SITE SPECIFIC FLOOD STUDY
PROPOSED SHAOLIN TEMPLE PROJECT
COMBERTON GRANGE
SOUTH NOWRA

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REFERENCES

Lyall & Associates Consulting Engineers (2006) – *Currambene Creek and Moona Moona Creek Flood Studies Rev 2.0* – Prepared for Shoalhaven City Council – 10 November 2006

The Institution of Engineers, Australia (1987) – *Australian Rainfall & Runoff – A Guide to Flood Estimation*

NSW Department of Infrastructure, Planning and Natural Resources (2005) – *Floodplain Development Manual – the management of flood liable land*

Commonwealth Bureau of Meteorology (2003) – *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* – June 2003

E.M. Laurenson, R.G. Mein and R.J. Nathan (2010) – Monash University Department of Civil Engineering and Sinclair Knight Merz – *RORB Version 6 Runoff Routing Program User Manual* –– January 2010

US Army Corps of Engineers Hydrologic Engineering Center (2001) – *HEC-RAS River Analysis System User's Manual* Version 4.1 – January 2010

US Army Corps of Engineers Hydrologic Engineering Center (2001) – *HEC-RAS River Analysis Hydraulic Reference Manual* Version 4.1 – January 2010

1 INTRODUCTION

Shaolin Temple Foundation (Australia) proposes to develop a site at Comberton Grange, South Nowra for tourist and residential purposes. A master plan has been developed by Conybeare Morrison architects, and an application was made to the Director-General of the Department of Planning for the requirements to be addressed in an Environmental Assessment of the proposed development.

The Director-General's Requirement (DGR) Number 8 is entitled Hazard Management and includes the following:

8.5 Provide a site specific flood study in accordance with Shoalhaven City Council's Flood Risk Management Policy and DCP No. 106 – *Floodplain Management*. Reference Draft *Currambene Creek Floodplain Risk Management Study and Plan*.

Identify the 10 year ARI, 100 year ARI and Probable Maximum Flood (PMF) extent associated with Currambene Creek and Georges Creek. Include identification of floodways, flood storage and flood fringe areas, determine high and low hazard areas as defined by NSW Floodplain Development Manual 2005. Reference flood levels outlined in Council's Currambene Creek and Moona Moona Creek Flood Studies (Council to make this study available).

8.6 Assess the potential impacts of sea level rise and increase in rainfall intensity on the flood regime of the site and adjacent lands with consideration of *Practical Consideration of Climate Change – Floodplain Risk Management Guideline* (DECC Oct 2007).

On behalf of Shaolin Temple Foundation (Australia), Conybeare Morrison engaged Brown Consulting (NSW) to investigate and prepare a report addressing the above DGRs.

It is important to note that this flood study is for planning purposes only and is not intended to set the Flood Planning Levels for the Georges Creek catchment or to define precisely the flood extents. This investigation and report are intended to form the basis for more detailed investigation and preliminary design of civil works to be carried out after the proposed master plan has been approved. This will require actual field survey to provide more accurate creek profiles and cross-sections. However, this investigation is considered to be sufficiently accurate for the purposes of master planning.

2 SITE DESCRIPTION

The proposed Shaolin Tourist and Residential Development is located at Comberton Grange, approximately 12km south of Nowra and 2km east of the Princes Highway, within the boundaries of Shoalhaven City Council.

The site comprises approximately 1,284 hectares and occupies seven separate parcels of land. The site is bounded on the south by Currambene Creek and is traversed by Georges Creek and its tributaries. Currambene Creek discharges to Jervis Bay at Callala Beach. Most of the site drains towards these creeks, although a small portion on the eastern side drains towards the upper reaches of Bid Bid Creek.

The site contains a former pine plantation of approximately 170 hectares and a further 110 hectares (approximately) has been cleared on ridges overlooking Currambene Creek. About 75% of the site is covered by forest, woodland and wetlands.

The landform can be generally divided between riparian and forest zones. Creek slopes at the lower reaches of Georges Creek are generally mild, with broad overbank areas and bed slopes of less than 0.1%. In the upper reaches and in the tributaries, the bed slopes increase and may be classified as steep mountain streams, with narrower overbank areas and bed slopes over 0.4% and up to 1.5% in the study area.

3 SURVEY AND ENGINEERING INFORMATION

This investigation draws heavily on the Currambene Creek and Moona Moona Creek Flood Sudies carried out by Lyall and Associates, Consulting Engineers for Shoalhaven City Council in 2006. The investigation of Currambene Creek included the Georges Creek catchment but did not extend up that creek or its tributaries. Their report recommended further investigation of Georges Creek. This investigation therefore is an extension of the previous flood study, which will be referred to as the Currambene flood study in this report.

The Currambene flood report was obtained from Shoalhaven City Council's web site. Council subsequently provided copies of the computer files on which the report was based. Council also supplied contours which they had extracted from their Geographical Information System (GIS), which covered most of the study area. This contour information was used to generate creek longitudinal and cross-sections, and sub-catchment boundaries.

Parts of the catchment of Georges Creek extend beyond the bounds of the GIS data held by Council. Fortunately, this did not affect the creek modelling, as the areas not able to be modelled were in the upper reaches which were well outside the study area. Catchment boundaries for these areas were derived from the 1:25,000 topographic map 90283S Nowra published by the Department of Lands

4 FLOOD INVESTIGATION BACKGROUND

4.1 METHODOLOGY

This flood investigation has been carried out in accordance with the guidelines in Australian Rainfall and Runoff, commonly abbreviated to AR&R, published by the Institution of Engineers Australia [1987]. Based on the size of the catchment and the nature of the investigation, peak flood discharges have been determined using the runoff routing program RORB developed by Laurenson, Mein and Nathan [2010] of Monash University and Sinclair Knight Merz Pty Ltd. The current version is 6.15. Flood levels were calculated using the computer program HEC-RAS, the River Analysis System produced by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers [2010]. The current version is 4.1.0

The RORB program estimates peak discharges based on the catchment area and an assessment of the catchment characteristics (principally the area and length of the creek reaches). The modelling treats the catchment as a whole, and apportions flows between the sub-catchments based on area and reach characteristics of each sub-catchment. The peak flows are calculated using a formula whose coefficients are adjusted until the peak discharges closely approximate known flood events. The coefficients derived from calibration for the known flood event are then used to derive peak discharges for other flood scenarios. Where previous flooding data is unavailable, coefficients are assumed from other flood studies on catchments of similar size and topography.

The HEC-RAS program calculates the water levels at selected points along the creek, where cross-sections have been measured. The program calculates flow areas for given flow rates, slopes and surface roughness. It then solves the equations of flow between successive cross-sections, progressing upstream from a known water level. The output from the program includes channel velocity to assist in assessment of flood hazard.

HEC-RAS is a one-dimensional modelling program, in that it can only consider flow along a linear channel. It cannot determine flows at an angle to the main creek alignment. For such detailed analysis, a two-dimensional modelling program is required. However, this more detailed modelling is not necessary for this investigation because flows are predominantly 1-D and confined to the creek channels.

4.2 FLOOD RISK TERMINOLOGY

Throughout this report, several terms are used to describe flood risks.

Rainfall intensities have been measured and collated by the Bureau of Meteorology over many years in order to determine the statistical relationship between rainfall of a particular intensity and the frequency of its occurrence. Based on this statistical data, the relationship between storm duration, rainfall intensity and probability of occurrence has been determined for the whole of Australia. This data was initially published as a series of maps and charts in AR&R [1987], and is now available on their web site.

The probability that a particular intensity might be exceeded in a storm in any one year is denoted as its *Annual Exceedance Probability* (AEP). Thus an intensity which has an AEP of 1% has a probability of 0.01 of being exceeded in any one year. This may also be considered as the intensity that might be exceeded on average once every 100 years (the inverse of 0.01). This intensity can thus also be termed as the 100-year *Average Recurrence Interval* (ARI) intensity, and the greatest rate of runoff generated from this rainfall would be termed the *Q100 peak runoff*.

The absolute worst case flood risk does not rely on extrapolation of rainfall records, but on the physical capacity to generate rainfall based on climatic considerations. The *Probable Maximum Precipitation* (PMP) is defined by the Bureau of Meteorology as the greatest depth of rainfall that is physically possible according to meteorological constraints for a given duration for a given size storm area at a particular location at a particular time of year, with no allowance for long-term climatic trends. The most extreme flood generated by any storm duration at a particular site is the flood generated by the PMP and is called the *Probable Maximum Flood* (PMF). The PMF is commonly considered to be approximately 10,000 years ARI.

Flows are measured in cubic metres per second, which is commonly abbreviated to *cumecs*.

4.3 CATCHMENT AREAS

The Currambene flood study identified catchments draining to Currambene Creek and Georges Creek. The catchments are shown on Figure 2.1 which has been reproduced in Appendix A. In that drawing, the catchments for Georges Creek are identified as S, T and U. The catchment boundaries had been determined by reference to the Nowra topographic map.

The catchments for Georges Creek and its tributaries have been further subdivided and the catchment boundaries adjusted in accordance with the more accurate contour information. The sub-catchment areas are summarised in Table 1.

Sub-catchment Number	Area (sq.km.)	Sub-catchment Number	Area (sq.km.)
S1	2.50	T1	2.08
S2	2.89	T2	1.60
S3	1.23	T3	1.34
S4	1.99	U1	2.23
		U2	3.11

Table 1. Catchments

The catchment plan C0-01 showing those sub-catchments is included in Appendix A. The total area of the catchment was found to be slightly lower than measured in the Currambene flood study, so an additional area was added to the lowermost sub-catchment (U2) so that the modelling would be consistent with the previous study. This is included in the table above, making the total catchment area for Georges Creek 18.97 sq.km. or 1,897 hectares.

4.4 RAINFALL INTENSITIES

Rainfall Intensity – Frequency – Duration data have been obtained from the Bureau of Meteorology for a location centred on the site at Latitude 34 º 58.5 " S, Longitude 150 º 38.0 " E. Rainfall intensities have been calculated for 10-year and 100-year ARI for the standard storm durations and have been compared against the intensities used in the Currambene flood study, which have been used in this study so that results are consistent. These are summarised in the following table.

ARI	10 years		100 years	
Duration (hrs)	Currambene	Georges	Currambene	Georges
1	65.57	66.1	100.64	101.0
3	33.51	33.7	53.23	53.8
6	21.77	21.4	35.34	35.3
9	16.9	16.6	27.8	27.5
12	14.17	13.8	23.51	23.0
18	11.01	10.8	18.02	17.7
24	9.19	9.0	14.89	14.7

Table 2. Rainfall Intensities (mm/hr)

For the storm duration determined in the Currambene flood study to give highest peak flows, the 12 hour storm, the differences are 2.6% for the 10-year ARI and 2.2% for the 100-year ARI. Thus using the Currambene intensities gives a slightly conservative result for Georges Creek.

4.5 PROBABLE MAXIMUM FLOOD

The method for calculation of Probable Maximum Precipitation used in the Currambene flood study is set out in the publication *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method* by the Commonwealth Bureau of Meteorology in June 2003.

The method involves overlaying a rainfall cell over the catchment, with contours of varying rainfall depths, and measuring catchment sub-areas between contours. The alignment of the overlay affects the results obtained, and thus it is somewhat subjective. Accordingly it has not been possible to accurately reproduce the rainfall contour overlay and thereby check the data used by the Currambene flood study, so that data has been simply accepted for this study.

5 DERIVATION OF PEAK RUNOFF

This part of the flood investigation is the Catchment Hydrology – the derivation of peak flows and graphs of the flow changes over time, known as Flow Hydrographs. This step provides the design flows for the creek backwater analysis, and was carried out using the RORB program.

5.1 RORB PARAMETERS

The Currambene flood study analysed catchment discharges in two sections: an upstream section which converges at The Falls, and the overall catchment discharging at Callala Beach.

The peak discharge at The Falls was compared to known flood data and calibrated by adjusting the RORB storage parameter 'kc' and the initial and continuing rainfall losses. For the overall catchment, the RORB parameter 'kc' was selected from a comparison of previously published results for catchments of similar size. These parameters are summarised in the following Table 3.

ARI – years	RORB Model Parameters				
	Initial Loss mm	Continuing Loss mm	Parameter 'm'	Parameter 'kc'	
				The Falls	Overall
100	40	2.5	0.8	10.8	13.9
50	50	2.5	0.8	10.2	13.9
20	55	2.5	0.8	10.5	13.9
10	60	2.5	0.8	10.3	13.9
5	55	2.5	0.8	12.6	13.9
PMF	0	0	0.8	10.8	13.9

Table 3. Rainfall Intensities (mm/hr)

It is important to note that the peak flow calculated at The Falls in the overall model is different from the peak flow calculated for that catchment by itself. This is because, as stated before, the RORB parameters vary with catchment size, and the flows from individual sub-catchments are adjusted to produce the final output hydrograph for the overall catchment.

A review of the data in the computer files provided by Council revealed two minor discrepancies.

The rainfall intensities used for modelling some storms at The Falls were multiplied by an Areal Reduction Factor of 0.96, while the report states that no reduction was applied (i.e. the ARF was 1.0). It appears that the initial modelling had used the ARF but this was subsequently changed based on later research. However, there is no impact on the flood study results, as the peak flows have been calibrated to known data while using that ARF. Changing the ARF to 1.0 would have required recalibration for no change in peak flows.

The second minor error involved the transposition of the areas for catchments F and G in the data for the overall catchment. This error resulted in changes to peak flows in Currambene Creek of less than 1% and negligible impact on flood profiles.

5.2 CALIBRATION AGAINST PREVIOUS STUDY

The version of RORB used in the Currambene flood study would have been an earlier version than the current Version 6.15. To ensure that there has been no changes to the software that affects the results, and to ensure that the data has been properly interpreted in this study, RORB runs were first carried out using the previous data.

The results of the test runs for all storm durations were compared against results tabulated in the Currambene flood report and summarised in the following Table 4.

Duration	The Falls	U/S Georges Ck Confluence	U/S Special Storage	Woollamia	Callala Beach
RORB heading:	**Combined flow ds falls**	**Currambene u/s P+Q+R**	**Currambene d/s Georges Ck**	**Currambene u/s area V**	**Flow at Callala beach**
10 Year ARI: Initial Loss 60.0mm, Continuing Loss 2.5mm/hr					
Table 4.1	**342**	**373**	**501**	**306**	**307**
1 hr	10.76	9.151	13.12)	7.724	7.259
3 hr	130.3	124.6	161.5	107.2	107.4
6 hr	208	223.8	297.7	188	188.7
9 hr	289.4	321.3	420.6	259.2	259.9
12 hr	**342.2**	**373**	**501.4**	**306.1**	**307.1**
18 hr	320.2	367	493.2	324.1	330.1
24 hr	305.5	362.8	466.2	347.7	361
100 Year ARI: Initial Loss 40.0mm, Continuing Loss 2.5mm/hr					
Table 4.1	**788**	**875**	**1190**	**683**	**771**
1 hr	236.1	237.7	306.1	187.7	183.8
3 hr	505.7	518	681.8	389.1	390.1
6 hr	648.8	707.1	936	555.9	558.2
9 hr	787.9	849.2	1125	681.5	686.6
12 hr	797.0	875.6	1191	764.6	771.6
18 hr	713.9	812.3	1093	763.9	789.6
24 hr	713.5	767.9	1021	731.6	775.6

Table 4. Peak discharge calibration results

Duration	The Falls	U/S Georges Ck Confluence	U/S Special Storage	Woollamia	Callala Beach
RORB heading:	**Combined flow ds falls**	**Currambene u/s P+Q+R**	**Currambene d/s Georges Ck**	**Currambene u/s area V**	**Flow at Callala beach**
PMF: Initial Loss 0.0mm, Continuing Loss 0.0mm/hr					
Table 6.2	**1980**		**2810**	**1805**	**1810**
6 hr	1978	2193	2807	1806	1810

Table 4. Peak discharge calibration results (continued)

Peak flows are highlighted in bold font. Good correlation was noted between the published and computed results. However, it was noted that the worst case flows in two locations (highlighted in yellow) occurred for different storm durations than stated in the report, and were higher than published. The differences are minor and have no impact on this flood study, as the peak flows were not used in the HEC-RAS backwater analysis. The actual flow hydrographs were used instead.

5.3 REVISED MODEL FOR GEORGES CREEK

The RORB data files were then corrected for catchments F and G and the catchments S, T and U were subdivided into smaller sub-catchments. The 10-year ARI, 100-year ARI and PMF storm events were analysed for the same range of storm durations. Peak discharges were also compared against previous results and summarised in Table 5 on the following page.

Peak flows generally increased by 0.5% to 1.1%. These small differences are caused by reorganisation of flows by RORB resulting from the changes in distribution of catchment sub-areas and reach length, and to a lesser extent by the correction to the catchment areas F and G as discussed in Section 5.1.

Generally, worst case peak flows occurred during the 12-hour storm event. Towards the lower reaches, longer storm durations tend to give higher peak flows, while at the upper reaches, shorter storm durations tend to give slightly higher peak flows. However, RORB does not take into account flood storage, while HEC-RAS does. Since the effect of flood storage is greater at the lower reaches, the peak flows in longer duration storms would be expected to reduce in the HEC-RAS analysis.

Accordingly, as for the Currambene flood study, the 12-hour storm duration was adopted for the backwater analysis using HEC-RAS. The RORB program output hydrographs generated for each sub-catchment, and these were used as flow inputs to the HEC-RAS unsteady flow modelling.

Duration	U/S Georges Ck Confluence	Georges Ck & Tributary 1	Georges Ck & Tributaries 1 & 2	Georges Ck u/s Confluence	U/S Special Storage	Woollamia	Callala Beach
RORB heading:	Currambene u/s P+Q+R	Georges Ck Central S1-S4	Georges Ck Combined	Georges Ck Section CC10	Currambene d/s Georges Ck	Currambene u/s area V	Flow at Callala beach
10 Year ARI: Initial Loss 60.0mm, Continuing Loss 2.5mm/hr							
Table 4.1	**373**				**501**	**306**	**307**
1 hr	9.114	4.945	8.853	3.62	13.45	7.555	7.02
3 hr	125.4	30.33	47.18	37.97	165.1	108.4	108.5
6 hr	224.9	36.88	56.94	62.26	302.0	189.7	190.3
9 hr	323.5	**61.04**	**94.41**	89.98	428.3	261.3	262.0
12 hr	**374.7**	49.32	78.31	**91.88**	**506.2**	**308.3**	**309.3**
18 hr	368.9	41.46	65.87	81.64	498.8	326.1	332.0
24 hr	365.6	57.33	89.99	89.64	471.1	348.7	362
100 Year ARI: Initial Loss 40.0mm, Continuing Loss 2.5mm/hr							
Table 4.1	**875**				**1190**	**683**	**771**
1 hr	239.3	71.03	113.5	74.28	313.5	189	184.7
3 hr	520.9	97.87	151.1	152.1	693.2	392.5	393.4
6 hr	711.5	105.8	163	174.2	949.1	559.8	561.7
9 hr	854.3	**121.5**	**195**	**208**	1139	**686.7**	691.4
12 hr	**878.7**	111.6	175	181.9	**1198**	**769.2**	**775.6**
18 hr	815.8	80.63	126.3	165.3	1099	767	**792.3**
24 hr	772	99.21	156.6	173.2	1035	733.1	777.6
PMF: Initial Loss 0.0mm, Continuing Loss 0.0mm/hr							
Table 6.2					**2810**	**1805**	**1810**
6 hr	2217	202.8	320.7	414.4	**2840**	**1822**	**1824**

Table 5. Peak discharges – Georges Creek

6 DERIVATION OF FLOOD PROFILES

6.1 HYDRAULIC BACKGROUND

There are two alternative methods for modelling flood events in HEC-RAS.

The simplest method is Steady Flow. The user nominates design flows at selected river stations and the program calculates the depth of flow and water surface profile based on the creek geometry, tailwater level, stream roughness and backwater principles. Typically the peak flows at each section are modelled, and this gives a conservative result as the analysis considers all peak flows to coincide (when in reality they may not coincide) and does not consider the effects of flood storage which tends to reduce peak flows and flood levels.

The more complex method is Unsteady Flow. This method must be used for estuaries where the effects of the diurnal tidal cycle are to be modelled. It takes into account the potential for flow peaks from creek branches to be offset due to time lags as the peaks travel along the river reach. Since the problem is complex, the program reaches a solution by taking an initial estimate of flow and then adjusting the water level until the flow and energy equations balance. This is called iteration, and the process of closing in on a solution is called convergence. In some cases, convergence is not possible, due to sudden changes in hydraulic characteristics, such as steep channel slopes or flow obstructions. In cases where the solution does not converge, the model is said to be unstable.

Modelling of rivers with gentle slopes is quite straightforward. However, modelling of steeper creeks and mountain streams with lower flows is more difficult. Cross-sections must be taken at much closer intervals and short sections with comparatively steeper slopes can cause the modelling to become unstable. The program must iterate to achieve a solution, and sometimes this iteration does not reach a solution because the depth of flow in a particular solution step at a particular location drops below bed level or rises above the top of river bank.

6.2 ROUGHNESS COEFFICIENTS

HEC-RAS calculates the hydraulic capacity (called conveyance) at each cross-section using the Manning formula. The conveyance is inversely proportional to a roughness coefficient ‘n’. Guidance on selection of appropriate values for ‘n’ are given in the HEC-RAS Reference Manual and many other references.

The critical element in determining the roughness coefficient is the amount of vegetation and the stream bed roughness. Photographs of Currambene Creek and Georges Creek were used as a guide. Typical examples are shown on the following page.

Photo 1. Currambene Creek

Photo 2. Georges Creek

HEC-RAS allows for 'n' values varying across different segments of the creek cross-section. In the simple form, the 'n' value is given for the left overbank, main channel, and right overbank areas. In more complex configurations, the 'n' value can vary over multiple segments of the cross-section.

For the lower reaches of Georges Creek, near the junction with Currambene Creek, roughness coefficients of 0.045 and 0.065 were adopted for main channel and overbank areas respectively. This was similar to the roughness coefficients adopted in the Currambene flood study.

For the upper reaches of Georges Creek and its tributaries, the effects of the denser creek bank vegetation were modelled by increasing the 'n' value to 0.12 in a band on either side of the main channel, and the roughness coefficient in the main channel was increased to 0.065.

6.3 MODELLING OF UNSTEADY FLOW

Flow hydrographs generated by RORB were entered as flow data to HEC-RAS as summarised in Table 6 below.

Reach	River Station	Catchments	Peak Flow		
			Q10	Q100	PMF
Currambene Creek	15206	A-L	342.2	757.4	1978
	13934	M	55.92	105.2	191.8
	11308	N	30.24	57.61	110.8
	10170	O	31.97	58.27	98.32
	4535	X	54.98	110.9	191.9
Georges Creek	6870	S1, S2	35.49	72.52	125.86
	5560	S4	23.44	38.42	57.34
	4086	U1	18.16	34.04	55.75
	1961	U2	40.0	62.69	92.34
Georges Tributary 1	1366	S3	10.38	19.47	30.63
Georges Tributary 2	3794	T1	15.71	29.67	52.25
	3160	T2	22.4	33.71	48.09
	1083	T3	18.64	28.13	40.3
Currambene Tributary 1	490	P,Q,R	114.2	246.6	484.4
Currambene Tributary 2	852	N/A	1	1	1
Currambene Tributary 3	1712	V, W	37.92	71.32	112.1

Table 6. Flow Data – Unsteady Flow Analysis

The HEC-RAS model exhibited signs of instability from the start, where initial flows are entered to establish a non-zero base flow. Additional cross-sections were interpolated to reduce the spacing between cross-sections to less than 50 m. A "pilot channel" was introduced to prevent the iteration from failing when water levels dropped to stream bed level, and base flows were increased to compensate for flow in the pilot channel. When a solution was reached, it showed an excessively high jump in water surface levels immediately upstream of the confluences of Georges Creek with its Tributary 1 and Tributary 2. As suggested by the HEC-RAS User Manual, the roughness was increased immediately downstream of the “jumps” but this only served to stabilise the solution process, not correct it.

To verify that the phenomenon that was shown on the unsteady flow creek profiles was not valid, a steady flow analysis was carried out using peak flows from the unsteady case. The creek profile showed a small jump at the confluence, which was expected, but the height of the jump was nowhere near as severe as the unsteady flow analysis predicted.

Further investigation revealed that these excessive “jumps” in water level occurred towards the start of the storm inflow, rather than at the peak flow stage. This indicated that the program was unable to resolve the sharp increase in flows from the base flow condition. Accordingly, it was decided to model the upstream reaches as steady flow rather than unsteady flow. This was considered to be valid because the upper reaches were not affected by the tidal cycle on tailwater level, which affects the downstream reaches of Currambene Creek and Georges Creek. In other words, the backwater effect from tidal variability does not extend to the upper reaches of Georges Creek or its tributaries.

6.4 MODELLING OF STEADY FLOW

Steady flow modelling is conservative, since peak flows are used which do not account for the attenuation that occurs as the flow progresses downstream through the reach. However, because the creek and its banks are typically steep, the conservative nature of this method would not result in large changes to the flood extents. In addition, the attenuation caused by flood storage is relatively small, because the depths of flow are comparatively small.

Peak flows used for the steady-state analysis in HEC-RAS were derived from the peak flow from the input hydrographs (entered as flow hydrographs for the unsteady flow analysis) and/or the peak flows generated by the unsteady analysis as described in the previous section. The choice of which data to use was determined from a review of the flood profiles generated by the unsteady flow analysis. Flows in Currambene Creek and its tributaries were not affected by the flow regime in Georges Creek, so the peak flows were taken from the results of the unsteady flow modelling in the Currambene flood study. Flows in the vicinity of the excessive “jumps” in Georges Creek would have been attenuated by the excessive volume of storage immediately upstream of the jump. Accordingly the unattenuated peak flow data from the input hydrographs were used instead.

These flows are summarised in Table 7 below.

Reach	River Station	Peak Flow		
		Q10	Q100	PMF
Currambene Creek	15206	339.32	753.76	1973.50
	13372	336.60	765.67	1936.78
	10170	258.33	705.75	1898.15
	8167	276.47	794.28	2095.72
	8117	287.93	854.93	2282.19
	7517	272.93	784.56	2155.68
	6035	236.76	718.17	1977.89
	2288	213.93	682.70	1796.59
Georges Creek	6870	35.49	72.52	125.86
	5934	58.93	110.94	183.2
	4662	88.30	128.44	196.61
	4086	103.46	160.61	251.98
Georges Tributary 1	1366	10.38	19.47	30.63
Georges Tributary 2	3794	15.71	29.86	52.29
	3160	31.18	55.94	94.0
	1083	39.88	77.05	128.86
Currambene Tributary 1	490	11.67	65.76	200.81
Currambene Tributary 2	852	1	1	1
Currambene Tributary 3	1712	32.08	71.3	112.1

Table 7. Flow Data – Steady Flow Analysis

For each of these cases, the downstream tailwater level (at the outlet of Currambene Creek at Callala Beach) was set to the tailwater level at the worst case Currambene flood study results, which corresponded to high tide level. The resultant flood profiles have been plotted on drawings C1-01 and C1-02 in Appendix B. The flood levels are summarised in Table 8 below, including comparison with the flood levels determined by the Currambene flood study. The flood levels show satisfactory correlation.

Reach	River Station	Flood Levels					
		Q10		Q100		PMF	
		Currambene	Georges	Currambene	Georges	Currambene	Georges
Currambene Creek	15206	4.66	4.59	6.58	6.53	9.80	9.75
	10170	2.95	2.90	4.32	4.30	6.36	6.32
	8117	2.27	2.32	3.58	3.62	5.53	5.50
	7517	2.15	2.19	3.37	3.36	5.23	5.14
	6035	1.94	1.97	3.10	3.08	4.90	4.78
	2288	1.08	1.06	2.01	1.96	3.79	3.53
Georges Creek	6870	-	20.09	-	20.40	-	20.76
	5934	-	16.29	-	16.65	-	17.03
	4662	-	8.32	-	8.69	-	9.19
	4086	-	5.54	-	6.06	-	6.87
	2859		3.38	-	4.01		5.54
Georges Tributary 1	1366	-	30.06	-	30.16	-	30.26
Georges Tributary 2	3794	-	32.54	-	32.77	-	33.03
	3160	-	29.02	-	29.27	-	29.50
	1083	-	16.25	-	16.66	-	17.09
Currambene Tributary 1	490	2.27	2.34	3.60	3.68	5.56	5.64
Currambene Tributary 2	852	1.96	1.99	3.10	3.10	4.90	4.81
Currambene Tributary 3	1712	2.11	2.16	2.52	2.51	3.75	3.67

Table 8. Flood Profile – Steady Flow Analysis

The indicative flood extents have been plotted on the contour map in drawing C0-02 for each of the 10-year ARI, 100-year ARI and PMF peak flood events by interpolation between contours. That drawing and two enlarged scale drawings C0-03 and C0-04 are included in Appendix A. The flood extents from the Currambene flood study have also been shown along the north bank of Currambene Creek. The flood extents in the Currambene flood report were interpolated between surveyed cross-sections rather than contours. These have been adjusted to fit to the actual landform as defined by the 2m interval contours.

Flood levels for the 10-year ARI, 100-year ARI and PMF have also been plotted on the creek cross-section drawings C2-01 to C2-08 in Appendix C.

6.5 SENSITIVITY CHECKS

Sensitivity checks were carried out on tailwater level and creek roughness.

The above results assumed the tailwater level which gave the worst case flood levels in the Currambene flood study. A sensitivity check was carried out by setting the tailwater to Mean Sea Level (RL 0.0m). A comparison of flood levels resulting from the steady flow analyses is given in Table 9 below.

Reach	River Station	Flood Levels					
		Q10		Q100		PMF	
		TWL 0.87	TWL 0.0	TWL 0.87	TWL 0.0	TWL 0.91	TWL 0.0
Currambene Creek	15206	4.59	4.59	6.53	6.53	9.75	9.75
	10170	2.90	2.90	4.30	4.30	6.32	6.32
	8117	2.32	2.31	3.62	3.61	5.50	5.51
	7517	2.19	2.18	3.36	3.35	5.14	5.14
	6035	1.97	1.95	3.08	3.07	4.78	4.78
	2288	1.06	0.67	1.96	1.88	3.53	3.53
Georges Creek	6870	20.09	20.09	20.40	20.40	20.76	20.76
	5934	16.29	16.29	16.65	16.65	17.03	17.03
	4662	8.32	8.32	8.69	8.69	9.19	9.19
	4086	5.54	5.54	6.06	6.06	6.87	6.87
	2859	3.38	3.38	4.01	4.00	5.54	5.54
Georges Tributary 1	1366	30.06	30.06	30.16	30.16	30.26	30.26
Georges Tributary 2	3794	32.54	32.54	32.77	32.77	33.03	33.03
	3160	29.02	29.02	29.27	29.27	29.50	29.50
	1083	16.25	16.25	16.66	16.66	17.09	17.09
Currambene Tributary 1	490	2.34	2.33	3.68	3.67	5.64	5.62
Currambene Tributary 2	852	1.99	1.97	3.10	3.09	4.81	4.81
Currambene Tributary 3	1712	2.16	2.16	2.51	2.51	3.67	3.61

Table 9. Flood Profile – Sea Level Sensitivity Check

We conclude that the difference between analysis with tailwater level at mean sea level or high tide level is minor in the lower reaches of Currambene Creek and negligible in Georges Creek. In the PMF analysis, there is essentially no difference.

The analysis of flood levels in Georges Creek adopted relatively high values of the roughness coefficient 'n'. It is possible that some reduction in the level of vegetation may occur as a result of bushfire or riparian management, and the true roughness may be slightly lower than has been conservatively assumed. To judge the effects of reduced roughness, the analysis was repeated with a 20% reduction in roughness coefficients. The results have been compared against the previous analysis in the following Table 10.

Reach	River Station	Flood Levels					
		Q10		Q100		PMF	
		Normal 'n'	Lower 'n'	Normal 'n'	Lower 'n'	Normal 'n'	Lower 'n'
Georges Creek	6870	20.09	20.01	20.40	20.29	20.76	20.60
	5934	16.29	16.18	16.65	16.51	17.03	16.85
	4662	8.32	8.13	8.69	8.47	9.19	8.92
	4086	5.54	5.31	6.06	5.78	6.87	6.54
	2859	3.38	3.21	4.01	3.86	5.54	5.45
Georges Tributary 1	1366	30.06	30.04	30.16	30.12	30.26	30.21
Georges Tributary 2	3794	32.54	32.47	32.77	32.68	33.03	32.91
	3160	29.02	28.93	29.27	29.17	29.50	29.38
	1083	16.25	16.13	16.66	16.51	17.09	16.89

Table 10. Flood Profile – Creek Roughness Sensitivity Check

In conjunction with these drops in water level, velocities in the creek channel increase. As the capacity is inversely proportional to the roughness, velocities typically increase by up to 20% in the upper reaches where flows are not affected by backwater from Currambene Creek, reducing progressively in the downstream reach of Georges Creek where velocity increases are in the order of 5%.

7 CLIMATE CHANGE

7.1 ASSESSMENT CRITERIA

DGR 8.6 requires an assessment of the effects of climate change in accordance with the October 2007 DECC publication *Practical Consideration of Climate Change – Floodplain Risk Management Guideline,* which will be referred to as the PCCC Guideline.

The PCCC Guideline has not given definitive predictions of the effects of climate change in terms of sea level rise or increases in rainfall intensity. The 2009 publication had set NSW policy for assessment of sea level rise, nominating a sea level rise of 0.4m by 2050 and 0.9m by 2100, but this policy has been withdrawn. Accordingly, this study will revert to the 2007 PCCC Guideline.

7.2 SEA LEVEL RISE

The PCCC Guideline notes that estimates of potential sea level rise as a result of climate change vary from a low value of 0.18m to a high value of 0.91m. Accordingly, it recommends assessment of three scenarios: 0.18m, 0.55m and 0.91m sea level rise.

These scenarios were modelled in HEC-RAS for the 100-year ARI 12 hour storm as unsteady flow to take into account variations in tailwater level of Currambene Creek with the diurnal tidal cycle. This is modelled in HEC-RAS as a stage hydrograph at STN -100 of the Curr4 reach. The sea level rise was modelled by adding the respective mean sea level increase to each sea level ordinate in the graph. In addition, the effect of a combination of 0.18m sea level rise and 10% increase in rainfall intensity was also modelled. The results are summarised at representative locations in Currambene Creek and the lower reach of Georges Creek in the following Table 11.

The modelling demonstrates that for this river system, sea level rise only affects flood levels in the lower reaches of Currambene Creek and Georges Creek. There would be no measurable difference upstream of STN 1961 in Georges Creek even for 0.91m sea level rise.

The varying flood profiles for the lower part of Currambene Creek have been plotted on drawing C3-01 and included in Appendix D. As there are no measurable differences for Georges Creek, corresponding flood profiles for Georges Creek and its tributaries have not been plotted.

Location	Q100	Q100 + 0.18m SLR	Q100 + 0.55m SLR	Q100 + 0.91m SLR	Q100 + 10% + 0.18m SLR
Georges Creek					
STN 4662	8.53	8.53	8.53	8.53	8.63
STN 4086	6.05	6.05	6.05	6.05	6.19
STN 3464	4.63	4.63	4.63	4.63	4.77
STN 2334	3.40	3.40	3.40	3.40	3.50
STN 1961	3.26	3.26	3.28	3.31	3.48
Currambene Creek					
STN 15206	6.57	6.57	6.57	6.58	6.91
STN 13934	5.45	5.45	5.45	5.46	5.68
STN 10170	4.20	4.20	4.21	4.22	4.42
STN 8317	3.51	3.52	3.53	3.55	3.72
STN 7517	3.26	3.26	3.28	3.31	3.48
STN 4535	2.75	2.76	2.78	2.83	2.95
STN 2328	1.92	1.96	2.06	2.22	2.13
STN 100	0.91	1.08	1.43	1.78	1.09
STN -100					

Table 11. Climate Change – Sea Level Increase

7.3 INCREASE IN RAINFALL INTENSITY

The PCCC Guideline notes that climate change modelling predicts varying changes in rainfall patterns between coastal and inland areas. In coastal areas, rainfall intensities in severe storms could increase by up to 30%. Accordingly, the guideline recommends investigation of the effects of increases in rainfall intensities of 10%, 20% and 30% relative to current IFD data.

Accordingly, the 100-year 12-hour storm was re-modelled in RORB for both The Falls and the Overall catchment, with the required increased rainfall intensities. The total rainfall increased from 282.1mm to 310.3mm, 338.5mm and 366.8mm respectively. The hydrographs generated from RORB for each sub-catchment were entered as inflows to HEC-RAS and modelled as unsteady flow. It was noted that the percentage increases in peak flows were greater than the percentage increases in rainfall intensity. This is because the initial losses of runoff as local storages are filled had already taken place, so the increased rainfall intensity was translated directly into increased runoff. The peak flows generated by the unsteady flow analysis were then modelled as steady flows to derive stable backwater profiles, as described in Section 4.7. The results are summarised in the following Table 12. Note that in each case the tailwater level was set at Mean Sea Level (0.0m AHD).

Location	Q100	Q100 + 10% RI increase	Q100 + 20% RI increase	Q100 + 30% RI increase
Georges Creek Tributary 2				
STN 3794	32.77	32.81	32.85	32.90
STN 3160	29.27	29.32	29.37	29.41
STN 2119	22.24	22.31	22.38	22.44
STN 1083	16.66	16.75	16.83	16.90
Georges Creek Tributary 1				
STN 1366	30.16	30.18	30.21	30.23
STN 453	22.83	22.86	22.90	22.93
Georges Creek				
STN 6870	20.40	20.46	20.52	20.58
STN 6110	17.59	17.66	17.74	17.81
STN 4662	8.68	8.79	8.89	8.99
STN 4086	6.05	6.18	6.32	6.45
STN 3464	4.72	4.87	5.02	5.16
STN 2334	3.74	3.92	4.12	4.28
STN 1961	3.66	3.84	4.05	4.22
Currambene Creek				
STN 15206	6.52	6.85	7.18	7.47
STN 13934	5.37	5.60	5.83	6.03
STN 10170	4.28	4.49	4.71	4.89
STN 8317	3.65	3.85	4.08	4.2
STN 7517	3.23	3.44	3.67	3.84
STN 4535	2.67	2.86	3.07	3.22
STN 2328	1.78	1.95	2.15	2.30
STN 100	0.03	0.10	0.36	0.55
STN -100	0	0	0	0

Table 12. Climate Change – Rainfall Intensity Increase

These results indicate that increases in rainfall intensity will have a noticeable effect on the overall system. The increases in flood levels will be greater in Currambene Creek than in Georges Creek, as the larger catchment will generate correspondingly larger increases in peak discharges. Typically, for each 10% increase in rainfall intensity, 100-year ARI flood levels would increase by 200-300mm in Currambene Creek, about 200mm in the lower reaches of Georges Creek, and between 30mm and 100mm in the upper reaches of Georges Creek and its tributaries.

These flood profiles have been plotted on drawings C3-02 and C3-03 for Currambene Creek and Georges Creek respectively. As the differences in flood level are so small for the Georges Creek tributaries, they have not been plotted.

8 FLOOD HAZARD

DGR 8.5 required the assessment of flood hazard areas based on Shoalhaven City Council's *Flood Risk Management Policy* and DCP No. 106 – *Floodplain Management*. It also required reference to a draft *Currambene Creek Floodplain Risk Management Study and Plan*.

DCP No 106 was adopted by Council on 26 September 2006. It incorporates and updates policies on flood risk management that had been previously published in their *Interim Flood Policy* in September 1987. The latter document has now been superseded by the DCP.

Council advised that as of March 2012 no Floodplain Risk Management Plan (FRMP) had been adopted for the Currambene Creek, and there were no immediate plans for its preparation. Accordingly, this report will reference DCP No. 106 only. However, FRMPs for other catchments in Shoalhaven were reviewed so that this report would be consistent with what has been adopted by Council elsewhere.

DCP No. 106 refers to the *NSW Floodplain Development Manual* [2005] and its terminology, which is discussed in the following section. It also defines two zones:

- The Flood Planning Area is the area below the Flood Planning Level, which is defined as the 100-year ARI (1% AEP) flood level plus freeboard.
- Flood Prone Land is the land below the Probable Maximum Flood level.

Within these zones, flood hazard may be defined as low hazard or high hazard, and areas may be defined as floodway, flood storage or flood fringe.

8.1 DEFINITION OF HYDRAULIC AND HAZARD CATEGORIES

The Floodplain Development Manual defines the three **hydraulic** categories as follows:

- **Floodways** are those areas where a significant volume of water flows during floods and are often aligned with obvious natural channels. They are areas that, even if only partially blocked, would cause a significant increase in flood levels and/or a significant distribution of flood flow, which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.

- **Flood storage areas** are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.

- **Flood fringe** is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

The manual further defines Flood Storage areas as those areas outside floodways that, if completely filled with solid material, would cause peak flood levels to increase anywhere by more than 0.1m and/or would cause the peak discharge anywhere downstream to increase by more than 10%.

The Manual defines flood **hazard** categories in Figure 1.2 in Appendix L. This is reproduced below.

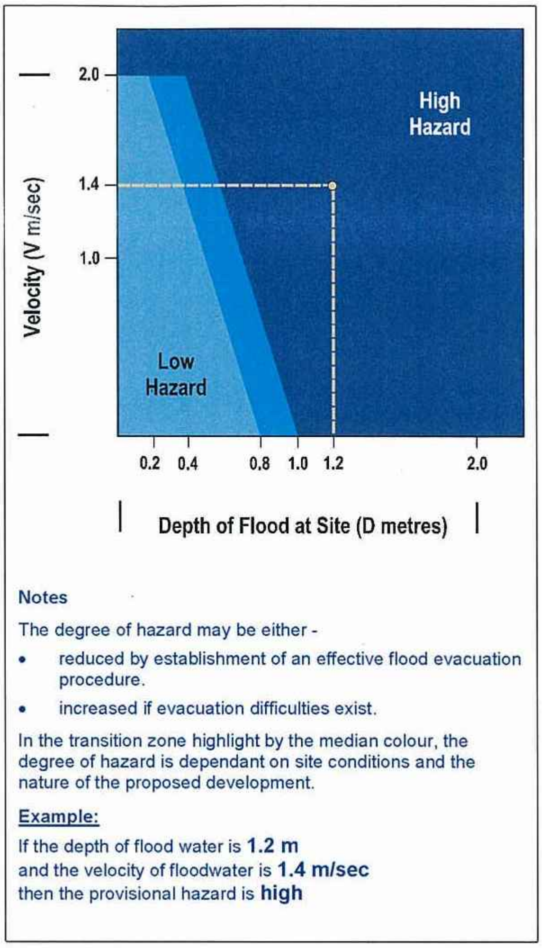

FIGURE L2 - Provisional Hydraulic Hazard Categories

As shown on this graph, any areas with a depth of flow in excess of 1.0 metre are considered High Hazard. Any areas with a stream velocity of 2.0 m/sec or greater are also considered High Hazard. For velocities less than 2.0 m/sec, High Hazard areas are defined by a combination of depth and velocity, as shown on the graph.

It should be noted that these categories of Hydraulic Flood Hazard are provisional, and should be adjusted to suit the individual conditions including ground surface materials and slopes, vegetation and evacuation routes.

8.2 ASSESSMENT OF FLOOD HAZARDS

For this study, floodways are confined to the main creeks which have been studied. There are no bank overflows that would cause floodways outside creek zones. All other flow paths may be considered as "overland flow paths" as the paths are local and the effects of filling or diversion would be confined to the immediate property.

All creeks have been assessed as high hazard along the main creek centreline except for the uppermost reach of Georges Tributary 2 above STN 900 – that is, approximately 0.9km from the confluence with Georges Creek. The width of high hazard zone across the creeks was then assessed on the basis of flood depth and stream velocity, as set out in Figure 1.2. In general, velocities in overbank areas are low, so that the high hazard zone does not extend far past the 1.0m depth limit.

For planning purposes, high hazard limit may be taken as the 10-year ARI flood line. The actual boundary plots so close to the 10-year ARI flood line that it cannot be distinguished from that line at the scale used on the drawings.

Although the Floodplain Development Manual requires an assessment of Flood Storage areas by determining the effects of filling on flood levels upstream and downstream of the site, in practical terms all areas assessed as High Hazard on the basis of depth are storage areas. It becomes an academic exercise to consider filling in those areas. In accordance with common practice, the delineation between High Hazard Floodway and High Hazard Storage has been made on the basis of flow characteristics and the relative effects of storage and flow on the hydraulic model.

In this study, High Hazard Storage has been defined as the lower reach of Georges Creek, up to approximately STN 2700 – that is, for a distance of approximately 2.7km from the confluence with Currambene Creek. In this area, the creek channel and overbank areas are significantly wider than for upstream reaches, and flow velocities correspondingly lower. This area is also the part of Georges Creek most affected by backwater from Currambene Creek. This demonstrates that its major impact on the stream hydraulics is its storage, not its conveyance. Upstream of Georges Creek STN 2700, high hazard areas are defined as "High Hazard Floodway".

According to Shoalhaven City Council's definition in DCP 106, Low Hazard Fringe Areas extend from the High Hazard zone to a level 500mm above 100-year ARI flood line. In the lower reaches of Georges Creek and its tributaries, this limit is located between the 100-year ARI and PMF extents, because the difference in level between the 100-year ARI and PMF flood levels is greater than 500mm. In the upper reaches, the difference between the 100-year ARI and PMF flood levels is less than 500mm, so the 100-year ARI + 500mm line would be outside the PMF. Accordingly, the PMF extents should be taken as the extent of Low Hazard Fringe Zone.

These zones have not been plotted, as the widths of Low Hazard Fringe Zone are typically so narrow that it would appear as a thick line at the scales used for the drawings.

9 MITIGATION AND MANAGEMENT MEASURES

For the proposed Shaolin Tourist and Residential development, it is recommended that:

- All residential allotments should be located above the 100-year ARI flood line.
- Additional clearance should be provided at the upstream side of all road crossings of the major creeks (Georges Creek and its two tributaries) to allow for possible backwater from bridges or culverts.
- Floor levels for all buildings should be at least 500mm above the 100-year ARI flood levels.
- The golf course may extend within the 100-year ARI flood extents but no associated structures should be located within this zone.
- Any structures within the PMF flood extents should be designed to withstand Probable Maximum Flows.

The flood contours were plotted on the proposed site plan. The resulting drawings (C04-01 to C04-04) are included in Appendix A to this report.

The drawings show that all residential allotments are well clear of not just the 100-year ARI flood contour, but also the PMF contour. Where proposed roads cross creeks, and culverts or bridges would be required, additional clearances have been provided. In addition, all proposed buildings in non-residential zones are also well clear of both flood contours.

Having regard to possible minor changes in layout and/or minor changes to calculated flood levels following detailed design, it is clear that the above recommendations can be readily achieved.

10 APPENDICES

Appendix A	Catchment and Flood Extents Plans
Appendix B	Flood Profiles
Appendix C	Georges Creek Cross-Sections
Appendix D	Effects of Climate Change

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APPENDIX A

Catchment and Flood Extents Plans

Drawings from Currambene Creek and Moona Moona Creek Flood Studies:

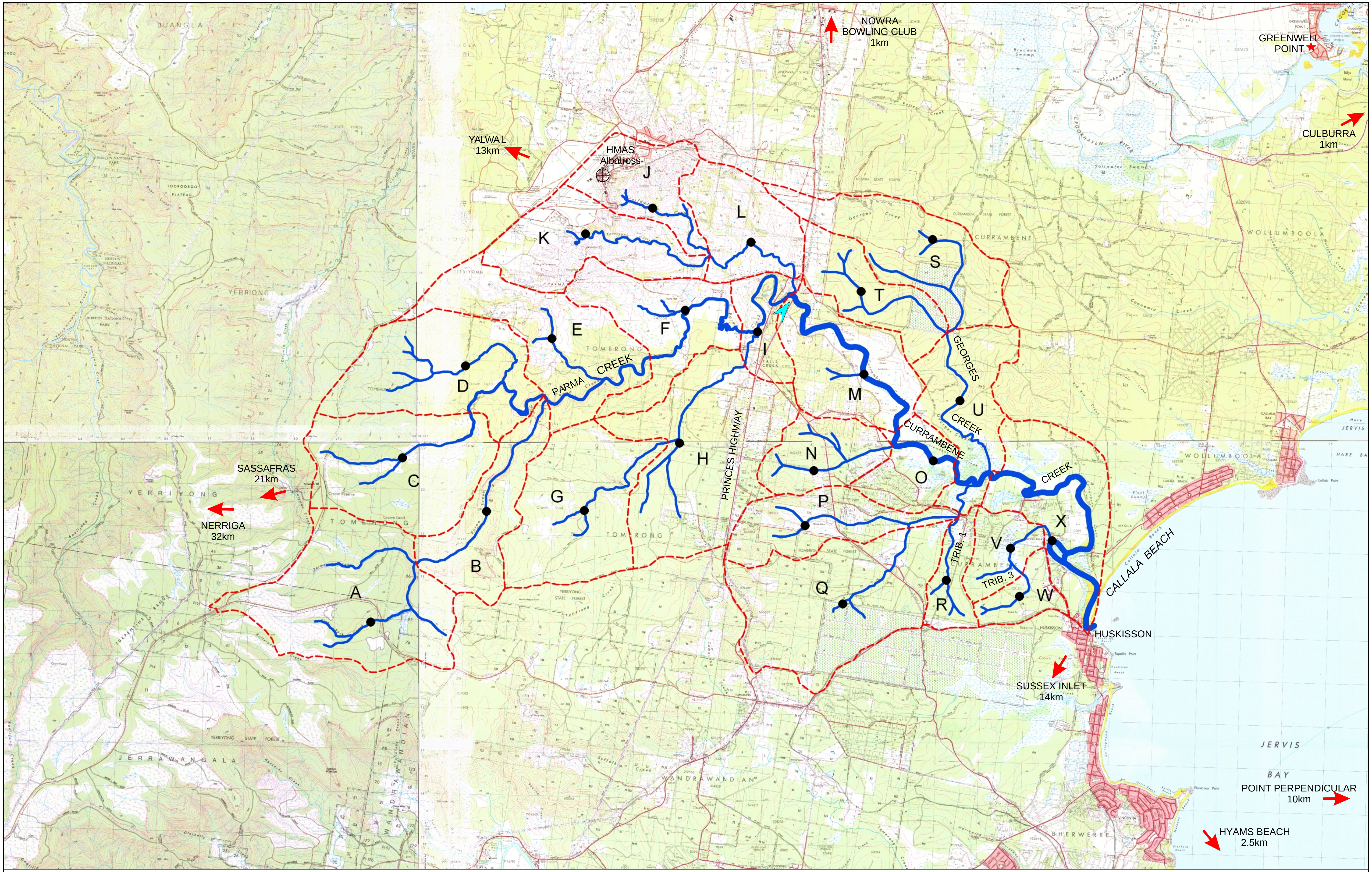
Figure 2.1	RORB Model Layout – Currambene Creek Catchment
Figure 4.5	Currambene Creek – Indicative Extents of Inundation 10 Year, 100 Year ARI and PMF

Georges Creek drawings:

C0110590-C0-01	Georges Creek Catchment Plan
C0110590-C0-02	Georges Creek Model – Flood Extents – Overall Plan
C0110590-C0-03	Georges Creek Model – Flood Extents Plan – Sheet 1 of 2
C0110590-C0-04	Georges Creek Model – Flood Extents Plan – Sheet 2 of 2

Site flood extents plans:

C0110590-C4-01	Stormwater Management Concept Plan Sheet 1 of 4
C0110590-C4-02	Stormwater Management Concept Plan Sheet 2 of 4
C0110590-C4-03	Stormwater Management Concept Plan Sheet 3 of 4
C0110590-C4-04	Stormwater Management Concept Plan Sheet 4 of 4



Currambene Creek at "The Falls" Stream gauging station. (STN 218004)

Pluviograph site

G ● Model sub-area and centroid

Daily Rainfall Station

YALWAL 13km Daily Rainfall Station and distance

CURRAMBENE CREEK AND MOONA MOONA CREEK FLOOD STUDIES

Figure 2.1

RORB MODEL LAYOUT
CURRAMBENE CREEK CATCHMENT