QUEENSLAND FLOOD COMMISSION OF INQUIRY

SUPPLEMENTARY REPORT – IPSWICH FLOOD FREQUENCY ANALYSIS

FINAL REPORT

OCTOBER 2011

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Project Supplementary Report Ipswich Flood Frequency Analysis	**Project Number** 111024
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SUPPLEMENTARY REPORT
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TABLE OF CONTENTS

PAGE

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1. INTRODUCTION

1.1. Overview

1 This report documents an assessment of flood frequency at Ipswich resulting from Brisbane and Bremer River flows. WMAwater have estimated the 1% AEP (100 year ARI) flood level at Ipswich, as well as the probability of the January 2011 flood event.

2 This report is supplementary to a previous WMAwater report, "*Brisbane River 2011 Flood Event – Flood Frequency Analysis*" (Reference 1), which documented similar investigations on the Brisbane River below the Bremer River confluence, from Moggill to the Brisbane River mouth. The analysis presented in this report should be read in conjunction with Reference 1, which contains general discussion of central concepts of flood frequency analysis and flood planning levels (FPLs), and also documents assumptions and limitations which are relevant to this study.

3 Determining design flood levels at Ipswich is a particularly complex task that has a considerable level of uncertainty. The prime cause of the uncertainty is the difficulty in quantifying the interaction between the Brisbane and Bremer Rivers, both of which exert a strong influence on flood behaviour in and around Ipswich and Moggill. The design flood estimates undertaken to date at Ipswich have not thoroughly addressed the joint probability of these two main flood mechanisms.

4 WMAwater have developed a flood frequency approach which incorporates a consideration of the joint probability effects of Brisbane/Bremer River floods at Ipswich. The approach can also be used to assess the influence of Wivenhoe and Somerset Dams on the frequency of flooding at Ipswich, via modification of the Brisbane River flow record as per Reference 1.

5 There are significant limitations to the analysis, particularly in the present understanding of backwater effects at the Brisbane/Bremer confluence and the conditional probability relationship of flooding between the two systems. WMAwater have attempted to identify the most important limitations and methods by which confidence in the results can be improved. The results from this preliminary analysis appear reasonably robust and consistent with historical data. Further efforts to reduce uncertainties in various parts of the analysis would be worthwhile.

1.2. Scope of the Report

6 Following the flooding of the Brisbane River and its tributaries in January 2011 the Queensland Flood Commission of Inquiry (The Commission) requested that Mark Babister of WMAwater prepare a report providing advice on the operation of Wivenhoe and Somerset Dams and the resultant flooding downstream.

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

a. 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.

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

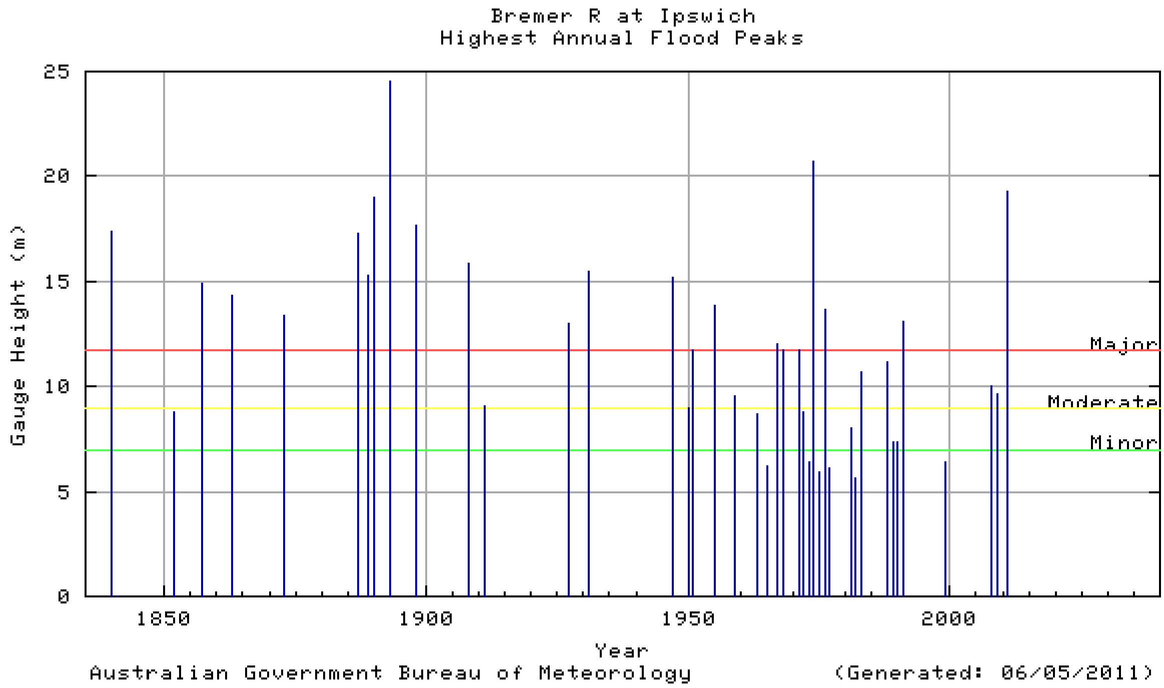
c. Using results of tasks (a) and (b) determine the ARI and AEP of the January 2011 floods at particular points along the Brisbane River and Bremer River.

2. BACKGROUND

2.1. Bremer River Catchment

8 The city of Ipswich lies approximately 40 km west of Queensland's State Capital, Brisbane, and has a population of 155,000. Ipswich can be impacted by floodwaters from the Brisbane and Bremer valleys and has a history of suffering significant flood events with 19 events having exceeded the "Major" flooding classification in the past 170 years (Figure 1).

Figure 1: Bureau of Meteorology Peak Flood Level Record and Classifications at Ipswich



9 The Bremer River passes through the southern and eastern suburbs of Ipswich, and its headwaters are in the Macpherson Ranges. The Bremer's total catchment area to the confluence with the Brisbane River is approximately 1,790 km^2 (Reference 15) of which Warrill Creek (also known as the Fassifern Valley) constitutes approximately two thirds at 1,150 km^2, entering the Bremer River approximately 10 km upstream of Ipswich.

2.2. Ipswich Flood History

10 Reasonably reliable flood records extend back as far as 1893, with other less reliable observations of large events from as far back as 1825 (Reference 34). The peak flood level record provided to WMAwater by the Bureau of Meteorology (BoM) dates back to 1840. Floods have traditionally been gauged at David Trumpy Bridge, which is also known as the Ipswich City Gauge.

11 The largest flood on record at Ipswich occurred in 1893 when the Bremer River reached a level of 24.5 mAHD. The largest flood of the 20th century was the 1974 event, reaching 20.7 mAHD at David Trumpy Bridge. This led to the inundation and partial or complete destruction of many homes. The January 2011 flood reached 19.25 mAHD, which caused significant residential and commercial damage. Each of the three highest recorded events at Ipswich (1983, 1974 and 2011) involved significant Bremer River flood flows occurring concurrently with major flooding of the Brisbane River.

2.3. Joint Probability of Brisbane/Bremer Flood Mechanisms

12 There are significant difficulties in estimating the frequency of a given flood level at Ipswich. The primary difficulty arises because whilst Ipswich is on the Bremer River and significant flooding can occur as a result of flows from the Bremer catchment alone, flooding may also occur as backwater from Brisbane River flooding (with or without concurrent elevated flows from the Bremer catchment).

13 Generally, the peak flood level experienced at Ipswich will be a result of the combined influence of the Brisbane and Bremer flood mechanisms, although the relative contribution may vary. It would generally be expected that the likelihood of significant concurrent flooding in both systems will increase for larger floods, as the large-scale meteorological systems that will generally produce large Brisbane River floods are also likely to produce considerable rainfall and runoff in the Bremer catchment.

14 This situation is a classic joint probability problem and while it is not uncommon that different mechanisms contribute to flooding, often the influence of the smaller catchment or secondary flood mechanism is relatively minor compared with the primary source of flooding. In such cases it is possible to assess flood behaviour from the dominant mechanism and use a reasonably simple assumption to account for the weaker supplementary mechanism. Such an approach is not suitable at Ipswich as the Bremer River has a substantial catchment size, and both sources of flooding (Bremer and Brisbane Rivers) have the potential to cause significant flooding.

15 The importance of the flood interaction is evidenced by the largest floods recorded at Ipswich, such as the January 2011 flood, when the recorded David Trumpy flood level was approximately 1.4 m higher than the level at Moggill near the confluence of the two rivers. From available data it appears that both the Brisbane and Bremer River flood mechanisms alone can produce flooding well above the “Major” level of 11.7 mAHD, and coincident Bremer River and Brisbane River flows can add in the order of 5 m on top of the level from Brisbane River backwater alone.

2.4. Use of Flood Frequency Analysis at Ipswich

16 Flood Frequency Analysis (FFA) is a preferred method to directly estimate flood probability in areas where variability in flood-producing mechanisms is hard to quantify. As discussed

in Reference 1, the at-site flood record includes all the variability in factors that influence flood behaviour such as rainfall intensity, runoff volume, storm characteristics, and relative contribution of tributaries.

17 Difficulties arise in undertaking traditional flood frequency analysis at Ipswich because the backwater influence from the Brisbane River makes the development of a rating curve (which is a difficult process for a non-backwatered gauge) even more challenging. Furthermore, a frequency analysis based solely on flows in the Bremer River will only estimate the probability of flood discharges in the Bremer River alone and will not capture the critical influence of the Brisbane River on the eventual flood level, and therefore will not be particularly useful for estimating flood height probabilities.

18 As a result, it is tempting to undertake FFA directly on observed flood heights. However there are pitfalls to such an approach as flood heights are dependent on localised topography in the vicinity of the gauge, and can therefore be subject to discontinuities. For example, a location with a narrow channel and a relatively wide flat floodplain will have a discontinuity at the level where flow breaks out into the floodplain, which can invalidate the fitting of a distribution to these data.

19 If a long record is available a meaningful estimate of flood probabilities can still be obtained by drawing a fit “by eye” through a plot of the recorded flood heights against their most likely probability, based on their rank in the historical record. However such an approach can be invalid in locations (such as Ipswich) where a major catchment change such as construction of a flood mitigation change introduces a substantial change to the at-site flood frequency.

2.5. Floodplain Management Challenges

20 In addition to the above challenges in estimating design flood levels at Ipswich, floodplain management at Ipswich is further complicated by the relatively large variation in observed flood levels. For example whilst the definition of “Major” flooding at Ipswich is a level above 11.7 mAHD at Ipswich City Gauge, the 1893 event reached a peak flood height of 24.5 mAHD whilst the recent January 2011 event reached 19.25 mAHD. Variation of this magnitude at the upper end of recorded flood levels is relatively uncommon for Australian catchments.

21 Another location where large variation in behaviour of extreme flood events occurs is at Windsor, located downstream of Warragamba Dam on the Hawkesbury-Nepean system in New South Wales. At Windsor under normal river conditions the river is tidal with an average level just above mean sea level (similar to Ipswich). In contrast the 1% AEP flood level is 17.3 mAHD, while the 0.5% AEP (200 year ARI) flood level is approximately two metres higher. This means that a house with a floor level at the standard flood planning level (FPL) of the 1% AEP plus 0.5 m freeboard will still be flooded in an event slightly larger than the 1% AEP.

22 From a planning and floodplain management perspective, particularly with regards to emergency response management, such large variation in flood levels is a major concern. Risk management involves consideration of both the likelihood and consequences of an event. In locations such as Ipswich and Windsor, the consequences of floods larger than the adopted FPL (such as the 1% AEP) can be far more severe than elsewhere, as the increased depths of water above the FPL can increase the risk of injury or death for inhabitants of the floodplain, and of structural failure of buildings built at the FPL.

23 As a result, Windsor has been identified as a location where traditional floodplain management methods need to be reconsidered and it is likely that such considerations are also applicable to Ipswich. Several variations to standard floodplain management measures have been proposed at Windsor, although consensus has not been achieved, which is partially a reflection of the magnitude of the challenges posed. Proposed measures include an increased focus on flood events larger than the 1% AEP event, and particularly on floodplain evacuation routes and procedures considering flooding up to and including the Probable Maximum Flood (PMF), to ensure that evacuated residents do not become stranded by rising floodwaters. Such areas may require:

a. higher flood planning levels to be used for certain types of development;

b. larger amounts of freeboard;

c. requiring two-storey dwellings for residences below the 0.5% AEP flood planning level, with flood compatible double-brick construction for the lower storey;

d. requiring buildings to have openings to reduce the likelihood of structural failure from differential flood level pressures; and

e. the incorporation of additional features to help manage the flood risk, such as dwellings with reinforced structures designed to withstand the forces of flooding, and the use of marine ply bracing that does not degrade and fail following extended periods of inundation.

3. PREVIOUS STUDIES

3.1. List of Key Reports

24 The following is a chronological list of key studies and reports relating to determination of design flood levels at Ipswich, and reviewed by WMAwater as part of this investigation.

- Queensland Survey Office (1975) – Maps of Inundation for Brisbane and Bremer Rivers as well as presentation of limited FFA analysis and damage estimates;
- Ipswich Council (Late 1970s) "Gamble" Maps – based on observations from the 1974 and 1955 events;
- SKM (2000) Ipswich Rivers Flood Studies – Phase 1 and 2 prepared for Ipswich Rivers Improvement Trust and Ipswich City Council
- Halliburton KBR (2002a) Ipswich Rivers Flood Studies – Lower Bremer River Flooding Report prepared for Ipswich City Council
- Halliburton KBR (2002b) Ipswich Rivers Flood Study Phase 3 – Final Report prepared for Ipswich City Council
- Sargent Consulting (2002a) Brief Review of Flood Frequency Analysis and Discharge Rating Curve for Brisbane River at Moggill Gauge prepared for Ipswich City Council
- Sargent Consulting (2002b) Composite Mapping for 20 Year ARI – Review and Recommendations prepared for Ipswich City Council
- KBR (2004) Bremer River Catchment Flood Risk Management Study – Final Report - prepared for Ipswich Rivers Improvement Trust
- DHI Water and Environment (2005) MIKE 11 Model Review Ipswich Rivers – Final Report prepared for Ipswich City Council
- DHI Water and Environment (2006) Ipswich River MIKE 11 Model Upgrade – Final Report prepared for Ipswich City Council
- Sargent Consulting (2006a) Ipswich Rivers Flood Study Rationalisation Project - Phase 3 "Monte Carlo" Analysis of Design Flows – Final Report prepared for Ipswich Rivers Improvement Trust and Ipswich City Council
- Sargent Consulting (2006b) Ipswich Rivers Flood Study Rationalisation Project - Re-estimation of Design Flows – Final Report prepared for Ipswich Rivers Improvement Trust and Ipswich City Council
- Sargent Consulting (2006c) Ipswich Rivers Flood Study Rationalisation Project - Re-estimation of Design Flood Levels – Hydraulic Model Calibration Report prepared for Ipswich Rivers Improvement Trust and Ipswich City Council
- Sargent Consulting (2006d) Ipswich Rivers Flood Study Rationalisation Project - Re-estimation of Design Flood Levels – Final Report prepared for Ipswich Rivers Improvement Trust and Ipswich City Council

3.2. Summary of Previous Studies

25 In 1975, following the large flood of 1974, the Queensland Survey Office published flood maps for the Brisbane and Bremer River systems. At a similar time Ipswich Council staff developed the "Gamble" maps for use in defining flood liable areas for development purposes. According to Sargent (2002b, Reference 13) no reports have been located documenting these maps and the maps have not been sighted by WMAwater for review. Reference 13 indicates that the 20 year ARI levels in the Gamble maps may have been based on observations of the 1955 event, which reached a level of 13.82 mAHD at the Ipswich gauge.

26 In 2000, SKM completed Phases 1 and 2 of the Ipswich Rivers Flood Studies (Reference 11). The study utilised models developed SKM Brisbane River work (1998, Reference 8), which were used to define Brisbane River flood levels. The study established, via flood frequency analysis conducted on flood levels rather than flows, a 1% AEP level at David Trumpy Bridge of 18.6 mAHD. SKM also undertook rainfall-runoff and hydraulic modelling work, resulting in an estimated 1% AEP level of 18.65 mAHD.

27 In 2002, Halliburton KBR completed a review (Reference 14) of the SKM 2000 report, which questioned the validity of the SKM (2000) design levels. KBR raised a number of issues primarily related to the hydraulic modelling work, including:

a. the use of an inappropriate hydraulic radius formulation, resulting in exaggerated conveyance (flow capacity);

b. excessively high roughness values;

c. poor model scaling;

d. a large proportion of cross-sections along the Bremer River reach (~70%) not extending to fully contain flood levels; and

e. an estimated reduction in modelled levels of approximately 1 m for events less than the 1% AEP when the above issues were addressed.

28 Sargent (2002b, Reference 13) made recommendations for generating composite maps from the Gamble maps, SKM (2000) and KBR (2002b) results (Phases 1, 2 and 3 of the flood study work). The report states that mapping of the 5% AEP event is far from straightforward since agreement between the studies from 2000, 2002 and the Gamble maps is poor (Table 1, Reference 13). A similar exercise was undertaken by KBR in 2004 (Reference 19).

29 In 2005 DHI (Reference 20) peer reviewed Ipswich City Council's hydraulic model (from SKM 2000 for the lower Bremer/Brisbane Rivers and from KBR (2002b) for the upper Bremer River). DHI were engaged to work on the model, and in 2006 DHI submitted a report detailing the changes made to the model and the impact of these on modelled calibration events (Reference 21). Recalibration is stated as being required and as per

other previous reports (Sargent 2002a, KBR 2002a) model schematisation was highlighted as an issue requiring further attention. In particular, DHI recommended separate schematisation of overbank and river flowpaths, and highlighted sensitivity analysis as a key issue (Reference 21).

30 In 2006, the Ipswich Rivers Improvement Trust undertook the Ipswich Rivers Flood Rationalisation Project, which led to a series of four reports from Sargent Consulting. These reports document the review and revision of hydrologic and hydraulic modelling to better define design flood levels in Ipswich. The main issue driving the project was the redefinition of the Q100 (1% AEP) flow estimate in the Brisbane River resulting from review and revision of SKM's Brisbane River study in 2003 (Reference 17). The progression of the Brisbane River work is discussed in detail in WMAwater's main report (Reference 1).

31 Sargent's first report (2006a, Reference 22) looked at Monte Carlo modelling of hydrology using CRC FORGE rainfall datasets. A finding from this work was that SKM (2003, Reference 17) had used underestimates of design rainfall depths for all durations except the 72-hour event. This discrepancy in the rainfalls could possibly explain the discrepancy that SKM (Reference 17) were finding between the flow estimates for Savages Crossing derived from the two different methods used – hydrologic modelling and flood frequency analysis. These discrepancies were also discussed by the Independent Review Panel headed by Mein (2003, Reference 16). Sargent queried the suitability of the RAFTS hydrologic modelling methodology used by SKM (2000, Reference 11), specifically the use of conceptual storages in RAFTS to emulate attenuation typically associated with flood routing.

32 Sargent's fourth report (2006d, Reference 25) re-defined design levels at the David Trumpy Bridge Gauge (Table 1) and indicated that a suitable freeboard for design flood levels may be one to two metres. The report also noted that the schematisation of the hydraulic model still required revision in order to reduce uncertainty associated with Ipswich design flood levels.

3.3. History of Design Flood Estimates

3.3.1. Ipswich

33 Several of the studies discussed above defined design flood levels and extents for Ipswich and surrounds. The design flood levels for the 5% AEP (20 year ARI) and 1% AEP (100 year ARI) events are summarised in Table 1.

Table 1: Summary of Previous Design Flood Estimates at Ipswich

Year	Author	ARI	Level (mAHD)
1975†	Queensland Survey Office	110 year	16.4
2000	SKM	100 year	18.60 / 18.65
2002	Halliburton KBR	100 year	18.65
2006	Sargent	100 year	15.28
1975†	Queensland Survey Office	28 year	12
Late 1970s	Ipswich City Council (Gamble)	20 year	13.5
2000	SKM	20 year	15.11
2002	Halliburton KBR	20 year	15.43
2006	Sargent	20 year	11.36

Notes:

† Results from the 1975 study do not consider tailwater (Brisbane River flooding) and therefore are not comparable with the other estimates.

3.3.2. Savages Crossing

34 Over time the Savages Crossing stream gauge location has shifted and hence each of the stations Lowood, Vernor and Savages Crossing all have the same gauge number of 143001 however the records are differentiated by suffix. Lowood is 143001A, Vernor is 143001B and Savages Crossing is 143001 or 143001C. Each of the stations has a similar upstream catchment area of approximately (10,100 km^2) and Vernor is 1.1 km downstream of Lowood whilst Savages is a further 200 m downstream of Vernor. (Table B.2, Reference 35). The Lowood gauge has a record of 41 years (1909-50), Vernor 8 years (1950-58) and Savages Crossing 33 years (1958 to 1991).

35 In 1993, the then Department of Natural Resources undertook at-site FFA for a variety of stations including downstream of the Brisbane River/Lockyer Creek confluence at Savages Crossing, Vernor and Lowood (Reference 35). The study estimated a 1% AEP flow of 5,633 m^3/s (pre-Somerset Dam), with an increased flow estimate of 9,511 m^3/s using the post-Somerset Dam record. This unexpected result is most likely explained by the relatively short record lengths used (as a result of splitting the record into pre- and post-dam series), and also the occurrence of the large 1974 flood in the post-dam series, but no floods above 6,000 m^3/s in the pre-dam series.

36 In 1998, SKM (Reference 8) undertook more detailed FFA work at Moggill, Lowood (Savages Crossing) and Port Office on the Brisbane River. In order to adjust the flow series to remove the effect of Somerset Dam, a relationship was derived between Woodford and Silverton. The study estimated a 1% AEP flow of 8,200 m^3/s at Savages Crossing (no dams) based on 75 years of record. This analysis did not include the flood of record (1893).

37 In 2003, SKM (Reference 17) revised the FFA work to make use of prior historical floods and regional information. The study used a Bayesian maximum likelihood approach with a range of at-site and regional methods, consistent with current best practice in FFA. Case 3 (using a record from 1890 to 2000 adjusted to remove dam effects), gave an estimated 1% AEP flow of 11,900 m^3/s using a Generalised Pareto fit, and that dataset forms the basis of flood frequency work at Savages Crossing in this assessment. Based on this work SKM gave 12,000 m^3/s as a best estimate within bounds of 10,000 m^3/s to 14,000 m^3/s.

Table 2: Summary of Previous 1% AEP flow estimates at Lowood/Savages Crossing

Report	Q100 Estimate (m^3/s)	Distribution	Continuous Record	Historic Period	Comments
SKM 2003	6,690	GP	1909-1951	1909-1951	Ignores data post Somerset, post Wivenhoe and the historical 1893 event. Excludes information from regional analysis.
	14,070	GP	1909-1951	1847-1951	Includes the best estimate of 1893 historic peak (13,000 m^3/s). Ignores data post Somerset and post Wivenhoe. Excludes regional information
	11,970	GP	1909-1951	1847-1951	As per previous case including prior regional information.
	15,690	GP	1909-1951	1825-1951	No prior regional information. 1825 and 1893 peak flows of 13,200 m^3/s. Plotting position of 1825 event is outside 90% confidence interval. Magnitude is highly questionable
	13,720	LP3	1909-1951	1847-1951	No prior regional information. Includes the best estimate of 1893.
	12,660	GP	1909-1951	1847-1951	1893 peak of 14,500 m^3/s estimated by BoM. Includes prior regional information
	11,560	GP	1909-1951	1847-1951	1893 peak of 12,000 m^3/s taken from BoM URBS modelling. Includes prior regional information
	7,667	LP3	1909-1951	1847-1951	Includes best estimate of 1893 historic peak (13,000 m^3/s). Q100 determined using ARR87 method for including historical data.
	7,870	GP	1909-1982	1909-1982	Includes prior regional information. Excludes best estimate of 1893. No correction for Somerset Dam
	11,500	GP	1909-1982	1847-1982	Includes prior regional information. Includes best estimate of 1893. No correction for Somerset Dam
	11,900	GP	1890-2000	1890-2000	Analysis of "No Dams". Excludes prior regional information.
	13,150	LP3	1890-2000	1890-2000	Analysis of "No Dams". Excludes prior regional information.
	3,590	GP	1909-2000	1890-2000	Analysis of "Post Dams". Excludes prior regional information
	4,920	LP3	1909-2000	1890-2000	Analysis of "Post Dams". Excludes prior regional information
SKM 1998	8,200	LP3	1910-1985	-	No Dams. FFA Fit by eye estimate. Annual series adjusted for those years with low or no-recorded flows.
	9,190	LP3	1910-1985	-	With Dams. Peak flow derived from RAFTS modelling
SEQWater 1993	5,633	LP3	1909-1942	-	No Dams.
	9,511	LP3	1943-1978	-	With Somerset Dam only. It is understood from this report that the FFA analysis was carried out based on observed flows post construction of Somerset Dam. Report concludes that post dam flows are higher than pre dam flows due to the post dam period being wetter than the pre dam period.

3.4. Comments on Previous Studies

38 The previous studies have tended to treat design flood estimation on the Brisbane and Bremer Rivers separately. SKM (Reference 11) recognised that backwater from the Brisbane River is the dominant flood mechanism at Ipswich. This was reflected in hydraulic modelling work undertaken for the assessment, which used an envelope approach, taking the design flood level at a given location as the maximum flood level obtained from either Brisbane River, Bremer River, or local catchment critical storm durations.

39 However, such methods generally require an assumption of the likely joint probability (for example by modelling a 5% AEP tailwater in the Brisbane River in conjunction with a 1% AEP design flood on the Bremer River), and a thorough assessment of appropriate joint probability assumptions has not generally been undertaken.

40 SKM (2000) undertook flood frequency based on recorded flood heights at the Ipswich gauge. However that analysis is subject to the limitations discussed in Section 2.4 above, and the historical data are not shown on the probability plot (Figure 7.6 of Reference 11) so the appropriateness of the distribution fitted to the data cannot be assessed.

41 The issues identified by Sargent with regards to the RAFTS modelling completed by SKM are important. If the rainfalls for durations other than 72-hours are indeed underestimates as suggested, the follow-on effects of the mistake may be considerable, as this body of hydrological modelling work has been used as an input for key assessments of design flood levels in the Brisbane River system, as well as investigations into the flood-mitigation effects of Wivenhoe and Somerset Dams.

42 The use of a small number of concentrated conceptual storages to emulate routing in the SKM (2000) RAFTS modelling (also identified by Sargent (2006a)) is highly unorthodox and WMAwater do not consider it to be an appropriate method in the context of the Brisbane River system.

43 For the task of estimating design flood levels at Ipswich, the modelling issues identified by DHI (2005b) and Sargent (2006a), while needing to be addressed, are likely to have less influence on the outcomes at Ipswich than a comprehensive treatment of the joint probability issues on flood behaviour.

4. FLOOD FREQUENCY ANALYSIS

4.1. Available Data

44 The following datasets were utilised for this analysis:

- Savages Crossing gauge continuous flow record, for which a composite record of the Lowood (143001A), Vernor (143001B) and Savages Crossing records (143001C) was created, received from DERM on 21 September 2011;
- The Savages Crossing annual maximum flow series, adjusted for the influence of Somerset Dam from SKM (Reference 17);
- Amberley gauge (143108A) continuous flow record, received from DERM on 21 September 2011;
- Walloon gauge (143107A) continuous flow record, received from DERM on 21 September 2011;
- Discontinuous peak flood height record at Ipswich gauge (040101), received from BoM on 29 September 2011; and
- Mike 11 model of the Brisbane and Bremer Rivers (Version 2), received from SKM on 6 July 2011 (refer to References 31 and 32).

45 Where flows records have been required, WMAwater have relied upon the flows provided by DERM, and have not checked the conversion of the gauge water level record against the applicable rating curve for the gauge.

4.2. Selection of Gauges

46 Previous studies have included flood frequency analysis at various gauges, and in the process have made an assessment as to the usefulness of the gauge record, accuracy of the rating, and other considerations. These assessments regarding the suitability of various gauge records were comprehensive and have been used by WMAwater to inform the selection of gauges for the present analysis.

47 The Moggill gauge was excluded from the analysis, as Sargent (References 24 and 25) indicated major issues with unstable channel shape at Moggill. Additionally, the continuous flow record provided to WMAwater is relatively short (1992 to present) and also contained spurious measurements (above 70 mAHD), which limited the usefulness of this gauge for this analysis.

48 Walloon and Loamside were also identified as being stations with relatively unreliable hydraulic characteristics and/or poor ratings by Sargent (2006d, Reference 25). The primary gauges selected for use in the flood frequency analysis were:

- Warrill Creek at Amberley (143108A); and

- Savages Crossing on the Brisbane River (143001A/B/C).

4.3. Methodology

4.3.1. Joint Probability Approach

49 The interaction between the Bremer and Brisbane River flood mechanisms at Ipswich is critical, and therefore was a central consideration in determining an appropriate methodology for the present assessment.

50 The approach used is based on an analytical technique proposed by Eric Laurenson (1974, Reference 5). The technique has a broad range of hydrologic applications, and its suitability for flood frequency analysis at locations where joint probability is important (such as a river confluence) was specifically acknowledged by Laurenson. Essentially, the approach allows for an at-site flood frequency analysis on one branch of the system to be transposed to another location, provided there is a sufficient understanding of:

a. the correlation between flows on the two contributing river branches (i.e. for a given flow on one branch, an estimate of the probability distribution of flow on the other branch); and

b. the physical interaction of the two branches at the confluence (i.e. an understanding of the flood level produced by coincident flows at varying magnitudes).

51 The data required to undertake this analysis at Ipswich are therefore:

a. a long continuous flood record on both the Brisbane and Bremer Rivers upstream of the confluence;

b. the gauges should preferably be far enough upstream from the confluence to be relatively free of backwater influence, but close enough to the confluence to capture a large percentage of the upstream catchment for the tributary, and

c. a series of rating curves giving flood heights at Ipswich for varying combinations of flow in the Bremer and Brisbane River systems.

52 For this analysis, the Savages Crossing gauge was selected as most appropriate for the Brisbane River component, and the Amberley gauge for the Bremer River component. It is possible that the Mt Crosby gauge could be used in place of Savages Crossing, as both gauges have a similar length of record. Savages Crossing was selected in this instance as considerable attention has already been given to FFA at this gauge in previous studies. The Amberley gauge on Warrill Creek was considered more suitable than the Walloon gauge, as it captures a larger proportion of the Bremer River catchment and is recommended by Sargent as having the more reliable rating curve.

53 Savages Crossing and Amberley represent good locations for the inputs into the joint probability analysis as they satisfy the criteria identified above. Savages Crossing also provides a good primary probability input as it has a relatively long record and the FFA work undertaken to date by SKM (Reference 17) has been comprehensive.

54 The relationship between flood level at Ipswich and the Brisbane/Bremer flows requires a large amount of data in order to be well defined across a broad range of flood magnitudes. As the gauge at Ipswich is non-continuous and only a limited number of historical data were available, the relationship was developed by supplementing the available historical data with hydraulic modelling results, using the Mike11 model provided to WMAwater by SKM (reviewed by WMAwater in Reference 31). While problems have been acknowledged with the Bremer River schematisation in the model, this was considered the most appropriate method to undertake the required analysis in the available timeframe.

55 A detailed description of the application of Laurenson's methodology to flood frequency analysis at Ipswich is provided in Appendix B.

56 The adopted FFA methodology combines the contribution of Brisbane River and Bremer River flooding. Additionally, the influence of Wivenhoe and Somerset Dams can be included in the analysis via appropriate adjustment of the Brisbane River data (at Savages Crossing in this instance) to represent "no dams" or "with dams" conditions.

4.3.2. Savages Crossing FFA

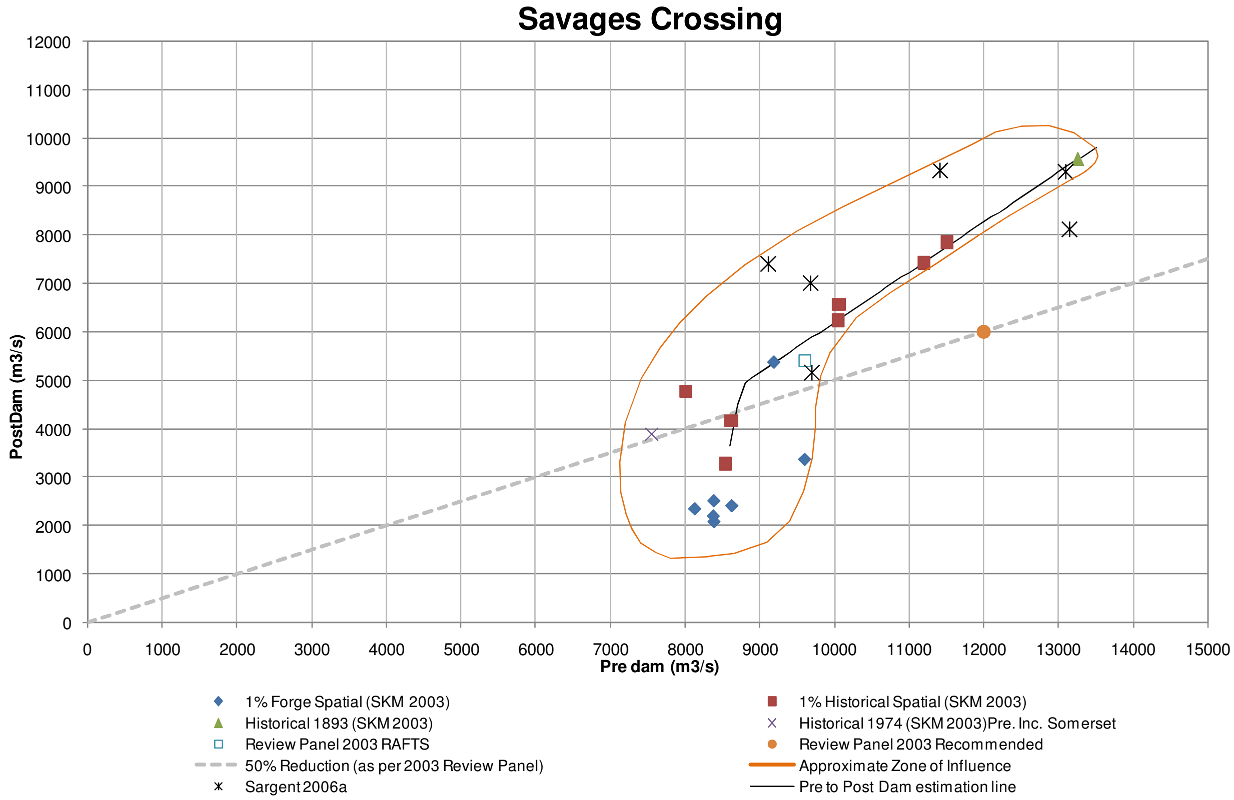
57 Under the adopted methodology, flood frequency curves at Savages Crossing are a key input for obtaining flood frequency estimates at Ipswich. Previous studies have investigated flood frequency at Savages Crossing using at-site and regional approaches under a wide range of assumptions, as summarised in Section 3.3.2.

58 For the purposes of this study, WMAwater utilised the annual maximum flow series provided to SKM by DNRM and utilised in the SKM (2003) study (Appendix D, Reference 17). The data series extracted from that Appendix was for the period from 1890 to 1955. The annual series from the SKM (1998) study (Appendix E, Reference 8) was used for the period after this, but prior to Wivenhoe Dam construction, from 1956 to 1985. The effect of Somerset Dam was already removed from this SKM (1998) dataset. Recorded flows from the DERM gauge data were used to complete the annual series period from 1985 to 2011. These data were adjusted by WMAwater to account for the influence of Wivenhoe Dam. The adjustment factor was determined by fitting a line to historical and modelled data points estimating the dam effects at Savages Crossing (Figure 2). The full annual series used by WMAwater is given in Appendix C along with the relevant sources.

59 Figure 2 is similar to Figure 5 of Reference 1 with additional points from Sargent 2006a (Reference 22). The additional Sargent data is consistent with the original SKM data and is based upon the same model. While the graph shows there is considerable scatter in the

mitigation of peak flow, it was necessary for this simplified joint probability assessment to assume a single relationship for flows above 3,600 m^3/s to represent average expected behaviour.

Figure 2: Flow Adjustment for Wivenhoe and Somerset Dam at Savages Crossing



60 WMAwater used the FLIKE program to undertake the FFA at Savages Crossing. The data were tested against the LP3 and GEV distributions, and the analysis was repeated with and without the January 2011 flood event.

4.4. Limitations

61 There are significant limitations for the application of the adopted Laurenson methodology at Ipswich, as follows:

a. The three-way relationship between flood level at Ipswich, discharge in the Bremer River, and discharge in the Brisbane River is not well-defined, particularly for larger floods. This relationship could be better understood via further hydraulic modelling, and the implementation of a continuous water level recorder at the Ipswich gauge.

b. The Savages Crossing gauge has been moved on two occasions, being originally located at Lowood and then briefly at Vernor before being placed at the current position. These moves may have interfered with the continuity of the gauge record. Additionally, the construction of Wivenhoe and Somerset Dams introduces discontinuities in the record.

c. The uncertainty surrounding the effect of the dams on flow at Savages Crossing is compounded by the Lockyer Creek component of flow, which is not subject to attenuation from the dams. Methods to address this uncertainty (such as Monte Carlo approaches) have been discussed in previous reports to the Commission (Reference 33 and 36).

d. As is generally the case for flood frequency analysis, there is some uncertainty regarding the rating curves for the gauges, as the stage-discharge observations that have been used to generate the ratings often do not cover very high levels of flow. These ratings can also be supplemented by hydraulic modelling; and

e. The length of record at Amberley (dating from 1961) is relatively brief.

4.5. Results

62 Figure 3 and Figure 4 display the results of flood frequency analysis at Savages Crossing with and without January 2011 data respectively for the "no dams" case. The estimated flows for various return probabilities are summarised in Table 3.

Figure 3: Flood Frequency at Savages Crossing including January 2011 data (GEV) – No Dams

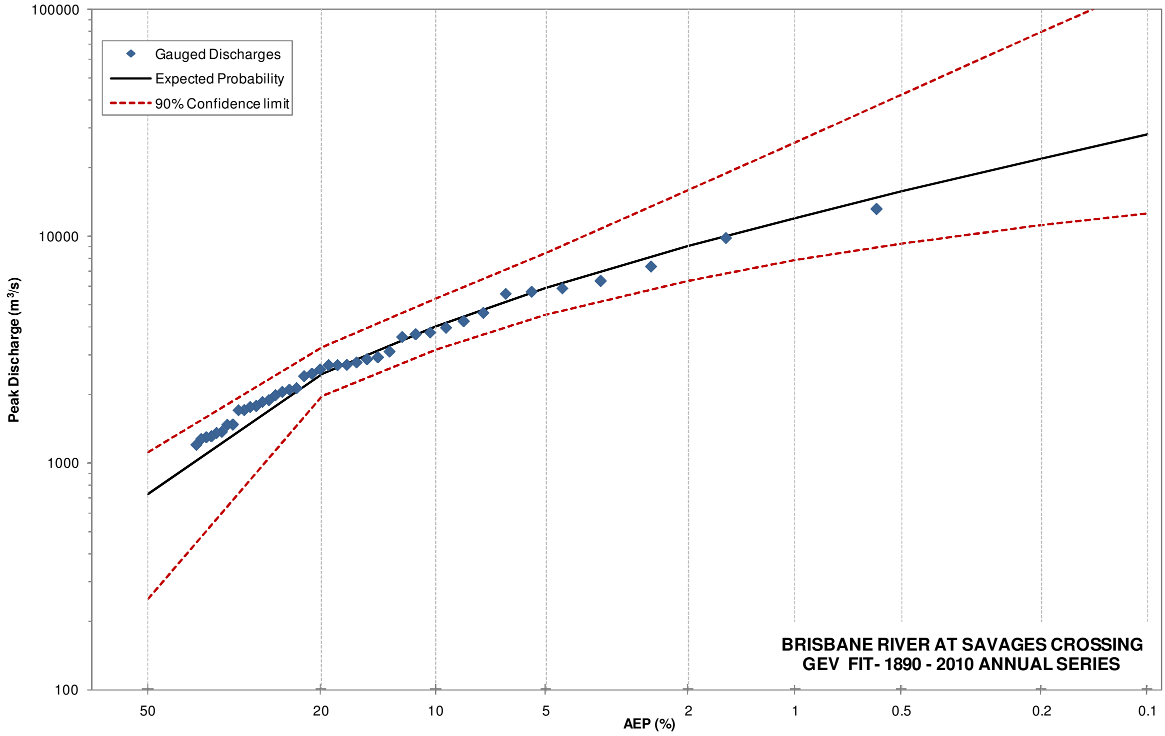


Figure 4: Flood Frequency at Savages Crossing without January 2011 data (GEV) – No Dams

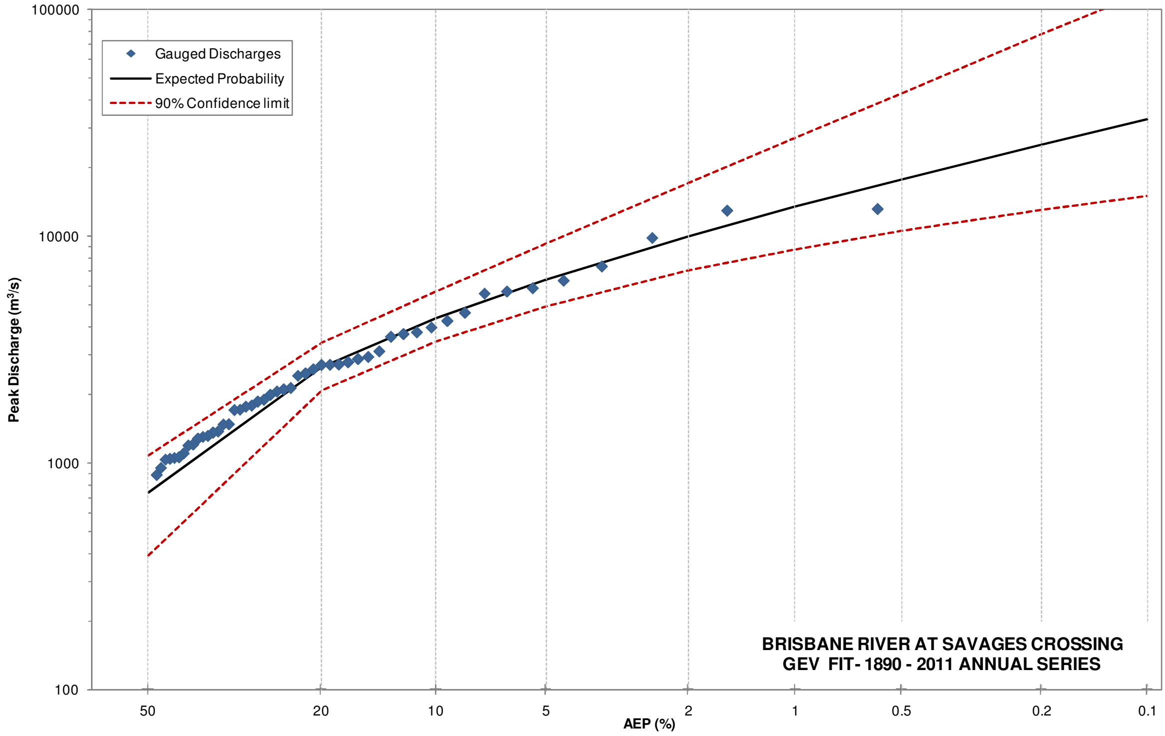


Table 3: Design flow estimates (m^3/s) from flood frequency at Savages Crossing

ARI	Excluding January 2011 Data		Including January 2011 Data	
	No Dams	With Dams	No Dams	With Dams
200 year	15,700	12,100	17,800	14,000
100 year	12,000	8,300	13,500	9,800
50 year	9,000	5,200	10,000	6,200
20 year	5,880	2,560	6,430	2,780
10 year	4,020	1,810	4,340	1,942
5 year	2,470	1,190	2,630	1,254

63 The adjusted annual series used for the Savages Crossing analysis is provided in Appendix C, along with LP3 fits to the data.

64 The flood frequency curves for both “no dams” and “with dams” scenarios obtained at Ipswich (David Trumpy Bridge), including information from the January 2011 flood, are presented in Figure 5.

65 Historical flood heights are also plotted on Figure 5, in two separate series. In water years (July to June) with multiple floods, only the annual maximum is included. The points marked with triangles represent floods with no mitigation from Wivenhoe or Somerset Dams, while squares indicate flood heights with both Wivenhoe and Somerset Dams in place. Solid markers indicate a recorded level at David Trumpy bridge, while hollow markers indicate that the recorded level has been adjusted to account for the removal/introduction of the dams. Adjustments were made based on the relationships developed in Figure 2 and Figure B6 (Appendix B). Error bars are provided as an indication of uncertainty involved with this procedure.

66 The flood frequency curves at Ipswich obtained without using the January 2011 flood data are plotted on Figure 6. Note that the plotting position of the historical data (particularly the larger events) also changes slightly as a result of the removal of the highly ranked January 2011 event.

Figure 5: Flood Frequency Curves at David Trumpy Bridge including January 2011 data

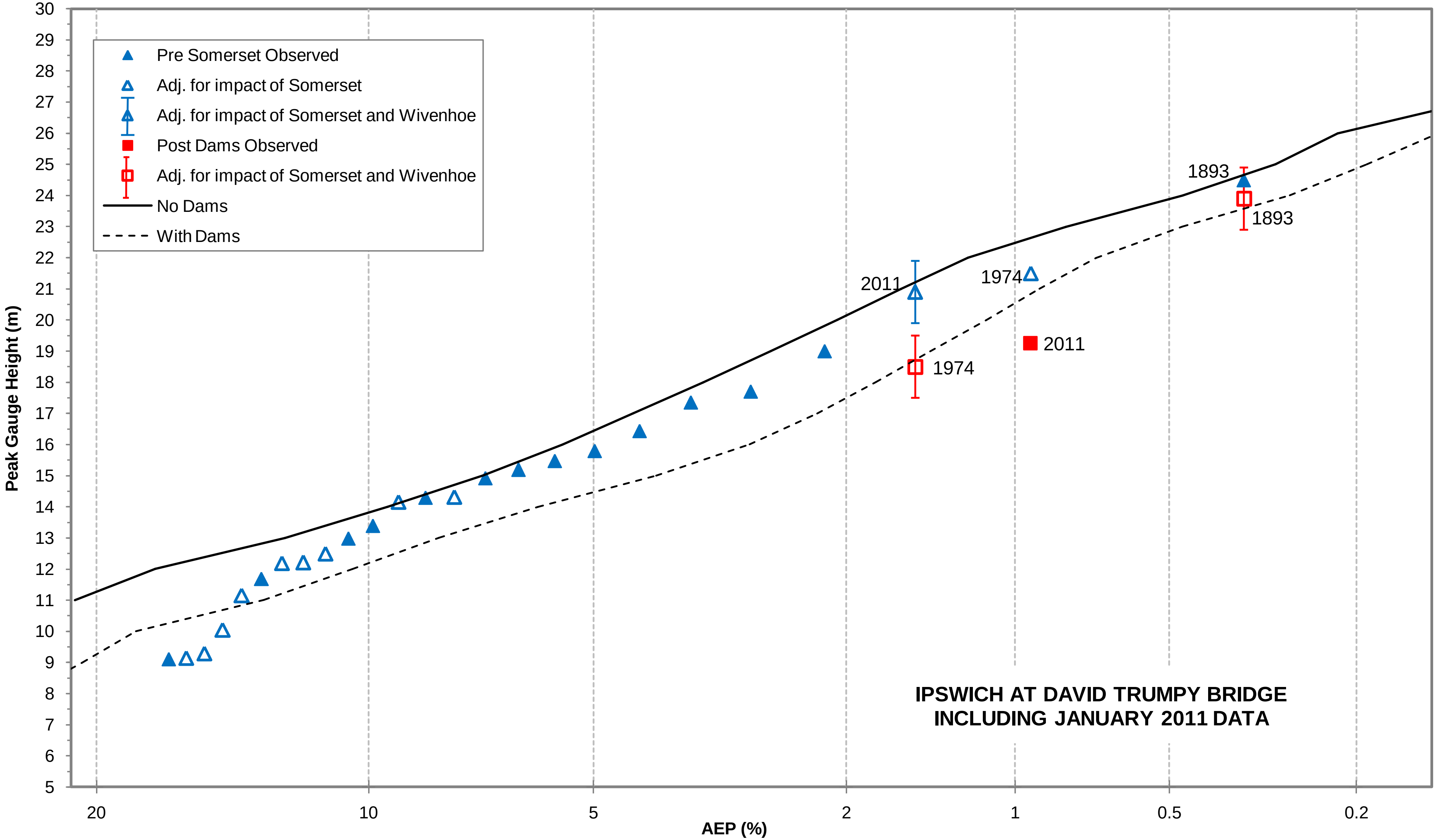
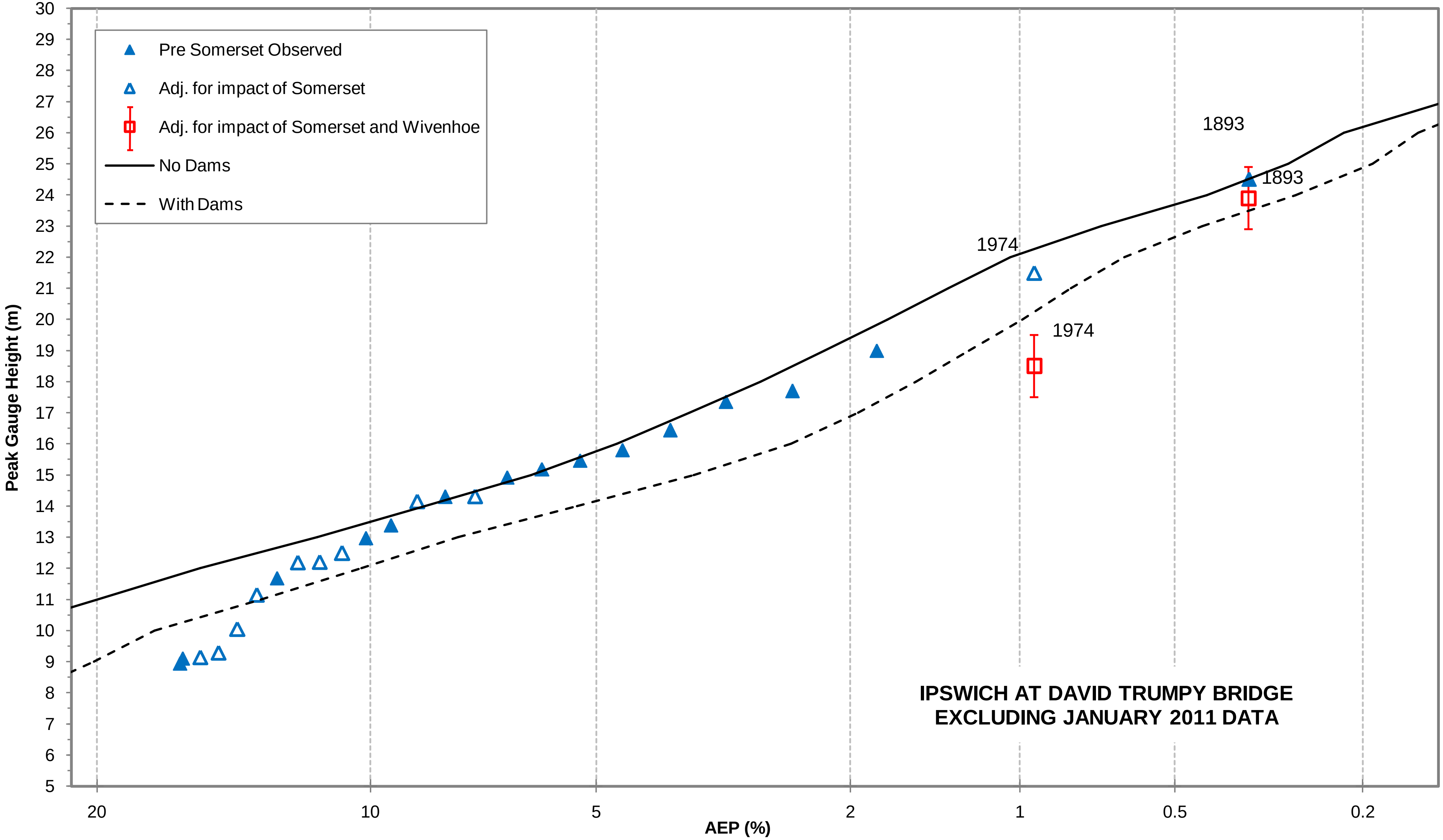


Figure 6: Flood Frequency Curves at David Trumpy Bridge without January 2011 data



67 It is important to note that on Figure 5 and Figure 6, the flood frequency curves are not actually derived from a distribution fitted to the plotted historical data points, as would typically be the case for FFA. The fact that the curves produce a reasonable match with the historical data provides some confidence that the methodology described in Appendix B is appropriate and robust, despite the limitations in the available data (as discussed in Section 4.4).

68 Another important observation is that the estimates at the rarer end of the flood frequency curve (such as the 1% AEP level) are not heavily influenced by the estimates for more frequent events (such as the 20% AEP to 5% AEP events). Therefore the results for the 1% AEP flood level are insensitive to the assumptions made about the influence of Wivenhoe and Somerset Dams on Savages Crossing flows below about 9,000 m^3/s (pre-dams). That is, although the effects of the dam are relatively uncertain below this level (Figure 2), the assumptions made in this flow range do not significantly affect the 1% AEP flood level estimate, which is primarily driven by the 1% AEP flow estimate at Savages Crossing (about 12,000 m^3/s for no dams without 2011 data), and by the correlation relationship with Bremer River flows.

69 The design flood levels at David Trumpy Bridge estimated from the analysis are summarised in Table 4.

Table 4: Design flood level estimates (mAHD) at Ipswich

ARI	Excluding January 2011 Data		Including January 2011 Data	
	No Dams	With Dams	No Dams	With Dams
200 year	23.7	22.7	23.9	22.9
100 year	22.1	20.0	22.5	20.6
50 year	19.4	16.9	20.2	17.5
20 year	15.8	14.2	16.5	14.5
10 year	13.5	12.1	13.8	12.2
5 year	11.0	9.1	11.3	9.3

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Summary

70 Estimation of design flood levels at Ipswich is a complex task, primarily due to the difficulty in quantifying the joint probability and physical interaction of the Brisbane and Bremer River flood mechanisms, both of which have the potential to produce major flooding at Ipswich. The difficulties are further compounded by the wide range of flood levels experienced historically.

71 Substantial effort was devoted to the estimation of design flood levels at Ipswich between 1998 and 2006. These studies included the development of hydrologic and hydraulic models, which appear to have been generally used to consider the Brisbane and Bremer River flood mechanisms independently when estimating design flood levels.

72 The most recent studies by Sargent (Reference 25) and DHI (Reference 21) recommended that further work was required, including substantial revision of both hydrological and hydraulic models (due to issues identified with the modelling methodology), recalibration of models, and re-estimation of design flood levels and extents.

73 The issues identified with the modelling work undertaken to date casts doubt on the validity of the design flood estimates, particularly in light of the lack of attention given to the crucial issue of joint probability.

74 WMAwater have presented a methodology for flood frequency analysis at Ipswich that can be used to address the joint probability issues identified above. The methodology has been used to estimate the probability of various flood levels at Ipswich, taking into account the mitigation effects of Wivenhoe and Somerset Dams. The estimated flood levels are generally higher than those estimated in previous studies, mainly due to higher design flows adopted for the Brisbane River.

75 The limitations of the adopted methodology are outlined in Section 4.4, and are primarily related to issues with the available data. WMAwater have attempted to identify methods for reducing these uncertainties, and in particular where data mining or modelling techniques could be used to supplement the data used for this assessment.

76 Despite the limitations identified, the adopted methodology directly assesses the crucial issue of joint probability of Brisbane River and Bremer River flood mechanisms at Ipswich, and produces a flood frequency curve that plots well against the likely probabilities of historical data.

77 The FFA work undertaken by SKM (Reference 17) at Savages Crossing is comprehensive and reflects best practice. While it would have been preferable to have access to these data for direct use in this assessment, the results were reproduced reasonably well with the relatively simple flow adjustment relationship indicated in Figure 2.

5.2. Ipswich 1% AEP Flood Level

78 The analysis undertaken by WMAwater gives an estimated 1% AEP flood level at Ipswich (David Trumpy Bridge) of 20.6 mAHD. Without the inclusion of data from the January 2011 flood event, the 1% AEP flood level estimate is reduced to 20.0 mAHD. A full range of flood levels from the analysis are presented in Section 4.5.

79 Due to limitations with the data used for the analysis, and recognising that Ipswich is subject to large variability in flood levels, these flood estimates have a relatively wide range of uncertainty. It would be reasonable to consider the estimates for the 2% AEP and 0.5% AEP flood levels (i.e. 17.5 mAHD to 22.9 mAHD) as an indicative range for the 1% AEP flood level.

80 Based on direct interpolation of the flood frequency analysis, the January 2011 event would be equivalent to approximately a 1.35% AEP (75 year ARI) flood at Ipswich (David Trumpy Bridge). The curve obtained appears to be somewhat high compared to the plotted historical data for rarer events, and therefore a more detailed analysis is likely to produce an estimate closer to the 1% AEP level.

81 Flood profiles within Ipswich and levels at locations of interest identified by The Commission were not produced as part of this assessment, as the available modelling tools and data were insufficient to complete such an analysis.

5.3. Recommendations

82 WMAwater have identified strategies to reduce the uncertainty of Ipswich design flood level estimates, which are generally consistent with the recommendations from previous WMAwater reports to The Commission (References 1, 31 and 33).

83 A high quality two-dimensional hydraulic model with a practical run time and a calibration focus on a range of recent events, including the 2011 flood, is required for the Brisbane and Bremer River systems to better understand their interaction. The model should be built using detailed and up to date bathymetric and topographic survey data.

84 Uncertainty associated with various aspects of the joint probability analysis undertaken for this assessment could be substantially reduced by further work. The physical relationship between Brisbane and Bremer River flows and levels at Ipswich could be better defined with access to reliable hydraulic modelling tools of this area (preferably two-dimensional).

85 The mitigation effect of Wivenhoe and Somerset Dams on flow at Savages Crossing has been treated deterministically for this study, although Figure 2 suggests there is significant variation in the attenuation factor. This aspect of the system could be incorporated into the analysis as a probabilistic variable to represent this variability.

86 Timing of flow in the Brisbane and Bremer systems has been implicitly accounted for in the flow correlation method. While this approach was sufficient for this analysis, the timing between flood peaks at Savages Crossing and Amberley could possibly be introduced into the analysis as another probabilistic variable to assess whether this is an important consideration. It is likely this could be undertaken with data already available from the gauge records, but this step was not undertaken in light of the time constraints on this project.

87 It should be investigated whether a better understanding of the correlation structure between flows on Bremer and Brisbane systems can be developed by considering historical catchment average rainfalls. Historical flow and rainfall data could be used in conjunction with calibrated models to investigate the relative timing of flows on the Bremer and Brisbane systems. The resolution of theses issues would allow flooding at Ipswich to be assessed in a Monte Carlo framework, and independently checked against the joint probability method used in this report.

88 As recommended by the Commission in its Interim Report, Stochastic/Monte Carlo analysis should be used to better understand the impact of Wivenhoe and Somerset dams on flows at Savages Crossing and (by extension) flooding at Ipswich.

89 The FFA work undertaken for Savages Crossing by SKM (Reference 17) should be updated to include the January 2011 event.

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Appendix A

APPENDIX A: GLOSSARY

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

Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.

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

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

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

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

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

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

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

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

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

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

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

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

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

Those areas of the floodplain where a significant discharge of water occurs during

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

mathematical/computer models The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.

minor, moderate and major flooding Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood:

minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded.

moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered.

major flooding: appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

peak discharge The maximum discharge occurring during a flood event.

Probable Maximum Flood (PMF) The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

Probable Maximum Precipitation (PMP) The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.

probability A statistical measure of the expected chance of flooding (see AEP).

risk Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.

runoff The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.

stage Equivalent to "water level". Both are measured with reference to a specified datum.

stage hydrograph A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.

water surface profile A graph showing the flood stage at any given location along a watercourse at a particular time.

Appendix B

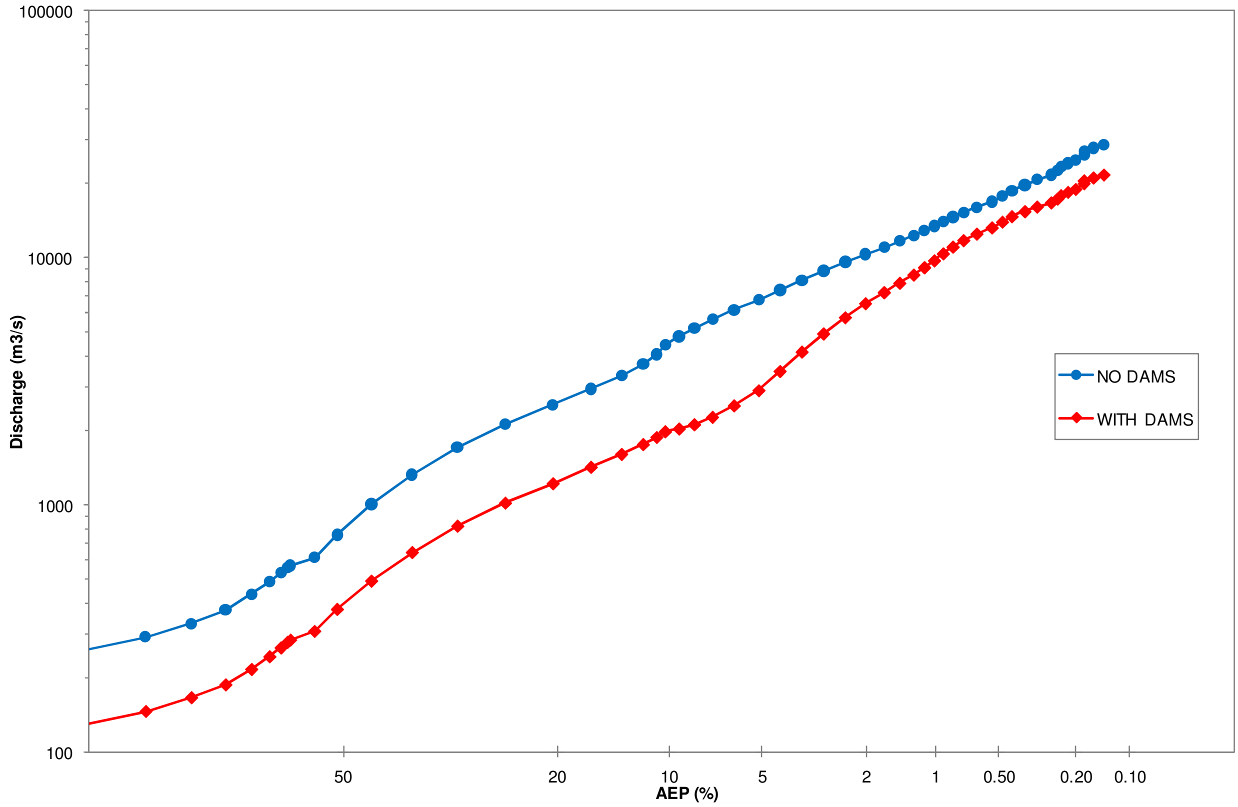
APPENDIX B: Ipswich Flood Frequency Methodology

B1 The methodology used to conduct the joint probability flood frequency analysis as applied at Ipswich is recounted step-by-step in this section. The methodology is adapted from Laurenson (1974), and particularly *Example 4* from the method published in Water Resources Research (Reference 5). The notation conventions from that paper are also utilised here.

B2 Step 1 – Estimate Flood Frequency Curve at Savages Crossing. This step was undertaken using a standard Bayesian flood frequency approach (implemented using the Flike program). The flood frequency curve was first estimated for "no dams" conditions using the adjusted annual series from Appendix C, for flows greater than 1,000 m^3/s.

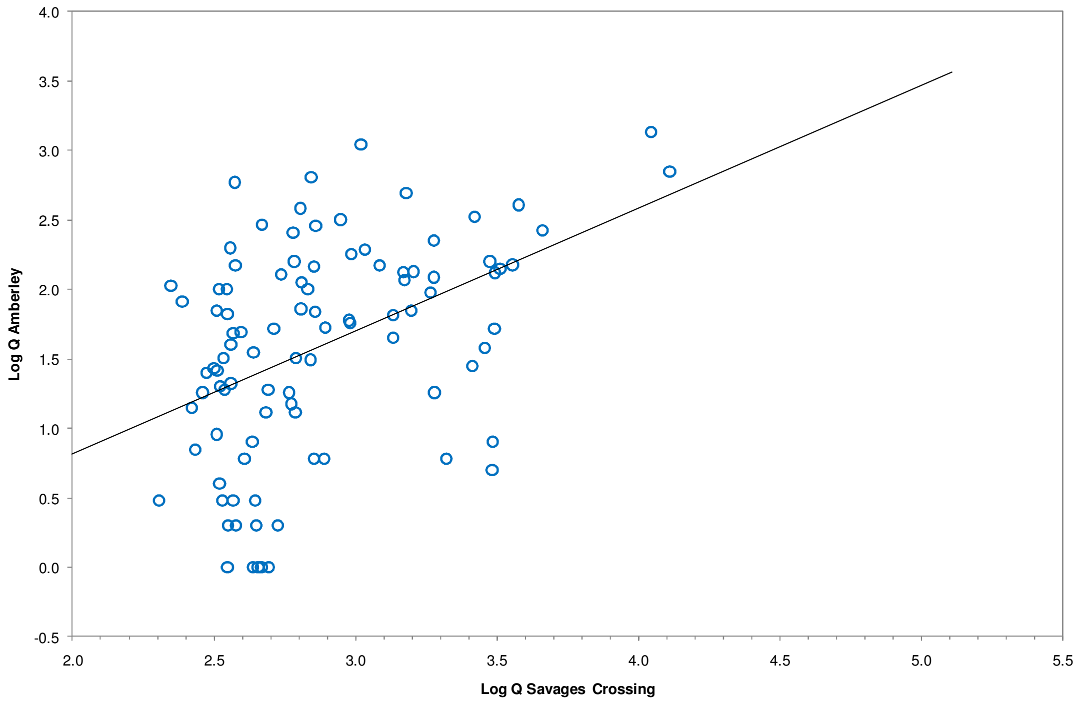
B3 The "no dams" curve was then adjusted to represent "with dams" conditions based on the relationship illustrated in Figure 2.

Figure B1: Input flood frequency curves at Savages Crossing



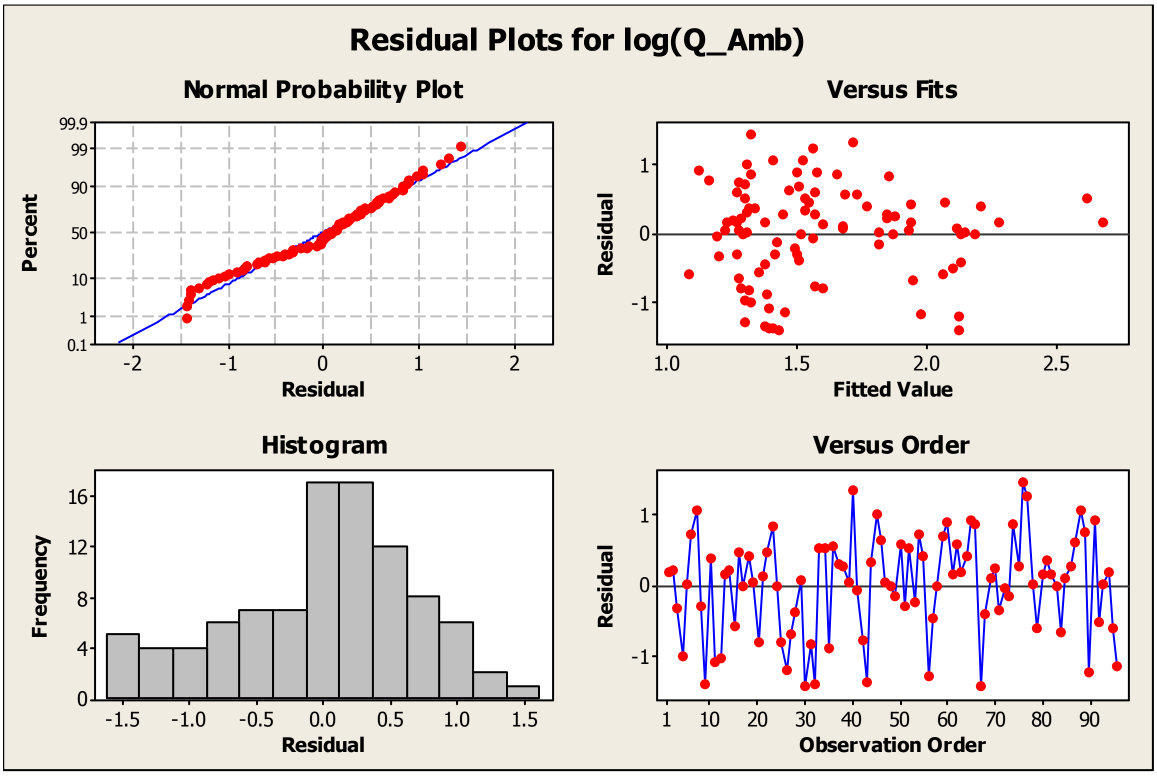
B4 Step 2 – Develop conditional flood frequency curves. The analysis requires a probabilistic description of the likelihood of various flows at Amberley (Q_{Amb}) being exceeded for a given flow at Savages Crossing (Q_{Sav}). First a series of flow pairs was extracted from the continuous gauge records. Flow peaks at Savages Crossing greater than 100m^3/s and separated by more than 5 days were identified. The continuous flow record at Amberley was then searched for flow peaks occurring within 12 hours of the Savages Crossing peak. 259 events meeting the criteria were identified at Savages Crossing, but of these only 96 had matching flows at Amberley, mainly as a result of the much shorter flow record. The flow pairs were log-transformed and a linear regression was fitted, as plotted in Figure B2.

Figure B2: Regression of Amberley discharge against Savages Crossing discharge



B5 The residuals of the log-log regression were found to be reasonably well represented by a normal distribution (Figure B3, upper left). When plotted against $\log(Q_{Sav})$, the variation in the residuals appears to reduce with increasing flow at Savages Crossing (Figure B3, upper right). If these data are representative of the general flow correlation between Amberley and Savages Crossing, this observation is consistent with the expectation that more closely correlated flow behaviour can be expected for larger floods (see Paragraph 13).

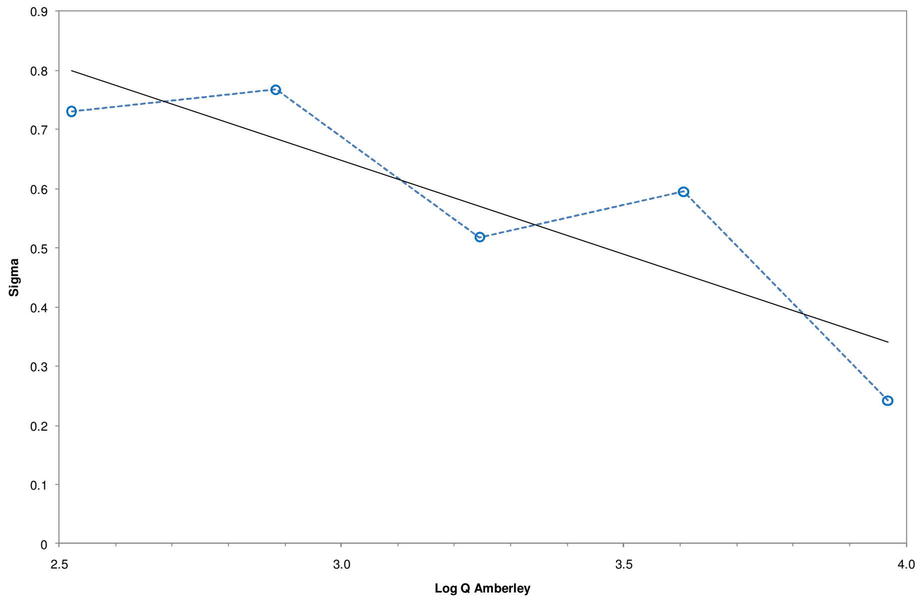
Figure B3: Residuals plots for regression of Q_{Amb} using Q_{Sav} as a predictor



B6 Based on the above findings, it was considered reasonable to separate the residuals into five bins based on Q_{Sav}, and estimate the change in standard deviation of the residuals based on the samples in each bin. The calculated standard deviations were then used to define a log-normal probability distribution of Q_{Amb}, conditional on Q_{Sav}:

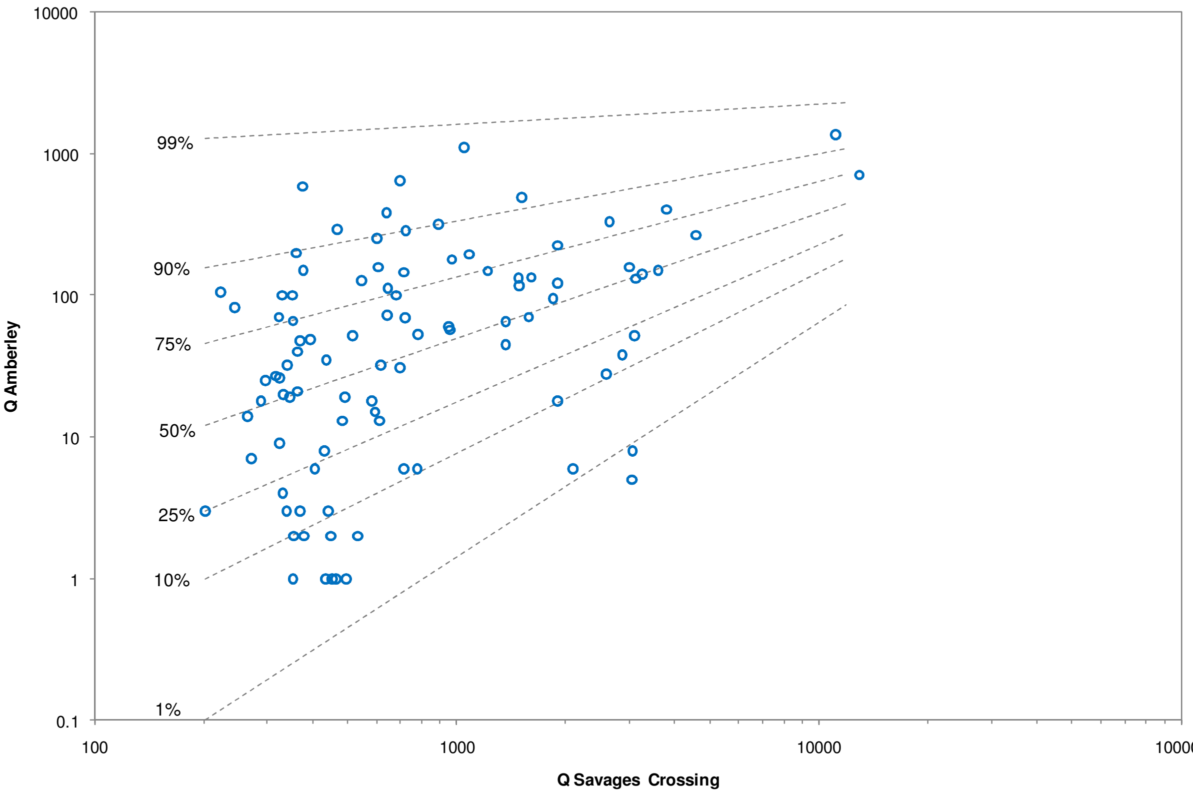
$$Q_{Amb}|Q_{Sav} \sim \log N(\mu, \sigma^2)$$

Figure B4: Trend of standard deviation of Q_{Sav}/Q_{Amb} regression residuals



B7 The mean (μ) of this conditional distribution is estimated from the log-log regression, and the standard deviation (σ) is estimated from the binned residuals described above. The conditional probability thus obtained is presented graphically in Figure B5, with dotted lines indicating the 99th, 90th, 75th, 50th, 25th, 10th, and 1st percentile respectively (top to bottom) of Q_{Amb} for a given observed value of Q_{Sav}.

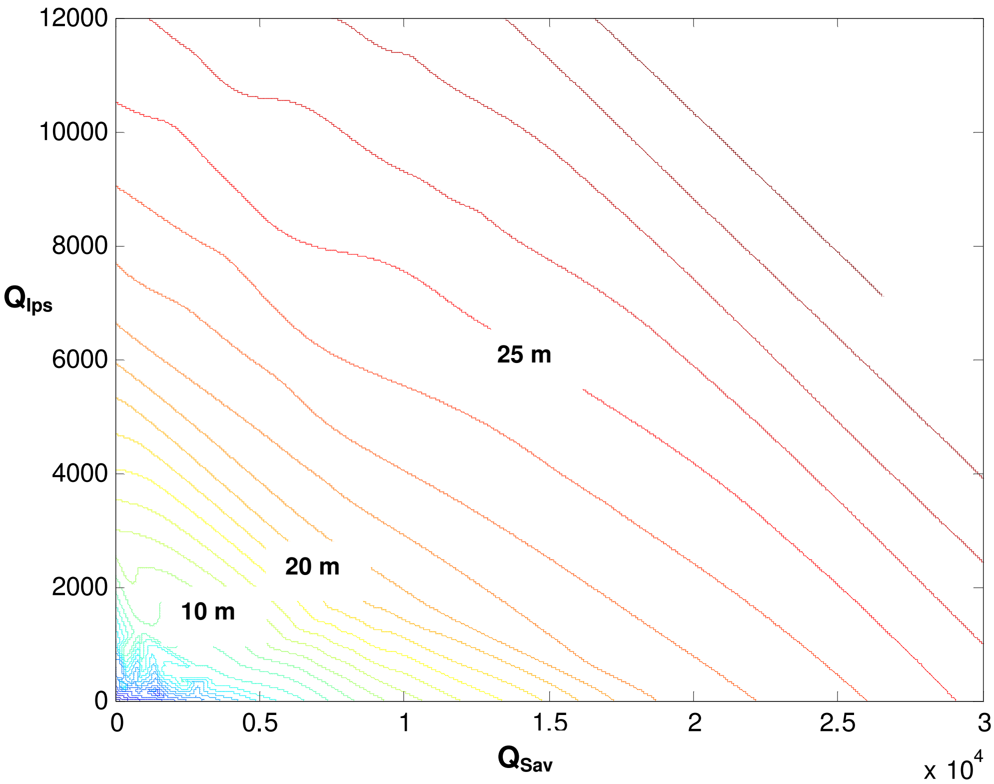
Figure B5: Probability curves for Amberley discharge conditional on Savages Crossing



B8 Step 3 – Develop backwater relationship at Ipswich. In addition to the correlation from Step 2, which represents the likelihood that flood producing rainfall on the Brisbane River system will produce a flood of various magnitudes on the Bremer River (using Warrill Creek at Amberley as a proxy), a relationship representing the physical interaction of Brisbane and Bremer River flows at the confluence is required.

B9 There is a paucity of historical data to develop this relationship, as water levels at Ipswich are not recorded continuously. 78 observations of peak height at Ipswich (H_{Ips}) are available in the period from 1840 and 2011, and of these only 45 concurrent observations are available for Q_{Sav} and Q_{Amb} or Q_{Wal} (flow at the Walloon gauge). There are only two events higher than 14 mAHD at Ipswich with recorded values at all relevant gauge stations (1974 and 2011). It was therefore necessary to supplement the data with results from the MIKE11 model. The model was used to estimate H_{Ips} for various values of Q_{Ips} and Q_{Sav}, particularly higher flows. The combined historical and modelled dataset was gridded to develop a relationship between flow at Savages Crossing, flow at Ipswich (based on flows at Walloon and Amberley for historical data), and flood height at Ipswich. Contours of the relationship developed are shown in Figure B6.

Figure B6: Contours of Ipswich flood level relationship with Q_{Sav} and Q_{Amb}



B10 Note that this relationship assumes coincident timing of flows at the Brisbane/Bremer confluence, as correlation of timing is implicitly included in the relationship developed at Step 2.

B11 Step 4 – Develop transformation matrix. A range of levels for the flood frequency curve at Ipswich was specified (from 0 mAHD to 28 mAHD in increments of 1 m). For each of the ordinates of Q_{Sav} in the Savages Crossing flood frequency curve, the relationship in Figure B6 was used to determine the required coincident value of Q_{Ips} that would result in each of the specified values of H_{Ips}.

B12 Each value of Q_{Ips} was factored to a corresponding flow at Amberley based on a simple relative catchment area relationship (assumed $Q_{Amb} = 0.6*Q_{Ips}$) The conditional flood frequency relationships developed at Step 2 were then used to estimate the probability of these values of Q_{Amb} being exceeded for the specified value of Q_{Sav}.

B13 For example, for a value of Q_{Sav}=10,000 m^3/s, the flow at Ipswich that would result in an Ipswich flood level of 21 mAHD is estimated to be approximately 2,000 m^3/s (Figure B6), which corresponds to an estimated flow at Amberley of 1,200 m^3/s. Based on the conditional flood frequency relationships, the probability of this flow being exceeded at Amberley for a Savages Crossing flow of 10,000 m^3/s is approximately 6%.

B14 Using this methodology a matrix, *A*, was established giving the conditional probability of Q_{Amb} based on Q_{Sav}, resulting in the specified values of H_{Ips}. The Savages Crossing flood frequency curve was sampled at 66 ordinates, giving a matrix with 29 rows (corresponding to the specified values of H_{Ips}) and 66 columns (corresponding to the ordinates of Q_{Sav}).

B15 The flood frequency curve at Ipswich $P(H_{Ips})$ was then obtained by matrix multiplication of the Savages Crossing flood frequency curve:

$$P(H_{Ips}) = A \times P(Q_{Sav})$$

B16 The analysis was repeated with and without the data from the 2011 flood event, and for both the “no dams” and “with dams” scenarios.

Additional Comments

B17 The flow correlation relationship developed at Step 2 suffers from complications for the “with dams” scenario, for two reasons. First, the underlying physical basis for the correlation is that flooding in the Brisbane and Bremer river catchments is often caused by rainfall from the same broad-scale meteorological systems. After the dam is constructed, this correlation does not necessarily change. That is, although the peak discharge at Savages Crossing may be reduced from say 13,000 m^3/s to 10,000 m^3/s by mitigation from the dams, the weather system which produced the “no dam” flow of 13,000 m^3/s would suggest a larger expected flow in the Bremer River. From this perspective, the conditional probability should always be determined using adjusted “no dams” flows at Savages Crossing.

B18 However the second consideration is that the Wivenhoe Dam flood mitigation procedures contain an explicit objective to avoid peak releases that coincide with peak Bremer River flows. The degree to which this objective can be achieved will vary with every flood.

B19 To some extent these two considerations will cancel each other out, suggesting that under "with dams" conditions the conditional probability of Q_{Amb} can be estimated using the reduced value of Q_{Sav} from dam mitigation. For this analysis, this approach was adopted for larger Brisbane River floods (greater than 6,000 m^3/s), as these floods are more likely to have "peakier" hydrographs that can be released with more favourable timing with regards to avoiding peak Bremer River flows. However it is recognised that this aspect of the analysis needs further attention.

B20 Another aspect of the analysis that could be substantially improved by further investigation is the physical backwater relationship developed at Step 3. In particular, further hydraulic modelling based on up to date topography between Savages Crossing, Amberley and Moggill could improve the definition of this relationship, as well as clarifying timing considerations for the flood peaks.

B21 Finally, although the method of implicitly incorporating timing considerations in the flow correlations at Step 2 was considered sufficient for this analysis, the timing between flood peaks at Savages Crossing and Amberley could possibly be introduced into the analysis as another probabilistic variable. It is likely this could be undertaken with data already available from the gauge records, however this step was not undertaken in light of the time constraints on this study.

Appendix C

APPENDIX C: Savages Crossing Flood Frequency Information

Year	Flow No Dams (m^3/s)	Source
1890	7343	SKM 2003
1891	1790	SKM 2003
1892	3953	SKM 2003
1893	13156	SKM 2003
1894	1060	SKM 2003
1895	1194	SKM 2003
1896	3699	SKM 2003
1897	432	SKM 2003
1898	5889	SKM 2003
1899	211	SKM 2003
1900	313	SKM 2003
1901	885	SKM 2003
1902	142	SKM 2003
1903	1717	SKM 2003
1904	351	SKM 2003
1905	816	SKM 2003
1906	745	SKM 2003
1907	302	SKM 2003
1908	6356	SKM 2003
1909	325	SKM 2003
1910	706	SKM 2003
1911	1316	SKM 2003
1912	461	SKM 2003
1913	416	SKM 2003
1914	234	SKM 2003
1915	1035	SKM 2003
1917	375	SKM 2003
1918	522	SKM 2003
1922	1280	SKM 2003
1924	173	SKM 2003
1925	778	SKM 2003
1927	2715	SKM 2003
1928	4225	SKM 2003
1929	2064	SKM 2003
1930	749	SKM 2003
1931	5574	SKM 2003
1934	614	SKM 2003
1935	120	SKM 2003
1936	139	SKM 2003
1937	1102	SKM 2003
1938	1052	SKM 2003
1939	460	SKM 2003
1940	697	SKM 2003
1941	425	SKM 2003
1942	1360	SKM 2003
1944	1207	SKM 2003
1946	1377	SKM 2003
1947	1302	SKM 2003
1948	613	SKM 2003
1950	2930	SKM 2003
1950	1043	SKM 2003
1951	2704	SKM 2003
1953	1863	SKM 2003
1954	2111	SKM 2003
1955	5692	SKM 2003
1956	2141	SKM 1998
1958	1770	SKM 1998
1962	152	SKM 1998
1963	502	SKM 1998
1964	258	SKM 1998
1966	2481	SKM 1998
1967	2706	SKM 1998
1968	3766	SKM 1998
1971	2779	SKM 1998
1972	1995	SKM 1998
1973	531	SKM 1998
1974	9807	SKM 1998
1975	407	SKM 1998
1976	1712	SKM 1998
1978	436	SKM 1998
1979	298	SKM 1998
1980	44	SKM 1998
1981	1478	SKM 1998
1982	2873	SKM 1998
1983	2420	SKM 1998
1984	456	SKM 1998
1985	166	SKM 1998
1986	623	Figure 2
1987	32	Figure 2
1988	1898	Figure 2
1989	3103	Figure 2
1990	1482	Figure 2
1991	375	Figure 2
1992	2588	Figure 2
1993	55	Figure 2
1994	47	Figure 2
1995	40	Figure 2
1996	4590	Figure 2
1997	87	Figure 2
1998	23	Figure 2
1999	3597	Figure 2
2000	195	Figure 2
2001	951	Figure 2
2002	39	Figure 2
2003	47	Figure 2
2004	515	Figure 2
2005	86	Figure 2
2006	24	Figure 2
2007	15	Figure 2
2008	109	Figure 2
2009	715	Figure 2
2010	244	Figure 2
2011	12926	Figure 2

Figure C1: Flood Frequency at Savages Crossing with January 2011 data (LP3) – No Dams

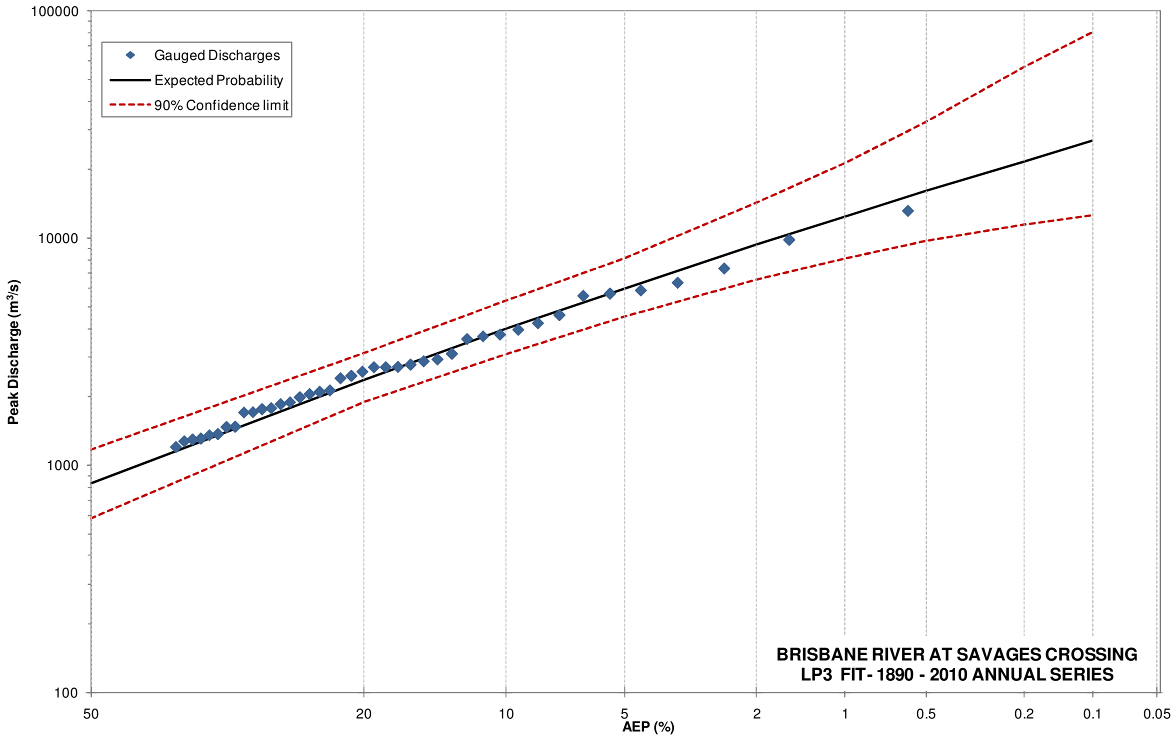
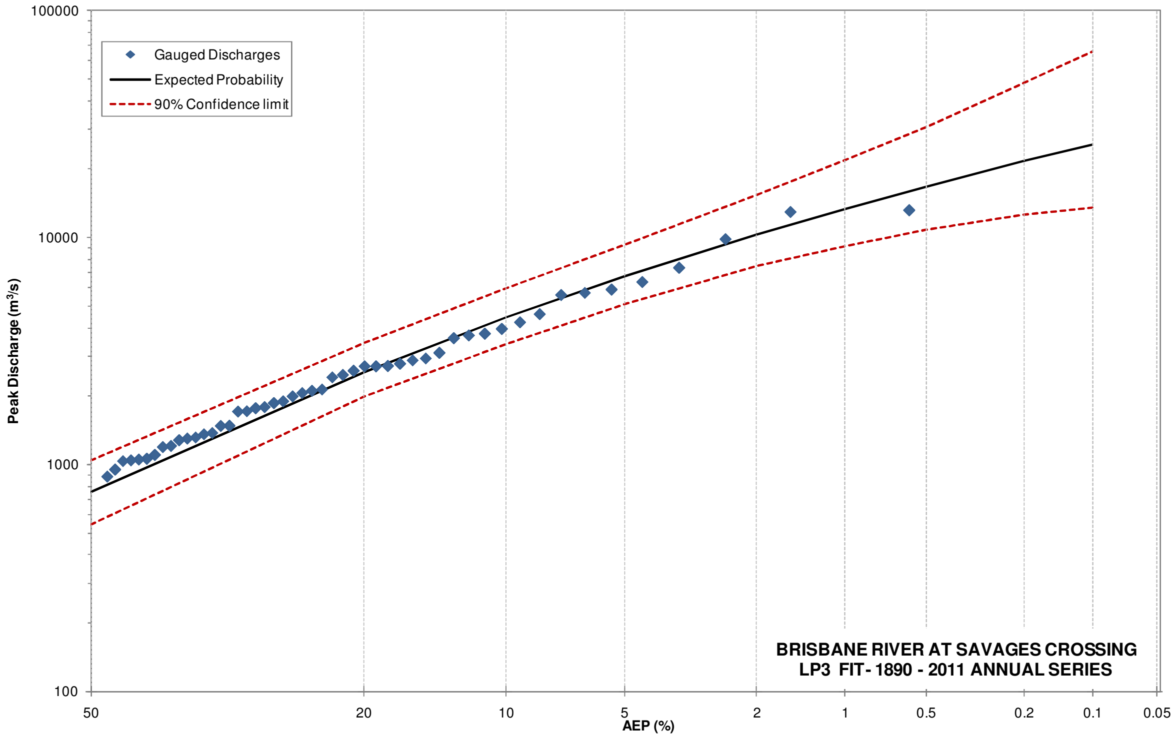


Figure C1: Flood Frequency at Savages Crossing without January 2011 data (LP3) – No Dams



Review of Supplementary Report, Ipswich Flood Frequency Analysis

PREPARED FOR

QLD Flood Commission of Inquiry

October 2011

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Executive Summary

This document is a review of the "WMA Ipswich Report" (WMA, 2011c) pertaining to flooding in the region of Ipswich (which incorporates flood frequency analysis of the Bremer and Brisbane rivers). It is a supplement to the "WMA Brisbane Report" (WMA, 2011a). The terms of reference are the same for both reports.

Since Ipswich is near the confluence of the Bremer and the Brisbane it is jointly influenced by both rivers. There are two aspects to this joint influence,

(i) the hydraulic join - where backflow from an elevated Brisbane River will cause higher water levels in Ipswich based on its proximity to the confluence and channel properties, and

(ii) the hydrologic join - because the flows down the Bremer and Brisbane Rivers are not independent of each other. Their correlation is due to the coincidence of rainfall on both catchments across the variety of storm events.

As the design levels at Ipswich are not independent of the flows down the Brisbane River, parts of the analysis are contingent on accepting the results of the WMA Brisbane Report. In fact, the WMA Ipswich Report presents significant additional reasoning on matters relating to the Brisbane River flows. This additional information requires discussion of the Brisbane River flows to be revisited. In short, they identify that data entry errors on a number of input rainfalls presented in the SKM report (2003) cause a significantly different interpretation of the results. The corrected input rainfalls support a higher post-dam estimate of flow than the SKM (2003) report.

Numerous studies of the Bremer River and flooding in the Ipswich region have been conducted previously. Putting aside questions over Brisbane River flows, there is a significant development in the methodology of the WMA Ipswich report. Where earlier studies have required limiting assumptions on the hydrologic coupling of the Brisbane and Bremer rivers, WMA have performed a joint probability analysis. This methodology is reviewed in detail because of the additional assumptions required beyond that of a standard flood frequency analysis. This review is not exhaustive on these matters, but is intended to highlight those assumptions which appear more critical.

The main short-coming of the WMA Ipswich report, as with the WMA Brisbane report, is dictated by the short time frame available to WMA: that a stochastic (Monte Carlo) assessment is required to provide the fullest assessment on the role of the dams. It is expected that modelling the variability in the dam conversion will cause significant additional uncertainty in the Q100 estimates. It is possible that this additional uncertainty is sufficiently large that any difference in the best-estimate from competing hypotheses is drowned by the variability in their resultant estimates. Such an analysis would favour greater conservatism from a risk-based analysis point of view in contrast to methods that use a deterministic dam conversion which can overstate the certainty in the resulting design estimate.

Contents

List of Tables

List of Figures

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 supplement (WMA, 2011c) is referred to here as the "WMA Ipswich report" whereas the main report (WMA, 2011a) is referred to as the "WMA Brisbane report". This report is best read in conjunction with the WMA Ipswich report. The terminology "the authors" or "WMA" is used to reference analyses expressed in either of these reports and "the reviewer" references analysis presented here or in the prior review of the WMA Brisbane report (Leonard, 2011). A summary of the reviewer's qualifications are provided in Appendix A.

The statistical methodology requires consistency in the catchment conditions such that conversions between the two must be made to account for the influence of the dams. This conversion is a central issue. A common terminology is that flow estimates as referred to as either being 'pre-dam' or 'post-dam' (similarly 'without-dams' or 'with dams'). Also note, that the term "dam" is typically used in the singular by the reviewer to refer to the combined behaviour of two dams (Wivenhoe and Sommerset). The terms Monte Carlo and stochastic analysis are used interchangeably, as both can be thought of generically even though they may taken on specific meanings in some contexts.

Another important term is the Q100. This is a design flow that will be exceeded 1% of the time in a *long run* average (1% AEP, annual exceedance 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. In the case of Ipswich which is influenced by both the Bremer and the Brisbane rivers, there is no single pair of flows that give the 1% AEP, rather it is determined from joint combinations of flows on both rivers. It is not appropriate to use the separately determined Q100 of the Brisbane River and the Q100 on the Bremer River as inputs. This is because the joint coincidence of two rare events implies the resulting combined event is even rarer.

1.1 Summary of WMA Brisbane Report

The claims of the WMA Brisbane report and the reviewer's response are summarised briefly here as background material. Given further reflection on the WMA Brisbane Report, some additional clarification and commentary is provided not otherwise made before.

WMA conducted a flood frequency analysis for the Port Office gauge and provided a Q100 flow estimate (pre-dam) of 13,000 m^3s^{-1}. The reviewer considers this estimate to be robust. A major reason for this is because it agrees well with the SKM (2003) estimate, yet it was based on a different methodology (equally valid) and a different set of data. The WMA preference of methodology is a pragmatic one. While questions over the reliability of the Port Office data are proper and the possibility of incorporating more detailed analysis along the lines of SKM (2003), these complexities take on a diminished importance because the pre-dam agreement is strong.

The pre-dam Q100 90% confidence limits provided by WMA of 10,000 m^3s^{-1} to 20,000 m^3s^{-1} are excessive. The reviewer demonstrated one method that reduces them to 10,000 m^3s^{-1} to 16,000 m^3s^{-1}. Incorporating regional techniques (as per SKM, 2003) offers another avenue for potential reduction. There is additional uncertainty due to the rating curve on top of this estimate. WMA suggest this issue is significant. Based on a qualitative analysis, the reviewer's opinion is that while this issue is important, it is not as significant as the pre-dam to post-dam conversion. A further clarification that was not given earlier is that the earlier analysis of rating curve errors was demonstrated for the case where they are equally likely to be positive or negative (Leonard, 2011). This causes the overall best estimate to remain unchanged but increases the uncertainty. If rating curve errors can be demonstrated to be significantly biased then the importance of this issue is reinstated.

The main discrepancy between the earlier SKM best estimate of 6500 m^3s^{-1} and the WMA best estimate of 9500 m^3s^{-1} comes down to assessment of the variability and average performance of the dams. There is considerable difficulty in this task as a rigorous assessment requires the input of a large variety (preferably 1000s) of large storms, whereas the historical record offers only a few. The method for achieving this is known as a Monte Carlo assessment or a stochastic method and the chief difficulty in its construction is showing that the relative occurrence rates of the storms are representative. This task is non-trivial and beyond the feasibility of the timeline imposed on WMA, yet the terms of reference require WMA to provide a post-dam Q100 best estimate. In lieu of a stochastic analysis, WMA have provided a loading curve which converts the response (post-dam flow) for a given load (pre-dam flow). The authors argue this should better match the true (but unknown) long-run average performance of the dam in contrast to prior estimates of approximately 50% reduction. Their argument, as presented in the Brisbane Report, rests on the observation that the 1893, 1974 and 2011 floods all have lesser reductions than a 50% curve and also on the inference that dam capacity to mitigate flooding should diminish for increasing flood sizes. Short of a stochastic analysis, the reviewer is persuaded that the WMA arguments for a higher estimate of 9500 m^3s^{-1} is suitable

The authors noted significant issues in determining the pre-dam post-dam conversion which revolve around unusually low estimates from rainfall methods. The authors had originally suggested climatic variability and issues constructing areal average estimates as likely reasons to explain the discrepancy. The reviewer has noted that either explanation is plausible but essentially unverified. The Ipswich report readdresses the issue of rainfall methods being biased low and they present significant additional evidence on this issue. This is discussed further in Section 2.

1.2 Overview of the WMA Ipswich Report

The authors note the complexity of inferring design levels for the Ipswich region and cite Windsor (NSW) as a precedent where sensitive changes in the AEP of the event (e.g. between a 1% AEP and a 0.5% AEP) are known to give large variation in the range of extreme events (~2 m). The authors point out the long record of events at the David Trumpy Bridge and that the three highest recorded events (1893, 1974, 2011) all have significant flows in the Bremer river that co-occurred with significant flows in the Brisbane River. The observation here is that the weather mechanisms that generate the flows in both catchments cause a level of correlation between the two that must be accounted for (i.e. a joint probability problem). The issue is relevant only for the region of the confluence of the two rivers such that elevated flows in one river will cause elevated levels upstream on the other branch (known as a backwater effect). The authors rightly point out that role of both rivers is significant to the Ipswich region, that floods can be caused either by the Bremer River alone, by the Brisbane River alone or by combinations of the two. For this reason, simplifying assumptions that ignore the dual behaviour are inappropriate (as noted in para. 38 and 39). There are two alternatives (i) perform flood frequency analysis on the observed river heights or (ii) perform a flood frequency analysis that accounts for probabilities of co-occurring flows.

The authors note that there can be pitfalls directly applying flood frequency analysis to observed river heights as the heights can be biased by localised effects. The authors instead use a flood frequency analysis of flows at two sites: Warrill Creek at Amberley and Savages Crossing on the Brisbane River. A hydraulic model is then used to account for local effects at the site of interest and determine the response heights for input flows. The quality of this model is a crucial element of the procedure and the authors highlight the need for model improvements and a better understanding of the backwater effects and timing effects.

As the heights at Ipswich are dependent on the Brisbane River, the estimates are therefore influenced by the assumed performance of the dams. For this reason the WMA estimates are tied to the assumptions presented in the WMA Brisbane report. In particular, that the reduction in flow provided by the dams is significantly less than in previous studies. This topic is reviewed further in the following section.

2. Biases in rainfall estimation

The WMA Ipswich report highlights a finding by Sargent (2006a) that earlier RAFTS modelling contained several spurious values of rainfall input (WMA, para. 31). This is a significant discovery as the low bias in rainfall estimates has been a confounding factor at the heart of understanding the behaviour of the Brisbane River dams. Since the Brisbane River system has myriad complexities to weigh up (tides, channel changes, catchment changes, gauge issues) and the number of assumptions in rainfall based analyses is large, it is not surprising that this source wasn't identified earlier. Standard flood frequency analysis is not dependent on rainfall analyses, but the presence of the dams requires rainfall based modelling techniques to determine the degree of mitigation. This degree has significant scatter depending on the incident rainfall patterns. Even floods not otherwise influenced by the dam are not immune to questioning (such as the 1893 flood) as the overall assessment must hold competing flow-based and rainfall-based information together. Either the discrepancy between the two can be explained or the reliability of one source over another is discounted. It should also be noted that this observation does not preclude other suggested possibilities for rainfall causing lower estimates (e.g. climate or rainfall gauge density).

This issue is pertinent to the Bremer in the vicinity of Ipswich as the backwater effects are a dominant flooding mechanism (WMA, para. 38). WMA observe that the corrected values, as verified by Sargent (2006a) cause higher post-dam flows than previously assumed. This supports the argument proposed by WMA in the Brisbane report that the attenuation of the dam should be less than previously assumed. While this observation gives a strong support for their argument there is still a significant degree of variability in the dam behaviour. The question is not whether one can point to a flood that was highly attenuated (e.g. 1999) or another that was poorly attenuated (e.g. 2011) but that if a great many storms were realised over the catchment, to know where does the overall average density of those storms lie? The average dam conversion performance is what defines the long-run average of the Q100. However, if the scatter of many hypothetical storms were known (and that those storms had representative occurrence rates) then more important questions could be answered regarding not just the long-run average, but the variability of flows down the Brisbane River and questions of the vulnerability in the event of a future flood. In short, the additional source of information is compelling for the argument of higher post-dam flows, but as noted from many sources, a stochastic method is needed to fully address questions over dam performance.

The reviewer accepts the estimation line and the zones of influence denoted by WMA in Figure 2. However. the revised estimates by Sargent (2006a) do not end all questions over the dam influence. Aside from debate over the significance of individual data points, the reviewer's opinion is in part due to a speculation that even if the estimation line were ultimately proven to be lower (with a stochastic analysis) that same analysis will reinforce the high level of variability and uncertainty – warranting higher greater conservatism in risk analysis. Nonetheless, some observations are made about the zone of influence in Figure 2 to indicate that judgements are still required about the dam behaviour,

- it has a significant range of post-dam flows, so questions over variability of attenuation remain
- it has been drawn skewed to suggest that the scatter may be more likely to go higher than the estimation line than below. This is only partly supported by the limited sample of data (even including revised information) as skewness is notorious for requiring a large number of points to estimate reliably.
- It has also been drawn with a sharp drop-off in the pre-dam flow vicinity of 8000 m^3s^{-1}. This is perhaps reflecting the data availability rather than a definitive statement on the physical function of the dam.

The review of WMA's joint probability analysis pursued in the following section makes use of the estimation line in Figure 2. However the methodology is generic so that it could be repeated with a 50% line, or some other functional form. The point being made is that any comparison of this nature should be done in an uncertainty framework so that any delineation of the confidence limits between the methods can be assessed (this informs whether you can statistically support one estimation line over another).

3. Joint flood frequency analysis review

A joint probability analysis is a complex task which rests on a large number of assumptions. Appendix B details a review of the WMA analysis for the purposes of testing some of these assumptions, but also to demonstrate a means for obtaining confidence inervals. The WMA report has not provided confidence intervals, no doubt due to a combination of the short time line and the computational demand involved. The reviewer feels that being able to provide confidence intervals is important for a number of reasons, amongst others (i) it causes one to assess assumptions, model parameters and their relative magnitudes of variability in detail (ii) it tests the model under a wide range of conditions (iii) it naturally cautions against over-confidence in the line of best fit by pointing to the range of possible scenarios.

To this end an attempt has been made here to determine uncertainty limits, but has fallen short because of time constraints and because of not having access to the underlying hydraulic model. An indication of the confidence limits has been determined for:

- a very small sample of 10 realisations (due to computational demand)
- a small AEP range aprox. 2% - 0.5% (due to hydraulic model approximation error)
- only the 'without dams' scenario, due to poor hydraulic model approximation

Thus the method would need to be revisited. The main benefit of an uncertainty analysis is to formally incorporate the variability implied by the pre-dam to post-dam conversion process which has otherwise assumed to be deterministic. The impact of different hypothesized conversion functions could be compared in light of the sensitivity on the final estimates. Where a significant increase in variability of the best estimate is determined this can be used to inform decision making and risk analysis.

Data files were obtained from WMA corresponding to the composite flow record at Savages Crossing (Q_{Sav}), the flow record at Amberley (Q_{Amb}) and the conversion functions used to account for the dams (Table C.1 and Table C.2). A number of datasets were extracted from these records including

- Amberley annual maximums (Table C.3),
- Coincident flows at Amberley for annual maximums at Savages Crossing (Table C.4)
- Entire record of Savages Crossing annual maximums (Table C.5)
- Peak over threshold (POT) flows at Savages Crossing with coincident Amberley flows (Table C.6)

Additionally the data underlying WMA Figure B2 and WMA Figure B6 was manually digitized based on their report,

- POT data (Figure B.6 and Table C.7) with little loss of precision
- Hydraulic model approximation (Figure B.10 and Table C.8) with significant impact on quality of results for heights less than 18m.

Two minor adjustments were made to the methodology of WMA, including

- Use of log-normal distribution (less parameters and more convenient to model)
- Formal implementation of standard deviation regression (facilitates parameter uncertainty)

A number of technical issues are raised in Appendix B, but none are considered to invalidate the general approach of WMA. A brief summary is listed here:

- 2 out of 47 of heights at Ipswich seem to be caused by annual maximums of Amberley flows rather than Savages Crossing. The issue is not significant enough to model formally as other assumptions are more critical and it is less likely to impact high flows.
- At least 3 out of 47 flows would cause a higher water level if timing considerations were given more attention

- The reviewer obtained more data points that WMA for the POT analysis although their scatter agrees well.
- An explanation for the 0.6 flow conversion factor is not given
- The Amberley flows modelled by WMA are likely to be underestimated in the lower tail. This should have little impact on the Q100 height estimates.
- Any ongoing work using a joint probability approach should present results for several alternative formulations regarding the correlation structure. The WMA model appears adequate, but the assumed correlation structure is likely to be a critical factor in controlling the exceedance estimates.

The output of the analysis in Appendix B is shown in Figure 3.1. This Figure validates the best estimate WMA results (cf. WMA Figure 5) for the region of 2% AEP to 0.2% AEP. For the method used here, AEP less than 0.2% start to have numerical precision issues and AEP greater than 2% are unreliable because the reviewer used a hydraulic approximation. The results are considered reasonable for the indicated region because the hydraulic approximation was suitably reproduced in this region. At the 1% level the simulation of 10 samples produced a range of approximately 4 m for the pre-dam scenario.

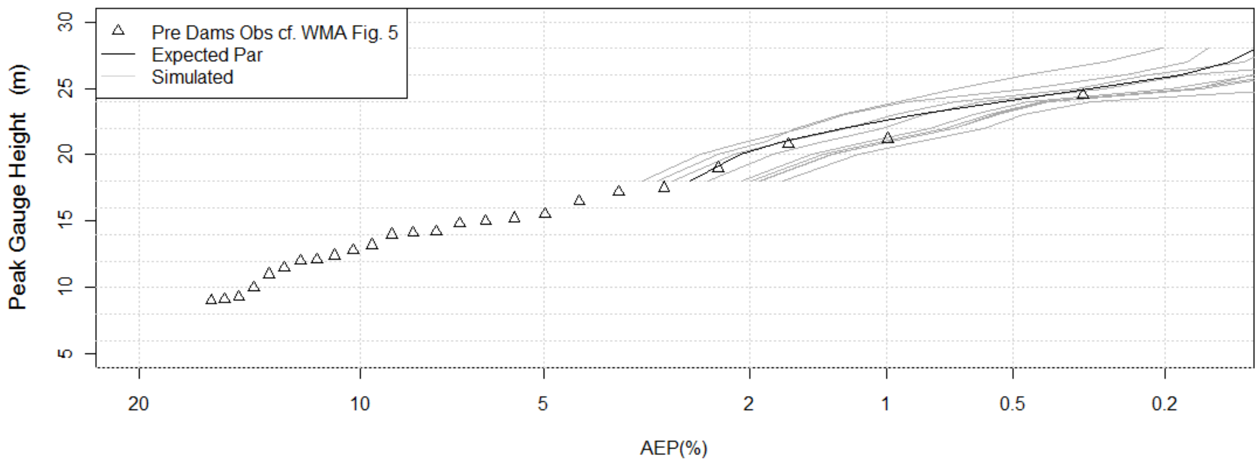

Figure 3.1 Modelled heights at the Ipswich gauge. The best estimate parameters similar to WMA water give the solid black line. 9 samples are presented (a very low amount for interpretation) which demonstrate variability in the method due to parameter uncertainty. The simulated lines have been clipped at a height of 18 m because approximations in the method below this level were unreliable (the estimated and simulated lines drop too quickly).

Other than verifying the general procedure by WMA, Figure 3.1 is of limited use because the "with dams" scenario is of greater interest. Unfortunately for this analysis, the nature of the dam conversion is to cause lower flows and this pushes more of the underlying probability distribution into the region where the hydraulic approximations were unreliable. Nonetheless, the method could be repeated with accurate knowledge of the hydraulic model and used to construct variability estimates[1]. The issue of the hydraulic approximation serves to show the centrality of the hydraulic model to the joint probability method (though, any alternate method would also require a strong reliance on this same model), so the model's quality is important.

[1] The without dams scenario should also be repeated because the flows since the dams were built require conversion to pre-dam estimates

The main aim of an uncertainty analysis should be to assess the statements made regarding the influence of the dams (WMA, Figure 2). Even though the Bremer River is not dammed, the backwater effects of the Brisbane River imply that the 'with dams' scenario will strongly influence the variability of the estimates in the Ipswich region. A method to do this would be to allow a multiplier on the error that scales with the magnitude of the pre-dam flow. The mean function could be either the 50% line or the estimation line suggested by WMA. Short of using a spatiotemporal Monte Carlo analysis to populate the scatter about the conversion line, it is not possible to know the true character of the conversion. So any analysis of this type would need to make stand-in assumptions, for example, that the variation is factored at 20% of the predam flow, that the errors follow a normal distribution and therefore that the scatter is not skewed or biased about the best-estimate of the mean line.

4. Conclusions

The aim of this review is to highlight the need for uncertainty in design estimates. WMA have adopted a joint probability approach to provide a best estimate of exceedance probabilities at Ipswich. This represents a significant advance on earlier methods, but it also rests on a large number of assumptions and can have high levels of uncertainty. Due to the imposed constraints WMA have been unable to provide formal uncertainty analysis of their estimate. The reviewer supports the WMA estimate but also notes that a variety of assumptions need to be tested in more detail and that uncertainty estimates need to be quantified. A method to achieve this which builds upon the joint probability framework has been demonstrated here, but results were only for a very limited case. The results presented here at least partially confirm the work by WMA, but significant additional work is required to demonstrate that the results are not sensitive to the joint dependence structure and to allow for the variable function of the dams (as already pointed out by WMA para. 85 and 87). The authors highlighted that their estimates will likely have a wide range of uncertainty and recommend a range based on the 2% AEP to the 0.5% AEP which is over 4.4m. Given the limited analysis presented here this estimate seems reasonable.

WMA have highlighted the two key problems in determining design estimates for the Ipswich region (i) understanding the hydraulic effects and (ii) understanding the joint hydrologic effects. There is a heavy reliance upon a hydraulic model in coming up with a design estimate in the confluence zone so that any improved understanding of the physical link will translate into better estimates. However, the reviewer’s main concern is that hydrologic uncertainties will overwhelm the design estimates.

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WMA (2011c) Supplementary Report – Ipswich Flood Frequency Analysis, Report 111024, October 2011, QLD Flood Commission of Inquiry

Appendix A – M. Leonard Curriculum Vitae

NAME: Michael Leonard

BORN: [redacted], Australia

CONTACT: School of Civil, Environmental & Mining Engineering
Engineering North N136, North Terrace Campus
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PRESENT POSITION: Research Associate, University of Adelaide

QUALIFICATIONS: B.E. (Civil), Hons. 1 (2002)
Ph.D (2010)

PHD THESIS: A Stochastic Space-Time Rainfall Modell for Engineering Risk Assessment

PUBLICATIONS

Books

1. Walker, D., **Leonard, M.**, Metcalfe A.V. and Lambert, M.F., (2008) Engineering Modelling and Analysis, Taylor and Francis, London
2. Lambert M.F., Daniell T.M. and **Leonard M.** *Proceedings of Water Down Under 2008 incorporating 31st Hydrology and Water Resources Symposium 4th International Conference on Water Resources and Environment Research*, Institution of Engineers, Australia, Adelaide, April 2008, 275 papers

Refereed Journal Publications

3. Thyer, M., **Leonard, M.**, Need, S., Kavetski, D., Renard, N. (2011) RFortran - an open source software library for linking to R from Fortran with applications in environmental modelling, *Environmental Modelling & Software*
4. Wong, G., Lambert, M.F., **Leonard, M.** and Metcalfe A.V. (2010) Drought Analysis using Trivariate Copulas, *Journal of Hydrologic Engineering*, 15(2), 129-141
5. **Leonard, M.**, Lambert, M.F., Metcalfe A.V. and Cowpertwait P.S.P. (2008) A space-time Neyman-Scott rainfall model with defined storm extent, *Water Resources Research*, Vol. 44, W09402, doi:10.1029/2007WR006110
6. **Leonard, M.**, Metcalfe, A.V., Lambert, M.F., (2008) Frequency analysis of rainfall and streamflow extremes accounting for seasonal and climatic partitions, *Journal of Hydrology*, 348 (1-2), pp. 135-147
7. **Leonard, M.**, Metcalfe, A.V., Lambert M.F., Kuczera G.A. (2007). Implementing a space-time rainfall model for the Sydney region. *Water Science Technology*, 55 (4), pp. 39-47.
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Refereed Conference Proceedings

11. **Leonard, M.,** Ball, J., Lambert, M. (2011) On the coincidence of extreme rainfall bursts with duration, *34th IAHR World Congress*, Brisbane, June 2011
12. **Leonard, M.**, Lambert, M., Metcalfe, A., Mohdisa, F. (2010) An analysis of shifts in rainfall across Southern Australia, *Water 2010*, Quebec City, July 2010
13. Qin, J., **Leonard, M.**, Kuczera, G., Thyer, M., Lambert, M. (2009) A High-Resolution Hierarchical Space-Time Framework for Single Storm Events and Its Application for Short-term Rainfall Forecasting, *8th IAHS Assembly, Hyderabad*, Sept. 2009
14. **Leonard, M.,** Osti, A., Lambert M., Metcalfe A. (2008) A Spatially Inhomogeneous Poisson Model of Rainfall Across Australia. *Water Down Under*, 2008, Institute of Engineers, Adelaide, South Australia, Apr. 2008.
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Consulting Reports

24. **Leonard M.,** Holmes, M. (2011) Temporal patterns of Australian Rainfall, for Engineers Australia, EngTest report C110303, (in preparation)
25. **Leonard M.,** Need, S. (2011) Spatial patterns of Australian Rainfall, for Engineers Australia, EngTest report C110304, (in preparation)

26. **Leonard, M.**, Thyer, M., Lambert, M., Maier, H., Dandy, G. (2011) Task 4 Milestone 1 Report, Application Test Bed, Onkaparinga Catchment Case Study: Surface Water Hydrological Modelling, for Goyder Institute (SA)
27. **Leonard, M.**, Lambert, M., Metcalfe A. (2009) Step change Analysis of Rainfall in Southern Australia, for Department of Land Water and Biodiversity Conservation (SA), EngTest report C090703
28. **Leonard, M.**, Lambert, M. (2003) Seasonal IFD Analysis of Adelaide Rainfall, for R Clark & Associates
29. **Leonard, M.**, Lambert, M. (2003) Seasonal Analysis of Simulated Adelaide Rainfall, R Clark & Associates

Discussion papers

30. Leonard M. (2009) Patterns of rainfall in space and time for use in flood risk estimation, discussion paper, ARR Technical Committee Workshop, 9th July, Melbourne pp. 1-30

TEACHING

The following is a list of subjects I have lectured

- Engineering Modelling & Analysis I (2007-2009)
- Water Engineering IIIB (2004, 2008)
- Advanced Water Resources Management IV (2003)
- Introduction to Geostatistics (Masters of Geostatistics) (2010, 2011)
- Computing for Geostatistics (Masters of Geostatistics) (2010, 2011)

REVIEWER FOR JOURNALS (in past 2 years)

Journal of Applied Mathematics and Decision Sciences (Hindawi) , *Journal of Hydraulic Engineering* (ASCE), *Hydrology and Earth System Sciences* (EGU), *Atmospheric Research* (Elseveir), *Water Resources Research* (AGU), *Journal of Hydrology* (Elseveir), *Environmental Modelling & Software* (Elseveir), *Hydrological Sciences* (Taylor & Franics), *Advances in Water Resources* (Elsevier), *Australian Journal of Water Resources* (IEAust)

REVIEWER FOR CONFERENCES

MODSIM 2005 International Congress on Modelling and Simulation. Modelling and Simulation Society of Australia and New Zealand, December 2005

Hydrology and Water Resources Symposium, 2006 – 2011, Institute of Engineers Australia

AWARDS

Best presentation by a student or recent graduate, Engineering Mathematics & Applications, 2009

Best presentation by a student or recent graduate, Water Down Under 2008

Postgraduate Research Award, AWA, 2006

CAMWE prize for best honours Civil & Environmental project thesis, Optimisation of water distribution systems including the effects of uncertainty (Industry partner: United Water International, 2002) (joint authorship - Zecchin, A., Berrisford, M., Leonard, M., Roberts. A.).

PROFESSIONAL ACTIVITIES

2008: Technical editor for Water Down Under 2008 Hydrology & Water Resources Symposium peer review process, Institute of Engineers, Australia

2007-2008: Delivered seminars to female secondary school students as part of 'Women in Technology' conference

2005-2008: Secretary for Engineering and Computer Science Faculty, Computer Aided Teaching Suite Steering Committee

2004-2009: Assistance in preparation of ARC research proposals

2003-ongoing: Member of the Hydrological Society of South Australia

2003: SAPAC Workshop High Performance Computing / Parallel Programming

Appendix B - Joint Analysis Method Detail

The aim of this section is twofold:

(i) Review the implementation and assumptions of the joint probability analysis by WMA
(ii) Formulate and document a methodology suitable for uncertainty analysis

The material in this Appendix is intended for technical readers, whereas the main document summarises the outcomes and discusses assumptions. Appendix C summarises data used in this study. Unfortunately the outcome of the study is limited by the use of an approximation surface to the hydraulic response (i.e. Figure B6, WMA, 2011).

The overall goal of the joint probability analysis is to construct a probability distribution of only those flows ($Q_{Amb,}$ Q_{Sav}) that will yield the maximum water height at Ipswich (H_{Ips}).

Figure B.1 summarises the joint distribution of **all** pairwise flows at Savages Crossing and Amberley from which the subset of pairs are required that yield annual maximums H_{Ips}. The Q_{sav}=100 threshold identifies occasions when Savages Crossing has high flow and the Q_{Amb}=100 threshold shows occasions when Amberley has high flows. The top right most corner is when both flows are high, but it is possible that annual maximums are contributed by two additional scenarios (i) Q_{sav} is high and Q_{Amb} takes on any coinciding value (right hand region) and (i) Q_{Amb} is high and Q_{Sav} takes on any coinciding value (top most region). The WMA analysis pursues the first case assuming that the Brisbane River is the dominant flood generating mechanism. In other words, the assumption is that the top-left corner of the plot does not contribute annual maximum water heights in Ipswich (or has sufficiently low occurrence rate to be ignored).

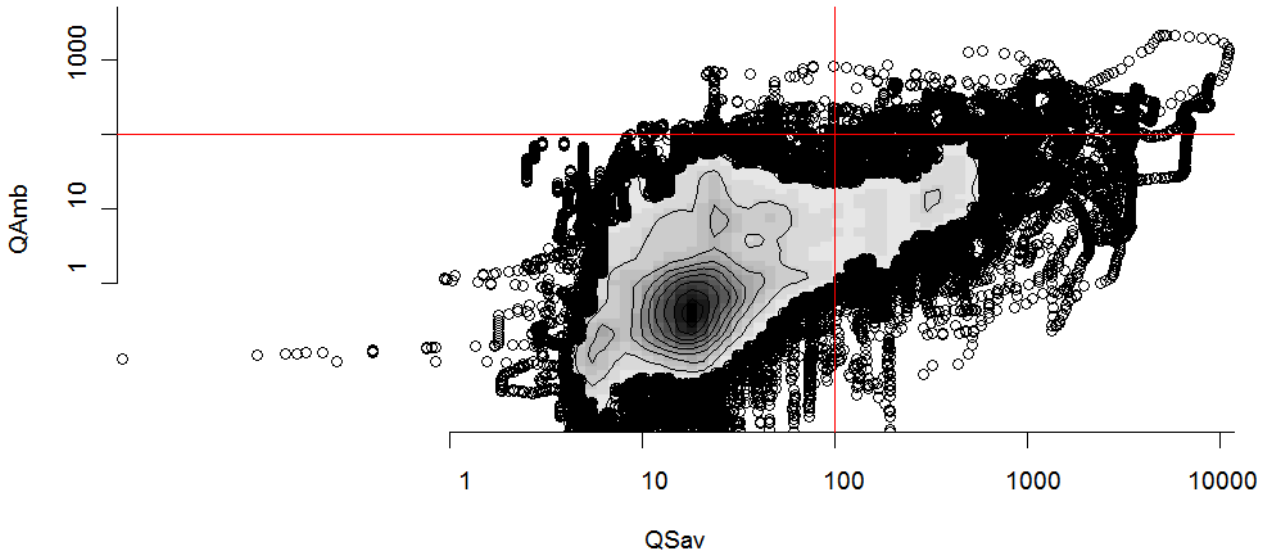

Figure B.01 Comparison of scatter produced by all pairs of Q_{Sav} and Q_{Amb} flows (not just selected independent or extreme values). Contours show density of inner data points whereas outer values are represented by points.

A basic check was conducted to see whether there were any years when the annual maximum flows at Amberley caused higher values of H_{Ips}, than choosing annual maximum flows at Savages Crossing. If this proves to be true, then the WMA analysis is biased by having ignored these cases. Table B.1 summarises the findings. It shows that there are 5 years out of 47 where using Q_{Amb} to identify annual maximums results in higher water levels than if Q_{Sav} had been used. Of these 3 of the cases appear to be a matter of timing, that with more attention the correct flow pair for that event (the one producing the maximum height) would be selected. Only 2 of the cases were from independent events. The ratio 2/47 is about 4% which suggests that the assumption used by WMA is reasonable. To otherwise accommodate this 4% would require double the number of parameters of that used by WMA and would cause significant uncertainty in the methodology. The reviewer also expects that the 4% of cases are not as significant for the upper tail of the distribution of water levels in Ipswich than those contained within the 96%.

Table B.1 A Comparison of flow pairs based. Case 1: based upon annual maximums at Savages Crossing and with coincident Amberley flows obtained via 12 hour timing rule. Case 2: : based upon annual maximums at Amberley and with coincident Savages Crossing flows obtained via 12 hour timing rule. Height at Ipswich obtained from gauge 40101. The classification "Timing" indicates flows are from the same event and "Different" implies two different events.

Case 1: Annual Max based on Q_{Sav}				Case 2: Annual Max based on Q_{Amb}				
Date	**Q_{Sav}**	**Q_{Amb}±12hr**	**H_{Ips}***	**Date**	**Q_{Amb}**	**Q_{Sav}±12hr**	**H_{Ips}***	**Classified**
20/02/1971	3123.2	130.9	7.7	4/02/1971	880.85	1473.87	11.71	Different
28/01/1974	11136.9	1359.5	20.7	27/01/1974	2107.53	9276.46	20.7#	Timing
22/01/1976	1844.3	95.4	5.3	11/02/1976	1288.92	973.41	13.65	Different
6/04/1988	1897.9	184.2	3.7	4/04/1988	542.13	1506.29	11.2	Timing
9/02/1991	374.5	372.1	7.2	8/02/1991	678.76	368.90	7.2#	Timing

* Nearest value read from gauge 40101. Does not necessarily correspond to exact flow from hydraulic model
# Where H_{Ips} appears the same, hydraulic model will show height difference (based on inspection of WMA Figure B6)

Proceeding with the joint analysis, it is reasonable to consider the case of annual maximum flows Q_{Sav} and the coincident flows Q_{Amb}. A fit of the annual maximum flows is shown in Figure B.2 using the lognormal distribution. The reviewer considers the skewness to be sufficiently negligible that a 2-parameter distribution is suitable. The alternatives presented by WMA using the GEV and LP3 offer similar quality of fit. The reviewer's preference is for the lognormal because of convenience in its implementation.

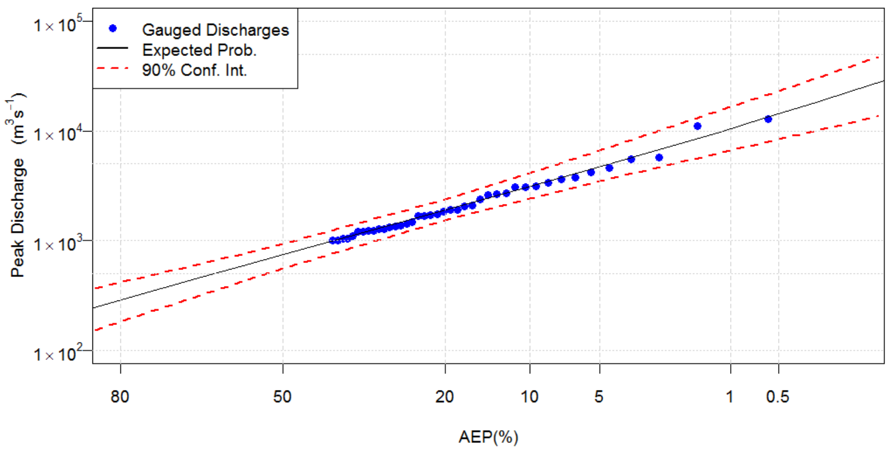

Figure B.02 Lognormal distribution fitted to Q_{Sav} using Flike

The next step is to model the Q_{Amb} flows that come from the same event corresponding to the Q_{Sav} annual maximum. In doing so a joint (bivariate) probability distribution is specified which can handle the case of correlated data. There are many alternate ways to model the joint distribution and the assumptions can be critical, especially when interest is in the region of the upper tail. WMA have opted to use a peak-over-threshold method (POT) which allows for more pairs of points to be collected than taking just one pair each year. Figure B.3 shows a comparison of the POT analysis[2] by the reviewer criteria (crosses) which obtained many additional pairs as compared to the points identified by WMA (circles). While there are many coinciding pairs, the reason for this difference is not clear and is likely to come down to a stricter independence criteria by WMA. Either way, this difference is unlikely to affect the WMA result as both datasets share the same overall scatter. The main challenge is to estimate the association between the pairs for increasing flows (i.e. if the Q_{Sav} flow is higher does this imply that the Q_{Amb} flows are similarly high, and how strong is the relationship?). Using a POT analysis bears the assumption that the association would be the same if only the Q_{Sav} annual maximums were used (triangles in Figure B.3). Although there are less of these points and they can occur at lower flows, it appears they have a similar association.

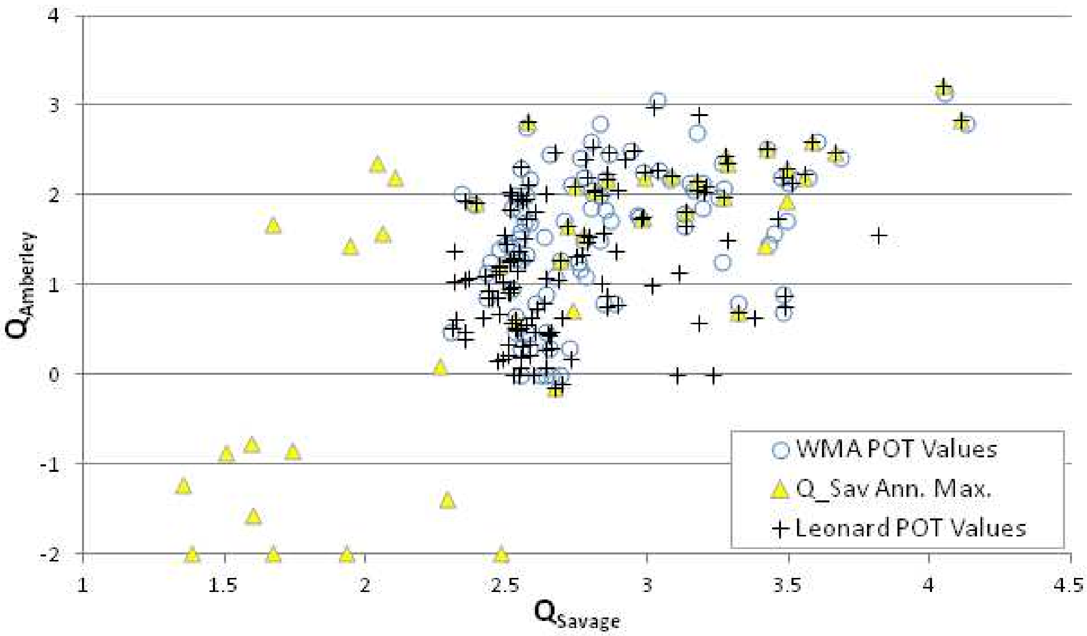

Figure B.3 Comparison of scatter produced by Q_{Sav} and Q_{Amb} flows. WMA POT values (Figure B2 Ipswich Report) are compared directly to Leonard POT values. Q_{Sav} annual maximums and coincident Q_{Amb} flows demonstrate similar correlation as POT values.

WMA have modelled the probability density of Q_{Amb} flows conditioned on a given flow of Q_{Sav}, denoted $f(Q_{Amb}|Q_{Sav})$. If the proability density of the Q_{Sav} annual maximums is specified $f(Q_{Sav})$ (i.e. the distribution in Figure B.2), then the joint distribution is obtained by the product of these two distributions.

$$f(Q_{Sav},Q_{Amb}) = f(Q_{Amb}|Q_{Sav})\, f(Q_{Sav}) \qquad \text{(B1)}$$

The distribution of Q_{Amb} flows (those that coincide on the day of annual maximum Q_{Sav}) is then obtained by integrating the joint distribution

$$f(Q_{Amb}) = \int f(Q_{Sav}, Q_{Amb})\mathrm{d}Q_{Sav} \qquad \text{(B2)}$$

The purported benefit of this procedure is that it uses the longer Q_{Sav} record and exploits additional information from the POT analysis, but one could have alternatively fitted the relevant pairs directly (the triangles in Figure B.3). It serves as a useful check to compute B2 to see how well the Amberley flows are

[2] 5-day independence criteria was used. Identified values shown in Appendix C. Also note, the threshold is not actually 100 m^3s^{-1} since it was applied to flows before the dam conversion and is closer to 300 m^3s^{-1} pre-Wivenhoe and 200 m^3s^{-1} post-Wivenhoe. The difference will have trivial impact on results.

modelled, even though the Amberley record is short. This will be done after giving further discussion to the joint relationship.

WMA have opted for a regression of the conditional mean and standard deviation of Q_{Amb} flows for given values of Q_{Sav}. This is a reasonably flexible approach but not the only one. Two more direct alternatives that use the Q_{Sav} annual maximums come to mind

(i) fit a joint log-normal distribution to the annual maximum pairs
(ii) fit the marginals using the best identified distribution (not necessarily lognormal) and then test a variety of copula functions to handle the dependence structure

Both of these approaches can be fitted using maximum likelihood methods which is convenient for an uncertainty analysis. A third option is to fit the POT distributions and then construct the annual maximums from the exceedances, but this would be much more complicated. Given a longer time frame the reviewer would be more confident with the results if a variety of cases were implemented to test the correlation structure between the two variables as the overall result is likely to be sensitive to this assumption.

As a verification of the WMA procedure, the regression approach has been re-implemented here. However a difference has been made to allow the standard deviation parameter to be formally regressed, rather than using a more ad-hoc estimation technique. The chief advantage is that this approach allows the uncertainty in the parameters to be assessed. The regression model is specified as

$$Q_{Amb} = m(Q_{Sav}) + s(Q_{Sav}) * \text{error} \tag{B3}$$

where $m(Q_{Sav})$ is the function of the mean which changes for a given value of Q_{Sav}

$s(Q_{Sav})$ is the function of the standard deviation which changes for a given value of Q_{Sav}

error explains the residual variance which is assumed to be normally distributed ~ N(0,1)

Linear regressions are assumed for the mean and standard and the model has 4 parameters

$$\frac{Q_{Amb} - (\mathrm{m}_0 + \mathrm{m}_1 Q_{Sav})}{\mathrm{s}_0 + \mathrm{s}_1 Q_{Sav}} = \text{error} \tag{B4}$$

where m_0 and m_1 are respectively the intercept and slope of the conditional mean

s_0 and s_1 are respectively the intercept and slope of the conditional standard deviation

This model was fitted using maximum likelihood techniques and results in the following parameters

Table B.2 Fitted regression parameters including covariance matrix of parameter variability

Param	Expected Value	Covariance	m_0	m_1	s_0	s_1
m_0	-0.94453		0.2246	0.0738	0.0200	0.0068
m_1	0.882515		0.0738	0.0248	0.0068	0.0023
s_0	1.249539		0.0200	0.00686	0.13928	0.0461
s_1	0.19652		0.0068	0.00235	0.04617	0.0155

The expected values of the parameters agree closely to those obtained by WMA. The residuals are comparable (Figure B.4) to those from WMA but the standard deviation obtained here is higher at high values of Q_{Sav} (Figure B.5). The WMA estimate has differing amounts of data in each sub-range they used to estimate the standard deviation. In the highest sub-range there were few data points and the WMA estimate of the slope in that region is less reliable. Figure B.6 shows the resulting conditional distribution, which is similar to WMA (cf. WMA Figure B2), but has noticeably higher variability in the region Q_{Sav}>5000.

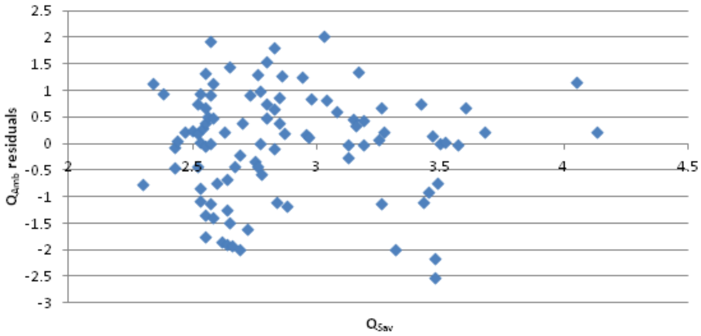

Figure B.4 Residuals of Q_{Amb} flows regressed against Q_{Sav} flows.

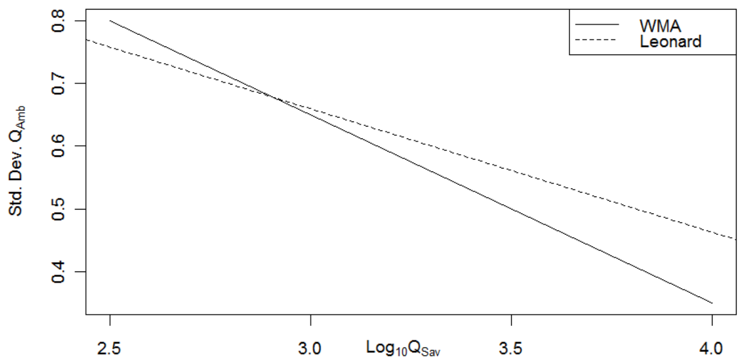

Figure B.05 Difference in regression of standard deviation of Q_{Amb} conditioned on Q_{Sav}

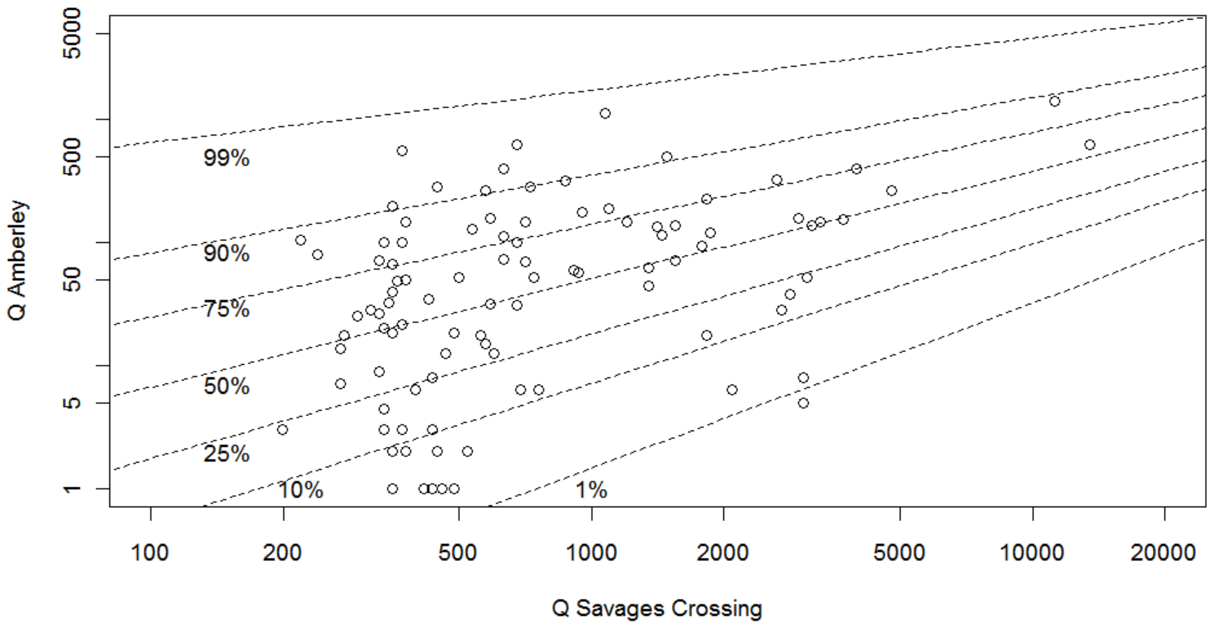

Figure B.6 Conditional distribution of Q_{Amb} given Q_{Sav} (Cf. WMA Fig B5)

The parameters in Table B.2 can be used to specify a multivariate normal distribution. Sampling this distribution allows random sets of parameters to be obtained which are then used in the uncertainty analysis.[3] A plot of the parameters in Figure B.7 shows the strong relationship between them. Fitting this regression line for a random sample of 1000 parameter sets shows any of the potential regression lines that could be followed (Figure B.8). For each of these regression lines Equation (B2) can be numerically integrated to obtain the distribution of Q_{Amb} flows. This check, as mentioned earlier is shown in Figure B.9

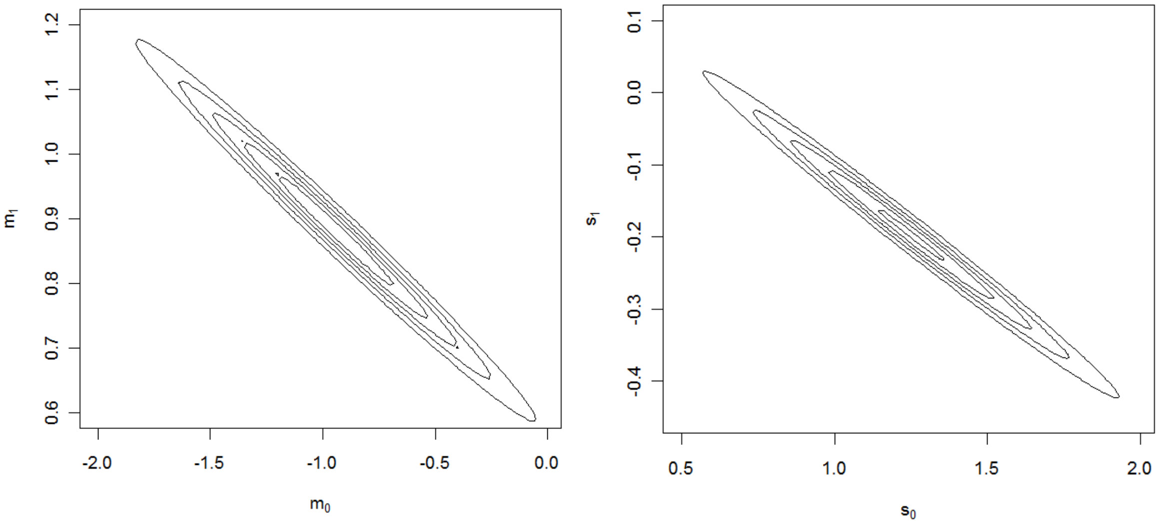

Figure B.7 Conditional distribution of Q_{Amb} given Q_{Sav} (Cf. WMA Fig B5)

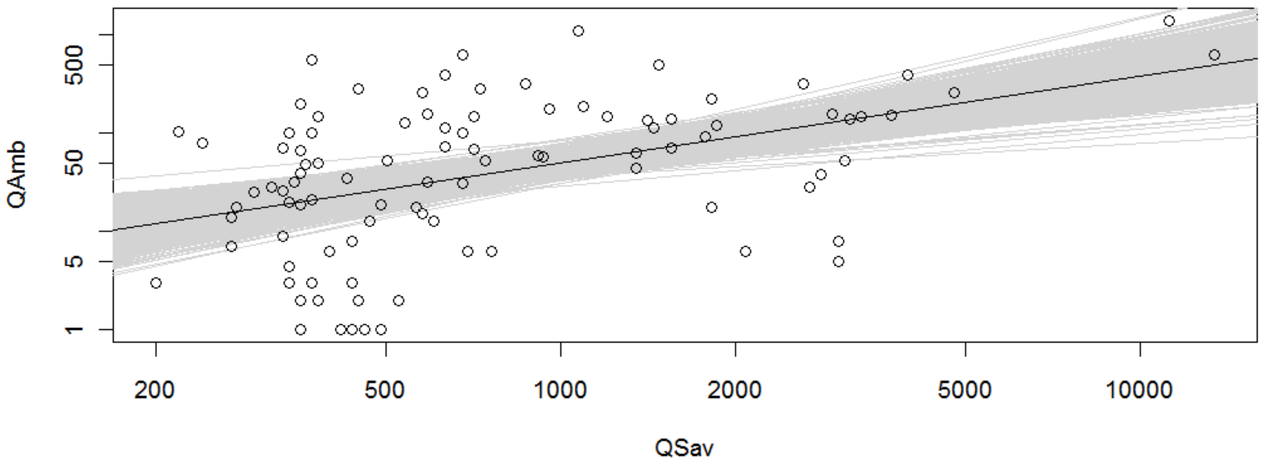

Figure B.08 Conditional distribution of Q_{Amb} given Q_{Sav} (cf. WMA Fig B5) Simulated values in grey. Expected value line in black.

Figure B.9 shows that the model adopted by WMA slightly underestimates the variability in the lower tail than if a more direct approach was adopted (leading to the lognormal or copula method mentioned earlier). This will have some impact on the final results, but it is likely to cause a slight underestimation in the variability at lower water levels of H_{Ips}. WMA Figure 5 currently have an overestimation in the lower tail

[3] Kuczera (1999) and FLIKE help files specify how covariance matrix is obtained from likelihood function. FLIKE uses the multivariate normal approximation to sample the true posterior distribution via importance sampling. Here, the multivariate normal is directly sampled (i.e. importance sampling not conducted). This is a reasonable first order approximation.

of H_{Ips}, which is likely due to compensating factors in the hydraulic model having masked out the effect being noted here (or that the observed values in this region are suspect).

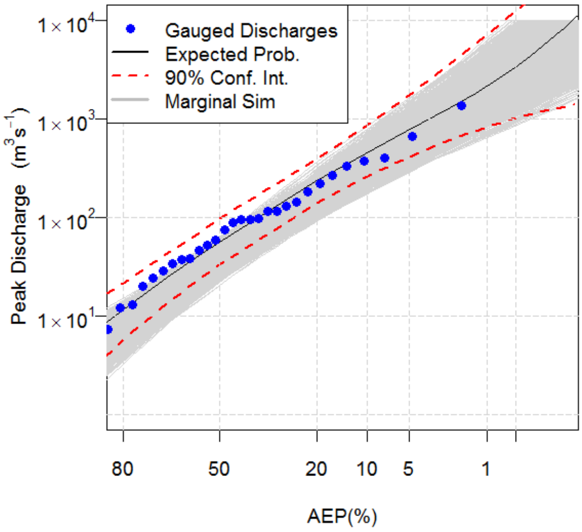

Figure B.09 Marginal distribution of Q_{Amb}. Simulated values in grey. 90% confidence limits obtained by directly fitting the Q_{Amb} distribution using FLIKE software (Kuczera, 1999)

Now that a probability model of the flows is constructed, it can be used with a transformation function to determine the probability of heights H_{Ips}. The reviewer did not have access to the hydraulic model results and manually constructed an approximation based on WMA Figure B6 (shown in Appendix C). Figure B.10 shows the comparison of the contours used here which are notably less detailed in the region of lower flows than WMA Figure B6. This has led to a significant restriction on the analysis presented in this review which was not otherwise expected when the approximation was constructed. Applying this methodology with actual contours developed from the hydraulic model should remedy the issue.

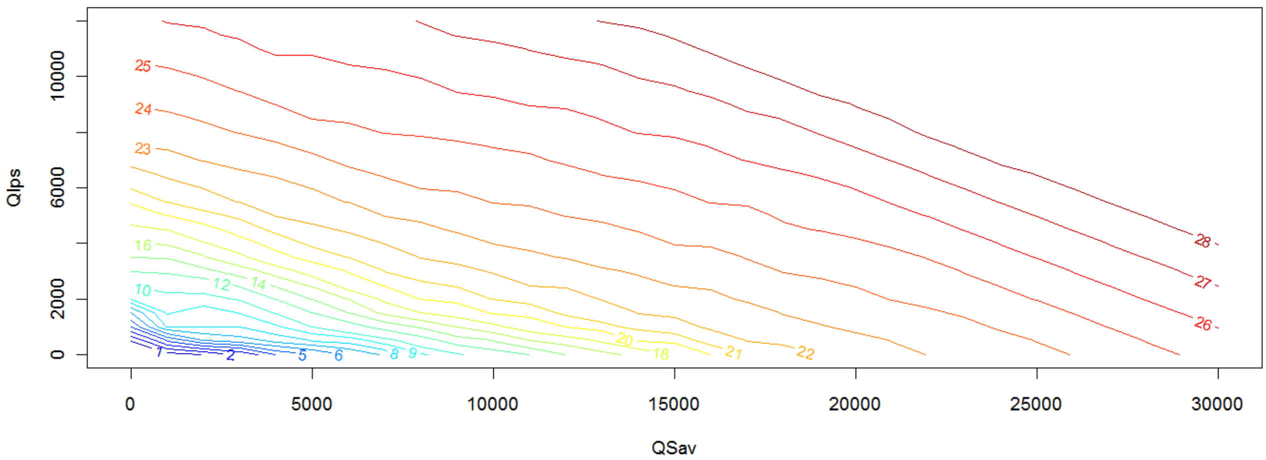

Figure B.010 Interpolated approximation of Figure B6 from WMA Ipswich Report. Converts flow inputs Q_{Ips} and Q_{Sav} to height of the Bremer River. The function is a summary and approximation of the hydraulic model.

Figure B.11 is identical to Figure B.10 except is uses the 0.6 conversion factor of flows between Q_{Ips} and Q_{Amb} (i.e. the y-axis labels have changed). Section 2.1 (para. 9) cites Warrill Creek as two thirds of the Bremer's total catchment area yet a conversion of 0.6 is used. More detail explaining this factor would be appreciated, e.g. is it a ratio of catchment rainfalls or is it based on the partial area to the location of the Q_{Amb} gauge?

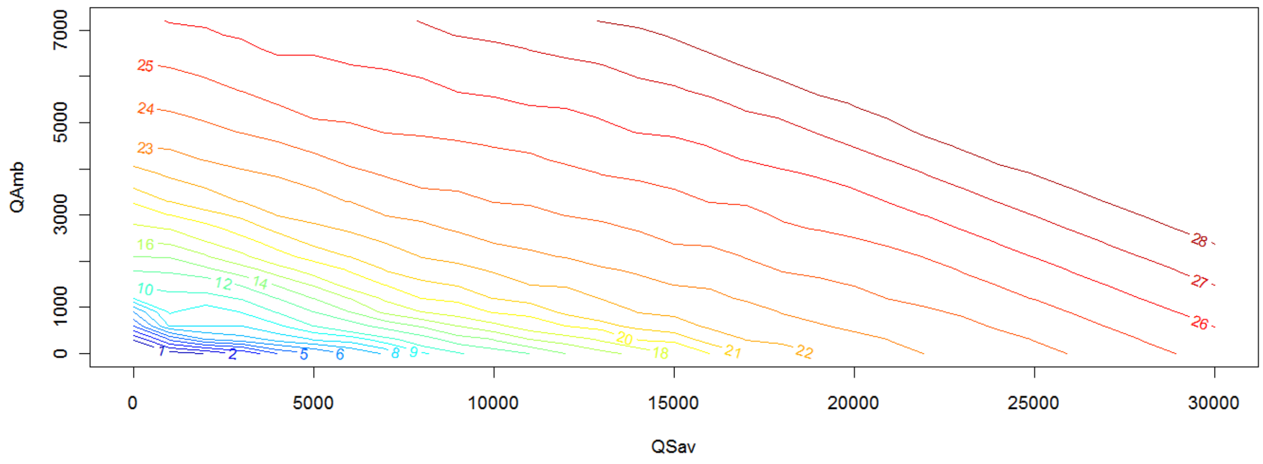

Figure B.11 Interpolated approximation of Figure B6 from WMA Ipswich Report. Converts flow inputs Q_{Amb} and Q_{Sav} to height of the Bremer River. The function is a summary and approximation of the hydraulic model.

The heights H_{Ips} are obtained by considering every possible pair of flows and weighting them by the probability that that pair of flows occurs and by the probability they exceed the specified height of interest. The notation presented here is different to the WMA Ipswich report but the procedure is fundamentally the same. A point of note is that the summation technique used here is computationally demanding and requires a small increments to achieve the desired precision whereas WMA required only 66 ordinates (WMA para. B14). The total probability theorem can be used to estimate the probability a height is exceeded,

$$Prob(H > h) = \iint Prob(H(Q_{Sav}, Q_{Amb}) > h) f(Q_{Sav}, Q_{Amb}) dQ_{Sav}\, dQ_{Amb} \qquad \text{(B5)}$$

where

- *h* is a threshold of interest

- *Prob*(*H>h*) is the probability the river exceeds a certain specified height. As an example *Prob*(*H>20 m*) is the probability that the height exceeds 20 m. If this is evaluated to, for example, 2%, it means that 20m is a 2% AEP.

- *H*(Q_{Sav},Q_{Amb}) is the function converting input flows to heights (Figure B.11)

- *Prob(H*(Q_{Sav},Q_{Amb})>h) is the chance a specific height exceeds the threshold, either 0 if it doesn't or 1 if it does.

- *f*(Q_{Sav},Q_{Amb}) is the joint density function of the input flows. (Figure B.12)

- $\iint dQ_{Sav}\, dQ_{Amb}$ means the values are to be summed over the entire range of flows

The procedure to implement this is quite straightforward to understand graphically. Figure B.12 shows the joint probability density function with height contours overloaded. An exceedance probability is by definition the probability of exceeding a certain height, so the procedure is simply to identify a height contour of interest and then summate the probabilities of the underlying distribution above the contour. The resulting estimate can be spurious if either the contours are incorrect (the hydraulic model) or the probability function is not representative. The issue of association between high flows is critical because, with reference to Figure B.12, more of the top-right portion of the shaded grey region can easily be pushed above contours of interest for a change in correlation parameter.

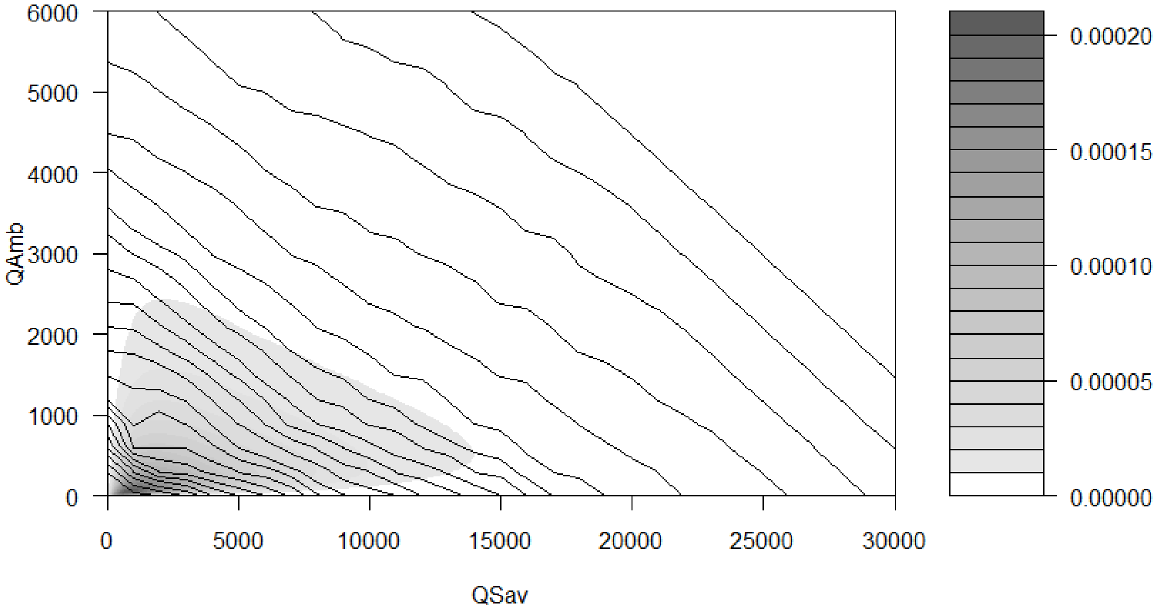

Figure B.12 Joint probability density function $f(Q_{Amb}, Q_{Sav})$ shown in shaded levels. H_{Ips} height contours are overlaid. The procedure for getting an exceedance probability is to locate a H_{Ips} contour of interest and then summate all the values of the probability function that lie above this line.

An uncertainty analysis proceeds by sampling different sets of parameters controlling the joint probability distribution. A given sample represents one possible characterisation for which the integration in Equation (B5) is performed at all water level heights of interest. This procedure is repeated for many replicates from which confidence limits can be constructed. The parameters of the Amberley flows have already been provided in Table B.2, but it is also necessary to allow the Savages Crosssing parameters to vary. These are provided in Table B.3. The procedure used here assumes that the Q_{Sav} marginal distribution parameters (Table B.3) are independent of the parameters used to specify the conditional distribution (Table B.2).

Table B.3 Savages Crossing marginal distribution fitted lognormal parameters

Scenario	Param	Expected Value	Covariance	Log_e mean	Log_e sdev
Without dams	**Log_e mean**	6.63131		0.0228	0.0109
	Log_e sdev	0.07775		0.0109	0.0158
With dams	**Log_e mean**	7.50498		0.0121	0.0000
	Log_e sdev	-0.54096		0.0000	0.0121

The parameters of the "With dams" scenario are presented in Table B.3 although final results are not able to be presented from this scenario. The reason is that the approximation used in Figure B.11 is unreliable at lower flows and the effect of the different parameters for the dams scenario is to lower the Q_{Sav} flows into this region so that results at the 1% AEP cannot be relied upon. The final result is shown in the body of the report in Figure 3.1.

Appendix C – Data Summary

This review was done in a short time frame so please do not rely on figures in this Appendix without checking them. They are presented here for reproducibility of results. The reviewer has performed only limited checks.

Table C.1 Savages pre-dam to post-dam flow conversion used by WMA Ipswich Report, 1943-1985 (Somerset only). Units m^3s^{-1}.

Pre	Post	Ratio
0	0	-
288	100	0.347
5692	5363	0.942
9807	7393	0.754
13922	11508	0.827
18037	15623	0.866
22152	19738	0.891
26267	23853	0.908
30382	27968	0.921

Table C.2 Savages pre-dam to post-dam flow conversion used by WMA Ipswich Report (Figure 2) for period 1986-present (Wivenhoe & Somerset). Units m^3s^{-1}.

Pre	Post	Ratio	Pre	Post	Ratio	Pre	Post	Ratio	Pre	Post	Ratio
0	0	1.00	10700	6917	0.67	13100	9400	0.71	15500	11882	0.77
1	1	1.00	10800	7020	0.67	13200	9503	0.71	15600	11986	0.77
2000	1000	0.50	10900	7124	0.67	13300	9606	0.71	15700	12089	0.77
8600	3657	0.42	11000	7227	0.67	13400	9710	0.71	15800	12192	0.77
8700	4486	0.53	11100	7331	0.67	13500	9813	0.71	15900	12296	0.77
8800	4952	0.56	11200	7434	0.67	13600	9917	0.71	16000	12399	0.77
8900	5055	0.56	11300	7538	0.67	13700	10020	0.71	16100	12503	0.77
9000	5159	0.59	11400	7641	0.67	13800	10124	0.71	16200	12606	0.77
9100	5262	0.59	11500	7745	0.67	13900	10227	0.71	16300	12710	0.77
9200	5365	0.59	11600	7848	0.67	14000	10331	0.71	16400	12813	0.77
9300	5469	0.59	11700	7951	0.67	14100	10434	0.71	16500	12916	0.77
9400	5572	0.59	11800	8055	0.67	14200	10537	0.77	16600	13020	0.77
9500	5676	0.59	11900	8158	0.67	14300	10641	0.77	16700	13123	0.77
9600	5779	0.59	12000	8262	0.67	14400	10744	0.77	16800	13227	0.77
9700	5883	0.63	12100	8365	0.71	14500	10848	0.77	16900	13330	0.77
9800	5986	0.63	12200	8469	0.71	14600	10951	0.77	17000	13434	0.77
9900	6090	0.63	12300	8572	0.71	14700	11055	0.77	24000	18400	0.77
10000	6193	0.63	12400	8675	0.71	14800	11158	0.77	31000	23366	0.77
10100	6296	0.63	12500	8779	0.71	14900	11261	0.77	38000	28332	0.77
10200	6400	0.63	12600	8882	0.71	15000	11365	0.77			
10300	6503	0.63	12700	8986	0.71	15100	11468	0.77			
10400	6607	0.63	12800	9089	0.71	15200	11572	0.77			
10500	6710	0.63	12900	9193	0.71	15300	11675	0.77			
10600	6814	0.63	13000	9296	0.71	15400	11779	0.77			

Table C.3 Annual maximums obtained from Amberley site. Water years begin in July, e.g. year=1961 implies 01/07/1961 to 31/06/1962. Units are m^3s^{-1}.

Date	Q_{Amb}	Date	Q_{Amb}	Date	Q_{Amb}	Date	Q_{Amb}	Date	Q_{Amb}
21/11/1961	271.948	29/10/1972	402.947	2/12/1983	187.811	18/02/1995	23.061	17/02/2006	30.662
8/05/1963	382.574	27/01/1974	2107.531	28/07/1984	156.37	3/05/1996	448.543	22/07/2006	0.461
23/04/1964	216.079	26/02/1975	286.573	13/02/1986	23.741	15/02/1997	29.956	6/02/2008	232.16
3/07/1964	64.113	11/02/1976	1288.919	29/01/1987	23.621	25/12/1997	35.513	21/05/2009	304.004
8/12/1965	133.568	11/03/1977	223.694	4/04/1988	542.134	9/02/1999	194.594	16/02/2010	127.34
12/06/1967	329.847	3/04/1978	127.122	6/07/1988	315.936	28/12/1999	81.38	12/01/2011	705.826
13/01/1968	402.947	21/06/1979	97.525	5/04/1990	345.566	3/02/2001	95.649		
15/05/1969	81.026	9/05/1980	158.4	8/02/1991	678.76	2/02/2002	2.084		
20/11/1969	36.814	7/02/1981	284.814	12/12/1991	807.292	4/06/2003	5.951		
4/02/1971	880.853	4/11/1981	236.462	19/07/1992	65.356	4/02/2004	146.157		
3/04/1972	101.996	22/06/1983	437.418	11/03/1994	79.147	11/12/2004	29.131		

Table C.4 Annual maximums obtained from Savages site for the period 1961-2011. Peak flows obtained from Amberley site where timing of ± 12 hours is allowed (note: the Amberly flows are not necessarily the annual maximum). Water years begin in July. Units are m^3s^{-1}. Savages flows converted to pre-dam.

Date	Q_{Sav}	Q_{Amb}	Date	Q_{Sav}	Q_{Amb}	Date	Q_{Sav}	Q_{Amb}
10/01/1962 06:00	595.14	28.74	22/01/1982 08:00	1217.81	145.005	18/12/2001 12:00	39.18	0.157
19/03/1963 06:00	1364.37	2.881	24/06/1983 00:00	1896.98	116.477	23/01/2003 12:00	46.56	0
01/04/1964 12:00	464.54	0.715	02/07/1983 13:00	545.20	5.122	06/03/2004 04:00	514.62	0.188
04/07/1964 12:00	113.51	19.976	14/11/1984 15:00	296.60	7.245	07/11/2004 18:00	85.59	0
21/07/1965 10:00	1689.64	0	09/07/1985 04:00	181.92	1.03	14/10/2005 12:00	24.00	0
12/06/1967 05:00	2644.09	329.847	01/10/1986 00:00	32.00	0.094	12/01/2007 10:00	301.15	0
13/01/1968 23:00	3792.18	402.947	06/04/1988 10:00	1897.95	184.179	05/02/2008 13:00	108.66	220.304
03/01/1969 04:00	339.06	2.829	05/04/1989 07:00	3103.45	33.798	21/11/2008 07:00	715.29	96.434
29/08/1969 07:00	348.47	0	29/05/1990 10:00	1482.29	87.69	03/03/2010 11:00	244.05	75.222
20/02/1971 03:00	3123.18	130.93	09/02/1991 10:00	374.52	372.104	12/01/2011 01:00	12926.21	667.386
14/02/1972 15:00	2092.72	3.04	17/03/1992 15:00	2587.87	11.937			
19/02/1973 15:00	957.56	38.321	20/08/1992 00:00	54.94	0.143			
28/01/1974 01:00	11136.89	1359.496	01/03/1994 07:00	46.78	0.6			
15/01/1975 05:00	393.65	0	01/02/1995 00:00	39.81	0.019			
22/01/1976 09:00	1844.33	95.398	05/05/1996 21:00	4590.43	264.558			
02/11/1976 06:00	641.78	37.178	17/02/1997 12:00	87.20	24.282			
03/04/1978 11:00	545.81	115.561	18/09/1997 12:00	22.55	0.043			
24/01/1979 12:00	491.63	13.018	10/02/1999 11:00	3596.54	94.494			
09/05/1980 12:00	126.41	52.226	05/08/1999 06:00	194.53	0.011			
09/02/1981 13:00	971.03	58.583	04/02/2001 16:00	951.34	46.376			

Table C.5 Annual maximums obtained from pre-dam Savages Composite site. Water years begin in July. Units are m^3s^{-1}.

Date	Q_{Sav}	Date	Q_{Sav}	Date	Q_{Sav}	Date	Q_{Sav}
28/12/1909 12:00	813.52	03/03/1940 15:00	697.33	29/08/1969 07:00	348.47	05/08/1999 06:00	194.5328
13/01/1911 13:00	1316.89	25/01/1941 23:00	425.27	20/02/1971 03:00	3123.18	04/02/2001 16:00	951.3354
04/03/1912 23:00	460.75	10/02/1942 18:00	1360.14	14/02/1972 15:00	2092.72	18/12/2001 12:00	39.1831
23/06/1913 08:00	416.42	31/12/1942 07:00	833.44	19/02/1973 15:00	957.56	23/01/2003 12:00	46.56479
27/02/1914 12:00	234.46	31/12/1943 16:00	1425.54	28/01/1974 01:00	11136.89	06/03/2004 04:00	514.6169
11/02/1915 06:00	1035.35	13/06/1945 08:00	328.65	15/01/1975 05:00	393.65	07/11/2004 18:00	85.59432
12/04/1916 04:00	159.20	26/03/1946 19:00	1265.58	22/01/1976 09:00	1844.33	14/10/2005 12:00	24.00151
29/01/1917 18:00	475.23	02/03/1947 23:00	1010.19	02/11/1976 06:00	641.78	12/01/2007 10:00	301.1502
13/12/1917 16:00	522.28	11/12/1947 09:00	1002.23	03/04/1978 11:00	545.81	05/02/2008 13:00	108.6639
08/05/1919 16:00	90.61	04/03/1949 16:00	1225.24	24/01/1979 12:00	491.63	21/11/2008 07:00	715.2873
22/01/1920 08:00	402.41	01/03/1950 07:00	3048.37	09/05/1980 12:00	126.41	03/03/2010 11:00	244.0536
11/06/1921 15:00	1237.26	01/02/1951 12:00	3394.09	09/02/1981 13:00	971.03	12/01/2011 01:00	12926.21
30/12/1921 23:00	1280.09	18/06/1952 12:00	50.90	22/01/1982 08:00	1217.81		
16/10/1922 01:00	46.34	24/03/1953 12:00	1214.10	24/06/1983 00:00	1896.98		
12/02/1924 14:00	173.20	13/02/1954 12:00	971.40	02/07/1983 13:00	545.20		
21/06/1925 09:00	778.43	30/03/1955 04:00	5692.98	14/11/1984 15:00	296.60		
06/01/1926 12:00	126.45	12/03/1956 20:00	2384.10	09/07/1985 04:00	181.92		
27/01/1927 15:00	2715.26	23/12/1956 12:00	405.08	01/10/1986 00:00	32.00		
20/04/1928 12:00	4225.43	11/06/1958 20:00	1746.15	06/04/1988 10:00	1897.95		
21/01/1929 13:00	2064.30	19/02/1959 10:00	1720.58	05/04/1989 07:00	3103.45		
30/06/1930 11:00	749.23	14/11/1959 14:00	1674.36	29/05/1990 10:00	1482.29		
06/02/1931 14:00	5574.73	27/02/1961 12:00	206.40	09/02/1991 10:00	374.52		
10/12/1931 18:00	327.63	10/01/1962 06:00	595.14	17/03/1992 15:00	2587.87		
20/01/1933 21:00	311.94	19/03/1963 06:00	1364.37	20/08/1992 00:00	54.94		
23/02/1934 16:00	614.28	01/04/1964 12:00	464.54	01/03/1994 07:00	46.78		
26/02/1935 16:00	119.91	04/07/1964 12:00	113.51	01/02/1995 00:00	39.81		
24/03/1936 09:00	138.63	21/07/1965 10:00	1689.64	05/05/1996 21:00	4590.43		
17/03/1937 17:00	1102.97	12/06/1967 05:00	2644.09	17/02/1997 12:00	87.20		
28/05/1938 23:00	1052.08	13/01/1968 23:00	3792.18	18/09/1997 12:00	22.55		
17/03/1939 09:00	459.89	03/01/1969 04:00	339.06	10/02/1999 11:00	3596.54		

Table C.6 Peaks over $100 m^3s^{-1}$ threshold at savages and separated by a minimum of 5 days. Peak flows obtained from Amberley site where timing of ± 12 hours is allowed. Water years begin in July. Units are m^3s^{-1}. Savages flows converted to pre-dam.

Date	Q_{Sav}	Q_{Amb}	Date	Q_{Sav}	Q_{Amb}	Date	Q_{Sav}	Q_{Amb}
19/11/1961 04:00	316.46	28.247	13/03/1974 06:00	1082.9	194.45	06/07/1988 20:00	888.44	315.936
21/12/1961 06:00	341.39	90.672	20/03/1974 00:00	437.0	1.935	23/12/1988 11:00	494.23	0.775
10/01/1962 06:00	595.14	36.442	15/01/1975 05:00	393.6	NA	12/04/1989 20:00	225.89	11.291
16/01/1962 12:00	291.38	7.205	27/02/1975 18:00	362.6	32.454	27/04/1989 06:00	2992.31	158.197
14/04/1962 09:00	353.62	1.612	03/09/1975 16:00	333.5	NA	02/05/1989 07:00	2366.44	4.316
02/01/1963 18:00	349.65	24.463	20/10/1975 17:00	308.7	36.442	09/05/1989 07:00	223.28	3.051
19/03/1963 06:00	1364.37	64.709	25/12/1975 02:00	718.9	174.515	17/05/1989 22:00	1608.77	124.063
31/03/1963 09:00	352.08	100.312	22/01/1976 09:00	1844.3	95.398	22/05/1989 23:00	273.38	8.891
08/05/1963 18:00	640.39	348.89	27/01/1976 12:00	322.5	19.114	28/05/1989 16:00	271.18	7.371
22/05/1963 12:00	324.03	8.954	12/02/1976 17:00	1045.7	946.404	03/06/1989 10:00	225.51	2.524
01/04/1964 12:00	464.54	0.715	01/03/1976 01:00	1272.1	NA	03/02/1990 02:00	264.63	12.324
24/04/1964 08:00	370.43	93.017	09/03/1976 07:00	697.1	38.314	31/03/1990 04:00	305.14	11.1
21/07/1965 10:00	1689.64	NA	15/03/1976 18:00	716.0	5.756	08/04/1990 12:00	1365.02	44.927
01/02/1967 02:00	448.20	2.726	31/03/1976 18:00	380.7	1.69	22/04/1990 15:00	376.91	131.201
26/02/1967 01:00	379.40	2.194	27/05/1976 18:00	434.4	1.225	27/04/1990 16:00	334.18	9.608
10/03/1967 20:00	324.53	17.968	02/11/1976 06:00	641.8	109.827	29/05/1990 10:00	1482.29	141.425
19/03/1967 23:00	1490.34	117.191	15/11/1976 19:00	322.9	108.058	03/06/1990 13:00	323.09	8.175
08/05/1967 15:00	405.73	5.5	12/03/1977 21:00	361.3	86.854	08/06/1990 14:00	260.92	4.289
12/06/1967 05:00	2644.09	329.847	03/04/1978 10:00	545.8	127.122	09/02/1991 10:00	374.52	657.611
27/06/1967 11:00	1579.91	107.176	08/09/1978 22:00	288.2	12.991	22/11/1995 00:00	436.05	104.93
02/07/1967 17:00	339.06	3.371	31/12/1978 19:00	345.6	14.385	11/01/1996 09:00	395.13	66.971
13/01/1968 23:00	3792.18	402.947	24/01/1979 12:00	491.6	18.758	05/05/1996 21:00	4590.43	297.607
19/02/1968 12:00	306.66	1.493	09/02/1981 12:00	971.0	178.552	10/05/1996 22:00	206.34	10.869
21/03/1968 17:00	333.49	20.39	17/02/1981 03:00	483.1	11.614	10/02/1999 08:00	3596.54	173.554
03/01/1969 04:00	339.06	4.211	22/02/1981 22:00	365.6	19.064	15/02/1999 09:00	1504.90	3.742
29/08/1969 07:00	348.47	NA	05/11/1981 00:00	352.6	207.425	09/03/1999 10:00	331.13	3.71
11/12/1970 08:00	782.82	113.47	05/12/1981 10:00	584.2	22.276	14/03/1999 12:00	318.32	1.652
01/01/1971 12:00	679.88	100.312	25/12/1981 16:00	329.7	99.7	04/02/2001 16:00	951.34	54.805
05/02/1971 05:00	1512.45	795.928	22/01/1982 08:00	1217.8	164.742	09/02/2001 17:00	320.23	2.197
11/02/1971 10:00	833.37	249.448	27/01/1982 09:00	296.6	15.863	06/03/2004 04:00	514.62	46.5
20/02/1971 03:00	3123.18	201.478	04/02/1982 08:00	353.4	1.214	21/11/2008 07:00	715.29	152.242
25/02/1971 04:00	764.74	23.708	28/02/1982 04:00	432.4	12.175	14/04/2009 04:00	231.55	12.061
02/03/1971 23:00	296.59	14.152	12/03/1982 07:00	618.2	35.262	20/05/2009 16:00	468.89	304.004
29/12/1971 13:00	778.29	5.942	17/03/1982 15:00	452.0	2.977	03/03/2010 11:00	244.05	81.834
14/02/1972 15:00	2092.72	5.074	04/05/1983 13:00	607.8	157.546	08/03/2010 12:00	206.31	23.704
20/02/1972 18:00	718.89	7.71	09/05/1983 17:00	341.1	3.192	14/10/2010 12:00	3059.85	7.717
05/04/1972 04:00	1903.17	32.302	14/05/1983 20:00	292.2	1.444	19/10/2010 13:00	208.44	4.232
13/11/1972 02:00	612.81	29.568	29/05/1983 08:00	724.2	303.763	06/12/2010 18:00	223.40	86.465
19/02/1973 15:00	957.56	56.75	03/06/1983 10:00	317.2	8.272	22/12/2010 04:00	2867.58	53.84
01/03/1973 23:00	388.23	4.403	24/06/1983 00:00	1897.0	272.066	30/12/2010 12:00	3249.00	139.643
09/07/1973 10:00	3045.21	5.867	29/06/1983 02:00	681.8	10.326	12/01/2011 01:00	12926.21	705.826
30/07/1973 11:00	369.68	3.602	04/07/1983 05:00	499.2	4.363	17/01/2011 03:00	6566.55	37.058
13/10/1973 00:00	452.59	1.972	09/07/1983 07:00	426.5	6.365	22/01/2011 04:00	368.89	55.381
08/11/1973 19:00	353.05	3.315	14/07/1983 11:00	395.5	2.978	27/01/2011 05:00	279.56	7.14
11/01/1974 15:00	645.98	117.191	19/07/1983 13:00	360.0	2.123	11/02/2011 03:00	202.00	3.365
17/01/1974 02:00	557.90	20.669	09/04/1984 04:00	323.0	69.836	01/03/2011 20:00	532.88	1.537
28/01/1974 01:00	11136.89	1664.289	14/11/1984 15:00	296.6	16.262			
06/02/1974 22:00	1037.15	10.192	06/04/1988 10:00	1897.9	225.258			
12/02/1974 02:00	295.28	4.868	12/04/1988 12:00	1283.0	13.638			
20/02/1974 03:00	442.48	2.881	05/06/1988 05:00	603.3	254.025			

Table C.7 Estimate of peak over threshold flow data underlying Figure B2 of WMA Ipswich Report (manually read from graph). Flows are Log_{10} m^3s^{-1}. There will be minor loss of precision compared to actual data due to ability to resolve symbol placement.

Q_{Sav}	Q_{Amb}	Q_{Sav}	Q_{Amb}	Q_{Sav}	Q_{Amb}	Q_{Sav}	Q_{Amb}
2.55	0	2.43	1.14	2.97	1.76	3.57	2.19
2.62	0	2.76	1.18	2.96	1.78	2.77	2.2
2.64	0	2.44	1.25	3.13	1.8	3.47	2.2
2.66	0	2.75	1.25	2.55	1.82	2.98	2.25
2.69	0	3.26	1.25	2.85	1.84	3.04	2.28
2.55	0.3	2.55	1.27	2.52	1.85	2.55	2.3
2.58	0.3	2.69	1.27	3.19	1.85	3.26	2.35
2.65	0.3	2.53	1.3	2.8	1.86	2.76	2.42
2.72	0.3	2.57	1.33	2.38	1.9	3.68	2.42
2.3	0.48	2.47	1.4	3.25	1.97	2.65	2.45
2.53	0.48	2.52	1.42	2.53	2	2.86	2.45
2.57	0.48	2.5	1.45	2.57	2	2.94	2.5
2.64	0.48	3.43	1.45	2.83	2	3.42	2.51
2.53	0.65	2.83	1.49	2.34	2.02	2.8	2.6
3.48	0.7	2.77	1.5	2.8	2.05	3.6	2.6
2.6	0.8	2.54	1.51	3.16	2.06	3.17	2.7
2.84	0.8	2.63	1.54	3.27	2.08	2.57	2.75
2.88	0.8	3.45	1.58	2.73	2.11	2.83	2.8
3.32	0.8	2.55	1.6	3.15	2.13	4.13	2.8
2.43	0.85	3.13	1.65	3.19	2.14	3.03	3.05
2.64	0.9	2.56	1.69	3.5	2.14	4.05	3.15
3.48	0.9	2.58	1.7	2.58	2.17		
2.52	0.95	2.7	1.72	2.85	2.17		
2.67	1.1	2.87	1.72	3.08	2.17		
2.78	1.1	3.49	1.72	3.52	2.17		

Table C.8 Approximation of Figure B6 in WMA Ipswich Report, Bremer River heights given Amberley and Savages flows. Units are m. Data manually determined by reading graph. Data captures the underlying relationship but loses significant precision for lower flows. Data is interpolated at a higher resolution to retrieve some precision.

Q_{amb} (rows) / Q_{Savage} (columns)

	0	1000	2000	3000	4000	5000	6000	7000	8000	9000	10000	11000	12000	13000	14000	15000
12000	25.75	26	26.1	26.3	26.5	26.6	26.7	26.8	27	27.2	27.4	27.6	27.8	28	28.25	28.5
11500	25.5	25.75	25.9	26.1	26.3	26.4	26.6	26.7	26.8	27	27.2	27.3	27.5	27.66	27.75	28.1
11000	25.25	25.5	25.6	25.8	26.2	26.2	26.3	26.4	26.7	25.7	26.8	27	27.2	27.33	27.5	27.8
10500	25	25.1	25.2	25.5	25.8	25.8	26	26.1	26.2	25.4	26.6	26.7	26.9	27	27.25	27.5
10000	24.6	24.8	25	25.2	25.4	25.6	25.8	25.9	26	25.2	26.4	26.5	26.5	26.8	27	27.2
9500	24.33	24.5	24.7	25	25.2	25.4	25.5	25.7	25.8	26	26.1	26.3	26.4	26.6	26.75	26.9
9000	24	24.2	24.4	24.5	25	25.2	25.3	25.4	25.7	25.8	25.9	26	26.1	26.3	26.5	26.7
8500	23.66	23.3	24.1	24.25	24.5	25	25.1	25.2	25.3	25.4	25.6	25.7	25.3	26	26.2	26.4
8000	23.3	23.5	23.75	24	24.25	24.5	24.8	25	25.1	25.2	25.3	25.4	25.5	25.8	26	26.1
7500	23	23.1	23.3	23.6	23.9	24.2	24.6	24.7	24.75	24.9	25	25.1	25.4	25.5	25.7	25.8
7000	22.33	22.75	23	23.2	23.5	23.8	24.2	24.4	24.5	24.7	24.8	24.9	25.1	25.3	25.4	25.5
6500	21.66	22.2	22.5	22.9	23.1	23.4	23.8	24.1	24.25	24.3	24.6	24.6	24.8	25	25.1	25.2
6000	21	21.5	22	22.33	22.7	23	23.33	23.66	24	24.1	24.2	24.3	24.5	24.8	24.9	25
5500	20.2	21	21.5	22	22.33	22.7	23	23.3	23.5	23.75	24	24.1	24.2	24.4	24.7	24.75
5000	19	20	20.7	21.2	22	22.35	22.5	23	23.2	23.5	23.75	23.8	24	24.1	24.3	24.5
4500	17.5	18	19.5	20.5	21.2	21.8	22.1	22.5	22.8	23.1	23.25	23.4	23.5	23.9	24	24.25
4000	16	16.2	17.8	19.5	20.5	21.2	21.6	22	22.33	22.66	23	23.1	23.3	23.5	23.8	24
3500	14	14.2	15.8	17.5	19.5	20.5	21	21.5	22	22.3	22.6	22.9	23	23.3	23.4	23.66
3000	12	12.3	13.5	15	17	19	20.1	21	21.5	21.7	22.1	22.3	22.5	22.9	23.1	23.33
2500	10	10.4	10.7	12.5	14.3	16.4	19	20.1	20.8	21.1	21.5	22	22.1	22.5	22.8	23
2000	9	9.7	9.5	10.1	12	14	16	18.4	20	20.5	21	21.33	21.66	22	22.25	22.5
1500	6	9.1	8.5	9	10.1	12	14	16.5	18	19	20.2	20.5	21.2	21.5	22	22.25
1000	4	8	8	8	8.8	10	11	12.8	14.2	16	17.5	19	20	20.75	21.25	21.5
500	1	4	6	6.5	7.3	8	8.5	9.5	11	13	14	16	17	18.3	20	20.5
0	0	0.5	1	2	4	5	5	7.2	8.8	9.8	11.1	12	14	15.3	16.6	18
	0	1000	2000	3000	4000	5000	6000	7000	8000	9000	10000	11000	12000	13000	14000	15000

	16000	17000	18000	19000	20000	21000	22000	23000	24000	25000	26000	27000	28000	29000	30000	
12000	28.75	29	29.33	29.66	30	30.25	30.5	30.75	31	31.3	31.7	32	32.2	32.4	32.6	12000
11500	28.5	28.75	29.1	29.4	29.7	30	30.25	30.5	30.75	31.1	31.5	31.75	32	32.2	32.4	11500
11000	28.1	28.5	28.75	29.1	29.4	29.7	30	30.25	30.5	30.8	31.25	31.5	31.75	32	32.2	11000
10500	27.8	28.1	28.5	28.75	29.1	29.4	29.7	30	30.25	30.5	30.9	31.25	31.5	31.75	32	10500
10000	27.5	27.8	28.1	28.5	28.7	29.1	29.4	29.7	30	30.5	30.6	31	31.25	31.5	31.75	10000
9500	27.2	27.5	27.8	28.1	28.4	28.7	29.1	29.5	29.66	30	30.3	30.6	31	31.25	31.5	9500
9000	26.8	27.2	27.5	27.8	28	28.4	28.7	29.1	29.33	29.65	30	30.3	30.6	31	31.25	9000
8500	26.6	25.8	27	27.5	27.8	28	28.4	28.7	29	29.33	29.66	30	30.3	30.6	31	8500
8000	26.4	25.6	26.8	27	27.4	27.8	28.1	28.4	28.8	29	29.33	29.66	30	30.3	30.66	8000
7500	25	25.2	26.5	26.8	27	27.4	27.8	28.1	28.4	28.65	29	29.33	29.66	30	30.33	7500
7000	25.8	26	26.2	26.5	26.66	27	27.4	27.8	28.1	28.33	28.66	29	29.33	29.66	30	7000
6500	25.4	25.7	25.9	26.1	26.33	26.56	27	27.4	27.8	28	28.33	28.66	29	29.33	29.66	6500
6000	25.2	25.4	25.6	25.8	26	26.33	26.66	27	27.4	27.65	28	28.33	28.66	29	29.33	6000
5500	25	25.1	25.4	25.6	25.8	26	26.33	26.66	27	27.33	27.66	28	28.33	28.66	29	5500
5000	24.66	24.8	25.1	25.3	25.5	25.8	26	26.33	26.66	27	27.33	27.66	28	28.33	28.66	5000
4500	24.33	24.5	24.9	25	25.2	25.4	25.66	26	26.33	26.65	27	27.33	27.66	28	28.33	4500
4000	24.1	24.3	24.5	24.7	24.9	25.1	25.33	25.66	26	26.33	26.66	27	27.33	27.66	28	4000
3500	23.7	24	24.2	24.4	24.6	24.8	25	25.33	25.66	26	26.33	26.66	27	27.33	27.66	3500
3000	23.5	23.8	24	24.1	24.3	24.6	24.8	25	25.33	25.65	26	26.33	26.66	27	27.33	3000
2500	23.1	23.4	23.7	23.9	24	24.2	24.6	24.8	25	25.33	25.66	25	26.33	25.66	27	2500
2000	22.8	23.1	23.3	23.5	23.75	24	24.2	24.5	24.8	25	25.33	25.66	26	25.33	26.66	2000
1500	22.5	22.7	23	23.4	23.5	23.56	23.9	24.1	24.4	24.65	25	25.33	25.66	26	26.33	1500
1000	22.2	22.33	22.7	22.9	23.25	23.33	23.5	23.8	24.1	24.33	24.66	25	25.33	25.66	26	1000
500	21.4	22	22.2	22.4	22.66	23	23.25	23.5	23.75	24	24.33	24.66	25	25.33	25.66	500
0	20	21	21.5	22	22.33	22.56	23	23.25	23.5	23.75	24	24.33	24.66	25	25.33	0
	16000	17000	18000	19000	20000	21000	22000	23000	24000	25000	26000	27000	28000	29000	30000	

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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 – SUPPLEMENTARY REPORT ON IPSWICH FLOOD FREQUENCY ANALYSIS PREPARED BY WMAWATER

1 BACKGROUND

WMAwater prepared a report for the Queensland Floods Commission of Inquiry (QFCI) entitled '*Brisbane River 2011 Flood Event – Flood Frequency Analysis*' dated 18th September 2011 (WMAwater, 2011a). The report estimated 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 estimated 100 year ARI flood levels along the lower reaches of the Brisbane River and compared them with the 100 year ARI flood levels currently adopted by Brisbane City Council.

WMAwater have since prepared a supplementary report to the above report entitled '*Supplementary Report – Ipswich Flood Frequency Analysis*' dated 12th October 2011 (WMAwater, 2011b). The supplementary report has estimated the 100 year ARI flood level and the ARI of the January 2011 flood at the Ipswich City Gauge.

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 above two WMAwater reports for the purpose of assisting the commission. WRM's preliminary findings on the first (Brisbane) report were presented on 14th October 2011 (WRM, 2011). WRM's preliminary findings on the supplementary (Ipswich) report are presented below.

2 SCOPE OF WORK

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

This review has been undertaken under significant time constraints, and hence, the findings of this report should be considered as preliminary. No independent hydrologic or hydraulic modelling or analysis has been undertaken by WRM as part of this review.

3 GENERAL FINDINGS

The WMAwater report states that the estimation of design flood levels at Ipswich is a particularly complex task that has a considerable level of uncertainty due to the difficulty in quantifying the interaction between the Brisbane River and the Bremer River. The WMAwater report also states that the past studies have not thoroughly addressed the joint probability of these two main flood mechanisms at Ipswich. I agree with these statements.

Due to time constraints, the WMAwater report has undertaken only a '*preliminary analysis*' to estimate the 100 year ARI design flood level and the ARI of the January 2011 flood at Ipswich. The WMAwater report has identified significant limitations and uncertainties in the analysis they have undertaken to date and has stated that further efforts to reduce uncertainties in various parts of their analysis would be worthwhile. The WMAwater report has also provided a number of recommendations to overcome some of the limitations and uncertainties identified in the current analysis. For these reasons and other reasons described in the following sections of this report, in my view, the results of the WMAwater report are not suitable for adoption at the present time.

The WMAwater report has not produced 100 year ARI flood profiles within Ipswich or estimated flood levels at key locations along the Bremer River as required under the scope of work provided by QFCI. According to the WMA report, this work has not been done because the available modelling tools and data were insufficient to undertake such an analysis within the available timeframe.

4 COMMENTS ON ADOPTED METHODOLOGY

WMAwater has adopted a flood frequency approach to determine design flood levels at the Ipswich City Gauge incorporating a consideration of the joint probability of coincident flooding in the Brisbane and Bremer rivers. The adopted approach, which is based on the methodology proposed by Laurenson (1974), is acceptable and is in accordance with guidelines given in Australian Rainfall and Runoff (IEAust, 1998) for the assessment of concurrent flooding.

The limitations and uncertainties identified in the WMAwater analysis (some of them identified in the WMAwater report) include:

- The insufficient consideration of backwater effects at the Brisbane and Bremer rivers confluence and the conditional probability relationship between the two river systems;
- The need for improved schematisation of the MIKE-11 model used to develop the relationship between the flood level at Ipswich and the discharges in the Brisbane and Bremer rivers;
- The use of discharge data in the analysis from previous studies without thorough review;

- The use of a single relationship between pre-dams and post-dams peak discharges at Savages Crossing;
- The inadequate consideration given to the difference in timing of the flood peaks in the Brisbane and Bremer rivers at Ipswich, particularly given the flood mitigation impacts and operating rules of Wivenhoe and Somerset dams;
- The inadequate review of the quality of rating curves used for different gauging stations, especially for high discharges; and
- The simplifying assumptions that had to be made to undertake the joint probability assessment within a limited timeframe.

The methodology adopted to convert the Brisbane River pre-dams peak flood discharges at Savages Crossing into post-dam peak discharges (with Wivenhoe and Somerset dams in place), in my view, is not satisfactory. The adopted methodology is too simplistic and, in my view, should not be used as discussed in our previous report (WRM, 2011).

Not all flood affected areas within the Ipswich Local Government Area (LGA) are affected by concurrent flooding from the Brisbane and Bremer rivers. Some areas are affected primarily by the Brisbane River, some areas are affected primarily by the Bremer River and its many tributaries, and some other areas are affected by other creek systems draining through the Ipswich LGA. A significant area through the central parts of Ipswich along the lower reaches of the Bremer River is affected by flooding from both the Brisbane and Bremer rivers. This is the area which is the subject of the WMAwater report.

5 COMMENTS ON DATA USED IN THE ANALYSIS

In the analysis undertaken in the WMAwater report, recorded discharges in Warrill Creek at Amberley and in the Brisbane River at Savages Crossing have been used to represent flows in the Bremer River and Brisbane River respectively near Ipswich. Warrill Creek is the largest tributary of the Bremer River. The Amberley station commands a catchment area of 914 km^2, which is about 48% of the total Bremer River catchment area of approximately 1,900 km^2. The Amberley gauge is about 9 km upstream of the confluence of Warrill Creek and the Bremer River.

The Walloon gauging station on the Bremer River commands a catchment area of 622 km^2 and is located about 6 km upstream of the confluence of Warrill Creek and the Bremer River. The Walloon station has the same length of discharge record as the Amberley station. However, the WMAwater report has not used recorded discharges in the Bremer River at Walloon apparently because a previous study (Sargent, 2006) identified this station as having '*unreliable hydraulic characteristics and/or poor ratings*'. Any justification for this finding could not be found from a review of the Sargent (2006) report. It is of note that, on the basis of data and information given in the Queensland Department of Environment and Resource Management (DERM) website (http://watermonitoring.derm.qld.gov.au/host.htm), the gauging station at Walloon appears to be quite well rated up to about 900 m^3/s and the rating for this station appears to be more reliable than for the Amberley gauging station.

It is suggested that any future analysis should consider routing recorded discharges at the Walloon and Amberley gauging stations to the confluence of the Bremer River and Warrill Creek and combining these routed discharges to produce a Bremer River discharge record just downstream of the confluence of the Bremer River and Warrill Creek. Such a record would provide a good representation of the total Bremer River discharge into the Brisbane River and enhance the accuracy of a joint probability analysis of the two river systems.

The Savages Crossing station on the Brisbane River is located approximately 58 km upstream of the confluence of the Brisbane and Bremer rivers, whereas the Moggill station is located just downstream of the confluence and the Mt Crosby station is located approximately 17 km downstream of the confluence. All data available for Moggill and Mt Crosby have not been collected and properly reviewed or considered in the WMAwater analysis, probably due to time constraints. It is suggested that a thorough review of this data be undertaken and used to enhance the accuracy of a joint probability analysis of the two river systems.

For use in the pre-dams flood frequency analysis (FFA) for Savages Crossing, post-dams data (1985-2011) has been converted to pre-dams data using the relationship in Figure 2 of the WMAwater report. In my opinion, this is unsatisfactory and subjects Lockyer Creek discharges downstream of the dams to the same conversion. Further, the adopted (single) conversion factor does not accurately account for the actual mitigation effects of the dams on individual flood events. For example, the February 1999 flood was larger than the 1974 flood in the upper Brisbane River, but this flood was fully mitigated by the dams (Seqwater, 2011). In the Savages Crossing pre-dams FFA, the 1974 discharge is taken as 9807 m^3/s whereas the 1999 discharge is taken as only 3597 m^3/s.

6 FLOOD FREQUENCY ANALYSIS RESULTS

The WMAwater report provides only very limited results on the FFA analyses. No FFA results and plots are given for Warrill Creek at Amberley, and there is no discussion on why the GEV distribution results appear to have been adopted for the Savages Crossing FFA in preference to LP3 results. Further, no statistics are given to assess how well the data fits the two probability distributions used to derive flood ARI's at Savages Crossing and Amberley.

It appears that Figure 3 and Figure 4, and Figure C1 and Figure C2, of the WMAwater report are labelled incorrectly. Figure 3 shows the FFA (GEV) results for the pre-dams data at Savages Crossing excluding the January 2011 flood and Figure 4 shows the FFA (GEV) results for Savages Crossing including the January 2011 flood. Similarly, Figure C1 shows the FFA (LP3) results for the pre-dams data at Savages Crossing excluding the January 2011 flood and Figure C2 shows the FFA (GEV) results for Savages Crossing including the January 2011 flood.

It is unclear whether FFA results in Table 3 of the WMAwater report are from fitting the peak discharges to the GEV or LP3 distribution.

It appears that post-dams 1974 and 2011 peak flood levels at Ipswich are labelled incorrectly in Figure 5 of the WMAwater report.

The estimated January 2011 peak flood discharge at the Savages Crossing gauge for pre-dam conditions given in the WMAwater report is 12,926 m^3/s. Based on results in Table 3, the January 2011 flood ARI at the Savages Crossing gauge for pre-dam conditions has been estimated to be over 100 years if the January 2011 flood is excluded from the FFA and less than 100 years if the January 2011 flood is included.

Although there are some apparent deficiencies in the data used in the FFA, the pre-dams 100 year ARI discharge estimate at Savages Crossing is within the confidence band of previous FFA results indicating that the adopted value is of the right order of magnitude.

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

- As discussed before, there is a heavy reliance on the accuracy of Figure 2 of the WMAwater report to estimate post-dams discharges at Savages Crossing which, in my view, is too simplistic.
- The estimated 100 year ARI flood level at Ipswich (20.6 mAHD) is significantly higher than all previous estimates. The estimated 100 year ARI flood level at Ipswich is also significantly higher than the 1974 (post-dams) peak flood level used in Figures 5 and 6 of the WMAwater report. This has been attributed to higher design discharges adopted for the Brisbane River.
- The ARI of the January 2011 flood at Ipswich has been estimated at 75 years. The WMAwater report suggests that a more detailed analysis is likely to produce an ARI estimate closer to 100 years. It is noted that WMAwater (2011a) estimated the ARI of the January 2011 flood at the Brisbane Port Office to be 120 years. The reasons for the significant difference between Brisbane and Ipswich estimates are not discussed in the WMAwater report.

7 CONCLUSION

WMAwater have described the analysis presented in their supplementary report as preliminary with significant limitations and uncertainties. The WMAwater report has also indicated that further efforts to reduce uncertainties in various parts of their analysis would be worthwhile and has provided a number of recommendations to overcome some of the limitations and uncertainties identified in their analysis. For these reasons and other reasons described in this report, it is my opinion that additional more rigorous analyses involving more comprehensive hydrologic and hydraulic modelling studies, including joint probability assessments and Monte-Carlo type analyses, are required to accurately estimate design flood levels in Ipswich.

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

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A part of BMT in Energy and Environment

Queensland Floods Commission of Inquiry Technical Review of Flood Frequency Analysis Report by WMAwater (12 October 2011) on Ipswich Flood Frequency Analysis

prepared for

Ipswich City Council

R.B18414.004.00.doc

October 2011

Queensland Floods Commission of Inquiry Technical Review of Flood Frequency Analysis Report by WMAwater (12 October 2011) on Ipswich Flood Frequency Analysis

prepared for

Ipswich City Council

October 2011

Offices

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

BMT WBM

CONTENTS

LIST OF TABLES

1 PURPOSE AND SCOPE OF THE REPORT

This Report has been prepared by Neil Collins. Neil's CV is included in Appendix A.

This Report documents a desktop review of a report prepared by Mark Babister of WMAwater dated 12 October 2011 (received 13 October 2011) for the Queensland Floods Commission of Inquiry, entitled 'Supplementary Report – Ipswich Flood Frequency Analysis – Final Report' (**Ipswich Frequency Report**).

In accordance with the timetable stipulated by the Queensland Floods Commission of Inquiry (**the Commission**) review reports of the Ipswich Frequency Report were required to be completed by 4 p.m. 20 October 2011, less than five full business days after receipt of the Ipswich Frequency Report. There has been insufficient time for a thorough review of the Report, to re-run models used or to construct independent models. We have therefore concentrated on three elements of the analysis that are critical to the conclusions drawn, being:

- The Savage's Crossing flood frequency analysis from which the ARI 100 year Brisbane River flow is derived.
- The 'conversion' of Warrill Creek / Brisbane River flow correlations to Bremer River / Brisbane River correlations.
- The derived flood levels at David Trumpy Bridge in Ipswich (Ipswich CBD) based on MIKE11 flood modelling.

The key conclusion drawn by WMAwater (at 78) is that the estimated 1% AEP flood level at Ipswich (David Trumpy Bridge/CBD) is RL20.6m. This conclusion is adopted despite the large uncertainty in predictions which are acknowledged in the Report (including at Section 4.5). We do not agree with either the inferred accuracy or the magnitude of this assessment, and this report details inaccuracies and uncertainties associated with the above three elements of analysis in this report which we consider make the key conclusion and other conclusions arrived at by WMWwater unreliable.

In summary, the analysis conducted by WMAwater:

(a) is likely to have introduced an overly conservative 'high bias' (Weinmann, October 2011) into the flood frequency analysis that would have led to an overestimation of flow for the 1% AEP event;

(b) is heavily reliant on a direct catchment area proportioning conversion of Warrill Creek / Brisbane River flow relationship to a Bremer River / Brisbane River, with the assumption that proportional flows would have occurred in the Bremer River to those that occurred in Warrill Creek. This was not the case in either the January 2011 floods or the 1974 floods;

(c) relies heavily on the use of the existing MIKE11 flood model to predict flood levels at Ipswich CBD when this model is known to be inaccurate as acknowledged in WMAwater's July 2011 report on 'Review of Hydraulic Modelling';

(d) 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

(e) 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 Brisbane and Bremer river systems, as significant scour and siltation occurred during the January 2011 flood event. The Ipswich Frequency Report by WMAwater has relied on the existing MIKE11 hydraulic model to translate flood levels for the ARI 100 year event 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).

[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

2 GENERAL COMMENTS

The joint probability flood frequency analysis approach to Bremer and Brisbane River flooding is supported, though there are a variety of ways this can be carried out, including Monte Carlo simulation modelling as recommended in the 2003 Independent Review Panel Report to Brisbane City Council. That Report also recommended the use of both flood frequency analysis and simulation modelling.

WMAwater acknowledge the need for substantial revision to both hydrologic and hydraulic models (paragraph 72), which is needed to provide a critical cross-check of the flood frequency analysis.

WMAwater's recommendations regarding risk management (paragraph 22) and the need for consideration of evacuation routes and procedures on all events up to the Probable Maximum Flood (PMF) are fully endorsed.

Flooding in Ipswich City can be significantly influenced by Brisbane River flooding and this is acknowledged in paragraph 74. WMAwater have relied upon the conclusions reached in its Brisbane Frequency Report when conducting the joint probability analysis for the Bremer River. Hence, our report of 14 October 2011 in relation to Brisbane Frequency Report is relevant to the WMAwater's Ipswich Frequency Report. In our report of 14 October 2011 we conclude that it is premature for WMAwater to reach the conclusion that the 1% AEP flood flow of 9,500 $m^{3/}$s for the Brisbane River at the Port Office gauge be adopted. In our view that conclusion is unreliable for the reasons explained in our report of 14 October 2011 The Ipswich flood frequency analysis derives a flow at Savage's Crossing consistent with the WMAwater Port Office flow and uses this to determine the flood level in Ipswich City. Given the influence of Brisbane River flooding on Ipswich City, any inaccuracy in the Brisbane River flow directly affects the reliability of flood level predictions in Ipswich.

3 REVIEW OF FLOOD FREQUENCY ANALYSIS AT SAVAGE'S CROSSING

The Ipswich Frequency Report relies heavily on the methodology used in the WMAwater September 2011 Brisbane Frequency Report for the Brisbane River, which has been subject to expert review by a number of experts. Having reviewed those reports, we support the key findings as follows:

Erwin Weinmann (October 2011)

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 9500m^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 of 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.

Rory Nathan, Sinclair Knight Merz (28 September 2011)

55 On the basis of the material presented by WMA Water, 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 13000m^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 estimation 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.*

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.

Further to these comments by other experts on the methodology, we comment as follows.

Flood Frequency Analysis

The Flood Frequency Analysis (FFA) carried out by WMAwater on Savage's Crossing uses an appropriate methodology that is consistent with current best practice for a site flood frequency analysis in Australia. However, a number of subjective decisions have not been reported including:

- Choice of flood distribution
- Selection of censored data
- Use of historic data

Decisions made in these choices and selections will directly affect the results of the FFA.

Additionally, output from Flike has not been presented which would include parameters and model diagnostics. This output would assist reviewers. Different flood distributions produce different results, as does the adopted cut off flow in analysis.

While extensive work on the FFA at Savage's Crossing has been presented, no FFA on Amberley Gauge has been presented. While the Savage's Crossing gauge is the primary gauge in the analysis, presentation of FFA at the Amberley Gauge would be beneficial.

Uncertainty

While various aspects of the analysis undertaken in the report identify uncertainty, either quantitatively or qualitatively, these uncertainties have not been propagated through the analysis and no uncertainty bounds (or confidence intervals) are presented for the flood level at Ipswich. Given the identified uncertainties and the statistical nature of the analysis this should have been provided. The assumption at 79 that the 2% and 0.5% floods encapsulate uncertainty is not statistically based and is subjective.

Conditional model

One of the key steps in the methodology of Laurenson (1973) is the determination of a relationship between discharges at the two upstream stations. WMAwater have determined a relationship for flows at Amberley (Q_{Amb}) and conditional flows at Savage's Crossing (Q_{Sav}). Despite being one of the key steps only limited detail is presented in the Report.

This inclusion of this detail in the Report would allow reviewers to assess and comment on the determination of the Log-Normal relationship including a justification of the selection of this model. Further there is no information presented on the appropriateness of the determined model of (Q_{Amb})

conditional on (Q_{Sav}). Documentation and reporting of this step would also benefit from the presentation of model diagnostics and plots of results.

The Log-Normal distribution has been parameterised using the log-log relationship between (Q_{Amb}) and (Q_{Sav}) to determine the mean (μ) with the standard deviation (σ) determined from the binned residuals. WMAwater note that the variance of the residuals reduces with increasing bin ranges and conclude that there is a stronger dependence between gauges at high flows. However, depending on the bin ranges the determination of the standard deviation may have been based on a limited number of data points and therefore the estimate of standard deviation may be sensitive. This is particularly relevant to higher flows and may affect the degree of uncertainty of the analysis.

Joint Dependence

The premise of the Report is that there is joint dependence between discharges on the Brisbane and Bremer Rivers and only through consideration of this joint dependence can reliable estimates of flood levels at Ipswich be obtained. However, the strength of the joint dependence has not determined.

The strength of the joint dependence can be determined using bivariate or multivariate extreme value analysis. The theoretical background to this is presented in Coles (2001) and this method has recently been applied in Australia by Westra (2011) to investigate the joint dependence between rainfall and storm surge.

This should be completed for the site before a reliable conclusion can reached regarding joint dependence. It is important to recognise that the Savage's Crossing discharge is strongly influenced by dam operation, whereas Warrill Creek has no regulation. This must affect the reliability of correlation used.

Alternative Joint Probability Approaches

The use of the Laurenson model (1973) is a little surprising given that it is nearly 40 years old and there have been a number of significant developments in the assessment of joint probability predominately between surge tide and flooding in coastal catchments. A number of recent examples are presented below.

For instance, McInnes et al. (2009) notes that joint probability methods are commonly applied to evaluate storm tide return periods. This study uses Monte Carlo method to estimate the Joint Probability distribution between tide and surge distributions. While this example assumes that tide and surge distributions are independent, which differs to the Brisbane / Bremer River case, the method could be readily adapted using the conditional probability distribution derived from the Amberley Gauge (notwithstanding the comments above).

There are also frequentist approaches such as the χ measure approach of Svensson and Jones (2004) to investigate the dependence of surge on rainfall and river flow. The assessment of bivariate and multivariate extreme value analysis has been covered by Cole (2001) and this has been recently been applied in Australia by Westra (2011) as noted above. The work by Westra is currently being extended as part of the Australian Rainfall and Runoff update and will provide a methodology for estimating the exceedance probability of a flood event (or AEP) when it is caused by multiple factors. While this is currently being developed for surge and flood events it is likely it could be readily applied to the Brisbane / Bremer River case.

Further, Bayesian approaches provide a natural framework to investigate joint dependence. For instance, Coles and Tawn (2005) note that the Bayesian approach provides for the management of uncertainties as well as a framework for the construction of complex statistical model that would be intractable using frequentist approaches. A Bayesian joint probability approach has been applied by Wang et al. (2009) to estimate seasonal stream flow in south–eastern Australia.

In summary, I consider that the finding that the ARI 100 year flow at Savage's Crossing is 9,800m^3/s premature and subject to a large amount of uncertainty, with the potential for an overestimation of the ARI 100 year flow. This has a direct bearing on flood levels predicted in Ipswich City, given the influence of Brisbane River flows.

4 REVIEW OF CORRELATION OF WARRILL CREEK FLOWS TO BREMER RIVER FLOWS

WMAwater use a flow correlation between Warrill Creek flows at Amberley and Brisbane River flows at Savages Crossing, as part of a joint probability analysis for flows in the Bremer River in Ipswich City and in the Brisbane River at Moggill.

This process is described in Appendix B of the WMAwater Ipswich Frequency Report.

A key step in this analysis is the translation of the Warrill Creek / Brisbane River flow relationship to a Bremer River / Brisbane River flow relationship.

Paragraph B12 states: "Each value of Q_{Ips} was factored to a corresponding flow at Amberley based on a simple relative catchment area relationship (assumed $Q_{Amb} = 0.6*Q_{Ips}$)". Based on the 0.6 factor, we assume WMAwater has proportioned catchment areas for Bremer River at Walloon to Warrill Creek at Amberley, as demonstrated below. Table 4-1 below summarises the catchment areas and proportion of the total contributing catchment upstream of Ipswich.

Table 4-1 - Contributing Catchment Areas

Catchment	Catchment Area (km^2)	Fraction of total area (no Purga Creek)	Fraction of total area (with Purga Creek)
Bremer River @ Walloon	638.6	0.41	0.36
Warrill Creek @ Amberley	913.3	0.59	0.52
Total area (no Purga Creek)	1551.9	-	-
Purga Creek @ Loamside	210.4	-	0.12
Total area (with Purga Creek)	1762.3	-	-

These figures show that the WMAwater catchment area relationship is true when Purga Creek is not considered. However, if Purga Creek is also included in the catchment area relationship, the proportion of contributing catchment for Warrill Creek drops to 0.52.

Using gauging station data extracted from the DERM website, Table 4-2 below gives the peak flows for a range of historic flood events for the major contributing catchments upstream of Ipswich.

Table 4-2 - Peak Flows Upstream of Ipswich for Historic Flood Events

Event date	Total combined flow (m^3/s)	Recorded Peak Flow (m^3/s)			Proportion of Total Flow		
		Bremer River @ Walloon	Warrill Creek @ Amberley	Purga Creek @ Loamside	Bremer River @ Walloon	Warrill Creek @ Amberley	PurgaCreek @ Loamside
Jan 1968	887	484	403	-	0.55	0.45	-
Jan 1974	4179	1660#	2108	411	0.40	0.50	0.10
June 1983	1182	602	437	143	0.51	0.37	0.12
Apr 1989	658	389	158	111	0.59	0.24	0.17
May 1996	1058	630	307	121	0.60	0.29	0.11
Feb 1999	706	451	195	60	0.64	0.28	0.08
Jan 2011	2501	1645*	706	150	0.66	0.28	0.06

Gauging records does not have data at the Walloon gauge for this event. Magnitude of flow has been taken from SEQWater data

* Gauging record indicates quality for this value as 'suspect'; value taken from URBS model data supplied by SEQWater

Paragraph 52 within the main body of the WMAwater Ipswich Frequency Report states that the Amberley gauge on Warrill Creek was considered more suitable for the FFA than the Walloon gauge on the Bremer River as it captures a larger proportion of the Bremer River catchment. Whilst this is true, based solely on catchment area, historical flow records from the gauging stations indicate that during flood events, flows at Walloon generally exceed the flows at Amberley, as given in the tables shown in Appendix B of this report.

Therefore, the following can be determined:

- The assumed catchment area relationship ($Q_{Amb} = 0.6*Q_{Ips}$) does not correlate with the flow data for the various gauging stations upstream of Ipswich.
- The historical data suggests that (on average) flows are greater in the Bremer River catchment to Walloon than the Warrill Creek catchment to Amberley, despite having a smaller catchment area.
- Purga Creek has an average contribution of 10%-11% to the total flow upstream of Ipswich.

5 IMPACT OF THE USE OF THE MIKE11 MODEL

WMAwater in their July 2011 'Review of Hydraulic Modelling' describe in detail the shortcomings of the SKM Version 2 MIKE11 model which is used in the Ipswich Frequency Analysis (Chapter 4). In particular, in paragraph 56 of WMAwater's July 2011 report, they state:

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

In Appendix B to our October 2011 report on WMWwater's Brisbane Frequency Report, we comment (in Chapter 4) on the SKM MIKE11 model and conclude that there is considerable uncertainty over the accuracy of flood wave timing and magnitude in the Ipswich area.

Our review of the Brisbane Frequency Report also provides comment on the URBS model used by SKM and prepared by SEQ Water. We conclude in that report that the URBS model represents the most reliable tool currently available for Ipswich City, as it matched very closely the recorded flood levels in Ipswich for the January 2011 event.

Therefore, in order to check the results by WMAwater in terms of levels in Ipswich City, for the Brisbane River and Bremer River flows assumed, we have utilised SEQ Water's URBS model. Appendix C provides details of the key rating curves of flow in the Bremer River versus gauge height at Ipswich City (the David Trumpy Bridge gauge). Each curve relates to a different Brisbane River tailwater level at Moggill.

Using WMAwater's published Brisbane River flood level at Moggill from their September 2011 Brisbane Frequency Report (Figure 13), the 100 year flow for the Brisbane River (9800m^3/s), and the corresponding flow in the Bremer River at Ipswich City (1900m^3/s) based on WMAwater's 12 October 2011 Ipswich Frequency Report, the URBS model produces a Q100 flood level at Ipswich City of RL18.3m, 2.3 metres lower than that predicted by the MIKE11 model. Had the 2003 Independent Review Panel design flow of 6,000m^3/s been used, flood levels predicted in Ipswich using the WMAwater methodology would have been between RL16 and 17m.

It is important to note that there is inadequate information provided by WMAwater to exactly define the assumed river flows or Moggill tailwater level, hence, we have had to rely upon interpolation.

The SEQ Water URBS model was not designed as a flood prediction tool, but rather as part of an overall rainfall / runoff and flood management system for the entire rivers system catchment. As such, whilst it performs a very useful cross-check of flood levels in Ipswich, we do not recommend its results be relied upon in isolation of alternate analysis. Such alternative analysis requires new river survey, re-building and recalibrating of the hydrodynamic model and flood frequency analysis, including Monte Carlo simulation and we again note that this work is all within the scope of the current WSDOS study.

6 CONCLUSIONS

I conclude that:

1 The conclusions reported by WMAwater at Section 5.2 of the Ipswich Frequency Report, and particularly the estimate that 1% AEP flood level at Ipswich (David Trumpy Bridge) of 20.6 mAHD (at paragraph 78), cannot be justified.

2 The methodology to convert the estimate of 'no-dam' ARI 100 year flows at Savage's Crossing to current conditions is simplistic and may have produced an overly conservative outcome, with over-estimation of the ARI 100 year flow.

3 The extent of uncertainty in the Savage's Crossing flow estimate should be reduced by alternative joint probability analyses, such as Monte Carlo simulation, as recommended by previous studies and reviews, including WSDOS.

4 The translation of the relationship developed between Warrill Creek and Brisbane River flows, to Bremer River and Brisbane River flows using catchment area alone ignores the effect of Purga Creek, leading to an incorrect estimation of Bremer River flows. In any case, the assumed relationship of flow at Amberley being 0.6 times flow at Ipswich does not correlate with any historic flow data. For example, a factor of 0.28 has been derived for the January 2011 event, and none of the 7 significant historic events exceed 0.5 and were generally lower. This translation method is not appropriate and is inaccurate and an alternate method taking account of spacial variability is required.

5 SEQ Water's URBS model is considered more accurate than the MIKE11 model used by WMAwater in predicting flood levels in Ipswich City for given combinations of Bremer River and Brisbane River flows, and the current MIKE11 model is considered unreliable for Ipswich City predictions.

6 Using the URBS model, I estimate that for the Brisbane and Bremer River flows assumed by WMAwater in their analysis (and I question WMAwater's assessment of the Brisbane River flows for the reason outlined report on the Brisbane Frequency Report), the peak flood level in Ipswich is RL18.3m AHD, some 2.3m lower than that predicted by the MIKE11 model. This analysis is not intended to suggest that RL18.m AHD is the correct flood level (given my views on the Brisbane River flows adopted by WMAwater) but it demonstrates the unreliability of the estimate at 78 of WMAwater's report. Given the very close match of the URBS model to the recorded January 2011 flood levels, the URBS model is a more reliable tool at present for assessments of flood levels in Ipswich City. Had the 2003 Independent Review Panel design flow of 6000m^3/s been used, flood levels predicted in Ipswich using the WMAwater methodology would be between RL16 and 17m.

7 Because of the very large uncertainty range inherent in the analysis, and because of a number of apparent overly conservative assumptions on which the WMAwater analysis for Ipswich has been based, it is not appropriate to rely on the reported findings in terms of ARI 100 year flood levels for Ipswich City.

8 The analysis by WMAwater has been carried out in a short period of time in isolation of the WSDOS study that is underway, and these works should be completed before any conclusions of event frequency and the ARI 100 year flood line can be determined.

7 LIMITATIONS

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

Due to the extremely short timetable for review, this report concentrates on three specific areas of uncertainty to demonstrate that the conclusions drawn are premature and that much more work is required before any firm conclusions can be reached.

8 REFERENCES

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APPENDIX A: CURRICULUM VITAE OF NEIL IAN COLLINS

A part of BMT in Energy and Environment

BMT WBM

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*

[redacted]

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.

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: ANALYSIS OF FLOW RECORDS FOR BREMER CATCHMENTS INCLUDING WARRILL CREEK

APPENDIX B - ANALYSIS OF FLOW RECORDS FOR BREMER CATCHMENTS INCLUDING WARRILL CREEK

Paragraph B12 states: "Each value of Q_{Ips} was factored to a corresponding flow at Amberley based on a simple relative catchment area relationship (assumed ($Q_{Amb} = 0.6*Q_{Ips}$))". Based on the 0.6 factor, I assume he has proportioned catchment areas for Bremer River at Walloon to Warrill Creek at Amberley, as demonstrated below. Table B-1 below summarises the catchment areas and proportion of the total contributing catchment upstream of Ipswich.

Table B-1 - Contributing Catchment Areas

Catchment	Catchment Area (km^2)	Fraction of total area (no Purga Creek)	Fraction of total area (with Purga Creek)
Bremer River @ Walloon	638.6	0.41	0.36
Warrill Creek @ Amberley	913.3	0.59	0.52
Total area (no Purga Creek)	1551.9	-	-
Purga Creek @ Loamside	210.4	-	0.12
Total area (with Purga Creek)	1762.3	-	-

These figures show that the WMA Water catchment area relationship is true when Purga Creek is not considered. However, if Purga Creek is also included in the catchment area relationship, the proportion of contributing catchment for Warrill Creek drops to 0.52.

Using gauging station data extracted from the DERM website, Table B-2 below gives the peak flows for a range of historic flood events for the major contributing catchments upstream of Ipswich.

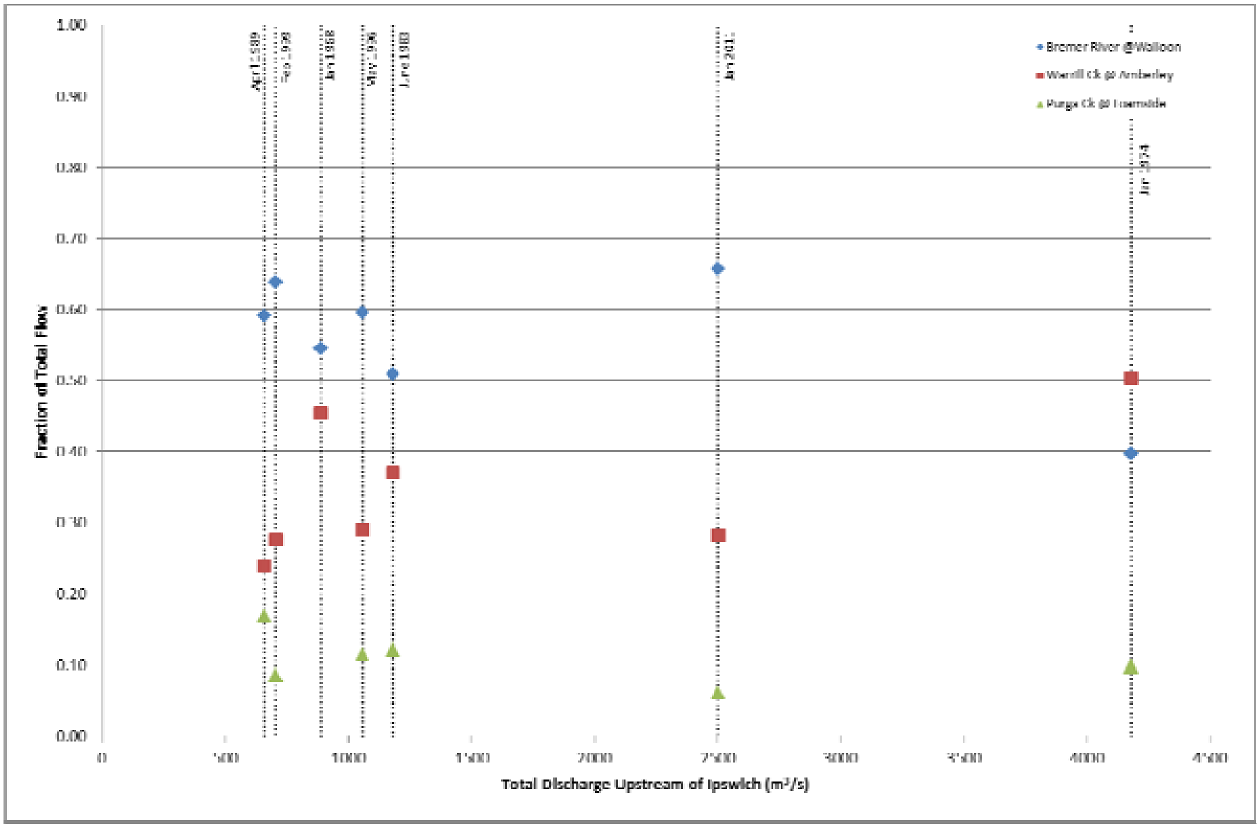
Table B-2 - Peak Flows Upstream of Ipswich for Historic Flood Events

Event date	Total combined flow (m^3/s)	Recorded Peak Flow (m^3/s)			Proportion of Total Flow		
		Bremer River @ Walloon	Warrill Creek @ Amberley	Purga Creek @ Loamside	Bremer River @ Walloon	Warrill Creek @ Amberley	PurgaCreek @ Loamside
Jan 1968	887	484	403	-	0.55	0.45	-
Jan 1974	4179	1660#	2108	411	0.40	0.50	0.10
June 1983	1182	602	437	143	0.51	0.37	0.12
Apr 1989	658	389	158	111	0.59	0.24	0.17
May 1996	1058	630	307	121	0.60	0.29	0.11
Feb 1999	706	451	195	60	0.64	0.28	0.08
Jan 2011	2501	1645*	706	150	0.66	0.28	0.06

Gauging records does not have data at the Walloon gauge for this event. Magnitude of flow has been taken from SEQWater data

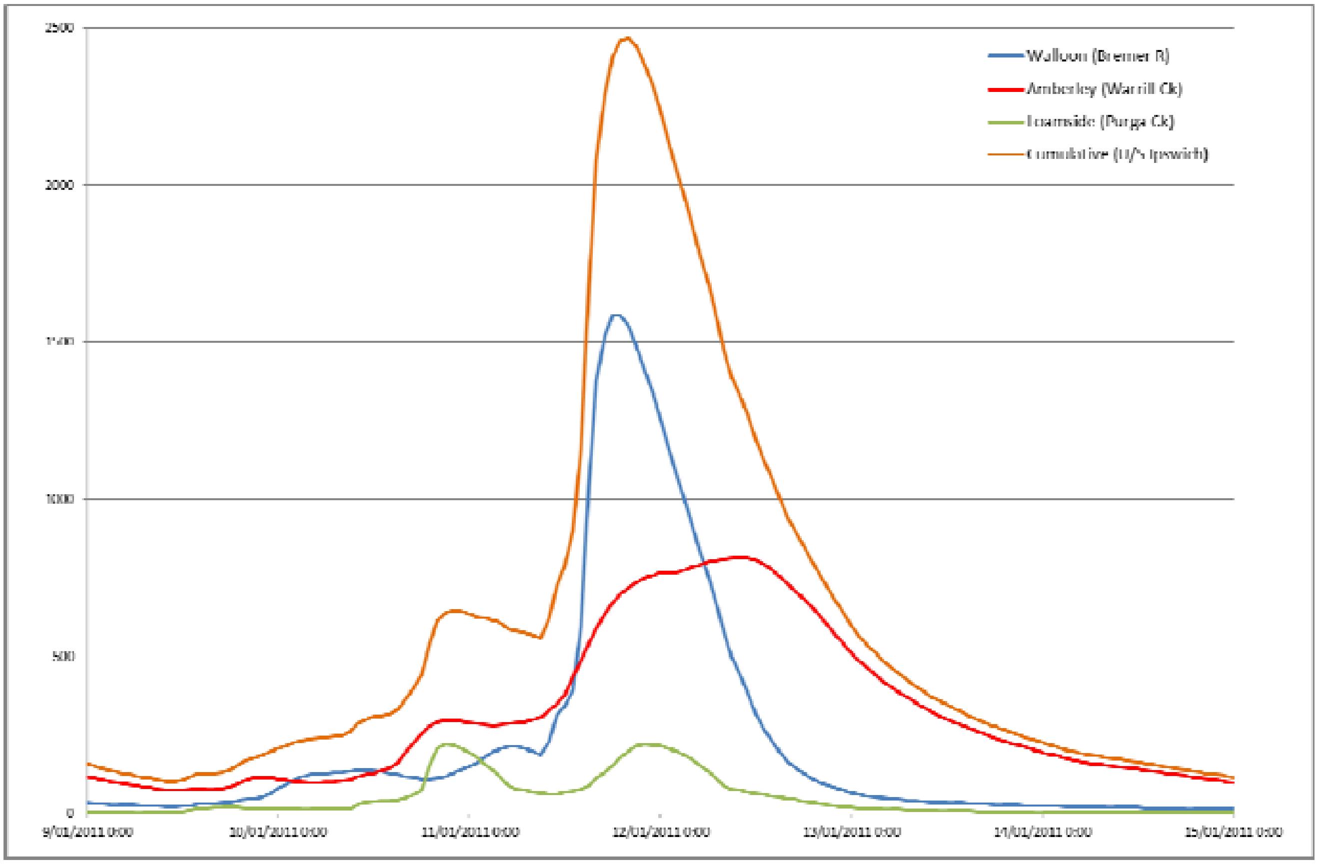
* Gauging record indicates quality for this value as 'suspect'; value taken from URBS model data supplied by SEQWater

Figure B-1 – Proportion of flows upstream of Ipswich for major contributing catchments



Flow hydrographs at the various gauges in the vicinity of Ipswich are given in Figure B-2 below. This shows that the majority of flow upstream of Ipswich is within the Bremer River catchment with a lesser contribution from the Warrill Creek catchment.

Figure B-2 – Flow hydrographs for January 2011 event



APPENDIX C: URBS MODEL RATING CURVES FOR IPSWICH CBD

Ipswich Flood Frequency Analysis – URBS Comparison

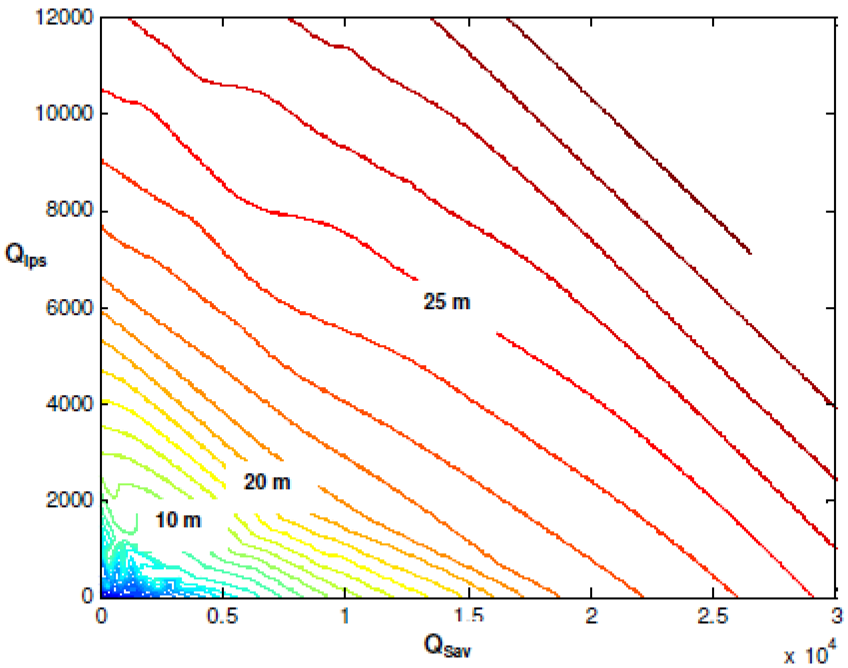
Using Ipswich 1% AEP flood levels from WMA Water's Supplementary Report – Ipswich Flood Frequency Analysis (FFA) to derive peak 1% AEP flows at Ipswich to compare estimated 1% AEP flood levels predicted with SEQ Water's calibrated URBS model.

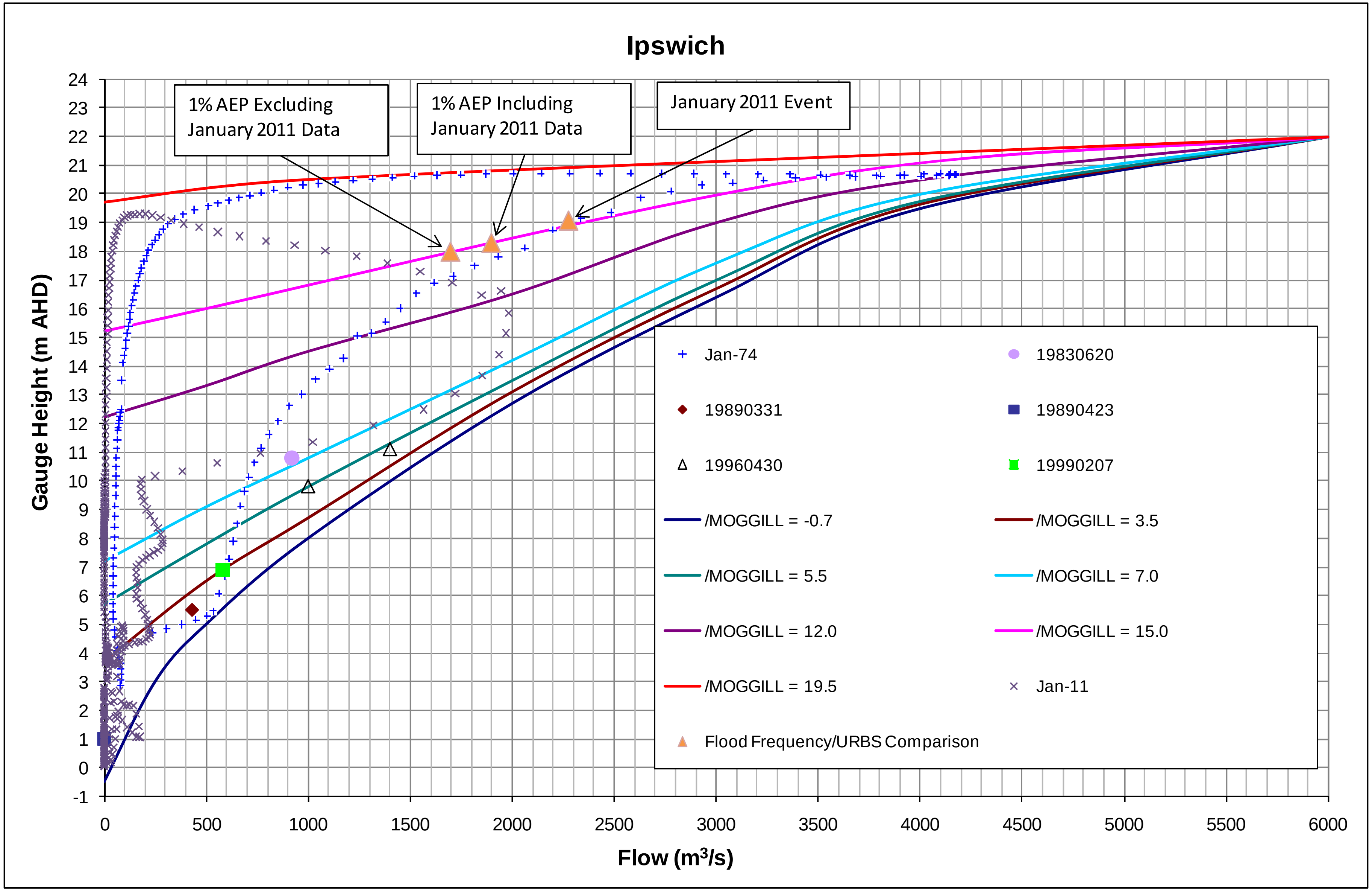
Table C1 – 1% Flow and Level Comparison

	1% AEP	
	Excluding January 2011 Data	Including January 2011 Data
Peak Water Level (mAHD) (WMA Water Ipswich FFA)	20.0	20.6
$Q_{savages}$ (m^3/s)	8300	9800
$Q_{Ipswich}$ (m^3/s)*	1700	1900
Peak Water Level (mAHD) (SEQWater's Ipswich URBS rating– Refer Figure C1)	18.0	18.3

* Derived from Figure B6, WMA Water's Supplementary Report - Ipswich Flood Frequency Analysis

Figure B6: Contours of Ipswich flood level relationship with Q_{Sav} and Q_{Amb}





Source: SEQWater

Figure C1

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