NSW RECONSTRUCTION AUTHORITY

HAWKESBURY-NEPEAN RIVER FLOOD STUDY TECHNICAL VOLUME 7: UPDATE TO THE MONTE CARLO FRAMEWORK AND SELECTION OF REPRESENTATIVE DESIGN FLOOD EVENTS

FINAL REPORT

Junction of Colo and Hawkesbury Rivers, 2021 flood. Photo by Adam Hollingworth, courtesy of INSW

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HAWKESBURY-NEPEAN RIVER FLOOD STUDY TECHNICAL VOLUME 7: UPDATE TO THE MONTE CARLO FRAMEWORK AND SELECTION OF REPRESENTATIVE DESIGN FLOOD EVENTS	**Project Number** 113031-15
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Revision History

Revision	Description	Distribution	Authors	Reviewed by	Verified by	Date
0	Draft Report	INSW	M.Babister M.Ward	H.Taylor		MAR 22
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Acknowledgement of Country

The NSW Reconstruction Authority and WMAwater acknowledge the Traditional Custodians of the lands where we work and live. We celebrate the diversity of Aboriginal peoples and their ongoing cultures and connections to the lands and waters of NSW.

We pay our respects to Elders past, present and emerging and acknowledge the Aboriginal and Torres Strait Islander people that contributed to the development of this report.

We advise this resource may contain images, or names of deceased persons in photographs or historical content.

Note

In July 2023, the Hawkesbury-Nepean Valley Flood Risk Management Directorate transitioned from Infrastructure NSW (INSW) to the NSW Reconstruction Authority. Any references to INSW should be read as referring to the Authority.

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TABLE OF CONTENTS

PAGE

LIST OF TABLES

APPENDIX B:

LIST OF FIGURES

APPENDIX B:

APPENDIX C:

LIST OF DIAGRAMS

LIST OF ACRONYMS

AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ALS	Airborne Laser Scanning
ARR	Australian Rainfall and Runoff
BOM	Bureau of Meteorology
DECC	Department of Environment and Climate Change (now DPE)
DNR	Department of Natural Resources (now DPE)
DPE	Department of Planning and Environment
IFD	Intensity, Frequency and Duration (Rainfall)
mAHD	metres above Australian Height Datum
OEH	Office of Environment and Heritage (now DPE)
PMF	Probable Maximum Flood
RORB	Runoff routing Software
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydraulic model)
WBNM	Watershed Bounded Network Model (hydrologic model)

ADOPTED TERMINOLOGY

Australian rainfall and runoff – A guide to flood estimation (ARR) (Pilgrim, 1987) is a national guideline document, data and software suite that can be used for the estimation of design flood characteristics in Australia. The fourth edition of ARR was published by the Commonwealth of Australia in 2019 (ARR 2019) (Ball et al., 2019). Geoscience Australia supports ARR as part of its role to provide authoritative, independent information and advice to the Australian Government and other stakeholders to support risk mitigation and community resilience.

ARR 2019 recommends flood frequency terminology that is not misleading to the public and stakeholders. Flood events are described in terms of the chance of occurrence in any one year, with this probability normally assigned to a flood based on its peak level. While there is a very high correlation between peak flow and peak level, individual floods show considerable variability in terms of flood volume, rate of rise and duration of inundation. This variability is caused by how wet the catchment is prior to an event and when, where and how much rain falls on the catchment. Floods occur randomly, so one flood event does not change the chance of a subsequent flood occurring. Rare events may occur in clusters: two floods with approximately a one per cent chance per year occurred in Kempsey in 1949 and 1950; the two largest floods in Brisbane occurred two weeks apart in 1893. Therefore, the use of terms such as ‘recurrence interval’, ‘return period’, and even ‘average recurrence interval’ (ARI), are no longer recommended as they imply that a given event magnitude is only exceeded at regular intervals such as every 100 years.

ARR 2019 recommends the use of Annual Exceedance Probability (AEP) to describe flood probabilities or frequency. Annual Exceedance Probability (AEP) is the probability of an event being equalled or exceeded within a year. AEP may be expressed as either a percentage (%) or 1 in X. Floodplain management typically uses the percentage form of terminology. Therefore a 1% AEP event or 1 in 100 AEP has a one per cent chance of being equalled or exceeded in any year. This report adopts the terminology of 1 in 100 AEP.

ARI and AEP are often mistaken as being interchangeable for events equal to or more frequent than 1 in 10 AEP. The table below describes how they are subtly different. It categorises flood events according to the ARR 2019 classification.

The probable maximum flood (PMF) is the largest flood that could reasonably be expected to occur for a catchment. For the purposes of floodplain management, and consistent with the NSW Government's *Floodplain Development Manual*, the PMF is estimated using the probable maximum precipitation (PMP) and a single temporal pattern. Due to the conservativeness applied to other factors influencing flooding, a PMP does not translate to a PMF of the same probability. But for the purposes of floodplain management, the probability of the PMP may be assigned to the PMF.

The Bureau of Meteorology (BoM) and NSW State Emergency Service (NSW SES) use the terms 'minor', 'moderate' and 'major' to describe floods. These terms do not relate to a particular probability at any location but are assigned based on local consequences. For this reason, they vary in probability and severity at different locations along the rivers. For example, at Windsor gauge, minor floods are those between 5.8 and 7.0 metres, moderate floods are between 7.0 and 12.2 metres, and major floods exceed 12.2 metres.

Design event quantiles such as a 1 in 100 AEP are used to refer to standard probabilities of events used in design flood estimation for example those listed in the table below.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very Frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	19.5
	0.02	2	50	49.5
	0.01	1	100	99.5
Very Rare	0.005	0.5	200	199.5
	0.002	0.2	500	499.5
	0.001	0.1	1000	999.5
	0.0005	0.05	2000	1999.5
Extreme	0.0002	0.02	5000	4999.5
			↓	
			PMP/ PMP Flood	

Note: EY = Exceedances per Year; AEP = Annual Exceedance Probability; ARI = Average Recurrence Interval
Source: adapted from ARR 2019 (Ball et al., 2019)

ACKNOWLEDGEMENTS

WMAwater acknowledges the contributions of the Hawkesbury-Nepean Flood Risk Management Directorate and other parties including but not limited to Hawkesbury Library, Hornsby Library, and Dharug and Lower Hawkesbury Historical Society.

EXECUTIVE SUMMARY

This Technical Volume describes a component of work completed as part of the 2024 Hawkesbury-Nepean River Flood Study, which updates and builds on the Hawkesbury-Nepean Valley Regional Flood Study completed in 2019 (WMAwater, 2019). The 2024 Flood Study includes a refined hydrologic model (WBNM) for all catchments other than the Warragamba catchment (**Technical Volume 2**), and a new two-dimensional hydraulic model (TUFLOW) (**Technical Volume 3**). This Technical Volume covers modifications and updates to the Monte Carlo framework used in the 2019 Regional Flood Study, which was then used to provide representative design inputs for TUFLOW modelling. The study area is depicted in Figure 1.

While it was necessary to update the Monte Carlo framework to incorporate the refined WBNM hydrologic model (Figure 2), the opportunity was taken to extend calibration of the RUBICON model to Wallacia based on a joint probability assessment undertaken as part of the 2024 Flood Study (**Technical Volume 6**) and to the downstream locations of Sackville Ferry, Colo Junction, and Webbs Creek Ferry. The stage frequency analysis at these locations informed changes to initial and continuing losses in the Monte Carlo framework, as well as to the timing of inflows on the Colo River and downstream tributaries (**Technical Volume 5**). A relationship between rainfall and coastal water levels was developed following work by Baird (**Technical Volume 4**). The fast RUBICON hydrodynamic model was modified to replicate the TUFLOW model (Figure 3).

Outputs of this study include updated representative events for existing conditions to simulate design events using the TUFLOW hydraulic model (**Technical Volume 11**). Climate change has also been assessed for rainfall increases of 9.5% and 19.7%. These represent the high emissions scenarios for mid-century and late century respectively. The 9.5% rainfall increase is also representative of the low emissions scenario for the late century. Representative events have been extracted from the 20,000 simulations in the Monte Carlo framework for these two scenarios for input into the TUFLOW hydraulic model.

These representative events are selected by assessing results from the WBNM-RUBICON model and then applying those same events to the WBNM-TUFLOW model developed by Rhelm and Catchment Simulation Solutions (Technical Volumes 2, 3, 11). This is valuable when running 20,000 events considering the time required to run a single event simulation through TUFLOW (15hrs) compared to RUBICON (less than 1min).

The representative design events will be used by a range of stakeholders including councils within the valley and the NSW Government to inform evacuation and emergency management, and land use and infrastructure planning.

1. INTRODUCTION

1.1. Project Context

Design flood estimation in Australia is moving from a single event per quantile (such as the 1 in 100 AEP) to Monte Carlo modelling where thousands of events are run. Particularly for large complex catchments, this enables the variability of floods to be better understood.

The Hawkesbury-Nepean Valley Regional Flood Study (WMAwater, 2019; hereafter referred to as the Regional Flood Study) included a Monte Carlo framework which simulated a range of design flood events that were representative of different AEP probabilities. A computationally fast one-dimensional (1D) hydrodynamic model called RUBICON was used in the Monte Carlo framework. The resulting 20,000 synthetic events represented the equivalent of 200,000 years of flood history and reflected the variability in flood drivers in the Hawkesbury-Nepean Catchment.

This Technical Volume forms part of the 2024 Hawkesbury-Nepean River Flood Study, as shown below:

- Technical Volume 1: Data Collection & Review
- Technical Volume 2: Hydrologic Model Refinement and Calibration
- Technical Volume 3: Hydraulic Model Setup and Calibration
- Technical Volume 4: Catchment/Ocean Level Joint Probability Assessment
- Technical Volume 5: Lower Hawkesbury Analysis
- Technical Volume 6: Wallacia Flood Frequency Analysis
- **Technical Volume 7: Monte Carlo Analysis**
- Technical Volume 8: March 2021 Flood Event Validation
- Technical Volume 9: March 2022 Flood Event Validation
- Technical Volume 10: July 2022 Flood Event Validation
- Technical Volume 11: Design Flood Modelling
- Technical Volume 12: Probable Maximum Flood (PMF) Modelling.

This report, **Technical Volume 7**, describes updates to the Monte Carlo framework as part of the 2024 study. One change has been to adjust the RUBICON hydraulic model to incorporate insights gained from detailed hydraulic modelling using a two-dimensional (2D) model called TUFLOW (see **Technical Volume 3**).

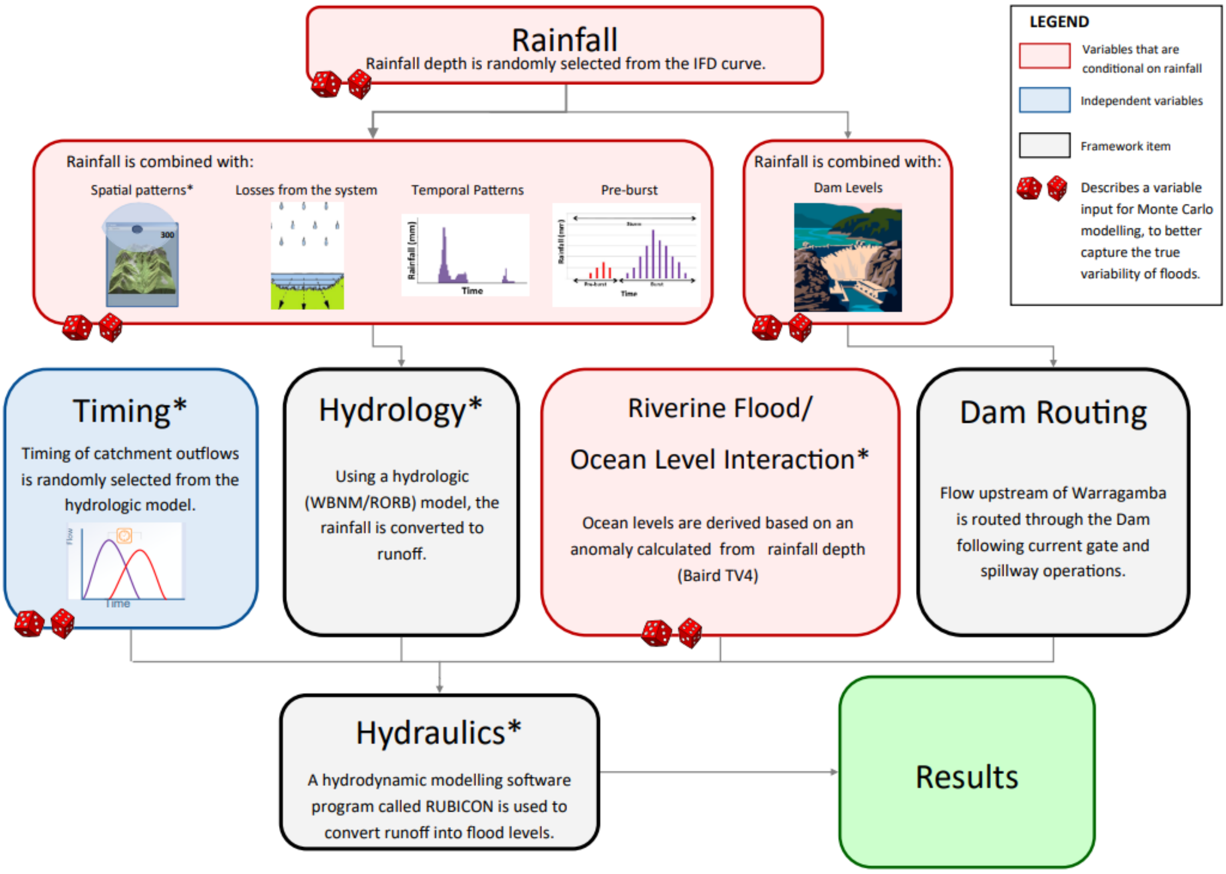
The outputs from the Monte Carlo modelling have been used as inflows and boundary conditions for the TUFLOW model, which is used to define the design flood events (see **Technical Volume 11**). While the TUFLOW model provides high resolution spatial results, it is not computationally feasible for TUFLOW to model the full suite of 20,000 synthetic design events generated from the Monte Carlo framework. This is why the RUBICON model used in the Monte Carlo framework is used to select representative events for detailed modelling in TUFLOW.

This Technical Volume is intended to be read in conjunction with the Regional Flood Study – elements of the Monte Carlo framework that have not been updated are described there.

1.2. Model Updates

This report documents the updates to the Monte Carlo framework and the selection of representative design events that have been undertaken since the completion of the Regional Flood Study.

As depicted in Diagram 1, the variables from the Monte Carlo analysis were inputs to the hydrological model, and the resultant flows, together with the other variables, were input into the RUBICON hydraulic model. This was used to assess flood behaviour.



** Indicates parts of the framework that have been updated for this study.*

Diagram 1: Flood Modelling Methodology

The following modelling updates have been undertaken since the Regional Flood Study report:

1. Downstream of Warragamba Dam, the existing RORB hydrologic model was replaced by a more detailed and refined Watershed Bounded Network Model (WBNM) (see **Technical Volume 2**). This model allows more detailed representation of the local tributaries and the interaction of these tributaries.

2. The computationally faster 1D RUBICON hydrodynamic model was adjusted to incorporate the insights gained from 2D TUFLOW hydraulic modelling completed as a part of the new flood study (see **Technical Volume 3**). The TUFLOW model was found to better represent floodplain storage and the constricted flow behaviour downstream of Windsor.

3. The riverine flood and ocean level interaction was updated to incorporate the correlation between extreme ocean levels and rainfall events in the valley (**Technical Volume 4**)

4. The timing of Colo River inflows to the Hawkesbury River was updated to incorporate a detailed historical analysis (**Technical Volume 5**)

5. Rainfall spatial patterns for the recent floods have been added to the suite of events, and initial losses have been varied in some downstream tributaries

6. The rating relationship at Windsor/Sackville was updated based on the results of the TUFLOW model

7. The calibration to flood frequency analysis and long-term flood records was extended to include flow records at Upper Colo and long-term stage records at Wallacia (**Technical Volume 6**), Sackville, Colo Junction/Lower Portland and Webbs Creek Ferry/Wisemans Ferry. This extended the calibration of long-term records to the Lower Hawkesbury River and the Nepean River above the Warragamba junction. In addition, flood frequency analysis for Warragamba Dam, Penrith and Windsor was extended to include floods up to 2022.

A summary of the updates made to the model framework is shown in Table 1.

Table 1 Monte Carlo Model Update Summary

	Component	Current Study	Relevant Section
Models	Hydrology	WBNM replaces RORB below Warragamba Dam	4.2
	Hydraulics	RUBICON recalibrated to match the outputs of TUFLOW	3.3
Fixed Inputs	Continuing losses	Recalibrated to match more locations: • Warragamba Dam • Nepean to Warragamba Junction, Warragamba Junction to Penrith • Penrith to Sackville including the Grose River, South and Eastern Creeks • Colo River to Ocean	4.3.1.2
Monte Carlo Inputs	IFD	unchanged	
	Temporal Patterns	unchanged	
	Spatial Patterns	Added spatial patterns from 2020, 2021, March 2022 and July 2022 events.	
	Initial Loss	Median initial loss adjusted from the Regional Flood Study in the catchments upstream of Wallacia and the Colo River Catchment and downstream tributaries, otherwise unchanged.	4.3.1.1
	Pre-burst Rainfall	unchanged	
	Dam drawdown	unchanged	
	Tributary Timing	Colo updated based on observed event timing (Technical Volume 5).	4.2.1
	Ocean Levels	Replaced by Baird (Technical Volume 4) for events rarer than a 1 in 2 AEP frequency. The new methodology creates a relationship between the maximum daily rainfall in the catchment and the maximum residual coastal water level. For events more frequent than a 1 in 2 AEP frequency, the levels remain unchanged	4.2.2

2. FLOOD FREQUENCY ANALYSIS

2.1. Overview

The flood frequency analysis (FFA) undertaken in 2019 provided strong validation for the Monte Carlo results at Warragamba, Penrith and Windsor (WMAwater, 2019). This has been expanded to include locations at Wallacia, Sackville, Colo Junction/Lower Portland and Webbs Creek/Wisemans Ferry. This additional work allowed for the improved calibration of the Monte Carlo model to the Lower Hawkesbury and the Nepean River above the Warragamba junction (see Section 4.3).

In addition, the period of record for the FFA at Warragamba, Penrith and Windsor has been extended to include the floods in 2020, 2021 and 2022.

FFA is the most robust method of estimating the probability of flooding where long flood records exist. It is a direct approach where a statistical distribution is fitted to the flood record. This analysis used an Annual Maximum Series (AMS) where the largest flood in each year is selected. The AMS approach is the recommended approach when the focus is on rarer floods. Peak Over Threshold (POT) analysis can also be used; however, this method is best adopted where a fit to floods less than the 1 in 10 AEP event is prioritised.

FFA is the foundation of design flood estimation in Australia for catchments where reliable streamflow records are available. However, it is generally necessary to use rainfall runoff modelling techniques in conjunction with FFA to produce a full hydrograph and assess catchment changes and flood mitigation options. The FFA documented in this section uses the most up to date techniques recommended in ARR 2019 (Ball et al. 2019). To reliably fit a statistical distribution to a flood record it is necessary to convert levels to flows and have a sufficiently long record. Where a rating curve is not present and the known large events have been recorded, it is possible to use the plotting position formula to assign an estimated frequency to the stage record for comparison with design flood levels. This approach has been adopted at Wallacia, Sackville, Colo Junction/Lower Portland and Webbs Creek/Wisemans Ferry.

Most of the data discussed in this section is described in the Regional Flood Study (WMAwater, 2019). This study combined more recent data with the extensive dataset collected in 1996 (Webb, McKeown & Associates). In the Lower Hawkesbury, data from the Lower Hawkesbury River Flood Study (AWACS, 1997) was also used, while at Wallacia data was sourced from the Upper Nepean River Flood Study (Department of Land and Water Conservation, 1995). Historical flood data and information were also sourced from the National Library of Australia's Trove database which contains digitised newspapers and gazettes dating back to 1803.

The generation of design flood estimates from FFA at Wallacia has been documented in the Wallacia Flood Frequency Analysis report (**Technical Volume 6**).

2.2. Rating Curves

Flood levels can be converted to discharge using site specific rating curves. This has been done for Penrith and Windsor-Sackville.

Some locations on the Hawkesbury-Nepean River do not have a unique relationship between stage and flow as their relationship is heavily influenced by backwater effects from downstream tributaries. This occurs at Wallacia, which is influenced by backwater discharges from Warragamba Dam, at Yarramundi, which is influenced by backwater from the Grose River, and at Sackville, which is influenced by backwater from the Colo River.

The following sections describe how model results were combined with gauging data to assist in the preparation of rating curves.

2.2.1. Penrith

Daily recording of water levels commenced at Penrith near Victoria Bridge in 1892. Flood records exist for some events before this date. This gauging site is currently managed by WaterNSW and rating curves are available for different historic periods. Historically, a new curve has been produced when alterations are made to the weir or there is a topographic change, such as occurred in August 1986 (bank reconstruction). Changes to the rating curve up to 1990 are detailed in the Regional Flood Study.

Further changes to vegetation near Penrith weir in the period between 1990 and the present have led to changes in the stage – flow relationship.

Figure 26 shows the adopted rating curve at Penrith for the post 1986 events, denoted “Original Rating Curve”. This relationship was used for calculating the discharge in the FFA for the post 1986 events that were not modelled under pre-dam conditions in TUFLOW. This is the same rating relationship used in the Regional Flood Study.

Figure 26 also shows gauged flows at Penrith in the period of 1986-1990. Further, the TUFLOW “Modelled Rating Curve” is presented, as per **Technical Volume 3**. The full event rating relationship for the TUFLOW modelled 1990 event is also provided.

2.2.2. Windsor-Sackville

The stage-discharge relationship between Windsor and Sackville was updated in the Monte Carlo framework based on the results of the TUFLOW model (Section 3.3, **Technical Volume 3**). The floodplain at Windsor is wide and complex, and flood level at Windsor is largely determined by the hydraulic constriction caused by the gorge downstream of Wilberforce. The DLWC (now DPE) took flood gaugings in the gorge at Sackville, but there was no recorded relationship between height at Windsor and flow in the gorge.

The TUFLOW model was used to estimate a relationship between flood height at Windsor and flow in the gorge for a series of historical and design floods. Sackville was chosen as the representative location of the gorge flow because it is downstream of a high flood breakout from the Windsor floodplain through Currency Creek, and all flow is concentrated to a single flow path.

The Monte Carlo results are plotted in Figure 27 for events larger than 8 mAHD at Windsor, this is the censoring threshold for the Windsor-Sackville FFA. Also plotted is the peak flow at Sackville and peak level at Windsor from the TUFLOW calibration events. Figure 27 includes the "Original" rating curve used in the Regional Flood Study and the "Adjusted" rating curve used in the current study, to derive the Windsor-Sackville FFA. The change to the rating relationship is due to an improved floodplain storage representation in the updated RUBICON model. This was informed by the latest LiDAR and the TUFLOW model, which indicated the old RUBICON model was underestimating floodplain storage in the Windsor area.

It was assumed that the relationship had remained unchanged throughout the period of record. While there have been substantial topographical changes on the Windsor floodplain, principally clearing of trees and sand mining downstream of Windsor Bridge, these changes are unlikely to impact the flood level at Windsor except in very small events as the water level is controlled by the restriction of the gorge, which has not changed.

2.3. Pre-Dam Adjustment

The major limitation of FFA is that it requires a good long-term homogenous dataset. This means that the method should only be adopted for locations with a good long-term flood record, that are either well gauged or where a reliable rating relationship can be calculated. In the Hawkesbury-Nepean, it is necessary to generate a homogenous dataset by removing the influence of Warragamba Dam. This is generally done by producing a pre-dam or without dam dataset as it is difficult to fit a statistical distribution to outflows from a gated dam. Many gated dams like Warragamba are operated in such a way that certain flows are not possible because of the predefined stage gate opening rules.

The formation of Lake Burragorang caused major changes to the hydraulic characteristics of the lower Wollondilly River, Coxs River and the Warragamba River. The lake had an impact on the size and timing of the outflow from the Warragamba catchment, and this in turn affected the flow regime at Penrith and further downstream, as well as flood levels upstream of the Warragamba and Nepean rivers' confluence at Wallacia.

Warragamba Dam impacts several aspects of the flow behaviour. If the dam is drawn down sufficiently, smaller floods' inflows can be significantly mitigated or completely captured as occurred in February 2020. However, inflows follow a cyclic nature with observed flood-dominated and drought-dominated cycles in the Windsor historical record. This means that the dam is usually relatively full during large events and the dam generally provides a very slight amount of mitigation. This mitigation does not always occur as peak discharges can be very close to pre-dam conditions when the peak dam level just triggers a gate opening.

The dam also changes the shape and timing of the dam discharge hydrograph. At full supply level, Lake Burragorang extends nearly 50km from the dam to the lake edge on the Wollondilly and Coxs rivers, and large parts of the lake are over 50m deep. This causes the flood wave to pass through the lake in less than an hour as inflows in the upper reaches raise the water levels of the whole lake. The additional storage in the lake largely offsets this accelerated travel time but peak discharges can occur earlier than if the dam was not present. The dam outflow hydrograph tends to be slightly mitigated and less peaky compared to the pre-dam scenario but can maintain flows close to the peak for a sustained period. This change in hydrograph shape and timing means that the dam outflow is more likely to coincide with the peak from the Nepean River. These aspects mean that the dam generally provides a slight level of mitigation. However, it is theoretically possible but unlikely for the dam to slightly increase flood levels downstream when the dam only provides a small level of mitigation, and when the early arrival of the Warragamba flows coincides with the Nepean flows.

In order to establish the magnitude of the dam's impact on flooding in the Hawkesbury-Nepean Valley, modelling was used to calculate the pre-dam discharge of large floods. Outflows from the dam were reverse routed to determine dam inflows and these inflows were then routed from the upper reaches of the lake back to the dam site under pre-dam conditions. When this was combined with the downstream modelling, pre-dam flows and levels could be estimated at all key locations. This modelling was carried out for the November 1961, June 1964, June 1975, March 1978, August 1986, April/May 1988 and August 1990 floods. A similar approach was used to estimate pre-dam flows for the February 2020, March 2021, March 2022 and July 2022 events. For the four recent events, downstream modelling under pre-dam conditions was completed using RUBICON whereas the events up to 1990 were modelled with TUFLOW.

While it would have been possible to carry out similar modelling for all the floods since 1960, a simplified approach was adopted as detailed in the 1996 Flood Study (Webb McKeown Associates, 1996, Appendix D.B). This simplified approach adjusts the level on the basis of pre- and post-dam flow at Warragamba. While the period of record there extends back to 1909, all of the larger events in this period occurred between 1949 and 1956. This was a very wet period, and only the 1949 event has any significant flow difference as Warragamba Dam would have been full for the other events.

To maintain consistency with the methodology used in the 1996 Flood Study the impacts of the dam on the 1867 event were calculated using modelling. This allowed for a comparison with the more complete post-dam record.

2.4. Pre-dam Adjustment for Sackville, Colo Junction and Webbs Creek Ferry

The flood events from 1961 onward were adjusted for the impact of Warragamba Dam by modelling the pre-dam scenario. Earlier flood events were adjusted to a nominal post-dam condition using a small flood adjustment technique.

At Sackville the dam tends to produce smaller changes as the flood level at Windsor and the subsequent flow downstream at Sackville are largely driven by the flood volume captured in the Windsor floodplain.

For the modelled floods (1961, 1964, 1975, 1978, 1986, 1988, and 1990) the average change in peak flow at the Sackville Ferry gauge was 200m³/s with 4 out of 7 of the events within 50m³/s. On this basis a fixed average adjustment of 200m³/s was assumed for the 1864, 1949, 1952, and 1956 events. All these events occurred in very wet periods, and other than the 1949 event they were proceeded by flood events within 3 years that would have filled the dam. The net result of this approach is that the smaller 1949 and 1952 events were reduced by approximately 300mm for post-dam condition and the larger 1864 and 1956 events by approximately 200mm.

At Colo Junction (Lower Portland), the floods of 1949 and 1956 required an adjustment to account for the effects of Warragamba Dam. Given the 1956 flood occurred during a wet period, we assume that Warragamba Dam would have been relatively full for this event. While it is unlikely the dam would be full in 1949, a more realistic change would not impact the rank of the event at Lower Portland. Therefore, the average of the change in flow from pre-dam to post for the modelled events (excluding 1961 where the dam started lower) was adopted at Lower Portland. This adjustment was equal to 170 m³/s.

At Webbs Creek Junction, an adjustment factor of 130m³/s was adopted for the 1889, 1949 and 1956 events to account for the impact of Warragamba Dam. Again, while the 1949 event did occur in a dry period, the average change was adopted. A larger flow adjustment would not change the post-dam event rank at Webbs Creek.

For the post 2020 events (February 2020, March 2021, March 2022, July 2022), the change in flow from the modelling was used to estimate pre-dam levels at the downstream sites.

2.5. Updated Flood Frequency Analysis

2.5.1. Locations

Table 2 lists the locations and the period of record for sites where FFA was undertaken. As part of this study, the number of sites where FFA has been undertaken was increased from the three sites of Penrith, Warragamba and Windsor used in the 2019 study. FFA at the additional sites is more challenging as the records are less complete, often shorter and are harder to adjust for the impact of Warragamba Dam.

Table 2. Flood record lengths used in this study

Location	Period of continuous record	Events prior to the continuous record
Wallacia	1909-2022	All large events since 1860
Warragamba	1909–2022	Estimates of 1864,1867, 1900 events
Penrith	1893–2022	Reliable measurement of 1867 event and some information on other large events in the 1860s
Windsor	1857–2022	Information on prior events back to 1791
Sackville	1909-2022	1864, 1867
Colo Junction/ Lower Portland	1949-2022*	No 1867 information available
Webbs Creek / Wisemans Ferry	1949-2022*	1867, 1889

*Events rarer than approximately 1 in 10 AEP considered

The pre-dam record was updated at Penrith and Windsor to use revised rating curves. The results are shown in Table 3 and discussed below. The Warragamba Dam pre-dam record did not require updating, except for the addition of the 2020-2022 events.

The FFA undertaken in the Regional Flood Study was carried out at Warragamba, Penrith and Windsor/Sackville with the emphasis on Penrith, which has a very good continuous record back to 1892, and was very well gauged up to 1992. Penrith gauge is located immediately upstream of Victoria Bridge and is often called the Victoria Bridge gauge. Actual flow gaugings are often carried out 3km upstream on the M4 Bridge where flow is confined to the river.

The very long record at Windsor was used with a model-derived rating curve based on flow at Sackville and level at Windsor. This record is very reliable back to 1855 because of the work of astronomer John Tebbutt (Babister et al., 2016a) and can be extended back to 1791 for large floods using multiple thresholds.

At both Penrith and Windsor, good records exist of the 1867 flood which is well in excess of the 1 in 100 AEP event. All flows were converted to pre-dam in order to undertake the FFA on a comparable basis (Section 2.3).

At Windsor, the FFA was fit using the continuous record from 1855, with the knowledge that 1867 is the largest flood since 1791. While the continuous record could be extended back to 1791, there is a risk of compounding errors due to the earlier flood levels being estimated relative to each other, and uncertainty about the size of the large event in 1809.

The Regional Flood Study trialled a range of distributions (LPIII and GEV) and fitting techniques and concluded that the best fit was produced using the LPIII distribution fitted using Bayesian techniques applying the Grubbs and Beck low flow test. As the annual series flows were only slightly changed, the same approach was adopted for the updated dataset.

The adopted fits are included as Figure 32 to Figure 34 and summarised in Table 3.

Table 3. Adopted Pre-dam FFA current study – Expected Parameter Quantiles (Flike)

Probability	**Flow (m^3/s)**		
1 in x AEP	**Warragamba**	**Penrith**	**Windsor**
10	5360	6020	4340
20	7640	8760	5730
50	10,660	12,680	7690
100	12,870	15,750	9260

The updating of the FFA has resulted in increases at Warragamba of around 4% for the 1 in 100 AEP with the inclusion of a gaussian prior distribution from Penrith. Using a prior is common practice where the statistical information from a gauge with a long record assists fitting at a nearby gauge, and is one of the major advantages of fitting using Bayesian methods. Using a prior at Warragamba is appropriate because Penrith has a longer, more reliable record, and typically about 80% of the flow at Penrith is from Warragamba.

There were very minor changes at Penrith due to improved pre-dam conversion estimates based on the updated model. There were decreases of around 11% for the 1 in 100 AEP Windsor estimate due to improved understanding of storage on the Windsor floodplain, which has influenced the rating curve between Windsor Bridge and Sackville.

Table 4 presents the flood frequency approach adopted for each of the key locations. Depending on the location, flow, stage (level) or both were used in the flood frequency analysis. The following sections detail the updates undertaken at Wallacia and at downstream locations where prior stage frequency analysis had not been undertaken.

Table 4. Adopted fits for Flood Frequency Approach

Location	Fitting method	Parameter	Description
Wallacia	Plotting position only	Stage	Fitting to pre- and post-dam stage records 1860-2022
Penrith	LP3	Flow	1791 to 2022 – Low flows censored applied with 1867 event included at 20,000 m^3/s and 101 years below 20,000 m^3/s
Warragamba	LP3	Flow	1791 to 2022 with 1864, 1867 and 1900 events included and 115 years below 9500 m^3/s, 50 years below 500 m^3/s A prior was included for Penrith.
Windsor	LP3	Flow	1791 to 2022 with 109 years below 2100 m^3/s (8m AHD).
Sackville	Plotting position only	Stage	Fitting to large pre- and post-dam stage records 1909-2022 – low levels censored below 1975 level of 7.1m AHD (unadjusted) with 1867 event included at 15.47m AHD
Colo Junction/ Lower Portland	Plotting position only	Stage	1949 – 2022 – low levels censored below 1988 level of 5.85 m AHD (unadjusted)
Webbs Creek/ Wisemans Ferry	Plotting position only	Stage	1949 to 2022 – low levels censored below 1988 level of 2.84m AHD (unadjusted), the 1867 event included at 9.14m AHD, the 1889 event included at 7.28m AHD and 1949 included at 5.57m AHD.

2.5.2. Wallacia flood record

The flood record at Wallacia dates back to 1860 with a continuous record from 1909. Additional detail on the compilation of this record is available in the Wallacia Flood Frequency Analysis (**Technical Volume 6**).

2.5.3. Downstream flood records

The Lower Hawkesbury River Flood Study, conducted by Australian Water and Coastal Studies (AWACS, 1997), collated flood records for Sackville at Sackville Ferry, Lower Portland (Colo Junction), and Wisemans Ferry (Webbs Creek), which have been adopted for use in the stage-frequency analysis. Where there are missing data in these records, they have been supplemented with data from other sources. While there is not a sufficient flood record to carry out a conventional flood frequency analysis at Sackville, Colo Junction and Wisemans Ferry, the top 6 events since

1909 were recorded, the next 6 events have generally been recorded, and the missing events can be infilled. Using the data at these three sites and Windsor it is possible to determine which events have not been recorded and to estimate the rank and level of any missing data using regression. The datasets have been censored where we are not confident that we have the complete record. In addition, an estimate of the 1867 flood level is available at Wisemans Ferry and a few kilometres upstream of Sackville. There is also information on other large floods from historical records.

The year 1909 is approximately the mid-point in the flood record at Windsor and also corresponds with the start of formal gauge records on the Colo and at Wallacia. This allows the plotting position of flood levels at these locations to be estimated for events with a return period above 1 in 10 AEP. For consistency, the same record period has been used at all three sites.

The other reason for using a stage analysis is that the three sites do not have unique rating relationships because of backwatering from the Colo and Macdonald rivers.

2.5.3.1. Sackville Flood record

A continuous daily record exists for Sackville from 1962 to 1968 and from 1972 to present. Additionally, the peak flood levels are available for floods between 1949 and 1962. A flood mark was surveyed for the 1867 flood, which is the largest on record. The Windsor flood record and other downstream records indicate that no significant events occurred between 1909 and 1949, which allows the plotting position to be calculated for a flood record above the 1975 flood level.

Table 5 includes the observed flood levels, the source of the events, and whether they were included in the analysis.

1952 and 1975 Event Estimation

At Sackville, two relatively small floods were missing from the record. These were the 1975 and the 1952 flood events.

An analysis of levels at Windsor and Sackville for other historical events was undertaken to assess whether a relationship could be established to estimate these missing events at Sackville. The initial relationship is shown in Diagram 2 which plots the Windsor level against the Sackville level.

A regression analysis was performed on the datasets which returned a root mean square error (RMSE) of 0.15 (presented in Diagram 2). A large component of the scatter of the results around the line of best fit in Diagram 2 can be attributed to the variation in the size of each event on the Colo River and the influence of flows at the Colo River junction on levels upstream at Sackville. To assist in understanding the influence of the Colo River flows, the data points have been coloured based on their corresponding Colo River flow.

To potentially improve the estimates, a regression was undertaken with multiple predictors, which also considered the peak flows on the Colo River. This produced a lower RMSE of 0.09. Diagram 3 shows how consideration of the flows on the Colo River improves the prediction of levels at Sackville. The levels produced from the joint Windsor and Colo River predictors were adopted for the AMS as they generally produced smaller residuals (errors) to the observed values (Table 6).

Table 5: Sackville flood record

Date	Peak Flood Level (mAHD)	Source	Adopted for AMS
06/1864	10.5	5	N
23/06/1867	15.47	5	Y
22/06/1949	8.4	1	Y
07/1952	8.25	2	Y
11/02/1956	9.4	1	Y
20/11/1961	10.4	1	Y
30/04/1963	4.6	1	N
06/06/1963	4.44	1	N
08/05/1963	4.22	1	N
30/09/1963	4.88	1	N
13/06/1964	10.97	1	Y
08/08/1967	4.8	1	N
16/11/1969	5.56	1	N
06/1975	7.45	2	Y
04/03/1977	5.4	1	N
21/03/1978	10.71	1	Y
02/06/1978	5.6	1	N
29/07/1984	3.0	1	N
07/11/1984	2.4	1	N
08/08/1986	8.16	3	Y
30/04/1988	8.55	4	Y
05/04/1989	5.36	1	N
04/02/1990	4.59	1	N
21/04/1990	5.65	1	N
01/08/1990	9.97	1	Y
09/02/2020	5.78	4	Y*
24/03/2021	9.70	4	Y
09/03/2022	10.68	4	N
05/07/2022	10.87	4	Y

1> Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)
2> Calculated in this study with joint probability assessment of Colo and Hawkesbury flows.
3> BoM gauge at Sackville Ferry
4> MHL Gauge Data at Sackville
5> Other Flood Data translated to the Gauge
*2020 event adopted for the pre-dam AMS at Sackville.

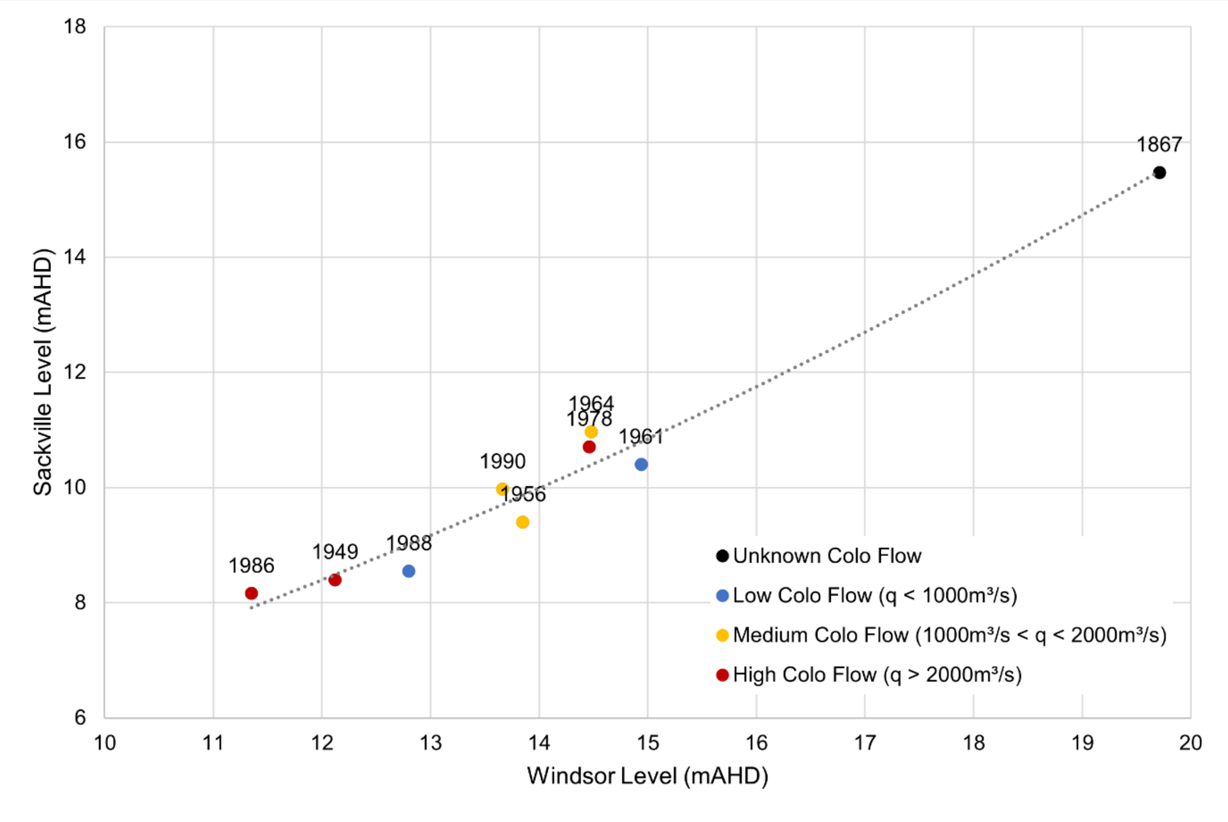


Diagram 2: Windsor and Sackville Level Correlation

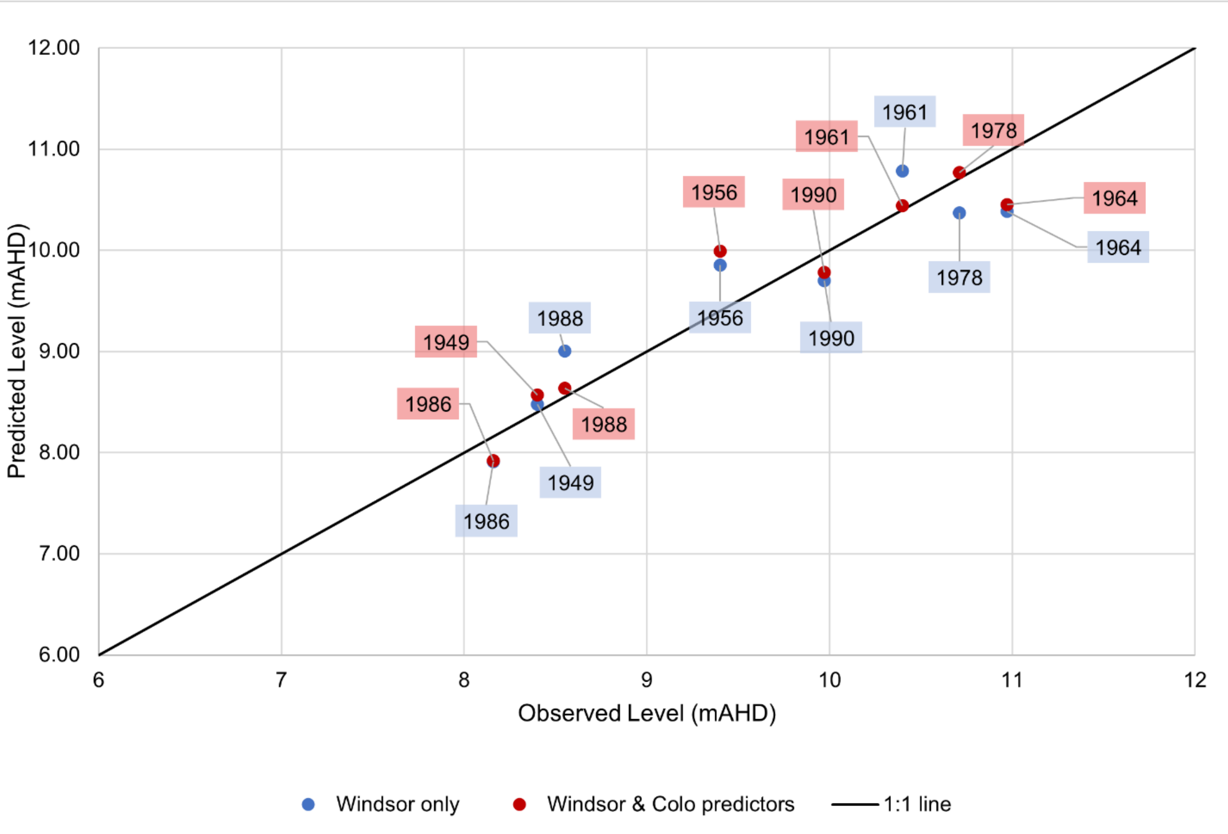


Diagram 3: Sackville Regression Comparing Regression Models

Table 6: Residual Error from regression models.

Year	Observed (m AHD)	Windsor only		Windsor & Colo predictors	
		Level (m AHD)	Residual Error (m)	Level (m AHD)	Residual Error (m)
1949	8.4	8.48	-0.08	8.57	-0.17
1956	9.4	9.85	-0.45	9.99	-0.59
1961	10.4	10.79	-0.39	10.44	-0.04
1964	10.97	10.39	0.58	10.45	0.52
1978	10.71	10.37	0.34	10.77	-0.06
1986	8.16	7.91	0.25	7.92	0.24
1988	8.55	9.00	-0.45	8.64	-0.09
1990	9.97	9.70	0.27	9.78	0.19
	RMSE		0.15		0.09

Table 7 presents levels at Sackville using the two fits. These two floods are the lowest in the historical record adopted for the AMS, and therefore are much less critical than the larger events at Sackville.

Table 7: Sackville Level Estimates for Missing Events

Missing Year	Level at Sackville using Windsor predictor only (m AHD)	Level at Sackville using Windsor and Colo River predictors (m AHD)
1952	8.21	8.25
1975	7.80	7.45

1867 Event Estimation

The 1867 flood was also estimated at the Sackville gauge. A historical flood mark exists upstream of the gauge at the church located at 597 Tizzana Road, which was built to replace a previous St Thomas Anglican Church building destroyed in the 1867 flood. Maximum levels were extracted at the location of the flood mark and the gauge at Sackville from TUFLOW results for the 1 in 200 AEP and 1 in 500 AEP events, which are most similar in magnitude. These results were used to extract a flood slope between the mark and the gauge to generate a level estimate for the 1867 event. Adopted levels for 1867 are shown in Table 8.

Diagram 4: Flood mark for the 1867 Event at Tizzana Road Church Obelisk

Table 8: Sackville Level Estimates for 1867

	Pre-dam (m AHD)	Post-dam (m AHD)
Sackville Ferry Gauge	15.47	14.81

1864 Event Estimation

The 1864 level is very similar to the 1961 event at Windsor after adjustment for the dam is considered. For the 1864 event, the level at Windsor is quite reliable while the level at Sackville is not. As a result, the same flood surface slope between Windsor and Sackville Ferry was assumed for the two events, placing the 1864 estimate 0.1m above the 1961 level of 10.4 m AHD. These two events rank 2 and 3 in the 230-year flood record at Windsor yet are exceeded at Sackville by the 1978 and 1964 events. Generating a plotting position for this event at Sackville is difficult as it sits in the first half (110 years) of the record, and it is unknown if other floods in this period would have produced higher levels at Sackville like the 1978 and 1964 events did. Due to this uncertainty, it has not been plotted on Figure 35. Levels for this event at the gauge are presented in Table 9.

Table 9: Sackville Level Estimates for 1864

	Pre-dam (m AHD)	Post-dam (m AHD)
Sackville Ferry Gauge	10.5	10.3

2.5.3.2. Colo Junction (Lower Portland) flood record

The flood record at Colo Junction (Lower Portland) extends back to 1949, with a daily record from 1961 to 1989, and a continuous record from 1989 to the present. Table 10 presents the events which were originally considered for the AMS and whether they were adopted.

Table 10: Colo Junction (Lower Portland) flood record

Date	Peak Flood Level (mAHD)	Source	Adopted for AMS
22/06/1949	6.8	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	Y
11/02/1956	7.0	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	Y
20/11/1961	7.77	BoM data (7.21m) + assumed gauge zero of 0.564m AHD	Y
30/04/1963	3.4	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N*
08/05/1963	2.7	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
13/06/1964	8.28	BoM data (7.72m) + assumed gauge zero of 0.564m AHD	Y
08/08/1967	3.5	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N*
04/03/1977	4.6	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N*
21/03/1978	8.31	BoM data + assumed gauge zero of 0.564m AHD	Y
02/06/1978	3.7	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
08/1986	6.06	Hawkesbury Nepean Valley Regional Flood Study	Y
30/04/1988	5.87	Lower Portland Staff Gauge	Y
27/04/1989	4.55	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N*
21/04/1990	5.1	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
01/08/1990	7.46	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	Y
10/02/1992	5.77	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N*
10/02/2020	4.73	Colo Junction Gauge record, MHL	N
24/03/2021	7.87	Colo Junction Gauge record, MHL	Y
09/03/2022	8.67	Colo Junction Gauge record, MHL	N
05/07/2022	8.99	Colo Junction Gauge record, MHL	Y

*although these are the largest floods in the year, floods below the 1988 level were not adopted as there is uncertainty in the completeness of the record below this level.

2.5.3.3. Webbs Creek (Wisemans Ferry) Flood Record

The flood record at Wisemans Ferry extends back to 1867, with daily records at the Webbs Creek Ferry gauge from 1964 to 1981 and a continuous record from 1981 to the present.

Table 11 contains the historical flood levels considered for the AMS and whether they were adopted. A low-level censor of 2.84 was used, as per Table 4. Discussion of some floods is provided below.

Table 11: Wisemans Ferry flood record

Date	Peak Flood Level (mAHD)	Source	Adopted for AMS
06/1867	8.6	Sydney Morning Herald, June 1 1889 (Wisemans Wharf)	Y
05/1889	6.8	Sydney Morning Herald, June 1 1889 (Wisemans Wharf)	Y
22/06/1949	5.57	Investigation of Major Flood Events at Wisemans Ferry (T. Young). Note AWACs is 3.1mAHD.	Y
11/02/1956	3.71	Calculated based on surrounding gauges and joint probability	Y
20/11/1961	3.20	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
11/1961	3.75	Webb McKeown Associates 1996 Study (Wisemans Wharf)	Y
13/06/1964	4.2	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	Y
21/03/1978	4.8	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	Y
21/03/1978	4.24	WMA – at Wisemans Ferry Wharf	N
08/1986	3.08	MHL record. Regression produced value of 3.05m AHD	Y
30/04/1988	2.84	BoM Gauge	Y
06/07/1988	2.78	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
05/04/1989	2.14	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
04/02/1990	1.97	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
21/04/1990	2.58	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	N
01/08/1990	4.30	Table A1 Lower Hawkesbury River Recorded Peak Heights (AWACS)	Y
10/02/2020	2.39	Webbs Creek Gauge record, MHL	N
24/03/2021	4.36	Webbs Creek Gauge record, MHL	Y
09/03/2022	5.18	Webbs Creek Gauge record, MHL	N
06/07/2022	5.78	Webbs Creek Gauge record, MHL	Y

1867 and 1889 Events

The May 1889 flood was a significant event at Wisemans Ferry, despite being less than 1 in 10 AEP event at Windsor. It was possibly the highest flood on record at St Albans in the Macdonald River Valley, with one source claiming it was higher there than the 1949 flood (Erskine, 1986). The Sydney Morning Herald on 1 June 1889 reported the level at Wisemans Ferry as 6ft below that of the June 1867 flood and 19ft above the high-water mark. This description equates to peak flood levels at Wisemans Ferry of approximately 6.8m AHD (1889) and 8.6m AHD (1867).

1949 Event

The June 1949 flood was a significant event at Wisemans Ferry, despite being about a 1 in 10 AEP event at Windsor. The level of this flood was surveyed as part of the *Investigation of Major Flood Events at Wisemans Ferry* (Young, 1984) at 5.57m AHD. This differs from a much lower level reported in AWACS (1997) (3.1m AHD).

For this analysis, the higher level of 5.57m AHD has been adopted with consideration of the following:

- At St Albans on the Macdonald River, this flood was estimated to be close to a 1 in 100 AEP flood event (Webb, McKeown & Associates, 2004). This coincident severe flooding in the Macdonald River lends support to the higher reading at Wisemans Ferry downstream (similarly to the 1889 flood).
- Young's level is corroborated by contemporaneous reports in newspapers as well as field interviews conducted with residents at Wisemans Ferry in 1980 (see Young, 1984).

1956 and 1986 Events

The 1956 and 1986 flood peaks have been estimated using a simple linear regression with Colo Junction (Lower Portland) levels and Webbs Creek Ferry levels. This linear regression only used 1978, 1964, 1990, 1961 and 1988. The 1949 event was not used because of the unusually high influence of the Macdonald River inflows. The regression is shown in Diagram 5 in blue. Subsequent to this analysis, a 1986 level of 3.08m AHD was received from both BoM and MHL, while the regression produced a level of 3.05m AHD.

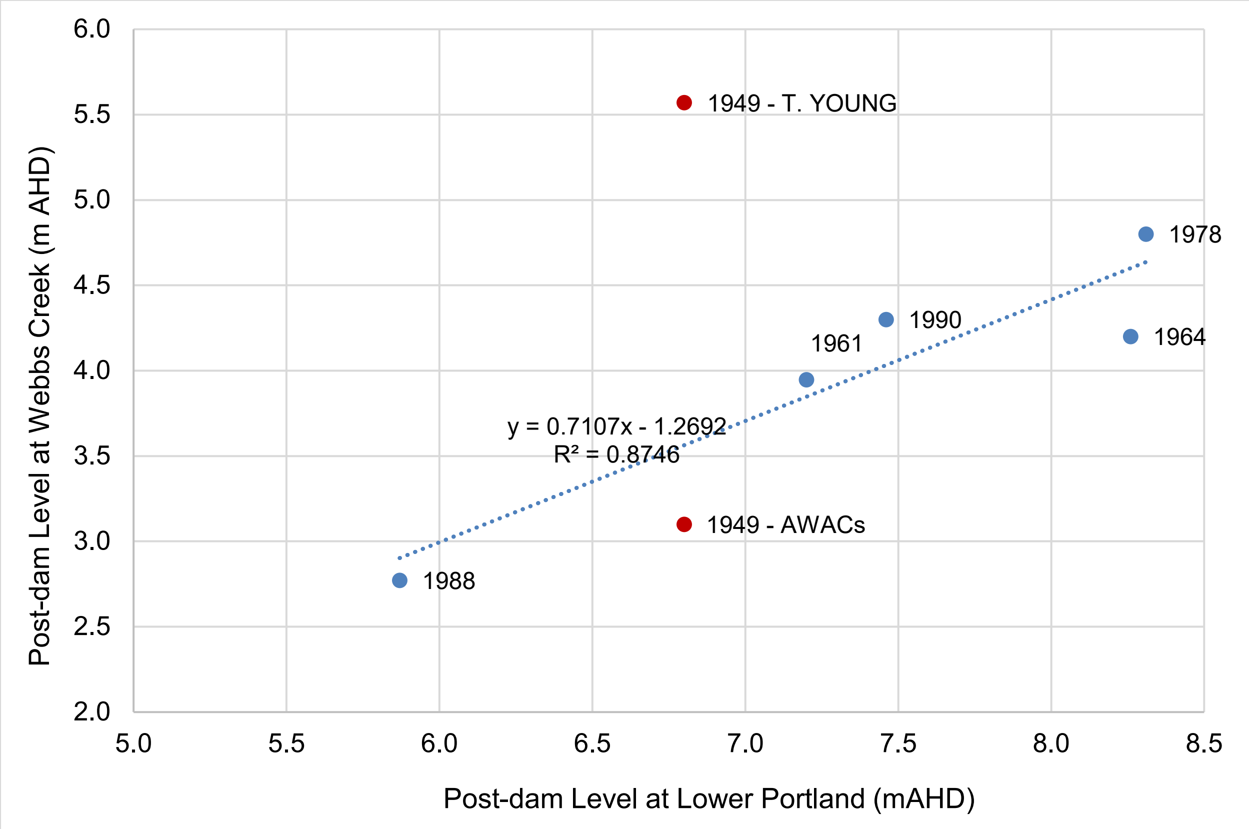


Diagram 5: Linear Regression between Colo Junction (Lower Portland) and Webbs Creek Ferry

Homogenous Flood Record

The Wisemans Ferry flood record is a mixture of records at the Webbs Creek gauge and the Wisemans Ferry Wharf. Webbs Creek has been adopted as the gauge for stage frequency analysis. It is located approximately 1.9 km upstream of the Wisemans Ferry wharf along the river. The first part of the assessment was to adjust any records from Wisemans Ferry Wharf so that they were representative of the upstream location of Webbs Creek Ferry. The TUFLOW model (**Technical Volume 3**) found a significant slope between the two sites. This slope is also evident from the peak levels recorded at MHL gauges during recent floods:

- the March 2021 flood recorded 4.36m AHD at Webbs Creek Ferry and 3.91m AHD at Wisemans Ferry Public Wharf
- the March 2022 flood recorded 5.18m AHD at Webbs Creek Ferry and 4.73m AHD at Wisemans Ferry Public Wharf.

Table 12 presents the homogenous flood record for Webbs Creek with adjustments for events observed downstream. The adjustments were made by interpolating the slope from modelled TUFLOW results for similar sized historical events and representative events up to 1 in 500 AEP.

Table 12: Webbs Creek flood record with adjustments

Year	Webbs Creek Level (m AHD)	Location of original record	Note
1867	9.14	Wisemans Ferry Wharf	Adjusted to be representative at Webbs Creek
1889	7.28	Wisemans Ferry Wharf	Adjusted to be representative at Webbs Creek
1949	5.57	Webbs Creek	Not adjusted
1956	3.71	Webbs Creek	Not adjusted
1961	3.95	Wisemans Ferry Wharf	Adjusted to be representative at Webbs Creek
1964	4.20	Webbs Creek	Not adjusted
1978	4.80	Webbs Creek	Not adjusted
1986	3.08	Webbs Creek	Not adjusted
1988	2.84	Webbs Creek	Not adjusted
1990	4.30	Webbs Creek	Not adjusted
2021	4.36	Webbs Creek	Not adjusted
2022	5.78	Webbs Creek	Not adjusted

2.6. Adopted Lower Hawkesbury Plotting Positions

Figure 35, Figure 36 and Figure 37 present the historical records and calculated plotting positions for Sackville, Colo Junction (Lower Portland) and Webbs Creek (Wisemans Ferry), respectively.

Table 13, Table 14 and Table 15 show the stage frequency results at Sackville Ferry, Colo Junction and Webbs Creek. These results are presented as pre-dam and post-dam levels with AEP estimates for each of the historical events considered. The note column indicates whether the record was observed in the dam case, for example, the 1867 event occurred prior to the construction of Warragamba Dam and in the pre-dam note column is shown as observed, but is adjusted to represent the post-dam case. Some results are calculated based on relationships to other gauges. Results for Wallacia are presented in the Wallacia Joint Probability report (**Technical Volume 6**).

Accurate confidence limits cannot be provided on stage flood frequency results. The reliability of these estimates is fair as we have a good record of the largest events in the last 113 years (at Sackville) and 73 years at Colo Junction and Webbs Creek. Larger events prior to the record have also been included at Sackville and Webbs Creek.

Table 13: Flood frequency analysis results - Stage pre-dam and post-dam at Sackville Ferry

Year	Level (m AHD)		Plotting Position (1 in X AEP)	
	Pre-dam	Post-dam	Pre-dam	Post-dam
1864	10.50	10.30	*	*
1867	15.47	14.81	380	380
1949	8.40	8.06	10	10
1952	8.25	7.93	9	9
1956	9.40	9.17	14	14
1961	10.93	10.40	37	28
1964	11.18	10.97	109	109
1975	7.59	7.45	*	9
1978	10.87	10.71	28	37
1986	8.25	8.16	9	11
1988	8.77	8.55	11	13
1990	10.19	9.97	19	19
2020	9.02	5.78	13	*
2021	9.82	9.70	16	16
2022	10.94	10.87	56	56
Notes	*Plotting positions not adopted for 1864, 1975 pre-dam and 2020 post-dam			
	Adjusted for Warragamba Dam Impact			
	Observed at Gauge			
	Calculated (See 2.5.3.1)			

Table 14: Flood frequency analysis results - Stage pre-dam and post-dam at Colo Junction (Lower Portland)

Year	Level (m AHD)		Plotting Position (1 in X AEP)	
	Pre-dam	Post-dam	Pre-dam	Post-dam
1949	6.80	6.59	14	14
1956	7.00	6.77	16	16
1961	8.35	7.77	28	22
1964	8.60	8.28	54	37
1978	8.56	8.31	37	54
1986	6.27	6.06	13	13
1988	6.03	5.87	11	11
1990	7.66	7.46	19	19
2020	5.35	4.66	*	*
2021	7.95	7.87	22	28
2022	9.02	8.99	104	104
Notes	*Plotting positions not adopted for 2020			
	Adjusted for Warragamba Dam Impact			
	Observed at Gauge			

Table 15: Flood frequency analysis results - Stage pre-dam and post-dam at Webbs Creek (Wisemans Ferry)

Year	Level (m AHD)		Plotting Position (1 in X AEP)	
	Pre-dam	Post-dam	Pre-dam	Post-dam
1867	9.14	8.96	380	380
1889	7.28	7.26	138	138
1949	5.57	5.45	54	54
1956	3.71	3.62	14	14
1961	4.46	3.95	27	16
1964	4.36	4.20	16	18
1978	4.91	4.80	36	36
1986	3.18	3.08	12	12
1988	2.96	2.84	11	11
1990	4.38	4.30	18	22
2020	2.84	2.39	*	*
2021	4.41	4.36	22	27
2022	5.80	5.78	85	85
Notes	*Plotting positions not adopted for 2020			
	Adjusted for Warragamba Dam Impact			
	Observed at Gauge			
	Calculated (See 2.5.3.3)			

3. HYDRAULIC MODELLING

The Monte Carlo framework relies on a fast hydraulic model that can run 20,000 simulations quickly. This allows events to be selected from the Monte Carlo results for detailed two-dimensional model runs. While the 2D model can take days to run, the 1D model runs in seconds. It is important that the two models produce similar results.

3.1. RUBICON Model

The computationally faster one-dimensional (1D) RUBICON hydrodynamic model was adjusted to incorporate the insights gained from TUFLOW modelling completed as a part of the Hawkesbury-Nepean River Flood Study (**Technical Volume 3**). The two-dimensional (2D) TUFLOW model was found to better represent floodplain storage and the constricted flow behaviour downstream of Windsor.

3.2. TUFLOW Model Results

The TUFLOW model was initially run for a suite of 8 calibration events with 7 of these events (excluding 2020) used to improve the calibration of the RUBICON model[1]. Two-dimensional models such as TUFLOW directly measure the storage within the floodplain while 1D models such as RUBICON represent the storage through a series of cross sections and stage storage tables. Until LiDAR became available it was very difficult to validate if 1D models accurately measured the storage. Where a 2D model is available it is possible to compare storage being used in different flood events and storage in different reaches. Direct comparisons of stage and flow hydrographs throughout the system can demonstrate where the 1D RUBICON model is underestimating storage, and modifications may be made to minimise the difference.

3.3. Comparisons to TUFLOW

3.3.1. Storage

In comparing the initial RUBICON results to that of the provided TUFLOW results, a noticeable difference in storage capacity around the Windsor floodplain was identified. This was also confirmed by comparing the storage in the RUBICON model with the latest aerial LiDAR Survey (ALS). While most of the storage in the RUBICON model is accounted for in the cross sections, the model included backwater storages for the floodplain areas of Upper Rickabys Creek, Upper Currency Creek, Oakville and Castlereagh. The comparison with the TUFLOW model indicated that additional storage was required for Cattai Creek, Little Cattai Creek, South Creek, Eastern Creek, Currency Creek, Bushells Lagoon, Gronos Point, Pitt Town and the Richmond lowlands. The RUBICON model was updated to accurately reproduce the floodplain storage. This additional storage required a slight recalibration which resulted in a minor increase in Manning's n for the main river downstream of Windsor. In addition to this, the storage factor for the cross sections

[1] The 2020 event was not of a sufficient size to generate additional benefit in calibration and model improvements for RUBICON. Sensitivity testing with the March 2021 event was carried out and is documented in **Appendix B**.

along the Colo River was increased. This was to reproduce the catchment response from the TUFLOW model which included a greater length of the Colo River.

3.4. Recalibration Strategy

The model was recalibrated to the historical events using inflows from **Technical Volume 2**. The model was fitted to stage and flow hydrographs throughout the system, first at Wallacia (stage only), and then downstream to key gauges at Victoria Bridge at Penrith, Yarramundi, North Richmond, Windsor Bridge (stage only), Sackville, Colo Junction, and Webbs Creek Ferry. The recalibration included additional storage at Wallacia and Camden and minor changes to Manning's n in the Hawkesbury-Nepean River. These are documented in Table 16 and were generally increases in the riparian area. These newly adopted Manning's n values were in line with those used in the TUFLOW model.

Table 16: Change in Manning's n values in the Hawkesbury Nepean River

Location	Change	New Manning's n
Cattai Creek to Hopefarm	Decrease of up to 0.005	0.029 – 0.034
Little Cattai Creek	Increase of up to 0.008	0.037 – 0.042
Windsor to Sackville	Increase by 0.005 above ~10 mAHD	0.029 - 0.042
Lower Portland	Increase of up to 0.004	0.037
Lower Portland to Webbs Creek	Decrease of up to 0.001	0.029
Webbs Creek to the Ocean boundary	Increase of up to 0.002	0.022

The final calibration for the historical events is shown in Figure 4 to Figure 23.

During the preparation of this report, large floods occurred on the Hawkesbury-Nepean Valley in March 2021, March 2022 and July 2022. The 2021 event was used to verify the RUBICON model, the analysis is detailed in **Appendix B**. Analysis of the 2022 floods has formed separate reports (**Technical Volumes 9** and **10**; Infrastructure NSW, 2023). These demonstrate that the TUFLOW model provides a good fit to the observed data for the March and July 2022 events. Due to time constraints, the 2022 events were not verified with the updated RUBICON model.

3.5. Results

3.5.1. Calibration floods

The following section details the calibration of the RUBICON model for selected historical flood events. The flows estimated in the RUBICON model are compared to the TUFLOW results and the available observed data.

3.5.1.1. Nash-Sutcliffe Efficiency

The hydrographs output from the TUFLOW and RUBICON models were compared using the Nash–Sutcliffe Efficiency (NSE). The NSE was calculated on flows greater than 500 m³/s and match for low flows was not calculated.

The results of the comparison are presented in the following sections and summarised in Table 17 below. An NSE of 1 is a perfect match. While it depends on the timestep, broadly an NSE > 0.7 is considered Good and an NSE > 0.8 is Very Good.

Except for the 1961 event, which as discussed below is not a like for like comparison, the scores are generally above 0.95. The matches between the two models at Penrith's M4 Bridge, Yarramundi and Sackville Ferry are excellent. The match at Wallacia is also very good.

Table 17: Nash–Sutcliffe model efficiency coefficient for RUBICON to TUFLOW

	1961	1964	1975	1978	1986	1988	1990
Wallacia Weir	0.725	0.986	0.875	0.886	0.952	0.957	0.942
M4 Bridge	0.328	0.999	0.996	0.992	0.998	0.997	0.998
Yarramundi Bridge	0.387	0.999	0.997	0.994	0.995	0.996	0.997
Sackville Ferry	0.876	0.996	0.973	0.980	0.985	0.993	0.990
Average	**0.579**	**0.995**	**0.960**	**0.963**	**0.983**	**0.986**	**0.982**

3.5.1.2. November 1961

Table 18 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 18: November 1961 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	**Gauged Data**	**TUFLOW**	**RUBICON**	**To Observed**	**To TUFLOW**
Wallacia Weir	41.35	41.44	41.73	0.38	0.29
Penrith (Victoria Bridge)	23.89	24.04	23.98	0.09	-0.06
Yarramundi Bridge	-	17.47	17.12	-	-0.35
North Richmond Bridge	16.64	16.08	16.16	-0.48	0.08
Windsor Bridge	14.9	15.4	15.38	0.48	-0.02
Sackville Ferry	10.4	11.63	10.95	0.55	-0.68
Colo Junction	7.77	8.42	8.06	0.29	-0.36
Webbs Creek	3.95	4.21	4.04	0.09	-0.17

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 4.

The modelled RUBICON results achieve an average catchment wide NSE of approximately 0.579 compared to the TUFLOW modelled hydrographs. For the 1961 event, the TUFLOW model placed the Nepean inflow at Wallacia instead of being input further upstream. For this reason, this event is not a like for like comparison, unlike the other events. As a result, the level hydrographs indicated by the TUFLOW model appear noticeably different in comparison to the RUBICON results and observed data (refer to Figure 5). Calibration to the observed data was achieved by changing the selected temporal pattern for the Cataract Dam catchment as well as adjusting initial and continuing losses upstream of Wallacia Weir. Given the limited pluviograph information for this event, it is not possible to further improve the calibration.

3.5.1.3. June 1964

Table 19 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 19: June 1964 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	**Gauged Data**	**TUFLOW**	**RUBICON**	**To Observed**	**To TUFLOW**
Wallacia Weir	43.93	42.44	41.96	-1.97	-0.48
Penrith (Victoria Bridge)	23.74	23.83	23.78	0.04	-0.05
Yarramundi Bridge	-	17.2	17.03	-	-0.17
North Richmond Bridge	15.99	15.88	15.84	-0.15	-0.04
Windsor Bridge	14.57	14.46	14.45	-0.12	-0.01
Sackville Ferry	10.97	10.84	10.32	-0.65	-0.52
Colo Junction	8.28	7.96	7.8	-0.48	-0.16
Webbs Creek	4.2	4.17	4.2	0	0.03

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 7.

Due to the limited observed stage hydrograph data, the modelled TUFLOW results were used as the basis of calibration for the 1964 event. An average NSE of approximately 0.995 is achieved between the RUBICON and TUFLOW modelled results with the shape of the stage hydrographs being mostly identical across the study area. However, an underestimation in peak flood levels at Sackville and Lower Portland can be observed (refer to Figure 8). This is due to the inability to find a parameter set that achieves a match between RUBICON and TUFLOW in both large and small events downstream of Sackville, with larger events generally underestimating peak levels and smaller events overestimating.

3.5.1.4. June 1975

Table 20 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 20: June 1975 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	Gauged Data	TUFLOW	RUBICON	To Observed	To TUFLOW
Wallacia Weir	38.79	39.39	39.08	0.29	-0.31
Penrith (Victoria Bridge)	21.49	21.68	21.78	0.29	0.1
Yarramundi Bridge	-	15.92	15.16	-	-0.76
North Richmond Bridge	-	14.93	14.26	-	-0.67
Windsor Bridge	11.2	11.42	11.72	0.52	0.3
Sackville Ferry	7.45	7.18	7.39	-0.06	0.21
Colo Junction	-	4.55	5.01	-	0.46
Webbs Creek	-	2.22	2.31	-	0.09

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 10.

Across the four locations, an average NSE of 0.961 was achieved between the RUBICON and TUFLOW models. Like the 1964 event, the shape of the stage hydrographs is mostly identical across the study area between the two hydraulic models (Figure 11). However, an underestimation of the peak level at North Richmond Bridge is apparent compared to the TUFLOW model, and a general divergence between Penrith and North Richmond (see Figure 10).

3.5.1.5. March 1978

Table 21 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 21: March 1978 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	Gauged Data	TUFLOW	RUBICON	To Observed	To TUFLOW
Wallacia Weir	42.24	42.19	41.92	-0.32	-0.27
Penrith (Victoria Bridge)	23.35	23.51	23.5	0.15	-0.01
Yarramundi Bridge	-	16.92	16.71	-	-0.21
North Richmond Bridge	15.59	15.66	15.69	0.1	0.03
Windsor Bridge	14.46	14.4	14.58	0.12	0.18
Sackville Ferry	10.71	10.99	10.49	-0.22	-0.5
Colo Junction	8.31	8.56	8.25	-0.06	-0.31
Webbs Creek	4.8	4.53	4.39	-0.41	-0.14

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 13.

An average NSE of 0.96 is achieved between the RUBICON and TUFLOW model results. The RUBICON model is also effective in matching the observed data, albeit to a lesser degree at Wallacia. While the difference in peak level between the RUBICON results and the gauged data is less than 1%, there are differences observed in the shape of the stage hydrograph. The modelled results match the falling limb of the gauged data however, the hydrograph rises earlier than the TUFLOW model at Wallacia (Figure 14). The 1978 event is one of the largest historical events alongside the 1964 event. As a result, and as detailed in the 1964 calibration (refer to Section 3.5.1.3), the modelled 1978 results underestimate the peak flood levels downstream of Sackville in comparison to the TUFLOW model.

3.5.1.6. August 1986

Table 22 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 22: August 1986 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	**Gauged Data**	**TUFLOW**	**RUBICON**	**To Observed**	**To TUFLOW**
Wallacia Weir	35.47	35.51	35.8	0.33	0.29
Penrith (Victoria Bridge)	20.06	19.77	19.76	-0.3	-0.01
Yarramundi Bridge	-	13.87	13.32	-	-0.55
North Richmond Bridge	13.02	13.25	12.8	-0.22	-0.45
Windsor Bridge	11.35	11.29	11.52	0.17	0.23
Sackville Ferry	8.16	7.71	7.66	-0.5	-0.05
Colo Junction	6.06	5.93	6.26	0.2	0.33
Webbs Creek	3.08	2.91	3.05	-0.03	0.14

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 16.

Both the shape and peak of the stage hydrographs are near identical between the RUBICON and TUFLOW model results and achieve an NSE of 0.98 between flow hydrographs at key locations. Neither model matches the observed data particularly well at Sackville, Lower Portland or Webbs Creek (Figure 17).

3.5.1.7. April/May 1988

Table 23 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 23: April/May 1988 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	**Gauged Data**	**TUFLOW**	**RUBICON**	**To Observed**	**To TUFLOW**
Wallacia Weir	40.81	40.91	40.18	-0.63	-0.73
Penrith (Victoria Bridge)	22.62	22.3	22.52	-0.1	0.22
Yarramundi Bridge	-	16.05	15.37	-	-0.68
North Richmond Bridge	14.68	15.06	14.48	-0.2	-0.58
Windsor Bridge	12.8	12.34	12.46	-0.34	0.12
Sackville Ferry	8.55	8.22	8.26	-0.29	0.04
Colo Junction	5.87	5.24	5.71	-0.16	0.47
Webbs Creek	2.84	2.36	2.55	-0.29	0.19

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 19.

The calibrated RUBICON model is effective at providing a close match to the modelled TUFLOW results with an average NSE value of 0.98. RUBICON is slightly higher than TUFLOW near Lower Portland and is a better match to the observed peak heights in this area (Figure 20).

3.5.1.8. August 1990

Table 24 compares peak flood levels for the observed, RUBICON and TUFLOW results.

Table 24: August 1990 model calibration results and available flood levels

	Level (mAHD)			RUBICON Difference (m)	
Location	**Gauged Data**	**TUFLOW**	**RUBICON**	**To Observed**	**To TUFLOW**
Wallacia Weir	39.21	38.82	39.21	0.00	0.39
Penrith (Victoria Bridge)	23.44	23.51	23.53	0.09	0.02
Yarramundi Bridge	-	16.86	16.47	-	-0.39
North Richmond Bridge	15.39	15.64	15.38	-0.01	-0.26
Windsor Bridge	13.46	13.44	13.47	0.01	0.03
Sackville Ferry	9.97	9.74	9.44	-0.53	-0.3
Colo Junction	7.46	7.1	7.08	-0.38	-0.02
Webbs Creek	4.3	3.79	3.76	-0.54	-0.03

The profile of the event along the modelled stretch of the Hawkesbury-Nepean River is shown in Figure 22.

The calibrated RUBICON model is effective at providing a close match to the modelled TUFLOW results with an average NSE value of 0.98. Additionally, the calibrated RUBICON model closely matches the gauged data. Similar to the 1964 event, peak flood levels are slightly underestimated downstream of Sackville (Figure 23).

3.5.1.9. Recalibration Summary

The recalibration has produced a close agreement between the RUBICON model and the TUFLOW model and a good match to the observed levels. This means that the RUBICON model is suitable for the selection of representative events to be run for the design AEP flood modelling within the TUFLOW model.

4. MONTE CARLO FRAMEWORK

4.1. Overview

The Monte Carlo framework was established in the Regional Flood Study to model flood events based on randomly sampling each variable from within the range of possible inputs:

- rainfall intensity
- spatial pattern of rainfall – where in the catchment rain falls
- temporal pattern of rainfall – when in the event rain falls
- initial loss – rain 'lost' at the beginning of an event through infiltration into the soil
- pre-burst rainfall – rain that occurs before the most intense burst of the storm
- dam drawdown – the level of Warragamba Dam before the start of an event
- relative timings of tributary inflows
- downstream ocean levels, including tides.

This section details the modifications made to the Monte Carlo framework for this study. The key changes made are:

- the hydrologic model (except for the Warragamba catchment) was changed from RORB to WBNM (**Technical Volume 2**)
- the timing of Colo River inflows was adjusted based on the timing of observed flood peaks (**Technical Volume 5**)
- the coincident ocean levels were changed to a relationship developed by Baird (**Technical Volume 4**)
- the storage within the RUBICON model was adjusted and the model was recalibrated (Section 3 of this report).

4.2. Hydrological Model

In the current study, the WBNM has been used for the hydrologic modelling of the Hawkesbury-Nepean, replacing the RORB model developed in the 1990s (Webb McKeown and Associates, 1996) for all catchments other than those that drain to Warragamba Dam. WBNM is widely used throughout Australia and particularly NSW.

WBNM simulates a catchment and its tributaries as a series of sub-catchment areas linked together to replicate the rainfall and runoff process through the natural stream network. Input data includes the definition of catchment characteristics including surface area of sub-catchments, proportion of impervious surfaces, stream length adjustments, initial and continuing losses, and temporal and spatial patterns over the catchment.

Key parameters for WBNM represent the response characteristics of a catchment. Typical model parameters include:

- Rainfall Losses: two values, initial and continuing loss, modify the amount of rainfall excess to be routed through the model sub-catchments
- Lag Parameter: this affects the routing of the runoff response to the rainfall excess

- Non Linearity Exponent: adjustment of the non-linearity of catchment response.

The WBNM hydrologic model for the Hawkesbury-Nepean was initially developed by WMAwater as documented in the Hawkesbury-Nepean Hydrologic Model Update Report (WMAwater, 2018). However, the model had not been incorporated into the Monte Carlo Modelling Framework used in the Regional Flood Study. As part of the current flood study, the model was further developed and calibrated by Rhelm to nine historical events (**Technical Volume 2**) and this version is used in the current study.

Diagram 6 compares the sub catchment layout of the RORB hydrologic model from the Regional Flood Study to the Rhelm WBNM hydrologic model used in the current study. The blue sub-catchments indicate where each of the hydrologic models were used in the updated framework. The RORB inflows were used upstream of Warragamba Dam, while WBNM was used in the upper Nepean and downstream of the Dam.. While the original RORB model has 121 sub-catchments covering the Hawkesbury-Nepean catchment (excluding Warragamba), the refined WBNM model has 794 sub-catchments, which allows for a more detailed representation of inflows into the hydraulic model. This has allowed the small tributary inflows to be individually represented between Wallacia and Windsor.

The WBNM model is displayed in Figure 2. Inflow locations into the RUBICON model are illustrated in Figure 3. The RUBICON model was modified to allow for the revised inflow locations developed from the WBNM model.

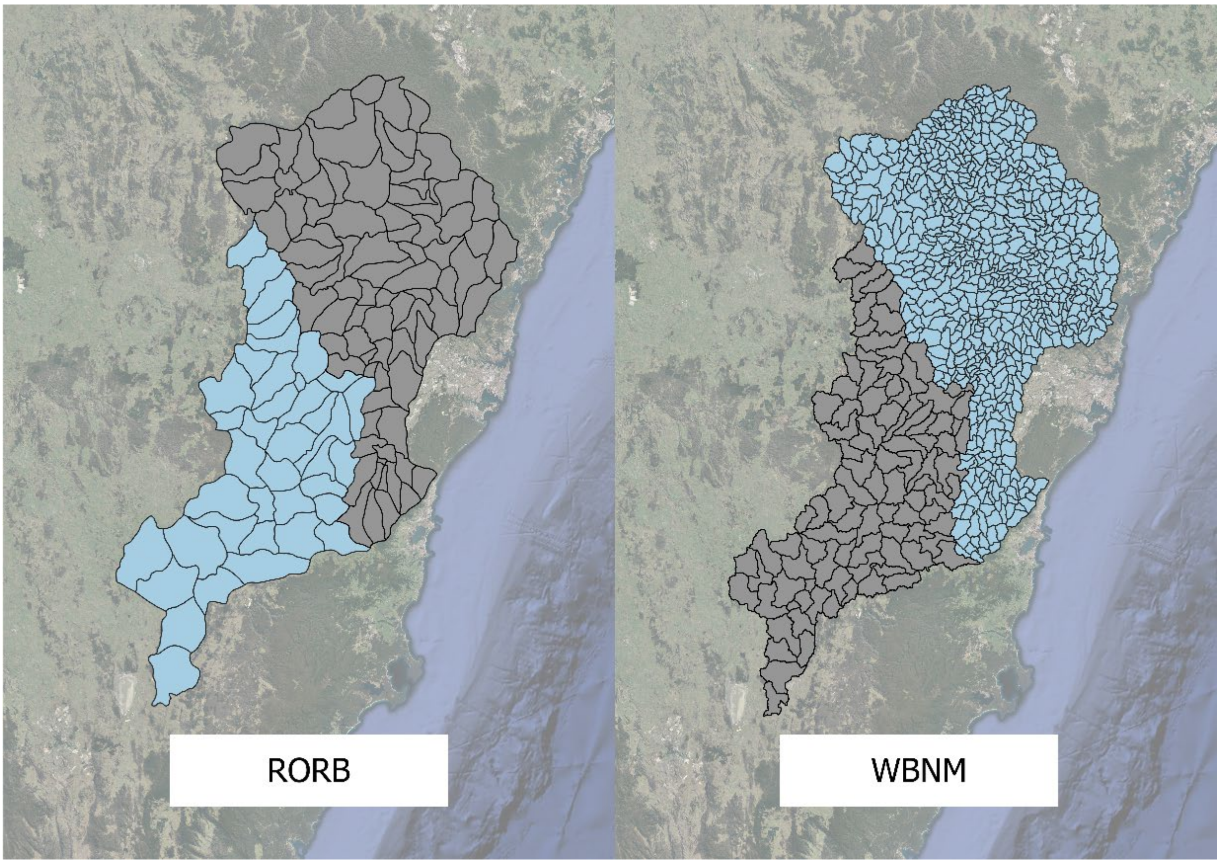


Diagram 6: Subcatchment Refinement in the WBNM hydrologic model.

4.2.1. Timing of tributaries

The coincident timing of tributary inflows can exacerbate flooding.

The timing of tributary inflows was calculated for the following catchments relative to the Warragamba River timing:

- Nepean River
- Grose River and South Creek catchments
- Colo River and lower tributaries.

The timing of the tributary flows is important for evacuation planning, particularly in the Richmond/Windsor area where interactions of local flows can significantly affect rate of rise and reduce evacuation times.

The Regional Flood Study used three-day catchment average rainfalls to determine the timing difference between 50% of the rainfall mass on the Warragamba, Nepean, Grose/South Creek and Colo/downstream tributaries. Generally timing differences were relatively small but some large and small events had quite large differences. The timing differences are applied to the inflow hydrographs from the hydrologic model before the hydraulic model is run.

While this approach worked well on the Warragamba, Nepean and Grose/South Creek systems, it wasn't reproducing the typical timing differences observed between the Hawkesbury River at Windsor and the Upper Colo gauging station. To address this, inflows on the Colo River have been adjusted to match the historical timing distribution between the Colo and Windsor (**Technical Volume 5**) for each of the 20,000 Monte Carlo events. This adjustment is discussed further in Section 4.3.2 as this shift was also used to assist in fitting the model to the flood record at Lower Portland and Wisemans Ferry.

4.2.2. Ocean levels

The Regional Flood Study independently sampled ocean levels from an ocean level frequency distribution. This approach is relatively simplistic and assumed that the annual maximum ocean level would occur concurrently with the annual maximum three-day rainfall event but that the events were not correlated. This approach produced design flood surfaces to the ocean (Broken Bay) but was not representing the interaction of ocean levels and flood events. To address this issue the approach used in rainfall events greater than 1 in 2 AEP was updated to consider the relationship between tidal anomalies and rainfall.

Baird undertook a joint probability assessment of catchment rainfall depths on the Hawkesbury-Nepean and elevated coastal waters using a combination of historical and synthetic events (**Technical Volume 4**). This work developed a relationship between the maximum daily rainfall within the catchment and the maximum coastal residual water level (excluding tide).

The adopted relationship for the Hawkesbury-Nepean is shown in Diagram 7. This was applied to the Monte Carlo framework by assessing the daily catchment average rainfall within the 3-day events and applying the relationship to generate the residual coastal water level stochastically.

Per Baird (**Technical Volume 4**), the residual water level timeseries over the event was generated using the following process, before being applied to a random point in the 18.5-year astronomical tide cycle.

1. For the 24-hour period of peak catchment-averaged rainfall, apply the calculated maximum water level residual from Equation 1.
2. For 36 hours prior to peak rainfall, linearly increase residual from 0 to the peak value.
3. For the 104 hours following peak rainfall, linearly decrease the residual water level to 0.

Equation 1

$$Max\ res.WL = 0.0029\ \times Max\ Daily\ Rainfall + 0.0267 + N(\mu, \sigma^2)$$

Where:

- Max res. WL = maximum coastal residual water level over a 24-hour period (excluding tide)
- Max Daily Rainfall = maximum 24-hour rainfall with catchment of a 0.05-degree geographic grid
- $N(\mu, \sigma^2)$ = standard normal distribution error function with μ = 0, σ = 0.095 m.

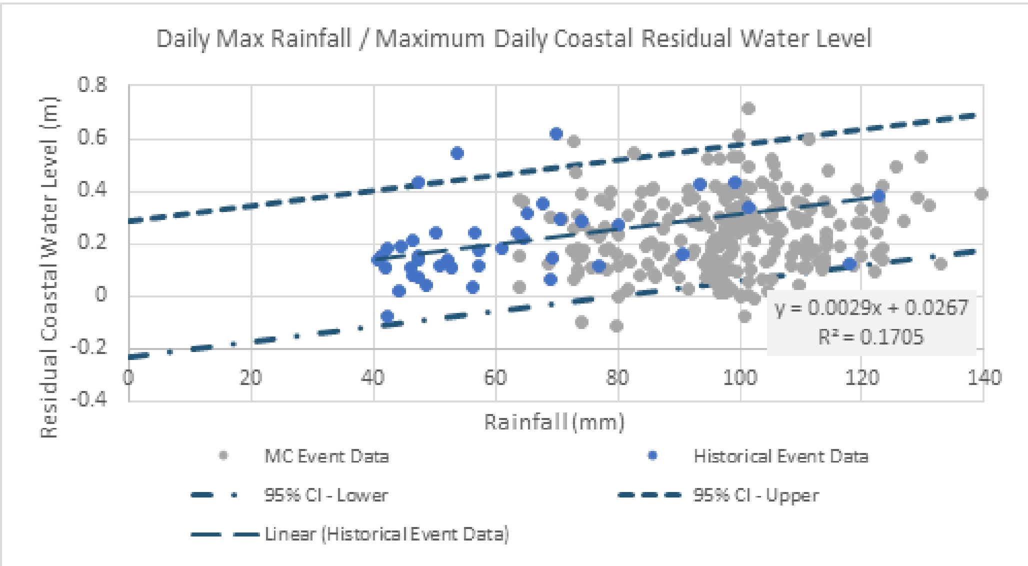


Diagram 7: Cross Plot of Catchment Averaged Maximum Daily Rainfall (in the Hawkesbury-Nepean Catchment) and maximum daily coastal water level residual (Fort Denison) – Historical and Monte Carlo Data (adopted from Baird – Technical Volume 4)

4.2.2.1. Ocean levels for small flows

While the Baird relationship was used for events with an AEP rainfall of 1 in 2 or larger, it was necessary to generate events with elevated ocean levels so the correct stage frequency was produced in the lower reaches. To achieve this, the previous simple approach was retained for the 5000 events in the Monte Carlo smaller than the 1 in 2 AEP rainfall event. While this assumption is technically incorrect as it assumes the highest annual ocean level occurs concurrently with a small but highest annual runoff event, it has the advantage of allowing the ocean level dominated cases to be captured in the framework without having to run separate additional ocean level dominated cases. The distribution sampled for these events replicated that described in the Regional Flood Study (WMAwater, 2019) and is listed in Table 25.

Table 25: Tidal peak distribution used

AEP (1 in x)	Peak tide level (m AHD)
5	1.228
5.52	1.315
10	1.345
20	1.375
50	1.415
100	1.435
200	1.455
500	1.473
1,000	1.484
2,000	1.494
5,000	1.504
10,000	1.51
20,000	1.515
50,000	1.521
100,000	1.525

4.3. Calibration

The Monte Carlo model was calibrated to the updated FFA at Wallacia, Warragamba Dam, Penrith, Windsor, Sackville, Colo Junction and Webbs Creek. The methodology adopted in this study is to approach the best overall fit at all locations. While this works well for larger events, there was some compromise with frequent events. This can be seen on the Penrith and Windsor FFA plots (Figure 33 and Figure 34) where the fit at Penrith is slightly above the expected probabilities for floods more frequent than the 1 in 10 AEP, and slightly low at Windsor for the same flood frequencies. This provides a good compromise between the two locations.

4.3.1. Primary parameters

The primary calibration parameters for the WBNM hydrological modelling were the initial and continuing losses. Where previously the losses in the Monte Carlo framework were spatially varied into areas upstream of Warragamba Dam and downstream of the dam (as described in Section 4.3.1.1), the current study updated the framework to allow for the initial and continuing losses to be spatially varied at a finer resolution. Losses have been assessed for the following areas:

- upstream of Warragamba Dam
- Upper Nepean River to the Nepean/Warragamba Junction
- Nepean Junction to Penrith
- Penrith to Sackville which includes South Creek downstream of the Eastern Creek confluence and the Cattai and Little Cattai catchments
- Grose River catchment
- South Creek catchment
- Eastern Creek catchment
- Colo River catchment
- Other, which includes the area downstream of the Colo Junction.

This has allowed an improved fit to the additional FFA locations (Figure 31 to Figure 37).

4.3.1.1. Initial Loss

The initial losses were drawn from a standardised ARR loss distribution curve (Table 5.3.13 in ARR 2019) with the medians shown in Table 26. Initial losses largely remained the same compared to the 2019 study, with the exception of the Upper Nepean which had the median initial loss reduced to 20mm, and the Colo and downstream catchments which had the median initial losses increased to 45mm to improve the Monte Carlo fit to FFA at the Upper Colo gauge (FFA assessed by Rhelm), Colo Junction, and Webbs Creek/Wisemans Ferry.

Table 26: Initial losses applied in the WBNM model for the Monte Carlo

Median Initial Loss (mm)								
Upstream of Warragamba Dam	Upper Nepean to the Nepean Junction	Nepean Junction to Penrith	Penrith to Sackville	Grose River	South Creek	Eastern Creek	Colo River	Other
30	20	30	30	30	30	30	45	45

4.3.1.2. Continuing Loss

Continuing losses were varied to improve the fit at a few locations. The losses upstream of the Dam were increased at the frequent end of the curve and decreased in the 1 in 20 AEP to 1 in 50 AEP range. This was done to improve the fit to the FFA at Penrith. As with the initial losses, the continuing loss was reduced on the Upper Nepean River, and increased on the Colo River and downstream of Colo Junction (Other). The continuing losses on the Nepean Junction to Penrith region were increased compared to the Regional Flood Study in order to improve the balance between the Penrith and Windsor FFA fits, and to ensure an appropriate stage-frequency curve was achieved at Windsor.

Table 27: Continuing losses applied in the WBNM model

AEP (%)	Continuing Loss (mm/h)								
	Upstream of Warragamba Dam	Upper Nepean to the Nepean Junction	Nepean Junction to Penrith	Penrith to Sackville	Grose River	South Creek	Eastern Creek	Colo River	Other
$1x10^{-5}$	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
0.5	1	1	2.2	2.2	2.2	2.2	2.2	3	3
1	1.2	1	2.4	2.7	2.7	2.7	2.7	3	3
2	1.39	1	2.6	2.7	2.7	2.7	2.7	3	3
5	1.78	1	1.9	2.4	2.4	2.4	2.4	3	3
10	2.5	1	1.5	1.5	1.5	1.5	1.5	3	3
20	2.7	2.7	1.2	1.2	1.2	1.2	1.2	1.7	1.7
50	3	3	1.2	1.2	1.2	1.2	1.2	1.2	1.2

4.3.1.3. Comparison to WBNM Model Calibration Losses

Table 3-23 of **Technical Volume 2** summarises the median loss values used in the calibration of the WBNM model. The Nepean catchments use an average initial loss of 73mm and continuing loss of 2 mm/hr. Downstream of Warragamba Dam to Colo Junction uses an average initial loss of 50mm and continuing loss of 1mm/hr. The Lower Hawkesbury catchments including the Colo River and Macdonald River use an average initial loss of 100mm and continuing loss of 2.2mm/hr.

When comparing these calibration losses to design events it is important to note that the calibration events are all more frequent than a 1 in 50 AEP event at Penrith.

The initial loss values adopted for the WBNM model calibration are higher than those adopted for the Monte Carlo calibration, because the WBNM model calibration includes pre burst in the temporal patterns.

While the continuing loss values are higher between Penrith and Colo Junction for events rarer than 1 in 10 AEP, they are in the reasonable 1-3mm/hr range.

The design losses used for the Monte Carlo calibration are therefore considered to be acceptable with reference to loss values obtained in the calibration of the WBNM model to historical events.

4.3.2. Secondary parameters

4.3.2.1. Spatial patterns

The Regional Flood Study sampled spatial patterns from 125 historic events. For each event a spatial pattern was selected from the closest 20 ranked patterns by catchment average depths, minimising the scaling of frequent event patterns in the framework. The framework was updated to include the spatial patterns from four recent events in the Hawkesbury-Nepean. These were the February 2020, March 2021, March 2022 and July 2022 events.

4.3.2.2. Tributary timing

The timing of flows from downstream catchments such as the Colo River, Webbs Creek and Macdonald River have a significant impact on the fit of the FFA at the Sackville, Colo Junction and Wisemans Ferry gauges on the Hawkesbury River.

The timing of the Colo River inflows was adjusted to match the historic distribution of Colo-Windsor peak differences. Due to backwater effects on the shape of the stage hydrograph from the RUBICON model, the timing difference between the Colo peak flow and Windsor peak level was extracted from the Monte Carlo model, and the Colo inflow timing was then shifted to match the historic distribution. Because the historical events have peak levels in the range 10-15 mAHD at Windsor, the analysis was carried out on Monte Carlo events in this range.

Diagram 8 compares the timing difference between the Colo River peak and Windsor peak for historical events and Monte Carlo events between 10 and 15 mAHD. Diagram 9 compares the modelled timing difference between the Colo River and Penrith peaks. These suggest that the framework is adequately capturing the timing differences observed in the historic events.

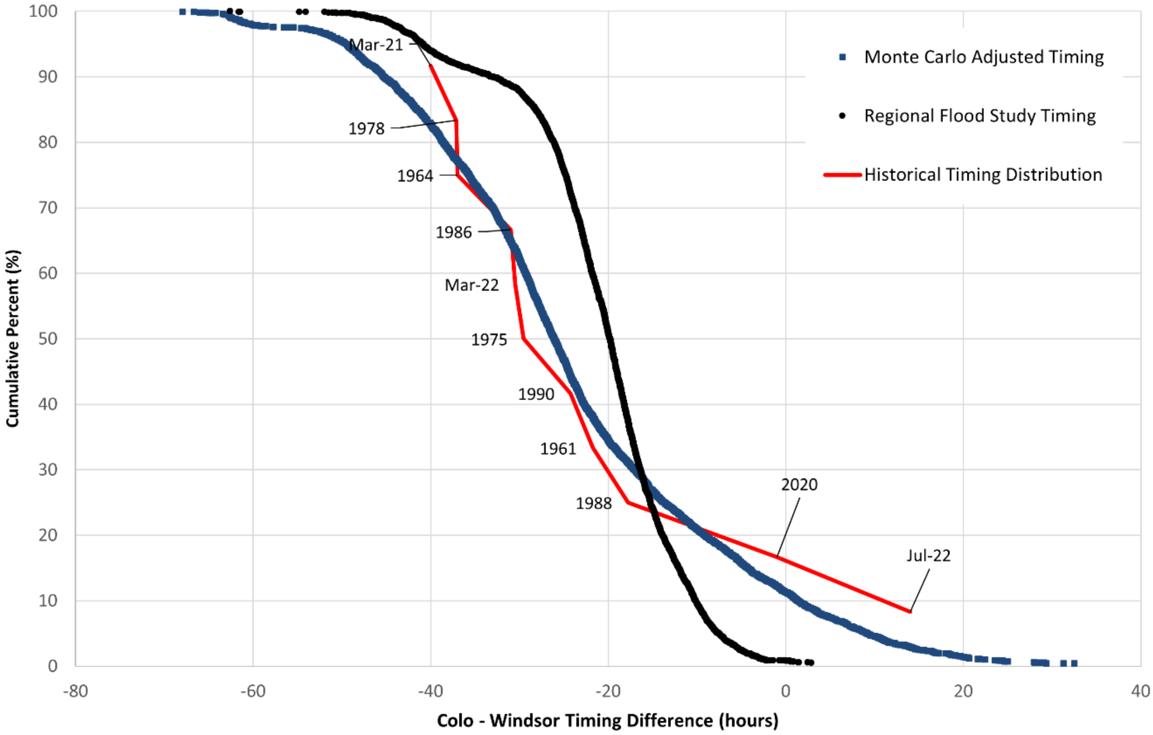


Diagram 8: Historical Colo-Windsor time to peak differences – for Windsor events 10-15 mAHD
Note: Negative values represent where the peak at Colo River occurs before the peak at Windsor

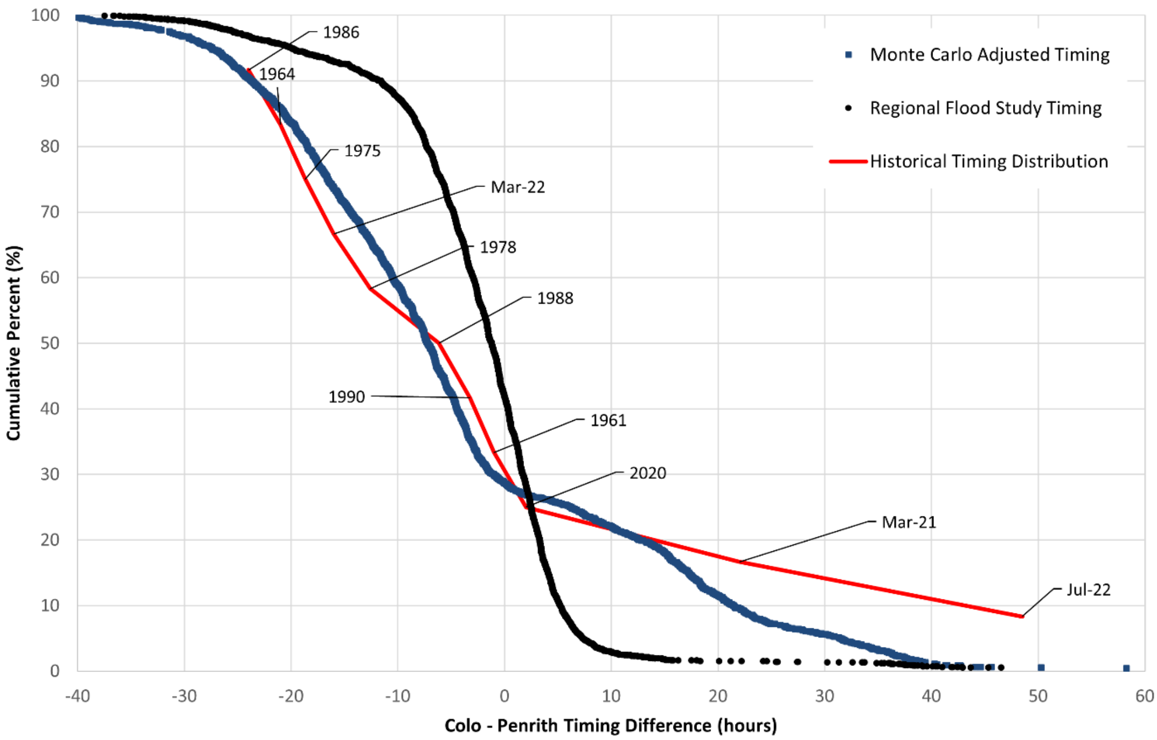


Diagram 9: Historical Penrith-Colo time to peak differences – for Windsor events 10-15 mAHD
Note: Negative values represent where the peak at Colo River occurs before the peak at Penrith.

5. RESULTS

This section presents the results of the updated Monte Carlo framework. Commentary is provided on the differences between the Regional Flood Study (WMAwater, 2019) and the Monte Carlo update in the current study. Further comparisons with the detailed Hawkesbury-Nepean River Flood Study results are provided in **Technical Volume 11**.

5.1. Key Reporting Locations

These reporting locations were adopted for consistency with the Regional Flood Study (WMAwater, 2019). The key locations are listed in Table 28.

Table 28: Key reporting locations

No.	Name	River / Creek	Description
1	Brooklyn Bridge (M1)	Hawkesbury River	Pacific Motorway (M1) crossing of Hawkesbury River
2	Spencer	Hawkesbury River	Approx. 2.5 kilometres upstream of Mangrove Creek confluence
3	Gunderman - Singletons Mill	Hawkesbury River	Between villages of Gunderman and Singletons Mill
4	Wisemans Ferry	Hawkesbury River	Wisemans Ferry crossing (Old Northern Road) - approx. 300 metres downstream Macdonald River confluence, approx. 1,300 metres downstream of Webb's Creek Ferry (St Albans Road), approx. 1,800 metres downstream of Webb's Creek confluence
5	Leets Vale	Hawkesbury River	Approx. 2.5 kilometres upstream of Leets Vale Caravan Park
6	Lower Portland	Hawkesbury River	Lower Portland Ferry crossing (West Portland Road) at Colo River confluence
7	Sackville	Hawkesbury River	Sackville Ferry crossing (Sackville Road) - approx. 450 metres downstream of Currency Creek confluence
8	Ebenezer	Hawkesbury River	At Riverside Oaks Golf Resort
9	Cattai Creek/Gronos Point	Hawkesbury River	Just upstream of Cattai Creek confluence (opposite Gronos Point)
10	South Creek at Richmond Road	South Creek	South Creek at Richmond Road
11	Windsor	Hawkesbury River	Hawkesbury River Bridge crossing at Windsor
12	Rickabys Creek at Blacktown Road	Rickabys Creek	Rickabys Creek at Blacktown Road
13	North Richmond	Hawkesbury River	Hawkesbury River Bridge crossing at North Richmond
14	Yarramundi Bridge	Nepean River	Between Hawkesbury/Springwood Road Bridge and Grose River confluence
15	Penrith	Nepean River	Victoria Bridge - approx. 600 metres upstream of Penrith Weir
16	Blaxlands Crossing	Nepean River	Blaxlands Crossing Bridge in Wallacia at Silverdale Road
17	Bents Basin	Nepean River	Bents Basin Campground downstream of gorge

5.2. Assigning Annual Exceedance Probabilities

As for the Regional Flood Study, for each variable of interest (flood level, flood flow etc), each of the Monte Carlo datasets was combined, ranked and AEPs assigned to each event based on its plotting position. The design event quantiles (for example, 1 in 100 AEP) for the variable of interest (flood level, flood flow etc) were extracted. Therefore, the event that results in a 1 in 100 AEP flood level at a particular location is not necessarily the event that results in a 1 in 100 AEP flow at the location. This is particularly the case in areas that are backwatered and have a large hysteresis in the flow relationship.

5.3. Peak Flood Levels

The peak flood levels for each of the AEP quantiles at each location are presented in Table 29.

Figure 31, Figure 35, Figure 36 and Figure 37 present the fit of the level results from Monte Carlo to historical records and calculated plotting positions for Wallacia, Sackville, Colo Junction and Webbs Creek, respectively.

The Monte Carlo results fit the observed Stage-Frequency analysis well at these locations, particularly at Webbs Creek and Sackville where the record length is long. The Wallacia stage-frequency results, including the method of generating the annual series, are discussed in further detail in **Technical Volume 6**.

Table 30 presents the difference between the flood levels found in the Regional Flood Study and the current study RUBICON results. Comparisons with the detailed TUFLOW results are provided in **Technical Volume 11**.

Differences downstream of Windsor are attributable to changes in Mannings ‘n’ values there (Section 3.4), the timing of the downstream tributaries including the Colo River (Section 4.2.1) and the update of the tidal surge levels to the work of Baird described in Section 4.2.2.

The changes in flood levels at Blaxlands Crossing (Wallacia) and Bents Basin are a result of changes to the losses in the Upper Nepean Catchment to better match the flood record at Wallacia Weir.

Table 29: RUBICON-modelled peak flood levels at key reporting locations – existing dam scenario
Note: Refer to **Technical Volume 11** for official TUFLOW-modelled peak flood levels

No.	Name	Peak Flood Levels (m AHD)											
		1 in 2 AEP	1 in 5 AEP	1 in 10 AEP	1 in 20 AEP	1 in 50 AEP	1 in 100 AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP	1 in 5000 AEP	PMF
1	Brooklyn Bridge (M1)	1.1	1.3	1.4	1.4	1.5	1.5	1.7	1.9	2.2	2.5	3.0	4.5
2	Spencer	1.3	1.4	1.6	1.8	2.2	2.6	3.1	3.9	4.7	5.6	6.4	9.2
3	Gunderman-Singletons Mill	1.4	1.8	2.4	3.0	4.0	4.7	5.6	6.8	7.8	9.1	10.1	13.5
4	Wisemans Ferry	1.6	2.4	3.3	4.2	5.3	6.2	7.3	8.6	9.8	11.2	12.4	16.3
5	Leets Vale	1.9	3.5	4.8	5.9	7.5	8.5	9.6	11.1	12.3	13.8	15.3	19.8
6	Lower Portland	2.4	4.7	6.2	7.6	9.3	10.4	11.6	13.2	14.6	16.2	17.8	23.0
7	Sackville	3.3	6.1	8.0	9.8	11.8	13.0	14.0	15.6	17.0	18.7	20.1	26.5
8	Ebenezer	4.3	7.4	9.6	11.7	13.9	15.2	16.2	17.8	19.0	20.4	21.8	27.7
9	Cattai Creek/Gronos Point	5.2	8.5	11.1	13.4	15.7	17.0	18.2	19.7	20.8	22.0	23.1	28.8
10	South Creek at Richmond Road	6.3	9.6	11.8	13.8	16.0	17.3	18.4	19.9	20.9	22.1	23.3	28.9
11	Windsor	6.0	9.6	11.9	13.9	16.0	17.3	18.4	19.9	20.9	22.1	23.3	28.9
12	Rickabys Creek at Blacktown Road	7.7	9.9	12.0	13.9	16.0	17.4	18.4	19.9	21.0	22.1	23.3	28.9
13	North Richmond	7.4	11.6	14.0	15.6	16.5	17.5	18.5	20.0	21.1	22.2	23.3	28.9
14	Yarramundi Bridge	7.9	12.3	14.9	16.7	17.4	18.0	19.0	20.3	21.4	22.5	23.6	29.0
15	Penrith	17.4	19.7	21.8	23.7	25.0	25.9	26.5	27.1	27.5	28.4	29.5	32.9
16	Blaxlands Crossing	30.9	35.4	38.3	40.5	43.2	45.0	46.7	48.9	50.7	54.5	58.4	66.4
17	Bents Basin	33.9	38.6	41.2	42.7	44.3	45.7	47.1	49.2	51.0	54.6	58.5	66.4

Table 30: Comparison of updated RUBICON flood levels to the 2019 Regional Flood Study (WMAwater, 2019)

No.	Name	Peak Flood Difference (m)											
		1 in 2 AEP	1 in 5 AEP	1 in 10 AEP	1 in 20 AEP	1 in 50 AEP	1 in 100 AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP	1 in 5000 AEP	PMF
1	Brooklyn Bridge (M1)	-	-0.2	-0.2	-0.2	-0.2	-0.2	-0.1	0.1	0.3	0.6	0.9	1.5
2	Spencer	-	-0.4	-0.3	-0.3	-0.2	-0.1	0.0	0.1	0.3	0.7	0.7	2.4
3	Gunderman-Singletons Mill	-	-0.4	-0.4	-0.5	-0.7	-0.8	-0.9	-0.8	-0.6	-0.2	-0.4	1.5
4	Wisemans Ferry	-	-0.4	-0.3	-0.6	-0.8	-0.8	-0.9	-0.8	-0.6	-0.1	-0.4	1.9
5	Leets Vale	-	-0.1	-0.3	-0.5	-0.6	-0.7	-0.5	-0.4	-0.3	0.2	0.4	2.6
6	Lower Portland	-	-0.1	-0.3	-0.6	-0.7	-0.7	-0.5	-0.4	-0.2	0.3	0.6	2.8
7	Sackville	-	-0.1	-0.5	-0.3	-0.4	-0.3	-0.3	0.0	0.3	0.6	0.9	2.9
8	Ebenezer	-	-0.9	-1.1	-1.0	-1.1	-1.1	-1.1	-0.9	-0.6	-0.5	0.0	1.7
9	Cattai Creek/Gronos Point	-	-0.3	-0.1	0.2	0.0	0.1	0.1	0.4	0.5	0.5	0.7	2.2
10	South Creek at Richmond Road	-	-0.3	-0.1	0.1	0.0	0.0	0.1	0.3	0.4	0.4	0.6	2.2
11	Windsor	-	-0.3	-0.1	0.1	0.0	0.0	0.0	0.3	0.4	0.4	0.6	2.1
12	Rickabys Creek at Blacktown Road	-	-0.2	0.0	0.1	-0.1	0.0	0.1	0.3	0.3	0.4	0.6	2.1
13	North Richmond	-	0.2	0.4	0.2	-0.1	-0.1	0.0	0.2	0.3	0.4	0.6	2.1
14	Yarramundi Bridge	-	0.3	0.4	0.3	0.0	-0.2	-0.1	0.1	0.2	0.2	0.4	1.9
15	Penrith	-	0.2	0.4	0.4	0.2	0.1	0.0	0.0	0.0	0.1	0.0	0.2
16	Blaxlands Crossing	-	0.3	1.1	1.1	0.6	0.3	0.2	-0.1	0.0	0.3	0.1	0.1
17	Bents Basin	-	0.3	0.9	0.8	0.5	0.3	0.1	-0.1	0.0	0.4	0.1	0.1

5.4. Peak Flood Flows

Figure 32, Figure 33 and Figure 34 show the results of the Monte Carlo model compared to the flood frequency analysis discussed in Section 2. Table 31 shows the flow frequency analysis for the pre-dam case from the Monte Carlo model at Warragamba Dam, Penrith and Windsor. The modelled pre-dam FFA in Table 31 can be compared to the expected parameter quantiles from Table 3.

The Monte Carlo results fit the pre-dam FFA well at the three key locations. At these locations, the Monte Carlo results plot within the confidence limits. While it is possible to improve the fit at any one of the three locations, this is deemed the best overall fit which does not compromise the results at any of the other locations.

The comparison of flow and stage frequency results against long-term flood records at seven key locations in the catchment verifies the framework, providing confidence that the Monte Carlo results are capturing the variability in the system and adequately representing reality.

Table 31: Flood frequency analysis results – flow pre-dam

AEP	Flow (m³/s)		
	Warragamba Pre-dam	**Penrith Pre-dam**	**Windsor Pre-dam**
2	840	1280	1330
5	2950	4030	2800
10	5000	6620	4250
20	7390	9450	5760
50	10,860	12,970	7640
100	13,390	15,730	9030
200	15,900	18,540	10,390
500	19,480	22,380	12,220
1000	22,090	25,150	13,150
2000	25,430	28,480	14,060
5000	30,380	33,730	15,000

The peak flood flows for each of the AEP quantiles are presented in Table 32 for some key flow locations. Flow has only been reported at locations in the floodplain where the flow is concentrated in one flow path. Peak flows have been extracted at the following locations:

1. The M4 Bridge on the Nepean River at Penrith
2. Yarramundi Bridge (Nepean River, upstream of the Grose River junction)
3. Sackville Ferry on the Hawkesbury River
4. Webbs Creek (Hawkesbury River, upstream of the Webbs Creek junction)

Changes to peak flows are produced in Table 33. Changes to the peak flood flows downstream of Windsor are influenced by a variety of factors. At Sackville the major cause is the increased floodplain storage at Windsor and reduced river flow. Downstream of Lower Portland, changes in the Colo hydrology and adjustments to the timing are also significant factors. The increase in floodplain storage at Windsor in the RUBICON model reduced peaks in the lower floodplain.

Table 32: RUBICON-modelled Monte Carlo peak flood flows at key flow locations – existing dam scenario

Note: Refer to **Technical Volume 11** for official TUFLOW-modelled peak flood flows

No.	Name	Peak Flood Flow (m^3/s)											
		1 in 2 AEP	1 in 5 AEP	1 in 10 AEP	1 in 20 AEP	1 in 50 AEP	1 in 100 AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP	1 in 5000 AEP	PMF
1	M4 Bridge	1080	3580	6190	9070	11,260	12,990	14,810	17,340	19,690	25,940	31,500	45,489
2	Yarramundi Bridge	1030	3500	6140	9060	11,170	12,650	14,170	16,270	18,160	23,710	28,330	49,802
3	Sackville	1230	2650	4130	5700	7570	8930	10,110	11,740	12,800	13,890	14,940	19,356
4	Wisemans Ferry	1860	3570	4920	6380	8510	10,100	11,910	14,610	17,120	20,370	24,200	38,903

Table 33: Comparison of updated RUBICON peak flood flows to the 2019 Regional Flood Study (WMAwater, 2019)

No.	Name	Change in Peak Flow (%)											
		1 in 2 AEP	1 in 5 AEP	1 in 10 AEP	1 in 20 AEP	1 in 50 AEP	1 in 100 AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP	1 in 5000 AEP	PMF
1	M4 Bridge	-8%	5%	9%	8%	4%	2%	1%	0%	1%	2%	1%	1%
2	Yarramundi Bridge	-7%	7%	10%	8%	4%	2%	2%	0%	0%	2%	1%	4%
3	Sackville	-3%	-6%	-8%	-10%	-17%	-18%	-20%	-20%	-21%	-23%	-24%	-27%
4	Wisemans Ferry	-22%	-16%	-21%	-25%	-26%	-27%	-25%	-25%	-26%	-21%	-20%	-7%

6. CLIMATE CHANGE

Climate change scenarios have been assessed in the Monte Carlo framework for rainfall increases of 9.5% and 19.7% based on ARR temperature scaling approach. These represent the high emissions scenarios for the mid-century and late century respectively. The 9.5% rainfall increase is also representative of the low emissions scenario for the late century. The standard approach is to factor up the rainfall equally at each location within the catchment. This was undertaken for the chosen climate change scenarios.

7. REPRESENTATIVE DESIGN FLOOD EVENTS

Within the Monte Carlo results, there is no one single design event of a given AEP at all locations, that is, while one event may be representative of a specific AEP at one location, it may not be for another. The purpose of the representative events is to provide a subset of the Monte Carlo events that can be used for detailed 2-dimensional modelling, as running all the Monte Carlo runs would be impractical. The aim is to select a limited number of events, which when enveloped, will match the design event flood surface across the entire Hawkesbury-Nepean River within a specified threshold.

7.1. Selection Methodology

The 2019 Regional Flood Study identified specific cross sections along the main river for comparing and selecting the representative events (refer to Table 34). Primary stations were identified as those that are key gauges for flood warning and future model set up whereas a less strict match criterion was applied for secondary stations. The representative events were selected by examining the difference between the peak flood level of the desired AEP quantile and the peak flood level in each Monte Carlo run at each station.

Table 34: Primary and secondary stations for representative event selection in the Regional Flood Study

<table>
<tr><th>Primary Stations</th><th>Secondary Stations</th></tr>
<tr><td>M4 Bridge</td><td rowspan="2">Sackville</td></tr>
<tr><td>Penrith</td></tr>
<tr><td>Yarramundi</td><td rowspan="3">Wisemans Ferry</td></tr>
<tr><td>North Richmond</td></tr>
<tr><td>Windsor Bridge</td></tr>
</table>

For the purposes of this study and to improve the representative event selection criterion, all cross sections between Bents Gorge and the Ocean inclusive (totalling 132 cross sections) were examined per AEP. Additionally, while the Regional Flood Study utilised a +/- 0.1m condition between the M4 Bridge and Sackville, and +/- 0.3m between Sackville and Wisemans Ferry, a single +/- 0.1m condition was applied for all cross sections in this study between the 1 in 2 and 1 in 2000 AEP frequencies. For the 1 in 5000 AEP, the condition was increased to +/- 0.2m due to the reduced number of viable events at that event frequency.

When selecting representative events at the ocean boundary, the tide distribution outlined in Section 4.2.2 was used instead of the AEP flood quantile at the ocean. Due to updates made to the ocean tide level distribution for the 1 in 2 and 1 in 5 AEP, flat tide events set to the respective ocean AEP level were constructed so that when enveloped will match the design event flood surface all the way to the ocean boundary.

7.2. Selected Representative Events

Where multiple sets of events were identified to satisfy the specified criteria, the set of events that most closely matched the AEP quantiles was selected with the objective of providing a minimal number of representative events to improve efficiency. The tables below outline the representative events selected for each AEP.

Table 35: Representative events selected for 1 in 2 to 1 in 100 AEP

Representative Events per AEP (1 in X)					
1 in 2	**1 in 5**	**1 in 10**	**1 in 20**	**1 in 50**	**1 in 100**
Bd08261	Bd01067	Bd02615	Rd00245	Bd05610	Rd04175
Bd03972	Bd03301	Bd05558	Rd03529	Rd05471	Rd02523
Bd04866	Bd04234	Bd05478	Rd08304	Rd08941	Rd09044
Bd05848	Bd08555	Bd05291	Rd09064	Rd06336	Rd01158
Bd01000	Bd02406	Rd09880	Bd07572	-	Rd03816
Bd01503	Bd07861	-	Rd04971	-	-
-	Bd10000	-	Rd06766	-	-

Table 36: Representative events selected for 1 in 200 to 1 in 5000 AEP

Representative Events per AEP (1 in X)				
1 in 200	**1 in 500**	**1 in 1000**	**1 in 2000**	**1 in 5000**
Rd00558	Rd08413	Rd03510	Rd00180	Rd02797
Rd09247	Rd00361	Rd08942	Rd01670	Rd01374
Rd04733	Rd00723	Rd02664	Rd08763	Rd04070
Rd05306	Rd06478	Rd08891	Rd07495	Rd09388
-	Rd01137	Rd09797	Rd00937	Rd05659
-	Rd03880	Rd05981	-	-
-	Rd09330	-	-	-

Diagram 10 illustrates the residual errors for the 1 in 100 AEP for the selected representative events. While the events have a residual error greater than -0.1m across several cross sections, the envelope of the four events will produce a flood surface that is within the +/- 0.1m of the true 1 in 100 AEP design surface.

The representative event difference plots are included for all design AEP events in **Appendix C**.

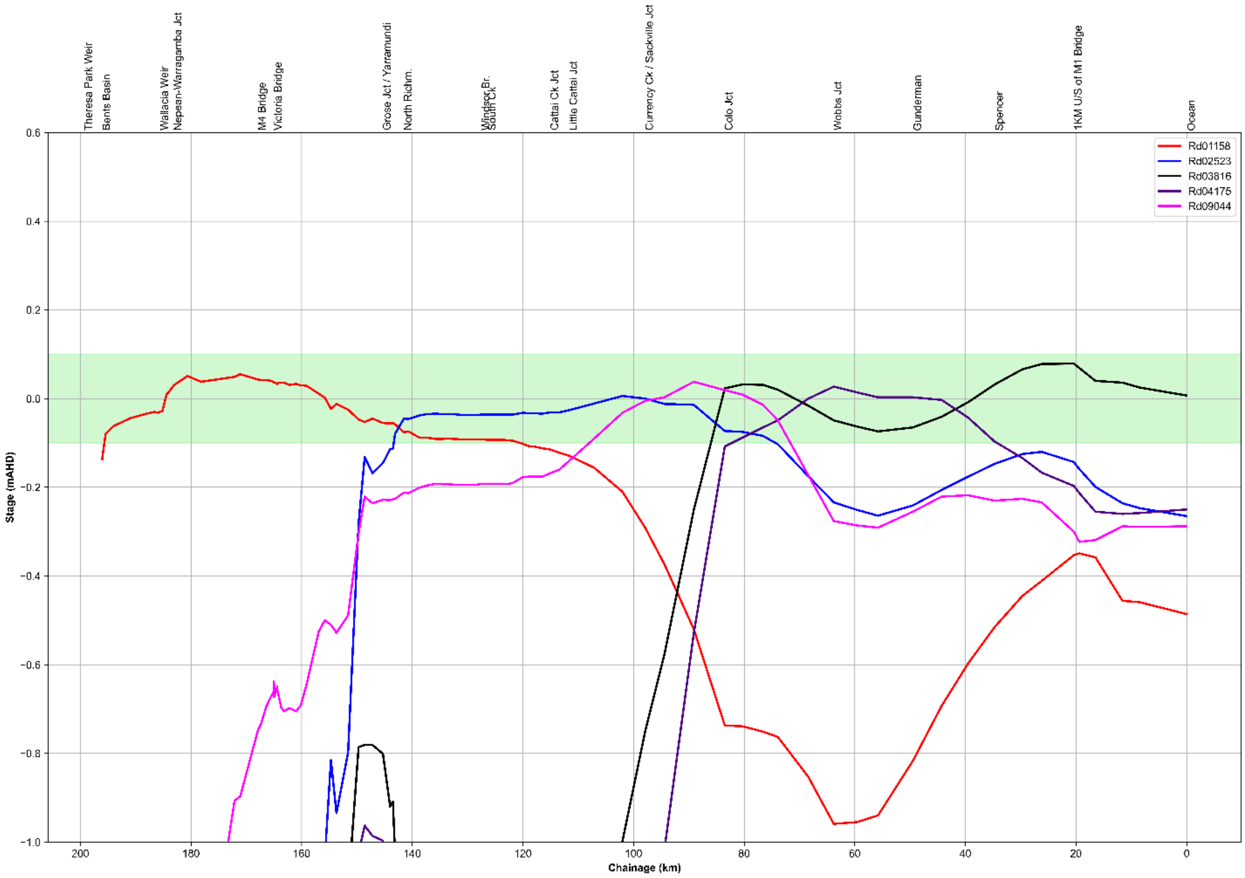


Diagram 10: Example representative event selection – 1 in 100 AEP

7.3. Representative Events for Climate Change

Climate change was assessed for the 9.5% and 19.7% rainfall increases representing the low range and high range emissions projections for the late century. Rainfall increase scenario 9.5% also represents the high emissions scenario for the mid-century. The representative events selected for the climate change scenarios are the same as those for the existing scenario (Section 7.2). This meant that comparisons between the scenarios did not introduce other variables present in the Monte Carlo such as spatial or timing differences.

Climate change scenarios were run for the 1 in 20 AEP, 1 in 100 AEP and 1 in 500 AEP quantiles. To confirm the selected events remained representative of the AEP of interest, they were re-ranked using the same methodology for the existing events outlined in Section 7.1. Some reordering of events was observed across the selected quantiles for 9.5% and 19.7% rainfall scaling. To avoid picking new representative events for these quantiles, the rainfall for several representative events was scaled to bring the events closer to the climate change AEP quantile values developed in the Monte Carlo analysis. The representative event selection diagrams for these two scenarios and scaling factors are in **Appendix C**.

This is a reasonable approach as it avoids selecting new representative events which would add +/- 100mm of noise to the results and ensures the modified representative events reproduce the Monte Carlo results.

7.4. How to Use the Representative Events

Running a more detailed 2-dimensional model of the entire catchment for all Monte Carlo events would take an extremely long time currently. However, running a detailed 2-dimensional model for select events is a more feasible undertaking. For most modelling situations it will only be necessary to run the representative events. Where major changes are made to the catchment response like building a mitigation dam or lowering full supply level at Warragamba Dam it will be necessary to run the full Monte Carlo framework and determine if the representative events change. For localised modelling generally only one representative event per AEP would need to be run.

If rate of rise or another variable is of interest, then a new set of events may need to be selected from the Monte Carlo set.

The individual representative events can also be used to understand the time dimension of design events as the design event quantiles produced for the mapping are an envelope of events and therefore a single time series of the event does not exist. This may be useful for emergency planning.

It is important to note however that the representative events were selected focusing on the Hawkesbury-Nepean River and may not be representative of the tributaries beyond the reach of backwater effects.

8. CONCLUSIONS

This report outlines the updates made to the flood study Monte Carlo framework originally developed as part of the Regional Flood Study (WMAwater, 2019). This work has led to improved calibration to historical events and a better representation of the floodplain and the coastal interaction.

New representative events for design AEP quantiles for the existing dam and climate were selected for input into the TUFLOW model developed by Rhelm and CSS.

Two climate change scenarios were run for the suite of Monte Carlo events. These were rainfall increases of 9.5% and 19.7%, representing the high emission scenarios for the mid and late century respectively. Representative events were provided for the 1 in 20 AEP, 1 in 100 AEP and 1 in 500 AEP climate change quantiles.

The results of the design flood modelling using the new TUFLOW model, for both existing and climate change scenarios, are reported in **Technical Volume 11**.

The representative design storms will be available to stakeholders, including councils within the valley and the NSW Government.

9. REFERENCES

Australian Water and Coastal Studies (AWACS)
Lower Hawkesbury River Flood Study (Final Draft)
prepared for Department of Land and Water Conservation (DLWC)
April 1997

Babister, M, McKeown, T & Retallick, M (2016a)
Windsor - Australia's longest flood record
37th Hydrology & Water Resources Symposium Queenstown NZ 2016

Babister, M, Trim, A, Testoni, I & Retallick, M
The Australian Rainfall & Runoff Datahub
37th Hydrology and Water Resources Symposium Queenstown NZ, 2016

Babister, M & Barton C (Editors)
Australian Rainfall and Runoff Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains- Stage 1&2 Report P15/S1/009
Engineers Australia, 2012

Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors)
Australian Rainfall and Runoff: A Guide to Flood Estimation
Commonwealth of Australia, Australia, 2019

Boyd, MJ, Rigby, EH, Van Drie, R
Watershed Bounded Network Model (WBNM) version 2017_01a
July 2017

Bureau of Meteorology (2006).
Guidebook to the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method, on CD-ROM 'Guide to the Estimation of Probable Maximum Precipitation: Generalised Southeast Australia Method'.
Bureau of Meteorology October 2006.

Commonwealth of Australia
Australian Disaster Resilience Handbook 7 Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia
AIDR 2017

Erskine, WD (1986)
River metamorphosis and environmental change in the Macdonald Valley, New South Wales, since 1949
Australian Geographical Studies 24, pp.88-197.
https://doi.org/10.1111/j.1467-8470.1986.tb00513.x

Infrastructure NSW
Hawkesbury-Nepean River March and July 2022 Floods Review: Hydrology, Riverbank Erosion, and Flood Mitigation Scenarios
February 2023

Kuczera, G
FLIKE Software Package
University of Newcastle 2013

Lyall and Macoun Consulting Engineers
Upper Nepean River Flood Study
prepared for Wollondilly, Campbelltown, Camden, Liverpool and Penrith Councils
Department of Land and Water Conservation (DLWC)
September 1995

Moriasi, DN, Gitau, MW, Pai, N, Daggupati, P (2015)
Hydrologic and water quality models: performance measures and evaluation criteria.
Transactions of the ASABE. 58(6): 1763 1785. https://doi.org/10.13031/trans.58.10715

NSW Government
Floodplain Development Manual: The management of flood liable land
April 2005

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

Webb, McKeown & Associates
Warragamba Dam Auxiliary Spillway Environmental Impact Study – Flood Study
October 1996

Webb, McKeown & Associates
Lower Macdonald River Flood Study
August 2004

WMAwater
Hawkesbury Nepean Hydrologic Model Update
prepared for WaterNSW
January 2018

WMAwater
Hawkesbury-Nepean Valley Regional Flood Study
prepared for Infrastructure NSW
July 2019

WMAwater
Climate Change and flooding effects on the Hawkesbury-Nepean
prepared for Infrastructure NSW
September 2021

World Meteorological Organization (2009).
Manual on Estimation of Probable Maximum Precipitation (PMP).
WMO - No. 1045, Geneva.

Young, T
Investigation of Major Flood Events at Wisemans Ferry (Draft)
November 1984

Figures

UPDATE TO THE HAWKSBURY NEPEAN MONTE CARLO FRAMEWORK AND DESIGN FLOOD EVENTS

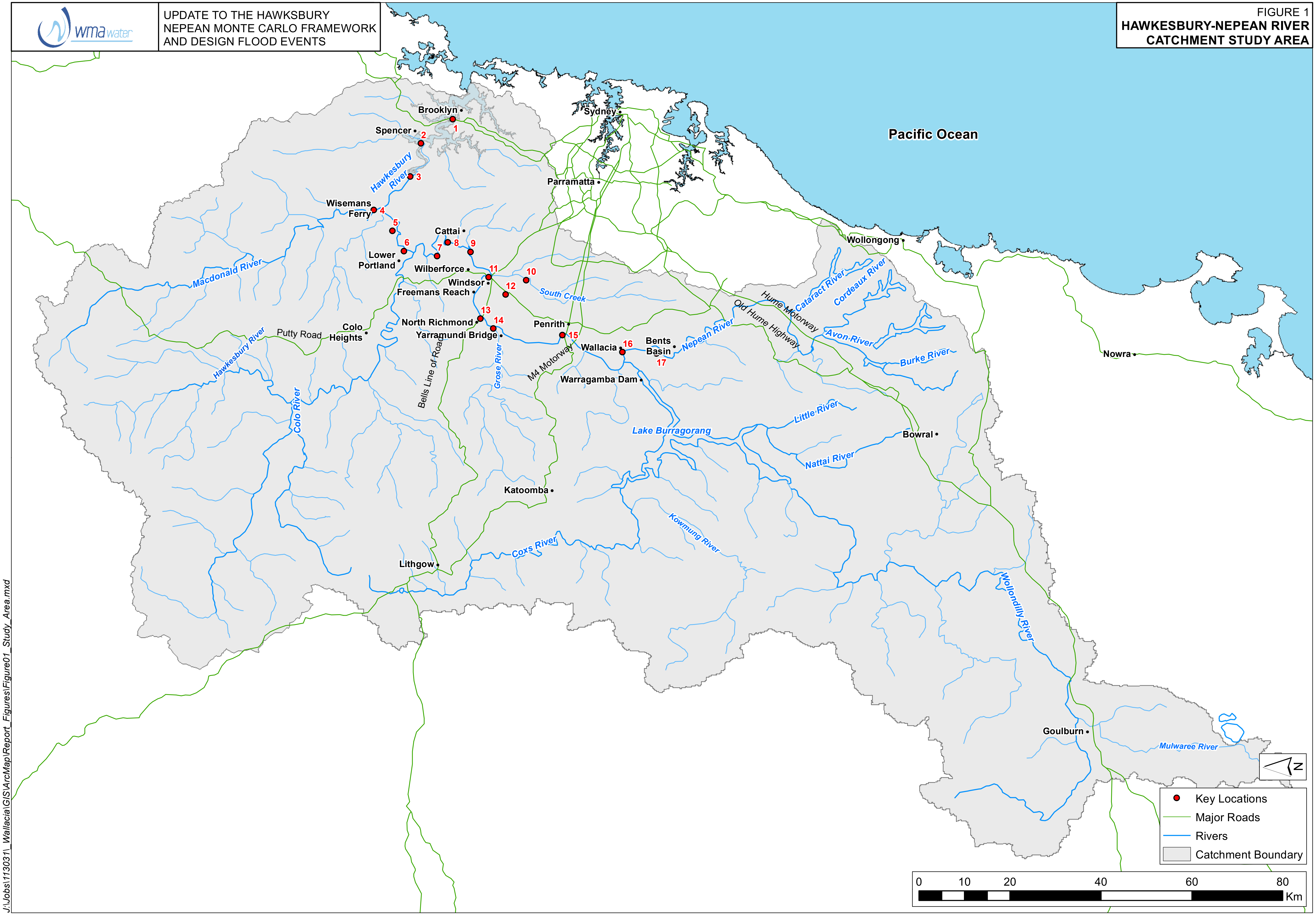


FIGURE 1
HAWKESBURY-NEPEAN RIVER CATCHMENT STUDY AREA

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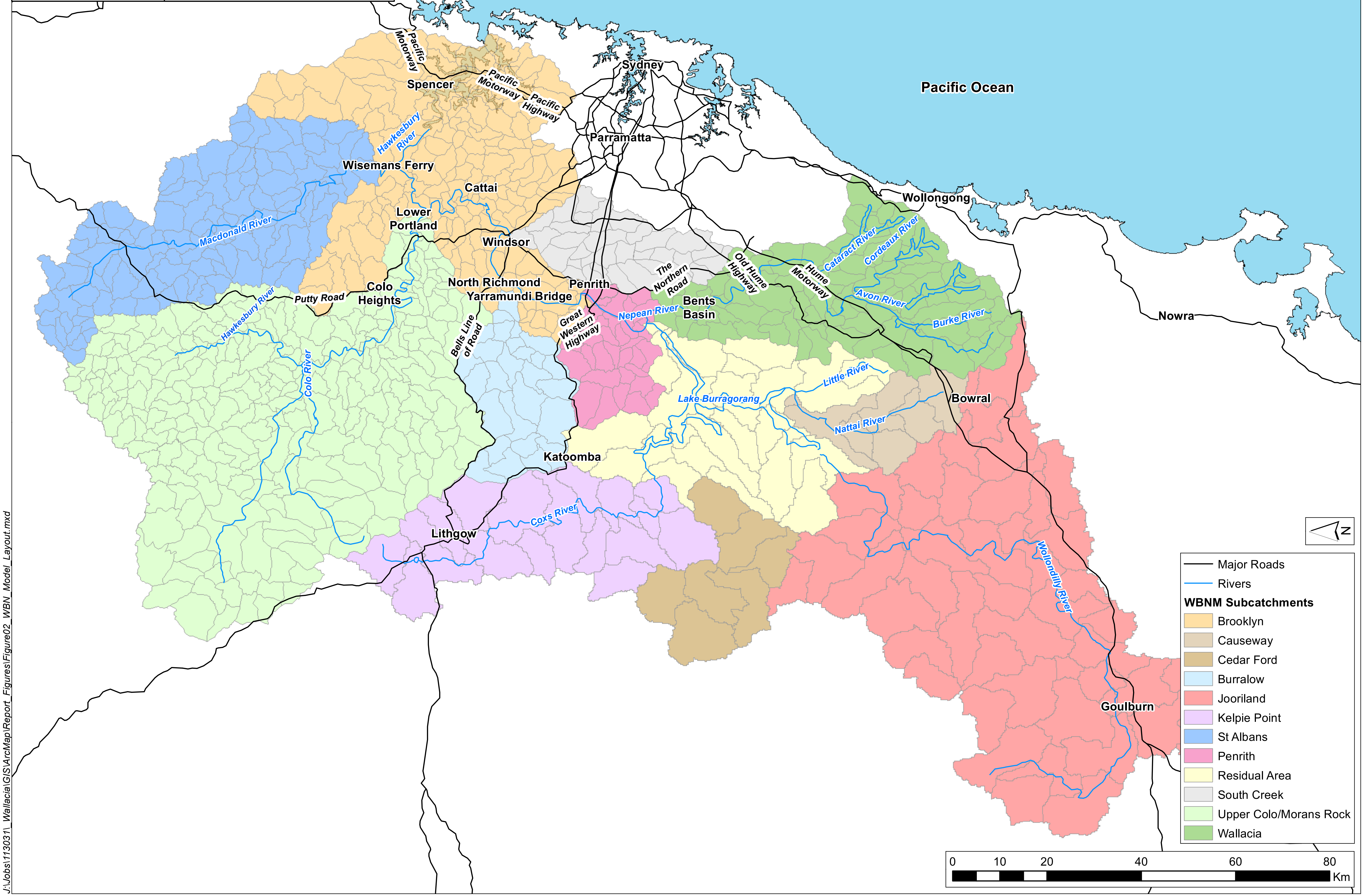
UPDATE TO THE HAWKSBURY NEPEAN MONTE CARLO FRAMEWORK AND DESIGN FLOOD EVENTS

FIGURE 2
HYDROLOGIC MODEL LAYOUT WBNM



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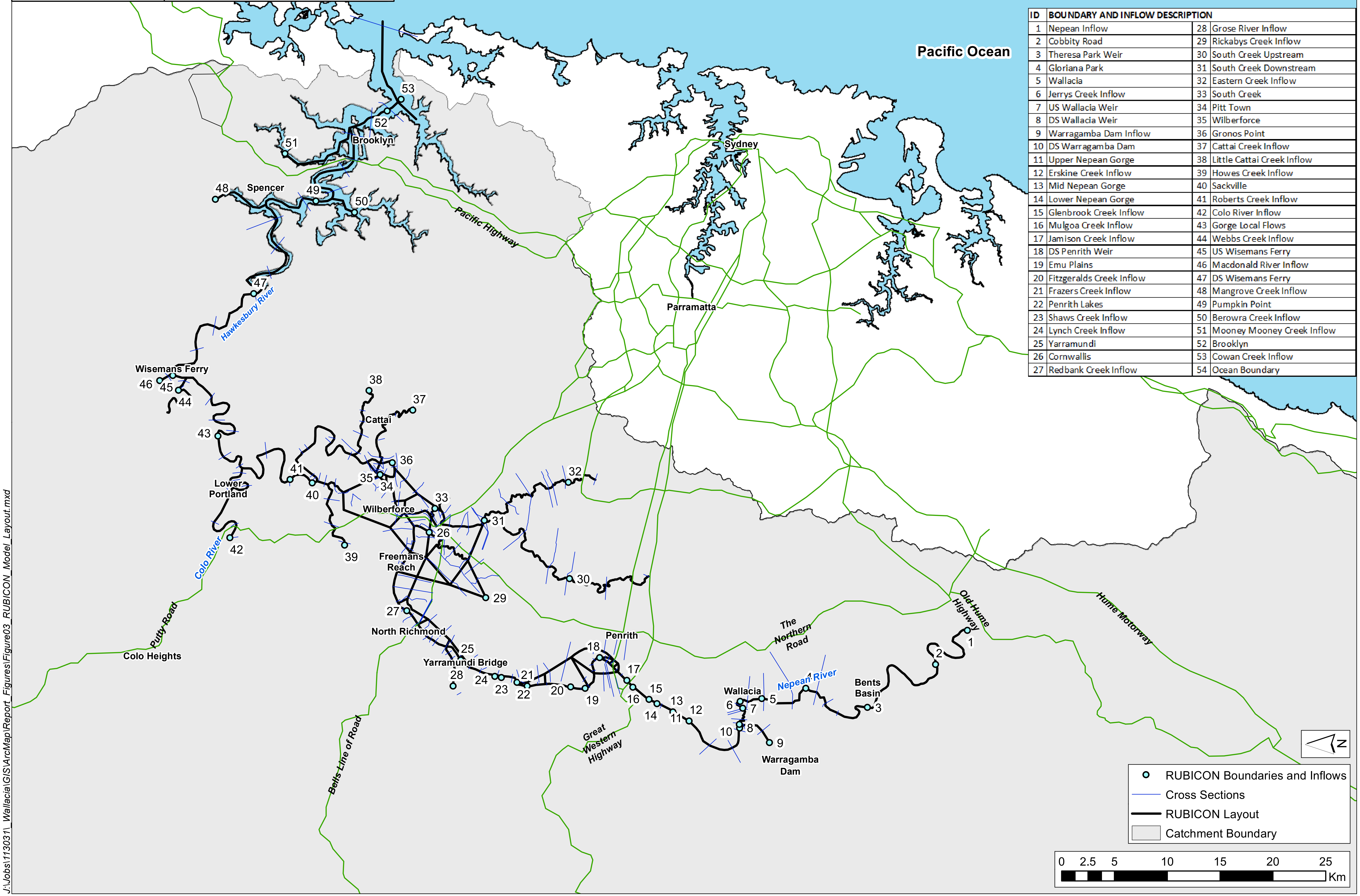
UPDATE TO THE HAWKSBURY NEPEAN MONTE CARLO FRAMEWORK AND DESIGN FLOOD EVENTS

FIGURE 3
HYDRAULIC MODEL LAYOUT RUBICON



ID	BOUNDARY AND INFLOW DESCRIPTION		
1	Nepean Inflow	28	Grose River Inflow
2	Cobbity Road	29	Rickabys Creek Inflow
3	Theresa Park Weir	30	South Creek Upstream
4	Gloriana Park	31	South Creek Downstream
5	Wallacia	32	Eastern Creek Inflow
6	Jerrys Creek Inflow	33	South Creek
7	US Wallacia Weir	34	Pitt Town
8	DS Wallacia Weir	35	Wilberforce
9	Warragamba Dam Inflow	36	Gronos Point
10	DS Warragamba Dam	37	Cattai Creek Inflow
11	Upper Nepean Gorge	38	Little Cattai Creek Inflow
12	Erskine Creek Inflow	39	Howes Creek Inflow
13	Mid Nepean Gorge	40	Sackville
14	Lower Nepean Gorge	41	Roberts Creek Inflow
15	Glenbrook Creek Inflow	42	Colo River Inflow
16	Mulgoa Creek Inflow	43	Gorge Local Flows
17	Jamison Creek Inflow	44	Webbs Creek Inflow
18	DS Penrith Weir	45	US Wisemans Ferry
19	Emu Plains	46	Macdonald River Inflow
20	Fitzgeralds Creek Inflow	47	DS Wisemans Ferry
21	Frazers Creek Inflow	48	Mangrove Creek Inflow
22	Penrith Lakes	49	Pumpkin Point
23	Shaws Creek Inflow	50	Berowra Creek Inflow
24	Lynch Creek Inflow	51	Mooney Mooney Creek Inflow
25	Yarramundi	52	Brooklyn
26	Cornwallis	53	Cowan Creek Inflow
27	Redbank Creek Inflow	54	Ocean Boundary

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FIGURE 4
STAGE PROFILES
NOVEMBER 1961

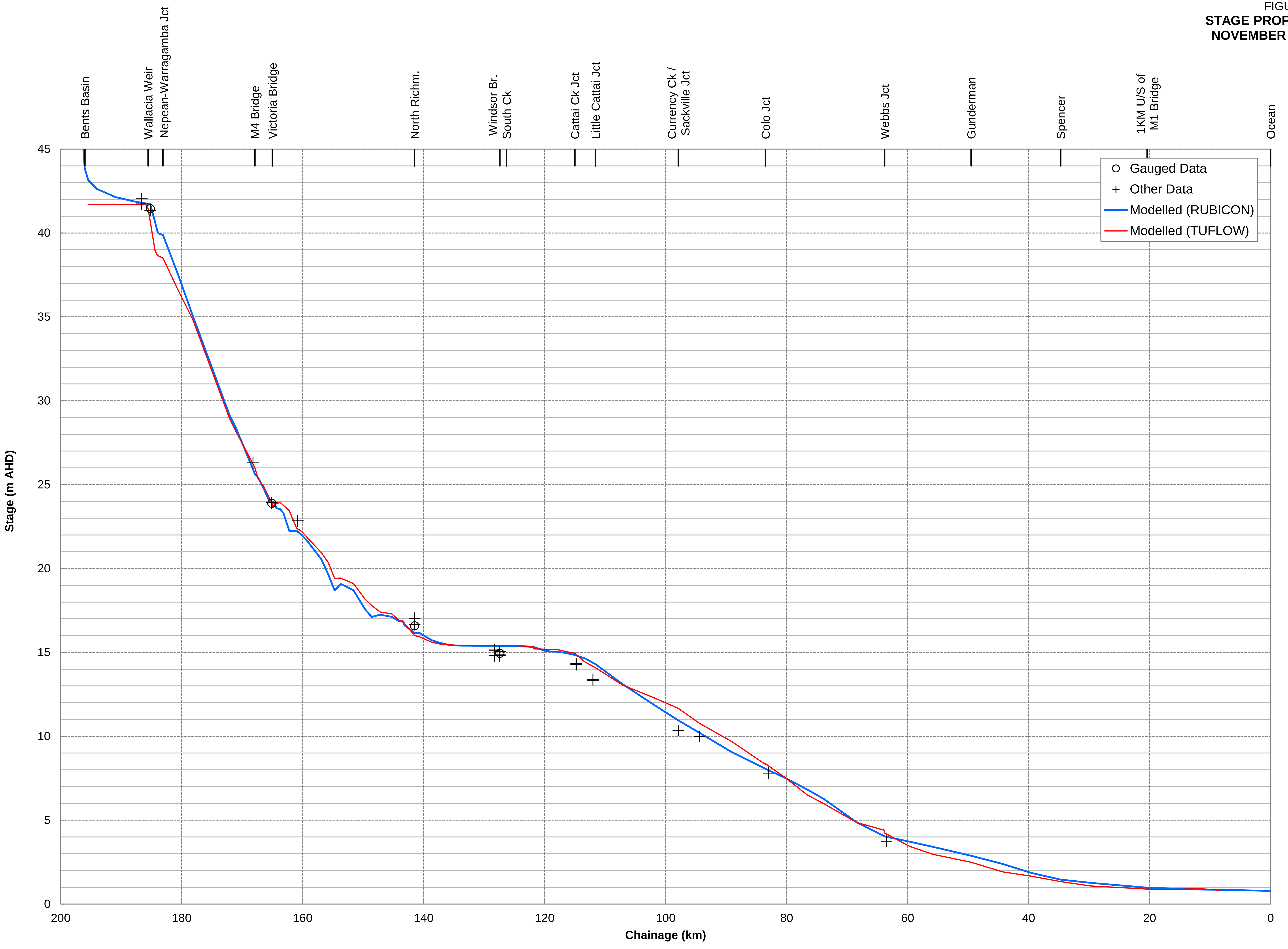

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FIGURE 5
STAGE HYDROGRAPHS
NOVEMBER 1961

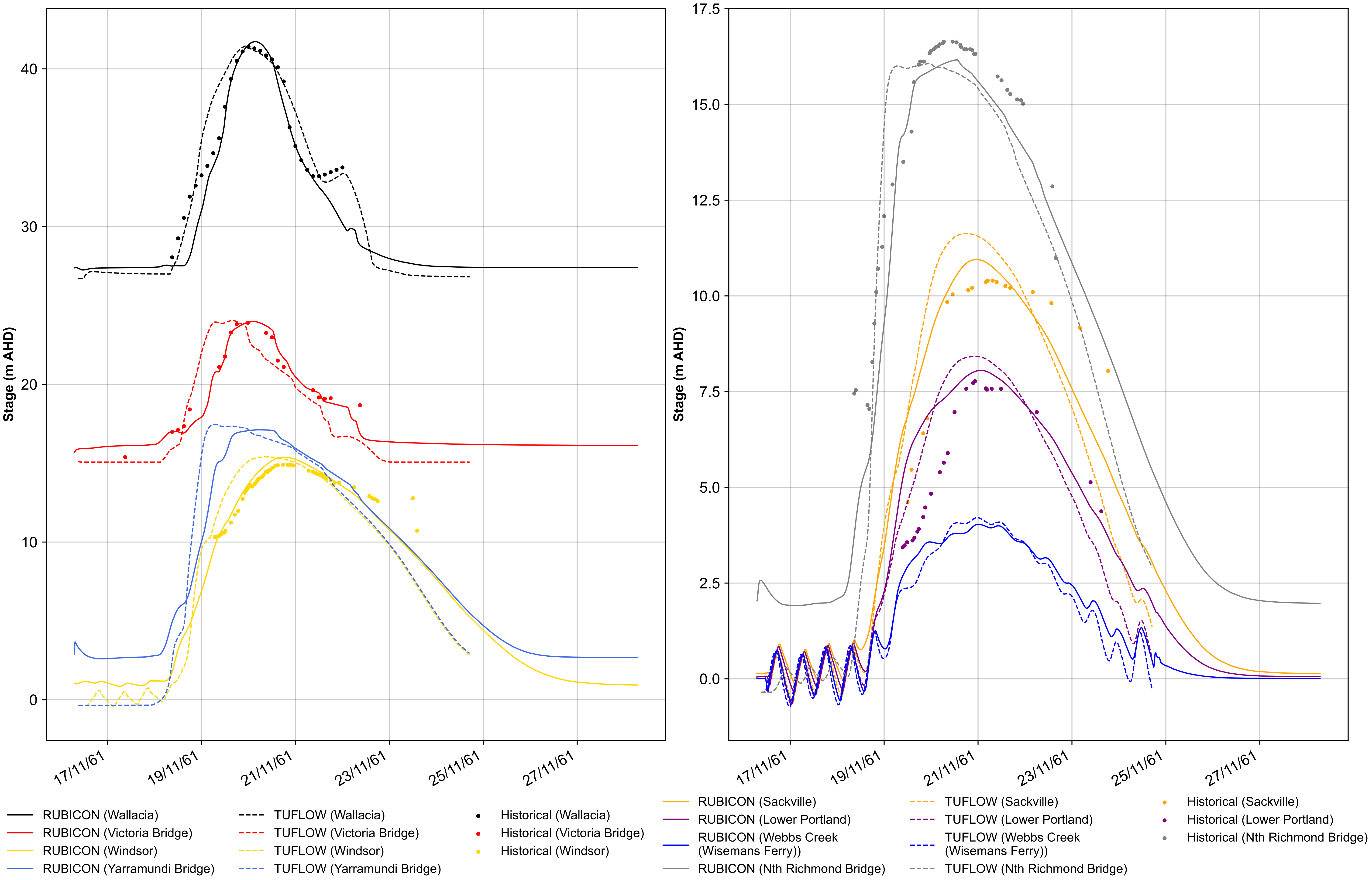

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FIGURE 6
FLOW HYDROGRAPHS
NOVEMBER 1961

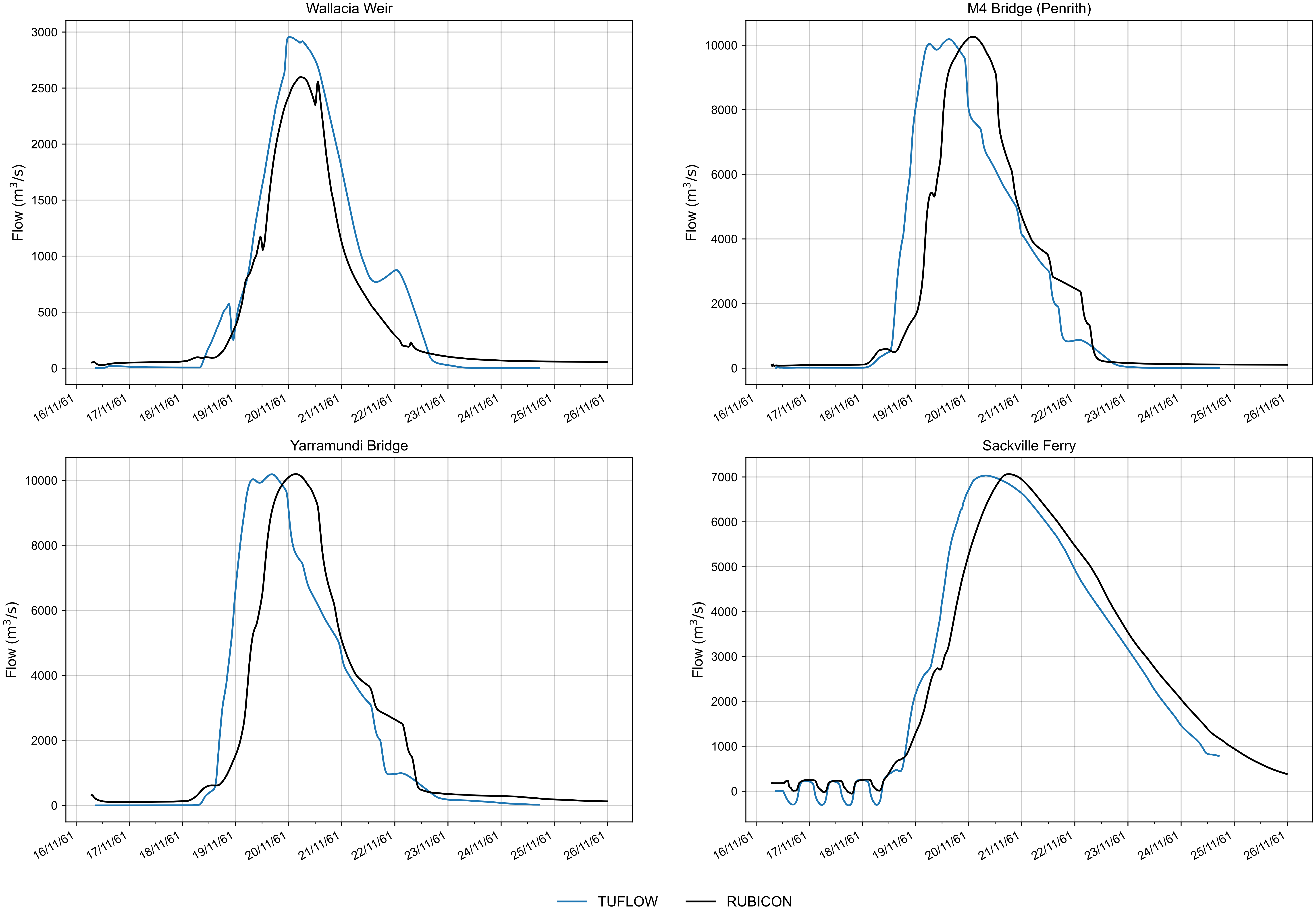

FIGURE 7
STAGE PROFILES
JUNE 1964

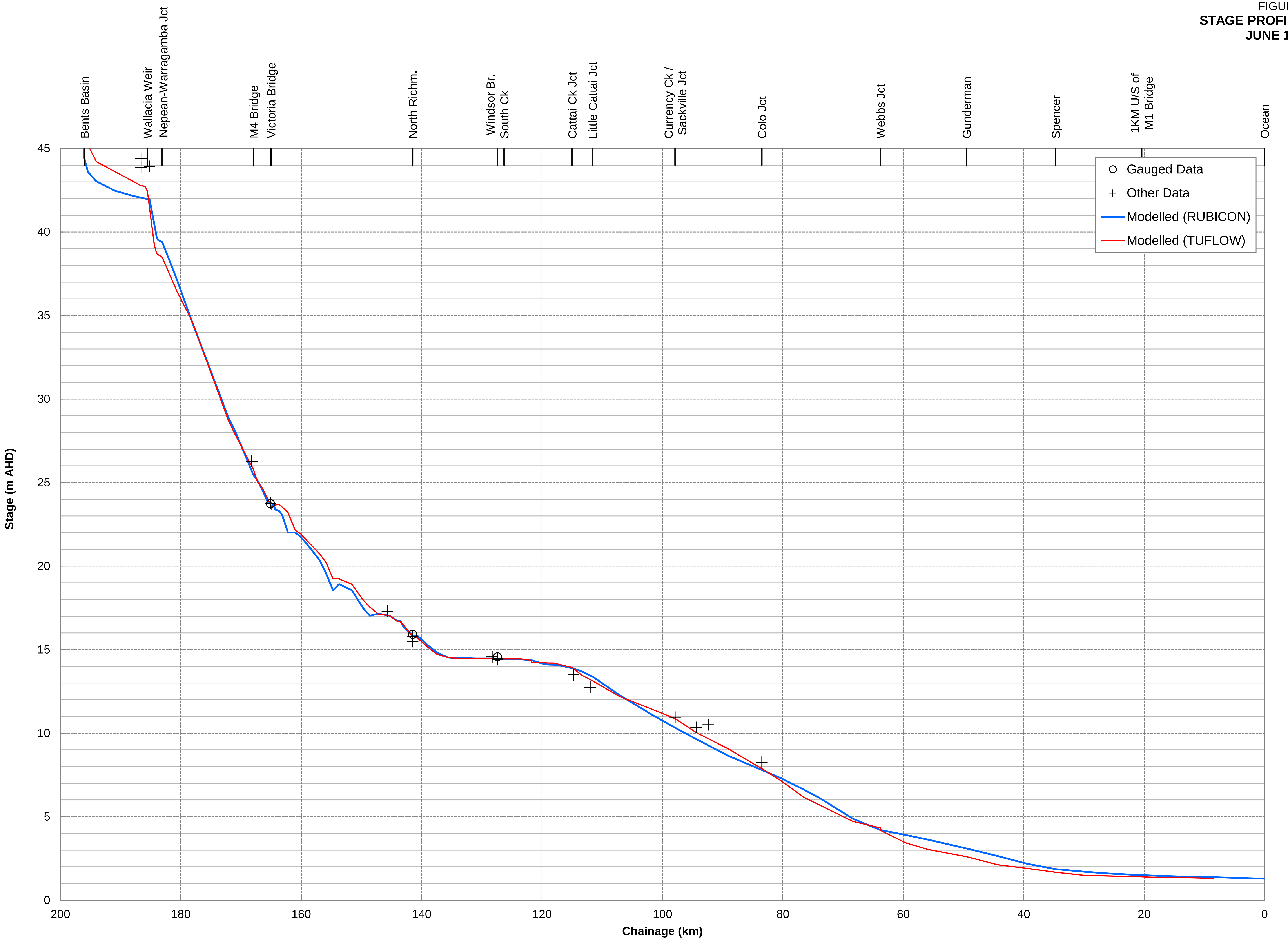

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FIGURE 8
STAGE HYDROGRAPHS
JUNE 1964

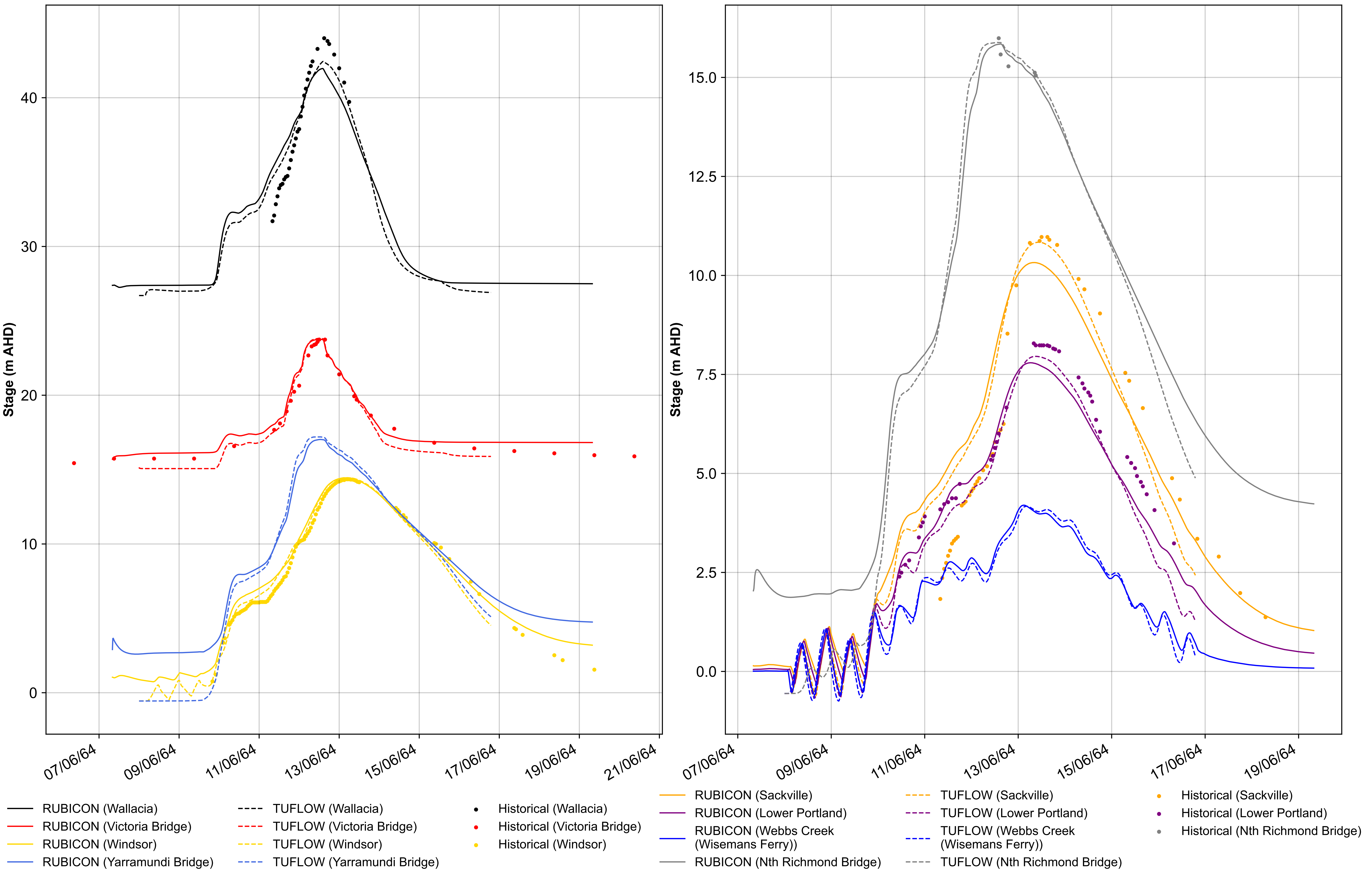

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FIGURE 9
FLOW HYDROGRAPHS
JUNE 1964

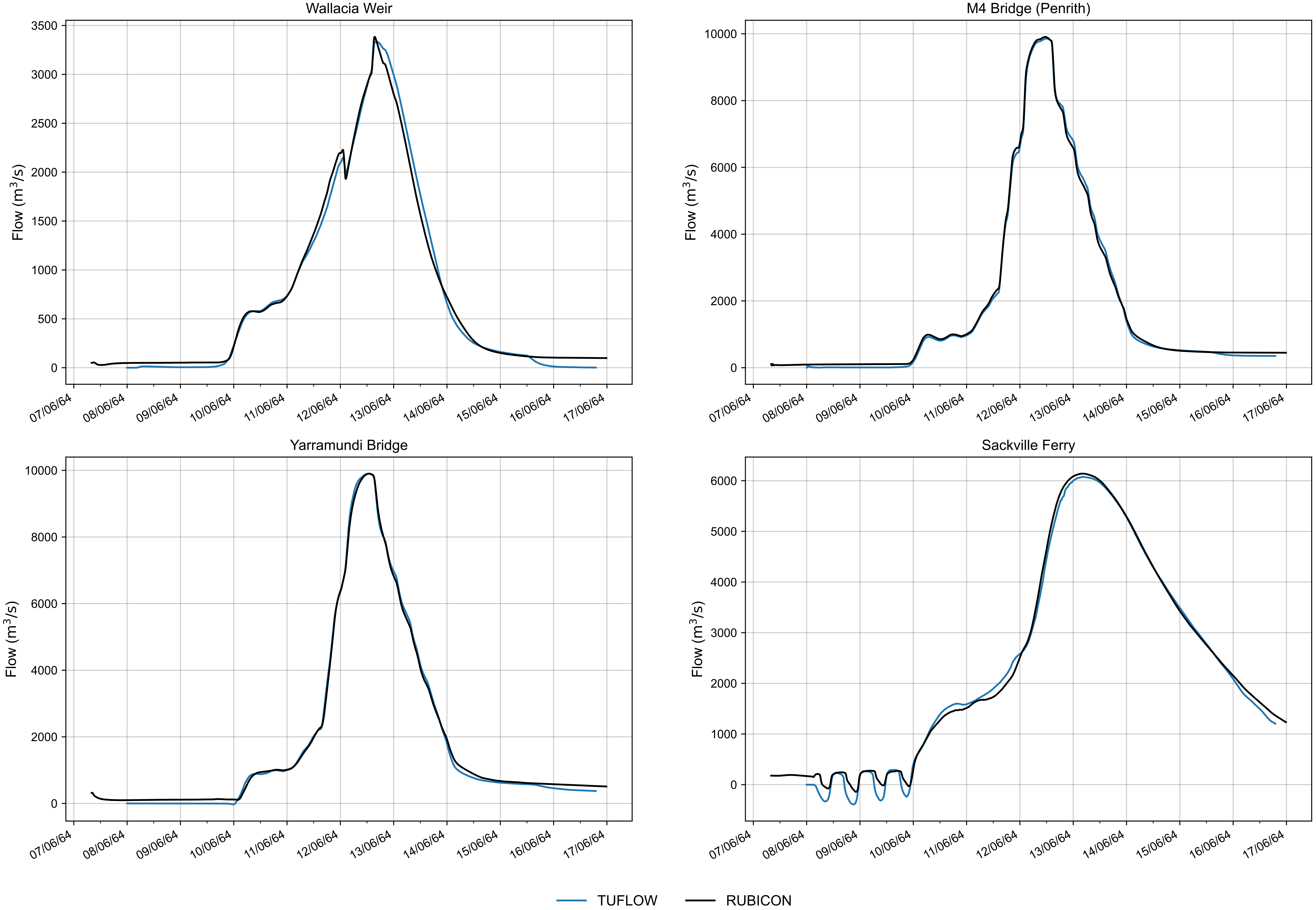

FIGURE 10
STAGE PROFILES
JUNE 1975

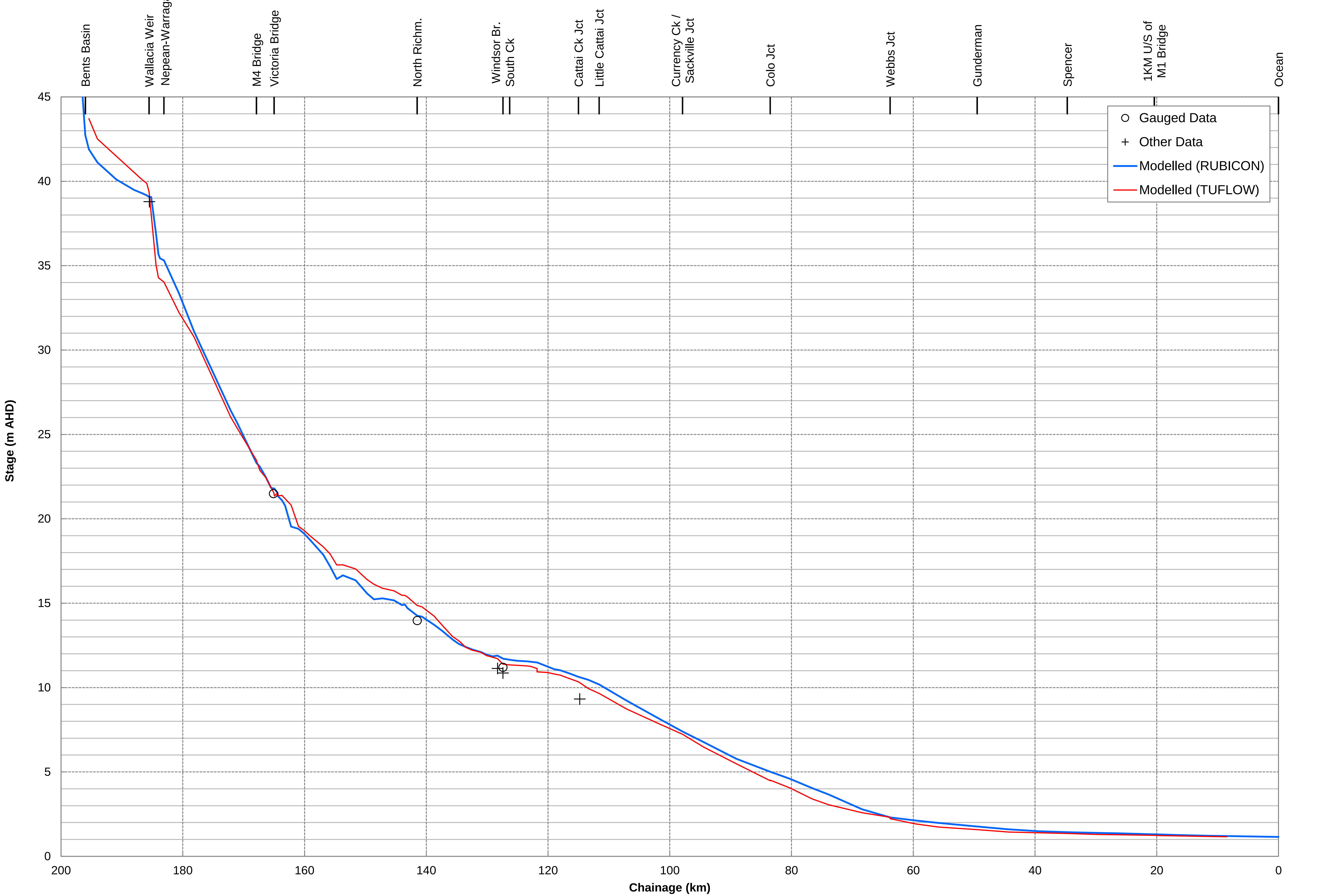

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FIGURE 11
STAGE HYDROGRAPHS
JUNE 1975

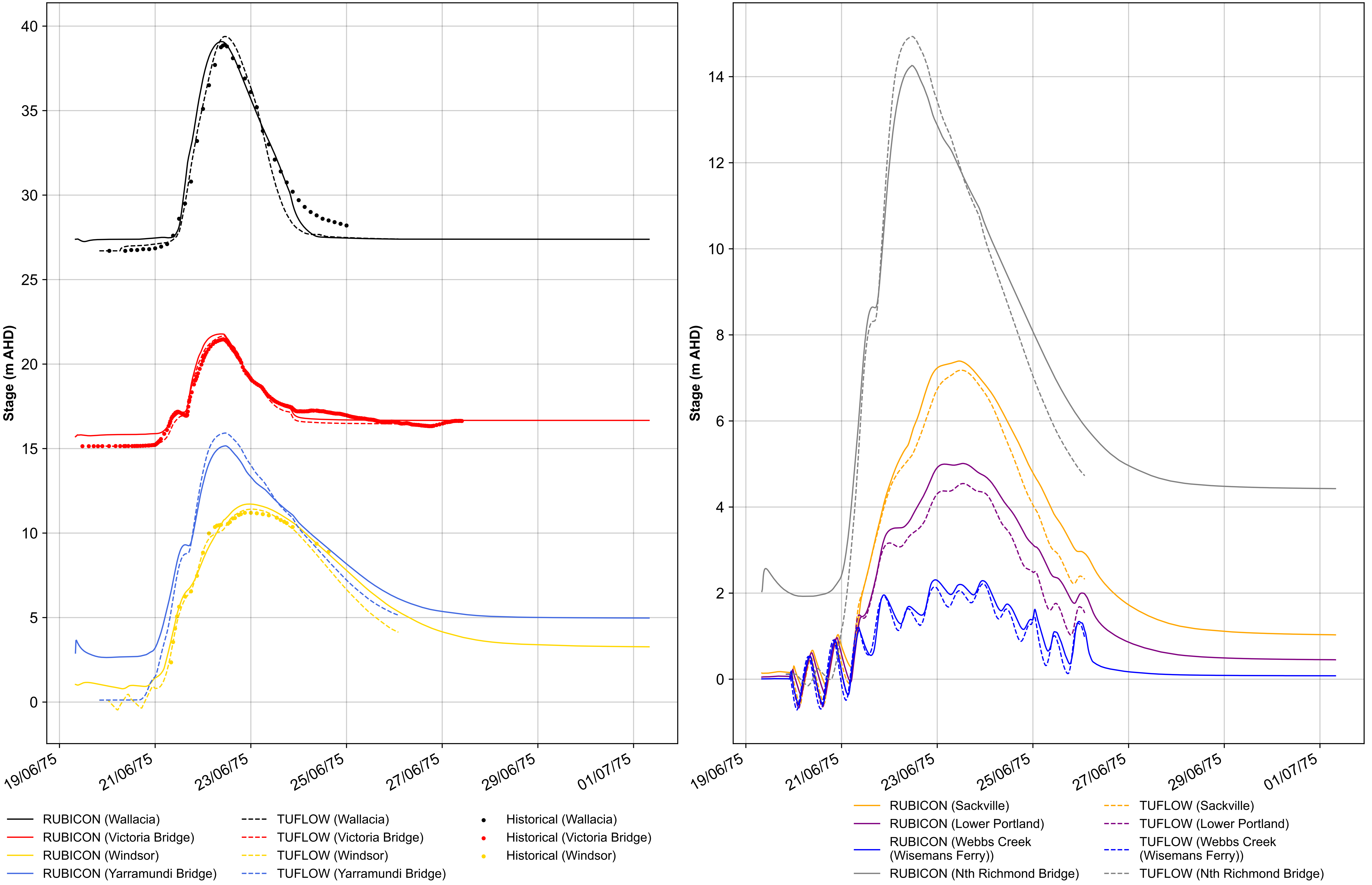

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FIGURE 12
FLOW HYDROGRAPHS
JUNE 1975

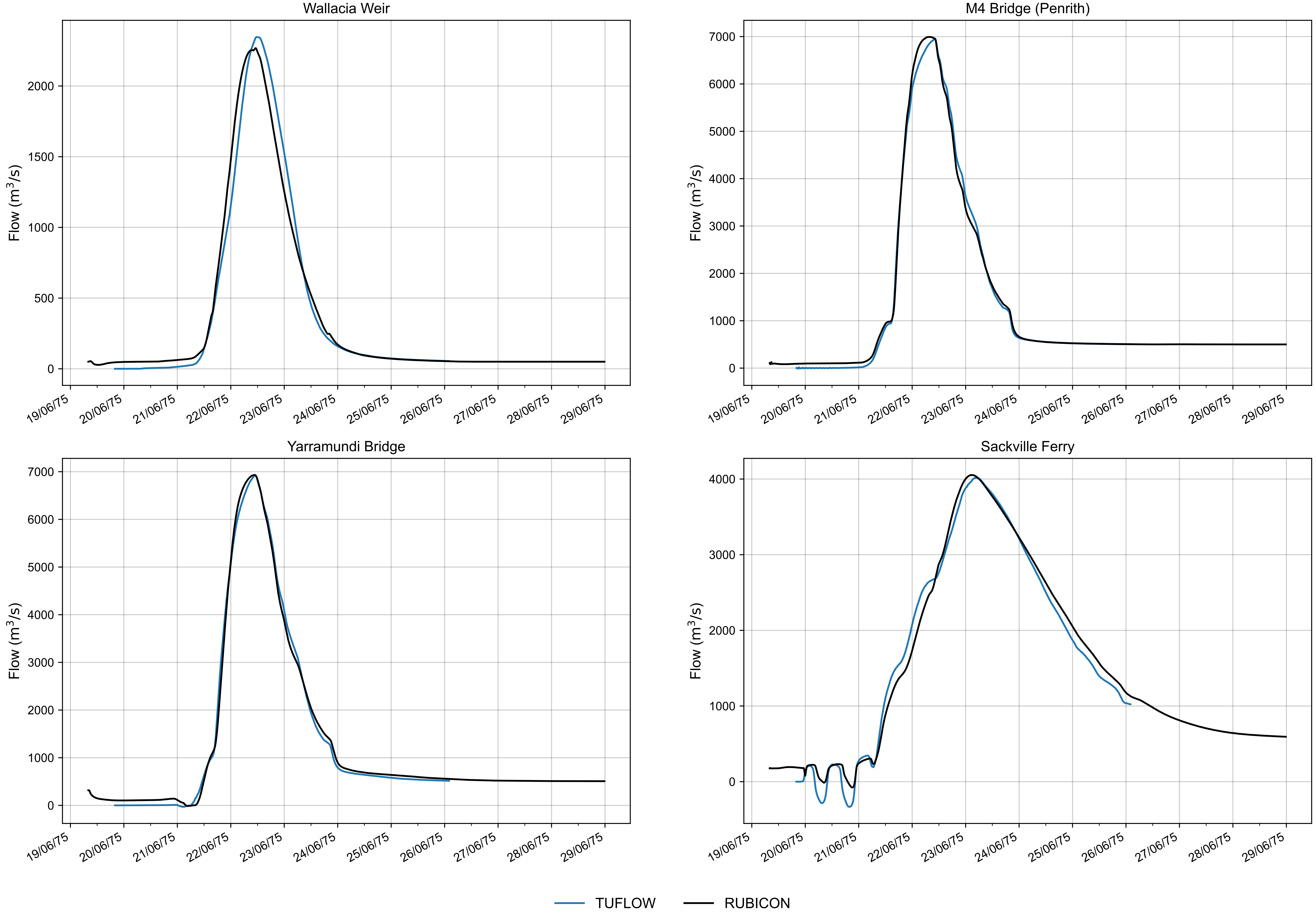

FIGURE 13
STAGE PROFILES
MARCH 1978

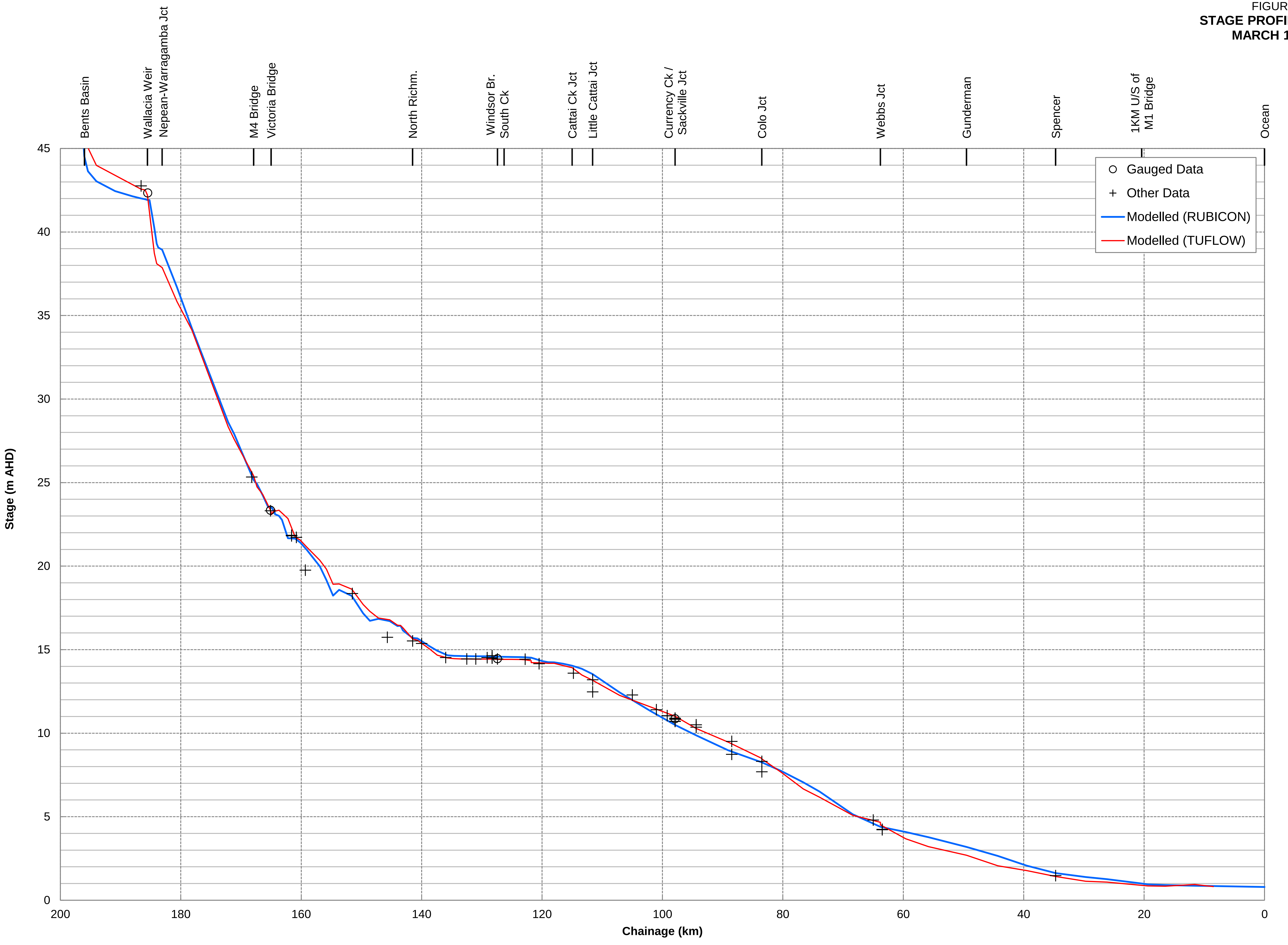

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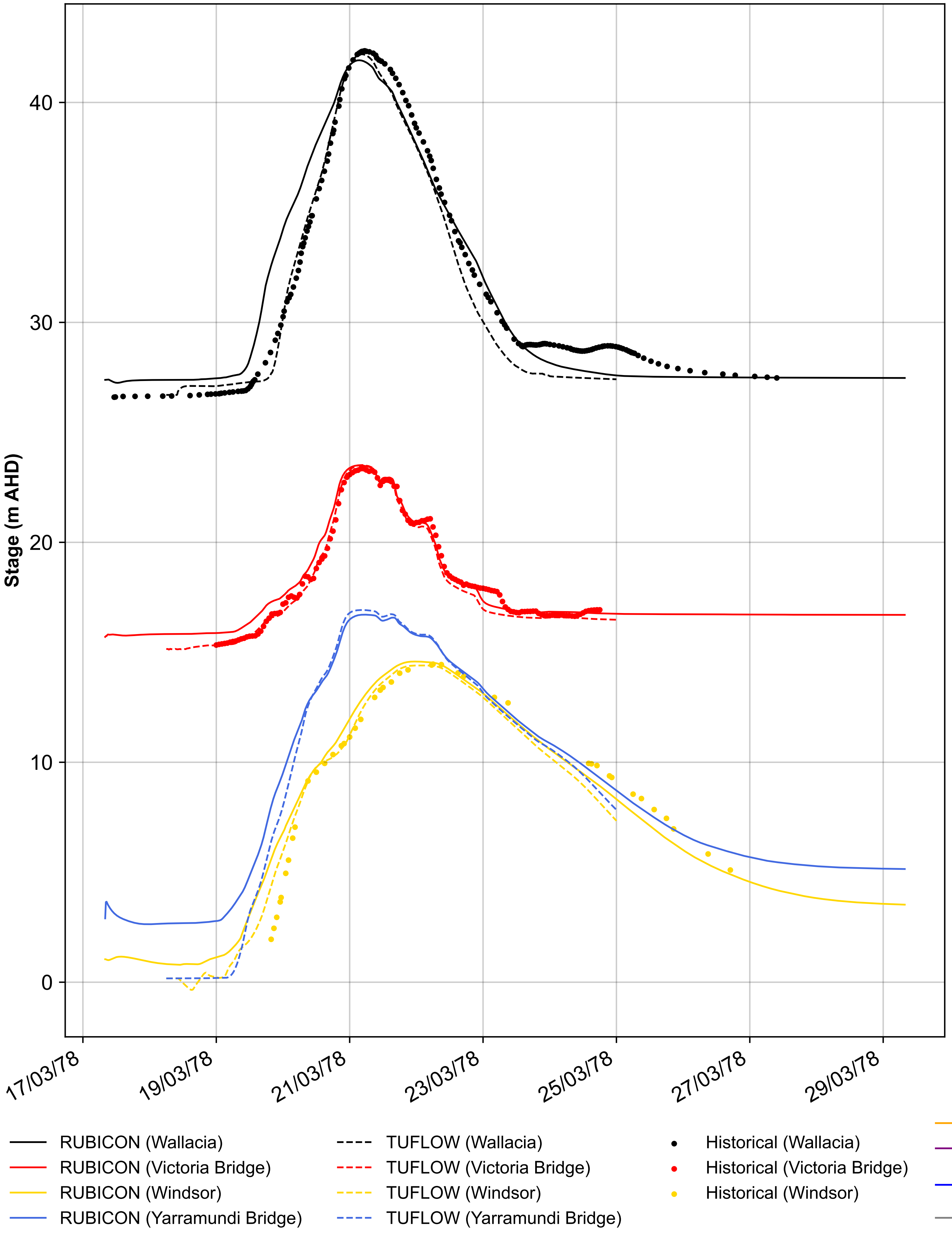
FIGURE 14
STAGE HYDROGRAPHS
MARCH 1978



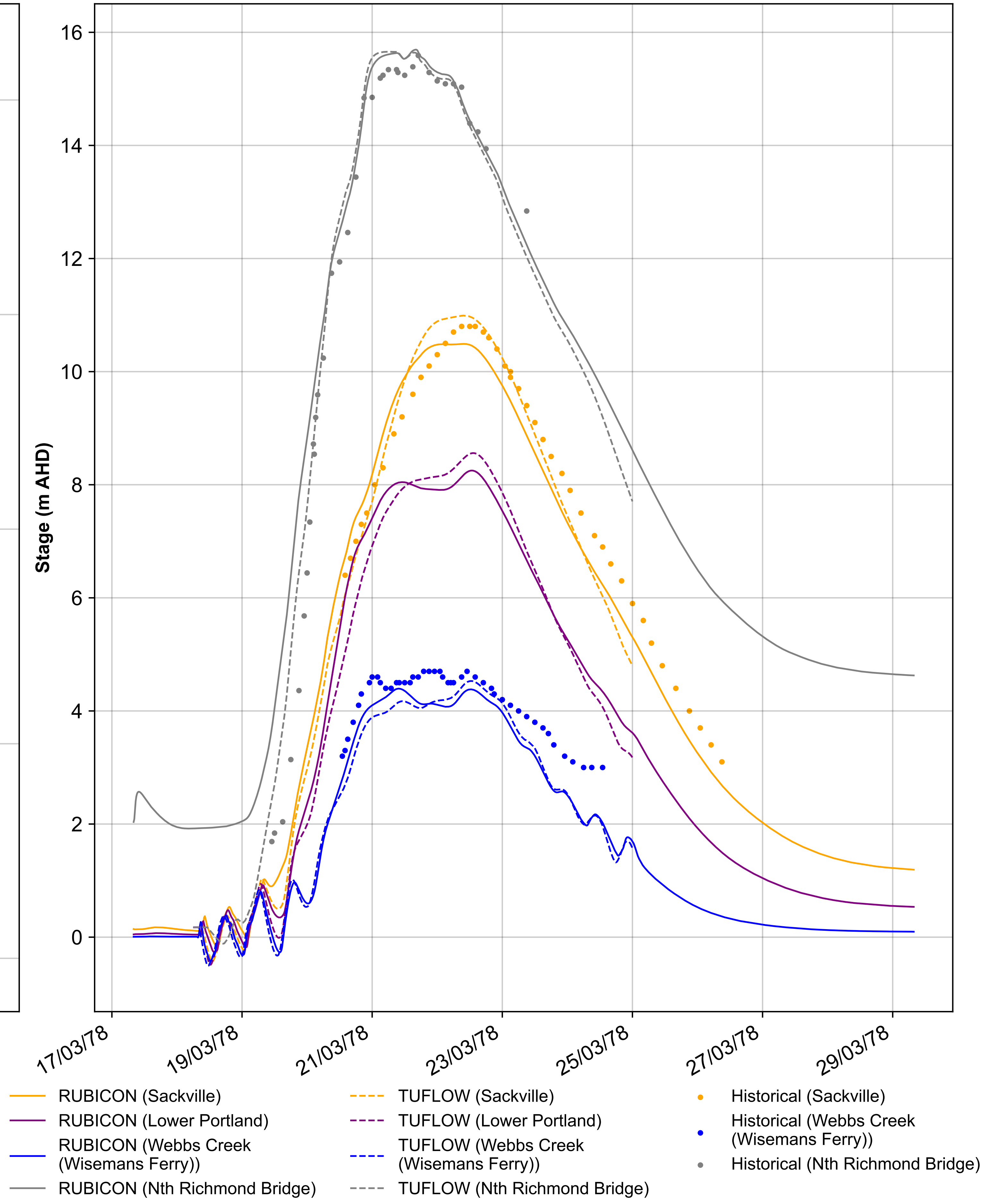


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FIGURE 15
FLOW HYDROGRAPHS
MARCH 1978

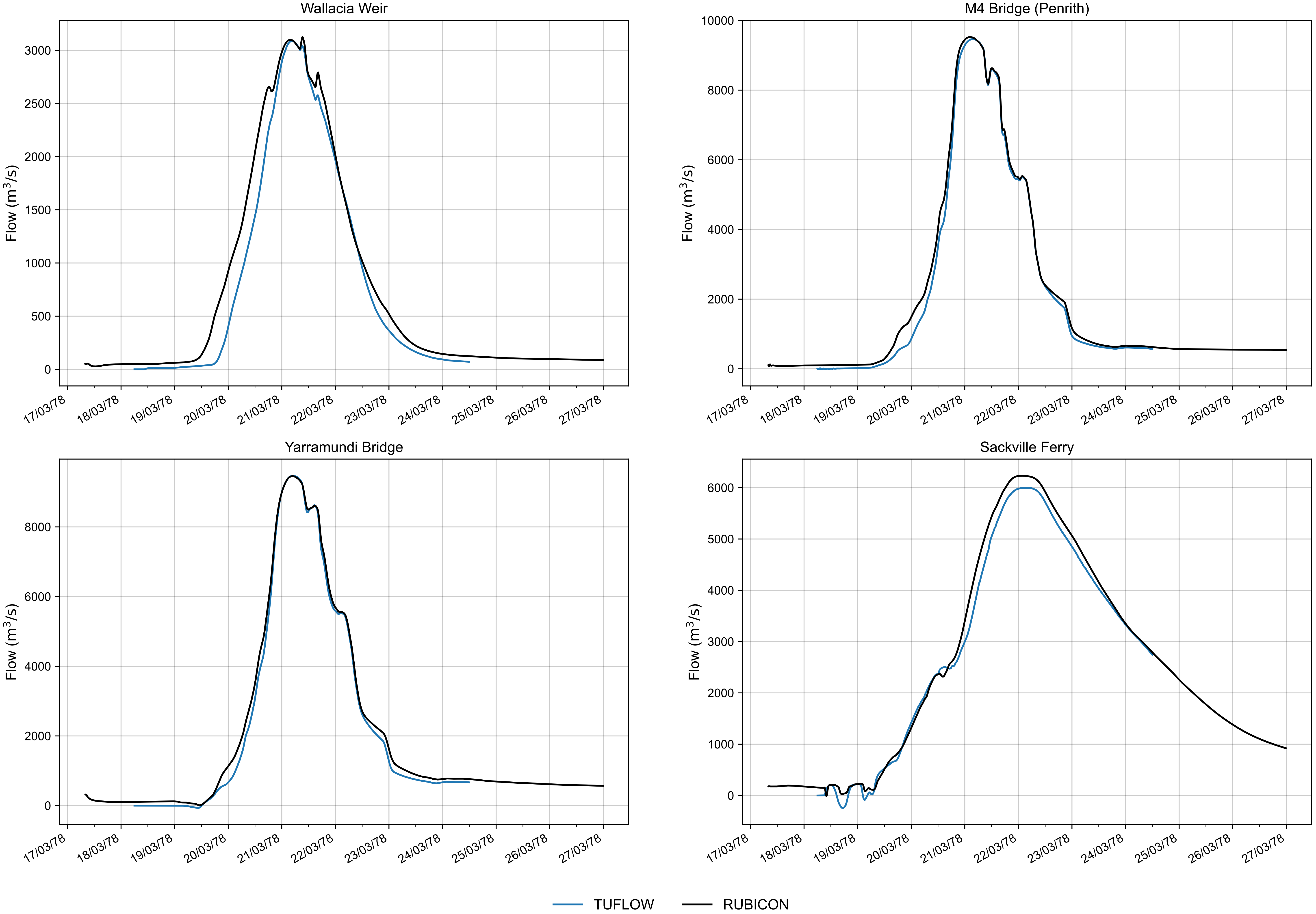

FIGURE 16
STAGE PROFILES
AUGUST 1986

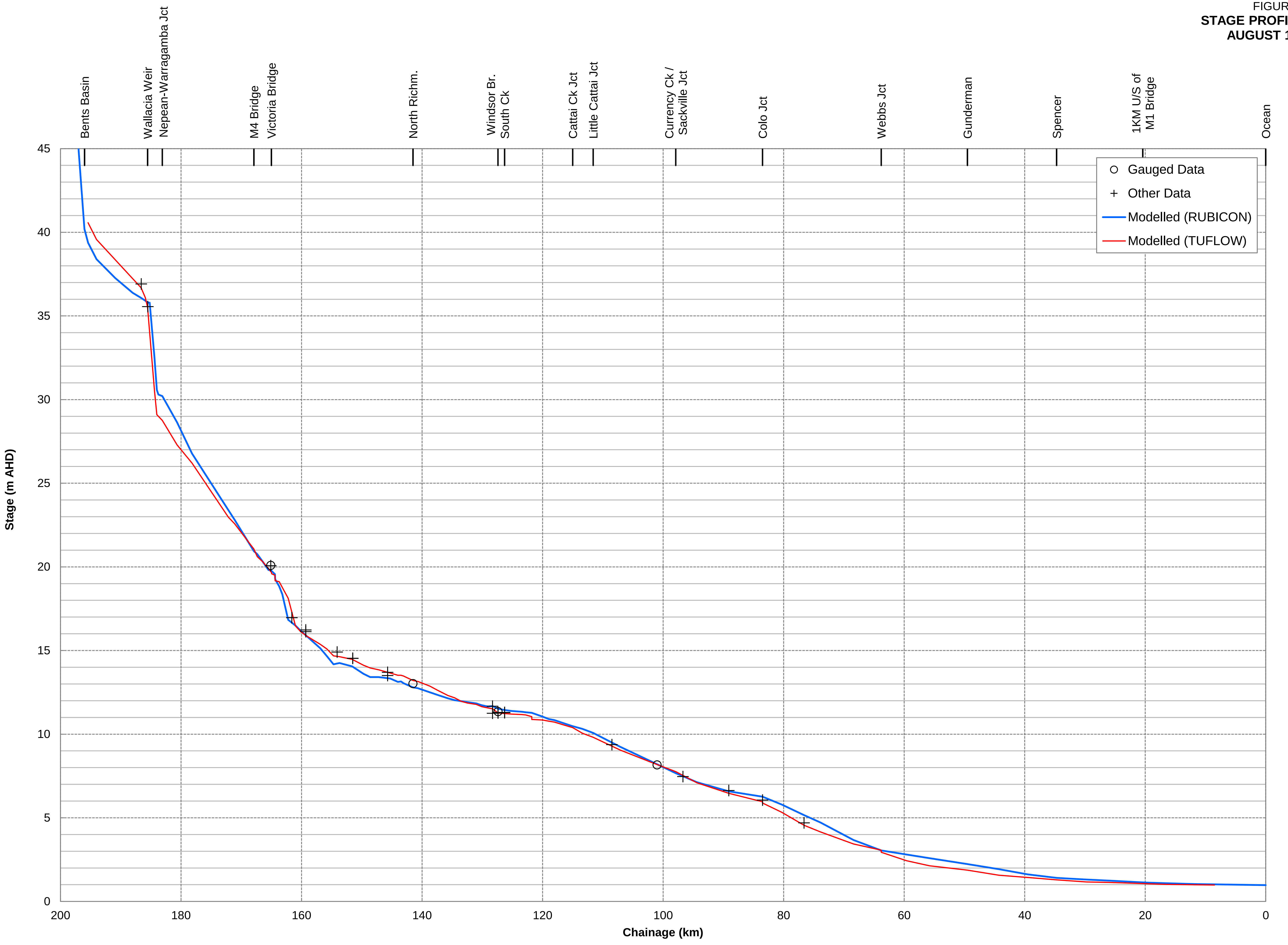

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FIGURE 17
STAGE HYDROGRAPHS
AUGUST 1986

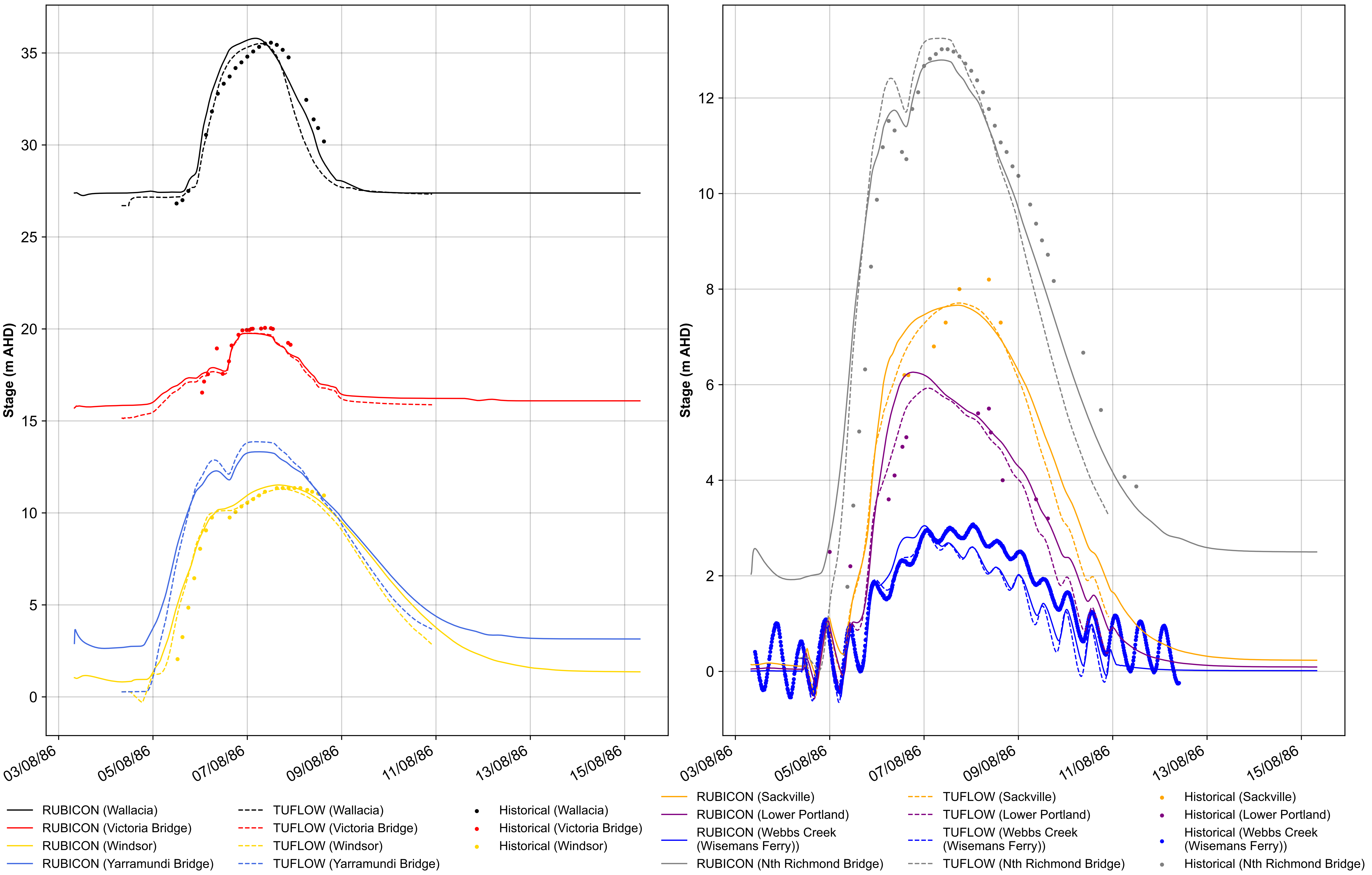

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FIGURE 18
FLOW HYDROGRAPHS
AUGUST 1986

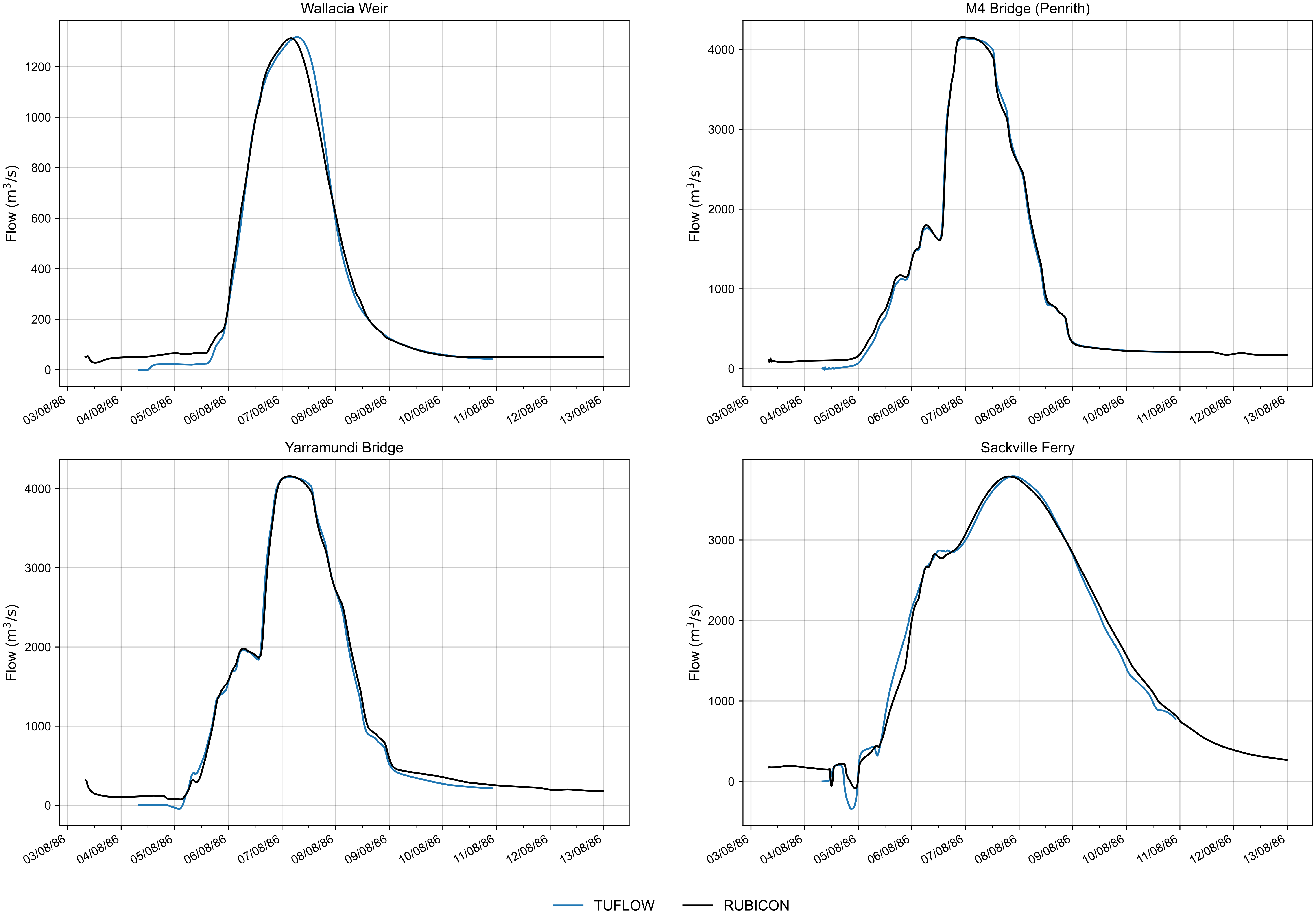

FIGURE 19
STAGE PROFILES
APRIL/MAY 1988

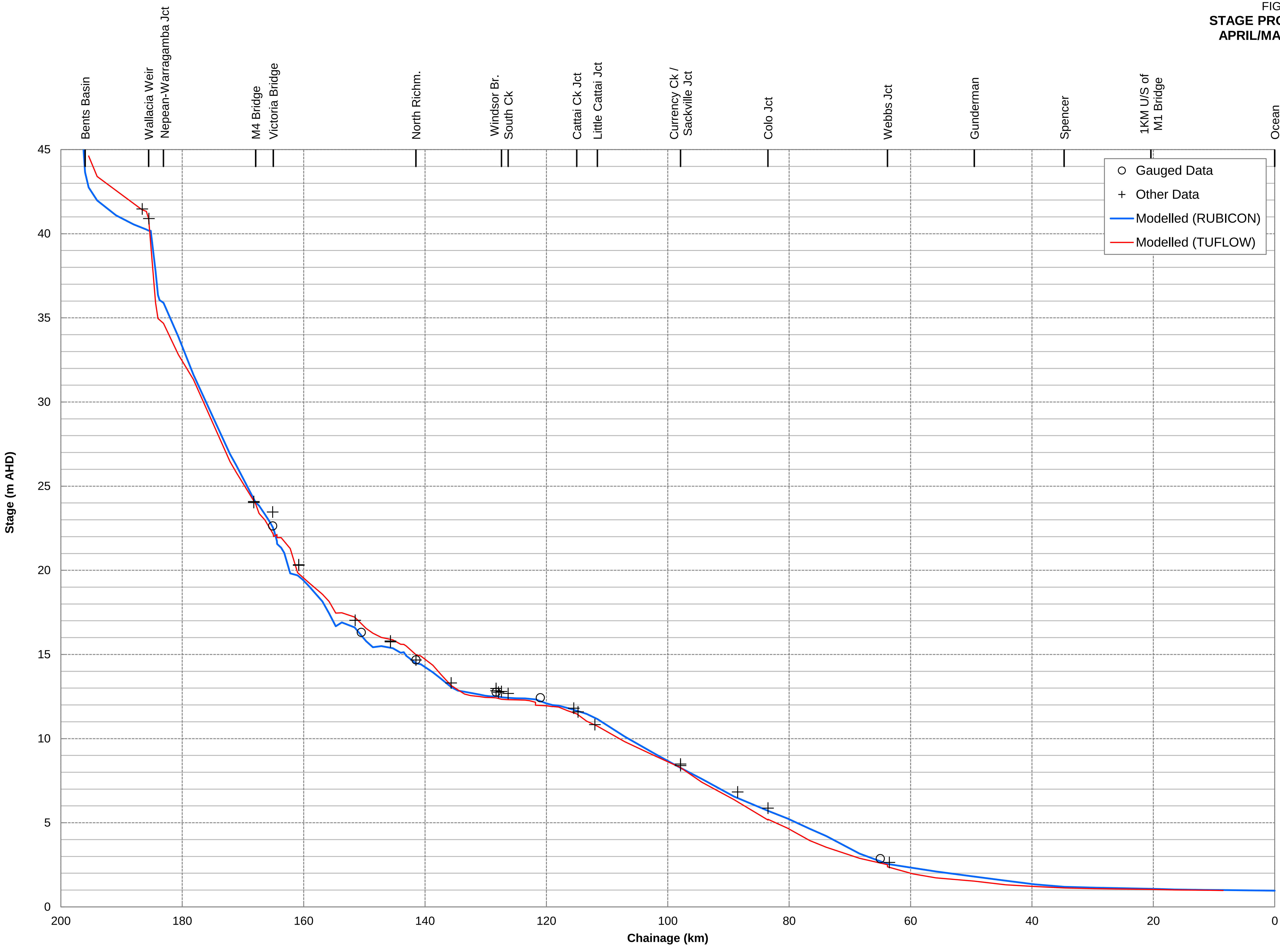

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FIGURE 20
STAGE HYDROGRAPHS
APRIL/MAY 1988

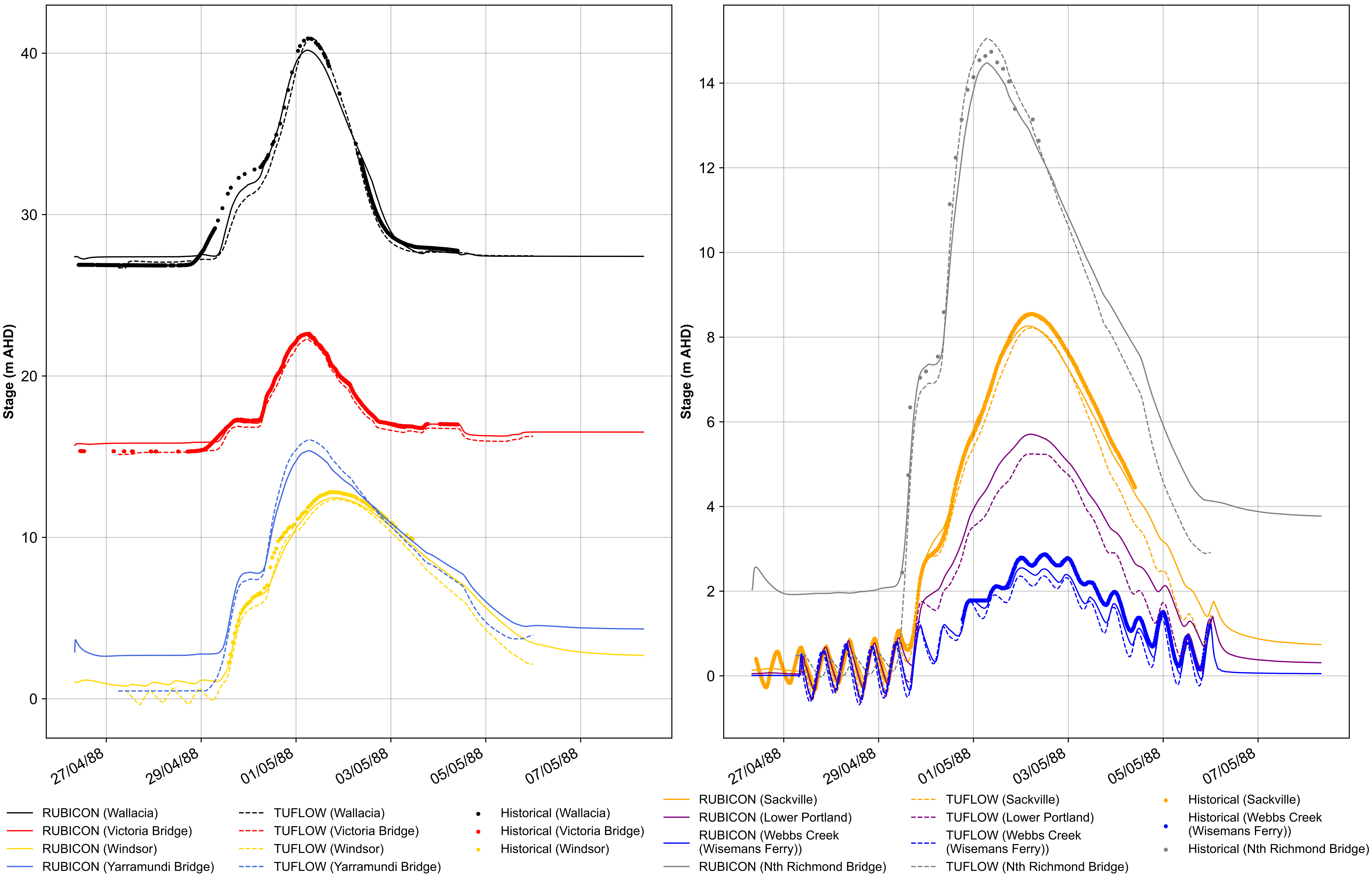

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FIGURE 21
FLOW HYDROGRAPHS
APRIL/MAY 1988

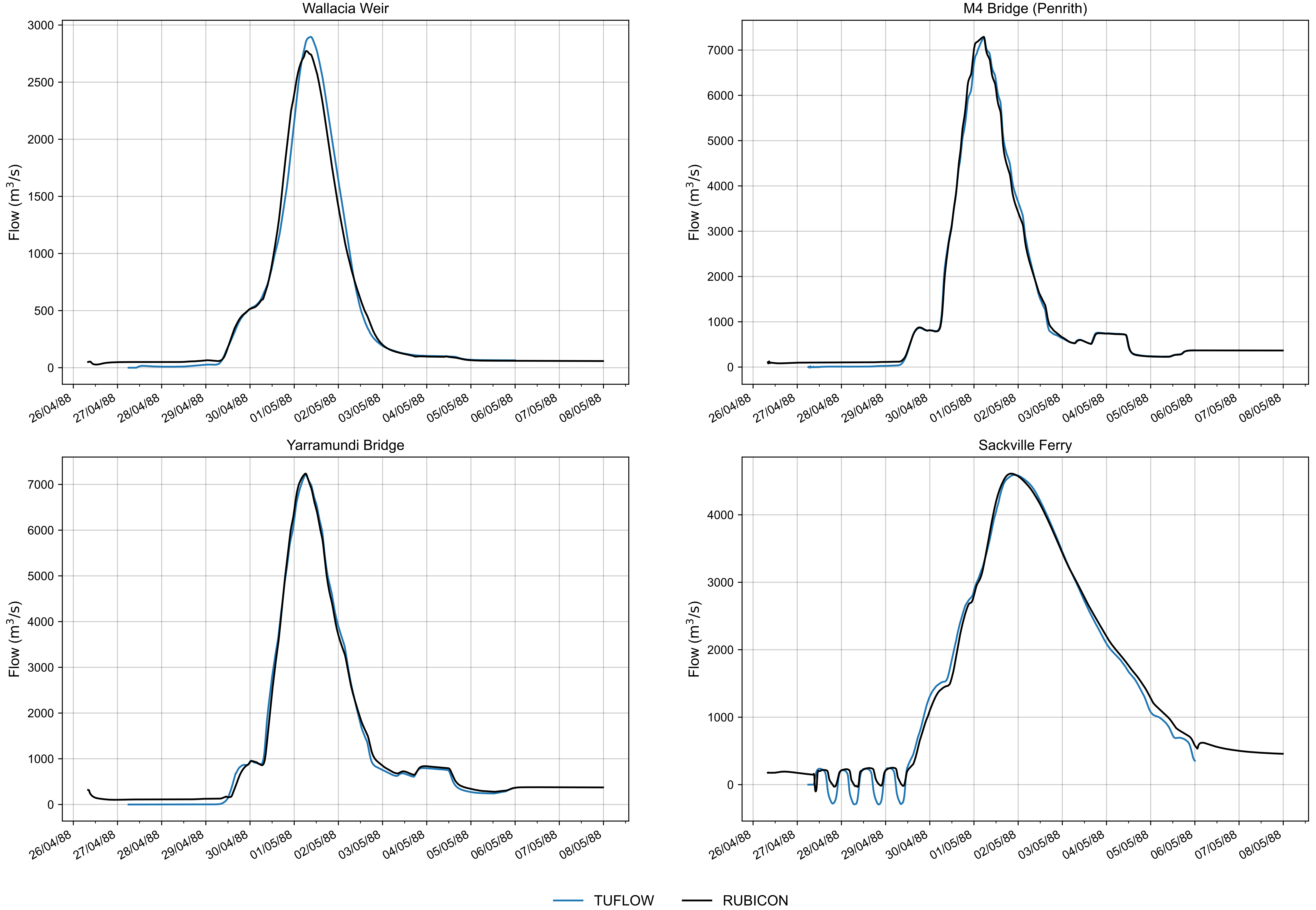

FIGURE 22
STAGE PROFILES
AUGUST 1990

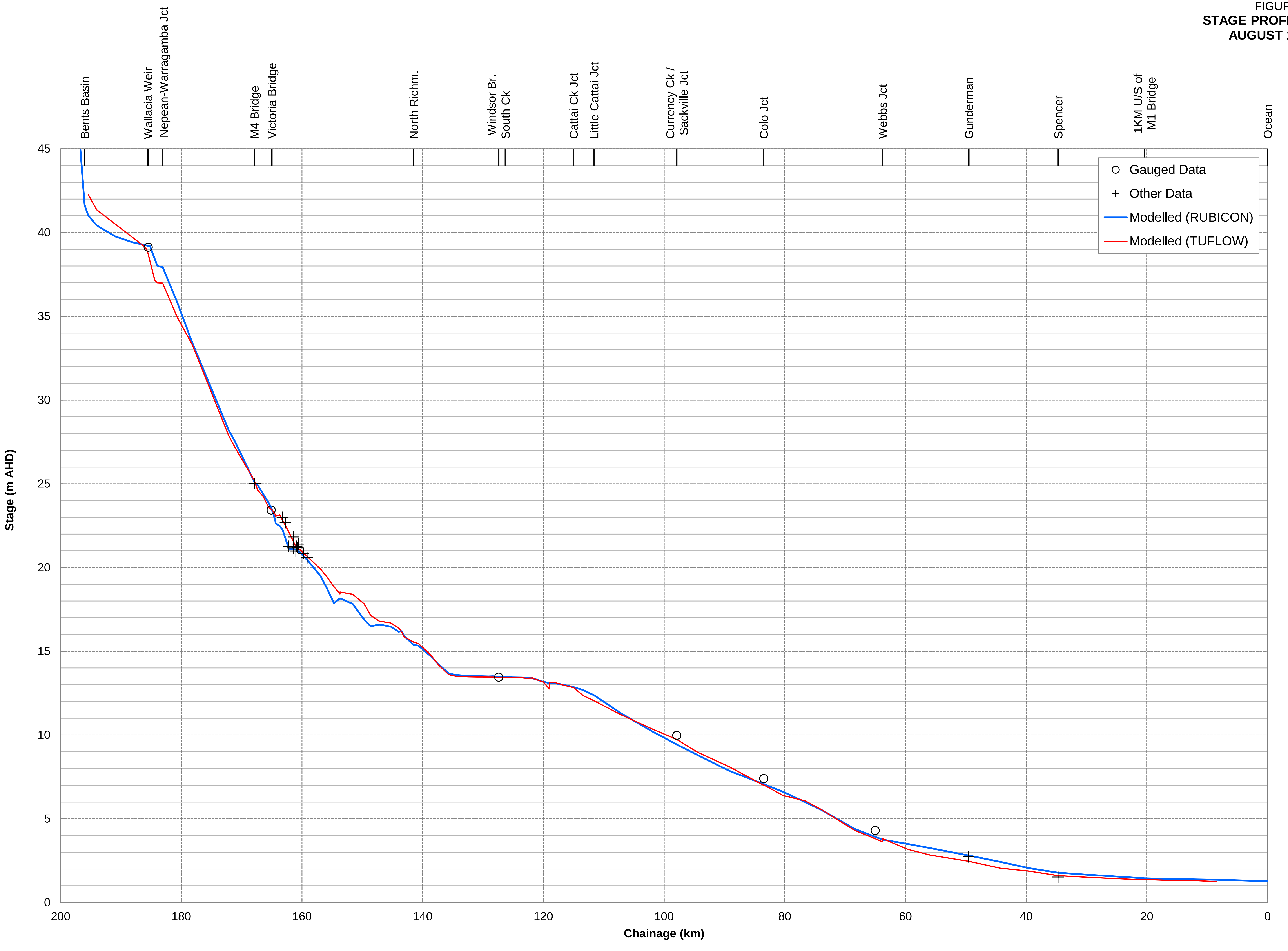

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FIGURE 23
STAGE HYDROGRAPHS
AUGUST 1990

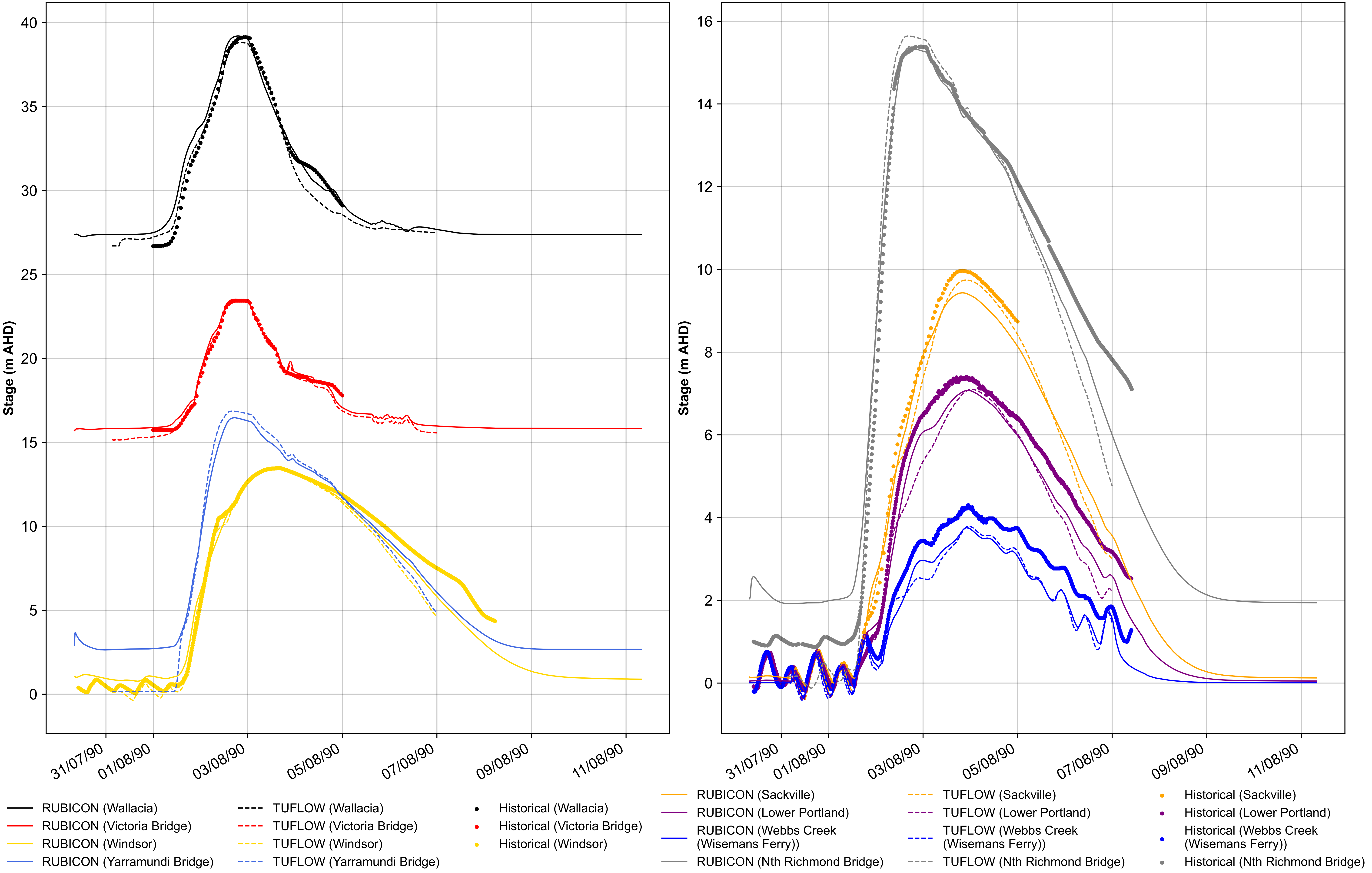

FIGURE 24
FLOW HYDROGRAPHS
AUGUST 1990

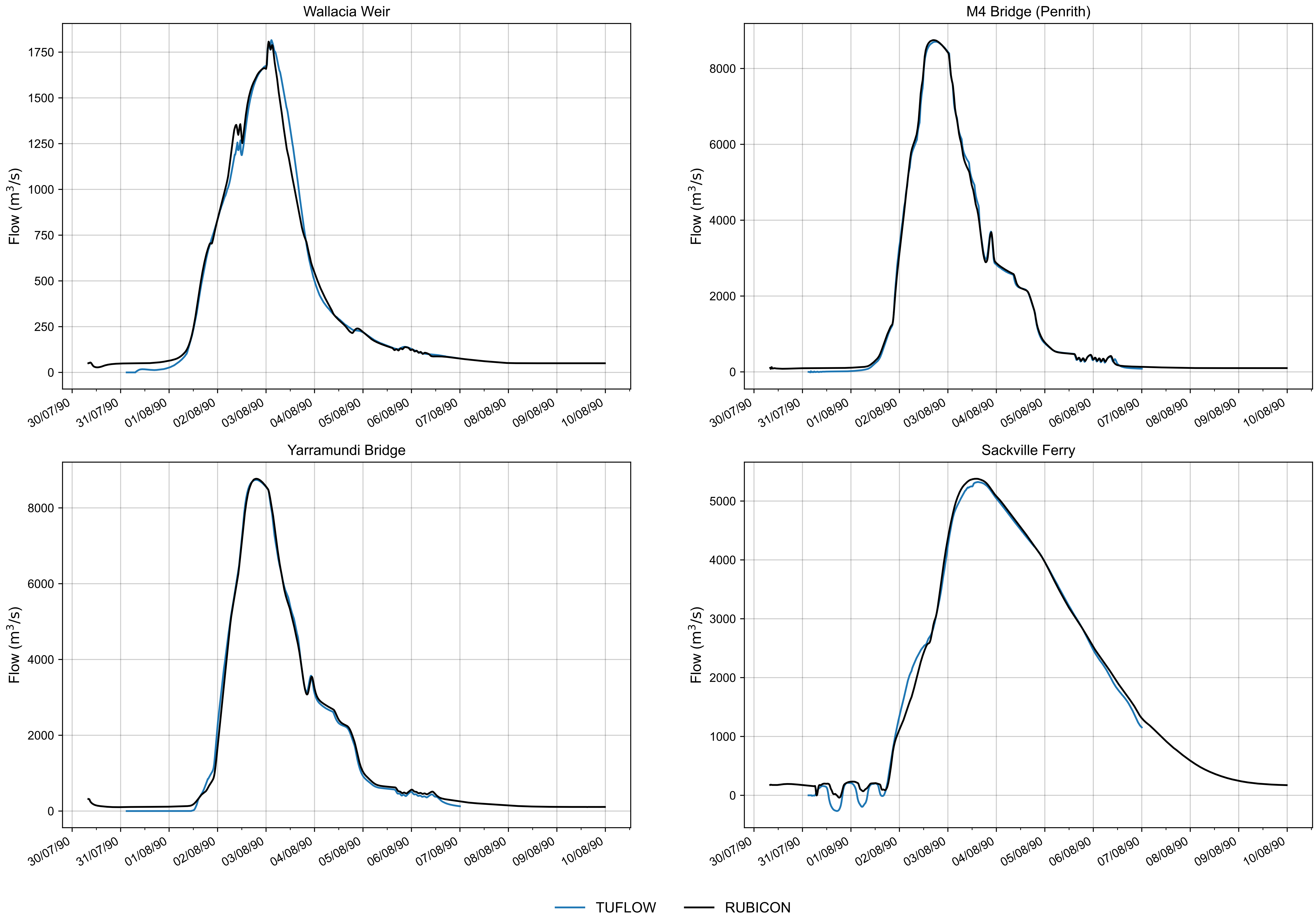

FIGURE 25
STAGE-FLOW RELATIONSHIP
WALLACIA WEIR

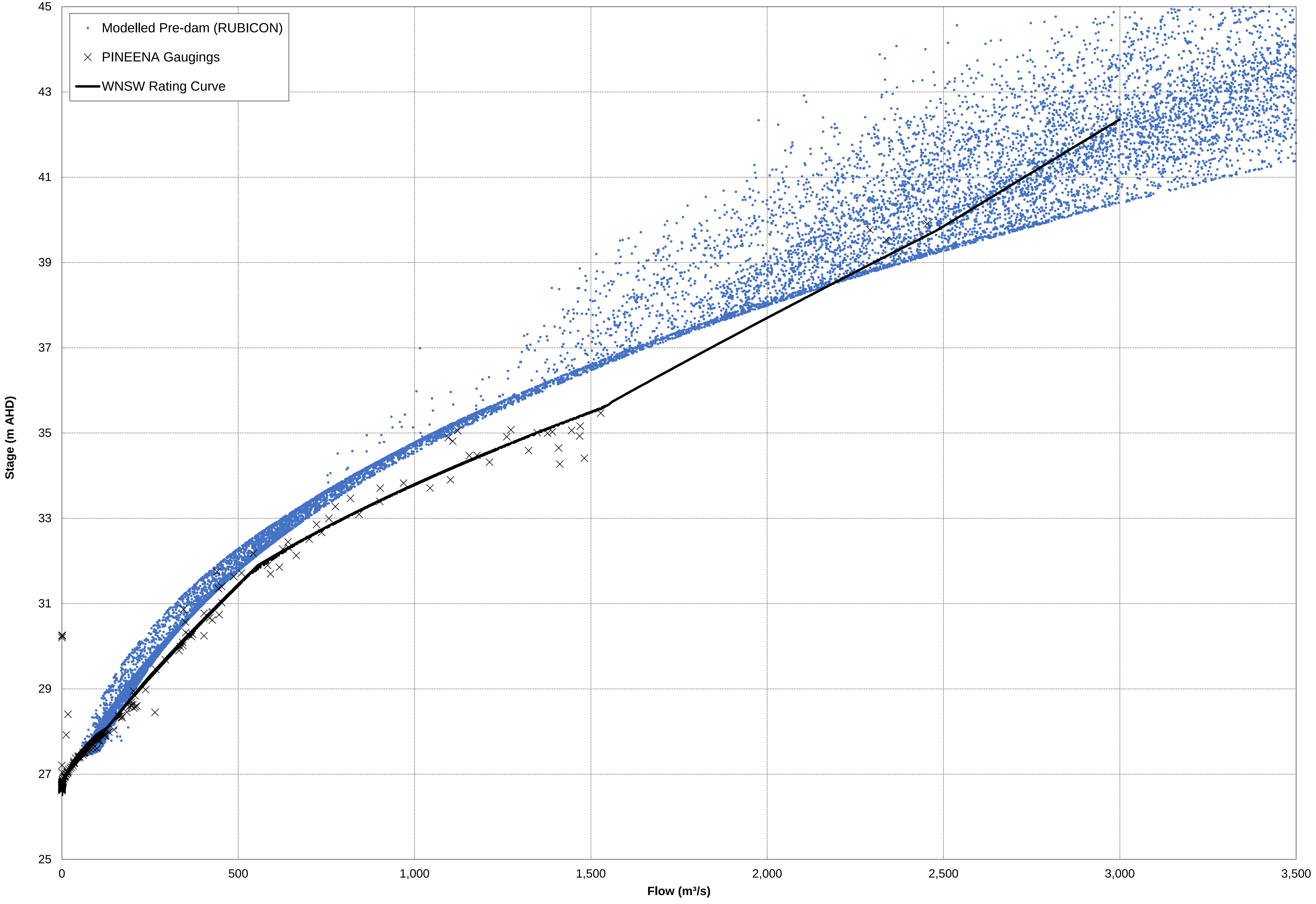

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FIGURE 26
STAGE-FLOW RELATIONSHIP
PENRITH (VICTORIA BRIDGE)

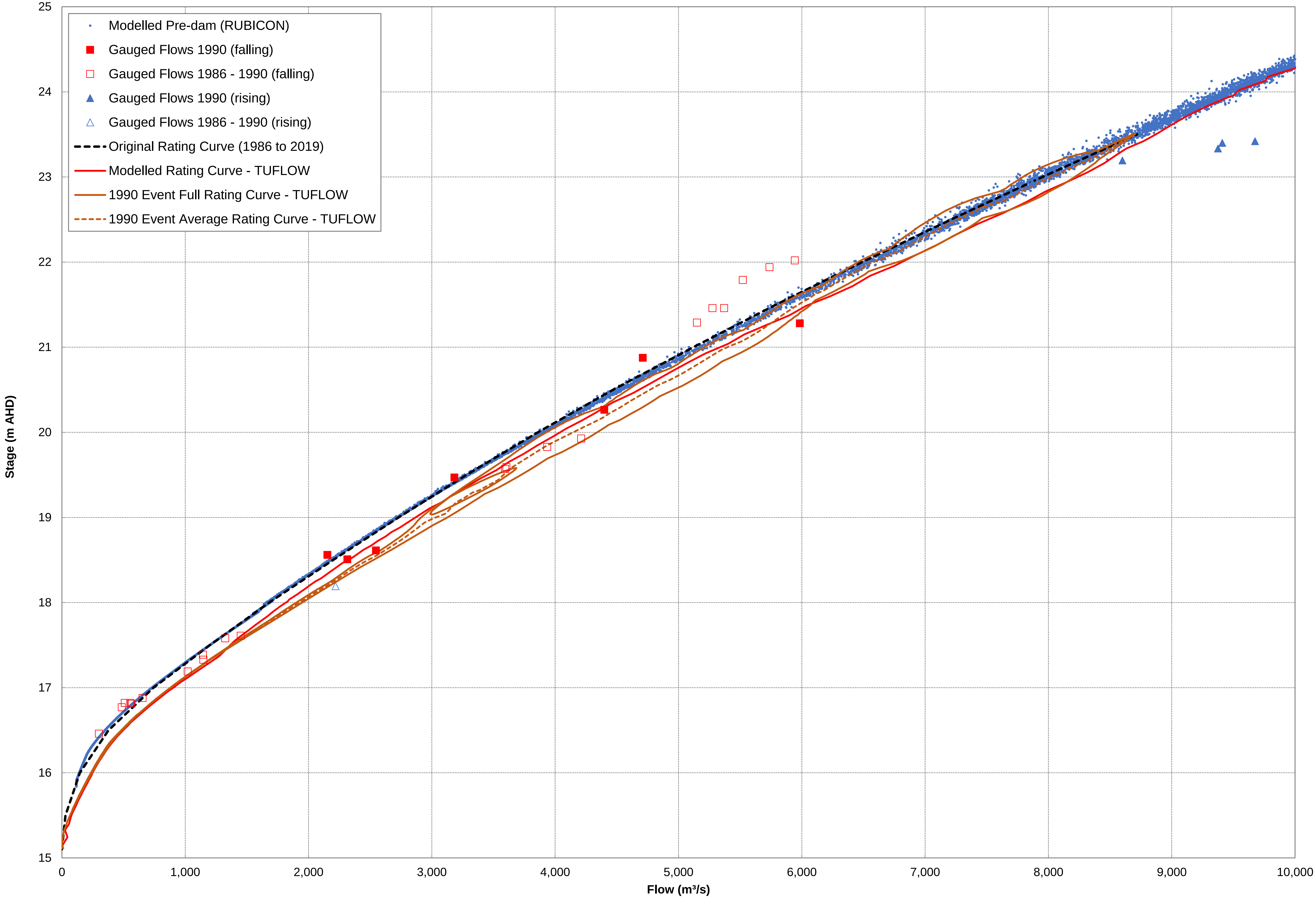

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FIGURE 27
STAGE-FLOW RELATIONSHIP
WINDSOR-SACKVILLE

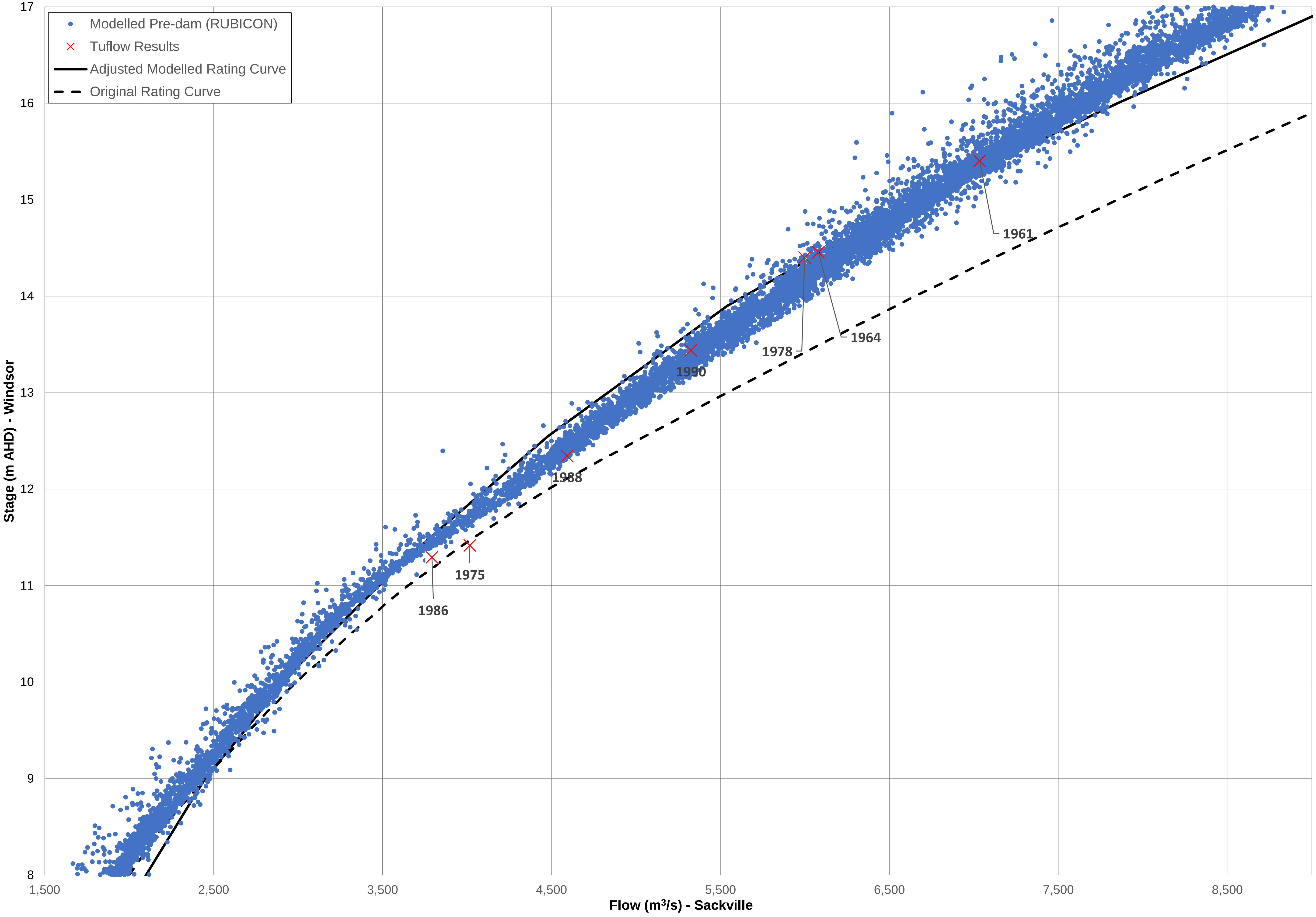

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FIGURE 28
STAGE-FLOW RELATIONSHIP
SACKVILLE

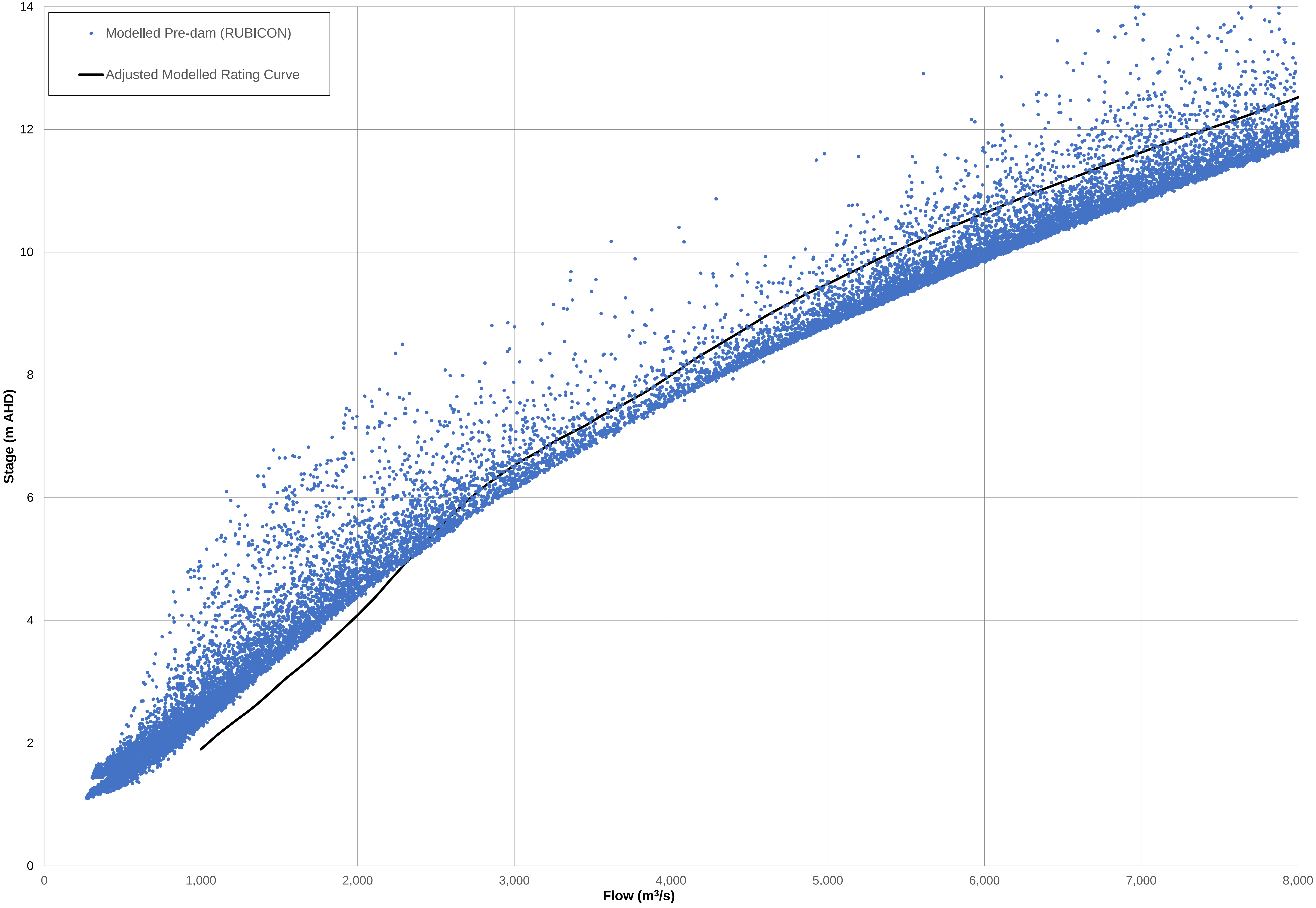

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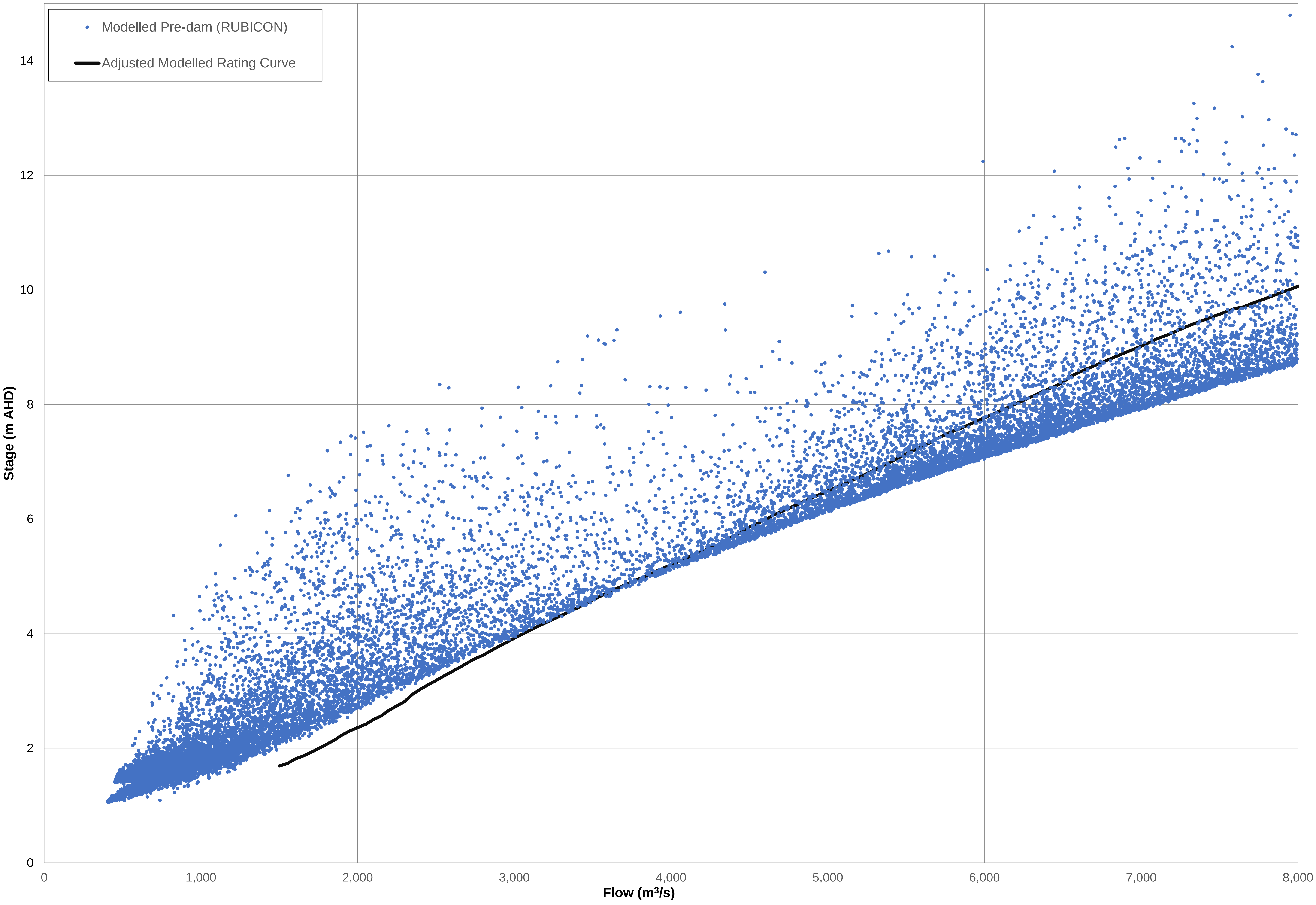
FIGURE 29
STAGE-FLOW RELATIONSHIP
COLO JUNCTION



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STAGE-FLOW RELATIONSHIP
WEBBS CREEK

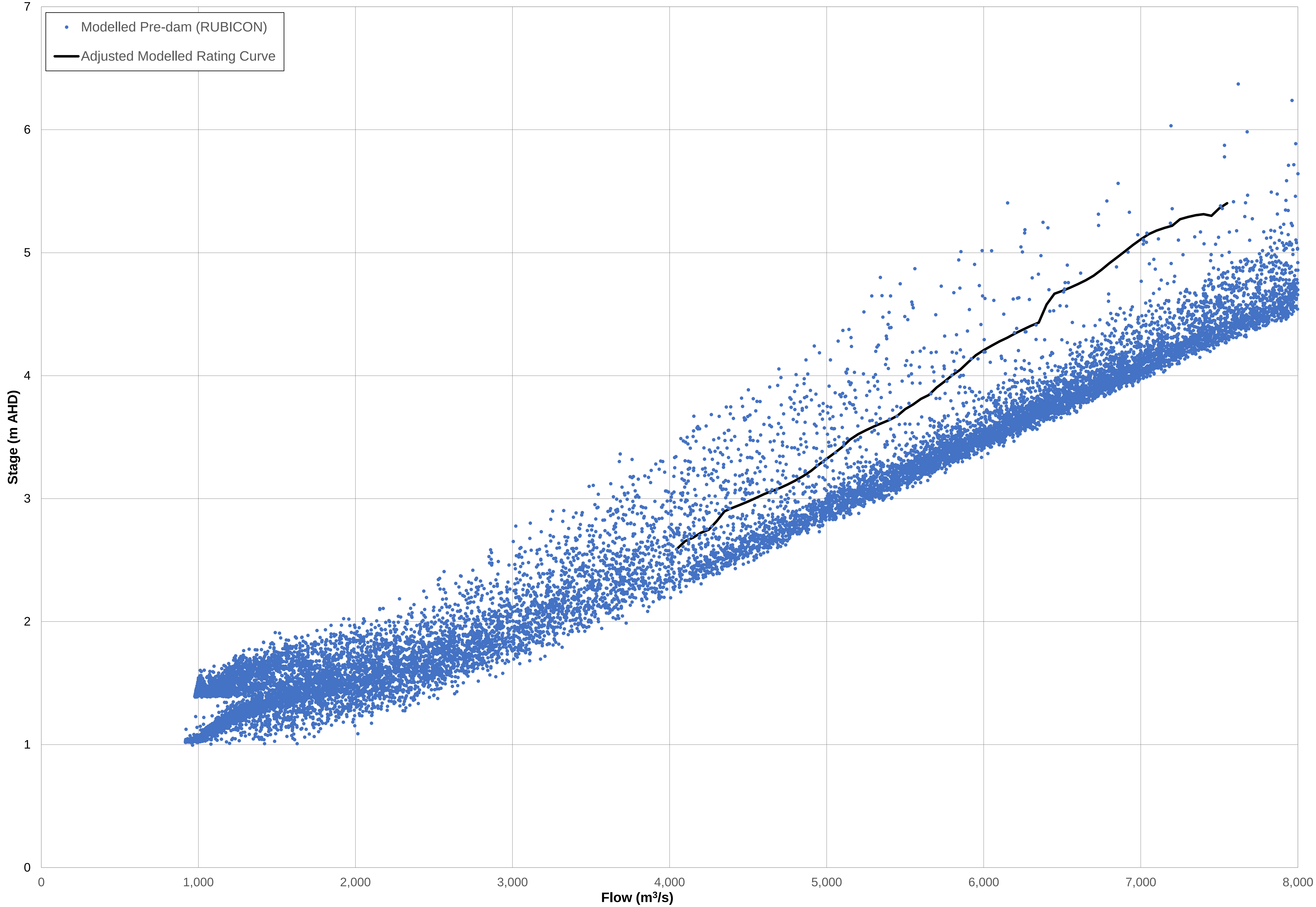

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FIGURE 31

STAGE FREQUENCY
WALLACIA WEIR

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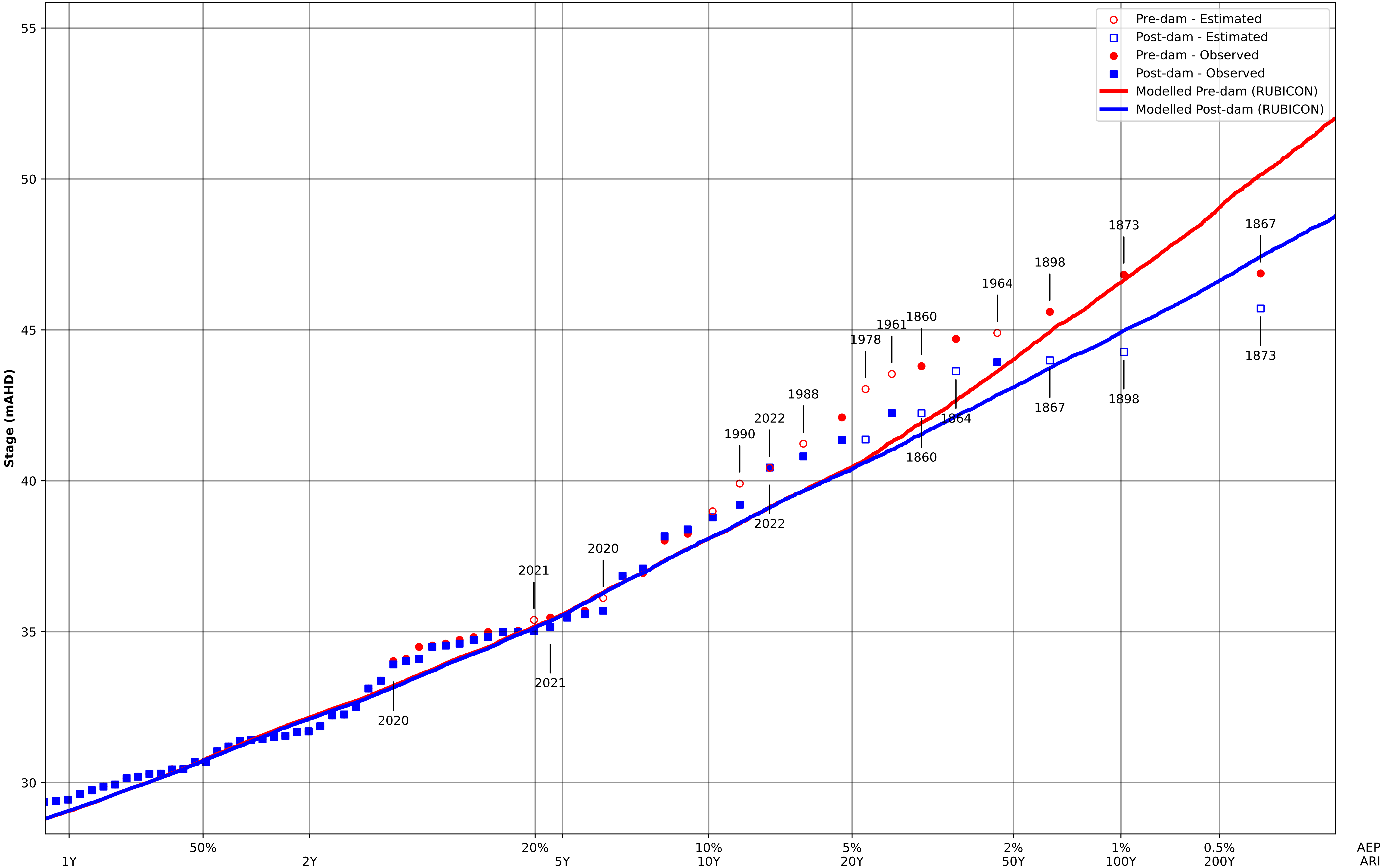

FIGURE 32

FLOW FREQUENCY
WARRAGAMBA DAM

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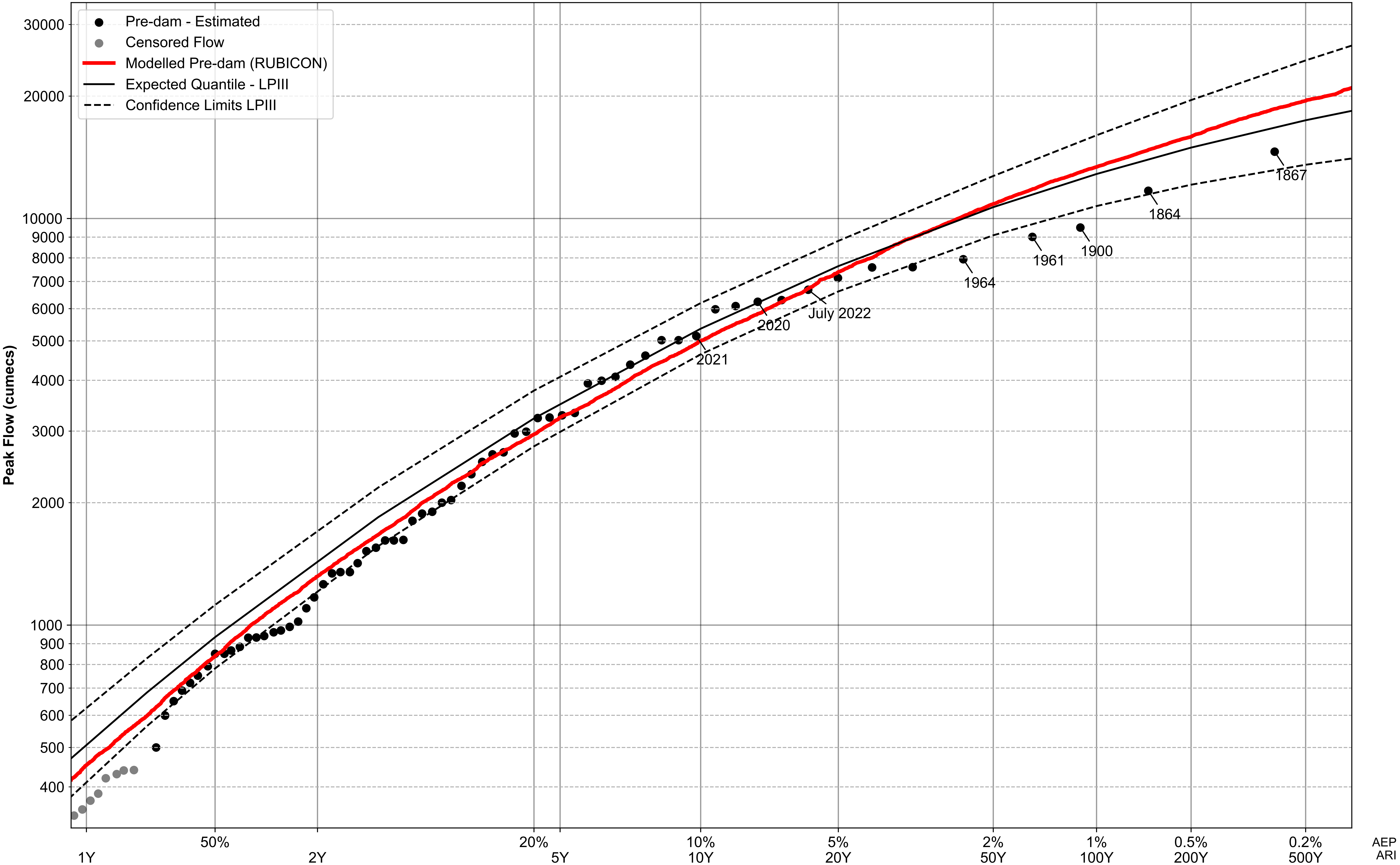

FLOW FREQUENCY
PENRITH

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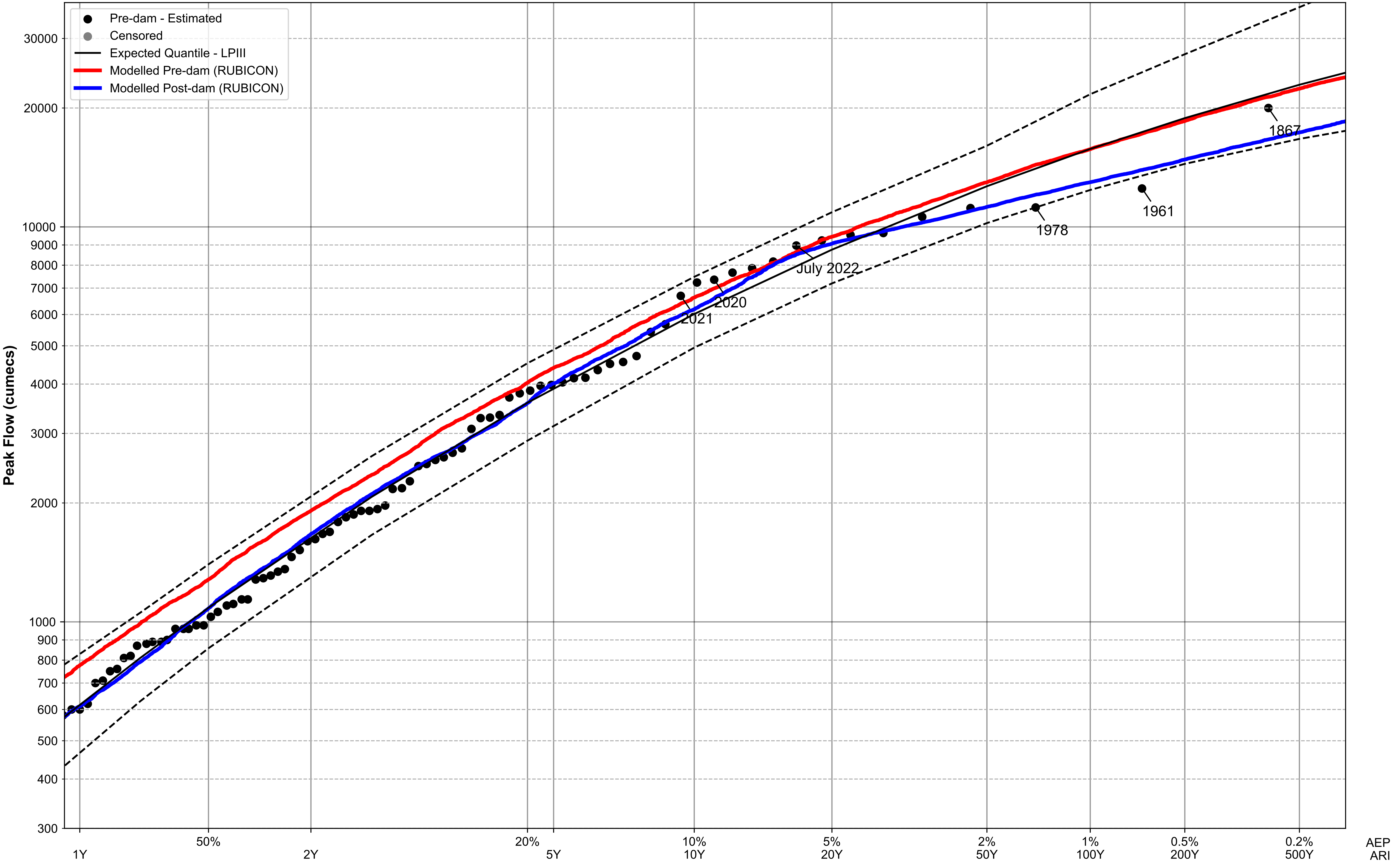

FIGURE 34

FLOW FREQUENCY
WINDSOR-SACKVILLE

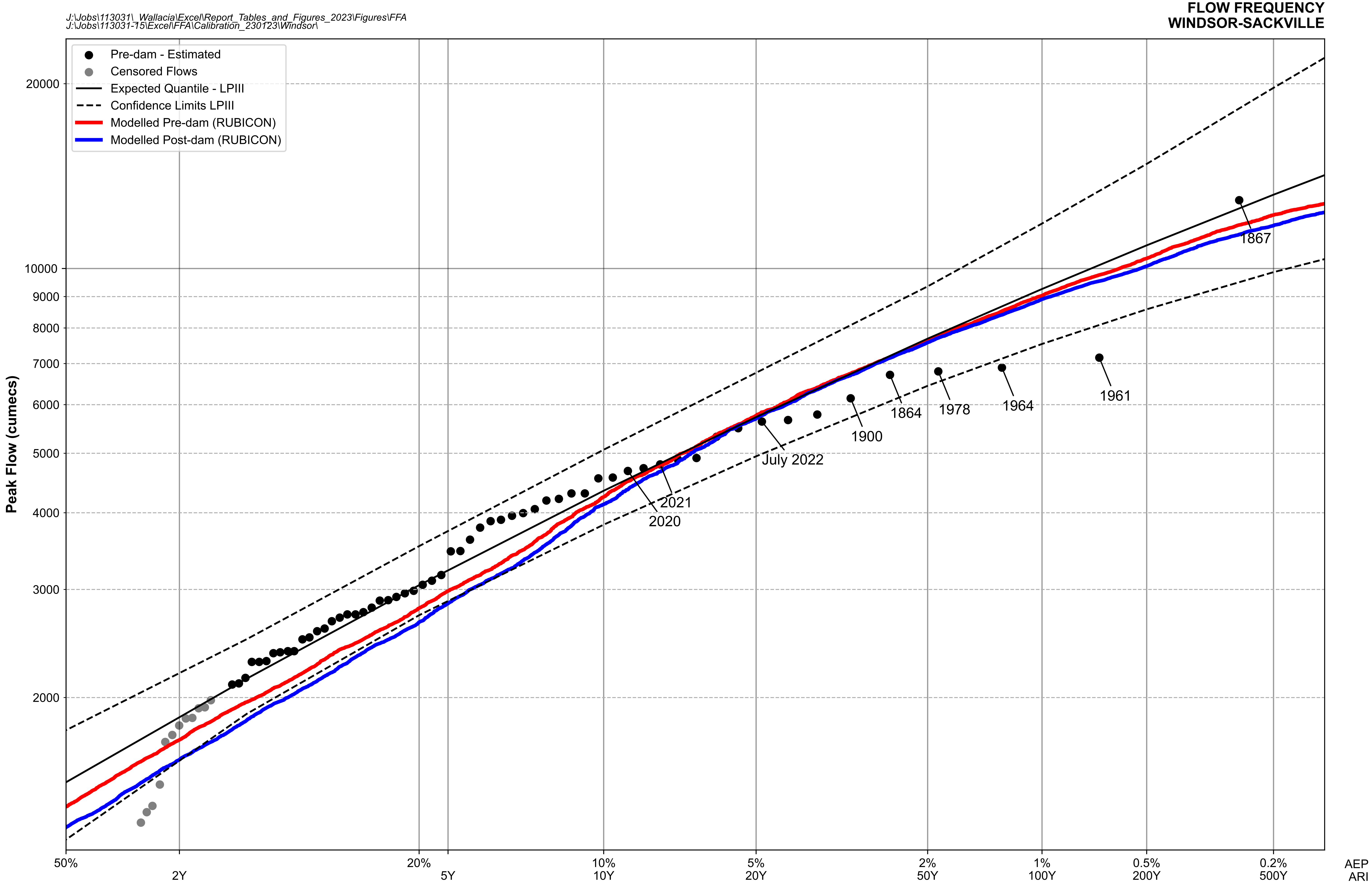

FIGURE 35

STAGE FREQUENCY
SACKVILLE

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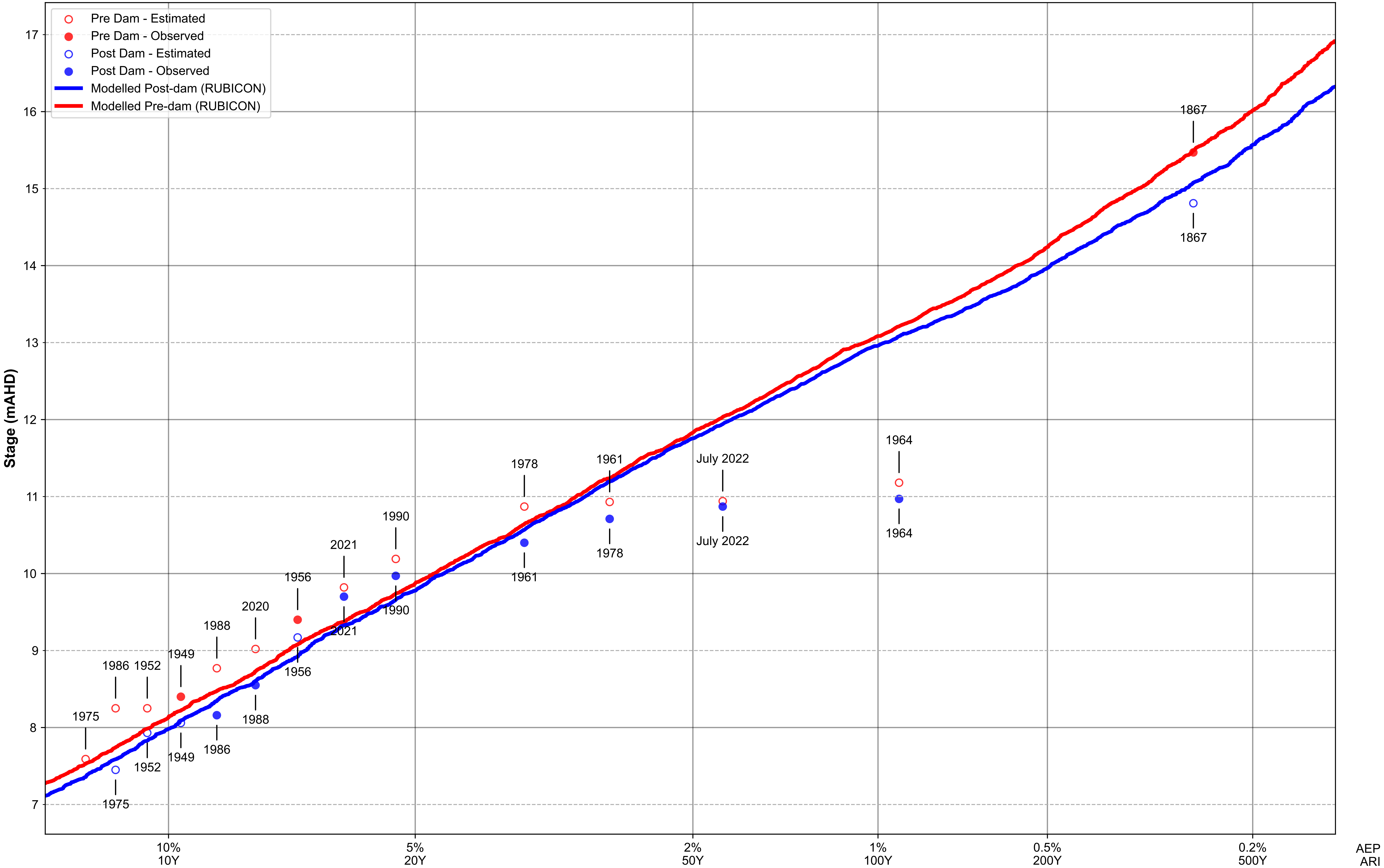

FIGURE 36

STAGE FREQUENCY
COLO JUNCTION

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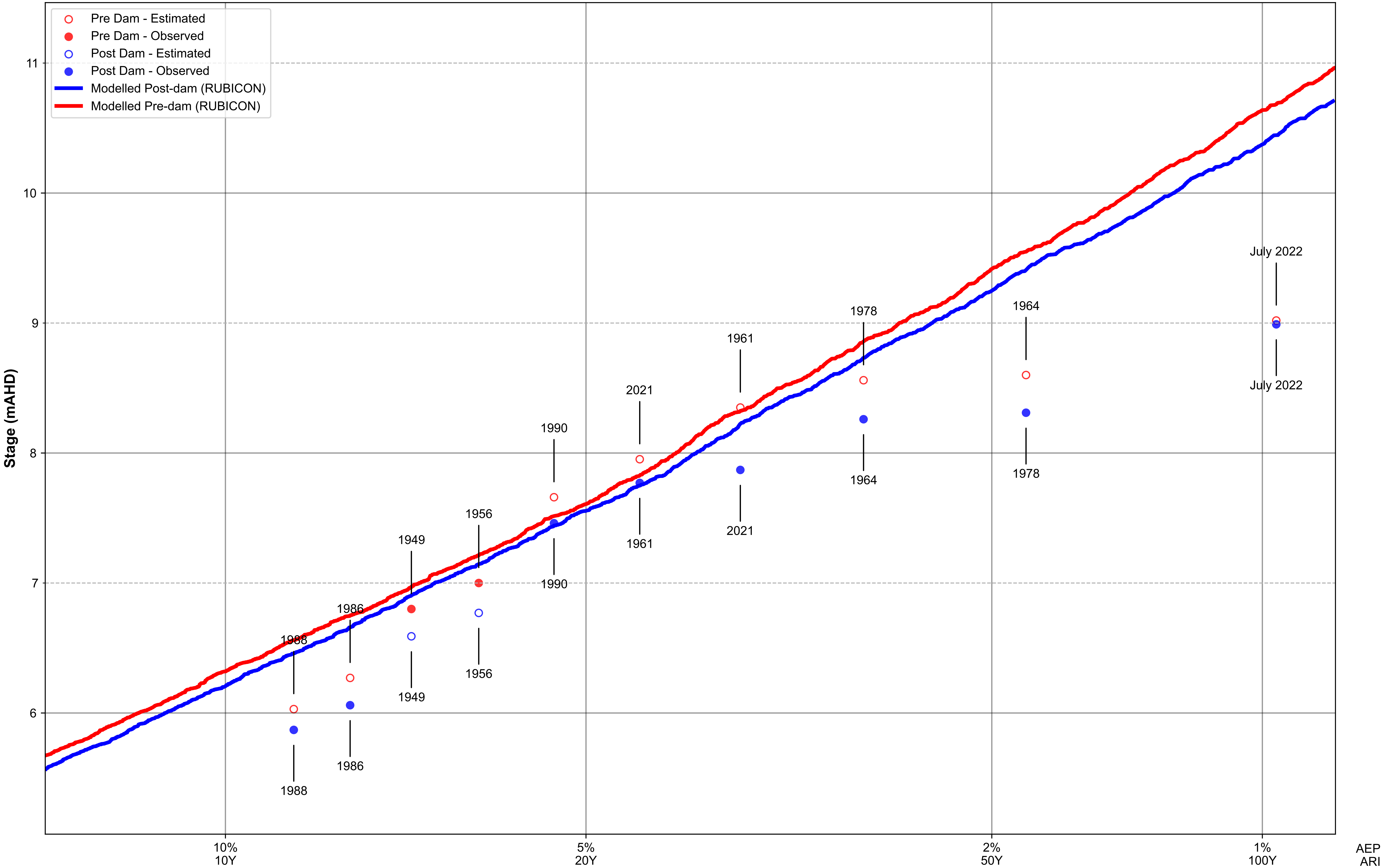
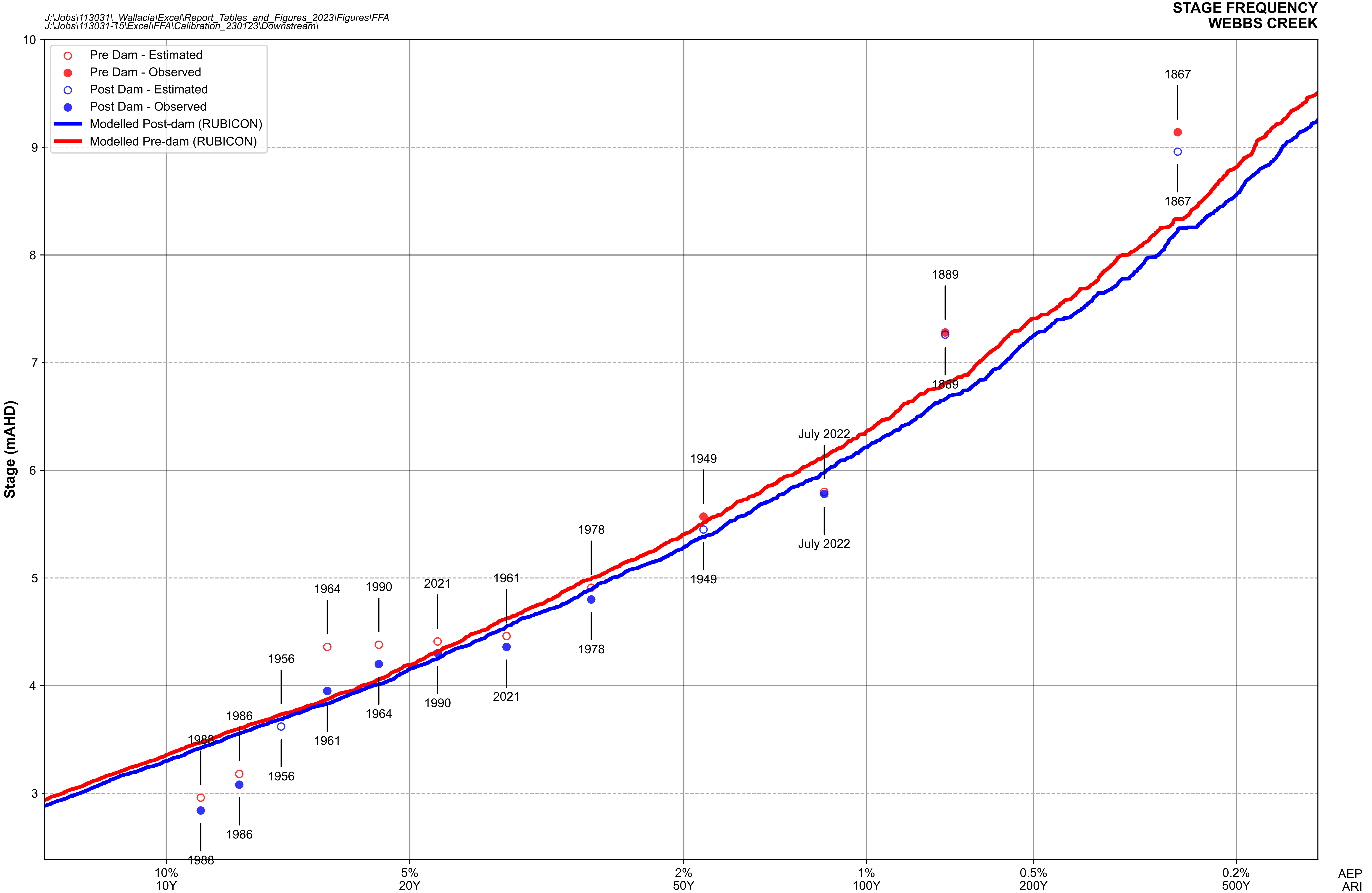


FIGURE 37

STAGE FREQUENCY
WEBBS CREEK



Appendix A

APPENDIX A. GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

	redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defenses excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).

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

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

floodplain risk management options The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.

floodplain risk management plan A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.

flood plan (local) A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.

flood planning area The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the ‘flood liable land’ concept in the 1986 Manual.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

- major overland flow paths through developed areas outside of defined drainage reserves; and/or
- the potential to affect a number of buildings along the major flow path.

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

merit approach The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State's rivers and floodplains.

The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.

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

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

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

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

modification measures Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.

peak discharge The maximum discharge occurring during a flood event.

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

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

probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

APPENDIX B. MARCH 2021 FLOOD EVENT VALIDATION

B.1. Introduction

March 2021 was the largest flood in 30 years on the Hawkesbury-Nepean River. It was important to validate the RUBICON model against this flood, given changes in the catchment and floodplain since the original model calibration. This is particularly notable at Penrith, where there has been significant revegetation downstream of Penrith Weir.

The March 2021 event was a prolonged double peaked storm with 536mm of rainfall at Blackheath between 16 March and 24 March. The following sections present the validation of the Monte Carlo Framework and RUBICON models to the event.

B.2. Warragamba Dam

WMAwater derived the dam outflow discharge using the official dam water level record that WaterNSW uses for its automatic gate opening system. This water level is based on an average of water levels at 3 gauges near the dam wall and in Hideaway Bay.

Flood inflows to the storage are discharged from the gated spillway using an automatic system known as the H14 dam opening protocol for the radial and drum gates. Discharge was calculated using the official gate equations and checked against the original ratings.

The outflow hydrograph was reverse routed using the methodology outlined by Boyd et al. (1989) to generate a dam inflow hydrograph.

The adopted outflow and inflow hydrographs are shown on Figure B1. The peak outflow of 5069 m^3/s was reached at 4:45 pm on 21 March 2021.

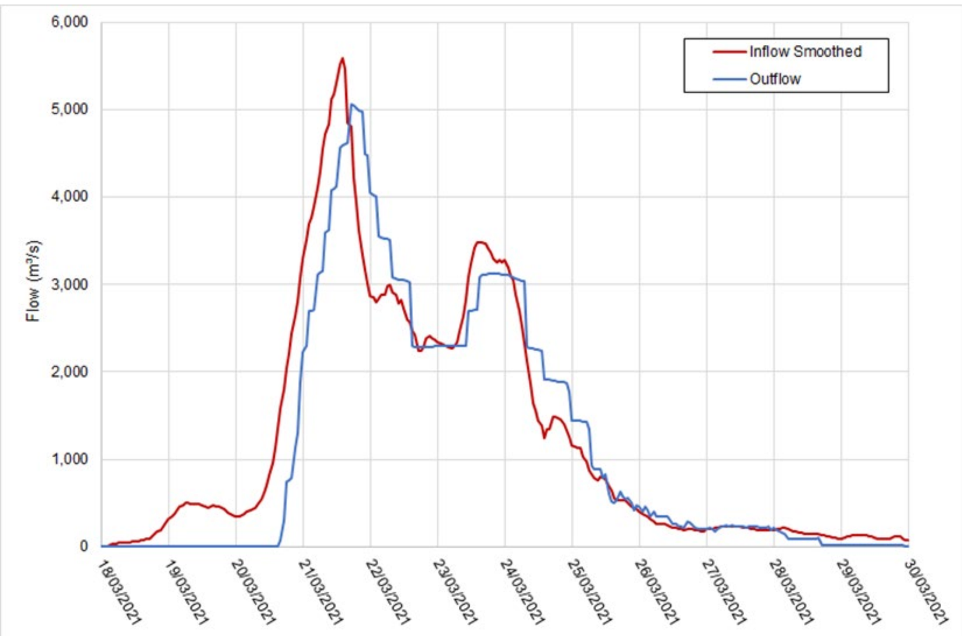


Figure B1: Warragamba Dam Inflow and Outflow Hydrographs

B.2.1. Inflow

Considering also the two floods in 2022 (see Infrastructure NSW, 2023), Table B1 indicates that the March 2021 event is the fifth largest on record in terms of its peak level in the dam. Due to its double peaked nature, it is higher in terms of inflow volume (ranked third since the dam was completed in 1960 – see also Figure B2). It ranks fairly low (eighth) on inflow peak.

Table B1: Historical event inflow comparison

Event	Peak Dam Level			Peak Dam Inflow			Total Dam Inflow Volume		
	Peak Level (m AHD)	AEP (1 in X)	Rank since 1960	Peak Inflow (m³/s)	AEP (1 in X)	Rank since 1960	Total Inflow Volume (GL)	AEP (1 in X)*	Rank since 1960
1867#	n/a	n/a	n/a	19593	330	-	2629	560	-
Nov-61	119.51	37	1	11033	40	1	1418	49	2
Jun-64	118.89	26	2	9322	27	3	1012	24	7
Jun-75	118.15	12	6	7293	16	5	710	14	8
Mar-78	118.01	10	8	9644	29	2	1212^	34^	4
Apr-88	118.06	10	7	7143	15	6	602	11	9
Aug-90	118.72	23	3	8817	23	4	1086	28	5
Mar-21	118.25	13	5	5591	9	8	1299	40	3
Mar-22	117.97	10	9	4880	7	9	1612	72	1
Jul-22	118.37	15	4	6909	14	7	1016	24	6

Notes

* The peak inflow and total inflow volume AEPs have been calculated following Australian Rainfall and Runoff (Ball et al., 2019) which uses critical duration assumptions for design events. As these assumptions use a single duration, they may slightly underestimate inflow hydrograph volumes. Alternative duration assumptions would result in more frequent AEPs for the historical volumes.

The 1867 inflow is only approximate and occurred before Warragamba was built. It is the largest historically recorded flood in the valley below the dam and has been included for context.

^ As the inflow hydrograph is based on the reverse routed outflow, and the March 1978 outflow hydrograph had limited data points, this is an estimate.

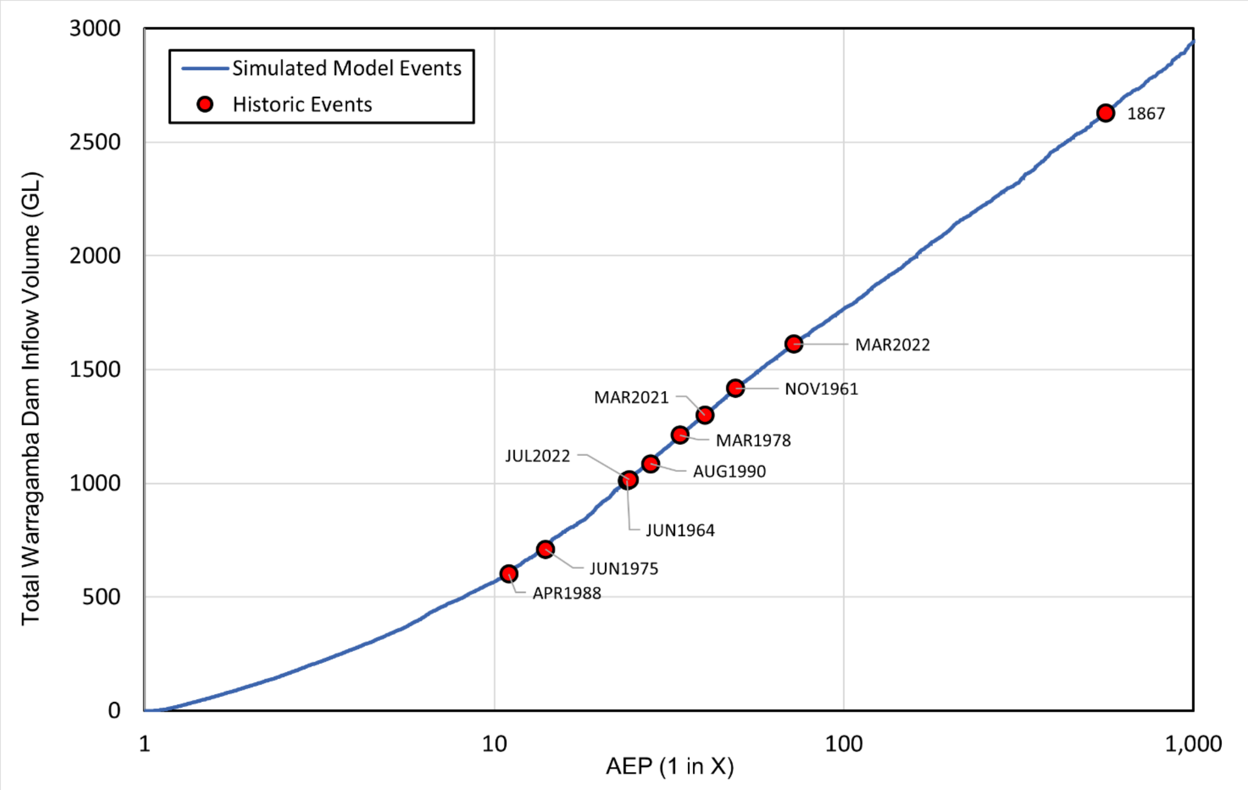


Figure B2: Warragamba Dam Total Inflow Volume

B.3. **Calibration**

B.3.1. Model Updates

The flows from Warragamba Dam and the WBNM hydrologic model were input to the RUBICON hydraulic model. The hydraulic model was originally calibrated and verified using seven historical floods from 1961 to 1990 (Webb, McKeown & Associates, 1996). However, as there have been some changes to the catchment in the 30 years since the 1990 flood particularly to vegetation downstream of Penrith, changes were made to the model so that it reproduced the behaviour of the 2021 event. These changes were guided by investigations carried out as part of **Technical Volume 8**.

B.3.2. Penrith

Technical Volume 8 documents work to assess the impacts of increased riparian vegetation at Penrith on flood behaviour. This has involved calibrating the WBNM hydrologic model and TUFLOW detailed 2D hydraulic model to conditions in 2021. In the TUFLOW model, this involved increasing the roughness of the channel downstream of Penrith, from the conditions observed in the early 1990s. Rating curves extracted from the TUFLOW model were used to guide changes to the RUBICON model at Penrith.

In order to represent the impacts of the increased riparian vegetation at Penrith in the RUBICON model, changes were made to the application of Manning's 'n' between Victoria Bridge and McCanns Island. In this area, an increased Manning's 'n' value (n = 0.12) was applied to the overbank areas. The effect of the vegetation on constricting flow from the river to the overbank areas was also represented in the model connections.

These changes resulted in a peak flood level that matched the recorded peak level at Penrith without impacting downstream results at Windsor (refer to Table B2).

The changes to Mannings 'n' adopted at Penrith for the 2021 calibration were not adopted for the entire framework and therefore the representative events. This is due to the impact of these changes being isolated to Penrith, where flood levels are primarily flow driven. (The increased roughness at Penrith is incorporated into the TUFLOW model and will be reflected in the final representative design event flood levels – **Technical Volume 11**).

Figure B3 shows the modelled March 2021 stage hydrograph at Victoria Bridge in Penrith.

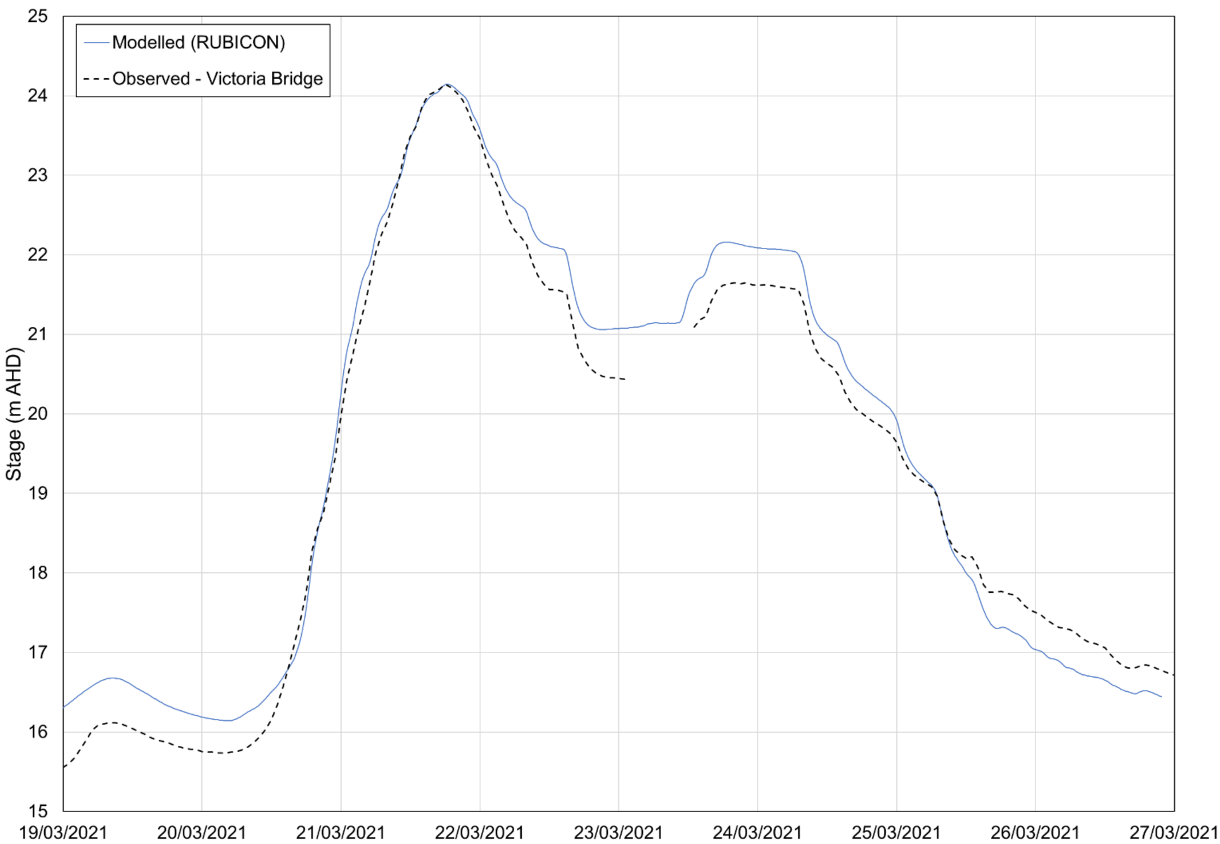


Figure B3: Penrith Stage Hydrographs

B.3.3. Windsor

Beyond the calibration described in Section 3.3.1 of this report, no further changes were required to achieve a reasonable fit to the peak at Windsor.

Figure B4 shows the modelled March 2021 stage hydrograph at Windsor.

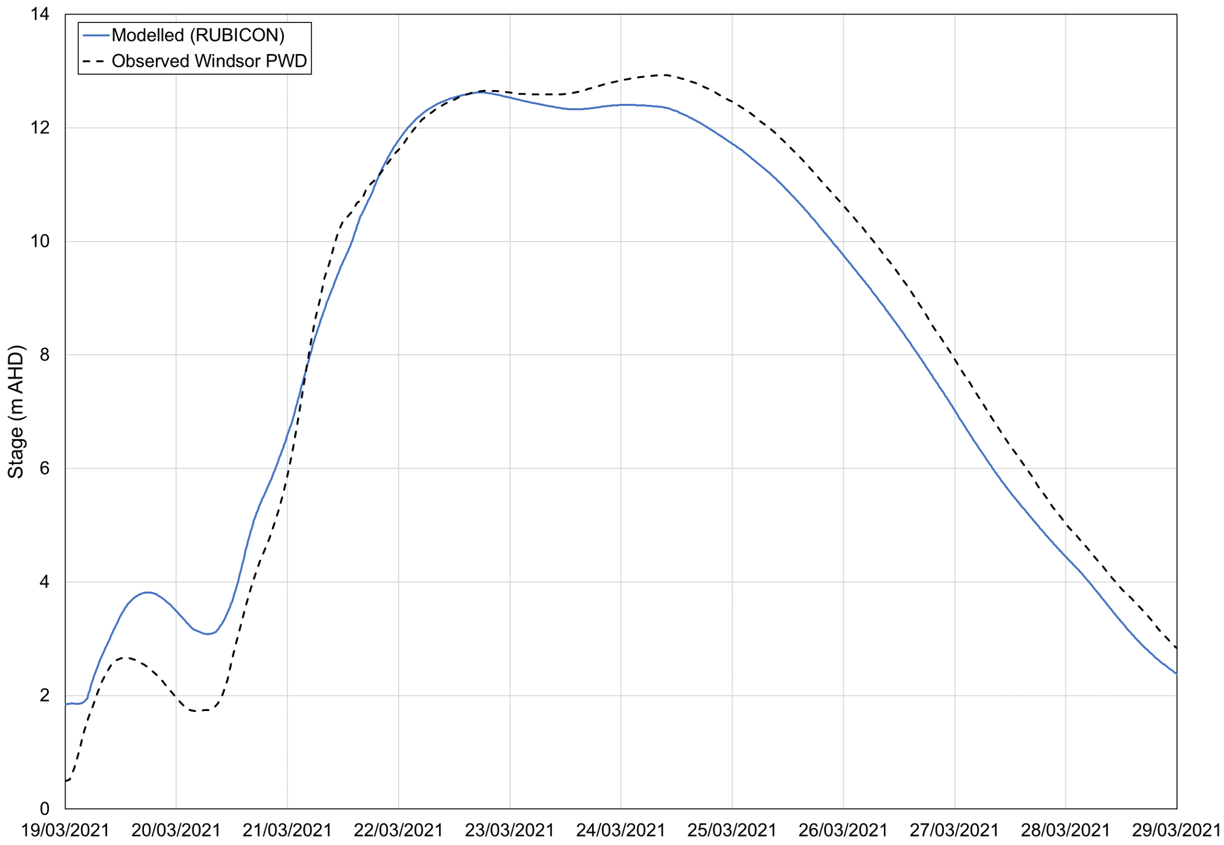


Figure B4: Windsor Stage Hydrographs

B.3.4. Calibration summary

The 2021 event was one of the larger floods on the modern record in the Hawkesbury-Nepean floodplain and was used as a verification of the WBNM-RUBICON model suitability. Table B2 shows the observed and modelled peak levels at Windsor and Penrith. The updated models provide a good representation of the observed flood hydrographs.

Table B2: Peak Level Comparison

	Penrith	Windsor
Observed Level (m AHD)	24.13	12.93
Modelled Peak Level (m AHD)	24.15	12.62
Difference (m)	0.02	-0.31

Appendix C

50% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 50% AEP LEVEL

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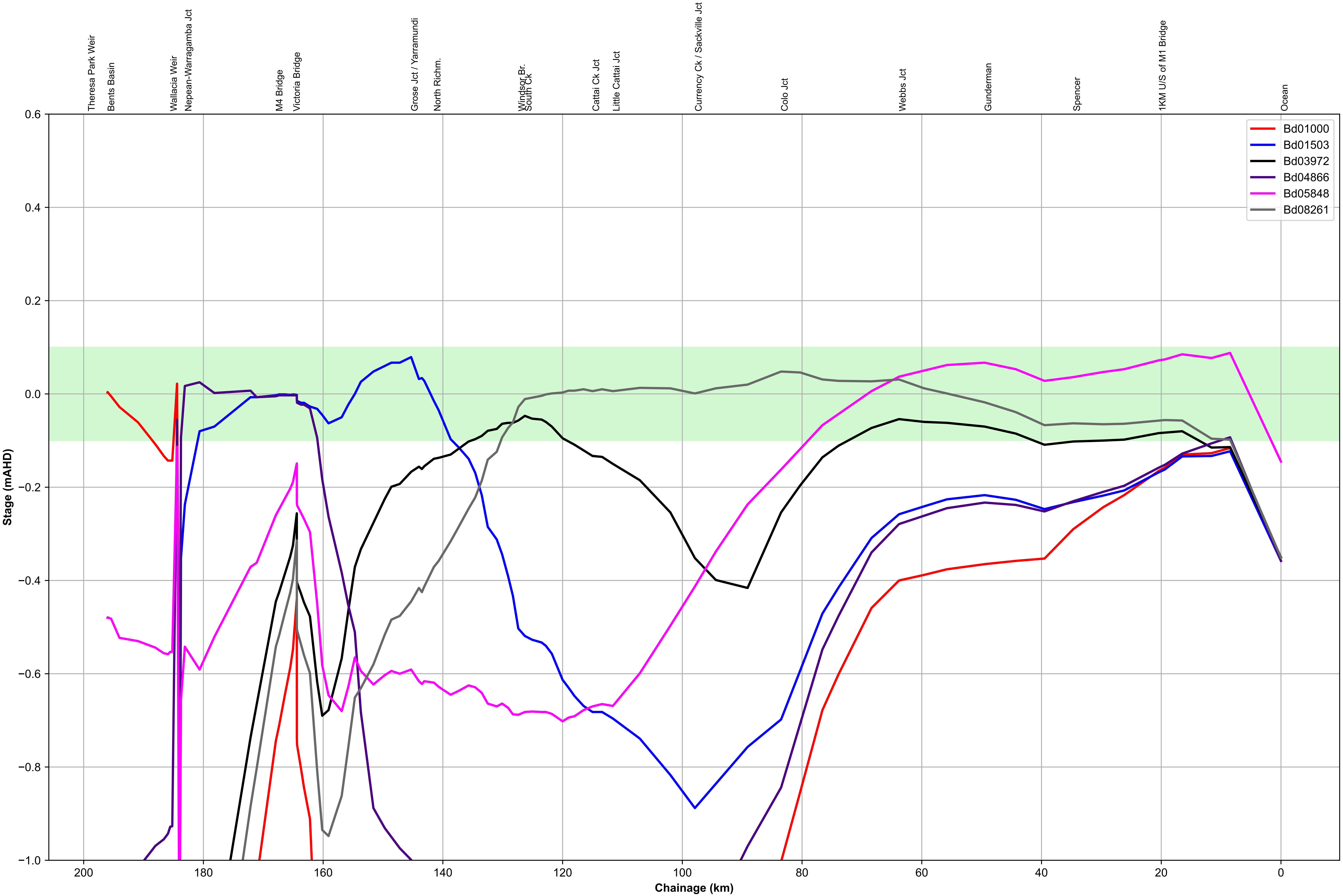

APPENDIX C2

20% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 20% AEP LEVEL

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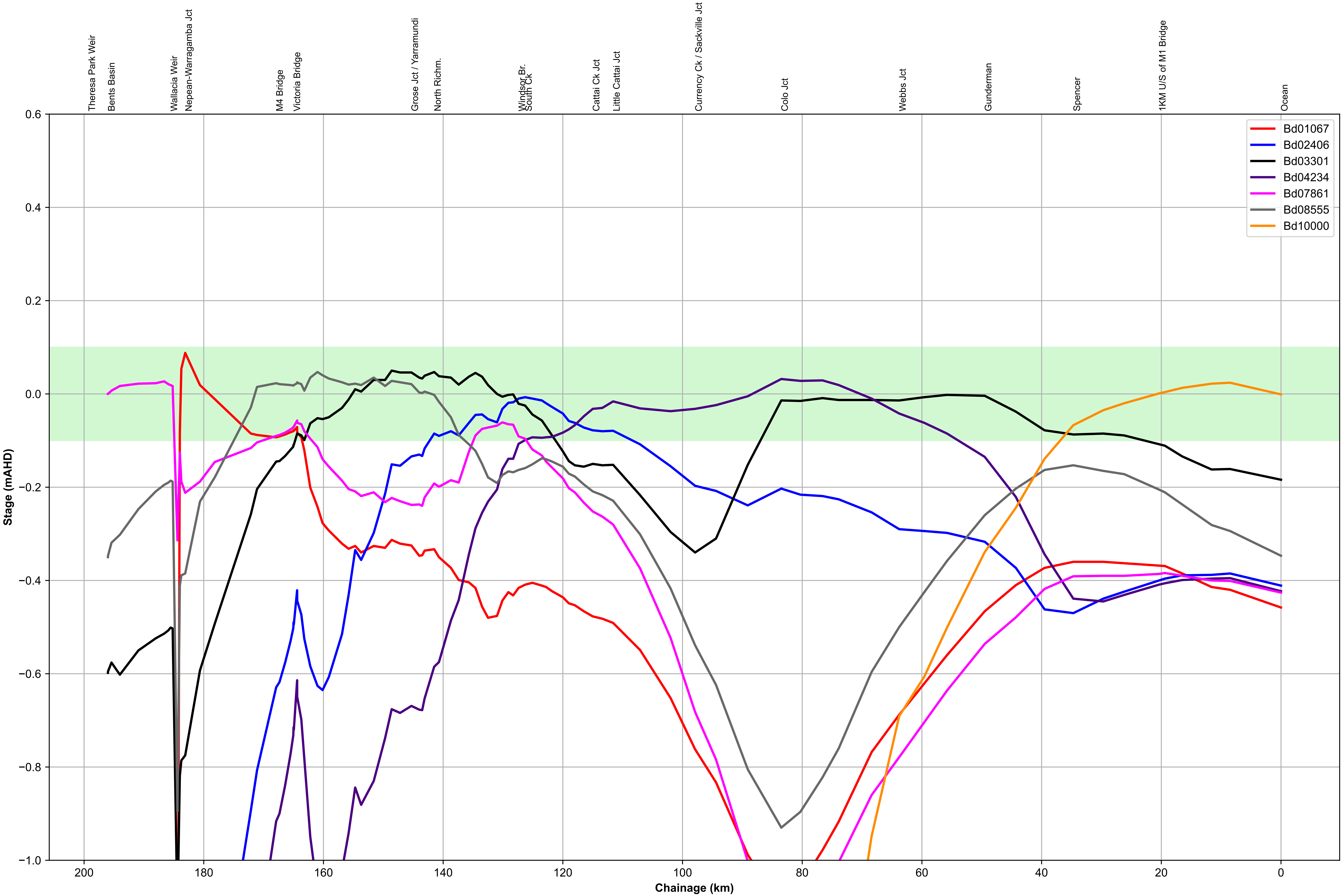

10% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 10% AEP LEVEL

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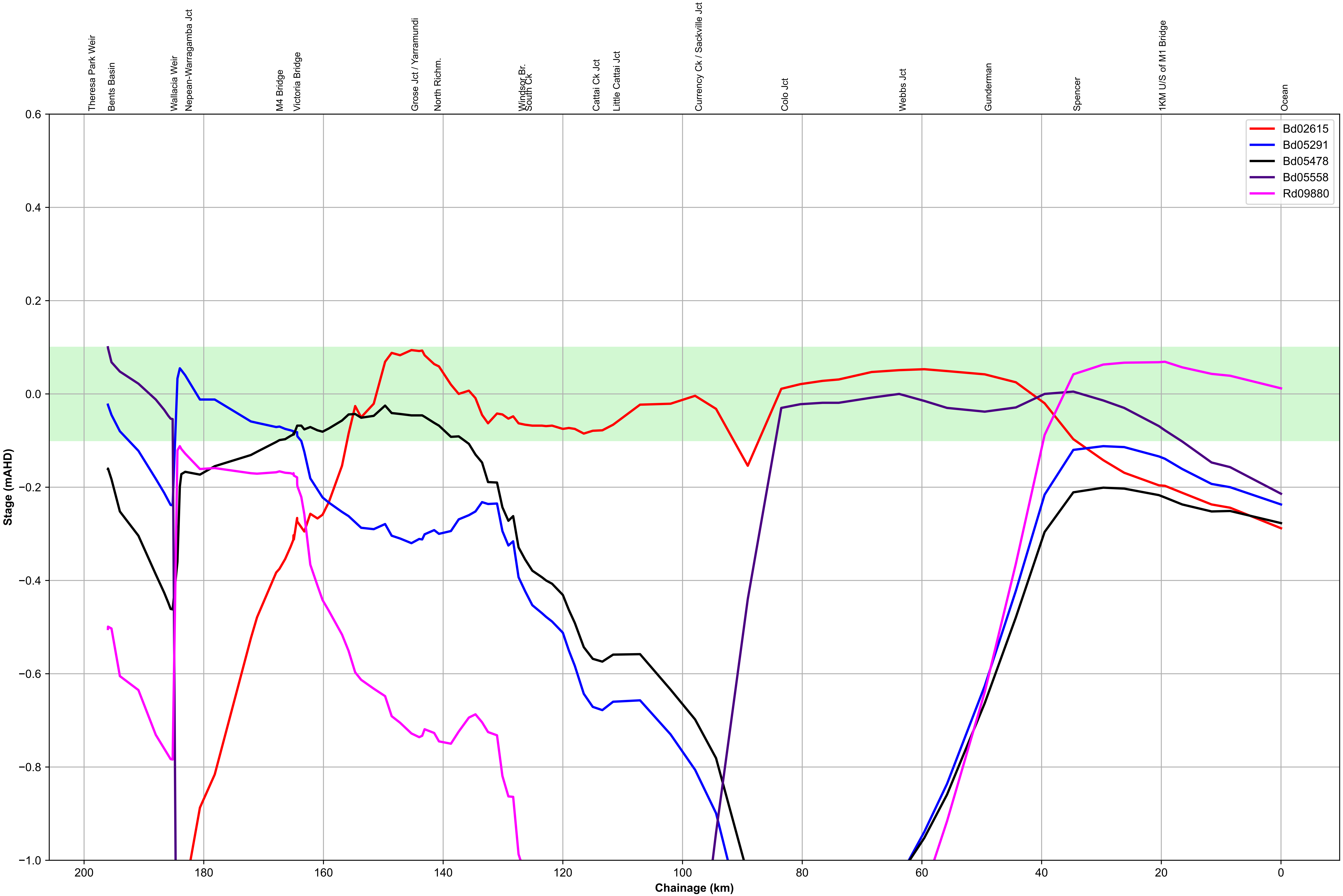

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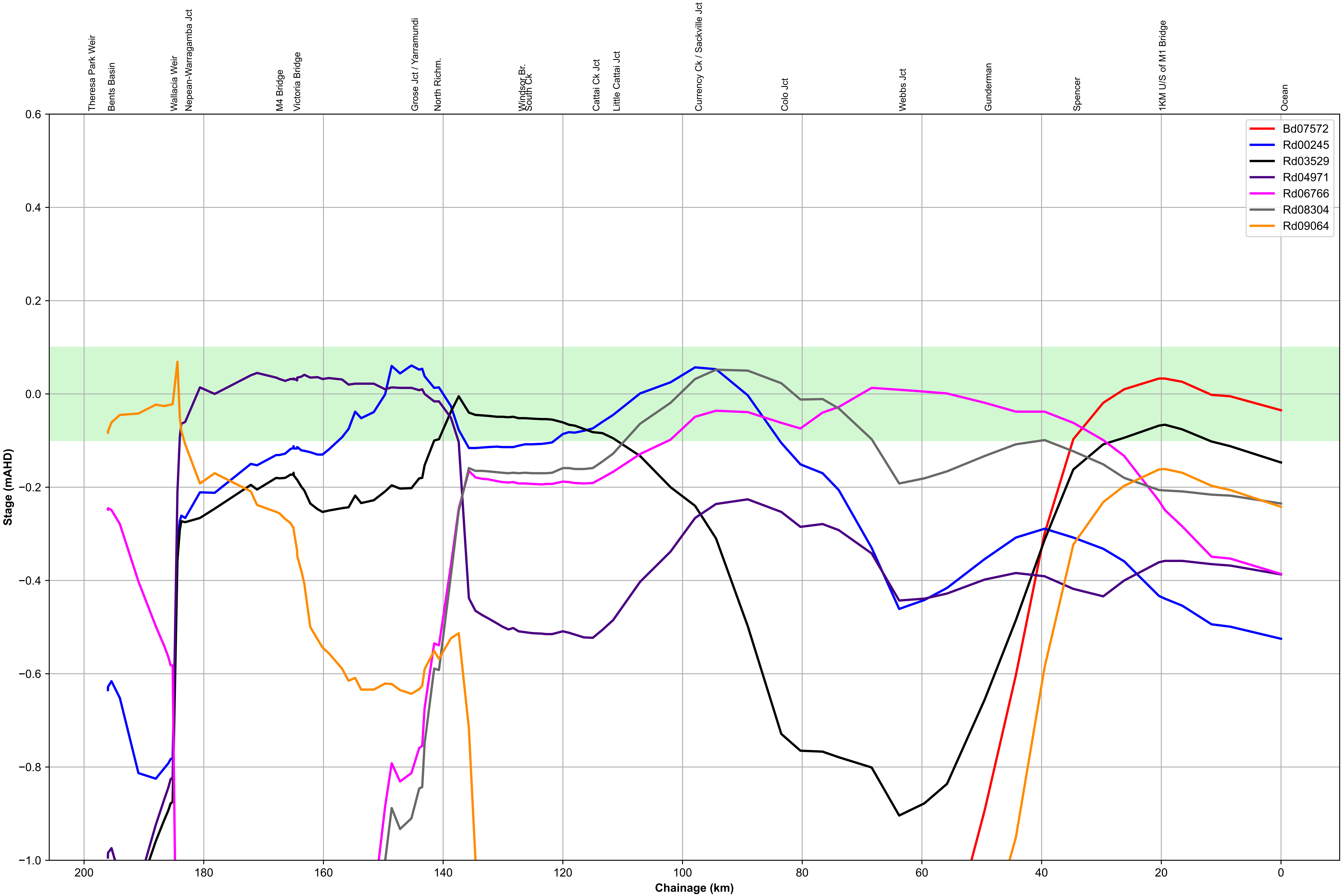
5% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 5% AEP LEVEL



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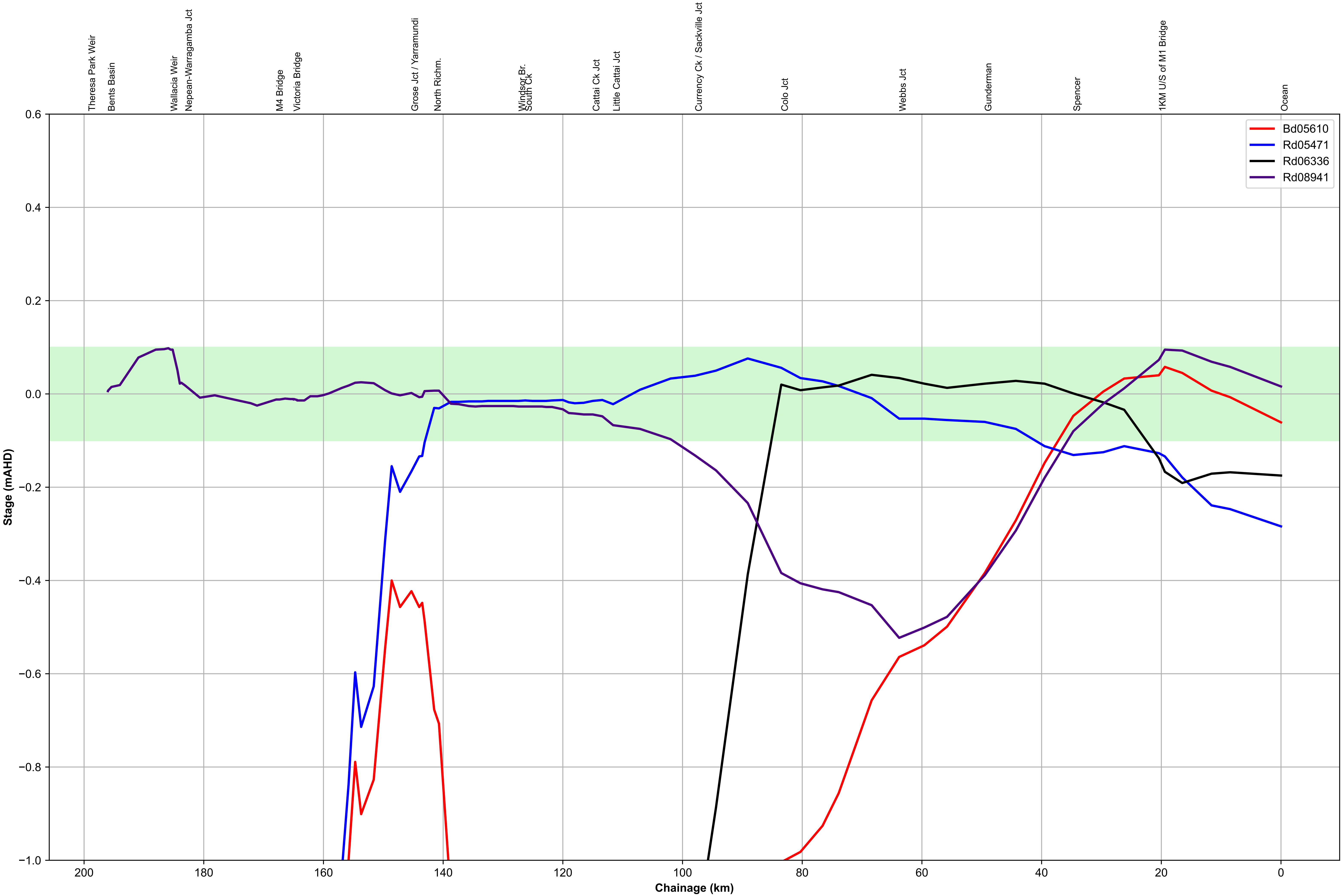
2% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 2% AEP LEVEL



APPENDIX C6

1% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1% AEP LEVEL

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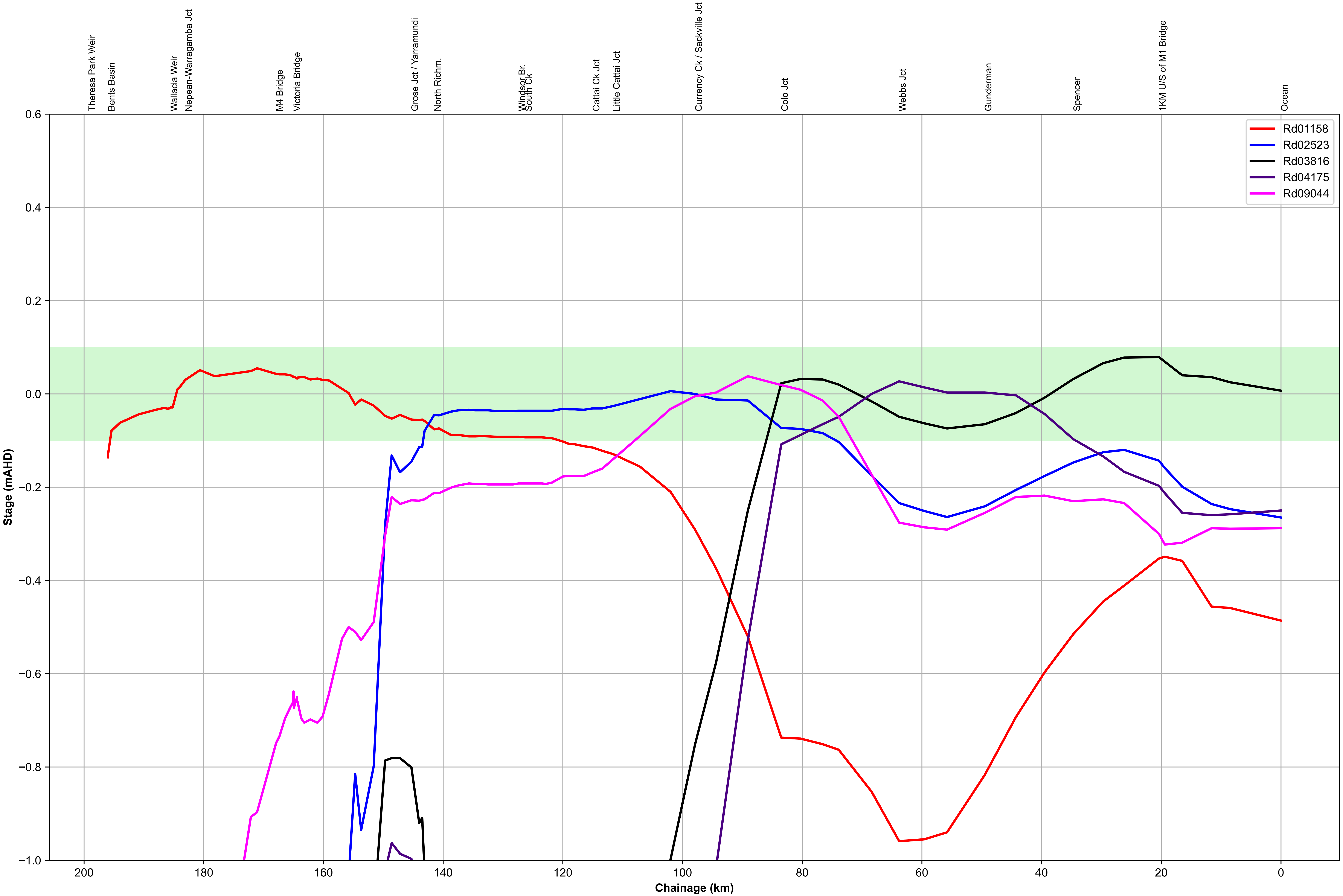

APPENDIX C7

1 IN 200Y AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1 IN 200Y AEP LEVEL

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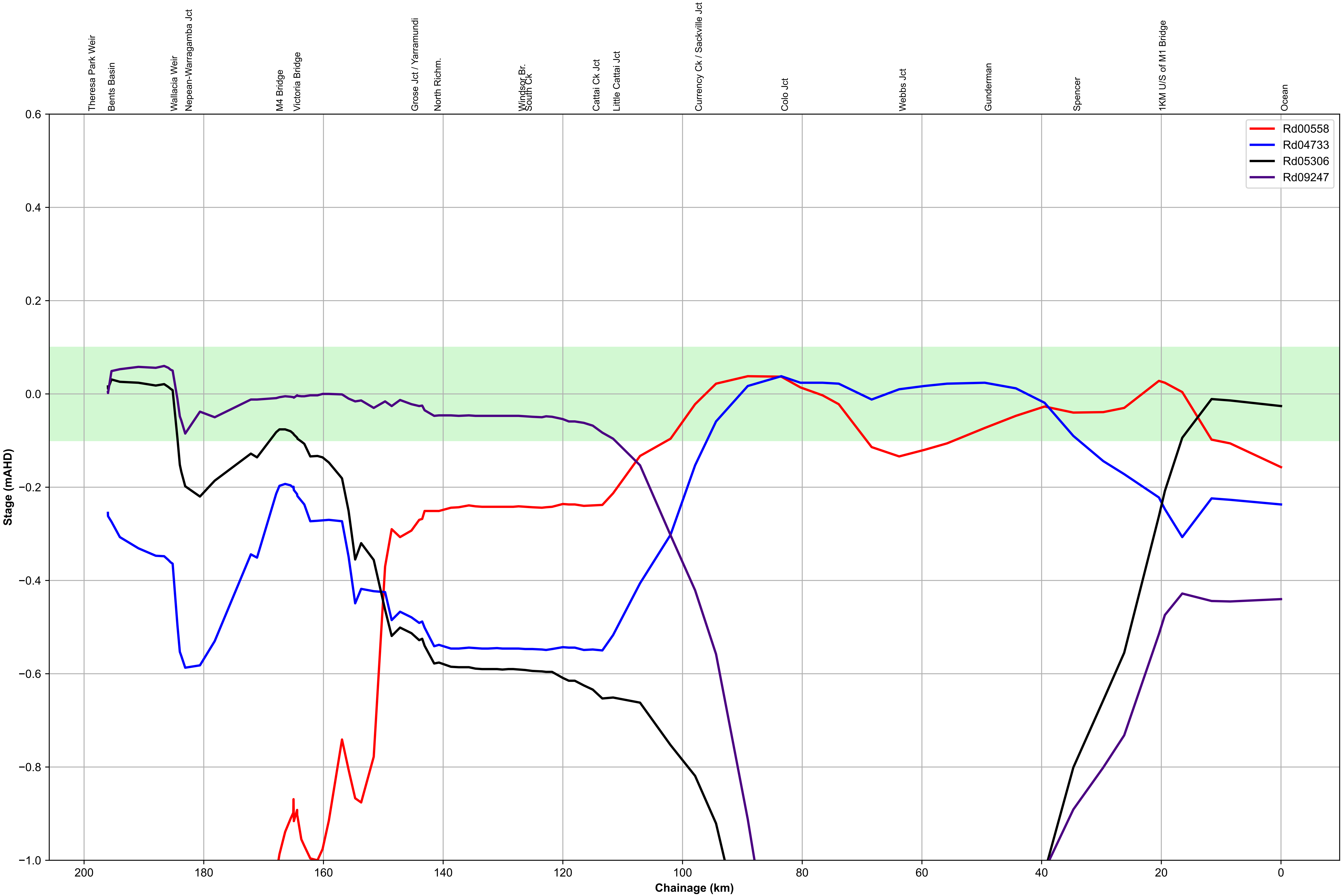

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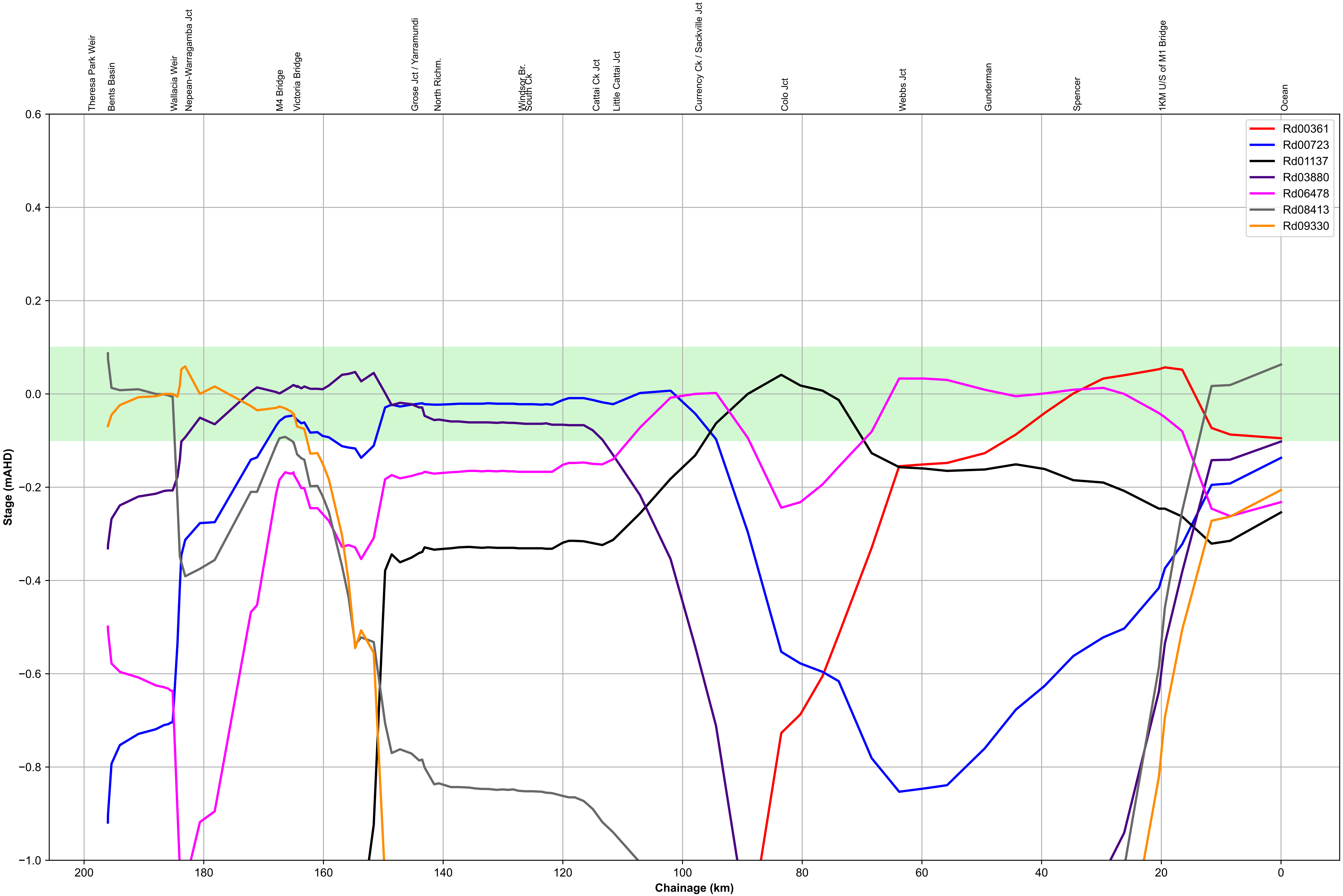
1 IN 500Y AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1 IN 500Y AEP LEVEL



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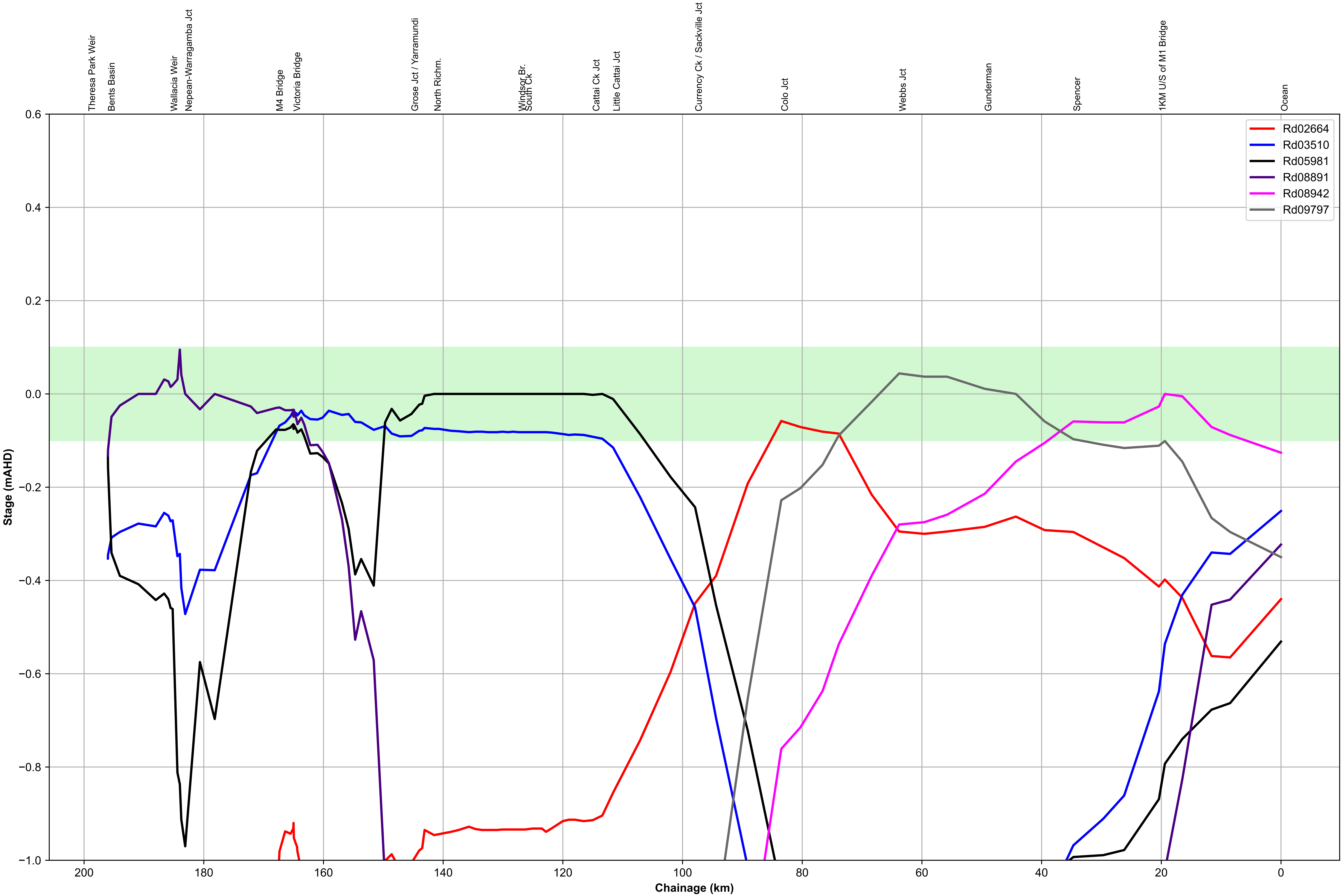
1 IN 1000Y AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1 IN 1000Y AEP LEVEL



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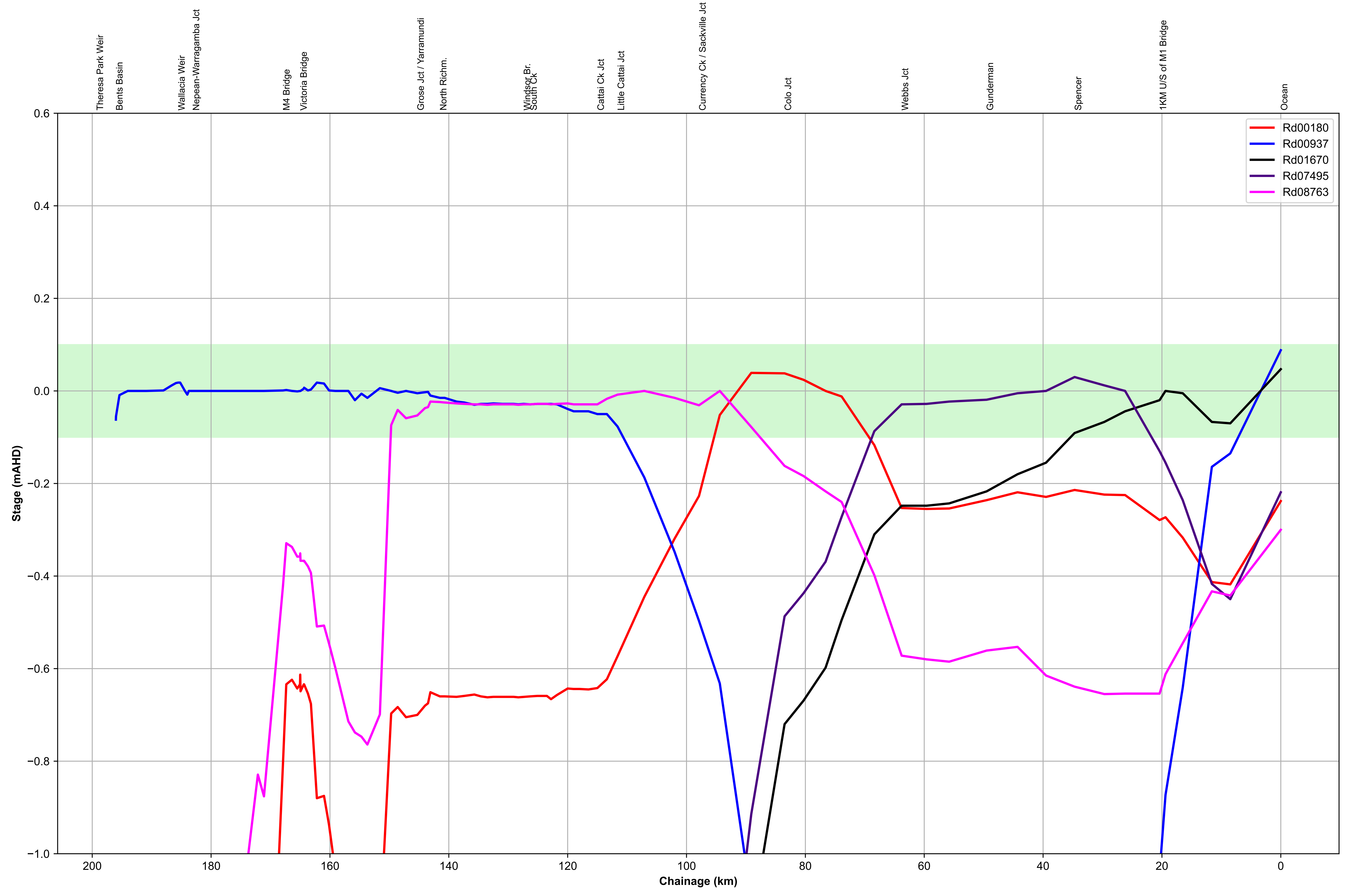
LEVEL DIFFERENCES TO 1 IN 2000Y AEP LEVEL



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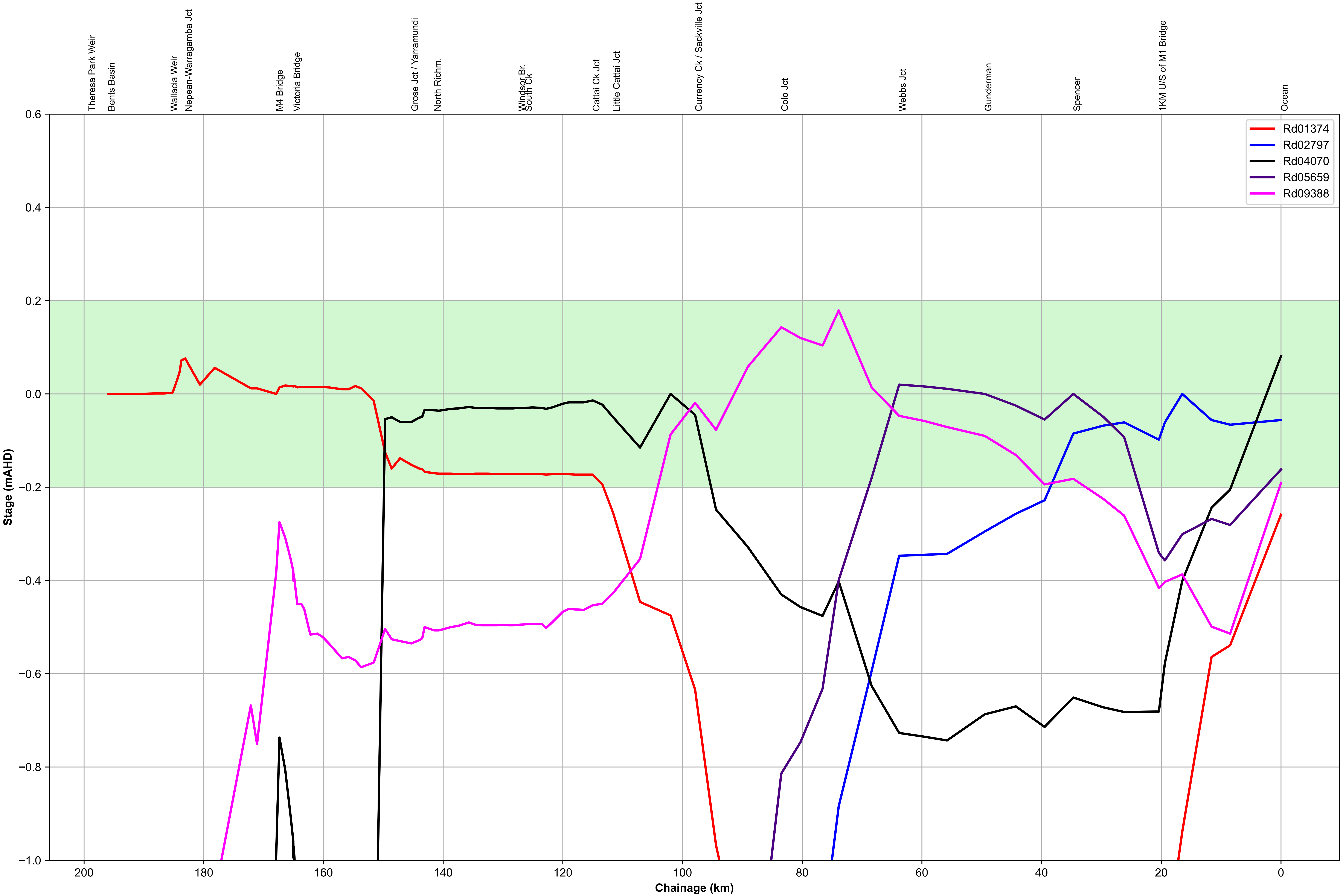
1 IN 5000Y AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1 IN 5000Y AEP LEVEL



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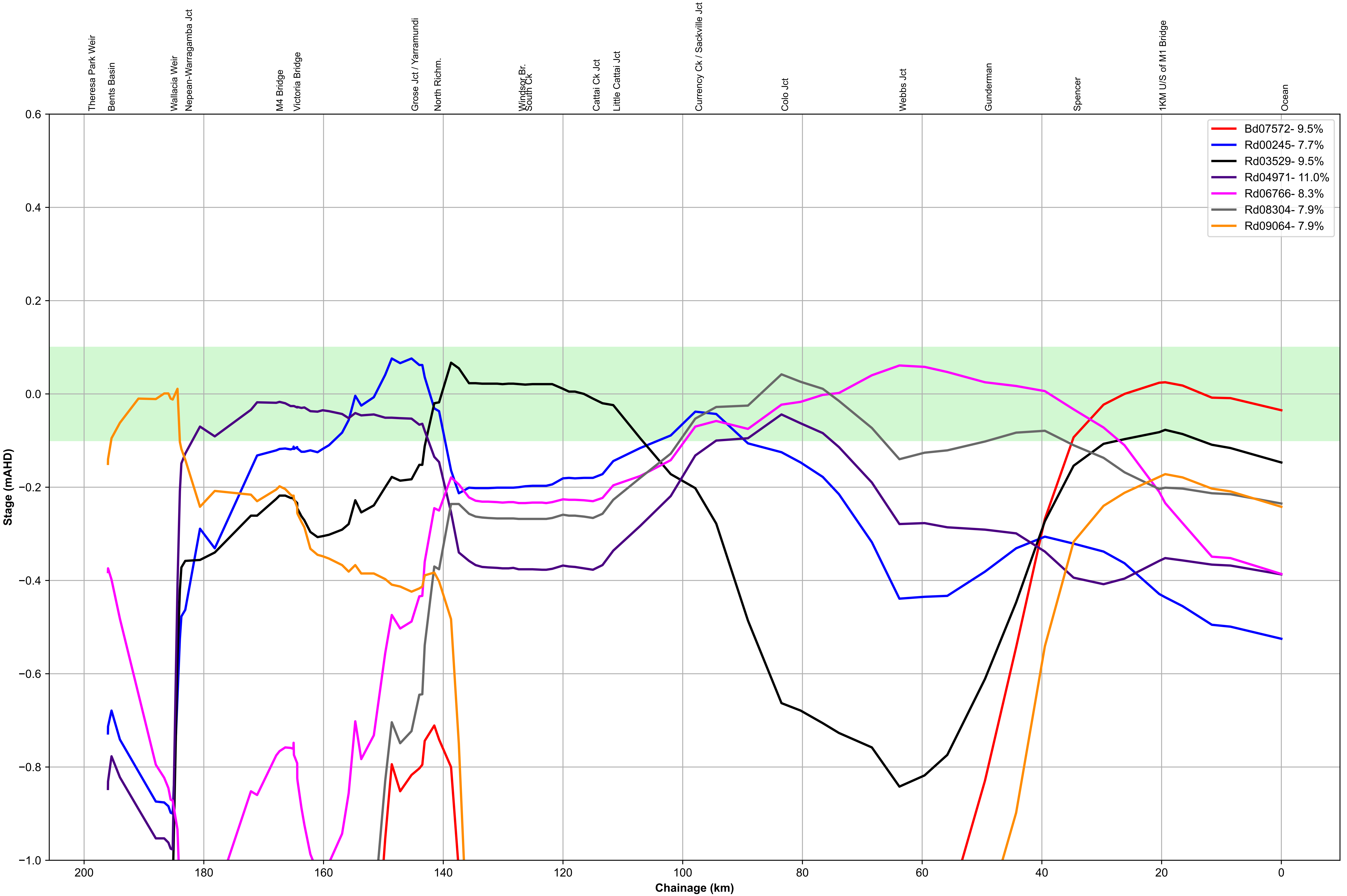
5% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 5% AEP LEVEL - CLIMATE CHANGE - 9.5% RAINFALL SCALING



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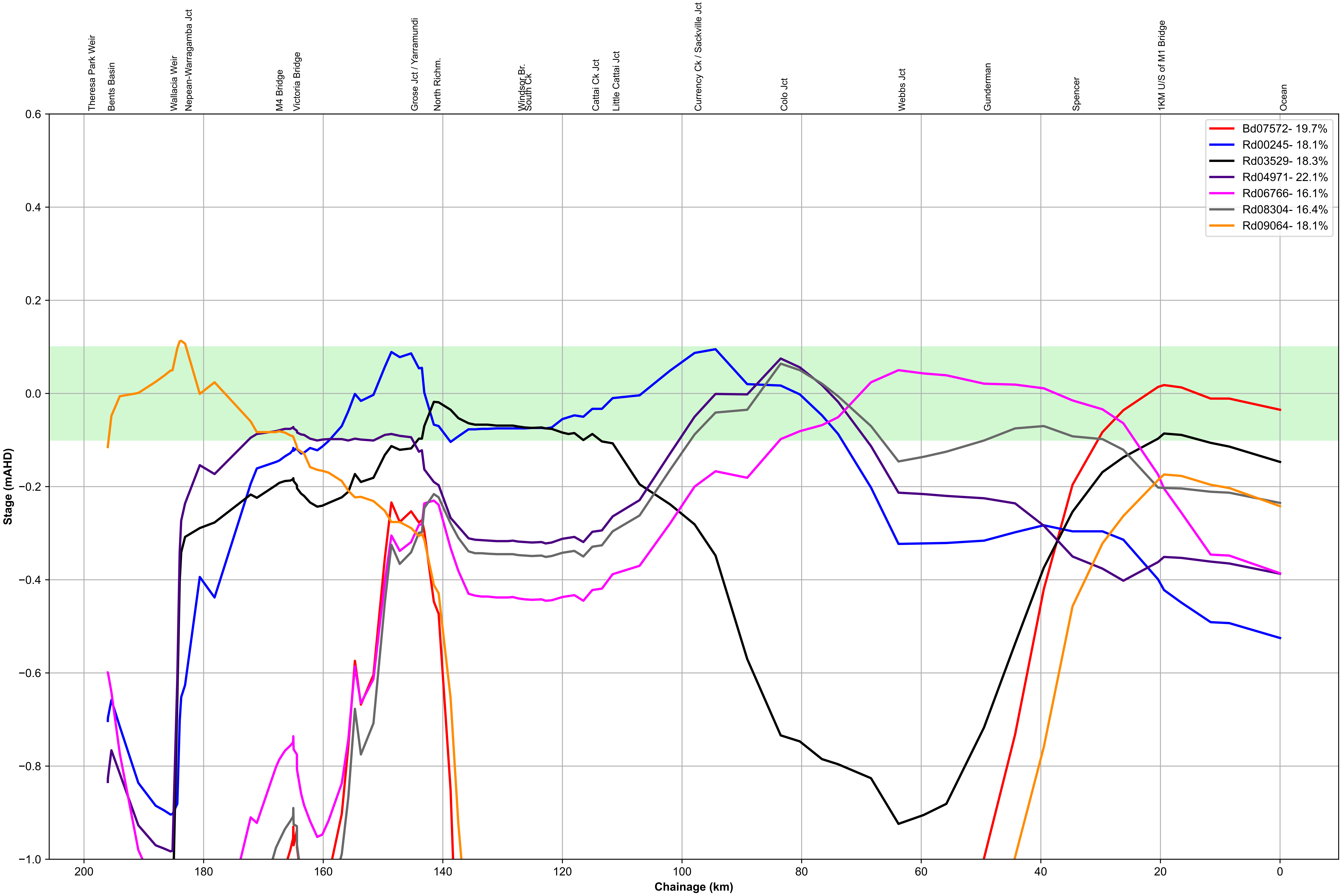
5% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 5% AEP LEVEL- CLIMATE CHANGE - 19.7% RAINFALL SCALING



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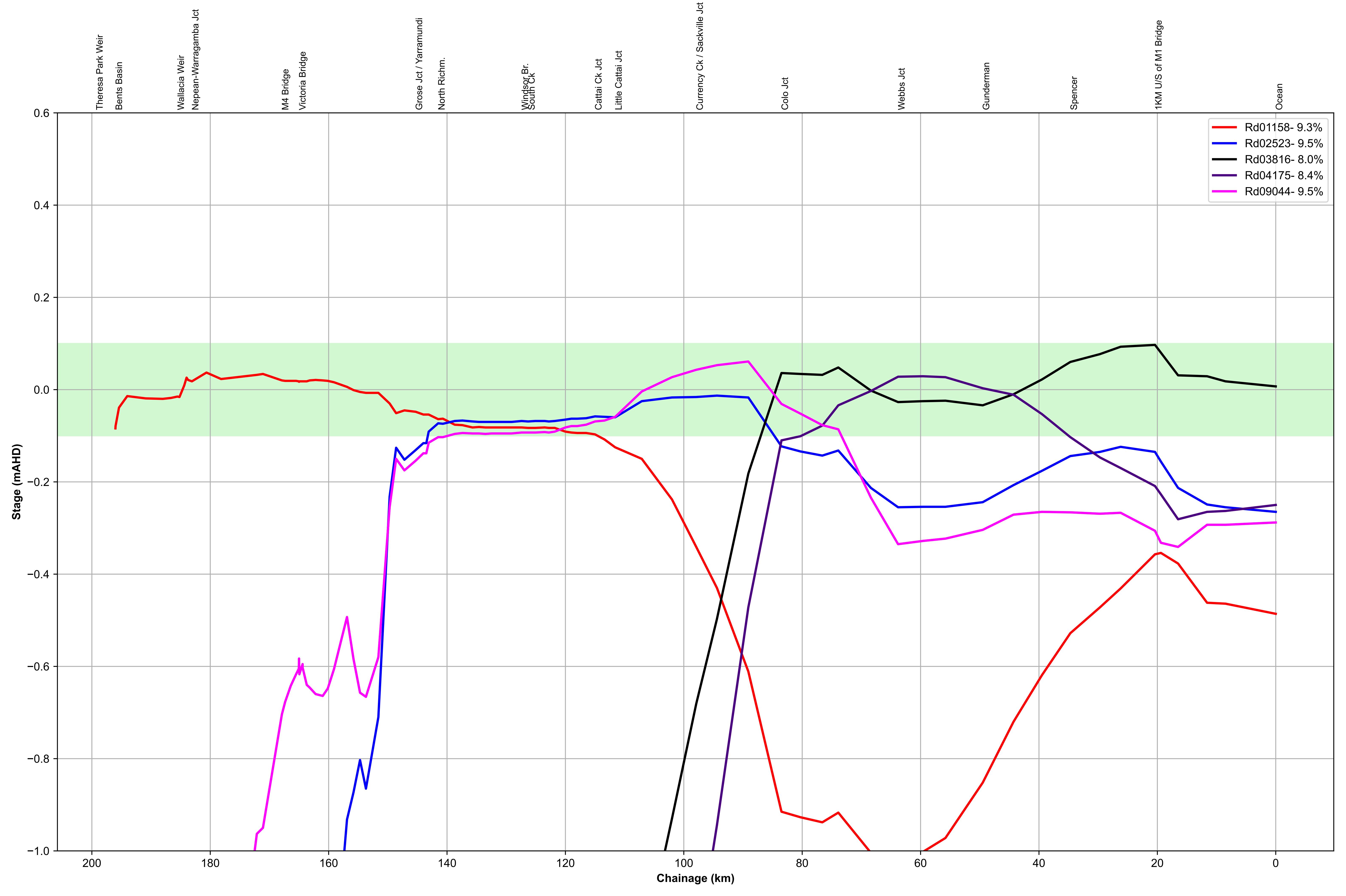
1% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1% AEP LEVEL - CLIMATE CHANGE - 9.5% RAINFALL SCALING



1% AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1% AEP LEVEL- CLIMATE CHANGE - 19.7% RAINFALL SCALING

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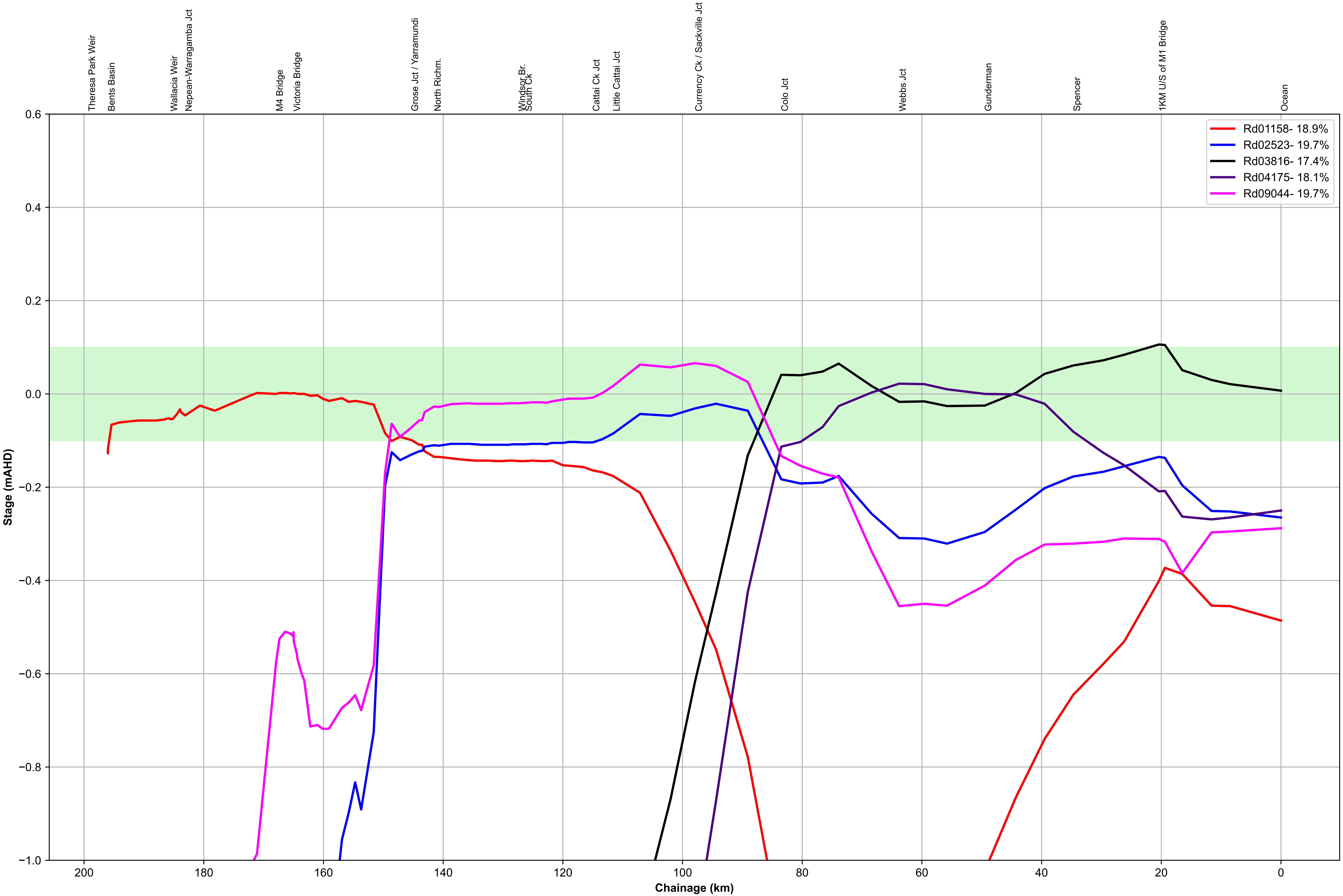

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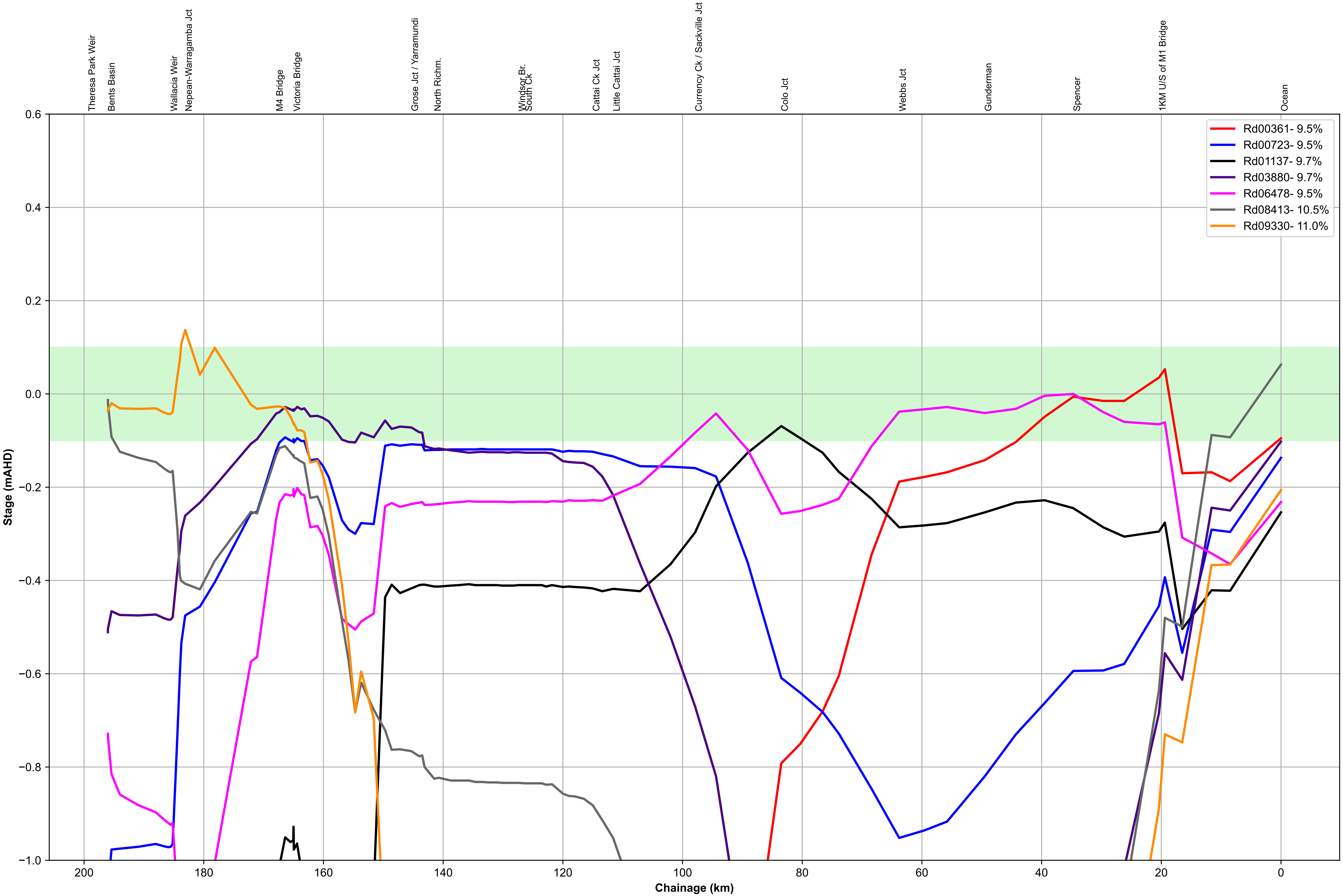
1 IN 500Y AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1 IN 500Y AEP LEVEL - CLIMATE CHANGE - 9.5% RAINFALL SCALING



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1 IN 500Y AEP REPRESENTATIVE EVENTS
LEVEL DIFFERENCES TO 1 IN 500Y AEP LEVEL- CLIMATE CHANGE - 19.7% RAINFALL SCALING