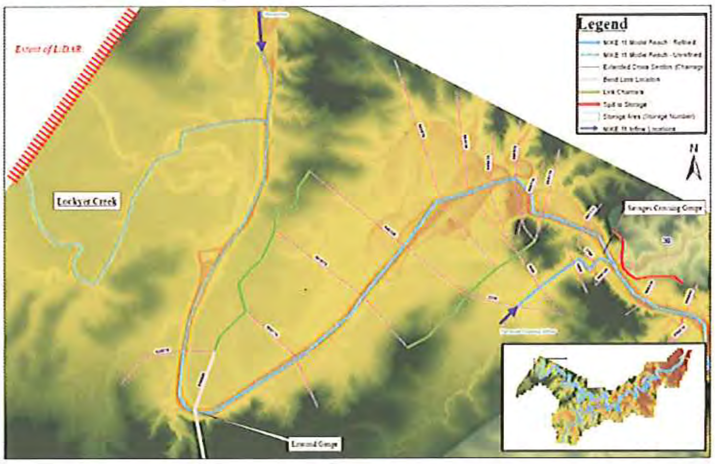
SINCLAIR KNIGHT MERZ
SKM

Joint Calibration of a Hydrologic and Hydrodynamic Model of the Lower Brisbane River



TECHNICAL REPORT

- Version 3
- 5 August 2011

Joint Calibration of a Hydrologic and Hydrodynamic Model of the Lower Brisbane River

TECHNICAL REPORT

- Version 3
- 5 August 2011

Sinclair Knight Merz
ABN 37 001 024 095
Floor 11, 452 Flinders Street
Melbourne VIC 3000
PO Box 312, Flinders Lane
Melbourne VIC 3000 Australia
Tel: +61 3 8668 3000
Fax: +61 3 8668 3001
Web: www.skmconsulting.com

Contents

1.	**Introduction**	**1**
2.	**Study Area and Data Availability**	**2**
2.1.	**Study Area**	**2**
2.2.	**Data Availability**	**3**
3.	**Hydrologic Modelling**	**6**
3.1.	**WT42 model**	**6**
3.2.	**URBS model**	**8**
3.3.	**January 2011 Rainfall Inputs**	**10**
3.3.1.	Method for deriving sub-area rainfalls	10
3.3.2.	Review of rainfall data	10
3.3.3.	Rainfall gauge locations	11
3.3.4.	Sub-area rainfalls	12
3.4.	**Initial Calibration of hydrologic models to January 2011 Event**	**14**
4.	**Hydrodynamic Model Development**	**19**
4.1.	**Past Hydrodynamic Models of the Lower Brisbane River**	**19**
4.2.	**Review of 2005 MIKE 11 Model**	**21**
4.3.	**Revised Hydrodynamic Model**	**22**
4.3.1.	Terrain Data	22
4.3.2.	Model Schematisation	23
4.3.3.	Link Channels	27
4.3.4.	Storage Areas	27
4.3.5.	Bend Losses	27
4.3.6.	Channel Geometry	28
4.3.7.	Roughness	29
4.3.8.	Fluvial Structures	32
4.3.9.	Boundary Conditions	32
4.3.10.	Model Setup and Parameters	33
5.	**Rating Curves at key Brisbane River gauges**	**43**
5.1.	**Available Data**	**43**
5.2.	**MIKE 11 Model Results**	**43**
5.3.	**Updated Rating Curves**	**44**
5.4.	**Implications**	**48**
6.	**Calibration of Hydrodynamic Model to January 2011 Event**	**50**
6.1.	**January 2011 Inflows**	**50**
6.2.	**Calibration methodology**	**52**
6.3.	**Comparison of modelled and recorded levels at key sites**	**53**

7. Scenario Modelling 57
8. Summary and Conclusions 60
8.1. Uncertainty and Limitations 60
8.2. Conclusions 61
8.3. Recommendations 63
9. References 64
Appendix A Summary of Hydrologic Data 65
A.1 Rainfall gauges investigated 66
A.2 Sub-area rainfalls 72
Appendix B Review of 2005 Model 76
B.1 Model Schematisation 76
B.2 River Channel Cross Sections 76
B.3 Link Channels 78
B.4 Hydraulic Structures 78
B.4.1 Bridges 78
B.4.2 Stream Flow Gauging 104
B.5 Representation of Roughness 104
B.6 Model Inputs and Boundaries 107
B.7 Model Setup 107

Document history and status

Revision	Date issued	Reviewed by	Approved by	Date approved	Revision type
Draft 0	22/06/2011	Peter Hill	Peter Hill	22/06/2011	Preliminary draft for discussion
Draft A	22/06/2011	Rory Nathan	Pat Nixon	22/06/2011	Draft
Version 1	24/06/2011	Rory Nathan	Pat Nixon	24/6/2011	Version 1
Version 2	07/06/2011	David Stephens	Pat Nixon	07/06/2011	Version 2
Version 3	03/08/2011	Peter Hill Rory Nathan	Pat Nixon	05/08/2011	Version 3

Distribution of copies

Revision	Copy no	Quantity	Issued to
Draft 0		1 via email	Terry Malone, John Tilbaldi
Draft A		1 via email	Terry Malone, John Tilbaldi
Version 1		1 pdf via email	Terry Malone, John Tibaldi
Version 2		1 pdf via email	Terry Malone, John Tibaldi
Version 3		1 pdf via email	Terry Malone, John Tibaldi

Printed:	5 August 2011
Last saved:	5 August 2011 09:50 AM
File name:	I:\QENV\Projects\QE09901\Technical\AdditionalModelling\Deliverables\Brisbane River Hydrodynamic Modelling_Vers3_WithTrackChanges.docx
Project manager:	Peter Hill
Report Author:	Peter Hill, Kristen Sih, David Stephens, Adam Cambridge
Name of organisation:	Seqwater
Name of project:	Brisbane River Hydrodynamic Modelling
Name of document:	Technical Report
Document version:	Version 3
Project number:	QE09901

1. Introduction

Following the significant flood in the Brisbane River in January 2011, SKM were appointed by South East Queensland Water (Seqwater) to jointly calibrate hydrologic and hydrodynamic models of the Lower Brisbane River. These models were developed to gain further understanding of the 2011 flood and allow a range of different operating scenarios to be assessed for Wivenhoe Dam.

Although there are a number of tools and approaches available for flood estimation, at the request of Seqwater this study has made use of the WT42 and URBS hydrological models and the 2005 version MIKE 11 model that have been developed in other studies. The URBS hydrologic model was developed by Seqwater and Terry Malone provided assistance during the course of the study in reviewing the available data and calibrating the model.

The following sections of the report cover:

- A summary of the Lower Brisbane River and the available hydrologic data
- Comparison of the WT42 and URBS hydrologic models
- The review of existing hydrodynamic models of the Lower Brisbane River and the development of an enhanced model
- Derivation of rating curves at key Brisbane River gauges
- Calibration of the hydrodynamic model to river levels recorded during the January 2011 Flood
- Application of the hydrodynamic model to assess alternate scenarios related to the prescence and operation of Wivenhoe Dam
- Summary and conclusions from the study

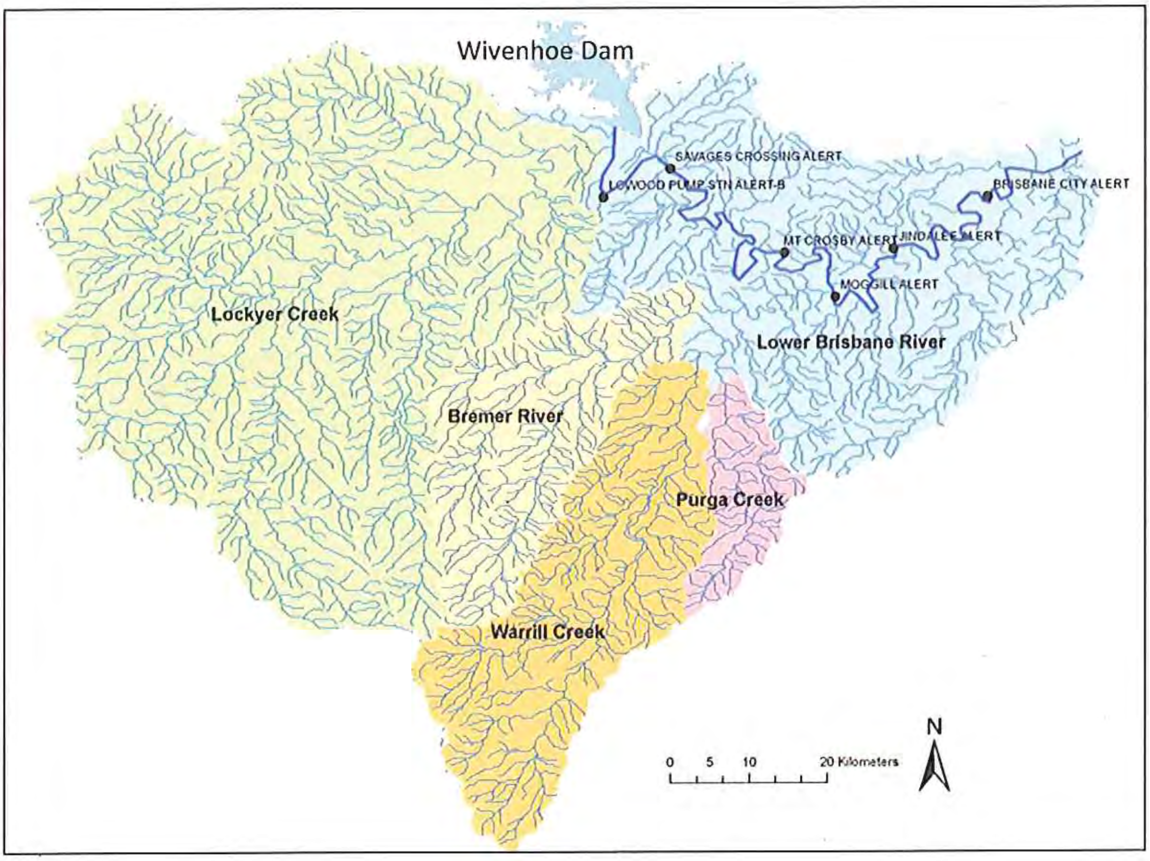
2. Study Area and Data Availability

2.1. Study Area

The focus of this study is on the Lower Brisbane River catchment, and in particular the portion that extends from Wivenhoe Dam to the mouth of the Brisbane River gauge, as shown in Figure 2-1. This study area was chosen due to the focus of the Commission of Inquiry on the operation of Wivenhoe Dam.

The hydrologic models (described in Section 3) extend across the whole of the Brisbane River catchment, but as Wivenhoe Dam controls the upstream catchment, the upper catchment has not been considered in the model calibration.

The hydrodynamic model (described in Section 4) has been based on a model developed by various consultants and used for various purposes (see Section 4.1). New terrain data has been made available, and this has been used to review and update the model schematisation for the Brisbane River. It should be noted that the Lockyer Creek and Bremer River reaches of the model have not been reviewed. If future work is undertaken using the model developed as part of this study then the remainder of the study extent should be examined and improved as appropriate.



- **Figure 2-1: Lower Brisbane River catchment study area that extends from Wivenhoe Dam to the mouth of the Brisbane River.**

2.2. Data Availability

There is an extensive network of rainfall and water level gauges in the Brisbane River basin that are owned and operated by the Bureau of Meteorology (BoM), Seqwater, the Queensland Department of Environment and Resource Management (DERM) and local councils including Ipswich and Brisbane City Councils. The majority of these gauges report data in real time either via radio (ALERT) or by telephone. There is a significant duplication in the reporting of rainfall and water level information in the network, with many ALERT water level gauges reporting data from DERM gauges. Further information on the hydrometric network is available in the Event Flood Report (Seqwater, 2011).

The rainfall gauges that were operational during the January 2011 event are described in Section 3.3, and the river gauges are shown in Figure 2-2. Although there are a large number of water level gauges in the basin, many are not DERM stations and therefore do not have an "official" rating to convert the recorded levels to estimated flow. This is the case for many of the ALERT water level gauges (shown as black dots in Figure 2-2), although unofficial ratings have been developed for many of these sites by interested agencies. There are at least 16 DERM river gauges in the Brisbane River below Wivenhoe Dam, and key stations from which data was collected for the January 2011 event are marked in red in Figure 2-2. Even some of these gauges did not provide reliable flow information for the January 2011 event for the following reasons:

- 143203c Lockyer Ck at Helidon – Failed on 10th January 2011 before recording the peak.
- 143210B Lockyer Ck at Rifle Range Rd – Large flows are understood to by-pass the gauge and the existing rating only reflects the in-channel flow.
- 143207A Lockyer Ck at O'Reillys Weir – This gauge is backwater affected by flow in the Brisbane River and cannot be reliably rated.

There is another gauge on Lockyer Creek at Lyons Bridge that is located at an old DERM site and therefore has some historical rating information available, but this rating only reflects the in-channel flow, and so was not useful during the January 2011 event.

A summary of the proportion of the catchments that have rated gauges that could be used to estimate flows in the January 2011 event is shown as shaded areas in Figure 2-2. This shows that across the whole of the Brisbane River catchment below Wivenhoe Dam, it is possible to estimate flow from about 40% of the area using gauges. In particular, only 30% of the Lockyer Creek catchment is gauged, which has caused the estimation of total outflow from the Lockyer Creek to be problematic. This highlights the importance of using a hydrologic model to estimate flows in the catchment.

It also needs to be recognised that in many cases, the rating at the DERM sites are based upon relatively small flows. This means that the relationship between river level and flow magnitude has been extrapolated for higher flows, resulting in there being less confidence in the flow estimates for higher stages. Appendix R of the *January 2011 Flood Event* report (Seqwater, 2011a) contains comments on the reliability of ratings at most stations. Section 5 provides an analysis of key river gauges along the Brisbane River using the hydrodynamic model, in order to provide greater rigour in estimates of higher flows.

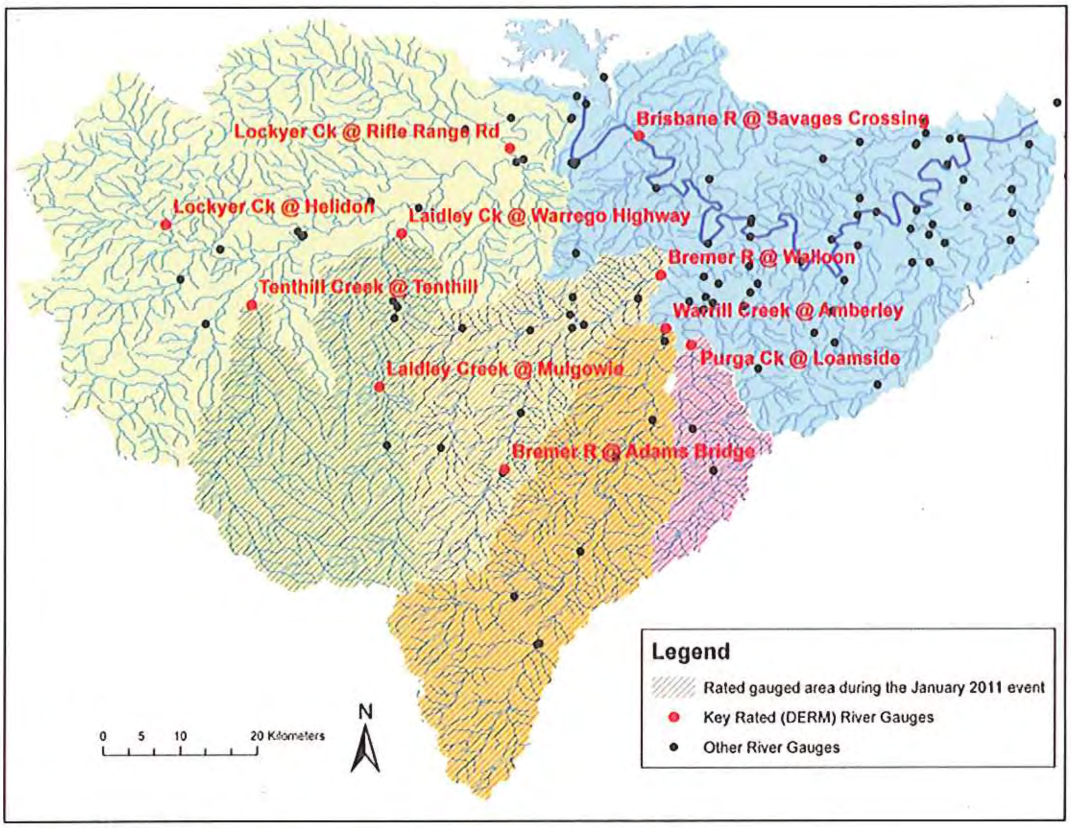


Figure 2-2: River gauges in the lower Brisbane River catchment.

3. Hydrologic Modelling

3.1. WT42 model

The latest version of the WT42 model was first developed in 1992 to assist in the operation of Somerset Dam and Wivenhoe Dam. The main aim of that project was to refine existing models so that they could then be used in a flood management model. However, the model was also used to revise design floods for the storages and undertake dambreak flood modelling downstream of the storages. The model layout is shown in Figure 3-1, with each of the sub-catchments shown having separate model parameters specified for them.

The WT42 model was calibrated to seven historical events (July 1965, March 1967, June 1967, December 1971, January 1975, January 1976 and June 1983) at up to 21 river gauge locations (South East Queensland Water Board and Natural Resources Queensland, 1992). The gauges in the lowest part of the Brisbane River, such as Ipswich, Moggill, Jindalee and Port Office, only had reliable records for 3 or less of these historical events. The model parameters chosen from the calibration were then verified to three more historical events (January 1968, April 1989a and April 1989b).

The key model parameters are the routing parameters, *k* and *m*, and initial and continuing losses. Consistent with the guidance from Australian Rainfall and Runoff (1999), the *m* parameter was held constant at 0.8, and a summary of the adopted *k* values adopted in design runs are shown in Table 3-1.

- **Table 3-1: WT42 model parameters from South East Queensland Water Board and Natural Resources Queensland (1993).**

Sub-Catchment	Name	Area (km^2)	*k*
Cooyar Ck at Damsite	TEN	980	43.6
Brisbane R at Linville	LIN	1,061	20.6
Emu Ck at Boat Mountain	EMU	913	53.0
Brisbane R at Gregors Ck	GRE	973	37.2
Cressbrook Ck at Cressbrook Dam	CRE	317	34.3
Stanley R at Somerset Dam	SOM	1,328	80.7
Brisbane R at Wivenhoe Dam	WIV	1,429	108.5
Lockyer Ck at Helidon	HEL	377	15.0
Tenthill Ck at Tenthill	TEN	465	19.0
Lockyer Ck at Lyons Bridge	LYO	1,590	75.0
Brisbane R at Savages Crossing	SAV	728	40.0
Brisbane R at Mt Crosby Weir	MTC	358	47.0
Bremer R at Walloon	WAL	626	44.0

Sub-Catchment	Name	Area (km²)	*k*
Warrill Ck at Kalbar	KAL	469	34.0
Warrill Ck at Amberley	AMB	449	35.0
Purga Ck at Loamside	PUR	223	49.0
Bremer R at Ipswich	IPS	265	15.7
Brisbane R at Jindalee	JIN	390	20.8
Brisbane R at Port Office Gauge	POG	339	19.3

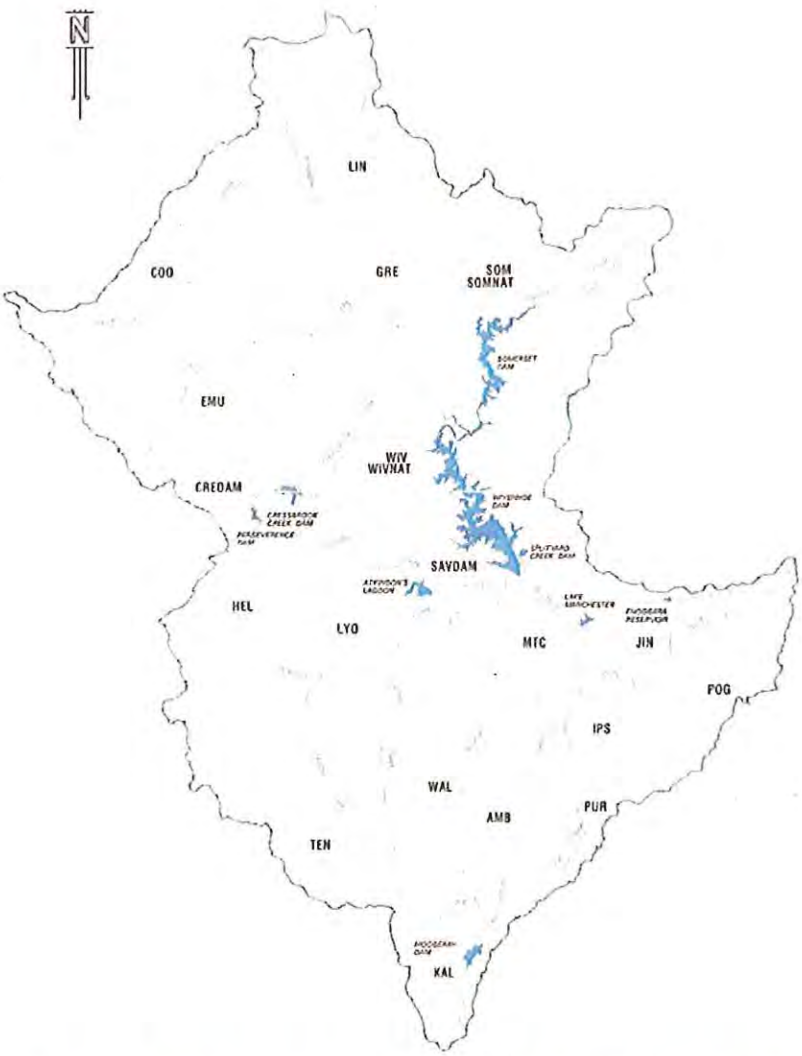


Figure 3-1: WT42 model layout (from South East Queensland Water Board and Natural Resources Queensland, 1993).

3.2. URBS model

The URBS model for the Brisbane River catchment was developed by Seqwater (2011). URBS is the most wide-spread hydrologic model for real time flood forecasting in Australia, and is used by the Bureau of Meteorology across Australia.

The model is similar to the WT42 model, but has a set of different routing parameters:

- *alpha* = channel lag parameter
- *beta* = catchment lag parameter
- *m* = non-linearity parameter (0.8, in accordance with Australian Rainfall and Runoff)

Similarly to the WT42 model, URBS characterises catchment losses using an initial loss/continuing loss model. However, an infiltration model has been included in URBS such that the continuing loss parameter is reduced from its initial value until a maximum infiltration capacity is reached, at which losses become zero. During periods of no rain, the infiltration capacity recovers and the continuing loss is reinstated.

The model layout is shown in Figure 3-2, which shows that the catchment has been divided into 7 sub-catchments. The areas associated with these catchments are shown in Table 3-2, along with an estimate of the *alpha* and *beta* values from calibration of the model to the January 1974 and February 1999 events.

- **Table 3-2: URBS model sub-catchment areas, and range of alpha and beta from 1974 and 1999 calibrations.**

Sub-Catchment	Name	Area (km²)	*alpha*	*beta*
Stanley R to Somerset Dam	STANL	1312	0.10 – 0.13	2.0 – 3.0
Upper Brisbane R to Wivenhoe	UPPER	5678	0.10	2.0 – 2.5
Lockyer Ck to O'Reilly's Weir	LOCKY	2974	0.15 - 0.20	3.0
Bremer R to Walloon	BREME	639	0.15 – 0.25	2.5 – 3.0
Warrill Ck to Amberley	WARRI	913	0.20 – 0.35	2.5 – 3.0
Purga Ck	PURGA	210	0.10 – 0.30	3.0 – 5.0
Lower Brisbane R	LOWER	1779	0.10 – 0.11	3.0

The URBS model of the lower Brisbane River has been configured with dummy storages at key locations to mimic the behaviour of the interaction between the river and its adjacent floodplain. Ratings for most locations, including dependent ratings, are included in the model. Dependent ratings derived from a hydrodynamic model reflect impacts such as the backwater impact at Ipswich due to high levels in the Brisbane

River and the tidal impact at Brisbane. The model has been calibrated on every large event since 1955.

The advantages of the URBS model are that it has a graphical user interface and automated procedures for preparing rainfall and streamflow information for use in the model and is well suited to real-time flood forecasting.

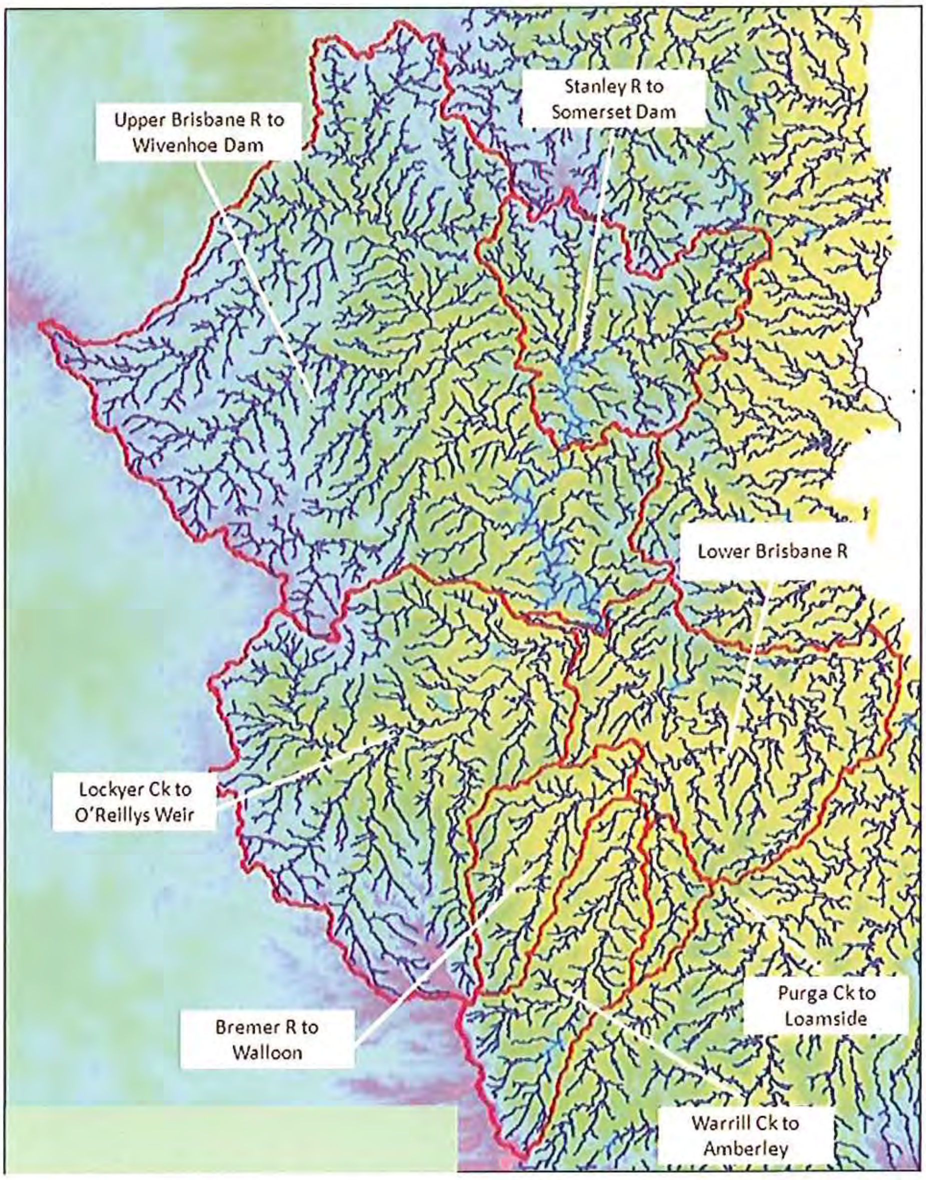


Figure 3-2: URBS model subcatchments for the Brisbane River.

3.3. January 2011 Rainfall Inputs

3.3.1. Method for deriving sub-area rainfalls

The distribution of rainfall across the catchment has been derived using the SUBRAIN utility. This program is based upon the same methodology adopted by the Bureau of Meteorology for flood forecasting (Malone, 1999). This derives a virtual pluviograph for each sub-area based on the nearest pluviograph and daily rainfall stations. This requires the following inputs:

- A list of the coordinates of the centroids of each sub-area (*.sub);
- A list of the coordinates of each of the daily and pluviograph rainfall gauges (*.net); and,
- Hourly rainfall at each gauge in separate files (*.r).

The SUBRAIN utility weights the rainfall data at each of the stations based on the inverse square of the distance to the centroid of each sub-area. The user is able to specify how many of the closest stations should be used, where the default value adopted historically for the Brisbane River catchment has been 4. The sensitivity of this assumption was tested (see Appendix A.2) and it was found that this has a minor impact on sub-area rainfalls; however a value of 6 has been used.

3.3.2. Review of rainfall data

All of the rainfall data received from Seqwater was reviewed. This was undertaken through a number of different checks:

- Where the same gauge has been recorded as both daily and pluviograph, the records were reviewed to determine the actual status of the gauge – only one instance of the data was used as inclusion of both would have biased the derivation of the sub-area rainfalls;
- Gauges that were commented out were investigated to determine the cause;
- Where gauges have been specified in previous files but no data was provided by Seqwater, a reason for this was sought;
- Where gauges have a similar name they were checked to ensure that a location is not included more than once (as this would bias the estimation of the sub-area rainfalls);
- Where gauges have been specified in the *January 2011 Flood Event* report (Seqwater, 2011a), but no data was provided by Seqwater, a reason for this was sought.

It was found that for some gauges for which Seqwater was not previously able to obtain daily data from BoM during the event, data is now available on the BoM website. In some cases this data had not yet been quality checked, but if this data was consistent with other gauges in the vicinity, it was used.

Pluviograph records could not be reviewed as they are predominantly ALERT gauges that Seqwater have the most up to date information for.

Once the verification of the data provided by Seqwater was undertaken, a comparison was undertaken of all the gauges provided by Seqwater and BoM gauges that recorded data in 2011. The BoM gauges included in this analysis were based on a search using the data bases available online (both through the Water Resources Station Catalogue and Climate Data Online) and this confirmed that there were no additional gauges available from BoM.

Finally, the rainfall totals for each gauge were spatially plotted to identify anomalies in the data recorded. Where anomalies were found, the gauges were reviewed against data from the BoM website. Five gauges were removed from the analysis through this process (40841, 40867, 40893, 40110, 40792 and 40963) and one gauge was revised using data available from the BoM (40914).

A summary of the findings of this investigation is provided in Appendix A.1.

3.3.3. Rainfall gauge locations

The rainfall gauges available during the event have been assessed to determine whether there is any bias in their locations. Figure 3-3 shows the distribution compared with mean annual rainfall across the catchment (although each event will have its own unique spatial distribution of rainfall the mean annual rainfall has been used as an indicator of generally drier and wetter areas in the catchment).

This shows that there is a slight tendency for rainfall gauges to be located in the drier parts of the catchment. For example, 30% of the catchment has a mean annual rainfall greater than 1,000 mm but this area only has approximately 15% of the rainfall gauges. This result is not unexpected given the higher rainfalls areas are typically associated with steeper topography which makes installation and maintenance of rainfall gauges more difficult.

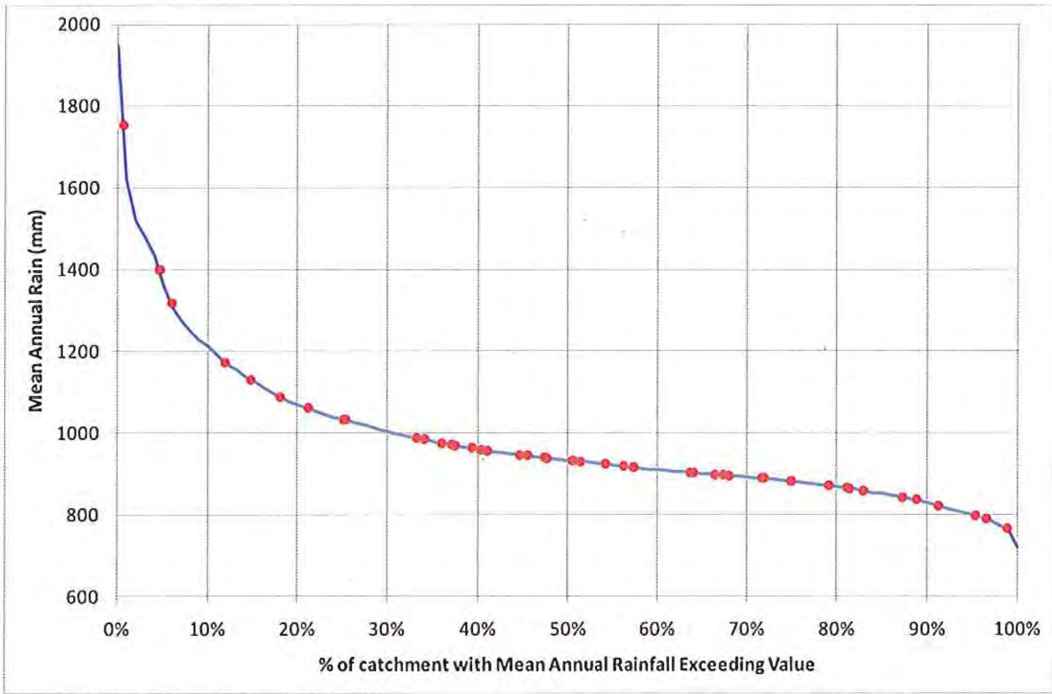


Figure 3-3: Distribution of mean annual rainfall throughout the Brisbane River catchment. Dots represent gauges available during the January 2011 event.

3.3.4. Sub-area rainfalls

The rainfall totals recorded at each of the gauges in the catchment during the January 2011 event (9am 2nd January 2011 to 9am 20th January 2011), and the associated sub-area rainfalls for the URBS and WT42 models are provided in Figure 3-4. Note that as the outflows from Wivenhoe Dam are known, the catchment upstream of the Dam has not been analysed. This shows differences in the sub-area rainfalls between the models which is due to the different locations of the sub-area centroids.

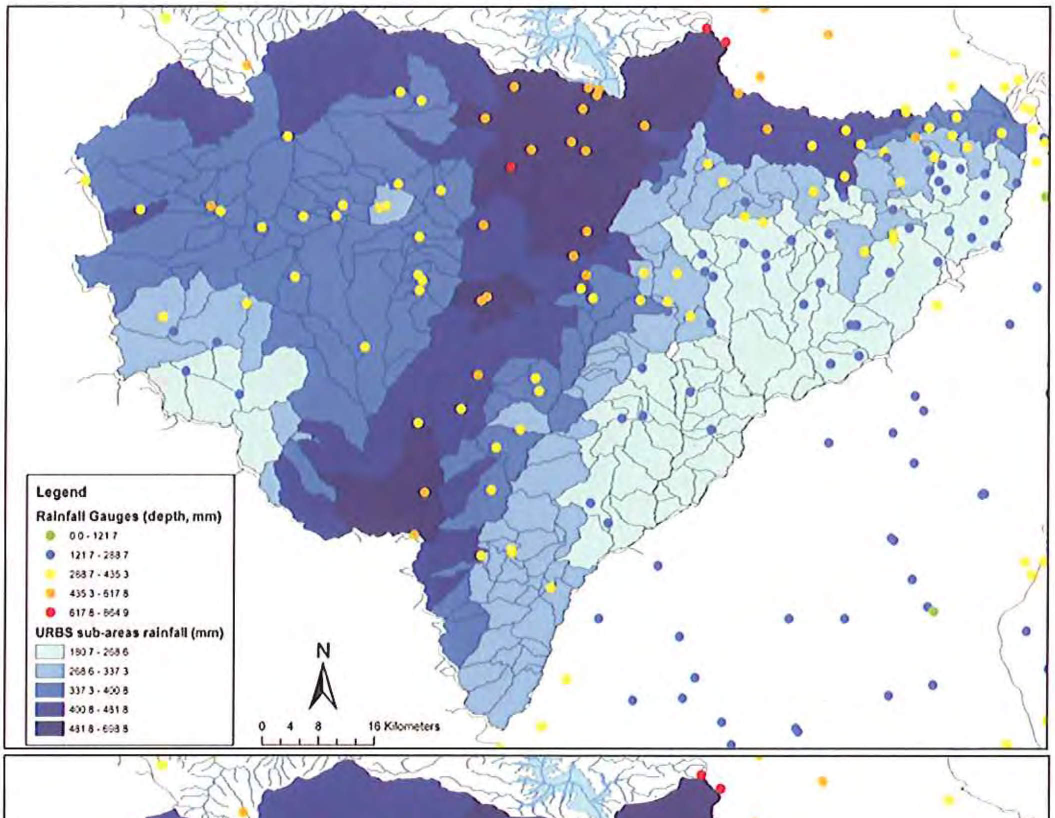


Figure 3-4: Sub-area rainfalls for the URBS (top) and WT42 (bottom) hydrologic models for the period from 2nd January 2011 to 20th January 2011.

3.4. Initial Calibration of hydrologic models to January 2011 Event

The initial calibration of both the URBS and WT42 models was focussed on understanding the differences in the models. The rainfall inputs described in Section 3.3 were input into the hydrologic models and the model parameters were altered in order to gain a good fit to the flows estimated from the river gauges. It should be noted that the flow estimated at the river gauges is based on recorded levels that are converted to flow using a rating table. For the majority of gauges in the catchment, these rating tables have been estimated, and are not reliable. Rating tables for key gauges on the Brisbane River have been revised using the hydrodynamic model, and more information on this is provided in Section 5.

The calibration fit at key locations is shown in Figure 3-5 to Figure 3-10, and these show that the URBS and WT42 models both provide similar results at key locations. The only exception to this is that the WT42 model provides a very peaky hydrograph at the site on Warrill Creek at Amberley, and the cause of this is unknown. Both hydrologic models were also used to check the estimated flow at other gauging stations within the catchment.

The adopted model parameters are provided in Table 3-3 and Table 3-4 and these are considered to be consistent both between sub-catchments and with previous calibrations. It should be noted that inflow for January 2011 event were further refined using the hydrodynamic model, and this is discussed in Section 6.1.

Table 3-3: URBS model parameters used to calibrate to the January 2011 event[1].

Sub-catchment	*alpha*	*beta*	Initial Loss (mm)	Continuing Loss (mm/h)
Lockyer Ck to O'Reilly's Weir (LOCKY)	0.15	2.5	50	1.5
Bremer R to Walloon (BREME)	0.25	2.5	20	2.5
Warrill Ck to Amberley (WARRI)	0.40	4.0	40	1.5
Purga Ck (PURGA)	0.40	3.0	50	1.5
Lower Brisbane R (LOWER)	0.10	2.5	50	2.5

[1] Note that *m* was held constant at 0.8 and infiltration was held constant at 500 mm across the catchment.

Table 3-4: WT42 model parameters used to calibrate to the January 2011 event[1].

Sub-catchment	*k*	Initial Loss (mm)	Continuing Loss (mm/h)
Lockyer Ck at Helidon (HEL)	17.0	30	1.5
Tenthill Ck at Tenthill (TEN)	40.0	30	1.5
Lockyer Ck at Lyons Bridge (LYO)	40.0	35	1.0
Brisbane R at Savages Crossing (SAV)	45.0	30	1.5
Brisbane R at Mt Crosby Weir (MTC)	45.0	30	1.5
Bremer R at Walloon (WAL)	50.0	50	2.5
Warrill Ck at Kalbar (KAL)	20.0	20	2.5
Warrill Ck at Amberley (AMB)	35.0	35	2.0
Purga Ck at Loamside (PUR)	45.0	45	1.5
Bremer R at Ipswich (IPS)	25.0	25	1.5
Brisbane R at Jindalee (JIN)	20.0	20	2.5
Brisbane R at Port Office Gauge (POG)	35.0	35	2.5

[1] Note that *m* was held constant at 0.8 across the catchment

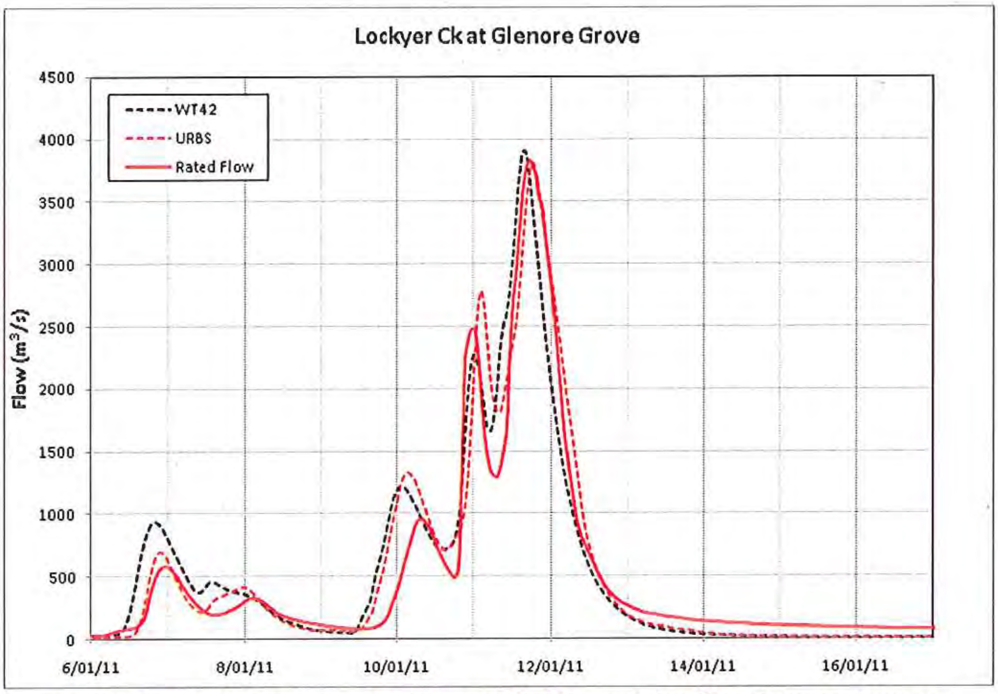


Figure 3-5: Comparison of URBS and WT42 model calibrations to January 2011 event at Lockyer Creek at Glenore Grove.

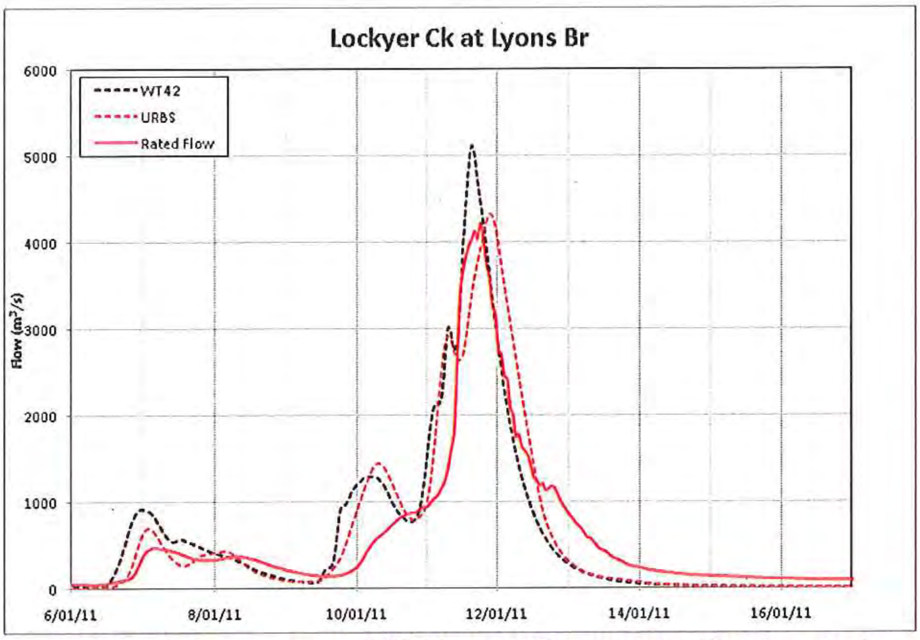


Figure 3-6: Comparison of URBS and WT42 model calibrations to January 2011 event at Lockyer Creek at Lyons Bridge.

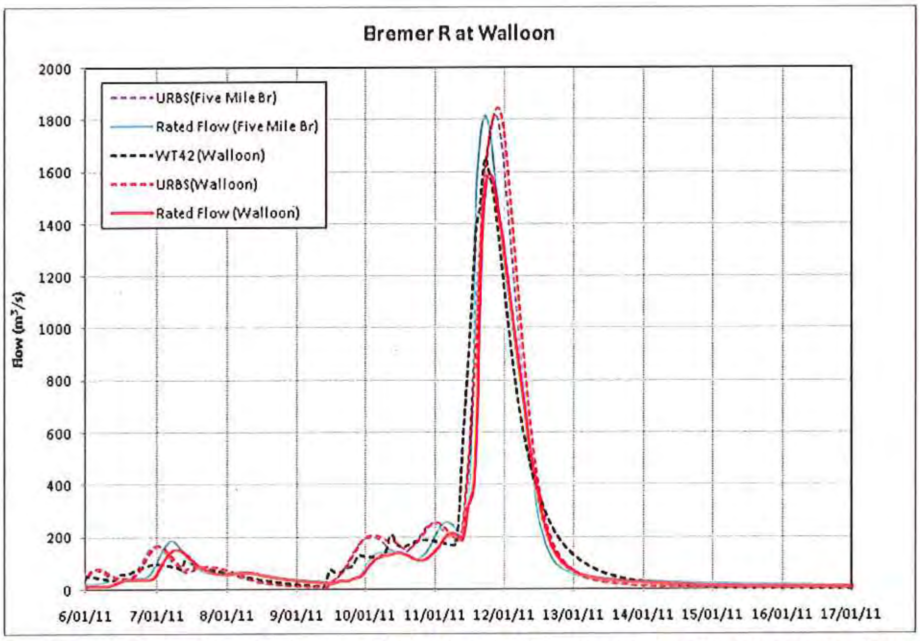


Figure 3-7: Comparison of URBS and WT42 model calibrations to January 2011 event at Bremer River at Walloon.

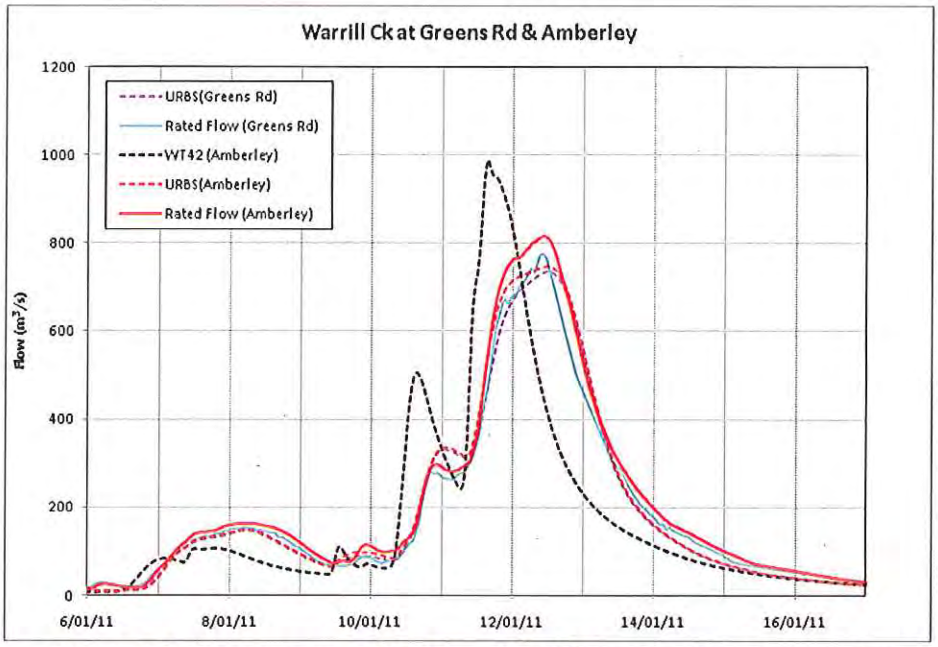


Figure 3-8: Comparison of URBS and WT42 model calibrations to January 2011 event at Greens Road and Amberley.

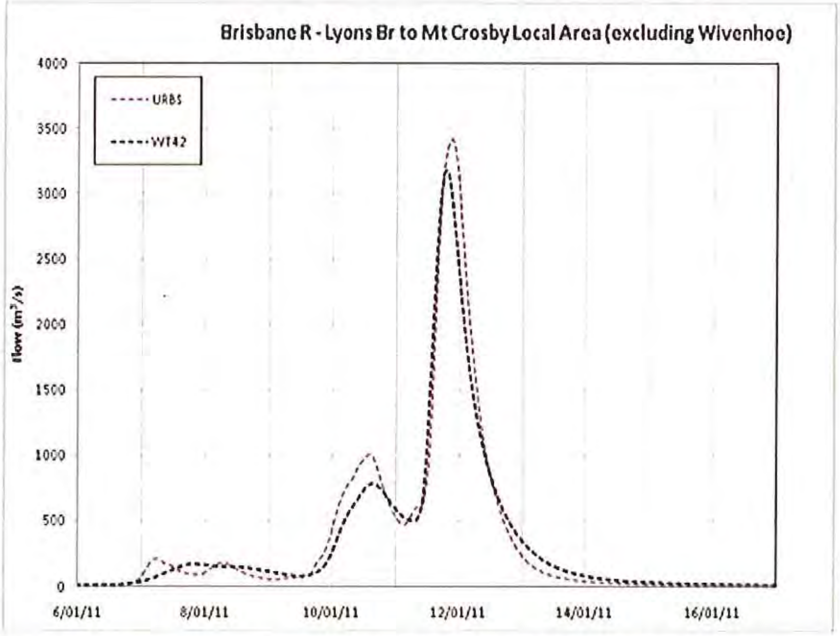
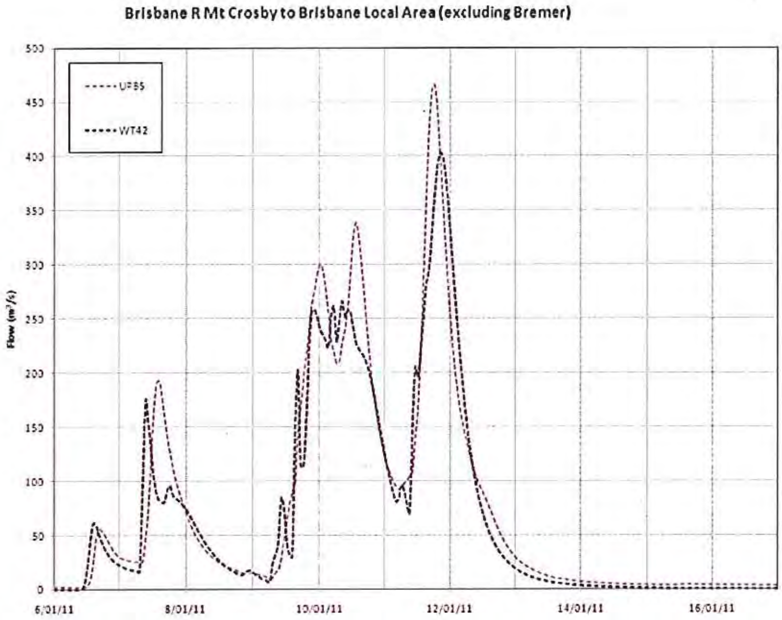


Figure 3-9: Comparison of URBS and WT42 model calibrations to January 2011 event for inter-station flows between Lockyer Creek at Lyons Bridge and Brisbane River at Mt Crosby.



- **Figure 3-10: Comparison of URBS and WT42 model calibrations to January 2011 event for inter-station flows between Brisbane River at Mt Crosby and Brisbane River at Port Office.**

4. Hydrodynamic Model Development

4.1. Past Hydrodynamic Models of the Lower Brisbane River

A number of reports have been reviewed and summarised to provide an overview of past hydrodynamic modelling of the Lower Brisbane River below:

1975 (November) "Brisbane River Flood Investigations" – Revision of the 1933 flood map prepared by the Queensland Bureau of Industry and a river stage-flood damage curve of the Brisbane River. *Queensland Cities Commission.*

1975 "Brisbane River Flood Plain Maps of Brisbane and Suburbs" – Flood maps were developed using flood levels estimated using a steady state gradually varied flow model of the Brisbane River which was calibrated to the January 1974 flood event. *Queensland Survey Office.*

1975 to 1976 "Wivenhoe Dam Tailwater Rating Derivation" – a backwater analysis of the Brisbane River from Savages Crossing past the then proposed Wivenhoe Dam site and up Wivenhoe Bridge that determined a tailwater rating for the dam. *The Irrigation and Water Supply Commission.*

1980 to 1981 "Simulation of Outflow from Wivenhoe Dam" – a calibrated (to events ranging from $200m^3/s$ to $7000m^3/s$) 1D implicit SHYDRO2 unsteady hydrodynamic model was developed of the Brisbane River from the Wivenhoe Dam to the Mount Crosby weir for investigating the consequences of a breach to the Wivenhoe Dam during its construction and preparing a flood manual for the Wivenhoe Dam. In 1981 the analysis was extended from the Mount Crobsy weir to the Brisbane River Mouth. *The Queensland Water Resources Commission.*

1985 "Report on Investigations into the Effects of Sewage Disposal to the Brisbane River" – a hydrodynamic model of the tidal reaches of the Brisbane River was developed for the purposes of investigating sewage disposal in the Brisbane River. *Department of Local Government.*

1989 "Preliminary Dambreak Analysis of Wivenhoe Dam" – an implicit unsteady DAMBRK hydrodynamic model was developed to investigate the effect of a '*Sunny day*' failure to the Wivenhoe Dam for assisting the State Emergency Service counter disaster planning. *Water Resources Commission.*

1994 "Brisbane River and Pine Flood Study" – a calibrated (to July 1973, January 1974, the early and late April 1989 events) hydrodynamic Rubicon hydrodynamic model of the Brisbane River from the Wivenhoe Dam to the Moreton Bay was developed to investigate the risks of flooding if the Wivenhoe Dam and/or the Somerset Dam were to fail, and provide a tool for potentially providing a real time flood warning and forecasting scheme. *South East Queensland Water Board.*

1998 "Brisbane River Flood Study" – a calibrated (to events January 1974, May 1996, June 1983, and late April 1989) hydrodynamic MIKE 11 hydrodynamic model was developed of the Brisbane River for informing flood plain planning decisions, flood forecasting (PROPHET), and a revegetation strategy. *Brisbane City Council.*

2000 "Ipswich Rivers Flood Studies" – the calibrated (to events December 1991, January 1974, May 1996, late April 1989, and June 1983) hydrodynamic MIKE 11 hydrodynamic model previously developed during 1998 was extended and re-calibrated to include the Bremer River and a number of its and the Brisbane River tributaries (Brisbane River model extended to the Ipswich City Council and Esk Shire Council boundary – located at grid reference -27.5, 152.72). The model was subsequently used for informing flood plain planning decisions, investigating potential mitigation options (Levees, detention basins, and Dam operations). *Ipswich Rivers Trust.*

2003 "Brisbane River Flood Study: Further Investigation of Flood Frequency Analysis Incorporating Dam Operations and CRC-FORGE Rainfall Estimates – Brisbane River" – the re-calibrated MIKE 11 model previously developed during 2000 was re-simulated to provide a '*best*' estimate of the likely 1 in 100 AEP flow at Savages Crossing and Brisbane Port Office Gauge, as well as flood levels at the latter. *Brisbane City Council.*

2004 "City Design – Flood Modelling Services: Recalibration of the MIKE 11 Hydraulic Model and Determination of the 1 in 100 AEP Flood Levels" – Based on the findings of the 2003 study the MIKE 11 model previously developed in 2000 was re-calibrated (to events January 1974 and March 1955) for the reaches of river within the Brisbane City boundary, since re-calibration during the 2000 study was primarily focused within the Ipswich City Council boundary. Once re-calibrated the model was used to assess the robustness of the "best" estimate of flow for the 1 in 100 AEP event at the Brisbane Port Office Gauge. *Brisbane City Council.*

2004 "City Design – Flood Modelling Services: Calculation of Floods of Various Return Periods on the Brisbane River" – Using the MIKE 11 model re-calibrated in 2004 the model was used to provide peak flood flows, levels, and velocities for a range of design flood events. *Brisbane City Council.*

2005 "Design Discharges and Downstream Impacts of the Wivenhoe Dam Upgrade – Q1091" – following revisions and improvements to the estimate of a Probable Maximum Precipitation (PMP) severity type event the Ipswich Rivers Trust version of the MIKE 11 model was used to assess what impact the proposed and required Dam improvements would have on flood risk downstream. To do this, the MIKE 11 model was extended up to the Lyons Bridge from the Ipswich and Esk Shire boundary (located at grid reference -27.5, 152.72), adapted so that it could be used to assess a higher severity event, the Probable Maximum Flood (PMF), and re-calibrated to the January 1974 flood event so as to correspond with the provided models predictions (note: this did not include the areas of the model that were built upstream as part of the study). *Wivenhoe Alliance.*

2005 "Dam Failure Analysis of Wivenhoe Dam – Q1091" – Using the amended and re-calibrated MIKE 11 model developed in 2005 the model was used to assess the impact of a dam breach for the existing Wivenhoe Dam and following the implementation of improvements to the Dam. *Wivenhoe Alliance.*

2009 "Flood Study of Fernvale and Lowood" – a calibrated (to the January 1974 and May 1996 flood events) 1D/2D linked hydrodynamic TUFLOW hydrodynamic model of the Brisbane River and Lockyer Creek within the Somerset Regional Council's region was developed for informing land use planning and development, and emergency planning. A 2D model was considered appropriate due to the large and complex floodplains of the study extent. *Somerset Regional Council.*

For the purposes of this study SKM were asked by Seqwater to make use of the MIKE 11 model developed in 2005 in conjunction with the WT42 and URBS models.

4.2. Review of 2005 MIKE 11 Model

The MIKE 11 model that was last refined in 2005 by the Wivenhoe Alliance was provided by Seqwater to be used as a basis for this study. This model was reviewed in order to understand its appropriateness and robustness for modelling the January 2011 event. A summary of the review is provided below and more details of the review are provided in Appendix A.

The following key issues were found with the 2005 MIKE 11 model:

- Representation of the cross-sections were not found to be appropriate for the magnitude of floods of most relevance to this investigation;

- Some cross sections were reversed (not critical to the processing of the hydraulic curves, but making auditing of link channels more difficult);
- Link channels specified at some locations were activated too early, or at a lower level than in reality;
- Some bridge details appeared inconsistent with the dimensions from other sources;
- The use of unrealistically high values of roughness (coefficient of Manning's n as high as 0.2 in some instances);
- A number of baseflow inputs were included (presumably to help improve stability) which mask the true inflows and affect modelled flood levels (e.g. Six Mile Creek)
- The provided model used a hot start file of a previous model run to provide initial conditions for the model – although certain situations and scenarios may dictate its use, this approach makes it less flexible for use as a flood warning and forecasting tool.

Significant effort was expended on trying to utilise the provided model for this investigation while making only minor modifications, however this was ultimately not possible due to above-mentioned issues. Accordingly, significant revisions were made to the model to ensure that it was suited to modelling the January 2011 event.

4.3. Revised Hydrodynamic Model

4.3.1. Terrain Data

No additional survey has been undertaken as part of this study. However, detailed terrain data was obtained from the South East Queensland LiDAR capture project[1], and was used to gain a better understanding of the key hydraulic features in the project area. The extent of the LiDAR data available is shown in Figure 4-1. The LiDAR covers the majority of the study area, but unfortunately does not extend up Lockyer Creek.

---

[1] © The State of Queensland (Department of Environment and Resource Management) [2010]. © Qld Bulk Water Supply Authority trading as Seqwater [2009]. To the extent permitted by law, SEQ Water gives no warranty in relation to the material or information contained in this Data (including accuracy, reliability, completeness or suitability) and accepts no liability (including without limitation, liability in negligence) for any loss, damage or costs (including indirect or consequential damage) relating to any use of the material or information contained in this Data

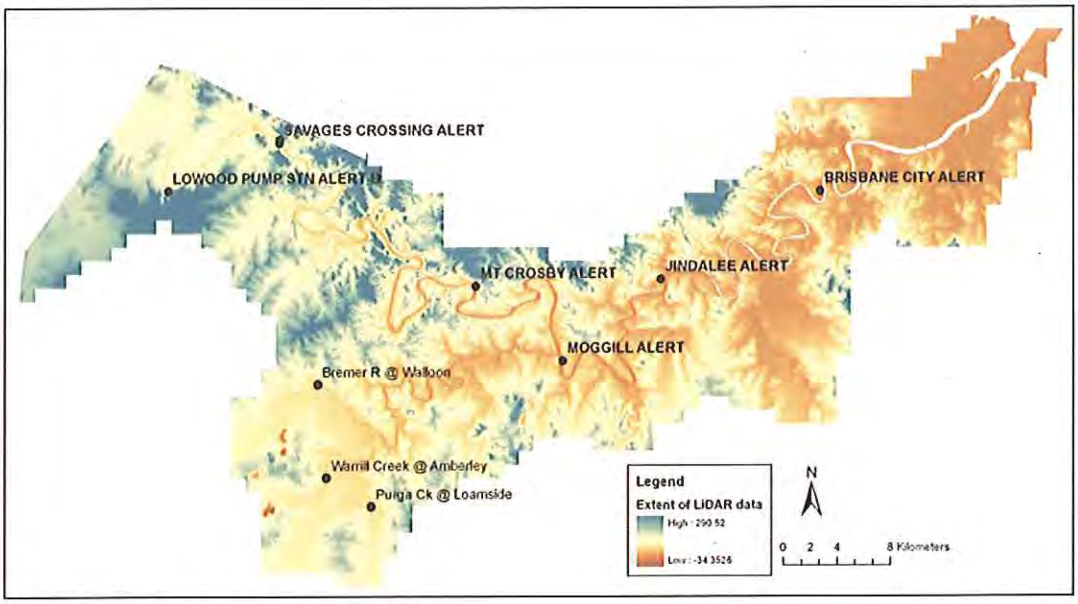


Figure 4-1: LiDAR data extent.

The LiDAR data was provided as a 1 metre resolution grid with an accuracy of +/ 0.15 m, and has had data processing routines applied to develop a '*bare earth*' model. This may, however, contain localised inaccuracies due to the presence of vegetation and buildings where as part of the data retrieval process the LiDAR laser strikes may have not reached the true ground surface.

The 1 m grid was processed into a 3 m grid by a routine where elevation points were retained at 3 m intervals. Appreciating that the aim of this study was to gain an understanding of the broad strategic routing of fluvial floodwaters to the City of Brisbane, this was considered to be an appropriate level of detail for representing the watercourses and their floodplains. If more refined routing characteristics are required of particular reaches of the Brisbane River (for example in the city itself where elevations at every 3 m will not be appropriate since this will not capture urban fabric details such as kerbs, walls, and other raised features which would influence flood flow dynamics) then this should be re-visited as appropriate.

4.3.2. Model Schematisation

The LiDAR data was used to better understand hydraulic controls throughout the Brisbane River to ensure that they are well-represented in the MIKE 11 model. The hydraulic processes along the Brisbane River are predominantly one-dimensional, but there are some two-dimensional aspects that needed to be carefully considered. Figure

4-2 demonstrates some key aspects of the model schematisation that have been included to better represent these two-dimensional attributes, namely link channels, storage areas and bend losses. These are described in Sections 4.3.3, 4.3.4 and 4.3.5, respectively.

The LiDAR data was also used to better refine the shape of cross-sections outside of the channel, and this process is described in Section 4.3.6. The model roughness, structures, boundary conditions and setup and parameters are described in Sections 4.3.7 to 4.3.10.

The total model schematisation is illustrated in Figure 4-5 to Figure 4-12 while Table 4-1 lists the chainages of key stream flow gauges, inflow locations, and hydraulic structures for the MIKE 11 model. It should be noted that the model has not been extended to represent any new additional reaches, but the model build has instead focused on resolving the issues that were identified during the review process.

Due to data constraints, only the Brisbane River reach from the Lowood Pump Station gauge to the mouth of the river have been re-schematised. All other contributing areas of the model have remained untouched unless amendments were required to improve the stability of the model or reduce the scope of what could be improved within the time constraints (e.g. Woogaroo Creek was changed to a storage area). It is important to stress, however, that the issues identified as part of the review part of this study are just as prevalent in other areas of the model and should be rectified to improve the confidence that can be placed in the predictions made by the model.

Table 4-1 Key Locations of the MIKE 11 model

Location	Type	Branch	Chainage
Wivenhoe Inflow	Boundary condition	BNE	930070
Mount Crosby Inflow	Boundary condition	BNE	988000
Savages Crossing Stream Flow Gauge	Channel Cross section	BNE	948120
Lowood Pump Station Stream Flow Gauge	Channel Cross section	BNE	936820
Allawah Road (Mount Crosby Weir Stream Flow Gauge) Bridge	Hydraulic Structure (Bridge)	BNE	988150
Moggill Inflow	Boundary condition	BNE	1004300
Moggill Stream Flow Gauge	Channel Cross section	BNE	1006300
Six Inflow	Boundary condition	BNE	1007780
Goodna Inflow	Boundary condition	BNE	1012475
Sandy Inflow	Boundary condition	BNE	1019490
Jindalee Inflow	Boundary condition	BNE	1025070
Jindalee Stream Flow Gauge	Channel Cross section	BNE	1026170
Oxley Inflow	Boundary condition	BNE	1040090
Oxley Stream Flow Gauge	Channel Cross section	BNE	1040090
Port Office Inflow	Boundary condition	BNE	1055280
Port Office Stream Flow Gauge	Channel Cross section	BNE	1055280
Breakfast Creek Infow	Boundary condition	BNE	1063125
Breakfast Creek Stream Flow Gauge	Channel Cross section	BNE	1063645
Bar Interstation Inflow	Boundary condition	BNE	1071520
Bulimba Creek Inflow	Boundary condition	BNE	1072020
Tidal Boundary	Boundary condition	BNE	1078660

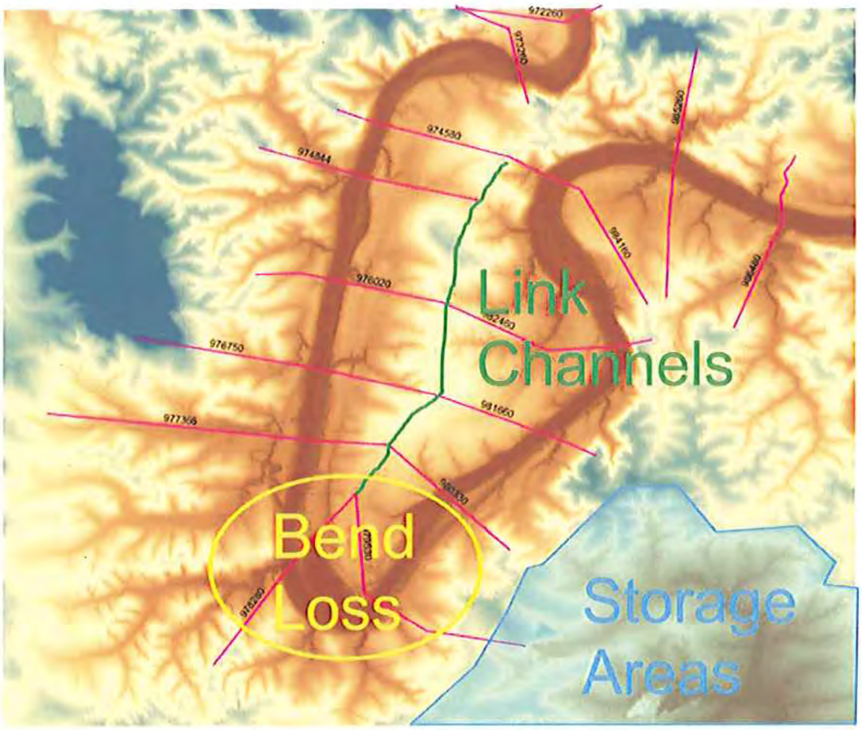


Figure 4-2: Key aspects of the model schematisation.

4.3.3. Link Channels

During high flows, water will not follow the main river channel, but rather will spill over low points in the terrain and "short cut" corners. Link channels simulate this process through defining a lateral weir that transfers water to nearby cross-sections once the weir is overtopped. The link channels have been defined as the high points between channel cross sections, as shown in Figure 4-2. Fifty-six link channels were added to the Brisbane River reach of the model.

4.3.4. Storage Areas

Areas of terrain that are not directly part of the Brisbane River, but which would serve to store water during times of flood, have been represented in the MIKE 11 model as a reservoir which is connected to the main river branch through a lateral weir. The lateral weir defines the terrain that links the storage to the Brisbane River, and this was defined using 2 m contour data, extracted from the 3 m terrain grid. The elevation-area relationship for the storage was developed using the 3 m terrain grid. Twenty-nine storage areas were added to the Brisbane River reach of the model.

4.3.5. Bend Losses

The Brisbane River has a number of large and sometimes severe meanders. To account for this in the overall representation of resistance to flow through the MIKE 11 model, the Manning's n roughness coefficient was increased for cross sections located at bends[2] in accordance with the recommendations provided in published guidance (Chow, 1959). The scaling factor used at these cross-sections is shown in Table 4-2. Appendix B.5 lists the locations of where these factoring values were specified and provides the locations of the cross sections.

Table 4-2: Factors used for the representation of river meanders (based on Chow (1959)).

Type of Meander	Factor applied to Manning's n
Appreciable	1.15
Severe	1.3

[2] This factoring was achieved through entering a higher factor at each bend cross-section in the HD parameters file which over-rides the global value.

4.3.6. Channel Geometry

Channel cross sections were developed by stitching together channel sections extracted from the provided MIKE 11 model with the extended floodplain representation extracted from the processed 3 m grid, as shown in Figure 4-3, to generate sections that spanned the entire floodplain. The channel sections were taken from the "*2003-x*" branch, which are believed to be original surveyed cross-sections.

In a number of locations the LiDAR data identified slightly raised areas of terrain which could have either been the presence of trees or small earth embankments serving to separate the main channel from field drains in the floodplain. In the majority of locations these slightly raised areas of terrain were ignored on the assumption that flood water would easily overtop the banks during flood conditions or reach the floodplain via a series of interconnected drains. However, where it was considered that in reality there would be a difference in the maximum water levels between the channel and the floodplain, these embankments were retained whereby floodplain sections were either included in the areas modelled as storage or raised so that a strategic representation of 1D flood routing could be represented. If future work is undertaken using the model developed as part of this study and/or if the routing detail of less severe events is required, then this should be reviewed and appropriately amended to represent the intended dynamics.

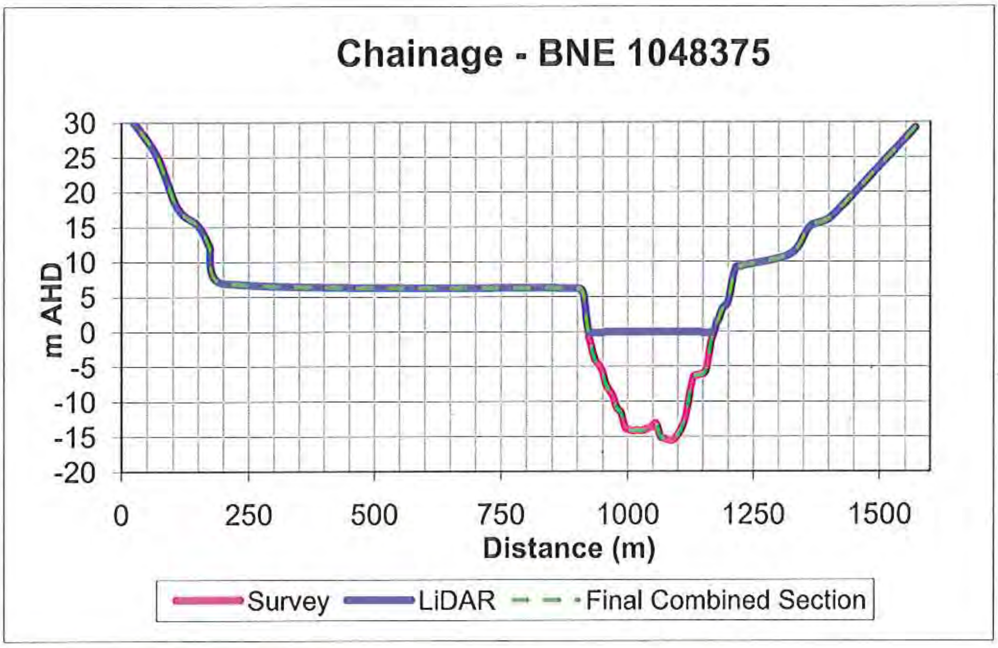


Figure 4-3: Example of cross section extension method.

4.3.7. Roughness

Channel and floodplain roughness are represented in the model through the use of an appropriate Manning's n value with low flow bank markers defining the transition to and from areas of higher roughness on the floodplains.

MIKE 11 provides the ability for the user to choose different radius types, which alters the method used to calculate conveyance for cross-sections with varying roughness values, for example where the floodplain roughness is specified as higher than the channel. The two main radius types are:

- *Hydraulic radius* (total area), which treats each of the different roughness zones as parallel channels, and calculates the total conveyance as the sum of these; and,
- *Resistance radius*, which develops an effective area for the cross-section, in line with the resistance factors specified, in order develop a single composite conveyance for the cross-section.

The hydraulic radius is considered to be better for use where flow is within deep and narrow cross-sections, as the resistance radius formulation does not fully take into account friction from the sides of the cross-section. However, resistance radius is considered to be more appropriate where there is significant variation in cross-section with flow magnitude, such as over bank flow paths (DHI, 2009).

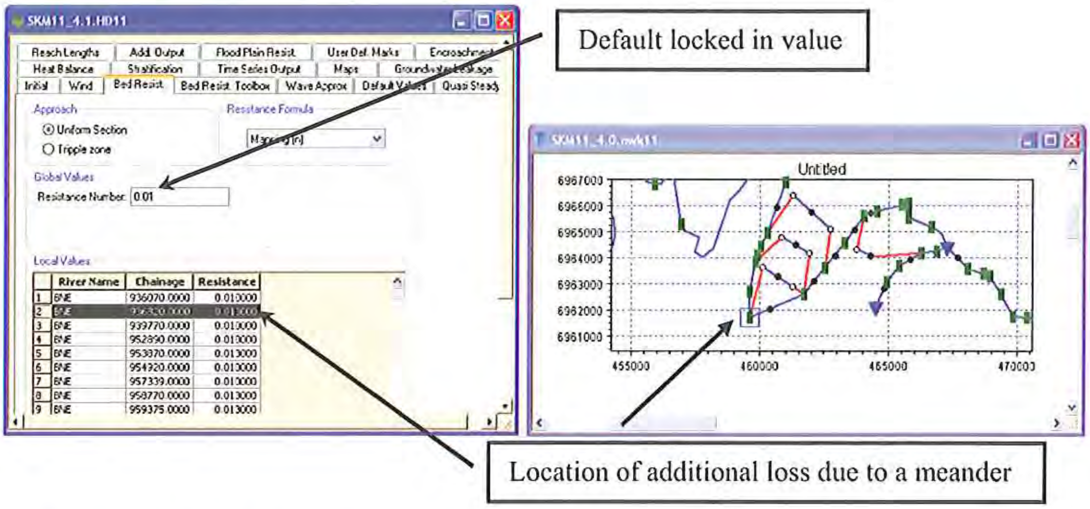
The adopted roughness values are a function of the selected radius type, and both approaches were explored in the initial calibration of the model to the January 2011 event. Both methods successfully reproduced the recorded levels, but the hydraulic radius type provided a slightly better hydrograph shape, while the resistance radius type provided slightly better timing.

The hydraulic radius type was ultimately adopted for the model runs on the basis that for the January 2011 event the majority of the flow is contained within the channel. It is important to note that the results of modelling scenarios within the flow range of the January 2011 flood were not sensitive to the selection of the radius type. However, the uncertainty will increase for flows in excess of the January 2011 event used for calibration.

The roughness values described in the remainder of this report relate to the hydraulic radius approach.

All channel cross sections are locked into a base value in the HD parameters (Figure 4-4) unless the cross section is located at a meander in which case it is overridden by a local factor to account for head losses that would occur (refer to Section 4.3.5). The differing channel and floodplain roughness of the sections are represented in each of the cross sections as being relative to this base value. For example, a channel Manning's n roughness of 0.03 can be specified as 3 in the cross sections channel geometry which is then multiplied by a base value of 0.01 in the HD parameters to derive a roughness value of 0.03 in the actual computation and simulation of the MIKE 11 model. This approach was applied for the following reasons:

1) It allows future users to readily see whether one cross section has a higher degree of roughness, as one section can be compared to the next.
2) It readily enables the use of the resistance number interpolation tool which can be used to calibrate one set of cross sections to a variety of past storm events, as changes to Manning's n are saved as the same cross section name, but with an additional extension name. This provides a more auditable trail for future model calibrations, as well as comparison and identification of sensitive parts, and/or potential errors in the model (e.g. higher roughness values used in one event but not in another, or higher roughness values used despite seasonal conditions or flood dynamic conditions cannot support their use).
3) To allow for sensitivity of the model to roughness, or large scale changes to the catchment roughness to be easily assessed, as users simply need to adjust the base Manning's n values in the HD parameters file.



■ **Figure 4-4: Application of roughness and bend losses.**

Initially, the roughness throughout the Brisbane River reach of the model was assigned as 0.05 for the channel and 0.08 for the floodplain, as these were the values used in the most recently developed *Tuflow* model of the catchment (Somerset Regional Council, 2009). Values of roughness for the channel and floodplain were then raised or lowered based on land use type by using aerial photography of the catchment. During this process, it was ensured that the adopted roughness values were consistent with those recommended by Chow (1959).

During these initial stages, the base roughness defined in the HD parameter file was set to 0.01. However, during the calibration process, this was raised to 0.0113, which effectively increased the roughness throughout the model by 13%.

Table 4-3 lists the values of roughness that were used to calibrate the model to the January 2011 event along the Brisbane River.

- **Table 4-3: Manning's *n* roughness values adopted in the MIKE 11 model.**

Branch	Chainage	Channel Roughness	Floodplain Roughness	Comments
BNE	930070 to 950270	0.079	0.090	Channel roughness raised to account for a more vegetated channel
BNE	951200 to 963595	0.054	0.090	
BNE	964170 to 994760	0.051	0.090	Floodplain roughness raised to 0.1 to account for the Corbould Land Trust and surrounding forested areas. Channel roughness raised according.
BNE	995690 to 1002785	0.051	0.090	
BNE	1003275 to 1019490	0.042	0.090	Channel roughness decreased to account for a less vegetated channel by Barellan Point to Moggill Country Club
BNE	1020115 to 1025590	0.042	0.090	Channel roughness decreased to account for a well maintained channel for suburbs of Brisbane
BNE	1026170 to 1036770	0.042	0.090	Channel roughness decreased to provide a better match to gauging at Jindalee.
BNE	1036915 to 1048035	0.034	0.113	
BNE	1048035 to 1078525	0.024	0.090	Channel roughness decreased to account for a well maintained and cleaned channel due to tidal processes

4.3.8. Fluvial Structures

Although there are a number of structures present on the Brisbane River, all but the Mount Crosby Weir and road crossing have been removed from the model. It was found that all of the bridges removed are either too small or large to significantly impact on the hydraulics of the river for the January 2011 event, and inclusion of the bridges in the model resulted in instabilities and known inaccuracies (see Appendix B.4). If future work is undertaken using the model developed as part of this study then additional structures should be included, since these will influence the local dynamics of floodwaters.

The Allawah Road Bridge, or Mount Crosby Weir, has been represented within MIKE 11 as a bridge structure solving the energy equation with FHWA WSPRO submergence and overflow default coefficients of discharge. The bridge details have been estimated using data provided by Sunwater and from publicly available photographs.

4.3.9. Boundary Conditions

The MIKE 11 model requires boundary conditions to be defined where river reaches start and end, and where additional inflows are included in the model. The model boundary conditions are described in Table 4-4. The Brisbane River upstream boundary condition is defined as the flow at Wivenhoe Dam, and the downstream boundary is set by tidal conditions in Moreton Bay. The tidal boundary has been defined as that as recorded at the White Island Gauge (CBM – 540495 / AWRC – 143891). Further details of the model inflows for the 2011 event are provided in Section 6.1.

Table 4-4: Boundary conditions in the MIKE 11 model.

Location	Boundary Type	Input	Description
BNE 930070	Open	Time-series flow	Wivenhoe Dam outflow
LOCKYER 3370	Open	Constant flow	Lockyer Creek at Lyons Bridge (dummy flow of 0.1 m^3/s)[1]
LOCKYER 9190	Point Source	Constant flow	Interstation flow from Lockyer Creek between Lyons Bridge and O'Reilly's Weir (dummy flow of 0.1 m^3/s)[1]
BNE 948120	Point Source	Constant flow	Interstation flow at Savages Crossing[1]
BNE 988000	Point Source	Time-series flow	Interstation flow between Wivenhoe Dam and Mt Crosby Weir
WAR 100000	Open	Time-series flow	Warrill Creek at Amberley

Location	Boundary Type	Input	Description
PURGA 100000	Open	Time-series flow	Purga Creek at Loamside
BREM 1000010	Point Source	Time-series flow	Bremer at Walloon
DEEB 10000	Closed		Deebing Creek[2]
DEEB 1005200	Point Source	Time-series flow	Deebing Creek
IRON 10000	Closed		Ironpot Creek[2]
IRON 18584	Point Source	Time-series flow	Ironpot Creek
BUND 10000	Closed		Bundamba Creek[2]
BUND 41030	Point Source	Time-series flow	Bundamba Creek
HWAY Left 0	Open	Constant flow	Small creeks within the Bremer River catchment whose flow contribution is included in other inflows (dummy flow of 0.1 m³/s).
LOW BRANCH1 0	Open	Constant flow	
LOW BRANCH2 0	Open	Constant flow	
UP BRANCH1 0	Open	Constant flow	
Small 1000	Open	Constant flow	
Reedy 1000	Open	Constant flow	
Mihi 10000	Open	Constant flow	
Mihi_br1 1292	Open	Constant flow	
Sch 10000	Open	Constant flow	
BREM 1020000	Point Source	Time-series flow	Interstation flow for Bremer R at One Mile Bridge
BNE 1007780	Point Source	Time-series flow	Six Mile Creek
BNE 1012475	Point Source	Time-series flow	Goodna Creek
BNE 1014610	Point Source	Time-series flow	Woogaroo Creek
BNE 1019490	Point Source	Time-series flow	Sandy Creek
BNE 1004300	Point Source	Time-series flow	Interstation flow for Moggill
BNE 1025070	Point Source	Time-series flow	Interstation flow for Jindalee
BNE 1040090	Point Source	Time-series flow	Oxley Creek
BNE 1050860	Point Source	Time-series flow	Interstation flow for Port Office
BNE 1063125	Point Source	Time-series flow	Breakfast Creek
BNE 1071520	Point Source	Time-series flow	Interstation flow for Bar
BNE 1072020	Point Source	Time series flow	Bulimba Creek
BNE 1078660	Open	Time-series water level	Tidal boundary

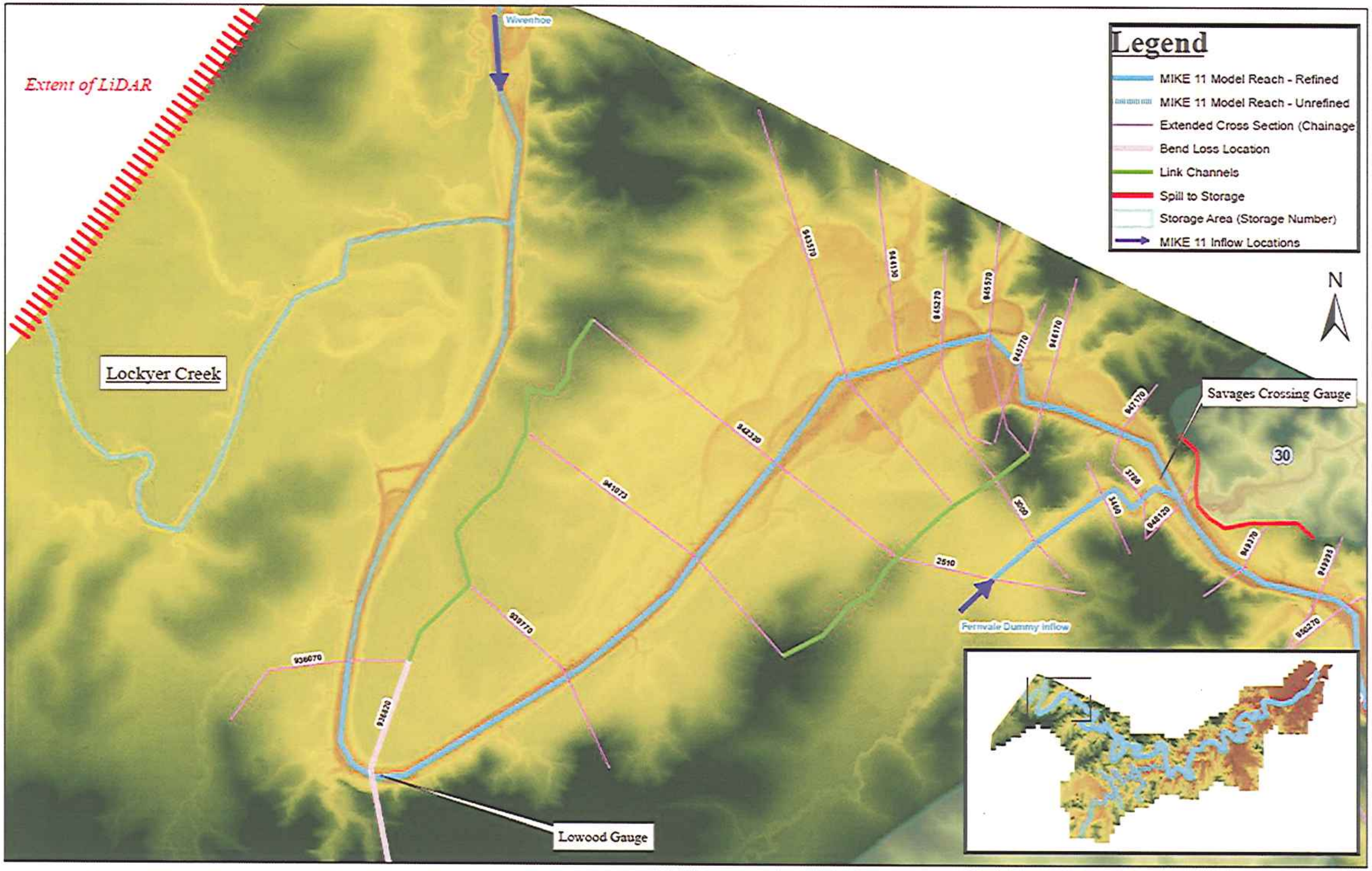
[1] Boundary conditions have been included in the model for Lockyer Creek and Brisbane River at Savages Crossing, but these are not used in the final model runs, and so small flows have instead been added at these locations. See Section 6.1 for more information.

[2] Deebing Creek, Ironpot Creek and Bundamba Creek have been treated as closed reaches as the inflows from the URBS model are extracted at the outlet of the creeks and are therefore entered into the MIKE 11 model at the outlets.

4.3.10. Model Setup and Parameters

The MIKE 11 model has been setup to run with the following parameters:

- Unsteady state;
- Adaptive time step with default parameters and limits of minimum time steps of 5 seconds and maximum time steps of 300 seconds;
- Initial conditions defined as the water levels recorded at stream flow gauges during the 2011 event (ie the initial water level before the arrival of the flood hydrographs); and,
- The *delh* value (a factor used to calculate the allowable distance to the bottom of an artificial slot to prevent drying out) was increased to 3 due to many areas of mismatching bed levels.



■ **Figure 4-5: MIKE 11 model layout – part 1.**

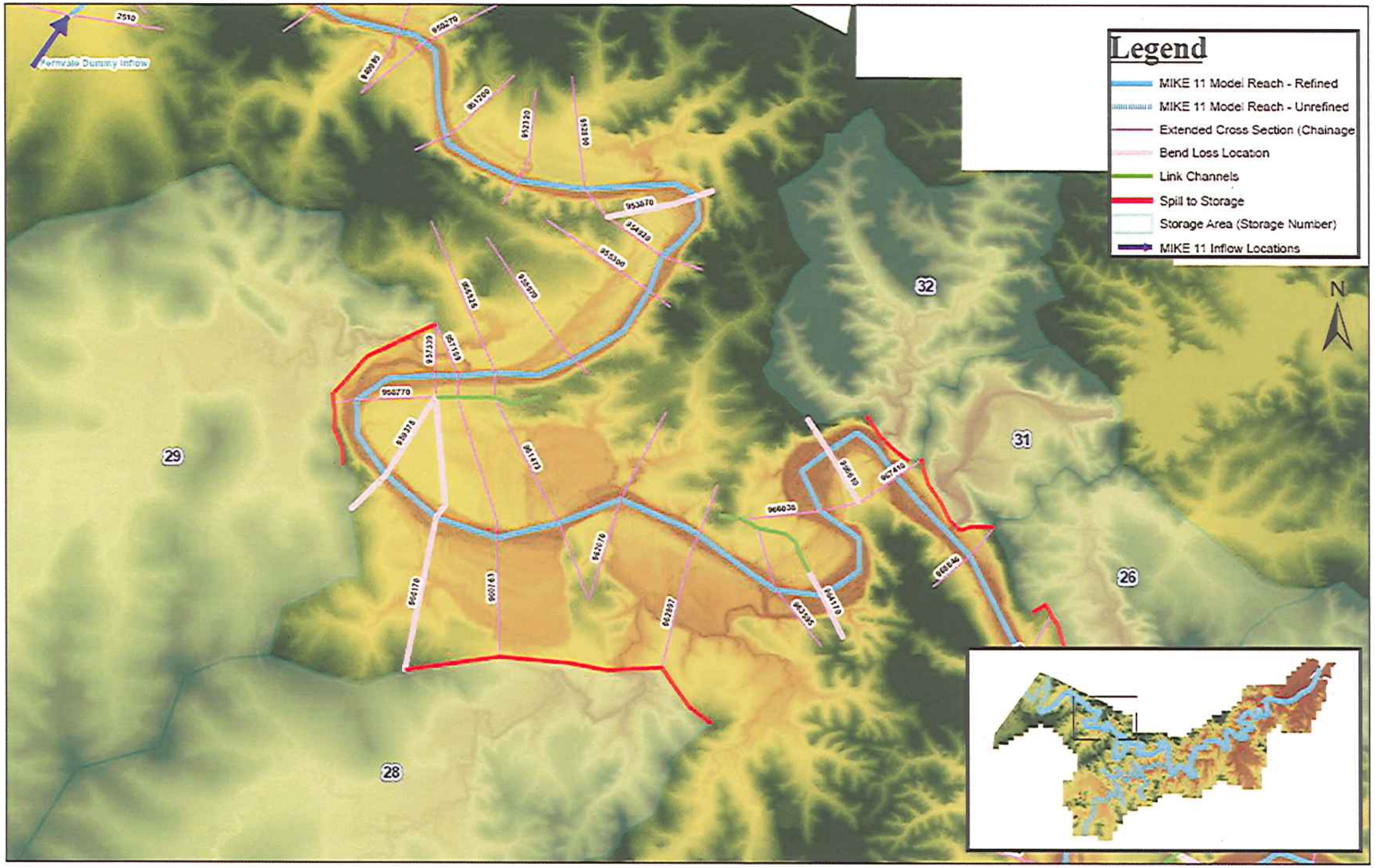


Figure 4-6: MIKE 11 model layout – part 2.

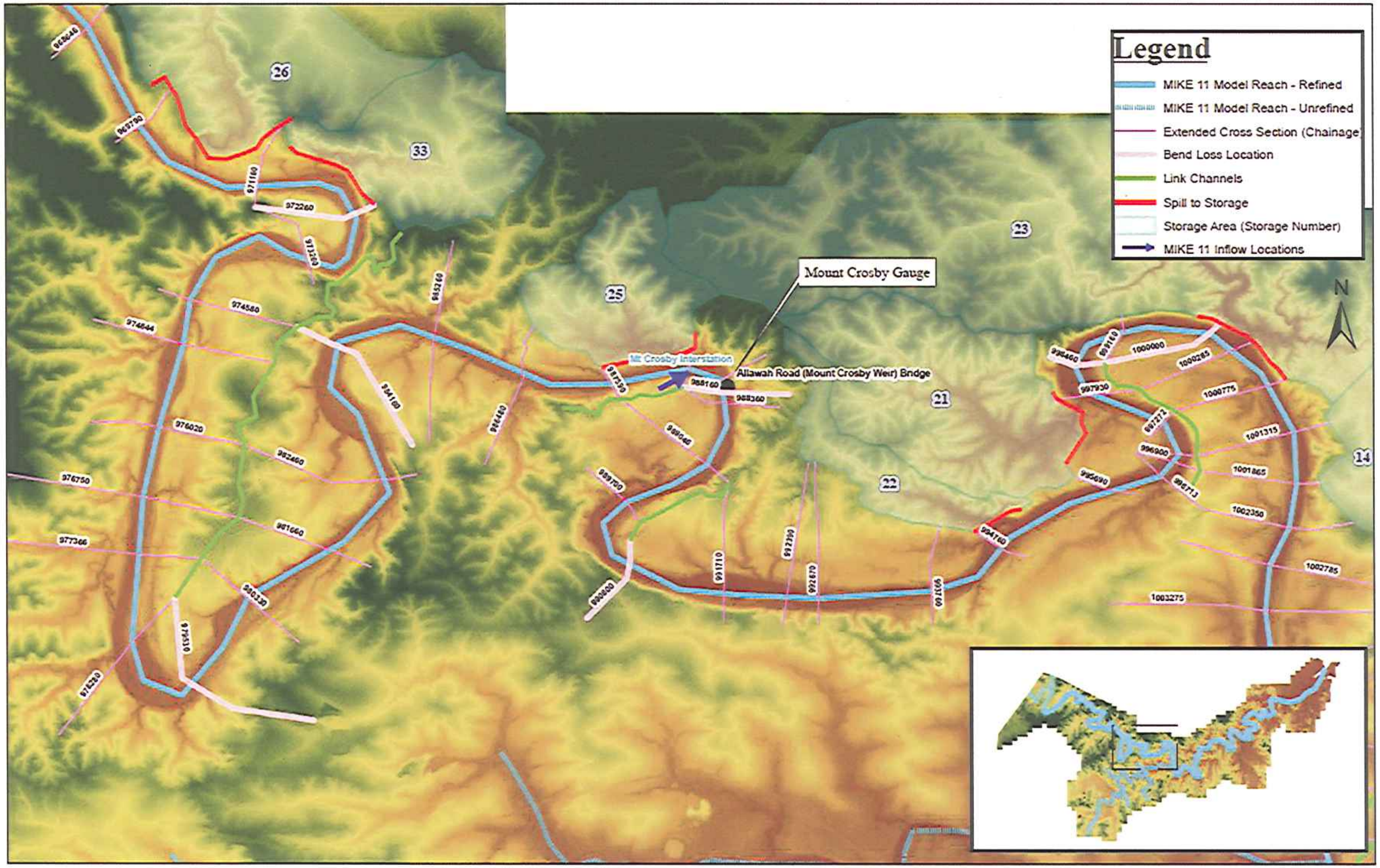
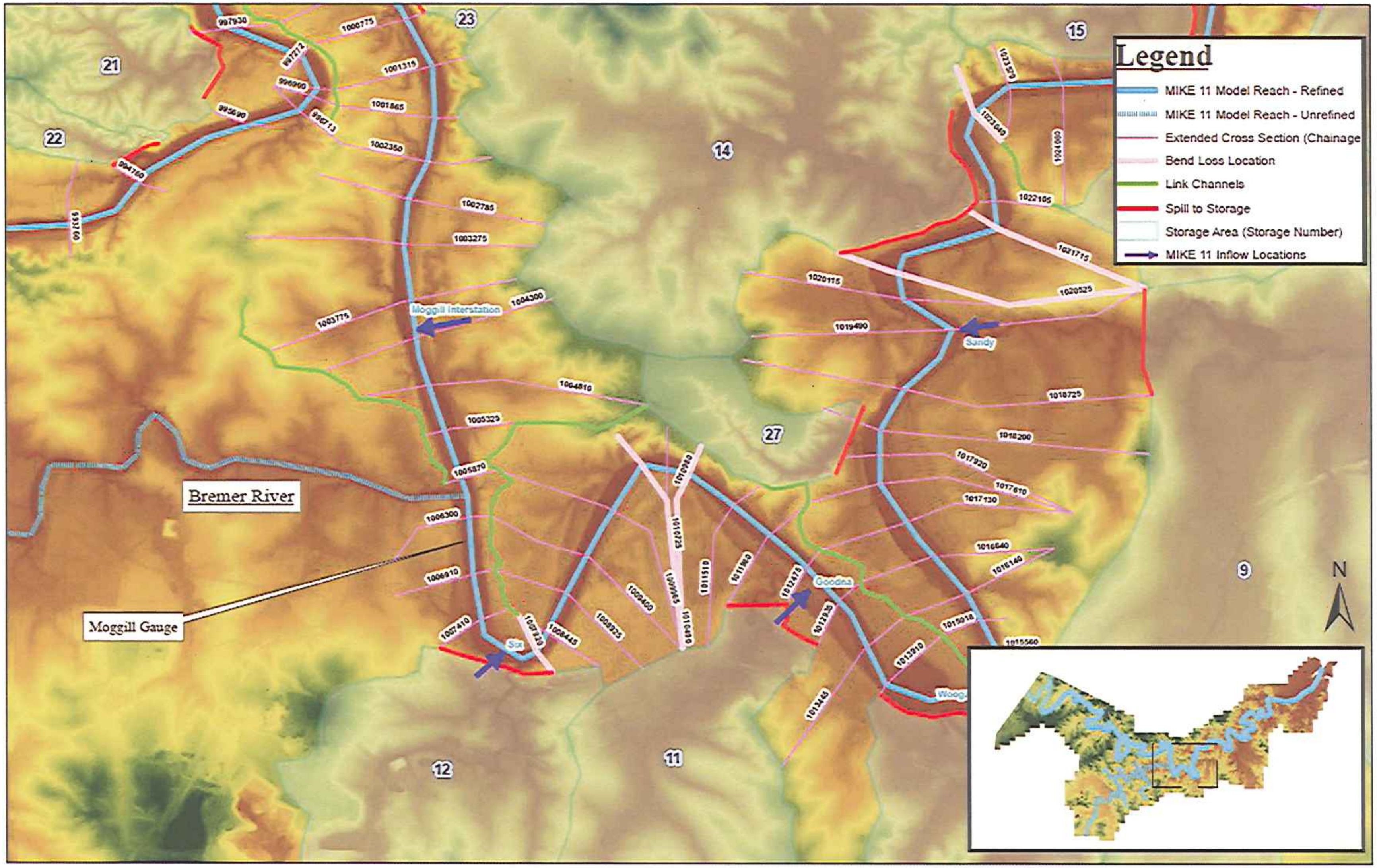
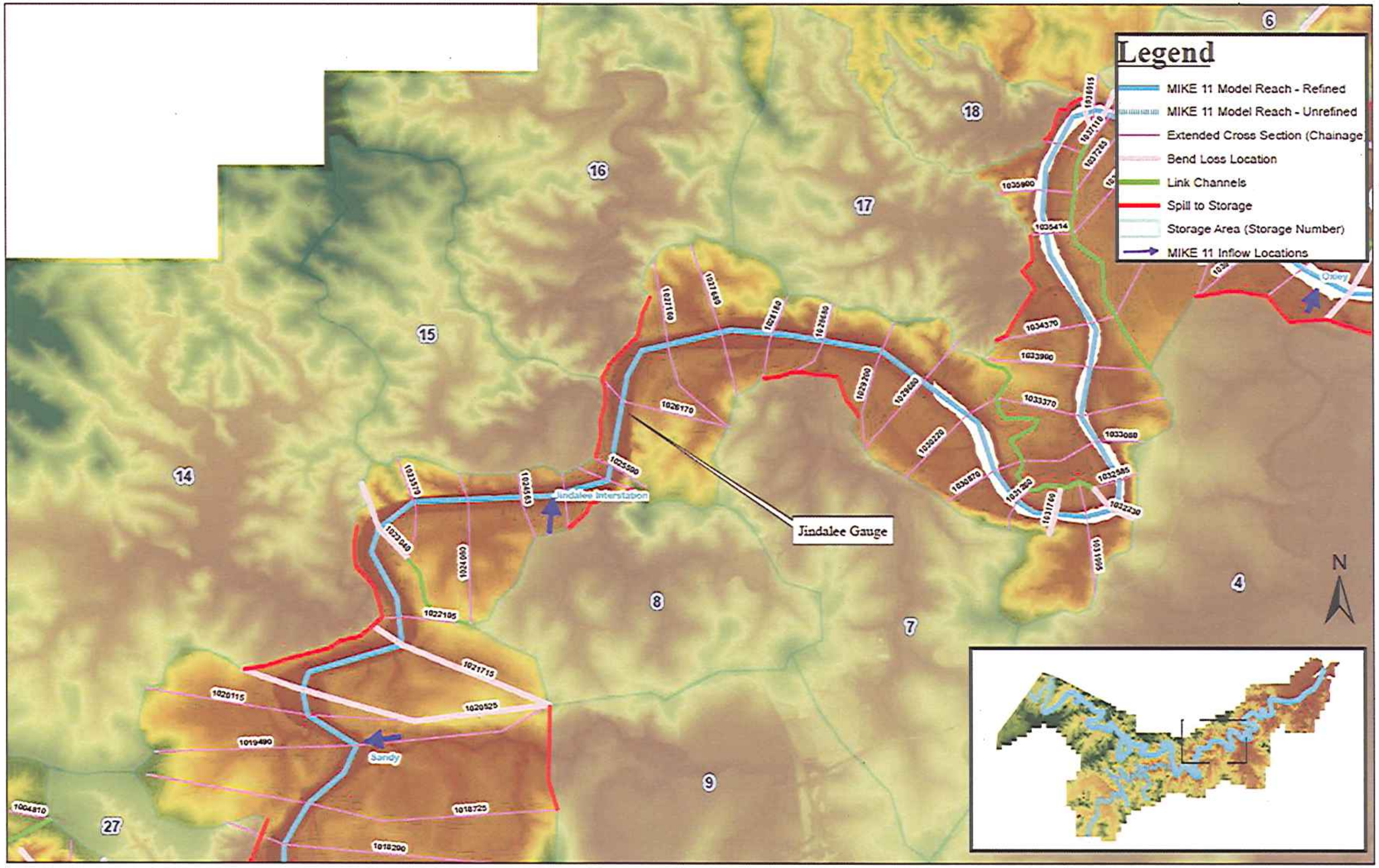


Figure 4-7: MIKE 11 model layout – part 3.



■ **Figure 4-8: MIKE 11 model layout – part 4.**



■ **Figure 4-9: MIKE 11 model layout – part 5.**

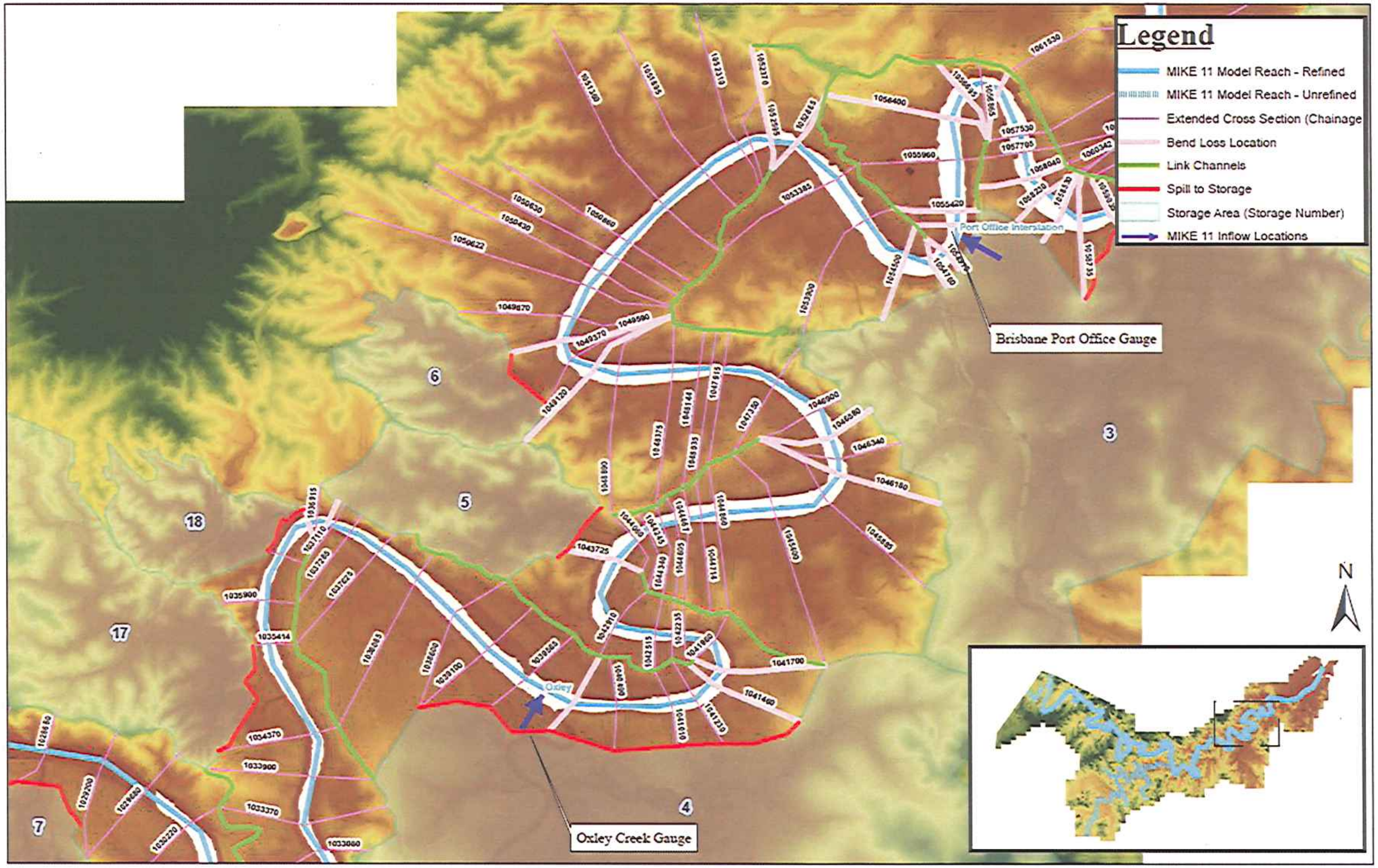


Figure 4-10: MIKE 11 model layout – part 6.

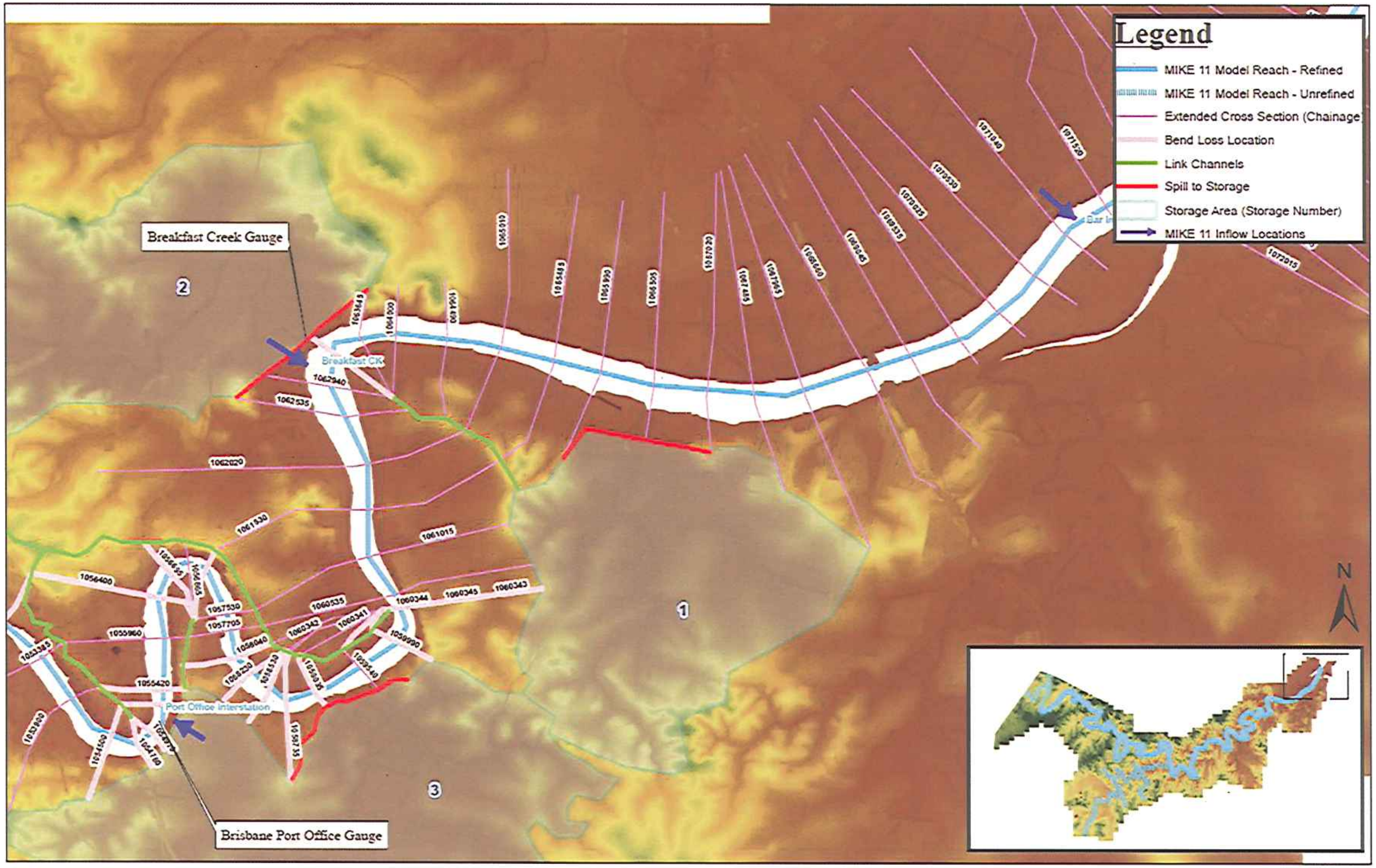
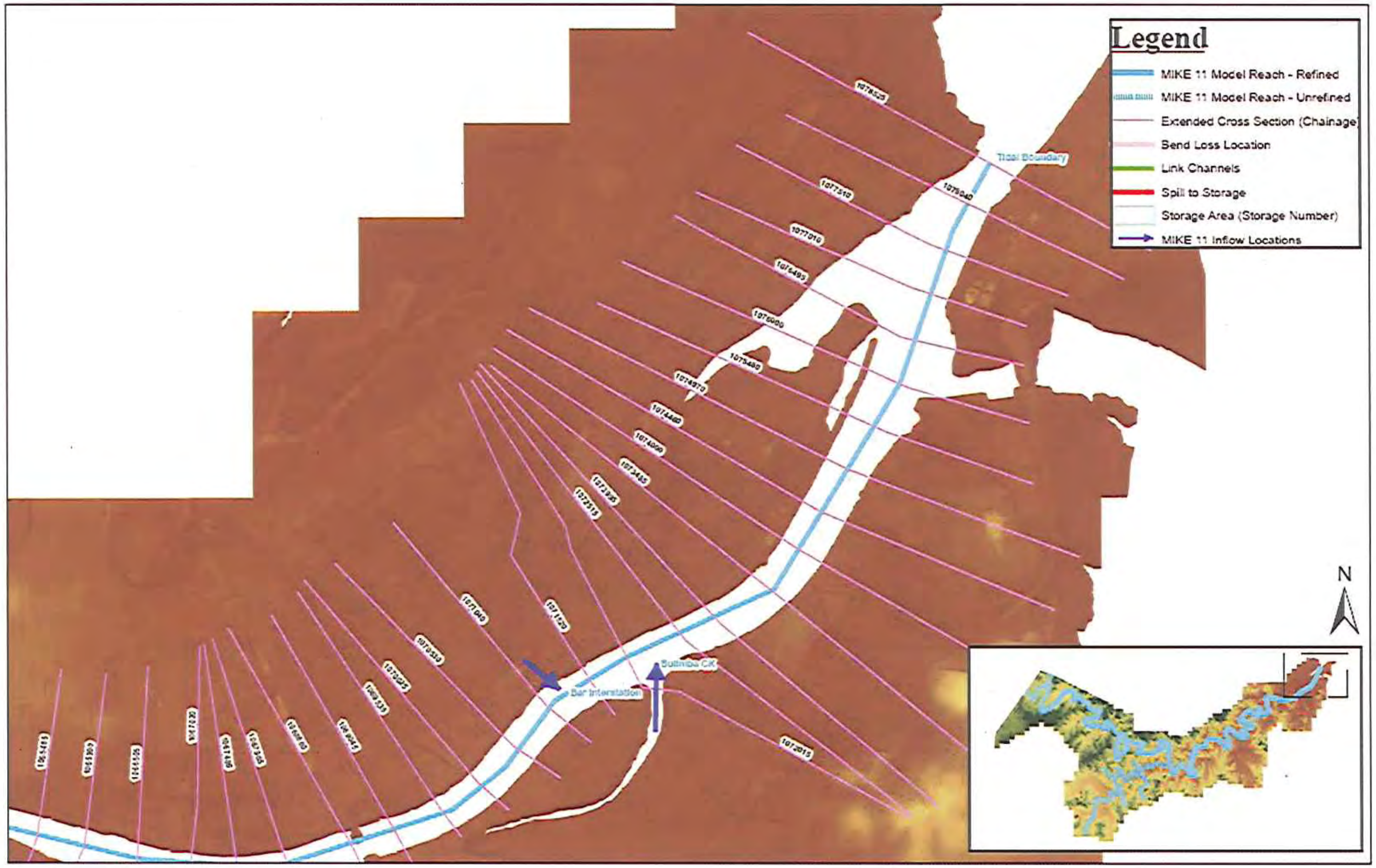


Figure 4-11: MIKE 11 model layout – part 7.



■ **Figure 4-12: MIKE 11 model layout - part 8.**

5. Rating Curves at key Brisbane River gauges

The calibration of hydrologic models has in the past been confounded due to the absence of rating curves for the key sites along the Lower Brisbane River. These sites record river level and the flows have had to be inferred from the results of a hydrologic model. The MIKE 11 model was used to develop rating curves at key gauges along the Lower Brisbane River.

5.1. Available Data

Data used to review and update the rating curves at Lowood, Savages Crossing, Mt Crosby, Moggill and Jindalee was obtained from a variety of sources, as listed below:

- streamflow gaugings undertaken during the January 2011 flood event;
- recorded river levels during periods of known constant releases from Wivenhoe Dam (including January 2011);
- DERM rating curves based on extrapolated flow gaugings;
- derived rating curves extracted from the calibrated URBS model provided by Seqwater; and
- modelled rating curves provided by BCC from a Brisbane River hydrodynamic model.

Not all data was available for each streamflow gauge location, but the available data was supplemented with modelled water levels at each gauge location extracted from a series of 'steady state' runs of the MIKE 11 model developed as part of this project. These runs were based on a simulation consisting of a constant inflow at the upstream end of Brisbane River model branch, with the simulation continuing for a sufficient time such that flow conditions were constant along the entire branch. This eliminated uncertainty in rating curves resulting from hysteresis.

5.2. MIKE 11 Model Results

The outputs from the MIKE 11 model steady state runs were used to derive rating curves at each gauge location. It was typically found that the MIKE 11 model results matched well with gauged estimates of streamflow, particularly for higher flows. It was also noted that the MIKE 11 results were significantly different to some of the supplied rating curves. In several cases the MIKE 11 rating curves showed that a larger flow would be expected for the same water level than was previously estimated.

On balance, it is believed that more weight should be given to the MIKE 11 model results for higher flows (greater than approximately 4,000 m^3/s) than the extrapolated DERM ratings and the derived ratings from URBS. The MIKE 11 model includes representation of the physical channel controls and floodplain storage present in the lower Brisbane River, which by definition cannot be directly accounted for in the URBS hydrological model. As such, it is suggested that rating curves derived from a combination of flow gauging and URBS estimates at lower flows, and MIKE 11 estimates at higher flows, be adopted for future use.

It should also be noted that the location of some gauges are problematic when considering the total channel flow at the gauge versus total flow in the river that may be carried on the floodplain or in an anabranch. This was an issue at Lowood for flows less than 15,000 m³/s, as the gauge is located on a river bend and significant overland flow occurs across the bend. For higher flows (e.g. approximately 15,000 m³/s and greater) similar floodplain bypasses are likely to occur at other locations such as Moggill, and to a lesser extent Mount Crosby, where downstream constrictions would cause flows to throttle back and flow over the floodplain. In this case, it was decided that the water level at Lowood representing a flow of 15,000 m³/s should be the water level corresponding with a total river flow of 15,000 m³/s, rather than the flow in the Brisbane River channel itself at this location (which is somewhat less).

5.3. Updated Rating Curves

Plots of the rating curves at each location were prepared and are shown in the following figures:

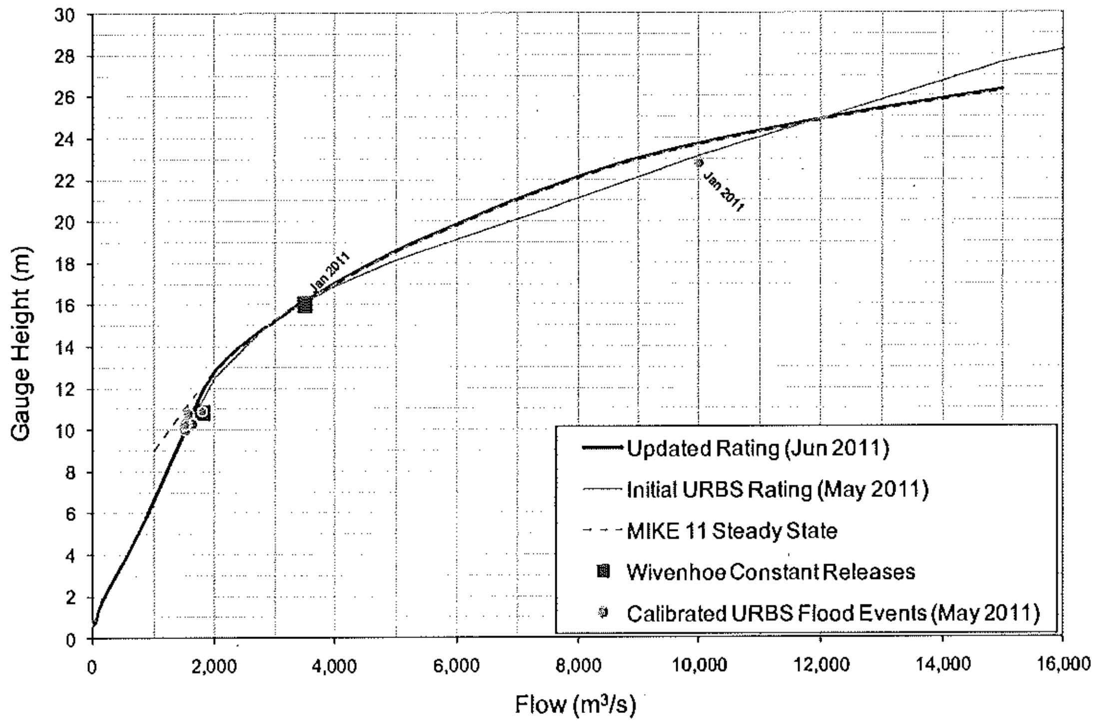


Figure 5-1 Rating Curve at Lowood Pump Station

It can be seen that the MIKE 11 model slightly overestimates the January 2011 constant release of 3,500 m^3/s at Lowood pump station. It is likely that this is due to the influence of a local channel control that has not been included in the model. At flow rates larger than 12,000 m^3/s, the MIKE 11 model also predicts higher flows than the rating derived in URBS. The updated rating curve was derived using the rating curve from URBS for flows up to 2,000m^3/s, with larger flows adopted from the MIKE 11 model results.

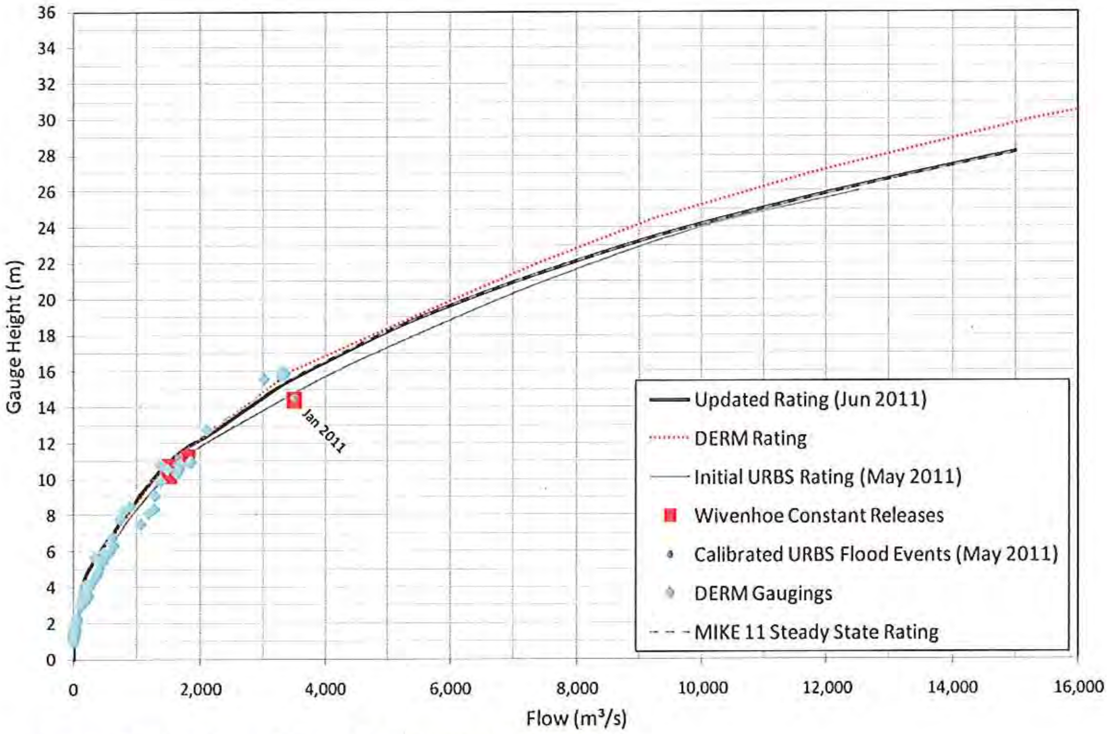


Figure 5-2: Rating Curve at Savages Crossing

The MIKE 11 model slightly over predicts the constant 3,500 m^3/s release from Wivenhoe Dam during the January 2011 flood at Savages Crossing. It slightly under predicts the lower flow constant releases from Wivenhoe Dam; however these differences are likely to be again due to a local channel control. The updated rating curve was derived from the DERM rating for flows up to 1,400 m^3/s and the MIKE 11 results for larger flows. The DERM gaugings for higher flows (3,000-3,500 m^3/s, undertaken during the January 1968 event) appear to be inconsistent with the constant Wivenhoe Dam release from the January 2011 event, and as such these gaugings have not been used to derive the updated rating.

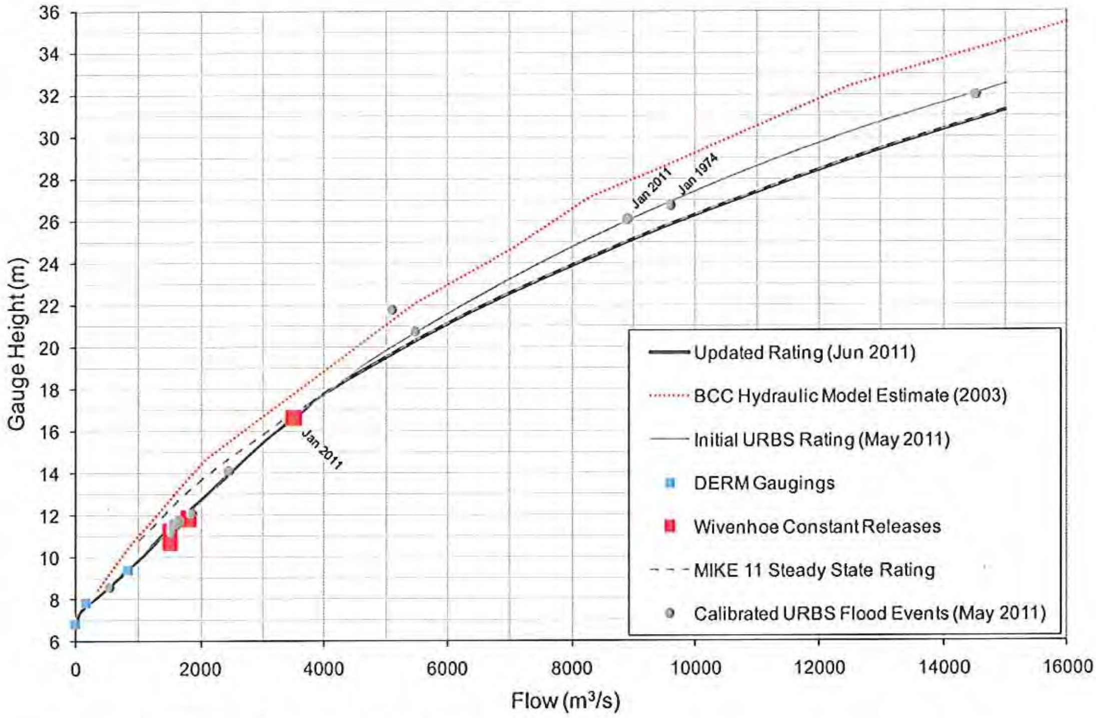


Figure 5-3: Rating Curve at Mt Crosby

At Mt Crosby, the MIKE 11 steady state results show that the higher flows had previously been underestimated. The previous estimate of peak flow in the January 2011 flood at this gauge was approximately 9,000 m^3/s; however using the rating curve from MIKE 11, this peak flow would be revised up to 9,800 m^3/s. At lower flows, the MIKE 11 steady state results provide a relatively good match to available DERM gaugings and also the constant Wivenhoe Dam releases. The updated rating curve was comprised mainly of the MIKE 11 results, adjusted slightly for flows less than 4,000 m^3/s to provide a better fit to the known low flow points.

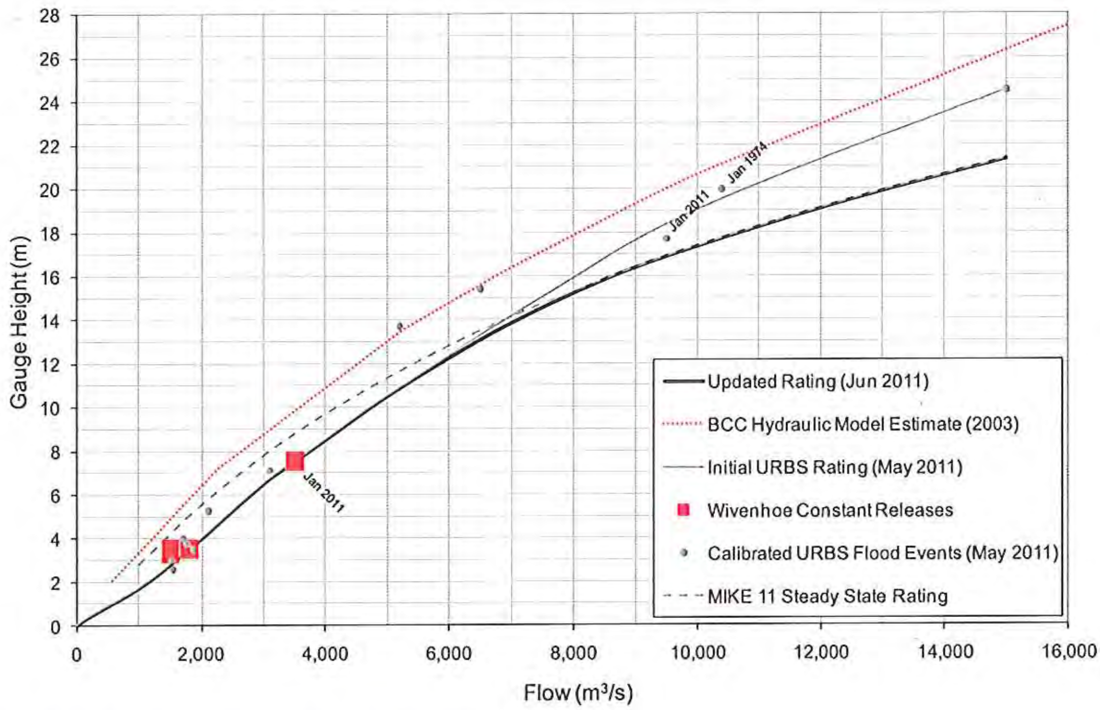


Figure 5-4: Rating Curve at Moggill

The rating curve at Moggill is similar to that shown for Mt Crosby, where again the MIKE 11 results show that the initial URBS rating was underestimating flows greater than approximately 7,000 m^3/s. The peak flow for the January 2011 flood event at Moggill, estimated using the initial URBS rating, is 9,200 m^3/s. Using the updated rating this increases to 10,400 m^3/s, an increase of approximately 13%. The updated rating curve at this site is composed of the initial URBS rating for flows less than 8,500 m^3/s and the MIKE 11 rating for larger flows.

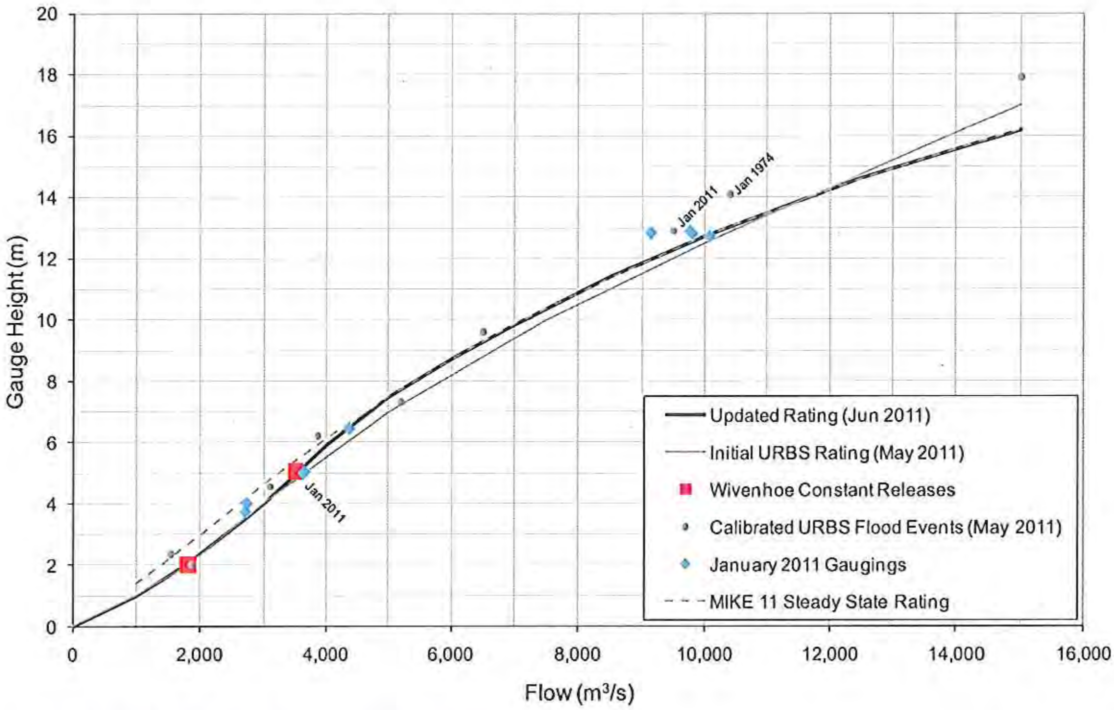


Figure 5-5: Rating Curve at Jindalee

The rating curve at Jindalee provides the best evidence that the MIKE 11 model is able to replicate the hydraulic conditions of the lower Brisbane River for high flows. It can be seen that the steady state MIKE 11 results fit very well to both the constant Wivenhoe releases and also the available DERM/Seqwater gaugings taken during low and high flows in January 2011. These gaugings are regarded as the only reliable gauged high flow information on the Brisbane River. The updated rating curve was adopted directly from the MIKE 11 results for flows greater than 5,000 m^3/s.

5.4. Implications

The updated rating curves have significant implications on the previous understanding of peak flows along the Brisbane River during large floods such as the January 2011 event and the January 1974 event. Initial and revised peak flows for both flood events are shown in Table 5-1 and Table 5-2. It is noted that the estimated flow at Moggill appears inconsistent with the values from the other sites. The Moggill gauge site is located very close to the confluence of the Bremer and Brisbane Rivers. Although the hydraulics of the Brisbane River have been reviewed and updated as part of this project, those of the Bremer River have not, and it is believed that there may be some local hydrodynamics that are not being correctly simulated in the model. For this reason, the rating and flows estimated at the Moggill site are considered to be less reliable.

Table 5-1: Initial and Updated Peak Flows for January 2011 Flood

Gauge	Recorded Level (m AHD)	Initial Peak Flow Estimate (m^3/s)	Updated Peak Flow Estimate (m^3/s)	Change (%)
Lowood Pump Station	46.47	9,700	8,800	-9%
Savages Crossing	42.58	10,100	9,900	-2%
Mt Crosby	26.12	9,000	9,800	+9%
Moggill	17.72	9,200	10,400	+13%
Jindalee	12.90	10,400	10,200	-2%

Table 5-2: Initial and Updated Peak Flows for January 1974 Flood

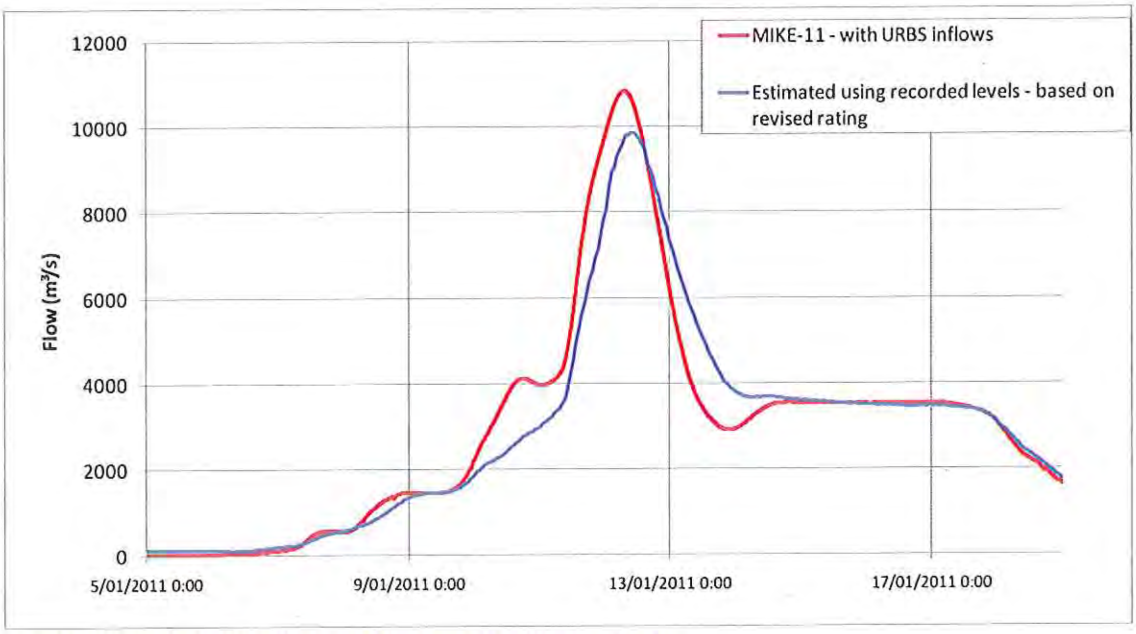
Gauge	Recorded Level (m AHD)	Initial Peak Flow Estimate (m^3/s)	Updated Peak Flow Estimate (m^3/s)	Change (%)
Lowood	45.70	9,520	8,600	-10%
Savages Crossing	42.22	11,500	11,200	-2%
Mt Crosby	26.74	9,500	10,400	+10%
Moggill	19.95	10,900	13,100	+21%
Jindalee	14.10	11,800	11,800	0%

6. Calibration of Hydrodynamic Model to January 2011 Event

6.1. January 2011 Inflows

The inflows to the hydrodynamic model for the January 2011 event were taken from the results of the URBS modelling. As discussed in Section 3.4 both URBS and WT42 gave similar results for the 2011 event and therefore the hydrodynamic modelling is not considered to be sensitive to the selection of either model.

It was found that when the URBS inflows were entered into the MIKE 11 model, the flow estimated in the upper reaches of the model did not adequately match the estimated flow from the gauges. An example of the match is shown in Figure 6-1 at Mt Crosby Weir. The URBS inflows result in a higher flow at the start of the hydrograph, and lower flows on the falling limb. Although these differences are relatively modest, this discrepancy was propagated downstream and resulted in poor calibrations to the recorded levels at the key sites along the Lower Brisbane River.



■ **Figure 6-1: Comparison of flows at Mt Crosby Weir.**

To better understand the difficulties with this reconciliation, the flow contribution between Wivenhoe Dam and Mt Crosby Weir was calculated as the difference between the estimated gauged flows at Mt Crosby Weir and the releases from Wivenhoe Dam (see Figure 6-2). This hydrograph is compared to the estimated contribution from the URBS model in Figure 6-3. This shows that interstation flow estimated from URBS (orange line) has a first peak around the 10th January 2011, which is caused by the flows in Lockyer Creek, that is not represented in the recorded flows. The second peak is also too high and the maximum flow from the third peak matches well, but the flow does not last long enough. The volume beneath both hydrographs is similar.

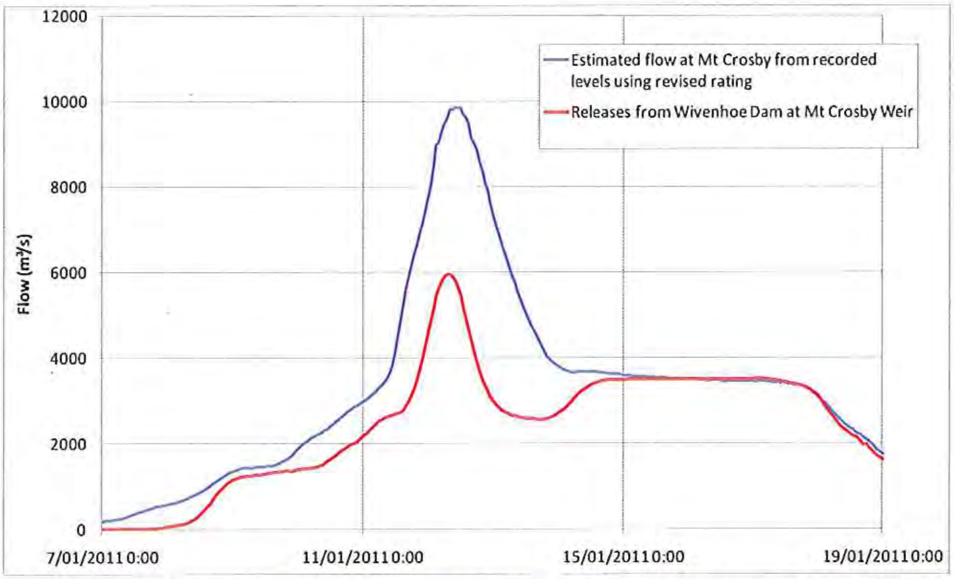
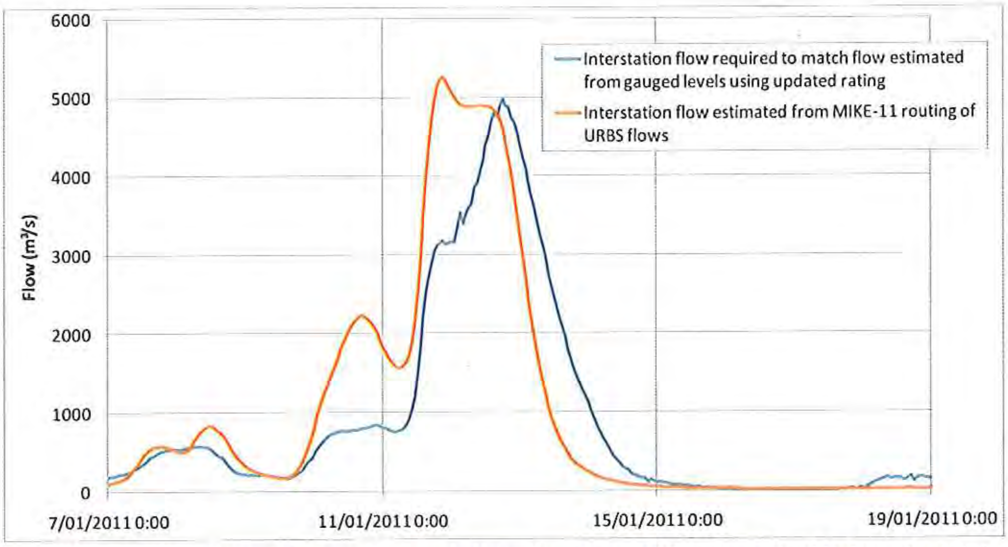


Figure 6-2: Flow hydrographs at Mt Crosby Weir based on releases from Wivenhoe Dam only, and estimated total flow using the updated rating.



- **Figure 6-3: Comparison of interstation flow from Wivenhoe to Mt Crosby Weir through simple subtraction (blue) and MIKE 11 routing with URBS inflow (orange).**

The difficulties in calibrating the MIKE 11 model to recorded levels in the upper reach of the model using the URBS inflows is likely to be due to the poor representation of Lockyer Creek in the MIKE 11 model. It was found that the cross-sections in this part of the model do not extend far beyond the creek channel, but they could not be extended as part of this project as LiDAR data was not available in the area. The lower end of Lockyer Creek is very flat, and it is known that levels in the creek are influenced by the levels in the Brisbane River. It is therefore believed that the runoff from Lockyer Creek that occurred around the 10th of January is likely to have been significantly attenuated before entering the Brisbane River.

The contribution of flows from upstream of Mt Crosby Weir was therefore calculated using the new rating curve at Mt Crosby Weir. All other model inflows were adopted from the calibrated URBS model.

6.2. Calibration methodology

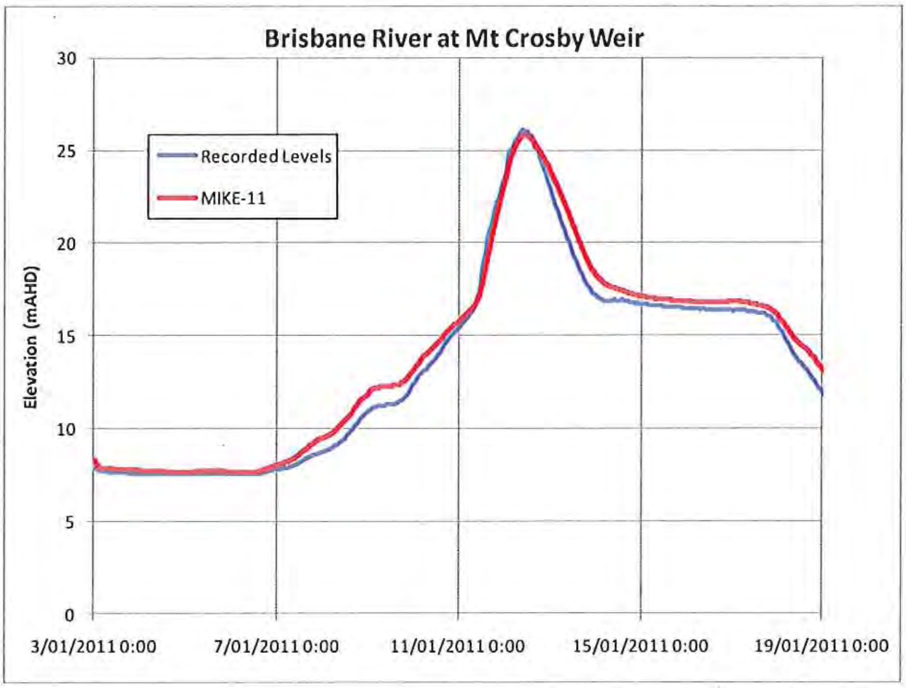
In summary, the strategy for calibrating the hydrodynamic model to the January 2011 event was to:

- adjust the roughness in the hydrodynamic model so that the rating curve at Jindalee gauge was consistent with the steady state flows and gaugings taken at Jindalee Bridge during the January 2011 event;
- use the hydrodynamic model to derive a rating curve at Mt Crosby Weir and then convert the recorded levels to estimated flows;

- estimate the inflow upstream of Mt Crosby Weir as the difference between the estimated flows at Mt Crosby Weir and the release from Wivenhoe Dam; and,
- estimate the inflow downstream of Mt Crosby Weir using the calibrated URBS model.

6.3. Comparison of modelled and recorded levels at key sites

The resulting calibration of the hydrodynamic model to the January 2011 event is shown in Figure 6-4 to Figure 6-9. The MIKE 11 model produced excellent calibrations to all gauges on the Brisbane River with the exception of Moggill. As mentioned in Section 5.4, the hydrodynamic processes at Moggill are likely to be affected by the Bremer River reach which has not been reviewed as part of this project. For this reason, the local water level results near Moggill are considered to be less reliable, particularly for low flows. It is also apparent that the MIKE 11 model under predicts water levels at the mouth of Oxley Creek. This is likely to be the result of a local channel control (for example the Green Bridge Busway at St Lucia) that has not been captured in the model due to insufficient data.



- **Figure 6-4: Calibration of MIKE 11 hydrodynamic model to recorded levels at Mt Crosby Weir during the January 2011 event.**

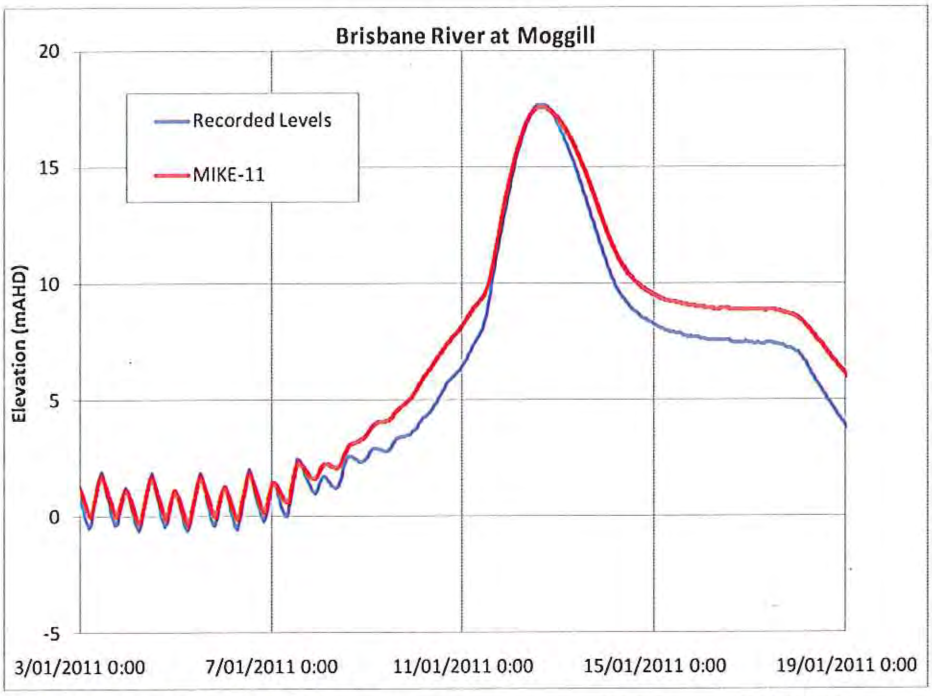


Figure 6-5: Calibration of MIKE 11 hydrodynamic model to recorded levels at Moggill during the January 2011 event.

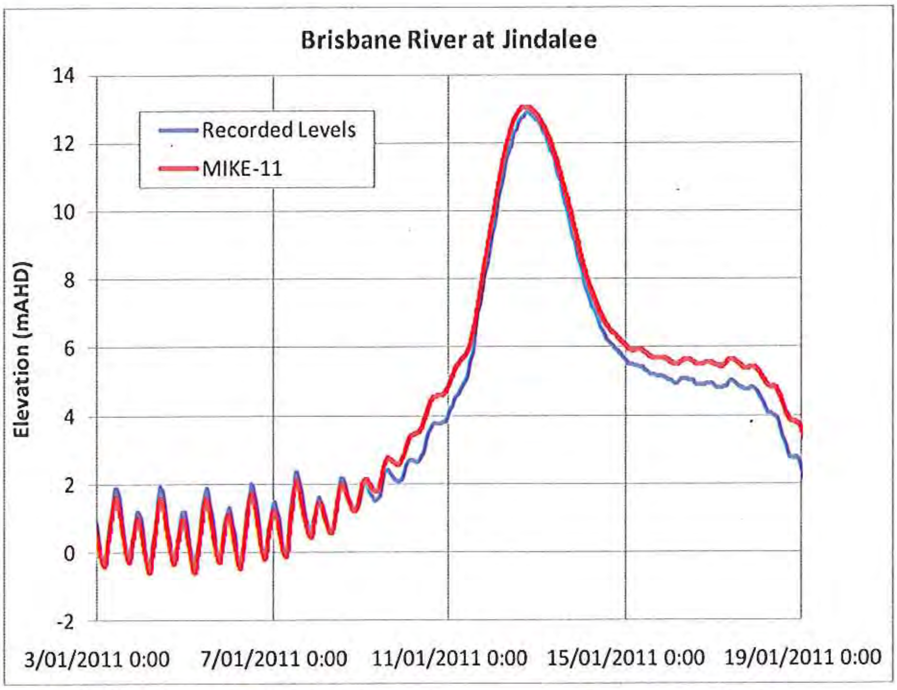
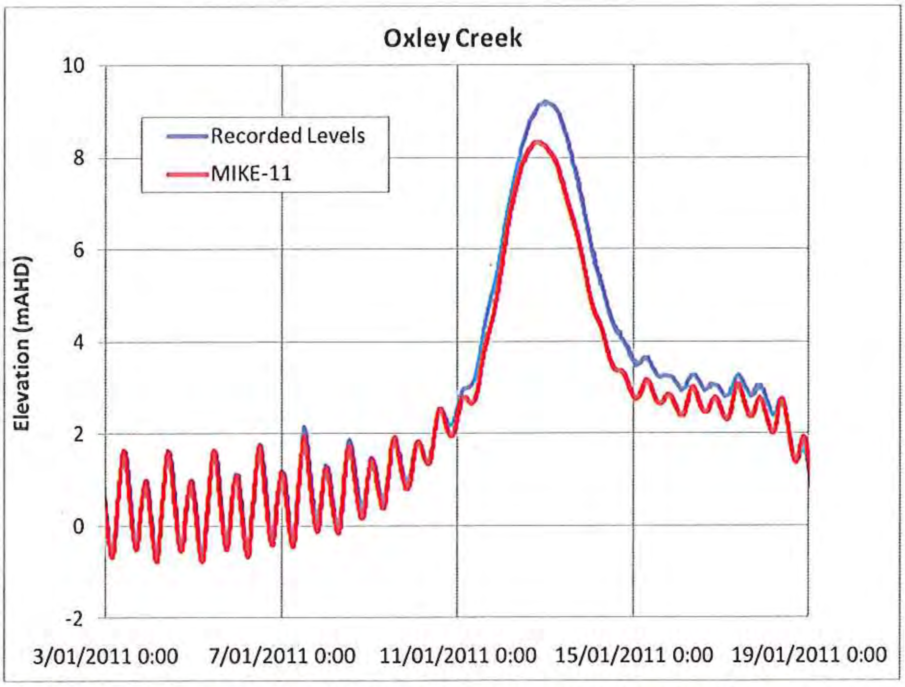
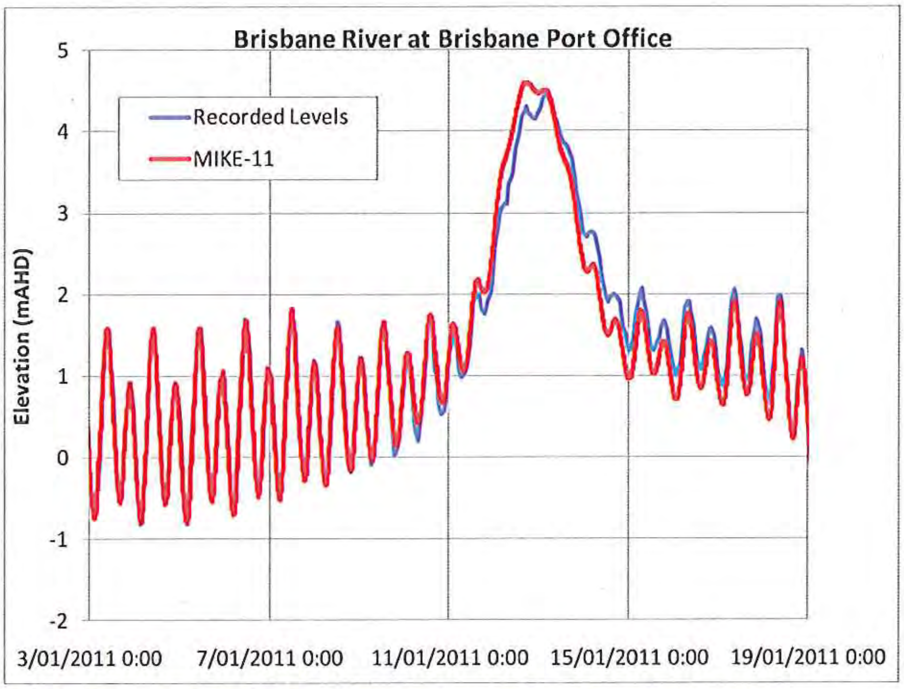


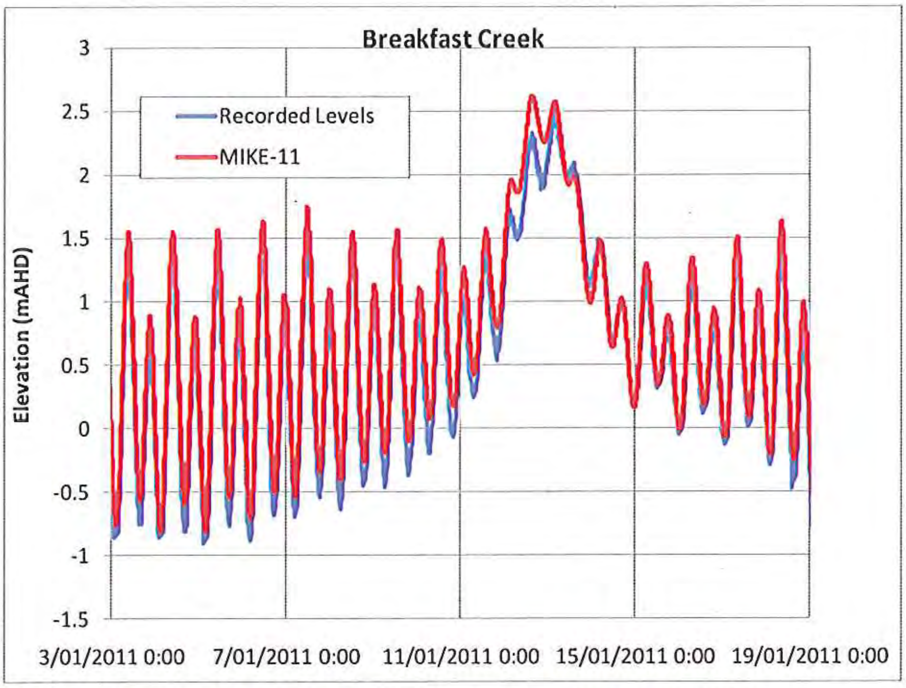
Figure 6-6: Calibration of MIKE 11 hydrodynamic model to recorded levels at Jindalee Gauge during the January 2011 event.



- **Figure 6-7: Calibration of MIKE 11 hydrodynamic model to recorded levels at Oxley Creek Mouth during the January 2011 event.**



- **Figure 6-8: Calibration of MIKE 11 hydrodynamic model to recorded levels at Brisbane River Port Office during the January 2011 event.**



- **Figure 6-9: Calibration of MIKE 11 hydrodynamic model to recorded levels at Breakfast Creek during the January 2011 event.**

7. Scenario Modelling

The calibrated MIKE 11 hydrodynamic model has been used to provide updated results for the 5 cases from the Seqwater report entitled *January 2011 Flood Event: Report on the operation of Somerset Dam and Wivenhoe Dam*. The scenarios are summarised in the Table 7-1 below.

- **Table 7-1: Scenarios from Seqwater (2011) report.**

Case Number	Case Description
1	Actual Wivenhoe Dam outflows combined with Lockyer Creek, Bremer River and other non-controlled catchment flows from the January 2011 Flood Event
2	Lockyer Creek, Bremer River and other non-controlled catchment flows from the January 2011 Flood Event only.
3	Actual Wivenhoe Dam outflows from the January 2011 Flood Event only.
4	Assumes Wivenhoe Dam removed and uses estimated flows in the Brisbane River at the location of Wivenhoe Dam combined with Lockyer Creek, Bremer River and other non-controlled catchment flows from the January 2011 Flood Event. This case provides an indication of the impacts of the January 2011 Flood Event at Brisbane City if Wivenhoe Dam had not been constructed.
5	Assumes both Wivenhoe Dam and Somerset Dam removed and uses estimated flows in the Brisbane River at the location of Wivenhoe Dam combined with Lockyer Creek, Bremer River and other non-controlled catchment flows from the January 2011 Flood Event. This case provides an indication of the impacts of the January 2011 Flood Event at Brisbane City if both Somerset Dam and Wivenhoe Dam had not been constructed.

The calibration of the model to the January 2011 event (see Section 6) provides confidence in the ability of the model to replicate a range of flow conditions up to the magnitude of the January 2011 event. Due to the physical representation of the river in the hydrodynamic model, its use is considered to provide superior predictions of water levels for higher flows than the hydrologic models (see Section 3), but it should be noted that the model results are particularly sensitive to:

- The tidal influence – the timing of the flood peak arriving results in significant differences in the water level results in the lower parts of the model;
- Structures – bridges have been removed from the model, and these are expected to influence water levels for higher flows; and,
- Overbank roughness – in some parts of the model, there was no overbank flow for the January 2011 event and so it was not possible to calibrate overbank roughness values, which introduces additional uncertainty when the model is used to estimate water levels for higher flows.

In addition, the choice of radius type for the cross-sections (see Section 4.3.7) also has an increasing impact on results for higher flows, where more significant overbank flow zones are activated. This means that the results for cases 4 and 5 are inherently more uncertain. A comparison of flow and water level hydrographs is provided in the plots below (Figure 7-1 and Figure 7-2, respectively), and a summary of the peak flows and water levels is provided in Table 7-2.

The results of the hydrodynamic modelling confirm the following conclusions in the Seqwater report:

- Even if the flood flows in the Stanley River and upper Brisbane River had been contained, and there were no releases from Wivenhoe Dam (Case 2), the flows from Lockyer Creek, Bremer River and other uncontrolled catchment flows would still have exceeded the threshold of urban damage;
- If there had not been any flows from Lockyer Creek, Bremer River and the other uncontrolled catchments, the actual releases from Wivenhoe Dam (Case 3) would have caused only minor flooding in Brisbane City.

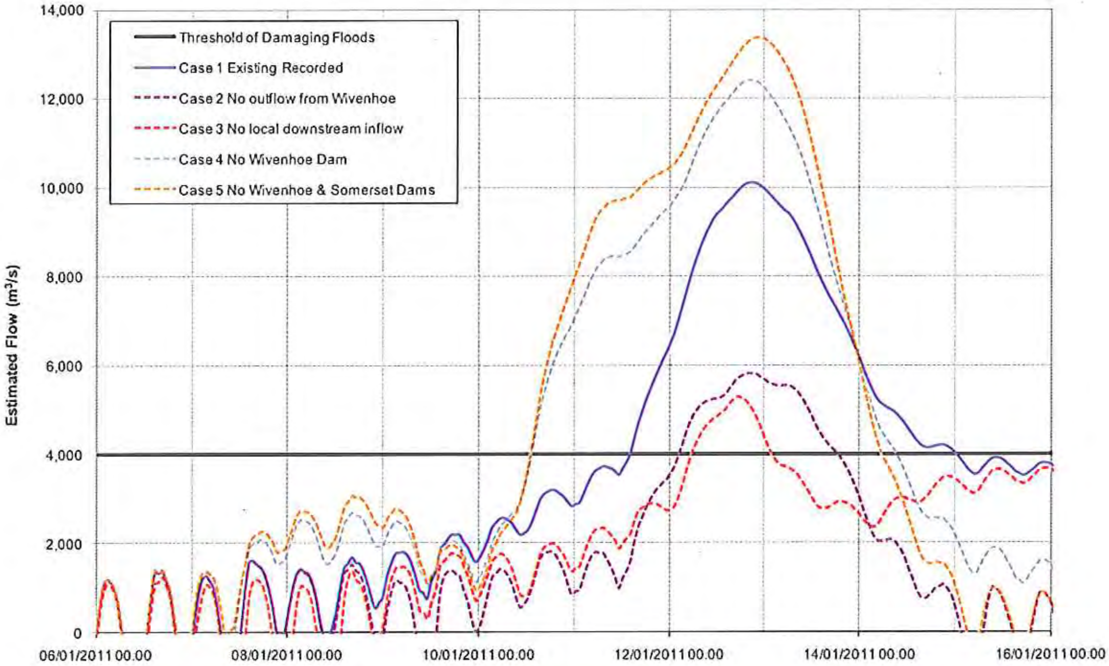
The hydrodynamic modelling provides updated results for the last two conclusions in the Seqwater report which were based upon the preliminary hydrologic modelling, namely:

- Without Wivenhoe Dam (Case 4), the peak flow would have been in the order of ***12,400*** m³/s and the peak height would have been in the order of ***1.1*** metres higher at Brisbane City;
- Without Somerset and Wivenhoe Dams (Case 5), the peak flow would have been of the order of ***13,400*** m³/s and the peak height would have been approximately ***1.4*** metres higher at the Port Office gauge.

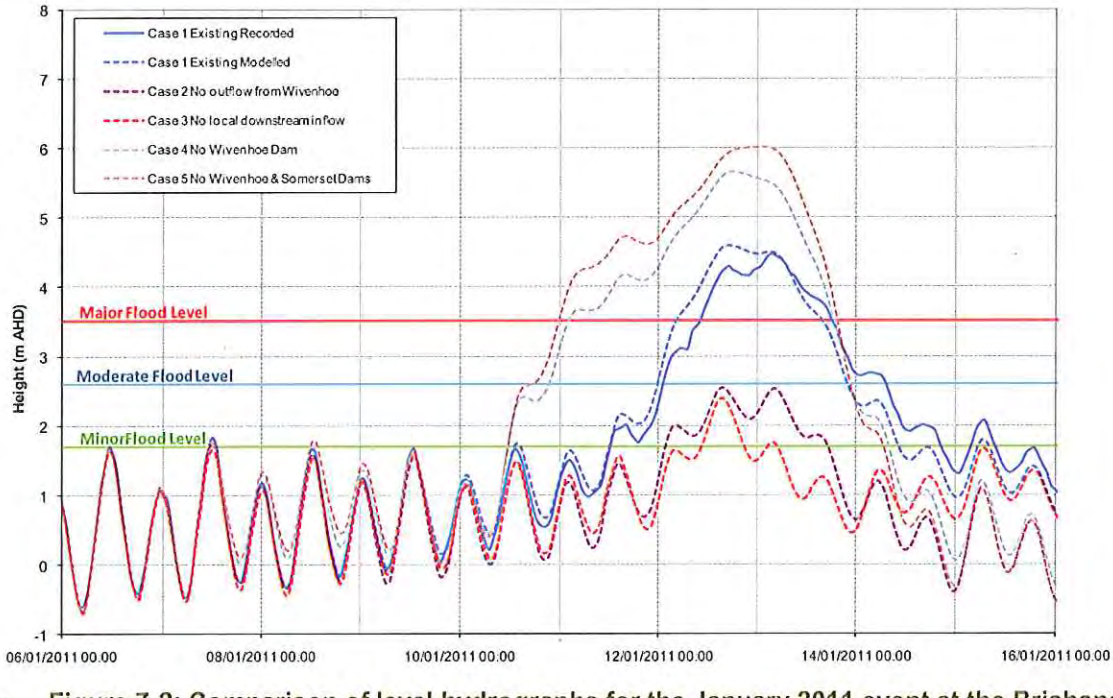
- **Table 7-2: Comparison of peak flow and water level estimates for the January 2011 event at the Brisbane Port Office under different scenarios.**

Case	Peak Flow (m³/s)		Peak Water Level (mAHD)	
	Hydrologic Model (March 2011)	Hydrodynamic model (June 2011)	Hydrologic Model (March 2011)	Hydrodynamic model (June 2011)
Case 1 Existing	9,400	10,100	4.5	4.6
Case 2 No releases from Wivenhoe	6,300	5,800	2.7	2.5
Case 3 Wivenhoe releases only	5,200	5,300	2.2	2.4
Case 4 No Wivenhoe Dam[1]	12,900	12,400	6.4	5.7
Case 5 No Somerset Dam or Wivenhoe Dam[1]	14,000	13,400	7.0	6.0

[1] The results for these scenarios are inherently more uncertain as they are for flows outside of the calibration range.



■ **Figure 7-1: Comparison of flow hydrographs for the January 2011 event at the Brisbane Port Office under different scenarios.**



■ **Figure 7-2: Comparison of level hydrographs for the January 2011 event at the Brisbane Port Office under different scenarios.**

8. Summary and Conclusions

8.1. Uncertainty and Limitations

The following uncertainties and limitations should be noted:

- It is important to appreciate that the MIKE 11 hydrodynamic model was developed for flood modelling purposes where the focus of attention is on the simulation of flood similar in magnitude to the January 2011 event; if the MIKE 11 model is used for purposes other than that for which it was intended it should be used with caution.
- The work undertaken as part of this study has been primarily concerned with estimating peak water levels in the Lower Brisbane River. During times of heavy rainfall localised flooding will also occur when the capacity of drainage infrastructure is exceeded and the MIKE 11 model is not suited to investigate such issues, as one of its primary assumptions is that all storm runoff will enter the river system.
- The channel cross sections have been developed using data within the provided MIKE 11 model. Whilst the model has been successfully calibrated and verified some effort should be made to review the appropriateness of these sections through additional survey, particularly as channel conveyance may have changed as a result of the January 2011 flood. A survey would serve to ensure the MIKE 11 model is representative and that ongoing stream flow gauging is reliable.
- The LiDAR data provided and used for the development of the model is of 1 m resolution and has an accuracy of around +/-0.15m. This may contain localised inaccuracies due to the presence of vegetation and buildings where as part of the data retrieval process the LiDAR laser strikes may have not reached the true ground surface and may affect the components which have been built as part of this study.
- The Brisbane River with its tidal influence, number of severe meanders, and anthropogenic influences such as releases from reservoirs located within the catchment is a highly turbulent environment which causes a number of channels to continually change in size and shape. Although for the severity of events considered during this study these changes would more than likely not be influential due to volumes of flow, during more frequent and less severe events such changes might be significant.
- As part of the model development all structures on the Brisbane River were removed apart from the Mount Crosby bridge/stream flow gauge. This will affect the local dynamics, especially in low flow events, and should be included in the MIKE 11 model if developed any further in the future.
- There are known limitations in the representation of Lockyer Creek and the Bremer River in the model. Model results from these reaches should not be relied upon.

8.2. Conclusions

The study has analysed the recorded rainfall and river level data available for the January 2011 event, and used this to calibrate the URBS and WT42 hydrologic models and an improved MIKE 11 hydrodynamic model of the Lower Brisbane River.

The rainfall information used in initial URBS hydrologic modelling undertaken by Seqwater immediately after the event was reviewed. This identified that the majority of rainfall inputs were appropriate however the review identified a small number of stations for which the data appeared suspect and these stations were removed, and also identified where additional data was now available from the Bureau of Meteorology. The sensitivity analysis undertaken to the selection of the rainfall stations demonstrated that the estimated rainfall depths at the model subareas was not sensitive to a slight increase in the number (from 4 to 6) of sites used in the analysis.

Although a large number of river gauges are located within the Brisbane River catchment below Wivenhoe Dam, most of these gauges do not have reliable rating tables (which relate the recorded level to flow). In lieu of this information, preliminary rating curves had previously been derived which related the recorded level to the flow modelled using an appropriately configured URBS hydrologic model for a range of historic flood events. Although such preliminary ratings are useful for making inferences regarding modelled levels they do not provide any additional information on which to calibrate a hydrologic model.

This means that during the January 2011 event, there was little reliable information available to calibrate modelled flows from the hydrologic models downstream of Wivenhoe Dam. This was exacerbated by a number of stations that failed during the event. Thus, for the 2011 event only 40% of the 6,515 km² catchment downstream of Wivenhoe Dam was covered by streamflow gauges that had recorded levels with a reliable rating curve. This reinforces the need to use a hydrologic model in order to estimate flows from the significant proportion of the catchment which is not covered by reliable recorded information.

A MIKE 11 hydrodynamic model was used to model the Lower Brisbane River. The model available for use on this project was that used by the Wivenhoe Alliance in 2005 to model the impacts of different upgrade options for Wivenhoe Dam. The model was reviewed for the purposes of this investigation and a number of deficiencies were noted relating to the schematisation, calibration and stability which meant that it was not suitable for modelling the 2011 event.

The MIKE 11 model was enhanced by using LIDAR data to extend and add cross-sections, lateral storages, link channels, bend losses and weirs. It is considered that the revised model provides a robust platform for investigating the hydraulic characteristics of the Lower Brisbane River.

The MIKE 11 model was run in steady state to derive rating curves for sites along the Lower Brisbane River. These ratings are consistent with gaugings undertaken at Jindalee Bridge during the 2011 event and also with the recorded river levels during periods of constant releases from Wivenhoe Dam.

This analysis shows that the initial rating curves developed using the URBS hydrological model were generally appropriate at the majority of locations along the Lower Brisbane River. However, at some locations (particularly Mt Crosby and Moggill) the initial URBS rating significantly underestimated the peak flow for the January 2011 event. It is estimated that the peak flow at Brisbane Port Office during the January 2011 event was approximately 10,100 m³/s. These new ratings provide an opportunity to refine the calibration of hydrologic models.

When flows from the URBS hydrologic model were included in the MIKE 11 model, it was found that the flows estimated at each the key gauge locations downstream of Savages Crossing using the revised rating curves could not be reproduced. The cause of this is likely to be due to poor representation of the Lockyer Creek in the MIKE 11 model which results in lower attenuation than actually occurred and/or the gaps in the available rainfall network that inadequately captured the intense rainfall that occurred in the vicinity of Mt Glorious. To ensure that the MIKE 11 model calibration is not hindered by uncertainty regarding inflows in the upper part of the catchment, the contribution of flows from upstream of Mt Crosby Weir were back-calculated from the flow derived using the new rating curve at Mt Crosby Weir and the outflows from Wivenhoe Dam.

The MIKE 11 model produced excellent calibrations to all gauges on the Brisbane River with the exception of Moggill and Oxley Creek (where the calibration is only fair). The calibrations provide only a slight improvement on the initial calibrations using the URBS hydrologic model, though the physical basis of the MIKE 11 hydrodynamic model gives greater confidence in extrapolating the model outside the range of calibration and hence for assessing the implications of different operating strategies.

The calibrated hydrodynamic model was used to update the preliminary modelling in Seqwater (2011b) which was undertaken using an URBS hydrologic model. The results of the hydrodynamic modelling confirm the following conclusions in the Seqwater report:

- Even if the flood flows in the Stanley River and upper Brisbane River had been contained, and there were no releases from Wivenhoe Dam (Case 2), the flows from Lockyer Creek, Bremer River and other uncontrolled catchment flows would still have exceeded the threshold of urban damage; and,
- If there had not been any flows from Lockyer Creek, Bremer River and the other uncontrolled catchments, the actual releases from Wivenhoe Dam (Case 3) would have caused only minor flooding in Brisbane City.

The hydrodynamic modelling provides updated results for the last two conclusions in the Seqwater report, namely:

- Without Wivenhoe Dam (Case 4), the peak flow would have been in the order of ***12,400*** m^3/s and the peak height would have been in the order of ***1.1*** metre higher at Brisbane City; and,
- Without Somerset and Wivenhoe Dams (Case 5), the peak flow would have been of the order of ***13,400*** m^3/s and the peak height would have been approximately ***1.4*** metres higher at the Port Office gauge.

8.3. Recommendations

The study has the following recommendations:

- Obtain LIDAR in the lower Lockyer Catchment to allow refinement of MIKE 11 to better represent the routing in the lower reaches of Lockyer catchment.
- Refine and improve the MIKE 11 model of the lower reaches of the Bremer and Lockyer catchments
- Use hydrodynamic models to derive rating curves for key gauges in the tributaries to the Brisbane River to inform the calibration of hydrologic models. The URBS and MIKE 11 models developed as part of this study should be calibrated/verified against further events over a range of flood magnitudes to improve the confidence in the modelled peak water levels.
- The MIKE11 model is currently based on a combination of surveyed cross-sections supplemented with LiDAR data. Given that this survey was collected prior to the January 2011 flood, it is possible that the magnitude of that flood has resulted in significant morphological change to the bed and bank shape of the river at key locations. It is recommended that consideration be given to updating the available survey data to ensure that the model reflects any alterations to the river bathymetry.
- If a hydrodynamic model is to be used for flood forecasting then careful consideration should be given to whether the model should be 1 or 2 dimensional or a coupled or linked 1D/2D model; or fully integrated hydrological and hydrodynamic models.

9. References

DHI, 2009. *MIKE 11: A Modelling System for Rivers and Channels – Reference Manual.*

Malone, T., 1999. *Using URBS for Real Time Modelling.* 25th Hydrology and Water Resources Symposium, Brisbane, 1999.

Chow, V.T., 1959. *Open-Channel Hydraulics.* International Student Edition. McGraw Hill Kogakusha, Ltd. 1959.

Seqwater, 2011a. *January 2011 Flood Event: Report on the Operation of Somerset and Wivenhoe Dam.* 2 March 2011.

Seqwater, 2011b. *Calibration of the Lower Brisbane River Hydrologic Model.* Internal Report. May 2011.

South East Queensland Water Board and Natural Resources Queensland, 1992. *Brisbane River Flood Hydrology Report: Report on Runoff-Routing Model Calibration (Volume I).* Brisbane River and Pine River Flood Studies. September 1992.

South East Queensland Water Board and Natural Resources Queensland, 1993. *Brisbane River Flood Hydrology Report: Design Flood Estimation for Somerset Dam and Wivenhoe Dam (Volume I).* Brisbane River and Pine River Flood Studies. March 1993.

Appendix A Summary of Hydrologic Data

A.1 Rainfall gauges investigated

No.	Name	Issue	Findings	Removed
541057	Mt Pechy AL	No files, even though ref in report	Upstream of Wivenhoe, so no need to include in modelling	NA
540168	Kluvers Lkt AL	No files, even though ref in report	Upstream of Wivenhoe, so no need to include in modelling	NA
540189	Baxters Ck AL	No files, even though ref in report	Upstream of Wivenhoe, so no need to include in modelling	NA
540207	Wilsons Peak AL-P	No files, even though ref in report	Only 1 data point, and daily data received from BoM under gauge 40876	Pluvio
40020	BLACKBUTT POST OFFICE	Same name as 540493	Same location as 540493	Daily
40024	BOONAH STARK AVE	Commented out	No rainfall recorded, no data available from BoM	Daily
40028	BROOWEENA LAHEY ST	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40060	COOYAR POST OFFICE	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40063	DAYBORO POST OFFICE	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40069	DUCKINWILLA CREEK	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40100	IMBIL FORESTRY	Commented out	No files supplied by SEQW, BoM data available	
40109	KIA ORA SANDY RIDGES	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40110	KILCOY POST OFFICE	Recorded 67.8mm	BoM data has lots of accumulated data, but looks like about 563mm recorded.	Daily
40113	KUMBIA POST OFFICE	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40118	LITTLE YABBA SFR	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40135	MOOGERAH DAM	Same name as 540474	Same location as 540474	Daily
40152	MURGON POST OFFICE	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily

No.	Name	Issue	Findings	Removed
40171	AMCOR PETRIE MILL	Commented out	No files supplied by SEQW, BoM data available	
40188	SIM JUE CREEK	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40196	TALLEBUDGERA GUINEAS	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40212	EAGLE FARM RACECOURS	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40245	TOOWONG BOWLS CLUB	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40246	WARRAGAI	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40255	WOOROOLIN POST OFFIC	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40289	EUMARELLA	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40310	MT BERRYMAN	Commented out	No files supplied by SEQW, BoM data available but not quality controlled	
40329	ATKINSONS DAM	Same name as 540479	Same location as 540479	Daily
40343	WAMURAN	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40394	MOUNT BARNEY	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40406	BEENLEIGH BOWLS CLUB	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40413	CENTRAL KERRY	Commented out	No files supplied by SEQW, BoM data available	
40416	CLEARVIEW TM	Same name as 40846	Same location as 40846	Daily
40424	WEST HALDON	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40440	KALBAR	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40460	MOUNT COTTON FARM	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40478	FRASER ISLAND EURONG	Commented out	No files supplied by SEQW, data from BoM has lots of accumulations	Daily
40486	YABBA STATION	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40492	YIELO	Commented out	No files supplied by SEQW, BoM data available but not quality controlled	
40496	CALOUNDRA WTP	Commented out	No files supplied by SEQW, data from BoM has lots of accumulations	Daily
40503	TALLEGALLA ALERT	Pluvio or daily?	Pluvio data available	Daily

No.	Name	Issue	Findings	Removed
40525	KIAMBA	Commented out	No files supplied by SEQW, BoM data available but not quality controlled	
40534	WUNBURRA	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40537	DUNWICH	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40542	MACLEAN BRIDGE	Same name as 40935	Same location as 40935	Daily
40558	GLENGAVEN	Commented out	No files supplied by SEQW, no data available from BoM	Daily
40583	WIDGEE	Commented out	No rainfall recorded, no data available from BoM	Daily
40606	UPPER MUDGEERABA WAT	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40686	BEENHAM VALLEY RD	Commented out	No rainfall recorded, BoM data available but not quality controlled	
40714	ROUND MOUNTAIN TM	Same name as 40945	Same location as 40945	Daily
40762	YARRAHAPPINI TM	Same name as 40940	Same location as 40940	Daily
40770	ORMISTON COLLEGE	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40784	CALAMVALE ALERT	Pluvio or daily?	Pluvio data available	Daily
40785	CAROLE PARK ALERT	Pluvio or daily?	Pluvio data available	Daily
40786	JINGLE DOWNS ALERT	Pluvio or daily?	Pluvio data available	Daily
40788	JOHNSON RD FORESTDAL	Commented out, pluvio or daily?	Pluvio data available, but suspicious	Daily and Pluvio
40790	Mt Gravatt AL	Commented out, pluvio or daily?	Pluvio data available, but suspicious	Daily and Pluvio
40792	RIPLEY ALERT	Commented out, pluvio or daily?	Pluvio data available, but suspicious	Daily and Pluvio
40793	LYONS ALERT	Commented out, pluvio or daily?	Failed during event	Daily and Pluvio
40794	THOMPSON RD GREENBAN	Pluvio or daily?	Pluvio data available	Daily and Pluvio
40795	OPOSSUM ALERT	Pluvio or daily?	Pluvio recorded 175.9mm – data from BoM website shows this is correct.	Daily

No.	Name	Issue	Findings	Removed
40808	CRESSBROOK DAM	Same name as 540142	Same location as 540142	Daily
40823	ROSENRETERS BRIDGE TM	Same name as 540148	Same location as 540148	Daily
40836	ONE MILE BRIDGE ALER	Pluvio or daily?	Pluvio data available	Daily
40839	Brisbane (Bcc) Alert	Commented out	No rainfall recorded	Pluvio
40841	CROFTBY TM	Recorded 11mm	No data available from BoM.	Daily
40867	KALBAR TM	Recorded 0mm	No data available from BoM.	Daily
40876	WILSONS PEAK ALERT	Pluvio or daily?	Pluvio out of action, but daily data available from BoM	Pluvio
40893	GOOMBOORIAN TM	Recorded 0mm	No data available from BoM.	Daily
40912	FRANKLYN VALE ALERT	Commented out, pluvio or daily?	Suspicious.	Daily and Pluvio
40914	MT TARAMPA	Recorded 205.7mm	Data from BoM website (not quality controlled) shows 643.8mm recorded. Inputs revised.	
40922	KINGAROY AIRPORT	Commented out	No rainfall recorded, no data available from BoM	Daily
40955	SPRINGVALE	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40960	CLEAR MOUNTAIN BURAN	Commented out	No rainfall recorded, quality checked data available from BoM	
40962	EBBW VALE	Commented out	No files supplied by SEQW, BoM data available but not quality controlled	
40963	FERNVALE BURNS ST	Low rainfall	Compared to other gauges close by, this appears to have not captured all of the rainfall	Daily
40977	SAMFORD KAY DRIVE	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40985	Bellbird Park AL	Pluvio or daily?	Pluvio data available	Daily
40991	ESK WHITE ROCK	Commented out	No rainfall recorded, data from BoM has lots of accumulations	Daily
40997	RUSSELL ISLAND	Commented out	No files supplied by SEQW, no data available from BoM	Daily
540059	PEACHESTER ALERT	Commented out	Not working	Pluvio
540065	PEAK CROSSING ALERT	Recorded 192	No data available from BoM. Assume correct.	
540101	Taringa Alert	Commented out	No files supplied by SEQW	Pluvio

No.	Name	Issue	Findings	Removed
540102	Indooroopilly Alert	Commented out	No files supplied by SEQW	Pluvio
540103	Morningside Alert	Commented out	No files supplied by SEQW	Pluvio
540125	Eight Mile Plains	Recorded 205.1	No data available from BoM. Assume correct.	
540135	Holland Pk AL	Commented out	No files supplied by SEQW	Pluvio
540140	GREGOR CK ALERT B	Out of action	Out of action and backup	Pluvio
540152	TENTHILL ALERT	Commented out	Not working	Pluvio
540159	SOMERSET DAM HW ALER	Commented out	Double counted rainfall	Pluvio
540164	TOP OF BRISBANE ALER	Pluvio or daily?	Pluvio data available	Daily
540175	LYONS BRIDGE ALERT B	Out of action from 09:00 11/01/2011	Data available through BoM Enviromon system. However, this is a backup and data was recorded at 540174, so not used.	Pluvio
540178	WIVENHOE DAM TW ALERT-P	Commented out	Keep in	
540179	WIVENHOE DAM TW ALERT-B	Commented out	Backup. Gauged data available at 540178.	Pluvio
540181	AMBERLEY ALERT B	Commented out	Backup. Gauged data available at 540180.	Pluvio
540182	LOWOOD ALERT P	Out of action from 15:00 11/01/2011	Data available through BoM Enviromon system. However, LOWOOD PUMP STN ALERT very close by, so not used.	Pluvio
540184	MT GLORIOUS ALERT B	Commented out	No longer exists	Pluvio
540194	KUSS ROAD ALERT	Out of action	Out of action	Pluvio
540195	WASHPOOL ALERT	Recorded 179	No data available from BoM. Assume correct.	
540196	WALLOON ALERT B	Commented out	Backup. Gauged data available at 540147.	Pluvio
540207	WILSONS PEAK ALERT P	Commented out	Did not work	Pluvio
540246	MT MEE ALERT P	Commented out	Backup. Gauged data available at 540185.	Pluvio
540249	HANLON ST BUNDAMBA A	Recorded 192	No data available from BoM. Assume correct.	
540298	Perseverance Alert	Commented out	Suspicious	Pluvio
540316	CHURCHBANK WEIR ALER	Commented out	Failed during event	Pluvio
540338	WOODFORD ALERT B	Commented out	Backup. Gauged data available at 540337.	Pluvio

No.	Name	Issue	Findings	Removed
540387	HARRISVILLE AL B	Same location as 540154	Comment out – recorded less rainfall than 540154	Pluvio
540388	ROSEWOOD ALERT B	Commented out	Backup. Gauged data available at 540193.	Pluvio
540456	MT ALFORD ALERT	Commented out	Not working	Pluvio
540458	HAYS LANDING ALERT	Commented out	Failed during event	Pluvio
540479	Atkinson Dam	Commented out	Failed during event	Pluvio
540486	WESTVALE AL	Commented out	Failed during event	Pluvio
540492	ESKDALE AL	Commented out	Yet to be installed	Pluvio

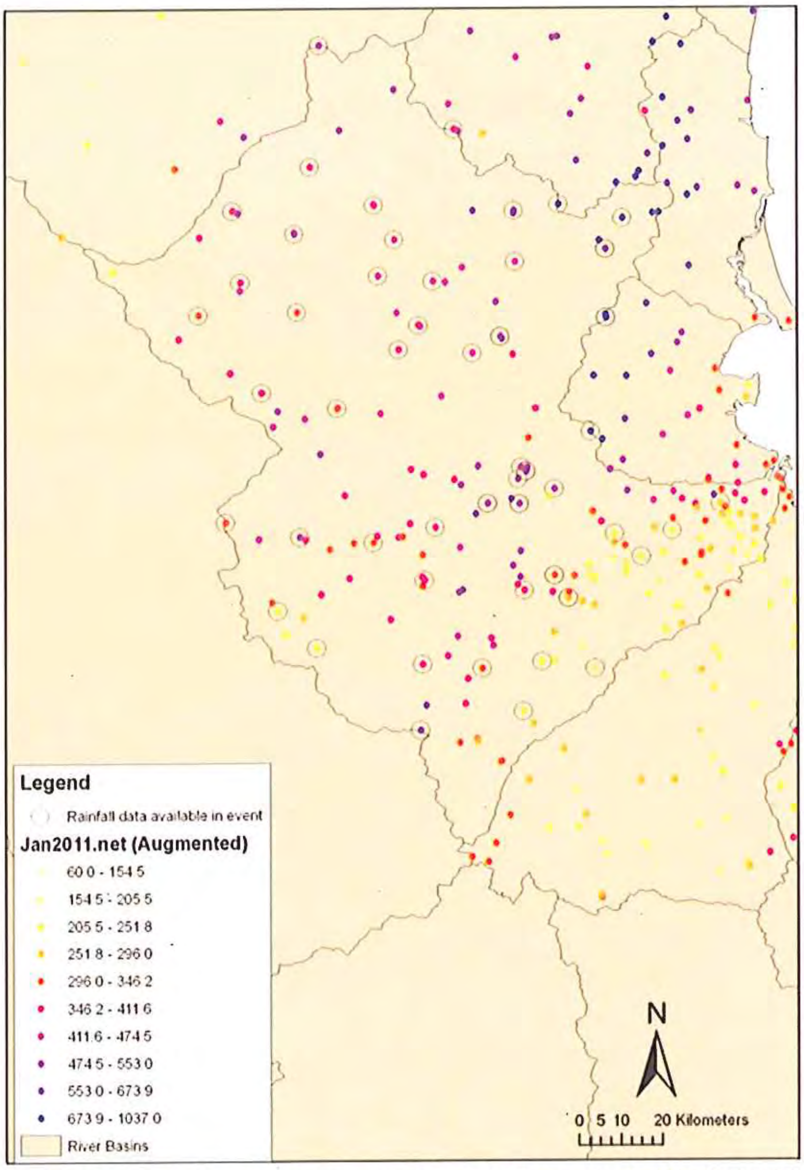
A.2 Sub-area rainfalls

The SUBRAIN utility weights the rainfall data at each of the stations based on the inverse square of the distance to the centroid of each sub-area. The user is able to specify how many of the closest stations should is used in this analysis, and the default value adopted historically for the Brisbane River catchment has been 4.

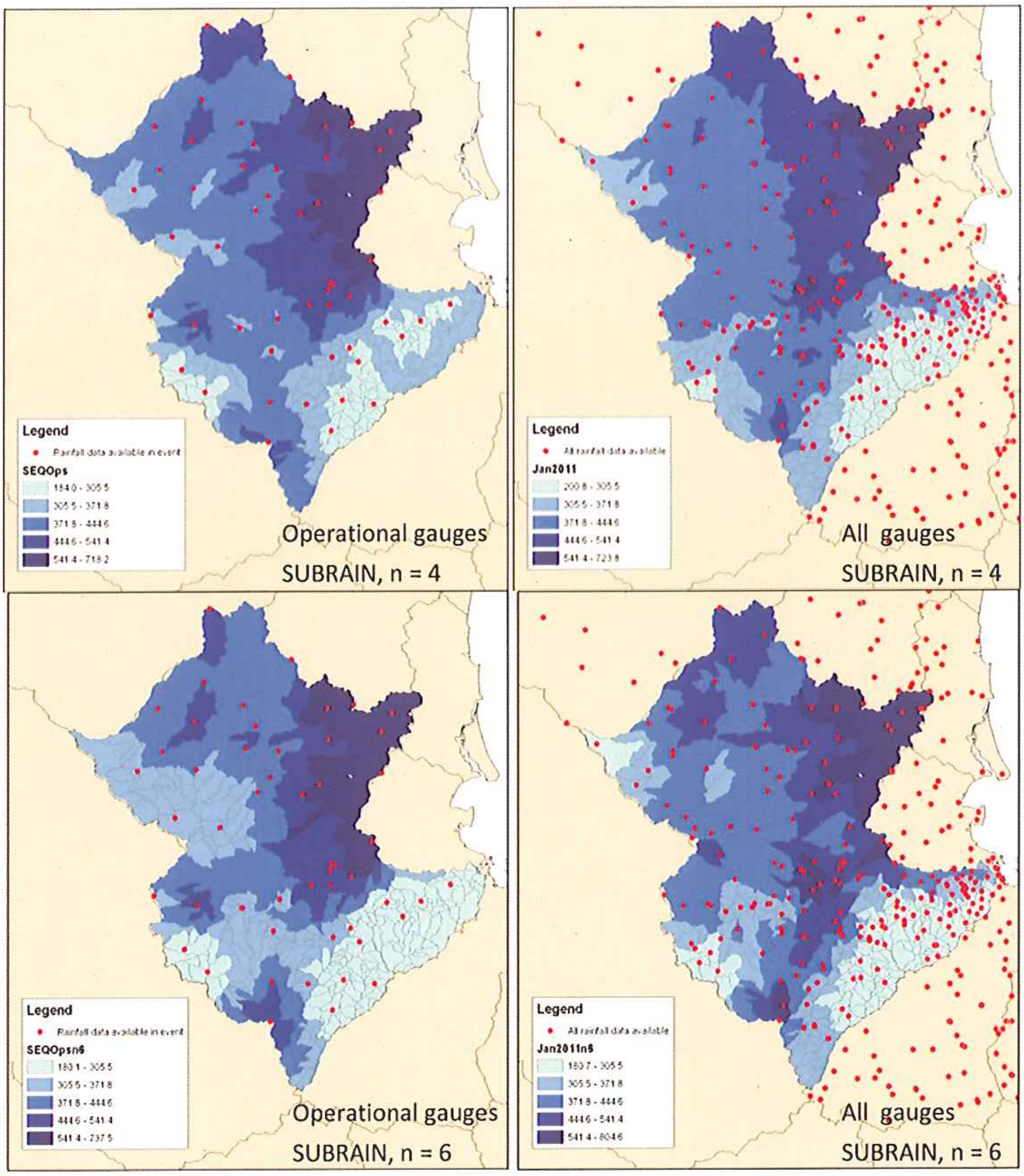
Figure A-1 compares the data rainfall available during the event (hollow black circles) to all of the data that is now available. It also displays the rainfall totals over the event (9am 2nd January to 9am 20th January 2011).

Using the two data sets shown in Figure A-1, the SUBRAIN utility was used to estimate catchment average rainfall depths over each of the URBS sub-areas, using the default of the closest 4 stations, as well as 6 stations. These are compared in Figure A-2 below. A comparison of the difference in the results when just operational or all available gauges is provided in Figure A-3. The results vary depending on the sub-catchment and the gauges available, but it shows that for the higher rainfalls recorded in the Somerset and Upper Brisbane River catchments, using only operational gauges results in higher sub-area rainfalls. This is consistent with the maps shown in Figure A-2. Figure A-4 shows that the difference between using n=4 or n=6 is minor.

It should be noted that this sensitivity analysis was performed before the rainfall gauges were finalised. For this reason, some of the plots shown may differ slightly from those shown in the body of the report.



- **Figure A-1: Comparison of rainfall stations available during the event (indicated by circles), with all data available after the event (dots).**



- **Figure A-2: Comparison of URBS sub-area rainfalls using data available during the event (left) and all rainfall data available after the event (right). All have been determined using the URBS SUBRAIN utility with either the closest 4 or 6 stations.**

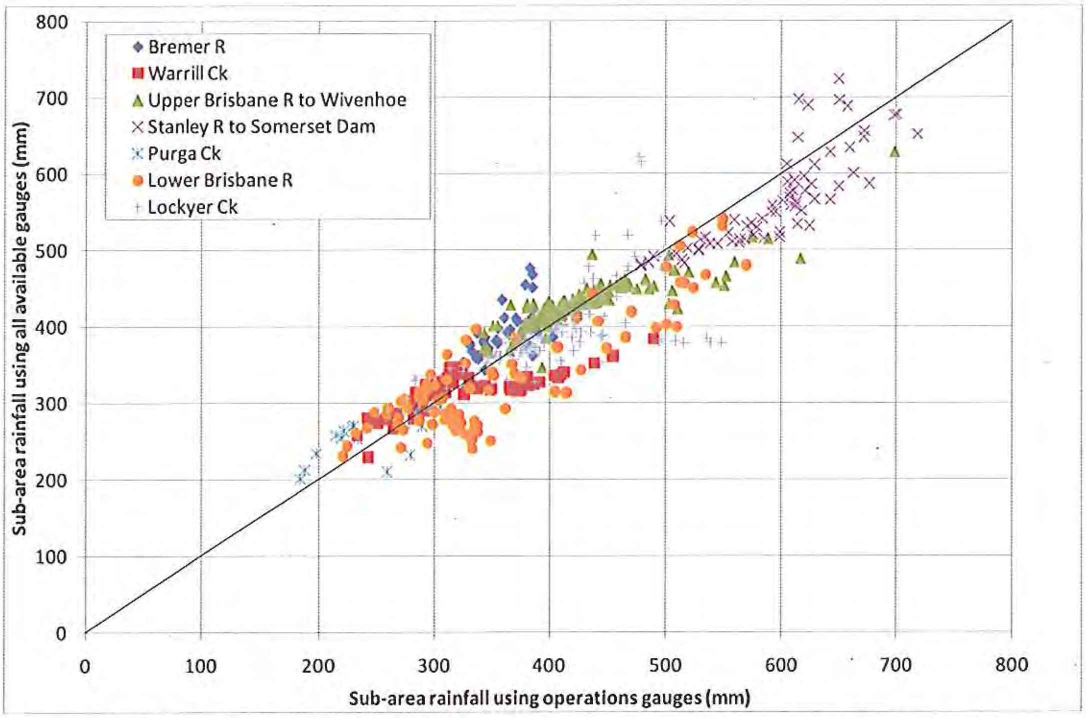


Figure A-3: Comparison of sub-area rainfall using stations available during the event and all data available after the event. Sub-area rainfalls were calculated using the SUBRAIN function with n=4.

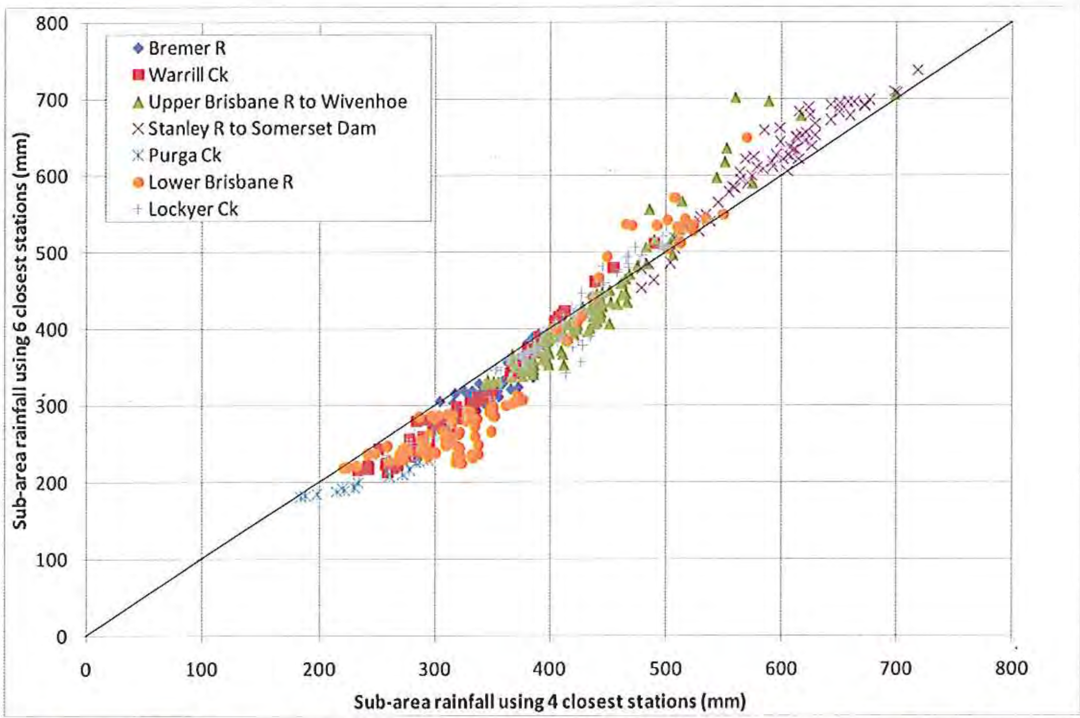


Figure A-4: Comparison of sub-area rainfall when SUBRAIN function is used with n=4 and n=6. Sub-area rainfalls were calculated using data available during the event.

Appendix B Review of 2005 Model

To assist Seqwater in understanding the appropriateness and robustness of the MIKE 11 model developed in 2005 and define what improvements could be made by SKM in the allotted time the provided model has been reviewed. The aspects of the model that have been audited are provided below with summaries of the findings (note: it has been assumed that readers of this report will have some understanding of hydrodynamic modelling).

B.1 Model Schematisation

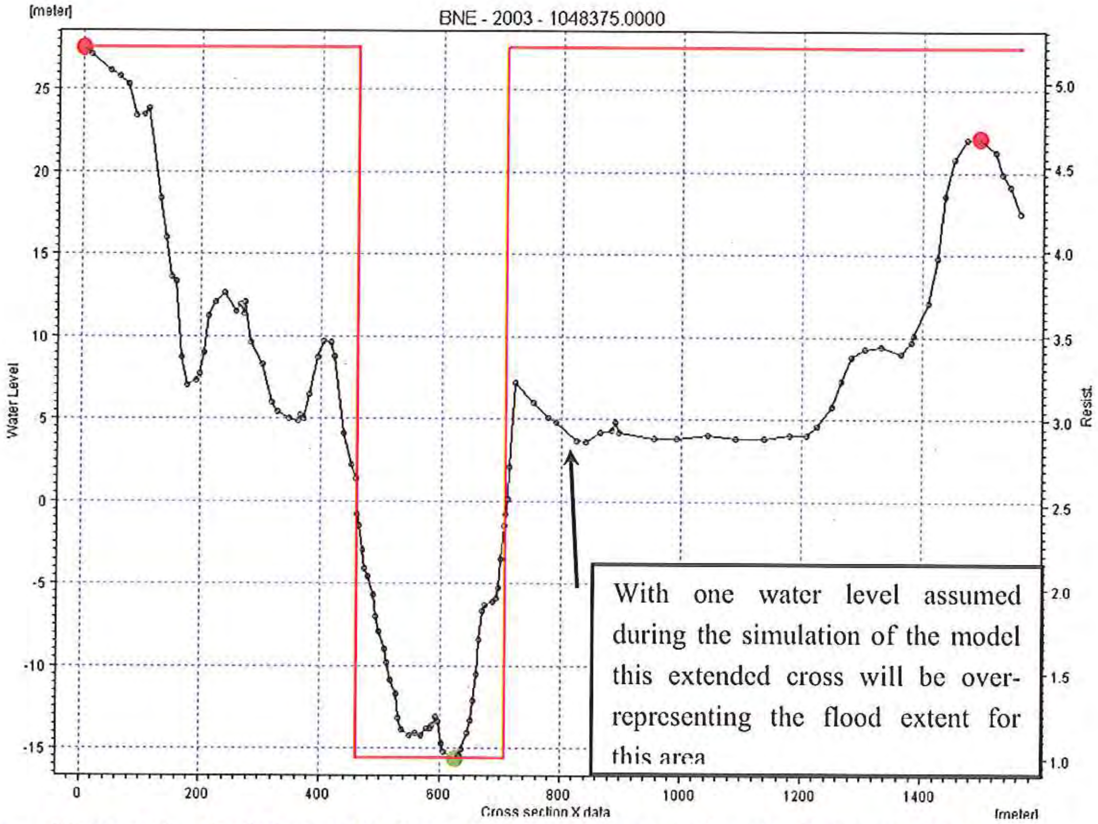
Due to the nature in which the MIKE 11 model has iteratively developed over time it is difficult to exactly determine how the model was schematically worked up to represent the routing aspects of the river and associated structures and features. In spite of this, following a review of several reports and associated spatial data sets obtained for the purposes of this study it is apparent that the overarching approach has been to use extended cross sections to represent the channel and floodplain with higher level linking channels connecting particular reaches of the system for when floodwaters would get out of bank (floodplain spills). This type of schematisation is typical in 1 Dimensional flood flow modelling.

B.2 River Channel Cross Sections

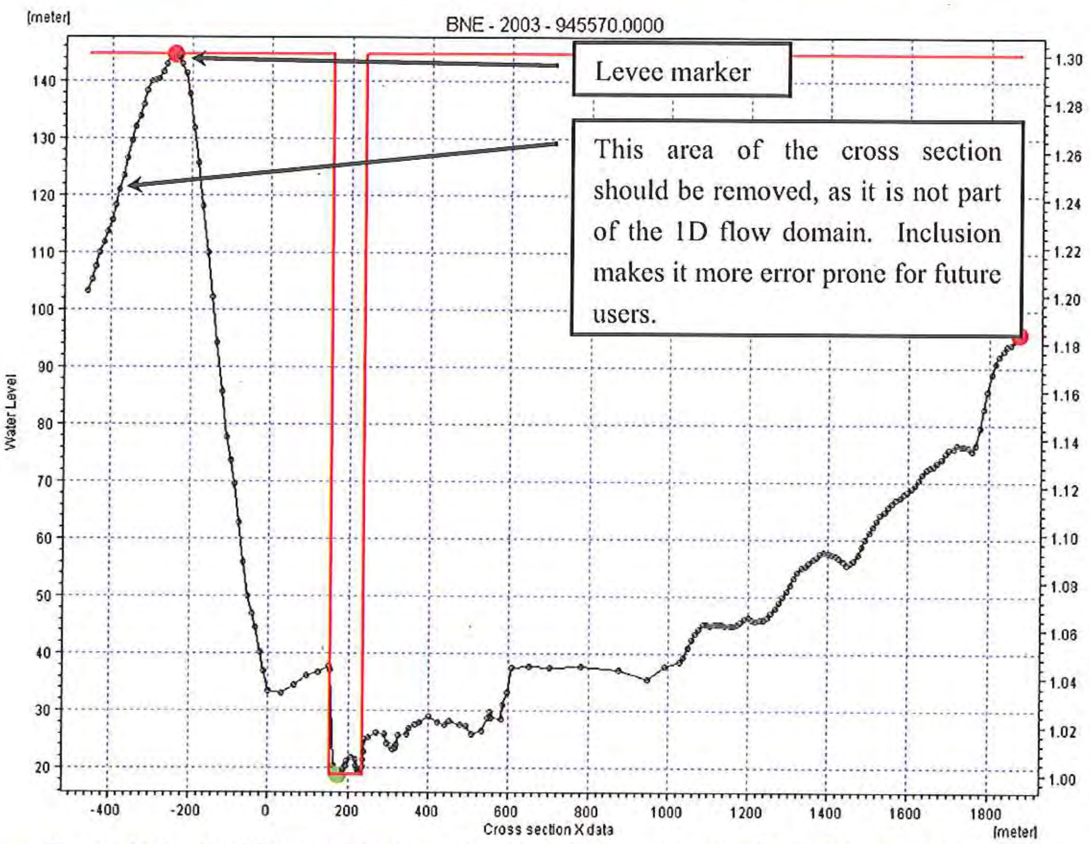
The MIKE 11 model is made up of a large number of river channel cross sections representing the Brisbane River and its associated tributaries. For the Brisbane River itself, there are 263 channel cross sections representing a river reach 149740m making the average cross section spacing of around 500m. With the model forming a strategic representation of flood flow processes this level of detail is considered appropriate.

Although an audit of each of the cross sections is beyond the scope of this review, a number of issues have been found with those sections that represent the Brisbane River itself. In the examples provided in Figures B1 and B2 below it can be seen that cross sections do not adequately represent the floodplain and include cross sectional area that should ideally be removed rather than separated from the processed data (the hydraulic curves which are used by the simulation engine) through the use of a levee marker (a modelling unit which acts like a glass wall). Typically, and it would be expected that, these sorts of occurrences would be represented as follows:

- compartmentalisation of the main 1D river and floodplain sections with spills along the river bank linking the 2 reaches together (one cross section representing the channel and one of the floodplain);
- defined areas of storage; and/or
- through the use of a 2-Dimensional flood spreading module.



- **Figure B-1 – Problems with Cross Sections Representing the Brisbane River**



- **Figure B-2 – Problems with Cross Sections Representing the Brisbane River**

Whilst not critical to the actual dynamics of the routing of flows through the river system (since the processed hydraulic curves will be processed similarly) there are also a number of cross sections that are in reverse. This sort of issue is problematic at locations where link channels are specified, as without supporting terrain data it is not apparent whether, or not, the link channel has been specified with the correct linking levels.

B.3 Link Channels

Although it is difficult to audit whether or not the link channels would represent the intended flood flow dynamics, comparing the modelling units with the LiDAR data that has been obtained for the purposes of this study it would appear that most would broadly represent the dynamics as intended. Where the link channels are not appropriate is where they connect to and from, as many appear to be situated between cross sections rather than actually at cross sections, and also in the cross sections themselves where problems with cross sectional units already discussed are common. The issue of the connecting of link channels could be appropriate, but from the simulations undertaken it would seem that the placement of these are being activated to early as they are placed between two cross sections at presumably a lower level than in reality.

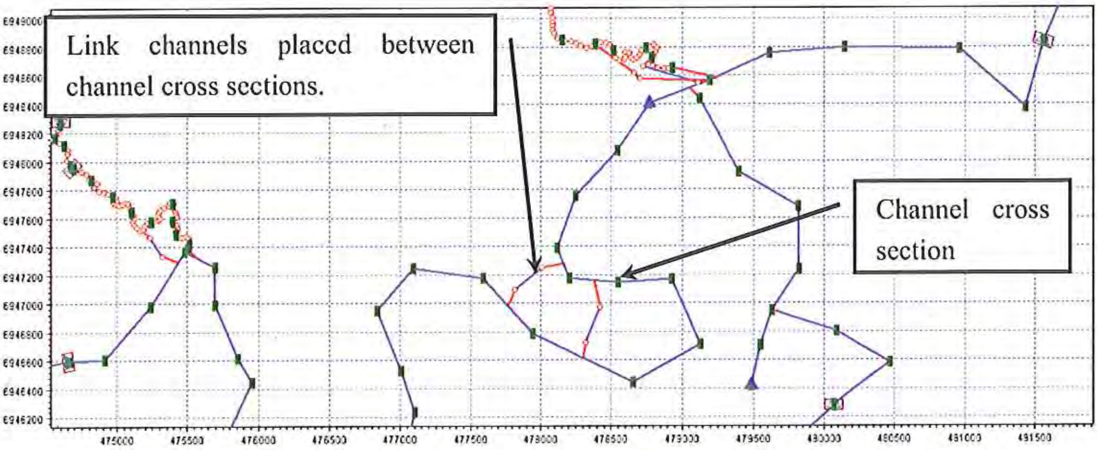


Figure B-3 – Problems with Link Channels (note: this is on the Bremer River)

B.4 Hydraulic Structures

B.4.1 Bridges

To assist in understanding what bridges are represented in the MIKE 11 model aerial mapping has been reviewed alongside GIS layers of road alignments to identify those bridges which cross the Brisbane and Lockyer Rivers. Table B.1 outlines whether, or not, the bridge has been represented in the MIKE 11 model provided.

Table B-1 – Review of Bridges

ID	Data Source	Bridge Type	Bridge Name	Used for Stream flow Gauging	Stream flow gauge name	Represented in MIKE 11	Comments in Ipswich (2000) and Brisbane and Pine (1994) Flood Studies	Summary of Review
1	Google	Track Bridge	Marschkes Farm Bridge			N		
2	Transport 1:250000	Road Bridge	Forest Hill Fernvale Road Bridge			N		
3	Google	Track Bridge	Fairmeadowl Farm Bridge			N		
4	Transport 1:250000	Road Bridge	Claredon Road Bridge			N		
5	Transport 1:250000	Road/Railway Bridge	Mahon Road (Disused Railway) Bridge			N		
6	Transport 1:250000	Road Bridge	Patrick Estate Road Bridge			N		
7	Transport 1:250000	Road Bridge	Wivenhoe Pocket Road (Twin Bridges) Bridge			N	Low level bridge - accounted for by roughness	
8	Transport 1:250000	Road Bridge	Banks Creek Road (Savages Crossing) Bridge	Y	SAVAGES CROSSING ALERT	N	Low level bridge - accounted for by roughness	
9	Transport 1:250000	Road Bridge	Summerville Road East (Burtons) Bridge	Y	BURTONS BRIDGE ALERT	N	Low level bridge - accounted for by roughness	
10	Google	Track Bridge	Corbould Nature Range Bridge			N		
11	Transport 1:250000	Road Bridge	Kholo Road Bridge	Y	KHOLO BRIDGE ALERT	Y	Multi span structure with 8 piers with a constant deck made of timber	This bridge seems to be fairly well represented. It is, however, worth noting that the width of the weir representing the deck way is smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge.
12	Transport 1:250000	Road Bridge	Allawah Road (Mount Crosby Weir) Bridge	Y	MT CROSBY ALERT	Y	River level = 0.5m AHD, soffit = 10.3 Modelled as a weir due to issues encountered during model building (based on a weir setup in HEC-RAS). The road is supported by 17 piers.	Although it is difficult to audit this structure, the weir would seem to be representing the throttling effect the bridge would have. It is also worth noting that the width of the weir representing the deck way is slightly smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge.
13	Transport 1:250000	Road Bridge	Mount Crosby Road (Colleges Crossing) Bridge	Y	COLLEGES CROSSING ALERT	Y	Multi span structure with 2 piers and a set 8-2700X900 RCBC culverts	This structure seems to be fairly well represented. With the structure represented as two separate modelling units it should be checked to ensure that the representation of the structure is adequate. It is also worth noting that the width of the weir representing the deck way is significantly smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge.
14	Transport 1:250000	Road Bridge	Centenary Highway (Jindalee) Bridge	Y	JINDALEE BRIDGE	Y	Soffit = 12.5m (average) Multi span structure with a constant deck way and 6 piers. During the 1974 flood event a barge was sunk immediately upstream of the bridge to avoid damage to the bridge.	With the deck level (11.067m AHD) effectively set at a level which is below the soffit (13.7m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.

15	Transport 1:250000	Road Bridge	Coonan Street (Walter Taylor) Bridge			Y	Soffit = 14.8m (average) The structure has been combined with Indooroopilly Rail Bridge for the purposes of the MIKE 11 model.	The representation of the weir which has a larger width than the upstream cross section will cause issues in calculating the relationships between discharges and levels, and with a left bank (14.6m AHD) lower than soffit level (15.01m AHD) the hydraulic effect of the bridge surcharging will not be represented.
16	Transport 1:250000	Railway Bridge	Main Line Railway Bridge			N		
17	Google	Foot Bridge	Albert Bridge			N	Soffit = 15.3m	
18	Google	Foot Bridge	Eleanor Schonell Bridge			N		
19	Google	Road Bridge	Go Between Bridge			N		
20	Transport 1:250000	Railway Bridge	South Coast Railway (Merivale) Bridge			Y	Soffit = 14.9m Multi span structure with 2 piers that was constructed after the 1974 flood	The weir arrangement effectively provides a level of cover of around 0.167 (15.367m AHD - 15.2m AHD). This could be correct, but needs to verified. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.
21	Transport 1:250000	Road Bridge	Grey Street (William Jolly) Bridge			Y	Soffit = 13.8m Multi span bridge with arch chords having little effect on conveyance.	This structure seems to be fairly well represented. Where the structural arrangement is not appropriate is in the representation of the weir that represents the deck way, since the larger than upstream cross sectional width will cause issues when calculating the relationships between discharges and levels. It is also worth noting that the very small slot located at the bottom of the structure in the bridge profile will cause stability issues for the MIKE 11 model.
22	Google	Foot Bridge	Kurilpa Bridge			N		
23	Transport 1:250000	Road Bridge	Victoria Bridge			Y	Soffit = 10.0m (average) Solid arch bridge which significantly reduces bore area during higher flood flows	With the deck level (10.267m AHD) effectively set at a level which is below the soffit (14.3m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The appropriateness of this structure is also to some degree compounded with the left bank (11.16m AHD) not extending above the soffit level. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.
24	Google	Foot Bridge	Goodwill Bridge			N		
25	Transport 1:250000	Road Bridge	Pacific Motorway (Captain Cook) Bridge			Y	Soffit = 11.0m (average)	With the deck level (9.867m AHD) effectively set at a level which is below the soffit (17.61m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The appropriateness of this structure is also to some degree compounded with the right bank (13.29m AHD) not extending above the soffit level. The width of the weir representing the deck way is significantly smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge. It is also worth noting that the very small slot located at the bottom of the structure in the bridge profile will cause stability issues for the MIKE 11 model.
26	Transport 1:250000	Road Bridge	Bradfield Highway (Story) Bridge			Y	Soffit = 30.8m (average) Unlikely to be overtopped.	With the deck level (30.867m AHD) effectively set at a level which is below the soffit (33m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The appropriateness of this structure is also to some degree compounded with bank levels (5.6m AHD and 31.9m AHD, respectively) not extending above the soffit level and also by the inclusion of a weir width (586.1m), or deck level width, that is greater than the cross section (523m) it is adjoined to. The representation of the weir which has a larger width than the upstream cross section will cause issues in calculating the relationships between discharges and levels.
27	Transport 1:250000	Road Bridge	Gateway Bridge			N	Modelled as a modified section in 1994	

Of the 28 bridges that have been identified 11 are currently represented within the MIKE 11 model to some degree (highlighted in bold in Table 1) in a culvert and weir arrangement (the culvert representing the bridge constriction and the weir representing floods flows overtopping the structure and floodplain). The audit has not been able to compare the representation of these bridges to survey, but has instead undetrtaken a sensibility check on the manner the structures have been represented. The detail of this review is provided below and a summary of the overarching issues that have been identified is provided below:

- *Bridge deck levels set below soffit levels* – This effectively does not represent the hydraulic effect of the bridge surcharging.
- *Bridge deck widths are either greater or smaller than the bounding upstream cross section* – This will either cause issues in calculating the relationships between discharges and levels or may throttle more severe flood flows, as the full deck way and cross section area are not being represented.
- *Bounding upstream cross sections do extend above the soffit level of the bridge* – This will not represent the hydraulic effect of the bridge surcharging.
- *Very small slots located at the bottom of structure in the bridge profile* – This will cause stability issues for the MIKE 11 model.

Brisbane Valley Highway (Fernvale) Bridge

The Brisbane Valley Highway (Fernvale) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 31.853m AHD

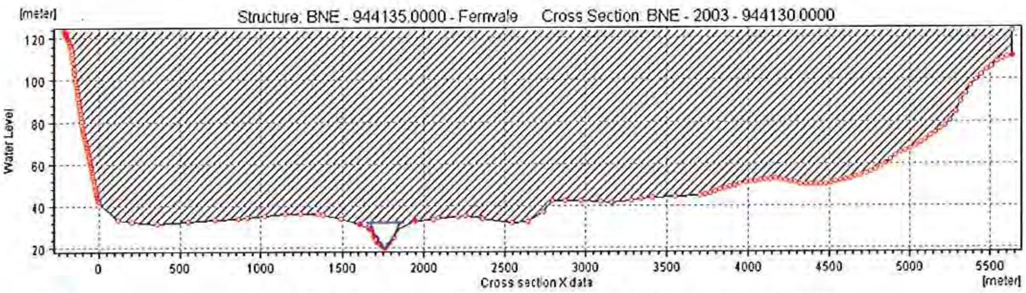
Invert = 20.26m AHD

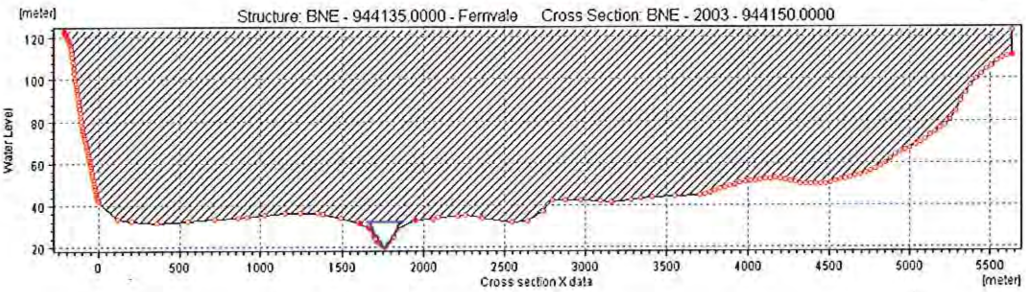
Maximum opening width = 228.133m

US cross sections invert = 20.25m AHD

Maximum US cross section width = 5631.31m

US cross section left and right maximum elevations = 122.774m AHD (left) 111.18m AHD (right)

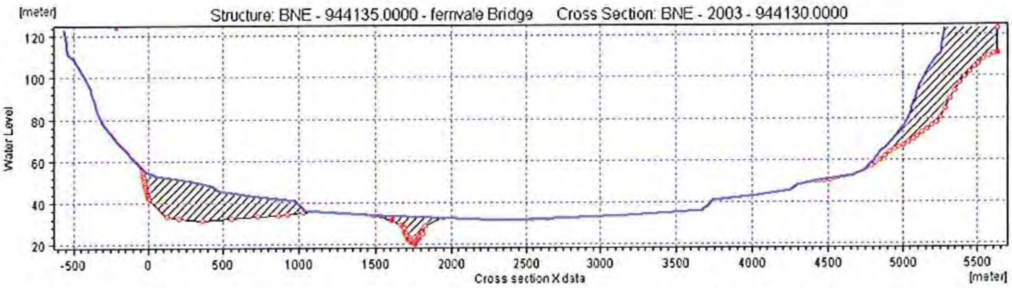


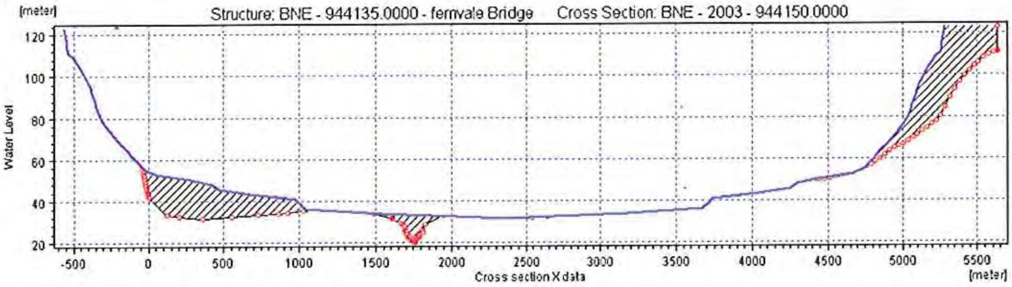


The weir that currently represents the deck level is currently represented as follows:

Invert = 31.657m

Maximum Width = 5851.6m





Summary

With the deck level (31.657m AHD) effectively set at a level which is below the soffit (31.853m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.

Kholo Road Bridge

The Kholo Road Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 11.28m AHD

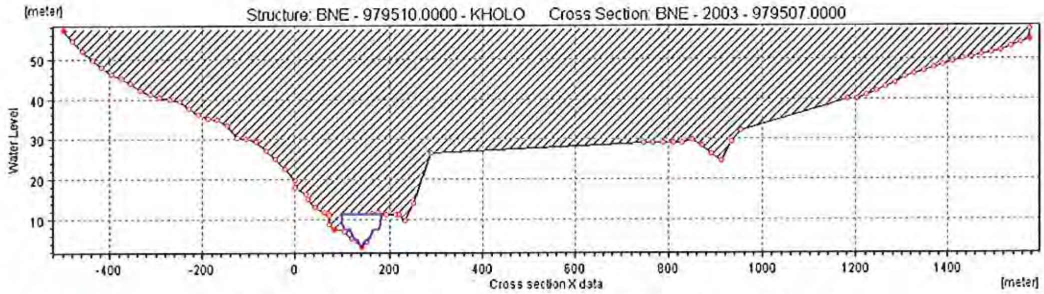
Invert = 3.32m AHD

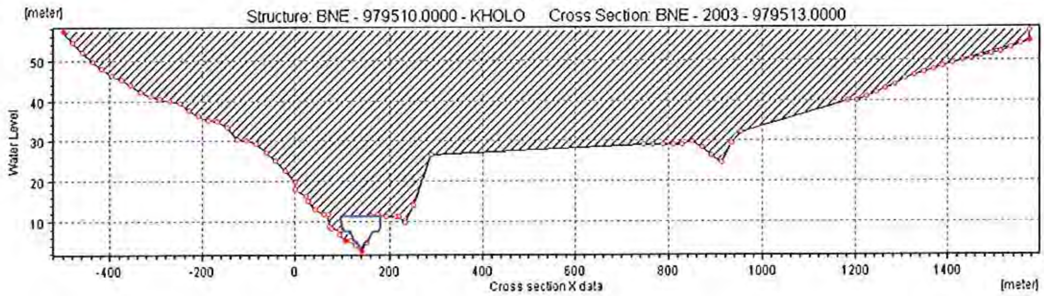
Maximum opening width = 89m

US cross sections invert = 3.32m AHD

Maximum US cross section width = 1575.88m

US cross section left and right maximum elevations = 57.1m AHD (left) 54.7m AHD (right)

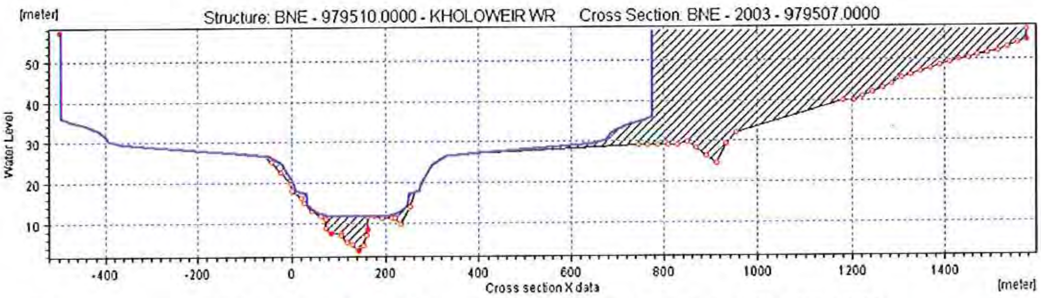


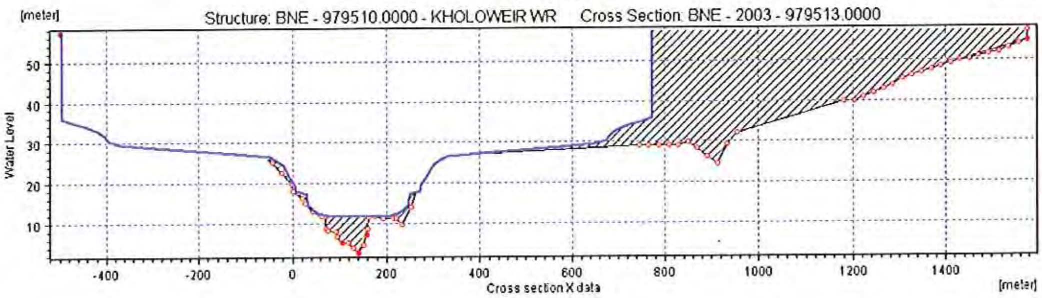


The weir that currently represents the deck level is currently represented as follows:

Invert = 11.73m AHD

Maximum Width = 1270m





Summary

This bridge seems to be fairly well represented. It is, however, worth noting that the width of the weir representing the deck way is smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge.

Allawah Road (Mount Crosby Weir)

The Allawah Road (Mount Crosby Weir) Bridge is currently represented as only a weir arrangement due to issues faced during the original development of the MIKE 11 model. It is reported that the details of the weir have been assessed and should appropriately represent the dynamics of flood flows at this location. The weir that represents this structure is currently represented as follows:

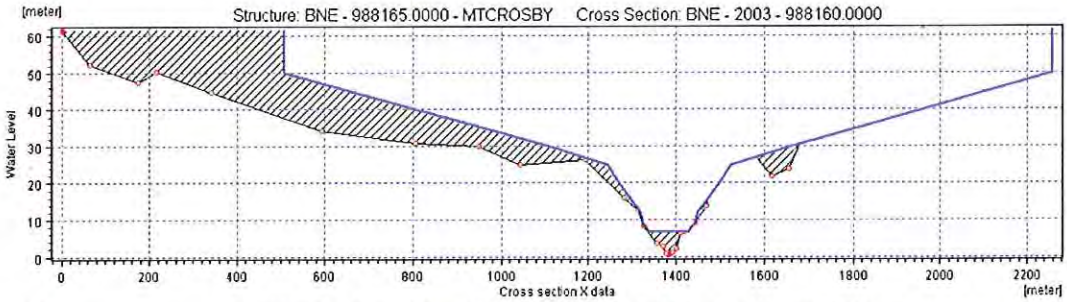
Invert = 6.71m AHD

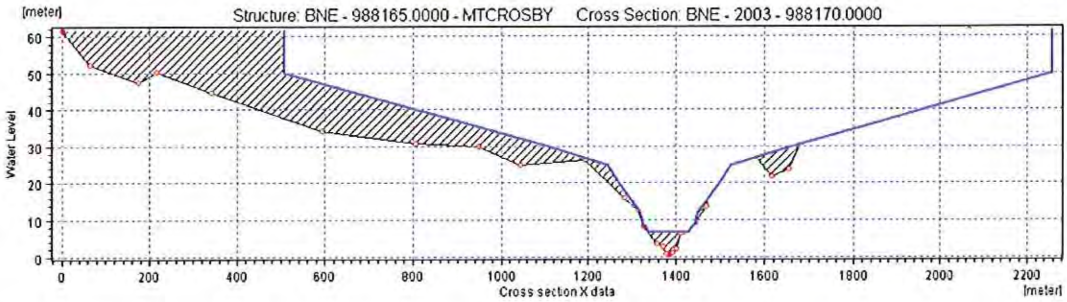
Maximum Width = 1750m

US cross sections invert = 0.52m AHD

Maximum US cross section width = 1958.4m

US cross section left and right maximum elevations = 61.43m AHD (left) 53.34m AHD (right)





Summary

Although it is difficult to audit this structure, the weir would seem to be representing the throttling effect the bridge would have. It is also worth noting that the width of the weir representing the deck way is slightly smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge.

Mount Crosby Road (Colleges Crossing) Bridge

The Mount Crosby Road (Colleges Crossing) Bridge is currently represented as 2 culverts and a weir arrangement. The culverts are currently represented as follows:

Culvert 1 – 8 x culvert openings with widths of 2.7m and 0.9m

Soffit = 2.18m AHD

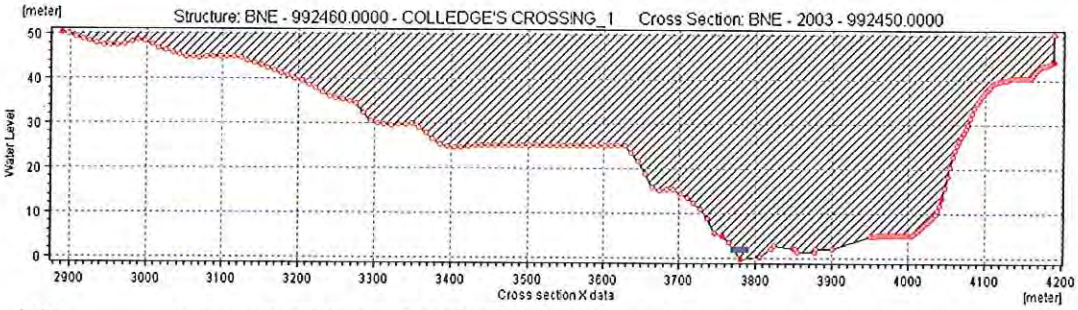
Invert = 1.28m AHD

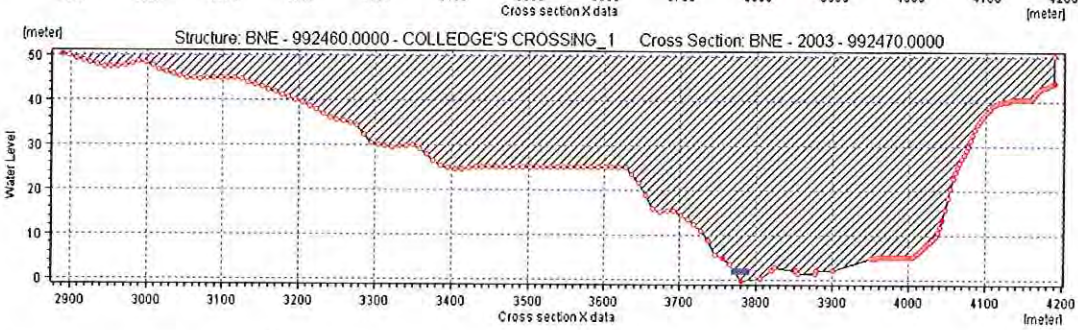
Maximum opening width = 21.6 (8m x 2.7m) m

US cross sections invert = -0.26m AHD

Maximum US cross section width = 4189.7m

US cross section left and right maximum elevations = 50.19m AHD (left) 44.26m AHD (right)





Culvert 2 – 1 defined culvert

Soffit = 2.748m AHD

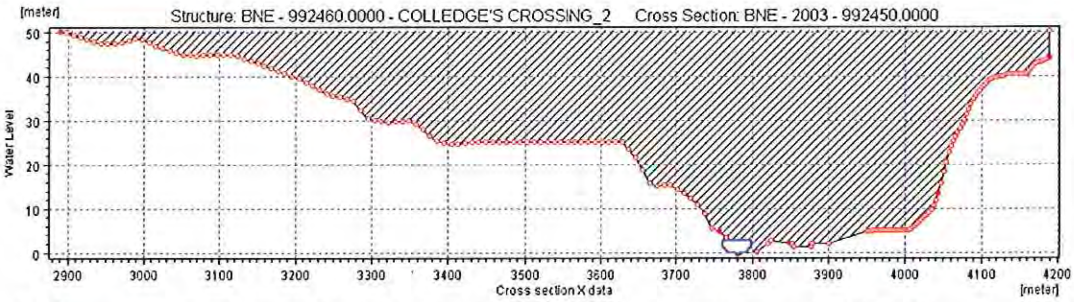
Invert = -0.262m AHD

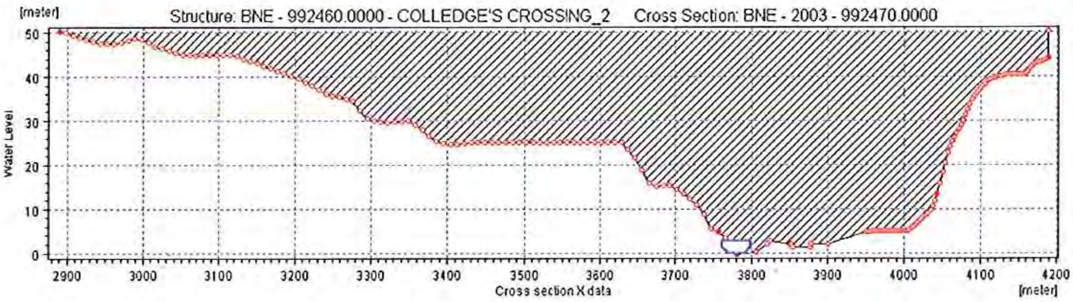
Maximum opening width = 37.9m

US cross sections invert = -0.26m AHD

Maximum US cross section width = 4189.7m

US cross section left and right maximum elevations = 50.19m AHD (left) 44.26m AHD (right)

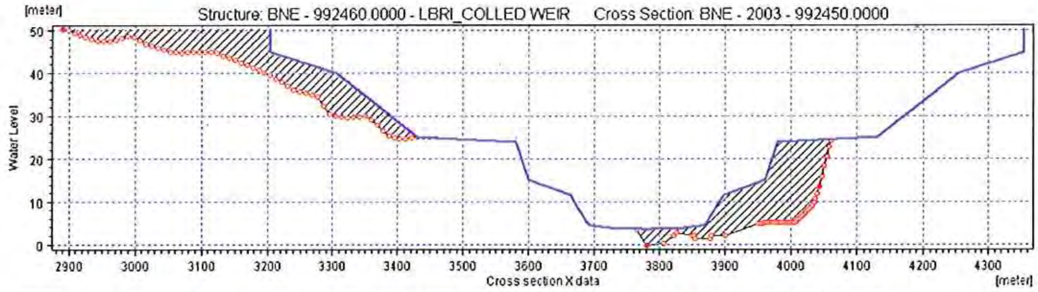


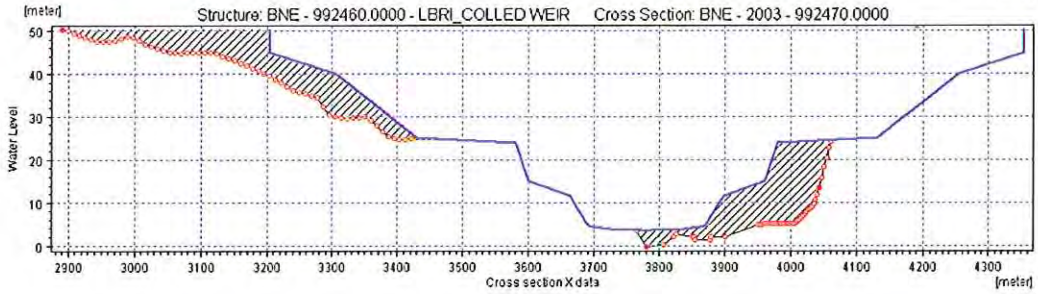


The weir that currently represents the deck level is currently represented as follows:

Invert = 3.38m AHD

Maximum Width = 1150m





Summary

This structure seems to be fairly well represented. With the structure represented as two separate modelling units it should be checked to ensure that the representation of the structure is adequate. It is also worth noting that the width of the weir representing the deck way is significantly smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge.

Centenary Highway (Jindalee) Bridge

The Centenary Highway (Jindalee) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 13.7m AHD

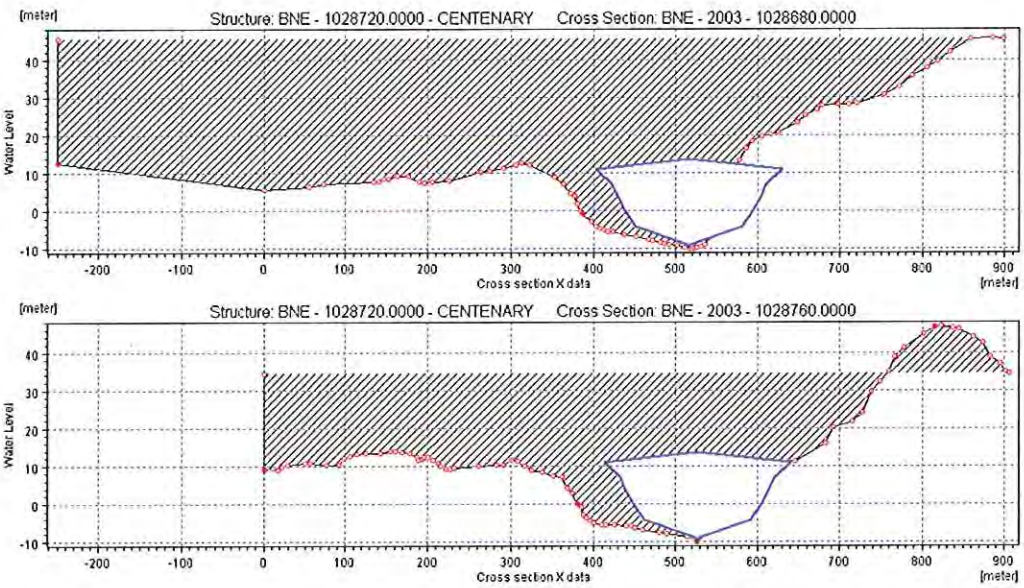
Invert = -9.9m AHD

Maximum opening width = 228m

US cross sections invert = -9.9m AHD

Maximum US cross section width = 566.8m

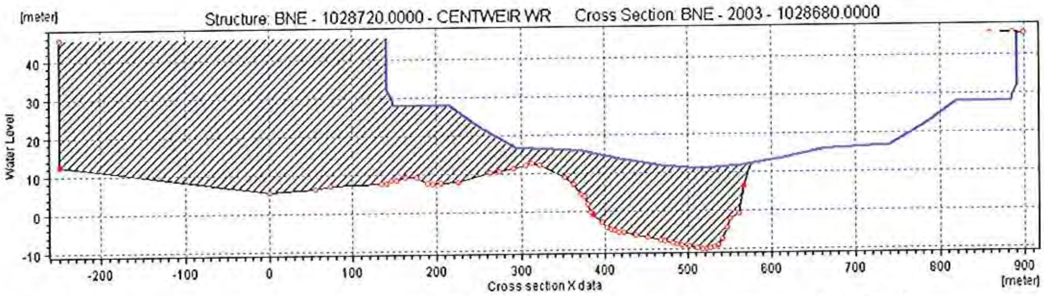
US cross section left and right maximum elevations = 14.6m AHD (left) 29.27m AHD (right)

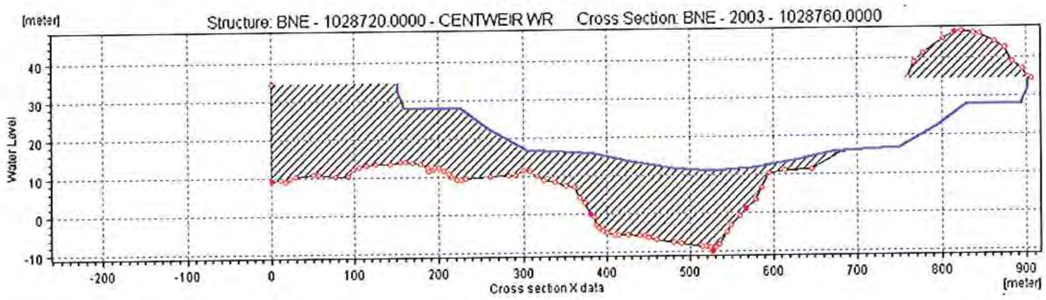


The weir that currently represents the deck level is currently represented as follows:

Invert = 11.067m AHD

Maximum Width = 748.87m





Summary

With the deck level (11.067m AHD) effectively set at a level which is below the soffit (13.7m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.

Coonan Street (Walter Taylor) Bridge

The Coonan Street (Walter Taylor) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 15.01m AHD

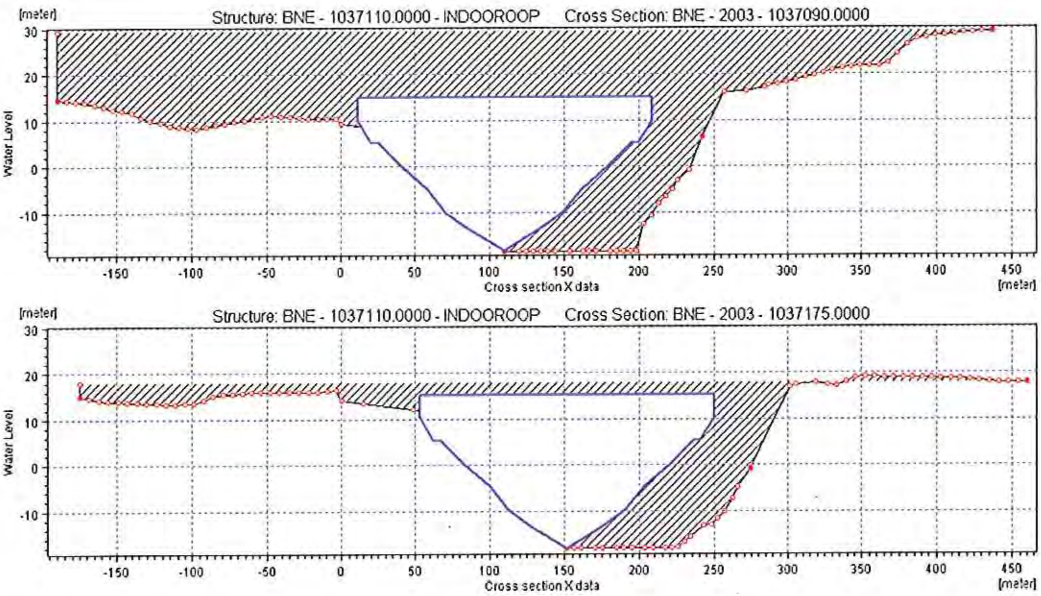
Invert = -18.39m AHD

Maximum opening width = 197m

US cross sections invert = -18.4m AHD

Maximum US cross section width = 436.7m

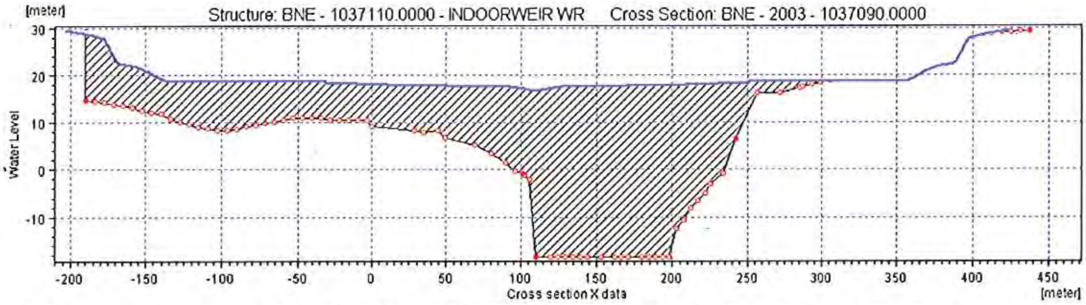
US cross section left and right maximum elevations = 14.6m AHD (left) 29.27m AHD (right)

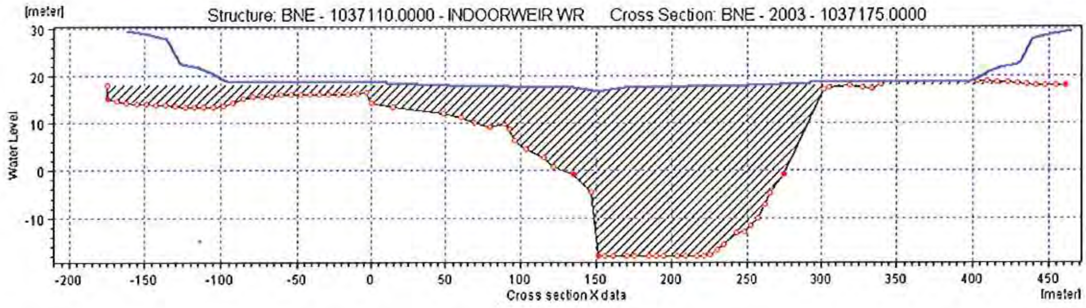


The weir that currently represents the deck level is currently represented as follows:

Invert = 16.567m AHD

Maximum Width = 626.78m





Summary

The representation of the weir which has a larger width than the upstream cross section will cause issues in calculating the relationships between discharges and levels, and with a left bank (14.6m AHD) lower than soffit level (15.01m AHD) the hydraulic effect of the bridge surcharging will not be represented.

South Coast Railway (Merivale) Bridge

The South Coast Railway (Merivale) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 15.2m AHD

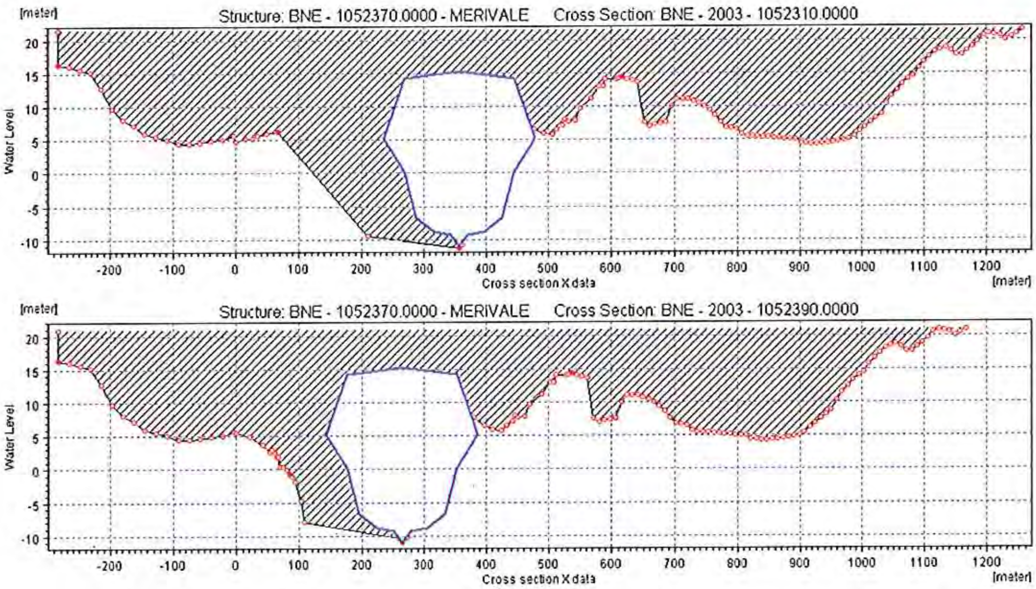
Invert = -10.9m AHD

Maximum opening width = 244m

US cross sections invert = -11.2m AHD

Maximum US cross section width = 615.35m (note: the use of a bank marker reduces the available cross sectional width to this)

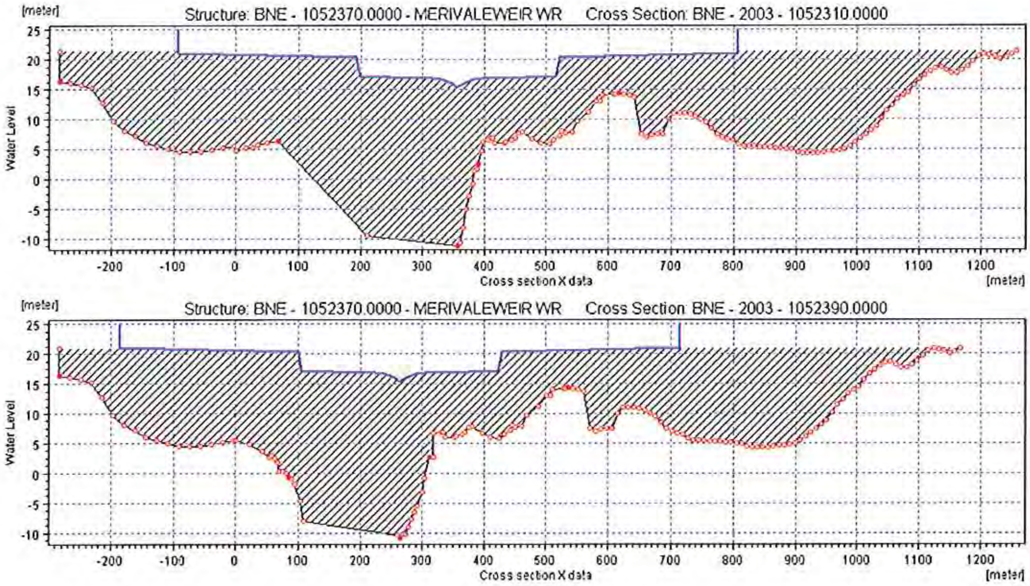
US cross section left and right maximum elevations = 16.32m AHD (left) 14.35m AHD (right)



The weir that currently represents the deck level is currently represented as follows:

Invert = 15.367m AHD

Maximum Width = 899m



Summary

The weir arrangement effectively provides a level of cover of around 0.167 (15.367m AHD - 15.2m AHD). This could be correct, but needs to verified. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.

Grey Street (William Jolly) Bridge

The Grey Street (William Jolly) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 11.9m AHD

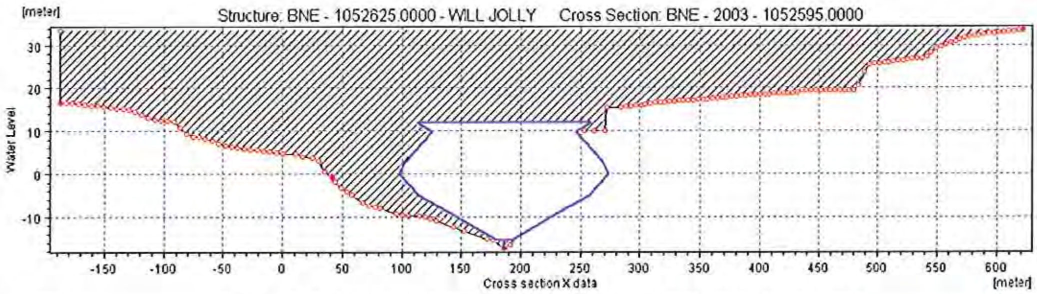
Invert = -17.2m AHD

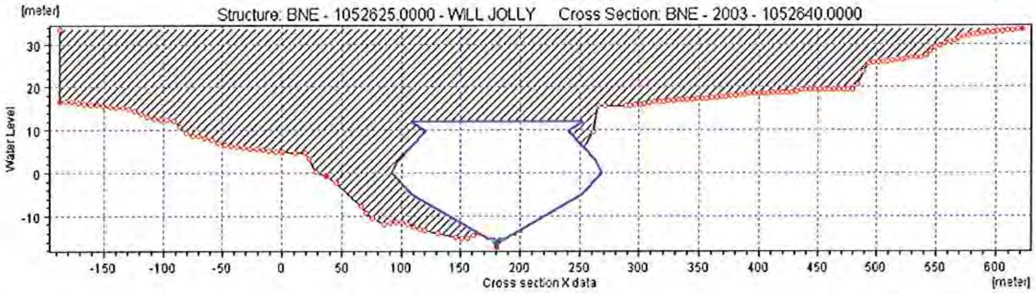
Maximum opening width = 176m

US cross sections invert = -17.2m AHD

Maximum US cross section width = 621.4m

US cross section left and right maximum elevations = 16.5m AHD (left) 33.34m AHD (right)

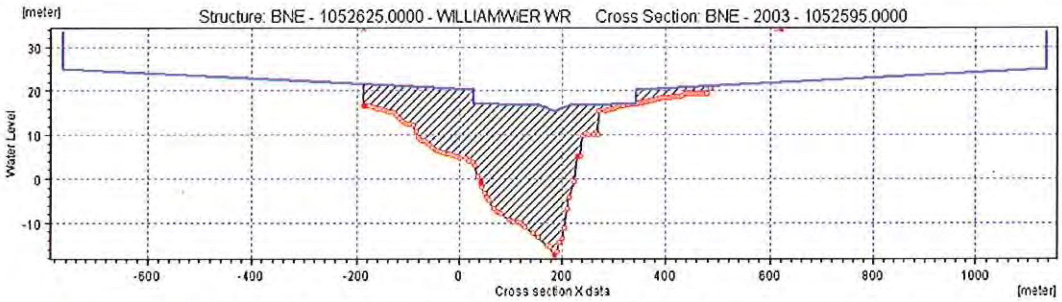


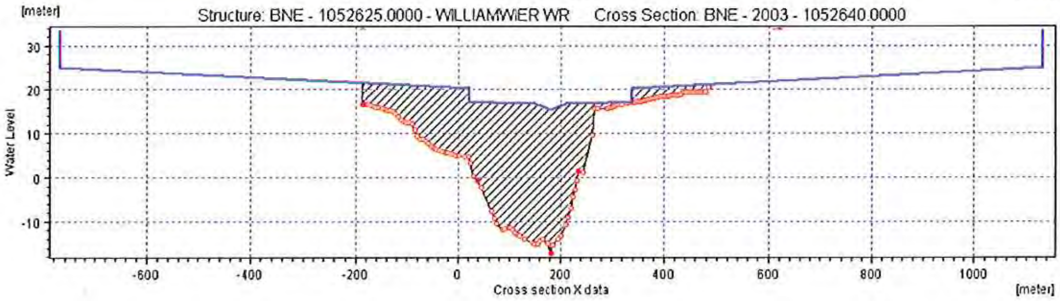


The weir that currently represents the deck level is currently represented as follows:

Invert = 15.367m AHD

Maximum Width = 1900m





Summary

This structure seems to be fairly well represented. Where the structural arrangement is not appropriate is in the representation of the weir that represents the deck way, since the larger than upstream cross sectional width will cause issues when calculating the relationships between discharges and levels. It is also worth noting that the very small slot located at the bottom of the structure in the bridge profile will cause stability issues for the MIKE 11 model.

Victoria Bridge

The Victoria Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 14.3m AHD

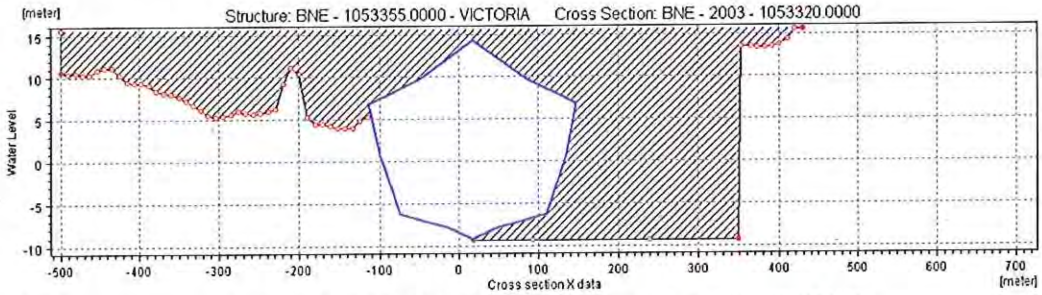
Invert = -9.2m AHD

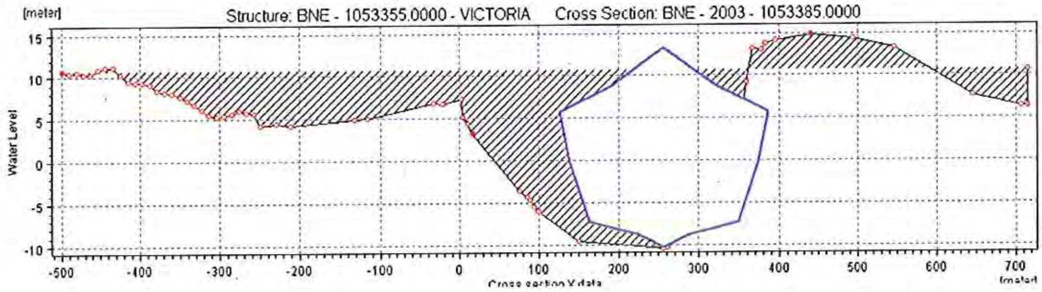
Maximum opening width = 261m

US cross sections invert = -9.3m AHD

Maximum US cross section width = 428.9m

US cross section left and right maximum elevations = 11.16m AHD (left) 15.5m AHD (right)

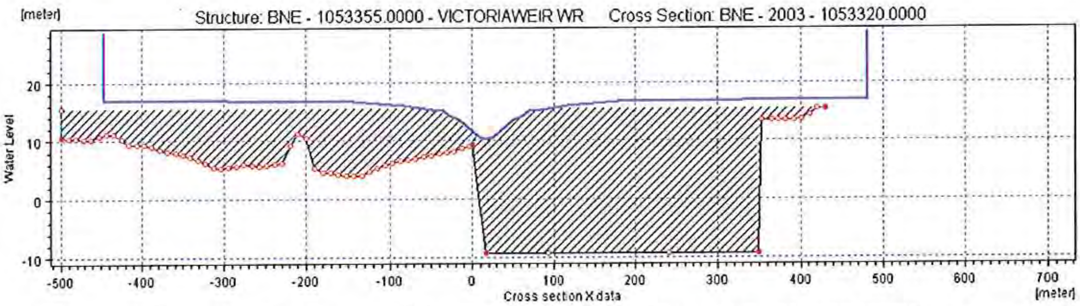


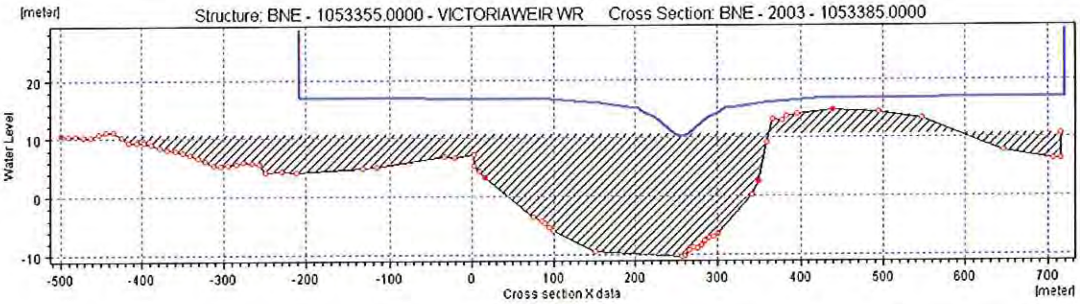


The weir that currently represents the deck level is currently represented as follows:

Invert = 10.267m AHD

Maximum Width = 928.2m





Summary

With the deck level (10.267m AHD) effectively set at a level which is below the soffit (14.3m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The appropriateness of this structure is also to some degree compounded with the left bank (11.16m AHD) not extending above the soffit level. The representation of the weir which has a larger width than the upstream cross section will also cause issues in calculating the relationships between discharges and levels.

Pacific Motorway (Captain Cook) Bridge

The Pacific Motorway (Captain Cook) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 17.61m AHD

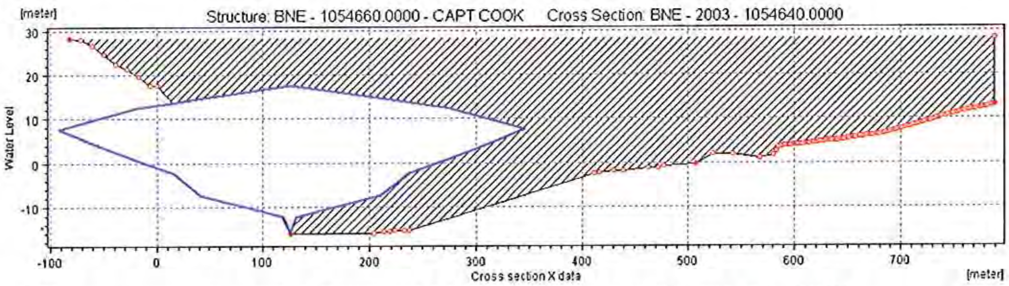
Invert = -15.89m AHD

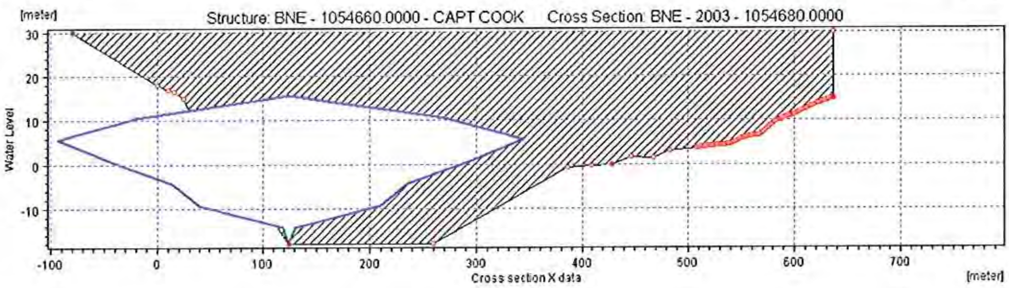
Maximum opening width = 438m

US cross sections invert = -16.0m AHD

Maximum US cross section width = 789.1m

US cross section left and right maximum elevations = 28.39m AHD (left) 13.29m AHD (right)

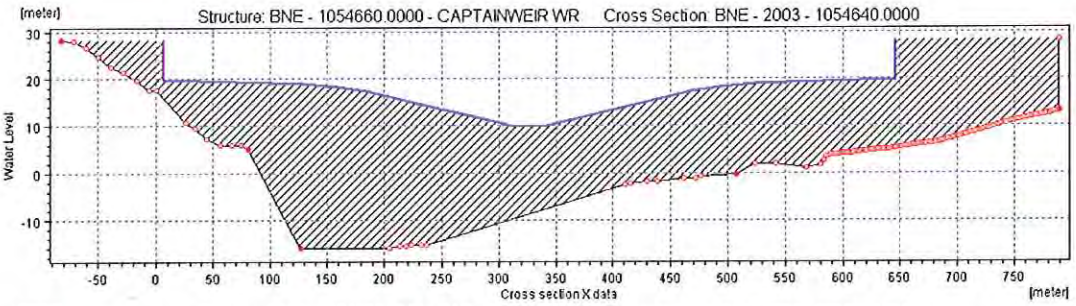


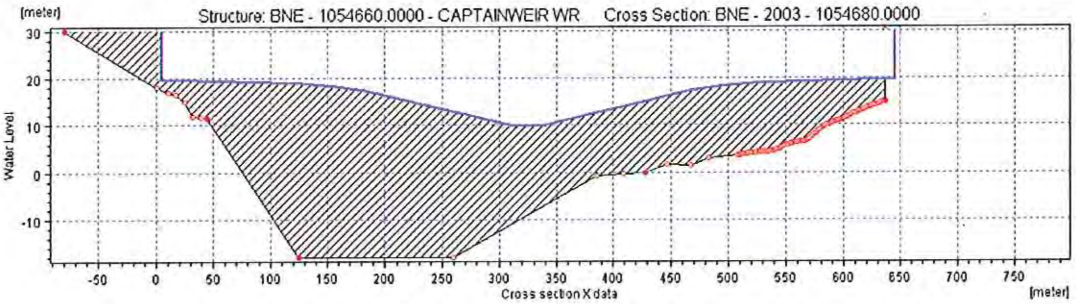


The weir that currently represents the deck level is currently represented as follows:

Invert = 9.867m AHD

Maximum Width = 640.2m





Summary

With the deck level (9.867m AHD) effectively set at a level which is below the soffit (17.61m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The appropriateness of this structure is also to some degree compounded with the right bank (13.29m AHD) not extending above the soffit level. The width of the weir representing the deck way is significantly smaller than that represented in the cross section upstream. This may throttle more severe flood flows and should also be checked to ensure that the full deck way and cross section area are being represented so as to allow floodwaters to flow over the top of the bridge. It is also worth noting that the very small slot located at the bottom of the structure in the bridge profile will cause stability issues for the MIKE 11 model.

Bradfield Highway (Story) Bridge

The Bradfield Highway (Story) Bridge is currently represented as a culvert and a weir arrangement. The culvert is currently represented as follows:

Soffit = 33m AHD

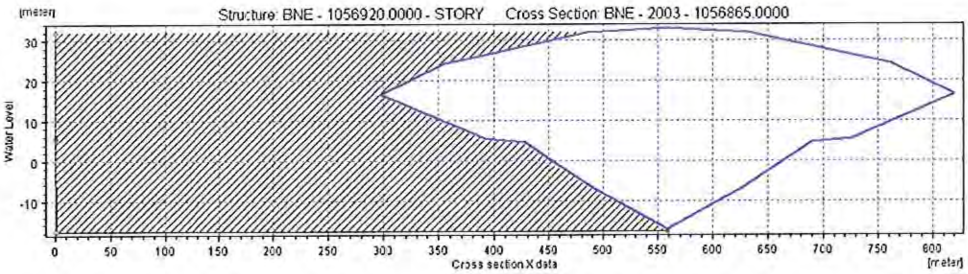
Invert = -17m AHD

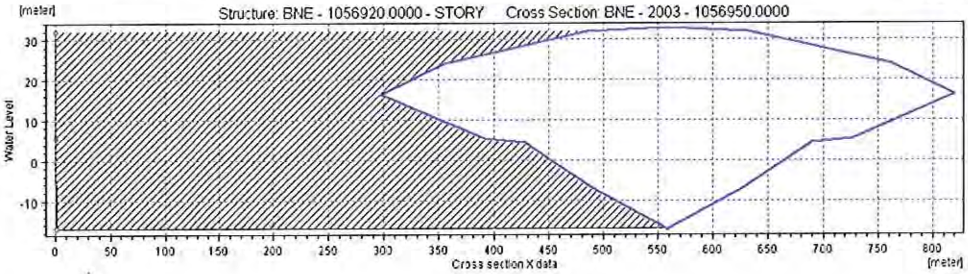
Maximum opening width = 523m

US cross sections invert = -17.5m AHD

Maximum US cross section width = 559.5m

US cross section left and right maximum elevations = 5.6m AHD (left) 31.9m AHD (right)

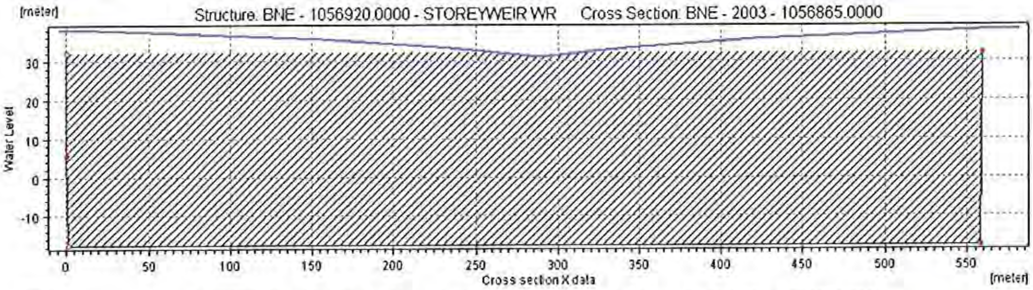


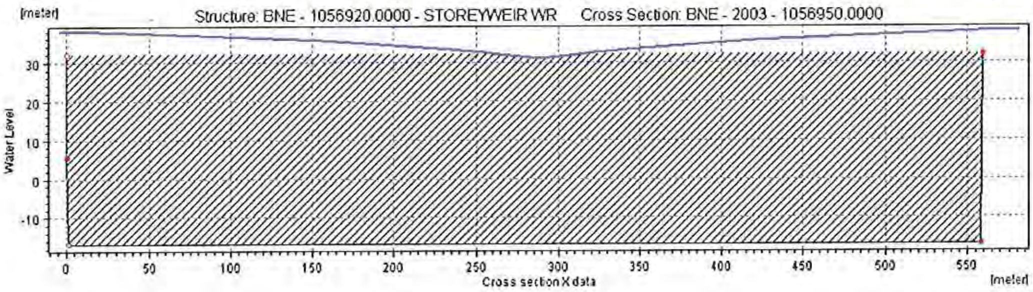


The weir that currently represents the deck level is currently represented as follows:

Invert = 30.867m AHD

Maximum Width = 586.1m





Summary

With the deck level (30.867m AHD) effectively set at a level which is below the soffit (33m AHD) the hydraulic effect of the bridge surcharging is not currently being represented appropriately. The appropriateness of this structure is also to some degree compounded with bank levels (5.6m AHD and 31.9m AHD, respectively) not extending above the soffit level and also by the inclusion of a weir width (586.1m), or deck level width, that is greater than the cross section (523m) it is adjoined to. The representation of the weir which has a larger width than the upstream cross section will cause issues in calculating the relationships between discharges and levels.

B.4.2 Stream Flow Gauging

Stream flow gauging on the Brisbane River which is represented in the MIKE 11 model is typically undertaken at bridge locations due to presumably the ease of access the transport network provides. Of those stream flow gauges that are currently operational, the following in Table B-2 are represented as part of the hydraulic representation of the bridge:

- **Table B-2 – Stream flow gauging represented in the MIKE 11 model**

Station Name	CBM Number	AWRC Number	Latitude	Longitude	Owner
COLLEGES CROSSING ALERT	540063	143868	-27.55	152.79	BUREAU/LOCAL GOVERNMENT (ICC)
KHOLO BRIDGE ALERT	540256	143864	-27.56	152.74	SEQWATER
MT CROSBY ALERT	540199	143839	-27.53	152.79	SEQWATER

With the auditing of the bridges already discussed in the previous section and no other stream flow gauging specifically represented in the MIKE 11 model on the Brisbane River itself no further auditing of the stream flow gauges has been undertaken.

B.5 Representation of Roughness

The representation of roughness in river system models is typically undertaken using the Mannings '*n*' coefficient of roughness. This coefficient accounts for a number of aspects representing the overall resistance a particular area would have on the flow in either the channel or floodplain. Values do vary from location to location and from season to season (more vegetation during the summer periods would result in higher values), but typically are within the order of the 0.03 to 0.07 other than at locations where the river meanders when values may be higher to account for the headlosses that would occur.

During the initial development of the MIKE 11 model during 1998 and 2000 the developers accounted for the meandering component in the overall estimation of roughness and applied this to cross sections through the use of either higher local resistance factors contained within the cross sections themselves or by setting higher local resistance factor within the HD parameter file. This amalgamated in the use of some very high roughness values (0.2 in some instances) in a number of locations. Although on occasions and/or situations this may be appropriate, the broad types of land uses discussed in the reports that have been reviewed as part of this study and a review of the aerial photography which is freely available on google maps underlines that the use of such high values of roughness cannot be justified and is erroneous. It is difficult to identify what effect this would have on past results with a number of errors built up into the model, but no doubt this would have acted to mask them.

In reviewing the actual locations of where roughness is specified in the channel cross sections it has also been found that a number of locations have either been mistakenly or incorrectly specified. For example, in Figure 4 it can be seen that the far left floodplain has an area of low roughness which would seem to be mistakenly specified, and in Figure 5 it can be seen that the area of lower roughness (or the area that represents the main river channel) is specified at a very low level. Both of these issues that have been identified will cause the model to be more unstable and less accurate.

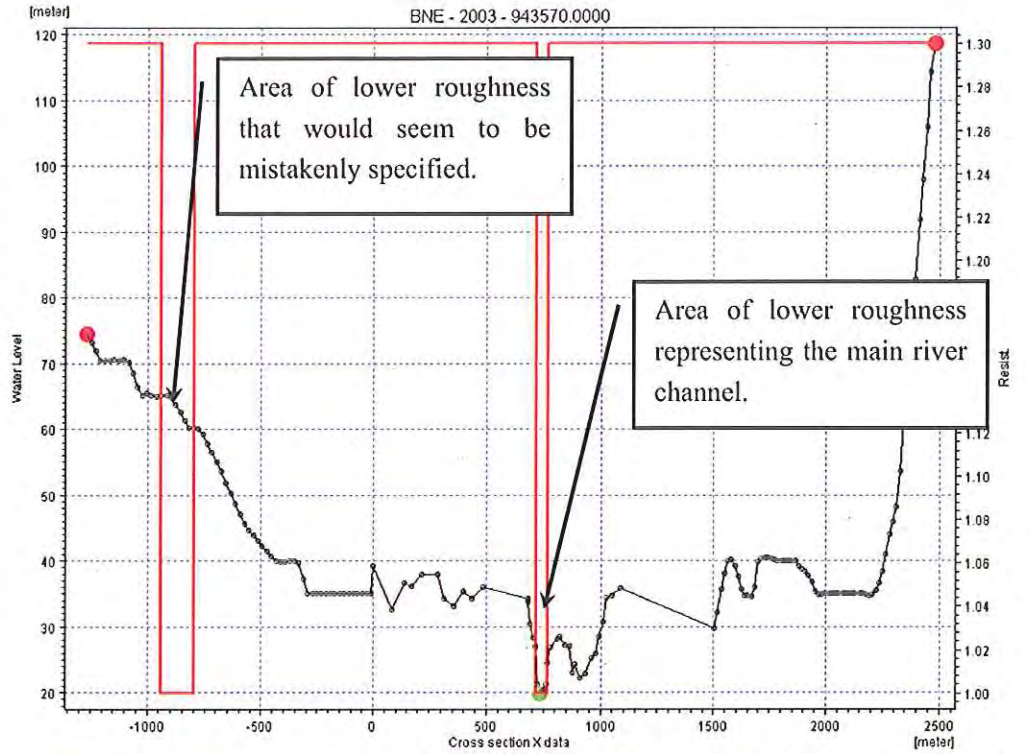


Figure B-4 – Problems with the representation of roughness in cross sections

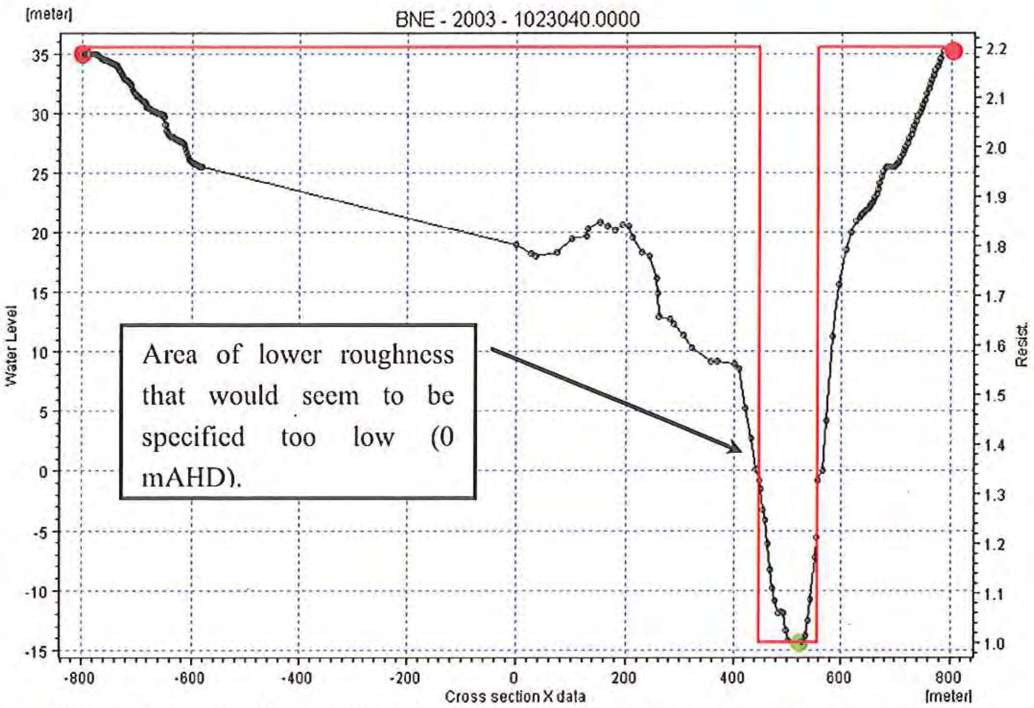


Figure B-5 – Problems with the representation of roughness in cross sections

Another aspect, which is not as critical as those already discussed, is the actual convoluted manner in which roughness has been applied in the model. The use of varying local and global roughness values in either the actual cross sections or HD parameters has made auditing the model more difficult as cross sections cannot be readily compared to one another and is prone to error.

B.6 Model Inputs and Boundaries

There are 12 upstream boundaries (or inflow locations), 1 downstream boundary (or the tidal mouth), and 26 '*baseflow*' locations. It is not quite clear why these baseflow inputs (additional $0.1m^3/s$) are included in the model, but it is likely they have been included to limit the drying out of river channel cross sections during simulation and thereby improve its stability (*"sweetener flows"*). Although with the amounts of flow added to the model this can be viewed as a minor error with the model, they can easily be mistaken to be representative inputs to the model and does then question the suitability of the flood levels that are predicted by the model on these reaches (e.g. Six Mile Creek).

B.7 Model Setup

The model provided was setup to simulate to solve the hydrographs inputted at intervals of 15 seconds (a fixed timestepping scheme). Whilst this level of timestep may be optimum for areas of the hydrodynamic model, it is likely that the use of a defined timestep will cause the routed flood hydrograph to either be dampened, or elevated, and thereby cause the model to be less stable. It is not understood why a fixed timestep has been used when typically an adaptive timestep is used in default so that the results of the hydrodynamic model are independent of timestep size whilst optimizing run times at the same time.

To provide initial conditions for the routing of flood hydrographs through the MIKE 11 model is currently setup whereby it relies on the use of a 'hot start file' (the results of a previous run). Although certain situations and scenarios may dictate its use, it does not make the model flexible as either a tool that can be furthered developed (particularly if storage areas are added, as a wrong initial condition may mean incorrect volumes are calculated) or for operational use as a flood warning and forecasting tool. The use of so many stabilising inputs demonstrates that the model has been poorly constructed, as the simulation engine is reliant on the *"fudging factors"* for the computation of hydrographs it is provided.

QUEENSLAND FLOODS COMMISSION OF INQUIRY

REVIEW OF HYDRAULIC MODELLING

FINAL REPORT

JULY 2011

wma water

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: 9299 2855
Fax: 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

REVIEW OF HYDRAULIC MODELLING
QUEENSLAND FLOODS COMMISSION OF INQUIRY

FINAL REPORT

JULY 2011

Project Review of Hydraulic Modelling Queensland Floods Commission of Inquiry	**Project Number** 111024
Client QLD Flood Commission of Inquiry	**Client's Representative** Lisa Hendy
Authors Mark Babister Stephen Gray Rhys Hardwick Jones	**Prepared by** [redacted]
Date 28 JULY 2011	**Verified by** [redacted]

Revision	Description	Date
1	FINAL	JULY 11
2	FINAL (Revised 28/07/11)	JULY 11

REVIEW OF HYDRAULIC MODELLING
QUEENSLAND FLOODS COMMISSION OF INQUIRY

TABLE OF CONTENTS

PAGE

1. EXECUTIVE SUMMARY 1

1.1. Background 1

1.2. Model Review 1

1.3. Conclusions 2

2. INTRODUCTION 6

2.1. Scope of the Report 6

2.2. Sequence of Events 7

3. AVAILABLE INFORMATION 8

3.1. Data Relied Upon 8

3.2. Reliance Statement 8

4. MODEL REVIEW 9

4.1. Introduction 9

4.2. Seqwater 2005 Mike11 Model 9

4.3. Version 1 SKM Model – Case 1 (January 2011 Calibration) 10

4.4. Version 2 SKM Model – Case 1 (January 2011 Calibration) 13

4.5. Review Caveats 13

4.6. Files Provided and Reviewed 13

4.7. Description of the Model 14

4.8. Assessment of Calibration 18

4.9. Comments 21

4.10. Model Limitations 22

5. ASSESSMENT OF JANUARY 2011 FLOOD EVENT 24

6. ASSESSMENT OF ALTERNATIVE DAM OPERATION STRATEGIES 32

Discussion – Early Transition to Strategy W4 (Option A) 32

6.1. Discussion – Prior Dam level at 75% FSL (Option B) 35

6.2. Discussion – Releases at Upper Limit During Strategy W3 37

6.3. Discussion – Optimised Strategy 38

7. RESPONSES TO QUESTIONS FROM THE COMMISSION 40

7.1. Additional Comments 42

8. REFERENCES 43

LIST OF APPENDICES

LIST OF TABLES

Table 1: Alternative Dam Operation Results 3
Table 2: Files reviewed as submitted by SKM 13
Table 3: SKM Roughness Values applied to Version 2 Model 16
Table 4: Relative contribution of Wivenhoe Dam flows to peak flood levels downstream 28
Table 5: Alternative Dam Operation Results 32
Table 6: Alternative Dam Operation Results (Table 5 reprinted here for convenience) 41

LIST OF FIGURES

Figure 1: Velocity Time Series (modelled) at Jindalee (SKM Model Version 1) 10
Figure 2: Discharge Time Series (modelled) at Mt Crosby Bridge (Version 1) 11
Figure 3: Water Level Time Series (modelled) at Mt Crosby Bridge (Version 1) 11
Figure 4: Model Results (Version 1) at Port Office versus “Fixed” model results 12
Figure 5: Mike11 Cross-Section with insufficient floodplain detail 16
Figure 6: Comparison of gauged and modelled water level – Mt Crosby 18
Figure 7: Comparison of gauged and modelled water level – Moggil 19
Figure 8: Comparison of gauged and modelled water level – Jindalee 19
Figure 9: Comparison of gauged and modelled velocities 20
Figure 10: Comparison of gauged and modelled water level – Port Office 20
Figure 11: Flow chart describing the derivation of All upstream Mt Crosby Input Hydrograph 21
Figure 12: Comparison of various input hydrographs from upper part of model 24
Figure 13: Percentage of flood volume from various sources 9-16 January 2011 25
Figure 14: Comparison of timing of Wivenhoe Dam release flow and flow from other sources 25
Figure 15: Impact of Bremer Flows on Case 2 and 3 runs 27
Figure 16: Impact of Wivenhoe Dam flows on Bremer Flows and Levels at Ipswich 29
Figure 17: Flood level profile in Bremer River for Calibration Event 30
Figure 18: Backwatering of Bremer River System – Calibration Event 30
Figure 19: Bremer River System – Gauge Locations 31
Figure 20: Option A4 Wivenhoe Dam Releases and Water Levels 33
Figure 21: Option A5 Wivenhoe Dam Releases and Water Levels 33
Figure 22: Option B Wivenhoe Dam Releases and Water Levels 35
Figure 23: Option C Wivenhoe Dam Releases and Water Levels 37
Figure 24: Option D Wivenhoe Dam Releases and Water Levels 38

1. EXECUTIVE SUMMARY

1.1. Background

1 The Queensland Floods Commission of Inquiry (the Commission) engaged, Mark Babister, Managing Director of consulting firm WMAwater, to provide expert technical advice and analysis to the Commission throughout the course of the Inquiry.

2 Following modelling of the January 2011 event by SKM on behalf of Seqwater, the Commission has asked Mr Babister to review the model and to make comment on its suitability for analysis of the January 2011 Brisbane River flood. Further, the Commission seeks answers to the questions below:

a) To what extent was flooding (other than flash flooding) in the mid-Brisbane River, the Lockyer Valley, Ipswich and Brisbane during January 2011 caused by releases from the Somerset and Wivenhoe Dams?
b) To what extent did the manner in which flood waters were released from the Somerset and Wivenhoe Dams avoid or coincide with peak flows from the Bremer River and Lockyer Creek?
c) Had the levels in Somerset and Wivenhoe Dams been reduced to 75 per cent of full supply level by the end of November 2010 (both with and without amendments to the trigger levels for strategy changes in the Wivenhoe Manual) what impact would this have had on flooding?
d) What effect would the implementation of different release strategies (to be identified by WMAwater) have had on flooding?

3 The hydrodynamic model has been built using hydrodynamic modelling software called Mike11 (Version 2009). A previous model sourced from Seqwater (Seqwater, 2005) was used as a base for the work. SKM have substantially revised the model within the Brisbane River, although modelled sections of Lockyer Creek and the Bremer River have been left unaltered. The revisions included incorporating up-to-date topographical data throughout the 149 kilometres reach of the Brisbane River downstream of Wivenhoe Dam.

1.2. Model Review

4 WMAwater's model review work began on 27 June 2011. Significant issues were identified with the model (Version 1) presented by SKM and utilised in the scenario modelling presented in SKM's report of 24 June 2011 (Reference 2). Following a meeting between WMAwater, SKM and Seqwater on 1 July 2011, SKM were able to revise the model to address the issues identified and subsequently WMAwater received new calibration results on 5 July 2011. Via a joint meeting between SKM, Seqwater and WMAwater on the same day agreement was reached on the model build and calibration. From WMAwater's perspective the agreement acknowledged that whilst not ideal, the model presented the best available opportunity to answer questions from the Commission

as noted above in Paragraph 2. WMAwater received a revised model (Version 2) on 7 July 2011.

5 The revised model exhibits good performance for standard quality control metrics – mass is conserved, the model is stable, utilises reasonable roughness parameter values and produces results that compare favourably with gauged data within its area of validity. Specifically the model has been demonstrated to match recorded flow level at three stream gauge stations downstream of the flow input location at Mt Crosby (i.e. Moggil, Jindalee and Port Office). Emulation of measured flow velocities at Jindalee is shown to be good and also the model matches peak flow at Jindalee as gauged during the January 2011 event (at or near the peak). Confidence in the model provided could be improved if the model was demonstrated to be able to replicate behaviour from other historical events without the need to substantially change model parameters (referred to as model validation). Nonetheless the revised model (Version 2) is considered fit for purpose to address most of the questions put forward to WMAwater by the Commission.

6 As the upper tributary flows are inserted into the model at Mt Crosby model results are only valid downstream of Mt Crosby. Also neither the Lockyer Creek or Bremer River systems have been calibrated or revised as part of SKM's work. As such the extent of the calibrated model is limited to the Brisbane River from Mt Crosby to its most downstream location in Moreton Bay. A full discussion of limitations of the model in its current form is provided in Section 4.10.

7 SKM provided the Version 2 model to WMAwater so that limited analysis, based on the Commission's specific enquiries, could be carried out. For consistency and to ensure that no contention existed around the model version used in analysis WMAwater utilised SKM's model without alteration except where explicitly noted.

1.3. Conclusions

8 Based on analysis of the calibrated model results for the January 2011 flood, as well as additional results from alternative scenario testing, WMAwater draw the following conclusions:

a. Flooding in the Brisbane River downstream of Mt Crosby occurred as a result of combined flow from Wivenhoe Dam releases as well as tributary inflows from Lockyer Creek, the Bremer River, and other catchments. Quantification of the relative contributions of each system is difficult, as the interactions between flows at confluences are complex, particularly with regard to timing of peak flows and backwater effects. The flooding caused by the combined flow from all tributaries is therefore not strictly comparable to the hypothetical flooding resulting from the flow of each tributary and results achieved from such comparisons are approximate only. Nevertheless modelling of isolated flow components has been undertaken in order to inform assessment work;

b. The total volume discharged from Wivenhoe Dam between the 9th and 16th of January was 59% of total flow volume in the lower Brisbane River during this period. However, the bulk of this flood volume was released after the flood peak, thereby providing flood mitigation benefits;

c. Modelling indicates that the peak of the Wivenhoe Dam releases reached the Mt Crosby gauge approximately 9 hours prior to the peak of all other flows upstream of Mt Crosby combined. However this assessment is limited by the modelling approach for inflows at Mt Crosby as discussed in Section 4.9;

d. Gauging at Jindalee during the event, and near the peak, indicates that peak flow was approximately 10,000 m^3/s. It is estimated that non-Wivenhoe Dam and Wivenhoe Dam flows were roughly equivalent contributors to this peak flow value;

e. Wivenhoe Dam peak flows, at the confluence of the Brisbane and Bremer Rivers, occurred near simultaneously with Bremer River peak flows. Significant backwatering of the Bremer River occurred within the lower Bremer River to a distance of approximately 15 km upstream of the confluence with the Brisbane River;

f. The combined flows of Lockyer Creek and Wivenhoe Dam had a significantly greater influence than the Bremer River contribution on total flood flow downstream of Moggill; and

g. If Wivenhoe Dam releases had occurred in isolation from any other flow in the Lockyer/Bremer tributaries and other downstream catchments, peak flood levels would have been lower at the Moggill, Oxley Creek, and Brisbane port Office gauges, than as a result of the inverse scenario (tributary flows without any flow from Wivenhoe Dam). This result is, however, in part attributable to the attenuating effect of the empty Bremer River system under the "Wivenhoe only" scenario. A more reasonable comparison where this effect is removed indicates that peak flood levels, at all locations downstream of the confluence, are roughly equivalent for the two scenarios.

9 Findings from alternative gate operation scenarios are summarised in the table below. Please note that scenarios are as per descriptions below:

a. Case 1 – The calibrated January 2011 model results supplied by SKM;
b. Option A – Earlier transition to Strategy W4;
c. Option B –Wivenhoe Dam at 75% of Full Storage Level (FSL) prior to the flood;
d. Option C – Discharge at upper limit during Strategy W3;
e. Option D – An optimised release strategy, as outlined by one of the Seqwater Flood Engineers in their statement to the Commission (Reference 3).

Table 1: Alternative Dam Operation Results

Location	Case 1	Option A	Option B	Option C	Option D
	Peak Flood Level (mAHD)	Peak Flood Level difference relative to Case 1 (m)			
Moggill	17.6	-0.3 to 0.4	-0.7	-0.7	-0.9
Jindalee	13.1	-0.3 to 0.4	-0.6	-0.6	-0.8
Oxley	8.3	-0.2 to 0.3	-0.5	-0.5	-0.6
Brisbane	4.6	-0.1 to 0.3	-0.3	-0.3	-0.4

10 Of these scenarios, Option D would have had the greatest impact with a reduction in peak flood level at Port Office of 0.4 m and a reduction at Moggil of 0.9 m. However of the scenarios investigated, Option D is also the least likely to be achieved in practice, as it would have relied on foreknowledge of the flood far superior to that available to the Flood Engineers, even taking forecast rain into account.

11 Option C is a more plausible alternative scenario, although it too would have required a level of foreknowledge of the flood event at key decision points that was not available at the time.

12 Option B, resulting from Wivenhoe Dam being at 75% FSL prior to the flood (either through policy or antecedent rainfall conditions), and using existing gate operations strategies from the Manual, would have resulted in a similar benefit to flood levels as Option C. If gate operations were revised to take advantage of the additional storage available under such a scenario, it is expected that the benefits on flood levels would improve further, although such scenarios have not been investigated here due to time constraints.

13 Various scenarios resulting from triggering Strategy W4 16 hours earlier were investigated as part of Option A. There is some flexibility under Strategy W4 as to the rate at which gate openings are undertaken to stabilise the dam level. An early transition to Strategy W4 may have either worsened or improved the severity of flooding downstream of Wivenhoe Dam, depending on the rate of gate opening adopted. Slower gate openings under an early Strategy W4 scenario would have improved flood impacts, but would also have required information about the timing and magnitude of the flood peak that was unavailable at the time.

14 There are a number of plausible alternative scenarios that could have been undertaken under Strategy W4 that would have resulted in worse (higher) flood levels downstream of Wivenhoe Dam.

15 Whilst the flood level reductions indicated in Table 6 would have been a benefit and reduced flood damages if they had been achieved, generally such scenarios could not have been reasonably achieved with the information available at the time and under the current operating strategies stipulated by the Manual. Nonetheless, these scenarios highlight that for this event, earlier increases in releases from Wivenhoe Dam during 9 and 10 January could have reduced the eventual peak outflow and the resulting severity of flooding experienced downstream.

16 With the information available during their operations, and using the strategies defined by The Manual, WMAwater believe the Flood Engineers achieved close to the best possible mitigation result for the January 2011 flood event.

17 Care must be taken with interpreting these findings, which are based on a single large flood event, in relation to the effectiveness of the strategies in The Manual for dealing with future events, some of which will be larger. WMAwater consider that the recommendations relating to gate operation strategies in the Report to the Queensland Flood Commission of Inquiry in May 2011 (Section 9.2, Reference 4) are further supported by the findings in this report, namely that:

a. *"Alternative gate operation strategies for flood mitigation should be reviewed … for a full range of flood events, with consideration of average annual flood damages resulting from each strategy."*

b. *"The review of gate operations should place particular emphasis on the hard transition between the W3 and W4 strategies. Modifications that specify an increasing target discharge at Moggill once key criteria are either reached or predicted to be reached should to be investigated."*

2. INTRODUCTION

2.1. Scope of the Report

18 WMAwater's work scope is defined by a letter from the Commission dated 17 June 2011 (ref: DOC20110617), as quoted below:

I write to confirm the Commission requests that you review the hydrodynamic model being developed by SKM for Seqwater. Further the Commission requests that if possible, you use the model to answer the following questions:

1. *To what extent was flooding (other than flash flooding) in the mid-Brisbane River, the Lockyer Valley, Ipswich and Brisbane during January 2011 caused by releases from the Somerset and Wivenhoe Dams?*
2. *To what extent did the manner in which flood waters were released from the Somerset and Wivenhoe Dams avoid or coincide with peak flows from the Bremer River and Lockyer Creek?*
3. *Had the levels in Somerset and Wivenhoe Dams been reduced to 75 per cent of full supply level by the end of November 2010 (both with and without amendments to the trigger levels for strategy changes in the Wivenhoe Manual) what impact would this have had on flooding?*
4. *What effect would the implementation of different release strategies (to be identified by you) have had on flooding?*

Please include in your report a detailed assessment as to any difficulties with the model, together with suggestions as to how (if at all), those difficulties may be remedied.

Please also provide a detailed explanation as to the limitations upon any results which you may obtain using the model.

19 WMAwater have undertaken the following tasks to address this scope of work, in chronological order:

a. Reviewed Mike11 modelling work done by SKM for Seqwater;
b. Made an assessment of issues with the model;
c. Provided suggestions as to how any issues identified in the above step might be remedied;
d. Provided, if possible, answers to Questions 1 and 2 from the Commission, as indicated above;
e. Run a range of alternative scenarios gate release and prior dam storage scenarios to assess impact on downstream flood behaviour; and
f. Provided discussion as to the limitations of the results achieved in modelling these scenarios.

2.2. Sequence of Events

20 The sequence of events that have occurred throughout the hydrodynamic model review and subsequent scenario analysis work is as follows:

a. 24 June 2011 5:35 pm – SKM advise WMAwater that model files are available for download (Version 1 SKM model);
b. 1 July 2011 10:30 am – Conference call including SKM, Seqwater and WMAwater. WMAwater provide preliminary feedback to SKM in regards to the reviewed model;
c. 4 July 2011 approximately 3 pm – Conference call between WMAwater and SKM in regard to WMAwater's preliminary findings of July 1;
d. 5 July 2011 approximately 11:30 am – WMAwater call to SKM to discuss progress toward revised model;
e. 5 July 2011 3 pm – Conference call between WMAwater, SKM and Seqwater in regard to model revisions and revised calibration. General concurrence on the model build and calibration of lower Brisbane River elements is achieved;
f. Model (Version 2 SKM model) subsequently issued to WMAwater (after COB 6 July 2011) and utilised for scenario modelling presented herein; and
g. 13 July 2011 – WMAwater issue report to Commission.

3. AVAILABLE INFORMATION

3.1. Data Relied Upon

21 Model files utilised are listed in Section 4.6. Please note the files listed are Version 2 model files for Case 1 – January 2011 calibration. Prior to Version 2 of the model SKM supplied WMAwater with Version 1 of the model.

22 Spreadsheets from Seqwater containing gate operations rating curves and flood event data, as reported in Reference 7.

3.2. Reliance Statement

23 This report has been prepared on behalf of The Commission, and is subject to, and issued in accordance with, the provisions of the agreement between WMAwater and The Commission.

4. MODEL REVIEW

4.1. Introduction

24 The model review focuses on the Mike11 hydrodynamic model (Mike11 version 2009) built by SKM (based on Seqwater's 2005 model) and calibrated to the January 2011 event. Two versions of the model are discussed. WMAwater have been involved from the point at which SKM first provided Version 1 of the model for revision up until SKM made Version 2 of the model available to WMAwater for further review and scenario modelling.

25 A general assessment of any hydrodynamic model will typically consider a variety of elements depending on the application. These elements generally include:

a. The model extent, location of boundaries, cross-sections, roughness values and other parameter settings used, boundary inputs and structure implementation;
b. Mass balance;
c. Stability;
d. Run-time (indicative of overall build and stability);
e. Calibration results; and
f. Fitness for purpose.

4.2. Seqwater 2005 Mike11 Model

26 SKM also provided a 2005 version Mike11 model previously developed by (or for) Seqwater. This same model is reviewed in SKM's report with findings and details presented in Appendix B of SKM's report (Reference 2). The SKM review found that the model was not in a condition suitable for use within Seqwater's overall flood forecasting system or for the establishment nor extension of stream gauge rating tables (in particular for larger events). Key shortcomings of the model, as noted in SKM's report are:

a. Cross-sections do not adequately represent the floodplain and include false areas of conveyance (page 73 and figures B-1 and B-2);
b. Improper schematisation of structures in some cases (e.g. Centenary Highway Bridge at Jindalee);
c. Roughness values were in excess of standard acceptable values when compared to available resources such as Chow (1959), for example;
d. Some errors in applying roughness to specific cross-sections;
e. A reliance on hot starts and steady state flow inputs to improve stability; and
f. Relatively small time step not suited to optimal run time.

27 WMAwater did not undertake a review of the 2005 version of the model.

4.3. Version 1 SKM Model – Case 1 (January 2011 Calibration)

28 The WMAwater review of the Version 1 model found some issues with the model build which undermined the legitimacy of calibration and scenario runs as presented in the recent report by SKM and Seqwater (Reference 2). Figure 1 to Figure 4 demonstrate the issues which are summarised below:

a. Flow velocities modelled were unrealistically high (cross-sectional average velocities greater than 10 m/s);
b. Model stability was poor;
c. Roughness values were artificially high, presumably to compensate for high flow velocities; and
d. Run time was excessive.

29 Overall the issue which led to most problems in the model was the resistance approach used. In summary, there are two possible issues with the use of the “Resistance Radius” approach (as adopted in the Version 1 SKM model). First, when used in conjunction with relatively high flow zone multiplier values it leads to artificially constrained cross-sectional area within the processed value table of the cross-section (*.xns11) files used in Mike11. Second, the “Resistance Radius” approach is less suited to deep cross-sections with steep side slopes as are found in many locations on the Brisbane River. Through some combination of these two mechanisms very high mean velocities were modelled (see Figure 1). The high modelled velocities were approximately 4-5 times what was achieved using an alternative resistance formulation and compared to gauged velocities at Jindalee were demonstrably false. The high modelled velocities in turn seemed to exacerbate stability issues and require the higher roughness values observed in the model. Please note that velocities presented are average velocity over the entire modelled cross-section, not peak in-bank velocity.

Figure 1: Velocity Time Series (modelled) at Jindalee (SKM Model Version 1)

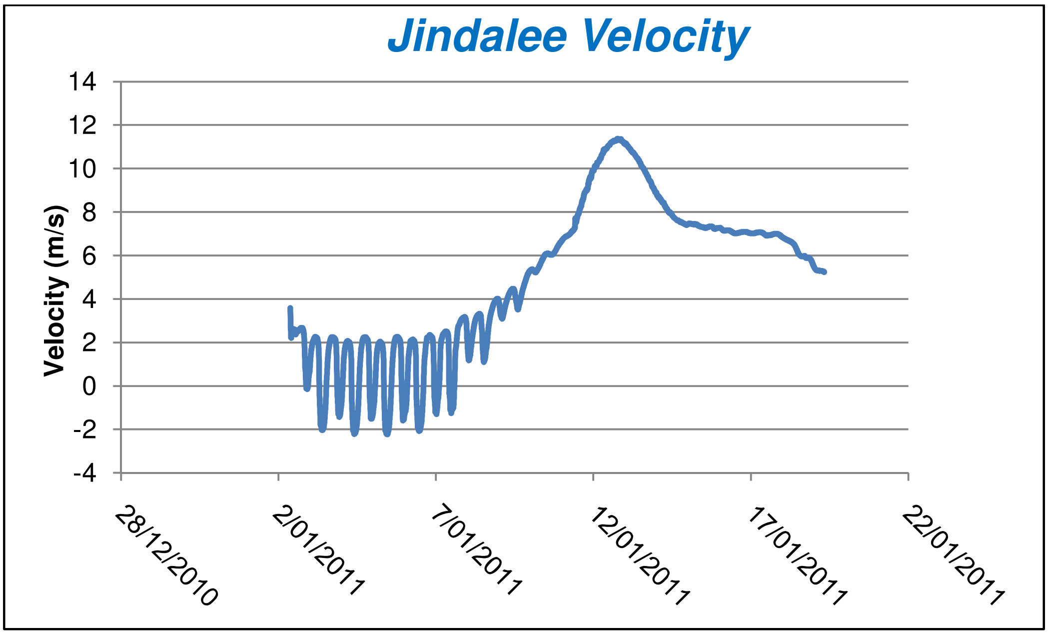


Figure 2: Discharge Time Series (modelled) at Mt Crosby Bridge (Version 1)

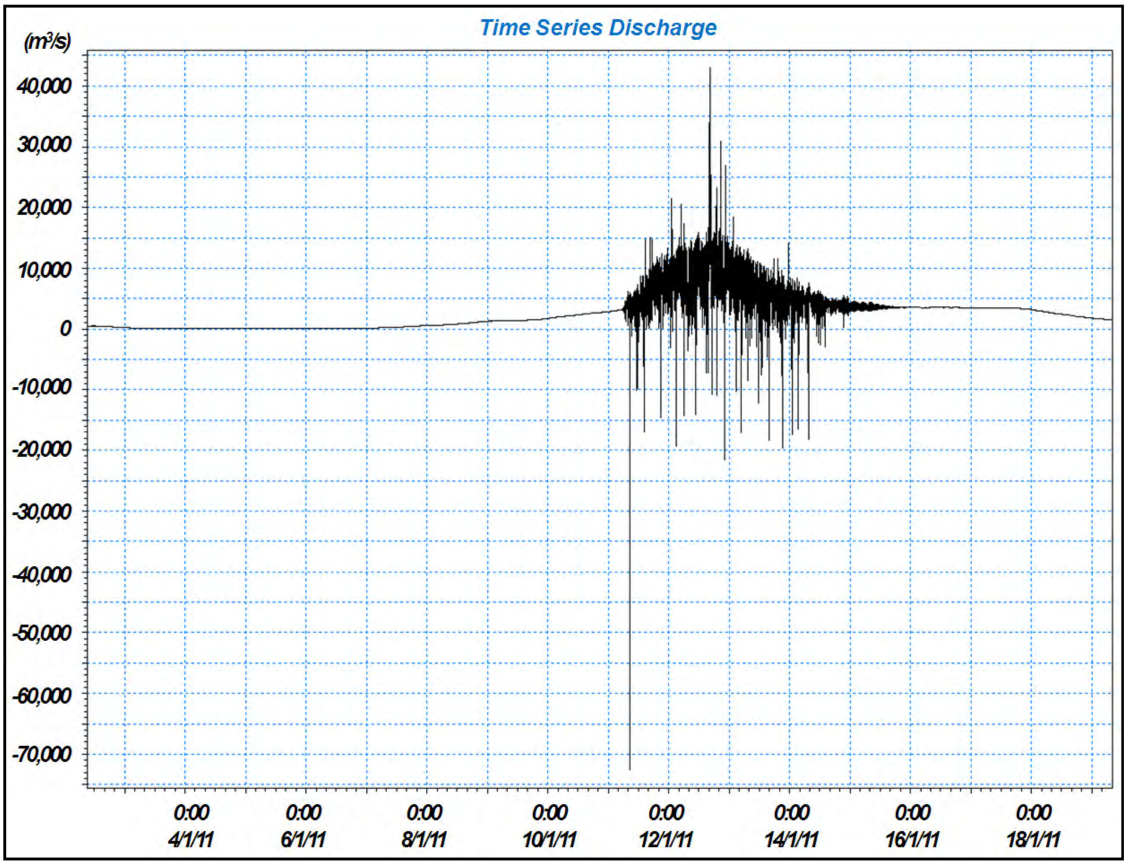


Figure 3: Water Level Time Series (modelled) at Mt Crosby Bridge (Version 1)

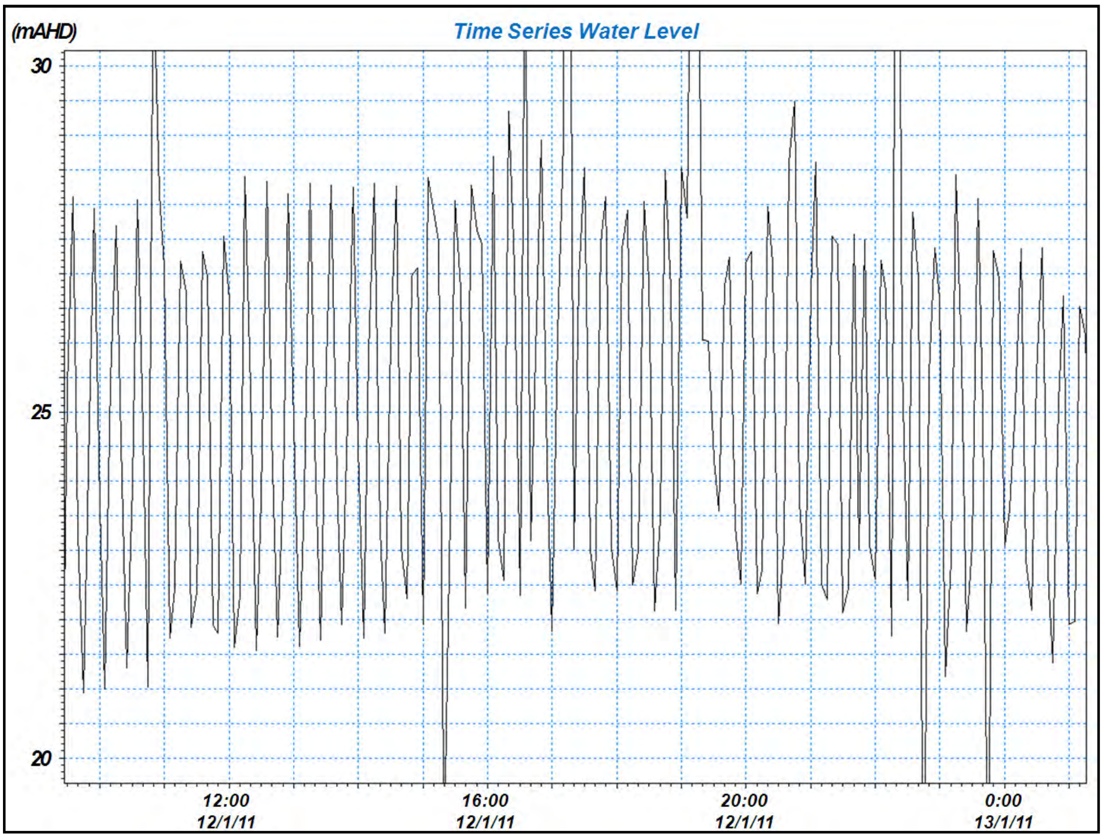
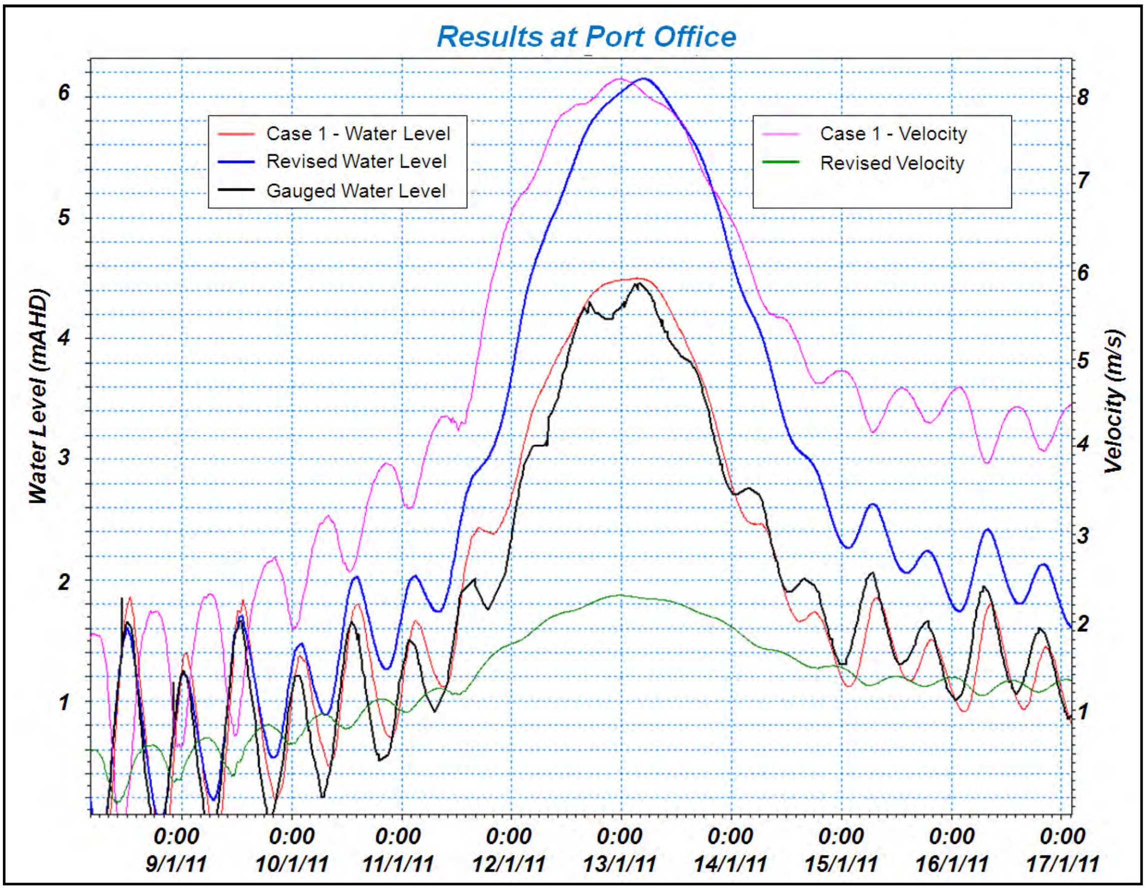


Figure 4: Model Results (Version 1) at Port Office versus "Fixed" model results



30 Figure 2 and Figure 3 indicate the Version 1 model's lack of stability with discharge fluctuating between 40,000 m^3/s and negative 70,000 m^3/s in Figure 2 (actual discharge peaks at approximately 9,000 m^3/s) and the water level fluctuating between approximately 21 mAHD and 28 mAHD in Figure 3 (actual peak water level is approximately 26 mAHD). Note both results are at Mt Crosby Bridge and both results are indicative of the worst of the stability issues in the model.

31 As part of the review process the Version 1 model was altered to a different resistance method and this reduced maximum cross-sectional average velocities in the Brisbane River from 10 m/s to approximately 2.5 m/s. The impact this change had on model results in the Version 1 model is shown in Figure 4. Note that whereas previously, with the unreasonably high velocities, the modelled water level was a good match for the gauged water level at Port Office, when the velocities are a more reasonable value (see "Revised Velocity" versus "Case 1 Velocity" in Figure 4), the modelled peak water level increases from 4.5 mAHD to approximately 6.2 mAHD. This demonstrates that the parameters used in the Version 1 model did not produce a reasonable match for both water level and velocity at the Port Office gauge. When there was a good match for water levels, velocities were too high, and when velocities were at a reasonable magnitude, water levels were too high.

32 WMAwater provided early feedback in regard to the model issues. SKM then proceeded to rapidly address these issues and provided WMAwater with a revised model late on 6 July 2011 (Version 2). Further review work herein will focus on Version 2 of the model as this is the model version used in all subsequent analysis carried out by WMAwater. It is noteworthy however that previous results obtained using the Version 1 model, presented in SKM's report (Reference 2) will require revision in light of the serious issues identified with Version 1 of the model.

4.4. Version 2 SKM Model – Case 1 (January 2011 Calibration)

33 The review of the SKM model (Version 2) was required within a limited period of time. For this reason the scope of the review is limited. In the first instance the review seeks to describe and then assess the model generally. Also the calibration of the model is assessed and comments are made as to the limitations of the model. The main purpose of the review was to assess whether the model was suitable for answering the questions put to WMAwater by the Commission.

4.5. Review Caveats

34 The review does not extend to the Lockyer Creek and Bremer River model elements as SKM make no assertion in regard to these parts of the model. Model behaviour upstream of Mt Crosby bridge is also not focussed on as the boundary conditions method used is not suitable for areas upstream of this point. This issue is further discussed below.

4.6. Files Provided and Reviewed

35 Files reviewed are as follows. Please note that 2005 Seqwater model files were also provided but not reviewed given limited time available and given SKM's review (Reference 2) had already deemed them unsuitable for use in modelling of the January 2011 event.

Table 2: Files reviewed as submitted by SKM

Case1_20110706-Info.Log	06/07/2011 5:46 PM	1 MB	LOG File
Case1_20110706-SimStat.Log	06/07/2011 5:46 PM	1 MB	LOG File
Case1_20110706.Log	06/07/2011 5:46 PM	1 MB	LOG File
Case1_20110706.omi	06/07/2011 5:46 PM	1 MB	Unix E...le File
Case1_20110706.res11	06/07/2011 5:47 PM	55.6 MB	RES11 File
Case1_20110706.sim11	06/07/2011 5:47 PM	1 MB	SIM11 File
Case1_20110706HDAdd.res11	06/07/2011 5:49 PM	62.9 MB	RES11 File
Case1.bnd11	06/07/2011 5:46 PM	1 MB	BND11 File
Jan 2011 Tide.dfs0	06/07/2011 5:49 PM	1 MB	DFS0 File
Jan2011_Case1_20110705.dfs0	06/07/2011 5:49 PM	1 MB	DFS0 File
MTCROSBY1_proc.dat	06/07/2011 5:50 PM	2.1 MB	Unix E...le File
MTCROSBY1.dfs2	06/07/2011 5:50 PM	10.5 MB	DFS2 File
SKM11_4.0.nwk11	06/07/2011 5:50 PM	1 MB	NWK11 File
SKM11_4.1.HD11	06/07/2011 5:50 PM	1 MB	HD11 File
SKM11_4.3.xns11	06/07/2011 5:50 PM	12.6 MB	XNS11 File

36 The main files constituting a Mike11 model are as follows:

a. Simulation file (*.sim11) – coordinates other model files found below and also dictates the period over which the simulation will occur, time step and the name of the result file and the save increment;
b. Network file (*.nwk11) – defines the spatial location of the model, the linkage between model branches and structures included in the model (bridges, weirs and culverts);
c. Cross-section file (*.xns11) – defines the topography of the branches modelled via a series of cross-sections with location along the branch specified by “chainage”;
d. Boundary file (*.bnd11 with linked time series files (*.dfs0) for boundary inputs) – indicates where inputs such as tidal data or inflow hydrographs should be applied within the model network and also links to the time series files which contain the boundary condition information ; and
e. Parameter file (*.hd11) – contains a variety of parameters, with the global roughness value being the most important of these. Also contains parameter settings pertaining to the solution scheme such as delta (forwardness value) and the iteration criteria.

4.7. Description of the Model

37 The overall model consists of 91 branches although all but 17 of these are link type branches rather than modelled creeks/rivers. The main focus of this review is on the Brisbane River section of the model from downstream of the Wivenhoe Dam spillway (chainage 930,070 m) to Moreton Bay (chainage 1,078,525), a total distance of approximately 149 kilometres. This reach is described by approximately 240 cross-sections. Only one structure is modelled on the Brisbane River and this is the Mt Crosby Bridge (chainage 988,150 m).

38 Key landmarks in the model are as follows. All landmarks relate to the Brisbane River unless otherwise specified:

a. Confluence of Brisbane River with Lockyer Creek (chainage 931,020 m);
b. Confluence of Brisbane River with Bremer River (chainage 1,006,200 m);
c. Lowood Gauge Station (936,820 m);
d. Savages Crossing Gauge Station (948,120 m);
e. Mt Crosby Gauge Station and Bridge (approximately 988,000 m);
f. Ipswich Alert Gauge Station on the Bremer River (1,014,640 m);
g. Moggil Gauge Station (1,006,300 m);
h. Jindalee Gauge Station (1,026,170 m);
i. Oxley Gauge Station (1,040,090 m); and
j. Port Office Gauge Station (1,055,280 m).

39 The main locations of boundaries within the model domain are at:

a. The upstream end of the Brisbane River representing Wivenhoe Dam releases (chainage 930,070 m);
b. Immediately upstream of Mt Crosby Bridge where all upstream flow not inclusive of Wivenhoe Dam releases is applied to the model (chainage 988,000 m);

c. Amberley and Walloon inputs within the Bremer River; and
d. Gauged tidal data applied at the downstream extent of the model.

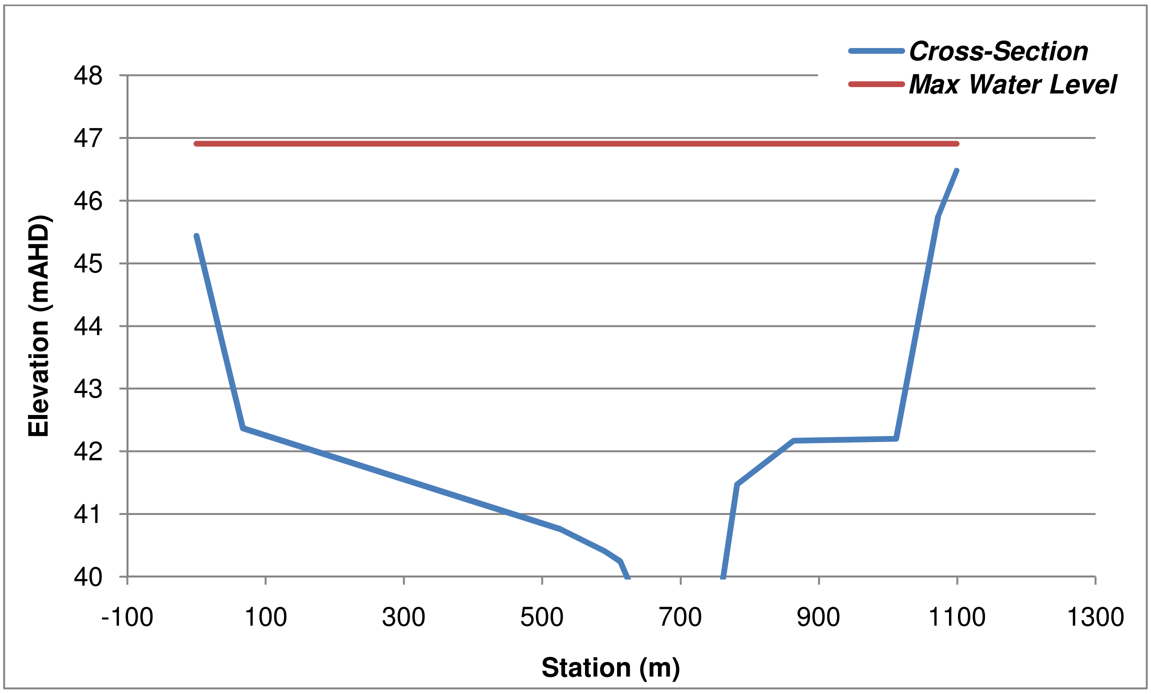
40 Generally the Brisbane River is schematised as one main flow branch with areas of off-branch storage represented in 28 discrete locations, distributed over the river from chainage 948,254 m (in the upstream) to chainage 1,066,425 m (in the downstream). Off-branch storage is represented via linked side storage areas (described in the *.nwk11 file using elevation / area relationships) and presumably this information was extracted from a digital elevation model (DEM) derived from aerial LIDAR survey. The amount of storage provided at these locations has not been reviewed nor has the capacity of linking structures to transfer flow (or the height at which such transfers occur).

41 In numerous other cross-sections significant floodplain area is modelled as being part of the main flow path, and this approach will in many cases over estimate conveyance and underestimate attenuation from overbank areas of floodplain. This will tend to lead to modelled hydrographs travelling downstream relatively quickly when compared to gauged flow.

42 Cross-sections, as per SKM's report (June, 2011) are composites of in-bank details surveyed previously (specific date unknown but TOPO-ID is “2003-x”) and overbank data extracted from a 3 m DEM (survey date unknown).

43 An issue noted with regard to the model cross-sections is that in some cases the cross-sections contain an inadequate amount of the floodplain and as such are subject to extrapolation error. This situation will typically overestimate peak flood level and lead to underestimation of system attenuation. An example is shown in Figure 5 for a cross-section at chainage 934,270 m on the Brisbane River, approximately four kilometres downstream of the Wivenhoe Dam outlet. Note that peak water level exceeds the defined topography. In such a situation Mike11 extrapolates vertically from the defined top left bank and top right bank. This issue was only observed in a small minority of model cross-sections and is unlikely to affect the model results significantly.

Figure 5: Mike11 Cross-Section with insufficient floodplain detail



44 Model roughness used throughout the model is based on "Total Area Hydraulic Radius". This approach is reasonable, particularly given that in many cross-sections, substantial portions of the flow remains within steeply banked flow channels (Reference 5).

45 Roughness utilised throughout the model is established via a combination of a global roughness value set in the *.hd11 file and lateral roughness multipliers set in the *.xns11 file. Effective roughness values (as Mannings 'n') used in the modelling have been summarised by SKM as per Table 3 below.

Table 3: SKM Roughness Values applied to Version 2 Model

Brisbane River model reach		Mannings 'n' Value	
From (m)	To (m)	Channel	Floodplain
930,070	950,270	0.074	0.084
951,200	963,595	0.053	0.084
964,170	994,760	0.055	0.105
995,690	1,002,785	0.053	0.084
1,003,275	1,019,490	0.042	0.084
1,020,115	1,025,590	0.047	0.084
1,026,170	1,036,770	0.045	0.084
1,036,915	1,078,525	0.024	0.084

46 Whilst it is likely that in some cases higher roughness values have been applied than might otherwise have been used, in order to aid model attenuation, i.e. as a solution to

schematisation and cross-section issues described above, generally the values used are reasonable and comparable to those found in the standard texts such as Chow (Reference 6). Lower in-bank roughness values are expected in downstream estuarine areas.

47 The main flow inputs to the model are as follows. The relative contribution of flow sources to total flow volume is discussed further in Section 5:

a. Wivenhoe Dam releases;
b. Other tributary Inputs upstream of Mt Crosby – these are lumped together in the “All inflows Mt Crosby” item in the flow time series file;
c. Bremer River inputs – there are several inflows within the Bremer River system but the main ones are Walloon and Amberley; and
d. Other miscellaneous tributary inputs – several relatively minor local flows are input into the model at appropriate locations.

4.8. Assessment of Calibration

48 As described above the calibration is valid only below Mt Crosby Bridge. Data available for assessment of the calibration includes the following:

a. mean measured velocities (via acoustic Doppler radar) at Jindalee stream gauge station during the event;
b. gauged discharge at Jindalee during event; and
c. recorded water level at Moggil, Jindalee, Oxley and Port Office.

49 Figure 6 to Figure 10 describe the calibration result. Overall the match between gauged and modelled water level is excellent at Moggil and Jindalee, particularly in regard to peak behaviour. The match is very good at the Port Office although the modelled peak does occur too early at this location. The match to mean velocity between modelled and observed data is excellent. Modelled discharge at Jindalee is also well matched with the model estimating discharge at close to 10,000 m^3/s, as per the gauging. The match to Mt Crosby is excellent but less relevant since this point was used to derive the input flow and also because it is located directly next to a major model boundary.

50 The model has a tendency to underestimate observed routing time, with the effect most evident at Port Office, the furthest distance (67 kilometres) downstream of Mt Crosby. The tendency of the model to have the flow arriving early relates to the likelihood that the model does not currently represent the storage of the system and resulting attenuation of flood flows, particularly between Jindalee and Port Office. The effect is slight however and likely exacerbated by the timing relative to the tide.

Figure 6: Comparison of gauged and modelled water level – Mt Crosby

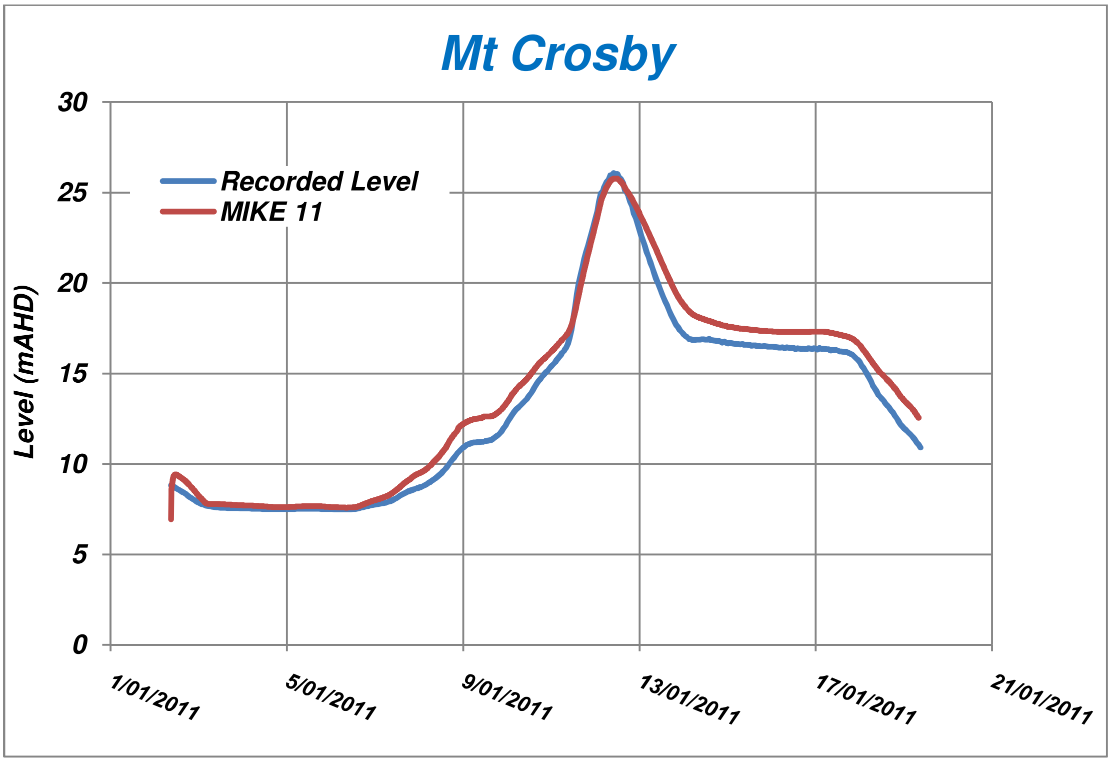


Figure 7: Comparison of gauged and modelled water level – Moggil

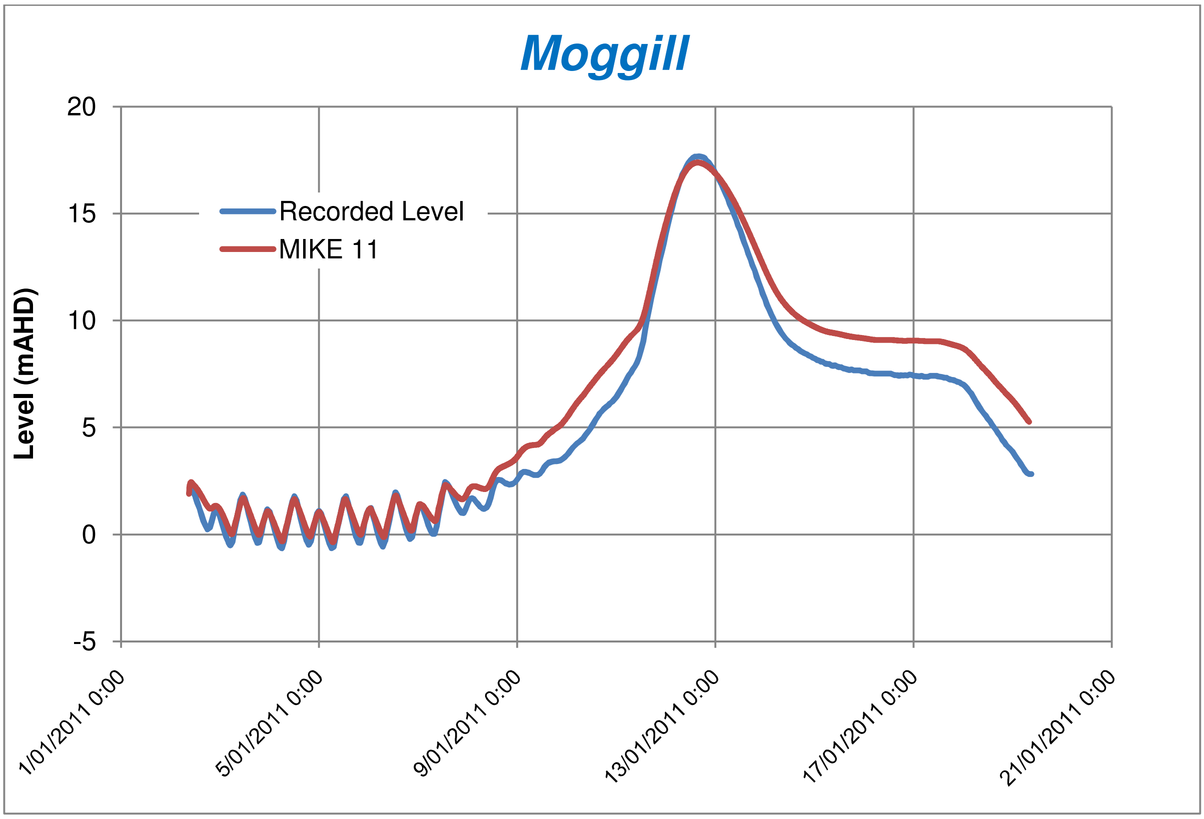


Figure 8: Comparison of gauged and modelled water level – Jindalee

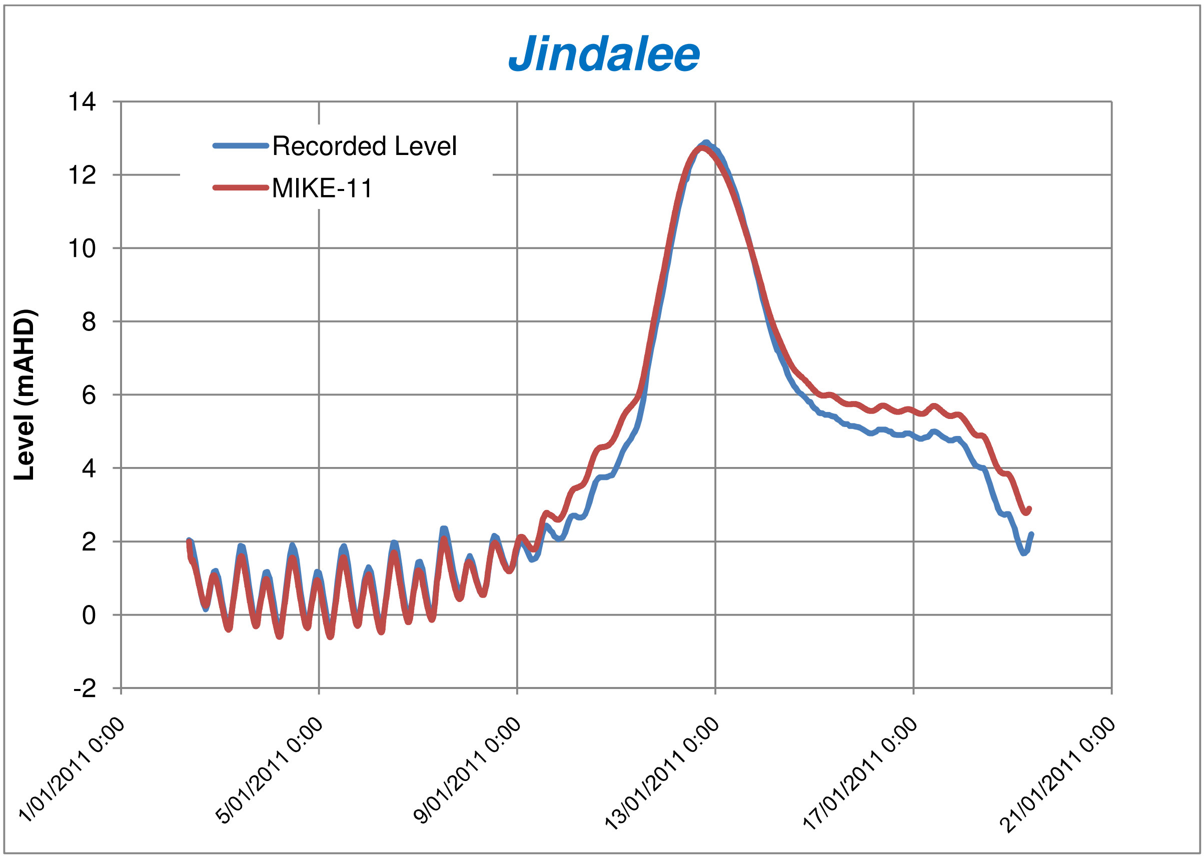


Figure 9: Comparison of gauged and modelled velocities

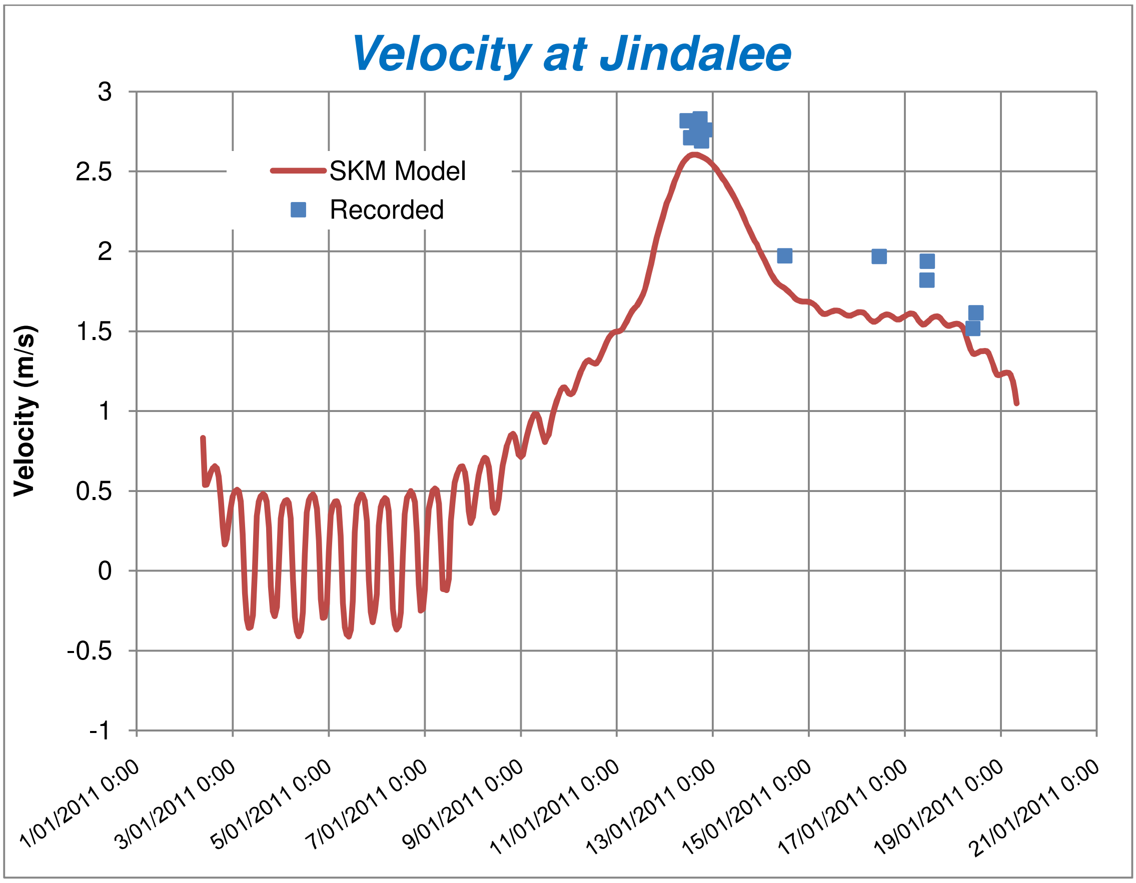
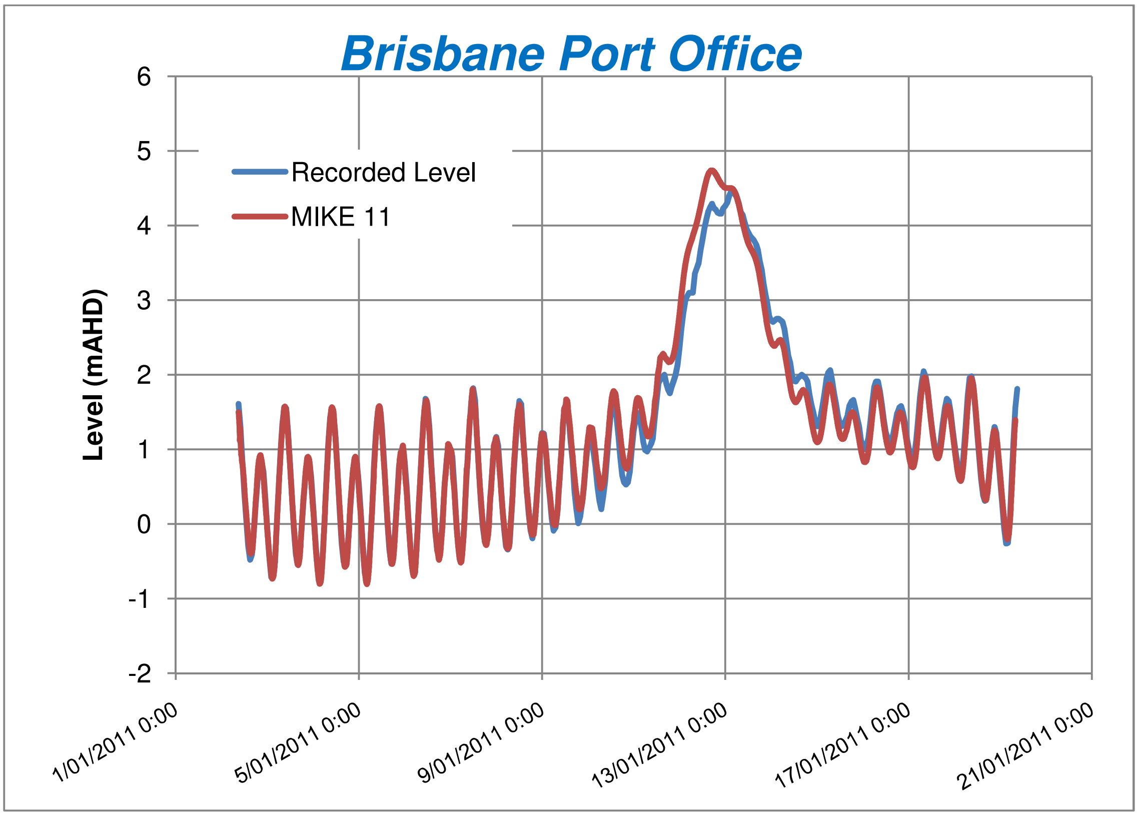


Figure 10: Comparison of gauged and modelled water level – Port Office

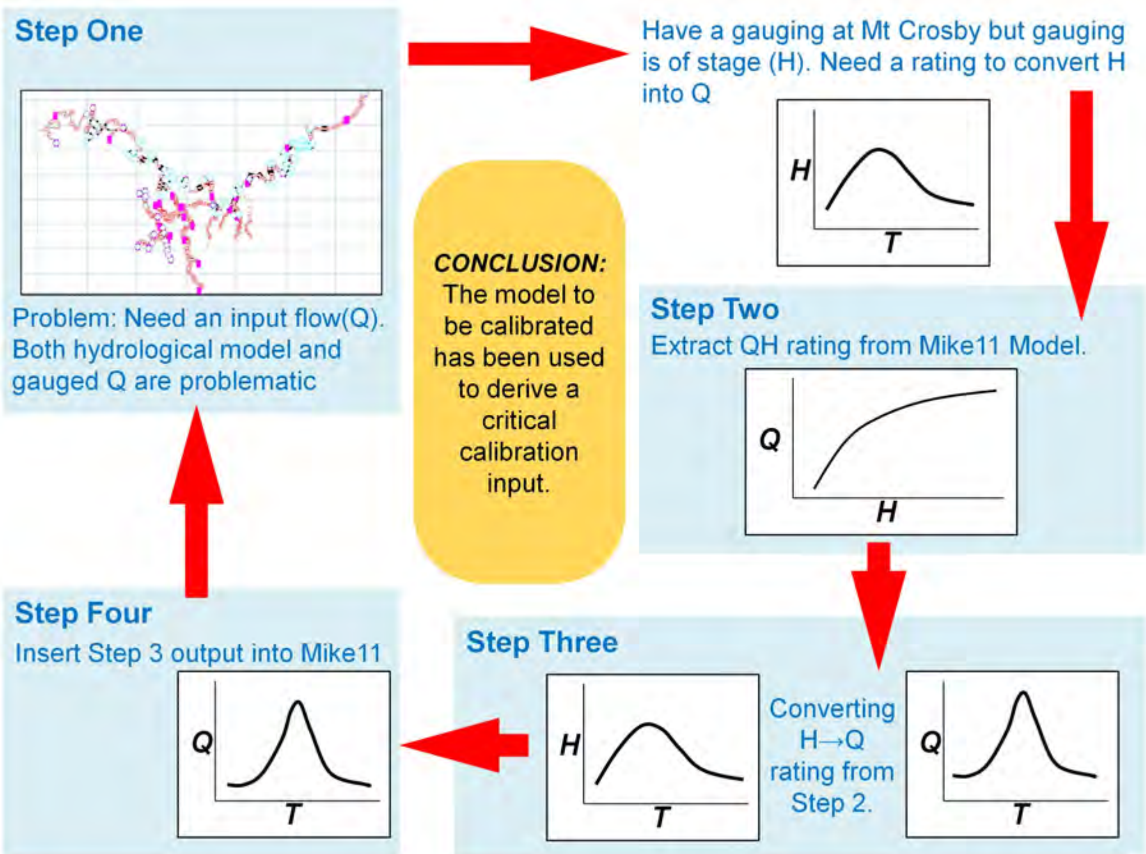


51 Overall the approach has provided a well calibrated modelling tool (between Mt Crosby and Moreton Bay) that can be used to answer the Commissions questions in regard to the January 2011 event and how flood levels downstream of the Dam were impacted by Wivenhoe Dam releases. Further it provides a basis for assessing how variations on the actual Wivenhoe Dam operations might have impacted peak flood level results downstream of the Dam.

4.9. Comments

52 Boundaries – Whilst the model domain includes the Brisbane River up to the outlet of Wivenhoe Dam the January 2011 event does not include tributary inputs such as Lockyer Creek inflows and other local inputs. Instead a lumped accumulation of inputs upstream of Mt Crosby Bridge (minus Wivenhoe flow), has been back calculated based on a Mike11 derived rating for the Mt Crosby stream gauge. Figure 11 describes the process and its inherent circularity i.e. the model to be calibrated is used to derive a key calibration input. Also the use of Mt Crosby as a major boundary is non-ideal because it doesn't allow for the adequacy of the model upstream of Mt Crosby to be assessed during the calibration. The same approach could presumably have been carried out at Lowood, approximately 50 kilometres up river, extending the overall portion of the model useful for analysis and interpretation.

Figure 11: Flow chart describing the derivation of All upstream Mt Crosby Input Hydrograph



53 Inadequate separation of floodplain storage from cross-section conveyance characteristics – It is noted that SKM have had a limited time to work on the model and that this has constrained their model development. Also the model build is based on a revision of the original Seqwater model and this dictated the methodology used to some extent. However the model as it currently stands appears to lack adequate attenuation, particularly between Mt Crosby and Port Office. It is likely that by incorporating parallel overbank flow paths, overall model conveyance could be more effectively limited and more attenuation/storage achieved. It is noted however that this model artefact may also be related to inadequate representation of the Bremer River which has not been included in work to date.

54 Inadequate detail in cross-sections – In some cases this can lead to extrapolation of cross-section data above supplied topographic information, leading to underestimation of flood attenuation and overestimation of water levels for a given flow (as per Figure 5). This issue was only observed in a small minority of model cross-sections and is unlikely to affect the model results significantly.

55 Non-optimal run time – Model run time is often an important indicator of general model build quality. The model currently utilises an adaptive time step, allowing the model to vary (based on criteria input by the modeller) the time step from between 30 seconds and 20 minutes. It is likely that the current criteria used with the adaptive time step mean that in reality the model runs using a 30 second time step most of the time. As part of the review the time step was changed to a fixed time step of 120 seconds and it was found that the model ran in approximately one quarter of the time relative to when the adaptive time step was used (total run time was less than four minutes) and that results are identical. It is likely that even shorter run times could be accomplished with further investigation and refinement of model schematisation.

4.10. Model Limitations

56 WMAwater consider the revised model (Version 2) fit for purpose for addressing most aspects of the Commission's questions (Section 2.1). Limitations of the Version 2 model are included below for completeness of the review process, indicating areas where attention may be required for further development of the model:

a) Quantification of the relative contributions of each system is difficult, as the interactions between flows at confluences are complex, particularly with regard to timing of peak flows and backwater effects. The flooding caused by the combined flow from all tributaries is therefore not strictly comparable to the hypothetical flooding resulting from the flow of each tributary. Because of this issue it is difficult to precisely resolve the impact Wivenhoe Dam releases have in addition to other flows by modelling Wivenhoe Dam flows only;
b) The method used to run the model (back calculation of flow input using a gauged hydrograph) is incompatible with use of the model in the Flood Forecasting system;
c) The model is unable to separately model Lockyer Creek flow and estimate its individual peak flow, volume and timing;

d) Reliability of Brisbane River model upstream of Mt Crosby is unproven by calibration;
e) Bremer River model is not successfully calibrated and results must be used with caution and as being indicative only; and
f) Given the model has been calibrated to the January 2011 event model but not validated against other historical floods, accuracy for other events is not established.

5. ASSESSMENT OF JANUARY 2011 FLOOD EVENT

57 Peak flow values for hydrographs input into the model include:

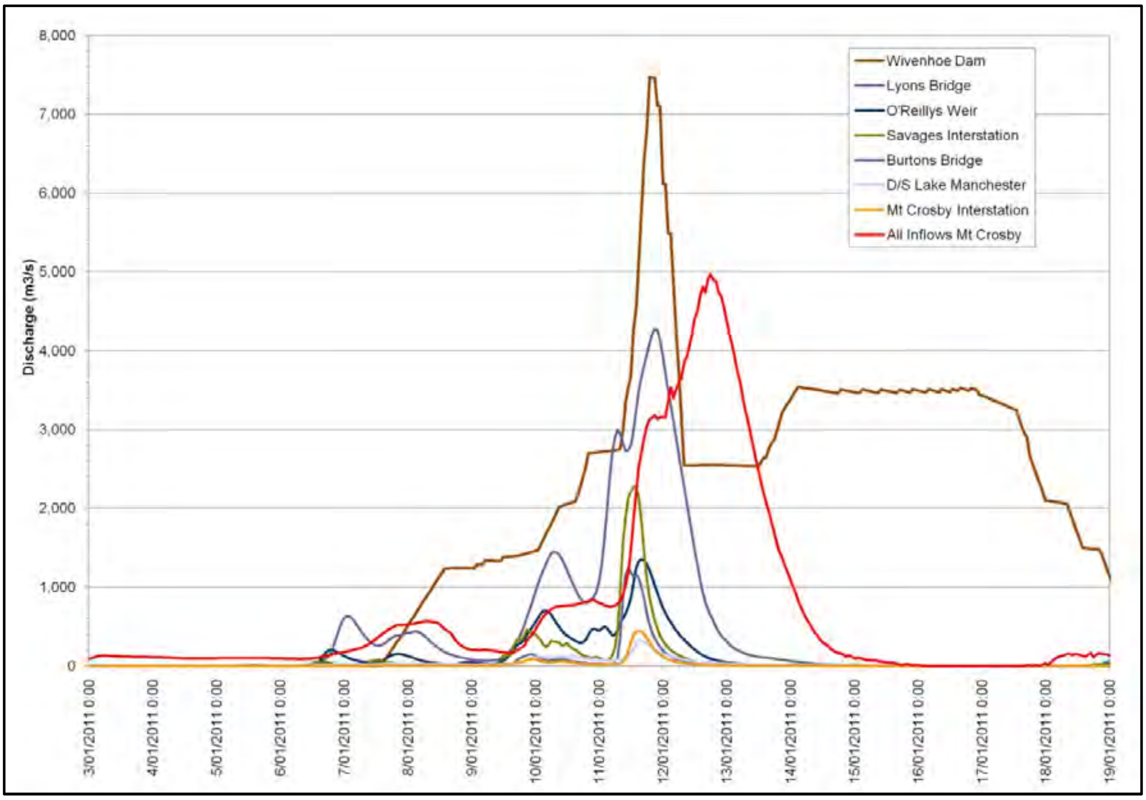
a. Wivenhoe Dam releases (peak flow 7,464 m^3/s);

b. All Inflows Mt Crosby (peak flow approximately 5,000 m^3/s); and

c. Bremer River (peak flow approximately 2,400 m^3/s).

Figure 12 shows hydrographs for the upper part of the model (upstream of Mt Crosby). Lockyer Creek (Lyons Bridge and O'Reillys Weir) and other tributary flows are shown. For the Case 1 model input, only "Wivenhoe Dam" and "All Inflows Mt Crosby" are used, as the latter combines the other inflows upstream of Mt Crosby Weir.

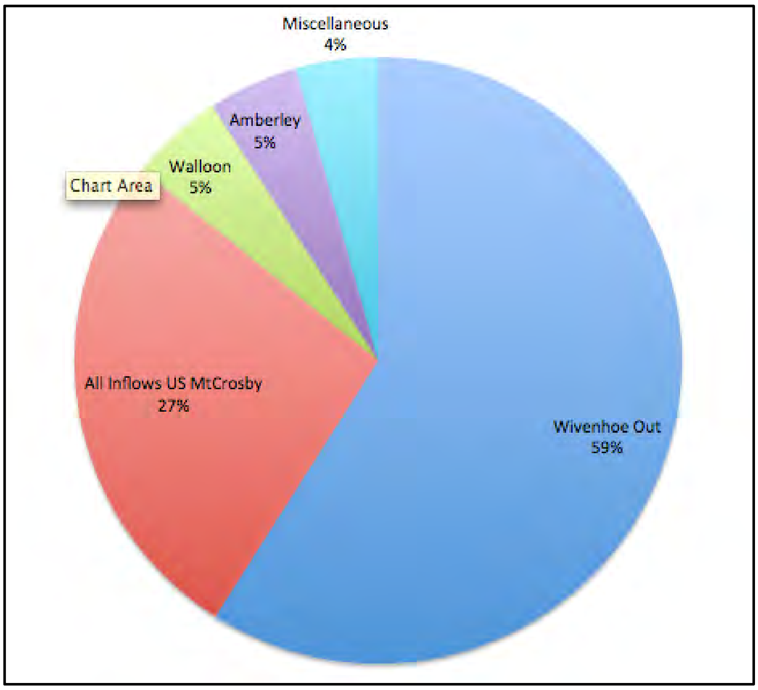
Figure 12: Comparison of various input hydrographs from upper part of model



58 Figure 13 describes the proportion of total flood volume contained in each of the model inputs from 9 January to 16 January inclusive (a period covering the majority of Wivenhoe Dam releases until the point where discharge was reduced after the sustained release of about 3,500 m^3/s).. Wivenhoe Dam releases constitute the greatest proportion of overall flow at 59%. Other inflows upstream of Mt Crosby account for 27%, the Bremer River inputs 10% and miscellaneous others account for the residual 4%. The bulk of the Wivenhoe Dam discharge was released after the flood peak, so these proportions are

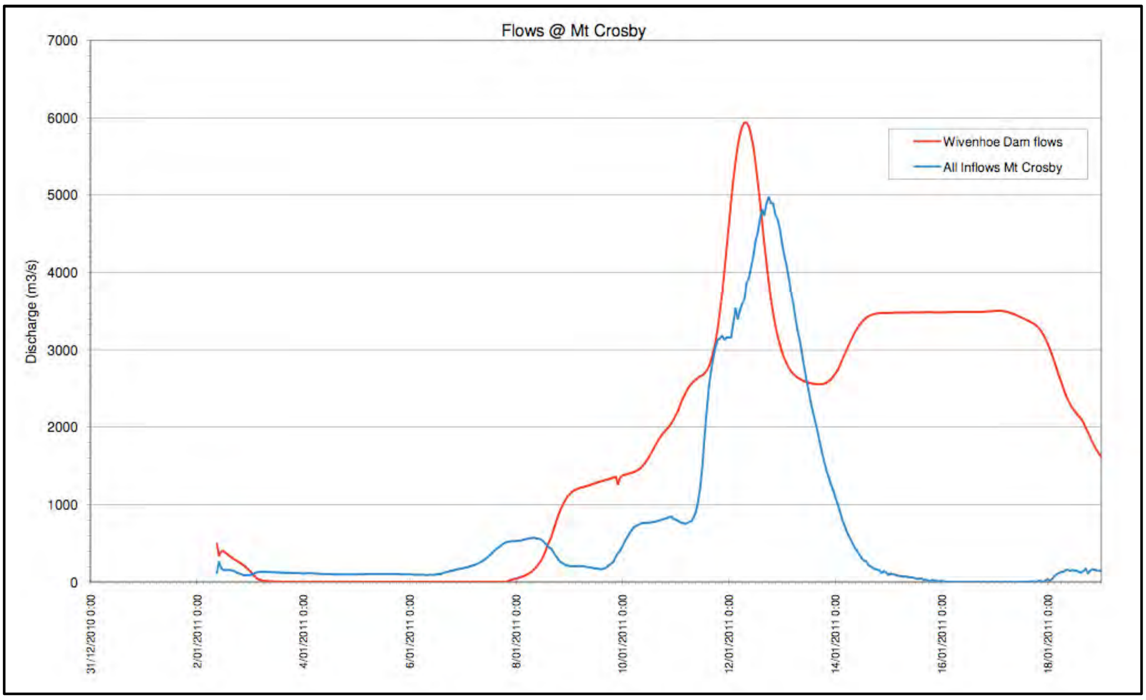
indicative of the total amount of flood runoff received from each of the tributaries, rather than the contribution to the flood peak

Figure 13: Percentage of flood volume from various sources 9-16 January 2011



59 SKM's calibrated model estimates that peak discharge at Mt Crosby was 9,500 m^3/s. Modelling of Wivenhoe Dam flow only indicates that Wivenhoe Dam peak discharge at Mt Crosby occurs 9 hours prior to the peak flow of other tributaries and 2.5 hours prior to the peak flow/stage at Mt Crosby. Figure 14 below shows a plot of routed Wivenhoe Dam flow versus flows from other tributaries ("All Inflows Mt Crosby") at Mt Crosby.

Figure 14: Comparison of timing of Wivenhoe Dam release flow and flow from other sources



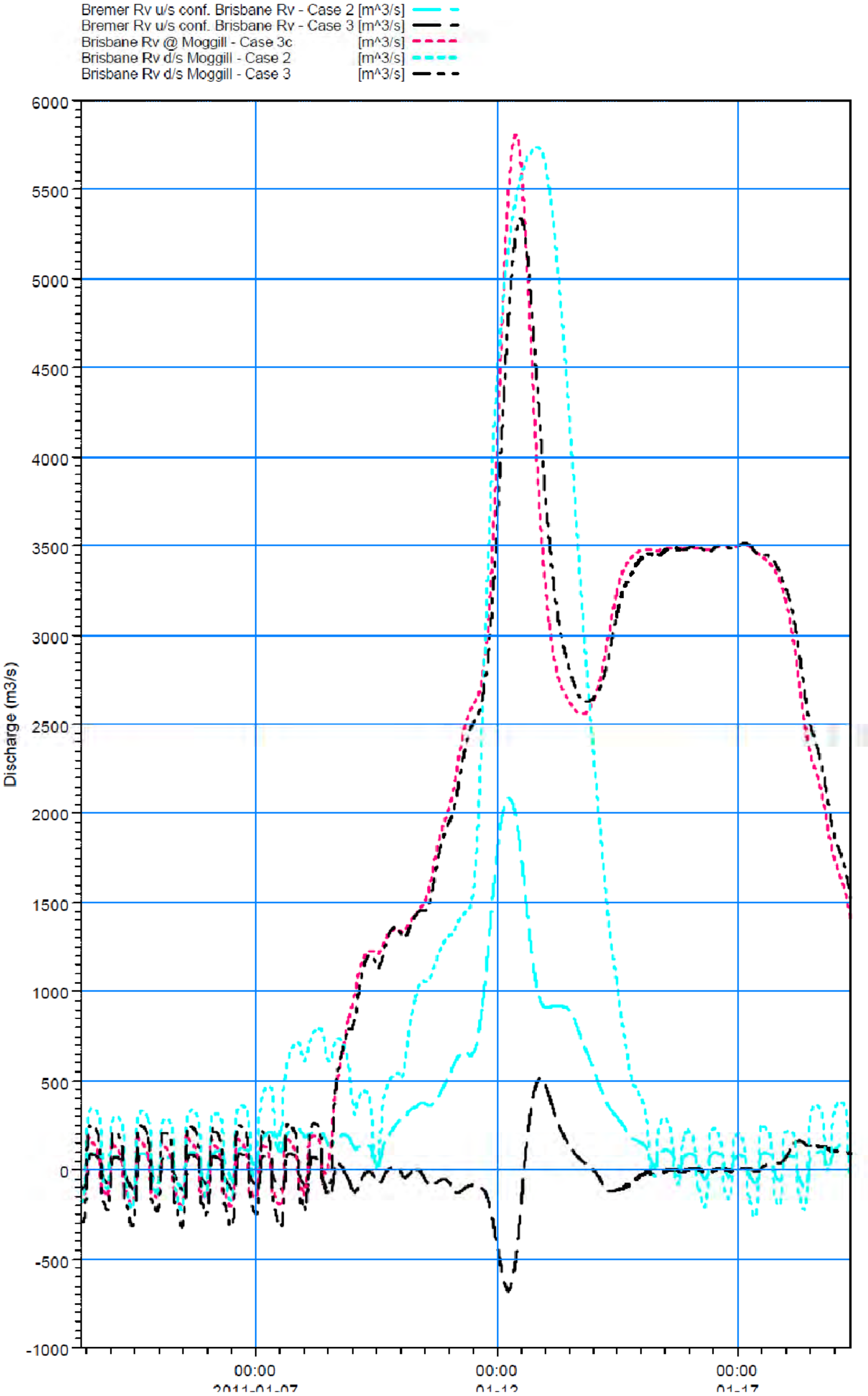
60 Analysis presented by SKM (Reference 2) presented two scenarios – Case 2 and Case 3. Case 2 was a model run of the January 2011 event without any Wivenhoe Dam contribution (but all other model flows as per the calibration run). Case 3 was again a model run of the January 2011 event although with no other flow contributions other than Wivenhoe Dam releases. A comparison of the two runs at Port Office (for stage) was used to indicate the relative contribution of Wivenhoe Dam and non-Wivenhoe Dam flows to resultant flooding.

61 The Case 2 / Case 3 comparison provide a basic understanding of relative contribution of Wivenhoe Dam and non-contributions to flooding during the January 2011 event. However the interactions between the various Brisbane River inflows are a significant component of the total observed flood behaviour, and removal of these interactions in Cases 2 and 3 results affects the outcomes of the comparison.

62 The most notable example is that in Case 3, the empty Bremer River system acts to attenuate the Wivenhoe Dam flow, as a significant portion of the peak discharge is diverted and stored in the lower Bremer River. Figure 15 shows the attenuating effect of the Bremer River by comparing Case 2 and 3 near the confluence of the Bremer and Brisbane Rivers. A negative flow up the Bremer River can be seen for Case 3 (Wivenhoe Dam flows only) whilst in Case 2 the Bremer River makes a substantial contribution to the Brisbane River flow.

63 In order to produce a more reasonable comparison WMAwater have run case 3c in which the additional storage provided by the Bremer River system has been removed.

Figure 15: Impact of Bremer Flows on Case 2 and 3 runs



64 Table 4 below indicates that at Moggil, Jindalee, Oxley and Port Office, Wivenhoe Dam (Case 3c) and non-Wivenhoe Dam (Case 2) flows result in approximately equivalent flood heights, indicating a roughly equivalent contribution to flood levels from both sources.

Table 4: Relative contribution of Wivenhoe Dam flows to peak flood levels downstream

Location	Case 1	Case 2	Case 3	Case 3c
	Peak Flood Level (mAHD)			
Moggill	17.6	12.5	11.8	12.4
Jindalee	13.1	8.6	7.9	8.4
Oxley	8.3	4.8	4.5	4.8
Brisbane	4.6	2.5	2.4	2.6

65 Figure 16 indicates that Wivenhoe Dam and Bremer River peak flows arrive at the confluence almost simultaneously. Ipswich flood behaviour is sensitive to backwater from Brisbane River flooding (caused by flows from either Wivenhoe Dam releases or other catchments below the dam). The exact additional flood height at Ipswich due to dam releases during the January 2011 event cannot however be ascertained with the current model. The susceptibility of large parts of the Bremer River system to backwatering are illustrated by Figure 17 which shows a relatively level pool at approximately 18 mAHD in the modelled profile of the Bremer River for the January 2011 event. Water level gauge observations at several stations within the Bremer River system (Figure 18) indicate the same, albeit at slightly higher heights.

Figure 16: Impact of Wivenhoe Dam flows on Bremer Flows and Levels at Ipswich

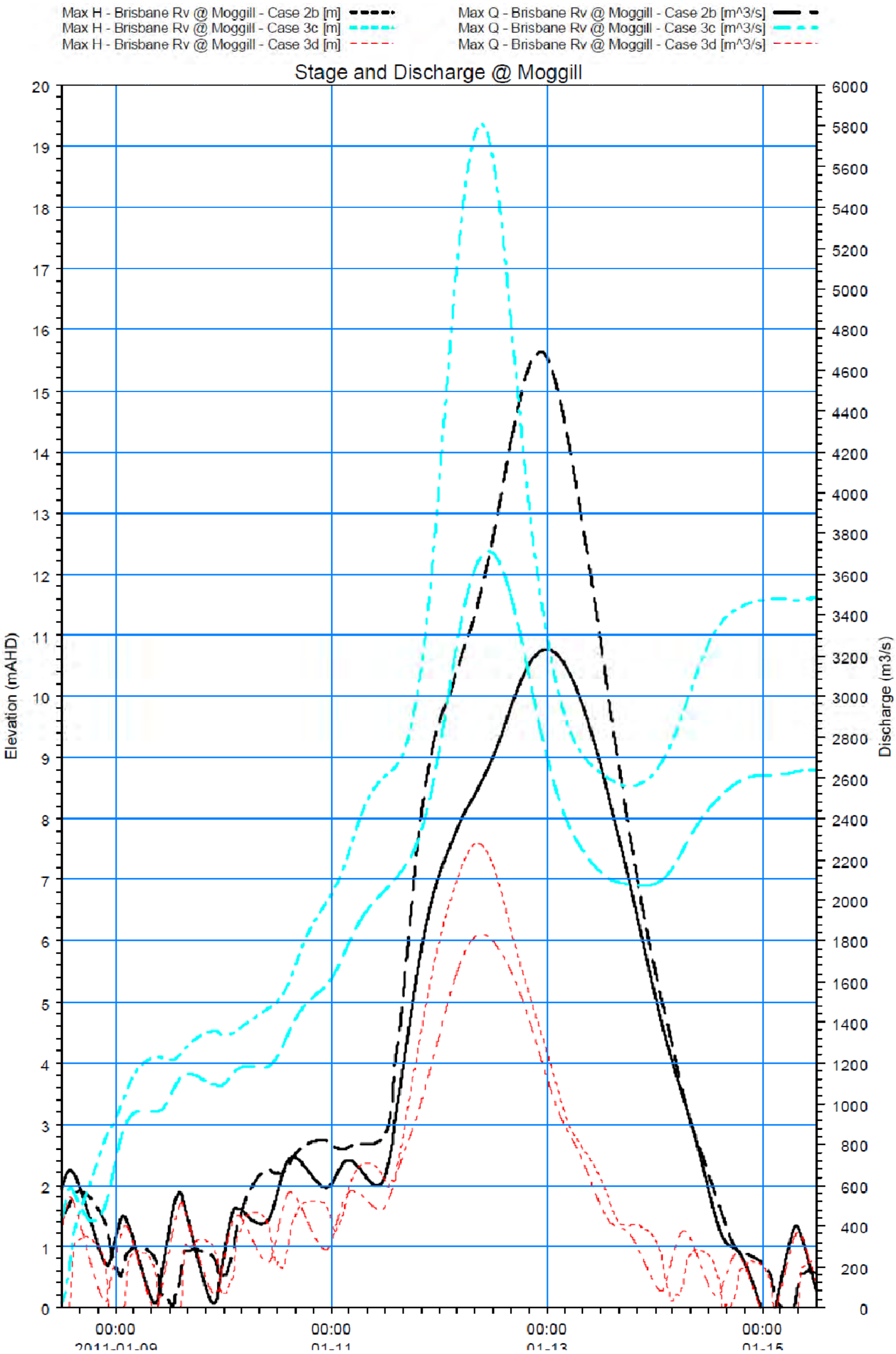


Figure 17: Flood level profile in Bremer River for Calibration Event

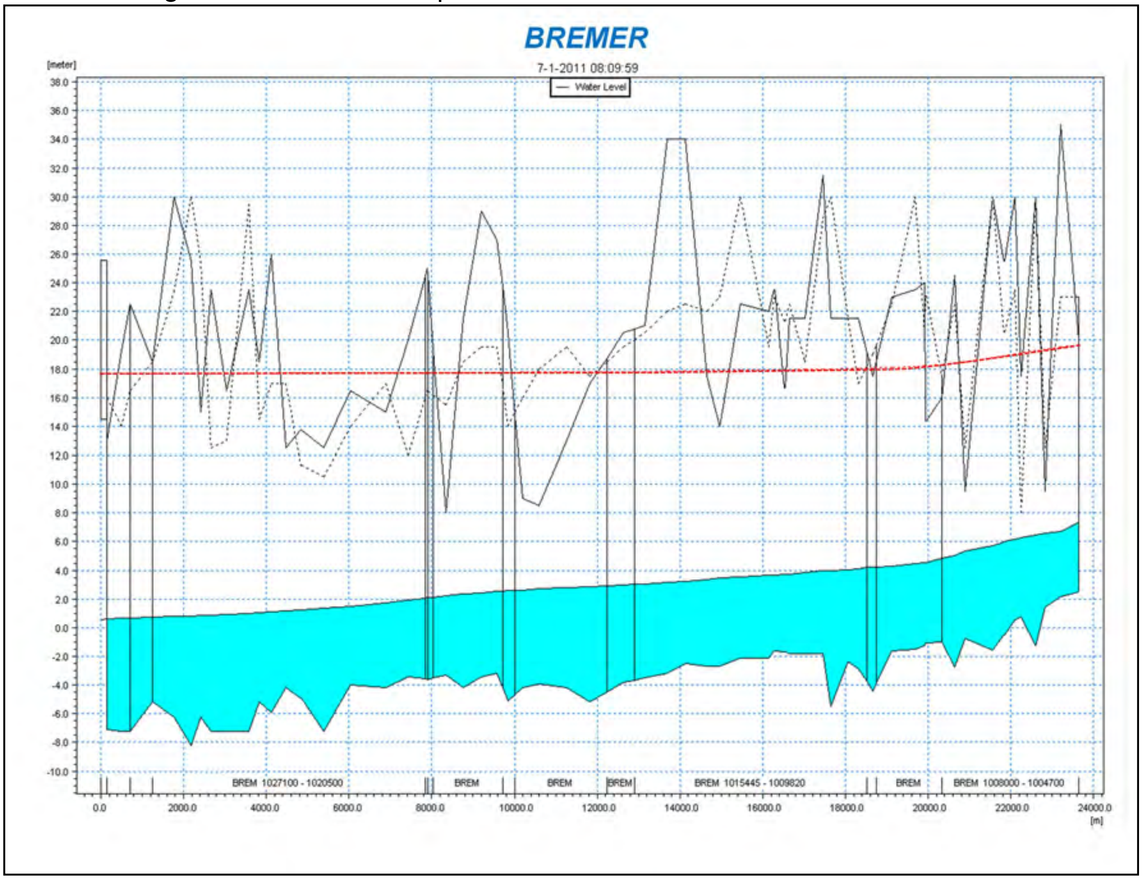


Figure 18: Backwatering of Bremer River System – Calibration Event

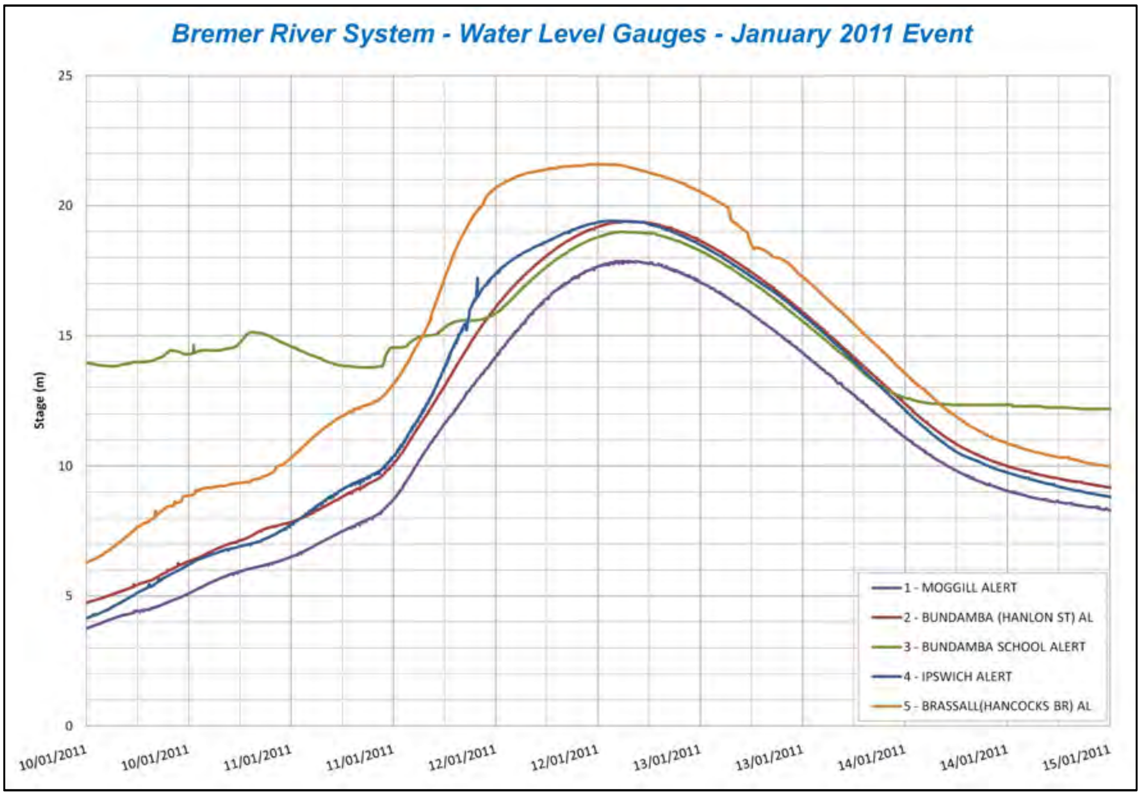
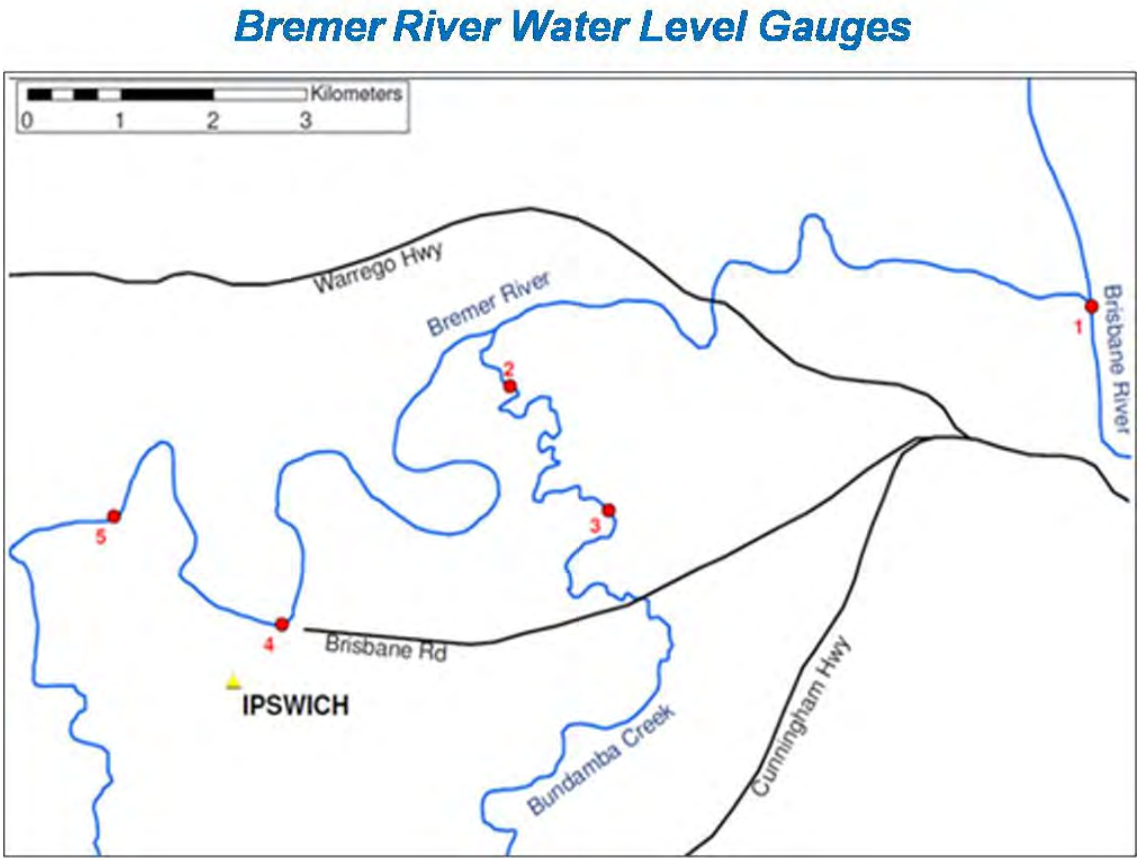


Figure 19: Bremer River System – Gauge Locations



6. ASSESSMENT OF ALTERNATIVE DAM OPERATION STRATEGIES

66 To address the Commission's questions about the potential effect of alternative dam release strategies on the January 2011 flooding, and the consequences of reducing the dams below full supply level prior to the flood, WMAwater investigated a range of hypothetical scenarios as follows:

a) Case 1 – The calibrated January 2011 model results supplied by SKM form the base case against which hypothetical scenarios are compared;
b) Option A – This scenario involves an earlier transition to Strategy W4 for the Wivenhoe Dam releases, at 4pm 10 January instead of 8am 11 January as actually occurred (16 hours earlier). This corresponds to the first prediction of a Wivenhoe Dam level exceeding 74.0 mAHD, based on modelling using scaled up forecast rain (Run 28, Appendix A, Reference 7).
c) Option B – The storage level in Wivenhoe Dam is assumed to be at 75% of FSL prior to the onset of the flood, but retaining the current operation rules.
d) Option C – This strategy explores the effects of increasing flows immediately after entering Strategy W3 to the upper allowable limit (keeping total flow at Moggill below 4,000 m^3/s).
e) Option D – An optimised release strategy, with the full benefit of hindsight and ignoring restrictions from the Manual on total flow at Moggill, to reduce flood impacts downstream, as outlined by one of the Seqwater Flood Engineers in their statement to the Commission (Reference 3).

67 Peak flood levels at key locations from the alternative scenario modelling are presented in Table 5 below. A negative value of "Peak Flood Level Difference" for a given scenario indicates a benefit (i.e. a reduction in flood levels compared to what actually occurred). Discussion of the results for each scenario are provided in the following sections.

Table 5: Alternative Dam Operation Results

Location	Case 1	Option A	Option B	Option C	Option D
	Peak Flood Level (mAHD)	Peak Flood Level difference relative to Case 1 (m)			
Moggill	17.6	-0.3 to 0.4	-0.7	-0.7	-0.9
Jindalee	13.1	-0.3 to 0.4	-0.6	-0.6	-0.8
Oxley	8.3	-0.2 to 0.3	-0.5	-0.5	-0.6
Brisbane	4.6	-0.1 to 0.3	-0.3	-0.3	-0.4

Discussion – Early Transition to Strategy W4 (Option A)

68 The primary goal in Strategy W4 is to maintain the safety of the dam, and the Manual states that Wivenhoe Dam gates should be opened until the dam level begins to fall. In order for the dam levels to fall, the outflow from the dam at a given time must exceed inflow.

69 There is some ambiguity in the Manual as to the rate at which gates should be opened once Strategy W4 is triggered. On one hand the Manual states under Strategy W4A that gate openings are occur at the intervals of 0.5 m every 10 minutes. On the other hand there is a requirement to consider the “impact if rapidly escalating discharge…on downstream reaches.” In practice during the January 2011 event, the Flood Engineers opened the gates at a rate of about 1.0 m per hour under Strategy W4, which produced an increase in outflow rate that mimicked the rate of increase of dam inflow. This appears to be a reasonable rate of opening to balance the requirements under Strategy W4.

70 However this flexibility of gate opening rates means that if Strategy W4 had been engaged earlier, two different courses of action would have been open to the Flood Engineers, either:

a. To quickly escalate outflows to match inflows and stabilise the level in the dam, resulting in a lower eventual peak lake level but a higher peak discharge than what actually occurred; or

b. To increase outflows at a slower but steady rate, to make more use of the remaining mitigation storage in the dam, resulting in a similar peak lake level as what occurred.

WMAwater investigated several alternative scenarios involving an early transition to Strategy W4. Of these scenarios, Options A4 (Figure 20) and Option A5 (Figure 21) respectively illustrate the two courses of action discussed above.

Figure 20: Option A4 Wivenhoe Dam Releases and Water Levels

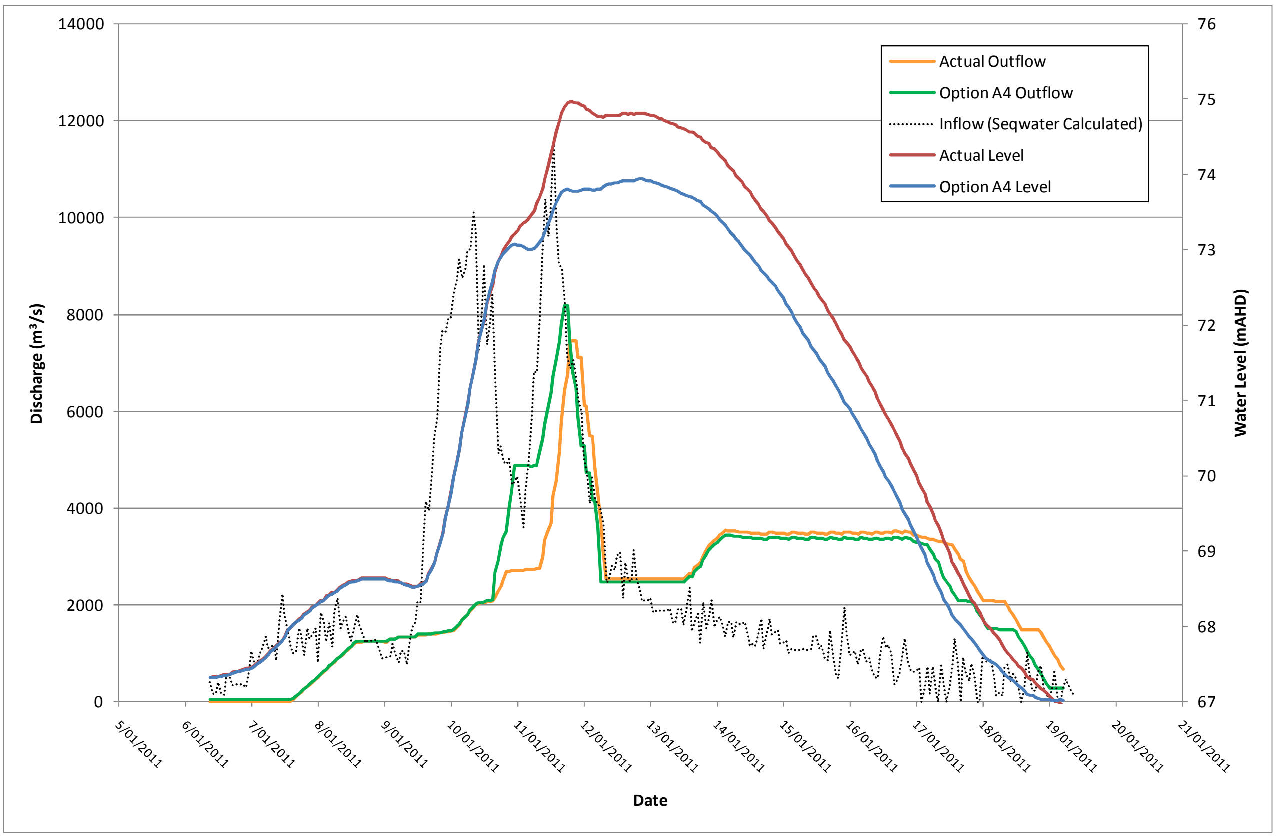
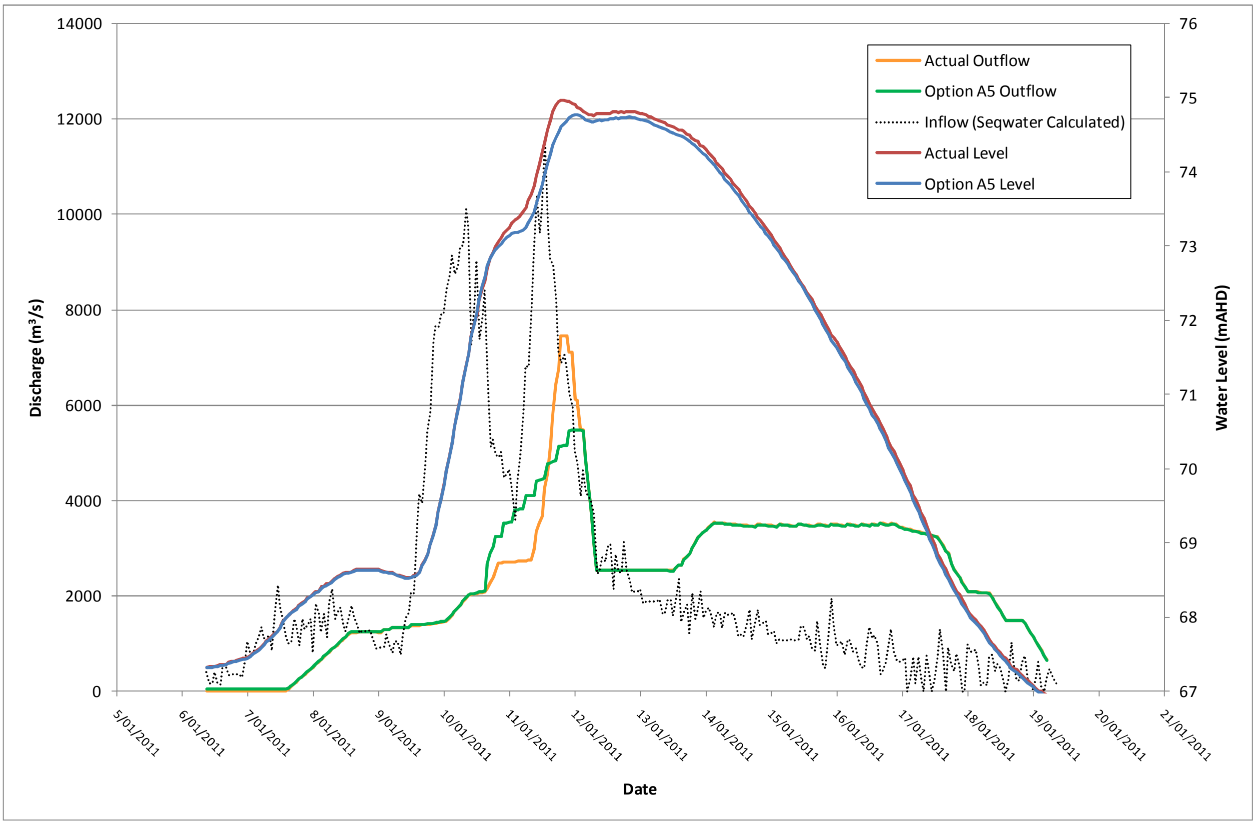


Figure 21: Option A5 Wivenhoe Dam Releases and Water Levels



71 Option A4 uses a rate of gate openings of between 0.5 m to 1.0 m per hour, similar to what was adopted for Strategy W4 during the January 2011 flood event. Option A5 uses a slower rate of 0.5 m every 4 hours to take advantage of the additional storage space due to the early transition.Modelling indicates that an early transition into Strategy W4 would have had mixed results, depending on the rate of gate openings then adopted by the Flood Engineers while under Strategy W4.

72 Option A4, where the gates are opened reasonably fast to stabilise dam levels, would have resulted in marginally worse flooding downstream of Wivenhoe Dam, with an increase of around 0.3 m to 0.4 m in peak flood levels at most locations on the Brisbane River. It is noted that under such a scenario, the peak lake level in Wivenhoe Dam would not have reached the 74.0 mAHD trigger level for Strategy W4, leaving a substantial amount of flood mitigation storage unused. The flood volume released from Wivenhoe Dam during the peak outflow period would therefore have been higher under this scenario.

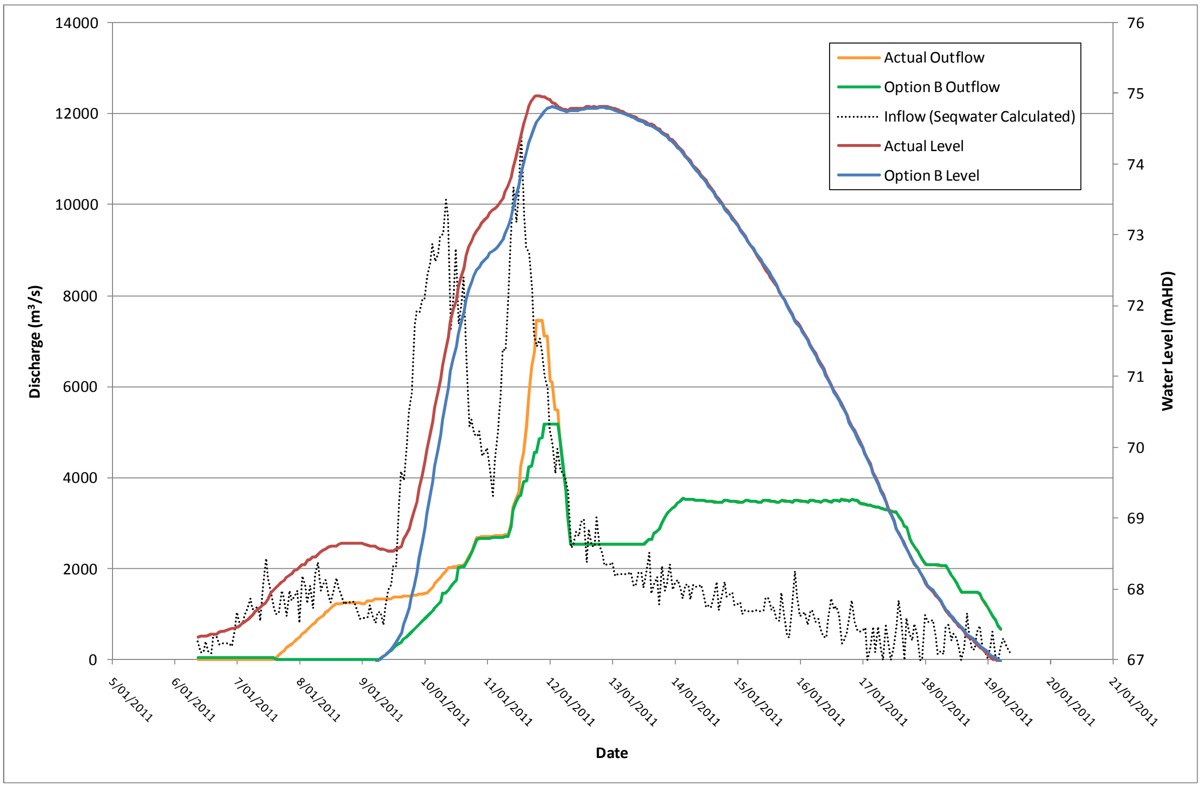
73 Option A5, where the gates are opened at a slower rate, resulting in a similar peak lake level but a lower eventual peak discharge, would have resulted in a relative benefit to flood levels with a reduction of between 0.1 m to 0.3 m at most locations. Further discussion of these outcomes is provided below. Implementation of the relatively slow gate openings in this scenario would have required some knowledge of the size of the second inflow peak to the dam. Given that additional rain of was forecast during the second peak (which did not eventuate), such a strategy probably would not have been justified.

74 It is likely that had Strategy W4 been implemented earlier, the rate of gate openings would have been somewhere between the Option A4 and Option A5 scenarios, and the resulting impact on flood levels would have been similar to what actually eventuated.

75 This analysis indicates that from around 10pm on 10 January 2011 onwards, when inflows to Wivenhoe Dam began to increase towards the second peak, the gate operations strategy adopted did not have a significant influence on flood severity downstream, and the strategy adopted by the Flood Engineers was towards the more effective end of the range of plausible scenarios.

6.1. Discussion – Prior Dam level at 75% FSL (Option B)

Figure 22: Option B Wivenhoe Dam Releases and Water Levels



76 The modelling indicates that this scenario would have reduced peak flood levels and extents along the lower Brisbane River, which a reduction of around 0.7 m at Moggill, tapering to a reduction of around 0.3 m at Brisbane Port Office.

77 If Wivenhoe Dam had been at 75% FSL prior to the commencement of the flood, it would not have reached the gate operation trigger level of 67.25 mAHD until around midday on 9 January, at around the same time as inflows to the dam began to increase substantially towards the first inflow peak (at 8am on 10 January). Under these conditions, according to the strategy flow chart on Page 23 of the Manual, Strategy W2 would have been engaged almost immediately, with Strategy W3 being triggered within a reasonably short time frame.

78 By 2pm on 10 January, operating under Strategy W3, it is reasonable to assume releases would have been similar to what actually occurred, although the dam level would have been approximately 0.7 m lower. This extra storage space would have resulted in Strategy W4 being triggered at a slightly later stage, and allowed for a lower peak release of around 5,200 m^3/s from Wivenhoe Dam, if the same peak eventual level in the dam was allowed to be reached.

79 This scenario would therefore result in a reduction in total flood volume released from Wivenhoe Dam (about 11% lower), and a reduction in the peak discharge from the dam from 7,500 m^3/s to 5,200 m^3/s (about 30%). This reduction in both total flood volume and peak discharge would have resulted in lower peak flood levels in the lower Brisbane River as per Table 5.

80 This scenario did not include the effect of reducing Somerset Dam to 75% of FSL. In the limited timeframe available for this work, the additional complexity of resulting interactions between the two dams prevented assessment of such a scenario. It is expected that such conditions would have resulted in additional reduction in flood impacts downstream of Wivenhoe Dam. However the incremental benefit would be lessened as the storage capacity of Somerset Dam at FSL is less than 33% of the Wivenhoe storage capacity at FSL.

81 This scenario did not include the effect of altering the trigger levels for dam release strategies stipulated in the Manual. There are several ways such changes could be made to re-allocate the additional storage available for flood mitigation that would come from lowering the lake level below the FSL, and time constraints prevented these changes from being assessed. The most likely would be to reduce the trigger levels in Strategy W1 and W2 by a similar amount as the lake level reduction, and leave the trigger for Strategy W4 the same, so that the additional capacity was available for use under Strategy W3.

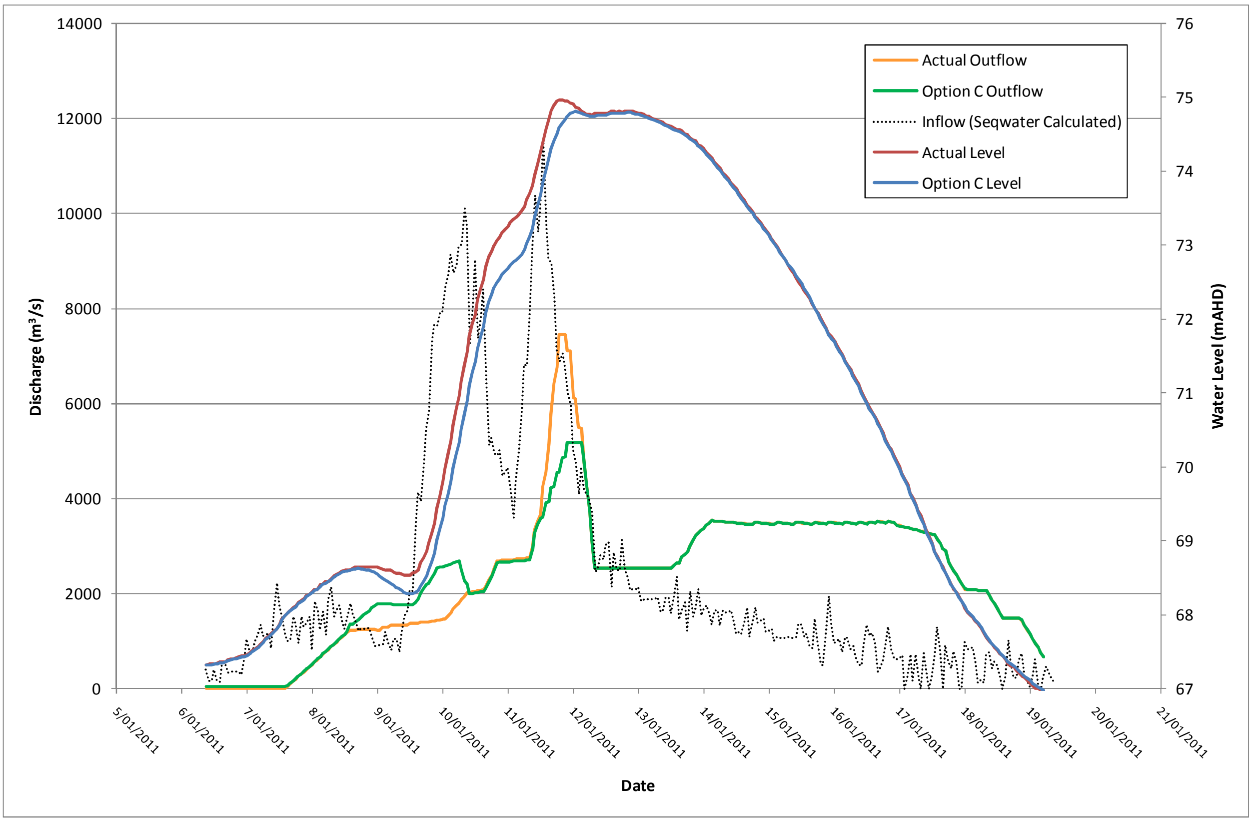
82 For the Option B scenario the effect of keeping the same gate operation strategies rather than changing them to re-allocate the additional storage for flood mitigation is as follows. Roughly 70% of the additional storage space available for flood mitigation (equivalent to 18% of FSL) would have been used up in the early part of the flood (by around 1pm on 10 January), as indicated by the first period of divergence between the green and orange lines on Figure 22. Only 30% of the additional storage (or 7% of FSL) would have been used up during the period of peak dam outflow, allowing the peak discharge to be lower (the second period of divergence between the green and orange lines on Figure 22).

83 This indicates that if the trigger levels for Strategies W1 and W2 were reduced corresponding to the reduction of lake level to 75% of FSL, such that the dam release in the early part of the flood had been similar to what occurred, the full additional 25% of FSL storage could have been used to reduce the dam outflow peak even further, and resulting in improved impacts downstream.

84 These results are for the January 2011 flood event, and the outcomes may not be the same for other floods. Any consideration of reducing water storage in the dam to improve flood mitigation should take into account the trade-off risks to water supply security.

6.2. Discussion – Releases at Upper Limit During Strategy W3

Figure 23: Option C Wivenhoe Dam Releases and Water Levels



85 Option C, whereby under Strategy W3 the Wivenhoe Dam releases would be increased to the upper allowable limit as soon as possible, would result in a similar reduction of peak flood levels and inundation extents as Option B (75% of FSL prior to flood).

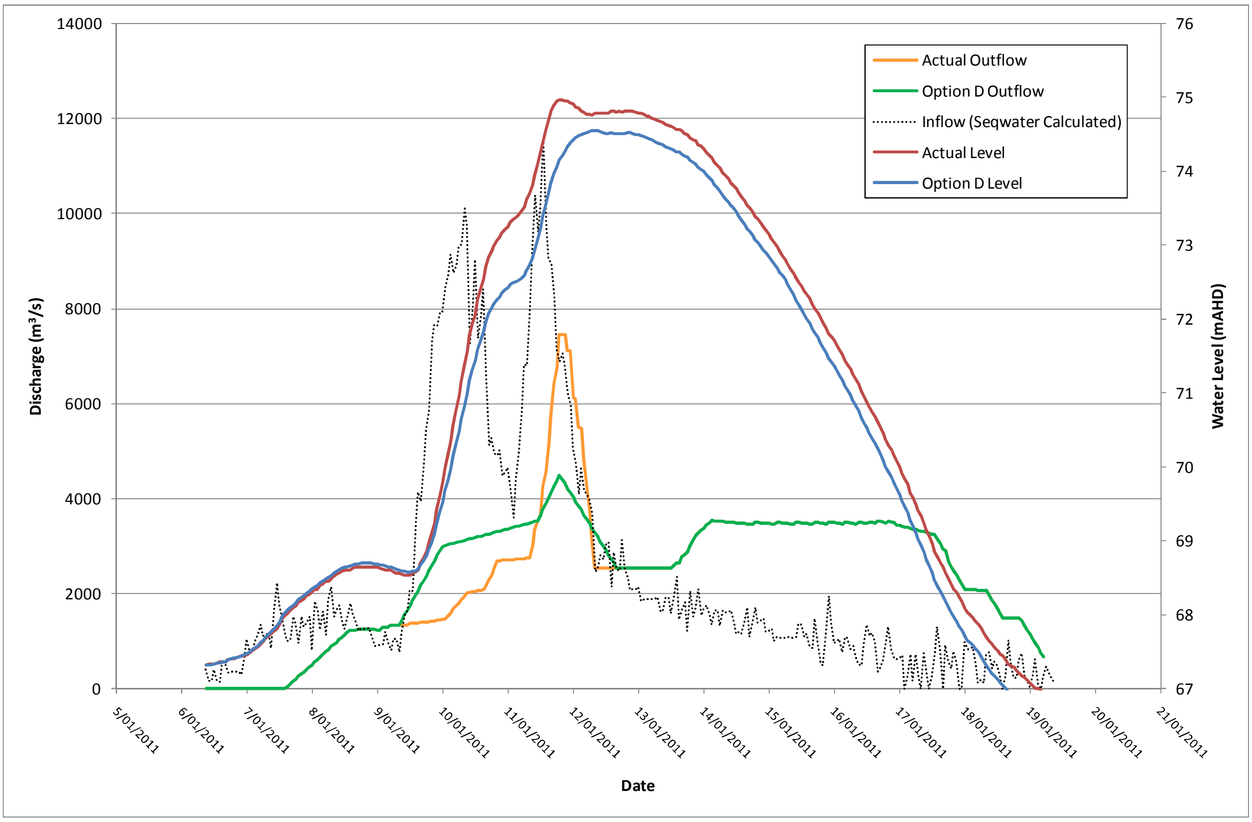
86 The reason for this similarity can be observed by comparing Figure 22 with Figure 23, and noting that from 2pm on 11 January the dam outflows and lake levels would have been very similar under of the two scenarios. This is because the additional flow (compared to what actually occurred) potentially released under this scenario between 12pm on 9 January and 2pm on 11 January, as shown by the divergence of the green line above the purple line on Figure 23 during this period, would have brought the total water stored in the dam back into line with the 75% FSL scenario.

87 It is important to note that enacting this scenario would have required the dam operators to increase Wivenhoe Dam outflow to around 1,800 m^3/s by 12am on 9 January, which is similar to the peak inflow that had been received into the dam until that time, and as such

the only real mitigation provided by the dam up until that point would have been to delay the flood peak rather than reducing it. The operators therefore would have required a high level of confidence that the peak dam inflows were going to increase dramatically, as they happened to do for the actual flood event, but were not expected to do based on information available at the time. Seqwater modelling at that time (Run 12, Appendix A, Reference 7) indicated that with or without forecast rain, the peak Wivenhoe Dam inflow had already occurred at 12pm on 7 January, at 1,890 m^3/s.

6.3. Discussion – Optimised Strategy

Figure 24: Option D Wivenhoe Dam Releases and Water Levels



88 Of the alternative scenarios assessed, Option D produces the largest reduction in peak flood impacts in the lower Brisbane River, with a reduction of 0.9 m at Moggill and 0.4 m at Brisbane Port Office.

89 If full foreknowledge of the dam inflows is available, the dam releases can be optimised to reduce peak discharge from the dam. Under this scenario, the peak outflow of Wivenhoe Dam is reduced from 7,500 m^3/s to 4,500 m^3/s (40% reduction). This significant reduction in peak discharge accounts for the majority of the beneficial effect on peak flood levels estimated in Table 5.

90 The implementation of Option D in reality would have been implausible, as it relies on using discretion to increase discharge from Wivenhoe Dam above allowable thresholds

under Strategy W3 during 9 and 10 January. It would have relied on foreknowledge of the large second inflow peak into Wivenhoe Dam, which modelling did not indicate was likely until early on 11 January (Run 35, Reference 7). As indicated above, by this point there were few if any reasonable options available to the Flood Engineers which could have significantly improved flood impacts compared to what eventuated.

7. RESPONSES TO QUESTIONS FROM THE COMMISSION

91 WMAwater undertook a review of the original model provided by SKM (Version 1), and identified issues that rendered the model unsuitable for use to answer the Commission's questions. SKM then provided WMAwater with a revised model (Version 2), which WMAwater consider fit for purpose for addressing most aspects of the Commission's questions. Details of the review are provided in Section 4.

92 Brief answers to the specific questions asked by The Commission are provided below. These answers rely on the information presented in this report for context.

To what extent was flooding (other than flash flooding) in the mid-Brisbane River, the Lockyer Valley, Ipswich and Brisbane during January 2011 caused by releases from the Somerset and Wivenhoe Dams?

93 Flooding occurred due to runoff from each of the Brisbane, Bremer and Lockyer Valley catchments. Looking at the total volume of the flood event between the dates 9-16th January 2011, Wivenhoe Dam releases accounted for 59%, Lockyer Creek and other tributaries upstream of Mt Crosby accounted for 27% and the Bremer River accounted for approximately 10% during this period. However the bulk of this flood volume was released after the flood peak, thereby providing flood mitigation benefits. With regards to contribution to the flood peak, from Moggil to the Port Office the proportion of peak flow contributed by Wivenhoe Dam and non-Wivenhoe Dam sources was roughly equivalent.

To what extent did the manner in which flood waters were released from the Somerset and Wivenhoe Dams avoid or coincide with peak flows from the Bremer River and Lockyer Creek?

94 Based on analysis of model runs it appears that at Mt Crosby, peak Wivenhoe Dam flow preceded the peak of other upper tributary flow inputs, including Lockyer Creek flows, by approximately 9 hours. Further downstream it seems likely that peak flows from the Bremer River and Wivenhoe Dam releases at Ipswich occurred almost simultaneously.

Had the levels in Somerset and Wivenhoe Dams been reduced to 75 per cent of full supply level by the end of November 2010 (both with and without amendments to the trigger levels for strategy changes in the Wivenhoe Manual) what impact would this have had on flooding?

95 For a reduction to 75% of FSL in Wivenhoe Dam prior to the start of the flood, and without amendment to trigger levels for strategy changes in the Wivenhoe Dam Manual, downstream flood levels are reduced by up to 0.7 m (at Moggil) and by 0.5 m and 0.3 m at Oxley and Port Office (Brisbane) respectively. If gate operations were revised to take advantage of the additional storage available under such a scenario, it is expected that the

benefits on flood levels would improve further, although such scenarios have not been investigated here due to time constraints.

What effect would the implementation of different release strategies (to be identified by WMAwater) have had on flooding?

96 Various options were run as follows:

a. Case 1 – The calibrated January 2011 model results supplied by SKM;
b. Option A – Earlier transition to Strategy W4;
c. Option B –Wivenhoe Dam at 75% of Full Storage Level (FSL) prior to the flood;
d. Option C – Discharge at upper limit during Strategy W3;
e. Option D – An optimised release strategy, as outlined by one of the Seqwater Flood Engineers in their statement to the Commission (Reference 3).

Table 6: Alternative Dam Operation Results (Table 5 reprinted here for convenience)

Location	**Case 1**	**Option A**	**Option B**	**Option C**	**Option D**
	Peak Flood Level (mAHD)	Peak Flood Level difference relative to Case 1 (m)			
Moggill	17.6	-0.3 to 0.4	-0.7	-0.7	-0.9
Jindalee	13.1	-0.3 to 0.4	-0.6	-0.6	-0.8
Oxley	8.3	-0.2 to 0.3	-0.5	-0.5	-0.6
Brisbane	4.6	-0.1 to 0.3	-0.3	-0.3	-0.4

97 Of these scenarios, Option D would have had the greatest impact with a reduction in peak flood level at Port Office of 0.4 m and a reduction at Moggil of 0.9 m. However of the scenarios investigated, Option D is also the least likely to be achieved in practice, as it would have relied on foreknowledge of the flood far superior to that available to the Flood Engineers, even taking forecast rain into account.

98 Option C is a more plausible alternative scenario, although it too would have required a level of foreknowledge of the flood event at key decision points that was not available at the time. While modelling indicates this approach would have produced a benefit during the January 2011 flood, no operational procedure can produce the optimal outcome for all floods. The option C approach would generally produce beneficial outcomes in floods that are large enough to eventually trigger Strategy W4, but would probably be detrimental in moderate-sized floods that remain in Strategy W3.

99 Option B, resulting from Wivenhoe Dam being at 75% FSL prior to the flood (either through policy or antecedent rainfall conditions), and using existing gate operations strategies from the Manual, would have resulted in a similar benefit to flood levels as Option C. If gate operations were revised to take advantage of the additional storage available under such a scenario, it is expected that the benefits on flood levels would improve further, although such scenarios have not been investigated here due to time constraints.

100 Various scenarios resulting from triggering Strategy W4 16 hours earlier were investigated as part of Option A. There is some flexibility under Strategy W4 as to the rate at which gate openings are undertaken to stabilise the dam level. An early transition to Strategy W4 may have either worsened or improved the severity of flooding downstream of Wivenhoe Dam, depending on the rate of gate opening adopted. Slower gate openings under an early Strategy W4 scenario would have improved flood impacts, but would also have required information about the timing and magnitude of the flood peak that was unavailable at the time.

101 There are a number of plausible alternative scenarios that could have been undertaken under Strategy W4 that would have resulted in worse (higher) flood levels downstream of Wivenhoe Dam.

102 Whilst the flood level reductions indicated in Table 6 would have been a benefit and reduced flood damages if they had been achieved, generally such scenarios could not have been reasonably achieved with the information available at the time and under the current operating strategies stipulated by the Manual. Nonetheless, these scenarios highlight that for this event, earlier increases in releases from Wivenhoe Dam during 9 and 10 January could have reduced the eventual peak outflow and the resulting severity of flooding experienced downstream.

7.1. Additional Comments

103 With the information available during their operations, and using the strategies defined by The Manual, WMAwater believe the Flood Engineers achieved close to the best possible mitigation result for the January 2011 flood event.

104 Care must be taken with interpreting these findings, which are based on a single large flood event, in relation to the effectiveness of the strategies in The Manual for dealing with future events, some of which will be larger. WMAwater consider that the recommendations relating to gate operation strategies in the Report to the Queensland Flood Commission of Inquiry in May 2011 (Section 9.2, Reference 4) are further supported by the above analysis, namely that:

a. *“Alternative gate operation strategies for flood mitigation should be reviewed … for a full range of flood events, with consideration of average annual flood damages resulting from each strategy.”*
b. *“The review of gate operations should place particular emphasis on the hard transition between the W3 and W4 strategies. Modifications that specify an increasing target discharge at Moggill once key criteria are either reached or predicted to be reached should to be investigated.”*

8. REFERENCES

1. Pilgrim, D. H. (Editor in Chief)
   Australian Rainfall and Runoff – A Guide to Flood Estimation
   Institution of Engineers, Australia, 1987.

2. SKM
   Joint Calibration of a Hydrologic & Hydrodynamic Model of the Lower Brisbane River
   Seqwater, 24 June 2011.

3. Malone, T. A.
   Second Statement to the Queensland Floods Commission of Inquiry
   11 April 2011.

4. WMAwater
   Report to the Queensland Floods Commission of Inquiry
   May 2011.

5. DHI
   Mike11 – A modelling System for Rivers and Channels
   Reference Manual
   DHI Software 2009.

6. Chow, V. T.
   Open Channel Hydraulics
   McGraw Hill Book Company Inc., 1959.

7. Seqwater
   January 2011 Flood Event
   Report on the operation of Somerset Dam and Wivenhoe Dam
   2 March 2011.

Appendix A

APPENDIX A: GLOSSARY

Taken from the NSW Floodplain Development Manual (April 2005 edition)

Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size 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 (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
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 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.

flood education
Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.

flood liable land
Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).

flood mitigation standard
The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.

floodplain
Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.

Flood Planning Levels (FPLs)
FPL's are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the "standard flood event" in the 1986 manual.

flood proofing
A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.

flood prone land
Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.

flood readiness
Flood readiness is an ability to react within the effective warning time.

flood risk
Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below.

existing flood risk: the risk a community is exposed to as a result of its location on the floodplain.

future flood risk: the risk a community may be exposed to as a result of new development on the floodplain.

continuing flood risk: the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.

flood storage areas
Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.

floodway areas Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.

freeboard Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.

habitable room **in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom.

in an industrial or commercial situation: an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.

hazard A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.

hydraulics Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.

hydrograph A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.

hydrology Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.

local overland flooding Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.

local drainage Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.

mainstream flooding Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.

major drainage Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves:

- the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or

- water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or

- major overland flow paths through developed areas outside of defined drainage reserves; and/or

- the potential to affect a number of buildings along the major flow path.

•

mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to “water level”. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.

Appendix B

APPENDIX B: MARK BABISTER CURRICULUM VITAE

Mark Kenneth BABISTER

POSITION: Managing Director

DATE OF BIRTH: [redacted]

NATIONALITY: Australian

PROFESSION: Civil Engineer

QUALIFICATIONS:

- Bachelor of Engineering (Civil) Honours
  University of NSW, 1988
- Master of Engineering Science
  University of NSW, 1993
- Graduate Diploma in Management
  Deakin University, 1997

MEMBERSHIP & COMMITTEES:

- Engineers Australia (CPEng, NPER)
- Registered Professional Engineer of Queensland (RPEQ)
- Chair of Engineers Australia, National Committee on Water Engineering
- AR&R Revision Steering and Technical Committees
- Former Chair of Sydney Division Water Engineering Panel, Engineers Australia
- Chair of Organising Committee for 2003 International Hydrology & Water Resources Symposium

SPECIAL FIELDS OF COMPENTENCE

- ***Community Engagement on Major Water Resource Projects***
- ***Hydrologic Modelling***
- ***Hydraulic Modelling***
- ***Floodplain Management***
- ***Flood Frequency, Joint Probability Analysis and Risk Assessments***
- ***Computer Programming***
- ***Data Collection, Analysis and Presentation***

PROFESSIONAL EXPERIENCE

WMAwater (formerly Webb, McKeown & Associates Pty Ltd)
September 1988 - to Date

Hydrological Studies
- Project Director - State of the Darling Basin Report for MDBC
- Project Director - Coxs River IQQM Review for Delta Electricity
- Project Director - Coxs River Mass Balance Review for DIPNR
- Project Manager - Hawkesbury-Nepean Water Use Study for DLWC
- Project Manager - Impact of Farm Dams on Streamflow in Hawkesbury-Nepean Catchment for DLWC
- Project Manager - Assessment of the Homogeneity of Streamflow on Hawkesbury-Nepean Catchment for DLWC
- Project Manager - Macquarie Marshes RUBICON programming for DLWC
- Project Engineer - HMAS Kuttabul
- Project Engineer - Buttonderry Landfill for Wyong City Council
- Project Manager - Review of the Bellinger, Kalang and Nambucca River Catchments Hydrology

Floodplain Management
- Project Manager - Riverstone Bypass Flood Study for RTA
- Project Manager - Penrith Lakes Development Flood Management Options for Bowdens
- Project Manager - Lord Howe Island for Lord Howe Island Board
- Project Manager - Investigation of Hawkesbury/Nepean Floodplain for Sydney Water Board
- Project Manager - Lochinvar for Maitland City Council
- Project Manager - Investigation of Floodplain Management Measures in Hawkesbury River for Hawkesbury-Nepean Flood Management Advisory Committee
- Project Manager - Wolli Creek Station Flood Study for NSR Transfield/Bouygues
- Project Engineer - Hunter River for Maitland Council
- Project Manager - Parramatta Rail Link for Maunsell McIntyre
- Project Manager - Cooks Cove for Maunsell McIntyre
- Project Manager - Upper Yarraman Creek FPM Plan for DLWC
- Project Manager - Wagga FPM Study for Wagga City Council

- Project Manager - Carroll Boggabri FPM Plan for DIPNR
- Project Manager - Kempsey Flood Study
- Project Manager - Newry Island Flood Study
- Project Manager - Deep Creek Flood Study
- Project Manager - Kurnell Flood Study, Floodplain Risk Management Study and Plan

Hydraulic Modelling

- Project Manager - M5 Motorway Cooks River Crossing Flood Study for Hyder Consulting
- Project Manager - Warragamba Dam Inter Departmental Committee Study for NSW Government
- Project Manager - Wooyung/Mooball Flood Investigation for Tweed and Byron Councils
- Project Manager - Warriewood Wetlands - Henroth Pty Ltd
- Project Engineer - Wombarra Hydraulic Study for State Rail Authority
- Project Engineer - Illawarra Railway Culvert Upgrading for State Rail Authority
- Project Engineer - Macleay River Flood Gate Operation for Kempsey Shire Council
- Project Manager - Emu Plains Local Hydraulics for DLWC
- Project Manager - Kempsey to Eungai Pacific Highway Upgrade for PPK Environment & Infrastructure
- Project Manager - Lane Cove River Crossing for Parramatta Rail Link Company
- Project Manager - Riverview Road Levee Gradient for DLWC
- Project Manager - New Southern Railway Cooks River Crossing for Transfield Bouygues Joint Venture
- Project Manager - Warragamba Dam Side Spillway for AWT
- Project Manager - Bethungra Dam PMF and Dambreak for DLWC
- Project Manager - South Creek High Level Crossing for DLWC
- Project Manager for various studies in Hawkesbury - Nepean catchment for DLWC
- Project Manager - Kempsey to Frederickton Pacific Highway Upgrade - Project Implementation
- Project Manager - Australian Rainfall and Runoff Research Project 15 - Two Dimensional Simulation

Design Flood Estimation

- Project Manager - Bethungra Dam PMF and Dambreak Assessment for DLWC
- Project Engineer - Review of Lower Hastings Design Flood Levels for Hastings Shire Council
- Project Manager - Lord Howe Island Design Rainfall Assessment for DLWC
- Project Manager - NSW - FORGE Project Data Compilation for DLWC
- Project Manager - Warragamba Mitigation Dam for Sydney Water Board
- Project Manager - Warragamba Dam Side Spillway, Freeboard, Dambreak and Sunny Day Failure Studies
- Project Engineer - Moruya River Flood Study for Eurobodalla Shire Council
- Project Engineer - Lord Howe Island Flood Study for Lord Howe Island Board
- Project Manager - Wombarra Drainage for RSA
- Project Manager - Australian Rainfall and Runoff Research Project 3 - Temporal Patterns of rainfall – Testing of an alternative temporal pattern approach

Stormwater Management

- Project Engineer - Sheas Creek for Sydney Water Board

Coastal & Estuarine Studies

- Project Engineer - Batemans Bay Coastal Management Study
- Project Engineer - Lake Cathie/Lake Innes Management Study for Hastings Council and National Parks
- Project Manager - Development of an Eroding Entrance Model for Breakout of Coastal Lagoons

Legal Proceedings

- Court Appointed Expert - Oceanic Developments vs Minister for Planning
- Expert Witness for the following:
    - Primo Estates vs. Wagga City Council
    - Kurnell Lodge
    - McGirr & Xenos - Woodford Street, Longueville
- Project Manager - EPA vs Camilleri's Stockfeeds Pty Ltd for NSW Environment Protection Authority
- Project Manager - EPA vs ADI Murray River for NSW Environment Protection Authority
- Project Manager - Warriewood Valley Pty Ltd vs Pittwater Council
- Project Manager - Davis-Firgrove Estate, Dubbo for North & Badgery
- Project Manager - Bourne ats Kurnell Lodge Pty Ltd

SYDNEY WATER BOARD
Southern Region - Systems Planning Group
July 1983 to August 1988

Involved in various aspects of water supply and sewer investigation. This included performance assessment of sewerage pumping systems and investigation, design and operation of reticulation and trunk watermains, modelling of network performance and water hammer, water supply operation and maintenance, reservoir design and stormwater construction. Construction experience included onsite supervision of stormwater channels at Woolloomooloo and Double Bay.

PUBLICATIONS

1993 RUBICON - An Unsteady Flow Branched Model

1993 Dealing with the Zero Depth Problem within the PIPENET Solution Algorithm

1994 A Review of Numerical Procedures for Routing Unsteady Flows Along a Dry Bed

1998 Batemans Bay Coastal Management: A Sustainable Future

1999 The Influence of the Illawarra Escarpment on Long Duration Design Rainfalls – Implications for Floodplain Management

2003 Editor 28th International Hydrology & Water Resources Symposium Proceedings

2005 Adding Value to Bureau of Meteorology Flood Prediction

2008 31st Hydrology & Water Resources Symposium Proceedings: *Can Fixed Grid 2D Hydraulic Models be used as Hydrologic Models*, joint authors with J. Ball and K. Clark

2008 Comparison of Two-dimensional modelling approaches used in current practice, *9th National Conference on Hydraulics in Water Engineering*, Joint author with M.Retallick

2009 A Hydroinformatic approach to development of design temporal patterns, *Hydroinformatics in Hydrology and water resources (Proc. Of Symposium JS.4 at the Joint IAHS and IAH Convention Hyderabad India,* Joint author with C.Varga and J.Ball

2009 Estimation of design flood flows considering climate change, *IAHR Congress Vancouver,* Joint author with J.Ball and B. Phillips

2009 An alternative approach for developing temporal patterns, *Proceedings of the 32nd Hydrology and Water Resources Symposium Newcastle 2009,* Joint author with C.Varga and J.Ball

2009 Two dimensional simulation in urban areas, *Proceedings of the 32nd Hydrology and Water Resources Symposium Newcastle 2009,* Joint author with M.Retallick and J.Ball

2009 Do filtered temporal patterns resemble real patterns? *Proceedings of the 32nd Hydrology and Water Resources Symposium Newcastle 2009,* Joint author with M.Retallick, C.Varga, J.Ball, and E. Askew

2010 Considering the impacts of Climate Change on flood risk *Practical Responses to Climate Change National Conference*, Joint author with D. McLuckie and R. Dewar

2011 Consideration of Sea Level Rise in Flood and Coastal Risk Assessments, *51st Annual Floodplain Management Authorities Conference* D.McLuckie, P. Watson and M. Babister.

2011 Revisiting the Design Flood Problem *Proceedings of the 34th IAHR World Congress* Joint author with J.Ball, and M. Retallick

2011 The Ineptitude of Traditional Loss Paradigms in a 2D Direct Rainfall Model *Proceedings of the 34th IAHR World Congress* Joint author with F.Taaffe and S.Gray

In Print *Australian Rainfall and Runoff Research Project 15: Two dimensional simulation in urban areas*, Editor