QUEENSLAND FLOODS COMMISSION OF INQUIRY

REVIEW OF HYDRAULIC MODELLING

FINAL REPORT

JULY 2011

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Project Review of Hydraulic Modelling Queensland Floods Commission of Inquiry	**Project Number** 111024
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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)

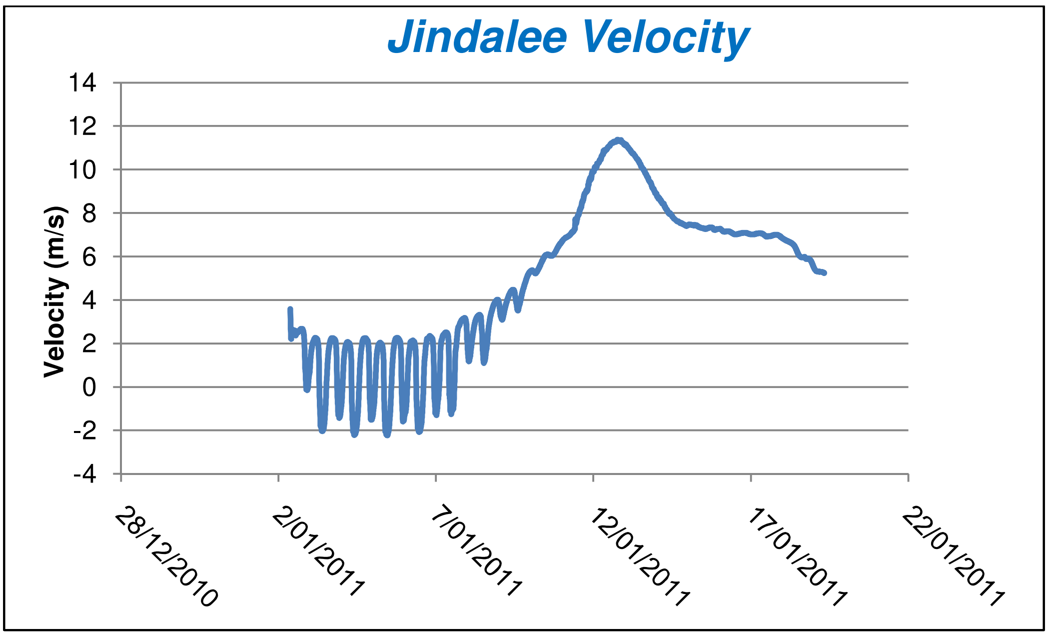


Figure 2: Discharge Time Series (modelled) at Mt Crosby Bridge (Version 1)

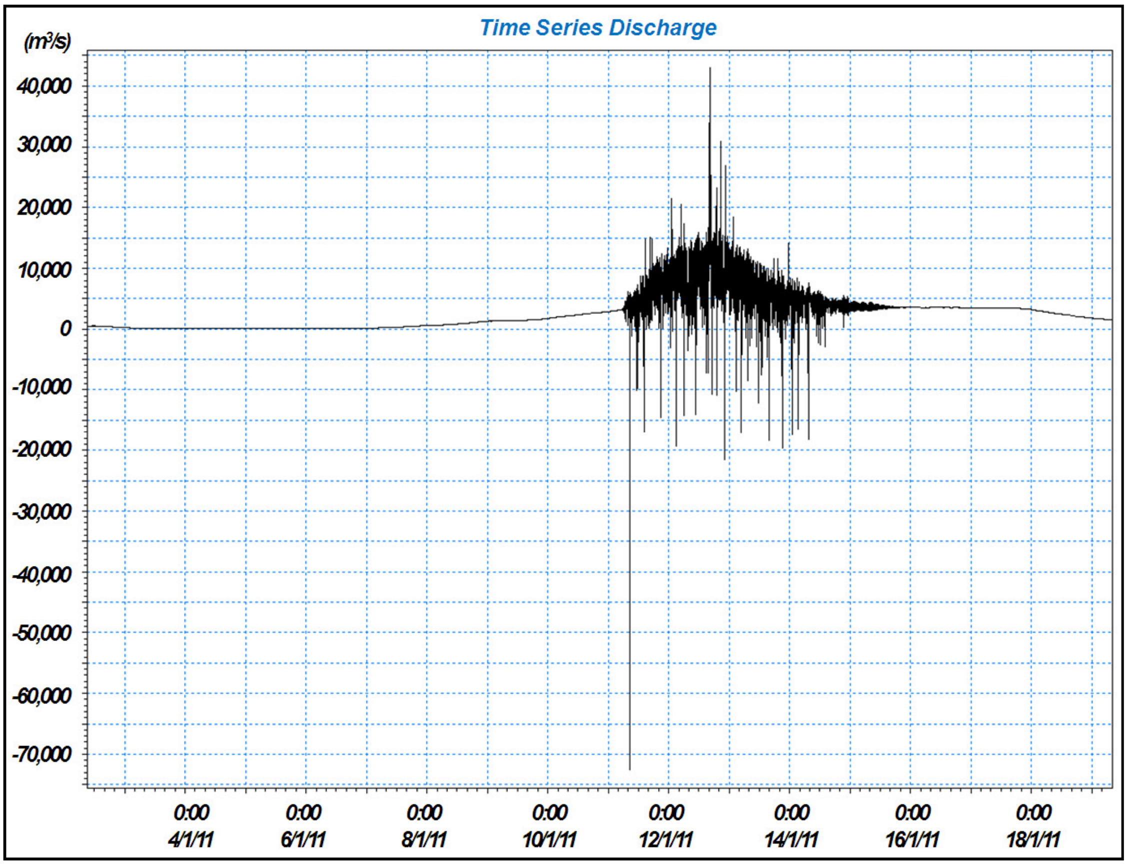


Figure 3: Water Level Time Series (modelled) at Mt Crosby Bridge (Version 1)

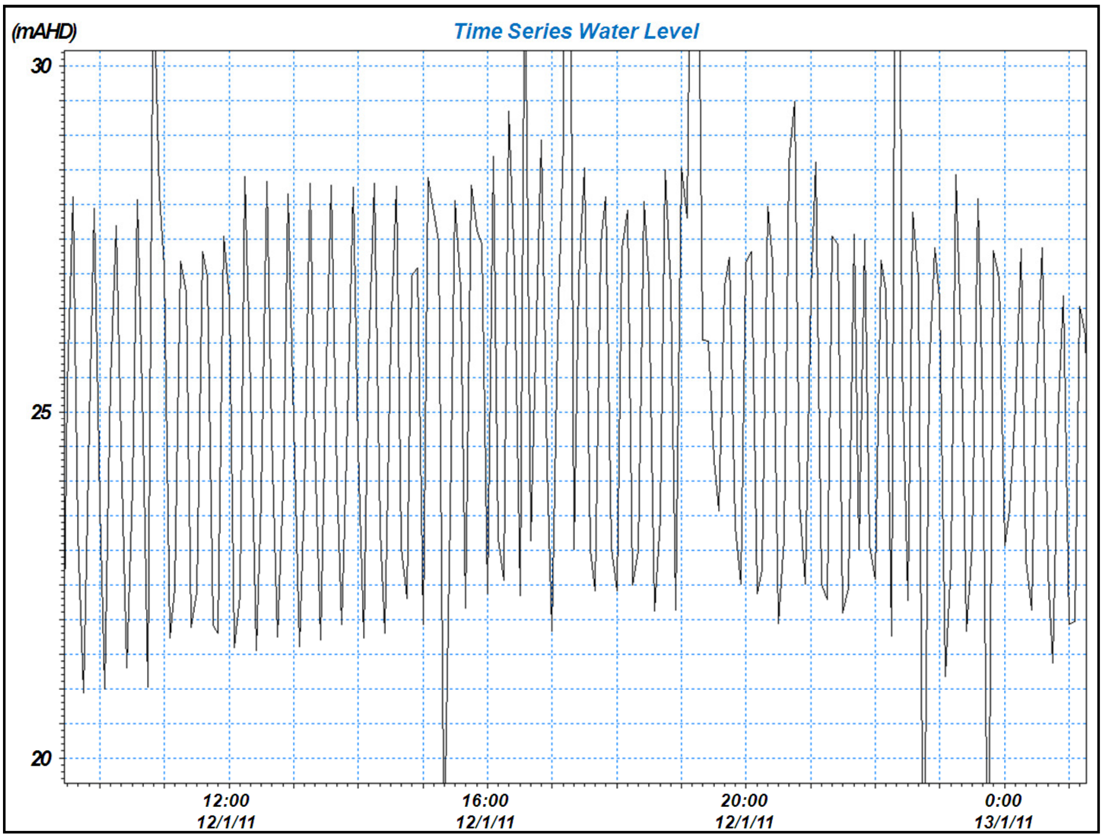
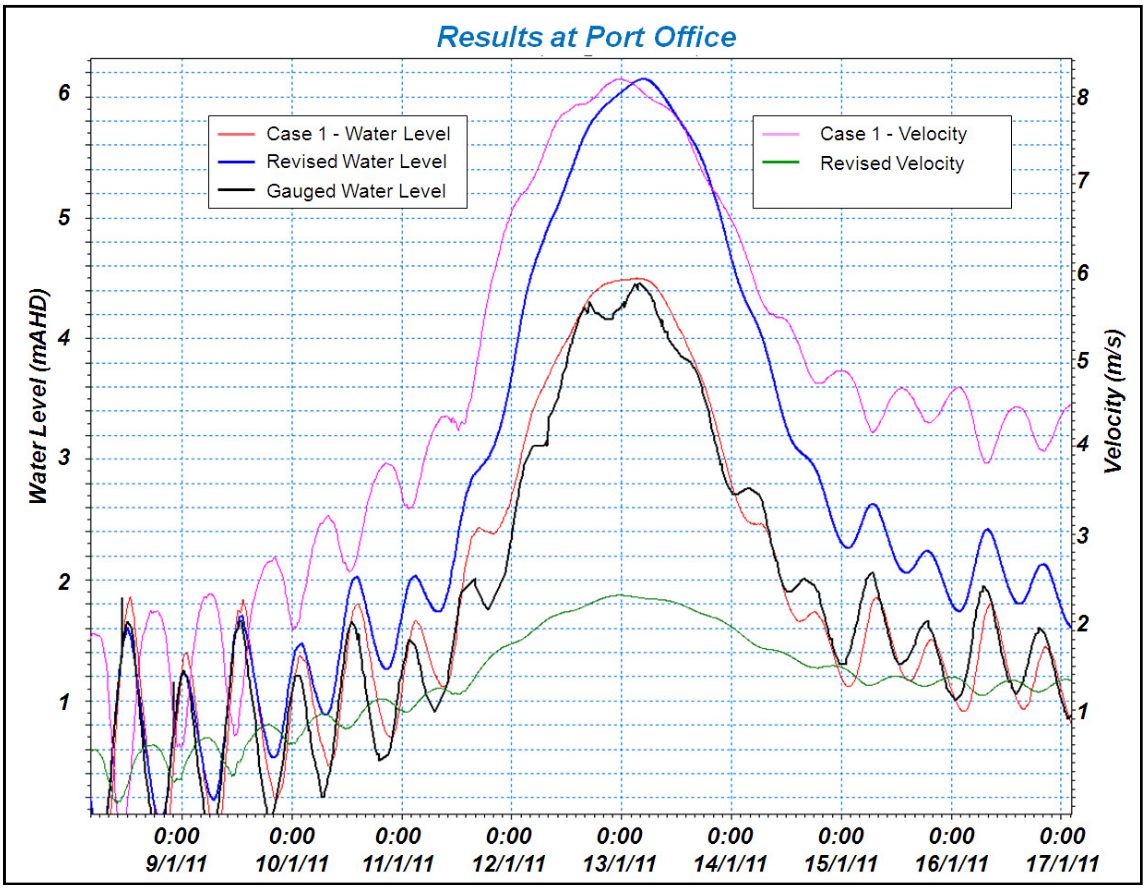


Figure 4: Model Results (Version 1) at Port Office versus "Fixed" model results



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.

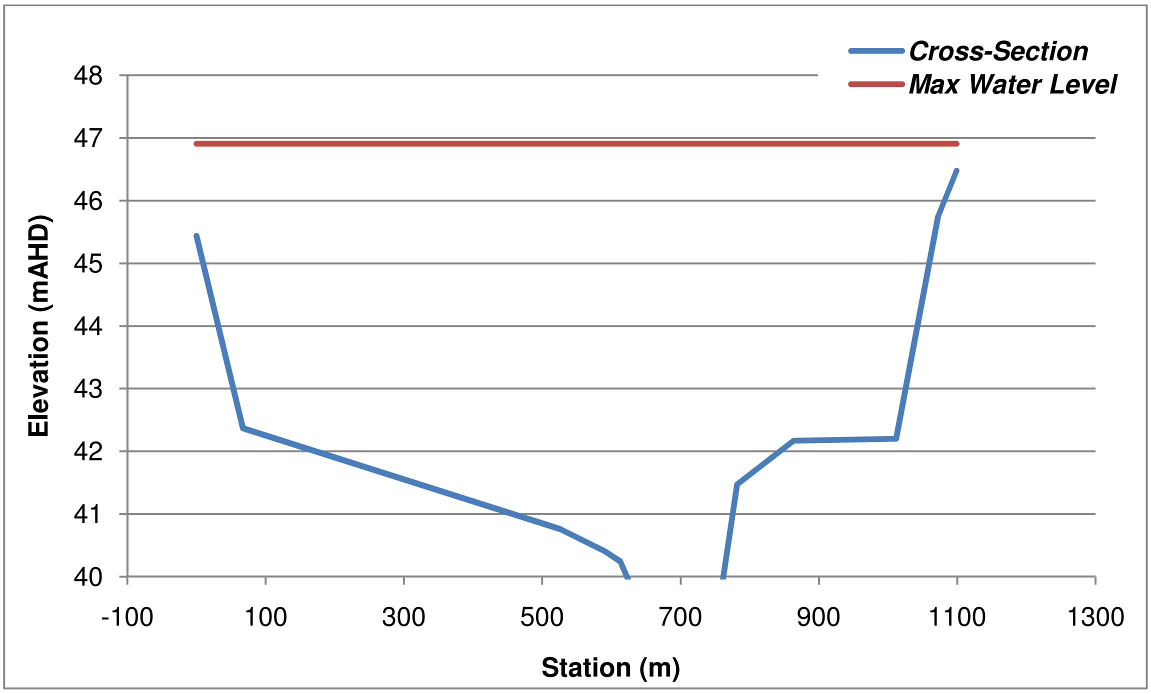
40 Generally the Brisbane River is schematised as one main flow branch with areas of off-branch storage represented in 28 discrete locations, distributed over the river from chainage 948,254 m (in the upstream) to chainage 1,066,425 m (in the downstream). Off-branch storage is represented via linked 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

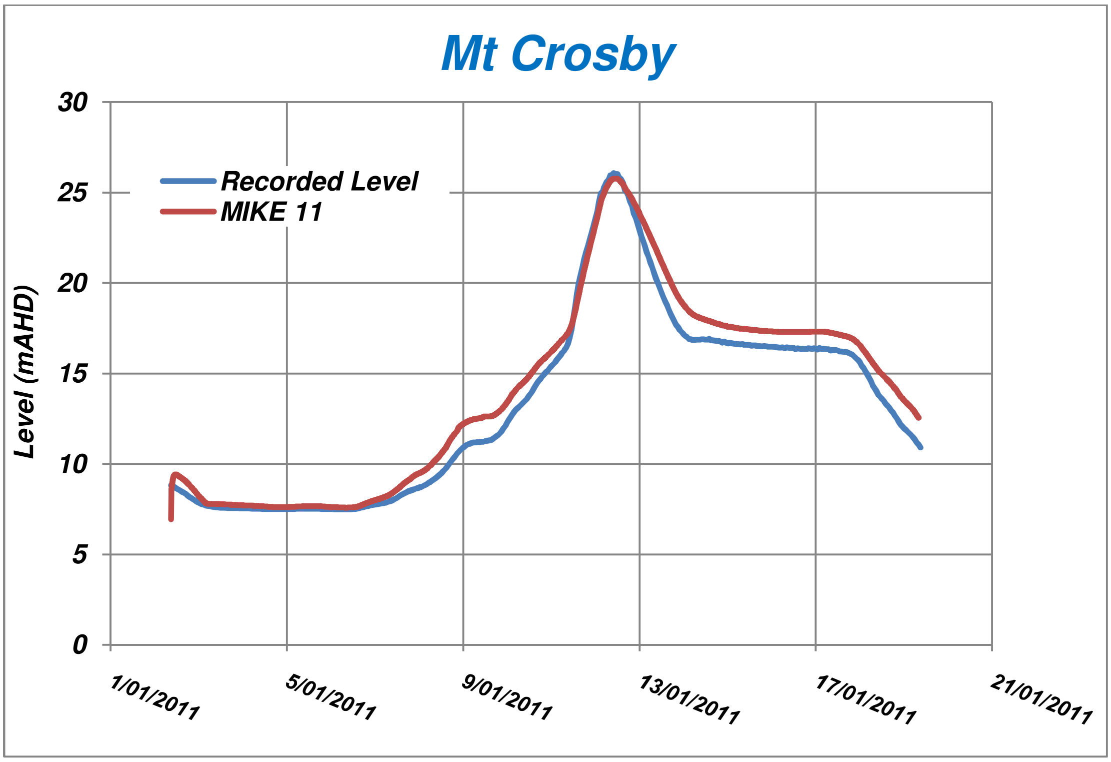


Figure 7: Comparison of gauged and modelled water level – Moggil

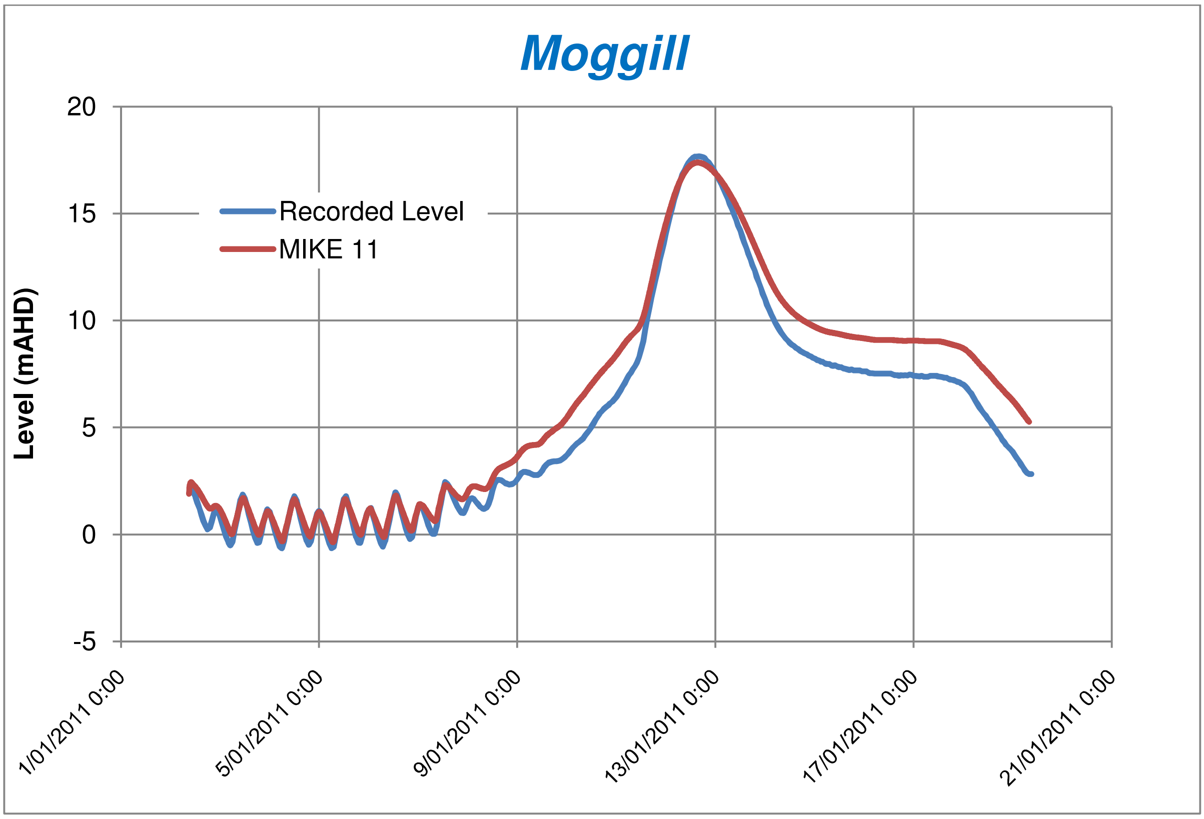


Figure 8: Comparison of gauged and modelled water level – Jindalee

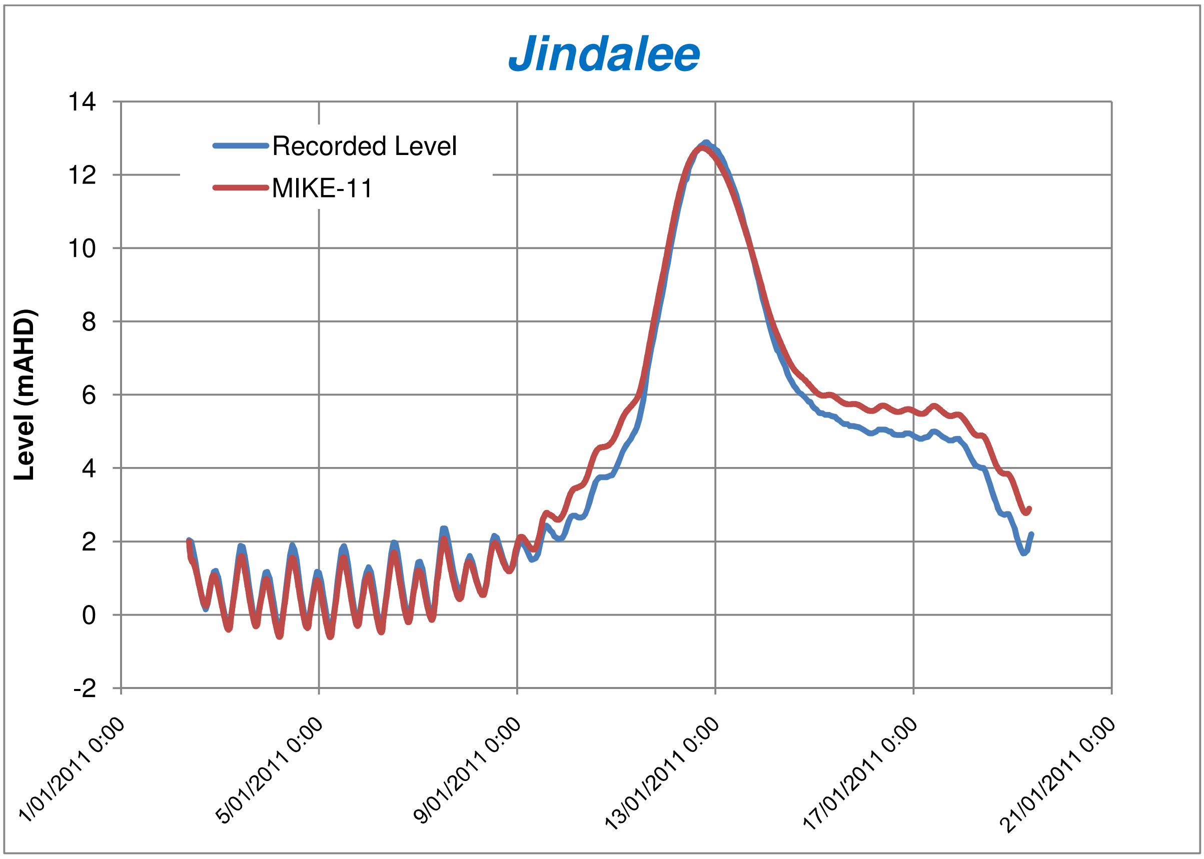


Figure 9: Comparison of gauged and modelled velocities

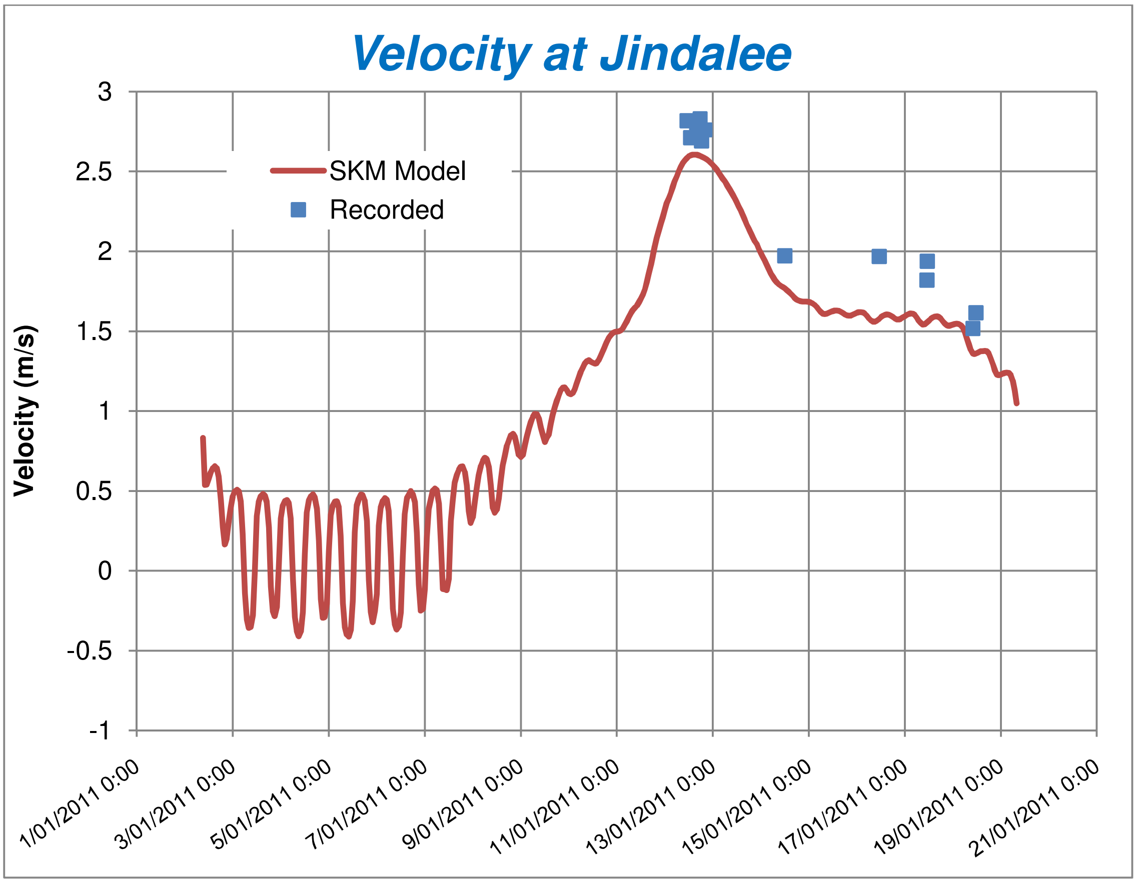
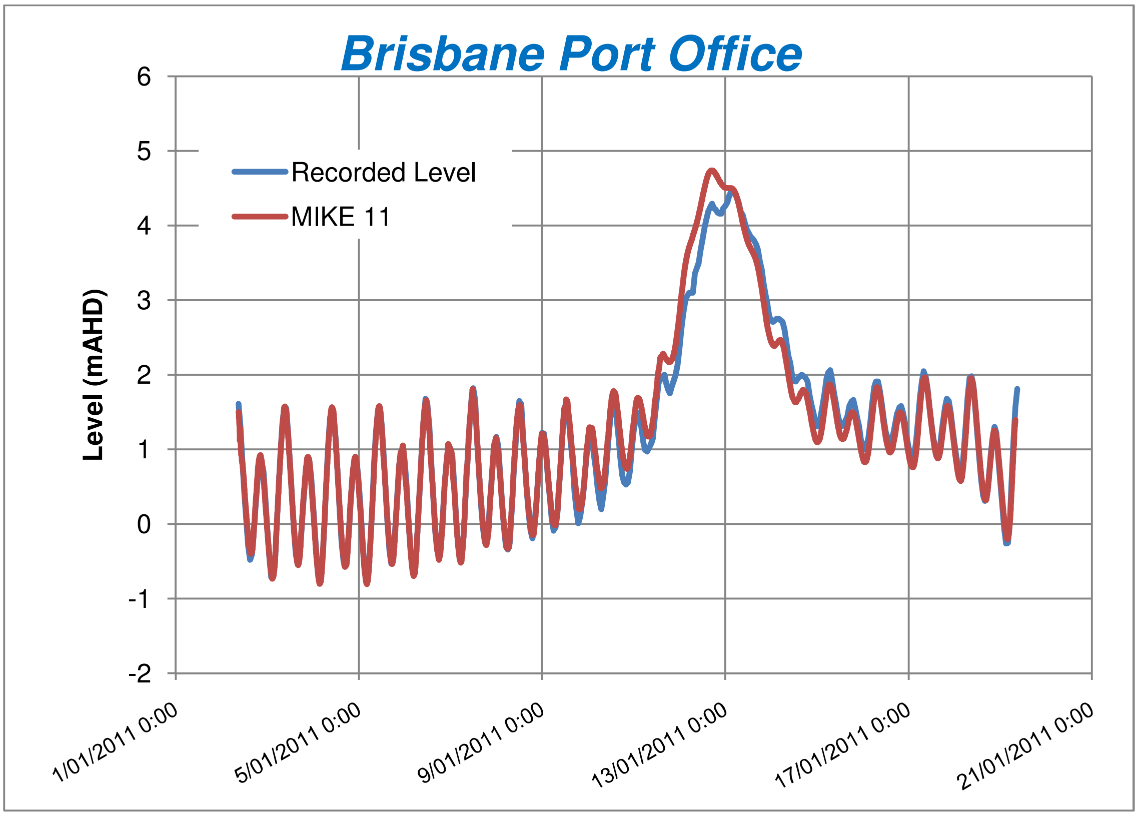


Figure 10: Comparison of gauged and modelled water level – Port Office

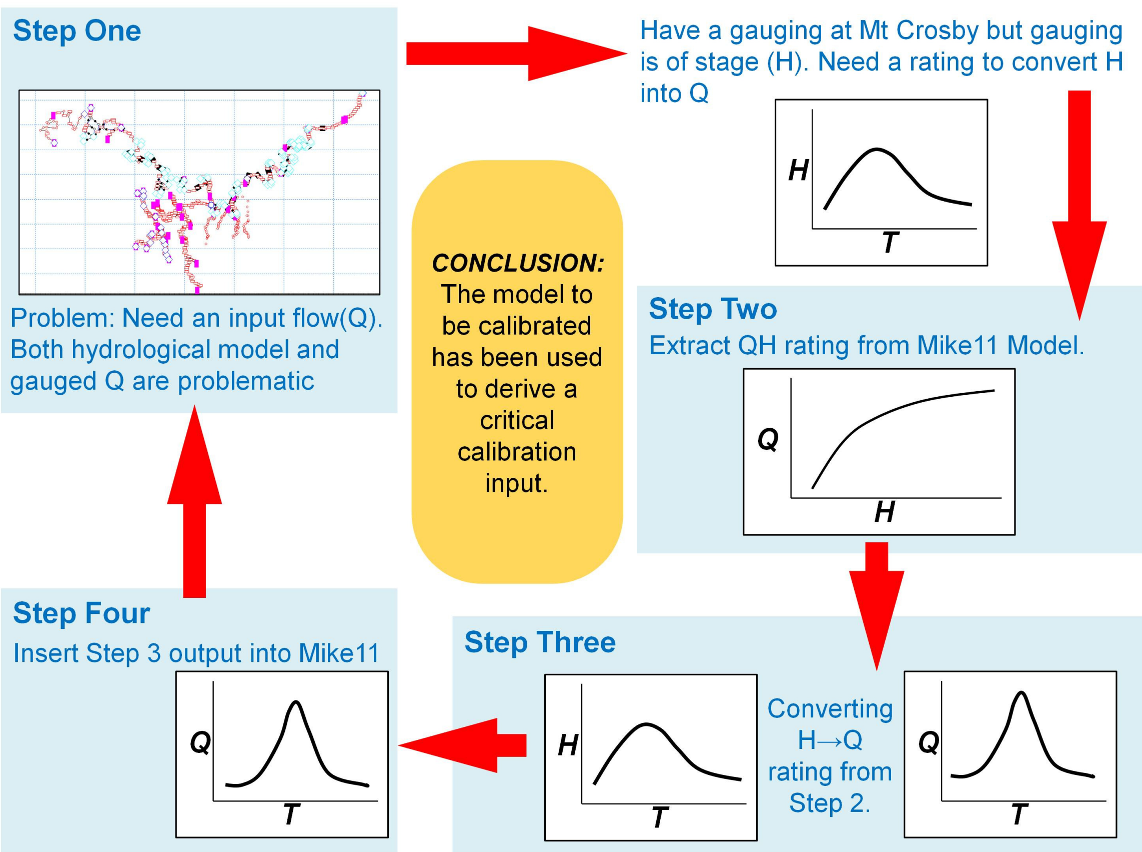


51 Overall the approach has provided a well calibrated modelling tool (between Mt Crosby and Moreton Bay) that can be used to answer the Commissions questions in regard to the January 2011 event and how flood levels downstream of the Dam were impacted by Wivenhoe Dam releases. Further it provides a basis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

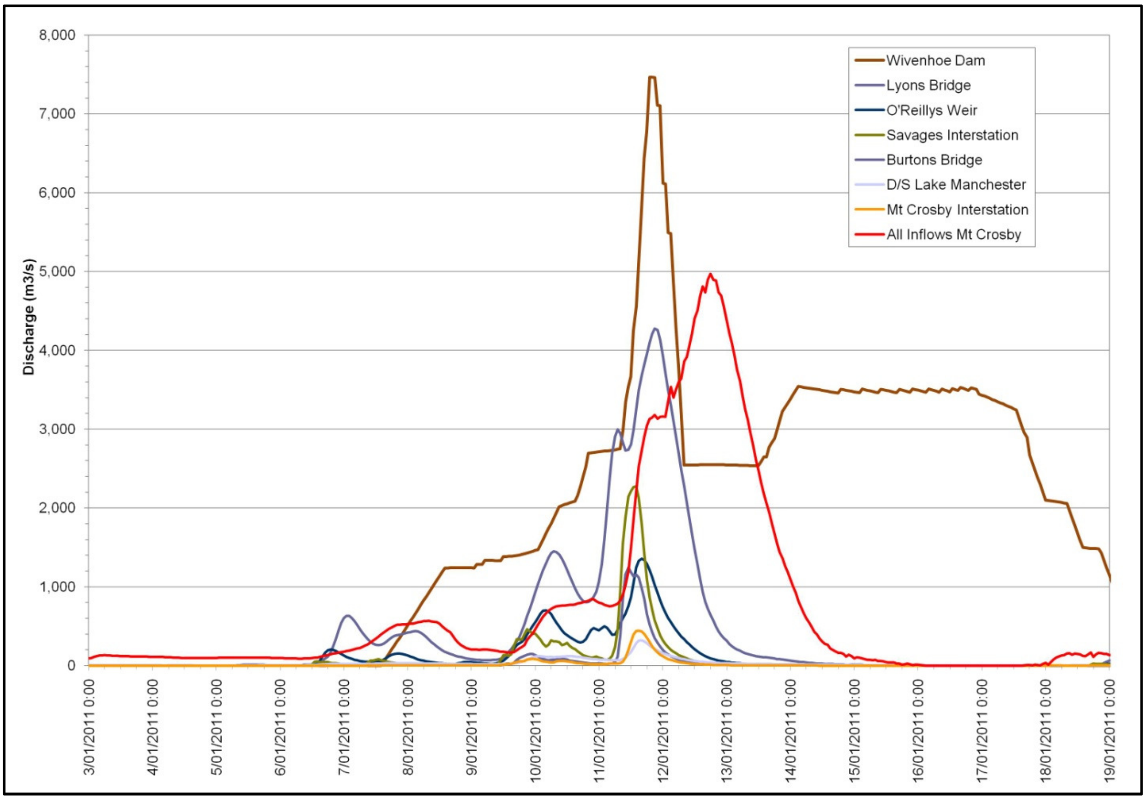
a. Wivenhoe Dam releases (peak flow 7,464 m^3/s);

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.

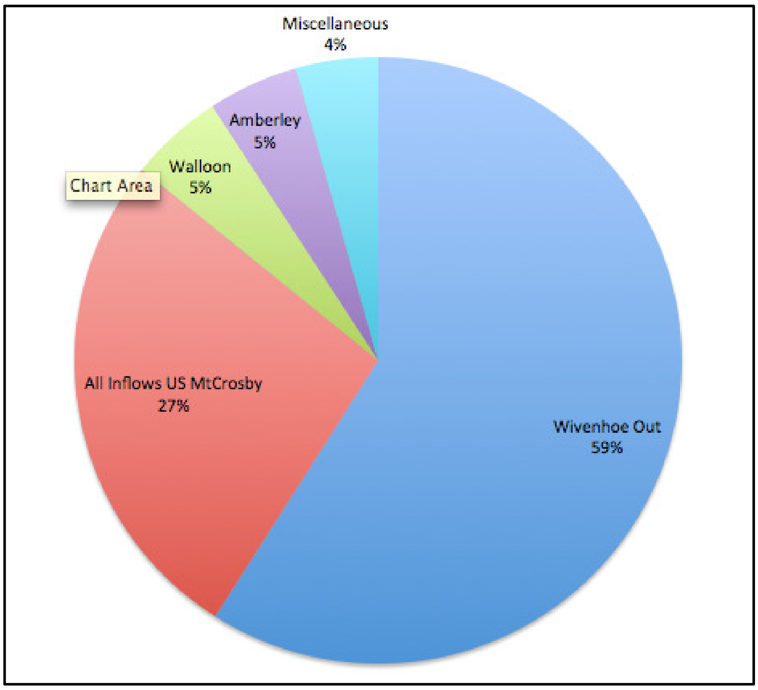
Figure 12: Comparison of various input hydrographs from upper part of model



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

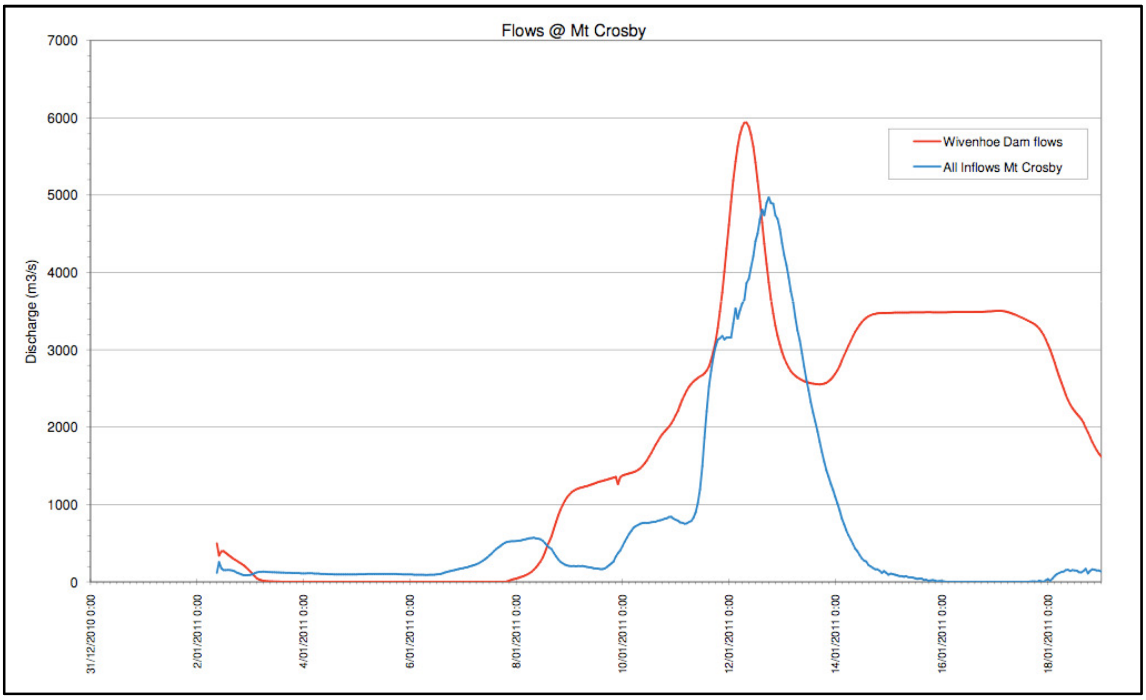
indicative of the total amount of flood runoff received from each of the tributaries, rather than the contribution to the flood peak

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



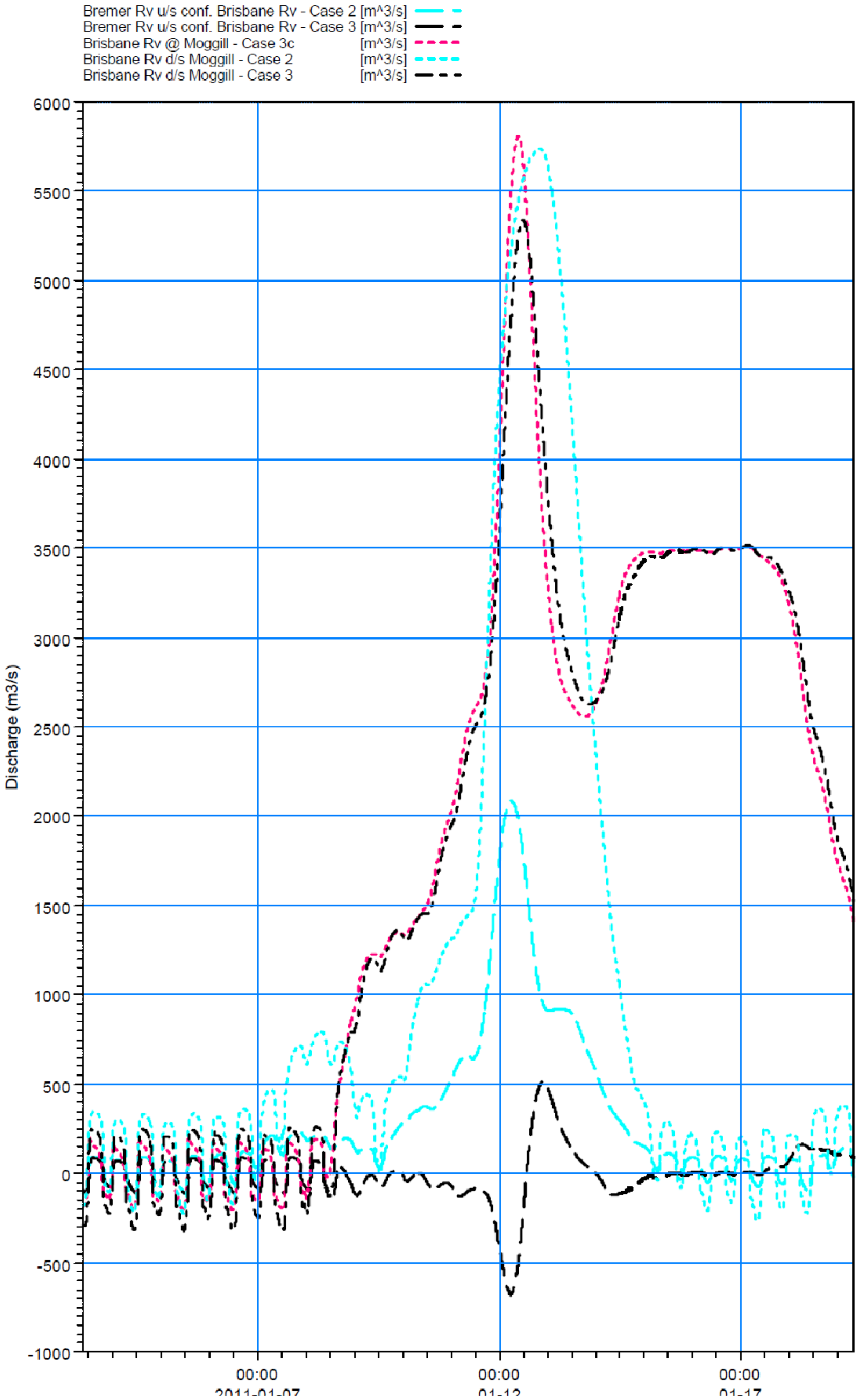
60 Analysis presented by SKM (Reference 2) presented two scenarios – Case 2 and Case 3. Case 2 was a model run of the January 2011 event without any Wivenhoe Dam contribution (but all other model flows as per the calibration run). Case 3 was again a model run of the January 2011 event although with no 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

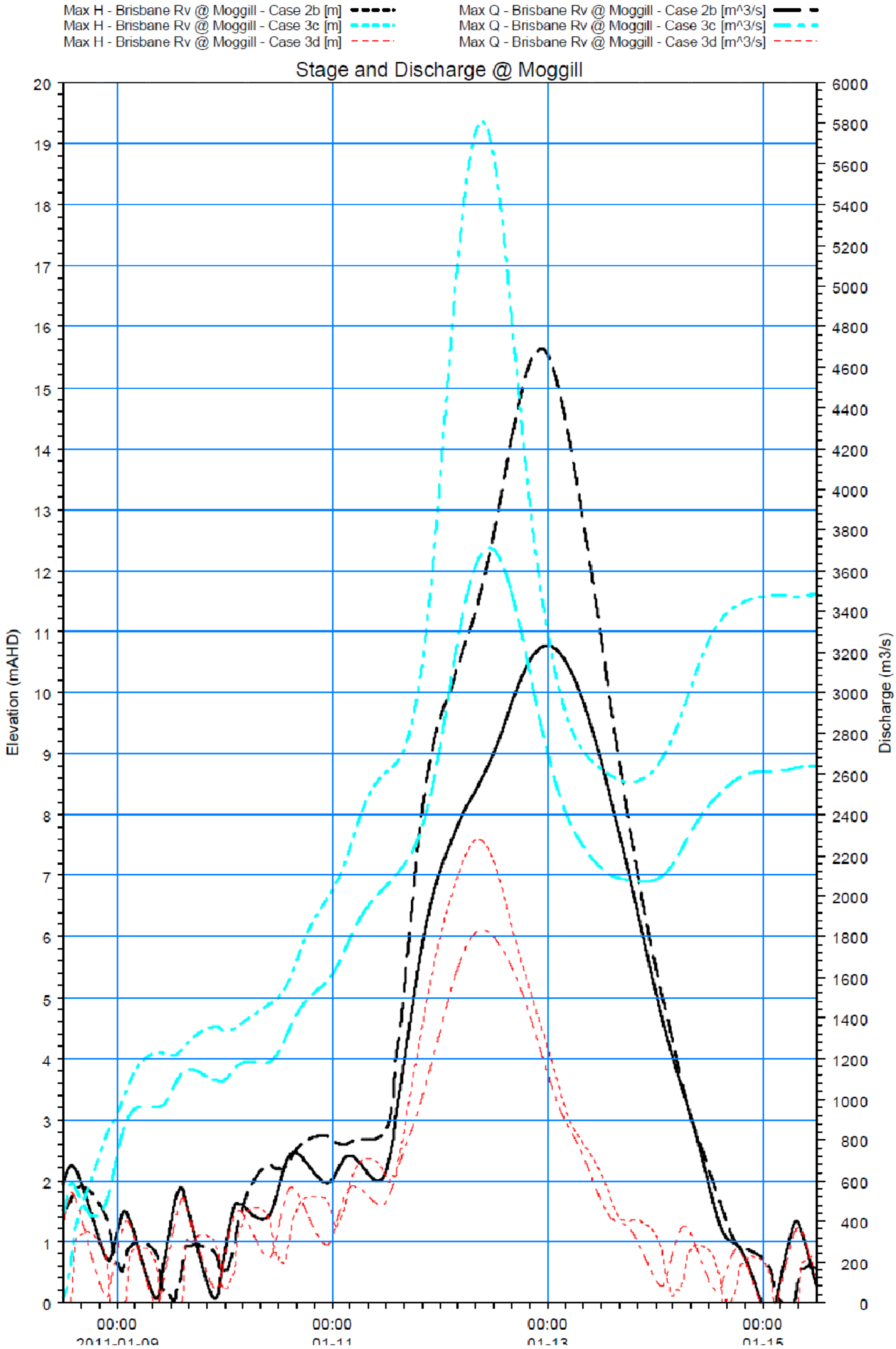


Figure 17: Flood level profile in Bremer River for Calibration Event

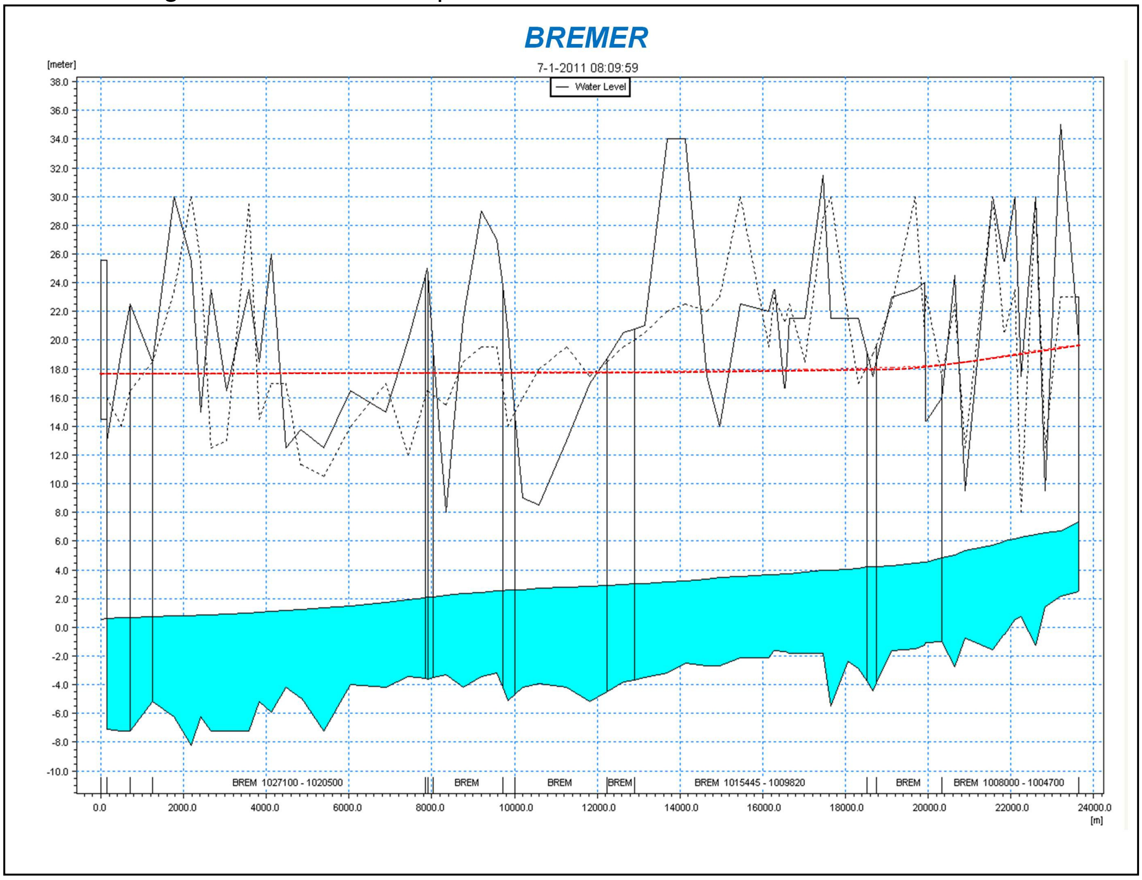


Figure 18: Backwatering of Bremer River System – Calibration Event

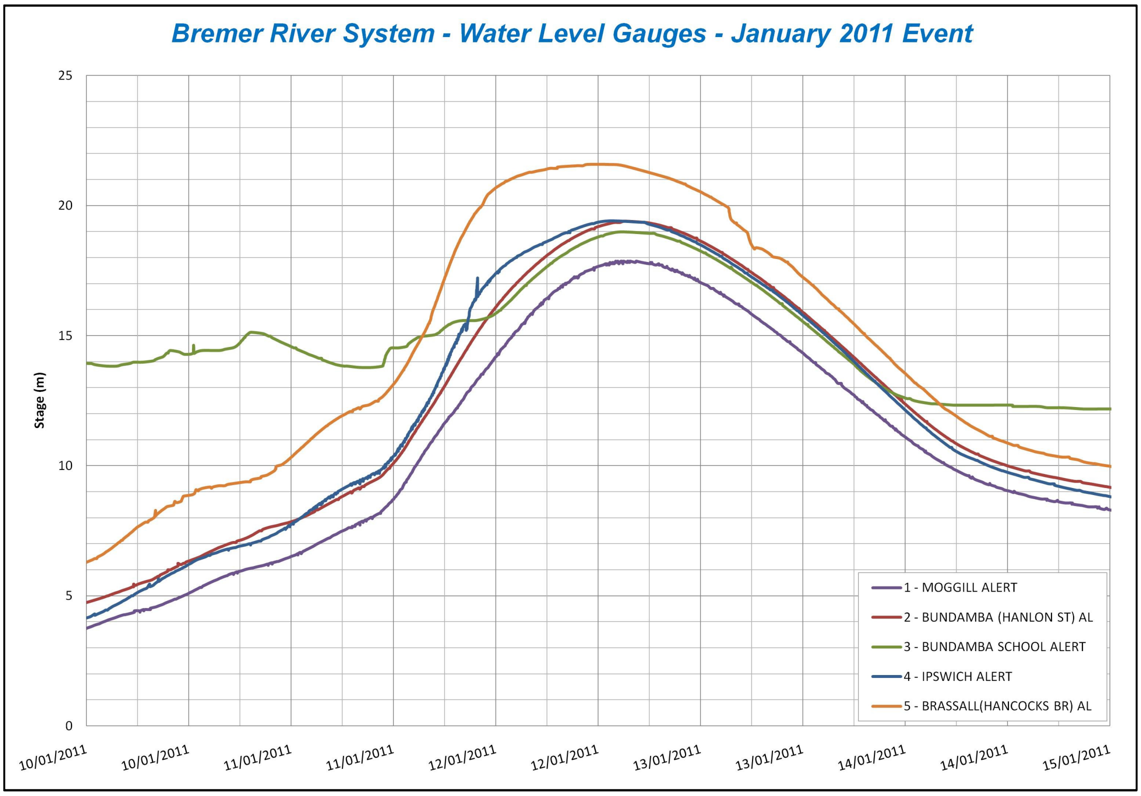
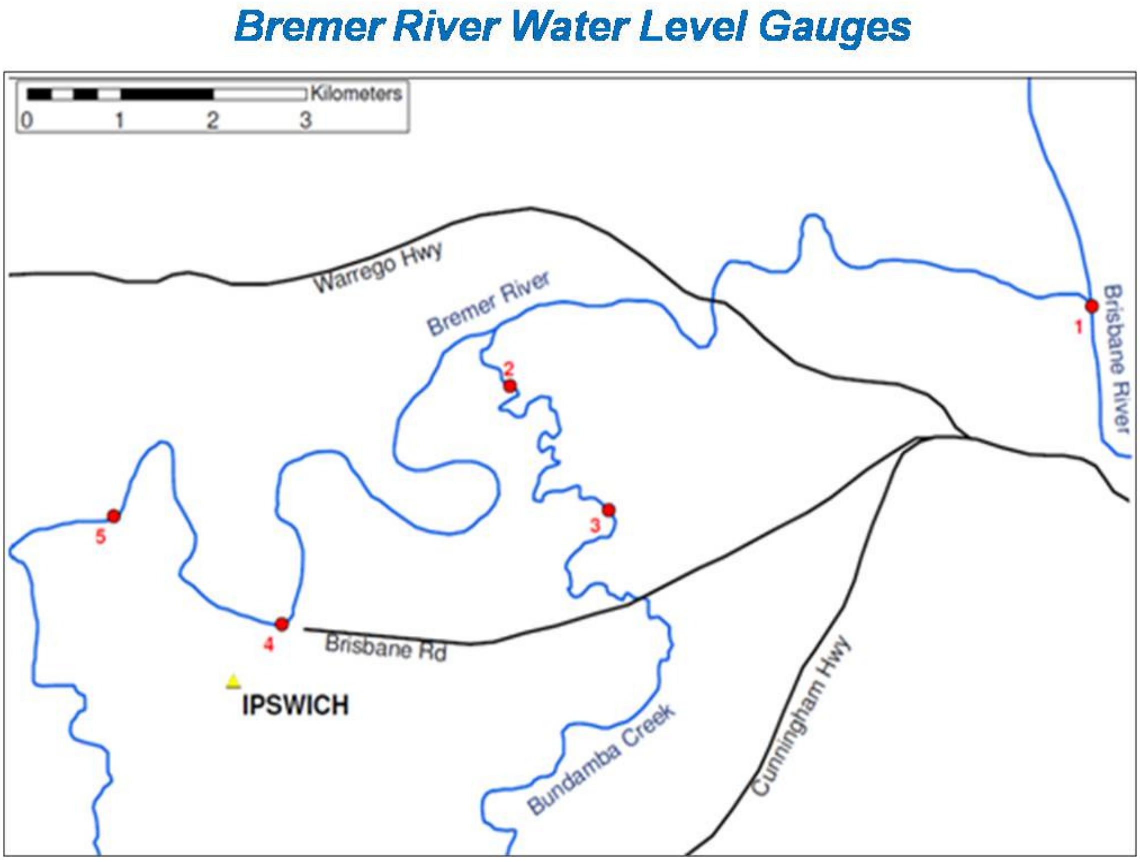


Figure 19: Bremer River System – Gauge Locations



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

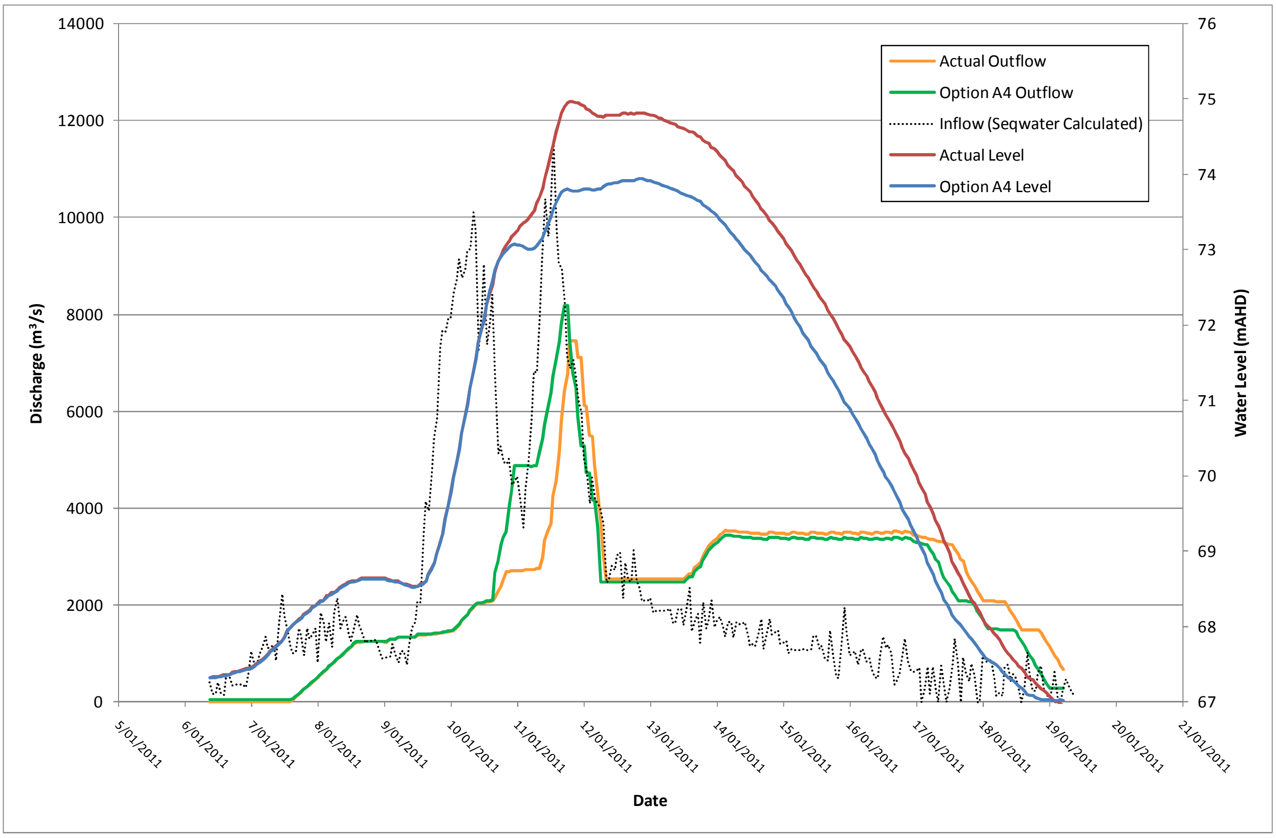
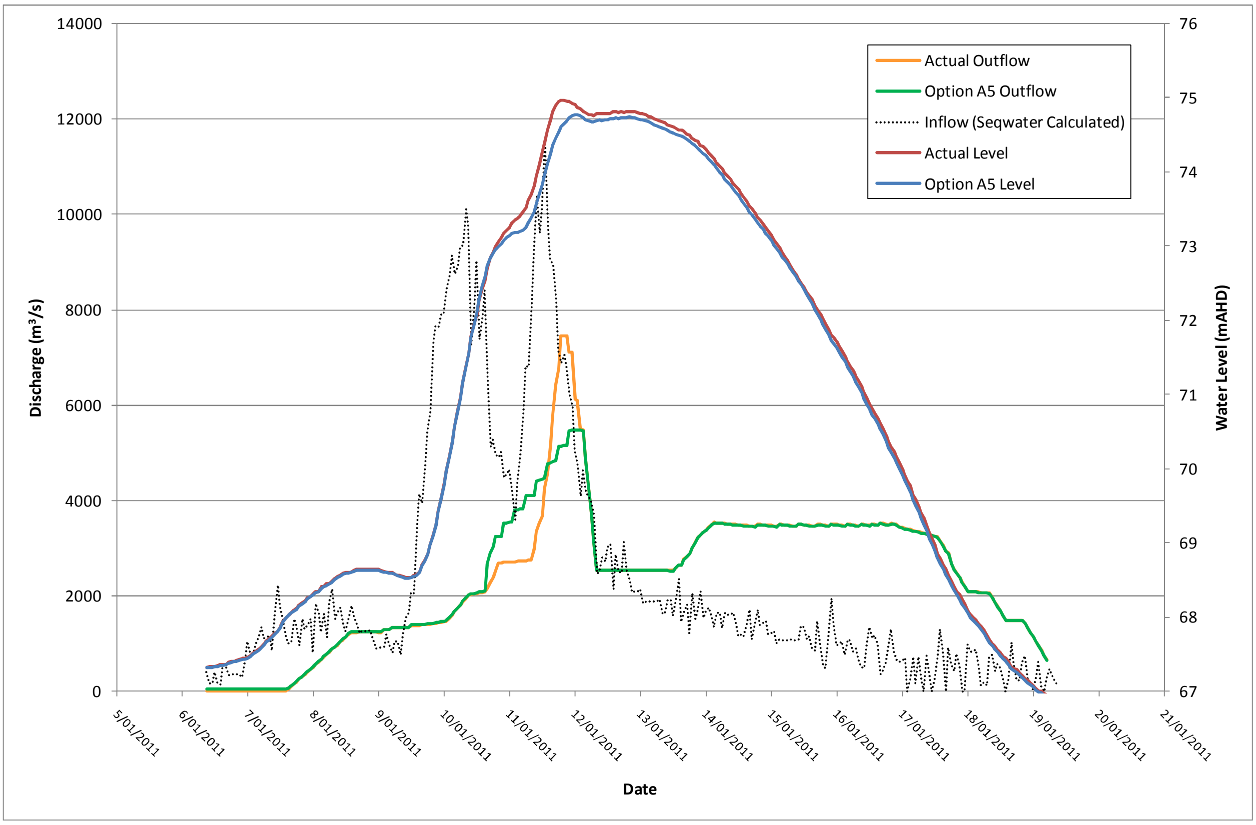


Figure 21: Option A5 Wivenhoe Dam Releases and Water Levels



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.

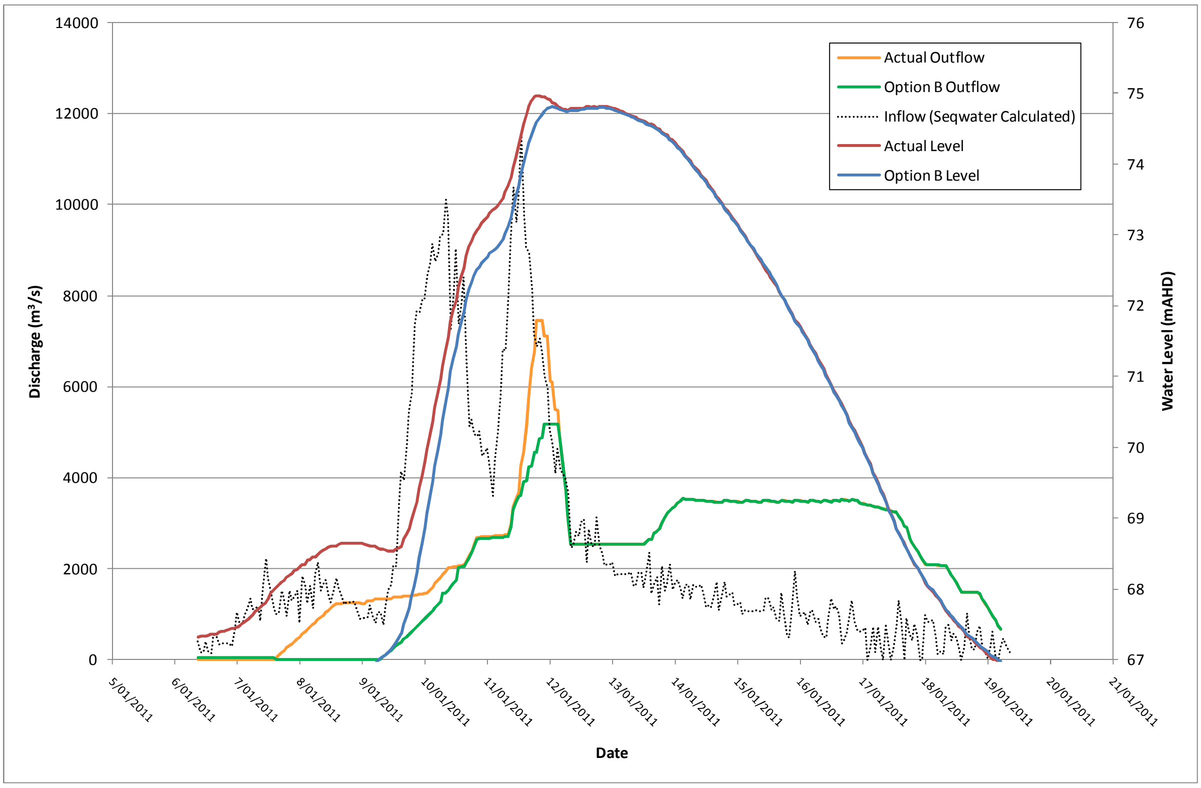
73 Option A5, where the gates are opened at a slower rate, resulting in a similar peak lake level but a lower eventual peak discharge, would have resulted in a relative benefit to flood levels with a reduction of between 0.1 m to 0.3 m at most locations. Further discussion of these outcomes is provided 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.

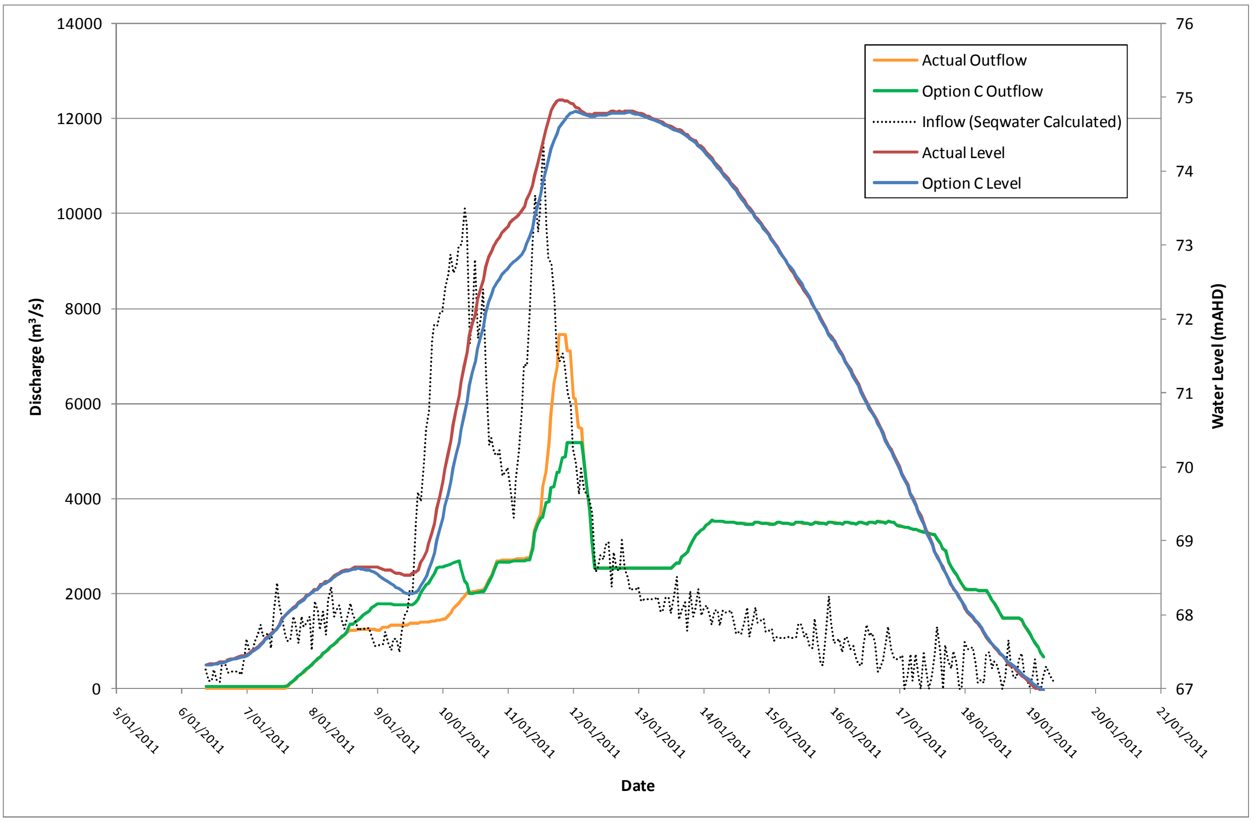
82 For the Option B scenario the effect of keeping the same gate operation strategies rather than changing them to re-allocate the additional storage for flood mitigation is as follows. Roughly 70% of the additional storage space available for flood mitigation (equivalent to 18% of FSL) would have been 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).

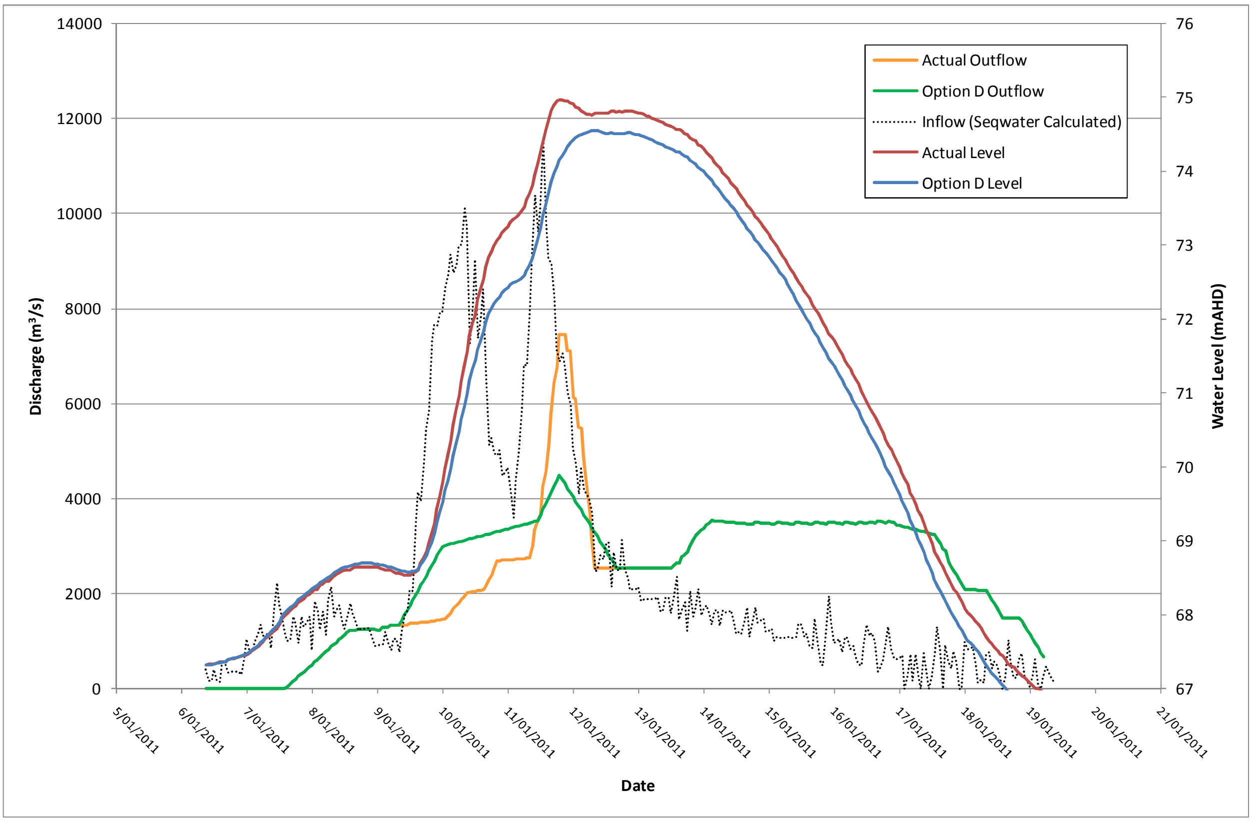
86 The reason for this similarity can be observed by comparing Figure 22 with Figure 23, and noting that from 2pm on 11 January the dam outflows and lake levels would have been very similar under of the two scenarios. This is because the additional flow (compared to what actually occurred) potentially 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

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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.

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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