Memorandum

TO: Queensland Floods Commission of Inquiry
FROM: Mark Babister
DATE: 7 October 2011
SUBJECT: Response to Peer Reviews of WMAwater's *Brisbane River 2011 Flood Event - Flood Frequency Analysis* (Sept 2011)
PROJECT NUMBER: 111024

1 This document addresses various comments and issues raised by the two independent reviews of our *Brisbane River 2011 Flood Event - Flood Frequency Analysis* (WMAwater, September 18th 2011, Reference 9) report:

- *Brisbane River 2011 Flood Event – Flood Frequency Analysis – Review of report by WMAwater*, Dr Rory Nathan, SKM, 28th September 2011
- *Review of Brisbane River 2011 Flood Frequency Analysis* – Dr Michael Leonard, Uni of Adelaide, 26th September 2011

Background

2 We thank both reviewers for their comments. We note that on most matters there is a commonality in opinion between the reviewers. However there are differences. This document does not attempt to address every minor issue raised by the reviewers, but instead addresses the main questions raised by the reviewers and adds clarification where our original explanation was not adequately understood. In preparing this response some discussion has been held with Dr Leonard.

3 Both reviewers have in broad terms endorsed the:
- methodology used to develop the high flow rating curve
- approach used in the flood frequency analysis, and
- pre dam Q100 estimate of 13,000 m^3/s (noting the uncertainty about the estimate).

4 The main issues raised by the reviewers are:

- Dr Nathan (Reference 11) has rejected the approach used to convert pre dam flows to post dam flows and hence the post dam flood levels.

- Dr Nathan has presented some additional observed debris marks for the 2011 event (Figure 3) that in some locations contradict the flood levels presented in the Joint Task Force 2011 report.
- Dr Nathan has raised questions, based on the debris marks, about the design flood profiles presented in our report.
- Dr Leonard (Reference 10) believes we have used implicit knowledge of the 2011 event to determine the post dam estimate.
- Both reviewers recommend the use of Monte Carlo (stochastic) analysis.

Monte Carlo Analysis

5 Both reviewers noted that our report (Reference 9) did not recommend Monte Carlo (Stochastic) Analysis to address the complex joint probability problem when determining flood levels under current conditions (post dam). It is our firm view that a Monte Carlo (Stochastic) would be the most appropriate method of addressing this problem as was recommended in our earlier report (May 2011, Reference 7) to the Commission and in The Commissions Interim Report. We did not reiterate this major recommendation because it had already been covered.

6 We understand that Dr Leonard was not aware of the recommendations in our May report when conducting his review but we find it curious that Dr Nathan who was well aware of this report and its associated expert testimony chose to ignore this fact. The two key quotes from our May 2011 report are set out below (we have used the term Stochastic instead of Monte Carlo).

> *"Substantial revision of the design hydrology methodology should be considered, preferably including a stochastic framework that can reproduce reasonable natural variability in the flood characteristics identified above, through the use of a suite of plausible temporal and spatial rainfall patterns." WMAwater, May 2011, Paragraph 123 (Reference* 7*).*

> *"The design modelling that was first developed in 1983 should be updated to take full advantage of new techniques for design hydrology and improvements in computing power. This should include an investigation of longer duration floods and larger inflow volumes, preferably using an ensemble or stochastic modelling process where a range of plausible temporal and spatial patterns are considered for a full range of flood events..." WMAwater, May 2011, Paragraph 173 (Reference* 7*).*

7 It is also part of the recommendations (2.10 to 2.13) of Queensland Floods Commission of Inquiry Interim Report that a "Stochastic or Monte Carlo or Probabilistic Approach" be used.

Pre Dam to Post Dam Conversion

8 While we thought that Figures 2 – 5 (Reference 9) and the associated text, on the pre to post dam relationship were largely self explanatory we accept the reviewer comment that more explanation was required.

9 The intent of these figures was to demonstrate the variability of pre dam to post dam flows and that there is no fixed ratio as this will vary from one event to the next. It was for this reason that the approximate zone of influence (solid orange line) was drawn around these values. The key aspect of Figures 2 – 5 (Reference 9) is that they include the best estimates of 1893, 1974 and 2011 events, as well as a number of synthetic (design) events. An updated version of Figure 3 is attached which includes a range of plausible adjustments for the impact of Somerset dam. It also shows the most likely zone of influence based on the scatter of the data points.

10 Figure 2 mainly contains Seqwater (Reference 12) and DNR data, while nearly all the points on Figures 3 - 5 were extracted from modelling work carried out by SKM (Reference 6). Even though Figure 2 is derived from different data sets it has a very similar shape to Figures 3 - 5.

11 It is important to understand that for all actual historical flood events either the pre or post dam value has to be estimated using complex modelling that adjusts for the presence or otherwise of the dam. For example the 1893 flood would have been modelled for post dam impacts. For synthetic events both pre and post dam flows are estimated using modelling. All of these events (historical or synthetic) represent the impact of a hydrologic loading on the catchment with and without dams. The historical events are important as while they do not represent the full complexity of the individual historical events they do embody the core characteristics (temporal and spatial patterns) and therefore are a good predictor of real catchments response. Of these the 2011 event is the most important as there is no uncertainty on how the operating procedures would have been carried out. The actual probability of the synthetic events are not that important in this application as they are only considered as hydrologic loadings, it is however important that they have plausible characteristics.

12 Using models it would be possible to add additional data points to these graphs. This would be best done by using a range of design rainfalls with observed temporal patterns and appropriately scaled spatial distributions. If the graph were sufficiently populated it could be used as part of a joint probability exercise to determine post dam flows from pre dam flows. The variability seen on these graphs represents the influence of many of the factors discussed by Dr Nathan in Paragraph 37 Reference 11. When the data points on these graphs are considered as hydrologic loadings there is no "circular" argument as suggested by Dr Nathan (Reference 11, paragraph 19, 55).

13 The 3 historical events (1893, 1974 and 2011) on Figure 3 are described by Dr Nathan as a "miserably small" dataset, they show a dam performance which is very different from the 2003 Review Panels 12 000 m^3/s pre dam to 6000 m^3/s post dam (Reference 5) and SKM (2003) 12 000 m^3/s pre dam to 6500 m^3/s post dam and all 3 events are larger than 6500 m^3/s. This raises serious concerns about the 2003 pre to post dam conversion as not one historical event supports it and it is hard to accept all 3 events are outliers.

14 While we still recommend a full Monte Carlo approach the advantage of this approach would be it doesn't need to make assumptions about the probability of design rainfall or design losses and can be used as a check.

15 Dr Leonard (Reference 10) has suggested that we have used implicit knowledge of the 2011 event to estimate post dam Q100 flow. While we were clearly aware of the dams impact on the 2011 event we believe that the estimate is valid without being aware of this event and makes best use of

the behaviour of large events that were known prior to 2011 (orange dashed line (estimation line) on Figure 3).

Comments of the SKM 2003 Models and Estimates

16 Dr Nathan (Reference 11) suggests that the SKM (2003) study (Reference 6) is the best available information and that *"the findings of the study were independently reviewed and endorsed by an independent panel."* The flood frequency analysis conducted by SKM at Savages Crossing represents best practice. The following commentary raises a number of issues have been documented by others in relation to the data and models used by SKM.

Rainfall

17 One of the concerns of the 2003 Review Panel (Reference 5) was a major misclosure between the Flood Frequency Analysis and the design rainfall method. The implications of this were that the modelling may have substantially underestimated the volume. Sargent (2006a, Reference 13) found a number of issues with the SKM (2003) work when conducting an analysis for Ipswich. Sargent (2006a, Reference 13) found that the CRC FORGE rainfall had been incorrectly input into the RAFTS model for the 24, 30, 36 and 48hr durations.

18 Sargent notes that of the SKM study that:

> *"… the effective rainfalls on the sub areas (i.e. input rainfall minus losses) were consistently lower than those applied in the current study. It was also confirmed that the applied losses were identical, so it was concluded that the input rainfalls were less than those provided in the CRC-FORGE spreadsheet." Sargent 2006a Section 5.1 , Page 11, para 2 (Reference 13)*

19 Once Sargent corrected the rainfalls the misclosure between the FFA and the design rainfall method was removed (refer to Table 1). The corrected pre dam RAFTS estimate is within 3% of the WMAwater estimate of 13 000 m^3/s (Reference 9).

Table 1: Comparison of RAFTS Model Peak Flow Estimates (Reproduced from: Sargent, 2006a, Reference 13). Note Current Study refers to Sargent 2006a.

Location	***Peak Flows (m^3/s) for Storm Durations of***				
	24 Hrs	***30 Hrs***	***36 Hrs***	***48 Hrs***	***72 Hrs***
a) Values from SKM (2003) Table 4-2					
Savages Crossing	*8,387*	***9,607***	*8,379*	*8,626*	*9,192*
Moggill	*7,607*	*9,015*	*7,588*	*8,004*	***10,101***
Brisbane Port Office	*7,608*	*9,015*	*7,589*	*8,005*	***10,106***
b) Current Study					
Savages Crossing	*9,700*	***13,140***	*11,400*	*9,700*	*9,100*
Moggill	*8,600*	***12,600***	*11,800*	*10,000*	*10,200*
Brisbane Port Office	*8,600*	***12,600***	*11,800*	*10,000*	*10,200*
Difference between b) and a) %					
Savages Crossing	*+16%*	*+37%*	*+36%*	*+12%*	*+9%*
Moggill	*+13%*	*+40%*	*+56%*	*+25%*	*+1%*
Brisbane Port Office	*+13%*	*+40%*	*+56%*	*+25%*	*+1%*
NOTE: Critical duration values shown in bold type					

RAFTS Modelling

20 Further, Sargent (2006a, Reference 13) also found the SKM (2003) RAFTS model has been set up in a very unorthodox way. Typically for a large rural catchment flow is routed through the each subcatchment or river reach. However, an approach often used for an urban situation has been used, where flow in each reach has been simply lagged without attenuation. Most of the attenuation takes place in a small number of large conceptual storages including the:

- Confluence of the Brisbane River and the Bremer River, and
- Confluence of the Brisbane River and Lockyer Creek.

21 Given these two large conceptual storages are just above two major calibration points, Moggill and Savages Crossing, there is serious concern that these storages are the only locations where the model estimates are reliable. Sargent (2006a, Reference 13) also found that these conceptual storages produced very different routing behaviour for different storm durations.

Hydraulic Model

22 KBR (2002, Reference 14) found that the use of the resistance radius method in the Mike 11 model developed by SKM was having major effects on the models behaviour for events that were not a similar order of magnitude to the calibration event. KBR (2002, Reference 14) recommended switching to the total area hydraulic radius procedure.

> *"In general, the conveyance value calculated in the previous study has been overestimated using the Resistance Radius procedure. The adoption of the new procedure for calculating hydraulic radius has increased water levels in some locations despite the significant reductions in Manning's n roughness".*
> *KBR, 2002, page 2* (Reference 14)

23 This finding regarding resistance radius is similar to the findings of the WMAwater 2011 Hydraulic Modelling Report (Reference 8) which also identified issues with the use of the resistance radius method in this catchment.

Observed peak flood level data for the January 2011 flood event

24 At the time of issuing WMAwater's Flood Frequency Analysis report (Reference 9), the only datasets available that contained information on peak flood levels for the January 2011 floods were the peak flood level marks indicated in Table 3 of the Joint Task Force March 2011 report (Reference 15) and stream gauge station observations along the Brisbane River.

25 In his response Dr Nathan (Reference 11, Figure 3) refers to observed data points for the 2011 event collected by Brisbane City Council that are different to those listed in the Joint Task Force March 2011 Report. The Joint Task Force (2011, Reference 15) did note that the observed levels contained within the report were draft and subject to verification.

26 The data points used by Dr Nathan were not made available to WMAwater and no source is included in Dr Nathan's review. As a result further assessment was not possible as the data points have not been tabulated. However, if these data points prove to be more reliable than the Joint Task Force March 2011 levels then these data points would suggest that the calibration of the Mike 11 model was not as poor as originally thought. Figure 3 would suggest that within 10km up and downstream of Jindalee the Mike 11 model fits the observed data reasonably well. There are still some issues with the calibration between Oxley Creek and the Port Office.

Clarification of method used to derive flood profiles

27 Dr Leonard (Reference 10) has wrongly interpreted that WMAwater calibrated the Mike 11 model to fit the 2011 Joint Taskforce data (Reference 15) and used this revised model to determine the 1% AEP levels.

28 WMAwater did not use the Mike 11 model to determine the 1% AEP levels because it was not practical to recalibrate the model in the time available. An alternative approach was undertaken by WMAwater to obtain a reasonable estimate of the Q100 levels along the Brisbane River (from Moggill to Brisbane Port Office). The basis of this approach was to utilise the observed peak flood level marks along Brisbane River (Reference 15), to derive the January 2011 peak flood profile. The Mike 11 model was used to estimate how far flood levels for design flows of 9500 m^3/s and 9000 m^3/s were below the January 2011 flood of approximately 9850 m^3/s.. Because the Joint Task Force flood levels were a fair distance apart a straight linear interpolation between the points was not considered appropriate. The shape of the flood profile between the observed flood levels was guided by the shape of the Mike 11 profile for the 2011 event.

29 While it would be better to recalibrate the model this approach is can be readily applied to the new dataset reported by Dr Nathan or any subsequent dataset. The flood levels for post dam design flows of 9500 and 9000 m^3/s can be presented relative to the 2011 observed flood levels. Table 2 shows the height of the revised Q100 lines relative to the 2011 flood levels. The values in the table

can be easily interpolated to determine design levels at any location where there is a reliable measurement of the 2011 level.

Table 2: Height of the Q100 below the 2011 flood level

Location	Height of the WMA Q100 (9500 m^3/s) below the 2011 level (mm)	Height of the WMA Q100 (9000 m^3/s) below the 2011 level (mm)
13 Bridge St., Redbank (off-bank)	400	980
Cnr. Ryan St. and Woogaroo St., Goodna	410	980
Cnr. Moggill Rd. and Birkin Rd., Bellbowrie (off-bank)	410	960
Cnr. Thiesfield St. and Sandringham Pl., Fig Tree Pocket	360	800
312 Long St. East, Graceville	340	730
Brisbane Markets, Rocklea	330	710
Softstone St., Tennyson (Tennyson Reach Apartments)	320	700
15 Cansdale St., Yeronga (off-bank)	270	610
42 Ferry Rd., West End	200	490
81 Baroona Rd., Paddington (off-bank)	180	460
Brisbane City Gauge	140	390

Flood Frequency Analysis - Data

30 While both reviewers have endorsed the pre dam flood frequency estimate as reasonable, the following issues raised by the reviewers are addressed.

31 Dr Nathan (Reference 11 Paragraph 33) writes that "*It is not clear why WMAwater did not critically review the extensive flood frequency analysis undertaken by SKM (2003).*". The main reason was that it was not undertaken at the Port office and could not be considered an "at site analysis" as Savage's Crossing, is considerable distance upstream (in the order of 100km) of the Port Office Gauge. While flood frequency estimates at Savages Crossing can be translated downstream it is necessary to assume that the attenuation over this long reach is exactly balanced by the Bremer inflows. For this reason the SKM (2003) (Reference 6) estimate was not included in the list of similar estimates in Paragraph 131 (Reference 9). It should be noted that the two estimates referenced were within approximately 5-6% of our estimate which is very different to the 30% bounds discussed by Dr Nathan.

32 Dr Nathan also questions why the 1999 December City Design (Reference 4) Q100 estimate was not included in the list of similar estimates. The Q100 pre dam in the version of the City Design December 1999 report provided to WMAwater contains no text regarding how this estimate (which is contained in a figure in the partial Appendix A provided) was derived.

33 Footnote 2 of Dr Nathan's review (Reference 11) suggests the flood level data used in Appendix B of our report was incorrectly attributed to SKM and should be City Design June 1999. WMAwater have been provided with 2 separate versions of the June 1999 Brisbane River Flood Study, neither of which are complete and one of which has an SKM logo on the front cover. Dr Nathan will no doubt understand the confusion with so many versions of reports floating around as he has himself mistakenly referenced the December 1999 City Design Report as the June 1999 report (Table 1, Reference 11).

34 Dr Nathan raised several questions about the assumptions behind some of the data used in the flood frequency analysis.

35 Two inconsistent flow values are given in Table 1 of City Design June 1999 (Reference 3) for the 1931 flood event: 7000 m^3/s and 6245 m^3/s. The table suggests that the removal of the dams reduce the flow. This makes no sense as even Somerset dam wasn't in place in 1931 and its removal would make flows go up. The flow value of 7000 m^3/s was used as it was more compatible with the other data.

36 The 1974 pre dam flow of was based on a consideration of estimates with Somerset dam (described in Paragraph 101 of Reference 9). These estimates range from 9800 to 10 900 m^3/s. Greater emphasis was put on the upper end of the range which is more consistent with the adopted rating curve. City Design (June 1999, Reference 3) suggests at the Port Office the adjustment to pre Somserset dam is 490m^3/s. While other sources suggest this adjustment may be higher. A pre dam estimate of 11 300 m^3/s was adopted.

37 The 2011 pre dam estimate was developed using the SKM pre dams 2011 flood level of 6.4mAHD (Reference 16, Table 7-2 Case 5) and the adopted rating curve.

References

1. SKM
   Brisbane River Flood Study
   Brisbane City Council, June 1998.

2. Mein, R.
   Brisbane River Flood Study Review of Hydrological Aspects
   Monash University December 1998.

3. City Design
   Brisbane River Flood Study
   Brisbane City Council, June 1999.

4. City Design
   Further investigations into the Brisbane River Flood Study
   Brisbane City Council, December 1999.

5. Mein, R., Apelt, C., Macintosh, J., Weinmann, E.
   Review of Brisbane River Flood Study
   Independent Review Panel, 3 September 2003.

6. SKM
   Brisbane River Flood Study - Further Investigation of Flood Frequency Analysis Incorporating Dam Operations and CRC-Forge Rainfall Estimates - Brisbane River
   December 2003.

7. WMAwater
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   May 2011

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9. WMAwater
   Brisbane River 2011 Flood Event – Flood Frequency Analysis
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10. Dr Michael Leonard, Uni of Adelaide
    Review of Brisbane River 2011 Flood Frequency Analysis
    26th September 2011

11. Dr Rory Nathan, SKM,

Brisbane River 2011 Flood event – Flood Frequency Analysis – Review of report by WMAwater,
28th September 2011

12. SEQ Water
Report on the Operation of Somerset Dam and Wivenhoe Dam
March 2011.

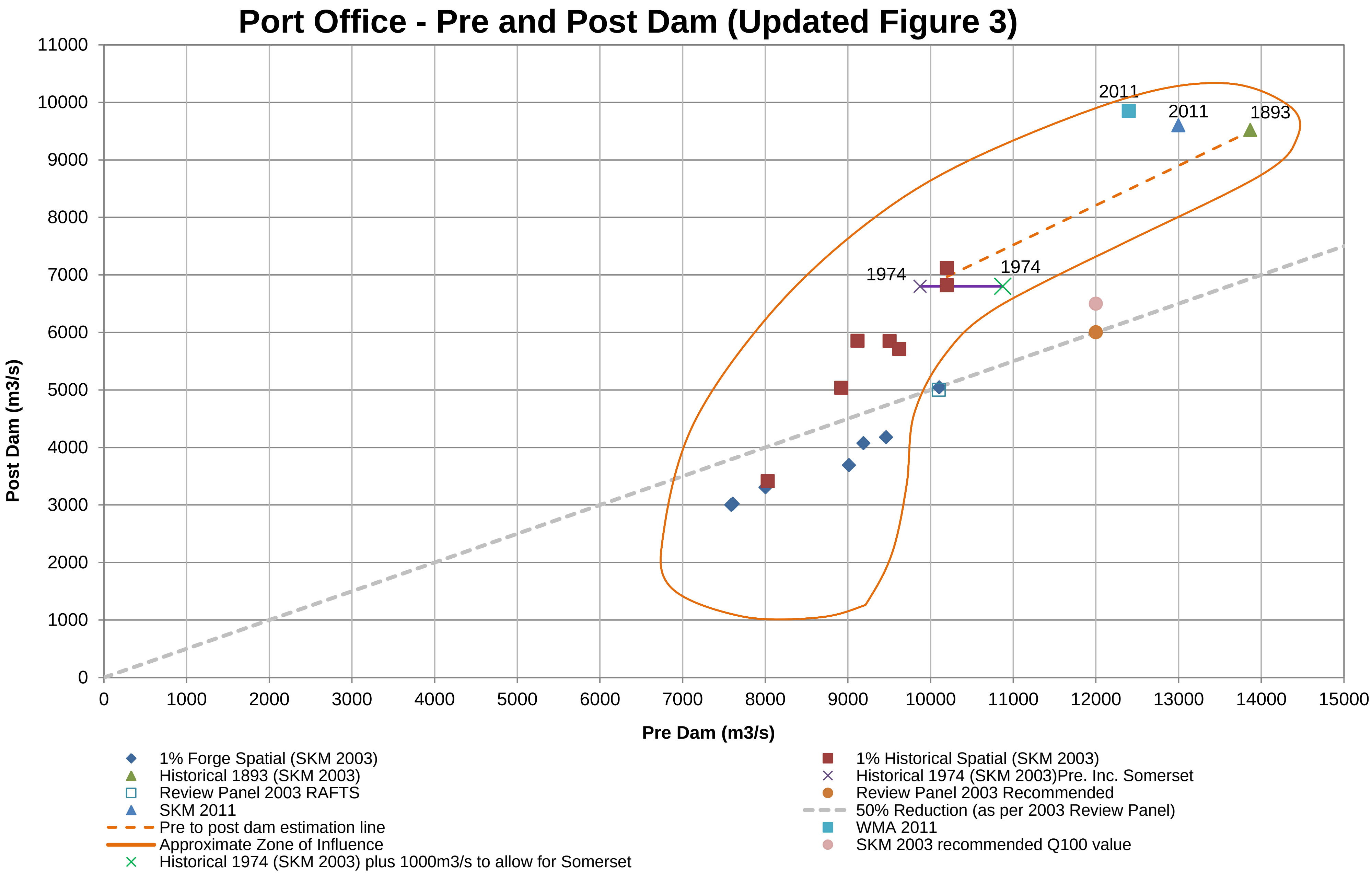
13. Sargent Consulting
Ipswich Rivers Flood Study Rationalisation Project Phase 3 – "Monte Carlo" Analysis of Design Flows
January 2006

14. Haliburton KBR
Ipswich Rivers Flood Studies – Lower Bremer River Flooding Report
8 May 2002

15. Joint Flood Taskforce
Independent Review: Joint Flood Taskforce Report
8 March 2011.

16. SKM
Joint Calibration of a Hydrologic and Hydrodynamic Model of the Lower Brisbane River –Technical Report
Seqwater, 24 June 2011

17. SKM
Joint Calibration of a Hydrologic and Hydrodynamic Model of the Lower Brisbane River –Technical Report
Seqwater, 5 Aug 2011



Port Office - Pre and Post Dam (Updated Figure 3)
Post Dam (m3/s)
Pre Dam (m3/s)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
12000
13000
14000
15000
2011
2011
1893
1974
1974
1% Forge Spatial (SKM 2003)
Historical 1893 (SKM 2003)
Review Panel 2003 RAFTS
SKM 2011
Pre to post dam estimation line
Approximate Zone of Influence
Historical 1974 (SKM 2003) plus 1000m3/s to allow for Somerset
1% Historical Spatial (SKM 2003)
Historical 1974 (SKM 2003)Pre. Inc. Somerset
Review Panel 2003 Recommended
50% Reduction (as per 2003 Review Panel)
WMA 2011
SKM 2003 recommended Q100 value

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14 October 2011
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[redacted]
Clayton Utz
Brisbane

By Email: [redacted]

Dear Mr [redacted]

QUEENSLAND FLOODS COMMISSION OF INQUIRY
REVIEW OF ASPECTS OF THE REPORT OF WMAWATER (SEPT 2011)

Following your recent telephone instructions I have prepared a review of aspects of the September 2011 report of WMAwater entitled "*Brisbane River 2011 Flood Event – Flood Frequency Analysis*". My advice is set out below. This review has been prepared under significant time constraints and has been limited to the matters listed below in bold type face.

What enquiries and investigations might affect any determination of the 1% AEP flood level following the January 2011 flood event?

1. This question must be answered in the context of information that is already available and the use to which the determination of the new flood levels will be put. Models are only approximations of real behaviour and it is always possible to improve model accuracy through additional effort but the law of diminishing returns applies.

2. For example, in the context of other parts of Queensland where no flood data exists, the sort of approach espoused in the temporary planning policy using the interim floodplain assessment overlay might be appropriate. (However such interim overlays would not be appropriate for Brisbane).

3. Given the (large – by Australian standards) number of flood studies that have already been undertaken and given the interim DFL that Council has implemented, the enquiries and investigations to be undertaken would, in my opinion, be similar to the new flood study that has been proposed, i.e. a full scale review of both flood frequency and rainfall-runoff approaches, in the light of the new information from the 2011 event.

4. In general terms, WMAwater have:

   (a) *Step 1* – used a flood frequency approach to estimate pre-dam flow frequencies;
   (b) *Step 2* – converted this to a post-dams flow regime using a mixture of past (i.e. 2003) and new (i.e. 2011) information; and
   (c) *Step 3* – applied these flows to calculate flood levels using a hydraulic model (that has been improved using 2011 data).

5. I do not find *Step 1* of much benefit. This uses a procedure previously applied by others (e.g. BCC, June 1999) which has also been rejected by others because of

uncertainties in the available data. WMAwater have not addressed these uncertainties (or undertaken sensitivity tests to examine the potential impacts of these uncertainties). I have provided further notes on these uncertainties in the Attachment.

6. In relation to *Step 2*, WMAwater have brought information to light which demonstrates that longer floods with bigger volumes get attenuated less by the dams. This is not in itself remarkable. There are also other factors affecting the attenuation (e.g. spatial and temporal distributions) and the 2011 event has and will continue to provide very valuable information to improve our understanding of hydrology and flood behaviour in the catchment during major flood events.

7. In relation to *Step 3*, the 2011 flood has allowed better calibration of the MIKE11 model and allowed flood level predictions to be made more accurately over more areas of the floodplain (once the design flows were determined).

8. Because of improvements in *Step 2* discussed above, WMAwater indicates the 1% AEP flood levels will be higher. I believe that this will be the likely outcome of the proposed new flood study (although at different levels from those determined by WMAwater). However I do not believe there is much benefit in publishing new estimates of flood levels from the WMAwater report at this time because:

   (a) the flood levels have uncertainties associated with them and the work has not been done to minimise these (e.g. allowing for tides, a more rigorous assessment of river bed/bank changes);
   (b) the proposed new flood study will produce new flood levels which will be different from the WMAwater flood levels; and
   (c) Brisbane City Council has already adopted an interim DFL that is equal to or higher than the 2011 flood level. WMAwater's new 1% AEP flood levels are lower than the interim DFL.

What factors affect the reliability of converting historically recorded levels into flows and whether or not the WMAwater report has had regard, or sufficient regard, to those factors?

9. These factors are:
   (a) uncertainties in the original measurements;
   (b) uncertainties in adjustments made for dredging and other river bed/bank changes;
   (c) uncertainties in discharge estimates due to tidal influences; and
   (d) uncertainties in discharge estimates due to the rating curve.

10. These factors are discussed in the attachment. In summary, based on the information I have been able to look at so far in the time available to me, it is appears:

   (a) there are significant uncertainties associated with the preparation of the discharge estimates prepared by WMAwater for use in their flood frequency analysis;
   (b) these uncertainties directly impact on the estimate of the 1% AEP discharge which WMAwater have derived;
   (c) many of these uncertainties are acknowledged by WMAwater. It would appear that they have had insufficient time available to address these uncertainties;

(d) it would have been good practice to test the sensitivity of the 1% AEP discharge estimate to potential changes in the discharges used to prepare the flood frequency analysis. This has not been done;

(e) the 90% confidence limits for the derived 1% AEP discharge are already moderate (i.e. 10,000m^3/s to 22,000m^3/s). These limits however assume no inaccuracies in the historical discharges. Therefore the true confidence limits will be somewhat larger than those quoted; and

(f) many of the difficulties with carrying out flood frequency analyses at the Port Office gauge have been recognised by previous studies which is why these studies have carried out their analyses further upstream beyond the influence of tides and dredging.

Review the appropriateness of Figure 3 (page 15, WMAwater report) and the use to which it has been put within the report.

11. Figure 3 contains a "*pre to post dam estimation line*" prepared by WMAwater. A copy of Figure 3 has been reproduced below.

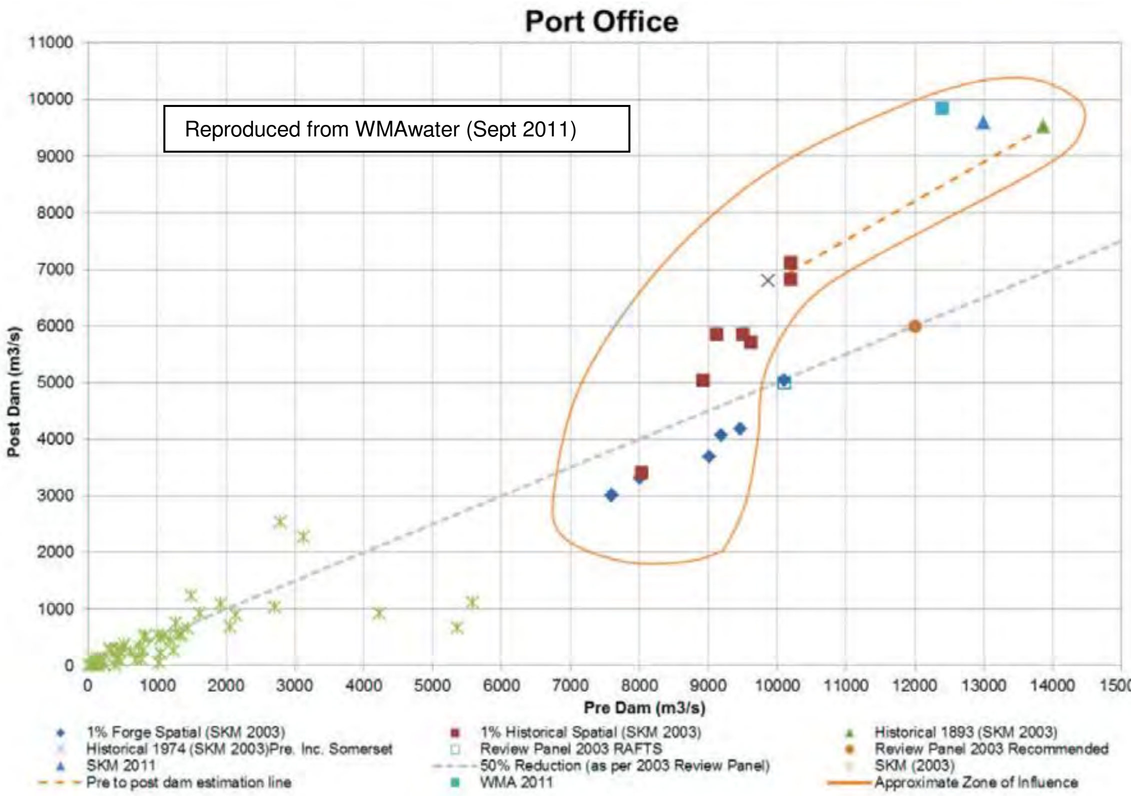

Figure 3: Port Office- Pre and Post dam flow

12. This estimation line was used by WMAwater to convert their pre-dams 1% AEP discharge estimate of 13,000m^3/s to a post-dams discharge of 9,000m^3/s (without the influence of the 2011 event). It was also used (together with 2011 flood data) to estimate the post-dams discharge of 9,500m^3/s (assuming knowledge of the 2011 flood). It is therefore a vital component of WMAwater's process for the determination of 1% AEP flows at the Port Office gauge.

13. The procedure used by WMAwater to prepare this line has not been explained in their report. The line appears to be either:

(a) a line representing a 32% reduction in pre-dam flows; and/or
(b) a line drawn between data for the 1893 flood and the highest two "*1% Historical Spatial*" data points.

14. The "*pre to post dam estimation line*" assumes a linear relationship between pre- and post-dams discharges. There will be a reduction in the amount of attenuation as the magnitude of flood discharges and volumes increases during major floods so a curve rather than a line is appropriate.

15. Some of "*1% Historical Spatial*" data on Figure 3 do not appear to be plotted in the correct position. The data likely originates from the "*critical flows*" presented in Appendix H of SKM (2003) for the seven historical spatial patterns considered by SKM. For some of the seven patterns however the data plotted are for different storm durations. Thus the pre-dam and post-dam discharges do not correspond to the same catchment event.

16. The "*Historical 1974*" data point included on the figure includes for the effects of Somerset Dam in the pre-dams discharge. WMAwater appear to be aware of this but it is unclear why it was included on the figure without first making an adjustment to remove the effects of Somerset Dam.

17. The probability of the "*1% FORGE Spatial*" data is known however the probability of the "*1% Historical Spatial*" data is not known. A difficulty arises in using such a diagram in the manner that WMAwater have used it because for a given pre-dam discharge, all of the data points do not have the same probability of occurrence.

18. There is considerable scatter in the data points on Figure 3. Floods come in all shapes and sizes. If all of them were plotted the scatter would likely be much wider than that shown. Floods that have more volume for a given pre-dam discharge will likely plot higher on the figure than those with a lower volume for the same discharge as these are subject to less attenuation by the dams. A range of attenuations is possible.

19. A key question is what attenuation amount from the range of possible attenuation amounts, should be used to attenuate the 1% AEP pre-dam discharge determined by WMAwater from their flood frequency analysis? This issue has not been explored by WMAwater.

20. In conclusion, assuming the pre-dam discharge has a probability of 1%, the post-dam discharge determined using the "*pre to post dam estimation line*" will likely have some bias away from a 1% AEP event.

21. WMAwater have no doubt been under significant time pressures in responding to the questions asked of them by the Commission. They have quickly and rather pragmatically produced an estimate of the 1% AEP post-dams discharge at the Port Office gauge which in my opinion, still contains significant uncertainties.

Yours sincerely

[redacted]

Drew Bewsher
Director

ATTACHMENT TO LETTER OF 14 OCTOBER 2011

Uncertainty in 'Measurements' of Flood Heights at the Port Office Gauge before 1875

22. The Port Office gauge was likely installed between 1875 and 1878[1]. Consequently records prior to 1875 must have been inferred[2]. This process must increase the uncertainty associated with these original 'measurements'.

23. WMAwater's flood frequency analysis makes use of 30 flood peaks[3]. Leaving aside 1893, 1974 and 2011, the two next biggest floods occurred in the period before 1875 where there was no formal tide gauge at the Port Office. These floods were 1841 and 1844.

24. The Brisbane Flood Notice Board[4] which was operational in 1911 as a flood warning system used information from historical floods and contained a diagram which "*includes all the floods about which accurate information is available*". No floods before 1893 are shown.

25. WMAwater have omitted the flood of 1845[5] without explanation. This may be a typographical error or it may have been intentional if they considered the flood data unreliable. It also occurred in the period before formal gauges were installed.

26. WMAwater ranks the 1841 flood as the next biggest flood after 1893 (and bigger than 2011). In relation to the 1841 flood, the Bureau of Meteorology have reported "*its exact height is uncertain*"[6]

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1 The Maritime Safety Queensland website at http://www.msq.qld.gov.au/Tides/Sea-level-measurement-in-Queensland.aspx records that "*The Engineer-in-Chief of the Harbours and Rivers Department from 1875 to 1889, William Nisbet, established permanent tide gauges in the Brisbane River for river works and dredging projects*". Further "*by 1878, six gauges were installed between Brisbane and Lytton*".

2 In the time available to me I have not found any records of the original measurements themselves or the method used to infer the flood height peak at the Port Office gauge site (except for the reference to the 1841 flood quoted by WMAwater at their Paragraph 40).

3 There is a general lack of detail in WMAwater's report (which is no doubt due to the limited time they had available for its preparation). It is not always obvious what data sources they have used. WMAwater do not list all the floods they used. Only the largest floods are reproduced in their Table 7. Therefore it's difficult to be confident about exactly which floods they used. Nevertheless their Table 6 shows they used 30 floods (or 31 if 2011 was included). As Table 1 of BCC (June 1999) also used 30 floods, I have assumed WMAwater used the same ones as in BCC's flood frequency analysis.

4 http://www.bom.gov.au/hydro/flood/qld/fld_history/brisbane_notice_board.shtml This BoM article contains an extract from G.G. Bond, Divisional Officer, Brisbane, 24th January, 1911 which states "*This was a notice board for giving information to the general public of the rise and fall of flood waters at the various flood warning stations on the Brisbane River and tributaries has been erected under the clock tower of the General Post Office at the main Queen-street entrance by courtesy of the Deputy Postmaster-General*".

5 Note also that WMAwater has not included 1845 in their list of big floods (refer their Table 8). BCC (June 1999) gave it an unadjusted height of 6.5mAHD making it a 'Rank 6' flood.

6 Brisbane Floods, January 1974. BoM, 1974. http://www.bom.gov.au/hydro/flood/qld/fld_reports/brisbane_jan1974.pdf.

Uncertainty in Adjustments Made for Dredging and Other River Bed/Bank Changes

27. In relation to these adjustments, WMAwater have followed the approach used in BCC (June 1999), i.e.

    (a) between 1864 and 1917 subtract 1.52m (i.e. 5ft) from the measurements to account for dredging over this period; and
    (b) prior to 1864 subtract 1.92m for the above dredging and the removal of the entrance bar.

28. I have a number of difficulties associated with this approach:

    (a) changes in the river bed and banks between 1841 and 2011 have been numerous and significant[7]. There have been many more changes than the two 'step' changes in 1864 and 1917 assumed by WMAwater;
    (b) even when considering the river conditions during the 1841 floods, Henderson, in his address to Parliament in 1896 stated "*it should not be forgotten that the river conditions were then very different from what they are now*". Changes since 1896 to today would also likely have been significant;
    (c) further with the relocation of the port to Fisherman Islands in the mid to late 1970s there will have been a reduction in maintenance dredging upstream (i.e. from the new port to the City) and likely aggradation of the river bed since that time;
    (d) the height adjustments assumed by BCC and used by WMAwater are tenuous and based on very old information of doubtful accuracy (my opinion)[8]. The adjustments would also likely vary with flood magnitude. More rigorous hydraulic modelling using different river bed/bank conditions should be undertaken to identify the adjustment to be made to every recorded river height in order to provide a homogenous data set from which to prepare a more rigorous flood frequency analysis;[9]
    (e) the results of this analysis could produce discharge estimates somewhat different from that currently used. For example by reference to WMAwater's Figure 8, a 1.0m change in height would produce discharge changes of from 1,500 m^3/s to 2,500m^3/s over the range of interest (i.e. stage heights ranging from 8mAHD to 2mAHD). These are significant discharge changes which could noticeably alter the flood frequency results and the prediction of the 1% AEP discharge.

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[7] The following document provides a more complete description of the river changes particularly over the 20th Century (which is missing from the references quoted by WMAwater):
http://www.marine.uq.edu.au/marbot/publications/CRC%20coastal%202003_HC%20report/Ch%209%20Appendices.pdf

[8] I note that WMAwater at their Paragraph 100 says that they tested dredging in SKM's MIKE11 model and found that the 1.52m adjustment was 'probably appropriate'. No details however are provided so it is impossible to test this. In particular it is not known what depth and width of dredging was assumed and over what sections of the river.

[9] This is supposedly what WMAwater have recommended at the end of their Paragraph 2. It is noted that river bed cross sections are available from a survey undertaken in 1873. There are also numerous and detailed records of the dredging works undertaken over more than a century. A comprehensive assessment of these records needs to be undertaken (together with review of other river bed surveys) and revised hydraulic models describing the river at various time periods. A more complete series of height adjustments could then be prepared to allow the historical records at the Port Office gauge to be related in 2011 river conditions. This would also need to allow for tide adjustments.

Uncertainty in Discharge Estimates due to Tidal Influences

29. Figure 2 of Cossins (1978) plots the recorded stage hydrograph at Port Office during the 1974 flood. This figure has been reproduced below and demonstrates how tides can influence the Port Office flood heights and how these effects are dampened as the flood height rises.

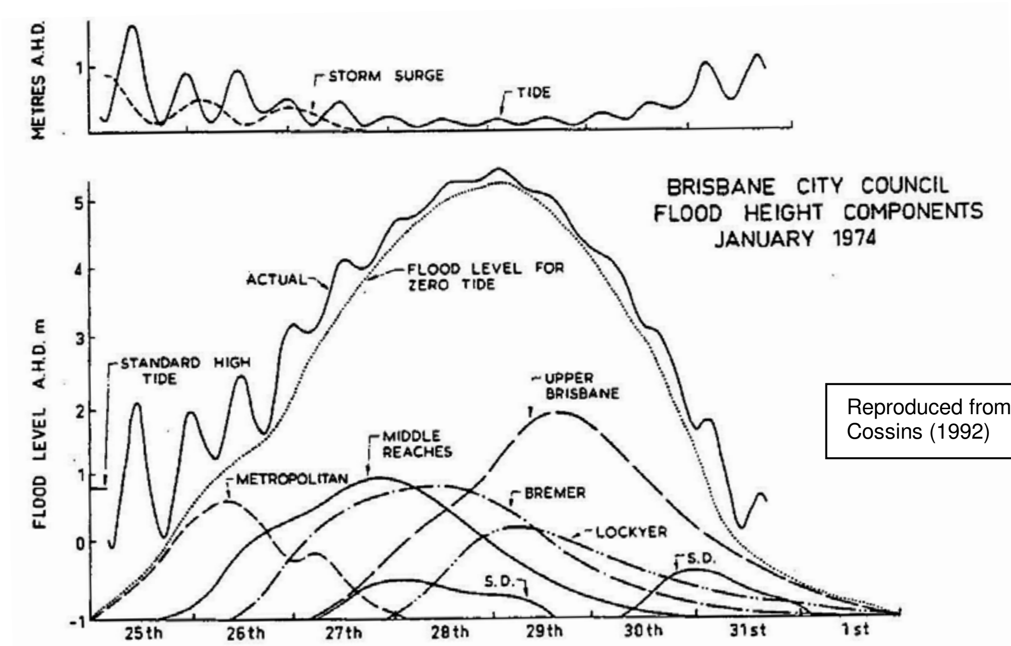

Fig. 2: Flood height components for January 1974 at Brisbane City Gauge in metres above Australian Height Datum.

Reproduced from Cossins (1992)

30. For example, when the flood height is say 1.5mAHD, at least ±0.5m or more could be due to normal tidal activity. Bigger ranges would likely occur if storm surges were present (as these often accompany floods) or spring tides. This tidal influence diminishes as the flood height rises but even at a flood level of 4mAHD, some tidal influence can be exerted.

31. The flood series used by WMAwater disregarded flows lower than 2,000m^3/s.[10] From the 'derived rating' curve in their Figure 8, this flow corresponds to a level of 1.4mAHD. Very considerable tidal influences would likely be present at this level.

32. It is likely that about two thirds of the 30 floods used in their flood frequency analysis had peak heights less than 3mAHD at the Port Office gauge. The calculation of these flows based on the derived rating will therefore be sensitive or very sensitive to the influence of tides.

33. A rigorous analysis would see each of these events analysed to remove the effect of tides.[11] (This approach has been suggested in previous studies).

[10] WMAwater's Table 10

Uncertainty in Discharge Estimates due to Rating Curve

34. This section addresses uncertainties in the conversion of flood height to discharge other than those due to measurement accuracy, dredging and tides which have been discussed above.

35. The adjustments for river bed/bank changes discussed above were made to ensure the rating curve remained stationary over the period of the flood frequency analysis. The rating curve shown in WMAwater's Figure 8 demonstrates how flood height and discharge are related at the Port Office under current conditions.

36. There are six data items included in WMAwater's Figure 8 which are discussed below. (The first two are listed under Item a):

(a) *SKM (June 1998)* and *SKM (June 1998)* – these appear to be based on recorded levels from historical floods with flows estimated from the MIKE11 models used in those studies - but I can't be sure;

(b) *SKM (2011)* – appears to be obtained directly from the MIKE11 model noting that this model was calibrated to the 2011 flood records;

(c) *1893 Event* – the orange box on Figure 8 is based on flood heights ranging from 8.35mAHD to 6.83mAHD.[12] The discharge range is 11,300m^3/s to 16,900m^3/s.[13] As discussed in the footnotes, the most likely estimate of the 1893 event is 6.83mAHD and 11,600m^3/s – so the orange box should be centred on this point (which approximates the lower left hand corner of the existing box).

(d) *1974 Event* – 5.45mAHD was the recorded height for this event. WMAwater states that previously the discharge was thought to be 9,800m^3/s but this has been revised upwards to 10,900m^3/s based on information learnt during the 2011 event.

(e) *2011 Event* – due to time limitations I have not had time to check these values.

37. By a review of the comments made in the previous paragraph, it would appear that the 2011 event has resulted in a shift of the upper section of the rating curve to the right, i.e. in the absence of the 2011 information, and based only on information from the floods of 1893 and 1974, and the previous SKM ratings, the upper part of the rating curve would have been further to the left. This 'shift' could be real (i.e. due to a better understanding of high flow behaviour obtained from the 2011 event and the better models), or, it could be due to other factors e.g. changes in the

[11] Note that whilst the bigger floods will be influenced little by tides, the impacts on the smaller floods can still influence the shape of the ultimate flood frequency curve and the prediction of the 1% AEP discharge, and the confidence limits surrounding it.

[12] 8.35 is the recorded flood height. 6.83=8.35-1.52 (i.e. includes the adjustment for dredging). There is an inconsistency here. The centre of the box has a flood height of 7.59mAHD. This is not the height of 6.83mAHD which WMAwater has used in their Table 8.

[13] These discharges are derived from Table 10.2 of Appendix A10 of BCC (June 1999) – and are also discussed from Page 7 onwards of BCC (Dec 1999). The analysis in this later document suggests the best estimate of the 1893 flood is 11,600m^3/s which is close to the lower bound of the discharge range in the orange box. Again an inconsistency exists.

bed/banks, channel roughness, etc that haven't been adequately explained/understood to date. It may also be a combination of both these causes.

38. Whatever the reason, to use the 'derived rating' on Figure 8 to estimate all the historical discharges (i.e. for use in the flood frequency analysis) does not seem appropriate and must introduce further uncertainties into the analysis which is subsequently presented by WMAwater in their Section 7.[14]

39. A likely consequence will be that the discharges used in the flood frequency analysis are over estimated, including the discharge estimate of the resultant 1% AEP event.

[14] This might best be seen by examining the discharges listed in WMAwater's Table 7. These discharges have been obtained from the 'derived rating' on Figure 8. For example the 1893 discharge is higher than 11,600m^3/s, and the 1974 discharge is higher than both 9,800m^3/s and 10,900m^3/s [refer Paragraph 36(c) and 36(d) above].

Bewsher

Drew BEWSHER

QUALIFICATIONS

Bachelor of Engineering (Hons), University of Tasmania, 1975.
Master of Science in Civil Engineering, California Institute of Technology USA, 1977

AFFILIATIONS:

Fellow, Institution of Engineers, Australia.
Past Chairman, Sydney Water Engineering Panel and Western Sydney Water Engineering Panel, Institution of Engineers, Australia.
Member, College of Civil Engineers, Institution of Engineers, Australia.
Chartered Professional Engineer, NPER-3 Registration

FIELDS OF SPECIAL COMPETENCE

Thirty years experience in water related projects in Australia, America and South East Asia. This work has included floodplain management studies, river hydraulics and flood studies, computer modelling, hydrological studies, irrigation and salinity modelling, urban drainage investigation and design, water quality investigations, dam break studies, environmental planning and environmental impact assessment, construction supervision and the project management and economic evaluation of water resources projects. He has also provided expert testimony in legal proceedings related to flooding and drainage matters in the Supreme Court, District Court, and Land and Environment Court, and other tribunals in NSW and elsewhere.

EXPERIENCE

1986 to date | Bewsher Consulting Pty Ltd, NSW, Australia, Director

Principal responsible for a number of projects including:

Floodplain Risk Management Studies and Plans for approximately 30 NSW councils including those for the Georges River, Mullet/Brooks Creek, Fairy/Cabbage Creek, Bowral, Coffs Creek, Camden Haven, Upper Parramatta River, Grafton, Lower Clarence River, Tweed River, Gwawley Bay, Prospect Creek, Billabong Creek, Berrima, Towradgi Creek, Haslams Creek, Mudgee, North Wentworthville, Carlingford, Brickfield Creek, Cabramatta Creek, Boundary Creek, Eastern Creek, Narrabri, Scone, Molong and the Paterson River.

Independent technical audits and expert advice associated with hydrologic modelling and water resources issues in Australia. This has included expert technical support to the Snowy Water Inquiry and numerous Government projects relating to water efficiency savings throughout the Murray-Darling Basin including Menindee Lakes, technical auditor of the Murray-Darling Basin Commission 'Cap' models involving over 20 valley models developed by four states and the ACT, expert advice on various projects related to the Murray-Darling Basin Salinity Management Strategy, expert review of IQQM models and environmental flow objectives for the NSW government, expert independent assessment of hydrological components of major infrastructure projects and EISs, together with numerous reviews carried out for the private sector.

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Flood risk assessment and computer modelling of flood behaviour for over 500 projects in urban and rural areas of Australia. This has included assessment of risks to life and property, simulation of flow behaviour using one and two dimensional models, stormwater and urban drainage assessments, and consideration of a range of related environmental, riparian corridor and water quality issues.

Expert testimony in excess of 30 court proceedings relating to development applications, valuation of floodprone land, personal injury claims and other issues relating to flooding, hydrology and stormwater drainage issues.

Policy formulation for floodplain development. This has included the preparation of over 20 Development Control Plans for local councils in NSW to ensure new developments meet best practice standards for floodplain management. The scope of these policies has also addressed floodprone caravan parks, on-site stormwater detention and a range of broader stormwater management issues.

Design and management of flooding and drainage infrastructure projects. These projects comprise detention basins, major trunk stormwater systems, creek rehabilitation, and the civil works associated with numerous floodplain and stormwater projects.

1980 to 1986 | Sinclair Knight & Partners Pty Ltd, Australia

Specialist Water Engineer working on numerous development, government aid, mining and World Bank projects in NSW, Malaysia, and the islands of Sumatra and Java in Indonesia.

1978 to1980 | River Murray Commission, Australia

Investigation Engineer responsible for modelling of water resources of the Murray-Darling Basin, assisting the Executive Engineer with river operations and various investigations into the water resources of basin, and the preparation of water accounting procedures**.**

1977 | Camp, Dresser & McKee, USA

Engineering investigations of flood behaviour and river hydraulics in the Los Angeles Basin.

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Prepared for
Clayton Utz

Subject
Provision of expert advice in relation to a report provided by WMAwater

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Author's Declaration

This report has been prepared in accordance with UniQuest's Quality Management System, which is compliant with AS/NZS ISO 9000:2000.

The work and opinions expressed in this report are those of the Author.

Signed by Emeritus Professor Colin Apelt

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TABLE OF CONTENTS

1. INTRODUCTION

Clayton Utz, advising Brisbane City Council with reference to the Queensland Floods Commission of Enquiry (The Commission), asked UniQuest Pty Ltd to engage Professor Apelt to provide expert advice in relation to a report prepared by WMAwater for The Commission with the title, „*BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS FINAL REPORT SEPTEMBER 2011'* .

As part of this advice, Clayton Utz requested that Professor Apelt prepare a list of the enquiries and investigations that, in his view, ought to be completed prior to any determination of the Q100 flood line following the January 2011 event.

This report consists of three main parts:

- The material of the first part, Section 2, provides the response by Professor Apelt to the request for a list of the enquiries and investigations that, in his view, ought to be completed prior to any determination of the Q100 flood line following the January 2011 event.
- The second part, Section 3, provides a summary of the methodology used in the WMAwater report for arriving at the estimate of 1% AEP flood flow and levels together with discussion of this methodology.
- In the third part, Section 4, some particular parts of the WMAwater report are discussed in more detail.

2. REVIEW RESPONSE - PART 1; DETERMINATION OF THE Q100 FLOOD LINE

The following material has been prepared in response to the request by Clayton Utz for Professor Apelt to prepare a list of the enquiries and investigations that, in his view, ought to be completed prior to any determination of the Q100 flood line following the January 2011 event. This work has been carried out in a limited time frame and without the opportunity for discussion with colleagues. Consequently, it is not guaranteed to be a final or complete list and it must not be taken as the detailed specification of work to be done.

2.1 Data required

All the data from the January 2011 flood event for the whole of the Brisbane River system catchment must be gathered, checked and assessed for accuracy: – rainfall; all flood levels, not just peak levels; all flood flows, not just peak flows; Moreton Bay tidal data; catchment conditions prior to the event. This will require all sources to share freely the data they have so that an archive can be created of all data including information about accuracy and plausible bounds. This archive must be accessible to all, without restriction.

An accurate digital terrain model must be generated for all areas likely to be flooded in extreme flood events. This will require

(i) Topographic survey of areas above dry weather water level
(ii) Bathymetric (stream bed and banks below water level) survey for the whole of the Brisbane River from the “mouth”, including the parts of Moreton Bay that can influence river heights during extreme flood events, up to Wivenhoe Dam and for the major tributaries.

Sediment characteristics must be measured for the same extent of the system for which the bathymetric survey data is required.

2.2 Analyses required - Generation of the best possible homogeneous data set for Flood Frequency Analysis (FFA) for pre-dams conditions

The generation of the best possible homogeneous data set for Flood Frequency Analysis (FFA) for pre-dams conditions requires the assessment of:

(i) how changes to the Brisbane River, especially those downstream from the Port Office Gauge, have affected flood levels;
(ii) the peak flow of each flood.

- A much more detailed analysis will be required than those that have been done to date.
- It will be necessary to develop appropriate hydrodynamic models of the Brisbane River to simulate the conditions that existed at the times of historic floods and to use them in conjunction with appropriate hydrologic models to produce estimates of flood flows consistent with the recorded / reported rainfalls and flood levels for each event.
- The models should then be used to produce estimates of the flood levels for the River system in its present state but excluding the Wivenhoe and Somerset Dams.
- Because the data available is limited in detail and accuracy it will be necessary to generate „best estimates' and „plausible bounds' for each historic flood event.
- The estimates of flood flows produced by this process can then be used for FFA.
- The FFA should be done with the „best estimates' and repeated using the two sets of „plausible upper bounds' and „plausible lower bounds'.
- The estimates of flood levels for the River system in its present state should be used to produce or extend rating curves for key locations.
- The rating curves should show the curve corresponding to the „best estimate' and also the „plausible upper bound' and „plausible lower bound'.

2.3 Analyses required - Generation of estimates for post-dams conditions

The effects of morphological (river bed level and cross section) changes due to sediment erosion and deposition during flood events must be studied for a range of flood magnitudes to determine what effects they can have on flood levels.

The model for simulating the expected operation and effects of Wivenhoe and Somerset Dams on flood flows, and associated data, should be independently peer reviewed.

The design hydrology must be determined for a range of Annual Exceedance Probabilities (AEPs), not just for 1% AEP. A Monte Carlo or similar type analysis will be required.

This analysis must take into account the observed variability:
(i) in temporal and spatial patterns of rainfall and the associated variability in relative timings of inflows from the dams and downstream tributaries;
(ii) in the correlations between event occurrences;
(iii) in losses; and
(iv) in reservoir drawdown.

An appropriate hydrodynamic model must be used to estimate the flood levels along the Brisbane River for the flood events that have been determined for the range of AEPs. When this is being done the effects of tidal variation on flood levels in the estuarine zone must be taken into account. This will require a Monte Carlo type analysis to examine the joint probabilities of flow rates and sea levels in Moreton Bay caused by tidal action and storm surge.

2.4 Comment

Section 2.3 calls for analysis of a range of AEPs, not just of the1% AEP. It is acknowledged that the full set is not required for „determination of the Q100 flood line'.

Nevertheless it is essential for a complete Flood Risk Management analysis for the area of Brisbane and for the whole of the Brisbane River system affected by flooding from the Brisbane River and its tributaries to be carried out.

It is essential to move from the "Q100 mentality" and to adopt a risk management approach in line with National Flood Risk Advisory Group (NFRAG) and other relevant guidelines. The risk management approach will require a detailed assessment of the benefits and costs of a full range of flood mitigation options for the full range of Annual Exceedance Probability flood events.

It will be most efficient for the full set of analyses to be done at the same time, rather than in a „piecemeal' approach.

3. REVIEW RESPONSE - PART 2; METHODOLOGY USED IN WMAwater REPORT FOR ARRIVING AT THE ESTIMATE OF 1% AEP FLOOD FLOW AND LEVELS

This part of the report provides a summary of the methodology used in the WMAwater report *„BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS FINAL REPORT SEPTEMBER 2011'* for arriving at the estimate of 1% AEP flood flow and levels together with discussion of this methodology.

Comments by Professor Apelt on some aspects of the report are presented below. Where References are cited the number of the Reference is that used in the WMAwater report named above. Unless stated otherwise, Figure numbers, etc are those from that report. Wherever anything in what follows is attributed to "the author", this refers to Professor Apelt.

3.1 Summary of methodology used in report for arriving at the estimate of 1% AEP flood flow and levels

- A rating curve is developed at the Port Office/City Gauge (see Note a).
- A Flood Frequency Analysis is done for the Port Office/City Gauge for the No Dams case to give an estimate of 13000m^3/s for the 1% AEP flood at the Port Office/City Gauge for the No Dams case (see Note b).
- A plot of peak flows at the Port Office/City Gauge versus peak flows for the No Dams case is constructed, Figure 3 (see Note c).
- The „relationship' in this plot is used directly to derive the estimate of the 1% AEP flood at the Port Office/City Gauge, 9500 m^3/s, as a ratio of the 1% AEP flood at the Port Office/City Gauge for the No Dams case (see Note d).
- A flood profile is calculated, starting from a level of 4.32 m AHD at Port Office/City Gauge to produce a profile of 1% AEP flood levels. The profile is produced after adjusting the MIKE 11 model results to match the observed data when it was used to reproduce the 2011 data (see Note e).

3.2 Notes and comments on the methodology for arriving at the estimate of 1% AEP flood and levels

a. This involves estimates of what the levels of historic floods would have been in the river in its present state, allowing for effects from lowering of the bar and of dredging along the river.
   i. The effect of bar lowering is estimated at 0.4 m and this is applied to ***all*** floods.

ii. The effect of river dredging is estimated at 1.52 m and this is applied to ***all*** floods.
iii. For cases where both „corrections are to be applied they are simply added for ***all*** floods.

Briefing material provided to the Independent Review Panel (by BCC officers from City Design) during its work in 2003 included the assessment of evidence on effects of the dredging of the river; "This suggests that small floods reduced by 1.52 m but little effect on large floods". It is noted here that the description „river dredging' does not encompass all of the changes – river training and reclamation of some tidal flats were involved also.

These estimates are taken directly from Reference 17 except for differences in two cases (detailed below in section 4.1). They are gross approximations. Further, their magnitudes are open to question; the briefing of the Review Panel included the advice that Bureau of Meteorology (BoM) provided a chart relating Moggill Alert to Brisbane City Alert "to show that the adjustment of pre-dredging levels should only be by 0.6 m rather than 1.8 m assumed in the December 1999 review" with the comment "This raises the estimate of the 1893 flood and all nineteenth century floods above those assumed in December 1999." The author quotes this comment, not to endorse the inferences about the magnitude of the nineteenth century floods, but solely to emphasise the point that, to his knowledge, there has never been a clear consensus about these matters.

The Independent Review Panel was briefed in 2003 on the uncertainties associated with any attempt to establish a rating curve for the Port Office site. It was obvious that it would not be possible to investigate this matter with sufficient thoroughness to establish a best estimate of a rating curve and its error bounds for the Port Office in the time available - approximately five weeks. In fact, there was insufficient time even to assess whether a rating curve **could** be developed that would be sufficiently reliable for the purpose of flood frequency analysis (FFA). In these circumstances, the Panel accepted the decision by Sinclair Knight Merz (SKM) to use the combined record from the gauges at Savages Crossing, Lowood and Vernor, referred to as "Savages Crossing" as the key site for FFA (Reference 20, para 4.2).

In further discussion of this matter in Section 4.1 the author's comments on paragraphs 148 and 149 of the WMAwater report warn that substantial margins of uncertainty about the rating curve at the Port Office may persist even after the extensive investigations have been completed, such that it may remain insufficiently reliable for use as the key site for FFA.

There have been several attempts to produce a rating curve for the Port Office/City Gauge for the No Dams case. There is no real consensus. As just one example, in an email to Ken Morris of Brisbane City Council dated 12 August 2003, Peter Baddiley of BoM provided a rating curve that is linear.

The writer acknowledges that persons involved in discussions about these matters in 2003 may have „moved on' from the views they held then. Nevertheless, the fact remains that most of the material used to produce the rating curve at the Port Office used in the WMAwater report (Figure 8), continues to have substantial uncertainties.

b. The results of the FFA are not greatly different from those from previous studies.
   i. The SKM study of 2003 estimated Q100 pre-dams at the Port Office/City Gauge to be in the range from 11000 to 13000 m^3/s and adopted 12000 m^3/s.
   ii. WMA gives results in Table 10 produced with GEV and LP3. As the number of larger floods omitted from the FFA increases, the results from GEV give progressively smaller estimates for Q100 (as would seem reasonable) whereas three of the estimates of Q100 from LP3 are virtually the same and the one for the case when five of the largest seven floods are omitted is the largest of all at 16610 m^3/s. This is counter-intuitive. This suggests that the GEV estimates should be preferred, not averaged with LP3 estimates. If the estimate for Q100 is based on GEV alone it becomes 12130 m^3/s.
   iii. The similarity in the estimates of Q100 produced by all of these FFAs gives little ground for comfort because they all suffer from the fact that most of the data are estimates of flows of uncertain accuracy, especially in the cases of most of the large floods.

c. It is not clear how Figure 3 was created. The plotted points are somewhat scattered. The data point identified as „SKM 2011' presumably was produced with the faulty Version 1 of MIKE11 since the only references for 2011 listed for SKM are No 35 (24 June 2011) and No 36 (11 March 2011), whereas the review that led to the production of the improved Version 2 of MIKE11 was carried out between 27 June and 5 July 2011 as described in the WMAwater report *'Review of Hydraulic Modelling Final Report 28 July 2011'*.

   The author is not convinced by the approach adopted to produce the data points in Figures 2 and 3 for the January 2011 flood event. In paragraph 64 of the report it is stated that "the 2011 event has a peak inflow of 11000 m^3/s (WMA) and 11150 m^3/s (Seqwater) and a peak discharge of 7500 m^3/s (giving a reduction of 32-35%)." This completely overlooks the complexity of the event in that there were two peak inflows approximately 29 hours apart.

- The first peak inflow was **10095** m^3/s at 08:00 on 10 January.
  At that time the outflow was 1944 m^3/s; it increased slowly to 2087 m^3/s at 15:00; then more rapidly to 2695 m^3/s at 20:00; then slowly to 2753 m^3/s at 08:00 on 11 January; it then increased rapidly to 7464 m^3/s as the second peak inflow arrived.
- The minimum inflow between the peaks was 3594 m^3/s at 02:00 on 11 January
  At that time the outflow was **2721** m^3/s, the dam level was 73.35 m AHD and the storage was 1,977,862 ML or 169.8% of FSV. The storage at FSL of 67.00 m AHD is 1,165,000 ML.
- From all of this one could argue that the attenuation of the first peak was 73.1%, corresponding to the peak outflow being 26.9% of peak inflow i.e. **2721/10095.** Even if one uses the initial objective of limiting outflow to 4000 m^3/s the attenuation would have been 60%, corresponding to the peak outflow being 40% of peak inflow.
- The second peak inflow was **11561** m^3/s at 13:00 on 11 January. The peak outflow rose to **7464** m^3/s by 19:00 on the 11th. This gives an attenuation of 35.4%, corresponding to peak outflow being 64.6% of peak inflow. (WMA has 32-35%). But the dam was at 170% of FSV at the start of this second peak inflow.
  The author does not consider that it is appropriate to use this figure of approximately 35% in isolation, ignoring the complexity of the event, as somehow representative of the attenuating effects of the dam. It is particularly of concern that no account appears to have been taken of the fact that a large proportion of the flood storage compartment was already used up when the second inflow peak began to arrive.

  Note: the figures used in the above discussion are taken from the Seqwater Report (Reference 26).

d. The 1% AEP peak flow at the Port Office/City Gauge is estimated by using directly the result calculated by SKM 2011 (see comment above in Note c). This gives for the Port Office/City Gauge a post-dams peak flow of 9500 m^3/s versus a pre-dams peak of 13000 m^3/s. So 9500 m^3/s is adopted for the 1% AEP peak flow at the Port Office/City Gauge. The author has severe reservations about this „conclusion'. The WMAwater report justifies this choice in paragraph 132 with "Using Figure 3 without applying any weight to the 2011 event a value of 9000 m^3/s is obtained as the post dam (Wivenhoe and Somerset dams) flow. The 2011 data provides the only real data point on the performance of the dam and suggests a post dam flow of 10000 m^3/s using WMAwater's estimate and 9500 m^3/s using SKM's estimate. On the basis of these three datasets a post dam flow of 9500 m^3/s was adopted."

If, for example one were to adopt 12130 m^3/s as the pre-dams 1% AEP peak flow at the Port Office/City Gauge and use the "estimation line" in Figure 3, the estimate of the post-dams 1% AEP peak flow becomes approximately 8300 m^3/s.

But, the author has a fundamental difficulty with the application of one specific result for a particular event (that may or may not be „typical') to convert a value obtained by statistical analysis to give another value and to treat that derived value as having statistical significance. There seems to some problem in the logic of that process. Further, it ignores the fact that the event from which the conversion multiplier is taken was unusual in that the inflow to Wivenhoe was double peaked. This is discussed further below. It also makes no allowance for the different combinations of flows from Wivenhoe and from the tributaries downstream from the dam.

The author considers that it is not appropriate to use one specific flood event as the basis for the estimate of the attenuation achieved by the dams that is to be incorporated in the estimate of the 1% AEP post-dams peak flow. The 1% AEP post-dams peak flow is a statistical, design concept and all the variable elements that contribute to its estimation, including the attenuation of the flood peak by the dams, must be estimated by statistical processes. In brief outline, these processes should include hydrological modelling of the whole Brisbane River catchment for a number of synthetically generated 1% AEP design storm events for the pre-dams case to produce inflow hydrographs at the site of Wivenhoe Dam, in addition to the pre-dam flood hydrographs at the Port Office. The inflow hydrographs at the dam site should be run through a dam operations simulation model to generate outflow hydrographs from Wivenhoe Dam for the post-dams case. These outflow hydrographs then provide the post-dams input from Wivenhoe Dam to the hydrological model of the Brisbane River catchment downstream from the dam to produce post-dams flood hydrographs at the Port Office. The end result of this modelling will be a set of estimates of pairs of 1% AEP design flood peaks for pre-dams and post-dams conditions at several locations along the Brisbane River, including at the Port Office. The best estimate of the pre-dams and post-dams 1% AEP design flood peaks will be determined from this set of „pairs'. (It is stressed that this brief outline must not be taken as a complete specification of the work required to produce the 1% AEP design flood estimate; it is given as an indicative outline only.)

e. It is not clear how the level of 4.32 m AHD was arrived at. No detail is given on how the MIKE 11 model results were “adjusted”. The resulting profile in Figure 13 looks less plausible than the „non-conforming’ one produced by MIKE 11 without adjustment (see Figure 12). The profile in Figure 13 has been made to conform to figures for peak flood levels given in the Joint Flood Taskforce Report (Reference 27). However, it was stated clearly in that report that the levels were “subject to final verification”. When the JFTF report was being produced, the author and another member of the Taskforce shared concerns they had about the accuracy of the flood levels given in Reference 27 for the stretch of the river from the West End Ferry to Seventeen Mile Rocks, because they seemed inconsistent with the rest of the flood profile. At the time, this was the only information available to the Taskforce and it was not feasible to have it checked during the three weeks between the first meeting of the Taskforce and the deadline for completion of the Report (8th March 2011). Floodwise Property Reports for the locations of these flood levels, downloaded in September 2011 from the BCC website, indicate that the current best estimates of many of these levels are lower than those given in Reference 27.

4. REVIEW RESPONSE - PART 3; EXTRACTS FROM WMAwater REPORT WITH COMMENTS ADDED

The numbering of paragraphs in the following is that used in the cited report. The text of each numbered paragraph is that in the Report. The Comments added are by Professor Apelt, referred to as "the author".

4.1 Matters of substance

91 Depending on the size of a particular event, rating curves can be sensitive to overbank conditions and topography. Often a change in slope is seen in the rating curve as flow enters the overbank. Future work utilising 2D hydrodynamic modelling to develop rating curves specific to a point in time needs to ensure that overbank topography is representative of the time of the event in question.

Comment This comment suggests that the writer of the WMAwater report believes that more investigation is needed to produce the best possible estimate of the rating curve at the Port Office.

92 If a 2D model is developed it should use high resolution survey data (LiDAR) for current conditions. To model earlier events it would be necessary to draw upon a range of data sets including aerial photography, ortho-photo maps, 1873 and 1974 survey details.

Comment As for paragraph 91.

99 The recorded stage for the 1893 flood event was 8.35 m AHD, however it is necessary to adjust this height for current conditions. Reference 17 adjusted all the recorded stages from 1864 to 1917 by 1.52m (5 feet) except for the 1893 event and assumed the discharge of this event was 14 600 m^3/s. *Table 10.2 of Reference 17* presents 5 estimates of flow ranging from 11 300 to 16 990 m^3/s for the 1893 event. Two of these estimates are based on velocity measurements taken during the event at Indooroopilly and Victoria Bridges (16 990 m^3/s and 14 600 m^3/s respectively). Reference 3 (Part 3, Section 2) details problems associated with reverse flow on the inside bend making measuring flow at Indooroopilly Bridge difficult during the 1930's and 1950's. Reference 18 modelled the 1893 event using cross sections from 1873 and estimated the peak flow at 11 600 m^3/s. (Italics added)

Comment There is no Table 10.2 in Reference 17, or in Reference 18.

119 For floods prior to 1917 the 1.52m dredging adjustment was used and for those prior to 1864 the 0.4m bar adjustment was also used. This is the same approach as used in Reference 17 other than the dredging adjustment has been applied to all events (including 1841 and 1893). For smaller events the flow adjustment used in Reference 17 was also used. For larger events the high flow rating derived as part of the current study was used (refer to Section 6.3.2).

Comment The blanket reductions applied to all flows are gross approximations. It will be necessary to develop appropriate hydrodynamic models of the Brisbane River to simulate the conditions that existed at the times of historic floods and to use them in conjunction with appropriate hydrologic models to produce estimates of flood flows consistent with the recorded/ reported rainfalls and flood levels for each event. The models should then be used to produce estimates of the flood levels for the River system in its present state but excluding the Wivenhoe and Somerset Dams. Because the data available is limited in detail and accuracy it will be necessary to generate „best estimates' and „plausible bounds' for each historic flood event. The estimates of flood flows produced by this process can then be used for FFA.

120 Adjustments were made to the 1974 event to account for Somerset dam and to the 2011 event to account for both dams. Every attempt was made to make adjustments in a consistent and non contradictory manner. The adopted high flow estimates are presented in Table 7 below. It is noteworthy that the 1841, the second 1893 event and the 2011 event are essentially the same size.

Table 7: Homogeneous Data Set of Flood Levels for the Brisbane River

Event	**Recorded Level (i.e. As measured during the event) (mAHD)**	**Adjusted Level (mAHD)***	***Adjusted Level (mAHD) Ref 17***	**Pre Dam Current Conditions Height (mAHD)**	**Pre Dam Current Conditions Flow (m3/s)**	***Pre Dam Current Conditions Flow (m3/s) Ref 17***
1893(a)	8.35(1)	6.83(1)	***8.35(1)***	6.83	13700	***14600***
1893(b)	8.09	6.57		6.57	12600	
1841	8.43(2)	6.51(2)	***8.03(2)***	6.51	12500	***14100***
2011	4.27	4.27		6.40	12400	
1974	5.45	5.45	***5.45***	5.50	11300	***10364***
1844	7.03	5.11	***5.11***	5.11	10400	***8924***
1890	5.33	3.81	***3.81***	3.81	8100	***6972***
1898	5.02	3.50	***3.45***	3.50	7500	***8500***

* Includes 1.52 m prior to 1917 and an additional 0.4 m adjustment for prior to 1864

Comment The two columns with entries in ***bold italics*** have been added to the Table 7 from the WMAwater Report by the author to include the corresponding data from Reference 17 for comparison. The differences between levels for 1893(a) and 1841 are explained in notes (1) and (2) below. Insufficient discussion is provided in the Report to support preference for the flow rates adopted over those from Reference 17.

Note (1) No adjustment was applied in Reference 17. The lower level adopted in the WMAwater Report results from a lowering by 1.52 m to account for dredging.

Note (2) Reference 17 lowered the level by 0.4 m to account for bar excavation but no adjustment was made for dredging. The further lowering is to account for dredging.

The differences between the entries in the original Table 7 of the WMAwater report and those in the extra two columns added by the author provide detailed illustration of the uncertainties associated with the rating curve at the Port Office, that has been discussed above in Section 3.2, Note a.

127 Results from the 4 flood frequency analyses undertaken at the Port Office gauge are shown in Table 10 (for both GEV and LP3 distributions).

Table 10: Comparison of Q100 Estimates for Considered Approaches

Data set/ Case	***Comments***	**Q100 (m^3/s)**		
		GEV	**LP3**	***Mean***
1841-2011[a]	***Includes all large floods***	12 130	13 730	***12 930***
1841-2010[a]	***Omits one large flood (3rd)***	11 740	13 900	***12 820***
1908-2011[b]	***Omits five of seven largest floods***	10 740	16 610	***13675***
1908-2010[b]	***Omits six of seven largest floods***	9 510	13 900	***11705***

[a] 141 censored flows lower than 2,000 m^3/s

[b] 90 censored flows lower than 2,000 m^3/s

Comment The two columns with entries in ***bold italics*** have been added by the author to the Table 10 from the Report.

128 Conducting the flood frequency analysis without the 2011 event changes the average of the GEV and LP3 estimates by only 95 m^3/s.

Comment The GEV estimate is reduced as more and more of the larger floods are omitted (as would seem reasonable). However, the LP3 gives the largest estimate for the case when five of the seven larger floods are omitted. This is counter-intuitive and it casts doubt on the LP3 estimates. This suggests that the GEV estimates should be preferred, and not averaged with LP3 estimates. If the estimate for Q100 is based on GEV alone it becomes 12130 m^3/s. However, the relatively small differences between estimates for Q100 are not really important here because the estimates of the individual flood flows have such uncertain accuracy, especially in the case of most of the larger floods. The similarity in the estimates of Q100 produced by all of these FFAs gives little ground for comfort because they all are based on data of such uncertain accuracy.

141 The Mike 11 Model (Version 2) developed by SKM for Seqwater as described in Reference 38 (and Version 1 described in Reference 35) was calibrated by SKM to Moggill, Jindalee and the Port Office. It was intended to use this model to fit a flood surface between Moggill and the Port Office for a peak post dam design flow of 9500 m^3/s. *When this model was compared to observed flood height data in the 2011 Joint Taskforce report (Reference 27, Table 3) problems were found with the fit (Figure 12)*. While the model fitted well at Moggill, Jindalee and the Port Office the fit was slightly low between Moggill and Jindalee and up to 1.8m low between Jindalee and the Port Office. This problem demonstrates the need for organisations to consider all agencies data when calibrating models. The Mike 11 model was therefore unsuitable to be used in profile generation. (Italics added)

Comment A note at Table 3 of Reference 27 states explicitly that the Jan 2011 levels given are "subject to final verification". The resulting profile in Figure 13 looks less plausible than the „non-conforming' one produced by MIKE 11 without adjustment (see Figure 12). The profile in Figure 13 has been made to conform to figures for peak flood levels given in the Joint Flood Taskforce Report (Ref 27). Floodwise Property Reports for the locations of these flood levels, downloaded in September 2011 from the BCC website, indicate that the current best estimates of many of these levels are lower than those given in Reference 27.

142 As part of the prescribed work scope The Commission required profile information on peak flood levels between Moggill and the Brisbane River mouth for the 2011 event and 1% AEP. January 2011 levels at each location were estimated *by adjusting the Mike 11 model results to match the observed data from 2011 Joint Taskforce (Reference 27).* From this, approximate flood levels at the points of interest identified by The Commission were determined. The same process was adopted for the 1% AEP flood levels using a peak post dam flow of 9500 m^3/s. These profiles are presented on Figure 13 and summarised in Table 13. (Italics added)

Comment No information is given about the nature of these adjustments.

148 A detailed study needs to be undertaken *to improve the rating relationship at the Port Office gauge.* This study needs to draw upon all the information held by Council and State Government. The rating information held by different organisations also needs to be consolidated and objectively reviewed. (Italics added)

149 The study needs to contain the following components:

- Development of a suitable industry standard 2D hydrodynamic model of the lower reaches of the Brisbane River. This model needs to be suitable for assessing historical changes to the river bathymetry and needs to have a run time that is practical for detailed calibration and assessment of changes,
- A detailed search of all data sources on the bathymetry of Brisbane River needs to be undertaken. This study needs to produce best estimate maps of the bathymetry at different times during Brisbane's development. A current survey of the bathymetry also

needs to be undertaken and the current morphological behaviour of the river needs to be understood,

- Astronomical tide need to be calculated for the flood events that occurred prior to the regular recording of tides,
- Where sufficient tidal and meteorological information is available the storm surge component at the river mouth needs to be estimated for each historical event,
- The methodology that has been developed under Research Project 18 of Australian Rainfall and Runoff for the calculation of the joint probability of river flooding and elevated ocean levels, should be applied to the lower reaches of Brisbane River so that flood risk can be properly quantified, and
- The sensitivity of flood levels to elevated ocean levels from climate change needs to determined.

Following the completion of the above tasks a revised flood frequency analysis should be carried out using the current best practice. This analysis should explore the use of a regional flood frequency approach.

Comment The statements and recommendations in paragraphs 148 and 149 are agreed with fully. Further, these paragraphs constitute implicit acknowledgement that the estimate of Q100 in the report is not a final figure to be adopted.

The author considers that extensive investigations of the kind discussed above and elsewhere throughout this report will be essential to improve the estimates of the magnitudes of the peak flows of historic floods to reduce the uncertainty of the results produced by flood frequency analysis.

The author would recommend against the use of any of the several extant versions of the rating curve at the Port Office. It needs to be recognised, however, that substantial margins of uncertainty about the rating curve at the Port Office may persist even after the extensive investigations have been completed. If that should turn out to be the case, the author would support use of “Savages Crossing” as the key site for flood frequency analysis.

4.2 Lesser matters of interpretation etc

63 "Figure 2 compares peak inflow and outflow for Wivenhoe dam from a number of sources for occasions when the dam is full. Also plotted on the graph is the 1:1 line (*or 50% reduction in flow by the dam as recommended by the 2003 Review Panel (Reference 20)*).While there is considerable scatter amongst the data points the graph shows below an inflow of 6000 - 8000 m3/s the attenuation is quite high while around 12000 m^3/s the attenuation is quite low. It is not unexpected that there would be some scatter as two floods could have a similar peak inflow and very different volumes and hydrograph shapes. There is however a reasonable correlation of volume and peak flow. For peak outflows up to 4000 m^3/s (max discharge allowed at Moggill under W3) the discharge is very dependent on the flow occurring in the Lockyer and Bremer Systems." (Italics added)

65 Figure 3 to 5 depict pre and post dam flow at Port Office, Moggill and Savages Crossing. The 50% reduction line*, as adopted by the 2003 Review Panel* is also shown. (Italics added)

<u>Comment</u> The 2003 Review Panel made no *recommendation* for a 50% reduction, nor did it adopt it. It noted that for the period 1890 to 2000 the DNRM model simulation of dam operations had indicated that it should be possible to operate the dams to reduce peak flood flow rates by about 60% on average and that it indicated a January 1974 flood attenuation of nearly 50%. (Reference 20, p15). The panel did not have access to the DNRM model and it recommended that it should be peer reviewed. (Reference 20, p23).

The work done in 2003 by SKM as consultants to BCC included hydrological modelling of the Brisbane River catchment for a number of synthetically generated design storm events (in this case 1% AEP design CRC FORGE rainfall events). In addition, hydrologic modelling was done using typical "real" event" spatial rainfall distributions with 1% AEP design CRC FORGE rainfall. The modelling of the pre-dams case produced inflow hydrographs at the site of Wivenhoe Dam in addition to the pre-dam flood hydrographs at the Port Office. The inflow hydrographs at the dam site were run through the DNRM dam operations model to generate outflow hydrographs from Wivenhoe Dam for the post-dams case. These outflow hydrographs provided the post-dams input from Wivenhoe Dam to the hydrological model of the Brisbane River catchment downstream from the dam that produced the post-dams flood hydrographs at the Port Office. The end result of this modelling was a substantial set of estimates of pairs of pre-dams and post-dams flood peaks at several locations along the Brisbane River, including at the Port Office. All had been calculated as 1% AEP design flood events. The Review Panel exercised its judgement to determine from this set of estimates the best estimate for the 1% AEP design flood flow at the Port Office. The fact that this turned out to be approximately 50% of the pre-dams flood flow is a consequence of the fact that most of the estimates of pairs of pre-dams and post-dams flood peaks were related approximately in this way. This ratio was applies only to the 1% AEP flood events considered in the SKM study. It was an **outcome** of the study, not an input to it. It cannot be applied to substantially different flood events.

Whatever ratio may be appropriate as the estimate of the attenuation for the 1% AEP design flood peak inflow to Wivenhoe Dam, it is fundamentally incorrect to use that ratio as applying to all magnitudes of inflow peaks, as is implied by the straight line plotted in Figure 2 of the WMAwater report. In fact, the magnitude of the **inflow volume** is of much greater significance than is the peak inflow rate. For floods with an inflow volume less than the flood storage compartment available, 100% attenuation can be achieved if so desired, regardless of the magnitude of the peak inflow rate. As the flood inflow volume becomes larger than the flood storage compartment available, the amount of attenuation of the peak flow that can be achieved will reduce but it will also depend on the dam operation strategy. For very rare flood events that are very much larger than that for which the dam was designed, the amount of attenuation will be very much reduced, though there will always be some attenuation because of storage volume resulting from rising flood levels upstream from the dam.

80 It would appear that this level was still not adopted as a planning level and SKM were commissioned by Council to prepare two further reports which were issued on 8th and 28th August 2003 to an Independent Review Panel (Reference 20)...... It is noted that the 2003 Review Panel terms of reference includes the following statement "*Even if the Q100 changes from 6800 m^3/s, it is likely that the Development Control Level will remain the same as is currently used in the Brisbane City Plan."*

<u>Comment</u> The 2003 Review Panel was not influenced in any way by the quoted statement.

82 SKM re-calibrated the 1998 Mike 11 hydraulic model to determine 1% AEP (100 year ARI) flood levels in a report from February 2004 (Reference 22). Although the December 2003 report had later found Q100 to be 6500 m^3/s, the 2003 Review Panel recommended Q100 flow of 6000 m^3/s at the Port Office gauge be used to giving a Q100 flood level at the Brisbane Port Office of 3.16 m AHD.

Comment The meaning of this paragraph is unclear. The Review Panel report (Reference 20) was issued in September 2003. Further, the Q100 level in that report is 3.3 m AHD, not 3.16 m AHD. The Review Panel saw a draft version of an SKM report with the same title as the December 2003 report (Reference 21). That draft was dated 28 August 2003 and it proposed the best estimates for the Q100 flow and levels at the Port Office as 6500 and 3.51 m AHD, respectively. The Review Panel cited this draft report in the List of References in Section 6 of its report (Reference 20). The contents of the draft SKM report were taken into account by the Review Panel when it formulated its considered judgements concerning the best estimates of the Q100 flow and level at the Port Office, aware that they differed from those proposed in the draft report. This is described in Section 4.8 of Reference 20.

5. CONCLUSION

Many of the matters discussed in this report are quite complex. The appropriate way to clarify many of the issues raised would be in thorough discussion and review with colleagues. Because of the very short time frame available, it has been necessary for the author to formulate the material presented in this report without any opportunity for discussion with colleagues or for lengthy review. With the benefit of sufficient time for such discussion and review of the issues in depth, the author’s views on some matters may evolve somewhat.

EMERITUS PROFESSOR COLIN J. APELT

Colin Apelt is Emeritus Professor and Honorary Research Consultant in the Department of Civil Engineering at The University of Queensland. He was appointed Professor of Civil Engineering in 1979 and served as Head of Department from 1982 to 1994. After retirement from full time academic staff in 1996, he was appointed Professorial Research Fellow until the end of 1999.

His research fields are in:
Computational Hydraulics applied to floods and tidal flows, to sediment transport and water quality in estuaries and marinas, to waves, and to transients in pipelines;
Experimental Fluid Mechanics applied to efficient passage of flood flows through waterways; to turbulence and energy losses in natural streams and engineered waterways; to flows past bluff bodies with relevance for flood and debris loads on bridges and for wind engineering; and to wave forces on berthing structures;
Computational Fluid Mechanics applied to flows in open channels and to bluff body flows;
He has substantial interests in all aspects of dredging and arranged four specialist courses on dredging in Brisbane since 1987.

He has acted as specialist consultant on hydraulic design of many major projects involving spillways, energy dissipators, bridges, culverts and flood mitigation works; and on computer modelling of flood and tidal flows in design and/or flood plain planning studies associated with many river and estuary systems in Queensland and New South Wales.

Since 1995 he has acted as specialist consultant in several major flood plain planning studies for the Brisbane River system and the Nerang River System. Those include;

- Member of Independent Review Board established to review the response of Brisbane City Council to the January 2011 flood event.
- Chair, Joint Flood Taskforce established to investigate the January 2011 flood event and to recommend interim standards and development guidelines to manage redevelopment of flood affected areas and new development activity within the Brisbane River floodplain for BCC. (2011)
- Review of Hydrodynamic Modelling for Tamar River & North Esk River Flood Study for Launceston City Council. (2008)
- Chair, Lord Mayor's Taskforce on Suburban Flooding, for BCC. (2005)
- Review of Brisbane River Flood Study for BCC as member of Independent Review Panel (R. Mein, C.J.Apelt, J Macintosh, E Weinmann) (2003)
- Review of Floodplain Hydraulic Modelling Methodology for Gold Coast City Council (with R Tomlinson). (2001)
- Review of Safety Analysis of Wivenhoe, Somerset and North Pine Dams (with Eric Lesleighter (SMEC)). (1992-1995)

The details of these and of other recent consultancies are given below.

Academic Qualifications
1952 BE(HonsI) in Civil Engineering, The University of Queensland.
1957 DPhil (Oxon), Fluid Dynamics and Numerical Analysis, Oxford University.

Professional Affiliations
Fellow of The Institution of Engineers Australia
Chartered Professional Engineer (retd)

Prizes and Awards

1952	University of Queensland Medal.
1954	Rhodes Scholar for Queensland.
1965	R.W.Chapman Medal of The IEAust.
1966	Fulbright Fellow, Senior Research Fellow Category.
1969	National Science Foundation Senior Foreign Scientist Fellowship Award.
1985	The Hawken Address, Queensland Division of The IEAust.
2001	Second Henderson Orator.
2011	Inducted into Engineers Australia National Committee on Water Engineering Hall of Fame.

Professional Experience

1951(Dec)-1954(Jul)	Design Engineer, Queensland Irrigation Commission.
1957(Jun)-1958(Nov)	Postdoctoral Research Fellow in Engineering Science, Oxford Univ.
1958(Dec)-1964(Dec)	Senior Lecturer in Civil Engineering, University of Queensland.
1965(Jan)-1979(Sep)	Reader in Civil Engineering, University of Queensland.
1966(Feb)-1966(Sep)	Research Associate, Maths Department, MIT.
1969(Feb)-1970(Jan)	Visiting Professor, Dept of Civil Engineering, Colorado State Univ. and Visiting Scientist, Nat. Centre for Atmos. Res.,Boulder, Colorado.
1977(Jan)-1977(Dec)	Visiting Professor, Civil Engineering, Univ. Canterbury, Christchurch.
1979(Sep)-1996(Jul)	Professor of Civil Engineering, University of Queensland.
1982(Jan)-1994(Apr)	Head of Department of Civil Engineering, Univ. of Queensland.
1996(Jul)-1999(Oct)	Emeritus Professor and Professorial Research Fellow, Dept of Civil Engineering, Univ. of Queensland.
1999(Oct)-Present	Emeritus Professor and Honorary Research Consultant, Dept of Civil Engineering, University of Queensland.

Competitive Research Grants

1972-1981	ARGC	$68,300
1981-1987	MST/AMSTAC	$143,200
1985-1990	ARC	$106,200
1985-1988	ARRB	$20,000
1993-1995	NCHRP(USA)	$60,300
1995	MARINAS	$53,400
1997-1998	CRC-TOURISM	$29,000
1998-1999	CRC-TOURISM	$110,000
1999- 2000	CRC-TOURISM	$120,000

Supervision of Postgraduate Research

PhD: 16 completed; MEngSc: 17 completed.

Publications Summary

2 Books; 91 refereed papers; 24 Commissioned Technical Reports; 2 Technical Movies; 1 Technical Video; 33 unrefereed papers and research reports.

Consultancies since 1990 (Current Consultancies not listed for reasons of Privacy)

Tweed River Entrance Sand Bypassing; Re-Assessment of Long Term Average Sand Transport Rate for Tweed River Sand Bypassing. C.J.Apelt as Independent Expert, UniQuest Project No 16066 for BMT WBM Pty Ltd Brisbane, 08/01/09 - 11/03/10.

Flagging of flood hazard areas, *Review of Methodology to Flag Flood Affected Areas in City Plan* 19/12/08 (4 pp) by C.J.Apelt and *Review of Final Methodology to Flag Flood Affected Areas in City Plan* 07/08/09 (5 pp) by C.J.Apelt, UniQuest Project Nos 15790 and 15790.02 for Brisbane City Council, 11/12/08 - 07/08/09.

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Queensland Floods Commission of Inquiry

Brisbane River 2011 Flood Event – Flood Frequency Analysis

Final Report by WMAwater
September 2011

Expert Comments by
Erwin Weinmann

RJ Keller & Associates

October 2011

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1 Introduction

These expert comments have been prepared to assist Clayton Utz, acting on behalf of Brisbane City Council, with the assessment of the report “Brisbane River 2011 Flood Event – Flood Frequency Analysis, Final Report, September 2011” prepared by WMAwater. This report will be referred to as “WMA (2011)”.

In the following, the basis and scope of the expert comments are further explained.

1.1 Appreciation of terms of reference for WMAwater report

Paragraph 5 of WMA (2011) gives the following terms of reference (TOR):

> “The Commission has requested that Mark Babister of WMAwater undertake the following:
>
> 1. Conduct a flood frequency analysis and determine the 1% AEP flood level for key locations on the Brisbane River below its junction with the Bremer River and on the Bremer River in the vicinity of Ipswich using information available prior to the January 2011 event. This work should be used to determine 1% AEP flood levels at up to 8 key locations in the Brisbane and Bremer Rivers and to produce 1% AEP flood profiles. This work should include a review of the SKM 1% AEP flood profile.
> 2. Repeat task 1 with the 2011 event included in the historical dataset.
> 3. Using results of task 1 and 2 determine the ARI and AEP of the January 2011 floods at particular points along the Brisbane River and Bremer River.”

It is not known to what extent these TOR have been supplemented by more detailed verbal instructions.

The following comments are relevant for the interpretation of these TOR:

- The term ‘flood frequency analysis’, in the first part of paragraph 1 of the TOR could be narrowly interpreted to mean that the study is restricted to the application of a statistical frequency analysis technique to an appropriate flood data set. However, the remainder of this paragraph makes it clear that a broader interpretation of the first task is appropriate: *the derivation by appropriate methodology of a relationship between flood magnitude (flows and levels) and frequency (expressed as annual exceedance probability – AEP, or average recurrence interval – ARI)*.
- The focus of the study is on estimates of the 1% annual exceedance probability (AEP) flood flows and flood levels at key locations. While this is not explicitly stated, these flood characteristics are understood to relate to the *current (post-dam) conditions of the Brisbane River catchment, river and estuary system*.
- Given the important flood management decisions that will be based on the study outcomes, *the estimated 1% AEP flood flows and flood levels should be as accurate as currently available data and methodology allow* (that is any remaining uncertainty about the adopted ‘best estimate’ should be as small as possible)
- The ‘best estimates’ of flood characteristics to be derived by the study should be *unbiased* (that is there should be no systematic tendency to under- or over-estimate, and any margin of safety to cover for uncertainties should be specified separately, as part of flood risk management measures).

- The reference at the end of paragraph 1 to the "review of the SKM 1% AEP flood profile" is read to imply that the review work should consider both hydrological approaches used in the SKM (2003) report to derive design floods (direct frequency analysis of flood data and simulation of design floods from design rainfalls) as well as the hydraulic modelling used to convert the derived design floods to design flood levels.
- The TOR recognise the *importance of the January 2011 flood as both a source of additional flood data and as a point of reference for flood plain management considerations.*

1.2. Criteria for assessment of report

The comments in the remainder of this report are based on the assessment of the data, methodology and analysis employed in the study against what is considered to represent accepted current practice, as documented in the current version of 'Australian Rainfall and Runoff' (ARR98), supplemented by more recent peer reviewed design information and methodology.

Appendix A to this report provides a summary of matters for consideration in design flood estimation, namely (i) the relationship between (probabilistic) design floods and actual flood events, (ii) the main flood producing and flood modifying factors to be taken account in flood estimation and (iii) the principal hydrologic approaches to design flood estimation. The main points to be taken from this summary are that the actual processes of flood formation and flood modification in the Brisbane River system are very complex, and that, for design flood estimates to be accurate and reliable, they need to be based on methodologies that take adequate account of these complexities and use the full range of flood data available.

It is recognised that the specified scope and available time frame and resources may have imposed limitations on the conduct of the WMAwater study, including the range of methods applied and the sources of data used. However, in these comments*, the assessment is against what is considered to be a desirable standard of rigorousness and completeness for a study whose findings can be expected to have very important and wide ranging implications.*

1.3 Scope and limitations of expert comments

The comments are based on the review of information contained in the following main documents:

- WMAwater (2011): Queensland Floods Commission of Inquiry, Brisbane River 2011 Flood Event – Flood Frequency Analysis Final Report, September 2011.
- SKM (2003): Brisbane River Flood Study – further investigation of flood frequency analysis incorporating dam operations and CRC-FORGE rainfall estimates – Brisbane River, Final Issue, 18 December 2003.
- Independent Review Panel (2003): Report to Brisbane City Council on review of Brisbane River Flood Study, 3 September 2003.
- BCC (2003): Joint Flood Taskforce Report, March 2011.

The comments are based on the information presented in these documents; they address the perceived strengths and limitations of the methodologies applied and compare the results produced by the different studies and reviews.

No additional analysis of basic flood data or information has been undertaken as part of this review.

2. Flood flow estimates derived by WMAwater

2.1 General

The methodology applied by WMAwater for the determination of 1% AEP flood levels in the study area involves three principal steps:

(i) Estimation of 1% AEP design peak flows at Port Office for pre-dam conditions by frequency analysis of peak flows
(ii) Conversion of pre-dam 1% AEP peak flows to post-dam 1% AEP peak flows at Port Office
(iii) Estimation of flood levels in study area for post-dam conditions

The first two steps involve hydrologic analysis techniques and are discussed in Sections 2.2 and 2.3 respectively. The third step involves hydraulic modelling and is discussed separately in Section 3.

2.2 1% AEP flood flows for pre-dam conditions

Data

Flood height data at Port Office

The basic data used to compile a series of maximum annual floods for the period of record are *recorded gauge heights at the Port Office*. Table 2 of WMA (2011) indicates that peak height records at the Port Office site commenced in 1841 but it is unclear how accurate and complete these records are for the early period (paragraph 40 implies that the 1841 flood level is sourced from a plan). Other floods reported for the period between 1824 and 1839 (including the 1825 referred to in the SKM 2003 report) have not been included in the analysis as they were judged to be either not significant or not reliably documented.

Notwithstanding some remaining uncertainties, it appears that the flood frequency analysis for the pre-dam conditions has been based on the most complete record of significant floods in the lower Brisbane River currently available.

To form a homogeneous record for flood frequency analysis, these recorded gauge heights need to be adjusted for the impacts of any significant changes in the conditions of the lower Brisbane River and estuary, notably dredging, river widening and major modifications to flood plain conditions. Section 4.2 of WMA (2011) details the significant changes in river conditions during the period of record, based on documentation in references. The adjustments to flood levels appear to be consistent with those used in BCC flood studies of 1999.

It appears that the adjustments to historical flood levels in the lower Brisbane River to compensate for changes in the conditions of the lower river are based on the most recent information that is readily available. However, as acknowledged in Paragraph 149 of the report, lack of detailed hydraulic modelling of the impacts of historical changes to the bathymetry of the Brisbane River and the possible impacts of storm surges on recorded flood levels, the adjustments are likely to have introduced significant additional uncertainty and possible bias into the 'homogeneous' record of flood heights at the Port Office gauge.

The conversion of these maximum flood heights to peak flows at the Port Office by means of a rating curve (including allowance for the impacts of the dams on recorded flood levels and flows in the period after 1959) is discussed in the next section (Methodology and results).

Peak flow data at other gauging sites
Section 7.1.2 of WMA (2011) discusses the availability of data at other gauging sites and the decision to base the flood frequency analysis on data from the Port Office gauge. The alternative gauges considered are (in downstream direction):

- Lowood/Savages Crossing – located some distance downstream of junction of Lockyer Creek with Brisbane River (1909 to date)
- Mt Crosby Weir – located some distance upstream of junction of Bremer River with Brisbane River (1900-1975)
- Mogill – located immediately downstream of junction of Bremer river with Brisbane River (1965 to date, after construction of Somerset Dam)

In contrast to the Port Office gauge, the rating curves for these three gauges are based directly on concurrent measurements of flood height and flow rate. However, these rating curves still require some degree of extrapolation to the magnitude of the largest observed flood events.

The decision by WMAwater to base the flood frequency analysis on flood height data at the Port Office gauge was based mainly on the fact that the significantly greater record length available at this site would better capture the long term climate variability affecting flood observations. *The discussion in paragraph 115 of WMA (2011) recognises the tradeoff involved between length of record and accuracy of flood data but provides only limited justification for the decision in favour of the longer but more uncertain flood data record at the Port Office.*

Figure 1 below shows a comparison of the pre-dam peakflow estimates for the Port Office site (as used in WMA 2011) and the values used by SKM (2003) for the Savages Crossing site (including simulated peak flows for the events that occurred after construction of the two Dams).

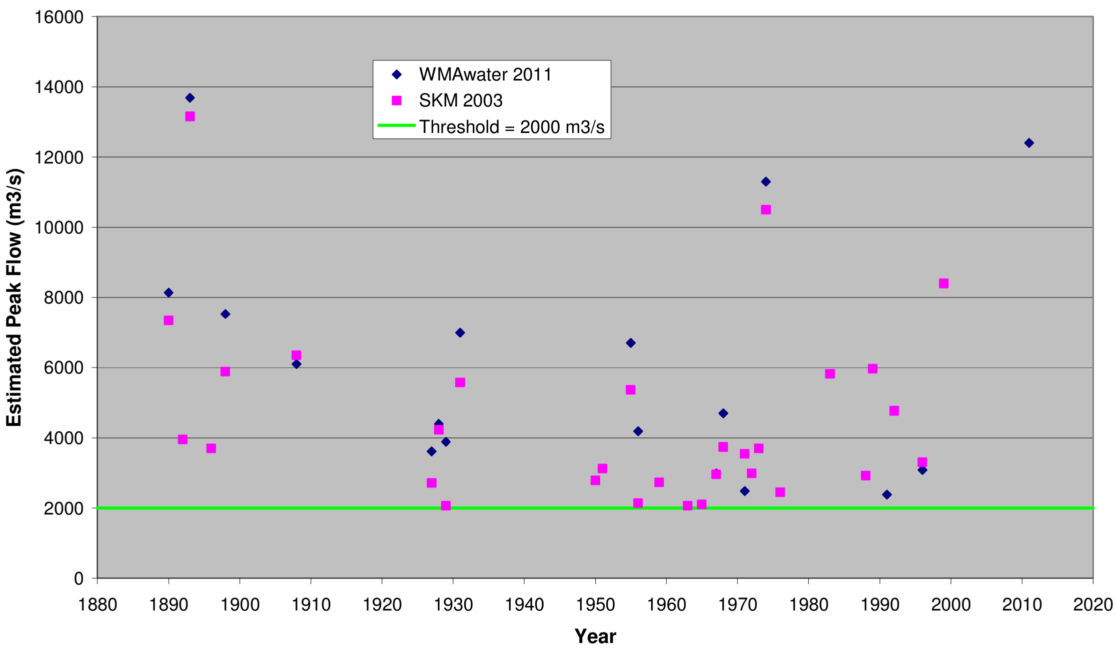

Figure 1 Comparison of annual peak flow data 1890-2011 (pre-dam conditions, flood peaks above threshold of 2000 m^3/s)

The comparison indicates that the estimates for the largest flood events (> 5000 m^3/s) are within about 10 to 20% of each other and thus quite consistent, given the likely influence of a range of factors that introduce variations in flood peaks between the two sites. However, the WMA (2011) pre-dam peak flow series misses some significant flood events which have occurred since the construction of Wivenhoe Dam and have been substantially mitigated by the dam (notably the 1999 event).

It would be highly desirable for any future detailed flood study to use the available flood data from all four sites in accordance with their special merits and limitations. This would allow some checking of flood estimates for consistency and would help to reduce the remaining uncertainty in design flood estimates.

Methodology and results

The main steps in the flood frequency analysis are:

(i) conversion of 'recorded' maximum flood heights to corresponding peak flows by means of a rating curve

(ii) adjustment of estimated flows for post-dam period for flood mitigation effects of Somerset and Wivenhoe Dams

(iii) fitting of a flood frequency curve to the series of adjusted annual maximum flood peaks using a selected probability distribution and fitting technique

(iv) determining confidence limits to express uncertainty in the flood quantile estimates (the flood magnitudes corresponding to selected ARIs or AEPs)

(i) Rating curve at Port Office

While there is anecdotal evidence of some height-discharge measurements at this site (Section 6.3.1), none such observation data was available to construct a rating at the Port Office. The 'best estimate of the high flow rating curve' shown in Figure 8 of WMA (2011) is based on the results presented in previous flood study reports and estimates of the flood height and flow ranges for the 1893, 1974 and 2011 flood events.

The use of a single valued rating curve relationship to convert recorded flood heights to flood peak flows in the range of flood magnitudes of specific interest (flows greater than 2000 m^3/s) involves the important assumption that the variations resulting from different hydrograph shapes and volumes, changing river bathymetry during major flood events and dynamic effects associated with different tidal boundary conditions are relatively minor, and the use of an average rating curve is sufficient.

There is limited information presented in the report to assess the validity of the simplifying assumptions embodied in the rating curve but they can be expected to introduce additional uncertainty into the basic data series used for flood frequency analysis.

(ii) Adjustments for flood mitigation effects of dams

The impacts of Somerset and Wivenhoe Dams on peak flood flows and flood levels in the lower Brisbane River system are discussed in Section 4.3 of WMA (2011), and Figure 3 presents data on the relationship between pre- and post-dam peak flows at the Port Office gauge site. The estimate of the pre-dam equivalent of the January 2011 flood shown in Appendix B (12,400 m^3/s) is consistent with the information presented in Figure 3, but the source of the pre-dam peak flow estimate for the 1974 flood event (11,300 m^3/s) is unclear.

It appears from the information in Appendix B to the WMA (2011) report that the adjustment of 'observed' peak flows for the 1967, 1968, 1971, 1991 and 1996 (and possibly 1974) flood events to pre-dam conditions was not based directly on the information in Figure 3, and Appendix B indicates that the estimated pre-dam flows for these events were sourced from the SKM (1999) report. The WMA (2011) report does not discuss the assumptions made in the SKM report to adjust the 'observed' peak flows to pre-dam conditions, and it is thus difficult to assess the degree of uncertainty (and possible bias) introduced by this step.

(iii) Fitting of flood frequency distribution

The adopted flood frequency analysis method described in Sections 3.3.1 to 3.3.3 of WMA (2011), as implemented in the FLIKE software, is considered to be in accordance with current best practice, as described in the draft of revised Book IV of 'Australian Rainfall and Runoff'. [It is of interest to note that the flood frequency analyses presented in SKM (2003) were also based on application of the FLIKE software.] No specific allowance has been made for differences in accuracy of individual flood peak estimates or for rating curve errors.

The comparison of the fitted flood frequency curves presented in Figures 9 and 10 indicates that the LP3 distribution (Figure 10) provides a better fit to the flood observations, but fails to reflect the apparent flattening of the flood frequency relationship at peak flows above 10,000 m^3/s. *The adopted 1% AEP peak flow estimate of 13,000 m^3/s for the pre-dam conditions appears to be an appropriate 'best estimate', given the flood data series used as basic input*.

(iv) Confidence limits on 1% AEP flood estimate

The confidence limits shown in Figure 10 of WMA (2011) reflect the uncertainty introduced into the flood estimates because of the high degree of natural variability in the flood data (including random errors) and the limited flood record length available for the estimation of the 3 parameters of the selected distribution. The effects of any systematic under- or over-estimation of peak flows for individual flood events (e.g. resulting from errors in adjusted flood heights or rating curve errors) are not included in these fitted confidence limits, nor do the make allowance for uncertainty in selecting the most appropriate theoretical probability distribution (GEV, LP3 or other candidate distribution).

Figure 10 indicates a 90% chance that the true 1% AEP flood peak estimate for pre-dam conditions is between about 10,000 and 22,000 m^3/s but this confidence interval would be wider if allowance was made for other uncertainty factors. In other words, even when the flood data from the largest flood events in a period of record of 170 years at the Port Office are analysed, there remains a substantial degree of uncertainty in the 'best estimate' of the 1% AEP flood peak under pre-dam conditions.

Appraisal of pre-dam flood estimation results

Notwithstanding the considerable degree of uncertainty with the 'best estimate' of the 1% AEP peak flow under pre-dam conditions, the estimate of 13,000 m^3/s produced by the WMA (2011) study is plausible in the light of the largest observed floods over the period of record and broadly consistent with the flood estimates derived by previous studies. Specifically, the 2003 report by the Independent Review Panel gives a peak flow estimate in the range of 10,000 to 14,000 m^3/s for pre-dam conditions at Savages Crossing, based on the flood frequency analyses reported in the SKM (2003) report, and the Review Panel adopted a peak flow estimate in the plausible range of 11,000 to 13,000 m^3/s for pre-dam conditions at the Port Office site[1].

[1] The uncertainty ranges given in the 2003 Review Panel report are labelled as 'plausible bounds'. This relatively narrow confidence interval should be interpreted as only a notional indication of uncertainty around the 'best estimate of 12,000 m^3/s. A formally derived 90% confidence interval about this design flood estimate would be expected to be considerably wider.

Given the large degree of uncertainty with the pre-dam flood estimates at the Port Office site, it would be highly desirable to make use of all available data for large historical flood events (including event rainfall data and flood records from other sites) and to check for consistency between the different sources of information.

2.3 1% AEP flood flows for post-dam conditions

Data

The estimation by WMA (2011) of design peak flows for post-dam conditions is also mainly based on flood data for the Port Office gauge. The actual flood height observations for historical floods are supplemented by results of simulation studies of flood events for pre- and post-dam conditions, apparently mostly sourced from the SKM (2003) report.

Methodology and results

The basic methodology applied by WMA (2011) to account for the flood mitigation effects of the two dams is to use a graphical approach to derive a relationship between pre-dam and post-dam flood peaks at the Port Office site, based on the limited data described above.

Figure 2 of WMA (2011) presents some selected results for the flood mitigation effect of Wivenhoe Dam immediately downstream of the dam, assuming that the dam is at full supply level at the start of the flood event. Figure 3 uses data from a number of sources to represent the flood mitigation effect of the two dams on peak flows at the Port Office. The methodology and assumptions involved in coming up with flood estimates for the pre-and post-dam conditions are not explained in any detail but it appears that most weight is given to pre-and post-dam estimates for the 1893 flood event, derived by SKM (2003), and the January 2011 flood event, derived separately by SKM and WMA.

Finally, the conversion of the 1% AEP peak flood estimate of 13,000 m^3/s for pre-dam conditions to an equivalent post-dam 1% AEP peak flow estimate of 9500 m^3/s was achieved by a single step, described very briefly in paragraph 132 of WMA (2011). These results imply a 27% peak flow attenuation effect of the two dams for the 1% peak flow at the Port Office.

It is considered that the hydrologic basis of this conversion step has not been sufficiently substantiated in the report. Given the complex array of factors that affect the relationship between pre-dam and post-dam flood characteristics in the lower Brisbane River, and the important implications of this conversion step on the 1% AEP flood profile, it should be based on a comprehensive analysis of how the relationship varies in response to different factors, and what can be considered to be a 'typical' degree of attenuation produced by the adopted flood operation of the two dams.

Appraisal of post-dam flood estimation results

The 2003 Review Panel report quotes results from DNRM simulations which indicate that the dams could be expected "to reduce peak flow rates by about 60% on average". It also noted that "the model indicates a January 1974 flood attenuation of nearly 50%, with a peak inflow rate of 10,500 m^3/s and outflow rate of 5,500 m^3/s". The 2003 Review Panel also took into account the results of the RAFTS model simulations by SKM for the pre- and post-dam conditions and concluded that "under post-dam conditions the Panel would expect Q100 flows downstream of Wivenhoe dam to be of the order of 50% of those under pre-dam conditions".

From basic hydrological considerations and experience gathered from other major dam systems, it can be expected that the potential flood attenuation effect (% reduction in peak flow) of Wivenhoe and Somerset Dams is generally largest for small to moderate floods and reduces with increasing flood magnitude (flood volume). However, the large degree of variability in the factors that determine the magnitude and frequency of floods for the post-dam situation (see Table A1 in Appendix A to this report for a summary of these factors) means that the relationship between pre- and post-dam peak flows will be a complex one, characterised by a large degree of scatter around any trend line. The relationship also depends on the assumed operating rules for the dams under major flood conditions.

Figure 2 gives a qualitative indication firstly of the *possible range of attenuation* by the two dams, from no attenuation when a flood is generated essentially in the parts of the catchment below Wivenhoe Dam, to virtually full attenuation when the flood is generated in the parts of the catchment above Wivenhoe Dam and when there is a large storage volume available relative to the flood volume. Secondly the figure shows a narrower *plausible range of attenuation within which most of the flood events could be expected to fall.*

The January 2011 flood appears to lie near the upper end of the plausible spectrum of variation, where the special characteristics of this event resulted in only a modest degree of attenuation. At the other end of the spectrum is the February 1999 flood, which resulted in a substantial inflow to the dams (Appendix D of SKM 2003 shows a simulated pre-dam flood peak flow of 8400 m^3/s at Savages Crossing) but there was only a minor flood recorded below the Dam. The estimated attenuation associated with the January 1974 event lies near the middle of the spectrum[2].

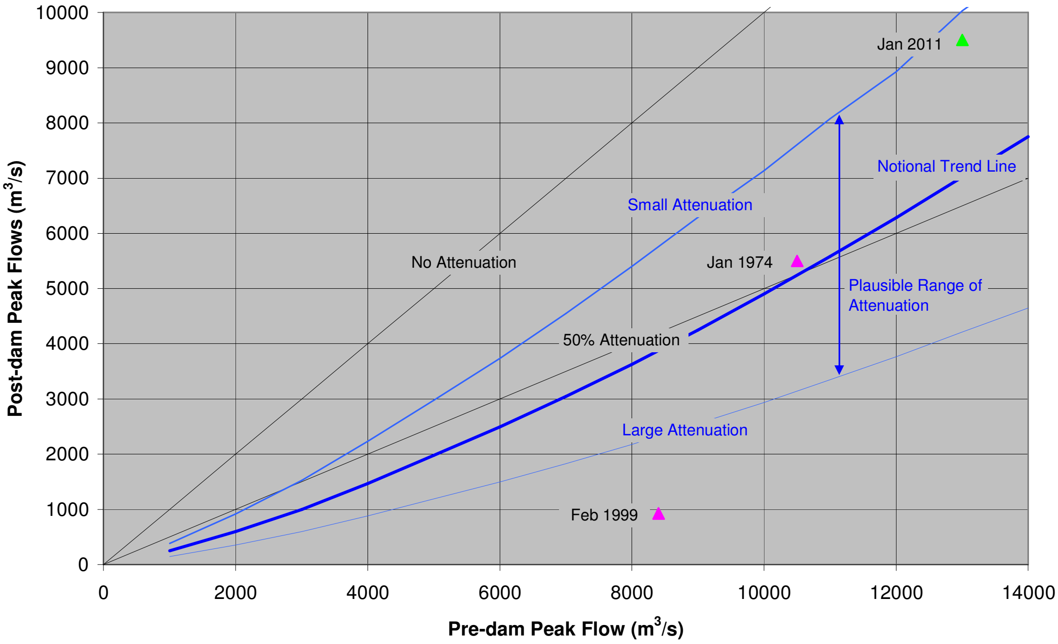

Figure 2 Conceptual diagram illustrating the possible and plausible ranges of flood peak attenuation by Somerset and Wivenhoe Dams

The methodology used in the WMA (2011) report to estimate the post-dam 1% AEP peak flow at the Port Office is based on very few data points which do not properly account for the large degree of variability and estimation uncertainty introduced by variations in storm characteristics and initial catchment/storage conditions. Use of the estimated attenuation effect for the January 2011 flood event as the main basis for determining the post-dam 1%AEP peak flow and corresponding flood level profile is considered to be arbitrary and likely to lead to biased flood estimates. Without confirmation from further analysis, the WMA

[2] In interpreting the estimated attenuation for these historical flood events it needs to be kept in mind that the results for each event are based on a set of specific assumptions which have not been fully documented. The estimated attenuation for the 1974 and 1999 events relates to the Savages Crossing site, while the attenuation for the 2011 event is for the Port Office site.

(2011) peak flow estimate of 9500 m^3/s can therefore not be considered to represent a 'best estimate' of the 1% AEP peak flow for the lower Brisbane River under post-dam conditions.

In my opinion a proper assessment of the likely attenuation effects of the dams on design floods for the lower Brisbane River system needs to be based on simulation modelling studies that examine the effects of likely variations in the key flood producing and flood modifying factors identified in Table A1. The variability effects could be assessed in an approximate fashion by sensitivity analyses, but preferably in a more formal joint probability framework, using Monte Carlo simulation methods. Such an approach would also give a more quantitative indication of the uncertainty in the post-dam design flood estimates.

Alternative estimation approaches

There are also two other approaches available for the estimation of the post-dam flood frequency curve in the lower Brisbane River:

(i) Adjusting each pre-dam annual flood peak recorded at a long-term gauging site, using a simulation model of the flood operation of the two dams to reflect the flood mitigation effects of the two dams, then undertaking a flood frequency analysis of this extended post-dam flood record. Such an adjusted data series for Savages Crossing was provided by DNRM and analysed in the SKM (2003) study as Case 4. However, the results of this analysis were discounted "as the method used [by DNRM] to obtain the adjusted data series was not assessed by SKM".

(ii) Using a design rainfall based approach to estimate the flood frequency curve for pre-and post-dam conditions, based on a well calibrated hydrologic model of the catchment and dam system, combined with a hydraulic river model to route the estimated design flood hydrograph to the point of interest. This approach was applied in the SKM (2003) study, using a calibrated RAFTS model of the Brisbane River catchment to Savages Crossing and a MIKE 11 model to route the flood hydrographs through the river reaches between Savages Crossing and the Port Office. The rainfall-runoff modelling approach adopted in SKM (2003) produced a post-dam 1% AEP flood peak estimate at the Port Office in the range of 5000 to 8000 m^3/s, with an adopted 'best estimate of 6500 m^3/s. It was noted in the 2003 Review Panel report that the flood estimates produced by this approach for the pre-dam conditions were significantly lower than the estimates from flood frequency analysis, and future work was suggested to address any apparent inconsistencies in the results from the two approaches. [Paragraph 138 of WMA (2011) explains this inconsistency by an apparent underestimation of catchment rainfalls (and consequently design rainfalls) in the more elevated parts of the Brisbane River catchment.]

Notwithstanding the limitations in the results obtained by SKM (2003) with the application of the design rainfall based modelling approach, with further development and additional data, and applied in conjunction with flood frequency analysis for additional validation, this approach is considered to have the potential to produce more accurate estimates of design floods in the AEP range from say 2% to 0.5% for the post-dam conditions in the lower Brisbane River.

2.4 Estimated AEP and ARI of January 2011 flood

The data and flood frequency analysis results presented in Figures 9 and 10 of WMA (2011) indicate that *the estimated AEP of an event similar to the January 2011 flood but occurring under pre-dam conditions is of the order of 1%* (equivalent to an ARI of 100 years).

Given the large degree of uncertainty in the estimation of the 1% AEP flood for post-dam conditions and the lack of a complete flood frequency curve for these conditions, it is difficult to assign a reliable AEP estimate to January 2011 flood event. The estimate of 0.83% AEP (120 years ARI) given in paragraph 133 of WMA (2011) indicates that the frequency of the post-dam flood is slightly lower than that of the estimated pre-dam flood. This can be *interpreted to imply that the catchment and storage conditions for this event may have been somewhat more severe than would be expected on average.*

The Joint Flood Taskforce report (2011) recommended the use of the January 2011 event as an interim standard for Brisbane City Council to base its decisions concerning new development and redevelopment. However, it clearly stated that a precautionary approach had been used in coming up with this recommendation which should only apply until the comprehensive flood study it recommended was completed. *This interim recommendation should thus not be interpreted as indicating that the flood flows and levels experienced in the January flood event represent an accurate and unbiased flood estimate for 1% AEP.*

Section 7.2 of the WMA (2011) report gives some information on rainfalls for the Brisbane River catchment, concentrating on a comparison of the 72-hour 1% AEP design rainfall estimates with the Seqwater estimates of 3-day rainfalls in the January 2011 event. The conclusion reached from this analysis is that "on a 72 hour basis the 2011 event upstream of the dam was slightly larger than a 1% AEP event and slightly smaller than a 1% AEP event downstream of the dam" (paragraph 139). *This conclusion appears to be inconsistent with the finding in paragraph 133 but the difference can be explained by the influence of the range of factors that affect the conversion of rainfall inputs to flood outputs, as discussed in Section 2.3 above.*

3. Estimation of 1% AEP flood level profile

The estimation of the 1% AEP flood level profile for the lower Brisbane River relies firstly on the results of the *hydrologic methods for estimating flood flows* and secondly on the *translation of these flood flows to flood levels at points of interest by means of hydraulic modelling*. My particular expertise is mainly in the area of *hydrologic design flood estimation methods and their application in different practical situations.*

My appraisal of the information provided in the WMA (2011) report has concentrated on the hydrologic aspects of the flood study methodology and the flood flow estimation results. My comments on the estimation of the flood level profile for the 1% AEP flood are therefore restricted to aspect that relate to the hydrologic inputs to the flood level determinations and their expected impacts on estimated design flood levels.

As pointed it out in the section on appraisal of post-dam flood estimation results, the 1% peak flood estimate for the Brisbane River reach below Mogill is associated with a large degree of uncertainty that also affects the hydraulic modelling of this design flood event and the flood level results obtained. *The WMA (2011) report does not provide any indication of the impact of this uncertainty in design flood flows on the estimated flood levels, other than stating that a 500 m^3/s reduction in post-dam peak flows would translate into an approximate flood level reduction of 0.5m at Mogill and 0.2 m at the Port Office (paragraph 143).*

It is also important to recognise that the WMA (2011) hydrologic analysis has been restricted to the estimation of peak flows for the pre- and post-dam conditions. The routing of flood flows through the lower reaches of the Brisbane River and the determination of the flood levels associated with these peak flows is also significantly influenced by the assumed hydrograph shape and flood volume associated with each peak flow. The WMA (2011) report does not detail the assumptions made for these flood characteristics or discuss their influence on the calculated flood levels.

4. Conclusion

My appraisal of the flood studies for the Brisbane River reported in WMA (2011) and comparison with information available from other flood study reports supports the following main comments:

1. The terms of reference for the WMA (2011) study appear to have been interpreted too narrowly to ensure that the estimated 1% AEP flood flows and flood levels are as accurate as currently available data and methodology allow, so that they can provide a firm basis for flood risk management decisions with wide ranging implications.
2. The 1% AEP peak flow estimate of 13,000 m^3/s for the Port Office site under pre-dam conditions is considered to be plausible and broadly consistent with estimates obtained by other studies but has a very wide margin of uncertainty associated with it. The WMA (2011) report recognises this uncertainty and suggests additional studies to improve the rating relationship at the Port Office. To reduce this uncertainty, it would be necessary to make use of other sources of data for large historical flood events (including event rainfall data and flood records from other sites) and to check for consistency between the different sources of information.
3. The conversion of pre-dam design peak flows to post-dam peak flow represents a challenging hydrological task, as it has to take account of the likely range of variability of the flood producing and flood modifying factors that affect this conversion. The WMA (2011) report does not demonstrate that this variability has been adequately allowed for in the determination of the post-dam 1% AEP peak flows at the Port Office site. The report does not include any suggestions for future work to address any limitations in the method used for this conversion.
4. The simplifying assumption used in WMA (2011) that the estimated attenuation effect for the January 2011 flood event is representative of typical conditions is considered to have introduced significant (high) bias into the estimated post-dam 1%AEP peak flow and corresponding flood level profile. Without confirmation from further analysis, the WMA (2011) peak flow estimate of 9500 m^3/s can therefore not be considered to represent a 'best estimate' of the 1% AEP peak flow for the lower Brisbane River under post-dam conditions.
5. For a more defensible estimate of the 1% AEP post-dam flood characteristics in the lower Brisbane River it will be necessary to use the combined results of a range of estimation methods based on all the relevant sources of flood data. The methods applied should include rainfall based design flood simulation for the pre- and post-dam conditions.
6. Given the high degree of variability in Brisbane River flood characteristics that can result from widely varying storm rainfall characteristics and initial catchment/storage conditions, it would be desirable to examine to what extent the estimation uncertainty could be reduced by the adoption a joint probability modelling framework (Monte Carlo simulation), as had been suggested in previous studies and reviews.
7. The large degree of uncertainty in the estimated 1% AEP peak flows for the post-dam conditions can be expected to be carried through into the determination of the flood level profile for this design flood event. Given the volume-sensitive nature of the lower Brisbane River system, it would be more appropriate to apply a hydrologic flood estimation method that produces complete flood hydrographs rather than just peak flows as inputs to the hydraulic flood level estimation model.

8. Finally, the outcomes of recent flood studies for the Brisbane River system, including the WMA (2011) study, appear to have been significantly restricted by the limited scope of the studies. Given the importance and wide ranging implications of the flood determinations emanating from the work of The Commission, it is considered essential that any future studies be given enough scope to adequately address the complexities of the Brisbane River flooding situation. The outcomes of these more comprehensive studies would also be helpful in supporting improved decisions on flood operation and management.

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APPENDIX A
Design flood estimation – matters for consideration

Relationship between design floods and actual flood events

Design flood estimates are probability-based estimates of flood characteristics (flood flows and flood levels) at specified locations (e.g. along a stretch of the Brisbane River) and for a specified set of conditions (e.g. the conditions existing in 2011, expected to remain applicable for the next few years). They reflect the outcomes of complex flood formation processes over the catchment and flood modification processes as the flood wave (or flood hydrograph) travels through the river/floodplain/estuary system. Each actual flood event results from different combinations of these factors within a typical range of variation, resulting in a large degree of variability in flood characteristics (e.g. flood magnitude, duration, 'peakedness'). Design floods should reflect the *'typical characteristics of floods that can be expected to occur at specified frequencies* (ARIs or AEPs).

Causes of floods and main factors affecting flood characteristics

Table A1 illustrates the main factors that affect the formation and modification of floods, and how they are concepualised for the estimation of design floods. While there are other possible causes of floods, for the Brisbane River catchment and the range of flood frequencies of direct interest here, the principal cause of floods is extended heavy rainfall over the catchment. Apart from the duration of a storm rainfall event and the total rainfall depth (average over catchment), the way this total rainfall is distributed in time and how it varies over the different parts of the catchment are also important in determining the resulting flood characteristics. The large range of possible flood modifying factors can significantly increase the degree of variability of flood outputs and adds further complexity to the design flood estimation problem.

Principal approaches to design flood estimation

The two principal approaches are distinguished by the basic data and methodology they use.

Approach 1: Flood frequency analysis (FFA) is based on statistical analysis of *flood characteristic outputs,* generally peak flows.

Strengths:

- based directly on data for flood characteristic at or near the location of interest
- requires few assumptions on how floods have been produced (if catchment and river conditions have remained relatively unchanged)
- allows relatively simple assessment of uncertainty in flood estimates arising from variability in data and limited record length (derivation of confidence limits)

Potential weaknesses:

- needs relatively long data records for reliable estimation of larger design floods (at least 50 years for estimation of 100 year ARI flood, longer for complex systems)
- flood data in record need to be for essentially unchanged conditions or have to be adjusted to a common set of catchment and river conditions
- adjustments to flood data for changes in conditions may introduce significant uncertainties into flood estimates (depending on the reliability of the data and methodologies used for the adjustments)
- extrapolation of fitted flood frequency curves to rarer flood events involves significant uncertainties
- applied mostly for peak flows – in estuarine flooding situations the influence of varying flood hydrograph shapes (flood volumes) and tidal conditions may invalidate the assumption of a one-to-one relationship between flood peak and flood level (as expressed by a rating curve).

Table A1 - Factors to be considered in design flood estimation

	ASPECT	FACTORS TO BE CONSIDERED	EFFECT ON FLOODS	RELEVANCE FOR FLOOD ESTIMATION
CAUSE OF FLOOD	**Catchment Rainfall**	• *Duration of storm event & total event rainfall* • *Distribution in time* • *Distribution in space*	• *Size of flood* • *Relative magnitude & timing of tributary flows*	**Basis for Simulation Modelling of Design Flood Events (based on statistical analysis of observed storm rainfall characteristics)**
FLOOD MODIFYING FACTORS (UPSTREAM SYSTEM)	Initial Catchment Conditions	• Catchment wetness • Initial content of dams • Initial condition of floodplains	• Proportion of rain becoming runoff • Flood mitigation potential of storages and floodplains	**Design assumptions on *hydrologic* flood modifying factors**
	Catchment Modifications	• *Major water storage development (for water supply & flood mitigation)* • Major rural land use changes • Urbanisation (large scale)	• *Reduced and delayed flood peaks downstream of storage* • Increased runoff • Faster flood response	
	River & Floodplain Modifications	• Changes to river & floodplain morphology • Riverside development • River crossings	• Changed flood routing conditions • locally changed flood levels	
FLOOD MODIFYING FACTORS (DOWN-STREAM SYSTEM)	River, Floodplain & Estuary Modifications	• *Changes to river , floodplain& estuary morphology* • Riverside development	• Changed hydraulic conditions • ***Modified rating curve (Port Office)*** • locally changed flood levels	**Design assumptions on *hydraulic* flood modifying factors**
	Tidal Boundary Conditions	• Astronomical tides • Tidal anomalies (effects of wind, waves, air pressure)	• Tide/flood interactions • ***Modified rating curve (Port Office)*** • Raised flood levels (lower system)	
FLOOD OUTPUTS	**Flood Characteristics**	**Flood flows** at key points (esp. peak flows)	Result of **hydrologic** factors (influenced by hydraulic factors)	**Basis for hydrologic flood frequency analysis**
		Flood levels at sites of interest (max. levels)	Combined result of **hydrologic** & **hydraulic** factors	**Directly observed flood data**

Note: Factors affected by the highest degree of variability are shown *italics*

Approach 2: Rainfall-based design flood estimation by simulation of the flood formation and flood modification processes

Strengths:

- basic probabilistic input: generalised design rainfall data which are based not just on rainfall data from the catchment of interest but also other catchments in the region – this allows more reliable extrapolation to rare events
- rainfall data for historic events are less affected by changes to catchment conditions than flood data
- flexibility to reflect various changes in catchment, storage and river conditions in hydrologic simulation model
- produces complete flood hydrographs – required in volume sensitive systems (where storage and tidal impacts play an important role in determining flood levels)
- can be used in a complementary fashion to flood frequency analysis

Potential weaknesses:

- quality of calibration/validation of simulation models depends on availability of concurrent storm rainfall and flood data for a range of flood events
- available rainfall data may give only an incomplete picture of the actual rainfall variation over the catchment
- requires a range of assumptions on flood modifying factors to ensure that design rainfall inputs are converted to design flood outputs of corresponding ARI or AEP ('probability neutral' conversion)
- quantification of uncertainties (confidence limits) not part of standard procedures (some indication of uncertainty from sensitivity analyses)

To allow best use of all available forms of flood data relevant to a particular catchment system, it is desirable to *use both approaches in a complementary fashion*. Where possible, design flood estimates for the catchment of interest should also be assessed for *consistency/compatibility with design flood estimates for similar catchments in the region*.

Curriculum Vitae

NAME: **WEINMANN, Peter Erwin**

PRESENT APPOINTMENT: Adjunct Research Associate of the Department of Civil Engineering, Monash University

CONTACT DETAILS:
[redacted]
Tel.: [redacted] Mobile: [redacted]
Email: [redacted]

QUALIFICATIONS:
- Dipl. Ing. ETH (Zurich), 1969 (Agricultural Engineering & Surveying)
- MEngSc (Monash), 1978 (major thesis on "Comparison of Flood Routing Methods for Natural Rivers")

FIELDS OF EXPERTISE:
- Hydrology (flood estimation, yield and low flow studies)
- Hydrologic risk assessments (especially dam safety)
- Flood risk management
- Water resource assessment and planning studies
- Design of hydrologic data collection networks

PROFESSIONAL AFFILIATIONS:
- Member, Engineers Australia, Chartered Professional Engineer
- Engineers Australia, Revision Committee of 'Australian Rainfall and Runoff'
- Editorial Panel, Australian Journal of Water Resources
- UNESCO International Hydrology Program, National Committee

AWARDS:
- W H Warren Medal - awarded by Engineers Australia for best paper in Civil Engineering in 1992 (jointly with R J Nathan)
- G N Alexander Medal (2002) - awarded for best paper on Hydrology and Water Resources in an Engineers Australia publication (jointly with A Rahman and R G Mein)
- C H Munro Orator (Engineers Australia, 2006)

PUBLICATIONS: Over 60 refereed papers, covering a wide range of topics in the fields of hydrology and water engineering (see list of publications).

CAREER SUMMARY:
- Since 2006: Independent consultant specialising in hydrologic studies and flood risk assessment/management
- 1993-2005: Senior Lecturer at Department of Civil Engineering, Monash University (Head of Water/Environmental Group 2002-2004)
- 2002-2005: Deputy Director of CRC for Catchment Hydrology (Monash Node)
- 1984-1992: Principal Hydrologist, Rural Water Corp., Victoria
- 1977-1984: Designing Engineer, State Rivers & Water Supply Commission, Victoria, Flood Plain Management Section
- 1970-1977: Engineer/Designing Engineer, State Rivers & Water Supply Commission, Victoria, Major Works - Designs Division

RELEVANT CONSULTING EXPERIENCE (RECENT):
- Brisbane City Council: Member of Joint Flood Taskforce 2011
- Gold Coast City Council: Advice on development of hydrologic models for Gold Coast City catchments (2007-2011)
- Snowy Hydro: Peer review of Talbingo Dam flood hydrology (2010/11)
- SKM for NSW State Water and ACTEW: Peer review of project on “Estimation of rare design rainfalls for NSW and ACT”
- SunWater:: Burdekin Falls Dam, design flood hydrology review (2010)
- Department of Water, Western Australia: Peer review of Murray Area flood study – hydrologic assessment (2010)
- Glenelg-Hopkins CMA: Review of flood studies for Port Fairy, Warrnambool and Portland catchments (2008-2010)
- State Water (NSW): Review of hydrological spillway adequacy assessments of portfolio of 16 State Water dams (2007-2010)
- Brisbane City Council: Member of Independent Expert Review Panel for Brisbane River Flood Study (2003)

A part of BMT in Energy and Environment

QUEENSLAND FLOODS COMMISSION OF INQUIRY

Technical Review of Flood Frequency Analysis Report by WMA Water (18 September 2011), specifically as it relates to Ipswich City Council

prepared for

Ipswich City Council

R. B18414.003.00.doc
October 2011

QUEENSLAND FLOODS COMMISSION OF INQUIRY

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prepared for

Ipswich City Council

14 October 2011

Prepared For:	Ipswich City Council
Prepared By:	BMT WBM Pty Ltd (Member of the BMT group of companies)

Offices

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Sydney
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CONTENTS

1 PURPOSE AND SCOPE OF THE REPORT

This report has been prepared by Neil Collins. A copy of his CV is included as Appendix A.

This report describes my desktop review of a report prepared by Mark Babister of WMA Water dated 18 September 2011 for the Queensland Floods Commission of Inquiry (Brisbane Frequency Report). The review is limited to those aspects of the Brisbane Frequency Report that are of relevance to flooding in Ipswich City, i.e. Brisbane River flooding of Redbank and Goodna. In the time available for the review, I have been unable to fully test the conclusions reached by WMA Water.

In my opinion, it is premature for WMA Water to reach the conclusions they have (specifically paragraph 145 where the 1% AEP flood flow of 9500m^3/s is adopted, and paragraph 146, where the 1% AEP flood line is said to be 1m higher at the Port Office and 3m higher at Moggill than previous estimates) because those conclusions are not underpinned by a consideration of all relevant factors. The analysis conducted by WMA Water:

(a) Is reliant on a single point flood frequency analysis that is itself subject to considerable uncertainty;

(b) Does not incorporate a probabilistic framework to assess natural variability across the catchment;

(c) Does not use both the statistical flood frequency analysis and simulation modelling of design flood events as previously used and recommended by independent expert panel reviews [1]; and

(d) Most importantly, the analysis has been prepared in isolation of the Wivenhoe and Somerset Dams Optimisation (WS DOS) study that is underway, and these works need to be completed before definitive conclusions of event frequency and the ARI 100 year flood line are reached.

The WS DOS study will carry out flood frequency analysis for several gauges, will update hydrologic and hydraulic models and will use these models to conduct simulation modelling of design flood events as a cross-check on the frequency analyses. In order to update the hydrologic and hydraulic models, new bathymetric survey is required of the river system, as significant scour and siltation occurred during the January 2011 flood event. WMA Water has relied on the existing MIKE11 hydraulic model to translate flood levels for the ARI 100 year event along the river despite significant discrepancies between actual and predicted flood levels for the January 2011 event having already been identified (WMA Water's July 2011 'Review of Hydraulic Modelling' Report).

Given that flood frequency analysis has not been carried out for other gauges and that simulation modelling of design flood events as a cross-check has not been completed, there is a high level of uncertainty with the conclusions drawn because additional work could affect the accuracy of conclusions.

On 13 October 2011 the Commission provided Ipswich City Council with a Report by WMA Water on Ipswich Flood Frequency Analysis. This report does not address the Ipswich Flood Frequency Analysis Report. However, WMA Water has made comment on flooding from Brisbane River in

[1] Independent Review Panel 'Review of Brisbane River Flood Study, to Brisbane City Council' September 2003, and 'Joint Flood Taskforce Report', to Brisbane City Council, March 2011

Redbank and Goodna, within Ipswich City and therefore we provide comment on this aspect of the Brisbane Frequency Report.

2 GENERAL REVIEW

My general review comments regarding the Brisbane Frequency Report are as follows:

1 The results of the frequency analysis are related only to Brisbane River flooding in Brisbane City and are specifically related to the Port Office Gauge. As such, it is of relevance to sections of Ipswich that are immediately adjacent to the Brisbane River. To translate the Port Office Gauge findings to Redbank and Goodna, WMA Water has relied on the existing MIKE11 hydrodynamic model, which is recognised from their July 2011 'Review of Hydraulic Modelling' Report to have a number of limitations and inaccuracies (refer Chapter 4).

2 WMA Water's Flood frequency analysis is based on the Port Office gauge in Brisbane. Whilst this gauge has over 170 years of record, there are many reasons why the use of this gauge has associated with it a considerable degree of uncertainty in results (refer paragraph 113 of the Brisbane Frequency Report). These include river changes including dredging, the extent and timing of dredging, removal of bars, and the construction of Somerset and Wivenhoe Dams. So whilst WMA Water concludes that the January 2011 flood event was a 1 in 120 year event for the current situation, the uncertainty based on 90% confidence limits (e.g. Figure 9 of the Brisbane Frequency Report) means that it could have been anything between a 50 to 100 year event, up to greater than a 200 year event but less than a 500 year event. This is before uncertainties of changes in river bathymetry and changes in catchment land use are taken into account.

3 River dredging was and still is carried out for shipping navigation. Both capital dredging to increase draft, and regular maintenance dredging has been carried out progressively, since the 1860's. In my view, it would be impossible to determine the exact river bathymetry at the commencement of specific flood events, because of the limited bathymetric survey available, and because the extent of siltation is unknown. In relation to the effects of both Somerset and Wivenhoe Dams on flood mitigation, it is not clear how the analysis has been adjusted for the flood mitigation effects for the period when Somerset Dam was in place, prior to the construction of Wivenhoe Dam.

4 The relevance of the results to Ipswich City are dependent upon the MIKE11 model used, which has considerable uncertainty (and is unsuitable for reliable level and flow predictions upstream of Moggill River). I discussed these limitations in Chapter 4 of my report included as Appendix B and these are summarised as follows. WMA Water also has identified shortcomings of the MIKE11 model in their July 2011 Report (Chapter 4).

- Floodplain representation with artificial vertical wall sections is poor in places.
- Channel roughness representing vegetation cover is abnormally high and unrealistic.
- Less than desirable model calibration to historic events.
- Lumping of catchments upstream of Mt Crosby.
- Over-prediction of Brisbane River flood levels.
- Under-prediction by one metre of flood levels in Ipswich CBD.
- Inadequate modelling of the Bremer River.

5 Paragraph 146 of the Brisbane Frequency Report says that the 1% AEP (Q100) flood line is approximately 3 metres higher at Moggill and one metre higher at the Port Office than previous estimates. WMA Water has not commented on many anomalies between the previous ARI 100 year flood line and the January 2011 flood slope. Whilst in many places the January 2011 level was higher than the previous ARI 100 year flood level, there are some places where it is either much higher or lower, which may be due to changes in bathymetry, effects of new bridges and effect of tide and tailwater conditions. These discrepancies are discussed in Brisbane City Council's 'Joint Flood Taskforce Report' of March 2011 (Section 2.2.3, 4th dot point). Changes to river bathymetry may have been substantial, and require re-survey and revision of the hydraulic model to provide accurate predictions along the river. This work is to be undertaken as part of WS DOS study. The MIKE11 model also uses bathymetry from a variety of sources all of which predate the January 2011 flood which changed the river bathymetry. The report would benefit from a comment on these factors.

6 Paragraph 147 of the Brisbane Frequency Report says that there is major uncertainty in the rating relationship at the Port Office gauge. This conclusion is supported, in terms of uncertainty. In relation to improving the Port Office rating, it is difficult to see how the uncertainties discussed above can be significantly reduced, but a review as proposed is supported.

7 WMA Water relies on statistical analysis of a single gauge at the Brisbane Port Office to predict the frequency of the January 2011 flood event, and as the basis of ARI 100 year flood predictions along the river including at Redbank and Goodna.

Previous estimates were based on two methods being statistical flood frequency analysis of a number of gauges, and simulation modelling of design flood events (refer Independent Review Panel Report, 2003 and Joint Flood Taskforce Report, March 2011). In particular, Savages Crossing Gauge data was the focus of flood frequency analysis.

In order to reduce the considerable uncertainty in WMA Water's predictions, further work is still necessary, both for additional gauge flood frequency analysis, and for simulation modelling of design flood events. The current WS DOS study is intending to carry out this additional work, and will address a number of specific recommendations from the Floods Commission Interim Report recommendation.

In my opinion, it is not appropriate to rely upon the findings of WMA Water unless they are verified and supported by the further additional work needed. WMA Water's conclusion in paragraph 146 should be qualified in terms of uncertainty bands. An example of an error on existing ARI 100 year flood level estimates is in the upper river reaches, including Moggill, Redbank and Goodna (refer Chapter 3 of this report), with previous level estimates being over a metre higher.

WMA Water has also not stated why they do not agree with the detailed reviews by recognised experts in the 2003 Independent Review Panel Report, or in the March 2011 Joint Flood Taskforce Report.

8 WMA Water has previously concluded in their report to the Commission in May 2011 that there should not be reliance on a single design hydrograph to determine flood frequency 'but rather a probabilistic framework that incorporates the natural variability of key characteristics from

observed storms/floods'. This suggests that WMA Water supports the need for two methods of analysis.

3 REVIEW OF ASSESSMENTS OF REDBANK AND GOODNA BRISBANE RIVER FLOODING

Because of all the uncertainties identified, the following is an example of differences identified between WMA's assessments and information from Ipswich City Council.

In Figure 12 of the Brisbane Frequency Report, WMA Water shows what they refer to as 'Existing 1% AEP Design Level – 2011 Review Panel' levels for two properties at 13 Bridge Street, Redbank and on the corner of Ryan Street and Woogaroo Street, Goodna, which are located in the area of the City of Ipswich. It is not clear from the Report how the plotted levels have been derived as the March 2011 Joint Flood Taskforce Report (which appears to have been used to identify the existing 1% AEP Design Level) does not include reporting at these locations. It is probable that interpolation has been used, but this should be explained more clearly in the Report.

Figure 12 of the Brisbane Frequency Report also shows surveyed and MIKE11 estimated levels for the January 2011 flood event for these sites.

Figure 13 shows the Figure 12 data plus WMA Water's estimate of the 1% AEP design level based on the WMA Water flood frequency analysis at the Brisbane Port Office gauge.

Advice from Ipswich City Council is as follows:

1. 13 Bridge Street, Redbank

   1% AEP: 15.33m

   2011 flood level: 16.8m

   1974 flood level: 19.22m

2. 20 Woogaroo Street (Cnr Ryan Street and Woogaroo Street), Goodna

   1% AEP: 14.78m

   2011 flood level: 16.92m

   1974 flood level: 17.67m

Extracts of Council's flood maps which supplement this data are included in Appendix C.

Set out below are two Tables which compare the levels determined by WMA Water and referred in the Brisbane Frequency Report for the Ipswich properties to the levels that have been advised by the Council.

13 Bridge Street, Redbank

Flood Event	Ipswich City Council Flood Level (mAHD)	WMA Water * Plotted flood level (mAHD)
1% AEP	15.33	14
Recorded 2011 peak flood level	16.8	17.55

*** Estimated from Figure 13, WMA Water, September 2011**

20 Woogaroo Street, Goodna

Flood Event	Ipswich City Council Flood Level (mAHD)	WMA Water * Plotted flood level (mAHD)
1% AEP	14.78	13.2
Recorded 2011 peak flood level	16.92	16.85

*** Estimated from Figure 13, WMA Water, September 2011**

From the above, it can be concluded that:

1. The existing 1% AEP Design Level used by WMA Water for the two Ipswich properties is over one metre lower than the 1% AEP design levels as advised by Ipswich City Council.

2. The 2011 surveyed flood level for the Bridge Street property do not accord with Ipswich City Council's reported level.

3. Given the 90% uncertainty range in the determination of the 1% AEP flood estimate by WMA Water, too great a reliance may be placed on the WMA Water 1% AEP design level line. I am strongly of the view that further review ought to be carried out before any conclusions as to the accuracy of the existing Council 1% AEP design levels are reached. In particular, the WS DOS study needs to be completed and the results considered.

4. Traditionally, the 1% AEP flood line has relied on the results of combined hydrologic and dynamic hydraulic flood modelling using historic and theoretical design storms rather than placing sole reliance on a statistical analysis of a single gauge. Hence, the WMA Water 1% AEP flood estimates require testing against refined hydrologic and hydraulic modelling results. The refinement of these models has already been recommended by the Commission.

4 LIMITATIONS OF THE REVIEW

This review is based solely on the published report and I have not had the opportunity to review the data relied upon.

The assumptions made by WMA Water for the actual state of the river, in terms of bathymetry on an annual basis over 170 years, and also how the impact of first Somerset Dam and then Somerset and Wivenhoe Dams were addressed, is critical to the conclusions made.

APPENDIX A: CURRICULUM VITAE OF NEIL IAN COLLINS

BMT WBM

A part of BMT in Energy and Environment

Neil Ian Collins

Position
Principal Hydraulic Engineer – Expert Services

Years of Experience
32

Professional Affiliations
PIANC
NPER-3
RPEQ

Qualifications
Master of Science Engineering, University of Queensland

Bachelor of Engineering (Civil) University of Queensland

Recent Employment Profile
2010 to Present
BMT WBM Pty Ltd – *Principal Hydraulic Engineer - Expert Services*

2007 to 2010
Gilbert & Sutherland Pty Ltd – *Principal Hydraulic and Water Resources Engineer*

2004 to 2007
Cardno Lawson Treloar – *Director*, *Queensland Manager*

1993 to 2004
Lawson Treloar - *Director*

Career Overview

Neil is BMT WBM's Principal Hydraulic Engineer; part of the Expert Services team, based in the Brisbane office. He has 31 years experience and is an acknowledged expert in the P+E, Land Court and Supreme Court of Queensland in flooding, water quality and coastal processes. He was also the independent hydraulic expert to the Queensland Government for the North Bank project. Neil has worked on major infrastructure projects as an Hydraulic Specialist including Sydney Third Runway, Sydney Harbour Tunnel, Gateway Bridge and Arterial and several coal ports in Queensland and in Indonesia, power stations in Queensland and Thailand, hydro-electric schemes in PNG and port dredging management at Cairns, Townsville, Weipa and Mackay.

Areas of Expertise

Hydraulics, Hydrology and Water Resources

Provision of Expert Witness Services in Flooding, Stormwater, Quality Control and Coastal Engineering

Summary of Major Projects

- Lauderdale Quay, Hobart – Coastal Hydraulics, Water Sediment Quality for IIS on a Major Marina Residential Reclamation Project.
- Brisbane Airport - International Terminal Drainage Design.
- Sydney Harbour Tunnel - Hydraulics Engineer for Immersed Tube Tow and Placement.
- Sydney Third Runway - Hydraulic Model Testing, Sea Wall Design and Environmental Management.
- Gateway Arterial - South East Freeway to Lytton Road - Civil and Hydraulic Design Manager.
- Gateway Bridge - Hydraulics and Approaches Services Relocations.
- Trade Coast Central - Flooding Review for BCC.
- Oak Flats to Yallah RTA Freeway Hydraulics.
- Kedron Brook Flood Impacts due to Airtrain.
- Tully and Murray River Floodplains Hydraulic Analysis and Modelling, for Drainage Scheme Design includes Large MIKE11 Modelling, with over 40 Bridges and 200 Channels.
- Expert Review - Mossman Daintree Road, Saltwater Creek Crossing: Independent Review of the Hydraulic Design of two Large Bridges.
- Hydraulic Design of Rock Armouring Works for the Barron River Bend at Cairns Airport.
- Eastern Corridor Study - Hydraulics and Hydrology investigation for Department of Transport.
- Relief Drainage Scheme Design for Albion Windsor Area Brisbane (Capital cost $2 million).
- Tarong Power Station - Design of Earthfill Dam (max. 23m height), Ash trench, Stormwater Diversion Channels.

Professional History

BMT WBM Pty Ltd

Principal Hydraulic Engineer providing expert witness services in flooding, stormwater, quality control and coastal engineering.

2010-2011: Over 25 appeals completed or still in progress
2010-2011: Flooding Commission of Inquiry – Technical expert for LGAQ and Ipswich City Council
2010-2011: Cairns Airport – Review of Airport Flood Immunity and Risk

Gilbert & Sutherland Pty Ltd

Wet ‘n’ Wild, Sunshine Coast – site and soil assessments, input to and review of AGE groundwater assessment, conceptual stormwater quality assessment, hydraulic and flooding assessments including yield, medli modelling for onsite and input to S&B water balance, contamination investigation.

- Stockland, Twin Waters – Flooding Assessment
- Mackay Boat Harbour – Wave Investigation
- Bourton Road, Alkira – Flooding and Stormwater Management Plan
- The Glades, Robina – Water Quality Compliance and Inspection Report

Expert Services:
2007: Truloff Pty Ltd -v- Gold Coast City Council
2008: Jimboomba Turf Co Pty Ltd -v- Logan City Council
2008: Lechaim -v- Gold Coast City Council
2008: Sunnygold International Pty Ltd -v- Brisbane City Council
2008: Bon Accord -v- Brisbane City Council
2008: Blue Eagle -v- Beaudesert Shire Council
2008: Brian Paddison -v- Redland Bay Shire Council
2008: Monarch Nominees -v- Brisbane City Council
2008: Kunda Park Pty Ltd -v- Maroochy Shire Council
2008: Owl Projects & Hyder -v- Gold Coast City Council
2008: Port Pacific Estates Pty Ltd -v- Cairns Regional Council
2008: Joanne Shepherd & Ors -v- Brisbane City Council
2009: Lenthalls Dam, Hervey Bay
2009: Testarossa -v- Brisbane City Council
2009: Heritage Properties & Ausbuild -v- Redland City Council
2009: Samantha Skippen -v- Miriam Vale Shire Council
2009: Anthony Wan Pty Ltd -v- Brisbane City Council
2010: Over 25 appeals in progress this year

Professional History (cont)

Cardno Lawson Treloar
Sovereign Waters, Wellington Point - flooding, tidal exchange and water quality management.

EMP Water Quality Management Plan preparation and site stormwater management, including hydrodynamic, advection/ dispersion and catchment pollutant yield modelling for:
• Emerald Lakes Project, Carrara
• Glenwood Estate, Mudgeeraba
• 'The Glades' (Greg Norman Design Course), at Robina
• Sovereign Waters, Wellington Point
• Pacific Palisades, Gavin
• Freshwater Valley Estate, Cairns
• Carrara Golf Course Re-development, Carrara
• The Broadwater Development, Mudgeeraba
• Over a Dozen Major Residential Development Projects.

• Full Two-dimensional (MIKE 21) Floodplain Modelling for Cairns Airport Inundation, Nerang River Floodplain and Martins Creek, Maroochydore.
• Noosa River System Flood Study: Includes full G.I.S. Interfacing, Colour Inundation Plan Production and MIKE11 Modelling.
• Detention Basin Design for Development Consulting, Calamvale, Brisbane: Hydrologic and Hydraulic Design using RAFTS.
• Hydraulic and Water Quality Design, Lucinda Drive Main Drain, Port of Brisbane, including Catchment Pollutant Runoff Management.
• Moreton Bay College Flood Investigation: MIKE11 Analysis of Flooding, Including Culvert and Channel Diversion Options.
• Input on EIS Report on Water Quality for Freshwater Valley Development, including EMP.
• Townsville Port Road and Rail Access Study - Hydraulics.
• Freshwater Creek Flooding, for Main Roads, included Bridge and Culvert Sizing and Positioning of Channel Training Works. (RORB/RUBICON).
• Mountain Creek Flooding Investigation Examination of 1992 Floods using detailed Hydrologic/Hydraulic Modelling and Design of Mitigation Works.

Expert Services:
2004: T.M. Burke Appeal
2004: East Point Mackay
2004: Dore Appeal
2004: 900 Hamilton Road, McDowall
2004: Milton Tennis Centre
2005: P&E Appeal Mount Samonsvale
2005: BCC & George Pasucci
2005: P&E Appeal 48 Comley Street Sunnybank
2005: P&E Appeal 398 Wondall Road, Tingalpa
2005: Cabbage Tree Creek Appeal
2006: 35 Suscatand Street, Rocklea Appeal
2006: Leong - v- Redland Shire Council Appeal
2006: Barry Hilson & Bach Pty Ltd - v- GCCC Appeal
2006: 57 Longhill Road Appeal
2006: 699 Bargara Road Appeal
2006: Chevellum Road Appeal
2006: 10 Karridawn Street, Nudgee Appeal
2006: Australian Hardboards Limited Appeal
2006: Dell Road and Hawkin Drive, St Lucia Appeal
2006: 106 Munro Street, Auchenflower Appeal
2006: 10 Adsett Road, P&E Appeal
2006: Saunders Creek Appeal
2006: 64, 70 & 74 Washington Avenue, Tingalpa

Professional History (cont)

Lawson Treloar

- Coastal Data Gathering and Analysis for Projects in Bali, Lombok and Malaysia.

- Pandorah Gas Project, Gulf of Papua. Neil was Responsible for Project Management of all Coastal and Oceanographic Aspects of this Project, including Preparation of the Relevant Components of EIS. This included Extreme Climate, Wind/Wave and Current Modelling.

Chevron PNG to Cape York Gas Pipeline Project, Gulf of Papua
Neil Carried out Project Management for all Coastal/Oceanographic Components of this Project, including:

- Wind/Wave Modelling
- Extremal Climate
- Bed Current Prediction
- Kumul Platform Berthing
- Endeavor Passage Landfall
- Wave, Current and Wind Data Gathering.

- Tidal Lagoon, Breakwater/Groynes, Water Quality and Quantity Management at Pecatu Indah Resort, Lombok.
- Marina and Reclamation, S-W Bali, (Putri Nyale) including Coastal Investigations and Hydraulic Design of Breakwaters and Revetments.
- Sediment Sampling and Monitoring Program for the Albatross Bay Dumpsite, Weipa, for Dept. of Transport. Job Manager for this Investigation which includes Monitoring of Movement of Material Following Dumping, and its Impact on Water Quality and Benthic Communities.
- Wellington Point Canal Estate - Coastal Hydraulic Investigation of Proposed Marina and Dredged Channel.
- Weipa, Embley Inlet Environmental Monitoring: Review and Planning for Long Term Monitoring and Assessment of Water Quality (for Comalco).
- Full 2D flooding assessments for Dept of Main Roads using MIKE 21 on Yarrabah, Cairns and Warrego Highway at Marburg.
- Current Profiling, Warrego River (1994).
- Sovereign Waters, Wellington Point - Flooding, Tidal Exchange and Water Quality Management.
- Responsible for all Flood and Water Quality aspects for several Gold Coast Projects, including Emerald Lakes, Nifsan's Glenwood and Broadlakes, including Lake, Wetland and EMP Design.
- Stream Diversion, including Sloping Drop Structure, Hydraulic Design, at 'Coops' Development, Brisbane (1993).
- Northumbria Lakes Estate, Flooding, Drainage, Gross Pollutant Trap and Trash Rack Modelling and Design (1994).
- Barron River Delta Prawn Farm I.A.S., including Flooding and Water Quality Monitoring and Modelling, using MIKE11 (1995).
- Hydraulic Manager for Cairns Airport Master Drainage Study, 1995, including Complex Hydrodynamic Flow and Catchment Management Analysis.

Expert Services:
1993: for Mulgrave Shire Council; Land Resumption Compensation Case in Land Court. (Flooding)
1993: for Mulgrave Shire Council; Development Appeal (Kamerunga Villas) in Planning and Environmental Court. (Flooding)
1994: for Pullenvale Residents Action Group, on Rezoning Appeal. (Flooding and Water Quality)
1994: for Development Consulting, on Rezoning Appeal for a Development with a Large Detention Basin at Calamvale. (Flooding and Drainage)
1994: for an Earthworks Contractor Regarding a Disputed Claim Over Levee Bank Construction at Mungindi. (Flooding)
1995: for a Developer on Bohle River Works. (Flooding and Water Quality)
1995: for Residents on Flooding, Murrumba Downs. (Flooding)
1995: for Residents on Flooding, Dayboro. (Flooding)

Connell Wagner

- Current Profiling, Warrego River (1994).
- Sovereign Waters, Wellington Point - Flooding, Tidal Exchange and Water Quality Management.
- Responsible for all Flood and Water Quality Aspects for several Gold Coast Projects, including Emerald Lakes, Nifsan's Glenwood and Broadlakes, including Lake, Wetland and EMP Design.
- Stream Diversion, including Sloping Drop Structure, Hydraulic Design, at 'Coops' Development, Brisbane (1993).
- Northumbria Lakes Estate, Flooding, Drainage, Gross Pollutant Trap and Trash Rack Modelling and Design (1994).
- Barron River Delta Prawn Farm I.A.S., including Flooding and Water Quality Monitoring and Modelling, using MIKE11 (1995).
- Hydraulic Manager for Cairns Airport Master Drainage Study, 1995, including Complex Hydrodynamic Flow and Catchment Management Analysis.
- Tarong Power Station. Design of earthfill dam (max. 23m height), Ash trench, Stormwater Diversion Channels.
- Callide B Power Station. Evaporation Ponds Simulation; Hydraulic Design and Stormwater Bypass Channel. Design of (25m) Ash Dam.
- Hay Point Multi-User Coal Export Facility. Design of Dams, Stormwater Drainage, Water Supply and General Civil.
- Townsville Container Terminal. Design of Stormwater Drainage and General Civil.
- Abbot Point Coal Terminal. Design of an Offshore Causeway.
- Subdivisional Design and Supervision, on over a dozen Projects.
- Bulk Sugar Terminal - Brisbane. Feasibility Studies, including Flooding.
- Gladstone Power Station. Ash Handling including Piping.
- Stanwell Power Station. Design Check on General Civil.
- Patrick Container Terminal - Port of Brisbane. Flooding and General Civil.

Expert Services:

1993: for Mulgrave Shire Council; Land Resumption Compensation Case in Land Court. (Flooding)

1993: for Mulgrave Shire Council; Development Appeal (Kamerunga Villas) in Planning and Environmental Court. (Flooding)

1994: for Pullenvale Residents Action Group, on Rezoning Appeal. (Flooding and Water Quality)

1994: for Development Consulting, on Rezoning Appeal for a Development with a Large Detention Basin at Calamvale. (Flooding and Drainage)

1994: for an Earthworks Contractor Regarding a Disputed Claim Over Levee Bank Construction at Mungindi. (Flooding)

1995: for a Developer on Bohle River Works. (Flooding and Water Quality)

1995: for Residents on Flooding, Murrumba Downs. (Flooding)

1995: for Residents on Flooding, Dayboro. (Flooding)

Expert Services for Phillips Fox; Caboolture Shopping Centre Extension Appeal in Planning and Environment Court. (Flooding)

Expert Services for Mulgrave Shire Council; Land Resumption Compensation Case in Land Court. (Flooding)

Expert Services for Mulgrave Shire Council; Development Appeal (Kamerunga Villas) in Planning and Environmental Court. (Flooding).

Papers/Publications

May 2007 QELA Conference Presentation – The Approval and Appeal Process in QLD and NSW, Experts view on soil and water issues.

Nov 2004 Publication - ‘Application of Australian Runoff Quality Draft Chapter 6 – A model approach’, Water Sensitive Urban Design Conference, 2004, Adelaide.

Jul 2004 ‘Integrated High Order Water Quality and Hydrodynamic Analysis', 8th National Conference on Hydraulics in Water Engineering, July 2004.

Nov 2002 Publication - ‘Hervey Bay Storm Surge’, 30th PIANC Congress, Sydney 2002.

Nov 2001 ‘The Use of Runoff Event Monitoring in Validating Sediment Control Measures’, 9th Annual Conference, International Erosion Control Association, Nov 2001.

Nov 2001 ‘Specialist 2D Modelling in Floodplains with Steep Hydraulic Gradients’, 6th Conference on Hydraulics in Civil Engineering, Nov 2001.

Mar 2001 ‘Planning Implications of New Technology in Floodplains’, RAPI Conference, Gold Coast, 2001.

Nov 1999 'Best Management Practices for Water Quality Control', and 'Zero Flooding Impact Assessments; the need for full two dimensional analysis', 8th International Conf. on Urban Stormwater Drainage, 1999.

Jul 1999 'Desktop Ship Simulation for a new Port Facility in The Gulf of Papua', Coasts and Port '99.
Mar 1997 ‘Implications of the Nifsan -v- G.C.C.C. ruling on floodplain hydraulics’, Qld Envir. Law Assoc., 1997.

Jul 1994 'What the Community Needs to Know – Approaches to Community Construction for Water Engineering Projects', I.E. Aust., Queensland Division, 1994.

Nov 1993 'Hydraulic Assessment of Floodplain Development: Case Studies', The Institute of Municipal Engineering, Goondiwindi, 1993.

Jul 1993 'Long Term Environmental Planning – Weipa Port Dredging', 11th Australasian Conf on Coastal and Ocean Engineering. Townsville, 1993.

Mar 1993 Integrated Hydrologic and Hydraulic Modelling', WATERCOMP '93. The Second Australasian Conference on Technical Computing.

Mar 1992 'Russell and Mulgrave River Catchment Management', Invited Guest speaker for Queensland River Trusts Conference, Cairns, 1992.

Nov 1990 'Recent Studies of Port Dredging and Offshore Spoil Dumps', Third Australasian Port and Harbour Conference 1990, IE Aust.

Aug 1990 'Barron River Airport Bend Study - An Exercise in Joint Numerical and Physical Modelling', Conf. on Hyd. in Civil Eng., 1990, IE Aust.

May 1989 'Comparison and Evaluation of Current Dynamic Flow Models', WATERCOMP '89. The First Australasian Conference on Technical Computing in the Water Industry, Melbourne, 1989.

May 1989 Publication - Dynamic Flow Modelling : Comparison and Evaluation of Current Models - final Report', ACADS International publication No. U-249, May 1989.

May 1988 'Comparison of Dynamic Flow Models', ACADS 2D Modelling of Flood Plains, Melbourne, 1988.

Jun 1985 'ACADS Project on Comparison of Unsteady Flow Models', ACADS workshop, Brisbane 1985.

APPENDIX B: TECHNICAL REVIEW OF HYDRAULIC MODELLING REPORTS BY WMA WATER (28 JULY 2011) AND SKM'S (5 AUGUST 2011) SPECIFICALLY AS THEY RELATE TO IPSWICH CITY COUNCIL – SUPPLEMENTARY REPORT PREPARED FOR IPSWICH CITY COUNCIL, SEPTEMBER 2011

A part of BMT in Energy and Environment

BMT WBM

QUEENSLAND FLOODS COMMISSION OF INQUIRY

Technical Review of Hydraulic Modelling Reports by WMA Water (28 July 2011) and SKM (5 August 2011), specifically as they relate to Ipswich City – Supplementary Report

prepared for

Ipswich City Council

R.B18414.002.00.doc
September 2011

QUEENSLAND FLOODS
COMMISSION OF INQUIRY

Technical Review of Hydraulic Modelling Reports by WMA Water (28 July 2011) and SKM (5 August 2011), specifically as they relate to Ipswich City – Supplementary Report

prepared for

Ipswich City Council

Offices

Brisbane
Denver
Mackay
Melbourne
Newcastle
Perth
Sydney
Vancouver

Prepared For:	Ipswich City Council
Prepared By:	BMT WBM Pty Ltd (Member of the BMT group of companies)

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1 PURPOSE OF THE REPORT

This report updates and replaces our 12 September 2011 report. The report corrects minor referencing and grammatical errors, and corrects some factual matters detected in review. Additional comment has also been added in relation to recommendations. The report has been prepared to provide a technical review of a report dated 28 July 2011 by Mark Babister of WMA Water for the Queensland Floods Commission of Inquiry, entitled ‘Review of Hydraulic Modelling’ (WMA, 2011). That report was prepared for the Commission to answer four specific questions related to the operation of Somerset and Wivenhoe Dams and the January 2011 floods.

In preparing his report, Mr Babister has relied on hydrologic and hydraulic models supplied by Sinclair Knight Merz (SKM), hence, we provide technical review comment on relevant components of the associated SKM report of August 2011 (SKM, 2011).

On 3 August 2011, the Floods Commission wrote to Ipswich City Council inviting comment from Council on the WMA Water and SKM reports. This review is intended to assist Ipswich City Council in providing a response to that request.

Hydrologic (URBS) and hydraulic (MIKE11) model files to allow us to complete a review were provided in late August 2011.

In conducting this review, we have focussed on issues of relevance to flooding the Ipswich City area.

To that end, additional information on flooding in Ipswich and of specific interest to Ipswich has been extracted from the previous work by Mr Babister, SEQ Water and SKM. A further hypothetical dam release option has also been analysed beyond those considered by SKM or Mr Babister, being a delayed release 12 to 24 hours later than actually occurred in the January 2011 floods. Such a strategy is not possible without enlarging the dam and was only carried out to determine whether a delay in releases from the dam could have a significant effect on flooding in Ipswich.

This report is **not** intended to be critical of actual releases made by SEQ Water, or of the hydrologic or hydraulic analysis by SEQ Water, SKM or WMA Water. It must be stressed that the presence of Wivenhoe and Somerset Dams prevented a much more severe flood occurring in the Brisbane River and in Ipswich City. Also, had the Strategy W4 not occurred when it did, a much larger flood could have also occurred, due to fuse plug spillways being activated.

The aim of this report is that, by focussing on Ipswich City flooding and results of the recent analysis work post the floods, a greater understanding of what affects flooding in Ipswich, and on what flood modelling predictions are based, may be achieved. This should assist in future reviews of the flood warning and flood management systems and approach, within the tight limitations of actual dam flood storage capacity and capability and operating rules. Flood warning, including the interpretation of forecasts into predicted impacts and appropriate responses, is of major importance to Ipswich City Council.

Limitations in the flood modelling tools have already been identified by others, and this report looks closely at whether additional limitations in relation to Bremer River flood predictions can be identified.

2 SPECIFIC QUESTIONS BY THE COMMISSION CONSIDERED BY WMA WATER

Section 2.1 of WMA Water's report details the scope of work, as reproduced below:

18 WMA Water'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 SEQ Water. 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 percent 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.

3 SPECIFIC ISSUES FOR IPSWICH CITY ARISING FROM THE REVIEW

The specific issues for Ipswich City, which we have considered in this Review, are as follows:

1. Timing of dam releases as it may affect flooding in Ipswich and Bremer River flooding.

2. How satisfactory is the calibration of the models are for Lockyer Creek and the Bremer River and therefore how reliable are the predictions in the WMA report for Ipswich City.

3. The benefits or disadvantages of alternative dam operating strategies for Ipswich, such as avoiding coincident Wivenhoe Dam release peaks with Bremer River peaks in Ipswich.

4. Why the release strategies for Wivenhoe Dam were not adjusted when assessing the 75% full supply level strategy and what effect such an adjustment would have had on flooding in Ipswich and Bremer River flooding.

4 REVIEW OF SKM'S MIKE11 AND URBS MODELS FROM PUBLISHED REPORTS

4.1 MIKE11 Model

Section 4 of WMA Water's report describes Version 1 and Version 2 by SKM, as well as the original 2005 SEQ Water MIKE11 model. Since Mr Babister completed his review, a Version 3 of the SKM report has been finalised and that version of the report is dated 5 August 2011.

We concur with Mr Babister's findings regarding serious shortcomings of the 2005 MIKE11 model used previously by SEQ Water, in Section 4.2 of his report.

We also agree with the shortcomings of the Version 1 model described by Mr Babister in Section 4.3 of his report. The prediction of 10m/s velocities in the Brisbane River is not credible and the model results are therefore unlikely to be reliable.

We have not reviewed Version 2 of the model in detail as we have focussed our review on Version 3 which is the most up to date version (SKM, 2011). Based on our review, the MIKE11 hydraulic model used has a number of assumptions and limitations, which could undermine the accuracy of the results for Ipswich City. These include:

- The floodplain has been represented using extended Sections. This may not always be appropriate;
- The extended Sections do not always extend far enough to capture the full extent of the January 2011 flood event, i.e. the model is represented by an artificial vertical wall at the end of Sections, which may cause an over prediction of water levels; and
- Channel roughness parameters (representation of channel and floodplain vegetation state) are abnormally high, which would again over predict water levels. This is more relevant to the upper reach (upstream of chainage 1,002,785) close to Mount Crosby. In the vicinity of the Bremer River (chainage 1,006,200) to Moreton Bay the roughness parameters are more within a normal range.

The calibration exercise of the MIKE11 is also less than desirable, for the following reasons:

- The model is used to create a rating curve at Mount Crosby. However, there is no discussion on how the rating curve was developed. The rating curve results in a good match for peak level at Mount Crosby, but flood levels are under predicted during the rise and ebb of the flood;
- The MIKE11 model does not attempt to individually represent tributary flows upstream of Mt Crosby (such as Lockyer Creek). All upstream inflows (excluding Wivenhoe Dam releases) are lumped together as a point source and inputted into the model immediately upstream of Mt Crosby. In version 3 of the SKM Joint Calibration Report, Figures 6-2 shows the peak flow at Mt Crosby as about 10,000m3/s with the routed Wivenhoe contribution of 6,000m3/s (suggesting that the contribution from the non-Wivenhoe flows are 4000 m3/s. Figure 6-3 shows a comparison of the difference between the two hydrographs in Figure 6-2 (red line) and the estimate from the URBS model (blue line) for the same interstation area. This suggests that the peak contribution of the catchment area between Wivenhoe and Mt Crosby, including the

Lockyer, is about 5,000 m3/s. The differences in the non Wivenhoe flow requires further investigation;

- If the model is generally over predicting water levels, the flow determined from the Mount Crosby rating curve will be overestimated and vice versa. However, the fact that the modelled and measured velocity at the Jindalee gauge are similar, gives the exercise credibility (with a small under prediction – velocities in the order of 2.6m/s);
- MIKE11 model underestimates the peak water level at the Ipswich CBD gauge by approximately 1m (Figure 6-1) when compared to the gauge level and URBS prediction. Whereas the peak flow is slightly overestimated by the URBS model but is reasonably matched by the MIKE11 flow. These discrepancies could be a result of inadequate schematisation of the model.

Most importantly for Ipswich, however, is the fact that there has been no revision to the model to correct issues identified above with the 2005 SEQ Water model and the Version 3 model, in relation to Bremer River and Lockyer Creek flooding. This leads to considerable uncertainty over the accuracy of flood wave timing and magnitude in the Ipswich area.

A major limitation to improving the flood modelling of the Bremer River and Lockyer Creek is the lack of suitably accurate survey data of the streams and floodplain.

In Section 1.3.9 (WMA, 2011) there is no quantification of the effects on flooding in Ipswich, which is required.

4.2 URBS Model

The URBS Model used by SKM was prepared by SEQ Water and has been calibrated to closely match the timing and flood heights that occurred during the January 2011 flood. From our review, a very close match for Ipswich City has been achieved with this model, and the URBS model, therefore, represents the most reliable flow, level and flood wave timing tool currently available.

5 DESKTOP REVIEW OF WMA WATER REPORT OF 28 JULY 2011 (WMA, 2011)

5.1 Introduction

Our review recognises that this exercise undertaken by WMA Water was undertaken in a short time frame.

An important consideration from the WMA Water analysis is that the calibrated model uses peak flow at Mount Crosby of 9,500m³/s. The Bremer confluence is 17km downstream of the Mount Crosby gauge. At Jindalee (which is a few kilometres upstream of Brisbane) the measured peak discharge is 10,000m³/s. This suggests that the Lockyer Creek flow is contributing a significant proportion of flow and that the flow in the Brisbane River is largely driven by flow from upstream of Mount Crosby, with lesser contribution from the Bremer River and other local catchments. More detailed analysis is required to determine the proportion of contributions from other local catchments.

We acknowledge that there was a significant rainfall event in the Bremer catchment which would have filled much of the Bremer floodplain before the Brisbane River peak from the Wivenhoe releases arrived. The Bremer River had a shallow gradient, which indicates that water was flowing down the Bremer River during the peak. Therefore it is not a purely backwater flood event in Ipswich. We believe that the flooding is due to a combination of high water levels in the Brisbane River and flows in the Bremer River on the receding limb of the flood. The inundation maps produced for the ICA Volume 3 report support this assumption (see Appendix A). They show the flood extent outline at 1800hours on 11 January 2011 (i.e. after cessation of the last rainfall event over the Bremer catchment) and the total maximum flood extent outline for the catchment up to 14 January 2011.

5.2 Contribution of Wivenhoe Dam Release Flows and Non-Wivenhoe Flows

With regards to the discussion on contribution of Wivenhoe Dam release flows and non-Wivenhoe flows on page 24 of WMA Water's report, the analysis does not isolate out the impacts of the Bremer River and Lockyer Creek. Our analysis of the URBS model suggests that the Lockyer Creek flow (4,796m^3/s) was a larger contributor to overall flow compared to the peak Bremer River flow (2,277m^3/s). This further supports the view that the flooding in Ipswich was contributed to by the backwater effect from the Brisbane River. The fact that in SKM's Case 3 a negative flow up the Bremer occurred (see paragraph 62 WMA, 2011) further supports the finding that Bremer River flood levels are significantly influenced by Brisbane River flooding. This has long been recognised by Ipswich City Council, SEQ Water and SKM. These figures require further analysis and review, , as the local catchment between Lockyer Creek and the Bremer River has an area of about 600km^2 and SEQ Water estimate that it generated a peak of about 3500m^3/s during the January 2011 flood.

5.3 Bremer River and Brisbane River Peaks

With regards to the comment in paragraph 65 of WMA Water's report that the Bremer River and Brisbane River peak occur at the same time, it is not reported how flows on the Bremer River were derived. The modelling carried out as part of this report (see Chapter 6) indicates that the peak water level from the Bremer only flow occurred at around 0440hours on 12 January 2011, whereas the peak in the Brisbane River occurred at the Moggill gauge at approximately 1400hours on 12 January. Figure 5-1 illustrates the timeline summarising rainfall, peak flood release from Wivenhoe Dam and associated warning information.

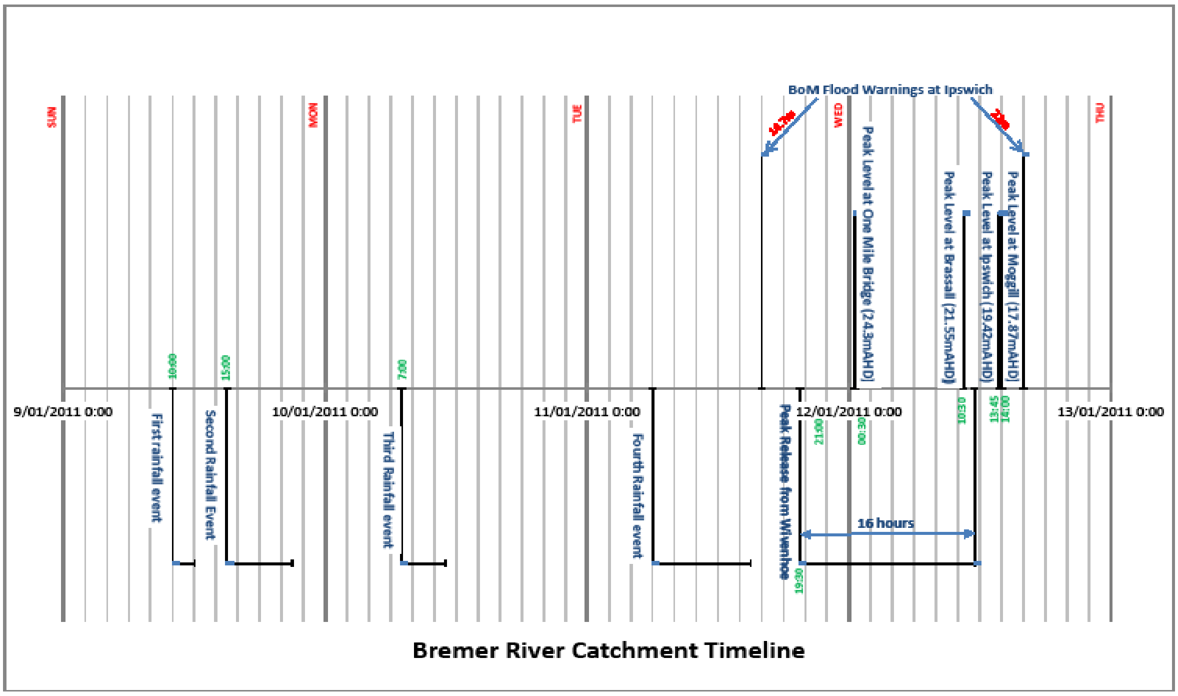

Figure 5-1 Bremer River Timeline

Peak flooding in Ipswich was significantly influenced by back-up flooding from the Brisbane River. Peak flooding in the Brisbane River at the confluence of the Bremer River was a result of runoff from the upper Brisbane River catchment system including major discharge from Wivenhoe Dam (which peaked at 1930 hours on 11 January 2011), combined with significant runoff from the Lockyer Creek catchment, with the peaks of dam discharge and Lockyer Creek coinciding. Hence, flooding in Ipswich was at least in part influenced by the timing of releases of water from Wivenhoe Dam but was exacerbated by the major flood event in Lockyer Creek, which was coincident with dam release flows.

There was also a significant flood event down the Bremer River due to Bremer River catchment rainfalls. The relative contributions to flooding in Ipswich from Brisbane River flooding, dam releases, Lockyer Creek flooding and Bremer River flooding are discussed in more detail in Chapter 7.

5.4 Option B Release Strategy

Paragraph 82 of WMA Water's report (Option B release strategy) does not address how this option may have affected the flood in Ipswich, including that the timing of peak releases would have changed, which could have reduced the coincidence of dam releases and Bremer River discharge to the Brisbane River. We consider this further in Section 6.3 of this report.

6 REVIEW OF MODELLING RESULTS USING SEQ WATER'S MIKE11 AND URBS MODELS

6.1 The January 2011 Event

The MIKE11 model estimates the peak water level at Ipswich to be 17.931mAHD. The recorded peak water level was 19.42mAHD as illustrated in Figure 6-1 below.

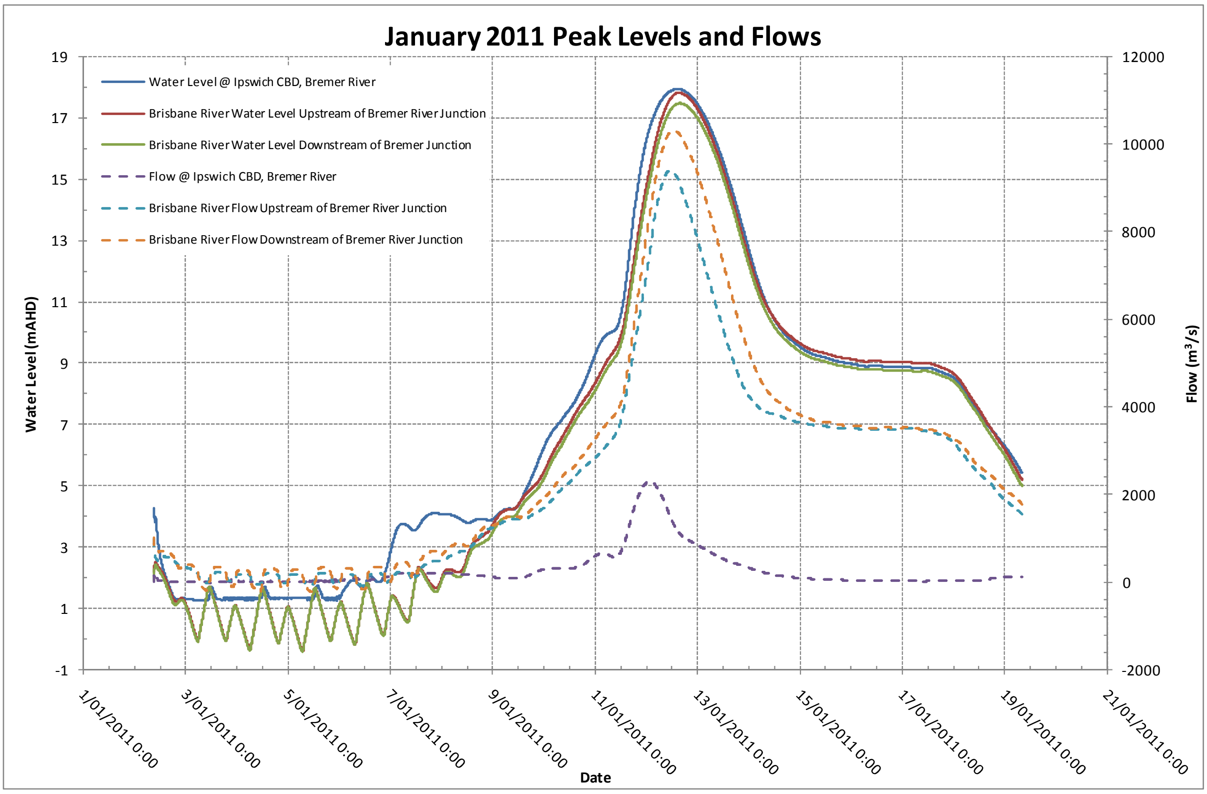

Figure 6-1 SKM Modelled Water Levels and Flows

URBS slightly overestimates the peak flow by approximately 150m^3/s (which is around 1% of the total flow). **The MIKE11 modelled peak flow of 2,300m^3/s equates to a design flow of between 20 and 50 year Average Recurrence Interval (ARI) (Sargent, 2006).** The actual measured peak flow at Ipswich was 2,277m^3/s.

6.2 Earlier Transition to Strategy W4 (Option A)

This scenario involves an earlier transition to Strategy W4 for the Wivenhoe Dam releases at 1600 hours 10 January 2011 instead of 0800 hours on 11 January 2011. Two cases were actually considered within this Option:

- Option A4 – to quickly escalate the outflows to match inflow and stabilise the level in the dam; and
- Option A5 – to increase outflow at a slower but steady rate to make more use of the remaining mitigation storage.

We have compared the output for both options against the January 2011 modelled results discussed in Section 6.1 of this report specifically for Ipswich City. Notwithstanding the inaccuracies described, the following results are presented.

Option 4A Scenario

Our analysis indicates the following for Ipswich Option A4 scenario as shown in Figure 6-2.

Results show that:

- Peak water level at Ipswich CBD increases over that predicted for the actual flood event (Figure 6-1) by 0.64m; and
- Peak flow at Ipswich CBD is 2,238m^3/s.

i.e. Option A4, if technically possible, could have worsened flooding in Ipswich by 0.64m.

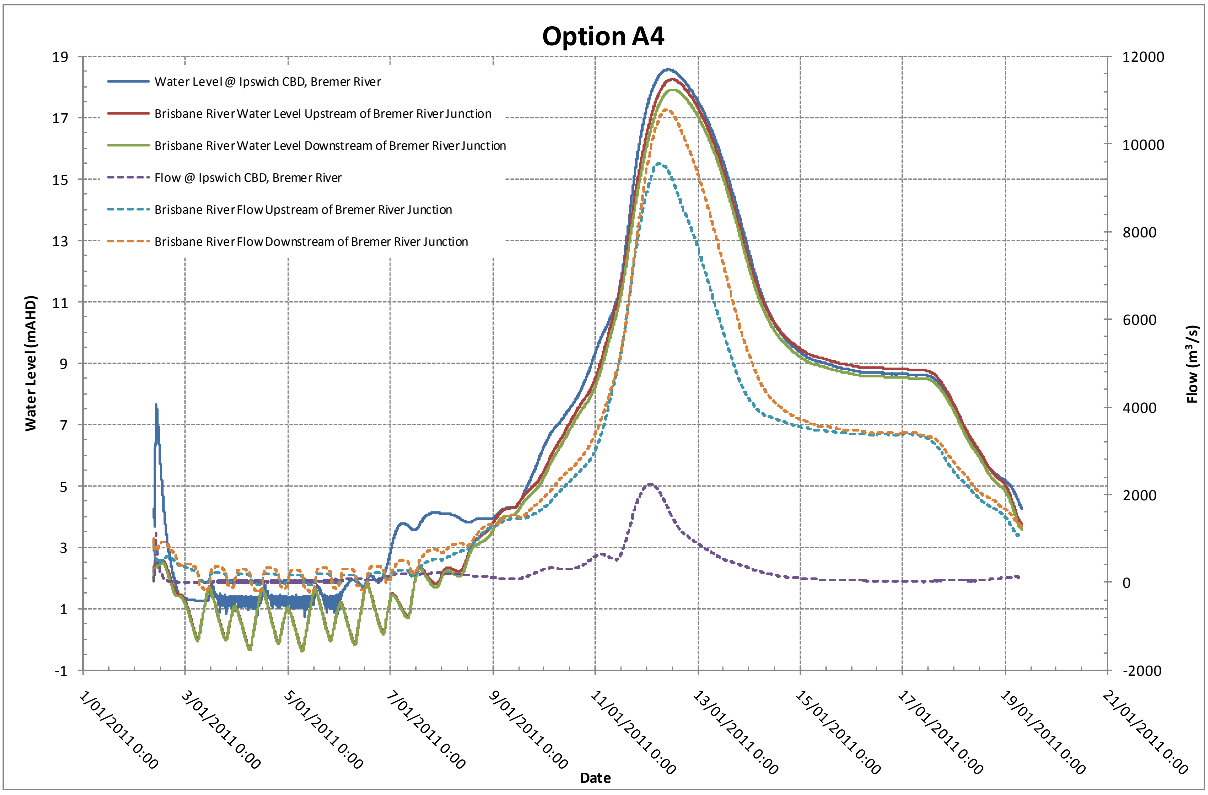

Figure 6-2 Option A4 Water Levels and Flow

Option 5A Scenario

Our analysis indicates the following for Ipswich Option A5 scenario as shown in Figure 6-3.

Results show that:

- Peak water level at Ipswich CDB decreases by 0.83m; and
- Peak flow at Ipswich CBD is 2,310m^3/s.

i.e. Option A5, if technically possible, could have reduced flooding in Ipswich by 0.83m.

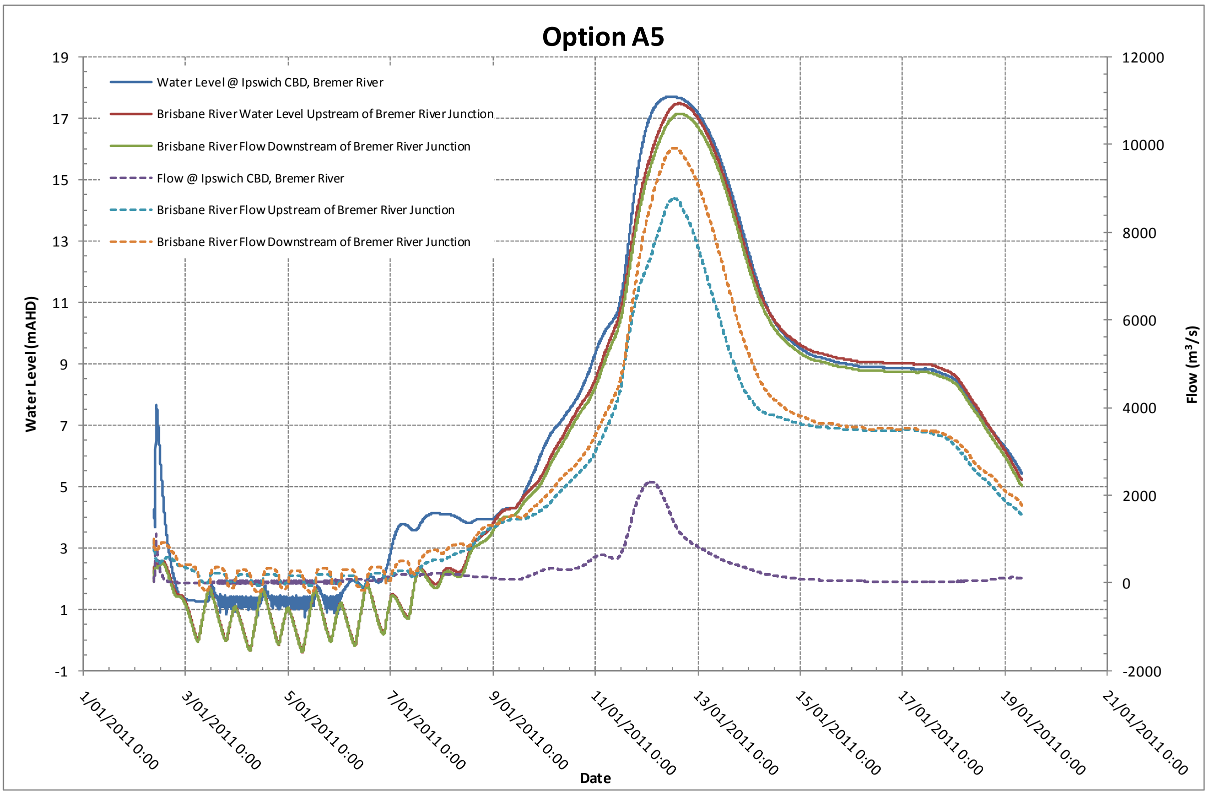

Figure 6-3 Option A5 Water Levels and Flow

For both Option A4 and A5, the impact of the earlier transition affects the peak water level in Ipswich but has little effect on the peak flow.

6.3 Wivenhoe Dam at 75% of Full Storage Level Prior to the Flood (Option B)

This scenario involves the storage level in Wivenhoe Dam being assumed to be at 75% of FSL prior to the onset of the flood but retaining current operating rules.

We have compared the output against the January 2011 modelled results discussed in Section 6.1 of this report. Notwithstanding the inaccuracies described, the following results are presented.

Our analysis indicates the following as shown in Figure 6-4.

Results show that:

- Peak water level at Ipswich CBD is decreased by 0.68m; and
- Peak flow at Ipswich CBD is 2,330m^3/s.

i.e. Option B could have reduced flooding in Ipswich by 0.68m.

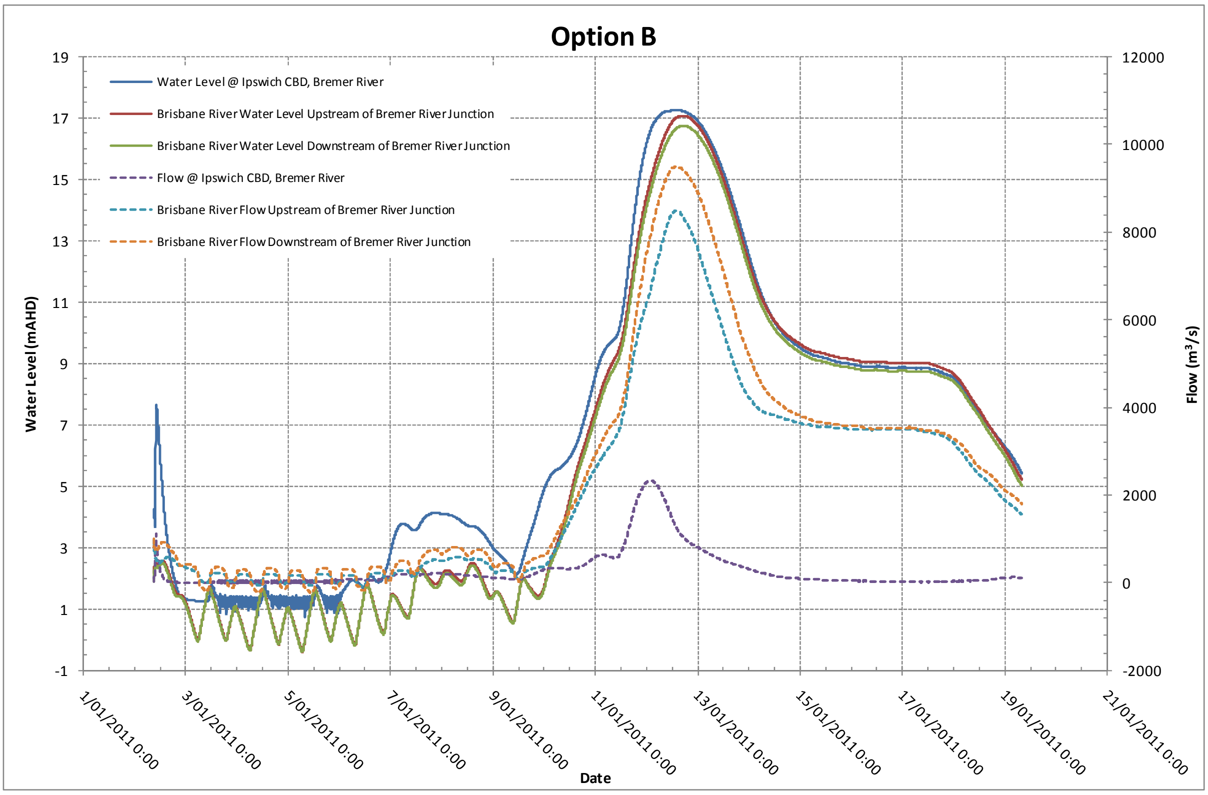

Figure 6-4 Option B Water Levels and Flow

6.4 Discharge at Upper Limit during Strategy W3 (Option C)

This scenario explores the effects of increasing flows immediately after entering Strategy W3 to the upper allowable limit.

We have compared the output against the January 2011 modelled results discussed in Section 6.1 of this report. Notwithstanding the inaccuracies described, the following results are presented.

Our analysis indicates the following as shown in Figure 6-5.

Results show that:

- Peak water level at Ipswich CBD decreased by 0.67m; and
- Peak flow at Ipswich CBD is 2,331m^3/s.

i.e. Option C, if technically possible, could have reduced flooding in Ipswich by 0.67m.

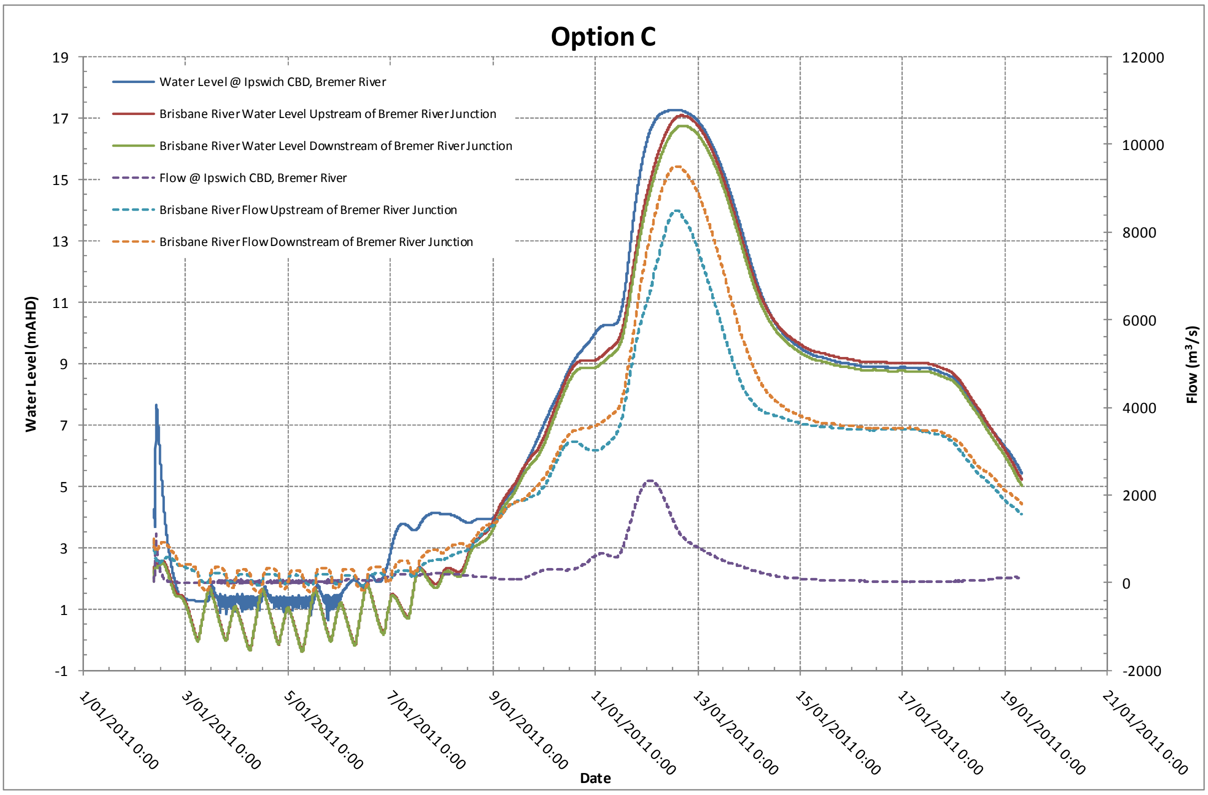

Figure 6-5 Option C Water Levels and Flow

6.5 WMA Water's 'Optimised' Release Strategy (Option D)

This scenario explores the effects of an optimum release strategy with full benefit of hindsight and ignoring restrictions from the Wivenhoe Dam Operating Manual on total flow at Moggill.

We have compared the output against the January 2011 modelled results discussed in Section 6.1 of this report. Notwithstanding these inaccuracies and assumptions described, the following results are presented.

Our analysis indicates the following as shown in Figure 6-6.

Results show that:

- Peak water level at Ipswich CBD decreases by 0.6m; and
- Peak flow at Ipswich CBD is 2,334m^3/s.

i.e. Option D, if technically feasible, could have reduced flooding in Ipswich by 0.6m, noting WMA Water's comments that, in reality, this option is not plausible.

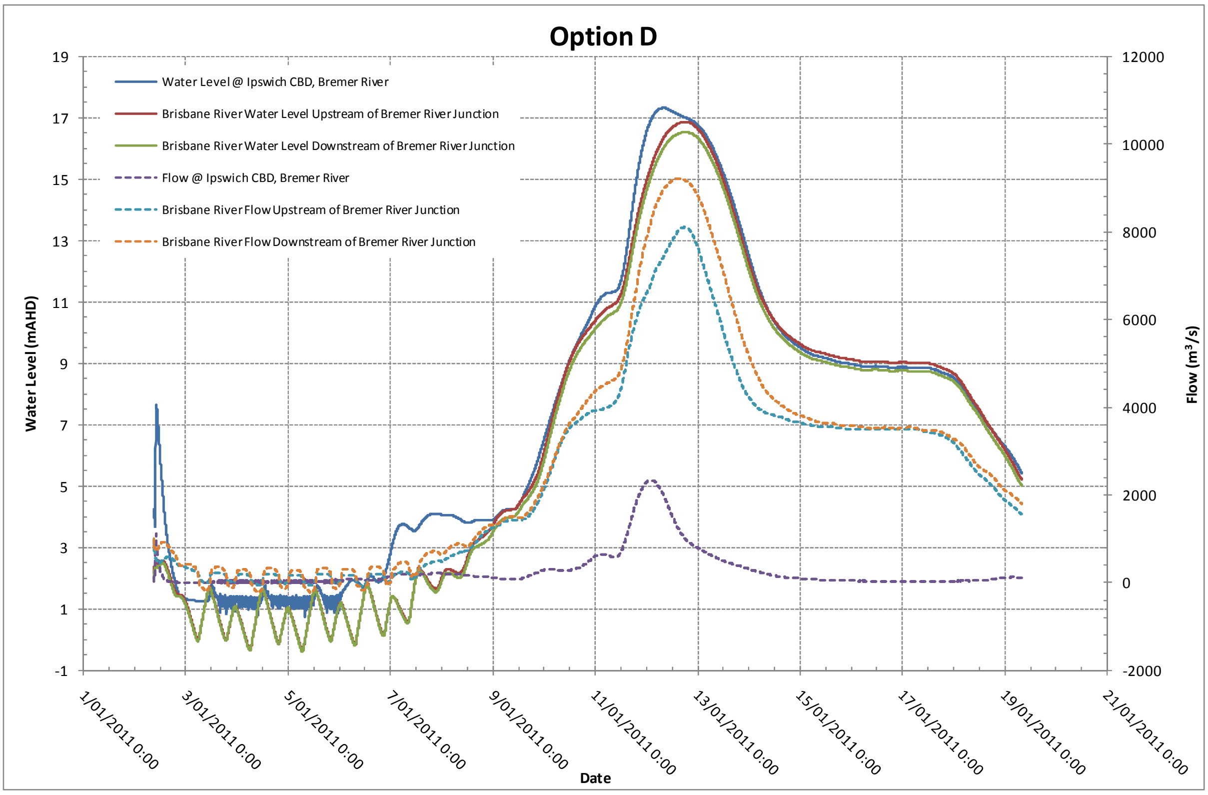

Figure 6-6 Option D Water Levels and Flow

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6.6 BMT WBM's Hypothetical Delayed Release Strategy (Option E) (enlarged dam option)

This scenario explores the effects of delaying the actual peak release from Wivenhoe Dam by 12 hours and by 24 hours. This option is not possible without increasing significantly available flood storage capacity in the dams through dam enlargement. It has only been assessed to allow a quantification of the interdependence of the timing of dam releases as they affect Ipswich and Bremer River flooding.

We have compared the output against the January 2011 modelled results discussed in Section 6.1 of this report. Notwithstanding the inaccuracies described, the following results are presented.

Our analysis indicates the following as shown in Figure 6-7.

Results show that:

- Peak water level at Ipswich CBD decrease by 0.1m for the 12 hour scenario; and
- Peak water level at Ipswich CBD decreases by 0.98m for the 24 hour scenario.

The longer the delay in dam release, the greater the reduction in flood levels in Ipswich.

A delay of 24 hours in Strategy W4 release would have reduced flooding in Ipswich by about a metre. This would not be possible without major dam flood storage compartment increases and dam raising.

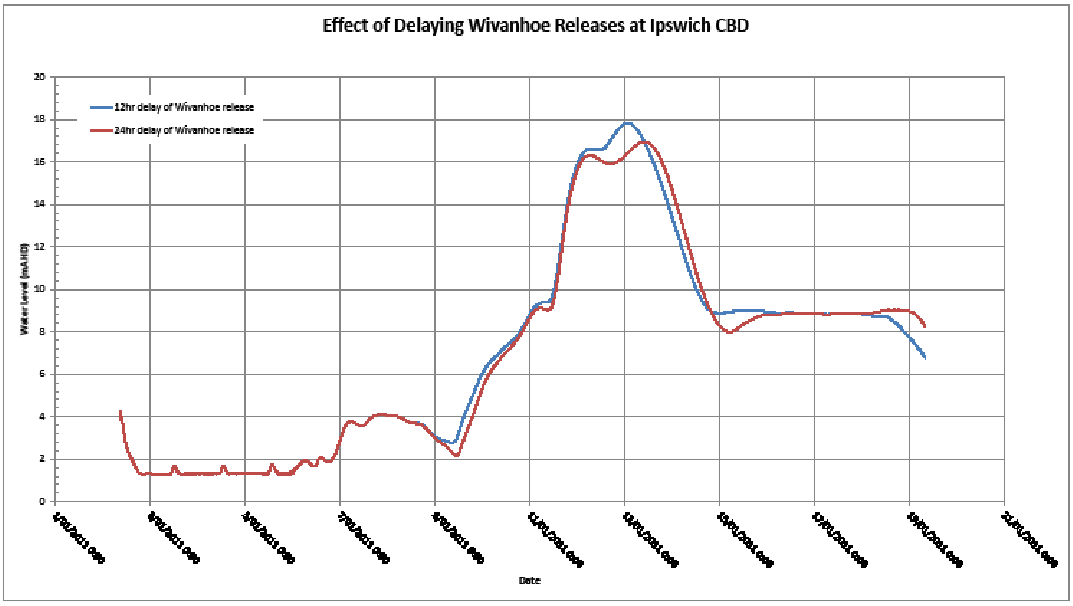

Figure 6-7 Delayed Release Scenarios

7 SUMMARY OF FINDINGS SPECIFIC TO IPSWICH CITY

7.1 Bremer River 2011 Flood Levels and Flows

A summary of model results is as follows in Table 7-1 and Table 7-2:

Table 7-1 Peak Water Levels

Location	Case1	Option A4	Option A5	Option B	Option C	Option D	Option E (12hr Delay)	Option E (24hr Delay)
	Peak Water Level (mAHD)	Peak Water Level (mAHD)	Peak Water Level (mAHD)	Peak Water Level (mAHD)	Peak Water Level (mAHD)	Peak Water Level (mAHD)	Peak Water Level (mAHD)	Peak Water Level (mAHD)
Ipswich CBD	17.931	18.566	17.699	17.250	17.259	17.325	17.836	16.949
Upstream of Bremer River Junction	17.816	18.246	17.484	17.075	17.081	16.862	17.799	16.967
Downstream of Bremer River Junction	17.472	17.914	17.149	16.741	16.748	16.534	17.585	16.749

Table 7-2 Peak Flows

Location	Case1	Option A4	Option A5	Option B	Option C	Option D	Option E (12hr Delay)	Option E (24hr Delay)
	Peak Flow (m^3/s)	Peak Flow (m^3/s)	Peak Flow (m^3/s)	Peak Flow (m^3/s)	Peak Flow (m^3/s)	Peak Flow (m^3/s)	Peak Flow (m^3/s)	Peak Flow (m^3/s)
Ipswich CBD	2299	2238	2310	2330	2331	2334	2361	2373
Upstream of Bremer River Junction	9394	9549	8784	8483	8484	8119	9305	8458
Downstream of Bremer River Junction	10304	10782	9914	9495	9501	9218	10464	9500

Table 7-3 summarises the changes in water levels and peak flows at Ipswich corresponding to the various options assessed within this report.

Table 7-3 Summary of Impacts at Ipswich

	Option A4	Option A5	Option B	Option C	Option D	Option E (12hr Delay)	Option E (24hr Delay)
Water Level (m)	-0.63	0.23	0.68	0.67	0.61	0.10	0.98
Peak Flow (m3/s)	61	-11	-31	-32	-35	-62	-74

7.2 What caused the January 2011 Flooding in Ipswich City?

Our analysis has clearly illustrated the effect of the Brisbane River flows at the Ipswich CBD. By isolating the effects of the Bremer River only flow, we have demonstrated that the peak Bremer River flow was between the ARI 20 year and 50 year ARI event based on design flood flow estimates by Sargent (2006). When the corresponding water level of 14.09mAHD derived for this design flow (in the absence of significant Brisbane River flooding) is compared to the Ipswich flood level classification from the BoM, it can be seen that this event is within the lower end of the major category.

When the effects of the flood in the Brisbane River, the releases from the Wivenhoe Dam and the flow from Lockyer Creek are taken into account, the measured peak was 19.25m AHD.

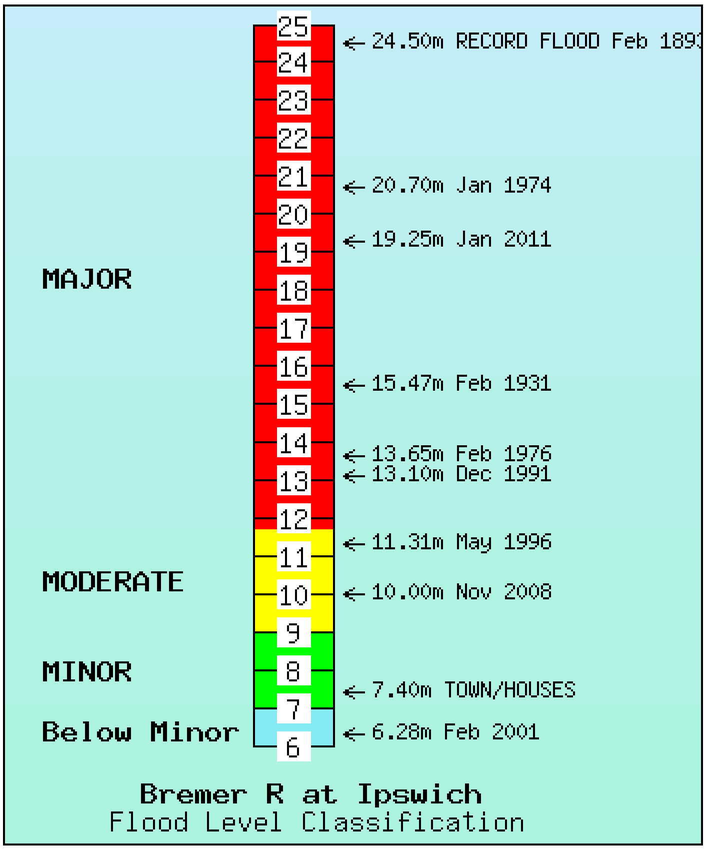

Figure 7-1 Flood Level Classification (BoM, 2011)

Flooding in Ipswich in the January 2011 event in the absence of major flooding (assumed Brisbane River level at Moggill of RL13.4m) in the Brisbane River, would have resulted in an ARI 20 to 50 year flood in the City, with water levels in the CBD reaching RL 14.1mAHD (using the work of Sargent, 2006).

Very large flooding occurred in the Brisbane River during this event, including in the Lockyer Creek catchment.

The sensitivity testing carried out by WMA Water and the delayed release strategy represented in this report, show that, under the scenarios tested in this report, at best, flood levels in Ipswich may have been able to have been able to have been reduced by about a metre for a January 2011 type event.

Further reductions in flooding in Ipswich, could be achieved with major capital works (e.g. bridge raising or dam raising) to allow further flexibility in either early or delayed releases.

Hence, we conclude that flooding in Ipswich was caused by a combination of a 20 to 50 year ARI event in the Bremer River, major to extreme rainfall events and flooding in Lockyer Creek and in the Brisbane River catchment, and the timing of the release strategy to move to Strategy W4. The rainfall event and its associated severity across the entire Brisbane and Bremer River catchments caused the flooding. Without Wivenhoe and Somerset Dams, flooding in Ipswich and Brisbane would have been many metres higher.

8 RECOMMENDATION OF ADDITIONAL WORK TO ASSIST IN ADDRESSING IPSWICH CITY COUNCIL'S CONCERNS

We recommend that additional work is required to address Ipswich City Council's concerns. These works include:

- A review and update of Lockyer Creek and Bremer River branches of the SKM MIKE11 model (acknowledging the limited survey data currently available for Lockyer Creek and Bremer River main channel);
- Review of calibration of the model for Bremer River reach and recalibration using the January 2011 flood event;
- Re-modelling of Option B with associated adjustments to operating rules to maintain 75% capacity - how these adjustments are made also needs to be subject to review. Such review should include, as a priority, not only reducing Wivenhoe Dam storage to 75% full storage level, but also adjusting the dam operating rules to attempt to maintain this level throughout the wet season;
- Further investigation of early or delayed release strategies independent of operating rules to assess the benefits and consequences of such strategies. Detailed and specific reporting for Ipswich for all options and strategies tested; and
- Consideration of what works or actions are required to allow early or delayed release strategies to be adopted, e.g. downstream bridge upgrades or dam storage compartment increases.

9 CONCLUSIONS

In conclusion, we find the following:

1 There is no specific reporting in the WMA Water report on the effects of the various strategies and options considered by WMA Water on the City of Ipswich and this report attempts to address those and other matters of concern for Ipswich.

2 Our analysis of the current models shows that flooding in Ipswich was influenced by Brisbane River flood back-up on the falling limb of Bremer River flooding.

3 Flooding in Ipswich in the January 2011 event in the absence of major flooding (assumed Brisbane River level at Moggill of RL 13.4m) in the Brisbane River, would have resulted in an ARI 20 to 50 year flood in the City, with water levels in the CBD reaching around RL 14.1mAHD (based on the work by Sargent, 2006). This requires further analysis and review to test the work of Sargent against current knowledgeWhen the effects of the flood in the Brisbane River, the releases from the Wivenhoe Dam and the flow from the Lockyer Creek are taken into account, the measured peak was 19.25mAHD.

4 We conclude that flooding in Ipswich was caused by a combination of a 20 to 50 year ARI event in the Bremer River, major to extreme rainfall events and flooding in Lockyer Creek and in the Brisbane River catchment, and the timing of the release strategy to move to Strategy W4. The rainfall event and its associated severity across the entire Brisbane and Bremer River catchments caused the flooding. Without Wivenhoe and Somerset Dams, flooding in Ipswich and Brisbane would have been many metres higher.

5 Despite the short time available and the limitations of available models, the modelling work by WMA Water has been useful is determining the potential significant positive benefits of adopting alternate strategies, including a 75% full supply strategy at the start of the next wet season. Flood levels reductions of up to 0.7m for a January 2011 event are predicted for Ipswich City. The strategy also needs to be expanded to determine the benefits of revised operating rules adjusted to maintain the 75% supply level, rather than maintaining the existing operating rules, that are designed around the 100% level.

6 The modelling work by WMA Water also shows that, subject to technical feasibility, options for gradual early release from the dam could reduce predicted flood levels in Ipswich by up to a metre for a January 2011 event.

7 By delaying the W4 release strategy by 24 hours, flood level reductions of about a metre for a January 2011 event may be feasible in Ipswich; however, this option would require a significant expansion of the dam flood storage compartment, and associated raising of the dam.

8 There are still significant shortcomings of the MIKE11 model in its representation of Lockyer Creek and the Bremer River that requires additional refinement and correction. It is acknowledged that insufficient survey data exists for Lockyer Creek and for the Bremer River main channel. Without accurate modelling of these two waterways, some uncertainty still exists over the timing of flood waves and coincidence of flood peaks.

9 Additional testing of the benefit of early or delayed release strategies, particularly on what benefit this could achieve to reduce peak flooding in Ipswich, ought to be carried out. This should include additional options for early release, beyond current operating rule restrictions, and

consideration of what additional dam storage may be required, to determine the benefits and dis-benefits to the entire system of such strategies.

10 Review of the costs and benefits of works required (e.g. bridge raising) to allow more flexibility in early and delayed release strategies is recommended.

11 All modelling work of alternative dam operation strategies tested to date as discussed in this report relate to the effect on a January 2011 type flood event. Any review of dam operating strategies, downstream capital works or increase in dam flood storage compartment require consideration of the consequences of these changes under a range of historic and design flood events, including those such as the 1974 flood where very different rainfall distribution patterns occurred, including events where the majority of the rainfall fell below the dam.

10 QUALIFICATIONS

This report must be read jointly with WMA Water's 24 July 2011 report and SKM's 5 August 2011 (Version 3) report. Terminology and definitions used are consistent with those of WMA Water.

No URBS input files were reviewed. No spreadsheets containing gate operations rating curves were provided or reviewed.

The review utilises the published reports quoted, and the SKM Version 3 MIKE11 model files, URBS results files, as provided by SEQ Water in August 2011, and model files as reported by WMA Water in their 5 July 2011 report.

The accuracy of this report is limited to the accuracy of this information and no independent verification of results from SEQ Water 's URBS modelling, SEQ Water's gate operational releases or from modelling work completed by WMA Water has been carried out.

11 REFERENCES

Sargent, 2006 Ipswich Rivers Flood Study Rationalisation Project Phase 3 Re-estimation of Design Flood Levels, Ipswich Rivers Improvement Trust

SKM, 2011 Joint Calibration of a Hydrologic and Hydrodynamic Model of the Lower Brisbane River, Technical Report, Version 3, August 2011

WMA, 2011 Queensland Floods Commission of Inquiry, Review of Hydraulic Modelling, Final Report

BOM, 2011 Flood Warning System for Bremer River to Ipswich

ICA, 2011 Flooding in Brisbane River Catchment January 2011 Volume 3 Flooding in Ipswich City LGA, Insurance Council of Australia Hydrology Panel

APPENDIX A: ICA 2011 INUNDATION MAPS

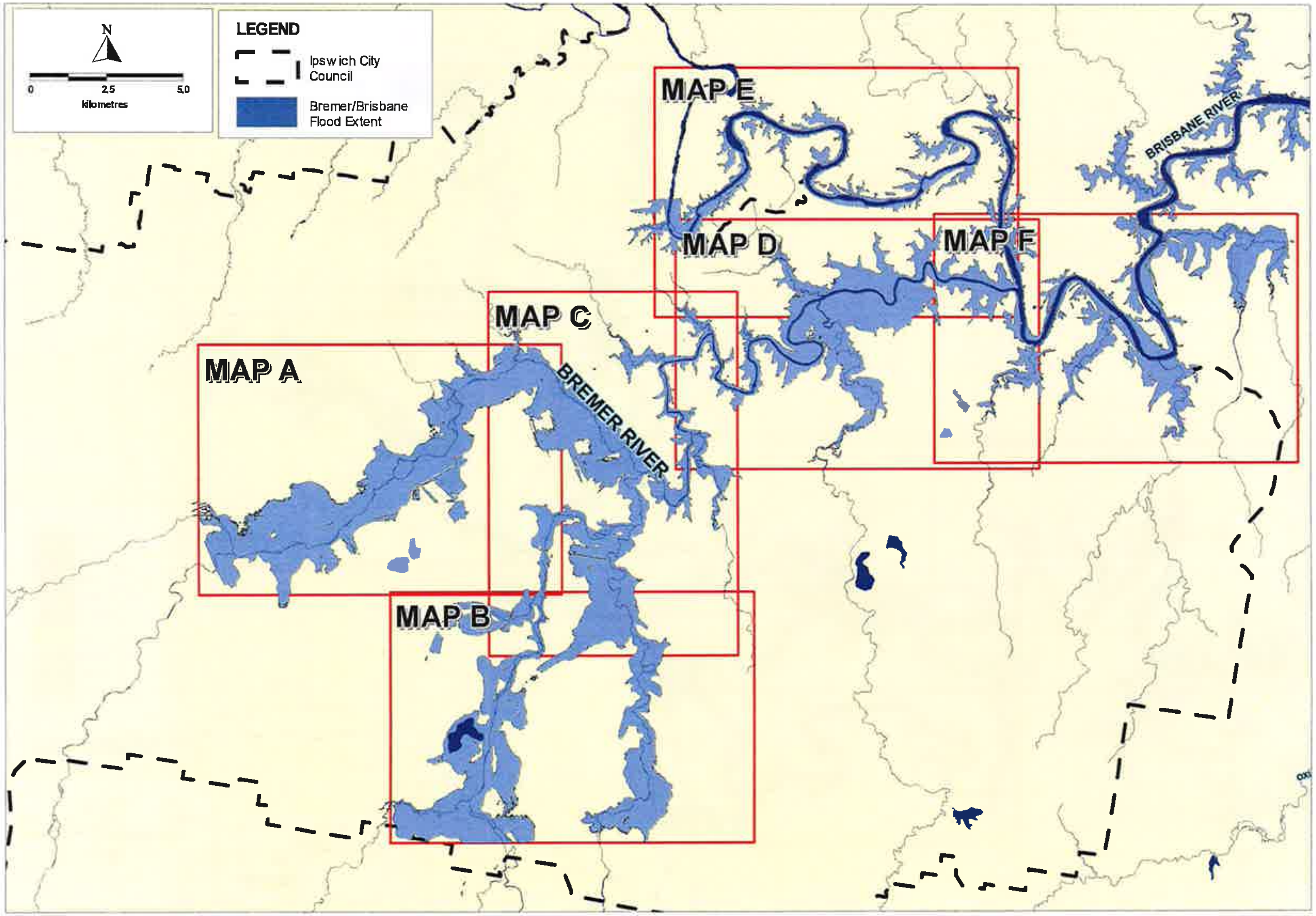

Figure 8.19 Maximum Extent of Inundation, Ipswich City LGA, 10-14 January 2011

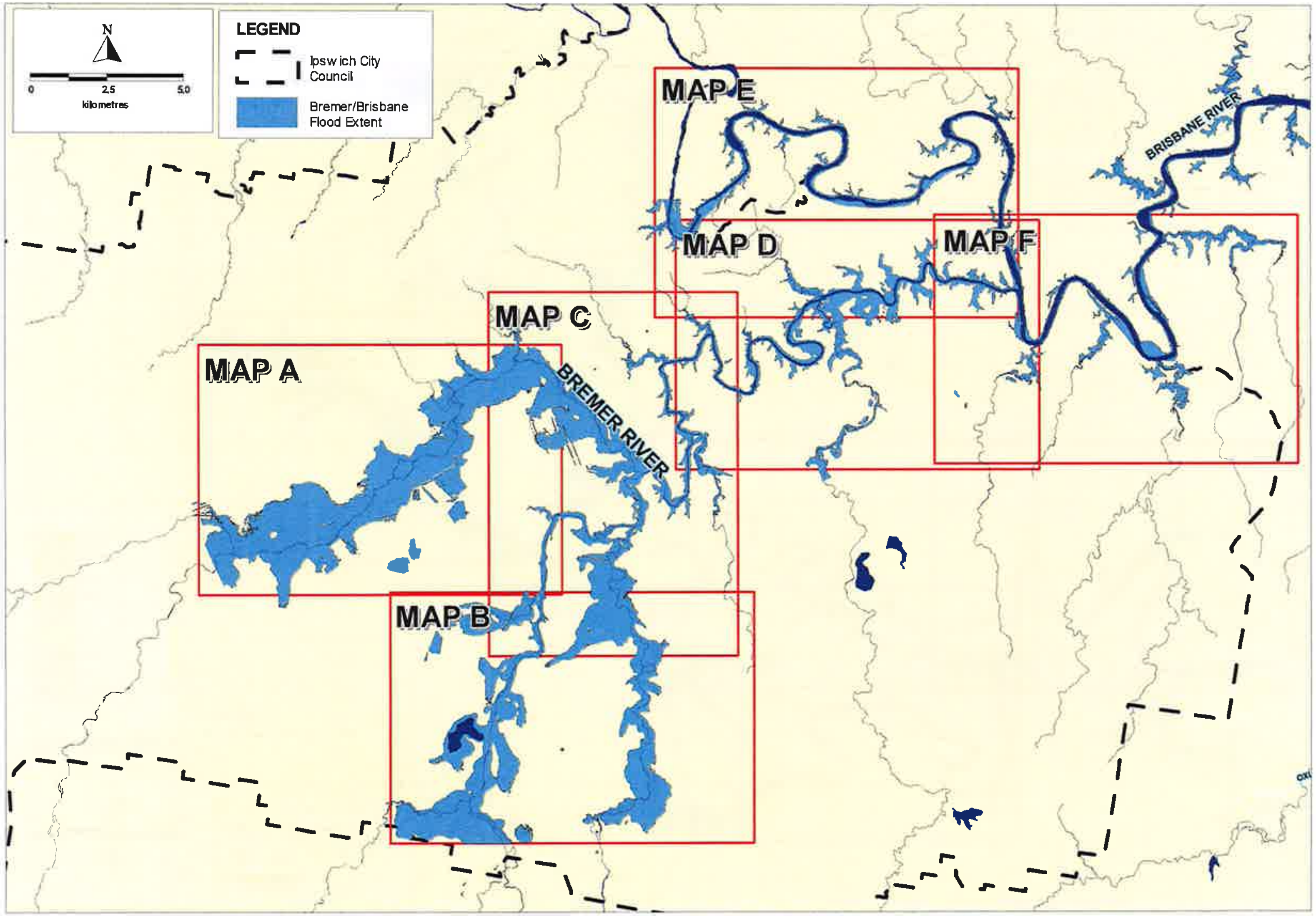

Figure 8.21 Extent of Inundation at 1800 Hours on 11 January 2011, Ipswich City LGA

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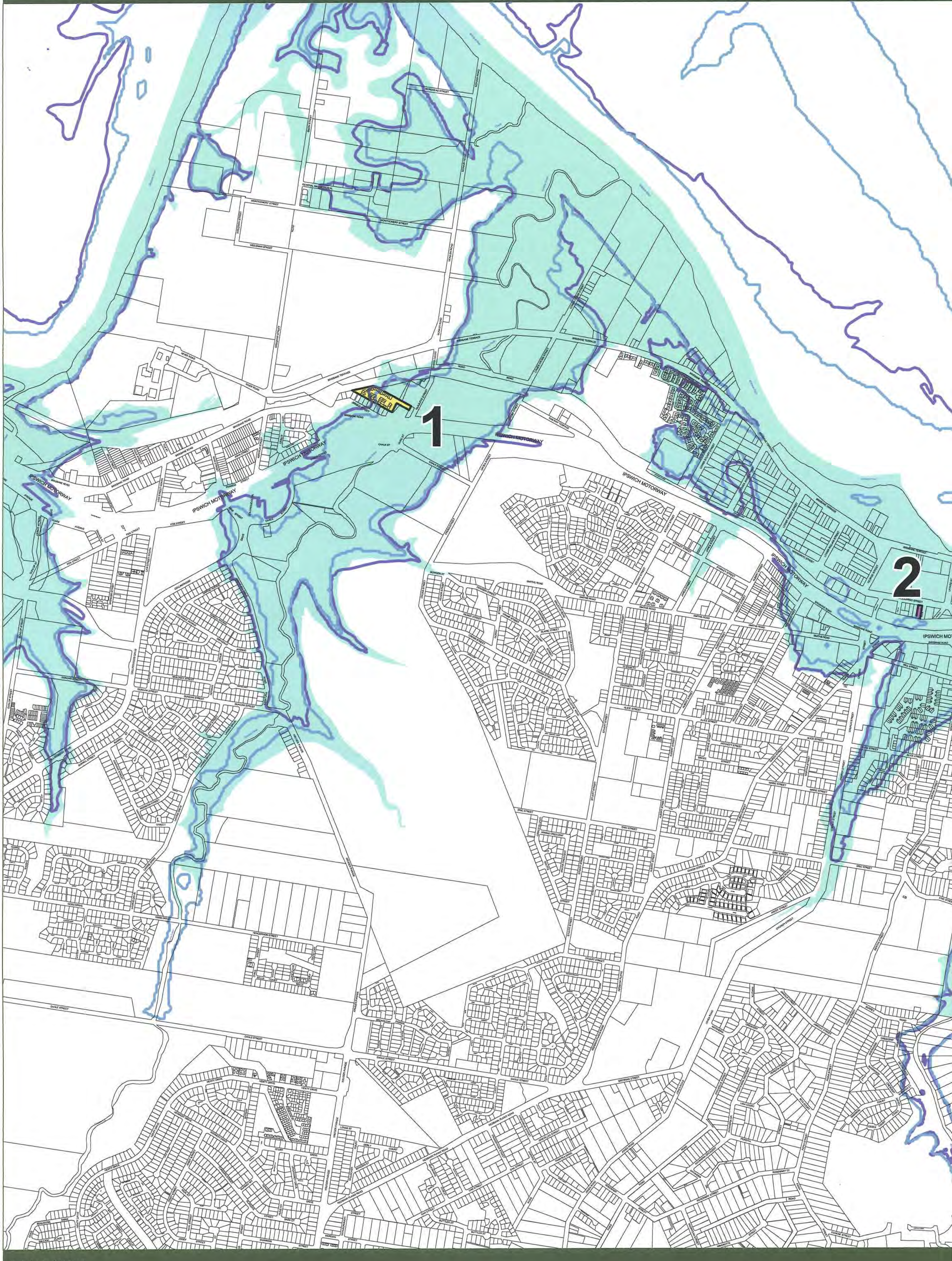
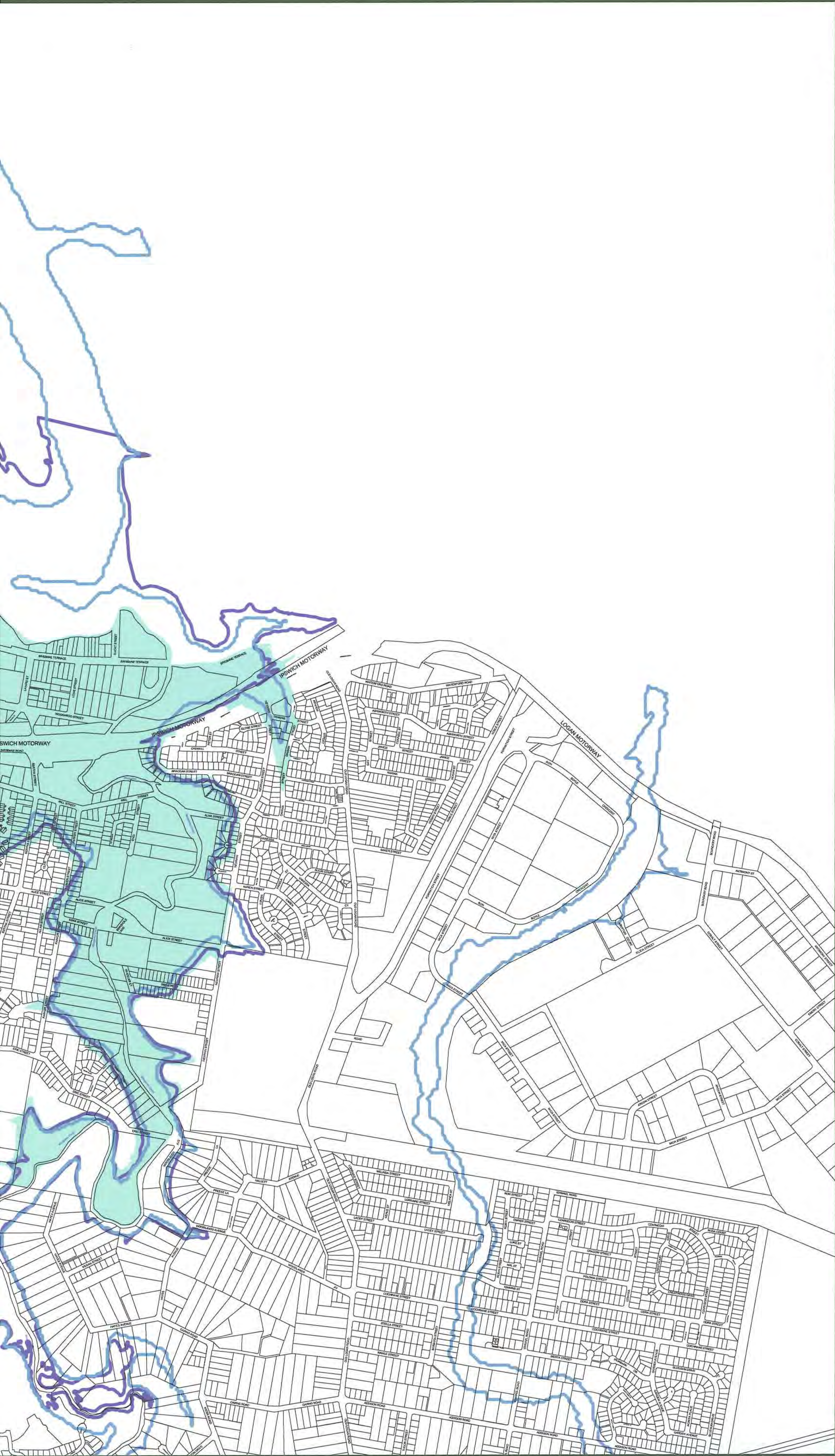
APPENDIX C: FLOOD LEVELS AND MAPPING DATA IN CURRENT IPSWICH CITY COUNCIL PLANNING DOCUMENT

Flooding Comparision Map



13/10/2011



City of Ipswich

IPSWICH CITY COUNCIL
45 Roderick Street, Ipswich Queensland 4305

Data Compiled by: Strategic Planning Branch, Planning & Development Department
Phone: (07) 3810 6888
Fax: (07) 3281 7085
Email: plandev@ipswich.qld.gov.au

Property Boundaries

1 in 100 Flood Line

2011 Flood Event

1974 Flood Event

1 13 Bridge Street, Redbank

2 20 Woogaroo Street, Goodna

N

GDA Map Grid : Geocentric Datum of Australia (G.D.A.)
Level Datum : Australian Height Datum (A.H.D.)

500 0 m

Scale 1:15,000

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0769-02-B
14 October 2011

wrm
water + environment

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Ms Jane Moynihan
Executive Director
Queensland Floods Commission of Inquiry
400 George Street
Brisbane
QLD 4000

Dear Ms Moynihan,

SUBJECT: PRELIMINARY FINDINGS OF PEER REVIEW - REPORT ON BRISBANE RIVER FLOOD FREQUENCY ANALYSIS PREPARED BY MARK BABISTER AND MONIQUE RETALLIK

1 BACKGROUND

Mark Babister and Monique Retallick (WMAwater) have prepared a report for the Queensland Floods Commission of Inquiry (QFCI) entitled '*Brisbane River 2011 Flood Event – Flood Frequency Analysis*' dated 18th September 2011. The report estimates the average recurrence interval (ARI) of the January 2011 flood and the 100 year ARI (1% AEP) flood discharge in the lower reaches of the Brisbane River (downstream of the Bremer River junction). In addition, based on its 100 year ARI discharge estimate, the report estimates 100 year ARI flood levels along the lower reaches of the Brisbane River and compares them with the 100 year ARI flood levels currently adopted by the Brisbane City Council.

DLA Piper Australia, acting on behalf of the Insurance Council of Australia (ICA), requested WRM Water & Environment Pty Ltd (WRM) to undertake a review of the WMAwater report for the purpose of assisting the commission. This report is in response to that request.

2 SCOPE OF WORK

This review has been undertaken on the basis of information and data gathered from a desktop review of the WMAwater report and supporting documentation provided by QFCI.

No independent hydrologic or hydraulic modelling has been undertaken by WRM as part of this review. Further, due to the limited time that was available to undertake this review, the findings of this report should be considered as preliminary.

The WMAwater report provided to WRM does not address Bremer River flooding in the vicinity of Ipswich as required under the scope of work specified by QFCI. WMAwater has since prepared a supplementary report on the Bremer River flooding dated 12th October 2011 but WMAwater's supplementary report has not been reviewed as part of this review by WRM due to time constraints.

3 GENERAL FINDINGS

3.1 Methodology

In my opinion, the analyses presented in the WMAwater report are not sufficiently rigorous to accurately estimate the ARI of the January 2011 flood or estimate the 100 year ARI flood discharges or levels in the lower reaches of the Brisbane River, especially for current (with Wivenhoe and Somerset dams) river conditions. It is possible that the limited time available for the study did not allow a rigorous investigation.

The methodology adopted for the pre-dam conditions flood frequency analyses (FFA) is generally acceptable (with some reservations) but the methodology adopted for the current post-dam conditions (with Wivenhoe and Somerset dams), in my view, is not satisfactory.

The key assumptions made, data used and data adjustments made in the FFA are not adequately explained or justified in the WMAwater report. Further, the level of detail presented on the FFA that has been undertaken and the results obtained from the FFA are inadequate to assess the validity of the results presented in the WMAwater report. In addition, the method used to correlate pre and post dams discharges at the Brisbane Port Office gauge is too simplistic and, in my view, should not be used.

The hydraulic model used to predict the 100 year ARI flood profile is acceptable but more accurate predictions could be made using a 2-Dimensional hydraulic model. There is insufficient information in the WMAwater report to assess whether the boundary conditions used in the hydraulic model to predict the 100 year ARI flood profile are acceptable.

3.2 Results Validation

The FFA has focussed solely on data at the Brisbane Port Office gauge. Although the Port Office gauge has the longest historical record, there are considerable uncertainties associated with this data set and, as a consequence, the results obtained using this data are also uncertain. The WMAwater report identifies the key limitations, difficulties and uncertainties associated with the Port Office data set. However, it has made no attempt to minimise the impact of these limitations and uncertainties by cross-checking and correlating the validity of Port Office gauge data and results against data and results for other lower Brisbane River gauge sites with long data records such as Mt Crosby, Moggill and Savages Crossing. Regional flood frequency analyses could have also been used to validate the results of the study. Sensitivity analyses to assess the impact of some of the uncertainties have not been undertaken.

The WMAwater study has undertaken a FFA of only the peak annual flood discharges in the Lower Brisbane River. A comprehensive FFA should also include an assessment of the peak annual flood event volumes particularly due to the flood storage affects of Wivenhoe and Somerset dams.

3.3 Inconsistencies and Potential Errors

There are significant uncertainties in the FFA results presented in the WMAwater report, including potential errors in the 100 ARI discharge estimate. These are discussed in Section 4.

There are also some apparent inconsistencies in the estimation of the 100 year ARI flood profile. These are discussed in Section 5.

3.4 Other Factors

The likely changes in the future to the Wivenhoe Dam and Somerset Dam operating rules may lower the 100 year ARI flood discharge in the lower Brisbane River. This potential impact may have to be considered when determining future 100 year ARI discharges and assessing the predicted 100 year ARI flood profiles.

4 FINDINGS ON FLOOD FREQUENCY ANALYSES AND RESULTS

4.1 General

The WMAwater report provides only very limited results on the FFA analyses. No statistics are given to assess how well the data fits the two probability distributions used to derive flood ARI's. Further, no FFA plots are given for the analyses that exclude the January 2011 flood.

4.2 Adopted Data and Analyses

In the FFA, the recorded peak flood levels for all pre-1917 floods, including the 1841 and 1893 floods, have been lowered by 1.52m to account for the effects of river dredging undertaken prior to 1917. Based on an assessment of continuous data from 1891 for the Port Office and Moggill gauges, SKM (1999) found that the 1841 and 1893 flood peaks at the Port Office were unaffected by river dredging and that the 1.52m adjustment should not be applied to large floods such as 1841 and 1893. The large floods generally have a significant influence on 100 year ARI estimates. The 1841 and 1893 floods were the largest and second largest floods on record. Hence, the FFA results, including the 100 year ARI flood discharge estimate, could potentially change significantly if the above adjustment to 1841 and 1893 recorded flood levels is removed from the analysis.

A new rating curve derived in the study (see Figure 8 of the WMAwater report) has been adopted for the Port Office gauge to convert recorded peak flood levels into peak discharges for use in the FFA. There are some uncertainties with this new rating curve. For example, the recorded peak flood level at the Port Office gauge for the January 2011 flood does not fit the derived rating curve. In addition, based on the shape of the January 2011 recorded water level hydrograph, the tidal influence appears to have affected flood levels at the Port Office gauge at least up to discharges of 9,500 m^3/s (Seqwater, 2011). Tidal influences are not taken into account in the adopted rating curve.

It appears that the flood events greater than 2,000 m^3/s have been classified as large and the remainder as small for the purposes of the FFA. The basis/justification for the selection of this threshold value is not known. Given that tidal influences affect flood levels for much higher discharges the adoption of a 2,000 m^3/s threshold appears unjustified. The adopted flood threshold has resulted in 141 out of 171 values

(82.5%) and 90 out of 102 values (88%) being 'censored' for the 171 year (1841-2011) and 102 year (1908-2011) data sets respectively. It is not clear what 'censored' means but it appears that these values have been omitted from the analysis. The recorded discharges at the upstream gauges should have been used to derive a discharge data set at the Port Office.

The February 1999 flood upstream of Wivenhoe and Somerset dams was larger than the 1974 flood (Seqwater, 2011). Based on data presented in Appendix B of the WMAwater report, it appears that the 1999 flood is not appropriately taken into account in the pre dams FFA.

4.3 Results

The following is of note with respect to the results for pre-dams conditions:

- The WMAwater report concludes that the 100 year ARI flood discharge at the Port Office estimated from the FFA is not sensitive to whether the January 2011 event is included or not. There is insufficient information in the WMAwater report to justify this finding.
- There is consistency in the pre-dam GEV distribution results for the two data periods analysed and for the two cases with and without the inclusion of the January 2011 flood. The results for the LP3 distribution for the shorter data set with and without the inclusion of the January 2011 flood are also consistent. However, the LP3 results for the longer data set with and without the inclusion of the January 2011 flood are not consistent. The reasons for this inconsistency are not discussed in the WMAwater report.
- The January 2011 flood ARI at the Port Office gauge for pre-dam conditions has been estimated to be 100 years. The estimated January 2011 peak flood discharge at the Port Office gauge for pre-dam conditions is 12,400 m^3/s. This finding is within an acceptable range for the data used in the analyses. However, based on the apparent better fit of the data for the LP3 distribution results, the ARI of the January flood for pre-dam conditions should be somewhat less than 100 years. It is also noted that if no river dredging adjustment had been applied to the 1841 and 1893 data the results may change significantly. These issues highlight the uncertainties associated with the results presented in the WMAwater report.
- The ARI's estimated from the FFA results are significantly higher than the likely probabilities estimated from plotting positions (see Table 9 of the WMAwater report). The reason for this is not discussed in the WMAwater report.

The following is of note with respect to the results for current (with-dams) conditions:

- It appears that no FFA has been undertaken for the current river conditions (with Wivenhoe and Somerset dams). Yet, an ARI for the January 2011 flood and a 100 year ARI discharge have been estimated for current river conditions. The basis for these estimates or justification for the adopted values is not adequately discussed in the WMAwater report.
- There is a heavy reliance on the accuracy of Figure 3 of the WMAwater report to explain and justify some of the study results and findings. Yet, there is no explanation about the data used to produce this figure and how the '*pre to post dam estimation line'* has been developed. There

are several reasons why the inferences made from Figure 3 may be inaccurate and may significantly overestimate post-dams peak discharge at the Port Office, including:

- The 1893 flood 'post-dam' discharge at the Port Office gauge is smaller than the equivalent 2011 discharge, although the 1893 'pre-dam' discharge is much larger than the equivalent 2011 discharge. The reason for this apparent inconsistency is not explained in the WMAwater report.
- The January 2011 flood had two peaks with the first flood peak inflow to the Wivenhoe and Somerset dams of the order of 12,000 m^3/s, whereas the corresponding peak discharge at the Port Office gauge for the first flood was only of the order of 2,000 m^3/s, a more than 80% reduction. This reduction in peak discharge at the Port office for the first flood event is not represented in Figure 3.
- The February 1999 flood was larger than the 1974 flood in the upper Brisbane River, but its impact on Brisbane was insignificant because this flood was fully mitigated by the dams (Seqwater, 2011). The peak flood level at the Port Office gauge was less than 1.7m AHD. The 1999 flood event is not represented in Figure 3.

- It is not clear how the ARI of the January 2011 flood or the 100 year ARI flood discharge for current (with dams) river conditions have been determined. The WMAwater report provides no analyses or justification for its results and findings on this issue. Further, the 100 year ARI peak flood discharge adopted for the lower Brisbane River for the current river conditions is not consistent with equivalent results for pre-dam conditions for the following reason. The WMAwater report has determined that the ARI of the January 2011 flood under current river conditions is 120 years. However, it has adopted a 100 year ARI flood discharge for the lower reaches of the river of 9,500 m^3/s, which is the same as the magnitude of the estimated January 2011 peak flood discharge at the Port Office gauge. The 100 year ARI discharge should be lower than the 120 year ARI discharge.

5 FINDINGS ON PREDICTED 100 YEAR ARI FLOOD PROFILE

It appears that there is an inconsistency in the predicted 100 year ARI flood profile. The adopted peak discharge for the January 2011 flood and the 100 year ARI flood (9,500 m^3/s) are identical. However, there is a significant difference in the predicted flood profiles for these two floods (see Figure 13 of the WMAwater report). The reason for this difference is not known and has not been explained in the WMAwater report.

The WMAwater report does not provide any details on the inflow boundary conditions (discharge hydrographs) or the downstream boundary condition (tide level) adopted to predict the 100 year ARI flood profile. Based on Figure 13 of the WMAwater report, it appears that the same downstream boundary condition has been adopted for both January 2011 and 100 year ARI Mike-11 model runs. If this is the case, the adopted downstream boundary condition may not be appropriate to predict the 100 year ARI flood profile.

6 CONCLUSION

In my opinion, the analyses presented in the WMAwater report are not sufficiently rigorous to accurately estimate the ARI of the January 2011 flood or estimate the 100 year ARI flood discharges or levels in the lower reaches of the Brisbane River, especially for current (with Wivenhoe and Somerset dams) river conditions. There are significant uncertainties in the FFA results presented in the WMAwater report, including potential errors in the 100 ARI discharge estimate. There are also some apparent inconsistencies in the estimation of the 100 year ARI flood profile. For these reasons, in my view, the findings of the WMAwater report should not be accepted until they are validated by more comprehensive hydrologic and hydraulic modelling studies. I understand that such studies are to commence in the near future.

Please do not hesitate to contact me if you have any queries.

For and on behalf of
WRM Water & Environment Pty Ltd

[redacted]

Dr Sharmil Markar *BSc(Eng) PhD FIEAust CPEng RPEQ*
Principal Engineer

References:

Seqwater (2011) '*January 2011 Flood Event - Report on the operation of the Wivenhoe Dam and Somerset Dam*', Report prepared by Seqwater, March 2011.

SKM (1999) '*Brisbane River Flood Study (Draft)*', Report prepared by Sinclair Knight Merz for Brisbane City Council, June 1999.

WMAwater (2011) '*Brisbane River 2011 Flood Event – Flood Frequency Analysis, Final Report*', Report prepared by WMAwater for the Queensland Flood Commission of Inquiry, September 2011.

Memorandum

To	Queensland Floods Commission of Inquiry	**Date**	21 October 2011
From	Rory Nathan	**Project No**	QE06544.01
Copy			
Subject	**Comment on Selected Issues Raised by WMAwater**		

Overview

1. The following comments are provided in relation to the Memorandum prepared by Mark Babister of WMAwater in his "Response to Peer Reviews" dated 7th October 2011. The comments presented below are restricted to matters arising from Paragraphs 17 to 23 of his response, and no other matters pertaining to the estimate of the current Q100 are considered here.

2. This paper is not intended to be a stand-alone document, and needs to be read in conjunction with WMAwater (2011[a,b]) and SKM (2011).

3. The WMAwater Memorandum raises a number of issues in the modelling undertaken by SKM (2003). The issues raised are based on comments contained in reports by Sargent Consulting (2006) and KBR (2002) and relate to:
    - apparent errors in the rainfall inputs used by SKM (2003);
    - apparent inadequacies in the RAFTS-XP model configuration; and,
    - the resistance approach adopted in the hydrodynamic model.

4. On the basis of the considerations detailed below it is concluded that:
    - the rainfall inputs used in the SKM RAFTS-XP model are materially correct;
    - the problems encountered by Sargent Consulting are associated with conceptual storage attributes that were not present in the SKM version of the RAFTS-XP model, and the calibration results demonstrate that the model adequately characterises the flood response of the catchment; and,
    - the resistance approach adopted in the SKM (2003) hydrodynamic model is considered reasonable, and given that the design simulations are within the range of flood magnitudes used in calibration, the choice of resistance model is of little consequence.

Rainfall Inputs

5. In paragraphs 17 to 21 a report by Sargent Consulting (2006) is relied upon to raise a number of apparent shortcomings in the rainfall-based modelling undertaken by SKM (2003). It should be noted that to our best knowledge SKM was not consulted at any stage of the investigation undertaken by Sargent Consulting, and SKM had no involvement in provision of the RAFTS-XP model or the rainfall data to Sargent Consulting.

6. Following receipt of the WMAwater Memorandum we reviewed in detail the model configuration and input files from the 2003 study. We can only speculate on the reasons why these problems were encountered by Sargent Consulting, however the information presented below does shed a little light on the nature of the issues raised.

7. The SKM RAFT-XP model rainfall volumes (for the whole catchment) have been compared with the CRC-FORGE rainfall volumes and found to differ by less than 1% for the 30 hour duration event. Further checks of the RAFTS-XP rainfall files and the CRC-FORGE rainfalls have indicated that there is no error in the inputs.

8. There is one minor discrepancy relating to the way that SKM used rainfall input files that impacts on the manner in which the input rainfalls are simulated in the RAFTS-XP model. The RAFTS-XP software contains an error that resulted in the first time increment of the input rainfall time series being ignored. It is assumed that Sargent Consulting used the in-built RAFTS-XP temporal patterns which does not have this problem (this is an issue that is associated with the RAFTS-XP software as distinct from the manner in which the model was configured to represent the Brisbane River catchment). Importantly, this issue has a very small influence (around 1%) on the magnitude of flows generated by the 30, 36, and 48 hour events due to the small proportion of rainfall in the first time increment. The difference in peak flows for the 24 hour event is in the order of 10% to 30%, however when corrected, the 24 hour event is still lower than the 30 hour and 72 hour events and thus this is of no consequence. The difference in the 72 hour event is around 5% and this event is the critical duration for the Brisbane River at Brisbane.

9. In summary, due to RAFTS-XP software ignoring the first increment in the input rainfall time series, the 2003 SKM results under-estimated the 100 year 30 hour event by 1% (the critical duration at Savages Crossing, and the point of comparison with the flood frequency analysis) and the 72 hour event by 5% (the critical duration at Brisbane).

10. The flows for the 1:100 AEP 30 hour “no dams” flood peaks listed in the Sargent Consulting report (2006; Table 3, p14) were compared with those from the SKM RAFTS-XP output files, and these are summarised in Table 1.

11. In comparing the flows derived by the Sargent Consulting (2006) RAFTS-XP model and the SKM RAFTS-XP model, two conclusions can be drawn:
    i. All flow comparisons *upstream* of Savages Crossing and at all locations in the unregulated tributaries are very similar (the minor disparities are due to differences in how the first rainfall increment is treated), however, for all locations *downstream* of this location the differences in peak flow are appreciable; and,
    ii. Given the presence of a conceptual storage at Savages Crossing, it is apparent that the differences in model results for downstream locations are due to differences in the way this conceptual storage was configured.

12. Evidence for differences in the conceptual storage configuration (the second conclusion note in the preceding point) is further reinforced by the results shown for the 30 hour event in Figure 4 of Sargent Consulting (2006); they report a discharge of 13,130 m^3/s for a peak stage of 7.5 m, and this is not consistent with the stage-discharge relationship adopted by SKM which would result in a discharge of only 6,430 m^3/s for the same stage (as reported in SKM, 1998).

Table 1: Difference in 30 hour flood estimates obtained using RAFTS-XP model developed by SKM (2004) and that reported by Sargent Consulting (2006)*.

LOCATION	RAFTS_NODE	SKM (m^3/s)	Sargent Consulting (m^3/s)	Difference (%)
Cooyar Ck	COO-OUT	1,501	1,500	0.1%
Bris R at Linville	LIN-OUT	3,424	3,420	0.1%
Emu Ck at Boat Mtn	EMU-OUT	1,381	1,380	0.1%
Bris R at Gregors Ck	GRE-OUT	5,904	6,010	-1.8%
Cressbrook Ck	CRE-OUT	686	690	-0.6%
Stanley R US Somerset Dam	SOM+++	2,234	2,230	0.2%
Bris R at Somerset Dam	SOM-OUT	3,592	3,620	-0.8%
Bris R at Wivenhoe Dam	WIV-OUT	10,981	11,150	-1.5%
Lockyer Ck at Helidon	HEL-OUT	882	860	2.6%
Lockver Ck at Gatton	GAT-OUT	2,949	2,970	-0.7%
Laidley Ck at Laidley	SHO-OUT	669	670	-0.1%
Lockyer Ck at Lyons Br	LYO-OUT	3,689	3,720	-0.8%
Inflow to Temp Storage Lock Ck Bris R jn	SAV10	14,382	14,560	-1.2%
Bris R at Savages Crossing	SAV-OUT	9,613	13,140	-26.8%
Bris R at Mt Crosby	MTC-OUT	9,621	13,170	-27.0%
Bris R at Moggill	JIN###	9,074	12,590	-27.9%
Bremer R at Walloon	WAL-OUT	1,125	1,130	-0.5%
Warrill Ck at Kalbar	KAL-OUT	1,020	1,020	0.0%
Warrill Ck at Amberley	AMB-OUT	1,700	1,700	0.0%
Purga Ck at Loamside	PUR-OUT	668	670	-0.3%
Bremer R at Ipswich	2C#	2,432	2,450	-0.7%
Bris R at Jindalee	JIN-OUT	9,075	12,590	-27.9%
Bris R at PO Gauge	POG-OUT	9,075	12,590	-27.9%

*Note: the number of decimal places (ie inferred accuracy) used in the above table is higher than can be justified, and has been adopted solely for the purposes of model comparison.

RAFTS-XP Model Conceptualisation

13. The configuration of the RAFTS-XP model is described in some detail in the SKM (1998) report. This report presents details of how the model was configured, where the main conceptual elements were based on:
    - Storage routing for overland flow;
    - Hydrograph lagging based on time of travel for upstream channels; and,
    - Storage routing to represent attenuation of channel flow in the downstream reaches.

14. Sargent Consulting found that the downstream model results were very sensitive to the conceptual storages. However, the SKM RAFTS-XP results do not reflect the same degree of sensitivity as observed by Sargent Consulting. The attenuation (ie reduction in flow) due to the largest storage (at Lowood) in the SKM model is 21% to 35% (for the range of durations considered); however, Sargent Consulting found that the attenuation at the same node varied between 6% and 34%. This difference in sensitivity provides further evidence to that presented in paragraph 12 above that the conceptual storage in Sargent Consulting's model was configured differently to that adopted by SKM. Somehow, either in the conversion of RAFTS-XP version 5.0 to RAFTS-XP 2000, or in the provisioning process, the operation of the conceptual storages used by Sargent Consulting differed from that originally devised by SKM.

15. Mr Babister expresses "serious concern" that the only locations where the model estimates are reliable are downstream of these conceptual storages. The basis for this view is not clear, as reasonable comparisons of historic flood events with model simulations (in terms of peak, shape, and timing) were derived for a large number of sites at locations upstream of these nodes. The locations of these points of comparison are shown in Figure 1, and plots of model performance at these locations for the 1955 and 1974 events are provided in SKM (1998, 2004).

16. It is noted that the comments made concerning the "very unorthodox" conceptualisation of the RAFTS-XP model reflect the views of Mr Babister; Sargent Consulting expresses their views in terms of sensitivity of the flows to the conceptual storages as they existed in their version of the model. It should also be noted that Prof Mein (1998) did not raise concerns with conceptualisation in his review.

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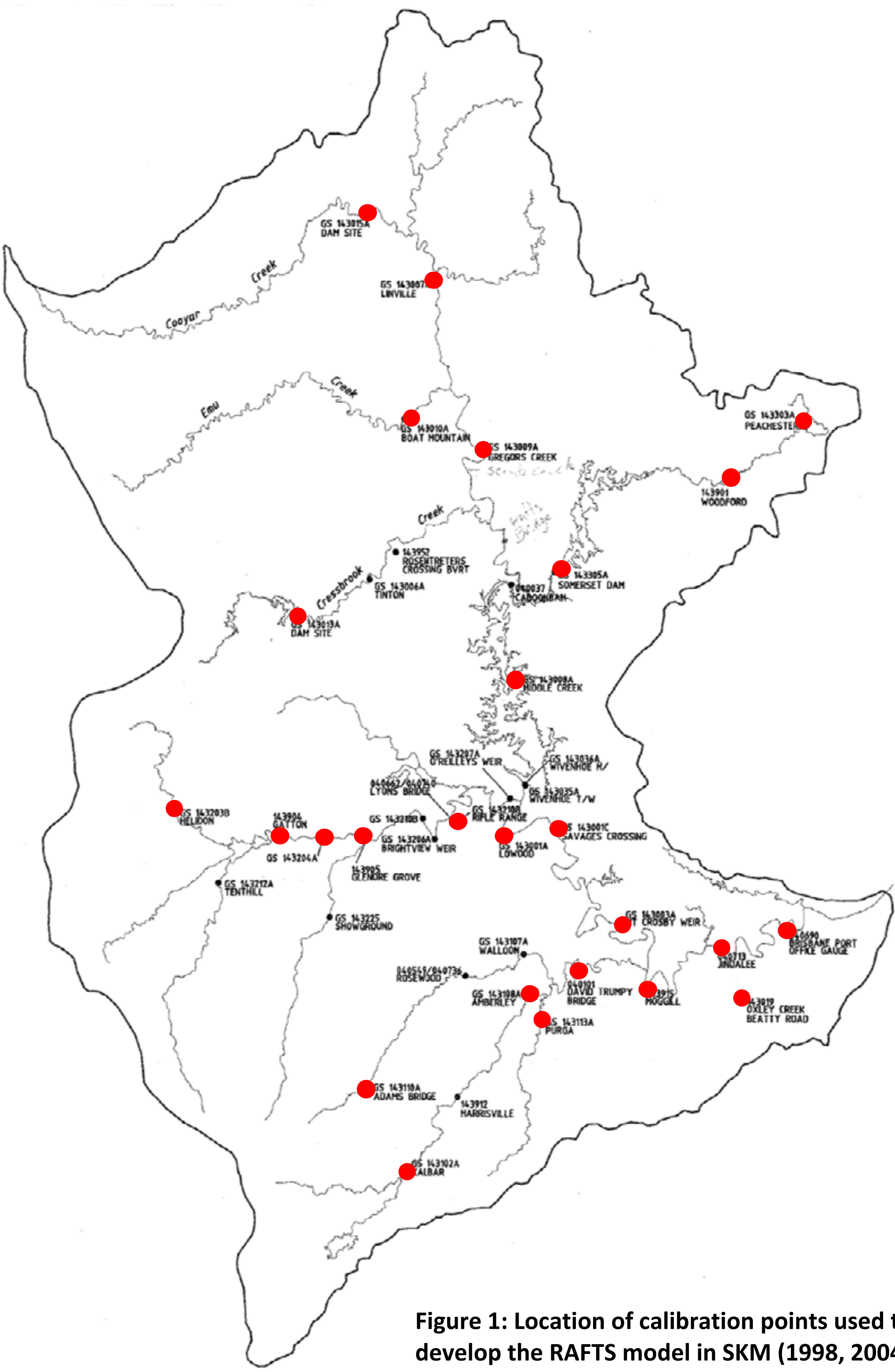

Figure 1: Location of calibration points used to develop the RAFTS model in SKM (1998, 2004).

Hydraulic Model

17. The general inference made in paragraphs 22 and 23 is that the use of the Resistance Radius method in the MIKE-11 model has a major effect on the performance of the flood model for flood events of different magnitude to the calibration events. Reports by KBR (2002) and WMAwater (2011[c]) are cited to support this view.

18. The text of the KBR (2002) report suggests that changes to *Manning's n* values were required when switching from Resistance Radius method (as adopted by SKM) to Total Area Hydraulic Radius (as adopted by KBR). This outcome is not surprising as the latter approach uses a depth-width averaged velocity in the non-friction parts of the momentum equations as opposed to the Resistance Radius method which uses a velocity which accounts for variations in *Mannings n* across the channel.

19. It is noted that the KBR comments were made in relation to the Bremer River which has different characteristics to the lower Brisbane River. The Bremer River is a more incised river which is deeper and narrower than the lower Brisbane River. The Brisbane River in its lower reaches would not be described as deep given its width (10m to 15m deep and 300m to 400m wide), particularly in large floods where the floodplain is activated. It is thus considered that KBR's comments have been used somewhat out of context as the focus of interest here is the appropriateness of the MIKE-11 model for use in the lower Brisbane River.

20. It is also worth noting guidance provided by the developers of the Hydraulic Model in regard to the use of the two methods (DHI, 2010):

    "Choice between resistance radius or hydraulic radius, effective area can depend upon the nature of the cross-section; if there are significant variations in shape (for example a river channel plus floodplains), resistance radius is appropriate. If the cross-section is narrow and deep, hydraulic radius could be more appropriate. Choice also will depend upon whether your personal experience (and knowledge of Manning numbers) is based upon one method as opposed to another.

    Remember that in most cases the differences between the two methods will be small. The momentum terms are dependent upon changes along the branch, so if you don't have significant variations between successive cross-sections there will be even less difference in the methods."

21. However, the most important point to note is that the design flood of interest, namely the "post-dam Q100", is similar to the magnitude of the historical floods used to calibrate the model. Indeed the adopted "post-dam Q100" along the lower reaches of the Brisbane River lies *between* the peak flows recorded in 1955 and 1974 that were used in calibration (as these occurred prior to the construction of Wivenhoe Dam). Model simulations undertaken for the Q100 do not require extrapolation, and thus we can be confident that the choice of resistance model under these conditions is of little consequence.

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[redacted]

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QUEENSLAND FLOOD COMMISSION OF INQUIRY

BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS

FINAL REPORT

SEPTEMBER 2011

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BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS

FINAL REPORT

SEPTEMBER 2011

Project Brisbane River 2011 Flood Event – Flood Frequency Analysis	**Project Number** 111024
Client Queensland Floods Commission of Inquiry	**Client's Representative**
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Date 18 September 2011	**Verified by** [redacted]

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1	FINAL REPORT	18 SEP 11

BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS

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1. INTRODUCTION

1 This report was prepared at the request of the Queensland Floods Commission of Inquiry (The Commission). It investigates the 1% AEP (Q100) flood level on the Brisbane River from Moggill to the ocean and assesses the probability of the 2011 flood event. An addendum report will address similar issues on the Bremer River.

2 While every attempt has been made to conduct a thorough, rigorous and scientific flood frequency analysis, there have been a number of difficulties in developing a dataset suitable for this purpose. In particular, the analysis relies on adjustments to recorded levels at Brisbane River Port Office to account for changes to river morphology, and is dependent on the use of a rating curve to convert recorded levels into flows. WMAwater consider that the flood frequency assessment could be improved by undertaking further steps to improve the rating curve and the adjustments to the historical record. Measures by which such improvements can be made are identified in this report, but require a much longer timeframe than that available for this assessment. It is recommended that further investigations be undertaken including a thorough review of the rating curves, assessment of astronomical tide influence in the historical record, and development of a suitable two-dimensional model of the Lower Brisbane River to assess the effects of geomorphological changes.

3 Section 2 of this report outlines the scope of the investigation. Section 3 provides background on determining design flood levels, the use of the 1% AEP level for planning purposes, freeboard and outlines best practice in conducting a flood frequency analysis. Section 4 details the history of flooding on the Brisbane River. Section 5 provides a brief summary of previous estimates of the 1% AEP flood level and flow. The rating curve and associated data used in this investigation and how it was derived is detailed in Section 6, while Section 7 details the analysis. Section 8 presents the main conclusions and outlines a process for developing robust flood frequency estimates.

4 This report interchangeably uses the terms 1% AEP, 100 year ARI and Q100. In Queensland the term "Q100" is regularly used to denote the level or flow that has a 1% chance of occurring in any one year. "Q" is normally used in water engineering to denote flow, so application of the term "Q100" to indicate flood level can create confusion. The term is not widely used in practice outside Queensland. The distinction can be particularly important in coastal areas, as the 100 year ARI flood level in the lower reaches of rivers is caused by a combination of ocean levels and flow (and other contributing factors) and is not necessarily a result of 100 year ARI flow alone.

2. SCOPE

5 The Commission has requested that Mark Babister of WMAwater undertake the following:

1. Conduct a flood frequency analysis and determine the 1% AEP flood level for key locations on the Brisbane River below its junction with the Bremer River and on the Bremer River in the vicinity of Ipswich using information available prior to the January 2011 event. This work should be used to determine 1% AEP flood levels at up to 8 key locations in the Brisbane and Bremer Rivers and to produce 1% AEP flood profiles. This work should include a review of the SKM 1% AEP flood profile.

2. Repeat task 1 with the 2011 event included in the historical dataset.

3. Using results of task 1 and 2 determine the ARI and AEP of the January 2011 floods at particular points along the Brisbane River and Bremer River.

6 The following locations were identified as being of interest between Moggill and Brisbane:

- 13 Bridge St., Redbank (off-bank),
- Cnr. Ryan St. and Woogaroo St., Goodna,
- Corner Moggill Rd, Birkin Rd, Bellbowrie (Coles),
- Corner Thiesfield St, Sandringham Pl, Fig Tree Pocket,
- 312 Long St East, Graceville,
- Brisbane Markets, Rocklea,
- Softstone St, Tennyson (Tennyson Reach apartments),
- 15 Cansdale St, Yeronga,
- 42 Ferry Rd, West End (Aura apartments),
- 81 Baroona Rd, Paddington (Epic Cycles), and
- Brisbane City Gauge.

3. BACKGROUND ON DETERMINING DESIGN FLOOD LEVELS

3.1. Use of 1% AEP Flood Level for Planning Purposes

7 For planning purposes it is necessary to decide what level of flood risk is acceptable for individuals and the community. Ideally planning levels should be decided on the basis of risk, where both probability and consequences are considered, but in most locations in Australia the 1% AEP (100 year ARI) flood is designated as having an acceptable risk for residential planning purposes regardless of the consequences. This approach often leads us to a planning level, line or map which defines whether flood-related controls on prospective development are applicable or not.

8 The 1% AEP (100 year) flood level has not always been utilised for flood planning purposes. Prior to the use of the 1% AEP (100 year) as a design flood standard it was common for communities to simply use the largest historical flood on record for planning purposes. The 1% AEP (100 year) planning level was first adopted for residential housing in the ACT in the 1970s and has subsequently been adopted in most locations throughout Australia. The 100 year standard is also used extensively in the USA.

9 While the use of a standard event such as the 1% AEP (100 year ARI) flood event for planning purposes may provide a level of consistency, ideally flood planning levels would be determined on the basis of a flood risk assessment. While such flood risk assessments have been carried out in many locations throughout Australia there has been a reluctance to move away from the 1% AEP flood standard. Floodplain risk management studies often show that there are strong social and economic reasons for considering a higher standard in some locations, such as:

- Where rare flood levels are significantly higher and likely to cause significant devastation (an example would include locations where the 200 year event is over 2m above the 100 year ARI event); and
- Where inundation of the location will have significant economic and social consequences for a much wider region (an example would be the inundation of the CBD and regional service section of a major city or town, which may disrupt/prevent the provision of essential services for a much larger regional population).

10 For these reasons the city of London is moving to a planning level above the 0.2% AEP (500 year ARI) level for the Thames estuary. Many parts of the Netherlands use planning levels above the 0.1% AEP (1000 year ARI) level as in many places inundation would have catastrophic consequences (including loss of life) and take many months to pump out.

11 It is very rarely possible to eliminate flood risk as this would require placing development above the Probable Maximum Flood (PMF) level which cannot generally be justified on economic grounds and in other cases may simply not be physically possible.

12 The 1% AEP (100 year) flood is a theoretical flood with a specified probability of being exceeded. An actual flood event is whatever happens to occur, and may be larger than, or less than, the 1% AEP (100 year) event and may vary in probability along the reaches of a long river.

13 Any actual flood event will vary in some manner from the 100 year event. Such variations are primarily due to differences in rainfall, as the rainfall that occurs in an actual event is different in duration, intensity and spatial and temporal pattern to that which is used to derive the 100 year flood. Variation in other flood producing factors, for example how wet the catchment is before the event, or the location of the storm centre within the catchment can also have an impact on the size of flood, and also contribute to differences.

3.2. Freeboard

14 Freeboard is used to account for several factors including uncertainty in the flood estimate, differences in water level across the floodplain due to local factors, wind waves, waves caused by passing vehicles and the cumulative effect of future development. Freeboard is in effect a factor of safety that allows for uncertainty in underlying data and is commonly used in both Australian and international practice. The NSW Flood Development Manual describes the purpose of freeboard as being “to provide reasonable certainty that the reduced risk exposure provided by selection of a particular flood as the basis of a FPL is actually provided.” (Reference 24, Appendix K7).

15 The additional buffer that freeboard includes an allowance for any minor increases in flood level due to the building of key infrastructure projects (such as roads and rail lines), which may have a cumulative impact on flood levels. In practical terms this ensures that minor changes in the 1% AEP flood do not result in houses falling into a high flood risk category.

16 Freeboard traditionally varies between 300mm (0.3m, or 1 foot) and 500mm (0.5m) but can be up to 1m in places. It is considered best practice in Australia to use 500 mm. In the coastal zone a separate allowance is often made for sea level rise resulting from climate change, as the impact of sea level rise decreases with distance from coast. Different freeboard amounts can often be applied to different types of development such as critical infrastructure or commercial/industrial development.

3.3. Flood Frequency Analysis Theory

17 The two basic methods for determining the probability of different flood levels are Flood Frequency Analysis (FFA) and the rainfall based Design Flood Method (DFM). FFA is the process of fitting a probability distribution to a series of flood peaks at a particular location. The DFM fits a probability distribution to observed rainfall and uses hydrologic and hydraulic modelling techniques to convert catchment rainfall of a certain probability to a flood level or flow, which is assumed to be of the same probability.

18 FFA provides a direct measure of flood probability and does not require assumptions about the different catchment wide processes and variables that contribute to peak flood levels or flow at a particular location. This allows for all the historically observed variability in rainfall intensity, storm volume, storm duration, storm type and antecedent conditions to be included. Unlike the DFM, FFA also provides a measure of the uncertainty of flood estimates. FFA is the method that most other flood estimation techniques used in Australian Practice are checked against.

19 While flood frequency has a number of advantages, as it can only be used when:

- A long flood record exists,
- The flood record is homogenous or can be adjusted to a near homogenous state,
- A reliable rating curve exists, and
- The probability of the event to be derived does not require extrapolation too far beyond the observed record length.

20 FFA should not be used to extrapolate far beyond the extent permitted given the period of record, as estimates are very dependent upon the assumed distribution. For example, a dataset with a 20 year length of record will not give a good estimate of the 1% AEP (100 year ARI) flow. When estimating rare events well beyond the period of record, rainfall based methods are recommend by Australian Rainfall and Runoff 1987 (Reference 7). In this situation rainfall based methods have advantages over FFA, as the probability of rare rainfalls can be estimated by regional techniques and extreme events can be approximated by methods that consider the limits of storm efficiency and the moisture holding capacity of the atmosphere.

21 The historical flood record can be analysed using either an annual series (where the largest event in a calendar or water year is extracted) or a Peak Over Threshold (POT) series (where the largest independent peaks are extracted). In South East Queensland a water year needs to start in winter as nearly all flood events occur in the wet season (over late spring and summer). For the analysis presented in this report, a water year is defined from July to June. While several floods occurred in 2010-2011 water year only the January flood (the largest) would be considered in an annual series. A POT series can be difficult to extract as there is no definitive way to determine if events are independent, which is a requirement of FFA. For this reason it is often not used. For example in February 1893, three floods occurred within 3 weeks, and while these events were probably caused by 3 separate meteorological events, the wet catchment and swollen rivers produced by the first flood influenced the magnitude of the subsequent flood peaks.

22 The current best practice advice on conducting FFA is contained in the ARR Draft chapter of the current revision of ARR (Reference 28). The major changes in the application of FFA (from those described in ARR 1987 (Reference 7) in Australia are:

- The removal of the recommendation to use the Log Pearson 3 (LP3) distribution, and
- The replacement of log space moment based fitting techniques.

3.3.1. Probability Distribution

23 Many probability distributions have been applied to FFA and this has been a very active field of research. However, it is not possible to determine the “correct” form of the distribution and no rigorous proof exists that any particular distribution is more appropriate than another. ARR (Reference 28) provides further discussion on this issue. Two broad approaches are possible. The first is to use a range of distributions and adopt the one which provides the “best fit”. The other is to use a single distribution for a region. While no distribution is recommended the Generalised Extreme Value (GEV) and Log Pearson 3 (LP3) are suggested as a starting point (Reference 28) as they have been shown to fit Australian data well.

3.3.2. Fitting Method

24 Recent research has suggested that the fitting method is as important as the adopted distribution. The traditional fitting method has generally been based on moments and this makes the fit very sensitive to the highest and lowest observed flow values. Recent research has shown that L-moment and Bayesian likelihood approaches are much more robust than traditional moment fitting and hence these are the current recommended methods.

25 For this analysis a Bayesian maximum likelihood approach has been adopted in preference to L-moments because the method allows the inclusion of large historical flood information outside the period of continuous record. While not necessary at the Port Office it would be required at other locations.

26 This study used the Flike flood frequency analysis software developed by Kuczera (Reference 29).

3.3.3. Historical Flood Information

27 In many locations in Australia data detailing the early flood record (from early settlement and hence prior to the establishment of continuous gauging stations) is incomplete and only the large events tend to be well documented. Where major floods are known to occur it is possible to include this information in modern flood frequency analysis via Bayesian methods. This is particularly important where these early floods are known to be larger than those contained in the continuous record even if little is known about the exact height or flow.

28 It is very important to include historical information when the continuous historic record does not contain many of the top ranking historic events. To not do so will probably result in underestimation of the probability of flooding.

3.3.4. Rating Curve

29 Flood frequency analysis is typically undertaken on flows, not flood levels, as flows lend themselves to the FFA methods and assumed distributions. As flow is not generally measured directly, a rating curve is required to convert observed peak flood levels into flow. The rating curve (height-discharge relationship) adopted for the estimation of stream flows from the recorded gauge heights is critical to the success of flood frequency analysis. A poor quality rating curve results in a poor estimate of flow. Where there has been a significant change in river cross section or where flow is affected by tidal effects or tributary inflows a family of rating curves is often produced.

3.3.5. Long Term Climate Variability

30 The flood record on the east coast of Australia exhibits periods of a decade or longer timescale that are flood or drought dominated. This was first recognised by Erskine and Warner in 1988 (Reference 30).

31 Short term climate variability on the east coast of Australia is characterised by the interannual El Nino/Southern Oscillation (ENSO). There is a marked increase in flood risk in eastern Australia during the La Nina phase. The El Nino phase typically contains few major floods (Reference 31).

32 There is also considerable evidence that longer term processes have a major impact on flood risk. The Inter-decadal Pacific Oscillation (IPO) is a pattern of Pacific Ocean temperature variation that shifts phase at a timescale typically lasting 15-30 years. On the east coast of Australia nearly all large events occur during an IPO negative period.

33 Figure 1 shows the IPO index from 1880 to 2000. Note that the two large flood events in that period in Brisbane occur close to IPO low points.

34 It is important that flood frequency analysis is carried out over a long period so that the results are not biased to either of these climate cycles.

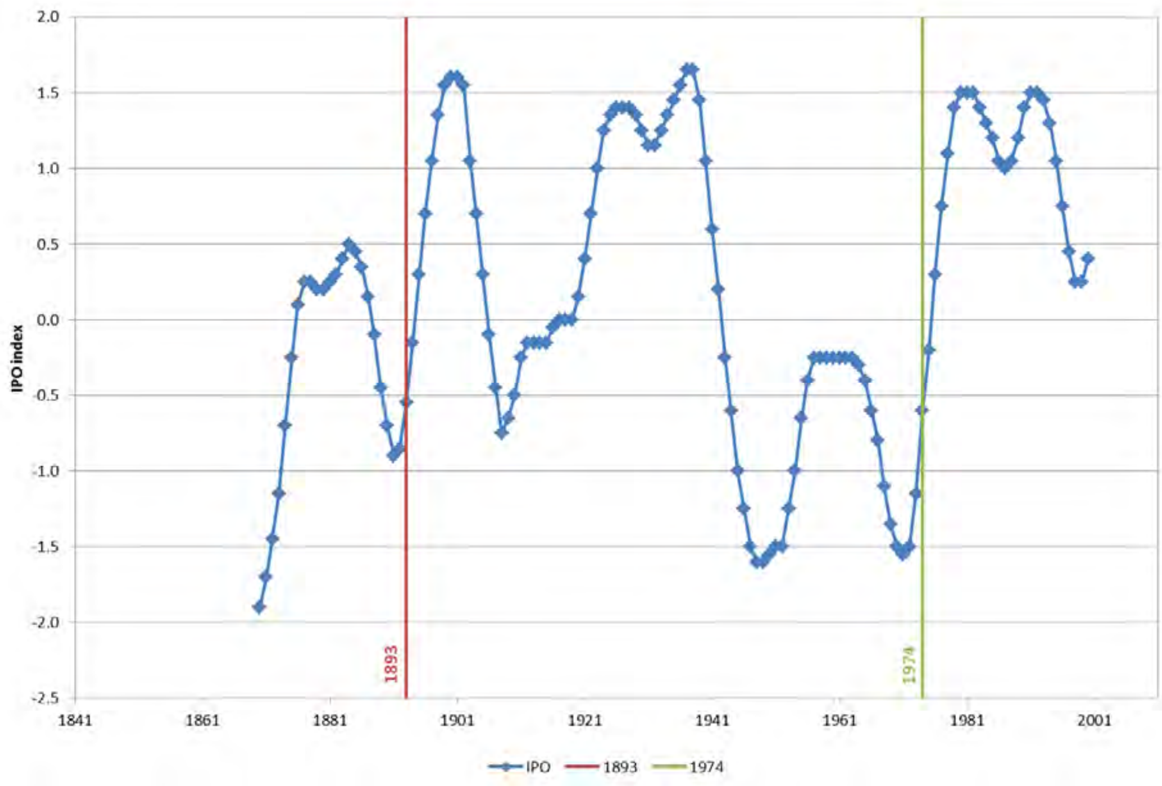


Figure 1: IPO Index 1880-2000

3.3.6. The Need for a Homogeneous Data Set

35 When conducting a flood frequency analysis a homogeneous data set, which is consistent and based on the same catchment conditions is required. The homogeneous data set needs to be free of the impacts of dams, levees, and significant changes in urbanisation and the river morphology. Creating a homogeneous data set is often not a simple process.

4. BRISBANE RIVER FLOOD HISTORY

4.1. Flood History

36 Little information is often available for floods that occurred early in the settlement of a region. However, historical flood information can be found from official correspondence, newspapers and even old parliament session documents. When examining historical flood records there is a desire to find accurately measured records. FFA techniques used in the past meant that where only vague information (such as "it was larger than the 1893 flood", or "it was the largest observed flood") was available it could not be incorporated into the analysis. However, the use of modern Bayesian techniques has meant this prior information can be incorporated. It is important if implementing this approach to determine whether the floods were actually significant. For example when people first settle a region they are still trying to establish what is the "norm" with regard to flooding, so there is often several floods called "significant" which are not that significant.

37 Historically studies of the Brisbane River have been reluctant to place too much weight on earlier events (Reference 16, Section 5). Investigation of historical records suggests there is plenty of evidence to prove the early events are credible and significant. The 1841 event was recorded in a number of locations and was discussed in the QLD Parliament after the 1893 event. Credible evidence also exists for the magnitude of the 1844 event. There are also references to earlier floods (1824, 1825, 1836, 1839 (Reference 32)) however, besides the 1825 event, they appear not to be significant, and for all these early events the evidence is not detailed enough for their inclusion.

38 Looking at historical events and comparing or ranking them with more modern events is complicated by the changes in the River that have occurred over the years since European settlement. These changes are discussed in Section 4.2 and include dredging, river widening and also the construction of Somerset and Wivenhoe Dams.

39 While the 1974 flood was very significant, when looking at the full flood record (1841-2011) with appropriate adjustments made for the impact of dams and dredging of the river, under current conditions the 1841, 1893 (2 events) and 2011 are larger than the 1974 under pre and post dam conditions.

4.1.1. 14th January 1841

40 The 1841 flood is the highest recorded in Brisbane's history at 8.41m AHD at the Port Office gauge. According to Reference 32:

> *"In 1896, JB Henderson, the Government Hydraulics Engineer in an address to Parliament reported that he found by examination of earlier plans that the 1841 flood was [7 centimetres] higher than the flood of 5th February 1893."*

41 While the 1841 flood produced the highest recorded flood level at the Port Office when proper adjustments are made for changes in the river it is no longer the largest known flood on the Brisbane River. In 1841 a sandbar was present at the mouth of the Brisbane river which exacerbated the flood levels upstream and no works had been carried out to dredge and straighten the river. Accounting for these factors would reduce the flood level by an estimated 1.92m to 6.48 mAHD at the Port Office gauge.

4.1.2. 10th January 1844

42 The 1844 event peaked at approximately 4 feet below the level of the 1841 event at 7.03 mAHD. The adjusted 1844 level is 5.11 mAHD.

4.1.3. February 1893

43 1893 was a wet year on the Brisbane River with several flood peaks occurring including 3 floods within February (peaking on the 5th, 12th and 19th). The first and largest event resulted in a peak of 8.35 mAHD recorded in Brisbane. The second event peaked at 2.15m AHD, with the third event peaking close to the level of the first at 8.09m AHD. Houses were washed away at Ipswich and Brisbane. During the first event the “Elamang” and the gunboat “Paluma” were carried into the Botanical Gardens area, and the "Natone" onto the Eagle Farm flats. In the third event these boats were refloated.

44 It is noteworthy that the 1893 flood peak of 8.35 mAHD occurred following the removal of the downstream bar in 1864 and in a period when the Brisbane River was being dredged on an ongoing basis to improve River navigability.

4.1.4. 25- 29th January 1974

45 The 1974 event was the highest flood recorded on the Brisbane River during the 1900’s with the river peaking at 5.45 mAHD. During the event 8,000 households were affected. The flood peak at the Port Office would have been marginally higher had Somerset Dam not been constructed in the 1940’s (completed in the 1950’s). The substantial river works carried out since 1893 are estimated to have lowered this flood level by approximately 1.5m (Reference 18).

4.1.5. January 2011

46 The January 2011 event was the largest experienced on the Brisbane River since the construction of Wivenhoe dam. The river peaked at a height of 4.27 mAHD at the Port Office gauge and 4.46 mAHD at the City gauge. Despite the gauges being located directly opposite each other on either bank of the river a discrepancy between the two was recorded. This issue was raised in Reference 39.

47 The Port Office gauge is located at a dock on the left bank of the River at the corner of Edward and Alice streets. The City gauge lies at approximately the same river chainage

but on the right bank. The Port Office gauge is operated by Marine Safety Queensland (MSQ) which is a State agency lying under the Department of Transport and Main Roads. Conversations with MSQ staff in the Tides Office indicate that the discrepancy is known and attributed to mechanical failure of the City gauge. The City gauge is operated by Seqwater and is the gauge which the BOM Brisbane Flood Alert System currently refers to.

48 Whether or not these discrepancies are attributed to the failure of the City Gauge or transverse slope on the river due to the river meander upstream of the gauges is not known. Of interest would be whether Seqwater believe they experienced any mechanical failure with respect to gauged water level at the City gauge during the event.

4.1.6. Flood Events on the Brisbane River

49 The largest events on the Brisbane River are summarised in Table 1.

Table 1: Brisbane River Flood History

Event	Recorded Flood Level (mAHD)
1841	8.43
1844	7.02
1890	5.33
1893 (5th February, a)	8.35
1893 (19th February, b)	8.09
1898	5.02
1974	5.45
2011	4.27 (4.46)

Note: Levels as recorded at Port Office (unadjusted for comparability)

50 Historical peak flood heights are available at Brisbane for the Port Office/City gauge, Lowood, Moggill and Savages Crossing. Table 2 summarises the period for which data is available. Several other gauges with shorter record lengths have not been included.

Table 2: Period of Record

Location	Start	End	Period of Record (Years)
Moggill	1966	2011	46
Lowood	1910	2011	102
Port Office/City Gauge	1841	2011	171

4.2. History of River Changes

51 Changes to the Brisbane River have been driven by three distinct priorities:

1. *Navigation* – The Brisbane River was an important transport link to agriculturally valuable lands in the Darling Downs. Dredging and works carried out to aid

navigation started in the 1860's and continued through until the 1940's (Reference 3);

2. *Flood Mitigation* – from the 1930's river widening works were carried out and training walls were installed. These works were aimed at mitigating floods and reducing problems with sedimentation of the river, as had occurred following the 1893 event (Reference 3); and
3. *Development* – In recent times there has been a large amount of infrastructure built on the banks of the river or within the river. These structures can restrict flow during large flood events and can have localised impacts on flood level. This study has not attempted to quantify the effects of development on flood behaviour. These changes are assumed to be small in comparison with the other changes.

52 With the construction of Wivenhoe dam cutting off sediment supply from a large part of the catchment, it is unclear if the river has reached a new equilibrium or if accretion is continuing to occur. This needs to be determined using up to date bathymetric survey and an appropriate assessment.

4.2.1. Details of River Works

53 The Brisbane River has a long history of dredging beginning in 1864. The following text and table have been largely adapted from References 5 and 19. Dredging locations and dates are summarised in Table 3.

54 Originally, a shallow bar covered the entrance to the river to a depth of 5 ft (approx. 1.5m) at low tide and 12 ft at high tide. Upstream of the bar, the river deepened to 24 ft (approx. 7.3m) until a rocky area at Lytton. In 1864 a channel was cut across the bar to allow larger ships to access Brisbane port. From 1866 until 1891 numerous smaller dredging projects were undertaken, including a channel through the Fisherman's Islands to Pelican Bay, a channel through Redbank Flats and Cockatoo Shoal in the upper reaches of the river and the deepening of the river near the Eagle Farm and Pinkenba Flats.

55 The flood of 1893 undid a lot of the dredging work, silting up the river and reducing its depth to only 6ft (approximately 1.8m). The bar was also reduced to 8ft 6inches (approximately 2.5m) in depth. The restoration work to restore the river to pre-flood conditions was completed in 1895. In the early 1900s, the curves of several river bends were adjusted to straighten the river and a series of training walls were built to improve scouring action. Deep dredging was undertaken from Brisbane City to the river mouth. The deposit from these works was used to create Bishop Island, which also has an influence on the river flow behaviour. These works were finished in 1912.

56 Dredging of the river reached its peak in 1940 (Reference 3). The abandonment of the city for port purposes has lead to the discontinuation of dredging in this region.

57 The 1999, Brisbane River Flood Study (Reference 17) accounted for the effects of dredging by adjusting flood heights for the initial bar dredging in 1864 (reduced flood levels

by 0.4m) and the major dredging works completed in 1912 (reduced flood levels by 1.52m). The adjustment of 1.52m came from an estimate by Henderson (as quoted in Reference 17) that dredging works would achieve a reduction in flow levels of 5 feet (approx. 1.5m). Analysis of Moggil and Port Office gauges in the December 1999 report (Reference 18) suggests that whilst dredging would impact flood levels in line with Henderson's earlier prediction the works would not significantly impact larger floods.

Table 3: Key Dredging Dates, Depths and Locations on the Brisbane River (Adapted from McLeod, 1977 and Thompson, 2002 (References 5 and 19)

Key Dredging Dates				
Date	Depth			Comments
	Bar	Draught	River	
1824	5-12'		24'	24ft until rocky area at Lytton was reached
1864		11'9"		Francis' channel through bar completed
1866			12'	Fitzroy dredge- created a channel through Redbank flats
1867		17'		Brisbane river mouth dredge- Fisherman Islands and across Pelican Bank
1871		17'	10'6"	Eagle Farm flats dredged, river depth is to town
1874				Cockatoo Shoal dredge
1877	17'		15'	Pinkenba Flats dredge
1879				Heath's Channel Dredged
1891b	20'			Maintenance dredge
1893	8'6"		6'	The 1893 flood silts up the river
1895	15'		15'-16'	restoration work from flood damages completed
1898			20'	Lytton Rocks removed
1900's				Tips of bends straightened: Kangaroo point, Garden point, Bulimba point, Kinellan Point
1900's				A series of Training walls was built to improve scouring action including the 8,600 Hamilton Wall
1908-12	24'		24'	A new straight bar cutting was made, spoil created bishop island. Major dredging undertaken
1965				Removal of Seventeen Mile rocks
N/A				Outer River Bar
				Inner River Bar

4.3. Impacts of Dams on Flood Levels

58 Wivenhoe and Somerset Dams and their operation have a significant effect on downstream flood levels. Table 4 summarises their characteristics.

Table 4: Dam Characteristics (Source: Reference 27)

	Wivenhoe Dam	Somerset Dam
Completed	1985	1959
Water Supply Storage (GL)	1150	370
Temporary Flood Storage	1450	524
Location	Brisbane River Upstream of Lockyer and Bremer	Stanley River Upstream of Brisbane River
Catchment (km^2)	7000 including Somerset Dam	1330
Reservoir Surface Area (km^2)	107.5	42.1

59 Both dams have a dedicated flood storage component which is used to mitigate floods. The mitigation benefits of the dam are larger effect on small events. The mitigation benefits of the dam are greater if the dam is below full supply level.

60 The 1999 flood was essentially captured by Wivenhoe dam. Modelling by WMAwater of the 2011 event showed that drawdowns between 0 and 17% had no impact on the peak flow and downstream flood levels. While a 25% drawdown resulted in a decrease in flood levels of 300 mm at the Port Office (Reference 38).

61 A 96 year simulation of Wivenhoe dam carried out by Reference 18 based on daily rainfall showed the dam was above 90% storage capacity 80% of the time and above 75% storage capacity 95% of the time. Reference 18 investigated the impact of different dam levels on the Q100 discharge hydrographs at Port Office using a MIKE 11 model. It was estimated that at full supply level the peak of the flood was approximately 8500m^3/s and at 50% supply level the peak will be reduced by 1800 m^3/s.

62 Because flood events tend to occur in wet periods it can be misleading to use the probability of different storage levels as the likely level before a flood. There is a much higher probability that the dam will be near full supply level at the beginning of the event. This is also confirmed by examining at the historic record. Reference 18 found that prior to 7 out of 9 historic events the dam level would be at or above full supply level. Prior to the 1974 event there would have been enough rainfall to fill the dam to spillway level (Reference 18). The 2011 event was preceded by 2 events (October and December) and was full before the event. While we are unsure whether the first 1893 event was preceded by a large amount of rainfall the subsequent large event (2 weeks later) was.

63 Figure 2 compares peak inflow and outflow for Wivenhoe dam from a number of sources for occasions when the dam is full. Also plotted on the graph is the 1:1 line (or 50% reduction in flow by the dam as recommended by the 2003 Review Panel (Reference 20)). While there is considerable scatter amongst the data points the graph shows below an inflow of 6000 - 8000 m^3/s the attenuation is quite high while around 12000 m^3/s the attenuation is quite low. It is not unexpected that there would be some scatter as two floods could have a similar peak inflow and very different volumes and hydrograph shapes. There is however a reasonable correlation of volume and peakflow. For peak outflows up to 4000 m^3/s (max discharge allowed at Moggill under W3) the discharge is very dependent on the flow occurring in the Lockyer and Bremer Systems.

64 The 2011 event has a peak inflow of 11000 m^3/s (WMA) and 11500 m^3/s (Seqwater) and a peak discharge of 7500 m^3/s (giving a reduction of 32-35%). If you average the peak inflow and outflow to remove some of the oscillations the numbers become much closer.

65 Figure 3 to 5 depict pre and post dam flow at Port Office, Moggill and Savages Crossing. The 50% reduction line, as adopted by the 2003 Review Panel is also shown.

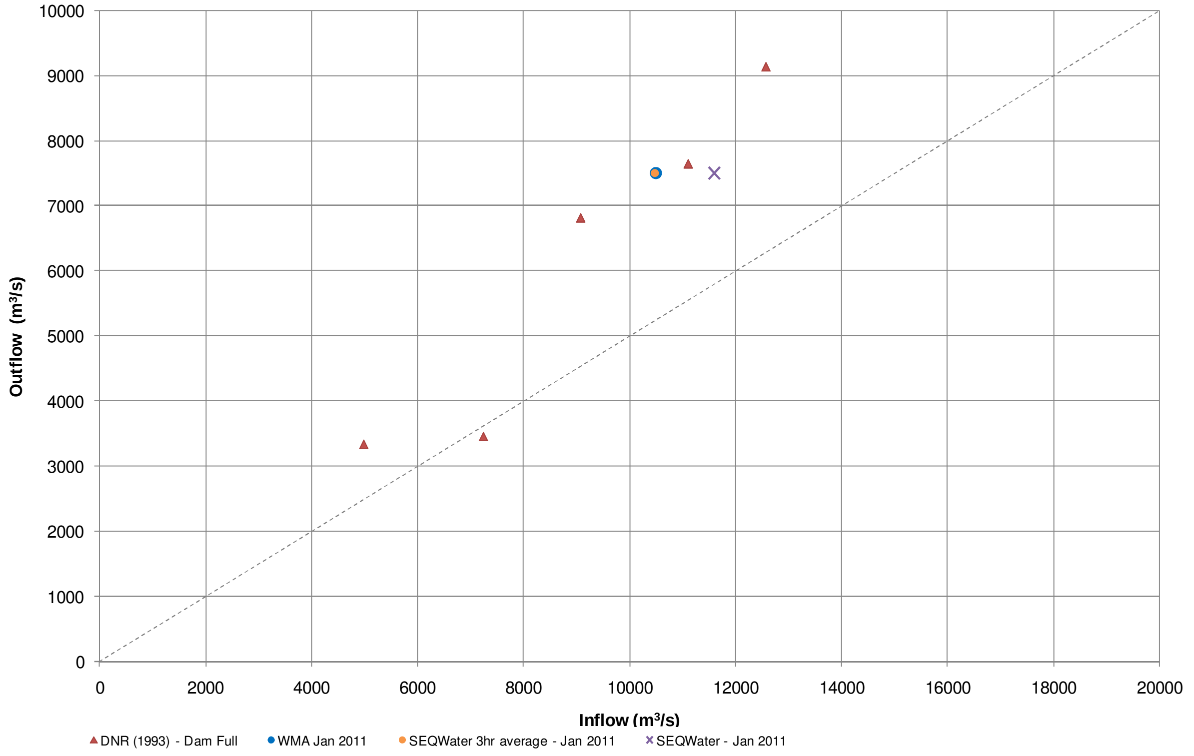


Figure 2: Inflow vs Outflow Wivenhoe Dam

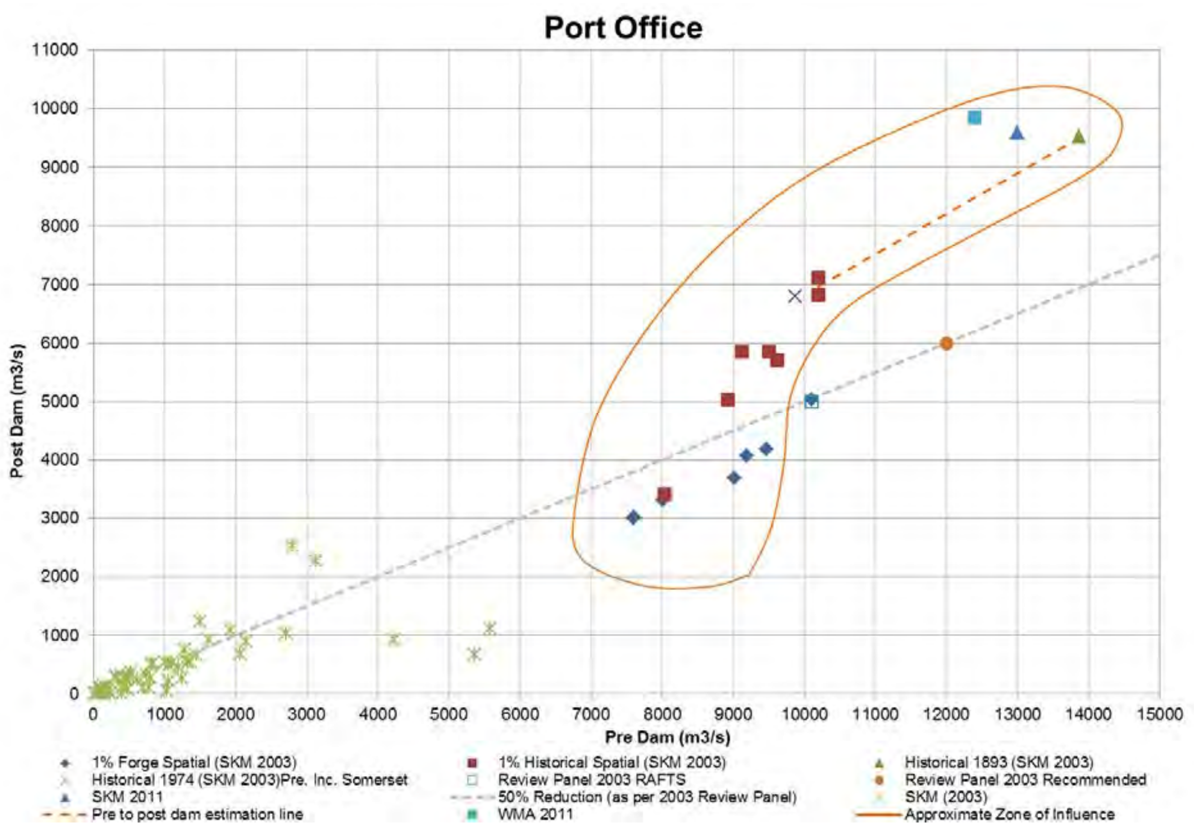


Figure 3: Port Office- Pre and Post dam flow

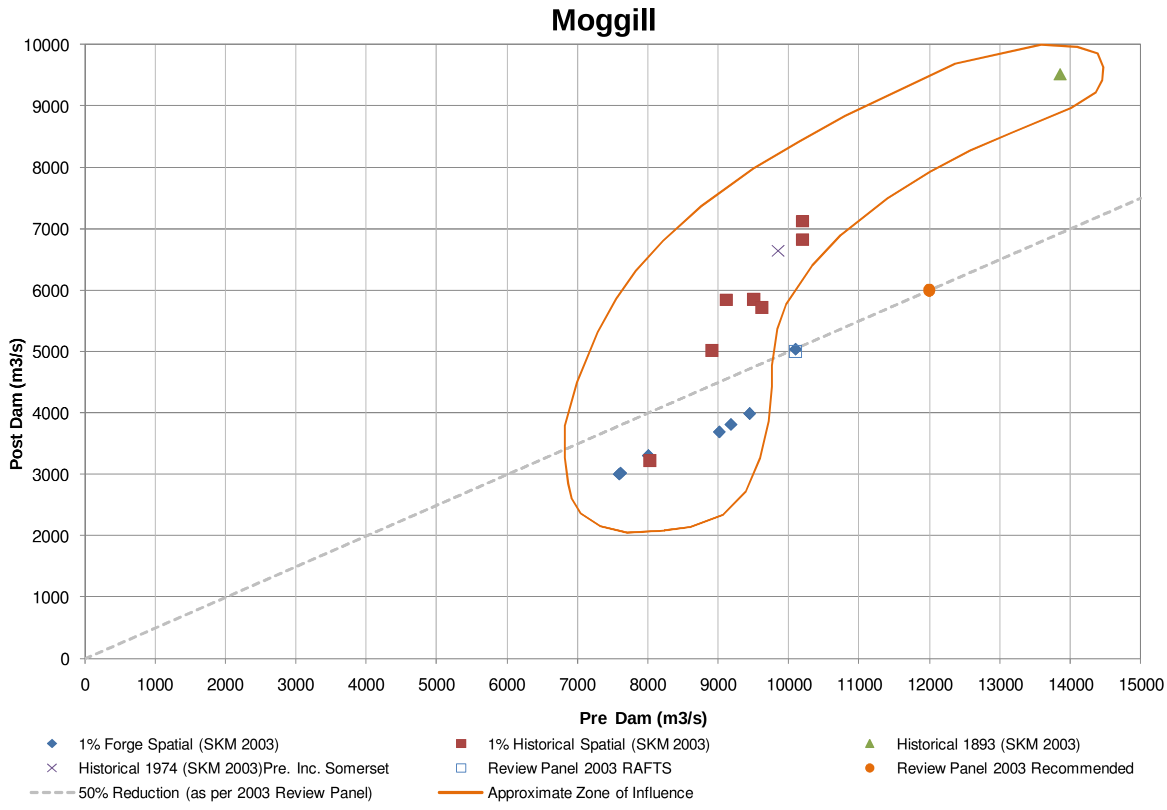


Figure 4: Moggill – Pre and Post Dam flow

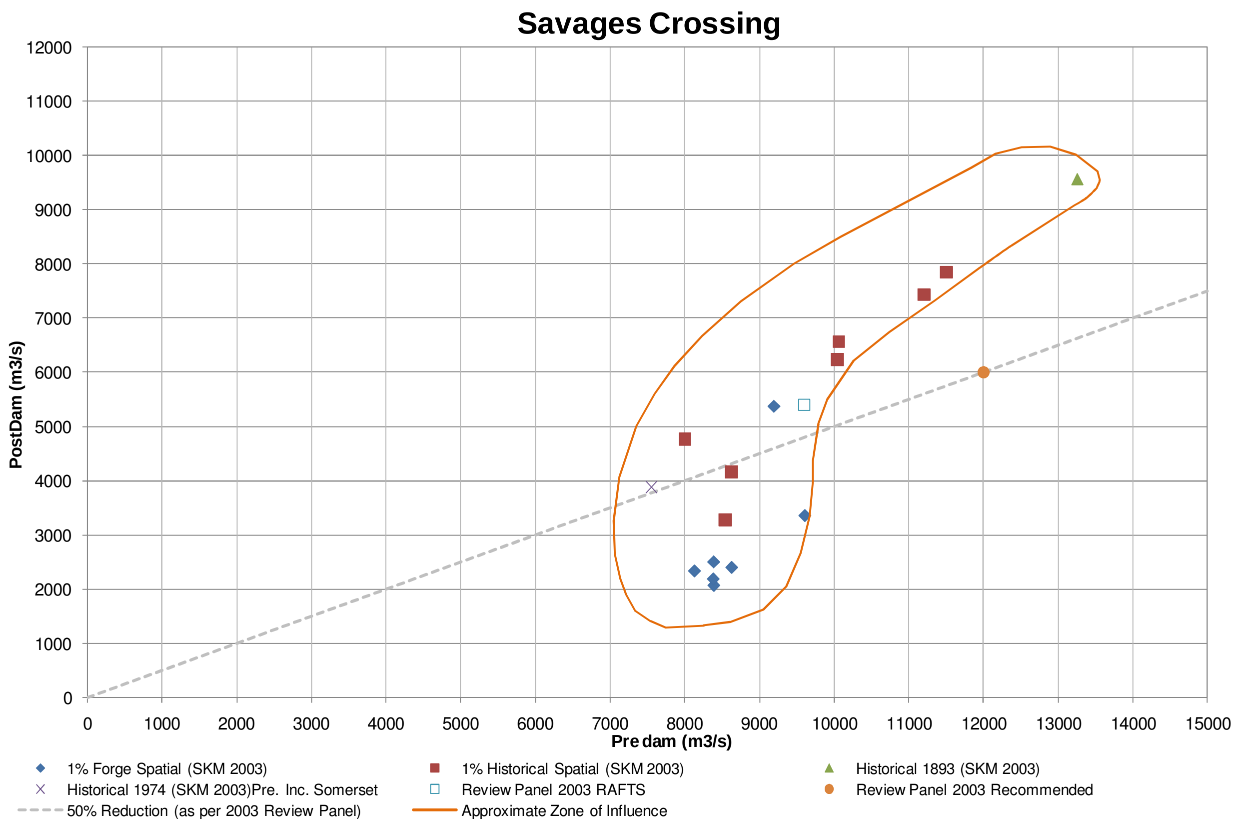


Figure 5: Savages Crossing – Pre and Post Dam Flow

4.4. Other Effects on Historical Flood Records

66 Other factors which influence historical flood record (either effecting the volume of runoff or effecting flow behaviour) and result in a non-homogeneous record include:

- Urbanisation,
- Extraction for water supply,
- Changes in catchment vegetation, and
- Obstructions in the river eg. bridges, walkways which restrict flow and reduce flood storage.

67 Urbanisation increases the amount of impervious surfaces, resulting in an increase in the runoff volume. The effects of urbanisation are most pronounced on small floods and on small catchments. Significant urbanisation has occurred in the Lower Brisbane River Catchment since the 1970's, but the total amount of urbanisation in the catchment is relatively small. Urbanisation can be neglected when assessing flood risk on large rivers like the Brisbane river, however it is important when looking at flooding on small creeks and tributaries.

68 Extraction of water for water supply (eg Wivenhoe dam) lowers dam levels and potentially increases the amount of mitigation that can be achieved by the dam. This is discussed in Section 4.3.

69 Deforestation or the removal of vegetation can also increase the runoff during an event. This effect is much more pronounced in small floods.

70 Obstructions that affect the flow behaviour or restrict storage will result in localised increases in flood levels. In the Brisbane CBD for example there are a lot of obstructions to the flow in the form of bridges and walkways. Most bridges only have a relatively minor and localised impact when the water level is below the underside of the deck.

5. PREVIOUS STUDIES OF Q100 AT PORT OFFICE

71 A significant number of flood studies and subsequent reviews have been undertaken on the Brisbane River. Since the 1970's there have been several revisions to the Q100 flow and flood level although WMAWater understand that the Flood Planning Level (FPL), established in 1984, has remained constant. Lower Brisbane River studies have generally referred to the Brisbane City and/or Port Office gauge. Although these gauges are at the similar chainage along the Brisbane River they are on opposite banks. Further discussion on the discrepancies between the two is discussed in Section 4.1.5 and 6.

72 Following the flood event of January 1974 the Cities Commission engaged Snowy Mountains Engineering Corporation (SMEC) to determine flood damage along the Brisbane River for floods of various magnitudes (Reference 3). This study also produced flood maps that show areas inundated for a range of flood heights between 2 mAHD and 10 mAHD at the City gauge. Included in the study was a flood frequency analysis carried out by Brisbane City Council using an annual series from 1887 to 1974.

73 Grigg (Reference 4) undertook a 'comprehensive evaluation of the proposed Wivenhoe Dam on the Brisbane River' in 1977. Although no Q100 design event flows were estimated for the Brisbane City gauge it is noted that the probable frequency of the Brisbane City gauge reaching 8 mAHD is 1 in 110 years (before the addition of the Wivenhoe dam) based on a flood frequency analysis. Hausler and Porter completed a report on the 'Hydrology of Wivenhoe Dam' in September 1977 (Reference 33) which, although completed before the full design of the dam was completed, includes the dams predicted effects. It was this study which provided the original design estimates of Q100 for Wivenhoe dam although it does not include flood estimates at the Brisbane City gauge.

74 The first study to establish design flows for the area downstream of the Wivenhoe dam was Weeks (1984, Reference 6). This report built upon the findings of his 1983 report on design floods at the dam itself. Design floods were calculated by using the design rainfalls as input into a calibrated runoff-routing model. Weeks (Reference 6) estimated a Q100 flow of 5510 m^3/s at the City gauge when Wivenhoe dam was in operation. This allowed for a peak outflow from the dam of 3500 m^3/s. By January 1985, for the purpose of the Wivenhoe Dam Operations Manual, Hegerty and Weeks (Reference 34) undertook a flood frequency analysis of flooding in the lower Brisbane River catchment taking into account operation of the Somerset and Wivenhoe dams. Flood frequency plots suggest a Q100 peak flow of up to 6800 m^3/s was derived for the Brisbane City gauge (Reference 34). A number of subsequent reports quote a Q100 peak flow of 6800 m^3/s derived in a 1984 study which was apparently used to set the flood planning levels. This 1984 report is referred to in the June 1999 City Design report (Reference 17) as being the "most recent study completed by Council's Water Supply and Sewerage Department". While WMAwater have not been able to obtain a copy of this report, it is thought this may be an earlier version of Hegerty and Weeks, January 1985 (Reference 34).

75 Following the completion of Wivenhoe dam 1985 a number of hydrology reports and design flood estimate studies were undertaken to establish peak flows both at the Wivenhoe dam and downstream in Brisbane City. Greer's 1992 study includes a summary of information from analyses completed prior to 1991 for Q100 and PMF pre and post dam scenarios (Reference 8). This study only references a pre dam peak flow of 11 500 m^3/s for the City gauge to Brisbane City Council 1984. Ayre, Culter and Ruffini undertook calibration of runoff-routing models in 1992 (Reference 9, which it is believed is also known as the DNR report Brisbane River Flood Study 7a). In March 1993 a DNR report (Reference 10) revised the peak flow estimate up to 8580 m^3/s. This report was apparently considered a 'draft' and revised in August 1993 (Reference 14). Reference 14 adopted the higher value of 9120 m^3/s for the Q100 peak flow at the Port Office based on a storm over the whole Brisbane catchment, as this spatial pattern was critical for the PMF. However, a value of 9380 m^3/s was also given for a storm in only the Upper Brisbane catchment and it seems it is this value which has been referenced in subsequent documents (Reference 17 and 23).

76 The next major study is the June 1998 SKM Brisbane River Flood Study - Final Report (Reference 15). This study used hydrologic and hydraulic models that were calibrated to four events and verified against another four events to establish a post dam peak Q100 flow of 9560 m^3/s at the Port Office. A design flood level for this event was estimated as 5.34 mAHD at Port Office. In December 1998, Brisbane City Council commissioned a review of this report by Mein (Reference 16). This review suggested that, although the approach used in the report was appropriate, the magnitude of the Q100 peak flow was an over estimate. The review considered that:

- Too much emphasis was given to historic events in the 1800's suggesting a higher emphasis should be placed on historic events from the 1900s, and
- Questioning the assumption that the dams were full prior to an event was questionable.

77 Revision the analysis based on the above considerations would have the effect of reduing the peak flow estimate. The review was also concerned about the misclosure between flood frequency analysis and the rainfall runoff approach. The concern was focused on the use of zero losses and the absence of an areal reduction factor to reduce the misclosure.

78 A year on from SKM's original Brisbane River Flood Study, City Design Brisbane City Council completed the Brisbane River Flood Study June 1999 (Reference 17). This report suggested that the most recent Flood Study prior to this was in 1984 and that SKM's 1998 study was only completed to draft status. This study addressed some of the recommendation from Mein (Reference 16). The study found that the 1% AEP (100 year) design flood levels in the river were significantly higher than the current development control levels (set by the 1984 study) by 1m up to almost 3m (Reference 17, Section 6). It estimated a Q100 peak flow of 8600 m^3/s and level of 5 mAHD at the Port Office gauge. It concluded that if current development control levels remained that these would have a return period of 1 in 55 years. One of the most important opinions expressed in this report

is *"The simple option of saying that the current development control level represents the 1 in 100 flood level is not valid."* (Reference 17, Section 8).

79 These new flows and level were not adopted and instead City Design prepared a *Further Investigations into the Brisbane River Flood Study* in December of the same year (Reference 18). In this report the peak flow for the 1893 event was reduced and thus reduced the estimate of Q100 peak flow by 600m^3/s to a value 8000m^3/s at the Port Office Gauge and the design flood level from 5 mAHD to 4.7 mAHD.

80 It would appear that this level was still not adopted as a planning level and SKM were commissioned by Council to prepare two further reports which were issued on 8th and 28th August 2003 to an Independent Review Panel (Reference 20). These reports have not been made available to WMAwater, although we have been assured that it is essentially the same as the final report issued December 2003 (Reference 21), it is not known what changes were made between the August and December SKM reports. Based on the two SKM August 2003 reports, the independent review panel concluded that a Q100 peak flow of 6000 m^3/s with dam with an estimated flood level of 3.3 mAHD at the Port Office gauge was a more likely estimate than previous estimates of over 8000 m^3/s. The panel also recommended that a plausible range of ±1000 m^3/s and ±0.5m for peak flow and level respectively was appropriate. The review proposed a pre dam flow of 12 000 m^3/s and that the dams reduced the flow by 50%. It is noted that the 2003 Review Panel terms of reference includes the following statement "*Even if the Q100 changes from 6800 m^3/s, it is likely that the Development Control Level will remain the same as is currently used in the Brisbane City Plan."*

81 In December 2003 the final report was issued by SKM (Reference 21). This report *"used the rainfall-runoff model developed as part of SKM's 1998 study with additional information and statistical techniques to reassess the plausible range of the Q100 flood".* The report gives an estimated Q100 peak flow at Port Office Gauge of 6500 m^3/s with a range from 5000 m^3/s to 8000 m^3/s corresponding to a flood level of 3.51 mAHD with a range of 2.76 mAHD to 4.41 mAHD. SKM suggest that the peaks are lower than in their previous 1998 study as areal reduction factors were used, there was more consideration of variation in temporal and spatial characteristics of rainfall, better knowledge of dam operating procedures and inclusion of regional streamflow information in the statistical flood frequency analysis.

82 SKM re-calibrated the 1998 Mike 11 hydraulic model to determine 1% AEP (100 year ARI) flood levels in a report from February 2004 (Reference 22). Although the December 2003 report had later found Q100 to be 6500 m^3/s, the 2003 Review Panel recommended Q100 flow of 6000 m^3/s at the Port Office gauge be used to giving a Q100 flood level at the Brisbane Port Office of 3.16 mAHD.

83 A number of other studies were undertaken between 2004 and the January 2011 flood event although none revised the Q100 estimate. Following the January 2011 event, the Joint Flood Taskforce released a report in March (Reference 27) which states that the

current Q100 peak flow was last estimated in 2003 to be 6000 m^3/s with a corresponding flood level of 3.3 mAHD including the uncertainty bounds as recommended by the 2003 Independent Panel Review. The Taskforce report (Reference 27) states that at the time of the 2011 flood, Brisbane City Council had defined the Defined Flood Event (DFE) to be 6800 m^3/s and the Defined Flood Level (DFL) to be 3.7 mAHD. This was first set in 1978 and reconfirmed in 2003 (Reference 27, page 17). WMAwater were not provided with any 1978 reports to confirm this, and as this is also prior to the construction of Wivenhoe dam, it is assumed this is may actually a reference to the works undertaken in the 1984 report (a copy of which has not been provided). As an interim approach to apply until the conclusion of the Commission of Inquiry and the conduction of a comprehensive flood study recommended by the Taskforce, the Taskforce recommended that the peak flood level from the January 2011 event now be used as the level on which Brisbane City Council bases its considerations for setting habitable floor levels and decisions concerning new development and redevelopment.

84 Table 5 below summarises the change in estimates of Q100 and flood levels at the Port Office gauge over time based on the reports reviewed. This information is also presented in Figure 6 and Figure 7. It should be noted that these estimates were not adopted by Council for flood planning levels and WMAwater believe that the flood planning levels have stayed constant since either 1978 or 1984.

Table 5: Estimates of Q100 peak flow (including effects of Wivenhoe dam) and flood level at Brisbane City / Port Office gauge

Report/Study Date	**Q100 Peak Flow (m^3/s)**	**Q100 Peak Level (mAHD)**
November 1984	5510	-
1984 (unknown report)	6800	3.3
January 1985	6800	-
March 1993	8580	-
August 1993	9120 / 9380	-
June 1998	9560	5.34
June 1999	8600	5.00
December 1999	8000	4.70
September 2003	6000 (±1000)	3.3 (±0.5)
December 2003	6500 (±1500)	3.51 (range 2.76 to 4.41)
February 2004	6000	3.16
March 2011	-	4.46 / 4.27*

* January 2011 Flood Level at City Gauge / Port Office Gauge

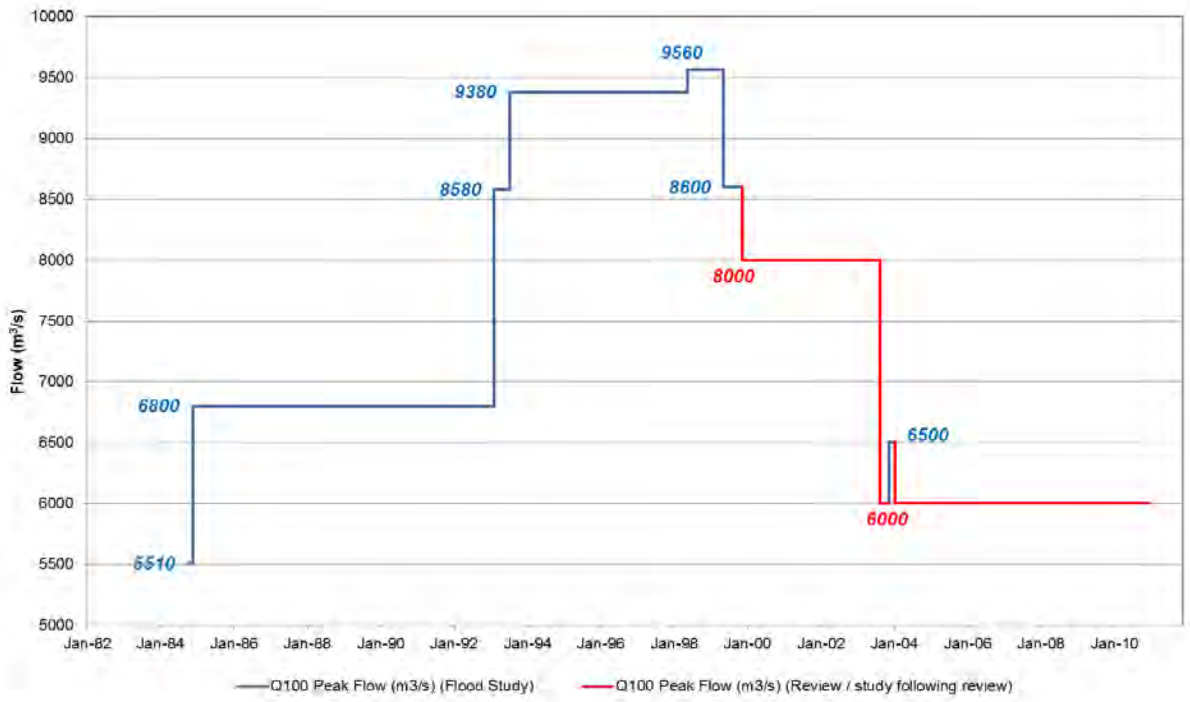


Figure 6: Changes in Q100 Peak Flow Estimates at Port Office/Brisbane City Gauges

Figure 7: Changes in Q100 Level Estimates at Port Office/Brisbane City Gauges

6. RATING CURVES

6.1. Introduction

85 Rating also known as height-discharge curves, are developed for a specific location, usually a gauging station, in order to convert height (stage) into a value of discharge (flow). Ideally a rating curve will be developed for a location with stable cross section geometry. Rating curve accuracy can also be compromised when the location is prone to backwatering or tidal influence. The rating curve is best developed using a series of gaugings (height and discharge observations) from a number of different sized events. The 2011 flood event highlighted the need to revise key rating curves within the Brisbane River catchment.

86 A series of rating curves were developed at the Port Office gauge using the current SKM/Seqwater Mike 11 model (Reference 35), earlier Mike 11 modelling by Brisbane City Council and SKM (Reference 18), estimates of flow from previous studies and information about tides and dredging.

6.2. Deriving a Rating Curve

87 A rating curve is often developed based on a series of data sources and techniques. This may include observations of height and discharge, which are typically limited to smaller events due to the rarity of and access issues associated with large flood events. In order to extend the rating curve beyond observed events, extrapolation techniques are often employed. Extrapolation methods include hydrodynamic models, simple hydraulic equations such as Manning's equation or curve fitting techniques.

88 While a rating curve is best derived from a series of observed height and discharge gaugings, if data is not available or are unreliable a rating curve may be developed using a calibrated hydrodynamic model which should be informed by accurate and relevant bathymetric and topographical data. To ensure the robustness of a rating curve derived from a hydrodynamic model sensitivity to downstream levels, tides and changes in cross section topography should be tested.

6.2.1. Bathymetric Data

89 The previous studies appear to have used three sets of bathymetric data. Detailed survey was conducted in 1873 from Victoria Bridge to Moreton Bay (Reference 18). Following the 1974 flood event the Department of Harbours and Marine carried out a detailed survey (Reference 3, Part 3 Section 3). The 1998 - 1999 SKM and BCC studies (Reference 15, 17, and 18) appear to have accessed newer survey however no date is given. We have not had access to this data in its original form. Data such as this is critical to the development of a rating curve and is important for understanding how the river changes with time. Reference 18 incorporated the 1873 survey into a Mike 11 model to estimate conditions for the 1893 flood event.

90 Collection and review of bathymetric data and its use in a two dimensional (2D) hydrodynamic modelling would substantially aid current efforts to construct rating curves for the Port Office which are representative of height-discharge conditions at the time of large flood events (eg. 1893 and 1974).

6.2.2. Topographic Data

91 Depending on the size of a particular event, rating curves can be sensitive to overbank conditions and topography. Often a change in slope is seen in the rating curve as flow enters the overbank. Future work utilising 2D hydrodynamic modelling to develop rating curves specific to a point in time needs to ensure that overbank topography is representative of the time of the event in question.

92 If a 2D model is developed it should use high resolution survey data (LiDAR) for current conditions. To model earlier events it would be necessary to draw upon a range of data sets including aerial photography, ortho-photo maps, 1873 and 1974 survey details.

6.3. Port Office Rating

93 The Port Office gauge is located approximately 1km upstream of the Storey Bridge on the left bank of the Brisbane River. It is operated by Marine Safety Queensland, part of the Department of Transport and Main Roads. A second gauge, the Brisbane City gauge, is located directly opposite on the right bank of the river. This gauge is operated by Seqwater and is central to the Brisbane flood warning system. In most instances the Port Office and City gauges will record identical information and as such the names of each are used interchangeably.

94 While previous studies have developed a rating relationship there appears to be no official rating curve for the Port Office/City gauge. In the many reports reviewed few details are given in regard to the rating curve used to convert historical stage observations (at Port Office) into associated peak discharges. A consistent feature of the reviewed reports is that the existence of a rating curve can only be implied, other than Reference 15. Although sufficient details of the rating curves are not provided in previous reports, various reports suggest that they have been provided by BoM, Brisbane City Council, or developed based on modelling.

6.3.1. Observed Height-Discharge Data

95 A key objective of reviewing previous reports was to establish observed height-discharge observations for the Port Office, or other locations, that could be used in establishing a reliable rating curve. There were very few documented instances where height and discharge were measured, many of which did not present the actual measurements. For most gauges, gaugings tend to be in the lower range of floods however because the Port Office is effected by tides most tended to be in the upper ranges.

96 During the review of previous reports, information was found that indicated that many gaugings had previously been undertaken. Reference 3 indicated that significant gauging of discharge had been carried out for events in 1931, 1951, 1955 and 1968 although none of the original information relating to these gaugings has been found in any of the available reports. Further investigation could likely unearth these and such work would be of some help in confirming the rating curve for Port Office.

97 Reference 18 shows that in large floods the peak flow at Moggill, Jindalee and Port Office tends to remain approximately constant. This is supported by model results in Reference 15 and Reference 21 as well as results from the model used in Reference 35. Modelling carried out by SKM (Reference 35) of the January 2011 event where ~ 9,600 m^3/s was measured at Jindalee and hence it can be assumed that ~ 9,600 m^3/s also occurred at Port Office.

6.3.2. Deriving a High Flow Rating at the Port Office

98 The 2011 flood event provides extra information about large floods that was not available to previous studies. By combining the limited information about height and discharge available for the 2011, 1974 and 1893 floods it is possible to estimate a high flow rating curve. This is not an easy task as there are conflicts between the data available.

99 The recorded stage for the 1893 flood event was 8.35 mAHD, however it is necessary to adjust this height for current conditions. Reference 17 adjusted all the recorded stages from 1864 to 1917 by 1.52m (5 feet) except for the 1893 event and assumed the discharge of this event was 14 600 m^3/s. Table 10.2 of Reference 17 presents 5 estimates of flow ranging from 11 300 to 16 990 m^3/s for the 1893 event. Two of these estimates are based on velocity measurements taken during the event at Indooroopilly and Victoria Bridges (16 990 m^3/s and 14 600 m^3/s respectively). Reference 3 (Part 3, Section 2) details problems associated with reverse flow on the inside bend making measuring flow at Indooroopilly Bridge difficult during the 1930's and 1950's. Reference 18 modelled the 1893 event using cross sections from 1873 and estimated the peak flow at 11 600 m^3/s.

100 WMAwater tested the impact of dredging using SKM's Mike 11 model. This relatively simplistic testing indicated that the dredging and river straightening works would effect the 1893 flood level and the 1.52m estimate used by Reference 17 for smaller floods was probably appropriate for the 1893 event. This gives a plausible range of levels for the 1893 event under current conditions of between 6.83 mAHD and 8.35 mAHD and a peak flow of between 11 300 m^3/s to 16 990 m^3/s.

101 Reference 3 discusses issues with gaugings from the Centenary Bridge during the 1974 event. There are several flow estimates for 1974 including 9800 m^3/s (Reference 15), 9873 m^3/s (Reference 17). Information learnt from ratings of the 2011 event at Jindalee suggests that these earlier estimates were slightly low, with a revised Jindalee estimate of

10 900 m^3/s (Reference 35). This gives a plausible range of peak flows of 9800 m^3/s to 10 900 m^3/s and a peak height of 5.45 mAHD.

102 For the 2011 event a flow of 9600 m^3/s has been used with a height range of 4.27 to 4.46 mAHD.

103 A best estimate high flow rating curve has been developed using the above plausible height-flow dataset. The lower end of the adopted curve is based on an average of the rating curves established in the 1998 and 1999 Flood Studies.

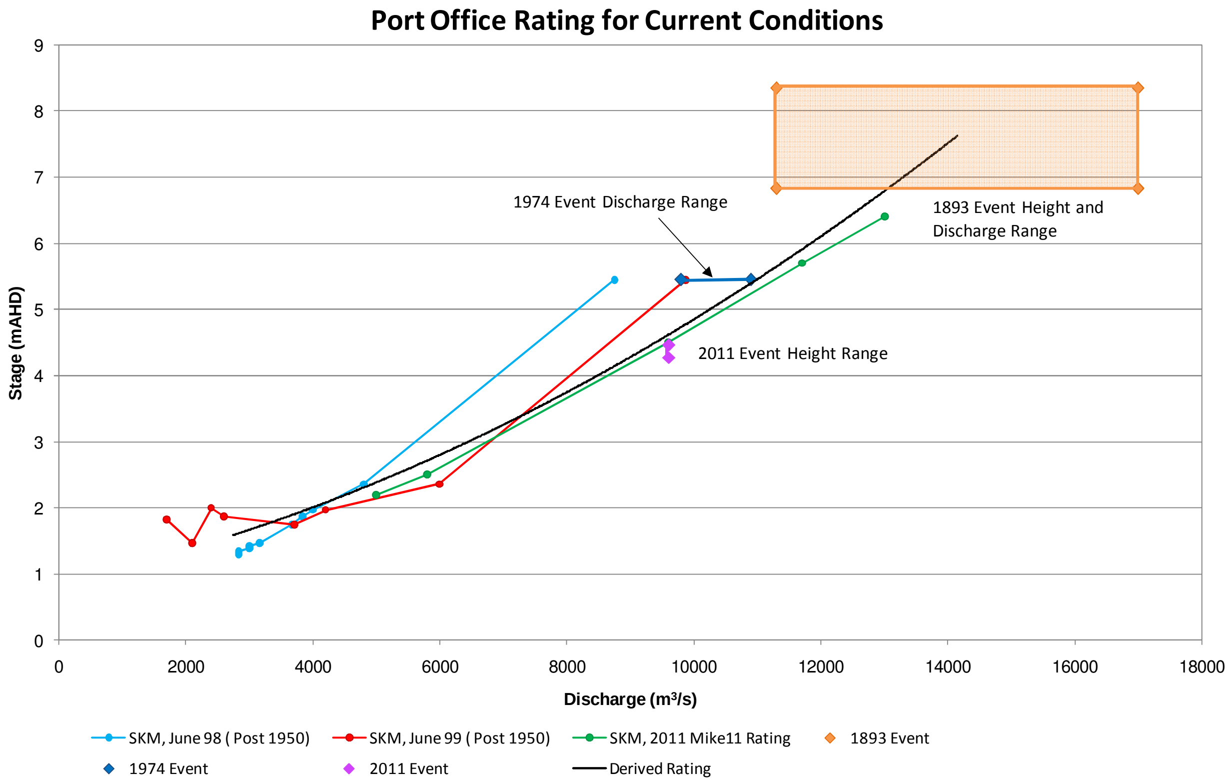


Figure 8: Port Office Rating for Current Conditions

7. ANALYSIS

7.1. Flood Frequency Analysis

7.1.1. Previous Flood Frequency Analysis

104 Flood Frequency Analysis (FFA) has been undertaken previously as discussed in Section 5 (Previous Studies). In particular, extensive FFA was undertaken in 1985 (Weeks), 1993 (DNR), 1998/1999 (SKM), and 2003 (SKM).

7.1.2. Choice of Location for At-Site Analysis

105 As discussed (refer to Section 3), important factors for reliable estimation of larger flood magnitudes include:

- long record length,
- homogeneous conditions over the length of record, which can be affected by:
    - dam construction,
    - changes to the gauge location and/or river cross-section at the gauge,
    - land-use changes, and
    - climate change or long-term climatic cycles, and
- preferably the inclusion of the largest known historical floods within the continuous period of record.

106 While there are several gauges on the Brisbane River, the majority of these gauges have relatively short record lengths (less than 35 years), making them of limited value in estimating larger events such as the 1% AEP (100 year ARI) flood magnitude. For short records FFA becomes an extrapolation method and estimates are heavily biased by what information is contained within the short record. The Port Office gauge at Brisbane is the notable exception with a record of large events from 1841 (over 170 years).

107 With regards to record length, other gauges which are potentially suitable for the estimation of the 1% AEP (100 year) Brisbane River flow include:

- gauge 143001 at Savages Crossing (established in 1909 at Lowood and relocated twice, to Vernor in 1950 and then to the current location in 1958, for a composite record length of 112 years),
- gauge 143003 at Mt Crosby Weir (record length of 76 years from 1900-1975 inclusive), and
- gauge at Moggill (record length of 46 years since 1965).

108 The relative merits of using the gauges identified above for FFA on the Brisbane River are discussed below.

7.1.2.1. Savages Crossing and Mt Crosby Weir

109 The gauges at Savages Crossing and Mt Crosby Weir both record flow from a similar area of the Brisbane River catchment as there are no major tributaries between them. Both records have the problem that they are affected by the construction of Somerset dam (and also Wivenhoe dam for Savages Crossing). Reference 12 (DNR, 1993) accounts for this change by undertaking a separate FFA for each of the two portions of the record, before and after Somerset dam construction. However the analysis gave the illogical result that the 1% AEP (100 year) flow for the later period (with Somerset Dam) was found to be substantially higher than the earlier period.

110 Reference 15 (SKM, 1998) accounted for the construction of Somerset dam by adjusting the flows recorded after the construction of Somerset dam to pre dam conditions and undertaking FFA on the entire record. This approach gave results that were reasonably consistent with results from the Moggill and Port Office gauges.

7.1.2.2. Moggill Gauge

111 The Moggill gauge was established in 1965, after the construction of Somerset dam. The current record length is therefore 47 years (inclusive of 2011), just over a quarter of the length of the Port Office record. The main source of heterogeneity in the Moggill record is the construction of Wivenhoe dam in 1985. Additionally, the gauge has been operational during a relatively dry period compared to other gauge records, with only been two major floods occurring in this period, including the January 2011 flood. The results for larger events from FFA at this site are therefore heavily influenced by whether the January 2011 flood is included or not.

112 Due to uncertainty about the effects of Wivenhoe dam on observed floods, Reference 15 only considered the period from 1965 to 1983 (19 years inclusive). During this period only one major flood event occurred (1974).

7.1.2.3. Port Office Gauge

113 This gauge is subject to heterogeneity from multiple sources. The most notable of which are:

- changes to the river bathymetry, both from natural sources and engineering works. Notable changes that have been identified include channel modification, construction of training walls and artificial islands to alter the natural tidal flow patterns, removal of sand bars, and dredging,
- construction of dams (most notably Wivenhoe and Somerset dams, introducing two step changes into the record), and
- changes to catchment land-use.

114 The major advantage of using the Port Office gauge is the significantly longer record length. This earlier period also captures much more information about large events on the Brisbane River and is summarised in Table 6.

Table 6: Recorded Floods by Period (Adjusted for Pre Dam Conditions)

Period	1841 to 1907 (67 years inclusive)	1908 to 2011 (104 years inclusive)
Number of recorded annual floods greater than 2,000 m^3/s	17	14
Number of recorded annual floods greater than 8,000 m^3/s	5	2

115 The Port Office gauge is therefore considered significantly more likely than the Moggill gauge to adequately capture long term climatic variation for the Brisbane River catchment. Despite the uncertainty introduced by changes to river channel conditions and the uncertainty over whether early flood measurements can be adjusted to current datum. It is therefore considered the best location for estimating the 1% AEP (100 year ARI) flow between Moggill and Brisbane City.

116 For these reasons previous flood frequency analyses have generally been focussed on the Port Office gauge, and it is also considered the most suitable site for conducting FFA for the purposes of addressing The Commission's questions within the scope of this report.

7.1.3. Creating a Homogeneous Data Set

117 Flood frequency analysis needs to be carried out on a homogeneous dataset. In order to do this at the Port Office gauge the flow record needs to be adjusted to consistent conditions. The gate operations used at Wivenhoe dam target specific flows at Moggill. This produces a stepped flow curve for smaller discharges. Flood frequency analysis assumes the flow curve will be smooth, and therefore it must be carried out on pre dam flows.

118 In order to construct a homogeneous data set the flow record was adjusted to represent the peak flow that each flood would produce under current catchment conditions without the presence of Wivenhoe and Somerset dams. The early floods had to be adjusted for dredging, river straightening and the bar that was removed in 1864, while the later floods required adjustment for Somerset and Wivenhoe dams.

119 For floods prior to 1917 the 1.52m dredging adjustment was used and for those prior to 1864 the 0.4m bar adjustment was also used. This is the same approach as used in Reference 17 other than the dredging adjustment has been applied to all events (including 1841 and 1893). For smaller events the flow adjustment used in Reference 17 was also used. For larger events the high flow rating derived as part of the current study was used (refer to Section 6.3.2).

120 Adjustments were made to the 1974 event to account for Somerset dam and to the 2011 event to account for both dams. Every attempt was made to make adjustments in a consistent and non contradictory manner. The adopted high flow estimates are presented

in Table 7 below. It is noteworthy that the 1841, the second 1893 event and the 2011 event are essentially the same size.

Table 7: Homogeneous Data Set of Flood Levels for the Brisbane River

Event	Recorded Level (ie. As measured during the event) (mAHD)	Adjusted Level (mAHD)*	Pre Dam Current Conditions	
			Height (mAHD)	Flow (m^3/s)
1893 (a)	8.35	6.83	6.83	13700
1893 (b)	8.09	6.57	6.57	12600
1841	8.43	6.51	6.51	12500
2011	4.27	4.27	6.40	12400
1974	5.45	5.45	5.50	11300
1844	7.03	5.11	5.11	10400
1890	5.33	3.81	3.81	8100
1898	5.02	3.50	3.50	7500

*Includes 1.52m prior to 1917 and an additional 0.4m adjustment for prior to 1864

7.1.4. Ranking of Events

121 Historic events need to be ranked from largest to smallest in order determine their plotting position. Ranking of events was carried out on the homogeneous dataset (ie. Pre dam levels described in Section 7.1.3. Only the larger events are included in Table 8 though all were included in the subsequent flood frequency analysis. The second 1893 event is not included in the ranked series as an annual series was used in the flood frequency analysis.

Table 8: Ranking of Historic Events (Annual Series)

Event	Pre Dam Flood Level (mAHD)	Rank
1893 (a)	6.83	1
1841	6.51	2
2011	6.40	3
1974	5.50	4
1844	5.11	5
1890	3.81	6
1898	3.50	7

122 Table 8 demonstrates that if the flood frequency analysis focus is only on events in the 20th century then it will result in a very different answer to one which includes the 19th and 21st century flood information.

7.1.5. Plotting Position

123 By considering the rank and period of record it is possible to estimate the most likely probability (AEP or ARI) of each event. The plotting position is used for plotting an observed event on a flood frequency diagram. The plotting position generated by

considering the rank is the most likely probability based on sampling theory and not the actual probability of an actual event. The Cunnane formula is used to determine the plotting position:

$$PP(m) = \frac{m - 0.4}{N + 0.2}$$

Where PP = plotting position
m= rank of the flood in the series.
N= number of years in the record (171 years for Port Office gauge)

Table 9: Plotting Position and Most Likely Probability of Historic Events

Event	Pre Dam Flood Level (mAHD)	Rank	Plotting position (AEP) %	ARI (1/PP)
1893 (a)	6.83	1	0.35	285
1841	6.51	2	0.94	107
2011	6.40	3	1.52	66
1974	5.50	4	2.10	48
1844	5.11	5	2.69	37
1890	3.81	6	3.27	31
1898	3.50	7	3.86	26

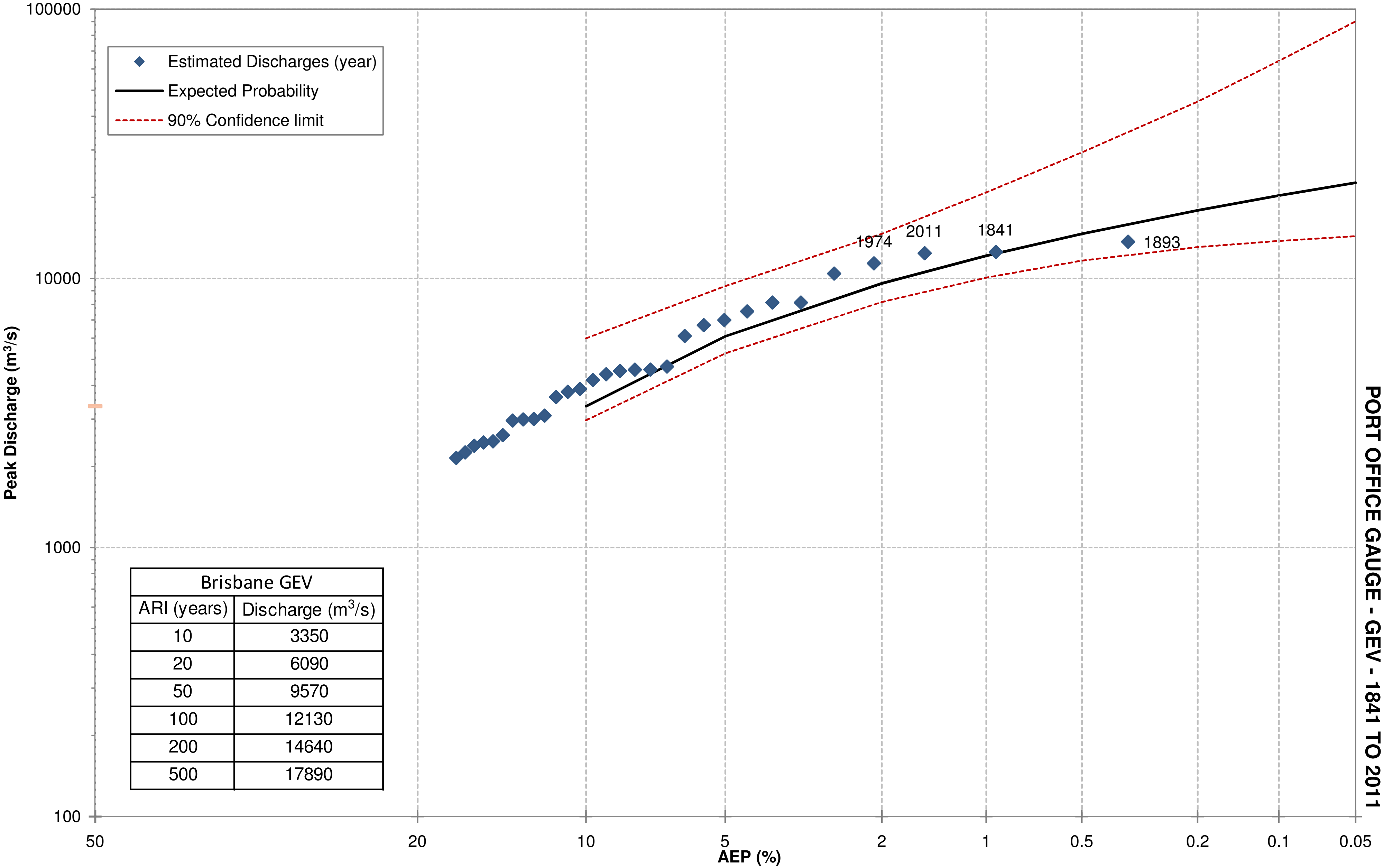
7.1.6. Flood Frequency Analysis – Port Office Gauge

124 Flow data series at Port Office (refer to Appendix B) were analysed using the Generalised Extreme Value (GEV) and Log Pearson 3 (LP3) distributions. Frequency analysis of this location presented a range of complications. While it is a very long record by Australian standards it is very hard to produce a consistent rating curve and properly account for the effects of the astronomical tide and storm surge. This causes the rating curve to be less reliable at low flows and causes a focus on high flows. Figure 9 and Figure 10 show the pre dam fit of the GEV and LP3 distributions. In both cases the fitting algorithm was challenged by the top few floods which have very similar flow values. The fits are sensitive to minor changes to the top few flows.

125 Analysis at the Port Office gauge was undertaken with and without the January 2011 flood data point. Additionally, the analysis was performed on two sections of the flood record:

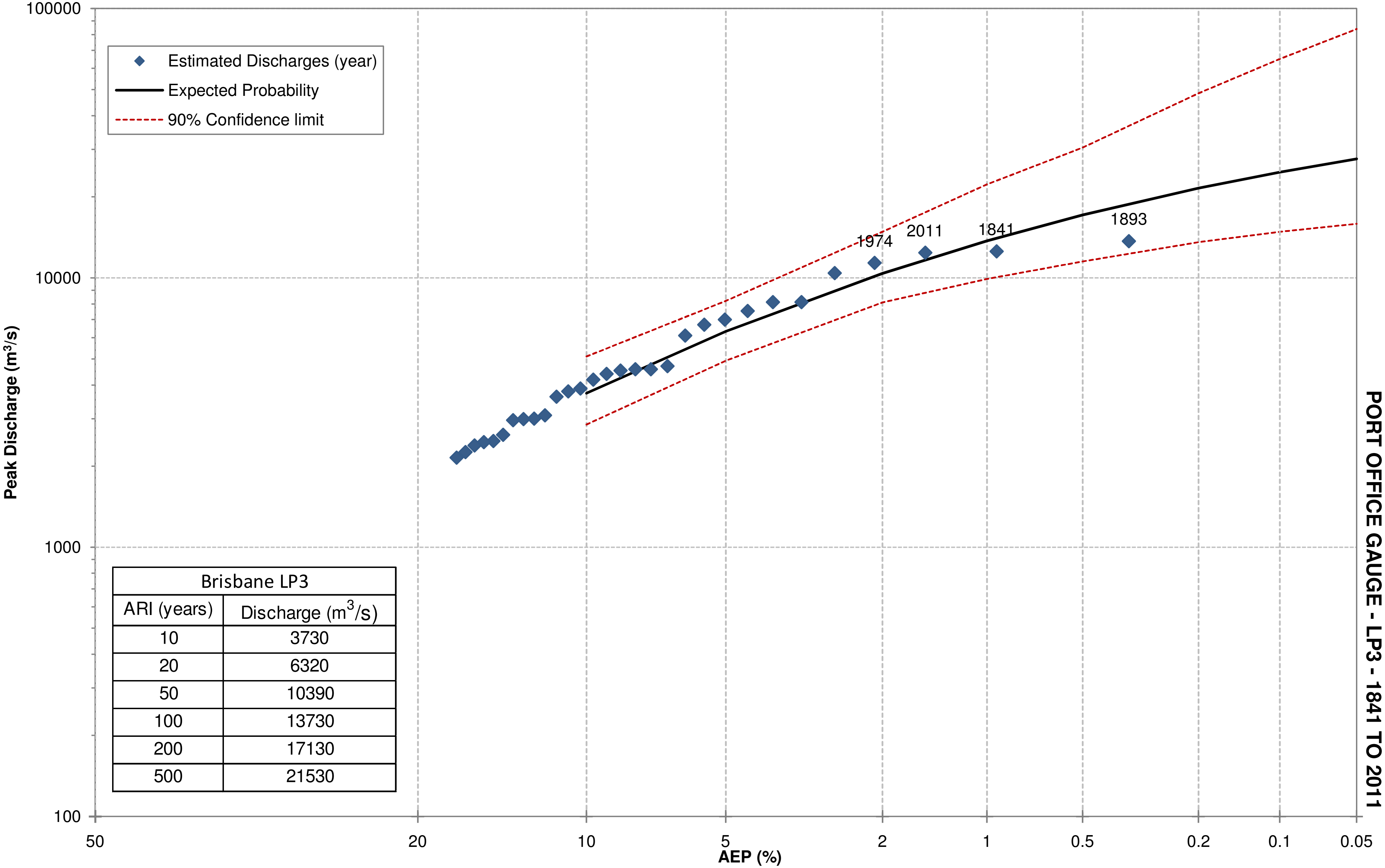
- The full record (1841 to 2010/2011), and
- A partial record, consistent with the period of record from the Lowood/Savages Crossing composite gauge (1908 to 2010/2011).

126 The main purpose of conducting the analysis on the partial record from the 20th century was to ascertain the influence of the "wetter" 19th century period of record on the results, bearing mind that the 19th century period is also more heavily affected by uncertainty from changes to river bathymetry from channel works. This comparison can therefore provide some understanding as to how flood frequency estimates from the gauges with 20th century records such as Moggill and Lowood might change with longer records.



Brisbane GEV	
ARI (years)	Discharge (m^3/s)
10	3350
20	6090
50	9570
100	12130
200	14640
500	17890

FIGURE 9
FLOOD FREQUENCY ANALYSIS
PORT OFFICE GAUGE - GEV - 1841 TO 2011



Brisbane LP3	
ARI (years)	Discharge (m^3/s)
10	3730
20	6320
50	10390
100	13730
200	17130
500	21530

FIGURE 10

FLOOD FREQUENCY ANALYSIS

PORT OFFICE GAUGE - LP3 - 1841 TO 2011

127 Results from the 4 flood frequency analyses undertaken at the Port Office gauge are shown in Table 10 (for both GEV and LP3 distributions).

Table 10: Comparison of Q100 Estimates for Considered Approaches

Data set/ Case	Q100 (m^3/s)	
	GEV	LP3
1841-2011[a]	12 130	13 730
1841-2010[a]	11 740	13 900
1908-2011[b]	10 740	16 610
1908-2010[b]	9 510	13 900

[a] 141 censored flows lower than 2,000 m^3/s
[b] 90 censored flows lower than 2,000 m^3/s

128 Conducting the flood frequency analysis without the 2011 event changes the average of the GEV and LP3 estimates by only 95 m^3/s.

129 The partial record results are influenced by the relative lack of data points during this period. Only 14 floods in the 104 year period are above the 2000 m^3/s threshold. Below this threshold tidal effects at the Port Office gauge have a far greater influence on recorded level than Brisbane River runoff, and the flows determined from a rating table below this level are therefore subject to significant uncertainty. The longer record has 30 gauged floods, resulting in an improved fit with less variability resulting from the distribution assumed for the analysis.

130 The distribution fits and confidence limits for the 1841 to 2011 period are illustrated in Figure 9 and Figure 10. The quantile estimates are shown in Table 11.

Table 11: Flood Frequency Analysis Results (1841-2011)

AEP (%)	ARI	Design Flows (m^3/s) at the Brisbane Port Office	
		GEV	LP3
20	5	440	1740
10	10	3350	3730
5	20	6090	6320
2	50	9570	10 390
1	100	12 130	13 730
0.5	200	14 640	17 130

Note: based on annual series 1841-2011, 141 censored flows lower than 2,000 m^3/s

131 In this case the 1% AEP estimates by the GEV and LP3 are relatively similar, with the LP3 providing a slightly better fit. On this basis a 1% AEP estimate of 13 000 m^3/s was adopted for the pre dam case. This estimate is similar to those of the more recent flood frequency estimates of 13 700 m^3/s (Reference 15) and 12 300 m^3/s (Reference 17).

132 Using Figure 3 without applying any weight to the 2011 event a value of 9000 m^3/s is obtained as the post dam (Wivenhoe and Somerset dams) flow. The 2011 data provides

the only real data point on the performance of the dam and suggests a post dam flow of 10 000 m^3/s using WMAwater's estimate and 9500 m^3/s using SKM's estimate. On the basis of these 3 datasets a post dam flow of 9500 m^3/s was adopted.

133 Based on these conclusions the 2011 flood event has a probability of 0.83% AEP (120 year ARI) under current conditions and under pre dam conditions would have a probability of 1% AEP (100 year ARI).

7.1.7. Uncertainty of Peak Flood Estimates

134 Design flow estimates by their very nature have a considerable level of uncertainty associated with them. The work presented herein trade off the benefits of a very long flow record with the uncertainty associated with the Port Office rating curve. The uncertainty limits shown on Figure 9 and 10, have a larger confidence limit above the flow estimate (expected probability line) than below. The information gained from the 2011 flood event shows that the dams mitigation potential can be considerably less than what was previously assumed by other studies (Figure 3). Weighing up these factors suggests that the uncertainty bound below the best estimate post dam flow (9500 m^3/s) is smaller than the uncertainty bound above the estimate.

7.2. Rainfall Comparisons

135 A major concern of the 2003 Review Panel was the misclose between flood frequency and rainfall runoff estimates. This section examines some of the causes.

136 The flood record is dominated by events between the 1840's and 1890's for which there is very little corresponding rainfall data. Figure 11 shows the number of long term rain gauges (with records longer than 30 years) in the Brisbane River Catchment. Only gauges listed in the BoM Water Resources Station Catalogue (Reference 37) were included in the analysis and obvious duplicates were removed or amalgamated. The 1840's events are potentially captured by only one gauge. The number of gauges increased towards 22 by the 1890. This means that any catchment average rainfalls developed for these events are likely to contain a high degree of uncertainty. This figure also demonstrates that rainfall based methods will be dominated by information from the 20th Century.

137 While the probability of rainfall is not usually the same as the probability of the flood, it can give a general indication of the likelihood of the event. The 2011 rainfall totals for 3 days were compared to the 1987 ARR design rainfalls and a separate series of catchment design rainfalls based on data up to 2009. This more recent analysis also makes use of spatial surface fitting techniques that were not available in 1987. Both design rainfall estimates were only based on official BoM gauges, which do not include alert gauges. Table 12 compares catchment average rainfalls for the catchment to Wivenhoe, the Lockyer, the Bremmer and total catchment to the Port Office gauge.

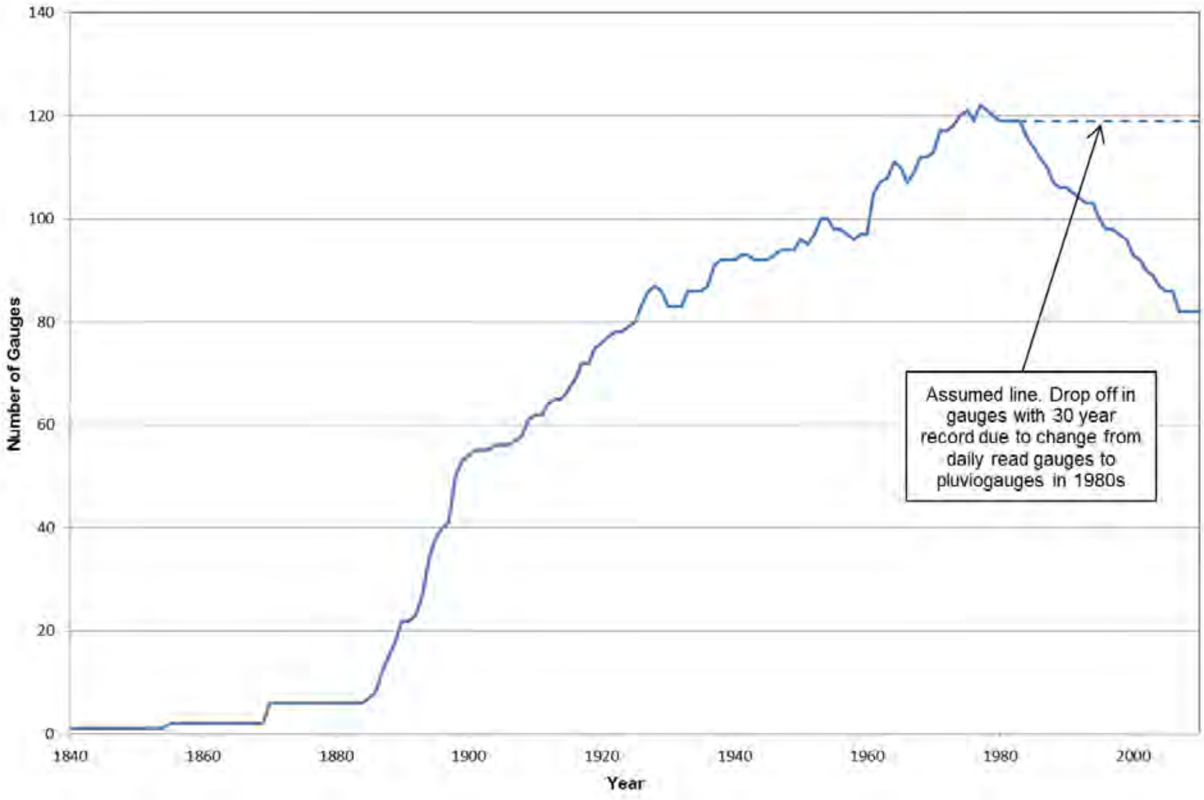


Figure 11: Number of Long Term Rainfall Gauges Available Compared to the Length of Historical Flood Record.

Table 12: Catchment Average Rainfall (mm)

Location	2011 3 Day Peak Rainfall (Seqwater Analysis) (mm)	72 hr 100 Yr ARR 87 (mm)			72 hr 100 Yr based on updated rainfall (mm)		
		No Areal Reduction	0.9 Areal Reduction	0.8 Areal Reduction	No Areal Reduction	0.9 Areal Reduction	0.8 Areal Reduction
To Wivenhoe	326	386	347	309	421	379	337
Lockyer	252	329	296	263	332	299	266
Bremmer	204	350	315	280	390	351	312
All Catchments	280	372	335	298	402	362	321

138 The design rainfall depths have been adjusted using an areal reduction factor. This factor adjusts point rainfall estimates to be used as catchment wide estimates. The use of areal reduction factors have been discussed quite considerably in the 1998 review (Reference 16). While the authors have no theoretical problem with the application of areal reduction factors we have the concern that the BoM rain gauges used in both design assessments (ARR 87 and revised design assessment) have a bias in their location that leads to an underestimation of catchment average rainfall. This is because rain gauges tend to be located on relatively flat land that is suitable for farming or a town. This problem was also found in the 2011 event by SKM (Reference 36) and Seqwater (Reference 26). This is also probably a source of some of the misclosure between rainfall based methods and

flood frequency based methods in some of the earlier studies. This misclosure has been found by the author on other catchments on the east coast of Australia with relatively rugged terrain.

139 Table 12 suggests that on a 72 hour rainfall basis the 2011 event upstream of the dam was slightly larger than a 1% AEP event and slightly smaller than a 1% AEP event downstream of the dam.

7.3. Determining the 1% AEP Line

140 The following locations were identified by The Commission as being of interest:

- 13 Bridge St., Redbank (off-bank),
- Cnr. Ryan St. and Woogaroo St., Goodna,
- Corner Moggill Rd, Birkin Rd, Bellbowrie (Coles),
- Corner Thiesfield St, Sandringham Pl, Fig Tree Pocket,
- 312 Long St East, Graceville,
- Brisbane Markets, Rocklea,
- Softstone St, Tennyson (Tennyson Reach apartments),
- 15 Cansdale St, Yeronga,
- 42 Ferry Rd, West End (Aura apartments),
- 81 Baroona Rd, Paddington (Epic Cycles), and
- Brisbane City Gauge.

7.3.1. Mike 11 Model

141 The Mike 11 Model (Version 2) developed by SKM for Seqwater as described in Reference 38 (and Version 1 described in Reference 35) was calibrated by SKM to Moggill, Jindalee and the Port Office. It was intended to use this model to fit a flood surface between Moggill and the Port Office for a peak post dam design flow of 9500 m^3/s. When this model was compared to observed flood height data in the 2011 Joint Taskforce report (Reference 27, Table 3) problems were found with the fit (Figure 12). While the model fitted well at Moggill, Jindalee and the Port Office the fit was slightly low between Moggill and Jindalee and up to 1.8m low between Jindalee and the Port Office. This problem demonstrates the need for organisations to consider all agencies data when calibrating models. The Mike 11 model was therefore unsuitable to be used in profile generation.

7.3.2. Profiles

142 As part of the prescribed work scope The Commission required profile information on peak flood levels between Moggill and the Brisbane River mouth for the 2011 event and 1% AEP. January 2011 levels at each location were estimated by adjusting the Mike 11 model results to match the observed data from 2011 Joint Taskforce (Reference 27). From this, approximate flood levels at the points of interest identified by The Commission were

determined. The same process was adopted for the 1% AEP flood levels using a peak post dam flow of 9500 m^3/s. These profiles are presented on Figure 13 and summarised in Table 13.

Table 13: Estimated 1% AEP Peak Flood Level and 2011 Peak Flood Level for Locations on the Brisbane River

Location	Estimated 1% AEP Peak Flood Level (mAHD)	Approximate January 2011 Peak Flood Level (mAHD)
13 Bridge St., Redbank (off-bank)	16.81	17.21
Cnr. Ryan St. and Woogaroo St., Goodna	15.96	16.37
Cnr. Moggill Rd. and Birkin Rd., Bellbowrie (off-bank)	14.63	15.04
Cnr. Thiesfield St. and Sandringham Pl., Fig Tree Pocket	10.86	11.22
312 Long St. East, Graceville	9.76	10.10
Brisbane Markets, Rocklea	9.51	9.84
Softstone St., Tennyson (Tennyson Reach Apartments)	9.58	9.90
15 Cansdale St., Yeronga (off-bank)	8.58	8.85
42 Ferry Rd., West End	6.55	6.75
81 Baroona Rd., Paddington (off-bank)	5.77	5.95
Brisbane City Gauge	4.32	4.46

143 Sensitivity testing using the flow estimate from the 1841 to 2010 data set found that the 1% AEP (Q100) height estimate at Moggill and the Port Office would reduce by approximately 0.5m and 0.2m respectively. While there are only minor differences in pre dam estimates (between the 1841-2010 and 1841-2011 datasets) the conversion to post dam, without knowledge gained from the 2011 event regarding dam mitigation ability, results in a post dam estimate of 500m^3/s less.

7.3.3. Review of the 1% AEP Flood Line

144 The 1% AEP (Q100) event as currently defined achieves has a peak level of 3.3 mAHD at Port Office/City gauge. Figure 13 shows Brisbane City Council's current "Q100" flood level profile (adapted from Reference 27). When this is compared to the revised 1% AEP flood profile based on 9500 m^3/s there is up to 3 metres discrepancy between the two near Moggill and a 1m difference at Port Office gauge.

FIGURE 12

SKM MIKE 11 JANUARY 2011 PEAK LEVEL PROFILE VS OBSERVED 2011 FLOOD LEVELS (2011 REVIEW PANEL)

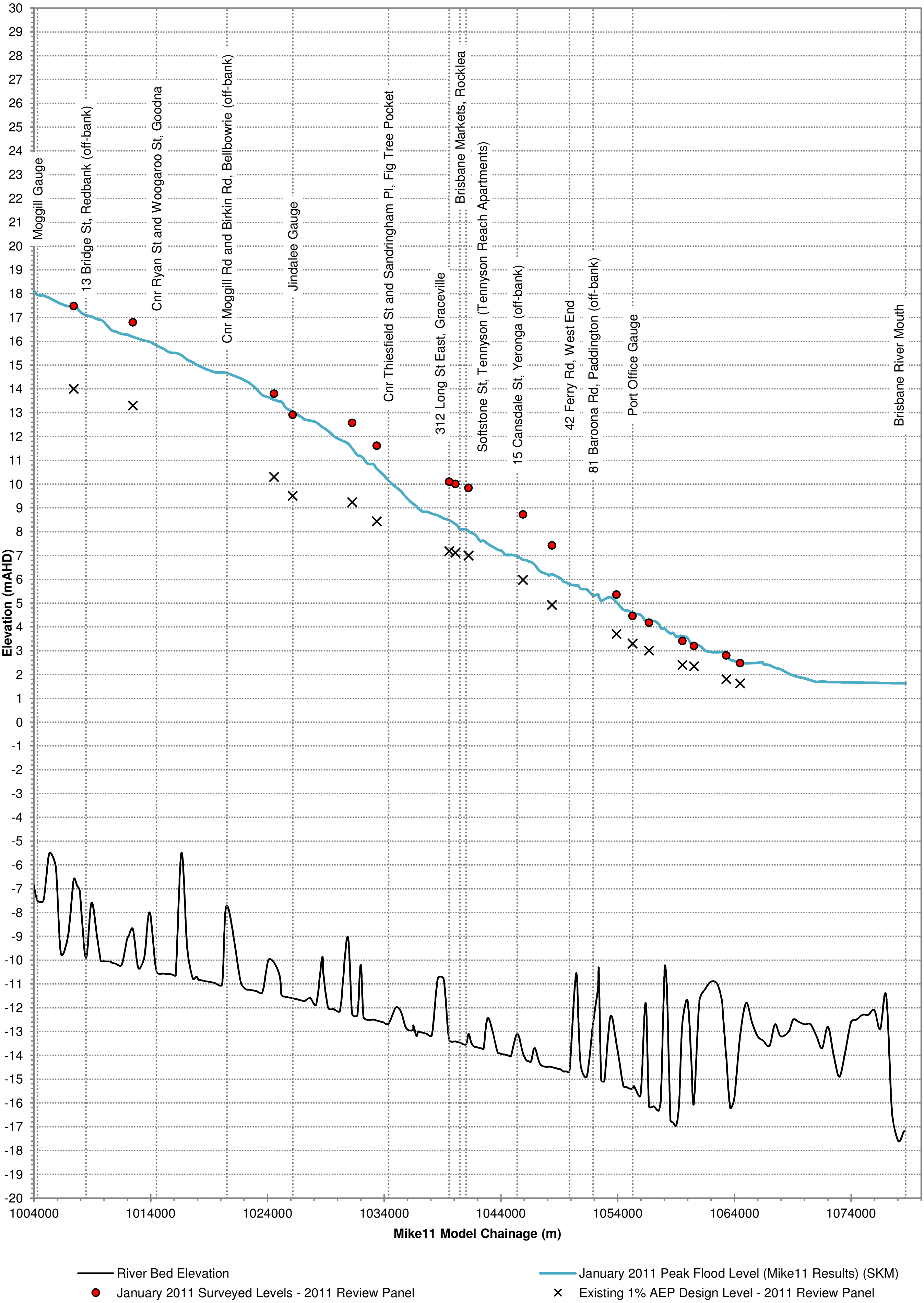
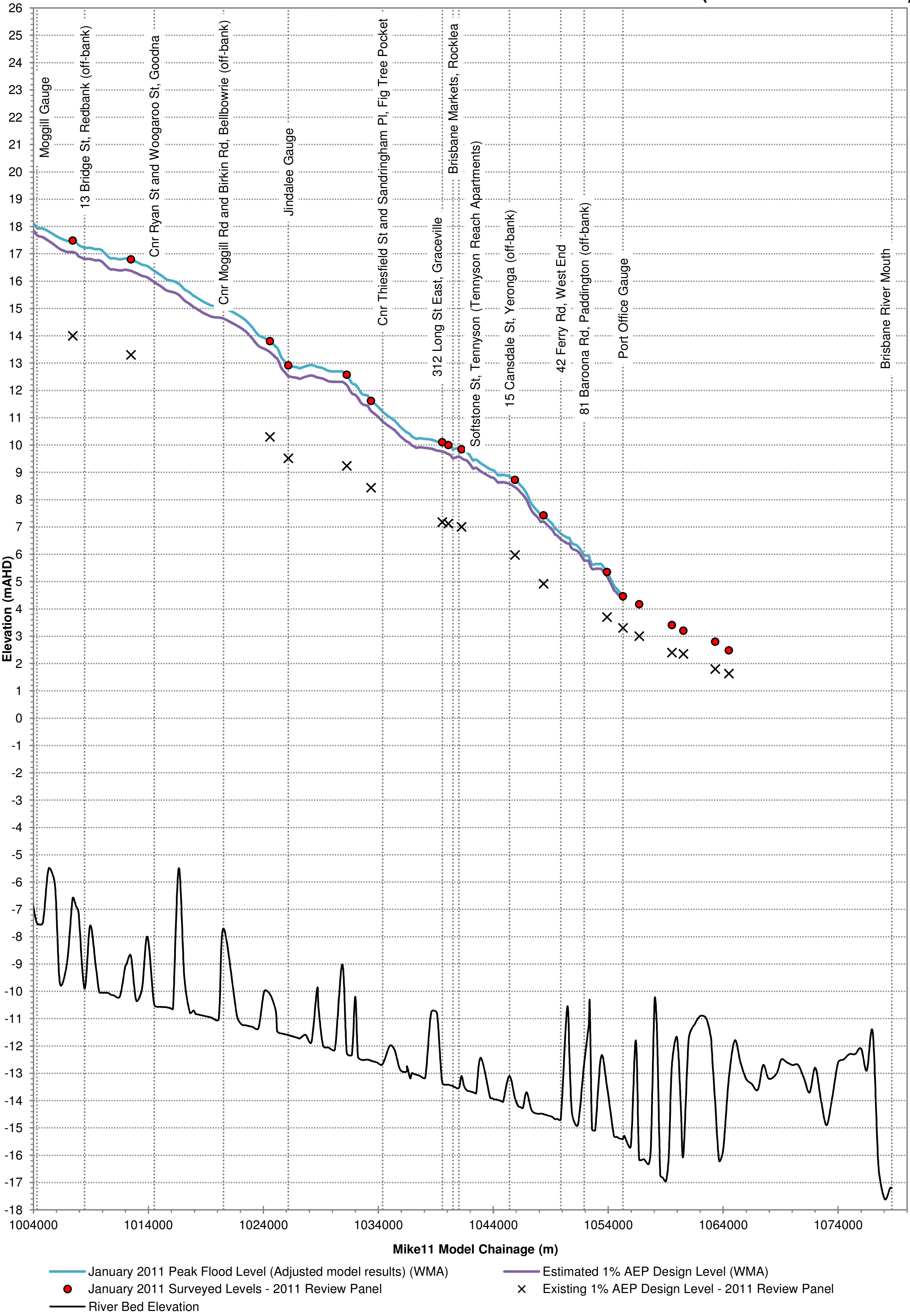


FIGURE 13

Q100 LINE & JANUARY 2011 PEAK LEVEL PROFILE VS OBSERVED 2011 FLOOD LEVELS (2011 REVIEW PANEL)



8. CONCLUSIONS AND RECOMMENDATIONS

145 A flood frequency analysis has been carried out on the Port Office gauge record for the period from 1841 to 2011. The 2011 flood shows the credibility of the large floods that occurred early in the settlement of Brisbane. This analysis gives a pre dam flow of 13 000 m^3/s which is consistent with many earlier estimates. This estimate is not sensitive to the inclusion of the 2011 flood. This pre dam estimate translates to a post dam estimate of 9500 m^3/s. This estimate is slightly sensitive to the new information gained from the January 2011 event on how Wivenhoe dam mitigates large floods. Without this new information the post dam flow would 9000 m^3/s (500m^3/s less).

146 The current Q100 flood line used by Brisbane City Council is significantly below the revised 1% AEP (Q100) flood line calculated by this study with a difference ranging from approximately 3m at Moggill to approximately 1m at the Port Office. The new line is slightly below observed levels of the 2011 flood event. The frequency analysis found that the 2011 flood has a return period of approximately 120 year ARI with Wivenhoe and Somerset dams in place (post dam) and a return period of approximately 100 year ARI under pre dam conditions.

147 The major source of uncertainty in estimating flood risk for Brisbane comes from the uncertainty of the rating relationship at the Port Office gauge. While this is not an easy location to generate rating curves it is necessary if the benefit of the long term gauge record is to be properly utilised.

8.1. Improving the Rating Relationship at Port Office Gauge

148 A detailed study needs to be undertaken to improve the rating relationship at the Port Office gauge. This study needs to draw upon all the information held by Council and State Government. The rating information held by different organisations also needs to be consolidated and objectively reviewed.

149 The study needs to contain the following components:

- Development of a suitable industry standard 2D hydrodynamic model of the lower reaches of the Brisbane River. This model needs to be suitable for assessing historical changes to the river bathymetry and needs to have a run time that is practical for detailed calibration and assessment of changes,
- A detailed search of all data sources on the bathymetry of Brisbane River needs to be undertaken. This study needs to produce best estimate maps of the bathymetry at different times during Brisbane's development. A current survey of the bathymetry also needs to be undertaken and the current morphological behaviour of the river needs to be understood,
- Astronomical tide need to be calculated for the flood events that occurred prior to the regular recording of tides,
- Where sufficient tidal and meteorological information is available the storm surge component at the river mouth needs to be estimated for each historical event,

- The methodology that has been developed under Research Project 18 of Australian Rainfall and Runoff for the calculation of the joint probability of river flooding and elevated ocean levels, should be applied to the lower reaches of Brisbane River so that flood risk can be properly quantified, and
- The sensitivity of flood levels to elevated ocean levels from climate change needs to determined.

Following the completion of the above tasks a revised flood frequency analysis should be carried out using the current best practice. This analysis should explore the use of a regional flood frequency approach.

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Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
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.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
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.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

	redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
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).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
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 fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
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.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the "flood liable land" concept in the 1986 Manual.
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.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State's rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
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.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
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.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

APPENDIX B: Port Office Adopted Annual Series

Table B 1: Port Office Adopted Annual Series

Year	Adopted Values in Current Flood Frequency Analysis (m^3/s)	Source
1841	12534	Section 6
1843	1940	SKM June 1999 report
1844	10410	Section 6
1845	8120	SKM June 1999 report
1852	2252	SKM June 1999 report
1857	2963	SKM June 1999 report
1863	3789	SKM June 1999 report
1864	4574	SKM June 1999 report
1870	3001	SKM June 1999 report
1873	2614	SKM June 1999 report
1875	2455	SKM June 1999 report
1879	2149	SKM June 1999 report
1887	4574	SKM June 1999 report
1889	4525	SKM June 1999 report
1890	8132	Section 6
1893	13690	Average of 5 estimates from SKM June 1999 report
1898	7528	Section 6
1908	6100	SKM June 1999 report
1927	3618	SKM June 1999 report
1928	4398	SKM June 1999 report
1929	3884	SKM June 1999 report
1931	7000	SKM June 1999 report
1955	6704	SKM June 1999 report
1956	4189	SKM June 1999 report
1967	2990	SKM June 1999 report
1968	4704	SKM June 1999 report
1971	2478	SKM June 1999 report
1974	11300	Section 6
1991	2387	SKM June 1999 report
1996	3087	SKM June 1999 report
2011	12400	Section 6

Appendix C

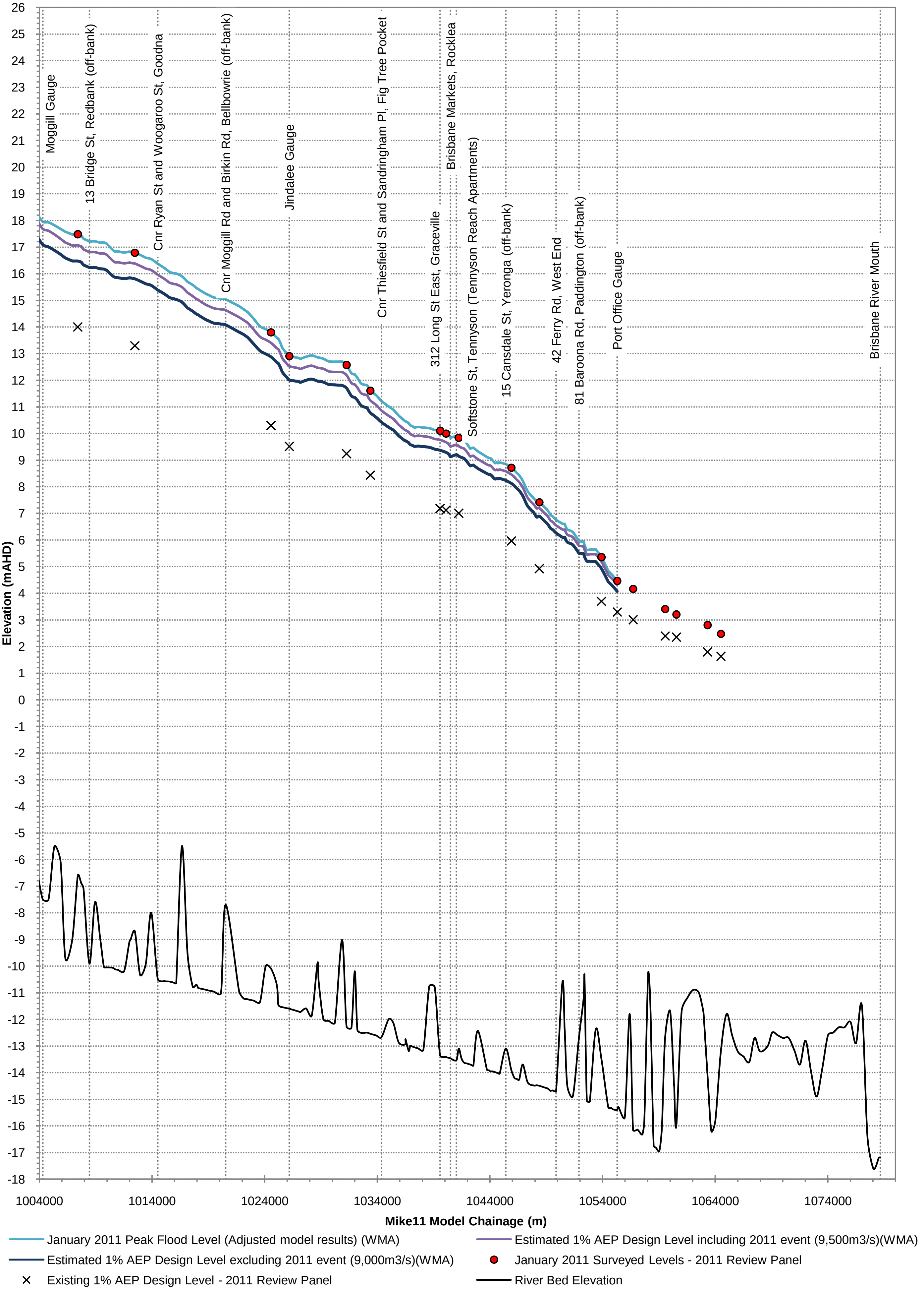
APPENDIX C: Q100 Including and Excluding the 2011 Event in FFA

Table C 1: Estimated 1% AEP Peak Flood Level (Including and Excluding 2011 Event in FFA) and 2011 Peak Flood Level for Location on the Brisbane River

Location	Estimated 100y ARI Peak Flood Level including 2011 event (9,500 m^3/s) (mAHD)	Estimated 100y ARI Peak Flood Level excluding 2011 event (9,000 m^3/s) (mAHD)	Approximate January 2011 Peak Flood Level (mAHD)
13 Bridge St., Redbank (off-bank)	16.81	16.23	17.21
Cnr. Ryan St. and Woogaroo St., Goodna	15.96	15.39	16.37
Cnr. Moggill Rd. and Birkin Rd., Bellbowrie (off-bank)	14.63	14.08	15.04
Cnr. Thiesfield St. and Sandringham Pl., Fig Tree Pocket	10.86	10.42	11.22
312 Long St. East, Graceville	9.76	9.37	10.10
Brisbane Markets, Rocklea	9.51	9.12	9.84
Softstone St., Tennyson (Tennyson Reach Apartments)	9.58	9.20	9.90
15 Cansdale St., Yeronga (off-bank)	8.58	8.24	8.85
42 Ferry Rd., West End	6.55	6.26	6.75
81 Baroona Rd., Paddington (off-bank)	5.77	5.50	5.95
Brisbane City Gauge	4.32	4.07	4.46

FIGURE C1

Q100 LINE (INCLUDING AND EXCLUDING JANUARY 2011 EVENT IN FFA) & JANUARY 2011 PEAK LEVEL PROFILE



Review of Brisbane River 2011 Flood Frequency Analysis

PREPARED FOR

QLD Flood Commission of Inquiry

September 2011

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Executive Summary

This document is a review of the report WMA (2011a) regarding flood frequency analysis of the Brisbane River. The report is herein referred to as the WMA report or as WMA without citing the year. The scope of work requested of Mark Babister by the Queensland Flood Commission of Inquiry was to:

1. Conduct a flood frequency analysis and determine the 1% AEP flood level for key locations on the Brisbane River ... using information available prior to the January 2011 event ... This work should include a review of the SKM 1% AEP flood profile.
2. Repeat task 1 with the 2011 event included in the historical dataset.
3. Using results of task 1 and 2 determine the ARI and AEP of the January 2011 floods at particular points along the Brisbane River and Bremer River.

Addressing Task 2, the WMA report concludes that the post-dam best estimate of the Brisbane River is 9500 m^3s^{-1}, which is based on a pre-dam best estimate of 13000 m^3s^{-1}. The methodology in obtaining both estimates seems justified. Furthermore, the pre-dam best estimate is consistent with earlier estimates, most notably the SKM (2003) best estimate of 12000 m^3s^{-1} ± 2000 m^3s^{-1}. Both flood frequency assessments have given detailed reasoning and used best-practice Bayesian techniques to obtain their estimates. The upward revision of 1000 m^3s^{-1} from the SKM estimate is minor given the uncertainty range and relative orders of accuracy involved. This difference can largely be attributed to the 2011 event and minor differences in methodology. The post-dam estimate of 9500 m^3s^{-1} is higher than the SKM (2003) estimate of 6500 m^3s^{-1} and largely relies on insight obtained from the 2011 flood to suggest the dams had less impact in the region of high flows than the 50% reduction estimate used in the investigations of SKM (2003).[1]

Addressing Task 1, it is the reviewer's interpretation that the task requires a critique of the data and methodologies used prior to the 2011 event rather than an analysis on the influence of 1 data point. The report offers a flow estimate of 9000 m^3s^{-1} using only data available prior to 2011, but the reviewer considers that the authors have implicitly used knowledge of the 2011 event in their argument for a different pre-dam to post-dam conversion of the flow (paragraph 132 and Figure 3). Nonetheless, the report goes a long way towards explaining discrepancies between their estimate and earlier estimates. Reasons offered include (i) confirmation by the 2011 event that early settlement flood estimates are plausible (ii) recent understanding of climate variability (iii) well-known discrepancies between flow-based and rainfall based techniques attributed to few rainfall records in the 1800s and to poor areal rainfall estimates (iv) uncertainty in the stage-discharge relationship (v) lack of large floods to validate the Wivenhoe dam and (vi) significant scatter in the pre-dam to post-dam flow conversion. The report concludes by emphasizing uncertainties in the stage-discharge relationship, but the reviewer feels that greater emphasis should be given to the scatter in the pre-dam to post-dam conversion. This is the largest differentiating factor between the WMA estimate and that recommended by SKM (2003). This point relates strongly to "joint probability" issues which are thorny obstacles in reliable flood estimation (relevant examples include flow peak with flow volume, and the joint distribution of rainfall over multiple catchments). These issues can only be addressed with detailed Monte Carlo assessment, which was a key recommendation of the SKM report (2003, page 48).

Regarding Task 3, this is the matter of applying a hydraulic model to the estimate 1% AEP flow. The hydraulic model is well documented in other reports and is not considered a main obstacle in coming up with flood design levels (as compared to the hydrological issues involved). Following from an upward revision of the 1% AEP flow, the authors note higher design levels ranging from 1m at the Port Office gauge to approximately 3m in the reaches approaching Moggill gauge.

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[1] The expert review panel (2003), on SKM reports, advised 6000 m^3s^{-1}. Only SKM (2003) is referenced for brevity.

Contents

1. Introduction

The scope of work requested by the Queensland Flood Commission of Inquiry to Mark Babister of WMA water was to:

1. Conduct a flood frequency analysis and determine the 1% AEP flood level for key locations on the Brisbane River below its junction with the Bremer River and on the Bremer River in the vicinity of Ipswich using information available prior to the January 2011 event. This work should be used to determine 1% AEP flood levels at up to 8 key locations in the Brisbane and Bremer Rivers and to produce 1% AEP flood profiles. This work should include a review of the SKM 1% AEP flood profile.
2. Repeat Task 1 with the 2011 event included in the historical dataset.
3. Using results of Task 1 and 2 determine the ARI and AEP of the January 2011 floods at particular points along the Brisbane River and Bremer River.

The requested work involves the flood frequency technique such that the overall method required to complete all three tasks is one and the same. The simplest and most ideal case for a flood frequency analysis is that a long record of gauged flows exist, that the catchment has not undergone significant changes in time and that the rate of flooding is relatively constant between differing periods. In this situation the annual maximum series assumes a statistical similarity so that an appropriate probability distribution may be fitted and the Q100 design flow[2] directly inferred. While the Brisbane River does have a long timeseries suited to this method, there are a number of complications in meeting the requirements for statistical similarity. This review will discuss these matters in separate sections by tracing through the main steps of a flood frequency analysis:

- Select annual maximums and homogenize them to reflect equivalent catchment conditions (‘pre-dam’)
- Perform the flood frequency analysis to obtain the Q100 estimate of pre-dam flow,
- Convert the pre-dam Q100 estimate to a flow estimate of current conditions (‘post-dam’)
- Use of the post-dam Q100 flow with a hydraulic model to obtain flood level estimates

Since the commission of Wivenhoe dam in 1984, the estimation of Q100 flows for the assessment of downstream flood planning has been contentious, with best estimate flows ranging between 5510 m^3s^{-1} and 9560 m^3s^{-1} and design levels ranging from 3.16 mAHD to 5.34 mAHD (summarised in Table 5, Figure 6 and Figure 7 of the WMA report). The WMA report, as with preceding reports, documents the history of these estimates and the considerations given over time to resolve known discrepancies. There are a number of related issues at the centre of debate:

1. The higher rate and magnitude of floods in the 1800s and the attendant reliability of their observation.
2. Discrepancies between streamflow based techniques (flood frequency analysis) and rainfall based techniques (e.g. design storm rainfall).
3. Converting measured heights to equivalent representative flows, i.e. the reliability of the stage-discharge curve (incl. correction factors for dredging, sediment build up, channel widening, etc.)

[2] Q100 is a design flow that will be exceeded 1% of the time in a *long run* average (1% AEP, annual exceedence probability). It is synonymous with the term 100 year ARI (average recurrence interval). While it is the 1% AEP flood height at any given point that is of interest, the design methodology requires the 1% AEP flow be defined and that 1% AEP heights are subsequently obtained from this flow.

4. The method for converting "pre-dam" flows to "post-dam" flows (and vice versa)
5. The use of deterministic rainfall methods (current standard practice) versus newer stochastic approaches, i.e. Monte Carlo (being proposed in current revision of Australian Rainfall & Runoff guidelines)

It is necessary, therefore, that the WMA report provide plausible explanations that reconcile these issues to accompany their best estimate of the Q100 flow. In brief, the explanations offered by the WMA report, listed in corresponding order, are:

1. A higher rate of flooding between centuries is plausible. They cite recent research into the influence of Interdecadal Pacific Oscillation as an example. (§3.3.5)
2. The density of rain gauges is sparse in the period of the 1800s (§7.2) and that, assuming it to be a genuinely wetter period, rainfall-based methods over-represent the more recent drier part of the record. Another factor is the use of biased (low) areal rainfall averages since rain gauges are not placed in regions of steeper terrain that coincide with higher rainfall.
3. The stage-discharge relationship is inherently less certain in the region of interest (large floods) but the 2011 floods offer a greater understanding of this relationship (§6). Improved understanding of this relationship is cited as a future means for reducing uncertainty in the Q100 estimate (§8.1).
4. The conversion of pre-dam to post-dam estimates for pre-dam flows above 8000 m^3s^{-1} has been poorly understood and overestimated the performance of Wivenhoe dam (Figures 2 to 5). This is perhaps the single biggest reason offered to explain the discrepancy in Q100 estimates (paragraph 134).
5. This issue is not highlighted in the WMA report, though the SKM (2003) report repeatedly cites this issue (page 6, 36, 37, 41, 42, 46, 48). In the reviewer's opinion this issue assumes central importance in its ability to explain issue 4.

While the WMA report offers satisfactory explanations on all of these issues, it does not place suitable emphasis on the 4th issue, even though this is the most significant factor explaining a difference in Q100 estimates between 6500 m^3s^{-1} (SKM, 2003) and 9500 m^3s^{-1} (WMA). In the reviewer's opinion, the most significant insight of the report is found in Figures 2 to 5 but the discussion in §4.3 and §7.1.7 is brief and deserves a fuller treatment. The emphasis on stage-discharge relationship in the conclusions, while valid, should not outweigh the issue of pre-dam to post-dam conversion and Monte Carlo methods.

A further matter addressed in this review is the role of hind-sight estimates of Q100 and the scope of work implied by Task 1. The WMA report cites a Q100 flow of 9000 m^3s^{-1} excluding all January 2011 flood information, but the reviewer feels the authors have used implicit knowledge of January 2011 in arguing for a different pre-dam to post-dam conversion than SKM (2003). The reviewer speculates that the intended question to be answered is why there might have been a discrepancy between pre-2011 and post-2011 estimates and how this can be reasonably explained.

2. Selection and homogenisation of annual maximums

There history of the discrepancy between the flow based and rainfall based techniques in estimating the Q100 has been well documented. The choice is either to assume the large floods in the 1800s are less reliable (or unreliable) or provide explanations as to why the rainfall based techniques are biased low. The WMA report suggests that the 2011 flood gives credibility to the observations of large floods in the early settlement of Brisbane (§8, paragraph 145). The authors offer detailed background on these early estimates in addition to explanations that might explain the discrepancy between rainfall and flow based techniques.

In an earlier review, Professor Mein (1998, §5) suggested that either the weather was genuinely more extreme in this period or that the flood observations in the 1800s on the Brisbane River should be regarded with suspicion (with endorsement for the latter). As a defence of the former scenario, the authors highlight the existence of climatic variability in §3.3.5, that given periods of a flood record spanning multiple decades can be biased toward either higher or lower flood values. The presentation of Figure 1 is only qualitative and it supports this assertion a little, but the reference to Kiem *et al.* (2003) provides a better and quantitative support for this observation. This reference demonstrates a regional flood frequency using 40 sites across NSW showing markedly different flood distributions between the +ve and -ve phases of the Interdecadal Pacific Oscillation (IPO) up to the 100 year ARI. To the reviewer's knowledge there are no quantitative studies of IPO phases of flooding for the Brisbane region anywhere close to 1% AEP events. The reviewer expects this would be difficult to establish for the Brisbane River record because analysis of the IPO in the 1800s would need to rely on plaeo-reconstructions which do not have the same temporal resolution as the post-1900 reconstructions (see Verdon and Franks, 2006). Nonetheless, it is plausible that natural climatic cycles can lead to multiple larger or more frequent floods in one epoch, followed by smaller or fewer floods in the subsequent epoch.

The report by City Design (1999, §5.1) addressed the concerns of the quality of flow estimates in the 1800s by detailing the methods used to account for the effects of river dredging and blockage of the river mouth. They conclude these flows can be reliably included in the flood frequency analysis and go on to obtain a pre-dam Q100 estimate of 12,300 m^3s^{-1}. WMA (§4, §6) have also chosen to include these observations (notably 1841, 1844, 1890, 1893, 1898) and further to the June 1999 report they provide a detailed account of the height estimates and the history of changes in the river such as dredging works. The authors also discuss the effect of dams on the flow estimates at the Port Office gauge. This discussion is largely concerned with the impact of Somerset Dam on the 1974 event and the impact of Somerset and Wivenhoe on the 2011 event. All these factors are drawn upon in §6.3.2 to construct a stage-discharge curve for the Port Office gauge that is also supported by information from existing stage-discharge curves and hydraulic modelling to test the dredging assumptions of the 1893 event. In this way the adopted flows are suggested to represent flows that would occur under pre-dam conditions for identical river section properties. Appendix B of the WMA report summarises the adopted homogenized flow estimates. Many of the lower flows are identical to the June 1999 report (Table 1) and a summary of changes in the larger flows is given here.

Table 1 Differences in adopted flows due to different stage-discharge relationship

Year	City Design, 1999 (m^3s^{-1})	WMA (m^3s^{-1})
1841	14100	12534
1844	8924	10410
1890	6972	8132
1893	14600	13690
1898	8500	7528
1931	6245	7000
1974	10364	11300
2011	n/a	12400

While the suggested revision of flows based on the WMA stage-discharge curve is justified by the reasoning offered in §4 and §6, a further reason is that the pre-dam Q100 estimate derived by WMA (13000 m^3s^{-1}) is not significantly different from the estimate of 12,300 m^3s^{-1} made by the June 1999 report or the SKM (2003) estimate of 12000 m^3s^{-1} ± 2000 m^3s^{-1} (a point noted by the authors in §8, paragraph 145).

The authors stress the need to improve the stage-discharge relationship (§8.1), which is a valid emphasis as this will help reduce uncertainty in the flood estimate, but it is the reviewer's opinion that this is not the most important emphasis. This issue of uncertainty in the pre-dam Q100 estimate and in the stage-discharge relationship is pursued further in the following section concerning the flood frequency analysis.

3. Flood frequency analysis

Numerous flood frequency studies have been performed on the Brisbane River and these are summarised in §5. Whereas earlier methods used rudimentary "fit by eye" techniques, WMA (and also SKM, 2003) have used a more advanced Bayesian technique (FLIKE, Kuczera, 1999) that has numerous advantages including the ability to (i) incorporate prior or regional information (ii) incorporate stage-discharge uncertainty (iii) assess parametric uncertainty and (iv) allow for thresholded values (censoring).

Whereas SKM (2003) used a regional approach that incorporated prior information, WMA have adopted a frequency analysis solely for the Port Office gauge. Both methods have their merits, and the contrast is not of interest here since they derive similar pre-dam estimates. WMA have adopted a threshold of 2000 m^3s^{-1} (based on Figure 8) so that tidally effected values below this threshold can be incorporated into the method without needing to specify their exact value. For the full record there are 141 values below this threshold and the 30 values above this threshold are listed in Appendix B.[3] The authors considered two common distributions the GEV and LP3 distribution and 4 different scenarios: (i) full record, 1841-2011 (ii) full record omitting the 2011 event, 1841-2010 (iii) partial record matching the Lowood/Savages period, 1908-2011 (iv) Lowood/Savages period omitting the 2011 event, 1908-2010.

The reviewer has repeated this analysis using the same software. It is important to stress the methodology used by WMA and the overall recommendation of a 13,000 m^3s^{-1} pre-dam Q100 are not being drawn into question. If anything the variation on analyses presented here further confirms the 13,000 m^3s^{-1} estimate.

[3] Note: 31 values are inadvertently listed in Appendix B, but the 1843 maximum is below the threshold

The method suggested here is nonetheless recommended as it will result in lower estimate uncertainty estimates and offers a suggestion on the relative influence of stage-discharge curve uncertainty.

The LP3 and GEV are standard 3-parameter distributions used in flood frequency analysis and the use of both distributions provides a comparative check of the methodology. Reviewing the fitted distributions, the reviewer's opinion is that the LP3 gives slightly poorer fits. Furthermore, with technical reasoning outlined in Appendix A of this report, the reviewer considers the 2-parameter Gumbel distribution (simplified from the GEV) to offer a comparable fit, with the chief benefit being a reduction in uncertainty due to one less parameter. The reviewer also recommends that the expected probability of the Q100 quantile is quoted in preference to the Q100 obtained from best expected parameters (assumed usage of WMA)[4]. The results of this analysis are presented in Table 2. Comparing these results to those presented for the GEV in Table 10 of WMA the estimates here are slightly higher due to the usage of the expected probability of the Q100 statistic[3] and the 90% uncertainty limits are smaller due to the use of the Gumbel distribution (limits of WMA Q100 GEV estimate inferred from Figure 9 as being 10,000 m^3s^{-1} to 20,000 m^3s^{-1}). From Table 2 the best estimate is on the order of 13,000 m^3s^{-1} ± 3000 m^3s^{-1} and the influence of the 2011 data point on this estimate is on the order of 500 m^3s^{-1} lower and corresponds well with earlier estimates prior to January 2011 (City Design, 1999; SKM, 2003). The 1908-2011 estimate is of a similar magnitude to the 1841-2010 estimate, but the uncertainty limits are much larger. The 1908-2010 estimate is on the order of 2000 m^3s^{-1} lower and is more sensitive to the removal of the 2011 event as there is less data in this series.

Table 2 Estimates of Q100 flow (m^3s^{-1}) from different scenarios using the Gumbel distribution. 4 different time periods are considered and the effect of stage-discharge curve errors is nominally demonstrated. 90% limits are supplied in brackets.

Year	Gumbel, No Rating Error	Gumbel Rating Error N(1,0.2) above 8000 m^3s^{-1}
1841-2011	13177 (9834,16661)	13351 (9886,16968)
1841-2010	12403 (9225,15688)	12522 (9271,15927)
1908-2011	12384 (7950, 17407)	12430 (7953,17505)
1908-2010	10597 (6803,14958)	10620 (6811, 15007)

Note: The exact values from a large sample (5,000,000) have been provided for sake of reproducibility, but given the magnitude of the values and nature of the estimation they should not be considered more specific than say the nearest 500 m^3s^{-1}.

The reviewer also notes that the FLIKE software provides facility to incorporate incremental errors in the stage-discharge relationship. A further test was done using the Gumbel distribution and allowing for normally distributed incremental errors in the stage-discharge curve (mean = 1, std. dev. = 0.2) for flows above 8000 m^3s^{-1}. These figures have not been determined from detailed consideration of information used in the construction of the stage-discharge curve and they are solely provided for their demonstrative purpose. Table 2 summarises the results of these repeated analyses. It can be seen that there is negligible difference in the best estimate after taking into consideration the orders of accuracy. The upper uncertainty limit is higher, but again this does not seem to be by a significant amount. The scenarios beginning in 1841 are more sensitive to the stage-discharge curve error as they have more events above

[4] The former corresponds to an actual best estimate of the Q100 statistic (the average of Q100s made over many parameter combinations), while the latter refers to the Q100 obtained when applying a single set of parameters (even though they are the "best" individual parameters). The Q100 obtained from the best estimate parameters will be close to, but not coincident with the true best estimate of the Q100 statistic.

the adopted 8000 m^3s^{-1} threshold and the 1908-2010 is the least influenced as only the 1974 event is above this threshold.

A comparison of the GEV and Gumbel distributions is provided in Figure 1 as it highlights the reduction in uncertainty and the similarity of best estimate flows for AEPs between 20% and 1%. In brief, the estimate of 13,000 m^3s^{-1} provided by WMA is confirmed here and its upwards revision from earlier estimates, based on Table 2, can be attributed to the influence of the 2011 event. It is recommended that the estimate is quoted with 90% confidence as 13,000 ± 3000 m^3s^{-1}. Table 2 demonstrates a method for explicitly allowing for uncertainty in the stage-discharge curve, but it is unclear whether the demonstrated values are appropriate and so the estimates are qualitative only. Nonetheless, this demonstration strengthens the reviewer's opinion that the estimation methodology is not as sensitive to the stage-discharge curve uncertainty as to other components.

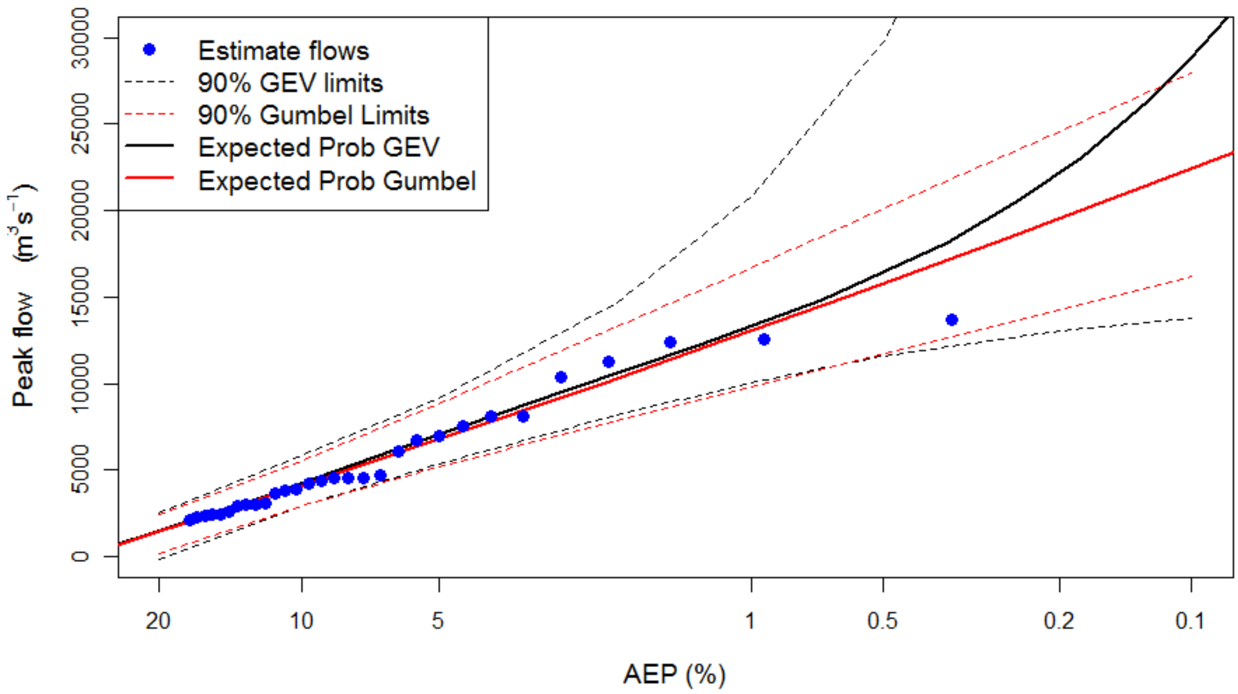

Figure 1 Comparison of GEV and Gumbel fits to 1841-2011 Brisbane River Flood data

4. Pre-dam to post-dam conversion

The previous section has highlighted that the pre-dam estimate provided by WMA is similar to earlier estimates, notably SKM (2003). However, the SKM (2003) post-dam best estimate is 6500 m^3s^{-1} whereas WMA suggest 9500 m^3s^{-1} is a better estimate. The assumption by WMA of a different pre-dam to post-dam conversion than SKM (2003) is based around Figure 3 of the WMA report. This figure summarises (i) extensive modelling undertaken by SKM involving a sensitivity analysis of spatial variation and temporal variation in the rainfall patterns, (ii) estimates derived using the CRC-FORGE method, (iii) the 1974 and 1893 historical events considered by SKM and (iv) the 2011 event. Very little discussion is given of the graph even though it contains a large degree of information. The challenge presented by Figure 3 is that estimates based on the CRC-FORGE and variation in rainfall patterns are supposed to represent 1% AEP estimates. However, these techniques only generate a scatter of pre-dam flow estimates in the range 7000-

10000 m^3s^{-1} whereas flood frequency analyses suggest a Q100 pre-dam estimate of 12,000 ± 2000 m^3s^{-1}. In determining a pre-2011 estimate of the post-dam Q100 flow the authors present a dashed line from the top end of this scatter (which also includes the 1974 event) up to the estimated flow of the 1893 event. Using this line the authors are able to convert the pre-dam Q100 of 13,000 m^3s^{-1} to 9000 m^3s^{-1} without explicitly using knowledge of the 2011 event (§7.1.6 para. 132). In the accompanying discussion the authors readily acknowledge that the 2011 event "provides the only real data point on the performance of the dam" and using the two actual estimates of the 2011 event (SKM 2011, §7, pg. 55, and WMA 2011b, pg. 3 respectively) estimates the post-dam Q100 flow to be 9,500 m^3s^{-1}. The only prior point in the region of high flows is the 1893 estimate, which was considered in the SKM report (2003, pg. 29) to be of questionable reliability (where the consideration was not in isolation but also included attempts to reconcile their method with the lower rainfall estimates and with the inherent scatter in the data underlying Figure 3). Therefore while an estimate of 9000 m^3s^{-1} could well be derived without knowledge of the 2011 event, it places disproportionate confidence in the reliability of the 1893 estimate (implicitly backed up by knowledge of the 2011 event).

The WMA report highlights the considerable scatter and sparseness of existing estimates of the dam's influence (§4.3, para. 63). The authors cite the example that "two floods could have a similar peak inflow and very different volumes and hydrograph shapes". This is very true. Consider for example that if the 2011 event did not have a second peak, the dams would not have required releases and there would be a different factor representing the attenuation of the dam for the same peak inflow. This observation of the joint nature of flood peaks and flood volumes is then countered with "there is however reasonable correlation of volume and peak flow". While true, this statement underplays the degree of difficulty implied by this scatter and seems at odds with the suggested zones of influence of the dam being depicted in Figures 3 to 5. With these zones of influence the authors are trying to highlight that the scatter in the dam's performance is not necessarily linear and likely departs from the assumed 50% line. It is because of a lack of understanding (at high pre-dam flows) and the considerable scatter in the pre-dam to post-dam relationship that a hindsight estimate can recommend a post-dam Q100 of 9,500 m^3s^{-1} where the previous best estimate based on the general scatter and other underlying issues in the data assumed a 50% reduction that gives 6500 m^3s^{-1}.

The conversion of pre-dam flow estimates to post-dam flow estimates is a complex function of the spatial and temporal patterns of rainfall. These patterns lead to the joint occurrence of flood volumes and peaks, but they also lead to other joint probability issues. For example, a rain event that lands exclusively below the Wivenhoe, or in the Lockyer or Bremer systems, will not be intercepted by the dams and so will undergo 0% attenuation. The same amount of rainfall falling in the catchment exclusively above the Wivenhoe may well undergo 100% attenuation. These types of problem are referred to as a joint-probability problems and they present a significant challenge to hydrologic design methods. Other types of joint probability problems include the initial reservoir level at the start of the storm (resolved previously, see §4.3, para. 61), the joint effect of tidal anomalies and freshwater flooding on flood levels in the lower reaches (less relevant to this report).

The chief issue with the overall flood frequency analysis is not with the fitting of distributions to selected data points, rather to the complication of the dams: dam response is volume dependent, but traditional flood frequency analysis is peak-based. There is significant scatter in the conversion of pre-dam to post-dam estimates and the relationship in Figure 3 is not well understood for the higher flows. One of the main recommendations of SKM (2003, pg. 48) was to implement a more exhaustive assessment of this scatter via a Monte Carlo approach. However, there is considerable challenge in implementing this method, since the discrepancy between rainfall–based and runoff-based techniques remains. The WMA report (§7.2) suggests two reasons why the rainfall methods are biased low. The first is that if the 1800s genuinely produced larger floods, then we do not have suitably dense rainfall networks in this period to have captured the events. In other words, the period which we have dense rainfall networks for is postulated to be a drier period. While this explanation is possible it is not clear to the reviewer how it could be tested or how rainfall scaling factors could be reliably estimated with correct long-term frequency in order to overcome this issue. The second reason offered by the authors is that areal averages of rainfall have been underestimated owing to the inhibited pattern of gauges in regions of steep terrain so that they do not capture the most intense parts of a storm. It is not clear whether this observation would account for the entire discrepancy between flow based and rainfall based estimates, but it at least has the benefit that it can be quantified more readily with attention to spatial interpolation algorithms and covariate elevation data. Even if these issues were overcome, it is a non-trivial exercise to generate the spatial patterns for a Monte Carlo estimate. While Monte Carlo estimation techniques are mature to the point of being included in the latest revision of Australian Rainfall and Runoff, the methods for simulating spatial rainfall data are complicated and remain less developed in engineering research and less tested in engineering practice.

5. Obtaining flood level estimates

This review is primarily concerned with the hydrologic aspects of the WMA report. Having obtained the Q100 post-dam estimate of 9500 m^3s^{-1}, the Mike 11 model was used to obtain the 1% AEP flood level estimates and the January 2011 flood level estimates for all lengths of the Brisbane River up to the Moggill gauge. Revisions of this model are documented in WMA (2011b) and SKM (2011). The authors note that while the model matched the January 2011 observations at Moggill, Jindalee and the Port Office well, discrepancies of up to 1.8m were observed at other locations recorded in the 2011 Joint Task Force report. The authors therefore calibrated the model to this data and used it to obtain the flood levels corresponding to the 1% AEP. As a technical matter, more information would have been appreciated on how the calibration of Figure 13 was achieved or whether this can be found in other reports. For example, what are the roughness values in the main channel and flood plain? Comparing the existing 1% AEP levels to the updated estimates (Figure 12), a difference of approximately 1m is seen at the Port Office gauge, at 3 km upstream the difference is approximately 2 m, at 10 km upstream the difference is 2.5 m, at 25 km upstream the difference is 3 m. Based on the frequency analysis, the 2011 event has an approximate ARI of 120 years (0.83% AEP) and the flood levels vary up to 0.5 m above the 1% AEP level in the upper reaches.

6. Conclusions

The estimation of design levels on the Brisbane River contains many complications and sources of uncertainty. The WMA report is concerned with the methodology of flood frequency analysis, which, in its most straightforward mode, is the fitting of a distribution to a set of statistically similar flood peaks. While this method is traditionally focused on streamflow, it seems that the methodology cannot be easily divorced from rainfall-based analyses because of the need to convert the pre-dam flow estimates coming out of the flood frequency analysis back into post-dam estimates that can be used to obtain the design levels for current conditions.

Over the successive reports on the Brisbane River there has been a discrepancy between the flow-based estimates and the rainfall-based estimates which existing studies have struggled to reconcile. This is in part due to the peculiar occurrence of more frequent and larger floods in the 1800s, where only 1 comparable flood exists in the 1900s. As suspicion has been cast over the accuracy of flood estimates in the 1800s, more emphasis has typically been placed on verifying the assumptions that underpin the rainfall analysis. With the occurrence of the 2011 flood, the WMA report places a renewed confidence in the credibility of flood estimates in the 1800s and offers detailed reasoning to establish the reliability of their estimation. In doing so they offer several reasons to support the plausibility of numerous large floods in the 1800s and to justify their perception that rainfall –based estimates are biased low. The first reason they offer is that there is a recent and growing understanding that flood peaks can be modulated by climatic oscillations spanning multiple decades. On this basis, the irregular rate and magnitude of flooding is more conceivable. They further suggest that, if this claim is proven true, then the lack of rainfall observations spanning the 1800s means that estimates of large rainfalls from the 1900s are biased towards a relatively drier period. While these explanations are plausible, it should be stressed that they remain quantitatively unverified for the Brisbane region (and verification is a non-trivial task). The other reason they offer to potentially explain the low bias in rainfall estimates is that rain gauges do not adequately sample higher rainfall areas that naturally occur in less accessible and steeper terrains. This suggestion is more amenable to being verified.

After detailed consideration the pre-dam Q100 estimate of 13,000 m^3s^{-1} ± 3000 m^3s^{-1} is robust and it agrees with earlier flood frequency estimates. The authors highlight the inherent uncertainty in the stage-discharge relationship, but it is suggested here that while this emphasis is valid, a better understanding of the pre-dam to post-dam conversion of flows is of equal, if not greater significance. As a hindsight exercise the existence of the 2011 flood along with the 1974 and 1893 floods is sufficient to establish the conversion of pre-dam estimates from the flood frequency technique to post-dam estimates (without recourse to rainfall based methods). WMA suggest the best post-dam estimate is 9,500 m^3s^{-1}, and this seems reasonable given the methodology they have followed. However, attempting to apply this conversion without knowledge of the 2011 event, whether explicit or implicit, is considerably more challenging and represents the issue facing the authors of the SKM report (2003) who determined an estimate of 6500 m^3s^{-1}. As a main recommendation the authors of this earlier report recommended a (rainfall based) Monte Carlo analysis as the best means for overcoming this limitation. This recommendation is repeated here as the zones of influence suggested by WMA in Figures 3 to 5 are still subject to uncertainty. However, correct implementation of this technique would face several challenges including the convincing simulation of spatial rainfall patterns and the outstanding issue that rainfall based estimates have yielded lower flow estimates.

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WMA (2011b) Review of Hydraulic Modelling – Queensland Floods Commission of Inquiry July 2011, QLD Flood Commission of Inquiry

Appendix A

Inspecting Figure 9 and Figure 10 of the WMA report it seems that the upper uncertainty bound extends to the vicinity of 20,000 m^3s^{-1}, which seems high. The GEV distribution has the following distribution function

$$F(x;\mu,\sigma,\xi) = \exp\left\{-\left[1+\xi\left(\frac{x-\mu}{\sigma}\right)\right]^{-1/\xi}\right\}$$

Where μ is the location parameter, σ is the scale parameter and ξ is the shape parameter. When the skewness is zero (or otherwise not significantly different from zero), the 3-parameter GEV distribution becomes the 2-parameter Gumbel distribution.

$$F(x;\mu,\sigma,0) = \exp\left\{-\exp\left(\frac{-(x-\mu)}{\sigma}\right)\right\}$$

The benefit of having one less parameter is a reduction in the uncertainty estimate (as the skewness parameter need not be estimated, but is fixed to have the pre-determined value of 0). Qualitatively this decision can be determined by inspecting any (lack of) curvature in the fitted GEV distribution. Quantitatively it can be made by assessing the distribution of the estimated skewness parameter (supplied by the FLIKE software of Kuczera, 1999). If the 95% limits of this distribution contain the value 0, then the skewness cannot be statistically distinguished from 0. In other words, a Gumbel distribution is suitable. Figure A1 shows this observation to hold for the 1841-2011 Port Office record (though perhaps a GEV with fixed skewness parameter at 0.1 would perform just as well or could be argued for on the basis of prior knowledge).

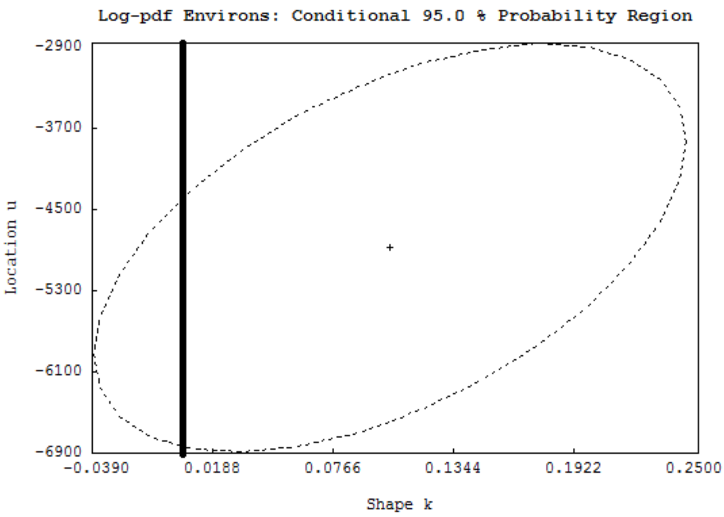

Figure A1 Distribution of GEV shape and location parameters for 1841-2011 data. Best estimate of shape parameter is 0.1, but the 95% confidence interval shows it is not statistically different from 0

It is important to note that this is a statistical observation and that other reasons may apply for retaining the skewness parameter. The most notable is for the extrapolation of estimates beyond the largest observed value. In this instance, the presence of the 1841 and 1893 events as indicated in Figure 9 show that the Q100 is within the interpolation region so that this simplification is justified.

Brisbane River 2011 Flood Event - Flood Frequency Analysis

REVIEW OF REPORT BY WMAwater

- Final A
- 28 September 2011

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1. Introduction

1. This report was prepared at the request of the Queensland Floods Commission of Inquiry. It provides a review of the report prepared by WMAwater (2011) that investigates the "Q100" flood along the lower reaches of the Brisbane River and the probability of the 2011 flood event.

2. For simplicity the term "Q100" is adopted throughout this report to denote the flood that has a 1 in 100 (or 1%) chance of being exceeded in any one year[1].

3. The scope of the report prepared by WMAwater was to provide estimates of:
   - the Q100 flood line on the basis of information and reports that existed prior to the 2010/2011 floods;
   - the Q100 flood line as it stands now, taking into account the data from the January 2011 event; and,
   - the severity of the January 2011 flood along different points on the Brisbane River expressed in terms of its annual exceedance probability (ie, the chance that it might be exceeded in any one year).

4. It needs to be recognised that the above scope represents a most difficult task, particularly as the investigation was undertaken in a very limited timeframe and without the involvement of the two key agencies concerned (namely, Seqwater and Brisbane City Council). In essence the scope requires WMAwater to resolve some vexed issues that have been the focus of a number of detailed investigations and independent reviews over the past three decades. It is thus inevitable that any conclusions drawn from such an investigation will be open to argument and be vulnerable to criticism. In short, this is a complex problem that is subject to considerable investigative constraints: it must be expected that any conclusions drawn are subject to the appropriate caveats, and would be superseded by the more detailed investigations contained in Recommendation 2.12 of the Interim Report prepared by the Queensland Floods Commission of Inquiry (2011).

5. This report should be read in conjunction with the report prepared by WMAwater (2011), but the main points are made in a fashion that should avoid the need for detailed cross-referencing.

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[1] The Q100 flood is more correctly referred to as the "1 in 100 AEP" or "1% AEP" flood, where AEP denotes the "annual exceedance probability" of the event. This flood is also referred to colloquially as the "100 year flood", which is a misleading term that does not correctly capture the notion that the event has a 1 in 100 chance of being exceeded in any one year.

6. It should be noted that the author of this report has been involved in a number of previous investigations relevant to the subject of this review, namely:
   - preliminary risk assessment of Wivenhoe, Somerset and North Pine Dams, commissioned by the (then) South East Queensland Water Board, as reported in Sinclair Knight Merz and Hydro Consulting Hydro Electric Corporation (March, 2000);
   - hydrological investigations into flood behaviour for the lower Brisbane River commissioned by the Brisbane City Council, as reported in SKM (2003);
   - review of hydrological issues relevant to the January 2011 event commissioned by Seqwater, as reported in SKM (2011[a]); and,
   - provision of advice to Seqwater on an ad-hoc basis since January 2011.

7. A summary of the qualifications and experience of the author of this report is provided in Section 2. An overall appraisal of the WMAwater report is presented in Section 3, and more detailed matters relating to the frequency analyses are discussed in Sections 4 and 5. Conclusions and recommendations arising from this review are presented in Section 6.

2. Qualifications and Experience of Reviewer

8. This report was prepared by Dr Rory Nathan, who is currently the General Manager Technology and Practice, and the Practice Leader for Hydrology, with Sinclair Knight Merz (SKM).

9. Dr Nathan holds the following academic qualifications:
   - Bachelor of Engineering (Agriculture) from the University of Melbourne (1980)
   - Master of Science (in Engineering Hydrology) from the University of London (1985)
   - Diploma of Imperial College, University of London (1985)
   - Doctor of Philosophy, University of Melbourne (1990)

10. He has the following professional affiliations:
    - Fellow, Institution of Engineers, Australia
    - Australian Representative, Floods Committee, International Committee on Large Dams
    - Member Hydrology Sub-committee, NSW Dams Safety Committee
    - Honorary Fellow, Department. Civil Engineering, Monash University
    - Past Honorary Fellow, Dept. Civil and Environmental Engineering, University of Melbourne

11. Dr Nathan has over thirty years experience in various organisations in Australia and overseas, covering academia, the public service, and private industry. Of particular relevance to the subject of this review, he was the lead author of the current Australian guidelines for the estimation of large to extreme floods (Nathan and Weinmann, 1999), and was a co-author of the current guidelines on the selection of acceptable flood capacity for dams (ANCOLD, 2000). He is also on the Engineers Australia's Technical Steering Committee for the ongoing revision of the general guidelines for design flood estimation. He has worked on numerous projects concerned with the assessment of flood risk across Australia, in every State and Territory. He has been contracted by the majority of major dam owning and other water resource agencies in Australia to provide consulting and advisory services, independent technical review, and participation in expert panels in formal flood risk assessment processes. He has also been contracted by several U.S. agencies to provide input to the development of flood estimation practice and related guidelines on the characterisation of flood risk.

12. He has published over 150 research papers on engineering hydrology in refereed journals, books, and conference proceedings, and has won several national and international awards for his contribution to professional practice, including:
    - Named as member of *Top 100 Most Influential Engineers* in Australia, 2009;
    - *National Civil Engineer of the Year*, awarded by the Institution of Engineers, 2000;

- Three-times awarded Engineers Australia's *W.H. Warren Medal* for the best paper in Civil Engineering (1992, 1998, and 2005);
- American Society Civil Engineering Journal of Irrigation and Drainage Engineering *Best Research Paper Award* (1997); and,
- *G.N. Alexander Medal* (1998) for the best paper in Hydrology and Water Resources, awarded by Engineers Australia.

13. A more detailed curriculum vitae is provided in Appendix B.

3. Overall Appraisal

14. The following briefly reviews the approach taken by WMAwater to address the scope of investigations as provided by the Queensland Floods Commission of Inquiry. The comments provided below are grouped according to the report sections adopted by WMAwater, and more detailed matters relating to the frequency analyses and assessment of event severity are discussed in Sections 4 and 5 of this report.

15. Section 3 of the WMAwater report presents background material on the context for the Q100 and the salient issues involved in its estimation. This material is well supported by the relevant guidelines and provides a useful overview of the issues involved.

16. Section 4 of the WMAwater provides a concise distillation of material relating to the history of flooding and river engineering works along the Brisbane River since early European settlement. The summary is well targeted to the needs of the investigation, though the information presented on the flood mitigation performance of Wivenhoe dam is based on a selective mix of historical and simulated analyses. This has important implications as noted in paragraph 19 below and in the following Section 4.

17. Section 5 of the WMAwater summarises the outcomes of the flood investigations previously undertaken for the catchment. The information is presented in a manner that emphasises the chronology of the estimates, and little analysis is provided on the differences in hydrologic assumptions, information content, and methodology that is associated with the different estimates. Such analysis would highlight the nature of the supporting evidence, and would clarify the extent to which the changes are due to re-examination of historical data, changes in methodology and operating assumptions, and/or the role of subjective judgement used in the investigations. In other words, while the discussion provides a comprehensive summary of how estimates have changed over time, it does not constitute a critical review that sheds light on the hydrological rationale for the changing estimates of Q100 over time. As discussed in Section 4 of this report, the nature of the factors that influence the flood estimates under "no-dam" conditions have changed little in comparison with those under current conditions where the mitigating impacts of Somerset and Wivenhoe Dams are considered.

18. Section 6 of the WMAwater report discusses the general issues involved in determining a rating curve, as well as a number of specific issues that confound the derivation of flows from flood level information at the Port Office. Determination of reliable rating curves over the period of available flood level information is a tractable problem, but its solution does require relevant bathymetric and tidal information, and careful hydraulic analysis. The rating curve derived by WMAwater makes good use of available information and is consistent with other analyses; the only point of minor disagreement relates to the averaging of rating curve

information over the lower range of discharges. The lower end of the curve is well defined by the constant release of 3500 m^3/s from Wivenhoe Dam made by Seqwater in January 2011 (SKM, 2011[b]); the artificial nature of these flow conditions are well suited to the derivation of the lower end of the rating curve and should be given high weight compared to other evidence.

19. Section 7 of the WMAwater report presents the substantial analyses used to derive the revised estimate of the Q100. Comment on the key issues arising from this is provided in the next section of this report, but in terms of overall appraisal it is suffice to note here that:

    - the broad approach used to undertake the frequency analysis using historical maxima is appropriate;
    - there is reasonably strong justification for the Q100 estimate under "no-dam" conditions as it is largely supported by observations over a 170 year period; but,
    - the justification presented for the Q100 estimate under current conditions is based on an assumption that reduces the operational complexity of Wivenhoe Dam and the associated joint probability issues to a single fixed reduction factor – the analysis involves a somewhat circular argument and relies heavily on the information contained in a single event, and as such, the estimate provided by WMAwater is not considered defensible; and accordingly,
    - the estimates of the Q100 flood levels along the Brisbane River are not supported – the primary reason for this view is because the Q100 flow estimates are not defensible, but it is also noted that more current information on debris marks has not been used.

20. Section 8 of the WMAwater report provides the conclusions of their investigations. As indicated in the preceding paragraph the conclusion drawn regarding the Q100 estimate under "no-dam" conditions is accepted, and the Q100 estimate relevant to current conditions is not. The recommendations made concerning the need to model the Port Office gauge with a hydrodynamic model are supported, but the lack of discussion (and associated recommendations) around the limitations inherent in the treatment of Wivenhoe Dam operations and the associated joint probability issues is considered a significant omission.

4. Flood Frequency Analysis

4.1. Overview

21. The approach adopted by WMAwater to derive an estimate of the Q100 event based on information available prior to 2011 may be summarised as follows:
    a) The Port Office gauge was selected on the basis of relevance and length of record.
    b) Historical *flood levels* prior to 1917 were adjusted to account for dredging works, and the *flow peak* for the 1974 event was adjusted (upwards) to represent "no-dam" conditions.
    c) A statistical distribution was fitted to the historical flood maxima and used to estimate the "no-dam" Q100.
    d) A single factor was applied to the "no-dam" Q100 estimate to derive an estimate of the Q100 under current conditions.

22. The above approach was repeated using data obtained from the January 2011 event (where in step *b* the flow peak for the 2011 event was adjusted upwards to represent "no-dam" conditions), and the Q100 estimates were recalculated to determine the impact of the recent floods.

23. The above steps involve varying degrees of subjective judgement and are underpinned by different levels of supporting evidence. These differences impact markedly on the defensibility of each successive step, as discussed below.

4.2. Adjustments to Historical Flow Estimates

24. *Adjustment of historical flood levels prior to 1917*. The estimates of flood peaks for the events prior to 1917 are largely based on estimates provided by City Design (1999[a]), where the highest events were revised in line with a rating curve derived from the hydraulic modelling[2]. These adjustments attempt to take account of the river engineering works that had taken place, as summarised by WMAwater. The rationale for this adjustment is clear, though it is recognised that the bathymetric information on which the estimates are based is uncertain. The sensitivity testing undertaken by WMAwater would suggest that the adjustments are "probably appropriate".

25. *Impact of Revised Rating Curve*. It would appear that WMAwater revised the flood estimates provided by City Design (1999[a,b]) for the highest historic events on the basis of their revised

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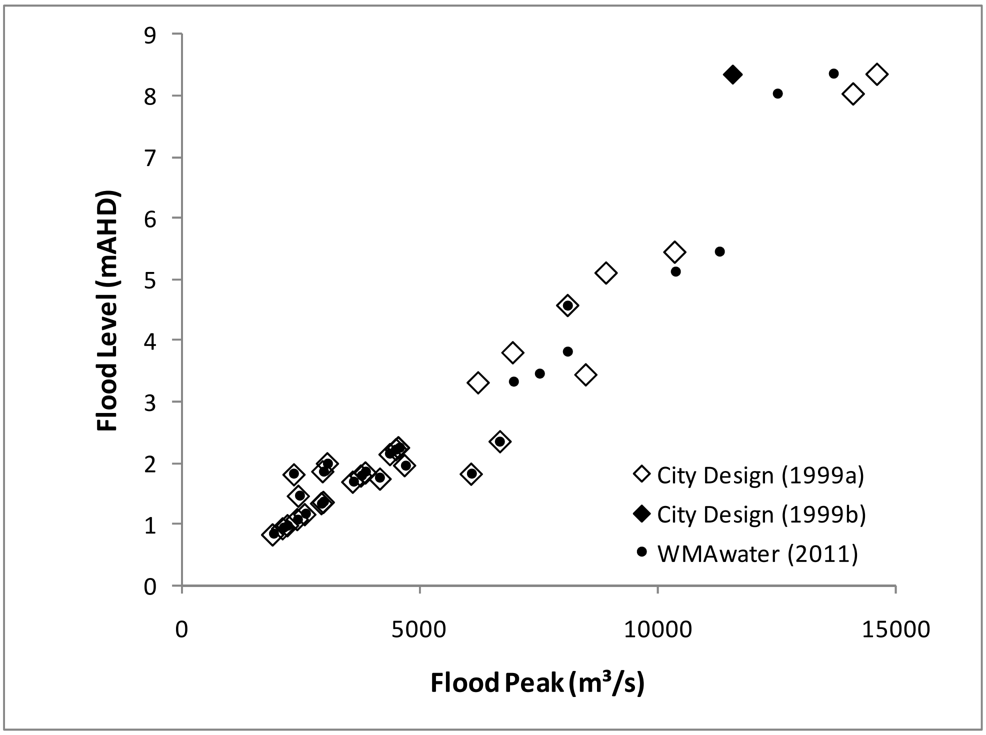
[2] Appendix B of the WMAwater report incorrectly states that the adjusted flood level data was obtained from "SKM June 1999 report" – this citation should read City Design (1999[a]). It is also noted that WMAwater also replaced the 1931 flood level estimated by City Designs (6245 m^3/s) with their own estimate (7000 m^3/s).

rating curve. The difference between the two sets of flood estimates is illustrated in Figure 1. With the information provided it is not possible to comment meaningfully on the appropriateness of the new flood estimates, other than to speculate that the hydraulic modelling approach used by WMAwater is likely to be more defensible than that available to City Designs in 1999. The point of the Figure 1 is to illustrate the influence of this step in their analysis, where at the high flow end the difference in the best estimate of flows is in the order of ~1000-2000 m^3/s.



- **Figure 1. Difference in estimates of historical floods derived by WMAwater and City Designs.**

26. *Derivation of "No-dam" estimate of 1974 peak flow*. No information is provided by WMAwater regarding the means by which the estimate of flow at the Port Office gauge was adjusted upwards to represent "pre-Somerset dam" conditions. Various estimates could be derived from earlier work that are within 1000 m^3/s of the adopted value of 11300 m^3/s (eg Hegerty and Weeks, 1985; SKM, 1998; City Designs, 1999; SKM, 2003) and it is unclear from the text what adjustments were made to account for the revised rating curve and for the removal of Somerset Dam.

27. *Derivation of "No-dam" estimate of 2011 peak flow*. Similarly, no information is provided by WMAwater regarding the means by which the estimate of flow at the Port Office gauge (~9600 m^3/s) was adjusted upwards to represent "no-dam" conditions (12400 m^3/s). It is

assumed that this was achieved using hydrologic inputs and hydraulic model provided by SKM (2011[c]), but adopting different assumptions in some fashion. It should be noted that the estimate derived by SKM (13400 m^3/s) was higher than that adopted by WMAwater, though without further information it is difficult to comment on the relative merits of the WMAwater estimate.

4.3. Frequency Analysis for "No-dam" Conditions

28. WMAwater used the FLIKE software developed by Kuczera (2001) to fit two different statistical distributions to the set of "no-dam" flood peaks. The conceptual nature of the statistical approach adopted (Bayesian maximum likelihood fitted to a censored historical series) represents best practice and is fully supported.

29. It is noted that the difference in estimates of the "no-dam" Q100 arising from choice of the distribution adopted for the full period of record is around 1600 m^3/s, or around 12% of the adopted estimate. This uncertainty reflects the "unknowable" nature of the distribution that governs the distribution of floods, and is additional to the uncertainty arising from the sample of historic maxima considered. It should also be noted that this uncertainty does not reflect the uncertainty inherent in the estimates of the flood maxima, as discussed above in paragraphs 24 to 27.

30. The nature of these uncertainties are common to all such investigations, and the preceding point is made to illustrate that, even if we assumed that we had 171 accurate observations of annual maximum floods, and that they represented a homogeneous sample, estimation of the Q100 is inherently an uncertain business. A reasonable estimate of the uncertainty of the "no-dam" Q100, even with these 171 years of observations, is around 30% to 40% of the adopted value.

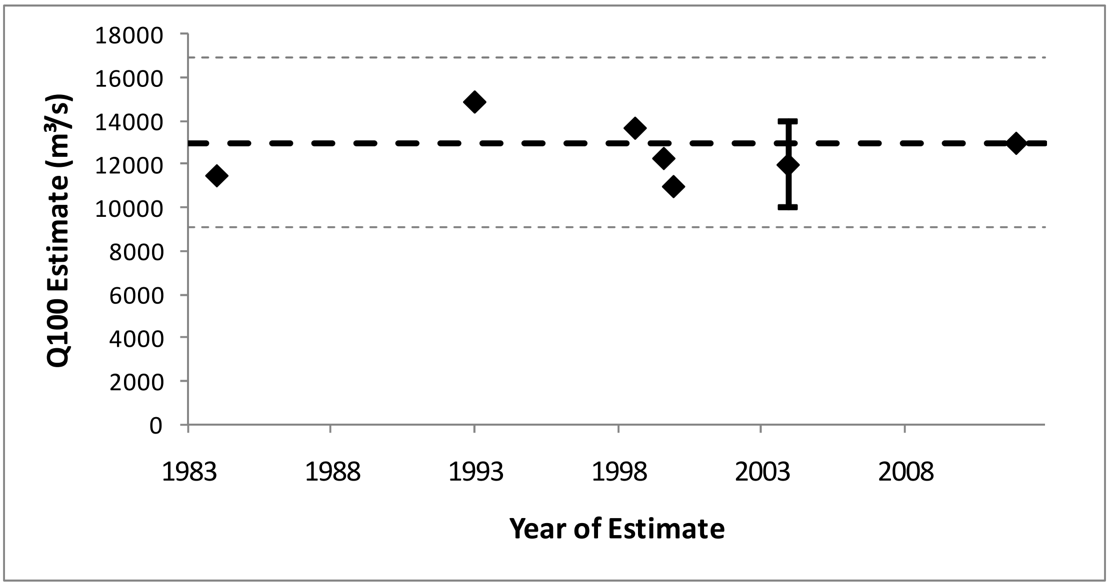
31. On the basis of the information presented by WMAwater, it is considered that 13000 m^3/s represents a reasonable estimate of the "no-dam" Q100.

32. WMAwater make the point that their estimate of 13 000 m^3/s is consistent with the "more recent" flood frequency estimates provided by SKM (1998) and City Design (1999[a]). This is a slightly curious observation to make, for it is noteworthy that this estimate is similar to *all* previous estimates of the Q100 in studies cited by WMAwater. The term "similar" is used here to denote a range of values that lie well within the notional (but optimistic) band of uncertainty of ±30%. The relevant estimates of interest are summarised in Table 1, and illustrated in Figure 2.

- **Table 1: Summary of estimates of the "no-dam" Q100.**

Estimate of Q100 (m^3/s)	Year Derived	Source
11500	1984	Weeks (1984) (also cited by Greer, 1992)
14910	1993	Dept. Natural Resources (1993)
13700	1998	SKM (1998)
12300	Jun 1999	City Design (1999[a])
11000	Dec 1999	City Design (1999[a])
12000	2003[1]	SKM (2003); Mein et al. (2003) Indicative uncertainty: 10000 – 14000 m^3/s
13000	2011	WMAwater

[1] See paragraph 33 for details.



- **Figure 2. Chronology of estimates of the "no-dam" Q100 (where the bold dashed line denotes the estimate made by WMAwater, and the notional associated band of uncertainty is shown by the narrow dashed lines).**

33. It is not clear why WMAwater did not critically review the extensive flood frequency analysis undertaken by SKM (2003). It represents the most recent investigations into the characterisation of flood risk along the lower Brisbane River, and the findings of the study were independently reviewed and endorsed by an independent panel (Mein et al, 2003). The study used a similar statistical approach as adopted by WMAwater, but reinforced the statistical inference by use of information from a number of other gauges. The SKM (2003) investigations focused on flood observations derived for Savages Crossing, which has a catchment area around 25% less than the Port Office gauge. While historical evidence (eg

City Design, 1999[b]) suggests that the peak flows along the lower Brisbane River tend to remain approximately constant (as noted by WMAwater, paragraph 97), evidence based on general Australian flood extremes (eg Nathan et al, 1994; Malone, 2011) suggests that peak discharges (for the larger catchment areas of interest) vary in a non-linear manner that is proportional to the ratio of the catchment areas raised to the power of 0.62. Thus, a Q100 estimate derived for Savages Crossing might be transposed to the Port Office under "no-dam" conditions by application of a factor that might vary between 1.0 and 1.20 (where greater weight should be applied to the lower figure on the basis of site-specific information). The SKM (2003) study concluded that the "no-dam" Q100 was likely to be between 10000 m^3/s and 14000 m^3/s, with a best estimate being 12000 m^3/s. If some account were made for the difference in catchment area between the two sites, the corresponding estimate at the Port Office gauge might be between 12000 m^3/s and 14300 m^3/s. This range neatly brackets the estimate derived by WMAwater, but given the additional weight that should be given to catchment-specific behaviour, it is concluded that the WMAwater estimate is around 5% higher than the estimate derived by SKM (2003).

34. This level of agreement between the SKM (2003) and WMAwater results is particularly important as they *were derived using largely independent data sets*. That is, using two separate sets of data representing long-term flood behaviour, both studies yielded similar estimates of the "no-dam" Q100. Such independent corroboration increases the level of confidence in the best estimate adopted. (The two studies do, however, diverge when it comes to the estimate of the Q100 for current conditions, and this is discussed in the following section.)

35. The information presented in Table 1 and Figure 2 actually illustrates an important point: the estimates of "no-dam" Q100 have changed little over the past 30 years because the underlying information from which they are derived is from a long period of observations that spans 170 years. The estimates of the "no-dam" Q100 are thus *statistically robust* as there is reasonably good evidence for the natural variability of flood behaviour in this catchment. Further efforts to refine the flood observations using better bathymetry data and hydraulic modelling will serve to reduce the band of uncertainty, but the "best estimate" is unlikely to change in a material fashion.

4.4. Estimation of the Q100 for Current Conditions

36. The reasonable consistency that is evident for estimates of the Q100 under "no-dam" conditions is not present when the corresponding timeline of estimates for "current" conditions are compared. This point is made quite strongly in the WMAwater report, and the reasons for this are worth exploring.

37. The difference between estimates of the Q100 under "no-dam" and "current" conditions is the mitigating effects afforded by the presence of Somerset and Wivenhoe Dams. These dams have considerable potential to reduce flood peaks for rainfall events that occur in the upper half of the Brisbane River catchment, but obviously have no ability to control floods from rainfalls that fall downstream of their location. Their potential for flood mitigation decreases as the magnitude of the flood increases, and this is in part dependent on the manner in which Wivenhoe Dam is operated. The ability of the dams to reduce peak levels at the Port Office is dependent on a number of key factors, namely:

- The average depth of the rainfall and the nature of the antecedent conditions;
- The location of the most intense areas of rainfall (ie, the spatial patterns of rainfall);
- The manner in which rainfall intensity varies during the event (ie rainfall temporal patterns);
- How the storm moves across the catchment during the event;
- The influence of tidal conditions on flood levels;
- The flood storage available in the dams just prior to the onset of the event; and,
- Operating procedures used to release stored floodwaters from the dams.

38. It should be recognised that all but the last two factors influence flood magnitude under either "no-dam" or "current" conditions. These factors are *stochastic* in that they depend on the capricious variability of Mother Nature, whereas the last two factors are (to different degrees) determined by human intervention.

39. There is an infinite manner in which the different stochastic factors may combine to result in a flood, and for this reason the longer the period of record the more confidence we have in our ability to characterise expected flood behaviour. As discussed in the preceding section, we have reasonable confidence in our estimate of Q100 under "no-dam" conditions as we have 170 years of observations to help us.

40. However, the introduction of the dams completely alters the manner in which these stochastic factors combine to yield a flood. The presence of the dams markedly heightens the sensitivity of the flood magnitude to these natural stochastic influences. For example, under natural conditions it makes little difference whether or not the most intense part of the storm is located over the exact centre of the catchment, or a small way upstream or downstream of it. However, with a dam in place, such a difference might mean that the bulk of the flood is impounded by the dam, or the flood might rise downstream of it. Examples of this variability may be seen in the spatial patterns associated with past major events, as reproduced in Appendix B from information presented in SKM (2003; 2011[a]). These plots show for example that the rainfall in the January 1893 event was largely restricted to the area above Somerset Dam, but in January 1974 it would have fallen below both dams. Thus, even if the

average catchment rainfall depth had been the same, the resulting peak flow at the Port Office would have been markedly different.

41. In short, the introduction of dams into the system vastly increases the complexity and uncertainty of flood behaviour as experienced by residents of Brisbane. While the availability of 170 years of record allows us to characterise flood risk with some degree of certainty, we need considerably more sophisticated tools to estimate this with the dams in place. Indeed, the historical information available to us that is relevant to "current" conditions (ie since the upgrade to Wivenhoe Dam in 2003) is in fact contained in a single event, in January 2011. With care, the historical information contained in a small number of events might be altered to represent current conditions, but by comparison with the information content that supports the estimate of the "no-dam" Q100, this is a miserably small data set to examine.

42. Given the foregoing, it is seen that the uncertainty surrounding the estimate of the Q100 for "current" conditions is inherently greater than that for "no-dam" conditions. Any estimate of the Q100 for current conditions is necessarily subject to simplifying assumptions, each of which will reflect the changing nature of what is "current". That is, the estimates will change according to the size of dam, the governing operating rules, and the nature of the design information that was thought to be most relevant at the time. It is thus to be expected that estimates of the Q100 for "current" conditions will be more volatile than those for "no-dam" conditions, and that these will vary over time as conditions change. It is for this reason that the only defensible way of estimating flood risk for current conditions is to analyse the joint probabilities in an explicit manner using such techniques as Monte-Carlo simulation, as recommended by SKM (2003) and Mein et al. (2003), and more recently by the Joint Flood Task Force (2011) and in the Interim Report prepared by the Queensland Floods Commission of Inquiry (2011).

43. The foregoing provides a broader context for commenting on the method used by WMAwater to estimate the Q100 under current conditions. WMAwater (as described in paragraph 32 of their report) converted the "no-dam" estimate of the Q100 to "current" conditions by the simple application of a single factor (0.73). In essence, this factor is intended to account for all the stochastic complexity as described in the preceding paragraphs. WMAwater make the under-stated comment that the "2011 data provides the only real data point on the performance of the dam". It is for this reason that the Q100 estimate is sensitive to consideration of the 2011 event, whereas that for "no-dam" conditions is not. No further comment is made concerning the limitations of their approach, and no recommendations are made concerning the need for improving this aspect of their inference.

44. By this means, WMAwater estimate the Q100 for current conditions to be 9500 m^3/s. This estimate is almost 50% higher than the estimate derived by SKM (2003), despite the fact that

the corresponding estimates derived by WMAwater and SKM for “no-dam” conditions are similar. It should be noted that the SKM (2003) estimate is derived using a model to simulate the operating procedures of the two dams, and a hydraulic model to route the outflows down to the Port Office. The inputs reflecting the stochastic behaviour of the system were conditioned to be consistent with the “no-dam” flood conditions, where the impacts associated with dam operations were handled by deterministic modelling.

45. The estimate derived by SKM (2003) was endorsed by an independent review panel established for the purpose. Both SKM and the independent review panel acknowledge the limitations of the adopted approach, and as a consequence both parties separately recommended the need for a more rigorous approach based on explicit joint probability procedures.

46. WMAwater’s estimate of “current” flood risk is heavily dependent on the results of a single flood event. Apart from the statistical sampling limitations involved, the simplicity of this approach introduces a degree of circularity in their argument. As noted above, the flood mitigation potential of the dams decreases as the magnitude of the flood increases. If the January 2011 flood is more representative of an event with an annual exceedance probability of 1 in 200, then it would be expected that a factor lower than 0.73 should be used, and thus the resulting estimate of the Q100 would be lower. The assumption that the characteristics of the January 2011 event are directly relevant to the Q100 biases the outcome in a very selective fashion. Giving more weight to the other evidence presented in Figure 3 of their report would alleviate this problem, but again the size of the sample compared to the nature of the stochastic influences does not provide compelling justification for the adopted approach.

47. In summary, it is considered that the approach taken by WMAwater does not give adequate consideration to the stochastic factors that influence the conversion of the “no-dam” estimate of the Q100 to “current” conditions. While it might be expected that the January 2011 event might result in an upwards revision of the Q100 estimates as derived in 2003, no compelling evidence to this effect has been presented.

5. Assessment of Event Severity

48. On the basis of their derived Q100 estimate for current conditions, WMAwater estimate that the 2011 event has an annual exceedance probability of 1 in 120. The exceedance probability is precisely stated given the large uncertainty inherent in its estimation. Following from the discussion presented in the preceding section, this author does not agree with either the inferred accuracy or the magnitude of this assessment.

49. WMAwater present supporting evidence in the form of rainfalls. It is not stated how the rainfall estimates provided in Table 12 of the WMAwater report were derived. Any estimate of total catchment rainfall is heavily dependent on the number and quality of the rainfall gauges used, and the manner in which the surface fitting procedures account for the influence in topography. It is thus not possible to comment on the validity of the catchment rainfall totals presented. It is noted that only gauges operated by the Bureau of Meteorology were used, and that there are other gauges (eg ALERT) that could be used to refine the estimate. It is not clear why a range of areal reduction factors were considered as this can be calculated – from Table 4-1 of SKM (2003) it may be inferred that the 3-day areal reduction factor relevant to the catchment is 0.86.

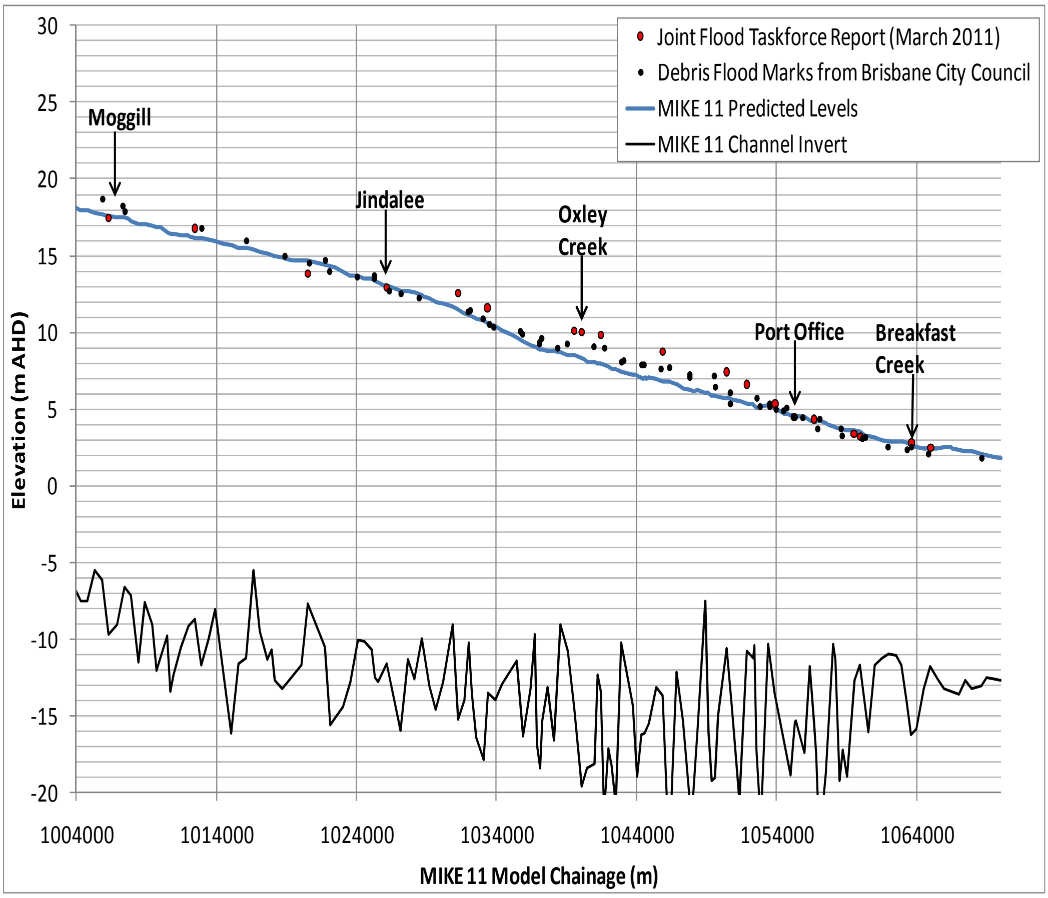
50. SKM (2011[a]) undertook an analysis of event rainfall data and concluded that the annual exceedance probability of the rainfalls for the whole dam catchment was around 1 in 100 to 1 in 200, though the annual exceedance probability of the most extreme point rainfalls that occurred in the centre of the Brisbane River catchment was likely to be between 1 in 500 and 1 in 2000. This interpretation suggests that the event rainfalls were rarer than that concluded by WMAwater, but it is fair to say that neither analysis was rigorously undertaken and there is no strong evidence to support one view over the other. To this author's knowledge no careful analysis of catchment rainfalls has yet been published that utilises all available data in a manner that takes account of the topographical gradients involved.

51. As discussed in the preceding section and noted by WMAwater, the exceedance probability of the rainfall event can only give a general indication of the severity of a flood event. This is particularly the case for Brisbane River, where the spatial and temporal characteristics of the rainfall can influence flood severity downstream of the dams in a manner that may be quite differentiated from the causative rainfall.

52. Analysis of estimates of the January 2011 flood under "no-dam" conditions would suggest that this was an event with an annual exceedance probability of around 1 in 100, but again this does not reflect the true flood risk as it stands under current conditions.

53. WMAwater present a flood profile on the basis of their estimate of Q100 under current conditions at the Port Office. This flood profile is then used to make inferences regarding the severity of the event at different locations down Brisbane River. Given that this author does not agree with the estimate of the Q100 then it follows that the assessment of severity as presented is also not endorsed. There are, however, a couple of additional minor points regarding this assessment that are worth noting, as discussed below:

   - WMAwater did not use the MIKE-11 model (version 2) developed by SKM as it is stated that the model underestimates peak levels by up to 1.8 m between Jindalee and the Port Office. It is noted that WMAwater did not have access to the SKM report that accompanies version 2 of the model (SKM, 2011c), as this (and other) limitations of the modelling were noted and discussed. This particular limitation did not impact on the results reported, and thus this aspect was not refined within the timeframe available.
   - It should also be noted that the maximum underestimation of water levels along this reach is actually about half that indicated by WMAwater. The reason for this discrepancy is that WMAwater relied upon interim flood levels referenced by the Joint Flood Taskforce (2011), which are flagged as requiring verification. The discrepancy between these interim values and recorded data may be discerned from the information presented in Figure 6-6 of SKM (2011[b]) and in Figure 6-7 of SKM (2011[c]). A plot illustrating model performance against flood debris data obtained by Brisbane City Council is shown in Figure 3. It should be noted that these debris marks provide only an approximate indication of flood level, but that at the confluence with Oxley Creek these indicators are consistent with recorded data.
   - Finally, it the WMAwater report states that the MIKE11 model was adjusted to match the surveyed flood levels, but no details of how this adjustment was undertaken nor the physical basis for this adjustment is provided. It is thus not clear on what basis the comparisons with the derived Q100 level are provided.



- **Figure 3. January 2011 peak level profile versus observed debris flood marks provided by Brisbane City Council.**

6. Conclusions and Recommendations

54. The scope of the investigations addressed by WMAwater relates to some vexed issues that have been the focus of a number of detailed investigations and independent reviews over the past three decades. It is understood that WMAwater had limited time to undertake their investigation, and it is appropriate that any conclusions drawn are subject to the appropriate caveats.

55. On the basis of the material presented by WMAwater it is this author’s opinion that:

    - the broad approach used to undertake the frequency analysis using historical flood maxima is appropriate;
    - there is reasonably strong justification for the Q100 estimate of 13000 m^3/s under “no-dam” conditions as this analysis makes use of flood behaviour observed over a 170 year period;
    - the method used to convert the estimate of “no-dam” Q100 to current conditions is overly simplistic and involves a somewhat circular argument that relies heavily on information contained in a single event;
    - the estimate of Q100 for current conditions is accordingly not supported; and,
    - as a consequence the Q100 flood level estimates along the Brisbane River are also not supported.

56. The estimate of the Q100 under current conditions is inherently more uncertain than the estimate of Q100 under “no-dam” conditions. It is considered that the only defensible way of estimating flood risk for current conditions is to analyse the joint probabilities in an explicit manner using such techniques as Monte-Carlo simulation.

57. This author agrees with the recommendations for improving the rating relationship at the Port Office gauge made by WMAwater, but it is recommended that higher priority be given to the application of more rigorous (joint probability) hydrological methods that reflect current operating procedures to allow the flood risk downstream of the dams to be characterised with confidence.

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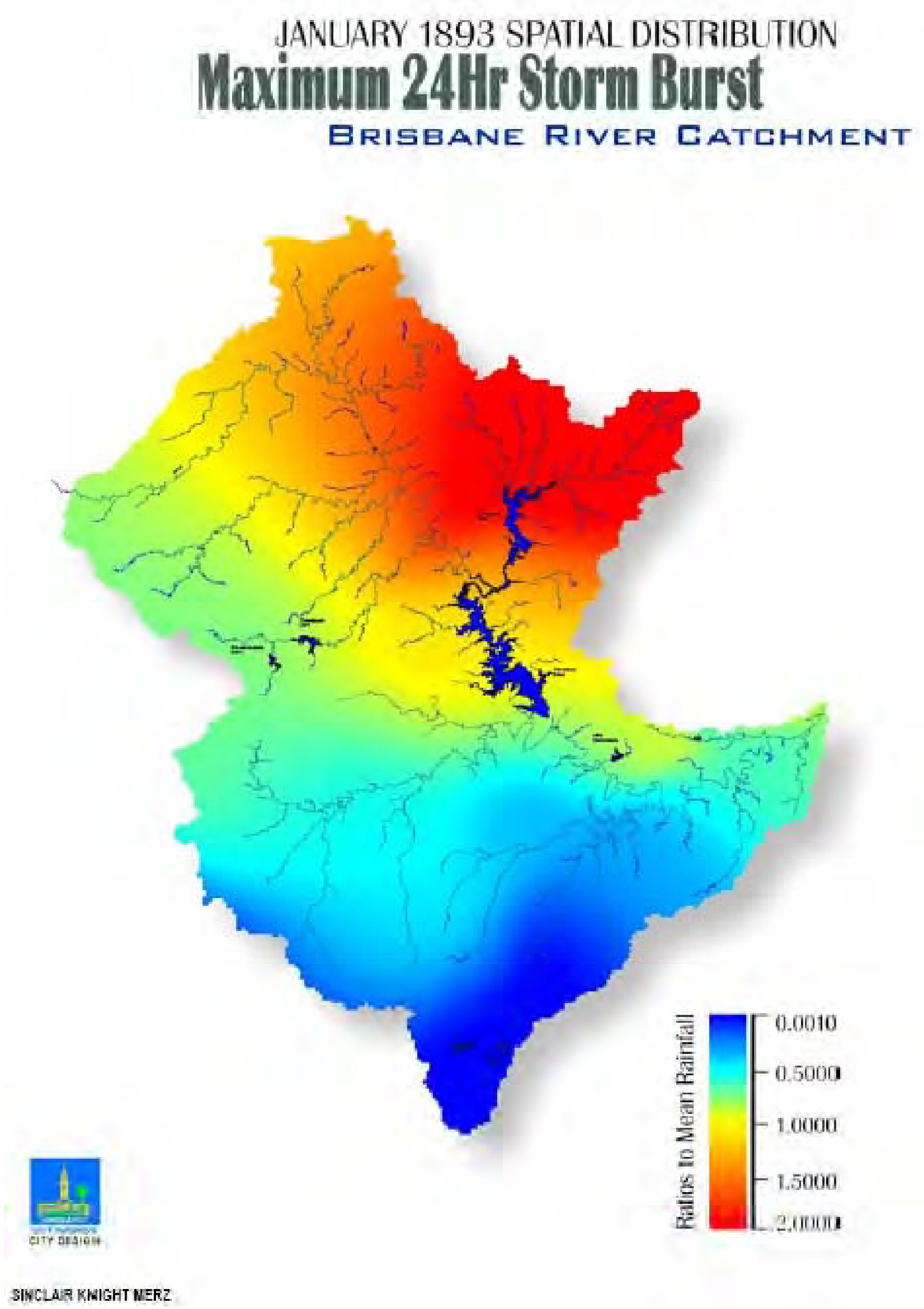
Appendix A – Spatial Patterns from Historic Events



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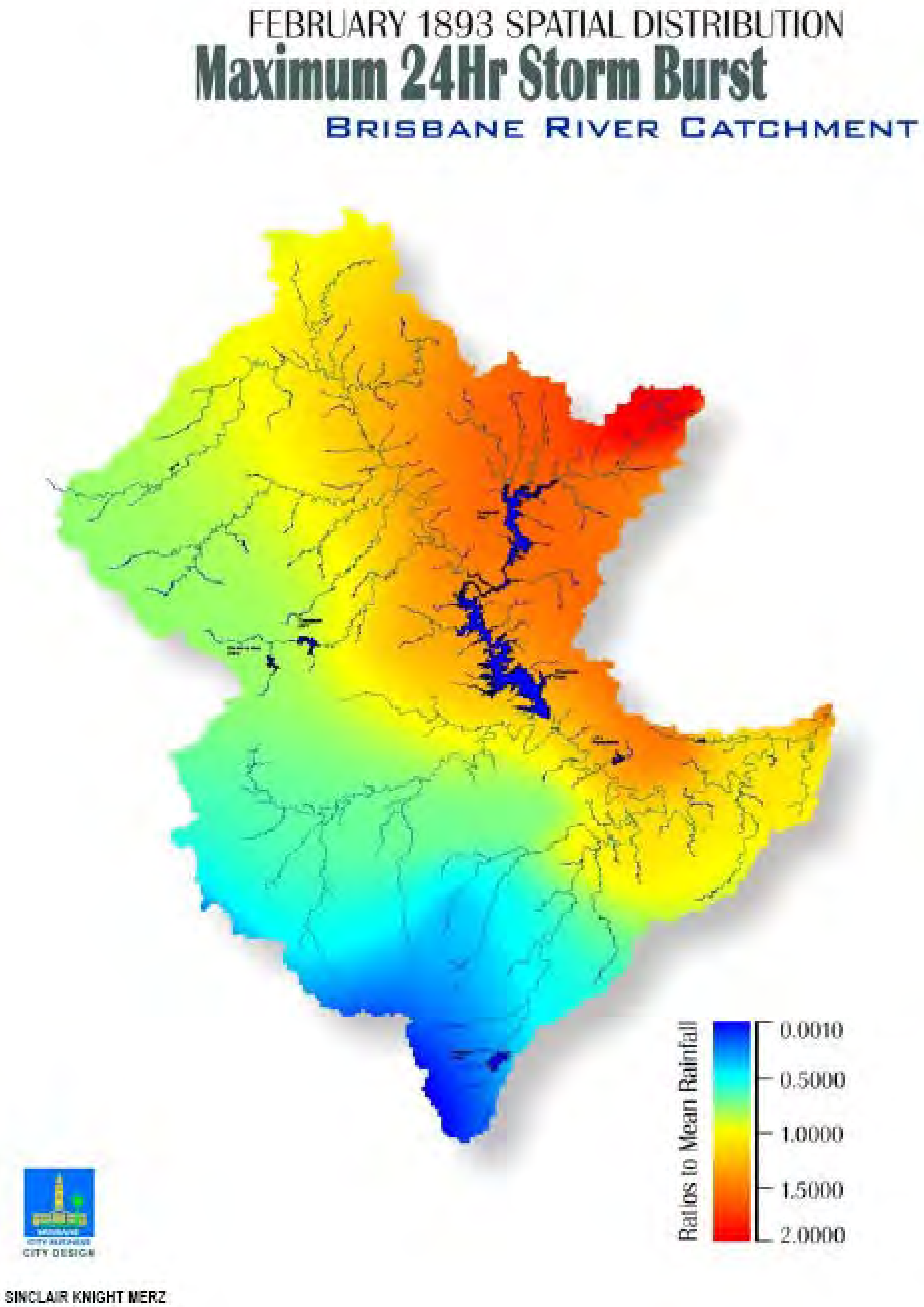

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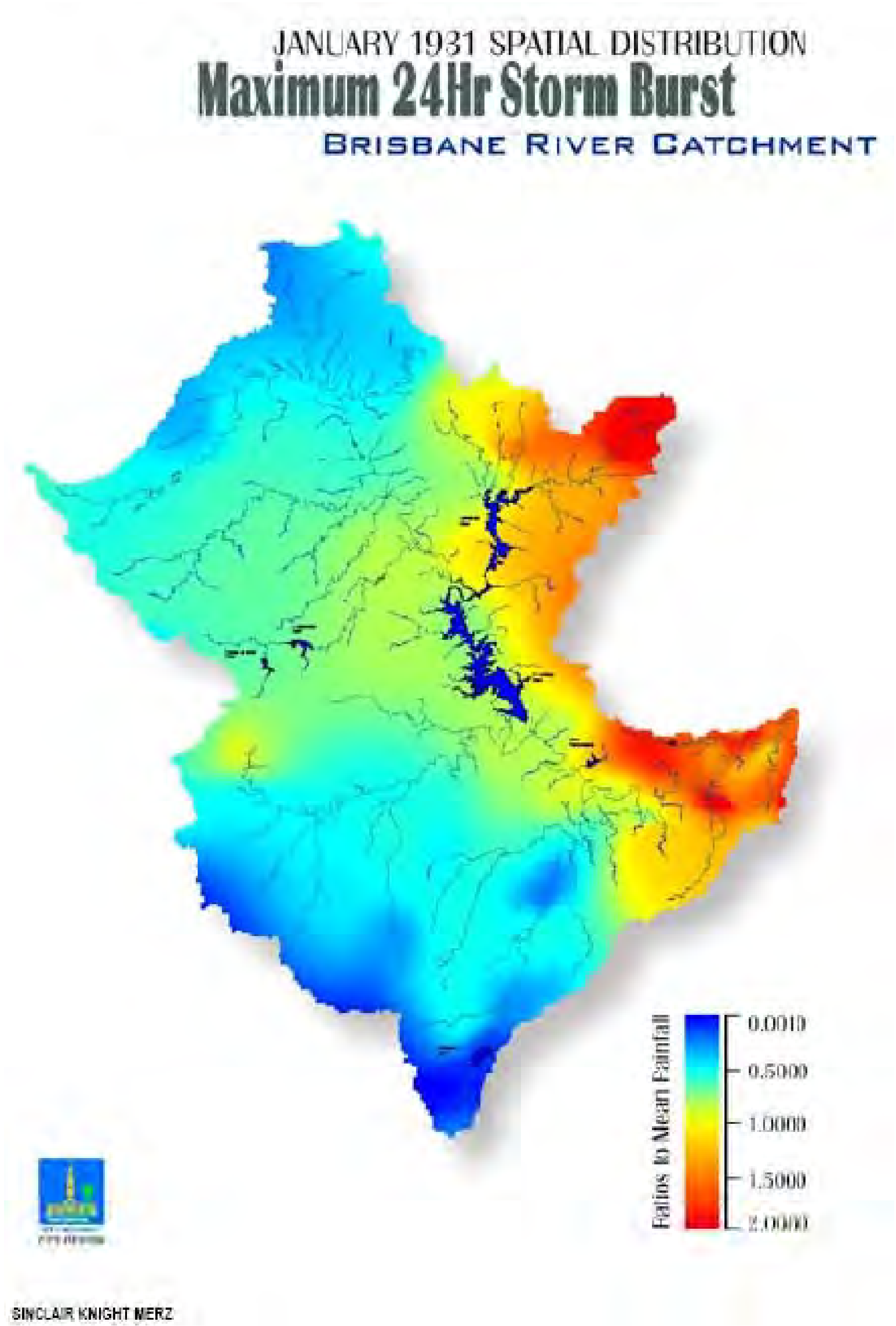
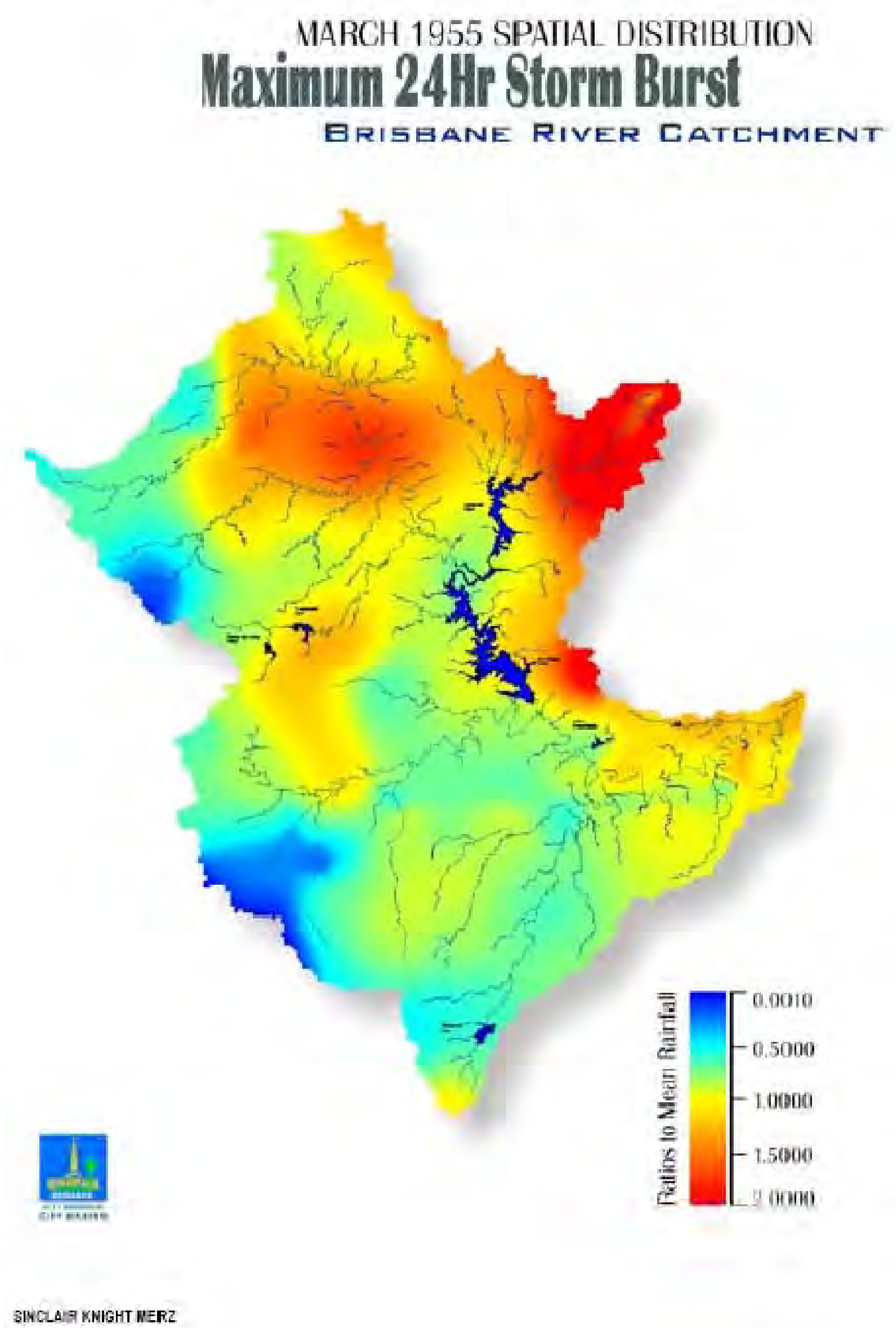


Reproduced from SKM (2003)



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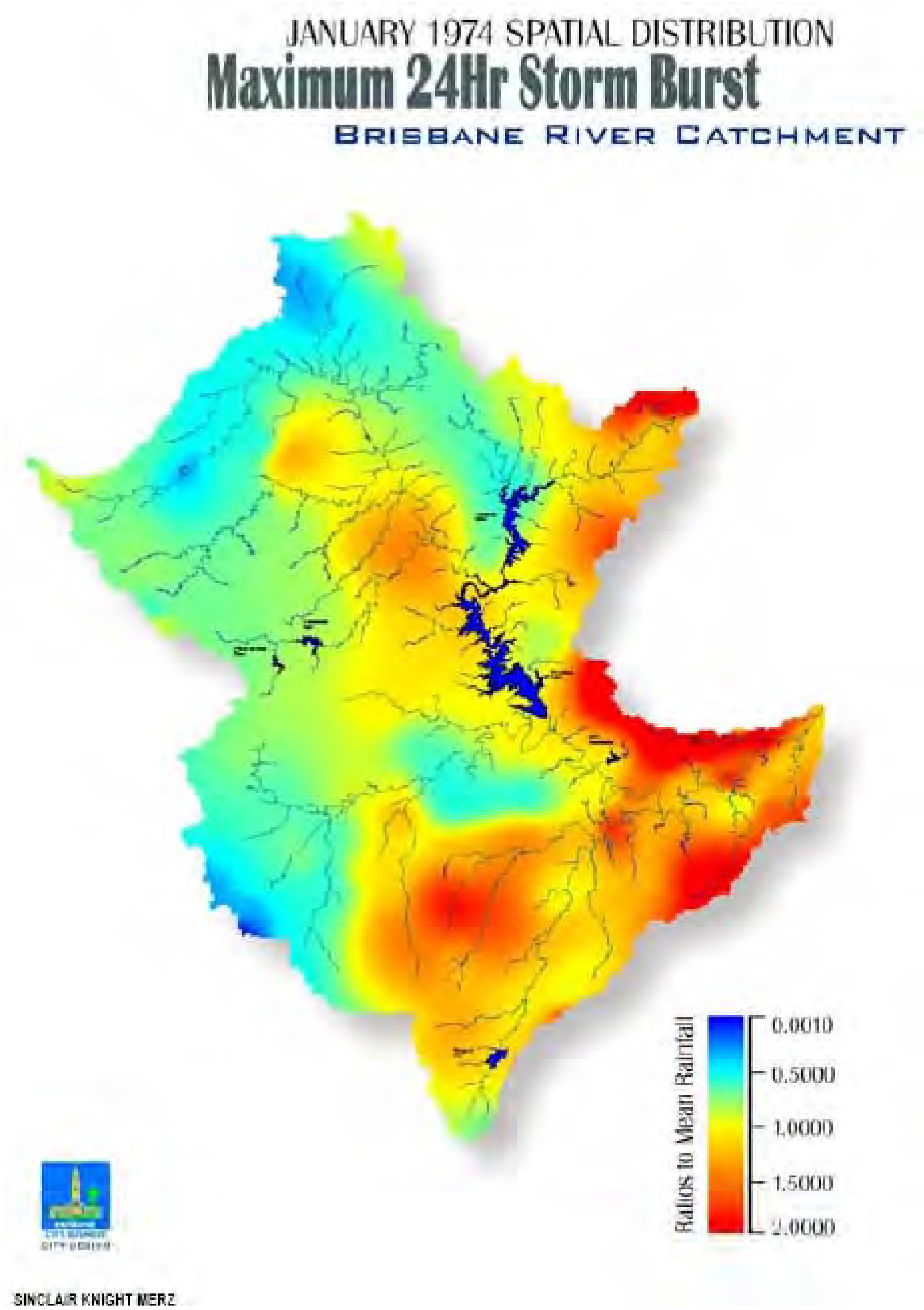

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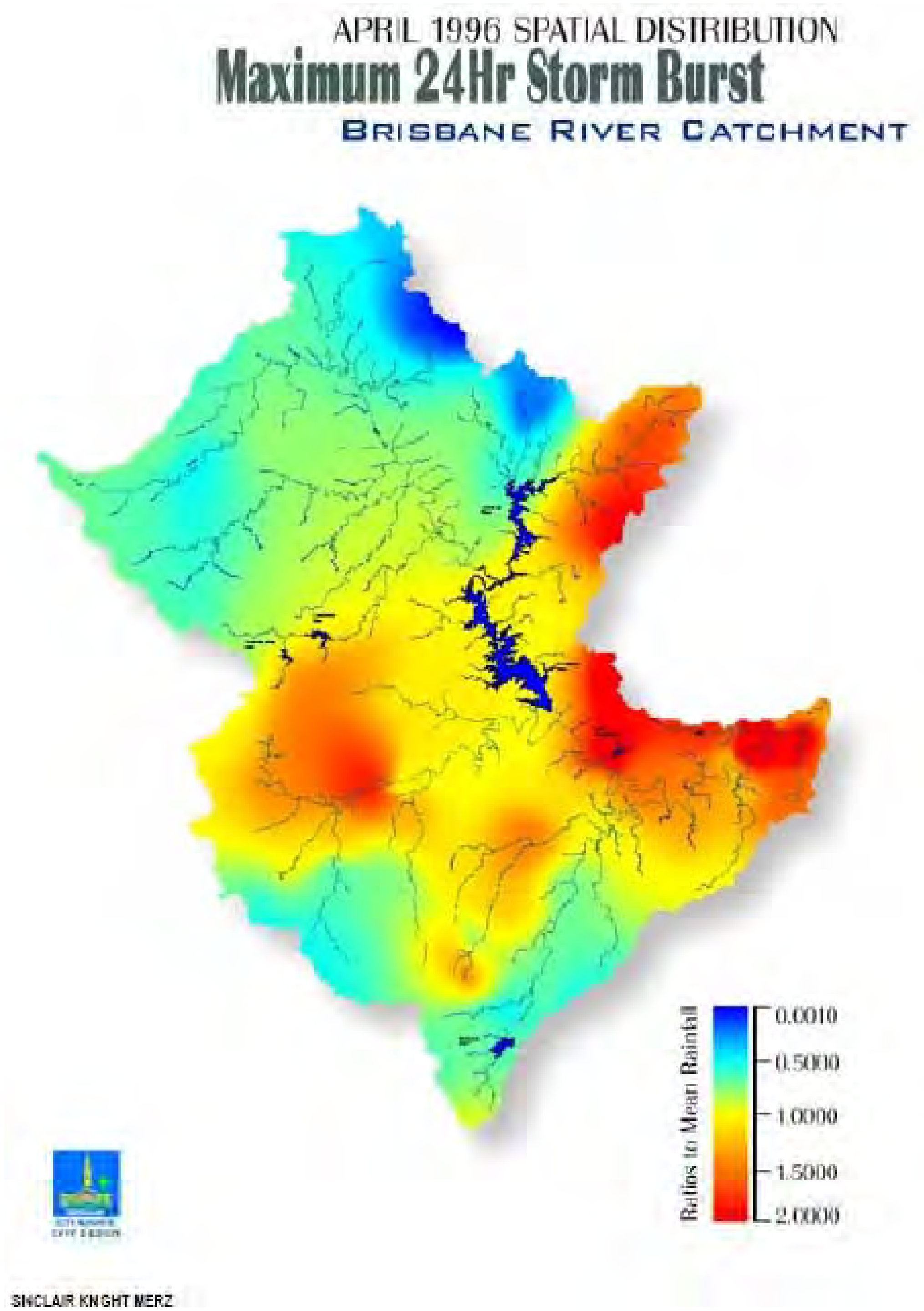
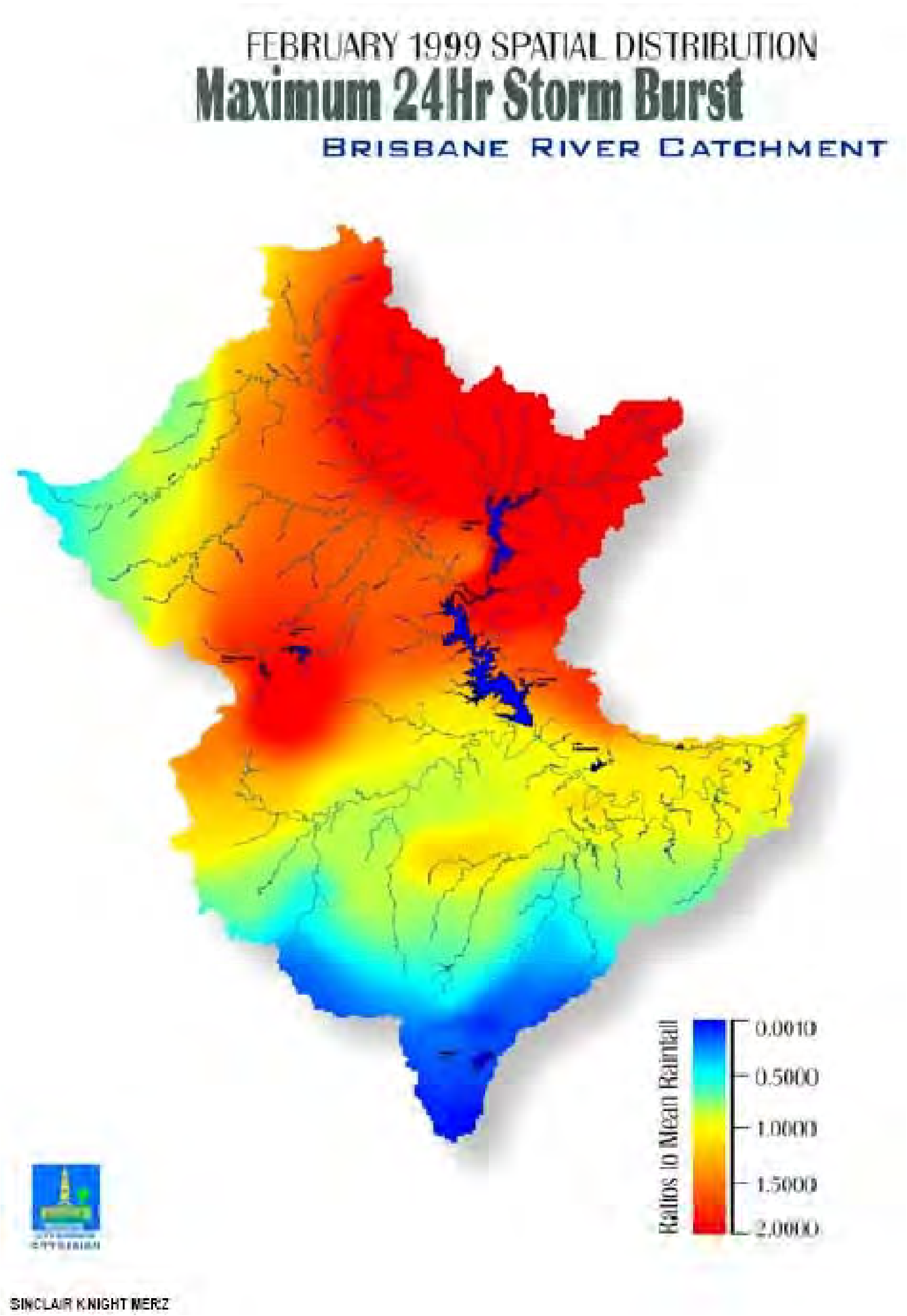
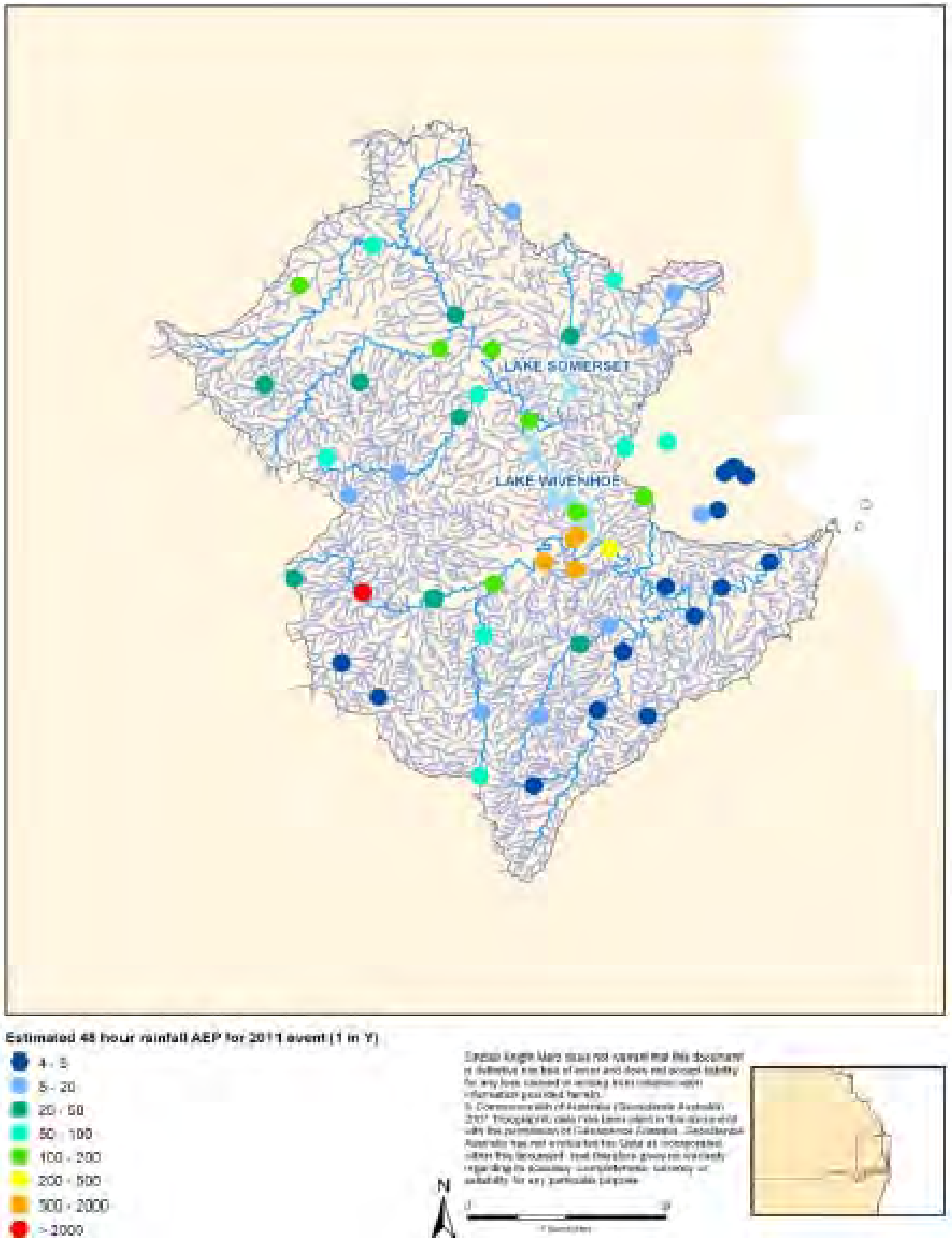


Reproduced from SKM (2003)



Reproduced from SKM (2003)



Spatial variation in exceedance probabilities of 48 hour rainfalls recorded in January 2011.

Reproduced from SKM (2011[a])

Appendix B – Curriculum Vitae for Dr Rory Nathan

Curriculum Vitae

Dr Rory Nathan

Practice Leader Hydrology

Qualifications

- B.E.(Agr), University of Melbourne, 1980
- M.Sc., D.I.C., University of London, 1984
- Ph.D., University of Melbourne, 1990

Affiliations

- Fellow, Institution of Engineers, Australia
- Australian Representative, Floods Committee, International Committee on Large Dams
- Member Hydrology Sub-committee, NSW Dams Safety Council
- Honorary Fellow, Department. Civil Engineering., Monash University
- Past Honorary Fellow, Dept. Civil and Environmental Engin., University of Melbourne

Awards

- Named as member of "Top 100 Most Influential Engineers" in Australia, 2009
- National Civil Engineer of the Year, awarded by the Institution of Engineers, 2000
- W.H. Warren Medal (1992, 1998, and 2005) for the best paper in Civil Engineering (national award by the Engineers Australia).
- ASCE Journal of Irrigation and Drainage Engineering Best Research Paper Award (1997)
- G.N. Alexander Medal (1998) for the best paper in Hydrology and Water Resources, (national award by the Engineers Australia)
- Best presentation of a technical paper at the Hydrology & Water Resources Conf. (1993)
- ACEA Award of Excellence (1998).
- Victorian Engineering Excellence Award (2003).

Fields of Special Competence

Dr Rory Nathan has around 30 years experience in engineering hydrology in both the academic and consulting fields. He is actively involved in a number of research projects under the auspices of Engineers Australia and with the University of Melbourne. While he has generally worked in areas of flood estimation, hydrological processes, regionalisation, and catchment hydrology, he has developed specialist skills in the following areas:

- Estimation of extreme hydrologic events (floods and low flows)
- Characterisation of risk for dam safety
- Hydrologic estimation in ungauged catchments
- Regionalisation of hydrologic information
- Characterisation of flow regimes for environmental flows
- Modelling and simulation of hydrologic processes
- Hydrologic model development and application

Relevant experience

- Convenor and senior author of the national guidelines for the estimation of large to extreme floods published by the Institution of Engineers Australia.

- Contracted by the U.S. Bureau of Reclamation to provide input to the development of guidelines on the characterisation of hydrologic inputs for risk analysis
- Contracted by the U.S. Army Corps of Engineers to help formulate research directions to be undertaken in the area of hydrologic risk using federal agency funding
- Contracted by the Murray Darling Basin Commission to oversee and review the flood risk assessment of Hume Dam being undertaken by NSW State Water (and SMEC).
- Member of panel undertaking risk review of the Dam Safety Program for Western Australia's South-West Irrigation Dams
- Member, Expert Review Panel for the Preliminary Risk Assessment of the portfolio of dams owned by the Hydro-Electric Authority, Tasmania
- Member, Expert Review Panel for the Preliminary Risk Assessment of Somerset, Wivenhoe, and North Pine Dams owned by the South East Queensland Water Board
- Member, Expert Review Panel for the upgrading of Rosslynne Dam owned by the Southern Rural Water
- Project Manager consequence assessment and risk characterisation of Dartmouth Dam (Goulburn-Murray Water)
- Project Director for the consequence assessment and risk characterisation of Hume Dam (DLWC, NSW)
- Variously Project Manager and Project Director for the estimation of hydrologic loads, risk characterisation, and consequence assessment of several dams owned by Goulburn-Murray Water (and its predecessor the Rural Water Corporation); Dartmouth, Eildon, Cairn-Curran, Nillhacootie, Laanacoorie, Mokoan, Waranga, Buffalo, Fyans, Bellfield, Rocklands,
- The estimation of hydrologic loads and review of spillway adequacy for many major water storages owned by the (then) Rural Water Corporation (Eildon, Dartmouth, Laanacoorie, Wartook, Bellfield, Fyans, Waranga, Lonsdale, Rocklands, Pine, Taylors, Cairn-Curran, Tullaroop, Upper Coliban, Lauriston, Malmsbury, Buffalo, and Pykes Creek).
- Responsible for event tree development and risk characterisation of hydrologic inputs to the Preliminary Risk Assessment of all dams owned by the Snowy Mountain Hydro-Electric Authority.
- Responsible for the derivation and characterisation of hydrologic and hydraulic inputs to the Preliminary Risk Assessment of all dams owned by South Australia Water.
- Use of quantitative risk analysis for evaluation of floodplain development options for AMP
- Provision of advice to ACTEW/AGL on how to best account for climatic variability in the development of options for their future water supply options (ongoing)
- Assessment of the vulnerability to climate change and variability for the water resources of the Fitzroy River
- Project Director for the consequence assessment of four major dams owned by the Dept. of Land and Water Conservation, NSW (Blowering, Burrinjuck, Split Rock and Keepit)
- Project Director for the consequence assessment and risk characterisation of the Kiewa Hydroelectric Scheme (Southern Hydro)
- Expert reviewer of extreme event hydrologic studies undertaken by Melbourne Water (Upper Yarra and Devils Bend reservoirs)
- The estimation of hydrologic loads and review of spillway adequacy for O'Shannassy Reservoir (Melbourne Water).

Refereed Journal and Book Publications

Nathan, R.J., T.A. McMahon, & B.L. Finlayson (1988): The impact of the greenhouse effect on catchment hydrology and storage-yield relationships in both winter and summer rainfall zones. In *Greenhouse, Planning for Climate Change*, G.I. Pearman (ed), CSIRO Publications and E.J. Brill Publishing Co., pp 273 - 295.

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Nathan, R.J. and Weinmann, P.E. (1996): Reply to Discussion by Green et al.: "The estimation of extreme floods - the need and scope for revision of our national guidelines". *Aus J Water Resources,* 1(2): 106-107.

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