is demonstrated in the results of the Resident Survey as shown in Section 3.6 and Appendix C.

The information provided by DNR included some valuable flood level information for the March 1978 flood. Specifically, there are records from 10 flood boards located along the river between Seaham and Clarence Town. These peak levels allow a calibration of flood gradient (and hence other parameters such as Manning's n) over an approximate river length of 13km and a flood level range of almost 2m (vertically).

5.4.2 Available Rainfall and Pluviograph Data

The observed flooding behaviour indicates that floods tend to result from two or three days of heavy rainfall. Hence, knowledge of the temporal distribution of that critical period of rainfall over 48 to 72 hours is very important. It would not be possible to run the hydrological model using daily data and expect meaningful results.

Therefore, historical flood events with good pluviograph information (i.e. rainfall data over short time steps such as six minutes) are critical to the choice of calibration / verification events. Given that most of the pluviograph stations in and around the catchment were installed in the 1970's, the study is essentially restricted to flood events after this time.

5.4.3 Available Streamflow Information

Similar to the issues regarding available pluviograph information, meaningful calibration of the flood models could only be gained from consideration of flood events with full recorded hydrographs (i.e. not just peak levels or daily readings). A time-varying record of peak levels (and calculated flows) is critical to the testing of the model performance in replicating the timing and shape of the flood wave as it propagates down the river.

Hence, the choice of calibration / verification events should ideally be limited to those events with at least two of the three flood gauges on the floodplain (i.e. Dungog, Glen Martin and Raymond Terrace) operational during the event.

5.4.4 Conclusions on Choice of Calibration / Verification Events

Based on all the above factors and the need to calibrate / verify the flood models to events as large as possible, it was decided to use the following three flood events for the reasons stated:

- ***February 1990:*** This event is a moderate to large flood event with adequate pluviograph and gauge records. Only five (5) peak flood levels were derived from the resident survey. However, this flood is also important because it is the only flood of recent time (i.e. after the installation of suitable flood and rainfall gauges) that was coincident with a Hunter River flood. This provides the opportunity to test the model's performance in replicating joint Williams River and Hunter River flood events, which is critical for the lower floodplain.
- ***March 1978:*** This flood event was a similar size to the February 1990 flood based on peak flows. It was also a double peak flood, although the gap between the first and second peaks (four days) would not provide an adequate test of a model's capability in replicating design multi-peak flood events (where the gap is in the order of hours). The peak flood levels recorded along the river reach between Seaham and Clarence Town provides critical calibration data.

- ***May 2001:*** This flood event was substantially smaller than the other two flood events. However, the Dungog gauge was operating as an automatic recorder for this event. As well, this is one of the first flood events following the installation of automatic recorders at Wangat (on the Wangat River) and Chichester (on the Chichester River), which will assist in the calibration of the hydrological model. Five (5) peak flood levels are available for calibration resulting from the resident survey.

The 1955 flood was considered for the calibration of the Williams River flood model. However, this flood was relatively small in the Williams River catchment and was primarily a Hunter River flood event that resulted in significant back-up flooding in the lower Williams River floodplain. Furthermore, the calibration of Lower Hunter River flood models to this flood event has been well documented in other studies (e.g. Lower Hunter River Flood Study).

5.5 Hydrological Model Calibration

5.5.1 March 1978 Flood

The recorded rainfall data for the March 1978 flood event is presented in Figure 5-3 along with the derived isopleths for this event. These isopleths were used to calculate rainfall totals over each hydrological model sub-area. From these rainfall totals, factors for each sub-area were derived for the most appropriate recorded pluviograph. These factors and the recorded pluviograph information were entered into the hydrological model.

The March 1978 flood was somewhat unusual in that there was a sharp rainfall gradient from the north to the south. The upper (i.e. northern) parts of the catchment received relatively intense rainfall in both rainfall bursts. However, the floodplain, southern parts of the catchment received only a fraction of that rainfall depth. This is demonstrated in the cumulative rainfall graph shown in Figure 5-4. The performance of the hydrological model in replicating the recorded flow data at the gauging stations is presented in Figure 5-5and Figure 5-6.

Parameters were varied to optimise the calibration to recorded data. Initial and continuing losses, Manning's n (both the main channel and overbank flow) were modified, as well as the non-linearity and B (Storage Coefficient Multiplication) factor. The location in which the recorded pluviograph were split between subcatchments was also varied based on the recorded daily rainfall data. Conversion from a lag time to a channel routing method for lagging hydrograph (see Section 5.2) had a significant effect on improving hydrograph timing and shape.

The records at Dungog for this flood event only cover the high flow period. Hence, there is no calibration data between the two peaks (or much data on the falling limb of either peak). Water levels were manually read by Mr B Hartcher and have been converted to flows estimates using the revised curve (See Section 5.3.3).

The records for Tillegra are based on daily read data; therefore calibration data are limited. Discrepancies between recorded data and modelled data are considered to be primarily due to the shortage of pluviograph and recorded flow data and the considerable uncertainties associated with the input data.

5.5.2 February 1990 Flood

The recorded rainfall data for the February 1990 flood event is presented in Figure 5-7 along with the derived isopleths for this event. These isopleths were used to calculate rainfall totals over each hydrological model sub-area. From these rainfall totals, factors for each sub-area were derived for the most appropriate recorded pluviograph. These factors and the recorded pluviograph information were entered into the hydrological model.

The February 1990 flood was due to relatively uniform rainfall over the catchment. This is demonstrated in the cumulative rainfall graph shown in Figure 5-8. Recorded totals of 339mm to 456 were recorded within the Williams River Catchment (totals from 9am 01/02 to 9am 05/02). Indeed, it was a relatively widespread flood event, resulting in minor flooding of the Hunter River as well. The performance of the hydrological model in replicating the recorded flow data at the gauging stations is presented in Figure 5-9 and Figure 5-10.

Initial and continuing losses, Manning's n, non-linearity and B factor were all modified to find the optimal calibration to recorded data. The location in which the recorded pluviograph were split between subcatchments was also varied based on the recorded daily rainfall data.

The records for Dungog for this flood event only cover the high flow period. Peak flood levels recorded occurred at 1am on the morning of the 4th. Discrepancies between recorded data and modelled data are considered to be primarily due to the shortage of pluviograph and recorded flow data and therefore the considerable uncertainties associated with the input data.

5.5.3 May 2001 Flood

The recorded rainfall data for the May 2001 flood event is presented in Figure 5-11 along with the derived isopleths for this event. These isopleths were used to calculate rainfall totals over each hydrological model sub-area. From these rainfall totals, factors for each sub-area were derived for the most appropriate recorded pluviograph. These factors and the recorded pluviograph information were entered into the hydrological model.

The May 2001 flood was due to relatively steady rainfall pattern over the catchment. This is demonstrated in the cumulative rainfall graph shown in Figure 5-12. However, it was a relatively small flood event and it is more likely to have spatial rainfall variations in the smaller events than the larger events. The performance of the hydrological model in replicating the recorded flow data at the gauging stations is presented in Figure 5-13 to Figure 5-16.

As for the other calibration events, the discrepancies between recorded data and modelled data are considered to be primarily due to the shortage of pluviograph and recorded flow data and the considerable uncertainties associated with the input data.

5.5.4 Chichester Dam

Chichester Dam has a relatively minor effect on the attenuation of flood flows for the three events simulated for the calibration exercise. Figure 5-17, Figure 5-18 and Figure 5-19 show the inflow and outflow hydrographs for these three events.

The event with the largest flows through the Chichester Dam was the February 1990 event. The hydrological model indicates that the peak flow into the dam of 1,200m^3/s was attenuated by approximately 20% down to 1,000m^3/s.

5.5.5 Calibrated Parameters

The parameters adopted for the hydrological model based on the calibration exercise are presented in Table 5-1.

Table 5-1 Calibrated Hydrological Model Parameters

Calibration Event	Initial Losses	Continuing Losses	Storage Coefficient Multiplication Factor (B Factor)	Storage Exponent
1978	75mm	2mm/hr	0.8	-0.285
1990	75mm	2mm/hr	0.8	-0.285
2001	25mm	2mm/hr	0.8	-0.285

5.5.6 Conclusions on Hydrological Model Calibration

The following conclusions can be drawn from the calibration of the hydrological model to three flood events:

- The hydrological model is quite sensitive to the spatial distribution of the rainfall (and probably the temporal distribution). The quality of the model calibration exercise is compromised by a lack of pluviograph stations in the catchment;
- The hydrological model adequately represents the attenuation and flood wave propagation characteristics of the catchment (upstream of Dungog);

In summary, the hydrological model (upstream of Dungog) provides an adequate representation of dynamic flows from the catchment for the purposes of this study and subsequent floodplain management studies.

5.6 Flood Frequency Analysis

5.6.1 Approach

Flood frequency analysis was completed by BMT WBM using available data from both the Dungog and Glen Martin gauges. The flood frequency analysis was carried out using MATLAB.

Professor George Kuczera (The University of Newcastle Research Associates) was commissioned to conduct an independent flood frequency analysis on the Williams River data.

5.6.2 Glen Martin Gauge

The Glen Martin has a relatively complete record of levels dating back to 1927. DNR converted recorded levels to flows using rating curves the have derived. DNR provided BMT WBM with annual maxima flow values from 1927 onwards. The flows used for this flood frequency analysis are presented in Table 5-2.

Table 5-2 Summary of Glen Martin Flows for Flood Frequency Analysis

Date	Height (m)	Peak Flow (m3/s)	Date	Height (m)	Peak Flow (m3/s)
28/07/1928	6.86	1,002	13/01/1968	8.20	864
09/02/1929	8.93	1,826	22/06/1969	9.31	1,184
17/06/1930	8.13	927	09/12/1970	5.22	341
22/04/1931	8.08	912	21/01/1971	9.82	1,316
11/09/1932	4.70	270	25/01/1972	9.75	1,300
29/09/1933	5.76	425	12/02/1973	4.85	282
12/12/1934	5.72	416	12/01/1974	6.05	438
17/01/1935	3.99	189	21/06/1975	6.40	493
02/03/1936	5.26	345	24/01/1976	7.96	792
17/03/1937	3.89	179	04/03/1977	8.40	900
11/04/1938	5.87	470	20/03/1978	10.85	1,775
11/03/1939	6.02	496	07/05/1979	6.05	418
22/12/1940	3.20	138	30/12/1980	1.79	48
08/02/1941	4.95	329	23/05/1981	5.68	364
15/10/1942	7.95	910	12/10/1982	5.44	331
24/05/1943	4.06	178	27/05/1983	2.88	111
25/08/1944	3.53	130	21/03/1984	5.23	305
11/06/1945	7.31	736	13/10/1985	9.35	1,161
18/04/1946	9.75	1,413	25/01/1986	6.22	445
11/12/1947	5.23	355	12/11/1987	8.01	803
04/05/1948	3.05	119	06/07/1988	6.47	487
17/06/1949	8.69	1,087	28/03/1989	6.42	478
19/06/1950	7.62	836	04/02/1990	10.95	1,827
19/01/1951	9.07	1,240	11/06/1991	3.16	129
06/08/1952	7.75	867	10/02/1992	5.81	383
08/05/1953	5.49	405	19/03/1993	3.62	161
21/02/1954	9.11	1,217	14/04/1994	2.56	91
17/02/1955	8.84	1,136	05/03/1995	5.60	353
01/03/1956	9.75	1,427	24/01/1996	2.69	99
19/02/1957	9.35	1,292	07/03/1997	4.84	266
09/02/1958	1.65	41	19/05/1998	7.33	654
30/10/1959	5.26	338	15/07/1999	7.92	783
29/02/1960	1.88	43	22/03/2000	8.46	914
11/06/1961	4.88	299	08/05/2001	8.92	1,036
13/05/1962	8.84	1,048	05/02/2002	4.04	194
19/03/1963	11.58	2,172	28/05/2003	5.38	323
10/06/1964	4.57	247	23/03/2004	7.52	668
22/07/1965	2.23	70	30/06/2005	6.72	512
08/12/1966	2.12	64	07/11/2006	4.56	237
22/10/1967	9.36	1,179	08/06/2007	9.30	1,141

K:\B16030.k.gjr_WilliamsR_FS\Flood_Frequency_Analysis\[Revised Glen Martin and Dungog Flows for FFA_26092007.xls]Glen Martin Data Table

Log Pearson III (LPIII) was fitted to annual maxima flow dataset. The resulting LPIII distribution is presented in Figure 5-20. Generalised Extreme Value (GEV) distribution was fitted to base 10 logarithms of annual maxima flow series.

Flows for key annual exceedance probabilities based on the flood frequency are presented in Table 5-3.

Table 5-3 Flow at Glen Martin from Flood Frequency Analysis

AEP	ARI (years)	Flow (m^3/s) at Glen Martin			
		Annual Max GEV	Annual Max LPIII	FLIKE	LHRFS
20%	5	1036	1027		1000
10%	10	1366	1333		1400
5%	20	1629	1590	1665	1800
2%	50	1883	1864	2008	2300
1%	100	2017	2031	2230	2680
0.5%	200	2114	2169	2424	N/A

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Also shown in Table 5-3 are the results of the flood frequency analysis for the Glen Martin gauge as reported in the Lower Hunter River Flood Study (PWD, 1994). In comparison with the results from the LPIII distribution, there are only minor discrepancies between the two results for the smaller flood events (20%, 10%, 5% AEP events).

However, for the 2% and 1% AEP flood events, the discrepancies are in the order of 25%. These discrepancies are due to the following two reasons:

- The flood frequency analysis presented in this report includes the period from 1994 to 2008 which is likely to be a period of lower than normal floods due to the extended El Nino event(s);
- The fit of the flood frequency analysis in the Lower Hunter River Flood Study is not as good as the fit produced in this study, especially for the higher flows. The Lower Hunter River Flood Study curve over predicted for the highest 9 flood events.

5.6.3 Dungog Gauge

Records for Dungog are less complete than the Glen Martin Gauge. DWE currently has an automatic gauging station (approximately 300m upstream of rail bridge), this began recording in 1995. Prior to 1995, readings were taken from a gauging station (maintained by BOM) read by SES/BOM volunteers. The BOM gauge is at the same site as the DWE automatic recorder. Prior to 1964 this gauge was located further downstream, at the caravan park toilet block (near the road bridge).

Rating curves (see Section 5.3) at the current gauge site and the pre-1964 gauge site were used to convert the recorded levels to flows. Flood frequency analysis is based on flows not levels. This allows the recordings at the two nearby locations to be used.

Partial series flood frequency analysis is derived using independent flow peaks. Multiple records deemed to be non-independent were removed from data set (e.g. peaks recorded on the 20th and 21st of January 1971. The full data set used for the flood frequency analysis is presented in Table 5-4.

A Generalised Pareto (GP) distribution was fitted to independent peaks over 500m^3/s. The distribution is presented in Figure 5-21. The flood frequency analysis derives an estimate of 1% flow at Dungog of 2,253m^3/s. A summary of estimated flows for a range of probabilities is presented in Table 5-5.

Table 5-4 Summary of Dungog Flows for Flood Frequency Analysis

Date	Source	Location	Level (mAHD)	Stage at DNR Recorder (m)	Flow Derived Using TUFLOW Rating Curve
15/06/1948	DMR 1985 Report	Gauge at Road Bridge	48.47	N/A	850
06/08/1952	DMR 1985 Report	Gauge at Road Bridge	47.63	N/A	580
01/03/1956	DMR 1985 Report	Gauge at Road Bridge	49.82	N/A	1800
27/12/1961	DMR 1985 Report	Gauge at Road Bridge	47.33	N/A	610
13/05/1962	DMR 1985 Report	Gauge at Road Bridge	48.93	N/A	1110
18/03/1963	SES	Mark in Toilet Block	50.17	N/A	2250
08/05/1963	SES	Gauge at Road Bridge	48.34	N/A	850
09/03/1967	SES	Current Gauge Location	48.14	6.92	543
06/08/1967	SES	Current Gauge Location	48.14	6.92	543
21/10/1967	SES	Current Gauge Location	49.10	7.88	928
12/01/1968	SES	Current Gauge Location	48.47	7.25	647
21/06/1969	SES	Current Gauge Location	49.18	7.96	973
21/01/1971	SES	Current Gauge Location	49.17	7.95	967
1972	Hunter River FPMS	Current Gauge Location	49.17	7.95	967
25/01/1976	SES	Current Gauge Location	48.32	7.10	596
02/03/1976	SES	Current Gauge Location	49.00	7.78	876
03/03/1977	SES	Current Gauge Location	48.31	7.09	593
19/03/1978	SES	Current Gauge Location	50.22	9.00	1722
07/11/1984	SES	Current Gauge Location	48.50	7.28	658
13/10/1985	SES	Current Gauge Location	48.15	6.93	546
13/11/1987	SES	Current Gauge Location	48.55	7.33	676
27/03/1989	SES	Current Gauge Location	47.60	6.38	426
20/05/1989	SES	Current Gauge Location	47.10	5.88	360
04/02/1990	SES	Current Gauge Location	50.20	8.98	1705
04/03/1995	SES	Current Gauge Location	48.70	7.48	736
19/05/1998	DNR	Current Gauge Location	48.39	7.17	619
15/07/1999	DNR	Current Gauge Location	47.69	6.47	442
08/03/2000	DNR	Current Gauge Location	48.34	7.12	601
22/03/2000	DNR	Current Gauge Location	49.12	7.90	937
09/03/2001	DNR	Current Gauge Location	48.68	7.46	728
08/05/2001	DNR	Current Gauge Location	48.80	7.58	779
27/05/2003	DNR	Current Gauge Location	47.49	6.27	408
23/03/2004	DNR	Current Gauge Location	48.67	7.45	721
30/06/2005	DNR	Current Gauge Location	48.17	6.95	550
08/06/2007	DNR	Current Gauge Location	48.17	6.95	550
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Table 5-5 Flow at Dungog from Flood Frequency Analysis

AEP	ARI (years)	Flow (m^3/s) at Dungog Generalised Pareto (Peak Over Threshold)
20%	5	833
10%	10	1105
5%	20	1408
2%	50	1863
1%	100	2253
0.5%	200	2688

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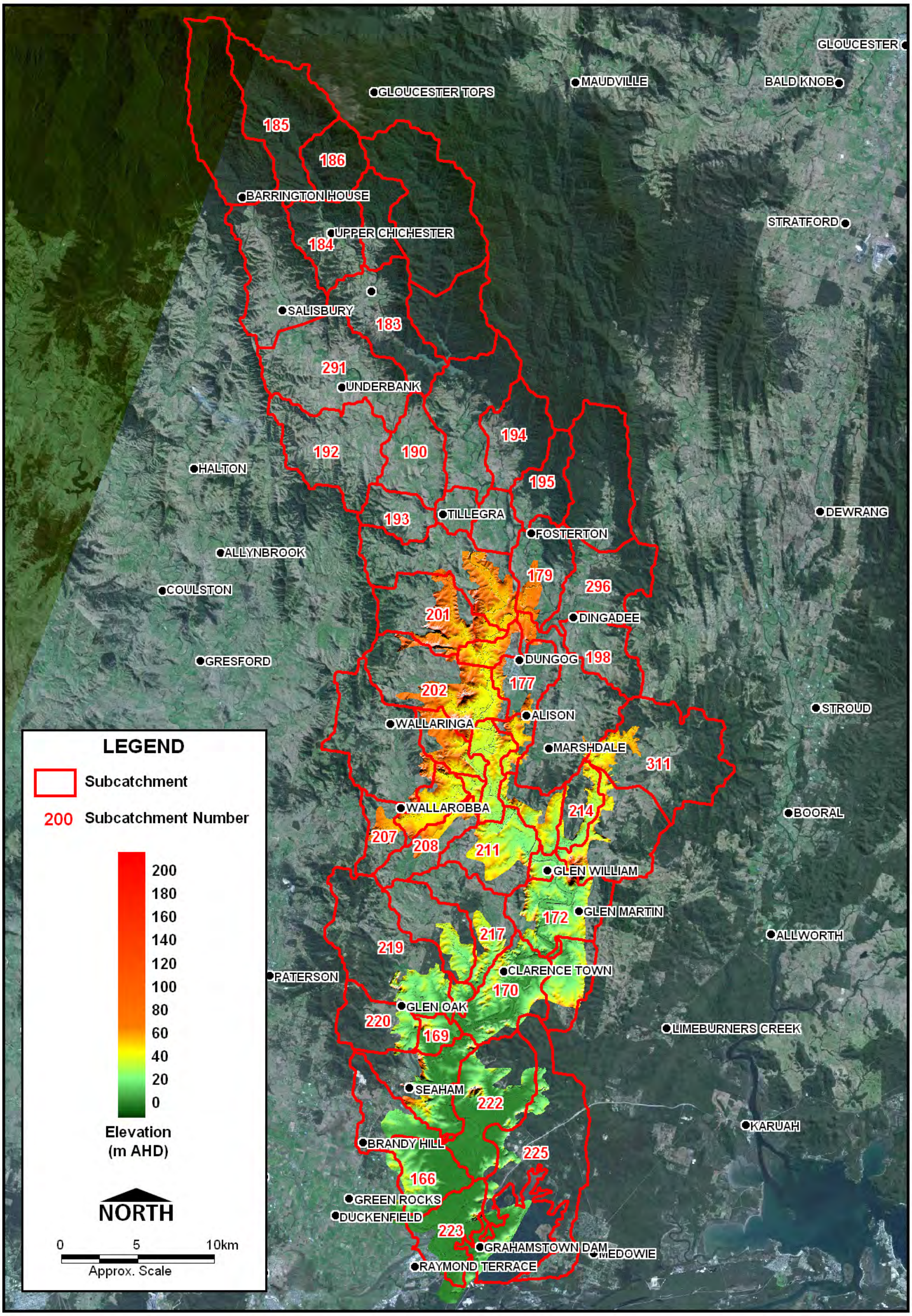

RAFTS-XP Catchment Layout

Figure 5-1

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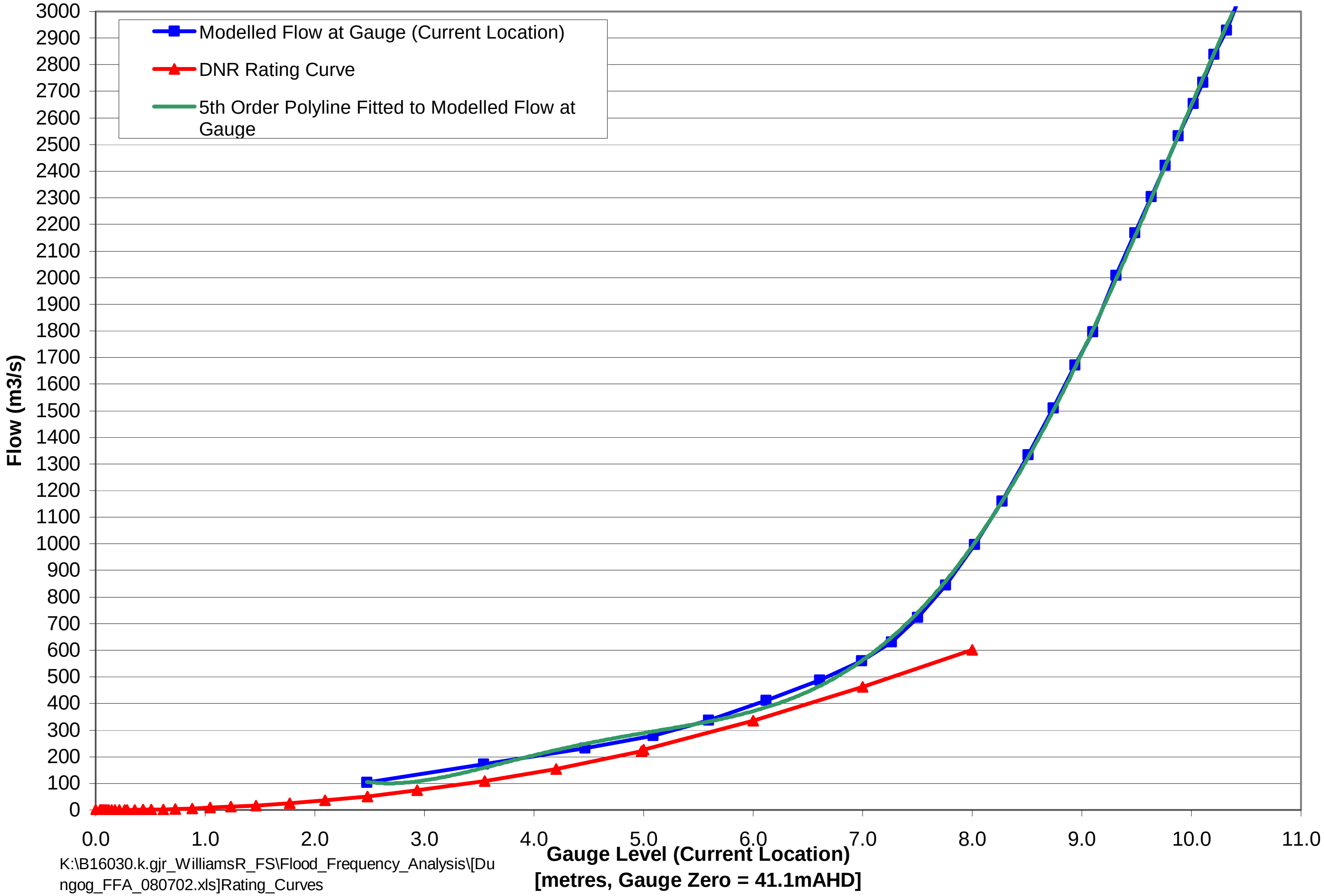
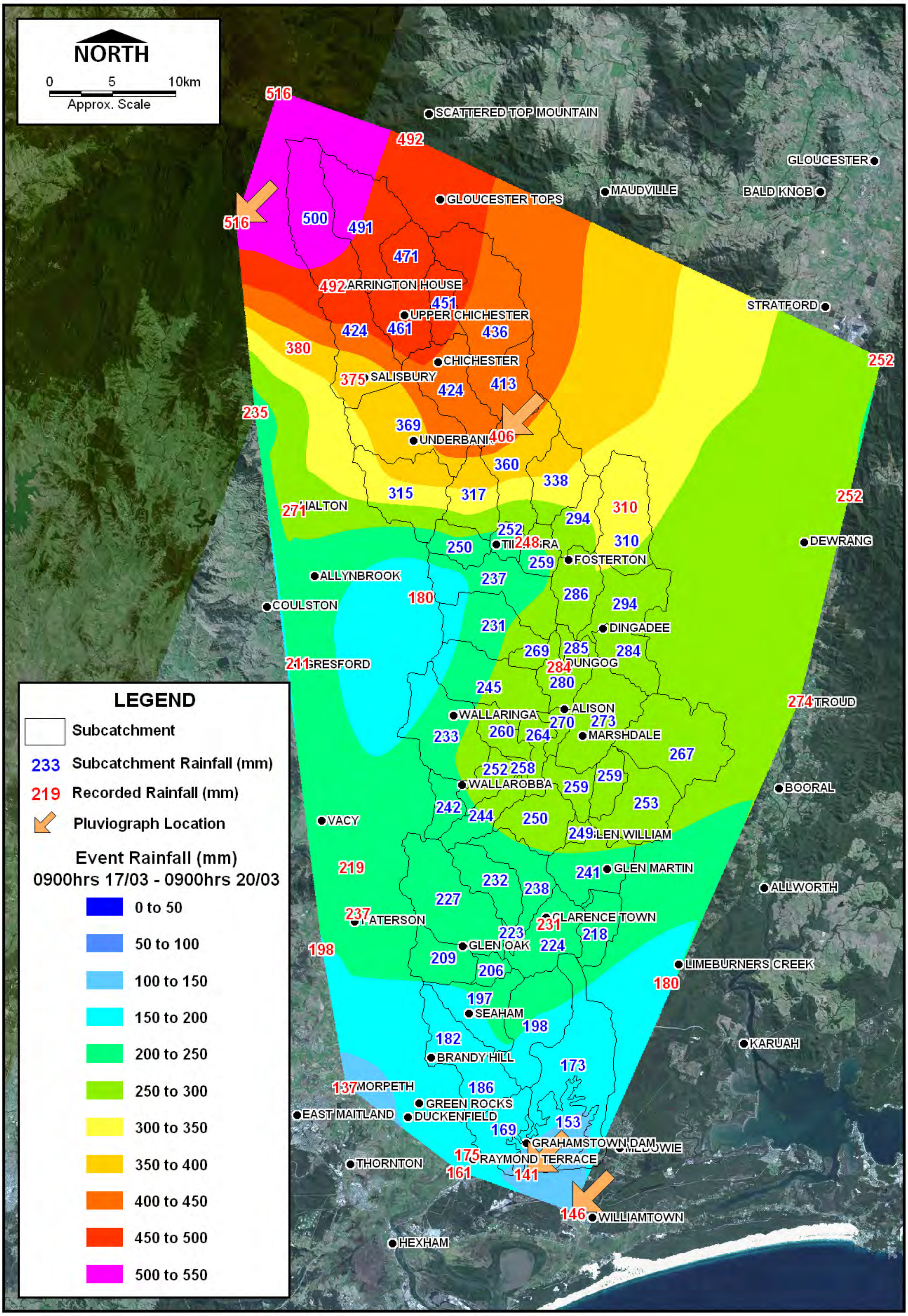


Figure 5-2 DNR and Revised Rating Curve for Dungog Gauging Station



March 1978 - Assumed Spatial Rainfall Distribution Figure 5-3

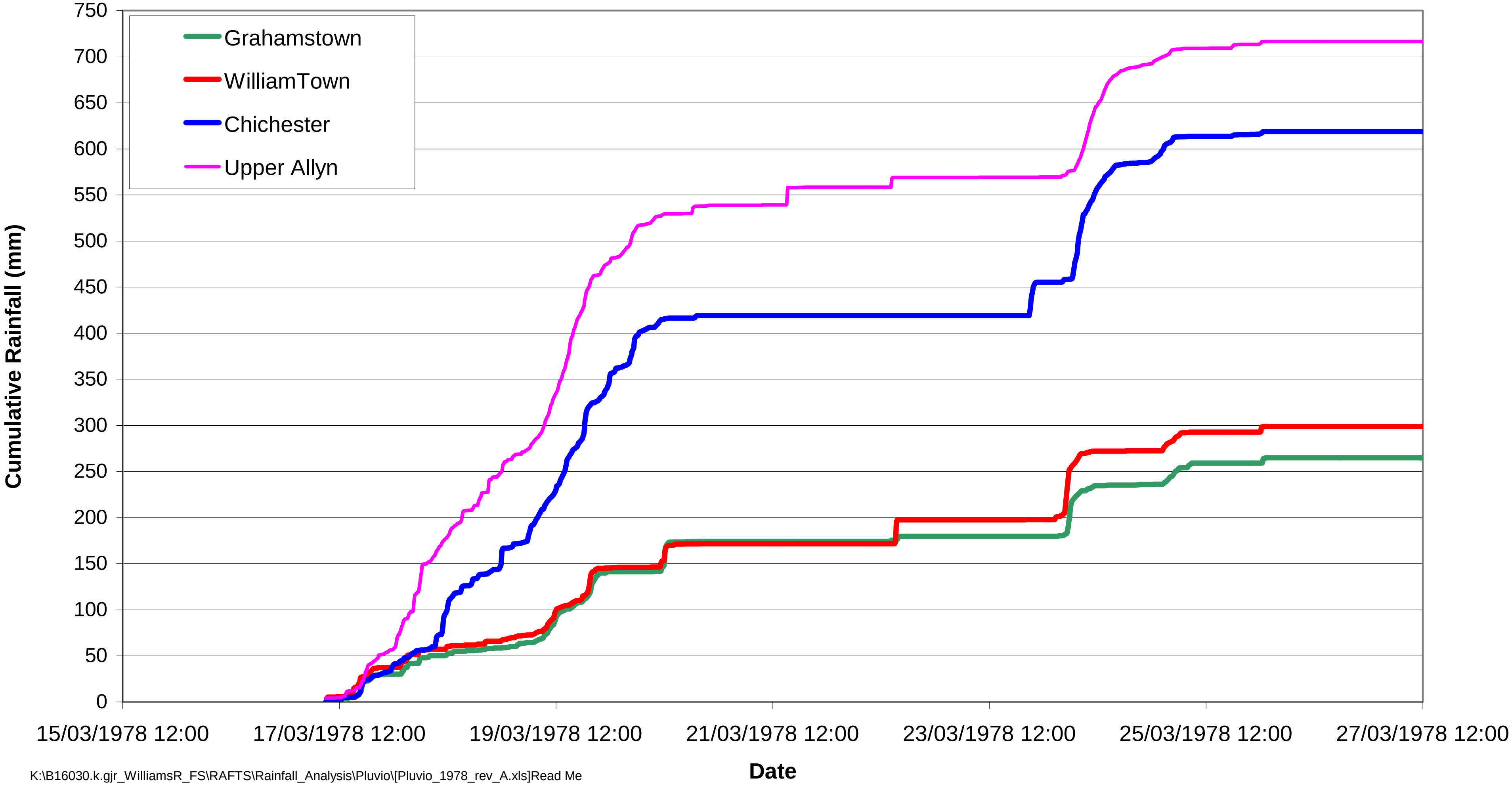


Figure 5-4 March 1978: Cumulative Rainfall Records

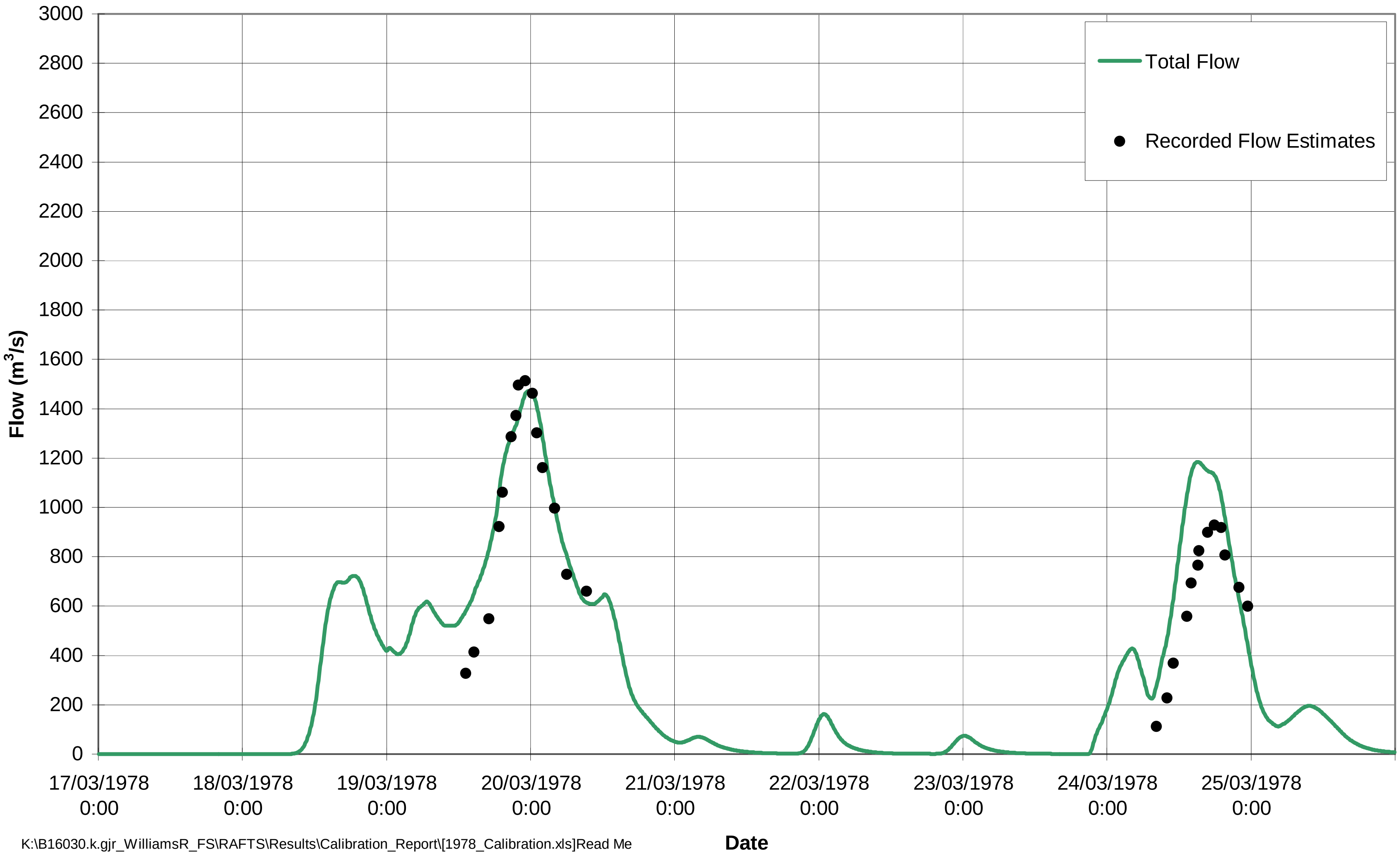


Figure 5-5 March 1978: Dungog Gauge Comparison for Hydrological Model

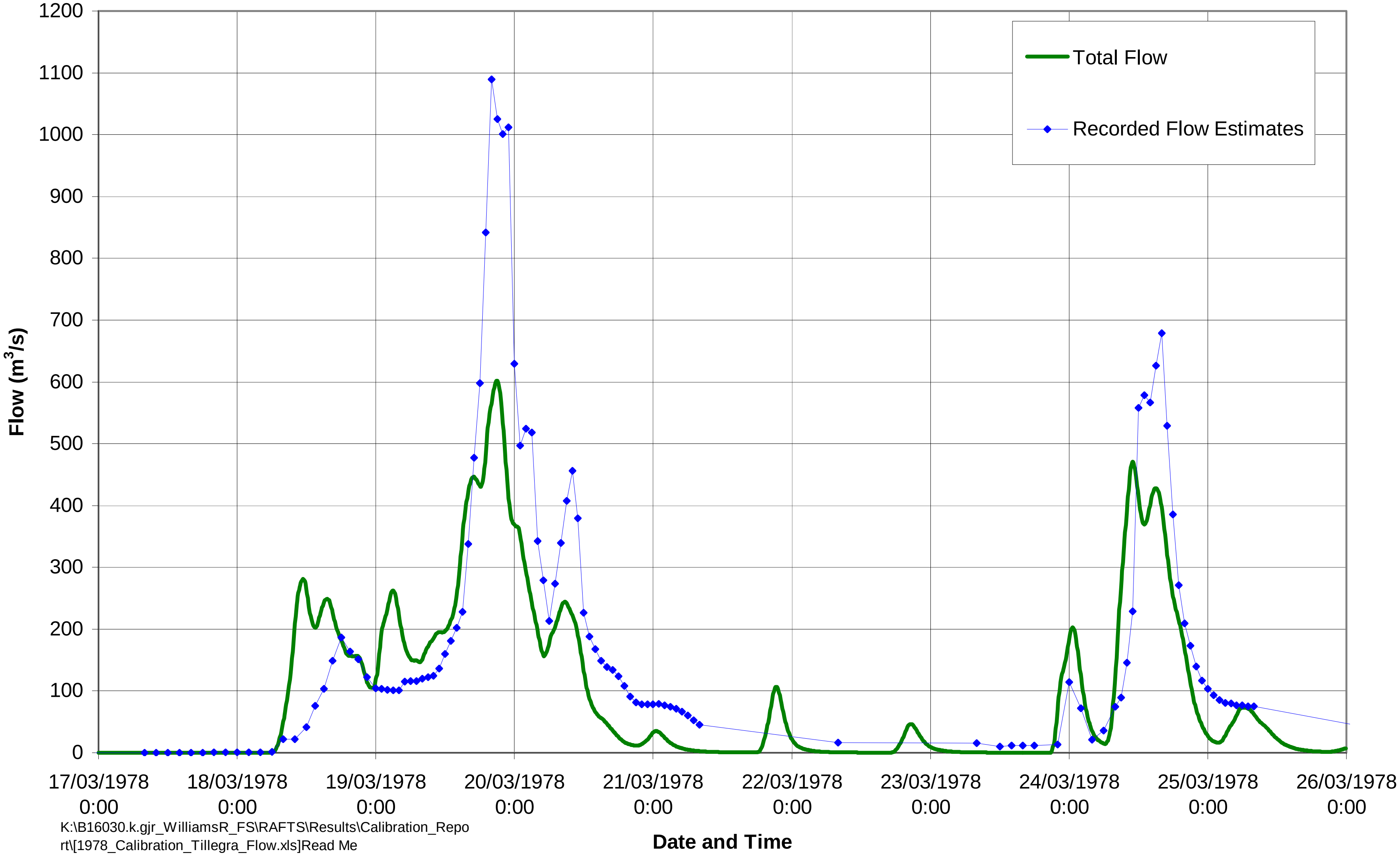
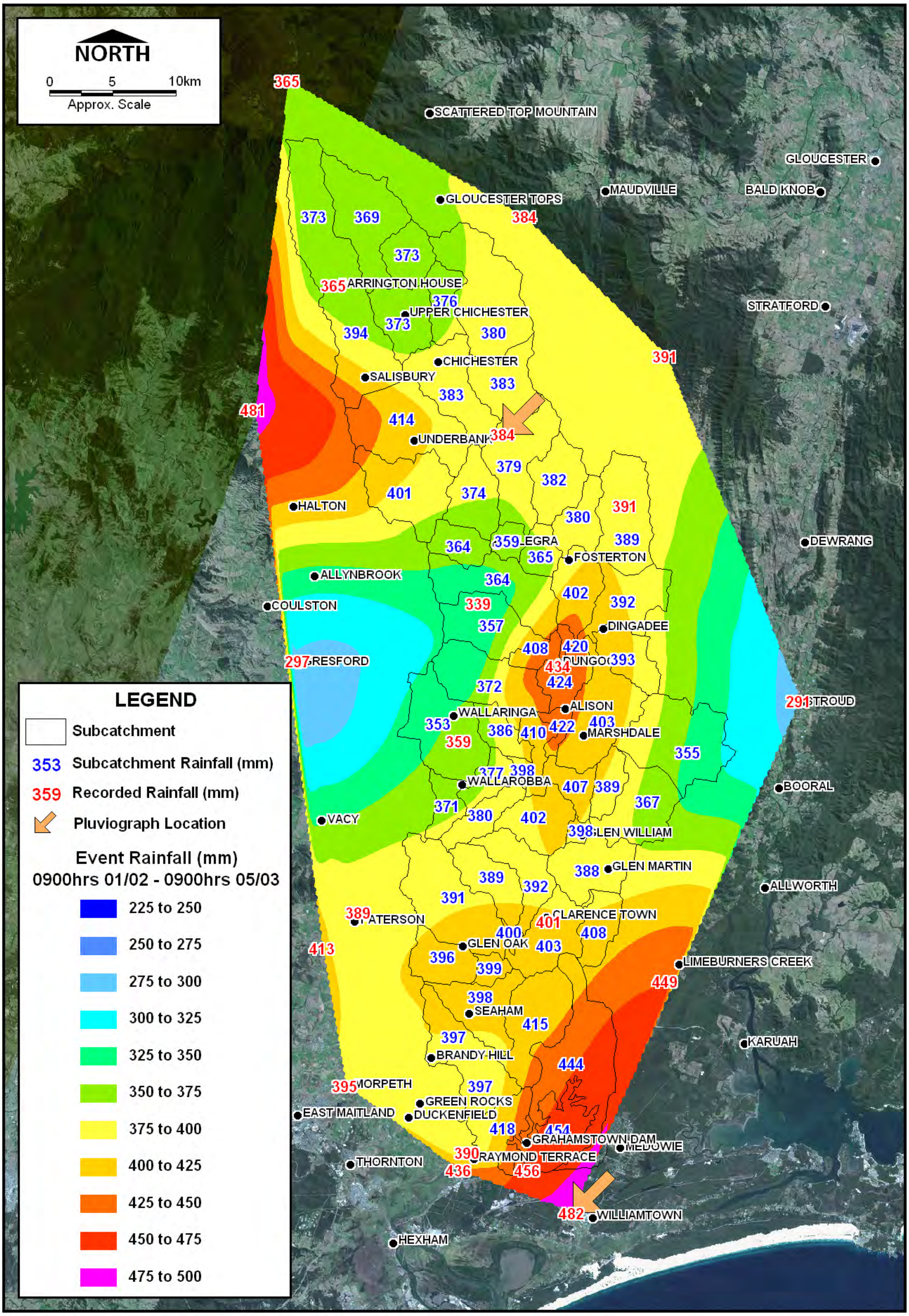


Figure 5-6 March 1978: Tillegra Gauge Comparison for Hydrological Model



February 1990 - Assumed Spatial Rainfall Distribution Figure 5-7

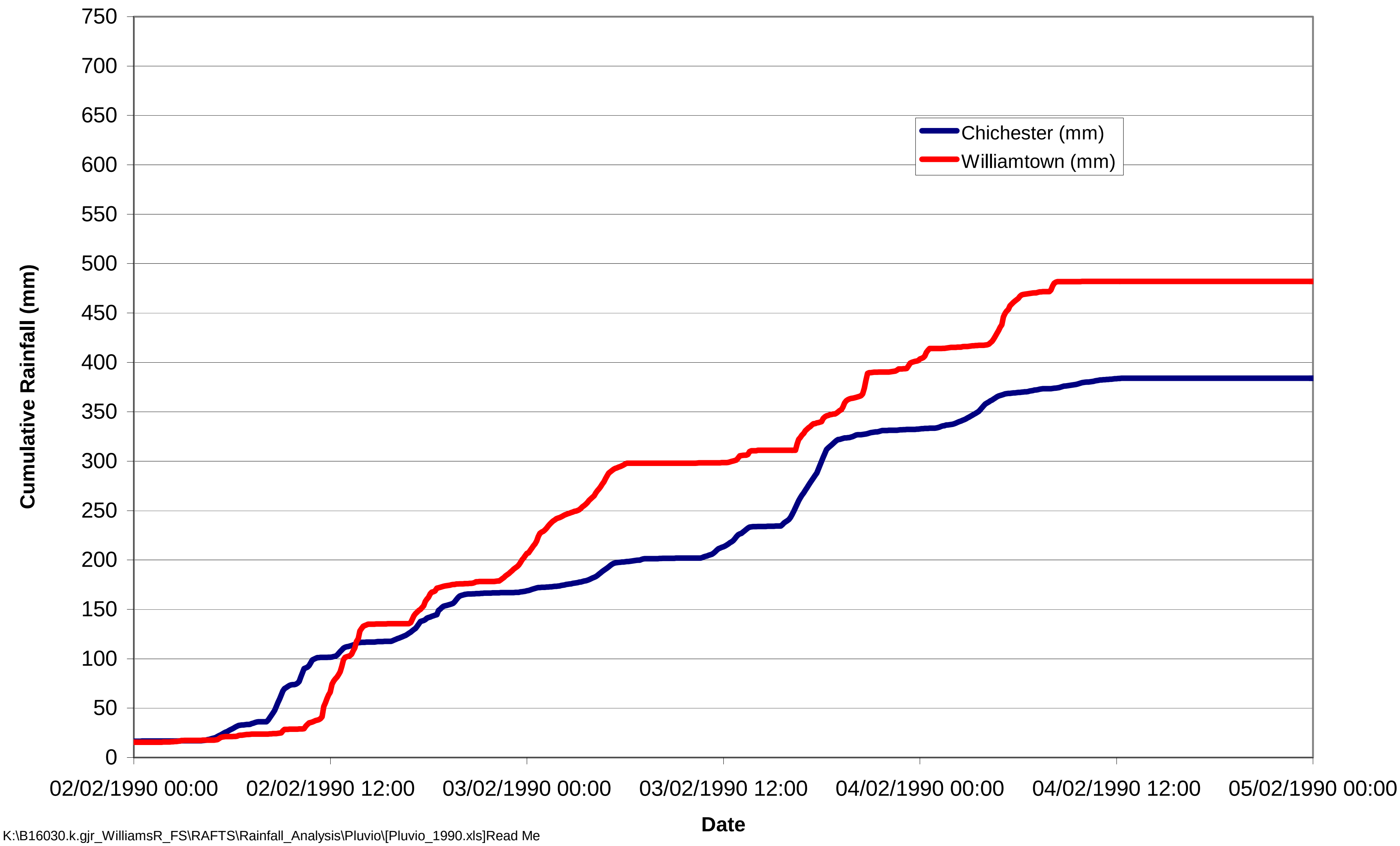


Figure 5-8 February 1990: Cumulative Rainfall Records

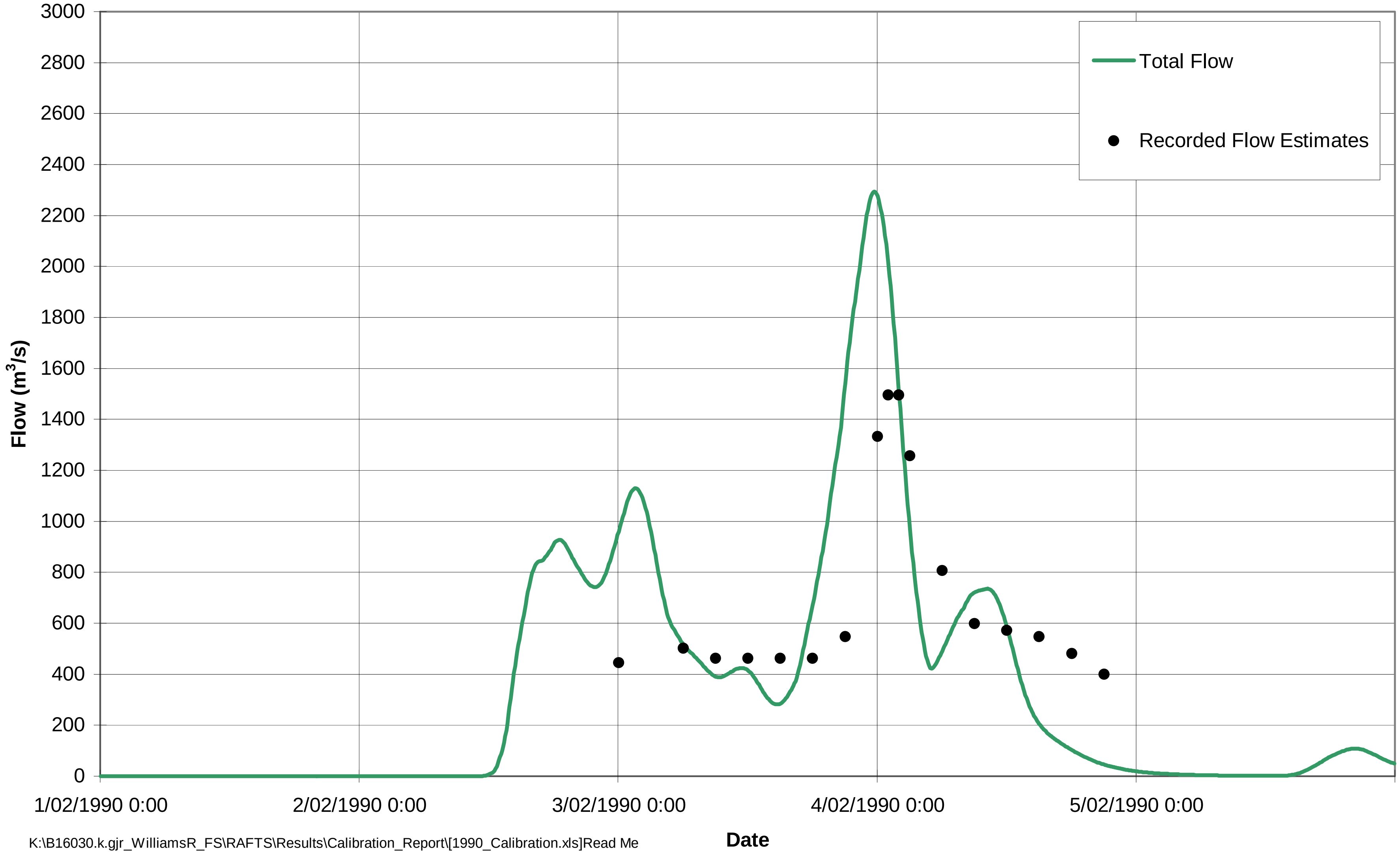


Figure 5-9 February 1990: Dungog Gauge Comparison for Hydrological Model

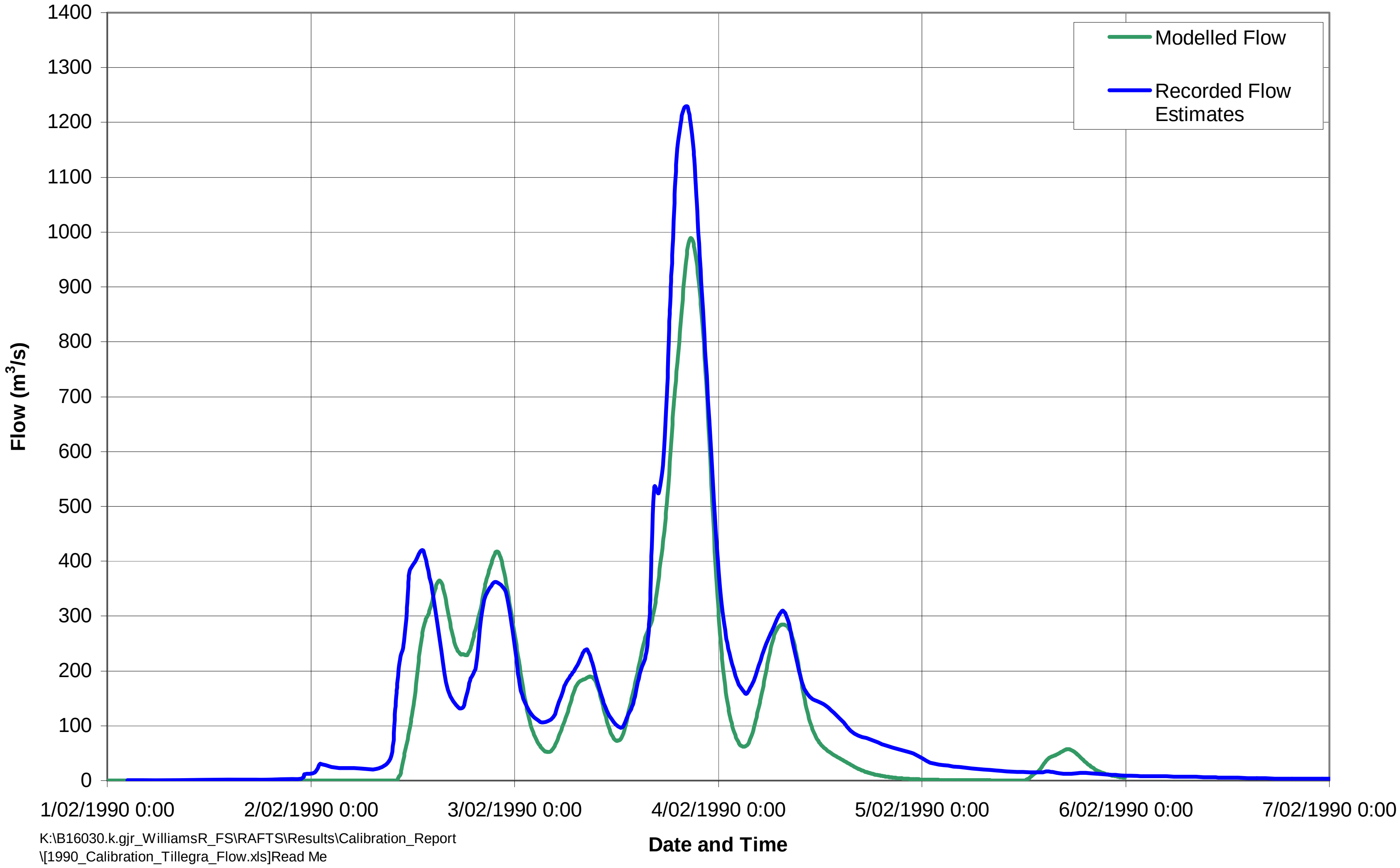
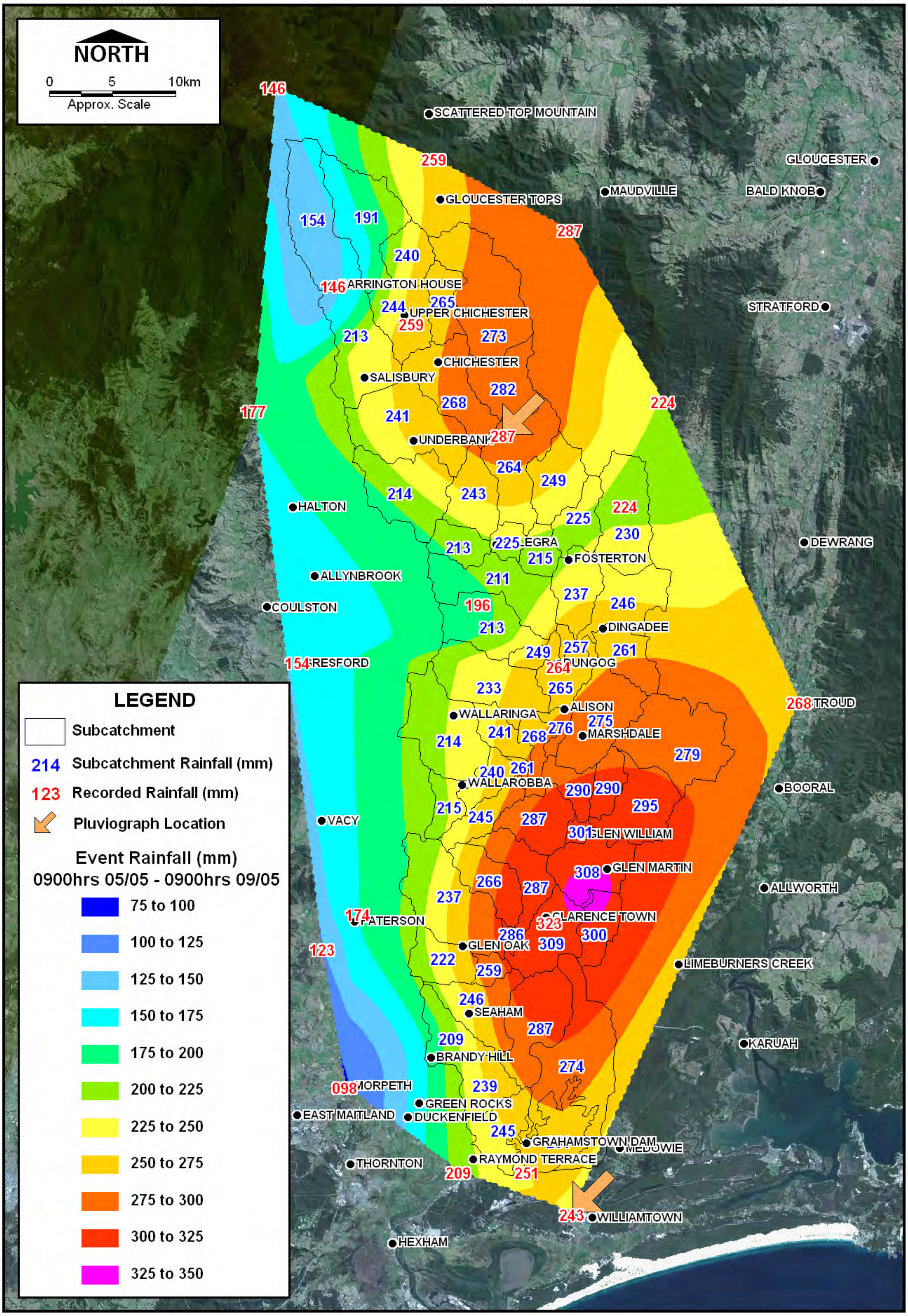


Figure 5-10 February 1990: Tillegra Gauge Comparison for Hydrological Model



May 2001 - Assumed Spatial Rainfall Distribution Figure 5-11

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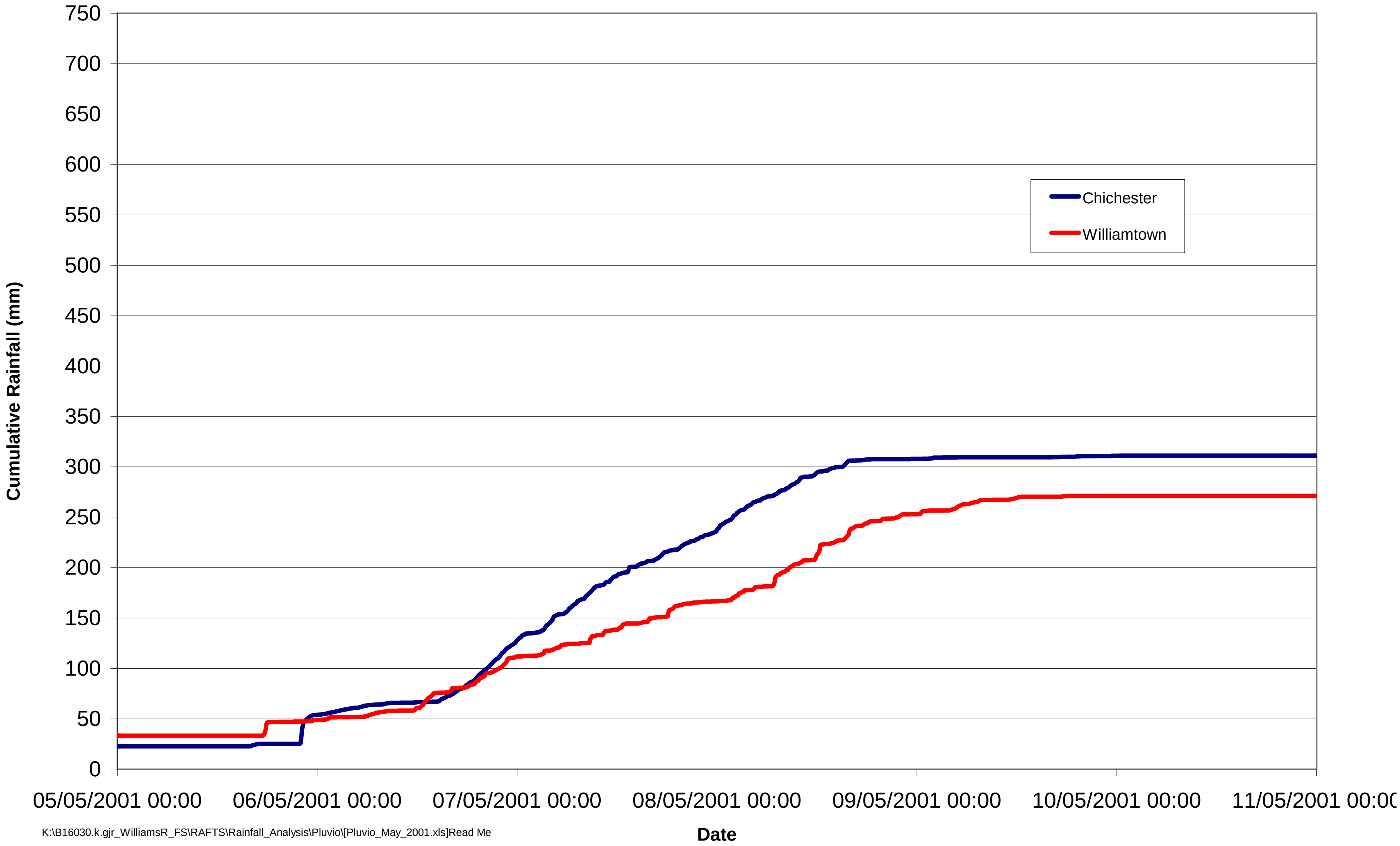

Figure 5-12 May 2001: Cumulative Rainfall Records

BMT WBM

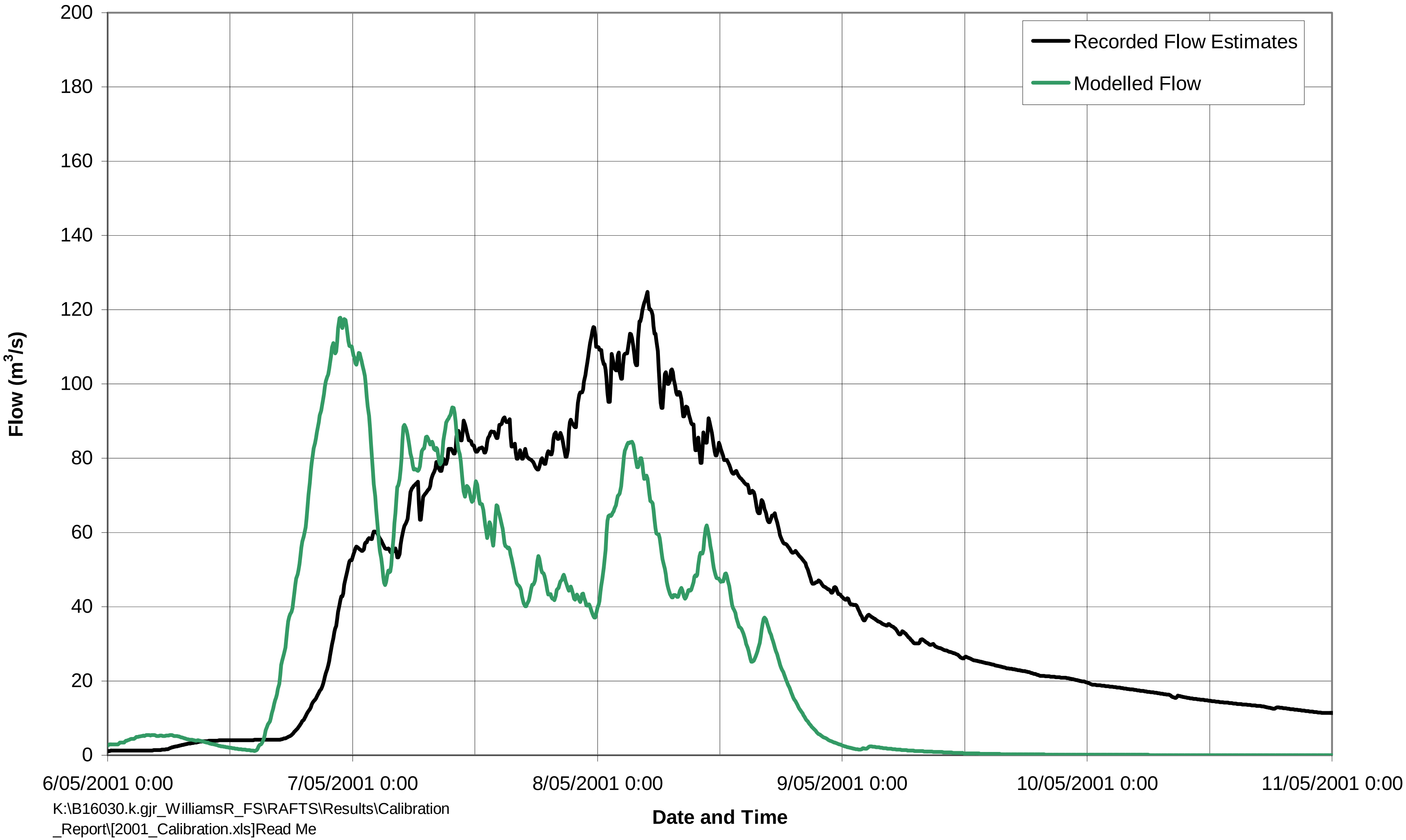

Figure 5-13 May 2001: Wangat Gauge Comparison for Hydrological Model

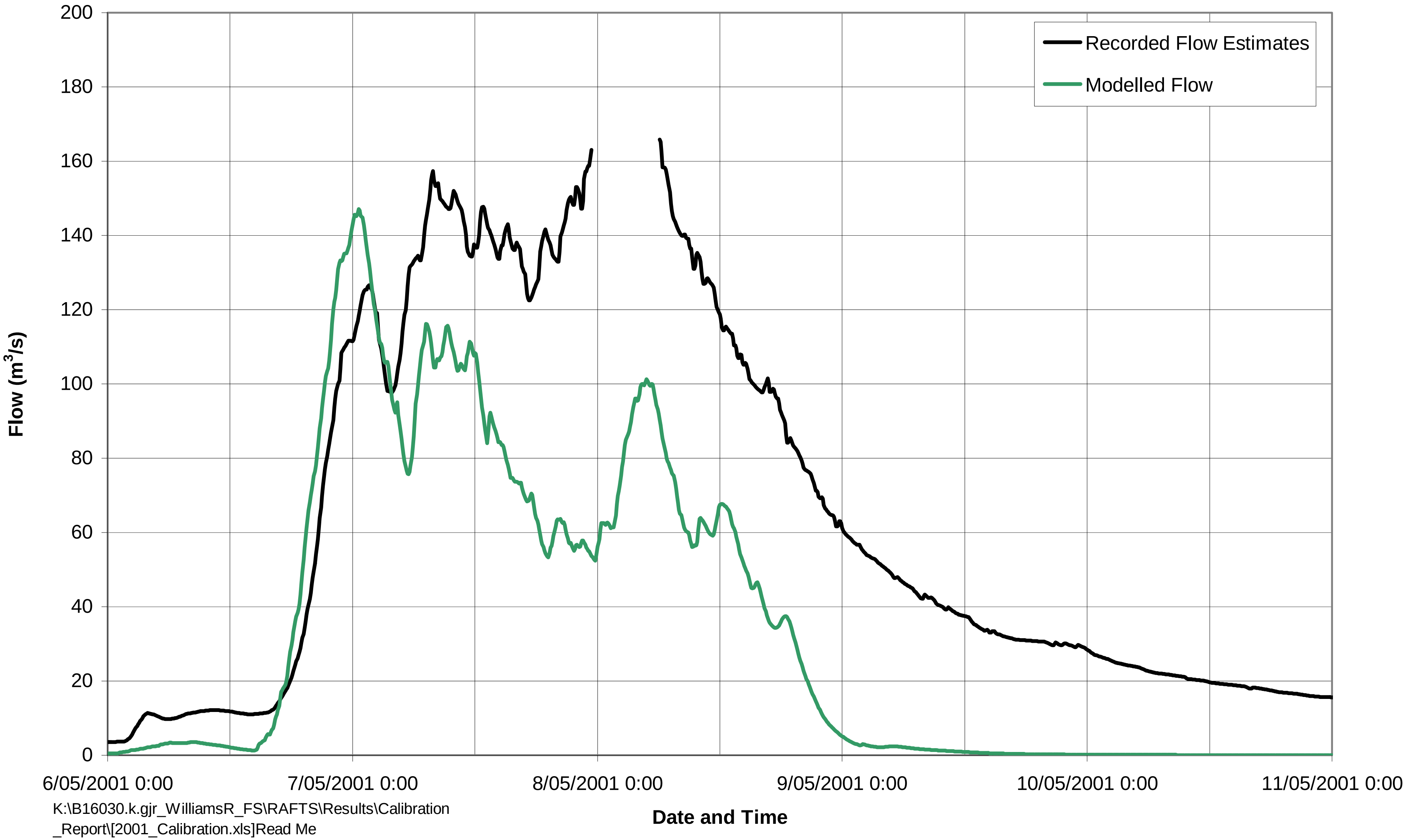

Figure 5-14 May 2001: Chichester Gauge Comparison for Hydrological Model

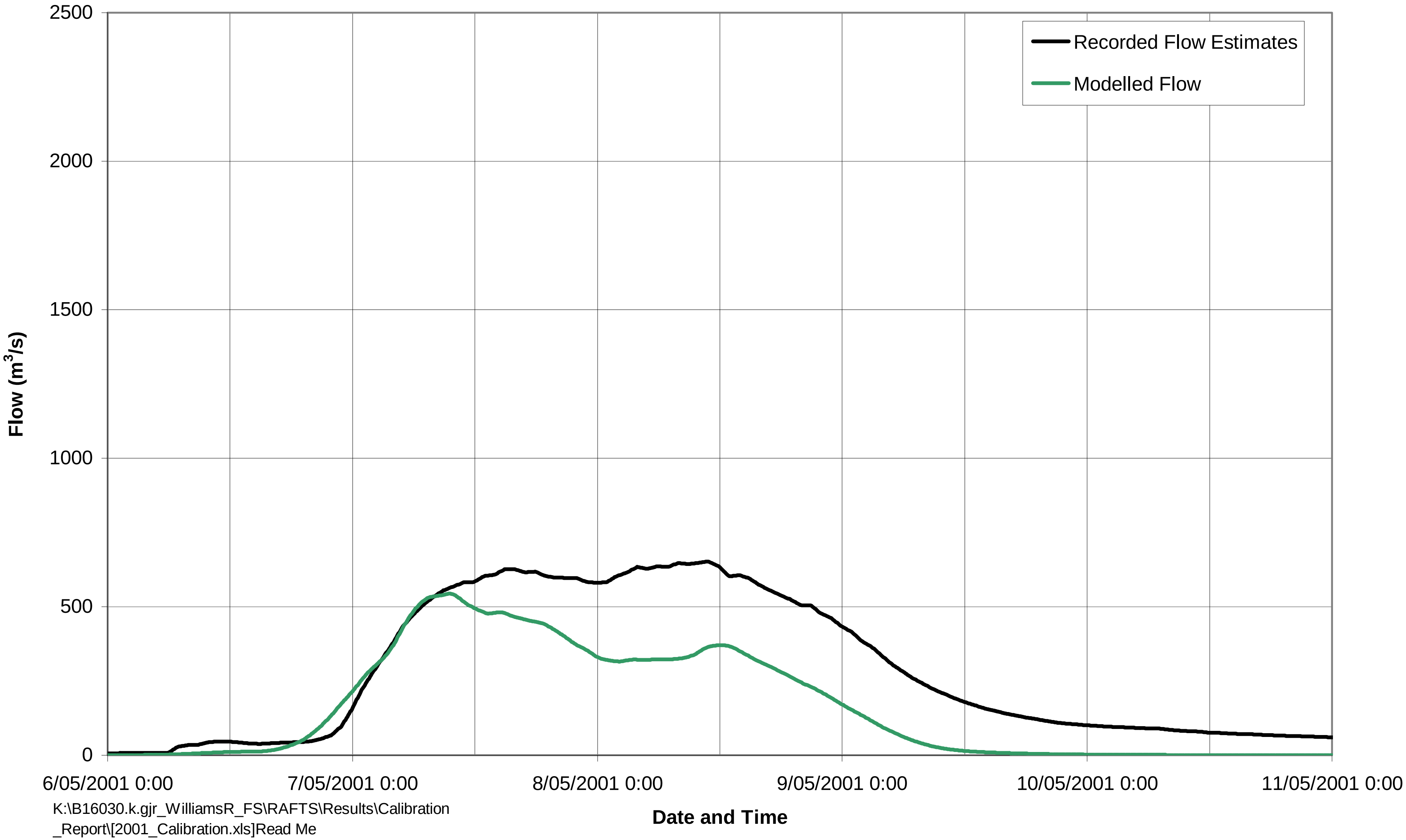

Figure 5-15 May 2001: Dungog Gauge Comparison for Hydrological Model

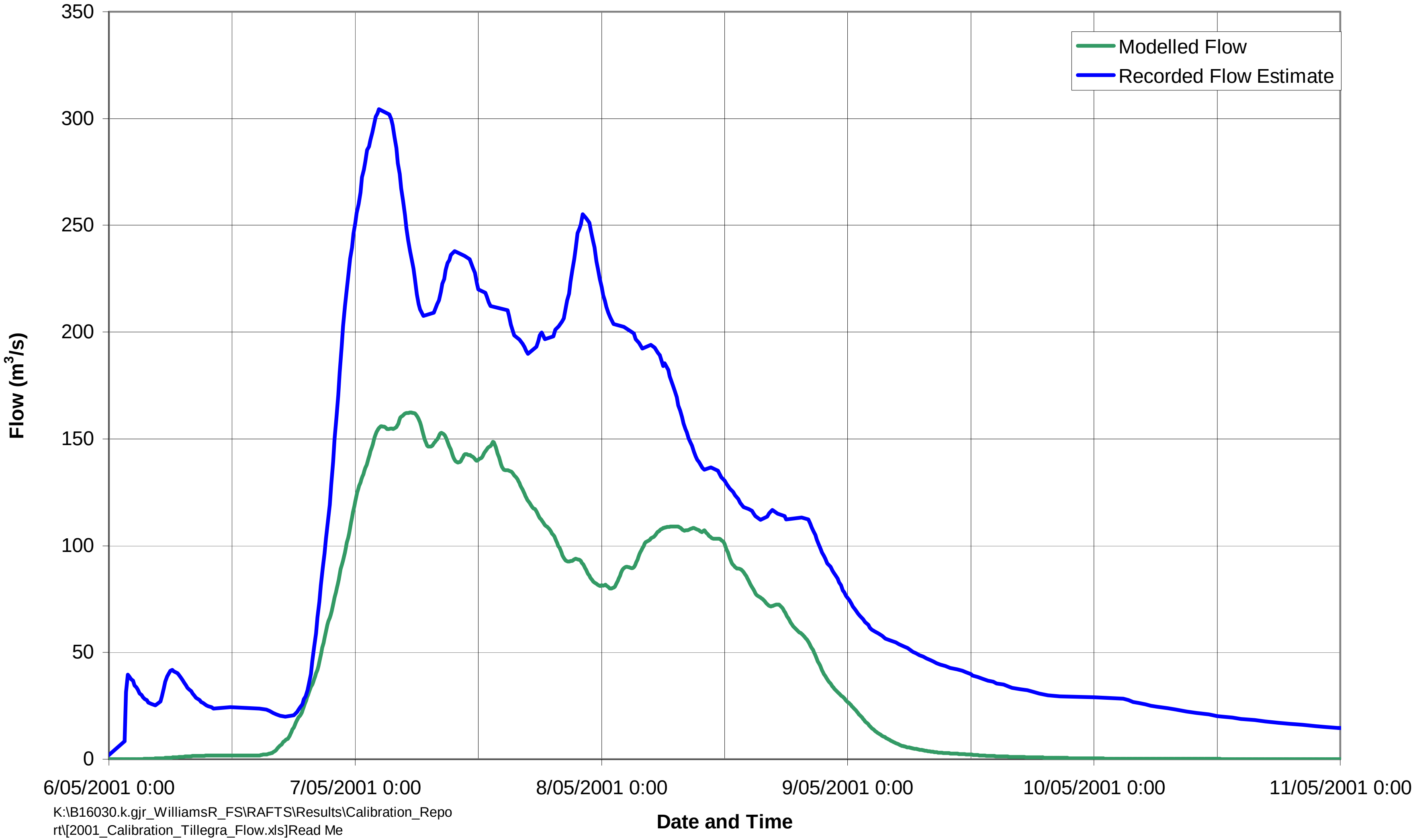

Figure 5-16 May 2001: Tillegra Gauge Comparison for Hydrological Model

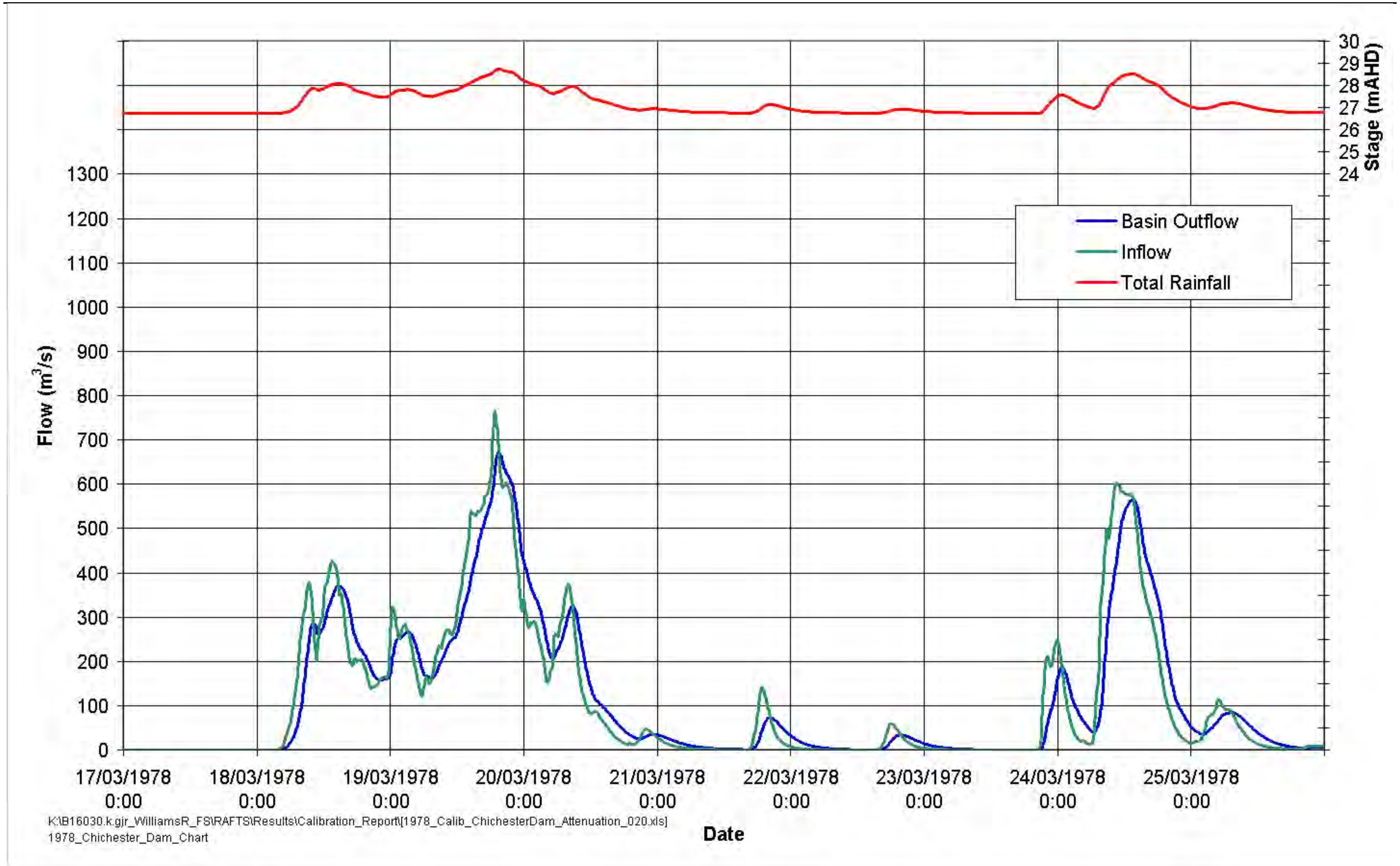

Figure 5-17 March 1978 Chichester Dam Routing of Hydrograph

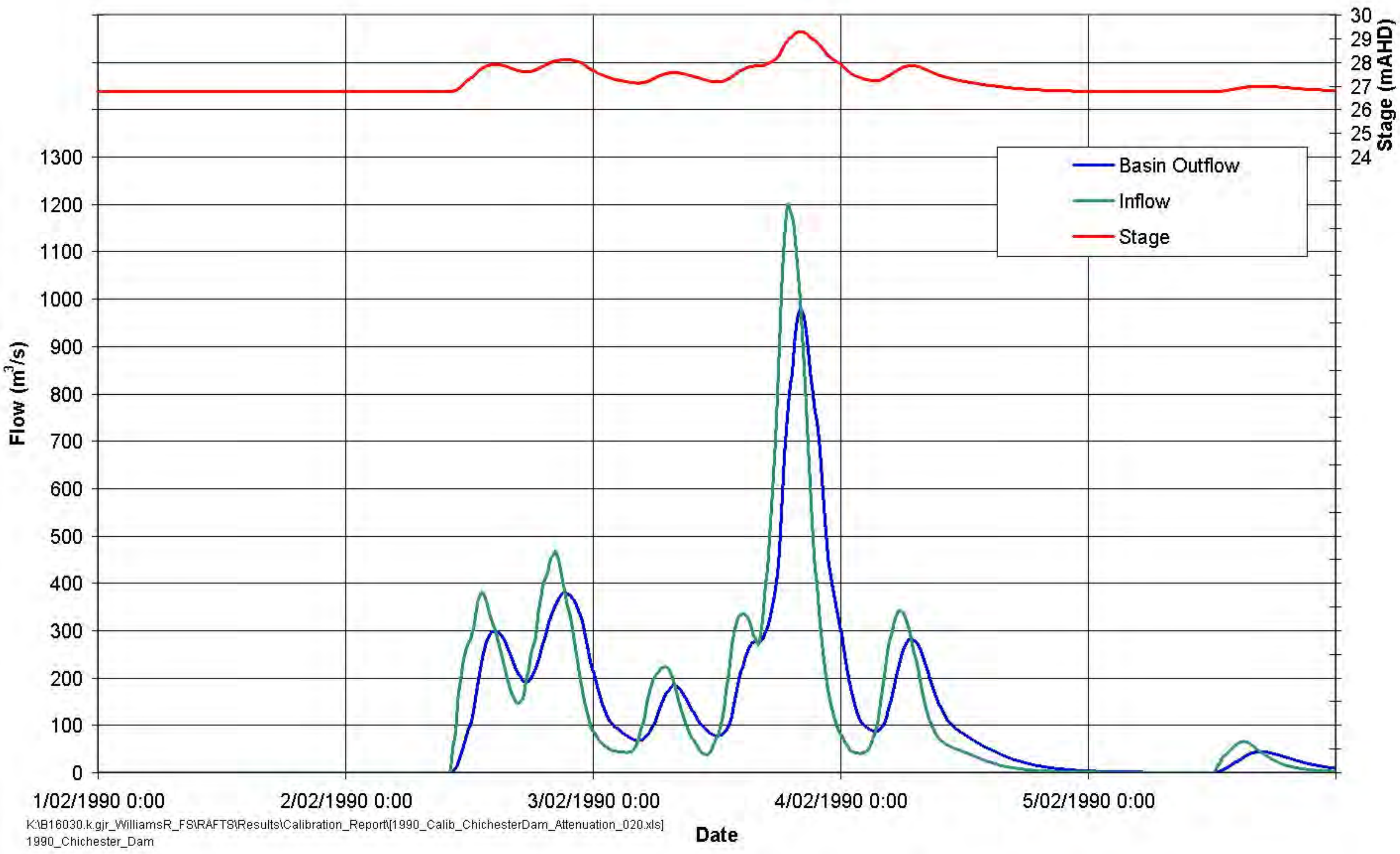

Figure 5-18 February 1990 Chichester Dam Routing of Hydrograph

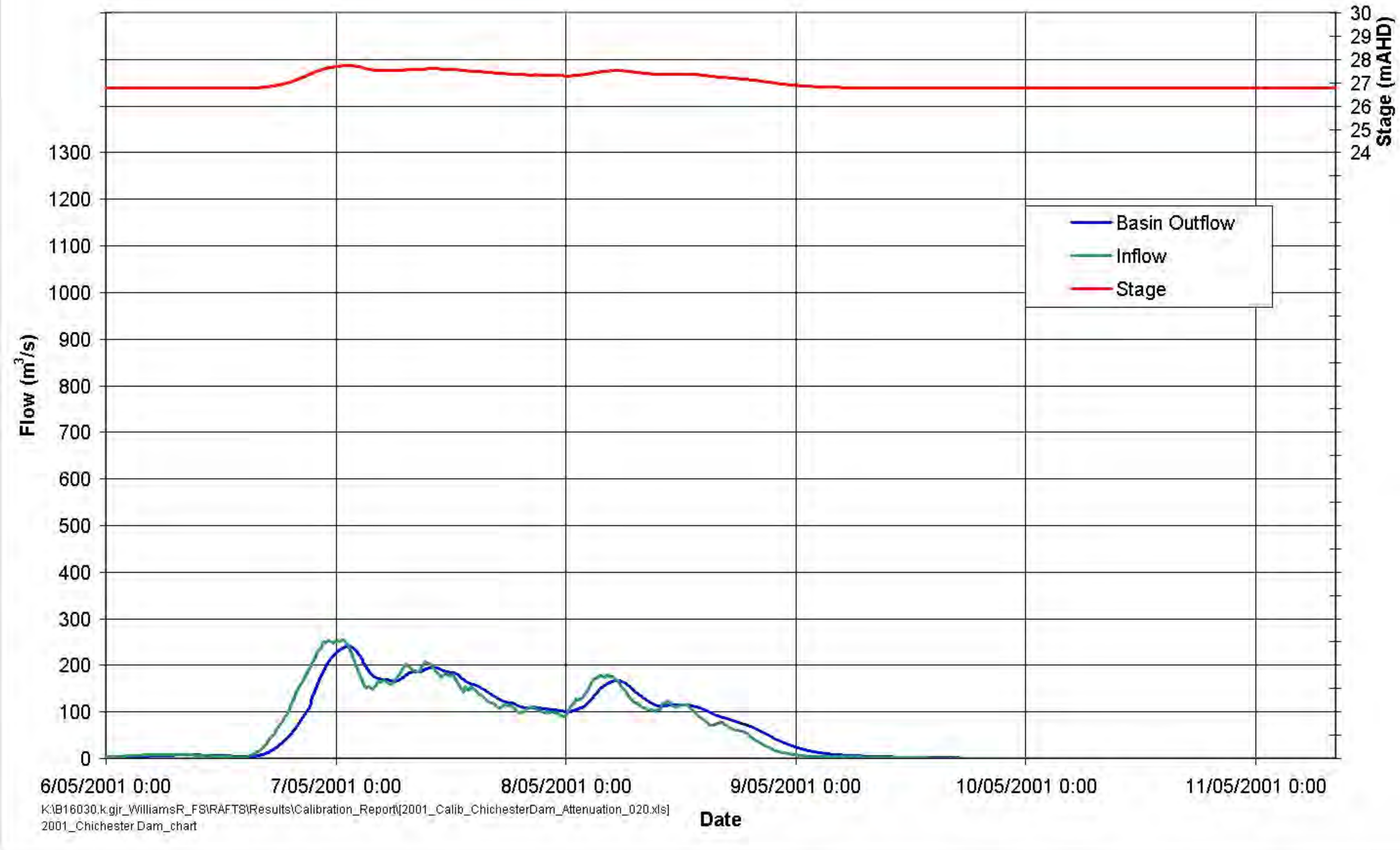

Figure 5-19 May 2001 Chichester Dam Routing of Hydrograph

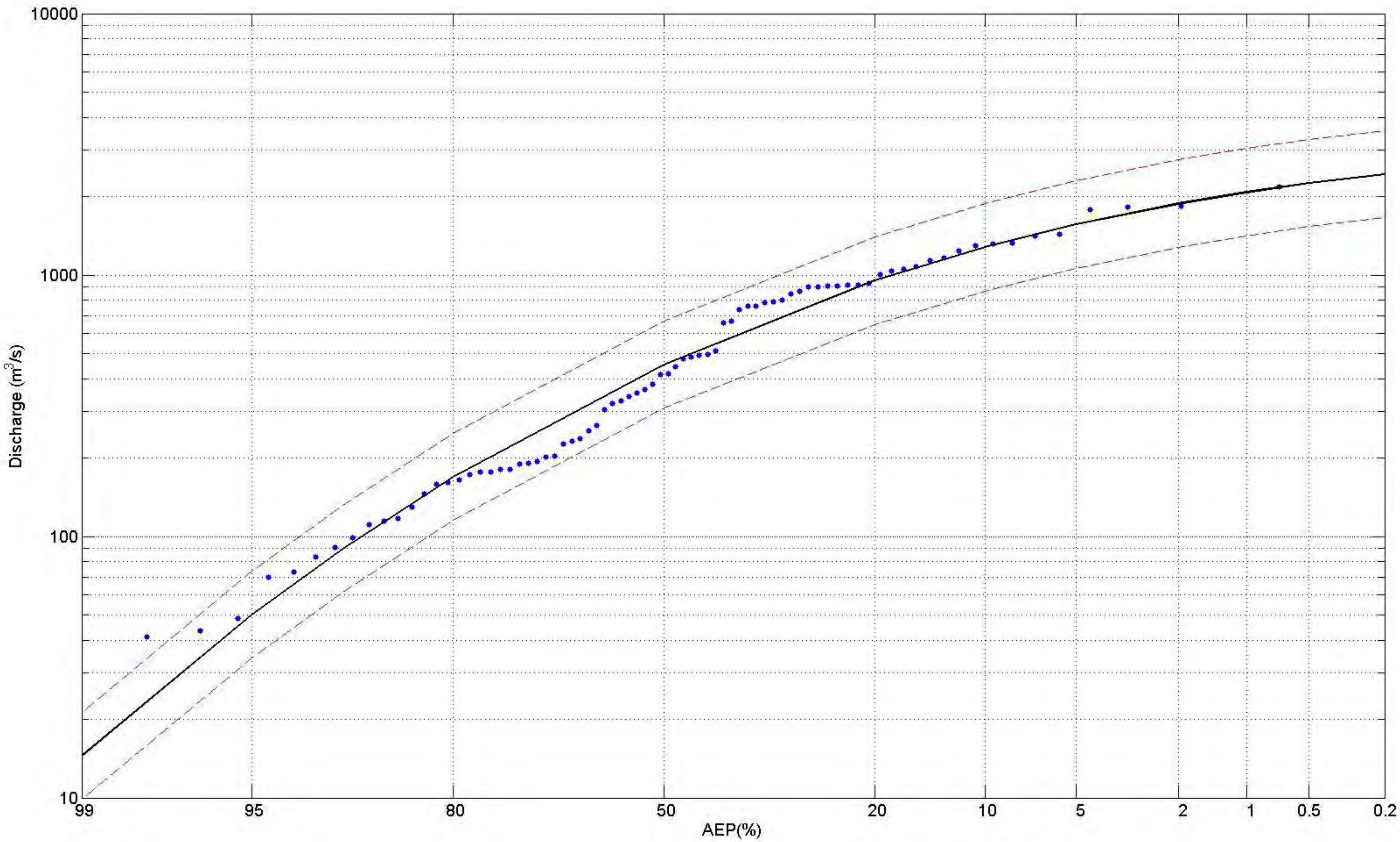

Figure 5-20 LPIII Distribution Fitted to Glen Martin Annual Maximum Discharge Series

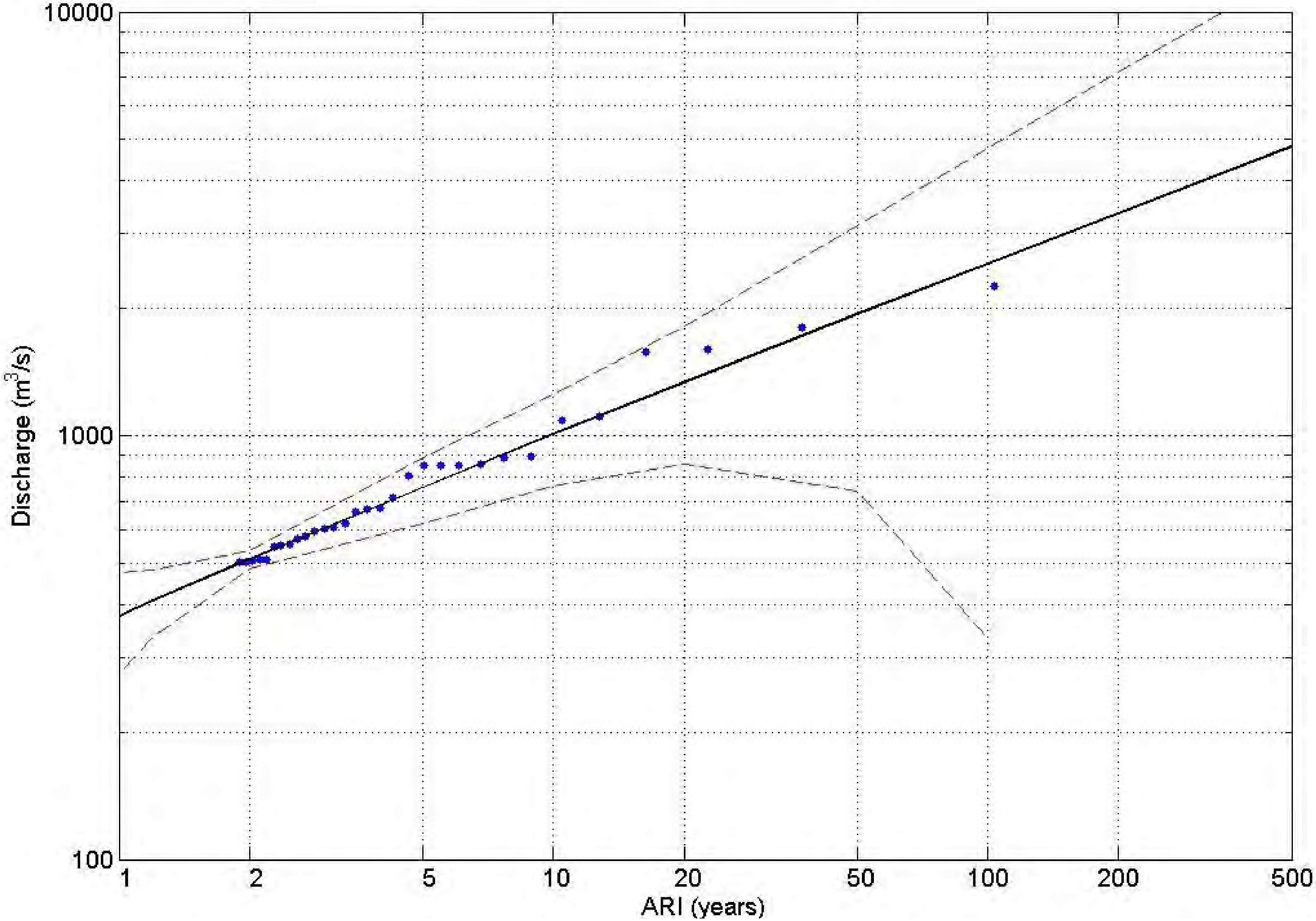

Figure 5-21 Generalised Pareto Fitted to Dungog Peaks Over Threshold Series

6 HYDRAULIC MODELLING

6.1 Hydraulic Model Development

6.1.1 Williams River 2D/1D Model Extent

The hydraulic model covers an area of 146 km^2 from approximately 5km upstream of Dungog down to Raymond Terrace (at the junction with the Hunter River). The model is based on a 40 m square grid, resulting in approximately 90,000 2D cells, with 163 1D sections representing the Williams River and tributaries. The TUFLOW software (www.tuflow.com) was used to develop and simulate the hydraulic model. The extent of the 2D/1D model is shown in Figure 6-1.

The basis of a hydraulic model is the representation of the underlying topography. This may be a DEM for use in 2D modelling, or cross-sections for use in 1D modelling. From this base, the model is 'built' up to represent the conditions that are required to be modelled. Major structures such as bridges and levees are added. Key model parameters are then defined. Parameters for the 2D cells such as the hydraulic roughness (e.g. vegetation density) and linkages to the 1D elements are specified. Finally, boundaries from the hydrologic model are established.

Twelve areas of different land-use type based on aerial photography and site inspections were identified for setting Manning's n values. The calibrated values for Manning's n for the 2D cells are listed in Table 6-1 and the spatial distribution of these varying land-uses is presented in Figure 6-2.

Table 6-1 Calibrated Manning's n Values for 2D Domain

Land Use Type	Manning's n Value
Road	0.030
Waterbody/Dam	0.030
Swamp	0.035
Pasture	0.038
Very Light Riparian Vegetation	0.040
Very Light Vegetation	0.045
Light Riparian Vegetation	0.050
Medium Riparian Vegetation	0.055
Light-Medium Density Vegetation	0.060
Medium Density Vegetation	0.070
Thick Vegetation	0.100
Plantation	0.100
Urban/Town	1

The 1D model network representing the Williams River (and Myall Creek) was developed as part of the TUFLOW 2D/1D model. This network was developed using the river cross-sections derived from various sources (see Section 3.2). The location and number of channels in the 2D/1D model can be seen in Figure 6-1.

The Manning's n values used for representation of the hydraulic roughness of the bank-to-bank section of the river were the subject of much of the calibration effort. The resulting values vary along the river and are presented in Figure 6-2. Discussion on how these values were derived is included in the discussion of the hydraulic model calibration events (see Section 6.2).

6.1.2 Structures and Levees

It is important to represent the crest of the levee system in the 2D/1D models used in this study. An accurate representation of these levee crests ensures that break-out flow from the river(s) into the floodplains occur at the correct river levels. As well, the rate of flow onto the floodplain in the early stages of overtopping is dictated by weir-like flow. The relationship between depth of flow over the levee and rate of flow in weir-like flow is such that the flow rate is very sensitive to the depth of flow (i.e. flow is proportional to depth to the power of 1.5).

Fortunately for this project, DNR has recently completed a full survey of the levee systems in the area using ground surveying techniques. As well, all major drains and floodgates were surveyed. These data were supplied to the study team. 3D breaklines were created to modify the cells along the river banks to ensure the levee system was correctly represented in the 2D domain. As well, 1D culverts (with uni-directional flow) were inserted into the model to represent the floodgates.

The location of all ridge breaklines (representing levee crests or road / rail embankments) and gully breaklines (representing small drains) are presented in Figure 6-3.

The major bridges along the Williams River were modelled in 1D using cross-sections to represent the open waterway area underneath the bridge deck and weirs to represent flow over the bridge deck. The specification of additional energy losses were based on bridge drawings obtained from the RTA, ARTC and Council. These drawings were used to calculate bridge loss coefficients using the techniques described in the Waterway Design: A Guide to the Hydraulic Design of Bridges, Culvert and Floodways (AUSTROADS 1994).

6.1.3 Linkage to Hunter River 2D Model

There is considerable interaction between flooding in the lower parts of the Williams River and the Hunter River. Hence, the 2D1/D TUFLOW model of the Williams River was linked to a 2D/1D TUFLOW model of the Hunter River. This Hunter River model was developed as part of a project for the Roads and Traffic Authority (RTA) and is being used for investigations into a new Pacific Highway (F3) crossing of the Hunter River.

The 2D/1D TUFLOW model of the Hunter River extends from Green Rocks to the mouth of the Hunter River. This is the same extent of the MIKE-11 model developed for the lower Hunter River Flood Study (PWD, 1994). The Hunter River model has a 40m cell size. This is the same as the lower Williams River model.

The TUFLOW Hunter River model included a relatively rough representation of the floodplain storage in the lower Williams River floodplain. Further, the model only included a small length of the lower Williams River. Hence, there was some degree of reconfiguration of the extents and structure of this Lower Hunter model in order to enable a proper link to the Williams River model.

The linkage between these models and the layout of the lower Hunter River 2D/1D model is shown in Figure 6-4.

6.1.4 Seaham Weir

Seaham weir was represented in the 1D model network as four parallel channels. Two channels were used to represent the floodgates. One channel represents the 120m wide concrete weir (elevation of 1.178mAHD), the final channel represents the approximately 220m wide rock weir (varying crest levels). Losses over the Seaham weir were calibrated to match those observed in calibration events.

Calibration of the losses over the weir was done to recorded levels in May 2001. A 1D only model of the Seaham weir was created for the purpose of loss calibration. Downstream water level of the calibration model was the time-series recorded by Manly Hydraulics Laboratory (MHL). Flow at the upstream boundary was extracted from the 2D / 1D hydraulic model. TUFLOW weir calibration factors were varied in order to match observed water levels upstream of the weir. Water levels upstream of the weir were recorded by Hunter Water, these are daily recordings with a 9am observation. The calibration is presented in Figure 6-5.

Due to constraints of the hydraulic model, floodgates are modelled as open initially. This acts to lower the modelled level upstream of the weir in the early stages of the flood. Floodgates would not have been open in the early stages as HWC operation policy aims to prevent saltwater intrusion into the weir pool. This is the probably explanation of the discrepancy between the observed and modelled level on the 7th of May (1st graph point).

The 1978 event was used to verify of the weir loss calibration. The MHL gauge downstream of the weir did not exist, thus it was necessary to use results from the 2D / 1D hydraulic model. Flow at the upstream boundary was extracted from the 2D / 1D hydraulic model. Peak water level upstream of the weir for the March 1978 event was recorded on a DNR flood board. Results of the weir loss verification are presented in Figure 6-6.

6.2 Hydraulic Model Calibration

6.2.1 General Approach

The hydrological model output for the historical events has significant uncertainties primarily due to the shortage of pluviograph data. To improve the inflows to the hydraulic model upstream of Dungog, the hydrographs were derived from the gauged levels (at the Dungog gauge on the Williams River). Inflows for tributaries downstream of this point were derived from the calibrated hydrological model as no gauges were available. The 2D model extent was reduced to reflect this inflow location upstream of Dungog (see Figure 6-1).

The hydraulic model calibration process involved the following general approach:

- The February 1990 flood event calibration was used to calibrate the lower section of the Williams River model and the lower Hunter River model as it was the calibration event with the largest Hunter River flows (coincident with a Williams River flood).
- The March 1978 flood event calibration was used to calibrate the section of river from Clarence Town to Seaham as it contained good peak level data along this reach. This flood event also had relatively well defined flood peaks, enabling calibration of the timing of the flood peak movement down the river system.
- The May 2001 flood event was used as a general verification event following the above two steps. The May 2001 flood is the only significant flood event since the installation of the automated recorder at Dungog (even though it was not a major flood event).
- The gauge at Glen Martin provides a useful record to allow calibration of the magnitude of flood flows throughout the calibration events. As well, it enables checks on the performance of the hydraulic and hydrological models in terms of the timing of flood peaks and rates of floodwater rise and recession. The flow records from the Glen Martin gauge have a relatively high degree of confidence due to the high flows actually recorded at the gauge site during flood events.

There was not any evidence of significant changes in floodplain geometry between 1978 and 2001 that warranted changes to the hydraulic model to represent each flood event.

6.2.2 March 1978 Flood

The March 1978 flood event is a two-peak event with the second smaller peak following the first peak by about four (4) days. In between the two peaks, little flow occurred. The downstream boundary for the model was the recorded levels at Raymond Terrace; this is presented in Figure 6-7. The performance of the hydraulic model in replicating recorded hydrographs (both level and flow) for the March 1978 flood event are presented in Figure 6-8 to Figure 6-13.

Manning's n values for the bank-to-bank flow were varied to provide the optimal calibration with recorded data. Floodplain Manning's values were also varied. Re-rating of the Dungog Gauging Station (see Section 5.3) had a significant effect on improving the modelled levels and flows.

A Manning's n value of 0.05 was required to match the flood model performance to the recorded levels along the Clarence Town to Seaham reach. This level may seem high for the degree of vegetation along the riverbanks. Typically, a straight river reach with this type of vegetation would require a Manning's n of between 0.035 and 0.045. However, the 1D Manning's n value also accounts for turbulence and energy loss around bends. This section of the river (and the remainder of the Williams River upstream) is relatively sinuous and was assigned an artificial increase in Manning's n of around 0.01 to account for this energy loss.

Furthermore, the long-section survey indicated that the bed levels along this stretch of the river (but possibly not the thalweg) could vary from the surveyed bed levels of discrete cross-sections by up to 3m. The depth of flow (at the peak of the March 1978 flood) in this reach of the river is in the order of 13m. Hence, an average over-representation of the river depth by 1.5m on this size of river and friction slope corresponds to an increase in Manning's n of 0.01.

Hence, the calibrated Manning's n value for this reach (and assumed for reaches further upstream) is considered to be within acceptable bounds.

The performance of the hydraulic model in replicating recorded peak flood levels for the March 1978 flood event is presented in Figure 6-10 (long section) and Figure 6-13 (plan plot). Observed and modelled levels for calibration points are presented in Table 6-2. Manning's n values were varied to provide the optimal calibration with recorded data.

Calibration points 1978-7 and 1978-10 (see Table 6-2) indicate a negative flood gradient (i.e. the points are higher than those immediately upstream). In one location (1978-4), this may be due to the recorded point representing flood levels at the outside of a river bend. However, the other occurrence is on a relatively straight stretch of river. Hence, this discrepancy with expected flood behaviour highlights the anticipated accuracy of these levels to be in the order of $\pm$ 0.15m.

Calibration point 1978-16 (see Table 6-2) is located approximately 100m from Glen Martin gauging station and indicates a peak level of 9.67mAHD. The peak level at the Glen Martin gauging station was 10.85mAHD. A flood gradient drop of 1.18m over 100m cannot be explained. The gauging station data is a higher quality data source. Hence, the recorded level at calibration point 1978-16 is ignored for the purposes of this flood calibration exercise.

The records at the Dungog gauging station for the March 1978 flood only cover the times of high flow. There is no recorded data for the receding limb of either flood peak below approximately 600m^3/s. For the rising limb there is no data below 300m^3/s for the first flood peak and no data below 100 m^3/s for the second flood peak. Hence there is no data to be used for generating inflows between the two peaks (or much data on the falling limb of either peak). Discrepancies at the Glen Martin gauging station between recorded data and modelled data are considered to be primarily due to this lack of recorded flow data at the low flow times and recession.

Table 6-2 Calibration Points – March 1978 Flood Event

Calibration Point	Data Source	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
1978-1	DNR - DS Weir	3.74	3.79	0.04
1978-2	HWA Seaham Daily Levels	4.25	4.22	-0.04
1978-3	DNR Floodboard-12	4.38	4.48	0.10
1978-4	DNR Floodboard-1A	4.47	4.52	0.05
1978-5	DNR Floodboard-11A	4.56	4.64	0.08
1978-6	DNR Floodboard-2A	4.64	4.72	0.08
1978-7	DNR Floodboard-3A	4.93	4.84	-0.09
1978-8	DNR Floodboard-10A	4.89	4.97	0.08
1978-9	DNR Floodboard-4A	5.16	5.08	-0.08
1978-10	DNR Floodboard-9A	5.36	5.31	-0.06
1978-11	DNR Floodboard-6A	5.32	5.48	0.15
1978-12	DNR Floodboard-8A	5.45	5.59	0.14
1978-13	DNR – Grey St	5.87	5.87	0.00
1978-14	DNR Floodboard-7A	6.3	6.39	0.09
1978-15	Glen Martin Gauging Station	10.85	11.13	0.28
1978-16	DNR Floodboard-Glen Martin	9.67	10.91	1.24
1978-17	SES Dungog (B. Hartcher)	50.22	50.40	0.18

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6.2.3 February 1990 Flood

The February 1990 flood event was a complex calibration exercise as it also included calibration of the Hunter River section of the hydraulic model. The calibration of the March 1978 and May 2001 flood events were achieved using the recorded levels at the Raymond Terrace gauge as the downstream boundary condition of the model. The calibration of the February 1990 used the entire Williams River and Hunter River 2D/1D flood models. Hence, the downstream boundary for this flood simulation was the recorded levels at the mouth of the Hunter River. The performance of the hydraulic model in replicating recorded hydrographs (both level and flow) for the February 1990 flood event are presented in Figure 6-13 to Figure 6-17.

Manning's n values for the bank-to-bank flow were varied to provide the optimal calibration with recorded data. Manning's n values for areas on the floodplain were also varied. Re-rating of the Dungog Gauging Station (see Section 5.3) had a significant effect on improving the modelled levels and flows.

At Raymond Terrace, most of the discrepancy between the model hydrograph and the recorded hydrograph is likely to be due to inaccuracies in the inflow hydrograph from the Hunter River flows. This inflow is not based on recorded data but on another 1D hydraulic model output (MIKE-11 model). The calibration of the Lower Hunter River MIKE-11 model shows a better comparison at Raymond Terrace. However, this model did not fully represent the lower Williams River and used a delayed hydrograph of the Glen Martin inflows for Williams River inflows. This assumption could be masking other discrepancies in the MIKE-11 model.

The performance of the hydraulic model in replicating recorded peak flood levels for the February 1990 flood event is presented in Figure 6-19 (long section) and Figure 6-22 (plan plot). Observed and modelled levels for all calibration points are presented in Table 6-3.

Downstream of Clarence Town there are only four recorded levels along this stretch of the river. Point 1990-6 (see Table 6-3) was provided by a land-owner as a level on a pump shed. This level was surveyed to be 3.77mAHD. However, this level is not in accordance with other flood information for the area. Flood levels of 5.16mAHD and 4.89mAHD were recorded approximately 500m upstream and downstream for the March 1978 flood. That flood was slightly smaller in peak flow than the February 1990 flood with similar Hunter River tailwater levels. Furthermore, the gauge at Seaham for the May 2001 flood recorded a peak level of 3.5mAHD. This gauge is 7km downstream and the May 2001 flood was significantly smaller than the February 1990 flood event.

A recorded point further downstream 1990-5 (see Table 6-4) was provided by a land-owner as a level on a tree. This was surveyed to be 2.86mAHD. This level is not in accordance with other levels in the area. The recorded level at Raymond Terrace was 3.01mAHD. The same land-owner provided a flood level as a mark on a caravan for the May 2001 event, which was a significantly smaller event. The May 2001 mark was surveyed to be 3.08mAHD.

Calibration point 1990-3 (see Table 6-3) is a floor level on King Street Raymond Terrace inside the levee system. Discussions with Mr. Bill Bobbins, who provided the flood level, indicated that 1990 peak level in the river was below levee spillway. Mr. Bobbins believes flooding occurred at this site in the February 1990 because the floodgates on the nearby culvert did not open. The recorded peak water level at Raymond Terrace gauging station was 3.01mAHD. The surveyed flood mark provided by Mr. Bobbins was 2.2mAHD. This validates Mr. Bobbins' hypothesis that the floodgates were unable to open for some period during the February 1990 flood event. Hence, flooding in this location is due to local runoff from behind the levee system. This localised runoff is not modelled for the purposes of the Williams River Flood Study.

The records at Dungog gauging station for the February 1990 flood event only cover the periods of high flow. There is no recorded data for flows below 400m^3/s. Hence there is no data to be used for the generation of inflows (or for calibration points) for the rising and falling limb of the hydrograph. Discrepancies between recorded peak levels and modelled peak levels are considered to be primarily due to the uncertainties associated with the data input to the model.

Table 6-3 Calibration Points – February 1990 Flood Event

Calibration Point	Data Source	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
1990-1	Stockton Bridge Gauging Station	1.08	1.06	-0.02
1990-2	Hexham Bridge Gauging Station	1.64	1.59	-0.05
1990-3	WBM Interviews	2.20	*Not Flooded See Discussion	-
1990-4	Raymond Terrace Gauging Station	3.01	3.02	0.01
1990-5	WBM Interviews	2.86	3.27	0.41
1990-6	WBM Interviews	3.77	5.09	1.32
1990-7	WBM Interviews	6.04	5.91	-0.12
1990-8	WBM Interviews	6.77	6.19	-0.59
1990-9	WBM Interviews	8.81	8.62	-0.19
1990-10	Glen Martin Gauging Station	10.84	10.97	0.13
1990-11	SES (A. Nash)	50.42	50.40	-0.02

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6.2.4 May 2001 Flood

The downstream boundary for the model was the recorded levels at Raymond Terrace; this is presented in Figure 6-23.

The performance of the hydraulic model in replicating recorded hydrographs (both level and flow) for the May 2001 flood event are presented in Figure 6-25 to Figure 6-26. This event was primarily used as a verification event for the previous two events.

The performance of the hydraulic model in replicating recorded peak flood levels for the May 2001 flood event is presented in Figure 6-29 (long section) and Figure 6-30 (plan plot). Observed and modelled levels for all calibration points are presented in Table 6-4.

Once again, the discrepancies between recorded and modelled data are considered to be primarily due to uncertainties associated with the data input to the model.

Table 6-4 Calibration Points – May 2001 Flood Event

Calibration Point	Data Source	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
2001-1	WBM Interviews	3.08	2.82	-0.26
2001-2	Seaham Gauging Station	3.43	3.46	0.03
2001-3	WBM Interviews	3.63	3.68	0.05
2001-4	Glen Martin Gauging Station	3.92	4.03	0.12
2001-5	WBM Interviews	8.80	9.01	0.21
2001-6	Dungog Gauging Station	32.36	32.25	-0.10
2001-7	Dungog Gauging Station	48.80	48.73	-0.07

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6.2.5 Hydraulic Model Calibration at Dungog

Calibration of the 10m Grid size model was undertaken as part of the re-rating of Dungog Gauge (See Section 5.3.3). This involved inputting a steadily increasing flow into the upstream of the 10m model. Long profiles of the results at a range of flow magnitudes are presented in Figure 6-31.

6.2.6 Conclusions on Hydraulic Model Calibration

The following conclusions can be drawn on the calibration of the hydraulic model to three flood events:

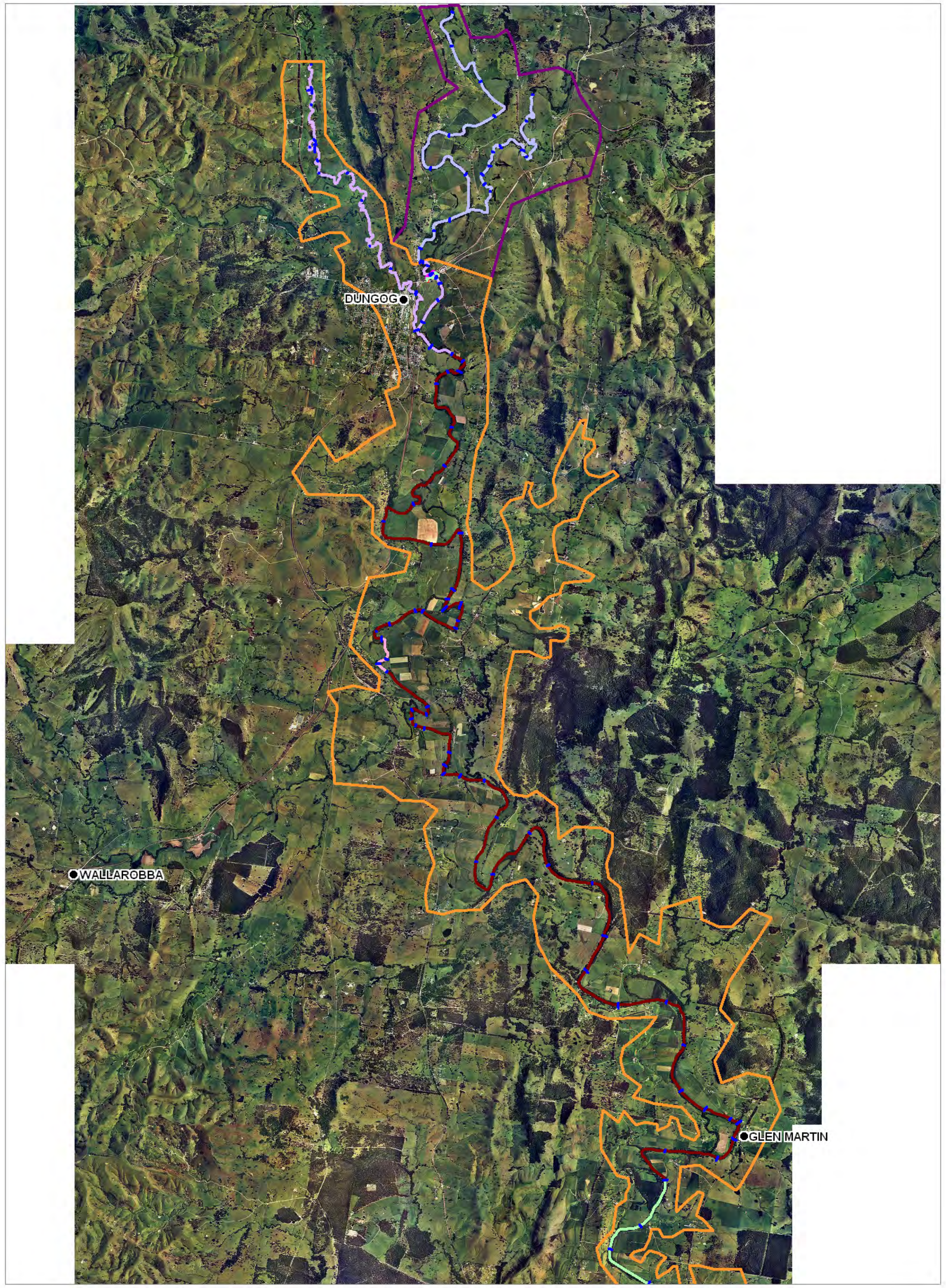
1 The Manning's n value for the lower reaches of the Williams River (upstream of Seaham) of 0.05 is derived from the calibration of the flood gradient in the March 1978 flood. The Manning's n value for the upper reaches of the Williams River (around Dungog) of 0.08 is derived from the calibration to the gauging station recorded flows and levels for the March 2000 flood event (as part of the re-assessment of the Dungog gauging station rating curve See Section 5.3). The Manning's n values for the reaches in between are a linear variation of these values.

2 The shape of the hydrograph and the speed of propagation of the flood wave along the river are considered acceptable given the uncertainties in the input data. This is demonstrated in the calibration to the March 1978 and May 2001 flood events;

3 The combined Williams River and Hunter River 2D/1D model is acceptable at replicating floods in both river systems as demonstrated by the February 1990 flood.

4 Unfortunately, there was only one calibration level available for all three flood events between Glen Martin and Dungog. This point (2001-5) was recorded for the May 2001 flood event, which is a relatively minor flood event. Hence, the quality of the model performance over this significant reach of river (approximately 35km) is largely untested.

In summary, the hydraulic 2D/1D model (linked to the Hunter River 2D/D model) provides an adequate representation of dynamic flood behaviour in the study area for the purposes of this study and subsequent floodplain management studies. However, it needs to be noted that the model is aimed at representing long duration flood events dominated by Williams River flows (and subsequent back-up in tributaries) and not the finer scale flood behaviour and steeper flood gradients of small tributary inflows.



Northern Area

Southern Area

LEGEND

- 2D Domain
- Additional 2D Domain - Design Events
- Cross Sections - HWA
- Cross Sections - DOC
- Cross Sections - DNR

1D Channels
(with Manning's n value)

- Bridge/Weir
- 0.015
- 0.020
- 0.025
- 0.035
- 0.040
- 0.050
- 0.055
- 0.060
- 0.065
- 0.080

Title:

Williams River 2D/1D Hydraulic Model Extent

Figure:
Figure 6-1

Rev:
A

N

0 2 4km
Approx. Scale

BMT WBM
www.wbmpl.com.au

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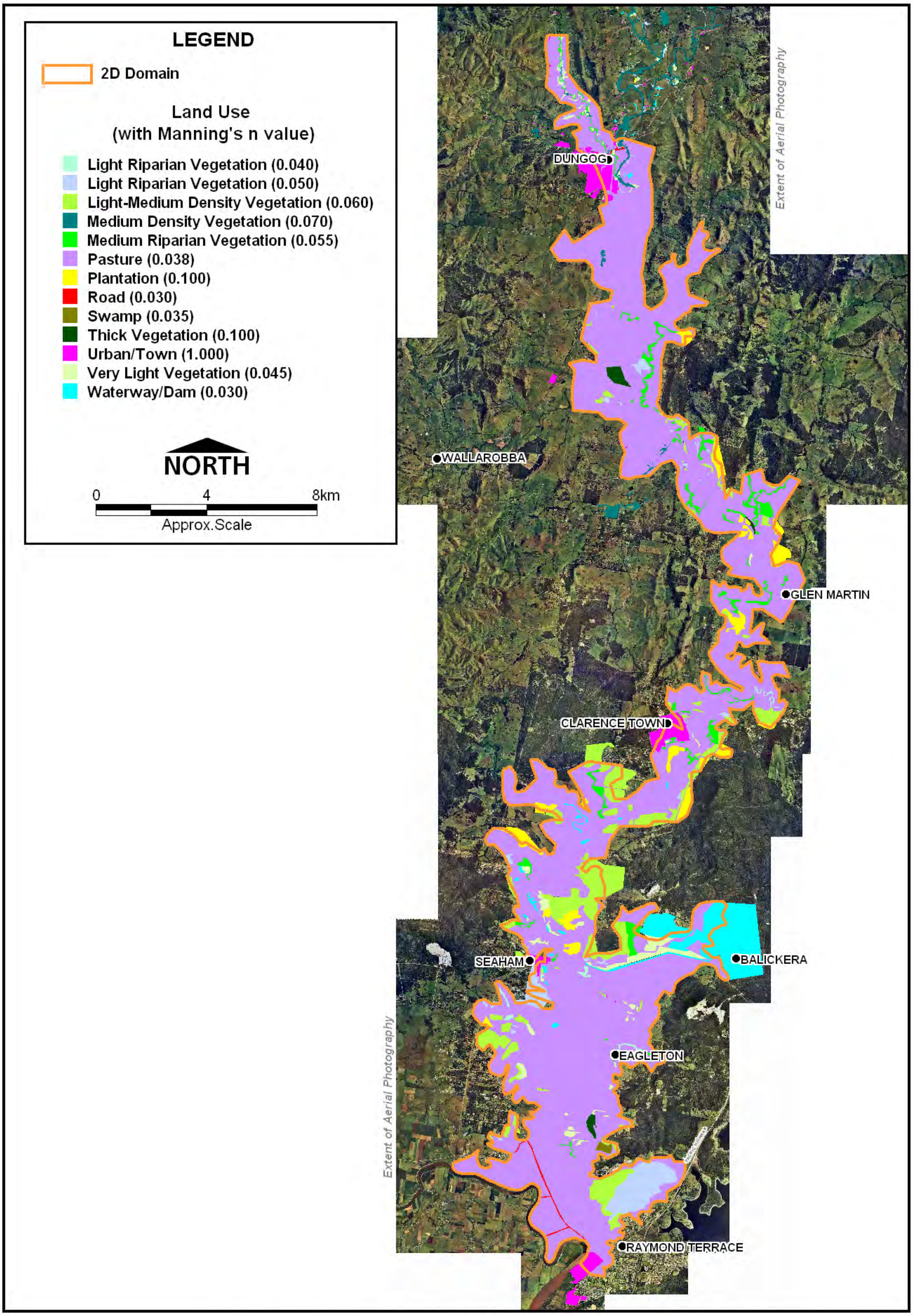

Williams River Land Use (Manning's n) for 2D Domain

Figure 6-2

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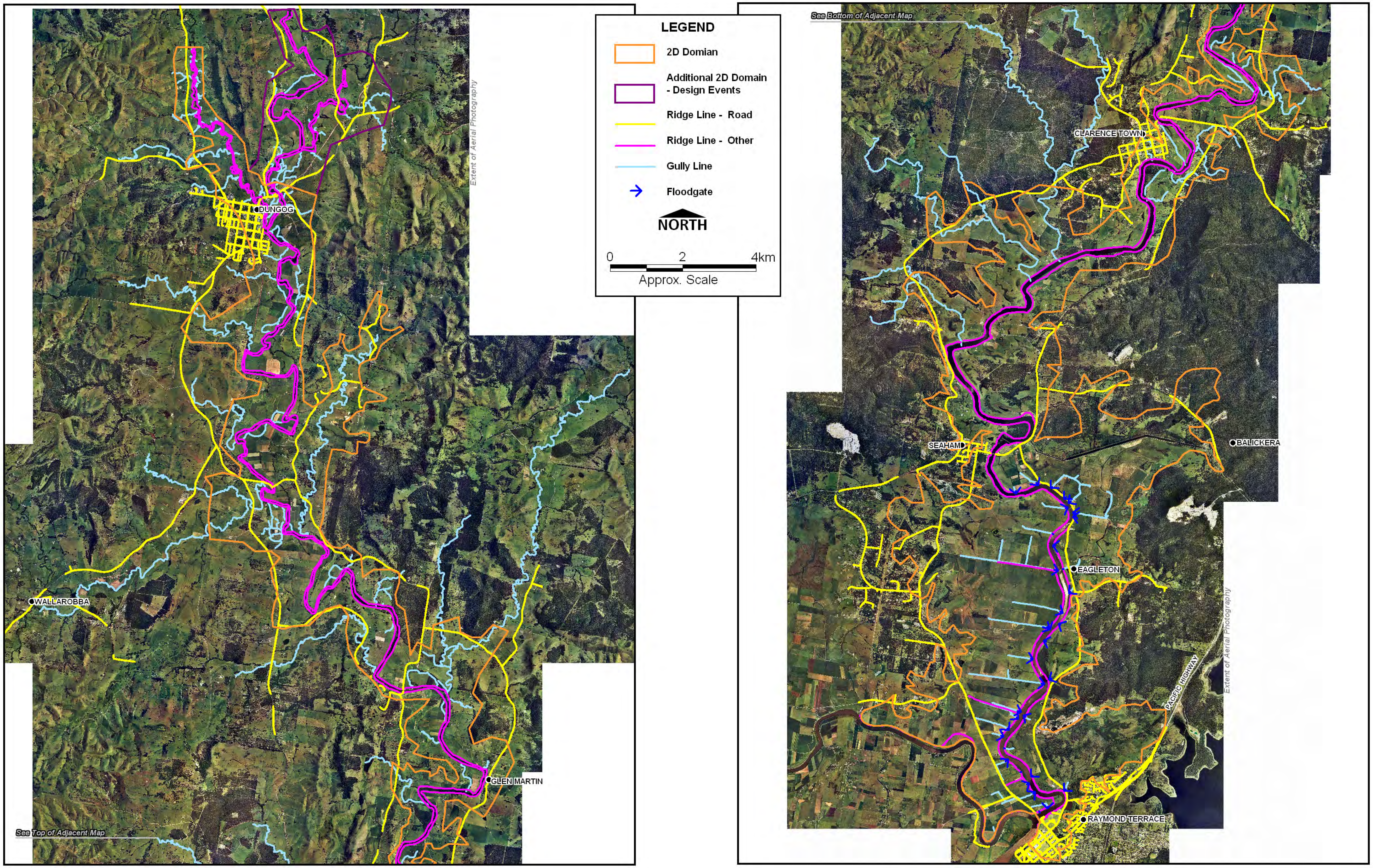
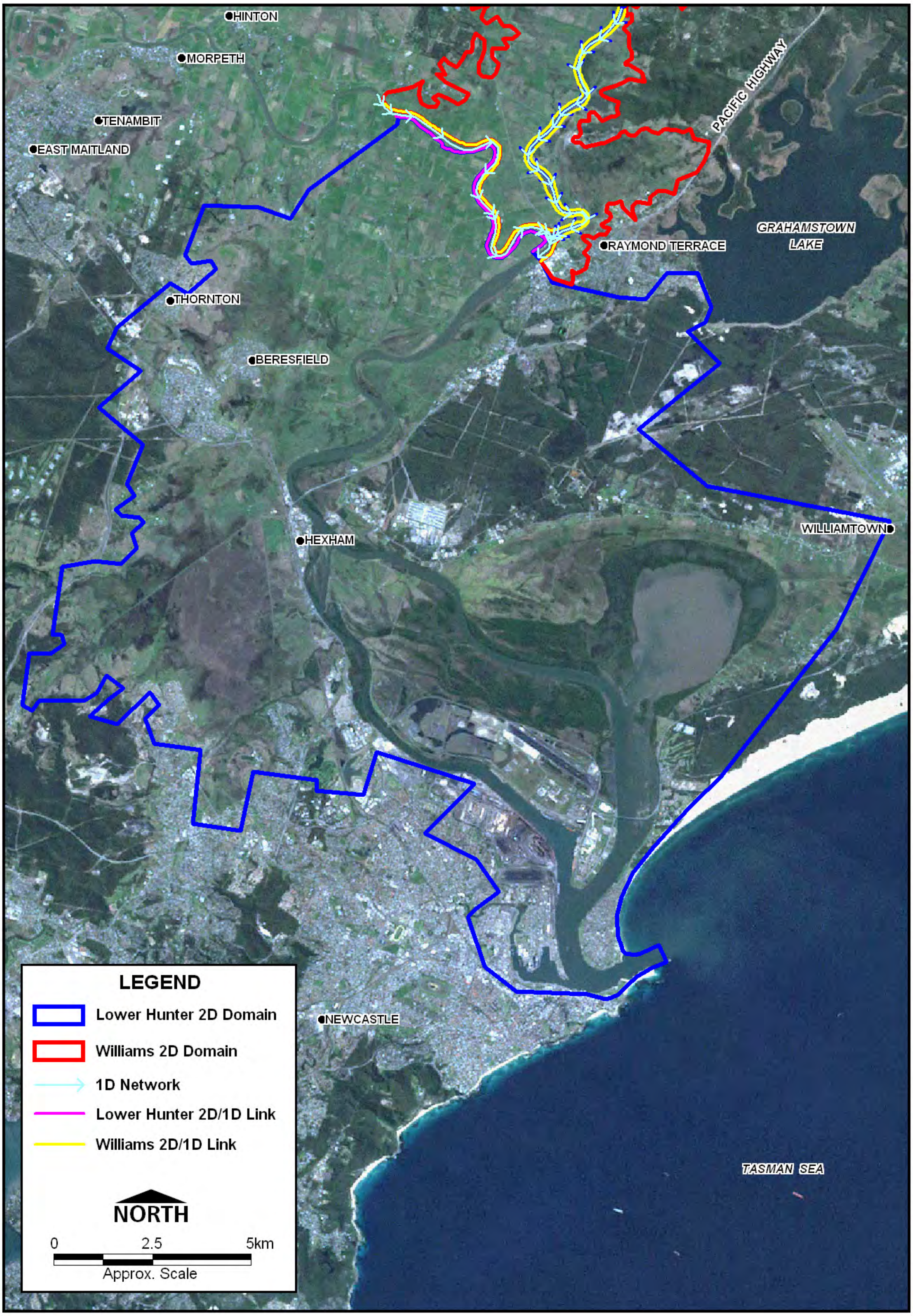


Ridge Lines, Gully Lines and Floodgates **Figure 6-3**



Lower Hunter River 2D/1D Hydraulic Model Extent and Linkages

Figure 6-4

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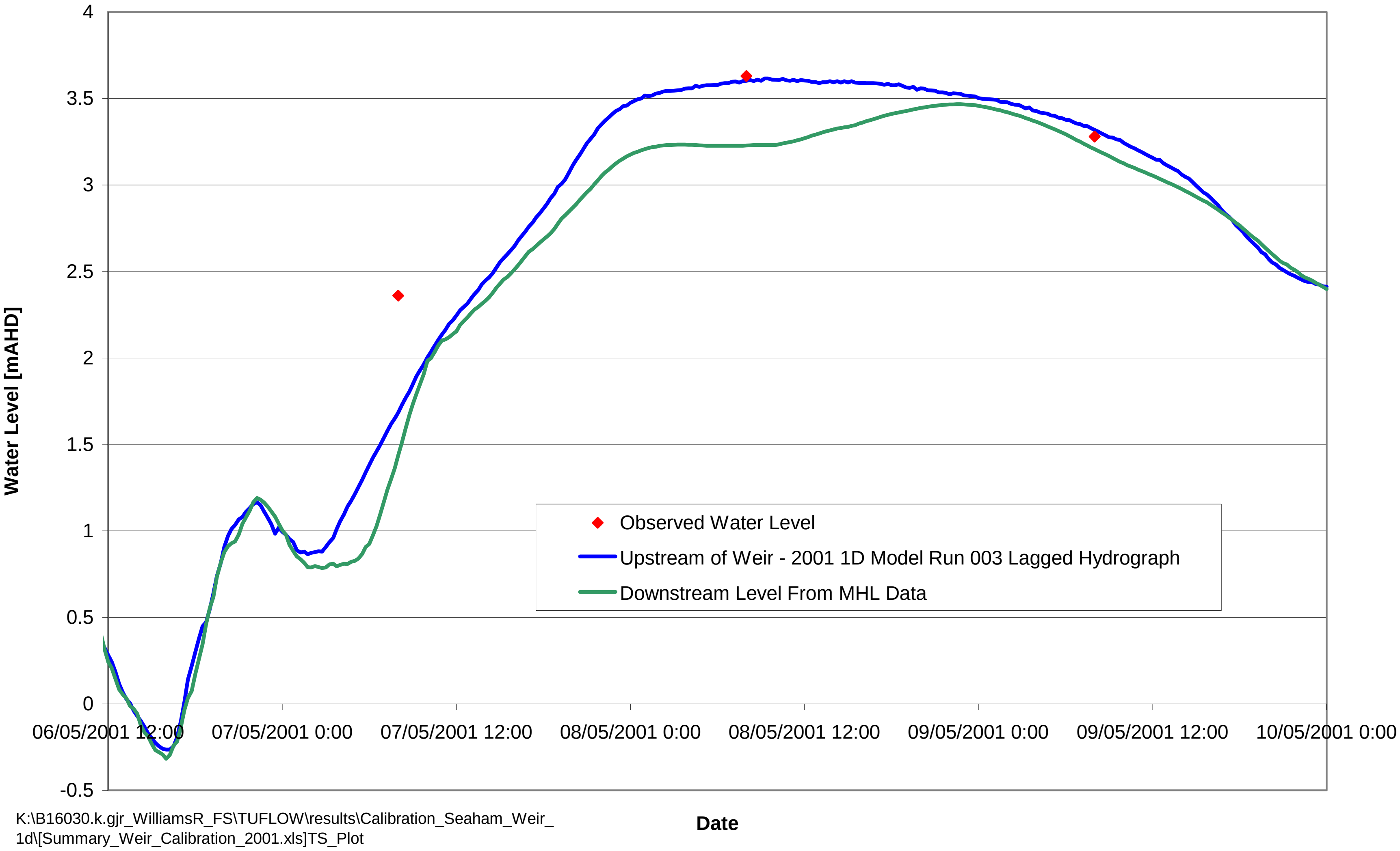


Figure 6-5 Seaham Weir Loss Calibration – May 2001

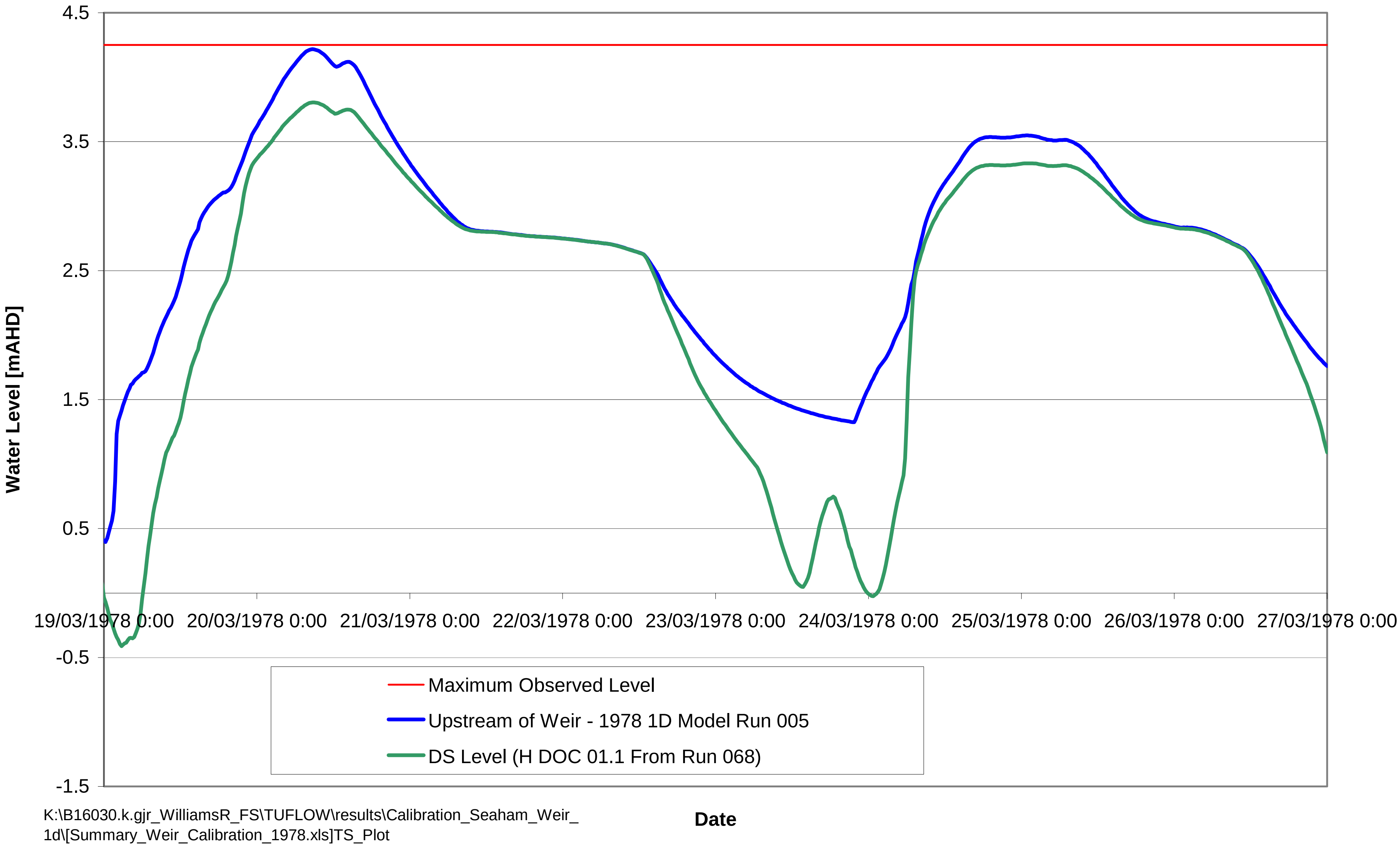


Figure 6-6 Weir Verification Results – March 1978

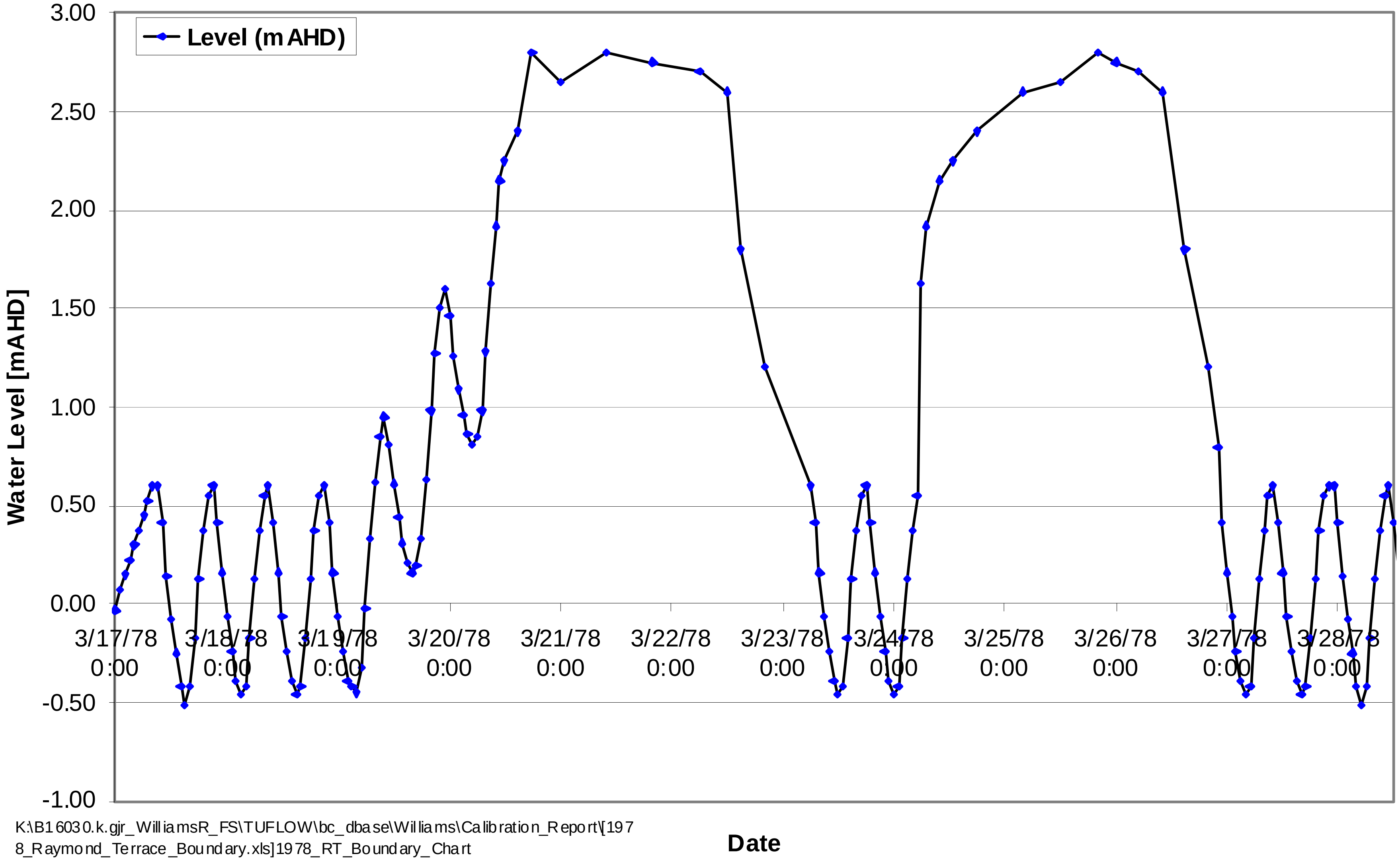


Figure 6-7 Raymond Terrace Recorded Levels as Boundary Condition for March 1978 Flood

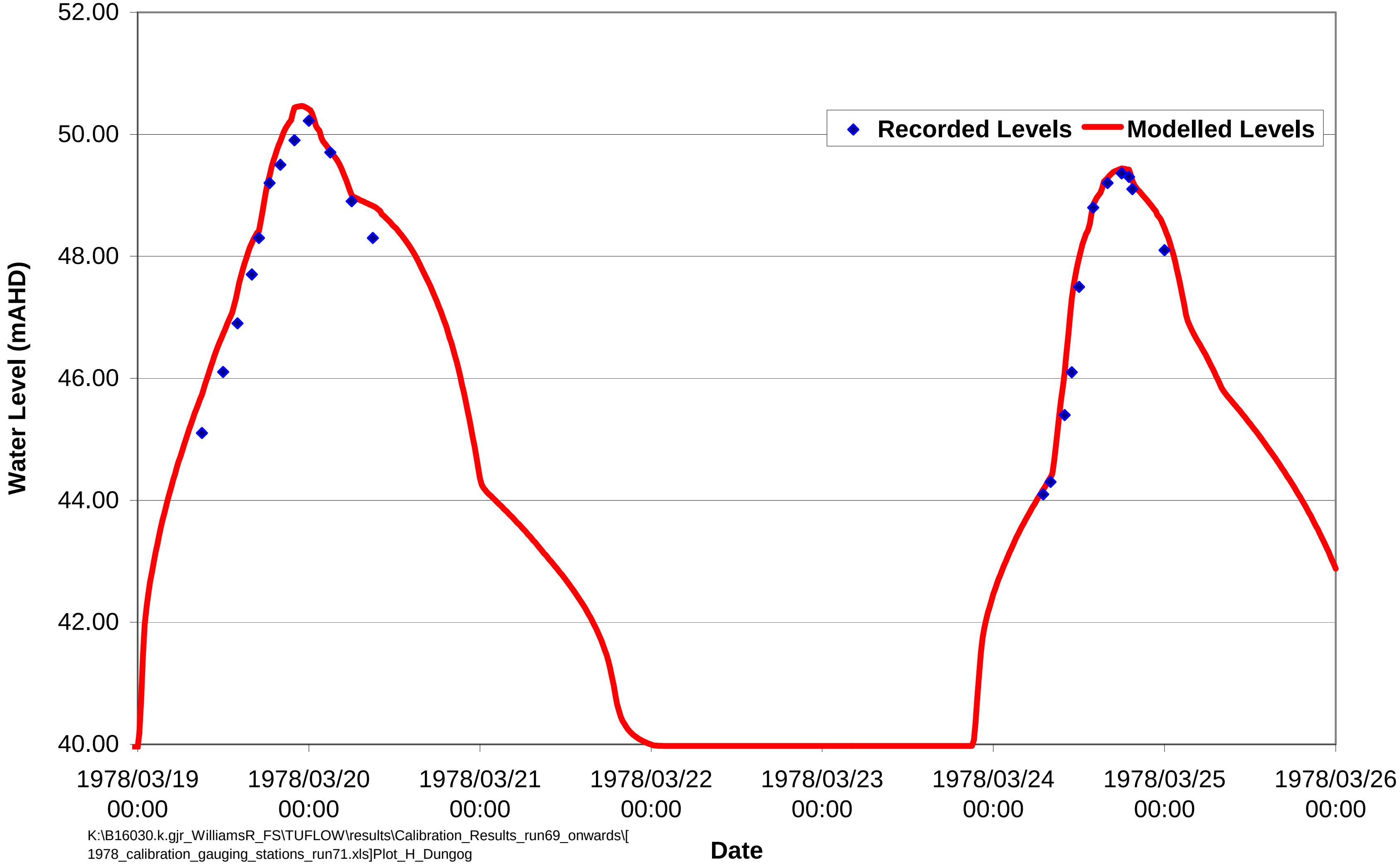


Figure 6-8 March 1978: Dungog Gauge Levels Comparison for Hydraulic Model

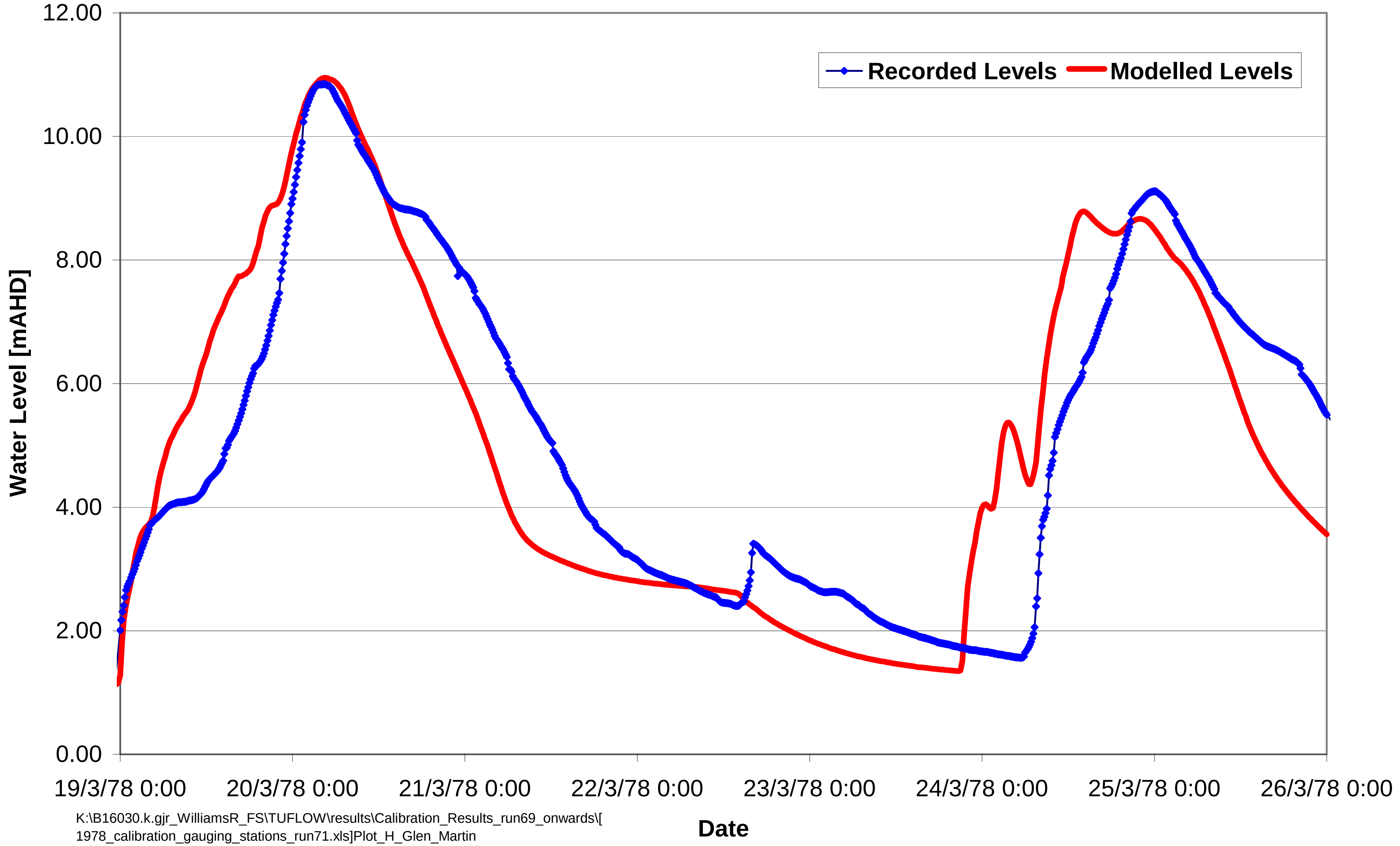


Figure 6-9 March 1978: Glen Martin Gauge Levels Comparison for Hydraulic Model

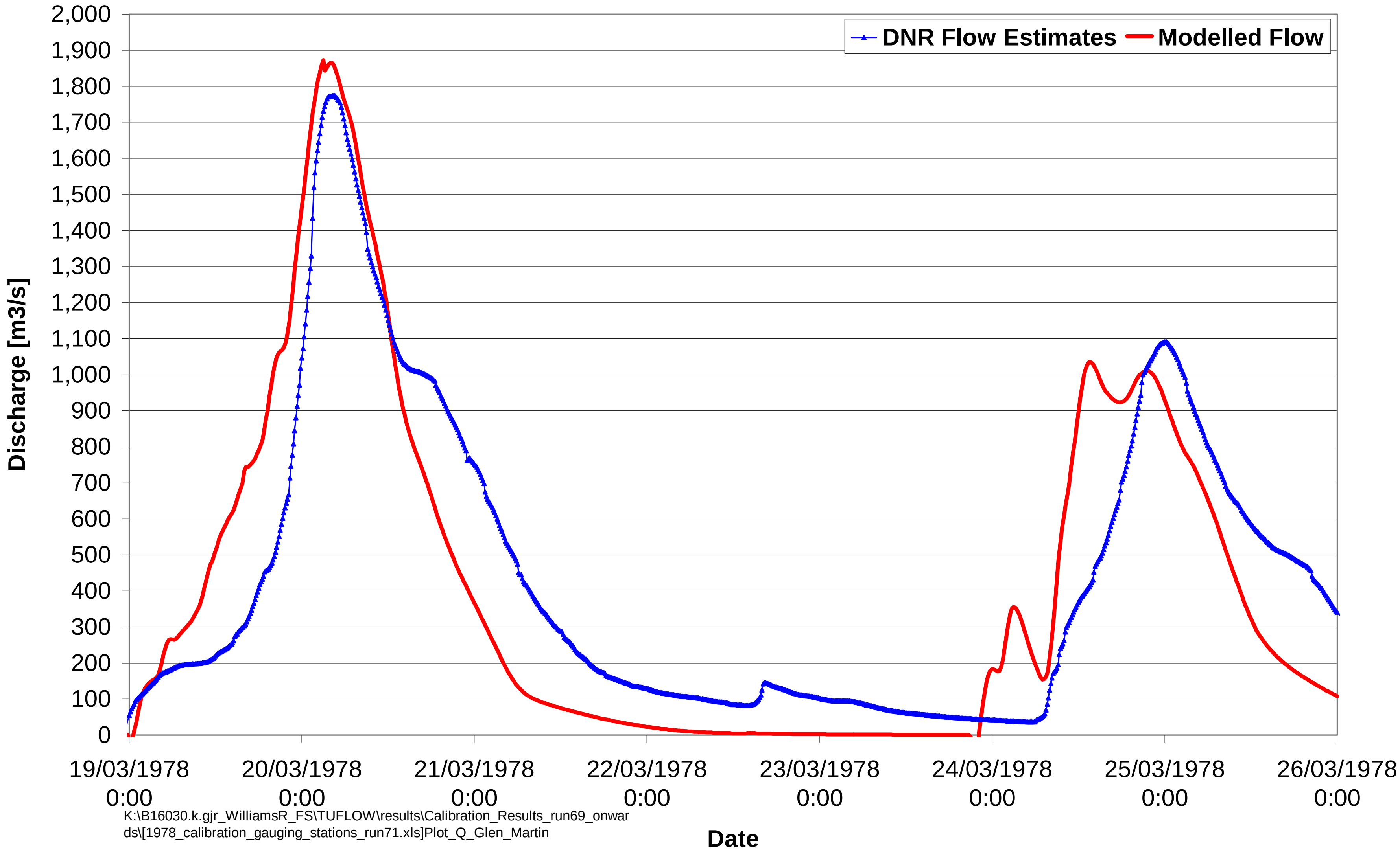


Figure 6-10 March 1978: Glen Martin Gauge Flows Comparison for Hydraulic Model

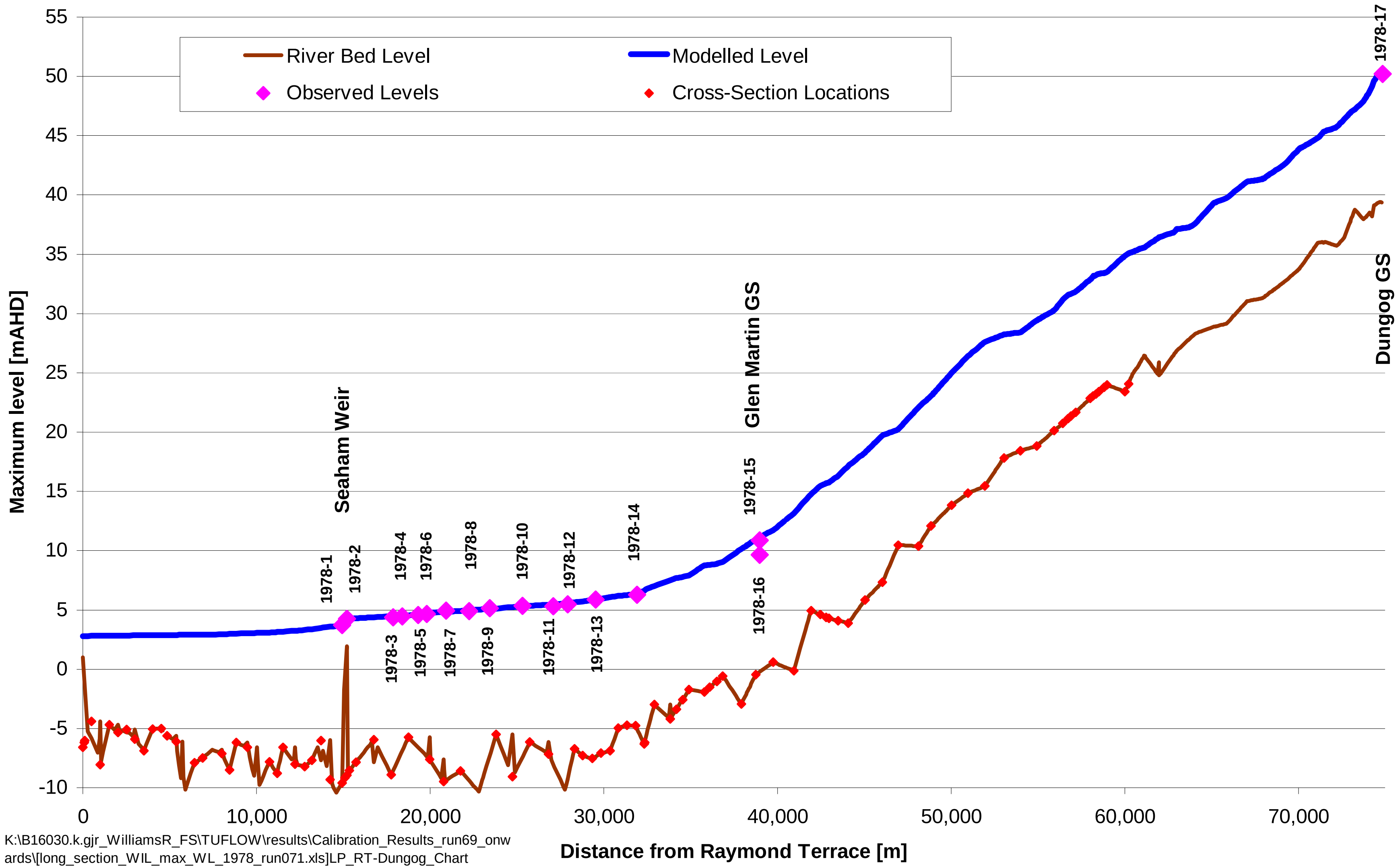
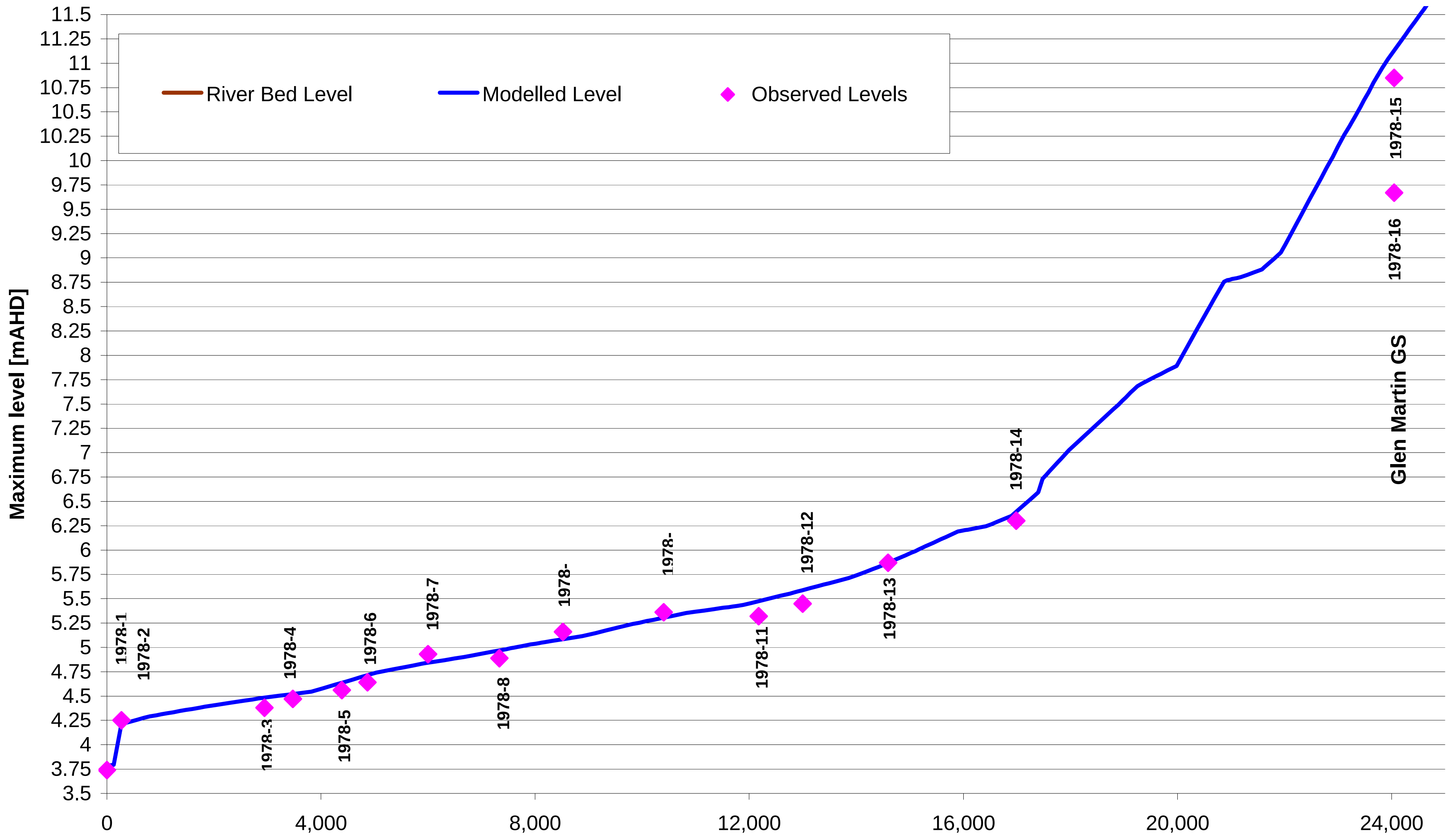
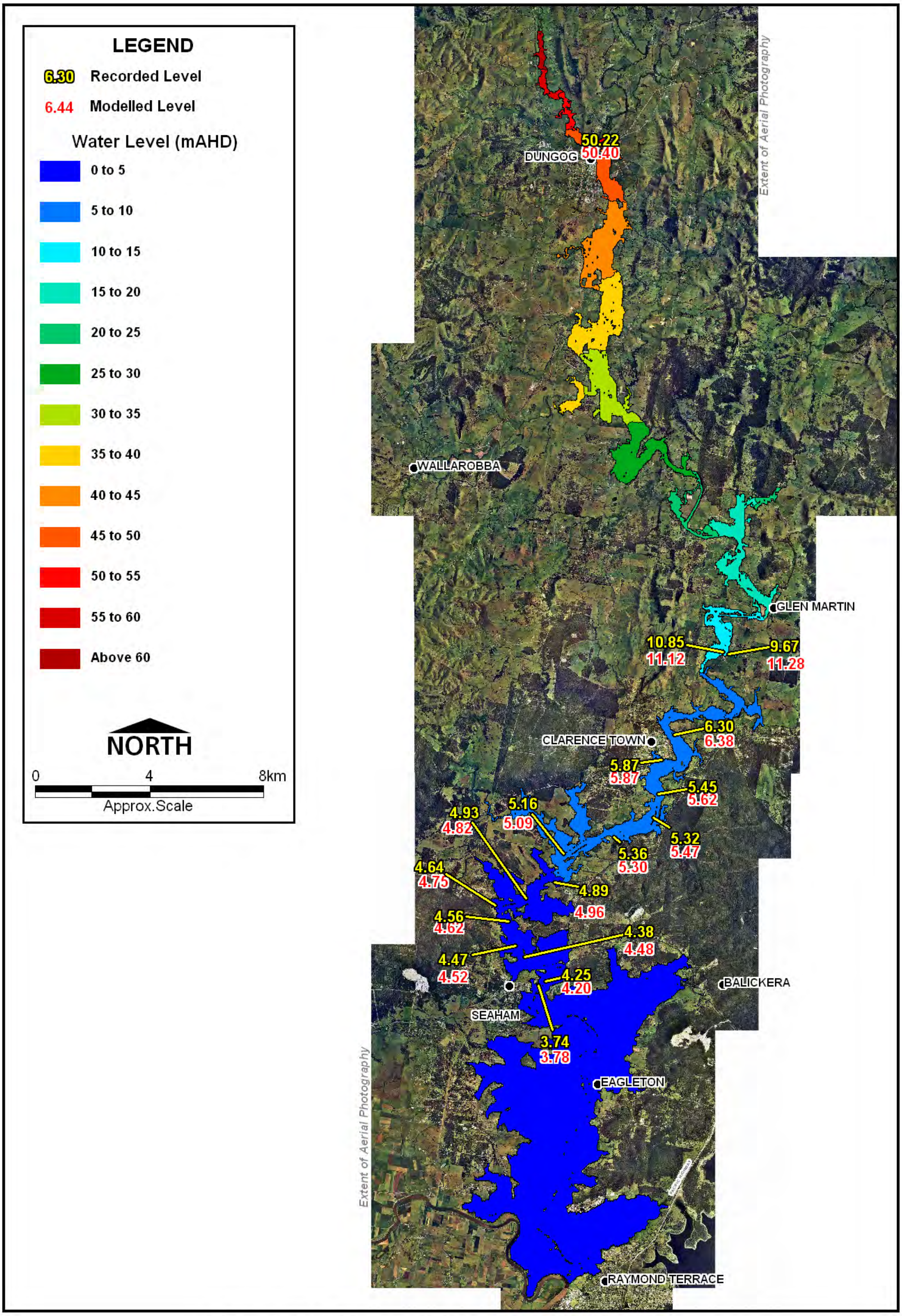


Figure 6-11 March 1978: Long Section Comparison for Hydraulic Model



K:\B16030.k.gjr_WilliamsR_FS\TUFLOW\results\Calibration_Results_run69_onwards\[long_section_WIL_max_WL_1978_run071.xls]LP_Seaham_Glen_Martin_Cha

Figure 6-12 March 1978: Long Section Comparison for Hydraulic Model – Seaham to Glen Martin



March 1978: Peak Levels Comparison for Hydraulic Model

Figure 6-13

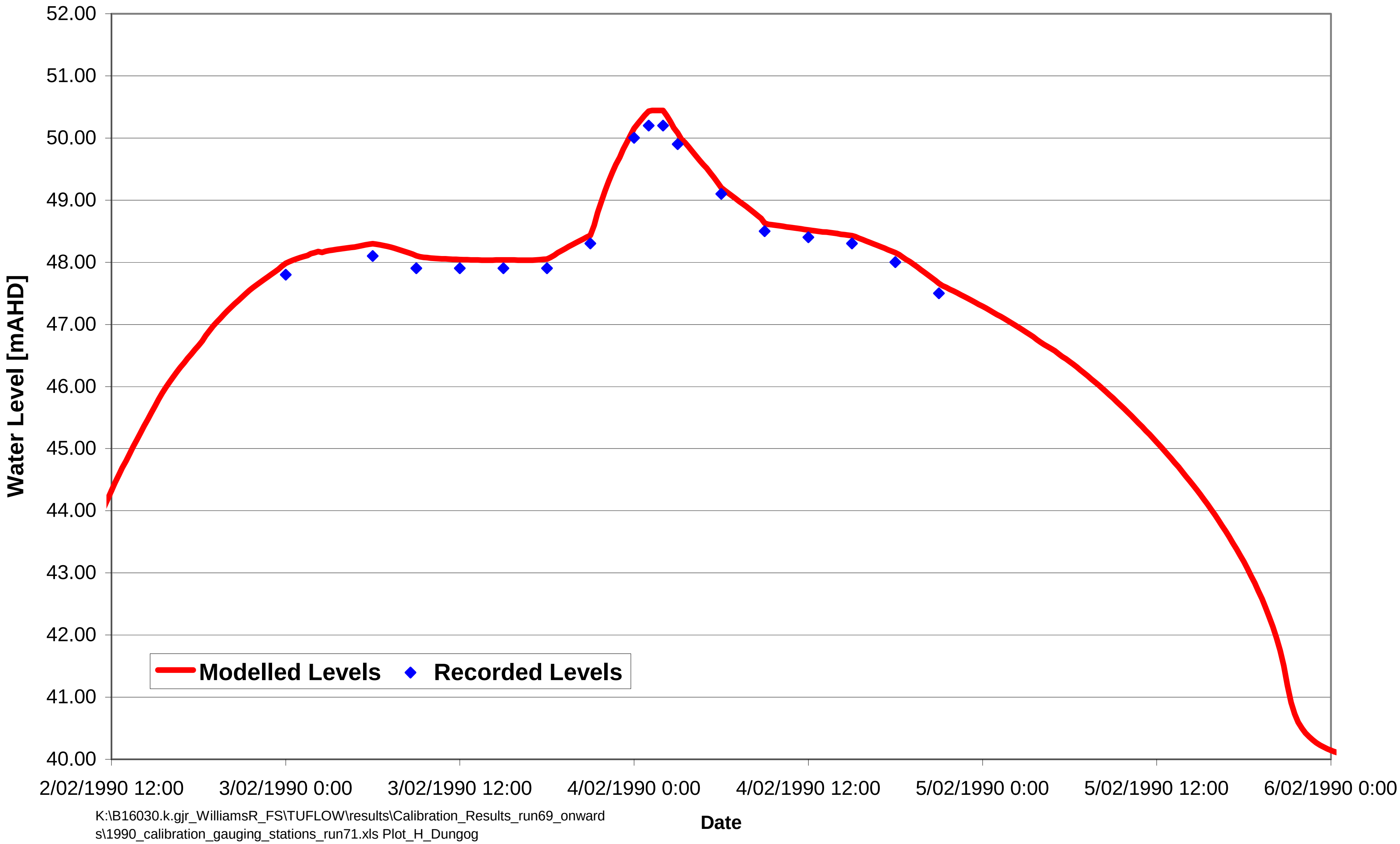


Figure 6-14 February 1990: Dungog Gauge Levels Comparison for Hydraulic Model

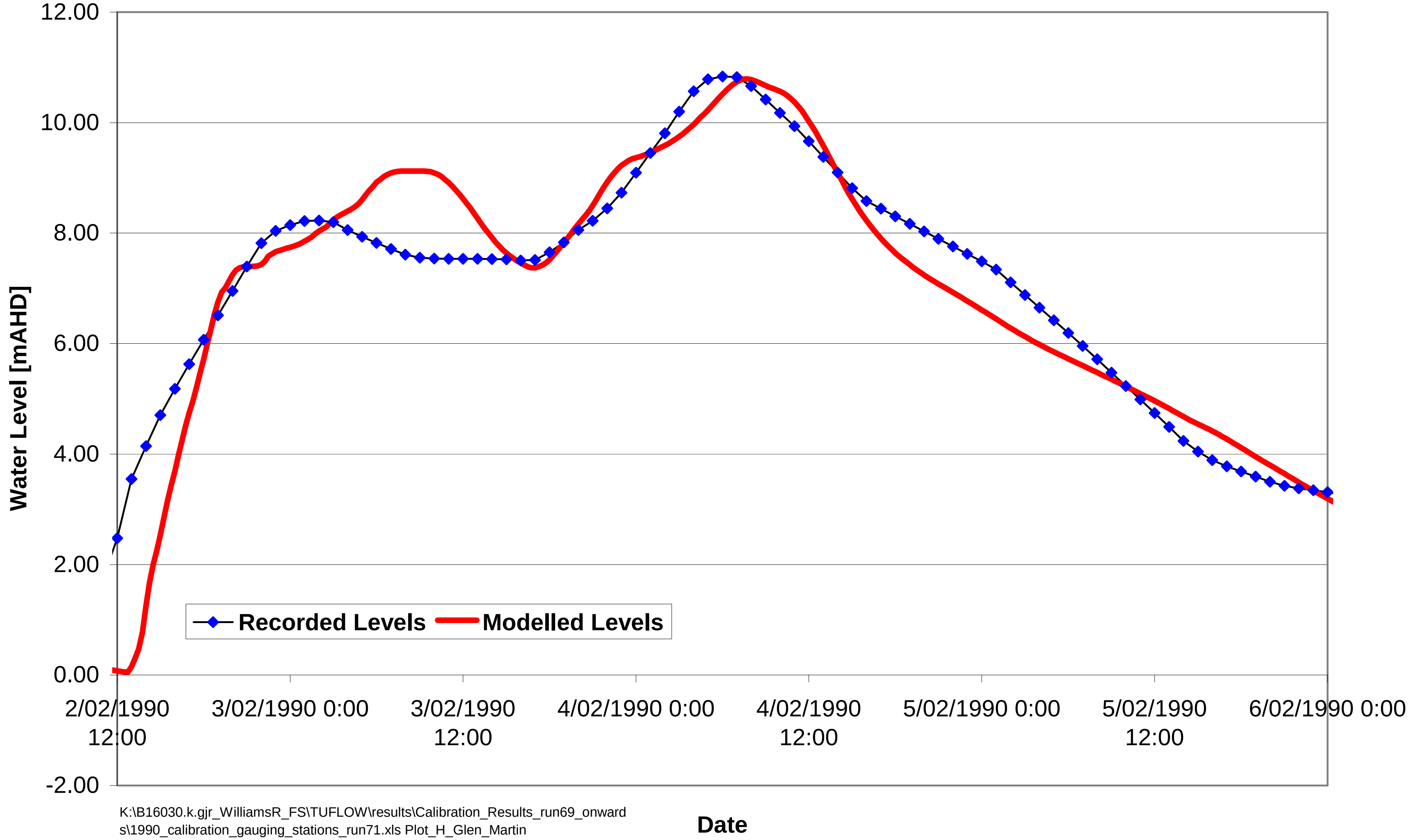


Figure 6-15 February 1990: Glen Martin Gauge Levels Comparison for Hydraulic Model

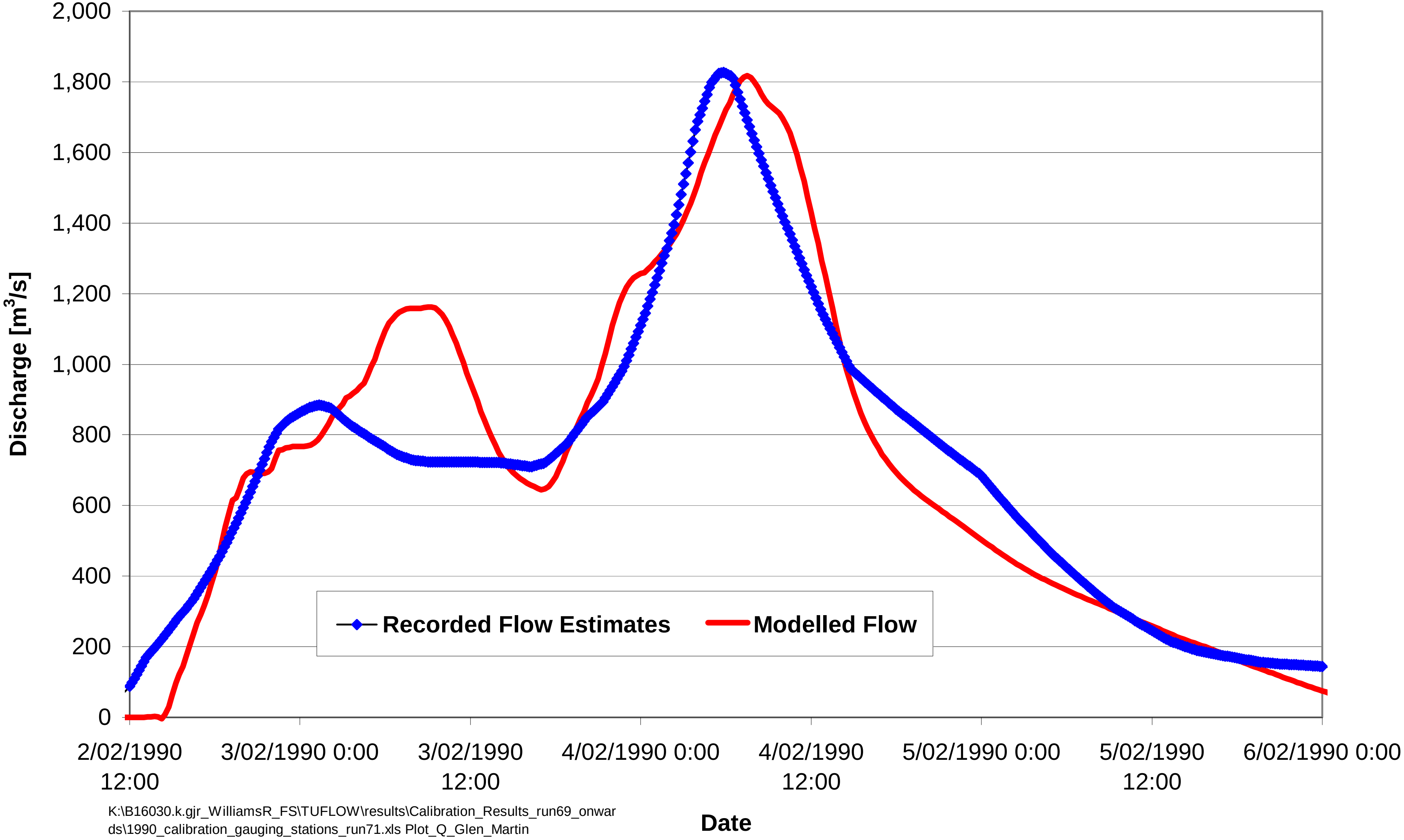


Figure 6-16 February 1990: Glen Martin Gauge Flows Comparison for Hydraulic Model

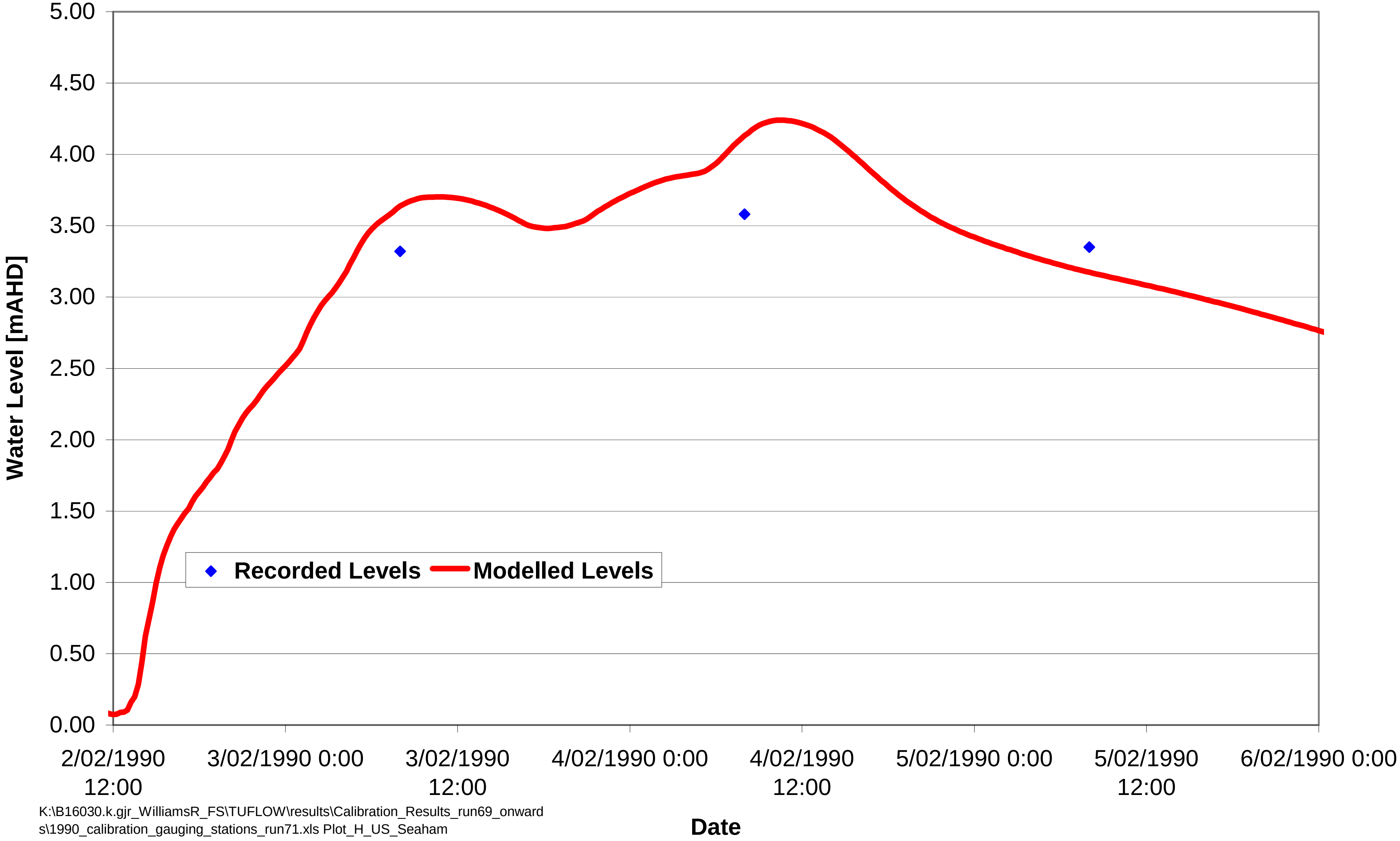


Figure 6-17 February 1990: Upstream Seaham Weir Levels Comparison for Hydraulic Model

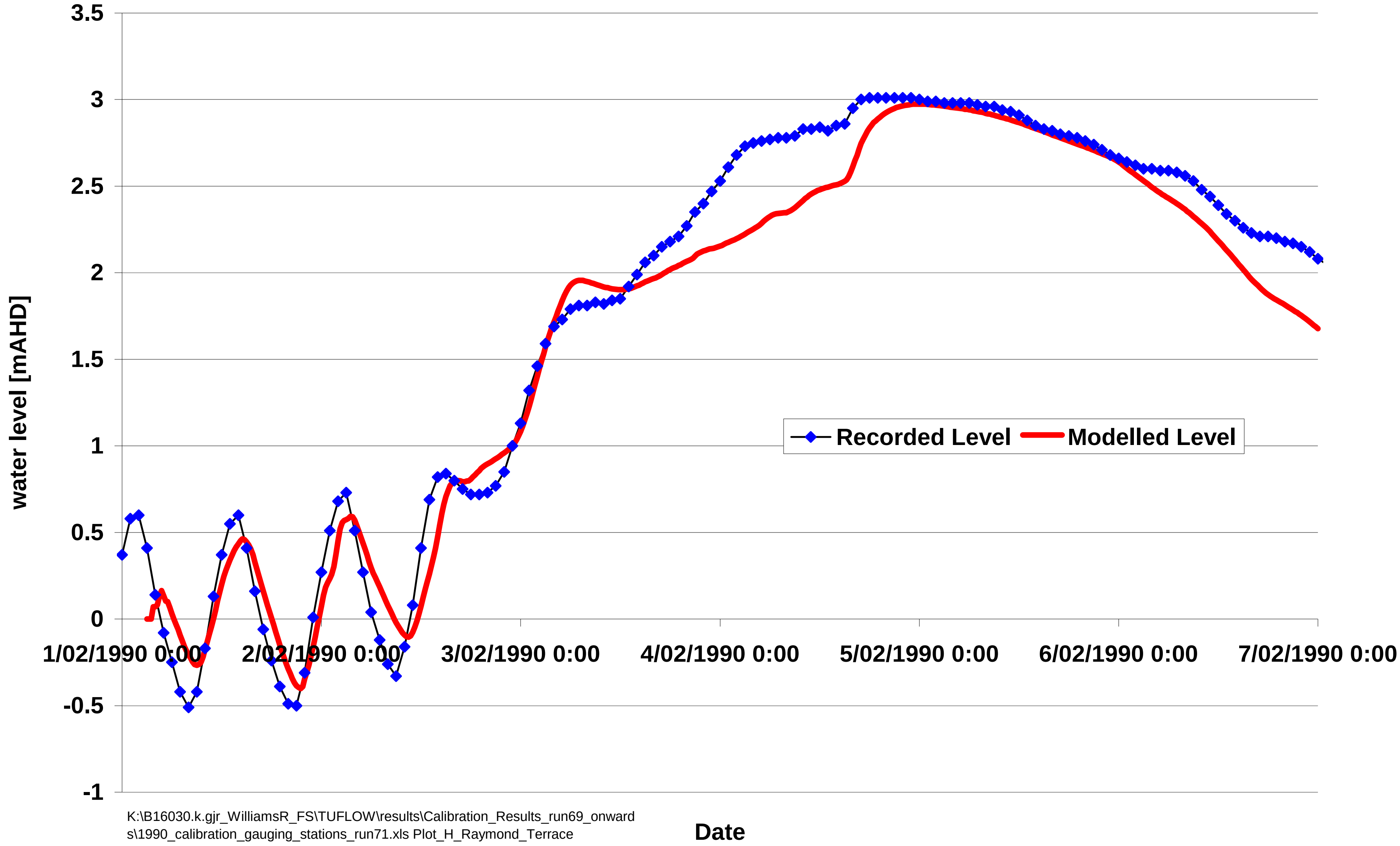


Figure 6-18 February 1990: Raymond Terrace Gauge Levels Comparison for Hydraulic Model

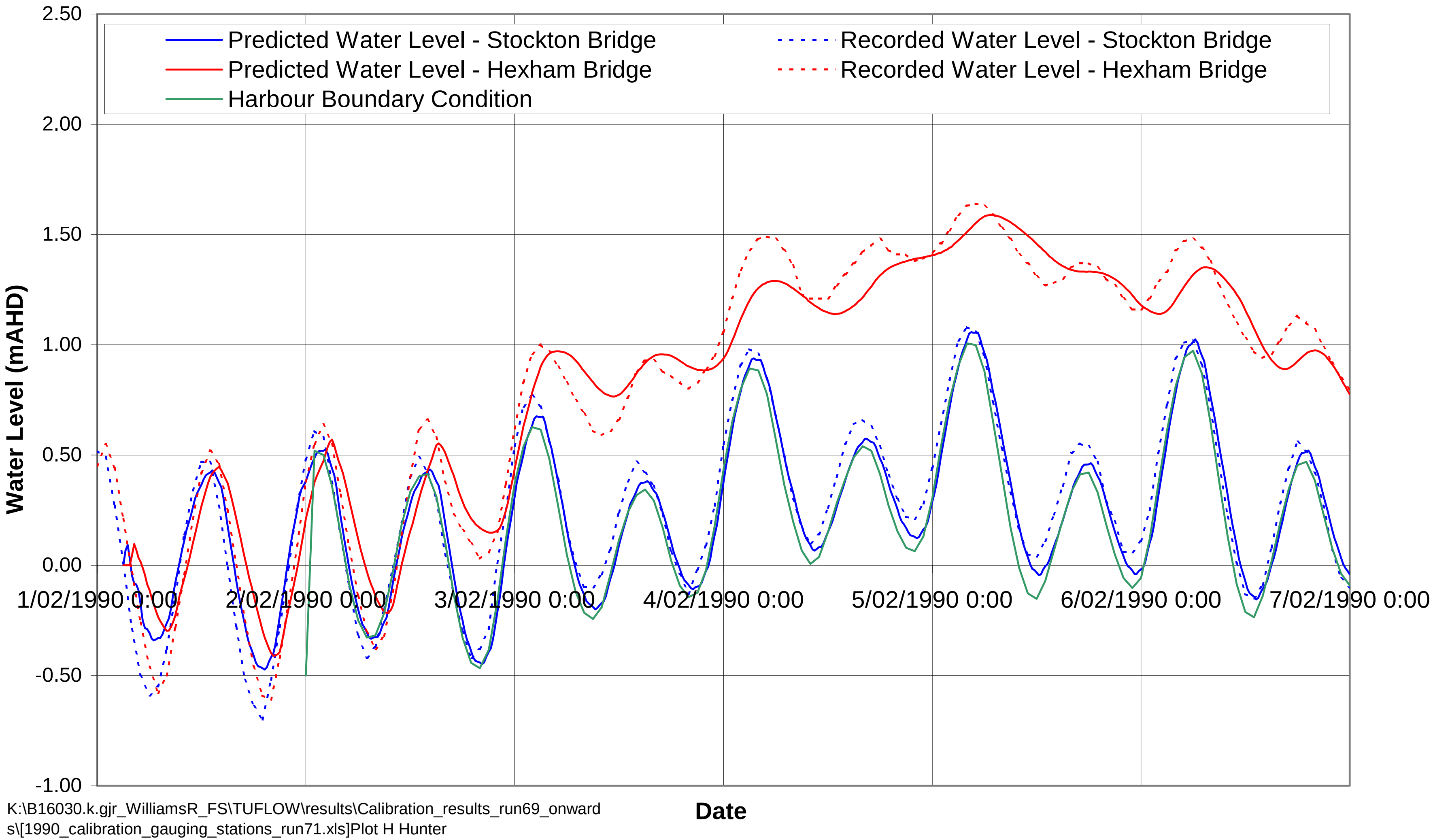


Figure 6-19 February 1990: Hunter River Gauge Levels Comparison for Hydraulic Model

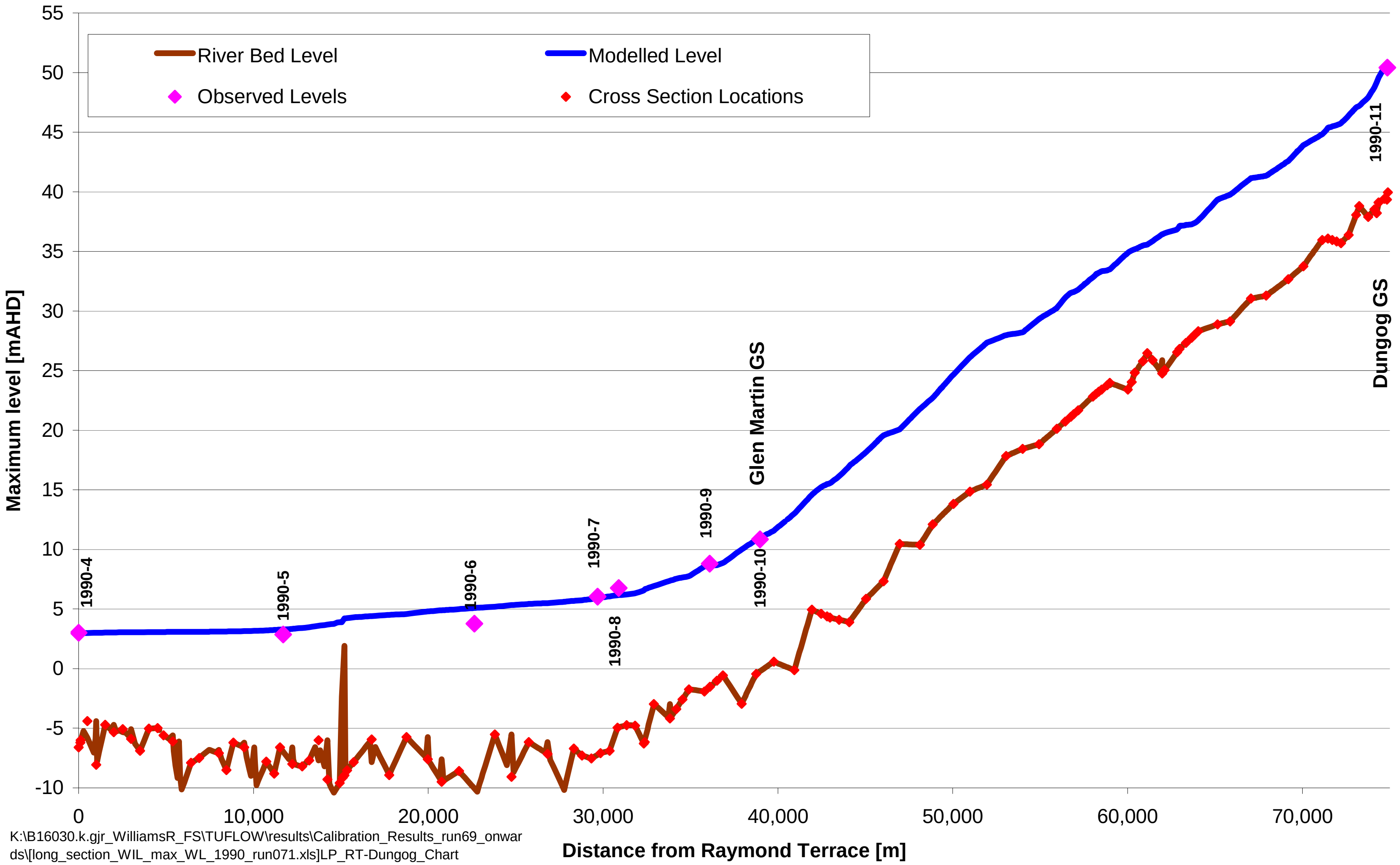


Figure 6-20 February 1990: Long Section Comparison for Hydraulic Model

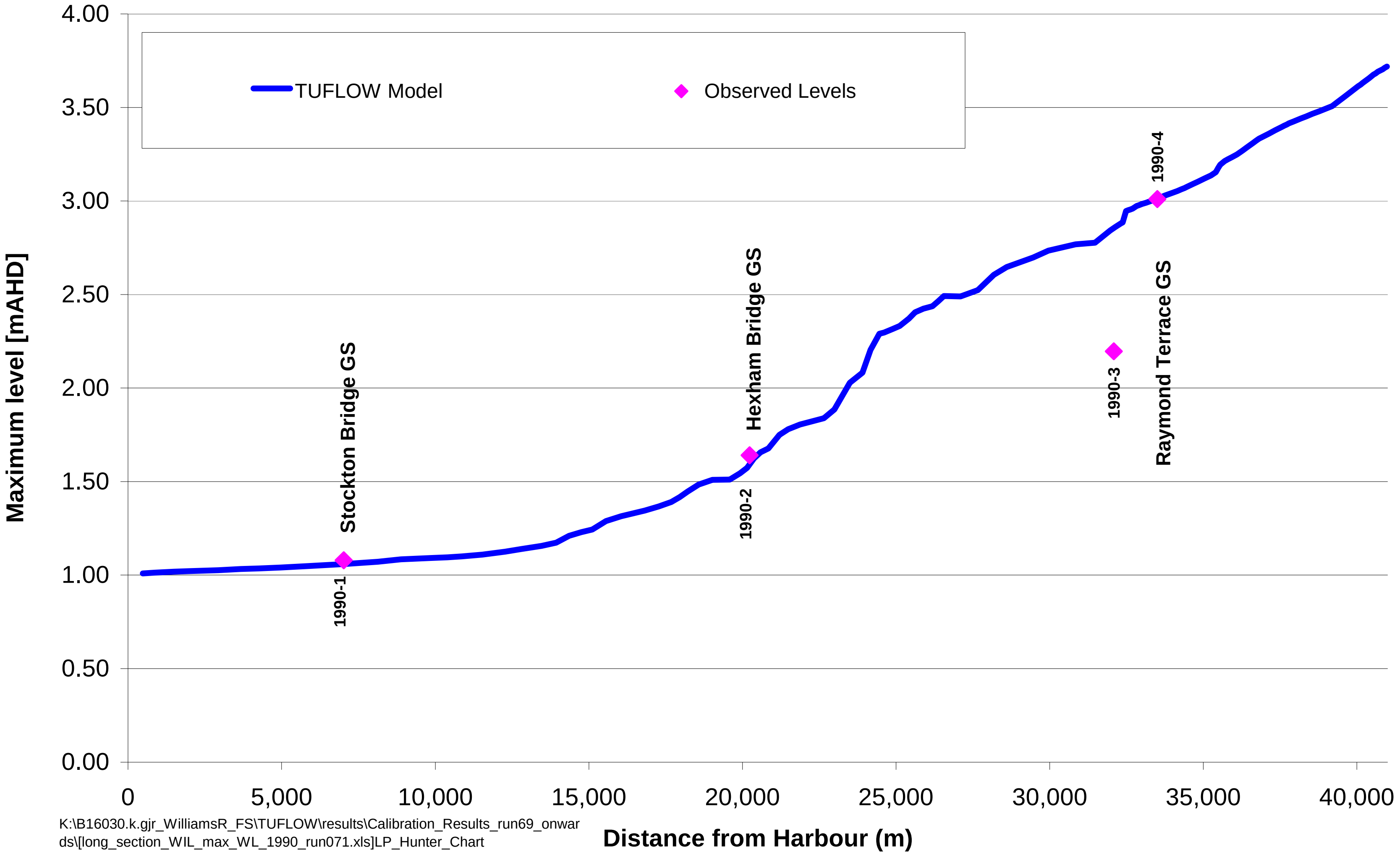
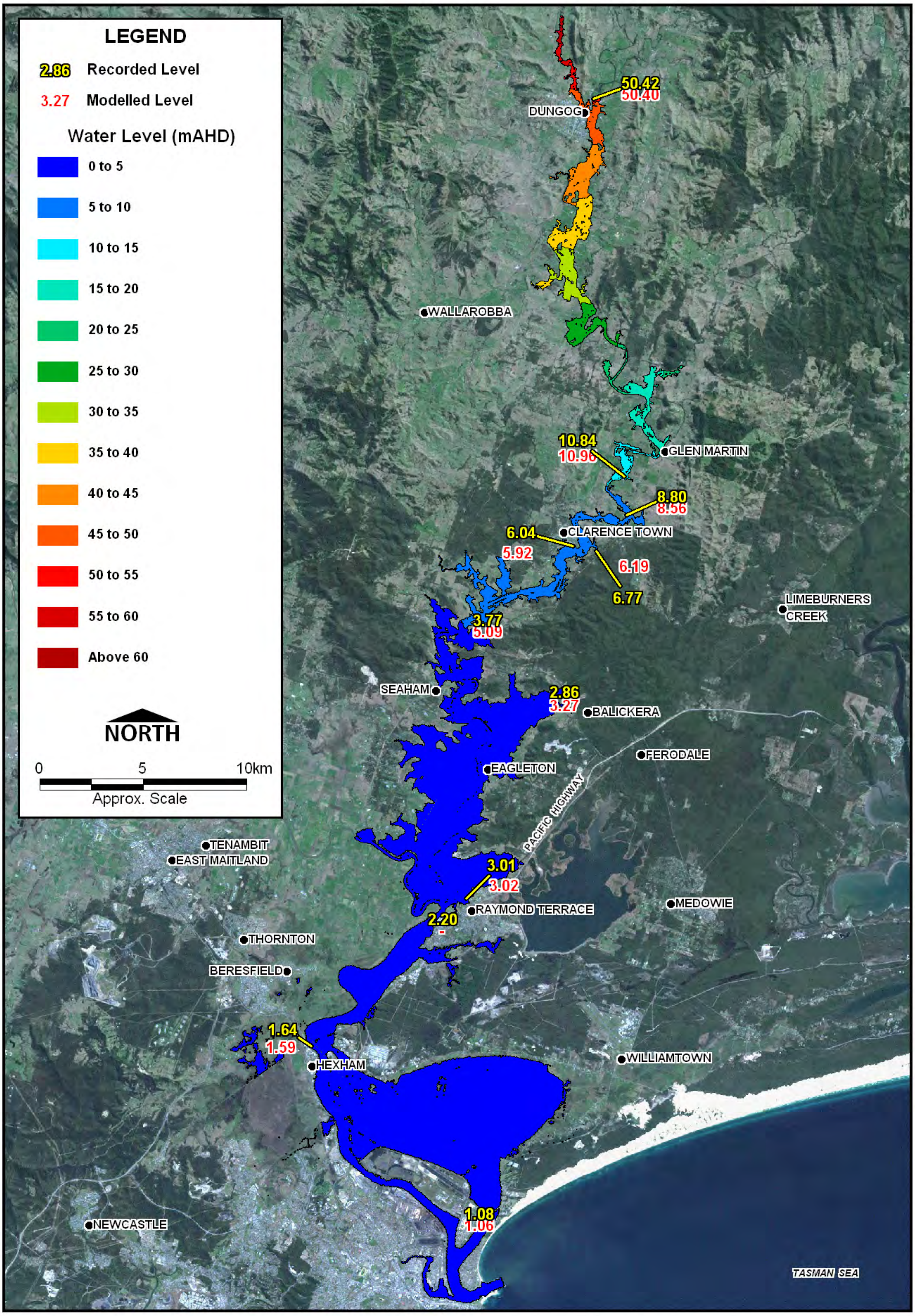


Figure 6-21 February 1990: Hunter River Long Section Comparison for Hydraulic Model



February 1990: Peak Levels Comparison for Hydraulic Model

Figure 6-22

I:\B16030_I_brh Williams River PEV\DRG\Calibration_Report\FLD_006_070503_2d_1990_Calibration.WOR

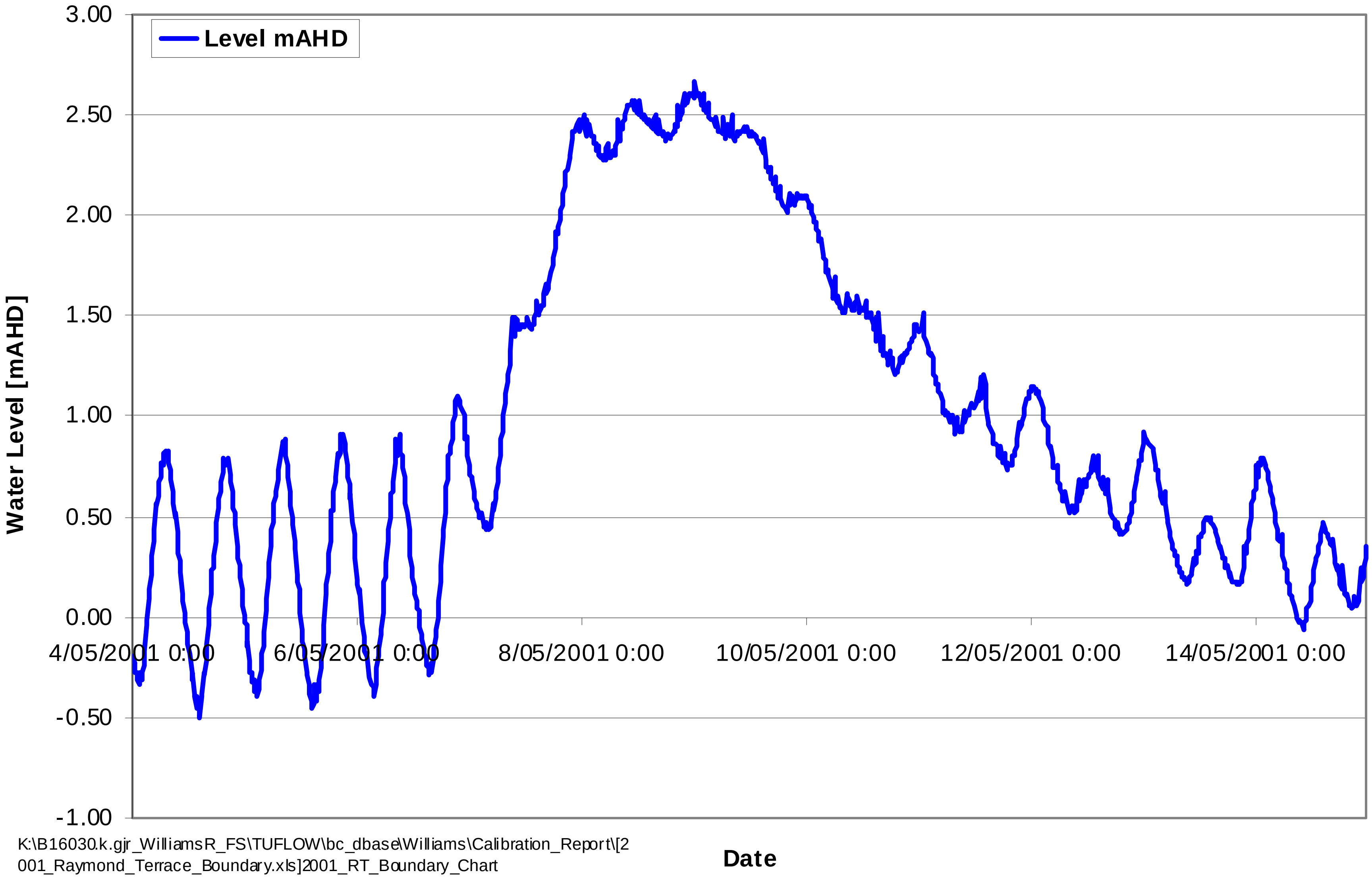


Figure 6-23 Raymond Terrace Recorded Levels as Boundary Condition for May 2001 Flood

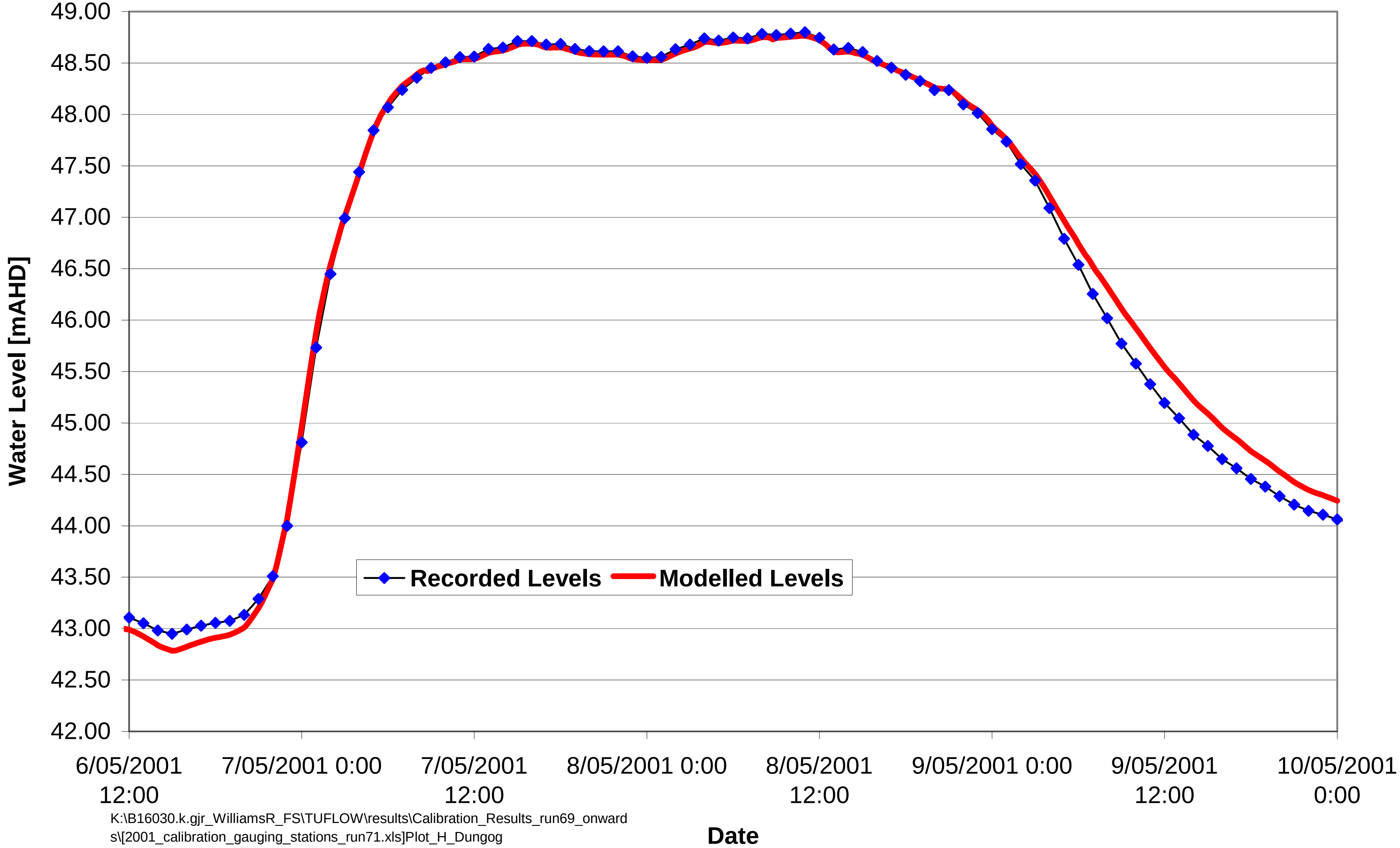


Figure 6-24 May 2001: Dungog Gauge Levels Comparison for Hydraulic Model

BMT WBM

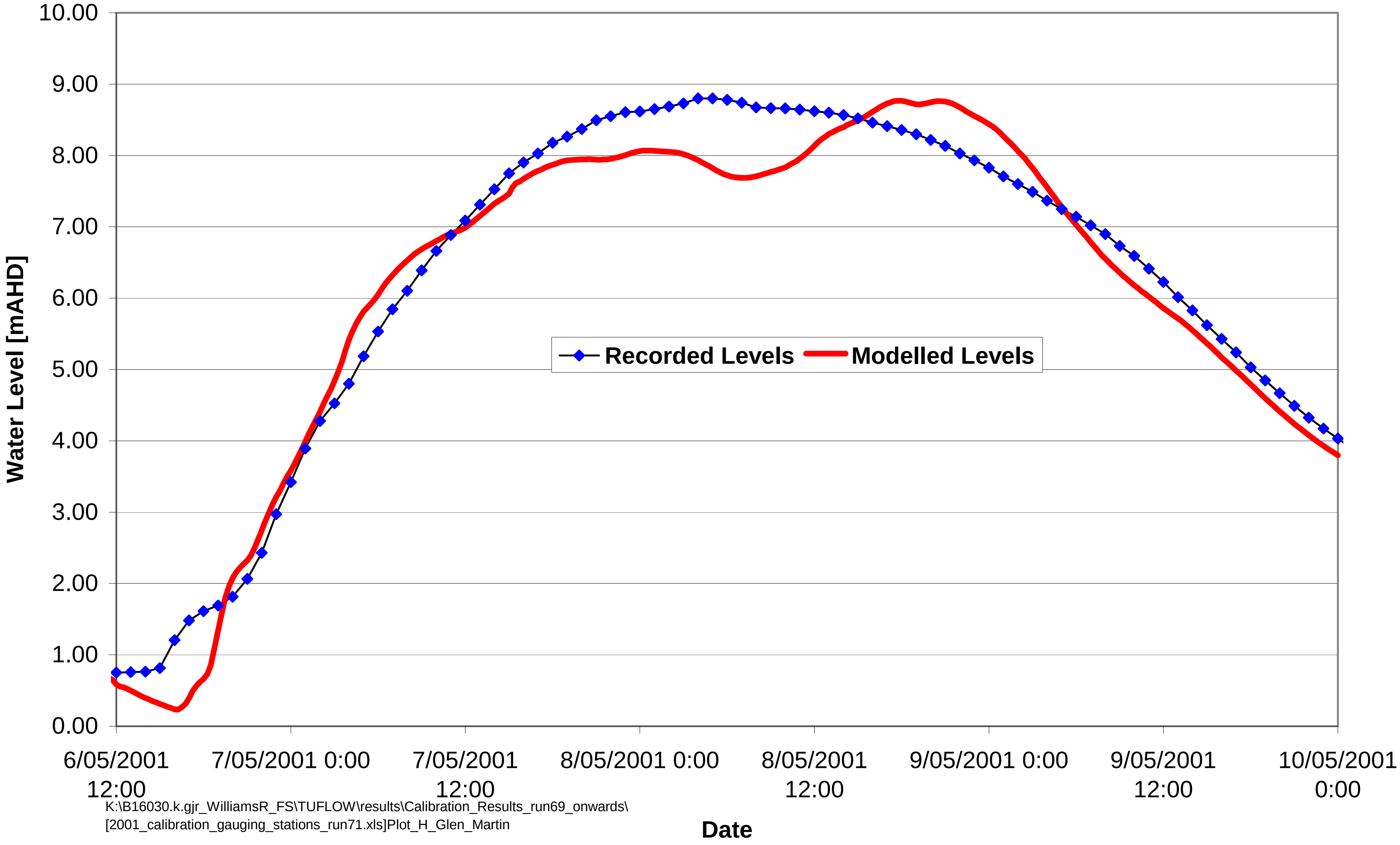


Figure 6-25 May 2001: Glen Martin Gauge Levels Comparison for Hydraulic Model

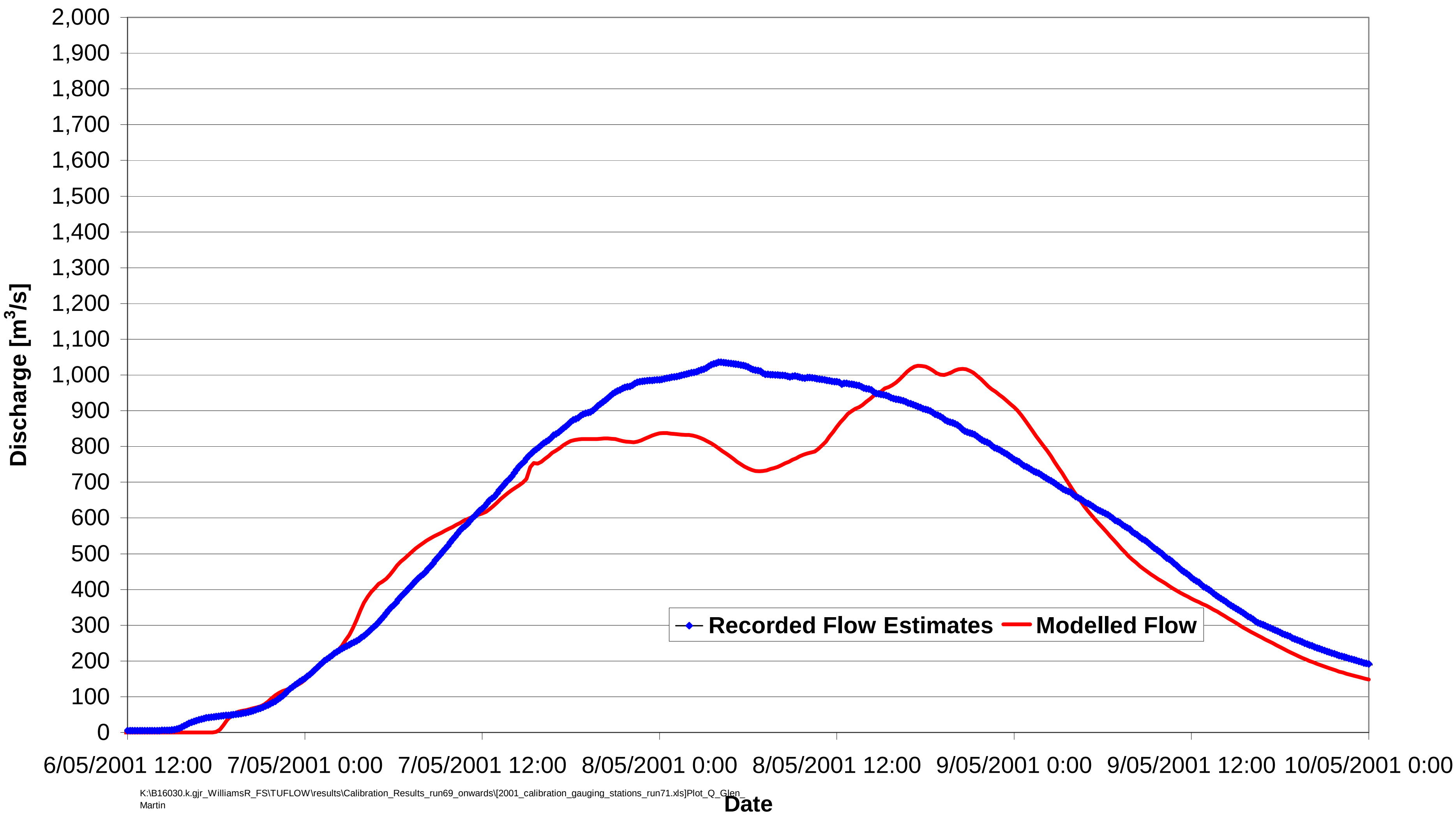


Figure 6-26 May 2001: Glen Martin Gauge Flows Comparison for Hydraulic Model

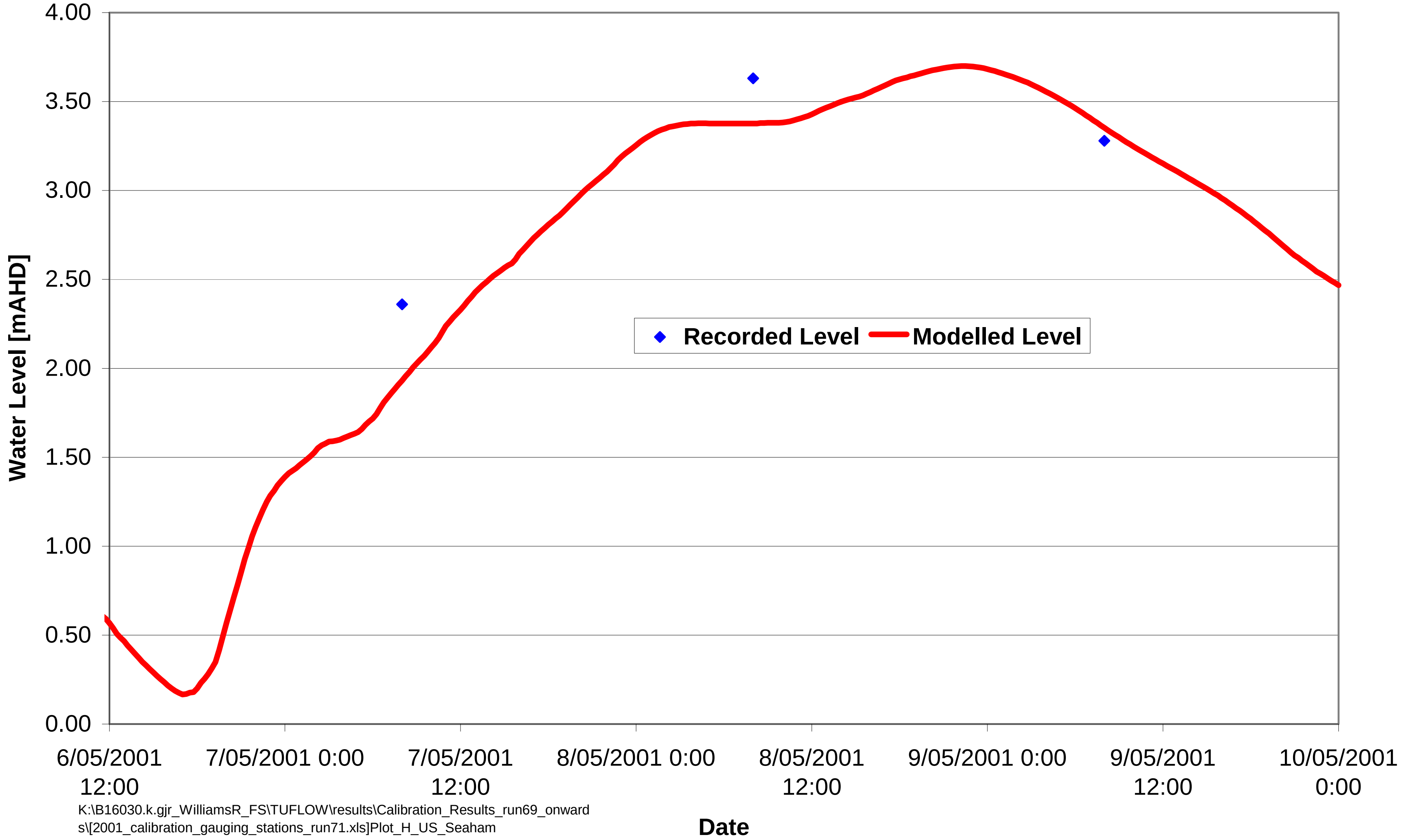


Figure 6-27 May 2001: Upstream Seaham Weir Levels Comparison for Hydraulic Model

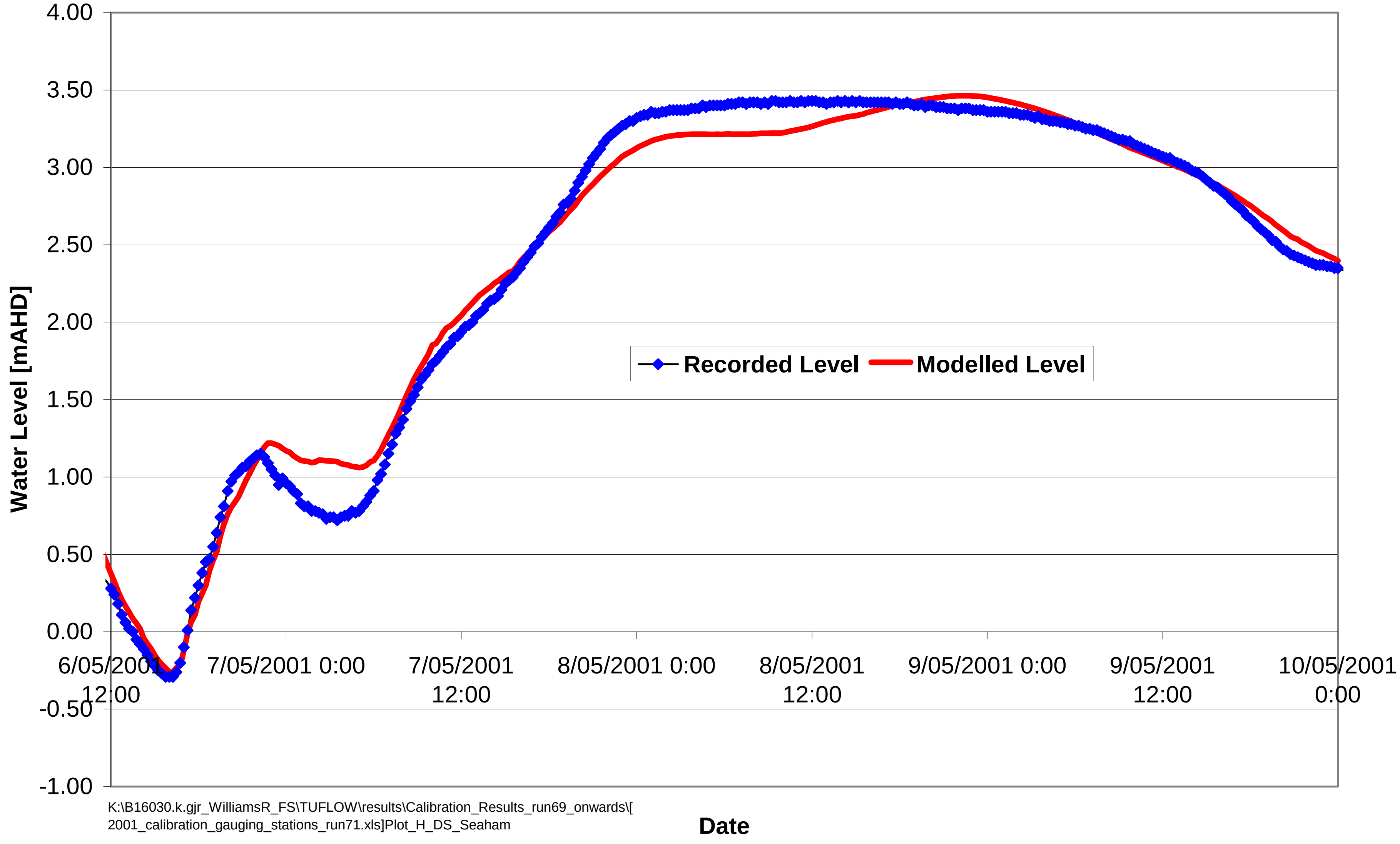


Figure 6-28 May 2001: Downstream Seaham Weir Gauge Levels Comparison for Hydraulic Model

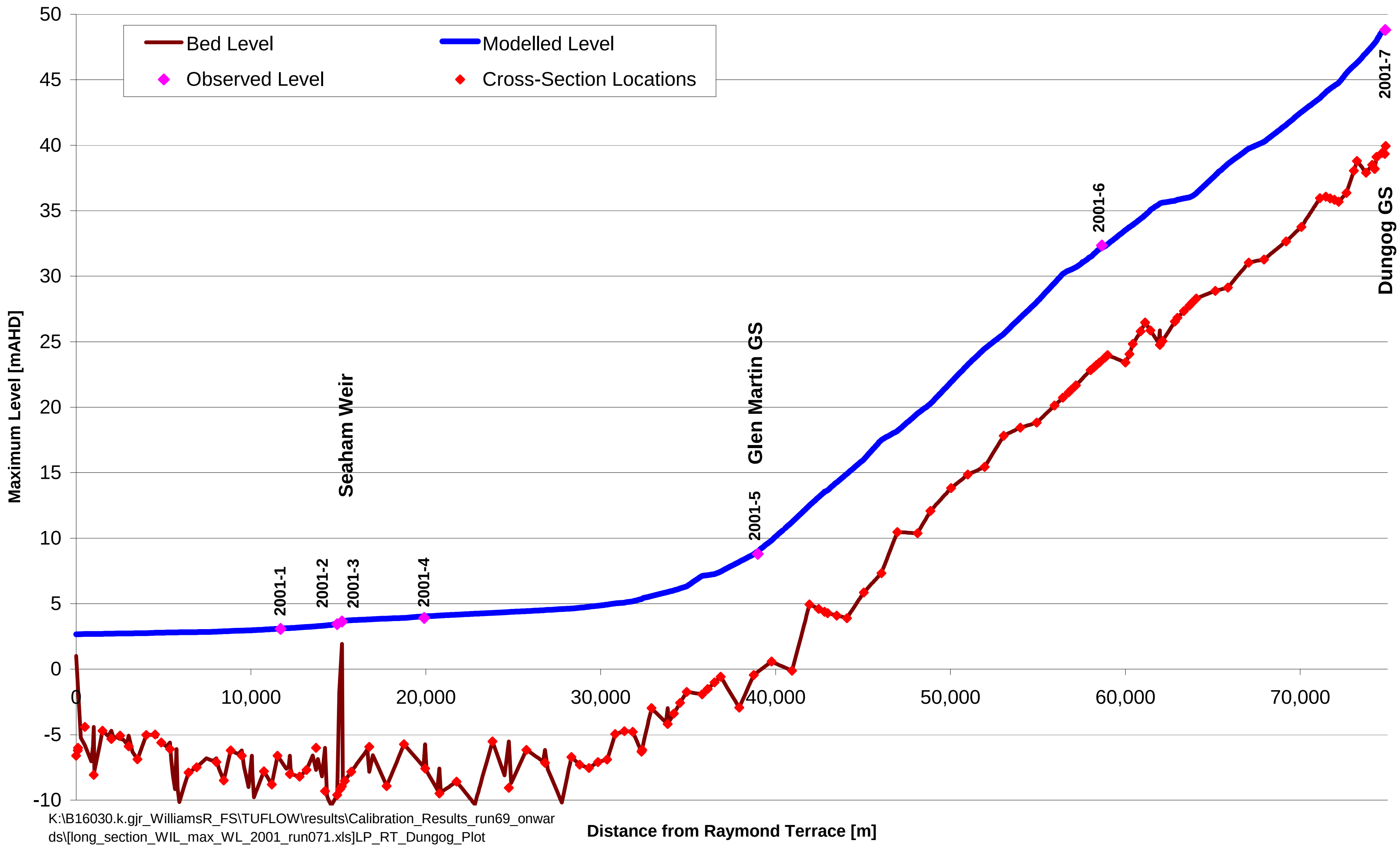
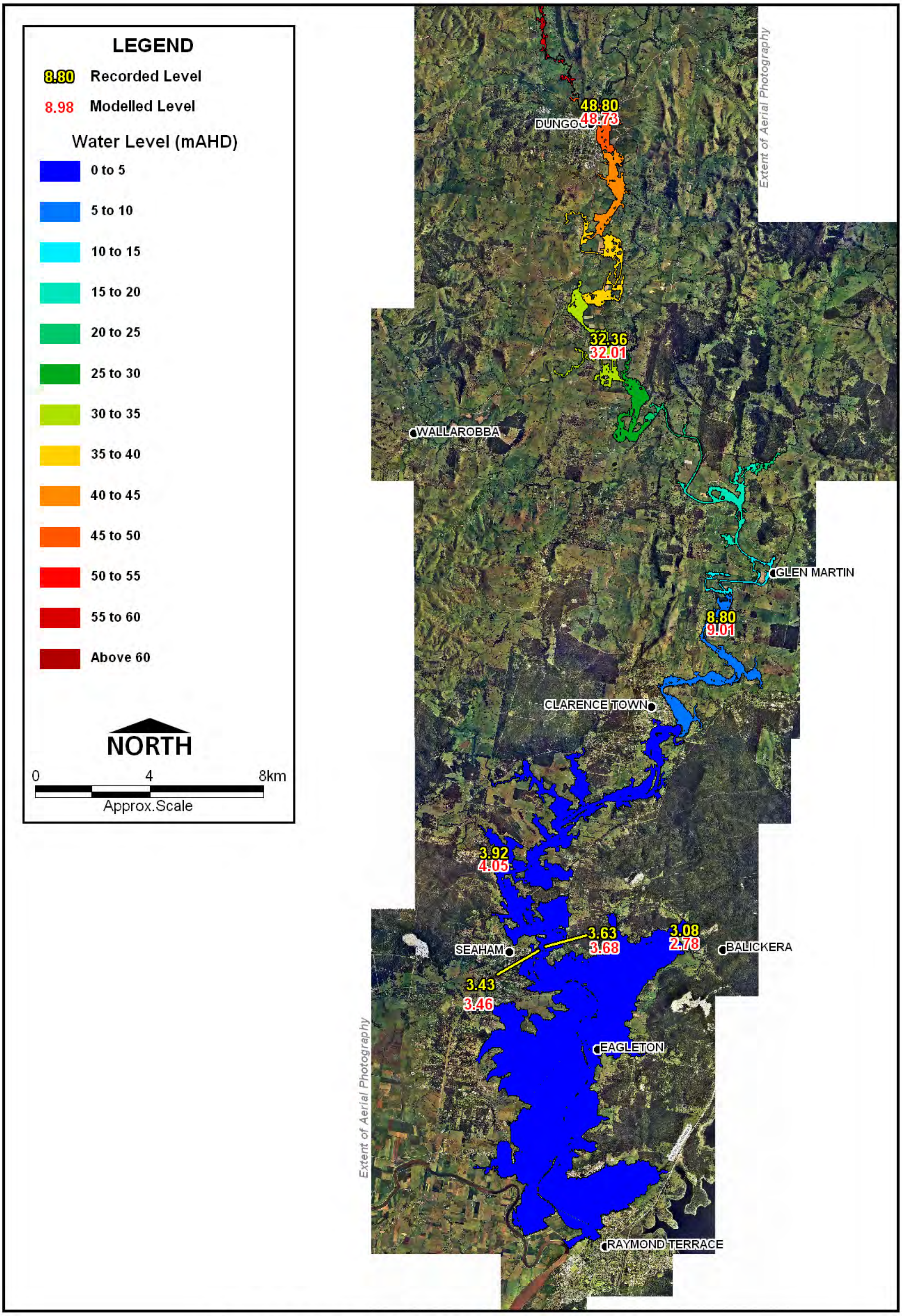


Figure 6-29 May 2001: Long Section Comparison for Hydraulic Model



May 2001: Peak Levels Comparison for Hydraulic Model

Figure 6-30

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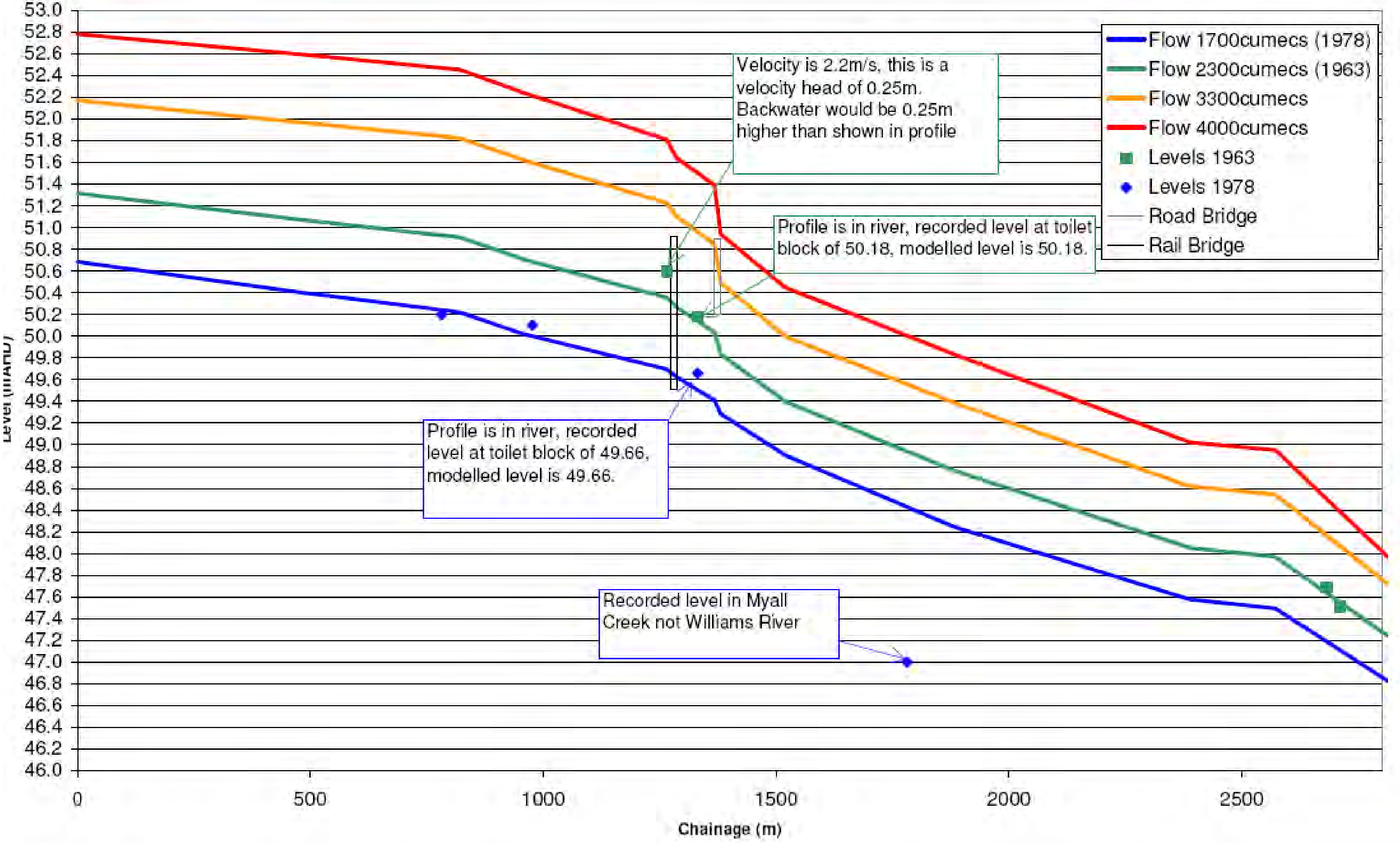

Figure 6-31 Hydraulic Model Calibration of 10m Grid Size Model at Dungog