

EUROBODALLA SHIRE COUNCIL

Moruya Flooding - Climate Change Assessment

301015-02079

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Advanced Analysis

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EUROBODALLA SHIRE COUNCIL MORUYA FLOODING - CLIMATE CHANGE ASSESSMENT

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PROJECT 301015-02079 - MORUYA FLOODING - CLIMATE CHANGE ASSESSMENT

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1. INTRODUCTION

1.1 Overview of Study Area

The Moruya Township is located in south-east New South Wales, approximately 7 km upstream from the coast along the Moruya River. The Moruya River is the name of the estuarine portion of the stream from the coast to the tidal limit which is approximately at the confluence of Wamban Creek, 20km from the coast. Upstream of this location is the freshwater portion of the stream known as the Deua River. The total catchment of the Deua/Moruya River is in the order of 1500 km². The Moruya River catchment alone is of the order of 500 km².

The hydrological study area consists of the entire Deua/Moruya River catchment including all major tributaries upstream of Moruya. The hydraulic study area covers the Deua/Moruya River to the location of the Wamban River gauge (approximately 23 kilometres upstream of Moruya Heads / 3 kilometres upstream of the Wamban Creek confluence and includes portions of tributaries in this region. Further details are given in subsequent sections.

Upstream of Wamban Creek the catchment is typically high energy with steep gradients and is heavily vegetated with dense uninhabited bushland. The lower reaches of the catchment, including the majority of the Moruya River, is generally low lying with large portions of the floodplain cleared for crops or grazing. The town of Moruya represents the small portion of the catchment that is urbanised. Downstream of Moruya, significant portions of the floodplain consist of salt marshes that extend to the trained ocean entrance of the river.

1.2 Objectives

The purpose of this Flood Study is to develop a calibrated 2D hydraulic model of the Moruya River and its floodplain to facilitate a climate change impact assessment.

As part of the development of the hydraulic model, a hydrologic model was developed.

1.3 Overview of Previous Flood Study

Hydrological and 1D hydraulic models were developed as part of the *Moruya River Flood Study*; (NSW Public Works Department, 1992). However, none of the model setup files were available and hence new models were developed for this study.

The *Moruya River Flood Study* (herein denoted as "the previous study") used the hydrological model WBNM to convert rainfall to stream flow, and the one dimensional hydraulic model RUBICON to determine water levels and velocities of these stream flows.

Calibration was achieved for both models against available historic flood data; however, design hydrographs were synthetically increased (a 10% increase in design rainfall) compared with published data in *Australian Rainfall and Runoff;* (Engineers Australia; 1987) at the time. This was done primarily because it was felt that *Australian Rainfall and Runoff* (therein denoted as AR&R) did not

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adequately cover the Deua/Moruya catchment and that three historic floods had exceeded the 100 year ARI level predicted using recommended rainfalls.

This resulted in the following estimates for the design flood levels referenced at Moruya Bridge:

- 4.2 mAHD for the 5% AEP event (20 year ARI)
- 4.7 mAHD for the 2% AEP event (50 year ARI)
- 5.1 mAHD for the 1% AEP event (100 year ARI)
- 6.9 mAHD for the Extreme event (PMF)

1.4 Flood History

The SES FloodSafe guide to Moruya indicates that a peak flood level less than 2.60 mAHD can be classified as 'minor', between 2.60 mAHD and 3.2 mAHD as 'moderate' and greater than 3.20 mAHD as 'major'. It must be emphasised that this flood classification system is based on the extent of human impact and not on recurrence interval. Therefore the level classified as 'major' is considerably below the level at which a hydrologist would so classify a flood.

Table 1 shows the floods that have exceeded 3.2 mAHD at Moruya Bridge since 1841 (when European records begin) taken from the *Moruya River Flood History 1841-1978*; (Public Works Department New South Wales). It should be noted that this table only includes events where a reasonable certainty of the peak level exists¹.

| Month and Year | Level (mAHD) |
|----------------|--------------|
| May 1925 | 5.4 |
| March 1975 | 3.70 |
| April 1945 | 3.7 |
| August 1974 | 3.62 |
| March 1978 | 3.40 |
| October 1976 | 3.35 |
| February 1992 | 3.20 |

| Table 1: Most significant floo | l events where a reasonable | certainty of the peak level exists |
|--------------------------------|-----------------------------|------------------------------------|
|--------------------------------|-----------------------------|------------------------------------|

¹ Other events, such as those in 1870, 1898, 1914, 1934, 1860 and 1867 were estimated from newspaper reports and have therefore been excluded from this table.

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2. HYDROLOGY

The primary outcome from the hydrological component of the study is to provide inputs for the hydraulic model. The hydrological model simulates the accumulation of rainfall over the catchment, its movement along defined flow paths, and the subsequent time-varying hydrograph produced at downstream locations.

Model setup files for the previous studies were not available and, as such, a new hydrologic model was developed using current hydrologic methods. To this end, a newer version of WBNM was used in this study (WBNM2000 Version 1.04 Jan 2007 (iWBNM)) in combination with a revised AR&R (1998) and recent studies providing more recent estimates for hydrological catchment parameters for use within WBNM (discussed further in subsequent sections).

Design input hydrographs were generated for the Deua River at the location of the Wamban flow gauge (where historic data was used to calibrate the model). Hydrographs were then generated at Wamban Creek (through the main channel of Wamban Creek and its primary tributary, Candoin Creek) and Mogendoura Creek.

2.1 Previous Flood Study's Hydrology

The previous WBNM model (WBNM Version 1; 1979) consisted of 42 sub-catchment areas and calibrated against the 1974, 1975, 1976, 1978, 1985 and 1988 events. Only a small number of rainfall gauges in the vicinity of the catchment recorded hyetographs for these events and these were used.

Outflow hydrographs at the Wamban gauge were used to calibrate the model against the historic events. The calibration process yielded a lag parameter of 1.68, an initial rainfall loss of 50 mm and a continuing loss of 2.5 mm per hour. The accuracy of these final calibration parameters in matching peak hydrograph flows and volumes for historic events were between -15% to 34% and -25% and 46% respectively.

The derivation of design flow hydrographs was then carried out according to AR&R (1987) where a critical storm duration for the 1% AEP event of 48 hours was also used for the 5%, 2% and PMF events. The peak flow of these hydrographs at Wamban was:

- 3250 m³/s (5% AEP)
- 4300 m³/s (2% AEP)
- 5200 m³/s (1% AEP)
- 11400 m³/s (PMF)

These flows were then adjusted by increasing design rainfall by 10%.

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2.2 WBNM Model Development

WBNM is an integrated hydrograph software package for hydrological studies on natural and urban catchments. The most recent version available, known as iWBNM, was used in this study. This software package uses a new graphical interface through Microsoft Excel® and Visual Basic (VBA). The WBNM software package is an event based hydrologic model and calculates flood hydrographs from storm rainfall hyetographs, using design storms from AR&R (1998).

The iWBNM model requires the sub-division of a catchment, such that runoff from each subcatchment is routed to the next along a defined flow path. The Deua/Moruya catchment was divided using air photos, Council's ALS DTM and Australian topographic maps in waterRIDE[™] FLOOD Manager into 43 sub-catchments. The coordinates of the centroid of each sub-catchment, its area and the coordinates of its outflow point were calculated and input into the iWBNM model.

Sub-catchments containing the primary flow paths (the primary flow channel) were identified and lag and loss parameters were set to initial estimates. The primary variables allowing calibration, assuming the catchment has been correctly sub-divided, are the lag parameter "C", nonlinearity exponent "m" and Stream Lag Factor. Other variables include the initial loss and continuing loss of rainfall to the soil. These two latter calibration variables can be associated with both pervious and impervious surfaces; however impervious surfaces were considered negligible for this catchment and were therefore omitted.

Design storms can be derived from AR&R 1998 (incorporating Bulletin 53, 2003) and storms with an AEP from 5% to the PMF can be simulated with durations that vary from 5 to 4320 minutes (15 to 360 mins for the PMF). In this way, the storm duration that produces the "worst case" response of the catchment for a given rainfall AEP can be determined and the associated hydrograph produced at any sub-catchment.

Figure 1 and Figure 2 show a visualisation of the hydrologic model for the Deua/Moruya River where the output hydrographs from sub-catchments "S33", "S22", "S23" and "S29" represent the hydrographs for the Deua River at Wamban, Wamban Creek and its primary tributary Candoin Creek, and Mogendoura Creek respectively.





Figure 1: Visualisation of the hydrologic model showing the entire catchment overlaid on a NSW topographic map of the region. The model sub-catchments are numbered from S1 to S43 with the routing channel shown in blue. Sub-catchment connectivity along this channel is represented as orange diamonds. Figure 2 shows a close view of this model in the vicinity of Moruya.





Figure 2: Close-up visualisation of the hydrologic model showing the portion of the catchment containing the Moruya River superimposed on an image of the DTM used to generate the model. The hydrograph for the Deua River at the Wamban gauge corresponds to the outflow of sub-catchment "S33". The hydrograph for Wamban Creek and its tributary correspond to the outflows of sub-catchments "S22" and "S23" whilst that for Mogendoura Creek is the outflow of sub-catchment "S29".



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2.3 Model Calibration and Verification

The hydrologic model was calibrated using historic events, the previous study, WBNM literature and current hydrologic techniques against recorded flow data for the Deua River at the Wamban gauge (#217002) from (PINNEENA / NSW Water Information; 2010).

An analysis of the historic floods of significance (Table 1) showed that only limited temporal and spatial rainfall data was available. More specifically, pluviograph data is only available for the western and northern portions of the catchment.

Two historic events were selected for calibration of the hydrologic model, being the 1974 and 1978 events, because these had the best data available with relatively high flows and levels.

2.3.1 Calibration to the 1974 Event

For the 1974 event, five pluviographs with rainfall data at 30 minute increments were available in the vicinity of the catchment:

- Oranmeir (Yarra Glen)
- Oranmeir
- Parkers Gap
- Braidwood (Reidsdale)
- North Araluen P.O.

Of these, only the North Araluen gauge is located *within* the Deua/Moruya Catchment, being located in the far north. The Yarra Glen and Oranmeir gauges are located close to the western boundary of the catchment, from which rainfall data for the western part of the catchment can be inferred. The Parkers Gap and Braidwood gauges are located further west and north respectively, and their use was deemed to be of limited value, and was not included in the model.

An analysis of daily rainfall gauges around the catchment showed a significant variance in the storm's spatial distribution (rainfall volumes). For example, the Bodalla P.O. and Moruya Heads gauges recorded a total accumulation of 249 mm and 267 mm respectively, whilst the pluviographs at Yarra Glen, Oranmeir and North Araluen gauges recorded 405 mm, 502 mm and 380 mm respectively.

As no pluviograph data was available for any part of the southern or eastern catchment, a synthetic pluviograph representing the daily rainfall records at the Bodalla P.O. gauge was generated by scaling the pluviograph data at other gauges by total rainfall accumulation (i.e. the same temporal pattern was assumed).

The Yarra Glen, Oranmeir and North Araluen recorded pluviographs were digitised and used as inputs for the model along with a synthetic Bodalla pluviograph based on scaled daily totals.

Recorded outflow hydrographs from the Wamban flow gauge (#217002) were obtained from the NSW Department of Infrastructure, Planning & Natural Resources (DIPNR).

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Initially, the iWBNM model was setup using the calibration data from the previous study. Hydrograph results were found to be generally smaller in peak flow and volume than those obtained in the previous study², but more closely matched historic recorded hydrographs.

A study in the *Australian Journal of Water Resources;* (Boyd and Bodhinayake, 2006) examined the lag parameter "C" of 584 storms in 54 catchments around Australia and found that the average for NSW was 1.74 and was independent of catchment area, stream slope and storm characteristics. Therefore, the "C" parameter was raised from the previous study's value of 1.68 to 1.74 in line with the updated research.

The nonlinearity exponent and stream lag factor were set to the recommended value of 0.77 and 1.0 as per current WBNM theory and empirical data (*Details of the Theory used in WBNM;* Boyd, Rigby and Schymftsek, 2007).

The initial and continuing losses were varied and simulations compared with recorded data until a "best fit" was obtained using an initial loss of 50 mm and a continuing loss of 1.5 mm per hour. The initial loss is the same as that used in the previous study but the continuing loss is smaller.

The calibrated 1974 simulation produced a hydrograph at the Wamban gauge that was within 4% of the peak flow of the recorded hydrograph, matching the double peak shape very well, and providing a closer fit than the previous model's parameters, (Figure 3).

Calibration parameters were as follows:

Lag Parameter "C" = 1.74 Initial Loss = 50 mm Continuing Loss = 1.5 mm per hour Nonlinearity exponent 'm' = 0.77 Stream Lag Factor = 1.0

² Some difference may be attributed to program updates to WBNM, updates to AR&R data and handmeasured/discretisation of the previous flood studies printed hydrograph and pluviograph plots.

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Figure 3: Summary of the 1974 hydrographs showing that the current model more closely matches historic data than the previous model

2.3.2 Validation to the 1978 Event

The same four pluviograph gauges provided data for the 1978 storm event for the north and western portions of the catchment and an analysis of daily totals again showed that the storm had a significant spatial variation in rainfall. Therefore a synthetic pluviograph was generated at the Bodalla gauge and used with the recorded pluviographs at Yarra Glen, Oranmeir and North Araluen as model inputs.

For reference, the previous studies hydrological model parameters were input and results showed a closer match to the recorded flows at the Wamban gauge than in the 1974 event.

The parameters obtained from the 1974 event calibration (refer section 2.3.1) were then incorporated into the model. The output hydrograph at the Wamban gauge was within 9% of the peak flow, (Figure 4). The rising limb of the hydrograph provided a good match to the shape of historic data; however, the falling limb was more drawn out.

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Figure 4: Summary of the 1978 hydrographs showing that the current model more closely matches historic data than the previous model, although with a drawn-out falling limb

2.3.3 Overall Calibration

The new calibrated model provides a closer match (peak flows, timing of peaks and overall slope) to the recorded flow for the 1974 and 1978 events than the previous study.

2.4 Design Storm Simulations up to the PMF

The hydrologic model was used to simulate the 5%, 1%, 0.5% and 0.2% AEP design storms in accordance with AR&R (1998) guidance (information on the PMF is provided in the following section). In order to calculate the critical storm duration for each design event, the model was setup to simulate these storms with durations varying from 5 to 4320 minutes.

Table 2 summarises the critical durations and peak flows of the calibrated model and provides a comparison with the previous studies results where available.



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Table 2: Summary of design flow hydrographs obtained from the calibrated hydrological model and a comparison with previous studies results where available

| Storm AEP | Current Model - Critical Duration | Current Model - Peak Flow at Wamban Gauge | Previous Model - Peak Flow at Wamban Gauge | |
|--------------|--------------------------------------|--|---|--|
| 5% | 48 hours | 3540 m³/s | 3250 m ³ /s | |
| 1% | 48 hours | 5510 m ³ /s | 5200 m³/s | |
| 0.5% | 36 hours | 6530 m³/s | N/A | |
| 0.2% | 36 hours | 8060 m³/s | N/A | |

Hydrographs for the Deua River, Wamban, Candoin and Mogendoura Creeks are shown in Figure 6 through Figure 9.

2.5 **PMP Design Storm Simulation**

The revised version of AR&R (1998), in combination with the Bureau of Meteorology's (BoM) Bulletin 53 (2003), provides guidance on determining the PMP. However it limits the use of the Generalised Short-Duration Method for the estimation of the Probable Maximum Flow for the Moruya / Deua River catchment as the catchment size is in excess of the suggested 1000 km² limit. It also suggests that the storm's critical duration should not exceed 6 hours.

Another approach suggested by DIPNR involves using a flood discharge equivalent to three times the peak 1% AEP discharge for the assessment of extreme floods where PMF estimates can not be reliably defined. This approach is likely to be overly conservative.

An alternative to the DIPNR approach is to extrapolate the peak flow from smaller design hydrographs using a log-linear distribution. This is not recommended by the BoM's Bulletin 53 and it would not be able to provide an estimate for the hydrograph shape (only the peak discharge).

Considering the above, the approach used in this study involved two steps and it is deemed to be generally conservative, giving a peak flow similar to that of the previous flood study but with a critical duration based on current recommendations from the BoM.

Firstly, in order to estimate the *shape and critical duration* of the PMF hydrograph, the hydrologic model was "cut-down" into two smaller catchment models of less than 1000 km² (where inflows from the upstream catchment were fed into the downstream catchment). Using AR&R (1998) and the BoM's Bulletin 53, the PMP was simulated on these "cut-down" models producing critical duration of 6 hours and a representation of the shape of a PMF hydrograph for the Moruya/Deua catchment (The "cut-down" model, according to Bulletin 53 requirements, would not give a conservative estimate of peak flows and therefore only the shape and critical duration were utilised).

Next, the peak flow was estimated based on peak flows of the other design storms using a log-linear extrapolation technique. This gave a PMF peak flow of approximately 11 500 m³/s which is similar to

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the previous flood study's estimate of 11 400 m³. This peak flow value was then used to produce the PMF hydrograph using the critical duration and hydrograph shape calculated using the "cut-down" model.

Figure 5 shows a plot of this data with the PMP represented by a 10 000 year ARI. The plot should be relatively linear for high recurrence intervals and taper towards the extreme flood. A value of three times the 100 year peak discharge is shown as a comparison, highlighting that the DIPNR estimate is overly conservative.



Figure 5: Extrapolation of the peak hydrograph flow for the extreme flood using a log-linear distribution. The plot should taper off with a low gradient on the right hand side of the plot. The pink data point represents the equivalent of the DIPNR estimate showing that this is overly conservative.

Hydrographs for the Deua River, Wamban, Candoin and Mogendoura Creeks are shown in Figure 6 through Figure 9.





Figure 6: Design hydrograph outputs for the Deua River at Wamban





Figure 7: Design hydrograph outputs for Wamban Creek





Figure 8: Design hydrograph outputs for Candoin Creek





Figure 9: Design hydrograph outputs for Mogendoura Creek



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3. HYDRAULICS

The hydraulic component of this study involved the development of an RMA-2 fully two dimensional finite element model (Version 8, March 2007).

The model utilises the design hydrographs generated in the hydrological component of this study for the Deua River at Wamban as well as Wamban, Candoin and Mogendoura Creeks. The outflow of the model is regulated based on a defined downstream relationship (ocean levels as per the *Updated Climate Change Working Paper* and the updated proposal letter both dated 22nd Of July 2010).

Historic data from the Deua gauge at Wamban was used to calibrate the model in combination with level measurements obtained at various locations within the study area.

3.1 Previous Flood Study's Hydraulic Analyses

The 1D RUBICON hydraulic model, used in the previous study, was calibrated to historic flood data available by primarily adjusting the Manning roughness. For historic floods, the ocean entrance channel area was also increased.

Historic hydrographs for historic event at Moruya Heads were not available and therefore the previous study estimated ocean levels (incorporating storm surge, wave and wind setup).

Design ocean hydrographs were generated incorporating storm surge and wave setup with the peak level set to coincide with the time of peak storm rainfall. Table 3 shows these peak levels used as well as the resulting peak levels estimated for each design flood at Moruya Bridge.

| Design Flood | Peak Level at Moruya Bridge | Peak Ocean Level Boundary Condition |
|--------------|-----------------------------|-------------------------------------|
| 5% AEP | 4.2 mAHD | 1.8 mAHD |
| 2 % AEP | 4.7 mAHD | 1.9 mAHD |
| 1% AEP | 5.1 mAHD | 2.0 mAHD |
| PMF | 6.9 mAHD | 2.0 mAHD |

Table 3: Summary of the RUBICON 1D model design flood simulations

3.2 RMA2 Model Development

The study area extended from several kilometres upstream of the confluence of the Deua River and Wamban Creek (at the approximate location of Deua River gauge 217002) downstream along the Deua River to the ocean entrance at Moruya Heads. The model incorporated a portion of Wamban, Candoin and Mogendoura Creeks to allow for inflows as well as a large portion of Racecourse and Malabar Creeks. Between these boundary conditions, all land that would become inundated during the design runs was included in the model. This meant that for the 500 year and PMF design flood



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simulations, the downstream boundary of the model was widened to incorporate the headlands either side of the Moruya River entrance.

The resulting study area was discretised using approximately 10,000 elements for the two model networks used in the analyses. Suitable element sizing was based on minimising the model run time whilst adequately capturing the topographical and terrain roughness characteristics. The flexibility in varying element sizes in RMA-2 models allows increased detail to be incorporated where required without increasing it throughout the whole model. Areas with large elevation gradients were discretised with many smaller elements whilst largely flat areas were approximated with larger elements. Key flow paths were given a greater spatial resolution across their flow cross-section and in areas where flow changed direction, elements with aspect ratios close to 1 were utilised. The network was built with the use of air photos, Council's ALS data, Council's DTM, and survey data extracted from the previous study (refer Figure 10 through Figure 12)



Figure 10: Air photos overlaid with Council's Aerial Laser Survey data. This data set is highly detailed, with topography measured to an accuracy of several centimetres although only covering some portions of the catchment.



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Figure 11: Air photos overlaid with Council's DTM. This data set is coarser than the ALS data, giving more broad-based information about the topography, although it covers the entire catchment area.



Figure 12: Air photos overlaid with a DTM and survey data generated from information utilised in the previous flood study. This data set provides broad information about the shape of the river channels and other important features that were surveyed for the previous model.

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Hydrosurvey data (as cross-sections) from the previous study was used to allocate elevations to nodes below the water surface. Air photos were used to assist in interpolating hydrosurvey data between cross-sections.

Available survey data and photos were used to incorporate the effects of bridges, culverts and other covered flow paths within the model. The piers of Moruya Bridge were included in the model.

Roughness parameters were estimated (prior to calibration) based on visible vegetative covering ranging from 0.025 to 0.080 (Manning roughness). The head loss associated with the reduction in flow area at bridges due to piers was modelled by increasing roughness to 0.100 over the span of the bridge.

Tailwater levels at the downstream end of the model were based on astronomical tide data overlaid with storm surge and wave/wind setup. The peak ocean levels were set to coincide with the time of peak rainfall intensity. The same peak ocean levels shown in Table 3 (from the previous flood study) were used, as discussed in the *Climate Change Working Paper*. It should be noted that because the peak ocean level coincides with the peak rainfall intensity, the peak flow in the downstream reach of the Deua River does not coincide with the peak ocean level. It was found that in all cases, the peak flow in the Deua River at Moruya Heads corresponds approximately to the next tidal peak sequence (after the ultimate peak) (this is shown in Figure 24).

Calibration hydrograph inflows for the Deua River were extracted from historic records whilst those for Wamban, Candoin and Mogendoura Creek were generated using the hydrologic model. Calibration and verification was undertaken using the 1974 and 1978 historic events, as was done for the hydrologic model.

Using the calibrated model, the 5%, 1%, 0.5% and 0.2% AEP design flood events as well as the PMF were simulated with all design hydrographs taken from the hydrological model.

Figure 13 and Figure 15 show the hydraulic model of the Deua/Moruya River.





Figure 13: Hydraulic Model showing air photos of the study area overlaid with the element mesh network (all calibration and design simulations up to and including the 200 years ARI event). More elevated areas around Moruya Heads have not been included because they are above the level of inundation for these floods.





Figure 14: Hydraulic Model showing air photos of the study area overlaid with the element mesh network (only the 500 year ARI and PMF event). The model incorporates a greater extent of the higher topography around the headlands that is affected during more extreme events.





Figure 15: Hydraulic Model showing air photos of the study area overlaid with the model network terrain surface and showing the location of boundary inflow and outflows and highlighting the captured topographical information within the network indicated by element shading



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3.3 Model Calibration and Verification

For each historic event, recorded flow hydrographs were input into the model and the resulting peak levels compared with recorded levels throughout the floodplain. Manning's roughness was then adjusted in order to reduce discrepancies. Naturally, recorded levels are limited in number and vary between nearby locations. The aim of the calibration process is to provide a best fit to all available data.

Sources for discrepancies in historic data can be generally linked to incorrect readings, damaged or sloping gauges, the location of gauges near hydraulic controls such as bridges or river bends, timing of readings (levels, photos, flood extents are not necessarily read or recorded at the peak of the flood and/or recorded with time) and changes to the environment (urban development, clearing vegetation, manmade or natural changes to the channel).

Two historic events were selected for calibration of the hydraulic model, being the 1974 and 1978 events.

3.3.1 Calibration to the 1974 Event

For the 1974 event, input hydrographs for the Deua River were based on the recorded data (refer Figure 3) whilst inflows for Wamban, Candoin and Mogendoura Creeks were generated by routing recorded rainfall through the hydrologic model (refer Figure 16).

The downstream boundary of the model requires ocean levels however they were not recorded for the historic events. Astronomical tide data was, therefore, calculated and used for the simulations whilst storm surge and wave/wind setup was unknown and not considered (Figure 17). This was consistent with the previous study.

In total, there are 16 historic data points for calibration in the study area with the majority being located in the town of Moruya and recorded by the NSW Public Works Department and Council. At some locations, two readings were available that were different by tens of centimetres, highlighting the limited accuracy of some of the data. Furthermore, some data had been collected indirectly through the use of photos at various times during the 1974 flood event. Measurements that were clearly related to a level on the rising or falling limb of the flood were excluded for calibration purposes (the peak of the flood occurred through Moruya approximately at 02:00 on the 29th of August, 1974). Typically, measurements derived from photographs related to a water surface outside of a 12 hour window of the peak of the flood. This was of a similar nature to the method used in the previous study where *"greater reliance"* was given to direct measurement made by Council and PWD as apposed to some levels derived from photos as *"it is likely that they were not taken at the peak of the flood"*.

All recorded data was obtained from the previous flood study. It has been assumed that this data was correct and that the location of each reading was accurately shown on provided maps (coordinates were not provided). The model roughness was iteratively varied to produce the best fit to the historic data. This was verified against levels obtained using the 1978 event inflows.

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Figure 16: 1974 historic flood simulation inflow hydrographs for major tributaries







The final calibrated hydraulic model provided a close match to the recorded data in the 1974 flood. Table 4 summaries the recorded, simulated and previous flood study data and the corresponding differences in levels (to recorded data). The calibrated RMA-2 model had an average of 0.11 m discrepancy with recorded data whilst the previous flood study's discrepancies were on average 0.13 m. Figure 18 shows the calibrated model results around Moruya overlaid on air photos with recorded data shown on top.

| Location Name ³ | Level Recorded ⁴ | Current Model | Previous Model ⁵ | Current Difference ⁶ | Previous Difference ⁷ |
|-------------------------------------|--------------------------------|------------------|--------------------------------|------------------------------------|-------------------------------------|
| Wamban Road | 9.18 / 8.54 | 9.20 | 9.10 | +0.02 | -0.08 |
| Kiora Bridge | 8.80 | 9.01 | 8.80 | +0.21 | +0.00 |
| Hospital | 4.05 | 3.90 | 3.88 | -0.15 | -0.17 |
| Bowling Club | 3.79 | 3.59 | 3.65 | -0.20 | -0.14 |
| Kindergarten | 3.66 | 3.59 | 3.65 | -0.07 | -0.01 |
| River Street | 3.64 | 3.62 | 3.65 | -0.02 | +0.01 |
| Church Street | 3.63 | 3.59 | 3.65 | -0.04 | +0.02 |
| Moruya Bridge | 3.62 | 3.47 | 3.49 | -0.15 | -0.13 |
| Princess Highway | 3.48 | 3.32 | 3.35 | -0.16 | -0.13 |
| Swimming Pool | 3.28/3.31 | 3.22 | 3.22 | -0.09 | -0.09 |
| Ford Street (South) | 3.18 | 3.18 | 2.95 | 0.00 | -0.23 |
| Ford Street (North) | 2.78 | 2.82 | 2.95 | +0.04 | +0.17 |
| Vulcan/Princess | 2.80 | 2.62 | 2.43 | -0.18 | -0.37 |
| Racecourse Ck | 2.87 | 2.57 | 2.43 | -0.30 | -0.44 |
| Mullanderee | N/A | 2.49 | 2.44 | N/A | N/A |
| Malabar Ck | 2.40 | 2.28 | 2.35 | -0.12 | -0.05 |
| | | | | | |
| Average Difference (absolute value) | | | | 0.11 | 0.13 |

Table 4: Summary of the 1974 Flood Event showing the recorded, current and previous flood study data

³ As referenced in the previous flood study.

⁴ Levels that were outside a 12 hour window of the peak of the flood were excluded. N/A signifies a comparison between the previous and current model only (No peak flood data on record)

⁵ Not all data was available in tabular form. Some results from the previous study were read from graphs and are subject to some error.

⁶ The difference was taken between the highest recorded level and the simulated level.

⁷ The difference was taken between the highest recorded level and the previous flood study's level.







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3.3.2 Verification against the 1978 Event

The calibrated hydraulic model was then used to simulate the 1978 event to verify that the set of calibrated parameters provided a best fit to both the 1974 and 1978 events.

For the 1978 event, input hydrographs for the Deua River were based on the recorded data (refer Figure 4) whilst inflows for Wamban, Candoin and Mogendoura Creeks were generated from the hydrology model and are shown in Figure 19.

Similarly to the 1974 event calibration, astronomical tide data was also derived as no recorded data was available. Storm surge and wave/wind setup was unable to be determined and therefore not considered (Figure 20).

In total, there were 5 historic data points for verification in the study area for the 1978 event. These were obtained from the previous flood study (which referred the source as the PWD) and it has been assumed that the location of each recorded level was accurately located on the figures (coordinates were not provided).



Figure 19: 1978 historic flood simulation inflow hydrographs for major tributaries

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Figure 20: 1978 astronomical tide at Moruya Heads

Table 5 summaries the recorded, simulated and previous flood study data and the corresponding differences in levels (to recorded data). The calibrated RMA-2 model had an average of 0.10m discrepancy with recorded data whilst the previous flood study's discrepancies were on average 0.16m.

Figure 18 shows the calibrated model results around Moruya overlaid on air photos with recorded data shown on top.



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Table 5: Summary of the 1978 Flood Event showing the recorded, simulated and previous flood study data

| Location Name ⁸ | Level Recorded ⁹ | Level Simulated | Previous Flood Study ¹⁰ | Simulated Difference | Previous Difference | | |
|----------------------------|--------------------------------|--------------------|---------------------------------------|-------------------------|------------------------|--|--|
| Kiora Bridge | 8.97 | 9.16 | 8.60 | +0.19 | -0.37 | | |
| Hospital | 4.00 | 3.97 | 3.73 | -0.03 | -0.27 | | |
| Kindergarten | 3.57 | 3.65 | 3.51 | +0.08 | -0.06 | | |
| GR 362 220 | 3.55 | 3.64 | 3.51 | +0.09 | -0.04 | | |
| Moruya Bridge | 3.40 | 3.52 | 3.35 | +0.12 | -0.05 | | |
| Mullanderee | N/A | 2.56 | 2.06 | N/A | N/A | | |
| Malabar Ck | N/A | 2.31 | 2.17 | N/A | N/A | | |
| | | | | | | | |
| Average (absolute value) | | | | 0.10 | 0.16 | | |

⁸ As referenced in the previous flood study.

 $^{^{\}rm 9}$ N/A signifies a comparison between the previous and current model only (No peak flood data on record)

¹⁰ Not all data was available in tabular form. Some results from the previous study were read from graphs and are subject to some error.

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3.3.3 Summary of Hydraulic Model Parameters

The model geometry was assigned based on ALS, DTM and survey data available whilst the bathymetry was assigned based on available survey data only. Some elevation data was quite sparse in some areas. In these regions, elevations were interpolated.

The following Manning "n" parameters were found to provide the "best fit" to the historical floods:

- o open river channel; 0.030
- o open grassland, open marshes, smooth floodplain; 0.035
- o long grass/light vegetation; 0.040
- o light shrubbery/vegetated floodplain; 0.045
- o medium shrubbery/vegetated floodplain; 0.055
- o heavy shrubbery/vegetated floodplain; 0.065
- o open urban / impervious areas; 0.030
- o built-up urban / impervious areas; 0.035 to 0.045
- o dense urban / impervious areas (including fenced areas); 0.055 to 0.065

A graphical representation of the variation in manning roughness parameters used in the hydraulic RMA-2 model is shown in Figure 22.



Figure 22: RMA-2 hydraulic model showing a visual representation of how the manning roughness varies in the calibrated model (overlaid on air photos).


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3.4 Design Flood Simulations

The hydraulic model design simulations consisted of input hydrographs for the Deua River, Wamban, Candoin and Mogendoura Creeks for the 5%, 1%, 0.5% and 0.2% AEP floods as well as the PMF as produced by the hydrological model (refer section 2.4 and 2.5).

The downstream boundary condition was based on an astronomical tide sequence overlaid with a storm surge peak level of 1.8 mAHD for the 20 year ARI event and 2.0 mAHD for all other design simulations (as was used in the previous Flood Study). The tidal boundary condition is shown in Figure 23 for all design floods. High astronomical tide was set to occur at approximately the time when peak discharge occurred through Moruya Heads whilst the peak tide surge was set to align with the peak rainfall intensity (as shown in Figure 24). As mentioned previously, this results in peak flood flows through Moruya Heads always coinciding with the second tidal peak (following immediately after the ultimate peak storm surge) – (refer Figure 24). Inflow hydrographs were determined from the hydrologic model (refer section 2.4)



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3.5 Results

Figure 26 shows the peak water surface profile along the Deua/Moruya River for all design flood simulations. The plot extends from several kilometres upstream of the Wamban Creek confluence and shows the relative location of key landmarks for reference.

Figure 27 to Figure 40 display the hydraulic model simulation results centred on Moruya showing the peak:

- Depth with velocity vectors (for all design flood simulations)
- Velocity times depth (for all design flood simulations)
- Water level (only for the 1% AEP and PMF design flood simulations)
- Preliminary Hydraulic Hazards (only for the 1% AEP and PMF design flood simulations). The five hazard categories used in this report are based on the two hazard categories defined in the NSW Floodplain Development manual, but with a greater resolution (refer to Figure 25).



Figure 25: The velocity and depth definition for the preliminary hydraulic hazard categories used in this report. The hashed area represents the "Low" hazard definition from the Floodplain Development Manual, with everything else being classified as "High".

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The 5% AEP Design Flood event shows peak levels in the town of Moruya that vary between 3.60m AHD and 4.15m AHD. Peak levels in the vicinity of the town of Moruya are generally less than those produced in the previous flood study. On Mullenderee Flats, peak levels vary between 3.45m AHD to 4.05 mAHD with depths exceeding 2 m over large areas. Around Moruya Heads (upstream of the training wall), levels are 0.2m higher than the previous study.

The 1% AEP Design Flood event shows peak levels in the town of Moruya that vary between 4.90m AHD and 5.20m AHD. Peak levels at Moruya Bridge are almost identical to those produced in the previous study (despite peak flows being larger); however the overall regional water surface profile is now flatter, meaning that velocities are reduced. Preliminary hydraulic hazards through the majority of the town of Moruya are "High" or "Very High". These hazards are primarily depth related although some areas adjacent to Racecourse Creek experience significant flow velocities. On Mullenderee Flats, peak levels vary between 4.75m AHD to 5.25m AHD with depths exceeding 3 m over large areas. Some preliminary Hydraulic Hazards through the majority of Mullenderee Flats are 'Very High' or 'Extreme' being driven by both large depths and flow velocities. Figure 41 shows a comparison of the 1% AEP flood extents with that from the previous study.

This flatter profile results in reduced peak 1% design flood levels upstream of Moruya but significantly increased levels downstream. At Moruya Heads, peak 1% levels are approximately 1.0m higher than the previous study. This is a result of the 2D model more accurately locating the position and magnitude of the significiant 'head loss' associated with the flow constriction at the heads, as evidenced by the longitudinal plot in Figure 26.

The 0.5% AEP Design Flood event shows peak levels in the town of Moruya that vary between 5.45 mAHD and 5.70m AHD. On Mullenderee Flats, peak levels vary between 5.35m AHD to 5.60m AHD. Flow velocities are generally much greater over Mullenderee Flats than through the town of Moruya.

The 0.2% AEP Design Flood event shows peak levels in the town of Moruya that vary between 6.25m AHD and 6.45m AHD. On Mullenderee Flats, peak levels vary between 6.20m AHD to 6.40m AHD. Flow velocities are significantly greater over Mullenderee Flats than through the town of Moruya.

The PMF Design Flood event shows peak levels in the town of Moruya that vary between 7.45m AHD and 7.65m AHD. Peak levels at Moruya Bridge are in the order of 0.6 m greater than those produced in the previous study; however the water surface slope is much flatter, indicating that velocities are reduced. Hydraulic hazards through the majority of the town of Moruya are 'Very High' with some 'Extreme' areas where flow from the Moruya River expands towards Racecourse Creek. On Mullenderee Flats, peak levels vary between 7.35m AHD to 7.60m AHD. Hydraulic Hazards through the vast majority of Mullenderee Flats is 'Extreme'. Peak flow velocities are generally much greater through Mullenderee Flats than the town of Moruya.

An interesting feature to note is the hydraulic control at Moruya Heads, as shown by the significant head loss in Figure 26. This indicates that the geometry of the heads plays a more significant role in determining flood behaviour than the ocean tailwater condition.

Table 6 shows a comparison of the design flood levels for the current and previous model at key locations for the 5% and 1% AEP flood events as well as for the extreme flood.

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| | 5% Flood AEP | | 1% Flood AEP | | Extreme Flood | |
|-------------------------------------|------------------|-------------------|------------------|-------------------|------------------|-------------------|
| Location on the Moruya River | Current Model | Previous Model | Current Model | Previous Model | Current Model | Previous Model |
| Kiora Bridge | 10.60 | 10.75 | 12.00 | 12.75 | 15.08 | 15.50 |
| Mogendoura Creek | 6.85 | 7.30 | 7.78 | 8.75 | 9.63 | 10.60 |
| Hospital | 4.52 | 4.70 | 5.48 | 5.65 | 7.80 | 7.50 |
| Moruya Bridge (U/S side) | 4.10 | 4.18 | 5.14 | 5.15 | 7.60 | 7.00 |
| Racecourse Creek | 3.43 | 3.55 | 4.80 | 4.60 | 7.38 | 6.40 |
| Malabar Creek | 3.30 | 3.40 | 4.68 | 4.50 | 7.20 | 6.30 |
| Garlandtown | 2.55 | 2.40 | 3.84 | 3.15 | 5.89 | 4.50 |
| Moruya Heads (U/S training wall) | 2.17 | 2.00 | 3.54 | 2.50 | 5.65 | 3.50 |
| Moruya Heads (D/S training wall) | 1.81 | 1.80 | 2.01 | 2.00 | 2.07 | 2.00 |

Table 6: Comparison of flood levels in the previous and current models¹¹

The main causes of the difference in levels at Moruya Heads between the current study and the previous study are:

- 1.2D entrance losses for flows entering the channel through the heads not 'seen' in the 1D model
- 2. More detail in the 2D model geometry gives a better representation of hydraulic gradients through the heads.

¹¹ Flood levels were read from 0.25m contour plots from the previous study and therefore may be subject to some rounding errors.

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Figure 26: Moruya River Peak Water Surface Profile within the model domain. Overlaid is the relative location of key landmarks.





Figure 27: 20 year ARI Design Flood; Moruya Overview - Depth Coloured with Velocity Vectors





Figure 28: 20 year ARI Design Flood; Moruya Overview – Velocity times Depth





Figure 29: 100 year ARI Design Flood; Moruya Overview - Depth Coloured with Velocity Vectors





Figure 30: 100 year ARI Design Flood; Moruya Overview – Velocity times Depth





Figure 31: 100 year ARI Design Flood; Moruya Overview - Levels





Figure 32: 100 year ARI Design Flood; Moruya Overview – Hydraulic Hazard





Figure 33: 200 year ARI Design Flood; Moruya Overview - Depth Coloured with Velocity Vectors





Figure 34: 200 year ARI Design Flood; Moruya Overview – Velocity times Depth





Figure 35: 500 year ARI Design Flood; Moruya Overview - Depth Coloured with Velocity Vectors





Figure 36: 500 year ARI Design Flood; Moruya Overview – Velocity times Depth





Figure 37: PMF Design Flood; Moruya Overview - Depth Coloured with Velocity Vectors





Figure 38: PMF Design Flood; Moruya Overview – Velocity times Depth





Figure 39: PMF Design Flood; Moruya Overview - Levels





Figure 40: PMF Design Flood; Moruya Overview – Hydraulic Hazard





Figure 41: Comparison of the current 100 year ARI extents with the previous model



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4. CLIMATE CHANGE ASSESSMENT

Climate change is recognised by the NSW Government as having the potential to significantly impact on both social and economic aspects of the community as well as the environment. The key impacts of climate change on flooding are attributed to sea level rise (SLR) and changes to rainfall behaviour (i.e. likely decreases in the occurrence of storms but increases in rainfall intensity).

The updated *Climate Change Working Paper; 22nd July, 2010,* discussed the likely impacts to the Deua/Moruya Catchment. From this, six scenarios were established as the basis for assessing the impact of climate change on the Moruya Catchment (Proposal 22nd July, 2010). These are associated with Sea Level Rises (SLR), rainfall intensity increases and "Existing Conditions" based on the previous Flood Study as well as updated parameters.

The scenarios analysed were the:

- a. Base Case ("Existing Conditions 1992 Study"):
 - 1 in 100 year ARI design storm hydrograph derived using the current WBNM model ("updated hydrology").
 - Peak ocean levels of 2.0m AHD
 - No "envelope approach"
- b. Change in "Base Case" due to Sea Level Rise Only Year 2050 Conditions (envelope approach maximum levels from):
 - i. Scenario 'a' (above) with 0.4m increase in ocean tailwater levels (ocean peak = 2.4m AHD). This represents the 1 in 100 year rainfall with 1 in 20 year tailwater level
 - ii. 1 in 20 year ARI design storm with 1 in 100 year ARI ocean levels plus 0.4m SLR allowance (ocean peak = 2.7m AHD).
- c. Change in "Base Case" due to Sea Level Rise Only Year 2100 Conditions (envelope approach maximum levels from):
 - i. Scenario 'a' (above) with 0.9m increase in ocean tailwater levels (ocean peak = 2.9m AHD).
 - ii. 1 in 20 year ARI design storm with 1 in 100 year ARI ocean levels plus 0.4m SLR allowance (ocean peak = 3.2m AHD).
- d. Change in "Base Case" due to Rainfall Intensity Increase Only
 - i. Scenario 'a' (above) with 10% increase in rainfall intensity
 - ii. Scenario 'a' (above) with 20% increase in rainfall intensity



- e. Change in "Base Case" due to Sea Level Rise (2050) and Rainfall Intensity Increase
 - i. Scenario 'b' (above) with 10% increase in rainfall intensity
- f. Updated "Existing Conditions" using current recommendations (envelope approach maximum levels from):
 - i. 1 in 100 year design rainfall (using updated techniques) with 1 in 20 year ocean levels (peak = 2.3m AHD refer figure 2 of discussion paper)
 - ii. 1 in 20 year design rainfall (using updated techniques) with 1 in 100 year ocean levels (peak = 2.6m AHD refer figure 2 of discussion paper)



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4.1 Results

The aforementioned scenarios were simulated and the peak water surface for each task was produced (as shown in Figure 42 to Figure 48)

Differences between the peak water level surface of task 'b' through 'e' and task 'a' as well as that between task 'f' and 'a' were also produced (Figure 49 to Figure 54). Comparisons of peak levels at key locations are shown in Table 7 with the difference compared with the "base case" shown in brackets.

The 2050 sea level rise (Task 'b') is essentially "absorbed" by the large head loss at Moruya Heads. This leads to a rise in peak levels of no more than 0.05 m in the Moruya River, which occurs just west of the heads. This increase in peak levels reduces to 0.02 m at the confluence of Malabar Creek and remains relatively constant through the town of Moruya. The difference in peak levels reduces to zero at the confluence of Mogendoura Creek.

The 2100 sea level rise (Task 'c') is somewhat absorbed by the head loss at Moruya Heads. This leads to a rise in peak levels of no more than 0.10 m in the Moruya River, which occurs immediately west of the Heads. This increase in peak levels reduces to no more than 0.06 m at the confluence of Malabar Creek and remains relatively constant through the town of Moruya, reducing to a 0.05 m increase upstream of the Moruya Bridge. The difference in peak levels falls to zero at the confluence of Mogendoura Creek.

The impact of rainfall intensity increases (Task 'd') has a much more pronounced affect on peak flood levels in the Moruya River. For a 10% increase in rainfall intensity at the 1% flood event, peak levels increase by up to 0.47 m in areas between Malabar Creek and Moruya Heads. This reduces to approximately 0.45 m for the area upstream of Malabar Creek to Moruya Bridge and to 0.30 m at the confluence of Mogendoura Creek.

With a 20% increase in rainfall intensity, peak levels for the 1% flood event increase by up to 0.90 m with this being approximately constant downstream of Moruya Bridge. The majority of the town of Moruya is affected by increases of 0.85 m. Upstream of the bridge; differences in peak levels begin to reduce with increases of 0.80 m immediately upstream, reducing to 0.50 m at the confluence of Mogendoura Creek.

Task 'e' provides an indication of the combined effect of SLR and rainfall intensity and the results show that peak levels increase by up to 0.55m, with the bulk attributable to the increased rainfall intensity. The difference is approximately constant downstream of the Moruya Bridge, with the majority of the town of Moruya affected by increases of 0.50 m. Upstream of the bridge, the increase in peak levels reduces to 0.30 m at the confluence of Mogendoura Creek.

The above results highlight the fact that Moruya Heads acts as a significant hydraulic control absorbing most of the modelled increase in sea level. In this regard, peak levels in the Moruya River are affected more by the rainfall increase component of climate change than the SLR component.

Task 'f', the updated existing conditions scenario, is based on current DECCW recommendations that suggests higher ocean levels should be used compared to the previous study. These results show only small differences to the flood levels from the base case. The increased storm surge level rapidly



decreases through Moruya Heads with peak level increases of no more than 0.05 m in the Moruya River, immediately west of the heads. This increase in peak levels reduces to no more than 0.02 m approximately 500 metres west of Moruya Heads. The peak water level increase at Moruya Bridge is approximately 0.01 m and reduces to zero approximately 2 km upstream of Moruya Bridge.

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Table 7: A comparison of the peak levels between the different cases at key locations. All inflows relate to the updated hydrology. The change in level relative to the base case is shown in brackets at the corresponding location.

| Location on the Moruya River | Task A; the base case (100 year flows plus 2 m tailwater) | Task B; 2050 SLR (Envelope of 100 year flows plus 2.4m TW and 20 year flows plus 2.7m TW) | Task C; 2100 SLR (Envelope of 100 year flows plus 2.9m TW and 20 year flows plus 3.2m TW) | Task Di; (10% increase in rainfall intensity for 100 year flows with 2 m tailwater) | Task Dii; (20% increase in rainfall intensity for 100 year flows with 2 m tailwater) | Task E; (Same as Task C but with a 10% increase in rainfall intensity for 100 and 20 year flows) | Task F; Updated existing (Envelope of 100 year flows plus 2.3m TW and 20 year flows plus 2.6m TW) |
|--|---|--|--|--|---|---|--|
| Kiora Bridge | 12.00 | 12.00 (+0) | 12.00 (+0) | 12.46 (+0.46) | 12.89 (+0.89) | 12.46 (+0.46) | 12.00 (+0) |
| Mogendoura Creek | 7.78 | 7.78 (+0) | 7.79 (+0.01) | 8.07 (+0.29) | 8.32 (+0.54) | 8.07 (+0.29) | 7.78 (+0) |
| Hospital | 5.48 | 5.49 (+0.01) | 5.52 (+0.04) | 5.85 (+0.37) | 6.19 (+0.71) | 5.90 (+0.42) | 5.49 (+0.01) |
| Moruya Bridge (U/S side) | 5.14 | 5.16 (+0.02) | 5.19 (+0.05) | 5.56 (+0.42) | 5.93 (+0.79) | 5.61 (+0.47) | 5.15 (+0.01) |
| Racecourse Creek | 4.81 | 4.84 (+0.03) | 4.88 (+0.07) | 5.28 (+0.47) | 5.68 (+0.87) | 5.34 (+0.53) | 4.83 (+0.02) |
| Malabar Creek | 4.68 | 4.70 (+0.02) | 4.74 (+0.06) | 5.15 (+0.47) | 5.54 (+0.86) | 5.20 (+0.52) | 4.70 (+0.02) |
| Garlandtown | 3.84 | 3.87 (+0.03) | 3.91 (+0.07) | 4.28 (+0.44) | 4.66 (+0.82) | 4.34 (+0.50) | 3.86 (+0.02) |
| Moruya Heads (U/S training wall) | 3.54 | 3.58 (+0.04) | 3.63 (+0.09) | 4.01 (+0.47) | 4.41 (+0.87) | 4.10 (+0.56) | 3.57 (+0.03) |
| Moruya Heads (D/S training wall) | 2.01 | 2.72 (+0.71) | 3.22 (+1.21) | 2.01 (+0) | 2.01 (+0) | 3.23 (+1.22) | 2.61 (+0.60) |





Figure 42: BASE CASE (TASK a; Existing Conditions); Peak Level with 0.25m contours





Figure 43: 2050 SLR CASE (TASK b; envelope approach); Peak Level with 0.25m contours





Figure 44: 2100 SLR CASE (TASK c; envelope approach); Peak Level with 0.25m contours





Figure 45: BASE CASE WITH 10% INCREASE IN RAINFALL INTENSITY (TASK d.i.); Peak Level with 0.25m contours





Figure 46: BASE CASE WITH 20% INCREASE IN RAINFALL INTENSITY (TASK d.ii.); Peak Level with 0.25m contours





Figure 47: 2100 SLR CASE WITH A 10% INCREASE IN RAINFALL INTENSITY (TASK e; envelope approach); Peak Level with 0.25m contours





Figure 48: UPDATED EXISTING CONDITIONS (TASK f; envelope approach); Peak Level with 0.25m contours





Figure 49: Water Surface Difference between the 2050 SLR case (Task 'b') and the Base Case (Task 'a')





Figure 50: Water Surface Difference between the 2100 SLR case (Task 'c') and the Base Case (Task 'a').




Figure 51: Water Surface Difference between the 10% increase in rainfall case (Task 'd.i') and the Base Case (Task 'a').





Figure 52: Water Surface Difference between the 20% increase in rainfall case (Task 'd.ii') and the Base Case (Task 'a').





Figure 53: Water Surface Difference between the 2100 SLR + 10% increase in rainfall case (Task 'e') and the Base Case (Task 'a').





Figure 54: Water Surface Difference between the Updated Existing Conditions (Task 'f') and the Base Case (Task 'a').



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5. CONCLUSION

The objective of this study was to assess the impacts of forecast climate change on the Deua/Moruya River.

The combinations of model runs in this study show that the action of the Moruya Heads as a hydraulic control tends to significantly reduce the impacts of the variance in ocean levels on peak flood levels in the Moruya Township. Essentially, the head loss associated with the Moruya Heads acts as somewhat of a 'buffer' against forecast increases in sea level. This was also evidenced in the previous (1992) study. Consequently, the current 500mm and 300mm freeboard for residential and commercial properties is likely to provide sufficient buffer against expected sea level rise by 2050 (ie increases in peak 100yr flood levels of less than 0.05m throughout the floodplain). Council may wish to evaluate planning controls in light of the slightly higher impact associated with sea level rise estimates by the year 2100 (general increases of between 0.05m and 0.1m in the populated parts of the floodplain).

The impact of forecast climate change increases in design storm rainfall intensity, however, has a significant impact on peak levels in Moruya. Consequently, predicted changes to rainfall patterns constitute a higher climate change risk to Moruya than Sea Level Rise and should be considered in any planning response to 'climate change'. However, there is currently significant uncertainty in scientific estimates of how 'climate change' may affect rainfall patterns. Engineers Australia is currently reviewing *Australian Rainfall and Runoff* and is expected to publish updated design rainfall information (and climate change impacts). This update is due for release in 2012 and it is recommended that when this data is available, Council:

- Update the flood/hydrologic models with updated rainfall parameters, and
- Undertake a review of the flood planning level based on the updated model results.

The 2D model results in differences in peak levels across the floodplain compared to the previous study. The most significant of these is in the upstream vicinity of Moruya Heads, where levels increase by up to 1.0m in the 1% design flood and up to 2.15m for the extreme design flood. Current planning controls will need to be reviewed in this area as these differences cannot be accommodated within current freeboard allowances.



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Appendix A; Flood Planning Area Maps

The Flood Planning Level (FPL) maps for Moruya are shown in Figure 55 through Figure 57. This show:

- the existing FPL based on the "Base Case" with a 0.5 m freeboard
- the predicted 2100 FPL based on a envelope approach with a 0.9m rise in ocean levels and a 10% increase in rainfall intensity (with a 0.5 m freeboard)
- the updated existing FPL based on an envelope approach with current storm surge recommendations (with a 0.5 m freeboard).

Each figure shows the relevant planning level with 0.25 m contours and the extent of flood prone land (that is, the extent of the PMF using the "Base Case" storm surge conditions).

Generally, the "base case" provides a FPL of approximately 5.5 mAHD for the town of Moruya, which is very similar to that of the FPL given by the "updated existing conditions". The 2100 scenario with sea level rise and rainfall intensity increase raises this level to approximately 6.0 mAHD.





Figure 55: Current Flood Planning Area Map (Base Case plus 0.5m freeboard) showing level contours in white and the extent of flood affected areas in red (PMF).





Figure 56: 2100 Flood Planning Area Map (2100 sea level rise (0.9m) with a 10% increase in rainfall intensity plus a 0.5 m freeboard) showing level contours in white and the extent of base case flood affected areas in red (PMF).





Figure 57: Updated Existing Conditions Flood Planning Area Map (Using current recommendations with an envelope approach plus 0.5m freeboard) showing level contours in white and the extent of the base case flood affected areas in red (PMF).



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