PAINKALAC CREEK FLOOD STUDY

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Glossary

AHD	Australian Height Datum. A universal reference level used in surveying.
ARI	Average Recurrence Interval, a term used in hydrology to describe the average interval in years between floods of a given magnitude or greater. Note that the actual intervals between such floods are subject to climatic variability and will not precisely match the average because their occurrence is irregular.
DTM	Digital Terrain Model – digital presentation of natural surface levels
HEC-RAS	River Hydraulic model. Developed by the US Army Corps for modelling river flows to determine water levels and other flow characteristics.
1D	One dimension – used to describe the type of hydraulic model. Flow is only considered to be in one dimension (ie. along the river)
2D	Two dimensions – used to describe the type of hydraulic model. Flow can be modelled in both down and across the river floodplain. (e.g. Cell models)
IFD	Intensity – Frequency – Duration. Relates to design rainfall intensity.
LSIO / FO	Land subject to inundation overlay / Floodway overlay
n	Manning n – a measure of stream roughness used to calculate stream velocity.
RORB	Catchment Runoff Model used to estimate catchment flood peaks.
Kc and m	Storage parameters used in the RORB model
IL	Initial loss (mm's) - Rainfall loss before runoff starts. Used in catchment runoff models
CL	Continuing loss rate (mm/hr) Rainfall loss rate during a storm event.
Rc	Runoff coefficient – Rainfall loss rate during a storm event.
Units	
Km	Length in kilometres
Sq km	Area in square kilometres
m3/sec	Flow rate - cubic metres per sec
mm	Length in millimetres
mm/hr	Rate millimetres per hour

1.0 PURPOSE

The Corangamite Catchment Management Authority has completed a detailed hydrological study of Painkalac Creek catchment to determine design flow estimates for the hydraulic floodplain modelling of Airey's Inlet.

This study was required to update the flood information held by the CCMA received from the Flood Data Transfer Project completed for DSE in 2000. This work calibrates a detailed RORB model with 4 historic events to improve the reliability of the design flow estimates for modelling the Painkalac Creek floodplain. This work forms the basis of new flood overlays to be introduced into the Surf Coast Shire Council Planning Scheme.

2.0 CATCHMENT HYDROLOGY

2.1 General Description of Catchment

Painkalac Creek at the Great Ocean Road has a catchment area of 61km^2. The headwaters of the river are located in the top of the Otway Rangers and approximately 95% of the catchment is forested within a State Park with moderate to steep stream gradients. The average slope of the main stream is 0.6% for the first 14kms, then steepens to a stream slope of 2.7% for the next 6km. Figure 1 shows the catchment extent and land cover.

The description of the land system is taken from "A Study of the Land in the Catchments of the Otway Ranges and adjacent plains" by A.J. Pitt for the Soil Conservation Authority in 1981. [Ref 6] The Painkalac catchment falls into the Moggs Creek land system (7.26) and is described by the following:

"*The terrain inland from Eastern View and Aireys Inlet consists of spurs and ridges with steep slopes and deep valleys. The outcropping Tertiary sediments are partly unconsolidated, but many beds are composed of quartzitic sandstones and siltstones. The lower parts of the landscape often posses outcrops of Cretaceous sediments.*

Open forests of Eucalyptus oblique, E. Sideroxylon and E.radiata occur over most of the landscape on duplex soils. The drier north- and west-facing slopes and steep slopes carry woodlands on shallow stoney soils. The Cretaceous outcrops can be recognized by the increase in understorey cover and the occurrence of species such as Acacia mucronata and Cassinia longifolia."

In the component table for this land system 83% of the soils have moderate to high permeability with depths > 2m.

2.2 Available Data Sources

Information available for this catchment consists of:

a) Observed flow data (Instantaneous flow from one site in the catchment)
Site 235232 – Painkalac Creek Dam (1974-1991, 1999-2012)

b) Daily read rainfall records for stations surrounding the Painkalac Creek Catchment.

BoM site ID	Name	BoM site ID	Name
87001	Anglesea	87119	Anglesea (VPG)
87124	Buckley	87126	Wurdiboluc Res
87160	Torquay Golf C	90004	Barwon Downs
90037	Eastern View	90061	Pennyroyal
90180	Aireys Inlet	90187	Boonah
90188	Benwerrin		

c) Pluviograph rainfall data is available from BoM AWS and flood warning stations from 2000. Very little pluviograph rainfall data is available in the 1970's near this catchment.

2.3 RORB Model Formation

A runoff routing model like RORB is required to give design hydrographs as input into the hydraulic model of the Painkalac Creek.

A 19 subarea catchment model of the Painkalac Creek was formed using GIS information held by the Authority (Figure 1). Ten metre contour intervals were used to determine the catchment boundaries and graphical tools used to calculate areas. Stream lengths were measured from the GIS hydro25 layer. Both the contours and stream data originated from the 1:25,000 topographic maps.

The subarea size was determined to ensure that sufficient subareas existed on both the main branch and Distillery Creek tributary to be able to make reliable estimation of peak flow runoff. The subarea size varied from 120 to 476 hectares, with an average size of 323 hectares.

The maximum distance of any subarea from the Great Ocean Road is 20.3 kms for Subarea A and the average distance for all subareas is 12.25km (i.e. Dav). There are 11 subareas above the gauging station at the dam site with a catchment area of 35.1 sq kms.

Figure 1 shows the layout of the subareas and reaches in the catchment. The location of the gauging station is also shown as a red triangle. Figure 2 shows the layout against the crown land shown shaded green.

Appendix A is the basic RORB catchment data file formed following the guidance of the RORB User Manual [Ref 2].

2.4 Calibration of the RORB Model

Four historic flood events are available that can be fitted to recorded hydrographs to calibrated the RORB catchment storage parameters (Kc and m). Storage parameters that gave the best fit to observed data was Kc= 9 with m= 0.75.

The storm pattern used and subarea rainfall totals for the calibration run are shown in Appendix E. Rainfall pluviograph data was taken from Boonah Rainfall station at the top of this catchment in the Otway Range.

Flood Event	Rainfall (mm)	Peak Flow at Dam (Site 235232) (m^3/s)	Peak Flow at Great Ocean Road (m^3/s)	Rainfall Losses IL (mm)/ RC
April 2001	255 in 72 hr	52	74	20 / 0.57
Feb 2005	120 in 60 hr	34	41	80 / 0.67
Nov 2007	122 in 26 hr	64	91	45 / 0.91
Jan 2011	152 in 120 hr	22	35	65 / 0.53

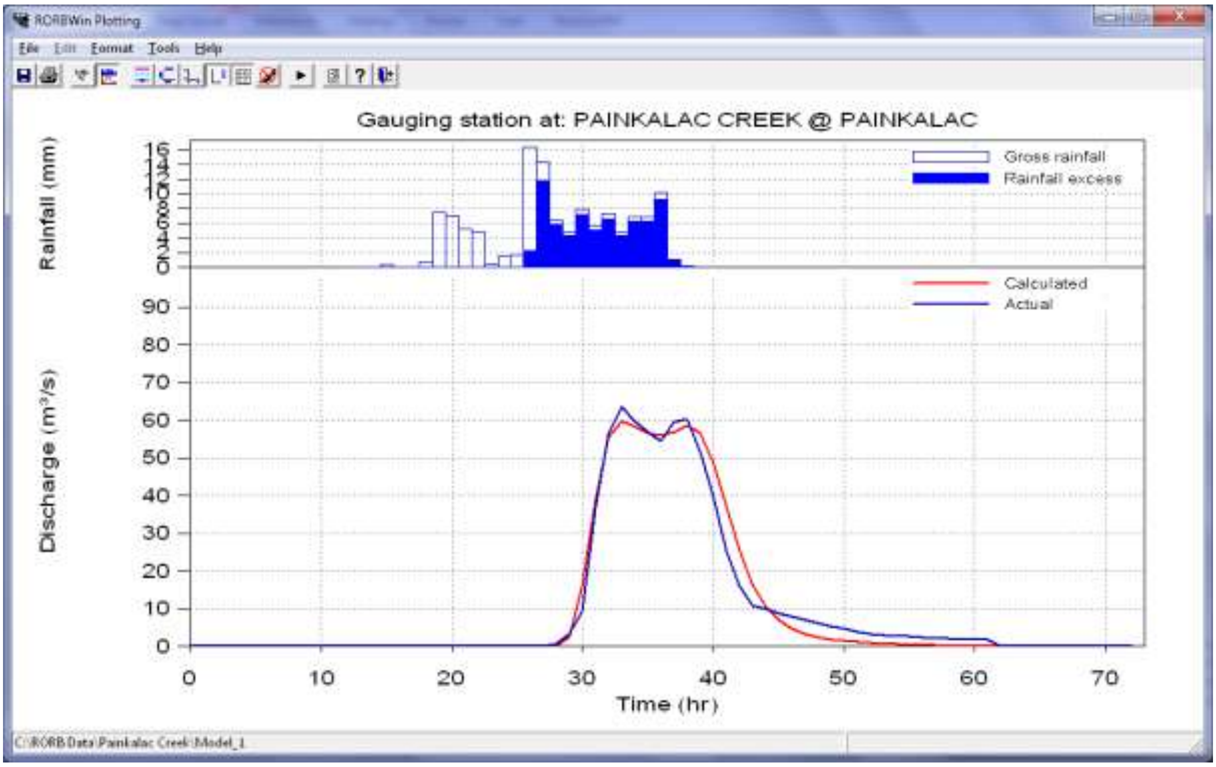
No base flow adjustment is required for this catchment (less than 0.1 m3/s).

Appendix B shows the RORB plots of the calibration runs for this flood event.

The fit to the observed flood hydrographs are reasonable in both shape and timing for the February 2005, November 2007 and January 2011. The April 2007 event does not represent a reasonable fit. The 2007 event pluviograph data is taken from Boonah on top of the Otways and likely to have a different pattern of rainfall than the lower subareas experienced. The majority of the catchment rainfall was taken up by the initial loss. This may be typical for this catchment being mostly forested.

The November 2007 fit results are shown below.

KC= 9 m= 0.75 IL= 45 mms RC= 0.91



Gauging station at: PAINKALAC CREEK @ PAINKALAC

	Hydrograph		Error	
	Calc.	Actual	Abs.	Percent
Peak discharge,m³/s	59.9	63.6	-3.7	-5.9
Time to peak,h	33.0	33.0	0.0	0.0
Volume,m³	0.25E+07	0.25E+07	-0.50E+04	-0.2
Av.abs.ord.err,m³/s			1.4	13.1
Time to centroid,h	36.8	37.4	-0.7	-1.8
Lag (c.m. to c.m.),h	5.0	5.7	-0.7	-12.0
Lag to peak,h	1.2	1.2	0.00	0.0

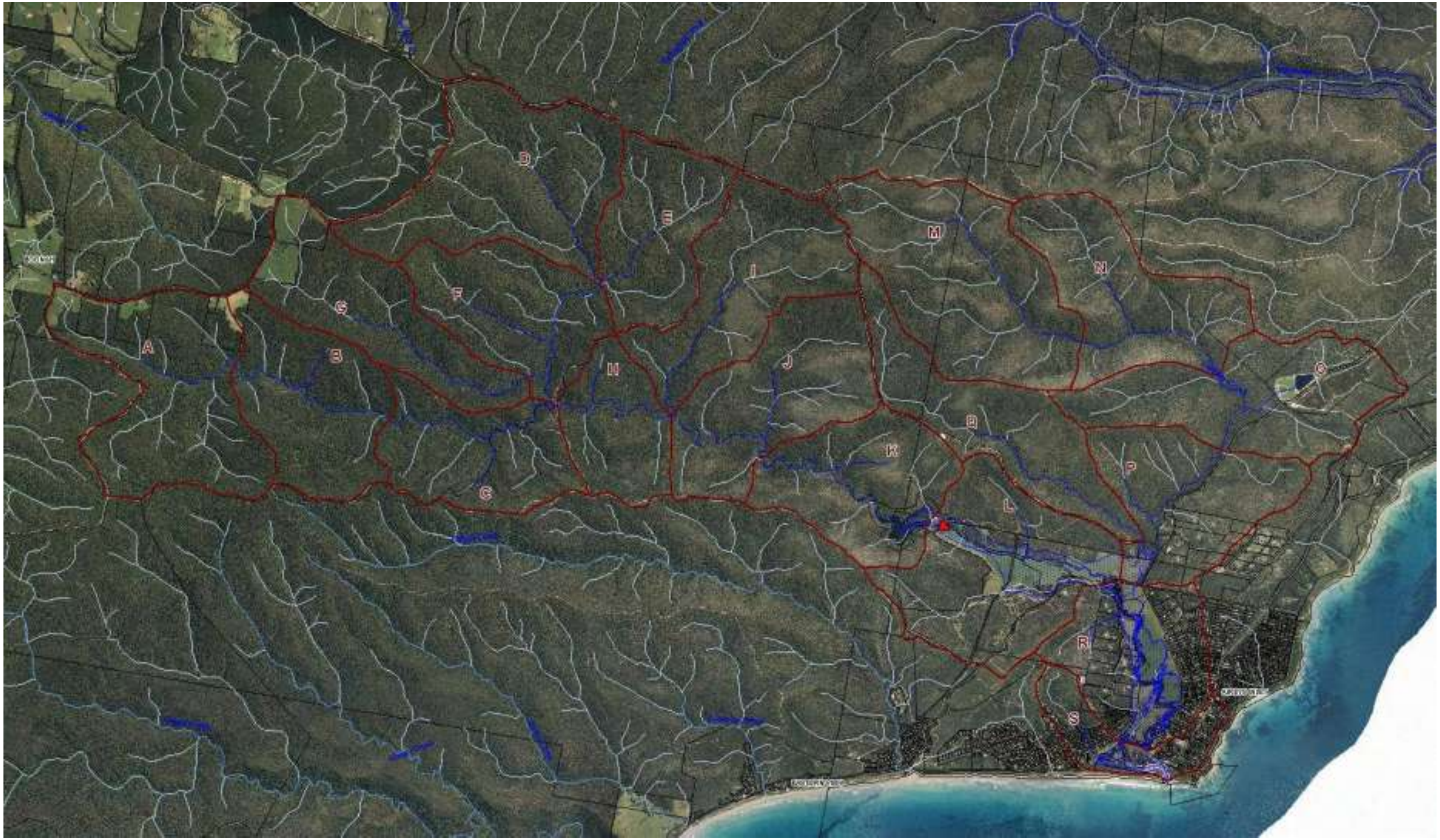


Figure 1 – Painkalac Creek Catchment

Figure 2 – Painkalac Creek Catchment Land Tenure (Crown land shaded light green)

2.5 Statistical Analysis of Peak Flows at Painkalac Creek Dam

A continuous flow record exists at site 235232 (Painkalac Dam) from 1974 to 1991 and from 1999 to today. The continuous flow record at this site is 32 years.

A standard statistical analysis (partial series) of peak flows at site 235232 (Painkalac Dam) has been performed. A log Pearson (LP3) distribution was fitted to the observed data points giving design peak flow estimates for frequencies 1, 2, 5, 10, 20, 50 and 100 yr ARI. Reliable estimates up to 50 yr ARI is expected from this data set.

The degree of confidence in this analysis does rely on the rating curve used at this site. Rural Water Commission and now Thiess Services have maintained this site and the highest gauging performed to date was at 1.73 m (853 Ml/day) in April 1980. The current rating curve extends up to 4.0 m staff reading (6600 Ml/day).

Data was downloaded from the Data Warehouse site for 1974 to 2013.

The five top flood events recorded at site 235232 are shown in the table below.

Rank	Flood Event	Peak Level (m)	Flow (Ml/day)	ARI (yrs)
1	Nov 2007	3.82	5533	23
2	Mar 1983	3.76	5229	18
3	Apr 2001	3.80	4486	9
4	Sept 1976	2.59	4039	6
5	Jul 1978	3.61	3731	5
	…..			
11	Feb 2005	3.15	2955	3
17	Jan 2011	2.70	1925	2

The estimated design discharges based on the best fit to the recorded data is shown in the table below for Site 235232 (Painkalac Dam).

The estimates have been based on a Log Pearson LP3 fit to the highest ranked partial series data. The LP3 fit was adjusted at the low end to fit the plotted points better.

Log 10 stats of this data in Ml/day are :		
	Mean	3.286
	Standard Deviation	0.271
	Skew	0.0

ARI (yrs)	Peak Flow (m3/sec)	Peak Flow (Ml/day)	95% Confidence Limit (Ml/day)	5% Confidence Limit (Ml/day)
1	9	780	280	730
2	22	1900	1590	2350
5	40	3500	2640	4040
10	50	4300	3350	5520
20	62	5400	3970	7330
50	78	6700	4670	10400
100	89	7700	5120	13330

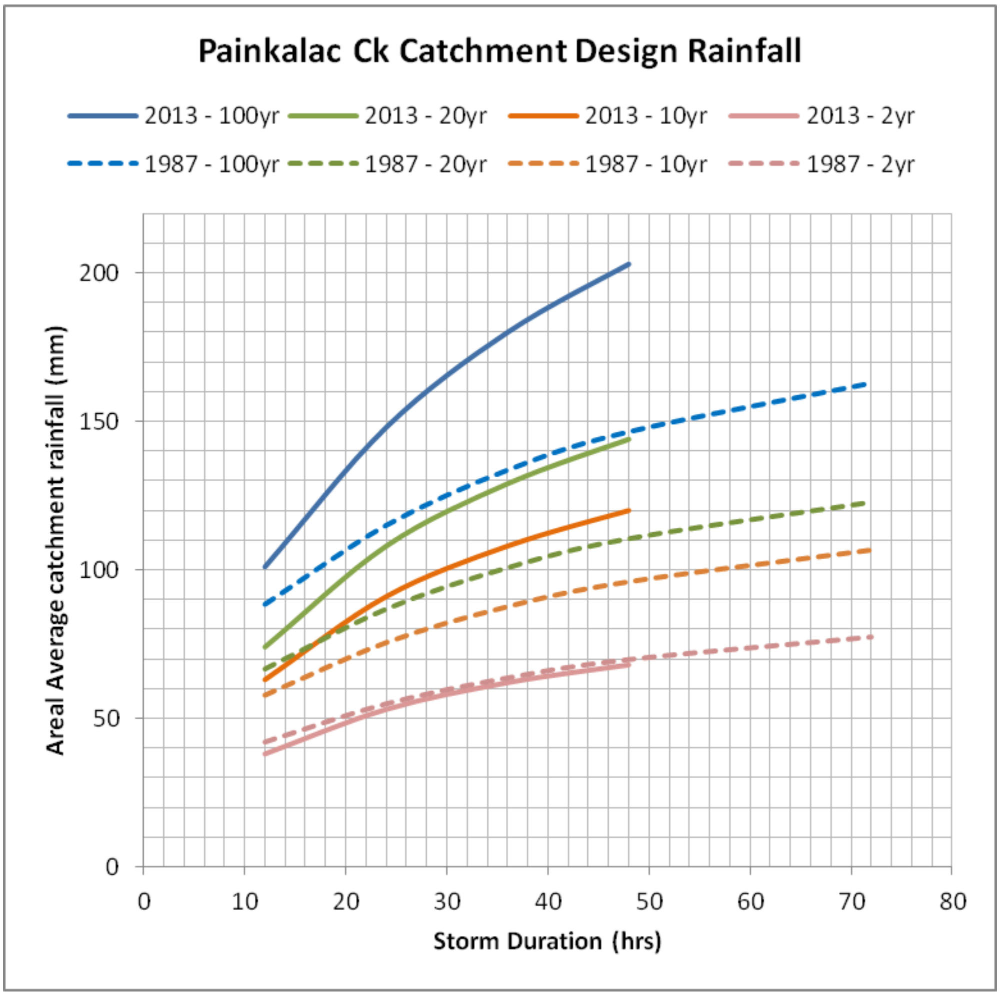
Appendix C shows the data and graph of the fitted frequency curve against recorded data with 5 and 95% confidence limits.

2.6 Design Run Parameters for RORB Model

The following calibrated RORB model catchment storage parameters were used :

Kc = 9.0 and m = 0.75

The Bureau of Meteorology has just released Intensity-Frequency-Duration (IFD) data for Australia. This new data has significant differences in totals from the previous data presented in Australian Rainfall & Runoff Vol 1 (AR&R) [Ref 1]. The graph below shows the difference in average catchment rainfall between the old and new estimates for the Painkalac Creek catchment. This study has used the new IFD data.

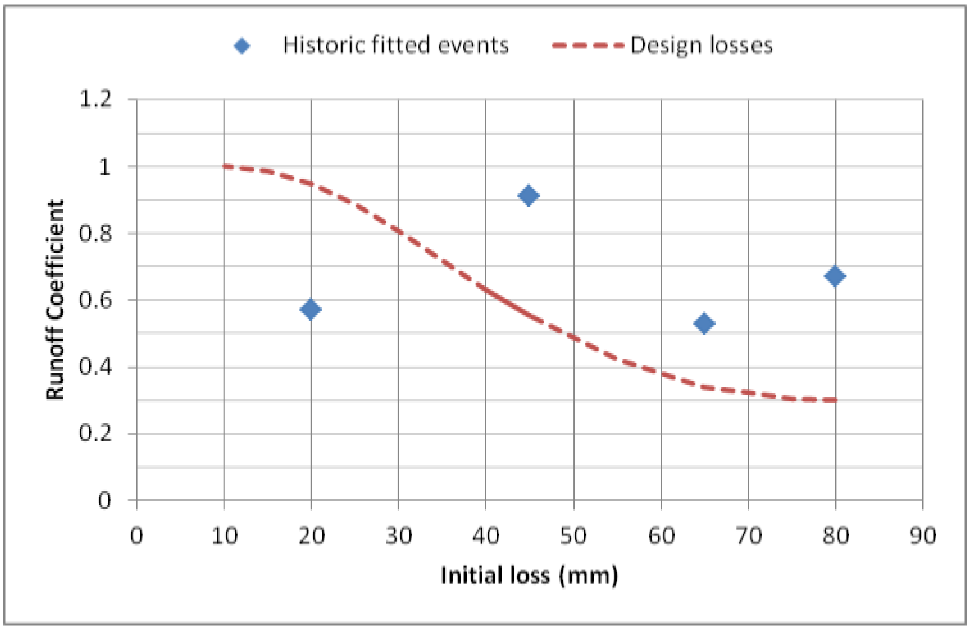


Design rainfall totals for each subarea were estimated for ten representative points evenly spaced over the catchment area. New IFD values for these 10 sites were taken from the BoM web site for 12, 24, 48 and 72 hour duration. Intermediate storm duration of 36 hours was created assuming a Log Normal relationship.

For each duration a graphical grid of these rainfall depths was created using triangulation in Mapinfo. The subarea design depths were then read from each graphical grid giving a variation from the top to the bottom for all subareas in the model. The new IFD data has not been incorporated into RORB multiple design run mode at this stage so individual design runs files were created for ARI and storm duration. Appendix D shows the BoM IFD output for a particular site (Anglesea VPG) for a range of ARI and storm duration. Appendix E shows the design subarea rainfall totals used for the RORB design runs and a map showing design rainfall isohyets for the 100yr ARI 48 hour duration storm.

The RORB model was then ran in design mode for each ARI / storm duration and design rainfall loss rates were calibrated to match the peak flow estimates from the statistical analysis at the gauging site 235232. The longer storm duration was found to generate the highest peak flow for ARI's 1 to 100 years.

The initial loss - runoff coefficient loss model gave the best fits in the calibration runs and the same model is used in the design runs. The relationship between initial loss and the runoff coefficient in design mode is difficult to estimate from the four fitted events. This forested catchment is expected to have some initial loss in a very wet condition of say 10-15 mm's with a Rc= 1.0. The runoff coefficient is also expected to flatten out for initial losses > 80 mm's to value of 0.3. The average initial loss from the fitted events is 50 mm's. The adopted design relation for losses is shown as the red line in the graph below. The part used for design runs is shown as the solid red line.



The table below shows the adopted rainfall losses used in the RORB design calibrations runs to best match the statistical analysis at Painkalac Reservoir (Site 235232). There is a slight increase in rainfall losses from 2 to 100 year ARI events.

ARI (yrs)	Losses		Peak Q (m3/s)		Peak Q (m3/s)
	IL (mm's)	RC	RORB Model	Stat Analysis	At Great Ocean Road
1	43	0.58	9	9	11
2	34	0.74	22	22	29
5	38	0.66	40	41	54
10	39	0.65	50	50	73
20	39	0.65	62	63	92
50	39	0.65	79	78	113
100	41	0.62	89	89	124
200	41	0.62	104	110	135

No allowance for base flow has been made. Base flows from observed events predict that base flow will be less than 0.1 m^3/s for most flood events.

2.7 Climate Change Effect on Design Rainfall

Climate change impact on flood producing rainfall events on the Painkalac Creek catchment is difficult to estimate at this time with both increasing and decreasing trends predicted. In a recent paper on climate change for the Corangamite Region prepared by CSIRO (Atmospheric Research) [Ref 3] indicates that :-

- Annual rainfall totals are likely to decrease, ranging from +10% to -25% by 2070;
- Projected rainfall decreases are strongest in the spring and then winter, the most likely time for floods from this catchment (i.e. 6 of top 10 from 1975 to 2012);
- Extreme short duration rainfall events may become more intense and more frequent with thunderstorms.

At this stage no change in design rainfall due to climate change is proposed in this study.

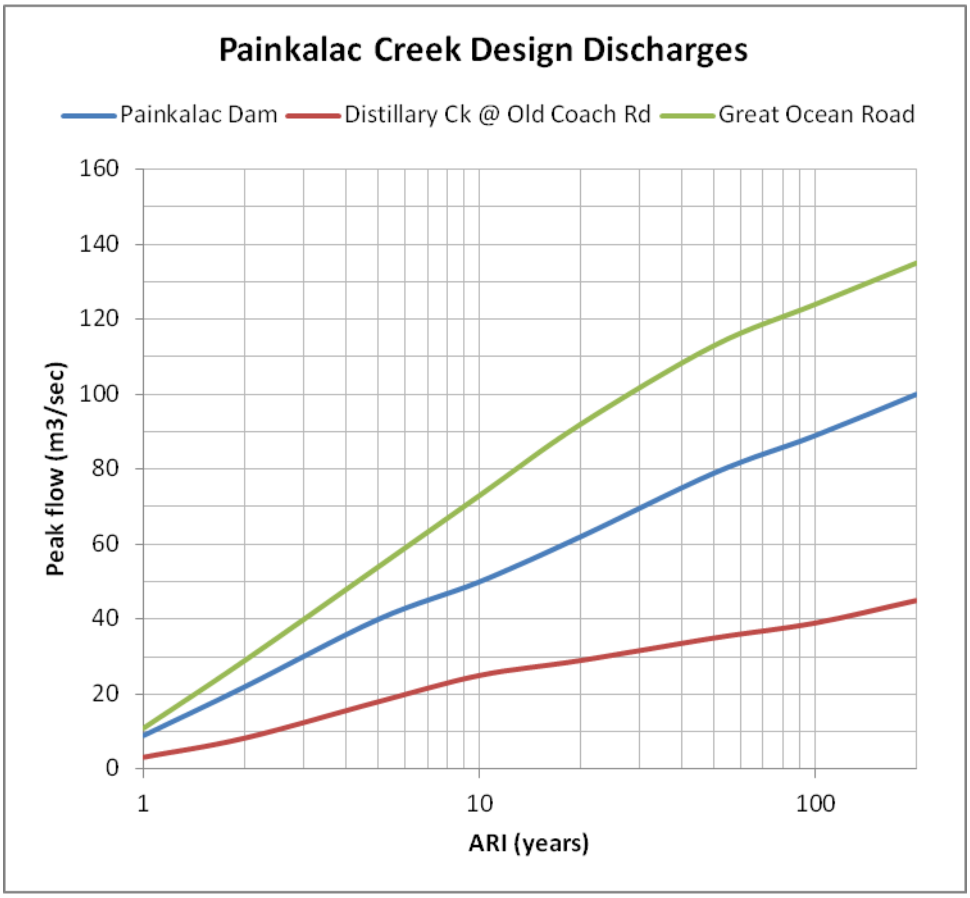
2.8 Results of RORB Modelling

The critical design storm duration was found to vary from 24 to 48 hours depending on design frequency.

The peak flows for the Painkalac Creek resulting from this work is shown in the table below. The 100yr ARI estimate at Great Ocean Road is 124 m^3/sec resulting from a design storm with a 36 hour duration. The critical storm duration for the 100 yr ARI estimate at Painkalac Reservoirwas 24 hours.

The adopted design flows for the Painkalac Creek are shown in the table below at four locations.

ARI (yrs)	Dam Site (235232) (m3/s)	Painkalac Creek at Old Coach Road (m3/s)	Distillery Creek at Old Coach Road (m3/s)	Great Ocean Road (m3/s)
1	9	9	3.2	11
2	22	24	8.3	29
5	40	42	18	54
10	50	52	25	73
20	62	65	29	92
50	79	80	35	113
100	89	89	39	124
200	104	104	50	135



Painkalac Creek Flood Frequency Curves

3.0 FLOODPLAIN HYDRAULIC MODELLING

The Painkalac Creek Estuary level is controlled by a sand bar (berm) at the ocean end of the estuary. The sand bar height builds up from sand blown along the beach blocked by the cliff and by normal wave action along this exposed beach. The height of the sand bar varies from 1.8 to 2.6m AHD and is mechanically opened when the estuary water level exceeds 1.8m AHD. The time for a catchment flood event to peak at Great Ocean Road varies between 12 to 24 hours from the start of rain allowing little warning time to lower the sand bar or open the estuary artificially to the sea. A flood event will scour a channel through the sand bar when it is overtopped with an estimated base width of 15 to 25 metres and a depth up to 1.8 metres.

Unfortunately there is little recorded data on the sand bar breach channel dimensions (ie. width, slope and invert level) and the CCMA will in future record this information after flood events. The boundary condition of this sandbar for any hydraulic model of the Painkalac Creek estuary is critical in estimating flood levels along the floodplain. The sensitively of the assumptions on flood levels by sand bar height and breach channel dimensions and timing are discussed in this report.

3.1 Sea Surge into Estuary

Current and future ocean tide levels are also is an important boundary condition for a hydraulic model of the Painkalac Creek. The CSIRO released a paper on “The effect of climate change on extreme sea levels along Victorian coast” in November 2009 [Ref 4].

The table below gives the expected peak tidal surge levels for the future assuming a sea level rise based on IPCC 2007 A1FI scenario. The levels shown are mean sea levels in deep water and therefore *do not include wave setup or wave run-up*.

ARI (yrs)	Current Climate (m AHD)	Year 2030 (m AHD)	Year 2070 (m AHD)	Year 2100 (m AHD)
	Sea Level rise	0.15	0.47	0.82
1	0.80			1.56
2	0.95			1.75
5	1.15			1.98
10	1.32	1.47	1.79	2.14
20	1.46	1.61	1.93	2.28
50	1.59	1.74	2.06	2.41
100	1.69	1.84	2.16	2.51

The current policy for CMA’s with respect to sea surge events is to use current climate levels with a higher freeboard of 500mm. For a starting height of the estuary berm at 2.1 m AHD, it is unlikely that sea surges will be a critical flood event for the Painkalac Creek floodplain for some time.

However, it is likely that sea level rise will naturally increase the height of the estuary berm with time that will result in an increase in flood frequency on the land side of the berm. This will require careful management of the berm height in the future.

3.2 Recommended Design Scenario

A HEC-RAS model of the Painkalac Creek was created using the following data:

- Cross sections of the floodplain taken from the Coastal Lidar data set with a reported accuracy of 0.05m vertically. The average distance between cross sections is 180 metres. The location of the cross sections are shown in appendix F maps as red lines across the floodplain;
- Run the HEC-RAS model in 1D full dynamic mode;
- Design hydrographs are taken from the RORB model output corresponding to the critical storm duration for the peak flow;
- Normal tide cycle is assumed at the downstream boundary condition. An average tide levels between the neap and king tide times are used;
- The starting level of the estuary berm is 2.1 m AHD and allow the berm to naturally begins to breach once it has overtopped with 50mm of water;
- The breach channel is assumed to develop to full size within 3 hours of the berm overtopping. The berm will be partially saturated due to flows passing through the berm to the ocean before overtopping occurs;
- The maximum breach channel is 25 metres wide and 1.5 metre deep during the peak flood flow. The breach channel width will continue to grow as the peak flood flow has passed.

During the assumed breach time of 3 hour, 2600 m^3 (average 22 m^3/min) of sand is scoured out requiring an average sand content of 3700ppm in the flow that passes in that time. The average flow velocity during the breach development is 2.1 m/sec and the breach time is considered a reasonable assumption at this time for the berm to fully breach. The stream power during the berm breach range from 20 to 140 N/m sec.

The photograph below was taken after the 4 November 2007 flood event and shows the extent of the breach channel. No measurements of the berm were taken before during or after the flood event.

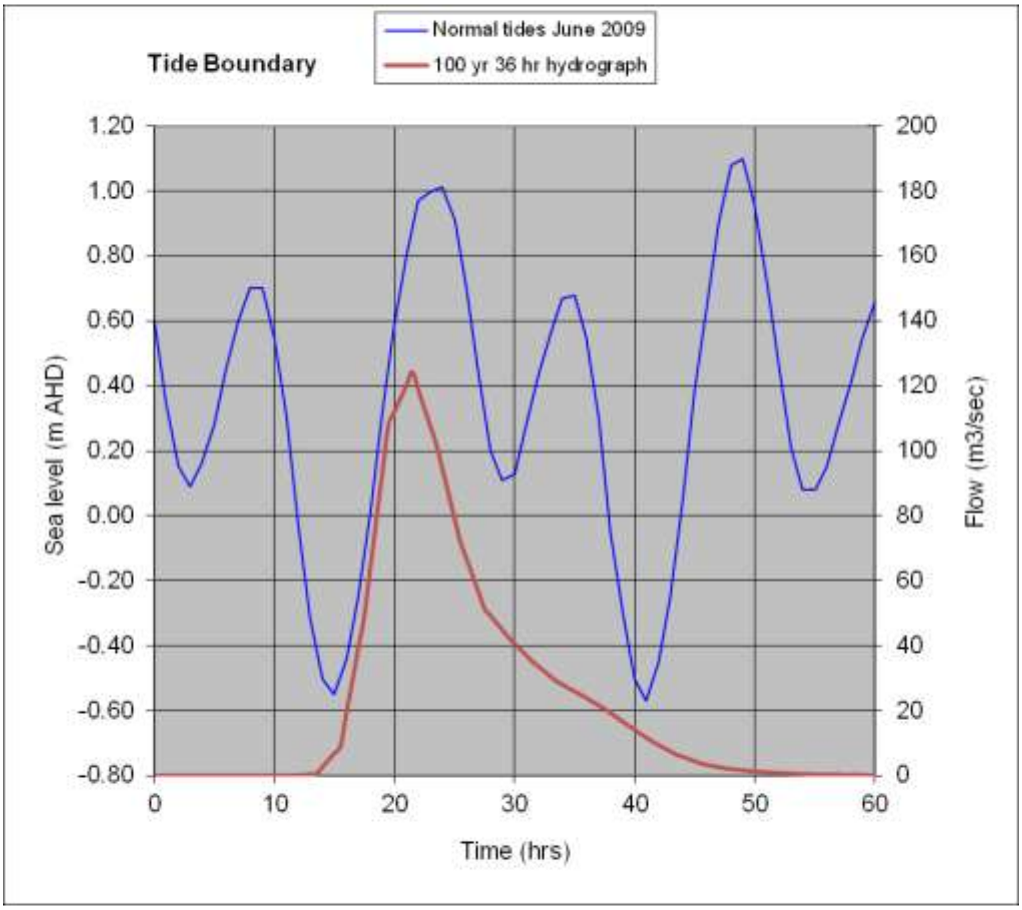
Estuary breach after the 4 Nov 2007 flood

3.3 Hydraulic Model parameters

The channel roughness in the hydraulic model has been estimated to be:

- Normal river section lower section, n = 0.025
- Outside normal river section, n = 0.04 to 0.05
- Breach channel in sand, n = 0.02

The boundary conditions for the tide and flow input is shown below. The peak of the flow coincides with a high tide.



The berm breach mechanism is setup in HEC-RAS as an inline dam breach structure.

The breach channel to the sea will have a slope steeper than 1/100 and be super-critical flow and therefore will not have hydraulic influence upstream on the dam break mechanism used in this unsteady model. Hydraulic calculations have confirmed that a breach channel fully formed with a slope of 1/100 will be super-critical flow for a range of flows 5 to 200 m^3/sec.

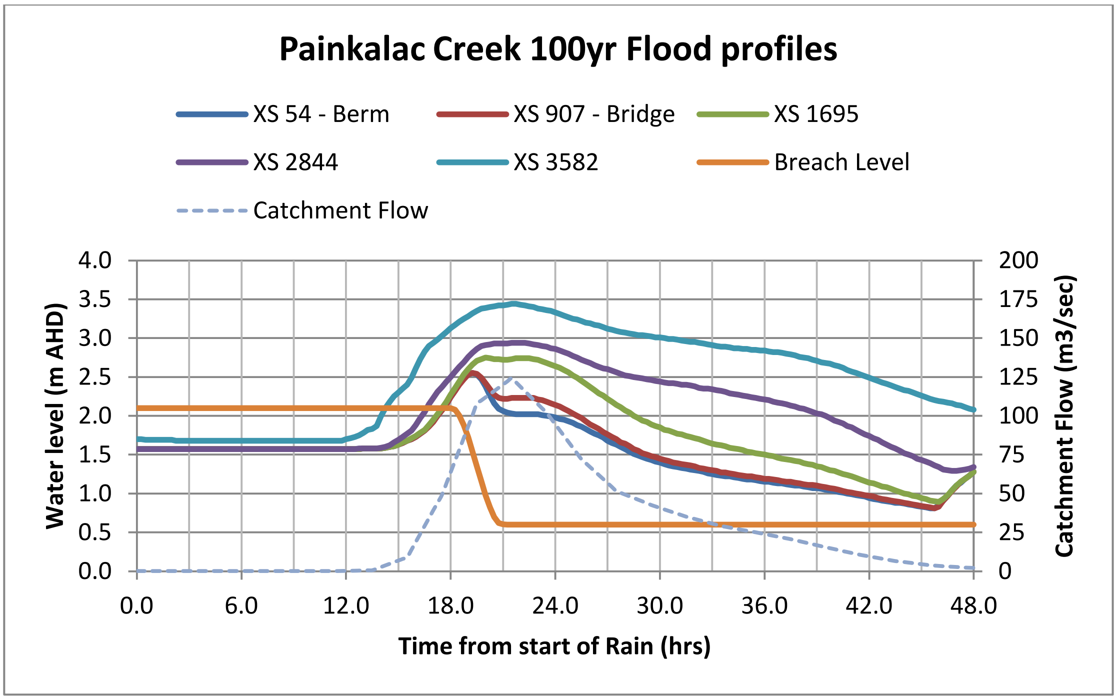
The breach channel is assumed to form and hold the channel shape specified in the dam break setup for the duration of the flood event. In reality the breach channel will continue to enlarge beyond that is specified for the dam break, especially at the channel sides. However it is considered that the dam break mechanism is a reasonable assumption in estimating peak levels on the Painkalac Estuary as flood level peaks occur during the breach channel is being formed.

The characteristic of the adopted dam breach mechanism is shown in Appendix H. Model results of sediment transport required by the assumed breach mechanism is confirmed by applying Yang Sediment Transport Function during the breach time.

3.4 Expected Effect on Flood Levels for Design Scenario

The figure below shows the water surface levels with time for five cross sections along the floodplain. The cross section names represent the distance from the ocean in metres, so XS 907 is 907 metres upstream of the ocean and is just upstream of the Great Ocean Road bridge. The orange line shows the berm height with the breach occurring between 18 and 20 hours, dropping the berm down to 0.6 m AHD. The breach is fully formed about 1.5 hour before the peak flow arrives from the catchment. The timing of the breach occurring with respect to the peak flow is critical to the resulting water surface levels, especially for the floodplain upstream of the Great Ocean Road bridge (XS 907 to XS 1695).

At each cross section the maximum water level occurs at different times due to different combine flows from the catchment flood and flows into and out of floodplain storage. The XS 3582 is far enough upstream to be not affected by the berm breaching. The kick in water surface level at time 46 hours for XS 54, 907 and 1695 is due to incoming tidal flow from the ocean.

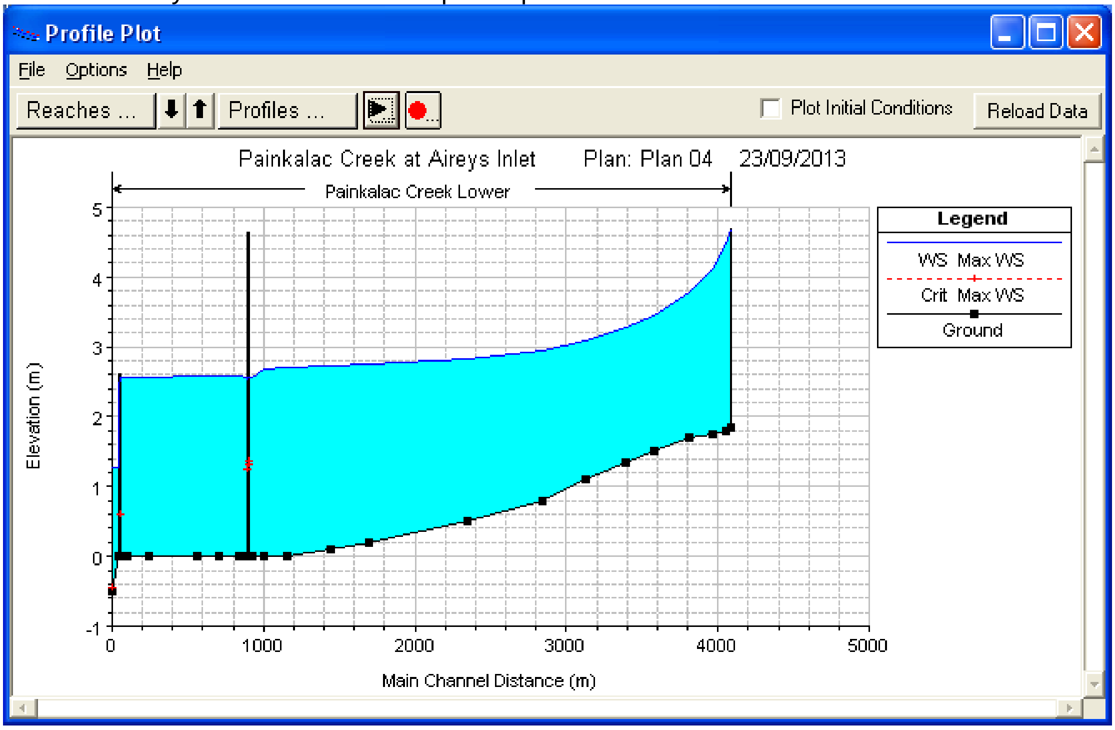


The maximum water surface levels along the floodplain for the 100 year ARI event is shown in the following table with the HEC-RAS profile plot. More detail for each cross section can be found appendix G.

	XS 53	XS 907	XS 1695	XS 2844	XS 3582
Distance from Ocean (m)	53	907	1695	2844	3582
Invert level (m AHD)	0.6	0.0	0.2	0.8	1.5
Maximum WSL (m AHD)	2.55	2.55	2.75	2.93	3.44
Time of peak level (hrs)	18.75	18.75	22.5	22.0	21.5
Velocity at peak level (m/sec)	0.4	1.3	0.5	0.9	1.4
Peak Flow (m3/s)	119	120	118	121	124
Max Velocity (m/s)	1.4	2.9	1.4	0.9	1.4

Maximum velocities at the lower cross sections occur after the peak level and reach 1.5 m/sec at best. The maximum velocity through the bridge section is 2.9 m/s.

HEC-RAS 100yr ARI maximum WSL profile plot.



Flood profiles have been modelled for 5, 10, 20, 50, 100 and 200 year ARI flood events using the same berm breach scenario. The results are shown in the table below.

ARI (yrs)	Peak Flow (m3/sec)	Maximum Water Surface Level (m AHD)					
		XS 53	XS 1003	XS 1438	XS 1695	XS 2844	XS 3582
5	54	2.40	2.44	2.46	2.47	2.60	3.09
10	73	2.45	2.52	2.54	2.55	2.71	3.21
20	92	2.49	2.58	2.61	2.63	2.80	3.30
50	113	2.53	2.64	2.68	2.70	2.89	3.39
100	124	2.55	2.67	2.72	2.75	2.93	3.44
200	135	2.57	2.74	2.81	2.83	3.01	3.49

The results for higher frequency floods show very similar results in the lower reaches of the floodplain with only cm’s difference. This is caused by the influence of the initial berm height and the breaching mechanism.

During the Nov 2007 flood event a flood level was recorded at 12a River Road of 2.57m AHD. This flood event has an average recurrence interval of 23 years and the location is at river chainage 1570m. No information is known of the condition of the berm before the flood but the recorded flood level is 5cm below the 20 year ARI model results presented in the table above and suggests that the results may be slightly conservative.

3.5 Sensitivity of model parameters

Testing of the sensitivity of model parameters on calculated flood levels has been completed for the 100 year ARI 36 hour flood event.

The following model parameters have been tested individually to determine impact:

- Starting berm height
- Final breach channel width and invert level
- Time for full breach channel to form
- Flood peak and high tide timings

The results are shown in the following table. Parameters breach channel width and invert level and timing of flood peak with high tide are shown not to be sensitivite to change. However starting berm height and time for full breach channel to develop are very sensitivity to change and can result in significant increases in water surface levels in the lower reaches.

Model sensitivity testing results:

100 year ARI - 36 hour flood event

Model Parameter	Starting Valve	Peak Water Surface Level (m AHD)				
		XS 53	XS 1003	XS 1695	XS 2844	XS 3582
Starting Berm Height (m AHD)	1.9	2.28	2.61	2.71	2.92	3.44
	2.1	**2.55**	**2.67**	**2.75**	**2.95**	**3.44**
	2.3	2.78	2.89	2.94	3.04	3.45
Full Breach channel width (m)	20	2.58	2.71	2.78	2.96	3.44
	25	**2.55**	**2.67**	**2.75**	**2.95**	**3.44**
	30	2.53	2.65	2.72	2.94	3.44
Full Breach channel invert (mAHD)	0.8	2.57	2.70	2.77	2.96	3.44
	0.6	**2.55**	**2.67**	**2.75**	**2.95**	**3.44**
	0.4	2.54	2.65	2.72	2.94	3.44
Time for breach channel to fully developed (hours)	2.0	2.41	2.61	2.71	2.95	3.44
	3.0	**2.55**	**2.67**	**2.75**	**2.95**	**3.44**
	4.0	2.67	2.81	2.87	3.00	3.45
Time between flood and high tide peaks (hours)	-3.0	2.55	2.67	2.75	2.95	3.44
	-2.0	**2.55**	**2.67**	**2.75**	**2.95**	**3.44**
	-1.0	2.55	2.67	2.75	2.95	3.44
	0.0	2.55	2.67	2.75	2.95	3.44

The starting values and results bolded in the table above are the design base case.

The properties that are flooded from the 100 year ARI flood event are located between chainage 1200 and 1800 metres and are represented in the table above by XS 1695.

The base case model peak water levels occur just after the berm breach begins so breach channel width and invert has little impact on the peak results.

The starting berm height has the most significant impact on resulting flood levels in the lower and middle reaches. If the berm height is higher than the base case, then the time when overtopping begins is delayed and closer to the peak flood flow. The next significant impact is the time for berm breach to fully develop. If the breach time is increased then flood levels increase as the delay again moves closer to the flood flow peak.

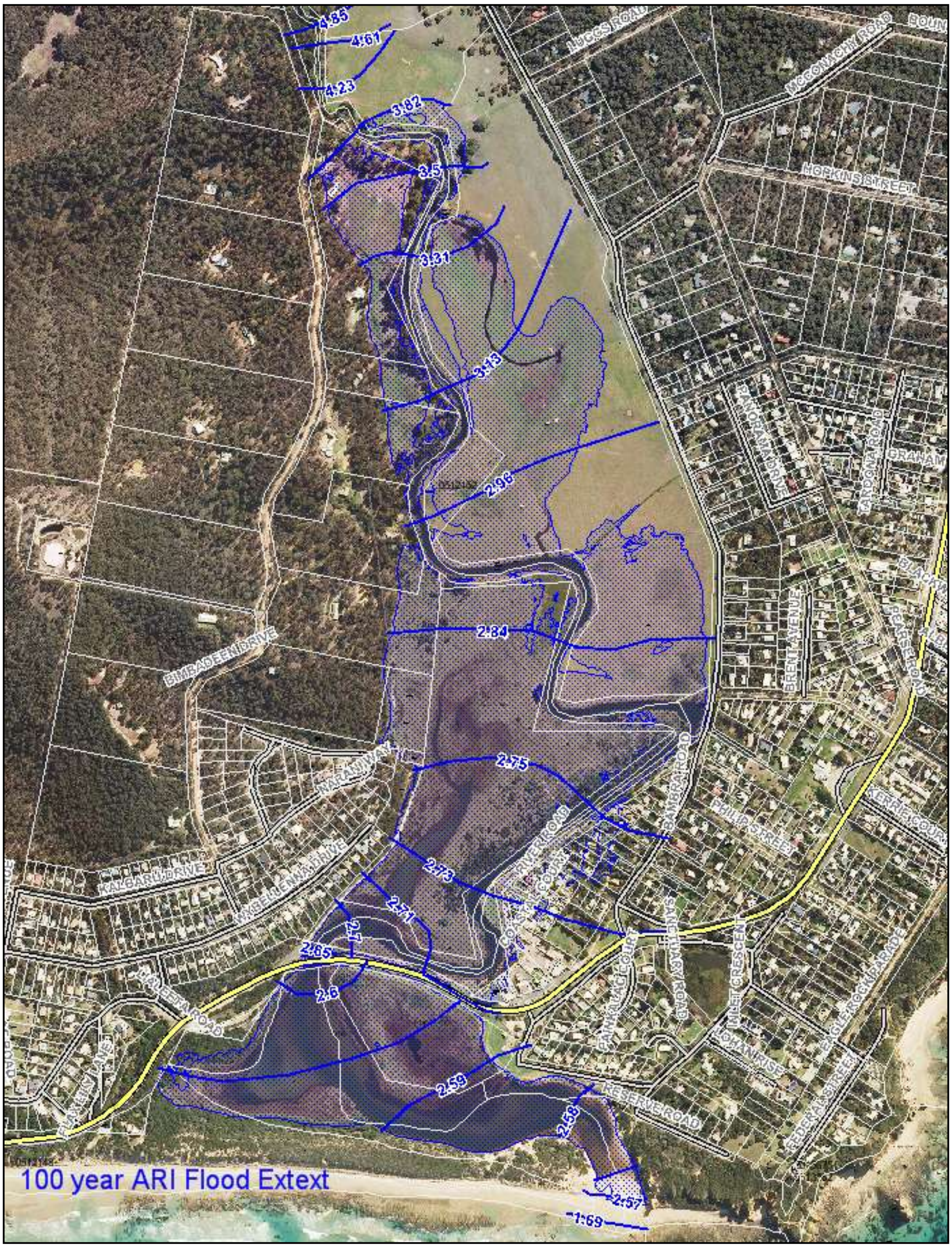
Further monitoring of the berm height of the estuary over time and documenting the berm breach development during flood and opening events is required. Survey of berm breaches after floods is also required to record channel width and invert level. Photographic record (say) every 15 mins could be taken from a safe distance during the breach development to confirm the assumptions made for the breach mechanism.

3.6 Recommended flood extent and levels

The design scenario for the hydraulic model is adopted at this stage with a recommendation of monitoring the berm height of the estuary and surveying the breach channel as soon after a flood event. Photographic record of the breach development should also be taken.

The 100 year ARI flood levels adopted are based on the total energy levels at each cross section reflecting the water level at the floodplain edge. The map below shows the resulting extent. The blue lines across the floodplain are the cross sections in the HEC-RAS model and the numbers represent the 100 year ARI flood levels.

The recommended freeboard for new development within the floodplain is 300 mm above the 100 year ARI flood level.



100 year ARI flood extent for Painkalac Creek

The photograph below shows the flood extent during the November 2007 event. The Great Ocean Road is still open for traffic.

Nov 2007 Flood taken from Western side looking back to Aireys Inlet.

Taken during opening 11 July 2013. Note the previous cut on the left hand side.

Opening 18 Aug 2012 – looking out to sea

Opening 18 Aug 2012 – looking back up the estuary

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1. "Australian Rainfall and Runoff - A Guide to Flood Estimation", by IEAust. Reprinted edition 2001.
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3. "Climate Change in the Corangamite Region", prepared by CSIRO (Atmospheric Research) on behalf of the Victorian Govt (DSE), 2004.
4. "The Effect of Climate Change on Extreme Sea Levels along Victorian Coast", Prepared by CSIRO for DSE 'Future Coast' program. McInnes etal, Nov 2009.
5. "HEC-RAS River Analysis System – User's Manual" by US Army Corps of Engineers. Version 4.0 March 2008.
6. "HEC-RAS River Analysis System - Hydraulic Reference Manual" " by US Army Corps of Engineers. Version 4.1, January 2010.
7. "A Study of the Land in the Catchments of the Otway Range and Adjacent Plains". TC-14 by A.J. Pitt for the Soil Conservation Authority, Victoria, Australia, 1981. (ISBN 0 7241 1908 6)
8. "Entrance Modelling and the Influence on ICOLL Flood Behaviour" by D Lyons and D Williams, BMT WBM Newcastle, 2012.

1 APPENDICES

A **RORB Model catchment file**

B **Calibration of RORB Model**

C **Statistic Analysis of peak flows at Site 235232 (Painkalac Creek downstream of Reservoir)**

D **AR&R IFD Data**

E **Design Subarea Rainfall totals**

F **Design Discharges – RORB output**

G **HEC-RAS outputs**

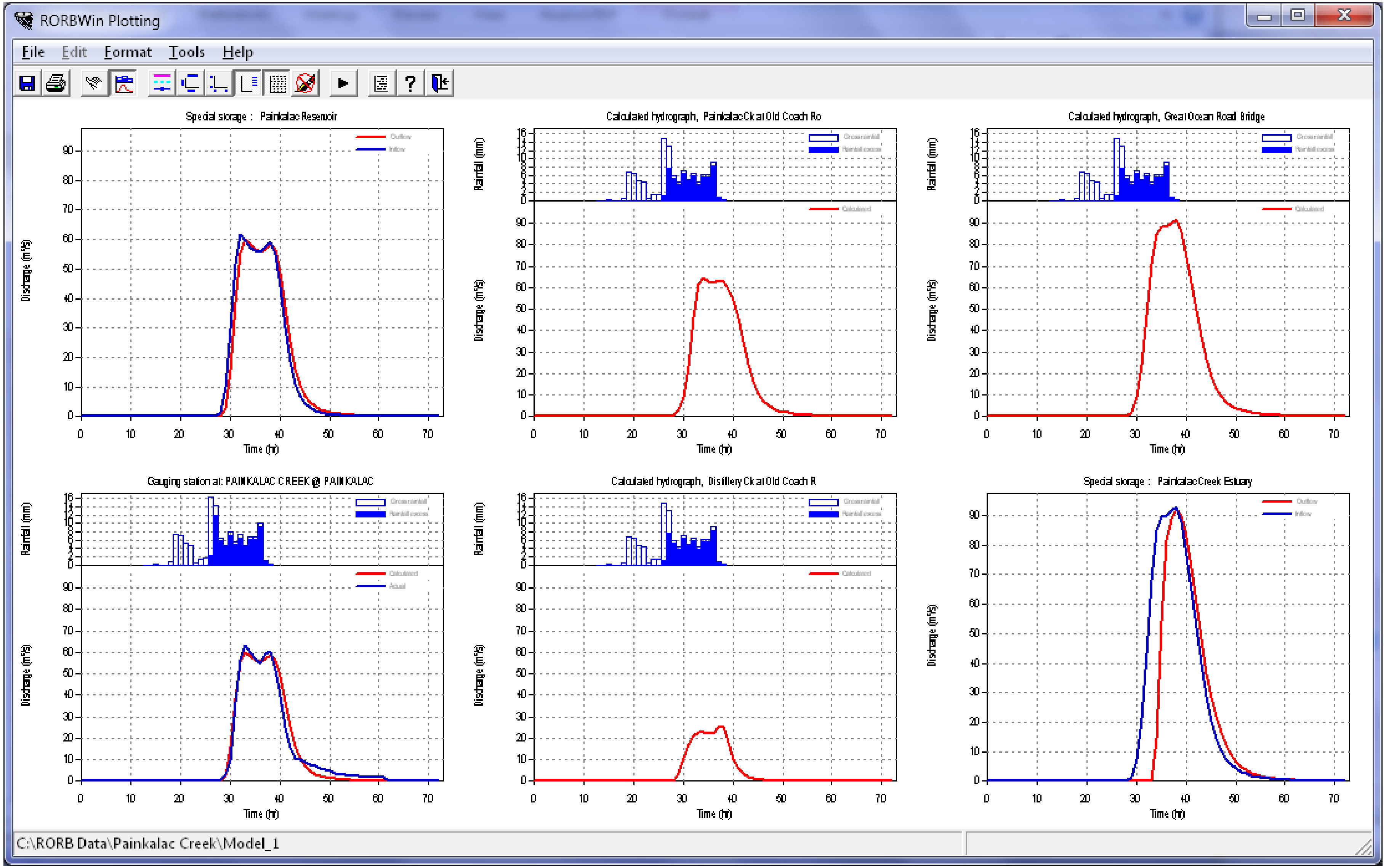
H **Berm Breach characteristics**

Appendix A – RORB catchment file

```
Painkalac Creek
C      Rorb Model - Design Runs
C      Existing catchment conditions
C      T Jones CCMA (June 2013)
C      ---------------------------------------------------
C      Model 1 - reaches based on L (kms)
C      Kc= 9.0           m= 0.75
C      Fitted to 3 events - Apr2001, Feb2005, Nov2007
C      Dav= 12.25 kms    Area= 61.3 sq kms
C      Design parameters
C      IL= 40mm    Rc= 0.63 (varies with ARI)
C                  or CL= 1.8 mm/hr for 100yr ARI
C      Painkalac Dam assumed 80% full
C
1,           all reaches natural
1,2.54,-99,  add Subarea A
3,           store hydrograph
1,0.5,-99,   add Subarea B
4,           add hydrograph
5,2.6,-99,   Route nature reach
3,           store hydrograph
1,0.65,-99,  add Subarea C
4,           add hydrograph
5,1.29,-99,  Route nature reach
3,           store hydrograph - start new trib
1,1.67,-99,  add Subarea D
3,           store hydrograph
1,1.07,-99,  add Subarea E
4,           add hydrograph
5,1.21,-99,  Route nature reach
3,           store hydrograph
1,1.09,-99,  add Subare F
4,           add hydrograph
5,0.49,-99,  Route nature reach
3,           store hydrograph
1,2.82,-99,  add Subarea G
4,           add hydrograph
5,0.42,-99,  Route nature reach
4,           add trib flow to main creek
5,0.68,-99,  Route nature reach
3,           store hydrograph
1,0.48,-99,  add Subarea H
4,           add hydrograph
5,1.37,-99,  Route nature reach
3,           store hydrograph
1,2.11,-99,  add Subarea I
4,           add hydrograph
5,1.56,-99,  Route nature reach
3,           store hydrograph
1,0.92,-99,  add Subarea J
4,           add hydrograph
5,1.61,-99,  Route nature reach
3,           store hydrograph
1,0.64,-99,  add Subarea K
4,           add hydrograph
5,1.82,-99,  Route nature reach
16
Painkalac Reservoir
3,29.78,1,   start reservoir 80% full below FSL at 29.78 m AHD
```

```
30.0,32,2.15,-99, 32m spillway @ RL 30.00 m AHD (best fit to BW Data)
1,9
26.8,165420, 27,184830, 28,292860, 29,419220 ,30,563920
30.5,643150, 31,726960 , 32,908330 ,33,1108030 ,-99
5,1.37,-99, Route nature reach
3,          store hydrograph
1,0.62,-99, add Subarea L
4,          add hydrograph
5,1.43,-99, Route nature reach
7
Painkalac Ck at Old Coach Road
3,          store main creek flow
C     start Distillery Creek catchment
1,3.13,-99, add Subarea M
3,          store hydrograph
1,0.79,-99, add Subarea N
4,          add hydrograph
5,1.92,-99, Route nature reach
3,          store hydrograph
1,1.03,-99, add Subarea O
4,          add hydrograph
5,1.81,-99, Route nature reach
3,          store hydrograph
1,0.67,-99, add Subarea P
4,          add hydrograph
5,0.56,-99, Route nature reach
3,          store hydrograph
1,2.96,-99, add Subarea Q
4,          add hydrograph
5,0.6,-99,  Route nature reach
7
Distillery Ck at Old Coach Rd
4,          add Distilley Ck  flow to main creek
5,0.2,-99,  Route nature reach
3,          store hydrograph
1,0.59,-99, add Subarea R
4,          add hydrograph
5,3.07,-99, Route nature reach
7
Great Ocean Road Bridge
3,          store hydrograph
1,0.85,-99, add Subarea S
4,          add hydrograph
5,0.74,-99, Route nature reach
16
Painkalac Creek Estuary
3,1.5,1,    start estuary at 1.5 m AHD
2.5,500,2.0,-99,  Sand bar 50m wide @ RL 2.5 m AHD (vary with
scenario)
1,4
1.5,0, 2.0,213000, 2.5,647000, 3.0,1350000, -99
0,          End of control codes
C    19 No. Subarea (Total  catchment 61.3 sq kms)
4.29,3.14,2.71,4.46,2.56,2.91,3.12,1.90,3.45,3.50
3.06,4.18,4.76,3.37,3.76,3.04,3.10,2.82,1.20, -99
0,-99,            All areas pervious
```

Appendix B – Calibration of RORB Model



Fit Event :4-6 November 2007 Flood Event

Routing results:
****
Painkalac Creek
Initial time:
4 Nov 2007 Flood (zero = 0am 3/11/2007)
FIT run no. 1

Parameters: kc = 9.00 m = 0.80

Loss parameters	Initial loss (mm)	Runoff coeff.
Interstation area above:		
PAINKALAC CREEK @ PAI	45.00	0.91
Catchment outlet	45.00	0.91

*** Gauging station at: PAINKALAC CREEK @ PAINKALAC

	Hydrograph		Error		
	Calc.	Actual	Abs.	Percent	
Peak discharge,m^3/s	59.87	63.60	-3.73	-5.9	
Time to peak,h	33.0	33.0	0.0	0.0	
Volume,m^3	2.45E+06	2.46E+06	-5.02E+03	-0.2	
Av.abs.ord.err,m^3/s			1.4	13.1	over dur. of calcs
Time to centroid,h	36.8	37.4	-0.7	-1.8	
Lag (c.m. to c.m.),h	4.98	5.66	-0.68	-12.0	
Lag to peak,h	1.21	1.21	0.00	0.0	

*** Calculated hydrograph, Painkalac Ck at Old Coach Ro

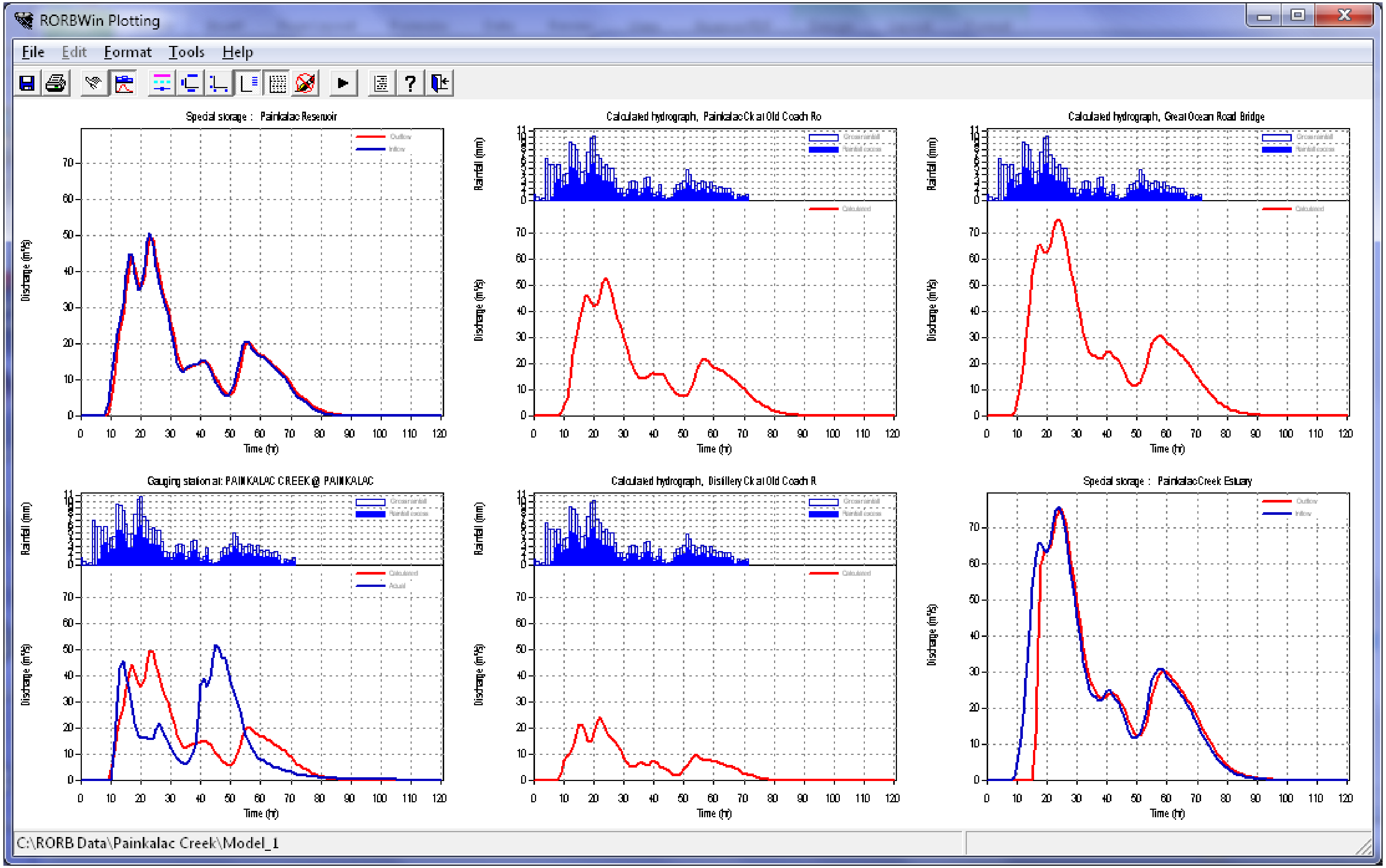
	Hydrograph
	Calc.
Peak discharge,m^3/s	64.55
Time to peak,h	34.0
Volume,m^3	2.65E+06
Time to centroid,h	37.5
Lag (c.m. to c.m.),h	5.61
Lag to peak,h	2.15

*** Calculated hydrograph, Distillery Ck at Old Coach R

	Hydrograph Calc.
Peak discharge,m^3/s	25.48
Time to peak,h	38.0
Volume,m^3	8.57E+05
Time to centroid,h	35.6
Lag (c.m. to c.m.),h	3.00
Lag to peak,h	5.43

*** Calculated hydrograph, Great Ocean Road Bridge

	Hydrograph Calc.
Peak discharge,m^3/s	91.65
Time to peak,h	38.0
Volume,m^3	3.62E+06
Time to centroid,h	38.0
Lag (c.m. to c.m.),h	5.91
Lag to peak,h	5.95



Fit Event :21-24 April 2001 Flood Event

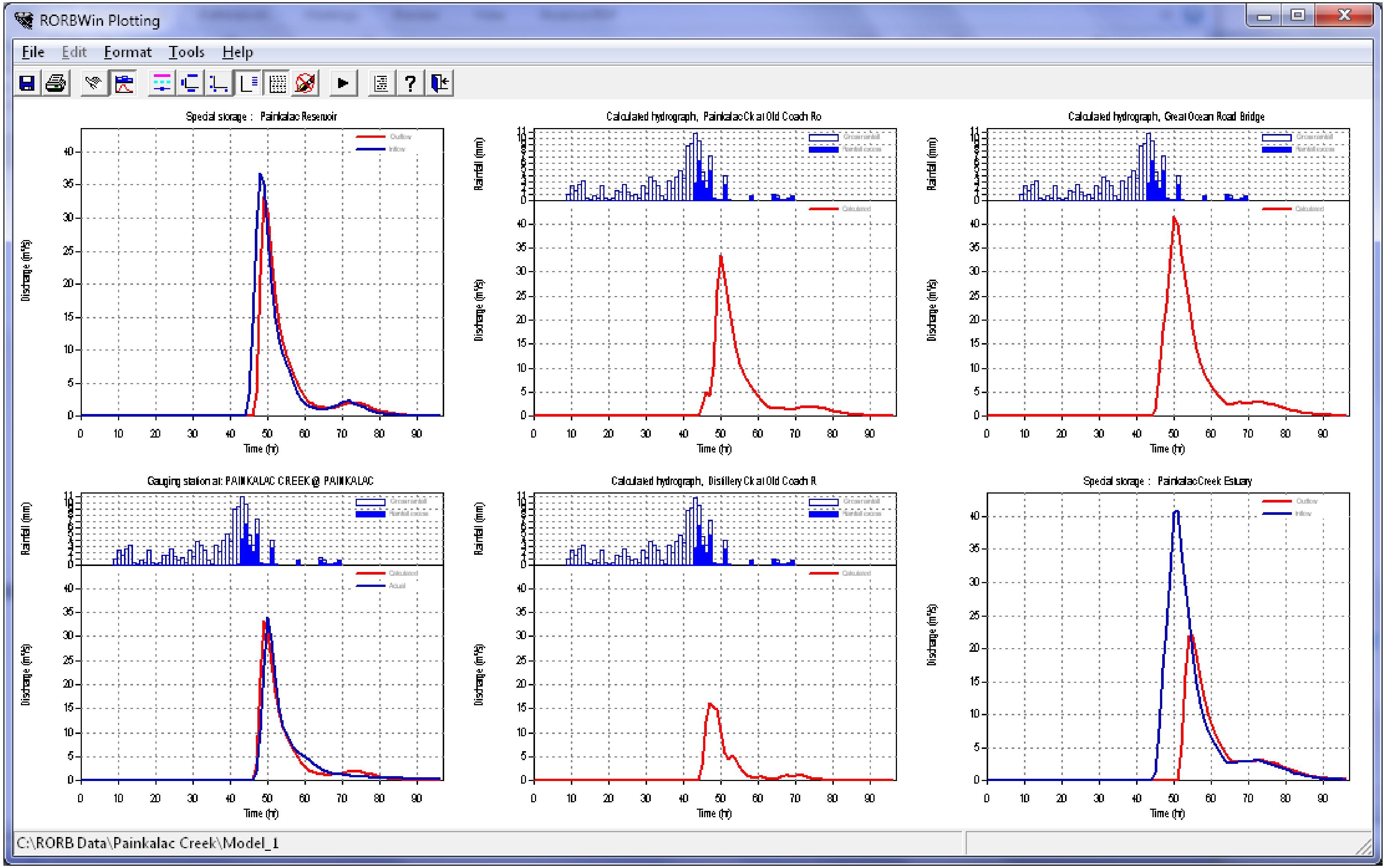
```
Routing results:
 ****************
 Painkalac Creek
 Initial time:
 21-24 April 2001 Flood (zero = 21-Apr-2001 0:00)
 FIT    run no.  1


 Parameters:  kc =     9.00    m = 0.75

 Loss parameters     Initial loss (mm)     Runoff coeff.
   Interstation area above:
   PAINKALAC CREEK @ PAI   20.00              0.57
   Catchment outlet        20.00              0.57


 *** Gauging station at: PAINKALAC CREEK @ PAINKALAC

                           Hydrograph          Error
                          Calc.   Actual    Abs. Percent
 Peak discharge,m³/s     49.23    51.80    -2.57  -5.0
 Time to peak,h           23.0     45.0    -22.0 -48.9
 Volume,m³            4.70E+06 4.71E+06-4.77E+03  -0.1
 Av.abs.ord.err,m³/s                         8.3  77.0 over dur. of calcs
 Time to centroid,h       35.4     38.7     -3.3  -8.5
 Lag (c.m. to c.m.),h     5.15     8.44    -3.29 -39.0
 Lag to peak,h            -7.2     14.8    -22.0-148.8


 *** Calculated hydrograph,  Painkalac Ck at Old Coach Ro

                           Hydrograph
                          Calc.
 Peak discharge,m³/s     52.91
 Time to peak,h           24.0
 Volume,m³            5.17E+06
 Time to centroid,h       35.9
 Lag (c.m. to c.m.),h     5.67
 Lag to peak,h           -6.24
```

*** Calculated hydrograph, Distillery Ck at Old Coach R

	Hydrograph Calc.
Peak discharge, m^3/s	24.27
Time to peak,h	22.0
Volume, m^3	2.13E+06
Time to centroid,h	33.5
Lag (c.m. to c.m.),h	3.08
Lag to peak,h	-8.45

*** Calculated hydrograph, Great Ocean Road Bridge

	Hydrograph Calc.
Peak discharge, m^3/s	74.89
Time to peak,h	24.0
Volume, m^3	7.60E+06
Time to centroid,h	36.1
Lag (c.m. to c.m.),h	5.75
Lag to peak,h	-6.32



Fit Event : Feb 2005 Flood Event

```
Routing results:
 ****************
 Painkalac Creek
 Initial time:
 3-5 Feb 2005 Flood (zero = 1-Feb-2001 9:00)
 FIT    run no.  1


 Parameters:  kc =     9.00    m = 0.75

 Loss parameters     Initial loss (mm)     Runoff coeff.
   Interstation area above:
   PAINKALAC CREEK @ PAI   80.00              0.67
   Catchment outlet        80.00              0.67

 *** Gauging station at: PAINKALAC CREEK @ PAINKALAC

                           Hydrograph          Error
                          Calc.   Actual    Abs. Percent
 Peak discharge,m³/s     33.13    33.90    -0.77  -2.3
 Time to peak,h           49.0     50.0     -1.0  -2.0
 Volume,m³             8.22E+05 8.24E+05-2.11E+03  -0.3
 Av.abs.ord.err,m³/s                        0.63  26.7 over dur. of calcs
 Time to centroid,h       54.7     55.4     -0.6  -1.2
 Lag (c.m. to c.m.),h     6.73     7.37    -0.65  -8.8
 Lag to peak,h            1.01     2.01    -1.00 -49.8



 *** Calculated hydrograph,  Painkalac Ck at Old Coach Ro

                           Hydrograph
                          Calc.
 Peak discharge,m³/s     33.39
 Time to peak,h           50.0
 Volume,m³             9.13E+05
 Time to centroid,h       55.3
 Lag (c.m. to c.m.),h     7.20
 Lag to peak,h            1.95
```

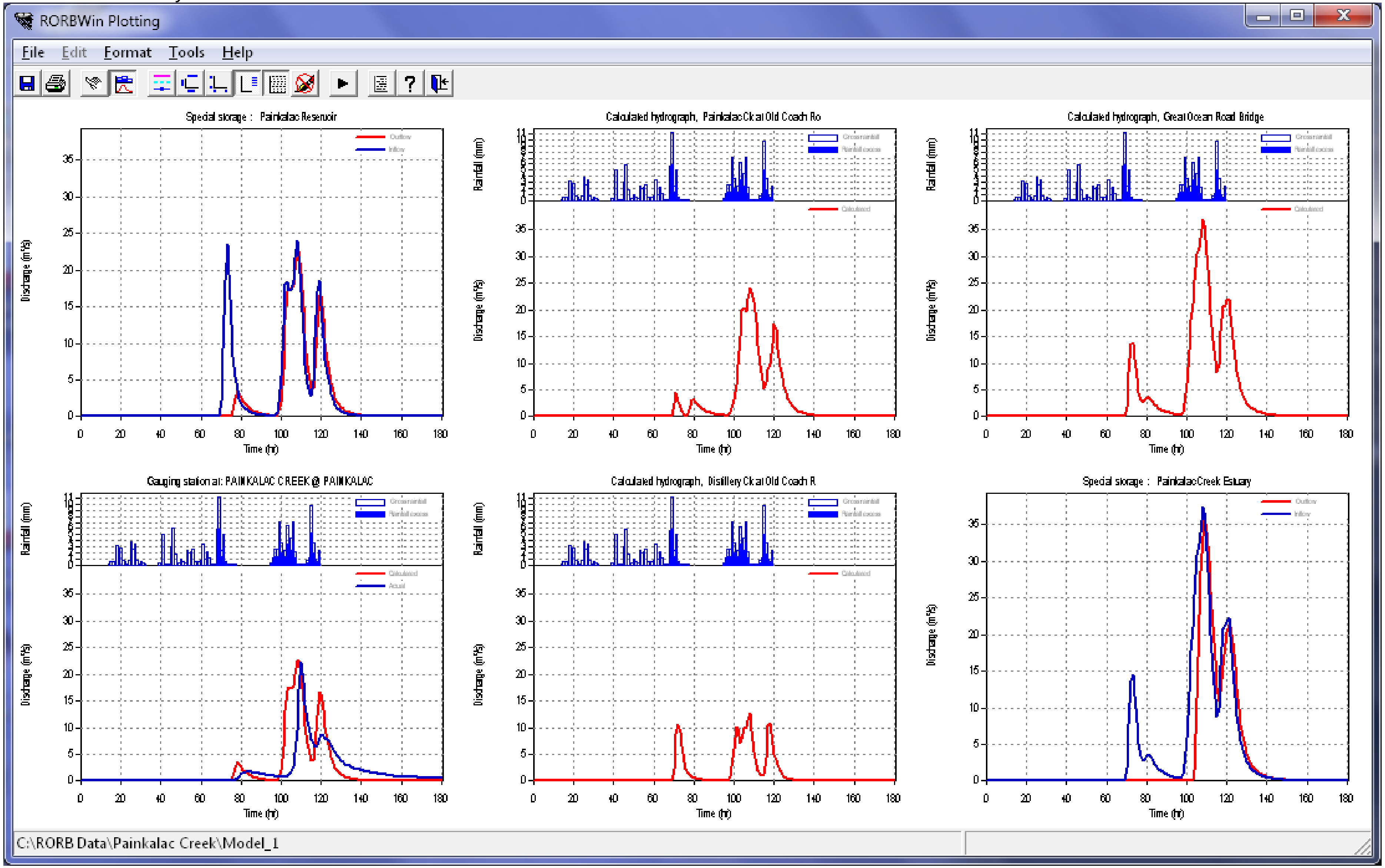
*** Calculated hydrograph, Distillery Ck at Old Coach R

	Hydrograph Calc.
Peak discharge, m^3/s	16.14
Time to peak,h	47.0
Volume, m^3	4.12E+05
Time to centroid,h	51.9
Lag (c.m. to c.m.),h	3.40
Lag to peak,h	-1.50

*** Calculated hydrograph, Great Ocean Road Bridge

	Hydrograph Calc.
Peak discharge, m^3/s	41.48
Time to peak,h	50.0
Volume, m^3	1.38E+06
Time to centroid,h	55.2
Lag (c.m. to c.m.),h	7.00
Lag to peak,h	1.79

Fit Event 4 : January 2011 Flood Event




```
Routing results:
```

```
****************
Painkalac Creek January 2011 Flood
Initial time:
9-27 January 2011 Flood (Start time 9am 9 Jan 2011)
FIT    run no.  1


Parameters:  kc =     9.00    m = 0.75

Loss parameters     Initial loss (mm)     Runoff coeff.
  Interstation area above:
  PAINKALAC CREEK @ PAI   65.00              0.53
  Catchment outlet        65.00              0.53

Results of routing through special storage Painkalac Reservoir
 Peak elevation=   30.47 m
 Peak outflow =    22.15 m³/s (spillway flow)
 Peak storage =     6.38E+05 m³


*** Special storage :   Painkalac Reservoir

                          Hydrograph
                       Outflow   Inflow
Peak discharge,m³/s     22.15    22.94
Time to peak,h           109.     108.
Volume,m³             1.28E+06 1.61E+06
Time to centroid,h       110.     102.
Lag (c.m. to c.m.),h     13.2      4.7
Lag to peak,h            11.7     10.7


*** Gauging station at: PAINKALAC CREEK @ PAINKALAC

                          Hydrograph          Error
                         Calc.   Actual    Abs. Percent
Peak discharge,m³/s     22.15    22.30    -0.15  -0.7
Time to peak,h           109.     110.      -1.  -0.9
Volume,m³             1.28E+06 1.15E+06 1.35E+05  11.8
Av.abs.ord.err,m³/s                         1.4  78.0 over dur. of calcs
Time to centroid,h       110.     119.      -9.  -7.5
Lag (c.m. to c.m.),h     13.2     22.1     -9.0 -40.6
Lag to peak,h            11.7     12.7     -1.0  -7.9


*** Calculated hydrograph,  Distillery Ck at Old Coach R

                          Hydrograph
                         Calc.
Peak discharge,m³/s     12.37
Time to peak,h           108.
```

```
Volume,m³             7.68E+05
Time to centroid,h       102.
Lag (c.m. to c.m.),h     3.36
Lag to peak,h            9.74



*** Calculated hydrograph,  Great Ocean Road Bridge

                          Hydrograph
                         Calc.
Peak discharge,m³/s     35.48
Time to peak,h           108.
Volume,m³             2.37E+06
Time to centroid,h       108.
Lag (c.m. to c.m.),h     10.3
Lag to peak,h            10.4

Calculated  drawdown= 0.647E+06 m³ : Painkalac Creek Estuary

Results of routing through special storage Painkalac Creek Estuary
 Peak elevation=    2.60 m
 Peak outflow =    34.48 m³/s (spillway flow)
 Peak storage =     7.95E+05 m³



*** Special storage :   Painkalac Creek Estuary

                          Hydrograph
                       Outflow   Inflow
Peak discharge,m³/s     34.48    35.90
Time to peak,h           109.     108.
Volume,m³             1.77E+06 2.42E+06
Time to centroid,h       116.     108.
Lag (c.m. to c.m.),h     18.6     10.4
Lag to peak,h            11.4     10.4
```

Appendix C - Statistic Analysis of peak flows at Site 235232 (Painkalac Creek Dam)

Partial Flow Series (1974-1991, 1999-2012)

32 year complete record

Record 32 years

All flows checked for independence

Rank	Date	Peak Flow (Ml/d)	Peak Flow (m3/s)
12	May-74	2892	33.5
14	Aug-74	2556	29.6
30	13-Sep-75	825	9.5
16	Oct-75	1985	23.0
4	22-Sep-76	4039	46.7
10	16-Oct-76	3119	36.1
32	20-May-78	772	8.9
7	4-Jun-78	3625	42.0
5	3-Jul-78	3731	43.2
20	Aug-78	1465	17.0
21	19-Nov-78	1465	17.0
6	17-Dec-78	3688	42.7
2	Mar-83	5229	60.5
18	May-83	1735	20.1
24	13-Sep-83	1116	12.9
15	16-Oct-83	2050	23.7
25	24-Jul-84	1051	12.2
27	20-Aug-84	898	10.4
13	19-Sep-84	2599	30.1
28	14-Sep-86	868	10.0
23	22-Oct-86	1144	13.2
8	Jan-87	3484	40.3
9	Oct-90	3445	39.9
26	Jan-91	984	11.4
31	Sep-91	781	9.0
3	Apr-01	4486	51.9
19	Aug-01	1521	17.6
29	Dec-01	843	9.8
11	Feb-05	2955	34.2

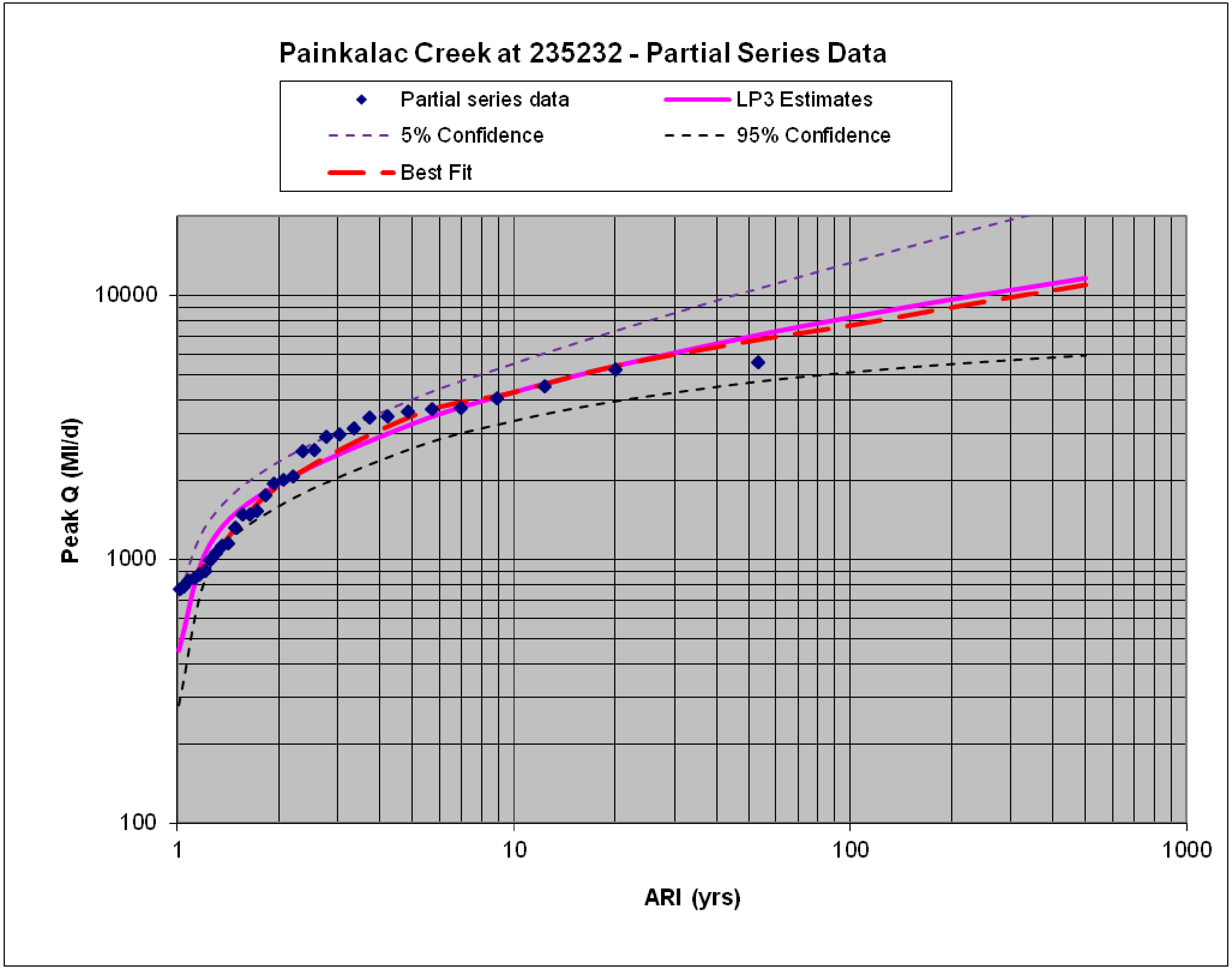
Rank	Year	Plotting Pos	Max Inst flow (Ml/d)	Log10(Q)	Max Inst flow (m3/s)
1	Nov-07	53.7	5533	3.743	64.0
2	Mar-83	20.1	5229	3.718	60.5
3	Apr-01	12.4	4486	3.652	51.9
4	Sep-76	8.9	4039	3.606	46.7
5	Jul-78	7.0	3731	3.572	43.2
6	Dec-78	5.8	3688	3.567	42.7
7	Jun-78	4.9	3625	3.559	42.0
8	Jan-87	4.2	3484	3.542	40.3
9	Oct-90	3.7	3445	3.537	39.9
10	Oct-76	3.4	3119	3.494	36.1
11	Feb-05	3.0	2955	3.471	34.2
12	May-74	2.8	2892	3.461	33.5
13	Sep-84	2.6	2599	3.415	30.1
14	Aug-74	2.4	2556	3.408	29.6
15	Oct-83	2.2	2050	3.312	23.7
16	Oct-75	2.1	1985	3.298	23.0
17	Jan-11	1.9	1925	3.284	22.3
18	May-83	1.8	1735	3.239	20.1
19	Aug-01	1.7	1521	3.182	17.6
20	Aug-78	1.6	1465	3.166	17.0
21	Nov-78	1.6	1465	3.166	17.0
22	Jun-12	1.5	1314	3.119	15.2
23	Oct-86	1.4	1144	3.058	13.2
24	Sep-83	1.4	1116	3.048	12.9
25	Jul-84	1.3	1051	3.022	12.2
26	Jan-91	1.3	984	2.993	11.4
27	Aug-84	1.2	898	2.953	10.4
28	Sep-86	1.2	868	2.939	10.0
29	Dec-01	1.1	843	2.926	9.8

1	Nov-07	5533	64.0	30	Sep-75	1.1	825	2.916	9.5
17	Jan-11	1925	22.3	31	Sep-91	1.1	781	2.893	9.0
22	Jun-12	1314	15.2	32	May-78	1.0	772	2.888	8.9

Flow Stats (Max Year Value)						Peak Flow Qy			Best Fit				Confidence Limits	
Average	2316	Ml/day	ARI	Ky	(Ml/d)	(m3/s)			(ml/d)	(m3/s)	ARI	δ	5%	95%
Medium	1955	Ml/day	1.01	-2.326	452	5	3.286	M logs	780	9	1.01	2.6363	729	280
Max	5533	Ml/day	1.25	-0.842	1141	13	0.271	S logs	1000	12	1.25	1.1698	1411	923
Min	772	Ml/day	2	0.0000	1931	22	0.01	G logs	1900	22	2	1.0801	2350	1587
Std Dev	1385		5	0.842	3268	38			3500	41	5	1.1698	4041	2642
			10	1.282	4301	50			4300	50	10	1.3748	5522	3351
			20	1.645	5396	62			5400	63	20	1.6845	7328	3974
			50	2.054	6967	81			6700	78	50	2.1988	10388	4673
			100	2.326	8257	96			7700	89	100	2.6363	13330	5115
			200	2.576	9653	112			9000	104	200	3.0983	16947	5498
			500	2.878	11657	135			11000	127	500	3.7212	22918	5930

G= 0.0

G= 0.0


Painkalac Creek at 235232 - Partial Series Data
Partial series data
LP3 Estimates
5% Confidence
95% Confidence
Best Fit
10000
1000
100
Peak Q (Ml/d)
1
10
100
1000
ARI (yrs)

Appendix D – AR&R IFD Data

The original Intensity-Frequncy-Duration (IFD) rainfall data is described in Australia Rainfall & Runoff (AR&R) volume one by the Institute of Engineers Australia. For a particular location in Australia a computation method is required to determine a design rainfall estimate using 9 parameters read from maps in Volume 2 of AR&R.

For the following stations the 9 parameters read from the maps are shown in the table below. Iron Bark Spur Track was the reference site for the multiple RORB design runs.

IFD Parameter	Aireys Inlet	Boonah	Anglesea (VPT)	Iron Bark Spur Tk
2yr – 1hr	19.12	20.0	18.86	19.69
2yr – 12 hr	3.82	4.29	3.94	4.03
2yr – 72 hr	1.12	1.25	1.19	1.25
50yr – 1 hr	34.93	35.0	34.99	35.00
50yr – 12 hr	6.19	7.49	6.50	7.08
50yr – 72 hr	1.81	2.00	1.89	2.00
Skew	0.46	0.47	0.45	0.47
F2	4.28	4.28	4.28	4.28
F50	14.76	14.75	14.77	14.75

Resulting point rainfall totals in mm's for Aireys Inlet, Boonah, Anglesea (VPT) and Iron Bark Spur Track are shown below for 100, 50, 20, and 10 yr ARI design events.

Frequency-Duration (ARI yrs – hours)	Aireys Inlet	Boonah	Anglesea (VPT)	Iron Bark Spur Tk
100 yr – 12 hr	89	110	94	104
100 yr – 24 hr	113	135	119	130
100 yr – 48 hr	141	160	147	158
50 yr – 12 hr	80	97	84	92
50 yr – 24 hr	102	120	107	116
50 yr – 48 hr	127	143	132	141
20 yr – 12 hr	68	82	71	77
20 yr – 24 hr	87	101	91	98
20 yr – 48 hr	108	122	113	121
10 yr – 12 hr	60	71	62	67
10 yr – 24 hr	76	88	80	85
10 yr – 48 hr	95	107	100	106

The new IFD design rainfall data is available from the Bureau of Meteorology web site at www.bom.gov.au\water\.

For each location the latitude and longitude coordinates are entered and the IFD data is displayed as shown on the following page.

Location

Label: Anglesea (VPG)

Latitude: -38.3686 [Nearest grid cell: 38.3625 (S)]

Longitude: 144.0842 [Nearest grid cell: 144.0875 (E)]

Table | Chart

IFD Design Rainfall Depth (mm)

Issued: 22 July 2013

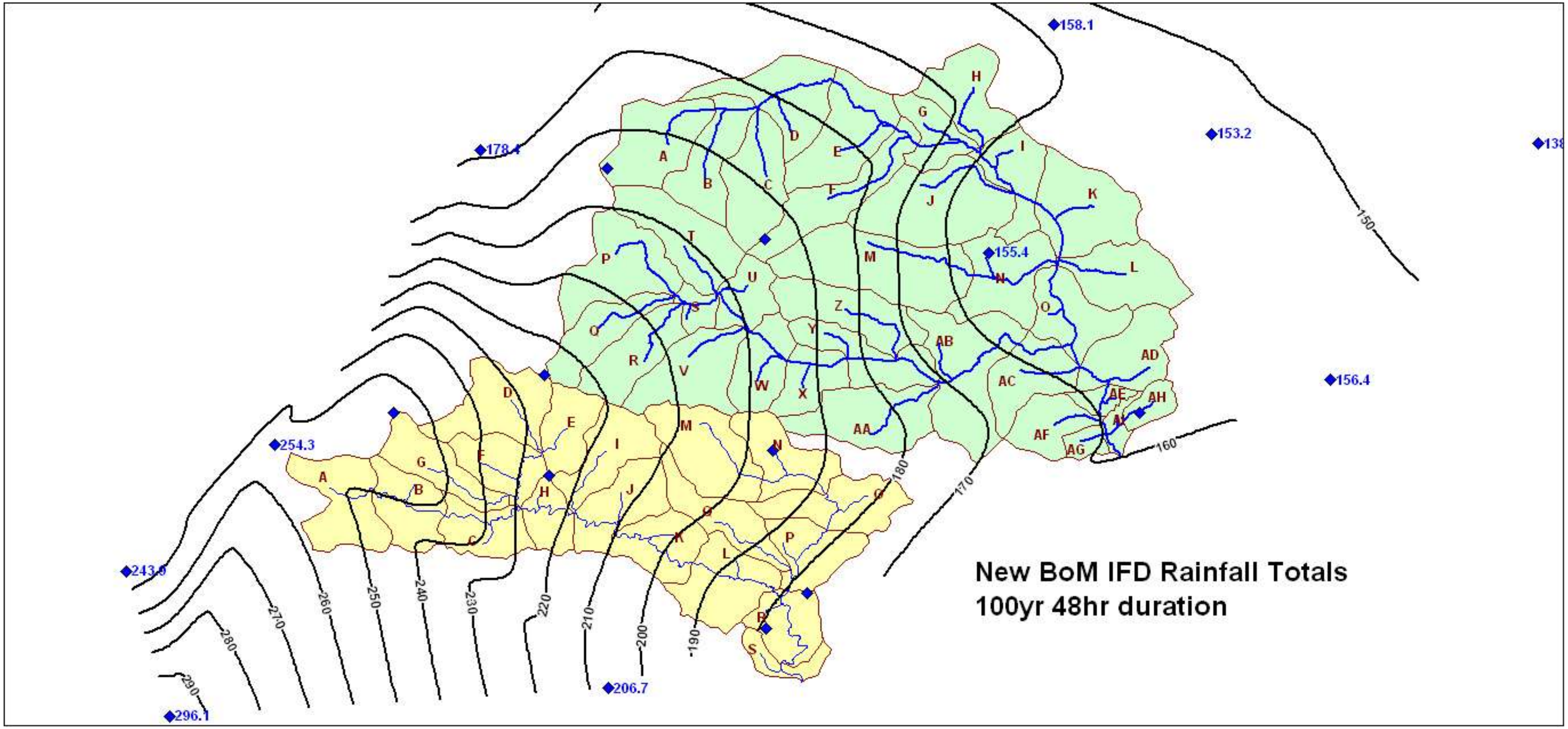
Rainfall depth for Durations, Exceedance per Year (EY), and Annual Exceedance Probabilities (AEP).

	EY	Annual Exceedance Probability (AEP)					
Duration	**1EY**	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
1 min	1.2	1.4	1.9	2.3	2.7	3.3	3.7
2 min	1.9	2.2	3.0	3.6	4.2	5.0	5.7
3 min	2.6	3.0	4.1	4.9	5.7	6.9	7.8
4 min	3.3	3.7	5.1	6.1	7.1	8.6	9.8
5 min	3.8	4.3	5.9	7.1	8.4	10.1	11.5
10 min	5.7	6.4	8.9	10.8	12.7	15.6	17.9
15 min	6.9	7.8	10.8	13.1	15.5	18.9	21.8
30 min	9.0	10.2	14.2	17.1	20.2	24.6	28.3
1 hour	11.6	13.1	18.0	21.6	25.3	30.6	35.0
2 hour	15.2	17.1	23.2	27.7	32.3	38.7	43.9
3 hour	18.1	20.3	27.5	32.7	38.0	45.4	51.3
6 hour	25.0	28.1	38.1	45.1	52.3	62.3	70.3
12 hour	34.7	39.2	53.7	64.0	74.5	89.2	100.9
24 hour	46.5	52.9	74.0	89.2	104.9	126.8	144.6
48 hour	58.1	66.4	94.7	115.6	137.6	168.4	193.8
72 hour	63.9	73.1	104.4	127.9	152.8	188.1	217.5
96 hour	67.8	77.3	109.7	134.1	160.0	197.7	229.1
120 hour	71.0	80.6	113.1	137.5	163.3	202.0	234.4
144 hour	74.0	83.5	115.6	139.4	164.4	203.5	236.3
168 hour	77.1	86.5	117.8	140.7	164.5	203.5	236.4

Appendix E – Subarea Storms and Design Rainfall Totals

Subarea	Area (km2)	F imp	F_Forested (%)	April 2001 4 day Rain Total	April 2001 Subarea Wts	Feb 2005 3 day Rain Total	Feb 2005 Subarea Wts	Nov 2007 2 day rain Total	Nov 2007 Subarea Wts	Average subarea rainfall wts
A	4.29	0	90	274	1.16	129	1.10	141	1.27	1.18
B	3.14	0	100	262	1.11	126	1.07	135	1.21	1.13
C	2.71	0	100	252	1.07	121	1.03	125	1.12	1.07
D	4.46	0	100	255	1.08	119	1.01	125	1.12	1.07
E	2.56	0	100	246	1.04	118	1.01	118	1.06	1.04
F	2.91	0	100	250	1.06	118	1.01	123	1.11	1.06
G	3.12	0	85	258	1.09	121	1.03	131	1.18	1.10
H	1.90	0	100	241	1.02	118	1.01	116	1.04	1.02
I	3.45	0	100	240	1.02	117	1.00	112	1.01	1.01
J	3.50	0	100	235	1.00	116	0.99	108	0.97	0.99
K	3.06	0	100	228	0.97	114	0.97	102	0.92	0.95
L	4.18	0	80	216	0.92	113	0.96	97	0.87	0.92
M	4.76	0	100	230	0.97	117	1.00	102	0.92	0.96
N	3.37	0	100	222	0.94	115	0.98	98	0.88	0.93
O	3.76	0	100	209	0.89	112	0.95	94	0.84	0.89
P	3.04	0	100	209	0.89	112	0.95	93	0.84	0.89
Q	3.10	0	95	222	0.94	114	0.97	99	0.89	0.93
R	2.82	0	70	208	0.88	111	0.95	92	0.83	0.88
S	1.20	0	70	205	0.87	111	0.95	91	0.82	0.88
Total/Aver	61.3	0	95.0	236		117		111		1.00

		Old AR&R 1987 IFD data					New BoM IFD Rainfall Pt Depths (mm's)				New BoM IFD Areal Rainfall Depths			
Subarea	Area (km2)	IFD 100yr 9hr Rainfall		IFD 20yr 9hr Rainfall		IFD Rainfall wts	100yr 12hr	100yr 24hr	100yr 36hr	100yr 48hr	100yr 12hr	100yr 24hr	100yr 36hr	100yr 48hr
		(mms)	(wts)	(mms)	(wts)	(wts)					0.90	0.92	0.94	0.95
A	**4.29**	**97**	**1.16**	**73.5**	**1.16**	**1.08**	**136**	**193**	**228**	**252**	**122**	**178**	**214**	**239**
B	3.14	96	1.15	72	1.14	1.07	131	190	224	250	118	175	211	238
C	2.71	93	1.11	70	1.11	1.04	126	180	214	239	113	166	201	227
D	4.46	94	1.12	71	1.12	1.05	123	177	209	233	111	163	196	221
E	2.56	92	1.10	69	1.09	1.03	118	168	201	224	106	155	189	213
F	2.91	93.5	1.12	70	1.11	1.04	125	178	219	245	113	164	206	233
G	3.12	95.5	1.14	72	1.14	1.06	132	191	224	250	119	176	211	238
H	1.90	91.5	1.09	68.5	1.08	1.02	117	168	199	222	105	155	187	211
I	3.45	90	1.08	67.5	1.07	1.00	114	164	196	218	103	151	184	207
J	3.50	89	1.06	67	1.06	0.99	111	159	190	212	100	146	179	201
K	3.06	87	1.04	65.5	1.04	0.97	105	151	180	201	95	139	169	191
L	4.18	84.5	1.01	64	1.01	0.94	98	140	169	188	88	129	159	179
M	4.76	86.5	1.03	65	1.03	0.96	107	153	183	204	96	141	172	194
N	3.37	84.5	1.01	64	1.01	0.94	102	146	175	195	92	134	165	185
O	3.76	84	1.00	63	1.00	0.94	95	136	163	182	86	125	153	173
P	3.04	84	1.00	63	1.00	0.94	95	135	162	181	86	124	152	172
Q	3.10	85	1.02	64.5	1.02	0.95	102	148	176	196	92	136	165	186
R	2.82	84	1.00	63	1.00	0.94	93	133	160	178	84	122	150	169
S	1.20	84	1.00	63	1.00	0.94	92	133	160	178	83	122	150	169
Total/Aver	61.3	89.3	83.6	67.3	63.2	89.7	112	161	192	214	101	148	180	203


New BoM IFD Rainfall Totals
100yr 48hr duration
158.1
153.2
178.4
155.4
156.4
254.3
243.9
206.7
296.1
150
160
170
180
190
200
210
220
230
240
250
260
270
280
290

Appendix F – Design Discharges (RORB)

Painkalac Creek Flood Study

RORB Model - Design Flow results

ARI	Storm Duration	Average Catchment Rain	Losses		Peak Flow with Kc=9 m=0.75 at: Inflow to Reservoir	Outflow (Site 235232)	Old Coach Road	Outflow Distillery Ck	Great Ocean Road
(yrs)	(hrs)	(mm)	IL (mm)	RC	(m3/s)	(m3/s)	(m3/s)	(m3/s)	(m3/s)
100	12	101	41	0.62	76	74	76	34	102
	24	148	41	0.62	96	89	89	39	114
	36	180	41	0.62	81	81	87	38	124
	48	203	41	0.62	88	82	82	38	121
	72								
50	12	89	39	0.65	68	66	68	30	88
	24	129	39	0.65	81	79	79	34	99
	36	155	39	0.65	74	73	80	35	113
	48	177	39	0.65	79	75	75	35	112
	72								
20	12	74	39	0.65	53	51	51	22	62
	24	108	39	0.65	63	56	60	24	74
	36	129	39	0.65	62	61	65	27	89
	48	144	39	0.65	66	62	63	29	92
	72								

10	12	63	39	0.65	39	36	35	14	40
	24	91	39	0.65	41	39	42	14	55
	36	108	39	0.65	53	49	49	20	68
	48	120	39	0.65	56	50	52	25	73
	72								
5	12	53	38	0.67	28	24	22	7.4	24
	24	75	38	0.67	27	26	29	12	41
	36	89	38	0.67	38	35	38	15	49
	48	98	38	0.67	42	40	42	18	54
	72								
2	12	38	34	0.74	9.5	6.9	6.2	0.3	5.4
	24	53	34	0.74	18	17	18	7.5	24
	36	62	34	0.74	23	20	22	8.3	28
	48	68	34	0.74	26	22	24	8.1	29
	72								
1	12	33	43	0.58	0	0	0	0	0
	24	46	43	0.58	5.5	3.9	3.6	0.2	3.2
	36	54	43	0.58	9.7	8.2	8.8	2.3	9.9
	48	60	43	0.58	9.8	9.1	9.3	3.2	11
	72								

Appendix G – HEC-RAS output

Profile Output Table - Standard Table 1

File Options Std. Tables User Tables Locations Help

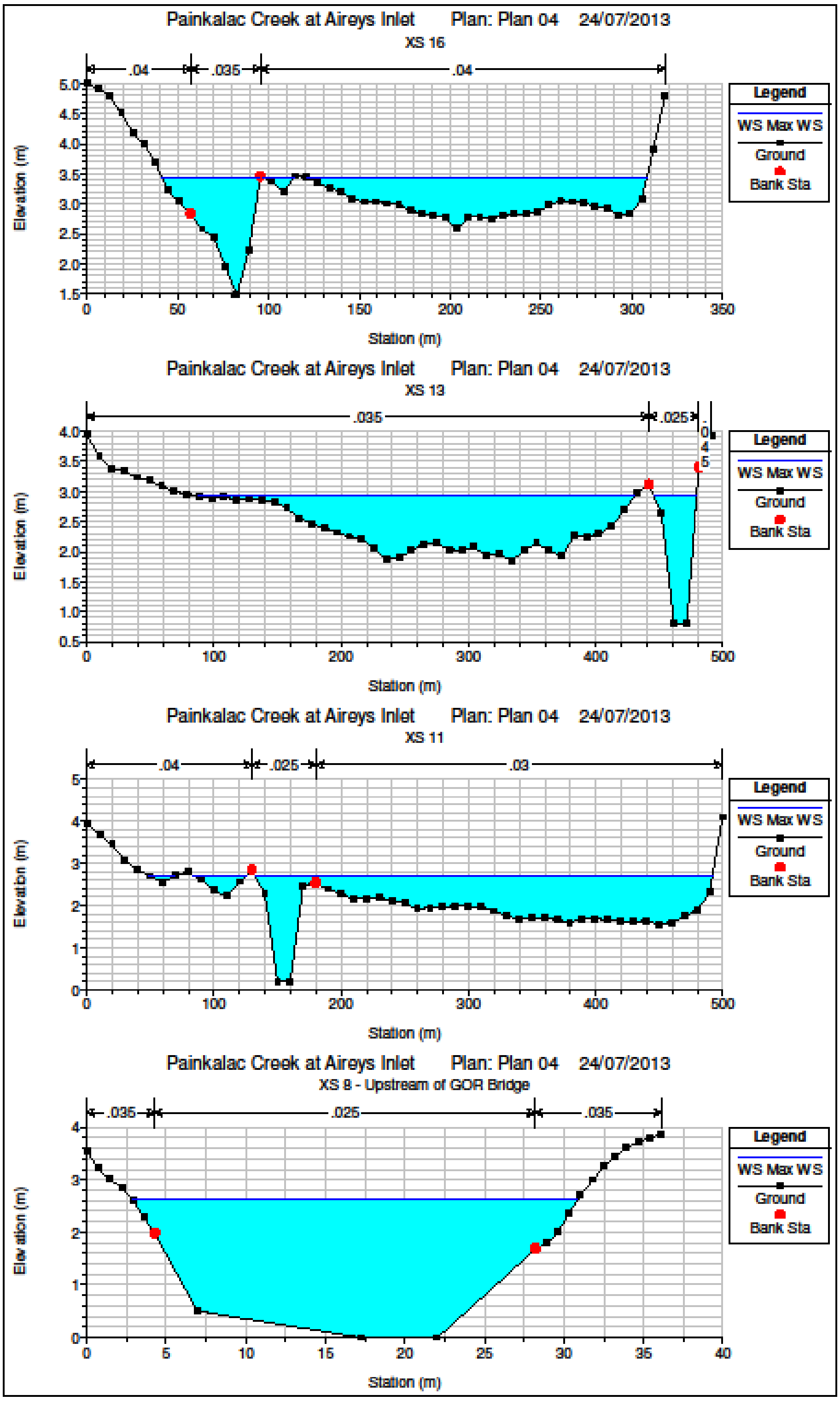
HEC-RAS Plan: Plan 04 River: Painkalac Creek Reach: Lower Profile: Max WS

Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Lower	4087	Max WS	124.30	1.85	4.68		4.85	0.007290	2.25	82.79	165.76	0.68
Lower	4050	Max WS	124.25	1.80	4.48		4.61	0.005282	2.19	91.05	148.09	0.59
Lower	3963	Max WS	124.18	1.75	4.11		4.23	0.004416	2.20	104.10	177.25	0.55
Lower	3815	Max WS	124.09	1.69	3.79		3.82	0.002535	1.06	157.32	287.46	0.38
Lower	3582	Max WS	123.82	1.50	3.44		3.50	0.001936	1.36	141.21	257.74	0.41
Lower	3395	Max WS	123.62	1.35	3.29		3.31	0.000410	0.74	235.54	243.54	0.22
Lower	3133	Max WS	123.21	1.10	3.10		3.13	0.001023	1.09	178.06	302.34	0.34
Lower	2844	Max WS	122.49	0.80	2.95		2.96	0.000366	0.88	256.67	381.03	0.25
Lower	2346	Max WS	121.00	0.50	2.82		2.84	0.000308	0.95	274.13	561.19	0.24
Lower	1695	Max WS	107.28	0.20	2.75		2.75	0.000097	0.45	341.09	428.67	0.13
Lower	1438	Max WS	104.52	0.10	2.72		2.73	0.000095	0.54	273.46	274.96	0.13
Lower	1160	Max WS	101.75	0.00	2.70		2.71	0.000065	0.50	290.37	224.61	0.11
Lower	1003	Max WS	96.83	0.00	2.67		2.70	0.000110	0.75	151.86	95.86	0.15
Lower	920	Max WS	68.28	0.00	2.57		2.65	0.000498	1.29	52.75	29.35	0.31
Lower	907	Max WS	65.81	0.00	2.55	1.32	2.64	0.000416	1.31	51.53	27.66	0.29
Lower	900		Bridge									
Lower	892	Max WS	66.29	0.00	2.55		2.63	0.000355	1.25	54.25	28.00	0.27
Lower	839	Max WS	72.86	0.00	2.59		2.60	0.000030	0.39	231.68	166.22	0.08
Lower	707	Max WS	72.83	0.00	2.59		2.59	0.000002	0.10	834.34	529.78	0.02
Lower	558	Max WS	71.48	0.00	2.58		2.59	0.000019	0.31	317.57	231.04	0.06
Lower	241	Max WS	66.44	0.00	2.57		2.58	0.000030	0.37	191.13	103.28	0.08
Lower	104	Max WS	61.34	0.00	2.56		2.57	0.000035	0.53	127.61	72.16	0.11
Lower	53	Max WS	59.88	0.00	2.55	0.60	2.57	0.000041	0.55	109.86	54.52	0.11
Lower	47		Inl Struct									
Lower	40	Max WS	-4.57	0.00	1.28		1.28	0.000002	-0.09	53.32	42.74	0.02
Lower	0	Max WS	-4.77	-0.50	1.28	-0.45	1.28	0.000000	-0.02	261.96	150.00	0.00

Total flow in cross section.

HEC-RAS100 year ARI Profile Output Table



HEC-RAS Cross Section plots for 100yr ARI event

The location of the HEC-RAS cross sections are shown as red lines with their chainage in metres from the ocean.

Appendix H – Berm Breach Characteristics

The breach channel is assumed to form and hold the channel shape specified in the dam break setup for the duration of the flood event. In reality the breach channel will continue to enlarge beyond that is specified for the dam break. However it is considered that the dam break mechanism is a reasonable assumption to estimate peak levels on the Painkalac Creek Estuary as they occur during the time the breach channel is being formed.

The breach characteristics are:

Berm starting height :	2.1 m AHD
Breach channel base :	25 m
Breach channel invert :	0.6 m AHD
Time to fully form channel breach :	5 hours
Breaching mechanism :	Overtopping
Water level to start breach :	2.15 m AHD
Breach progression :	Half sine curve
Volume of sand removed by breach :	2200 m3
Average flow velocity during breach development :	1.5 m/sec

The berm erosion rates and the associated Yang Sediment Transport concentrations in the development of the breach channel are shown in the graphs below. It is expected that the breach channel to the ocean would initially be steeper than 1/100 and flatten out as the channel is fully formed to 1/200. The breach channel length to the sea at Painkalac is expected to be half the distance than Anglesea Estuary, therefore steeper slopes. The assumed breach mechanism concentrations required is shown as the blue line on the two graphs on the next page (a 2 and 3 hour breach time). The Yang sediment transport concentration potential is shown as dashed lines for two breach channel slopes. The 3 hour breach time is best suited for the Painkalac Creek estuary with Yang concentrations exceeding the model requirement.

The maximum erosion rate is 25 m3/min (3700 ppm) with associated stream power of 120 N/m sec. The stream power increases significantly to more than 400 N/m sec when the flow is contained within the breach channel. This occurs after the breach channel has formed.

The procedure outlined in HEC-RAS Hydraulic Reference Manual for Yang sediment transport functions (Sample Calculations) has been used to determine a suitable breaching mechanism.

Key parameters used in the Yang Sediment Transport procedure:

Water temperature : 55^{o} F (12.8^{o} C)

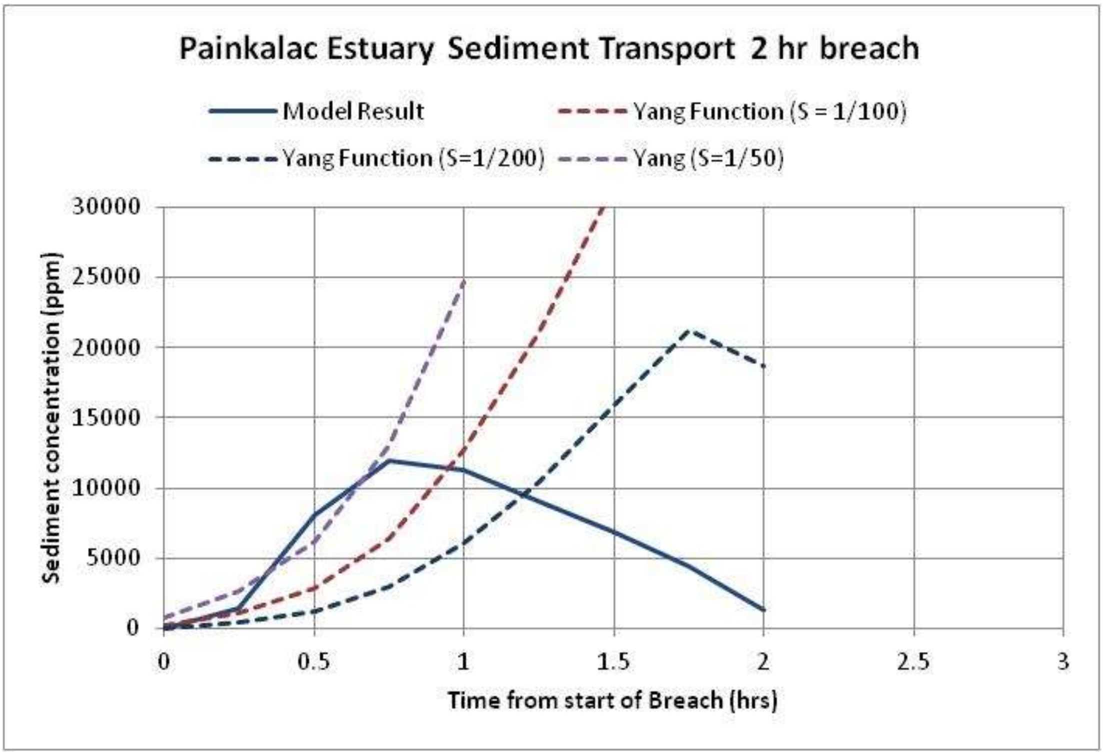
Median Particle Dia : 0.354 mm (Medium Sand 0.25 - 0.5 mm)

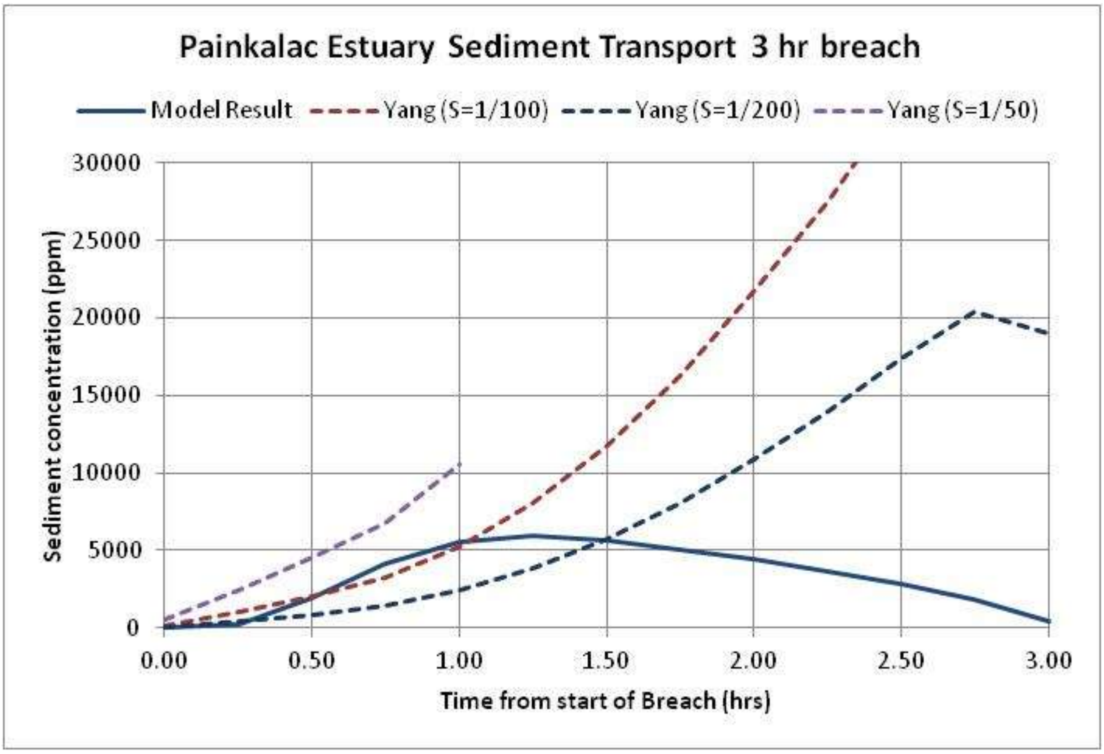
Specific g sediments : 2.65

The Painkalac Creek has more energy (head and flow) available to form a breach channel than Painkalac Creek. A five hour breach time with a 20 metre base channel has been used for Anglesea Estuary berm.

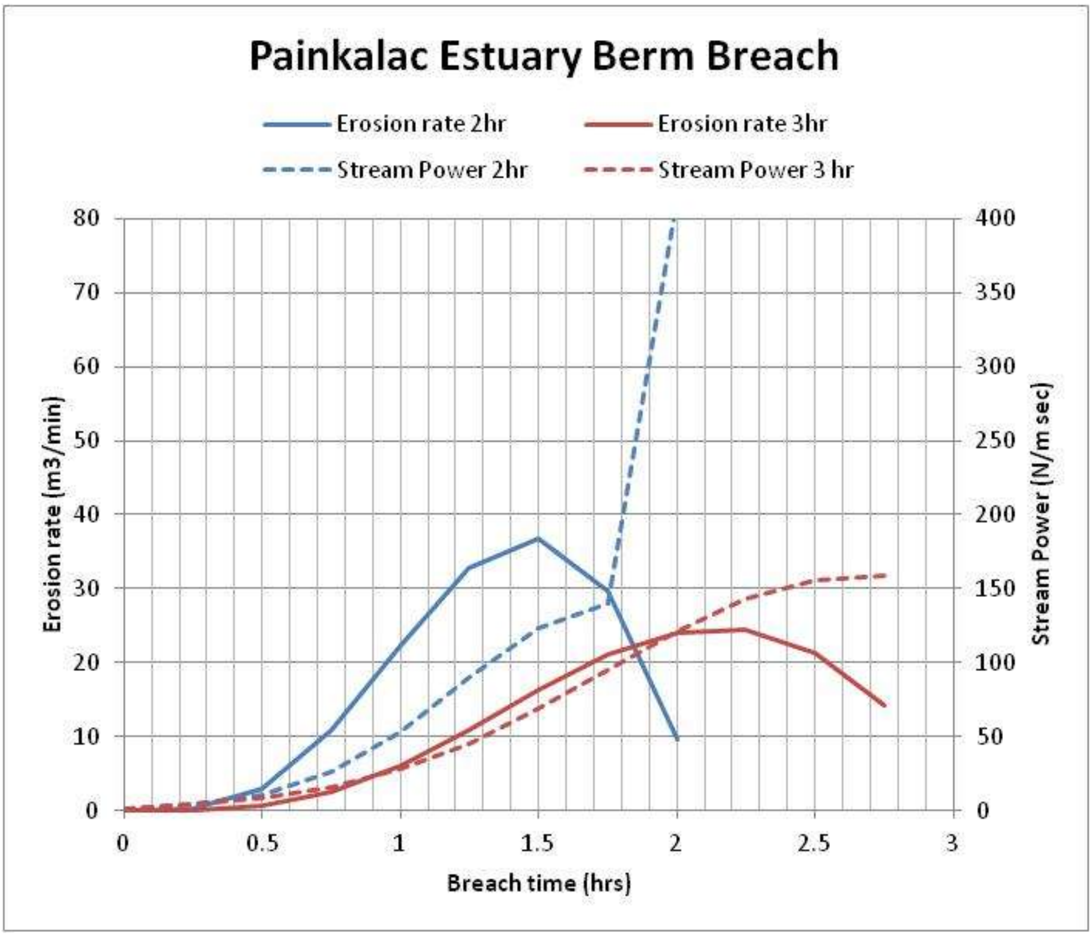
Recently a few papers have been written that discuss the highly dynamic nature of berm breakouts and the challenges in defining appropriate initial conditions and breach channel diminsions. Most of the examples are along the NSW coast and East Gippsland with ICOLL (intermittently closed and open lakes and lagoons) that are much larger systems than Painkalac Estuary. A paper by D Lyons and D Williams of BMT WBM in 2012 is a good example [Ref 8].

More data is required to compare these rates with other events and estuaries to confirm the assumptions made for the breaching mechanism.





A comparison of erosion rate of the breach channel with stream power is shown on the graphs below. The peak erosion rate of 25 m3/min with a stream power of 120 N/m sec occurs about 2 hours after the breach begins for Painkalac Estuary.


Painkalac Estuary Berm Breach
Erosion rate 2hr
Erosion rate 3hr
Stream Power 2hr
Stream Power 3 hr
Erosion rate (m3/min)
Stream Power (N/m sec)
Breach time (hrs)
0
10
20
30
40
50
60
70
80
0
50
100
150
200
250
300
350
400
0
0.5
1
1.5
2
2.5
3


Anglesea Estuary Berm Breach Characteristics
Anglesea Erosion rate (3hr)
Anglesea Erosion 5hr
Anglesea Stream Power (3hr)
Anglesea Stream Power 5hr
Eroded Volume rate (m3/min)
Stream Power (N/m sec)
Time (hrs)
0
5
10
15
20
25
30
35
40
45
50
0
10
20
30
40
50
60
70
80
90
100
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00