# FLOODPLAIN AND OCEAN INUNDATION ASSESSMENTS THAT DON'T BREAK THE BANK.

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In many areas, there is a lack of information on flooding or ocean inundation due to climate change and sea level rise (SLR) until detailed studies are completed.

Eurobodalla Council's Interim SLR Adaptation Policy requires affected development proposals to address these hazards. This paper illustrates two cost effective and robust assessment methods to predict climate change impacts.

## CASE STUDY 1: SURFSIDE BEACH COASTAL INUNDATION

### Background

Much of the village of Surfside at North Batemans Bay is a sand plain at an elevation of about 2m AHD. The beach dune crest is low at 2.7 - 3.3m AHD.

The Surfside plain is vulnerable to inundation from a 1% ocean storm overtopping the low dunes. In the future, sea level rise could potentially exacerbate inundation. Flood levels behind the dunes would be mainly controlled by the inflow of water from dune overtopping. Additional flow into the hinterland would input from the surcharge of stormwater pipes.

**Figure 1 Surfside Beach**

## Coastal Storm Data

The Eurobodalla Coastal Hazards Scoping Study (SMEC, 2010) and the Batemans Bay Coastline Hazard Management Plan (Webb McKeown 2001) provide the following parameters for Surfside Beach for a 1% AEP ocean storm occurring today:

- Tailwater level = 2.66m AHD
  (Storm surge 1.5m AHD + 1.16 m nearshore wave setup)
- Theoretical nearshore wave height $H_s$ = 1.5 m
- Peak wave period $T_p$ =12 s

## Dune Levels

Levels for the dune crest at Surfside Beach are derived from Council's LIDAR survey data and summarised in Table 1.

**Table 1 Dune crest levels Surfside Beach**

| Precinct | Shoreline Length | Dune Crest Levels (m AHD) | |
|---|---|---|---|
| | | Range | Mean |
| | | | |
| East | 270 m | RL 2.9 – RL 3.1 | RL 2.99 |
| Central | 430 m | RL 3.2 – RL 3.5 | RL 3.33 |
| West | 120 m | RL 2.5 – RL 3.0 | RL 2.74 |

## Dune Overtopping Flows

### *Current Year*

Calculation of wave runup overtopping applied physical modeling data reported in Figure VI-5-15 of Coastal Engineering Manual. This quantified 1% AEP mean overtopping rates occurring today at Surfside, factored up to achieve a 95% confidence level (Table 2).

**Table 2 Surfside Beach Current Wave Overtopping Rates**

| Zone | Overtopping Rate |
|---|---|
| East | 73 L/s per m |
| Central | 32 L/s per m |
| West | 173 L/s per m |

The current overtopping flow rate integrated over the whole of the beach is **54 cumecs**. Note that once runup from an individual wave overtops the dune, the nature of the flow then approximates open channel flow. Estimates of critical depth and critical velocity can be applied to consider loadings and damage potential to beachfront dwellings.

*Future Sea Level Rise (SLR)*

The 1% AEP tailwater level for Surfside Beach is predicted to increase from 2.66m AHD today, to 3.00m and 3.50m AHD for years 2050 and 2100 respectively.

Parts of the beach have dune crests below the 1% nearshore ocean level with SLR. At face value, this suggests extreme potential inundation of the dune crest and hinterland at Surfside Beach. However, where low beach barriers occur, sea level rise is expected to cause a sand feed to the dune field which recedes landwards. Therefore as sea level rise progresses, sand deposition would be expected to maintain the same freeboard to the design ocean water level as at present.

Overtopping rates under future SLR scenarios would therefore be identical to the current year. As a sensitivity check, if expected increases in dune elevation were not to occur, increased overtopping flows at year 2050 are estimated at **300 cumecs** over the whole length of beach.

At year 2100 without the expected dune height increase, inflows would occur over the top of the tide with setup from a 1% ocean storm. In this extreme case, the design nearshore stillwater level would be higher in elevation than the current dune height along the whole length of the beach.

Under this scenario the wave overtopping flow rates are meaningless as this Surfside area would fill to a level of 3.5m AHD. Serious damage to beachfront dwellings would occur. A reasonable assumption taken by this paper is that the dune crest at Surfside would either naturally build up, or if this did not occur, that the dune would be progressively artificially nourished.

## Overtopping Flow Depth and Velocity

*Current Year*

As hydraulic loads are sensitive to velocity, design loads are calculated for the relatively high velocity associated with critical-depth flow conditions. Attenuation with distance from the crest would occur but is ignored to be conservative. Structural wave loads impacting onto a flat wall assumed perpendicular to the flow are also derived.

For the east and west precincts, the calculated design conditions just landward of the dune crest are shown in Table 3 for current wave and ocean conditions.

**Table 3 Surfside Beach Current Wave Overtopping Conditions**

| Zone | Critical Flow Depth (m) | Overtopping Velocity (m/s) | Wave Loading (kPa) |
|---|---|---|---|
| East | 0.082 | 0.9 | 0.9 |
| West | 0.145 | 1.2 | 1.6 |

*Year 2050 Conditions without Dune Deposition*

Using the integrated wave overtopping rate across the whole beach of 300 cumecs gives the following extreme design conditions (Table 4).

**Table 4 Surfside Beach 2050 Extreme Wave Overtopping Conditions**

| Zone | Critical Flow Depth (m) | Overtopping Velocity (m/s) | Wave Loading (kPa) |
|---|---|---|---|
| Whole beach | 0.24 | 1.52 | 2.6 |

## Hinterland Flood Levels

If they were sufficiently sustained, wave overtopping could feasibly flow across properties and along roads in Surfside to pond in a lower 'basin' area. The components of inflow to this 'basin' would be:

- wave overtopping flows;
- direct rainfall; and
- surcharging of stormwater pipes, which drain to four locations across Surfside Beach.

The basin would drain by overland flow across a weir located on Figure 1.

Inflow and flood level parameters are calculated as shown in Table 5.

**Table 5 Surfside Beach Hinterland Flood Levels**

| SLR SCENARIO | Wave Overtopping (cumecs) | Stormwater Surcharge (cumecs) | Direct Rainfall (cumecs) | TOTAL (cumecs) | Flood Level (m AHD) |
|---|---|---|---|---|---|
| **Current** | 54 | 9 | 2 | 65 | 2.235 |
| **2050** | 54 | 10 | 2 | 66 | 2.24 |
| **2100** | 54 | 12 | 2 | 68 | 2.25 |

Hydraulic weir calculations show the current peak 'basin' level for a discharge of 65 cumecs would be less than **2.3m AHD** and insensitive to changes in inflow.

The year 2050 conditions without dune deposition would result in higher wave overtopping flows and hence a higher basin level than Table 5 estimates. Weir calculations show the 2050 peak 'basin' level for a discharge of 312 cumecs would be approximately **2.8m AHD**. We propose that this level be adopted as a conservative year 2050 scenario for planning purposes, until council adopts a firmer position on protection of the dune system.

## Acknowledgements

We are indebted to Gary Blumberg of GBA Coastal Pty Ltd who has performed calculations of wave overtopping conditions and loadings.

## References

SMEC Australia 2010, *Eurobodalla Shire Coastal Hazards Scoping Study*

US Army Corps of Engineers 2002, *Coastal Engineering Manual,* EM 1110-2-1100

Webb McKeown 2001, *Batemans Bay Coastline Hazard Management Plan*

# CASE STUDY 2 - MORUYA INDUSTRIAL DEVELOPMENT

## Introduction

In 2009 the NSW Government released their NSW Sea Level Rise Policy Statement (Department of Environment Climate Change and Water, 2009) which specified sea level planning benchmarks for the NSW Coastline. For areas affected by freshwater flooding, and within the area of influence from sea level rise this meant that existing flood planning levels set through the flood study, floodplain risk management plan and Council development control plan process may not have allowed for the risks associated with sea level rise into the future, as well as other potential climate change impacts.

For some areas of the Eurobodalla this left Council in a difficult position. On the one hand, the State Government had issued a policy statement recommending the amount of sea level rise to be considered within the planning process, however the flood studies that Council relied on to set flood planning levels did not take this rise into consideration. This led to a period of uncertainty until Council was able to update their flood studies to incorporate sea level rise impacts.

## Development on the Moruya Floodplain

In this specific case, a development application was being prepared for an industrial development on the outskirts of Moruya, on the edge of the Moruya floodplain (Figure 2).

A flood study of the Moruya River had been prepared by the Department of Public Works and Services (DPWS) in 1992 and a floodplain management study was prepared by Patterson Britton in 1996. These studies did not consider sea level rise.

Council were hesitant to apply existing flood planning levels to the site and were unsure about how to apply sea level rise impacts to a development site a considerable way (7km) upstream of the river entrance . Funding was made available for an update of the floodplain management study, however this work would take at least 6-12 months to complete, whilst the development remained in limbo.

The development proponent did not have the funding available, nor the inclination to prepare a stand-alone flood study for the Moruya floodplain to set levels for the development, however wanted the development assessed as quickly as possible. A simplified, less expensive approach was developed to attempt to assess the impact of sea level rise on flood levels in this part of the Moruya River to provide some security for Council in setting flood planning levels for the development site.

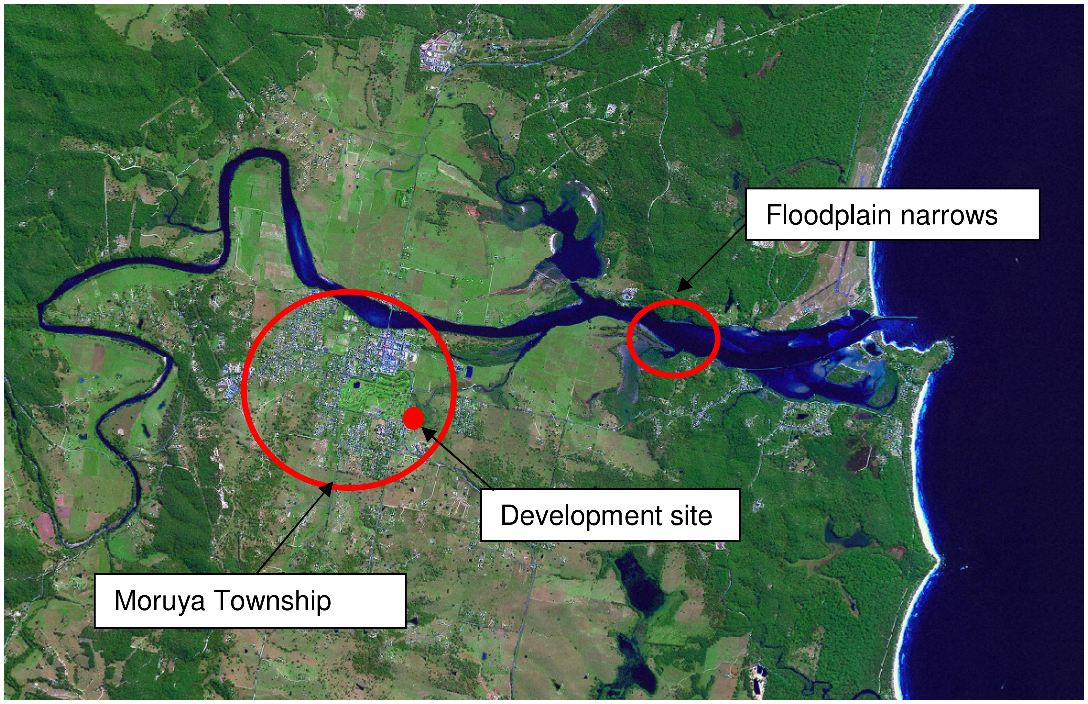

**Figure 2 Moruya River Floodplain**

## Floodplain behavior

The Moruya Flood Study tested a range of ocean entrance (tail water) conditions for the 1% AEP freshwater flood event as part of the hydraulic modeling of the Moruya River (DPWS, 1992). Results showed that an increase in tail water level from 1.6m AHD to 2.4m AHD led to an increase in flood level at the Moruya River bridge of only 0.05m. Clearly, in the case of a large freshwater flood, the influence of tail-water levels 7km upstream is limited; nonetheless, Council required that the impact of sea level rise, particularly the State adopted increases be assessed.

A primary reason for the limited transfer of tail water levels upstream relates to a geomorphic characteristic of the Moruya River floodplain. Approximately 3.5km downstream of the Moruya bridge, the floodplain and river is “choked” at a point where the floodplain narrows considerably as the river passes through a relatively narrow gap associated with areas of granitic geology around the floodplain.

This choke point acts as a primary control on Moruya River flood levels upstream of that point for some distance. This is clearly shown in flood level long sections through the river from both historic and more recent flood modeling (Figure 3 and Figure 4)

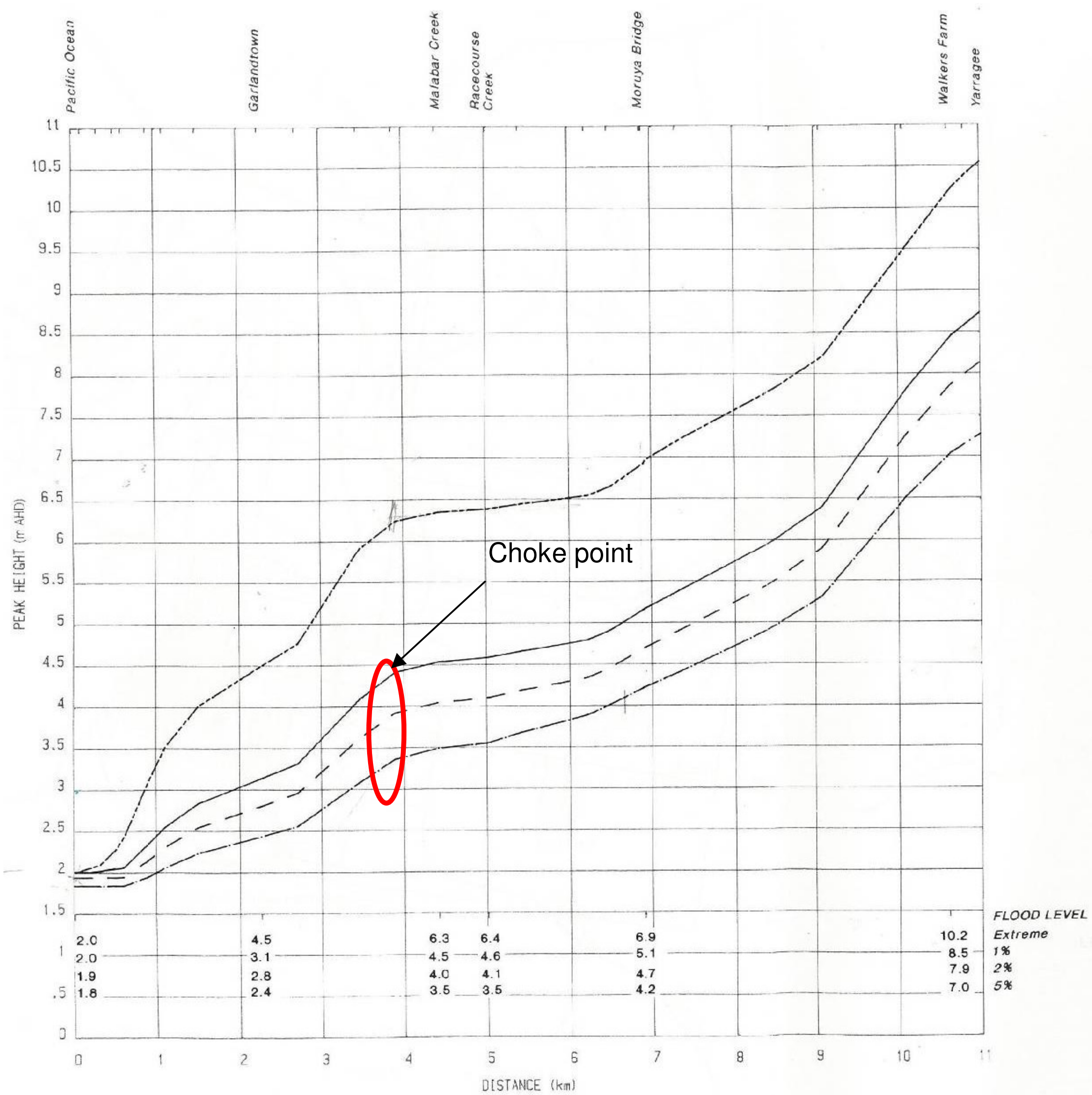


**Figure 3. Flood longsection from 1992 Moruya Flood Study (DPWS, 1992).**

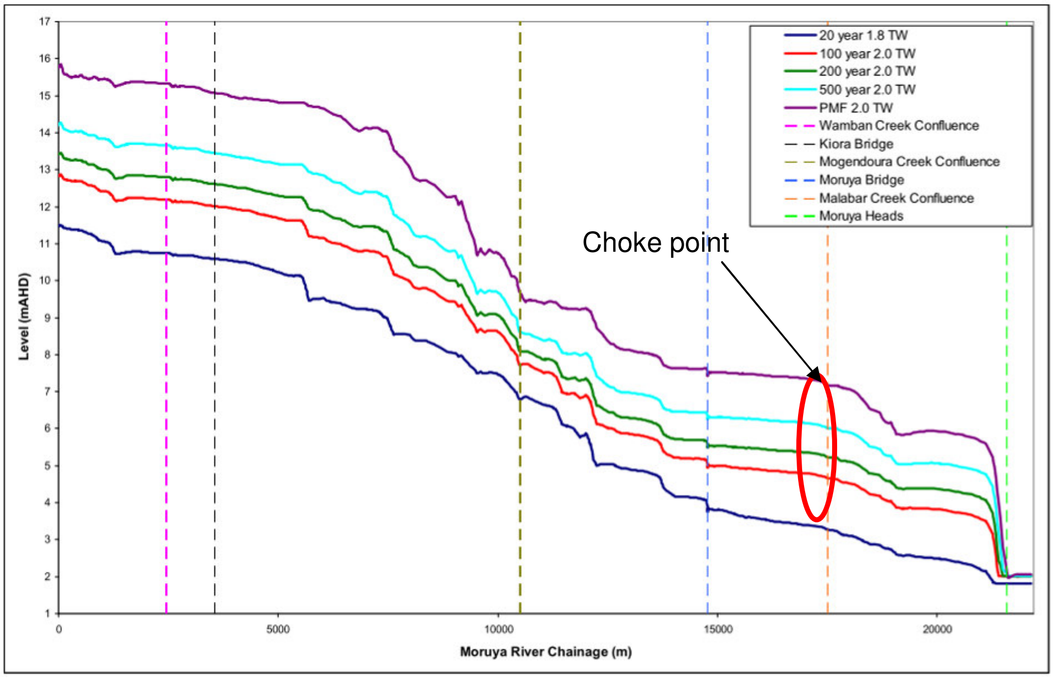


**Figure 4. Flood long section from 2011 Flood Study (Worley Parsons, 2011)**

Although the sensitivity analysis clearly shows that for the tail water levels tested, the impacts at Moruya township are limited (DPWS, 1992), the range considered does not extend to future tail water levels that include sea level rise and climate change impacts. This needed to be tested to satisfy Council's requirements.

We proposed to make use of this floodplain characteristic to create a simple stage storage relationship for the area upstream of the choke point and use this relationship, and the impact of sea level rise to estimate flood levels for the Moruya Township.

## Flood level assessment methodology

At the time of the assessment we had limited access to terrain information. A surface model of the floodplain based on 2m contours on the southern side of the Moruysa River and a surface model extracted from Google Earth was prepared. The Google Earth surface model was compared with 2m ALS contours as well as on ground survey and found to be sufficiently accurate for the purposes of these estimations. The extent of the surface model created was from the choke point to approximately 800m upstream of the Moruya Bridge (Figure 5).

Flood levels at the Moruya Bridge were used to estimate storage volumes in the area shown (Figure 5). The storage volume was then broken down into the component associated with ocean inundation, and that associated with freshwater inflows. In essence the two flood components were assumed to be layered on top of one another for the purpose of this assessment (
Figure 6). We understood that this was not an accurate representation of the real interaction between freshwater floodwaters and ocean inflows, however was estimated as a conservative approach given that the time varied nature of inundation was ignored.

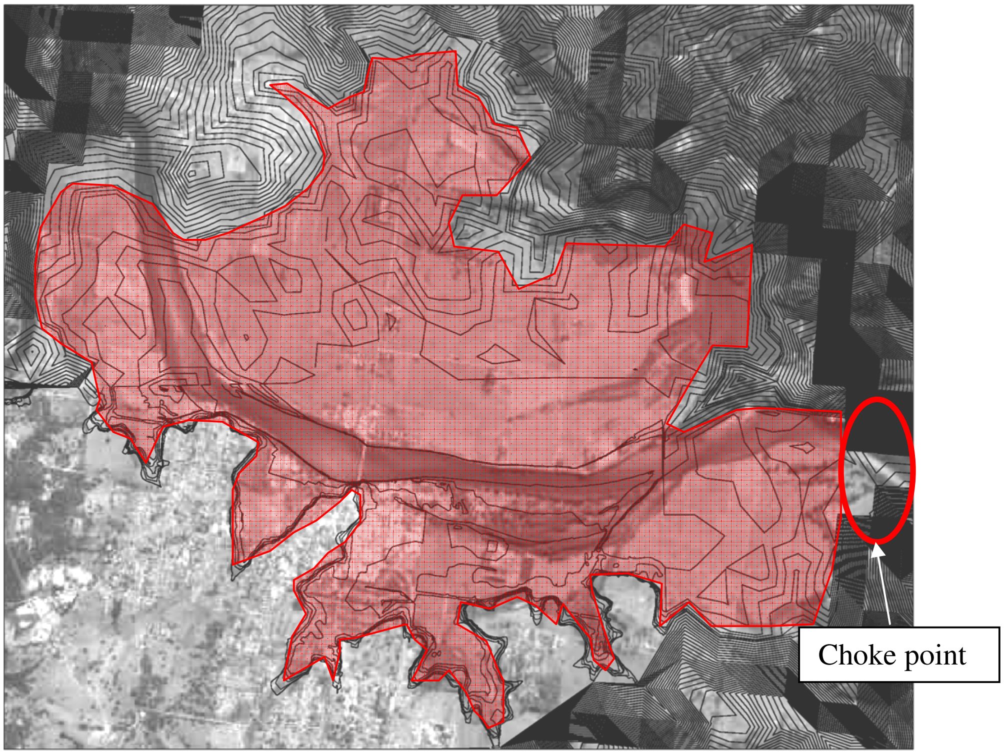


**Figure 5 Area of floodplain used for stage storage (terrain model combined Council data and Google earth data)**

In this way, additional volume associated with an increased ocean inundation component could be added to a freshwater flood and the associated flood level estimated using the stage storage relationship.

The tail water level of 2.0m AHD as used in the Moruya Flood Study was assumed as the base line for the addition of projected sea level rise.

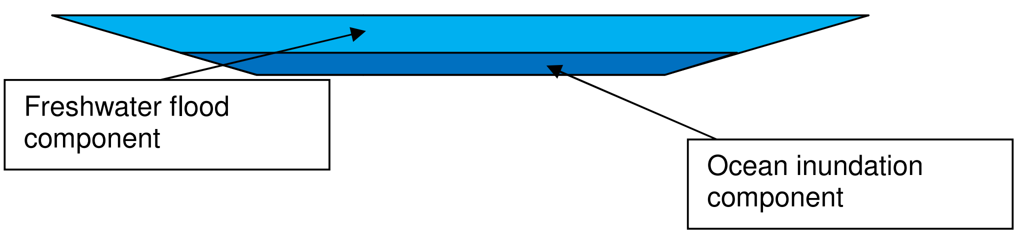


**Figure 6 Cross section schematic through floodplain**

## Comparison of Stage – Storage approach with Moruya Flood Study (1992) data

Given the limited accuracy of surface data, and the fact that the area in question would have some hydraulic gradient a comparison using the stage storage methodology was made with the Moruya Flood study (DPWS, 1992) sensitivity analysis results, where a tail water level of 2.4m AHD was tested with the hydraulic model.

The freshwater and ocean inundation volumes associated with the current 1% AEP level at the Moruya Bridge were determined based on the stage storage relationship. This is a tail water level of 2.0m AHD and flood level at the Moruya Bridge of 5.10m AHD.

Ocean inundation volume associated with 2.0m AHD was then subtracted from total to give "freshwater flood" component volume and associated stage (flood level). Then the volume associated with an ocean inundation level of 2.4m AHD was added to give a total flood volume for the area behind the 'choke' which was then transformed into a flood level using the stage storage relationship (Figure 7). The estimated flood level for a 1% event with a tail water level of 2.4m AHD is 5.3m AHD using the stage storage methodology.

The hydraulic model used in the Moruya Flood Study (DPWS, 1992) estimated a flood level of 5.2m AHD for the same combination of flood event and tail water level, a 0.1m difference, suggesting that the stage storage approach is a reasonable estimation, and conservative in this case.

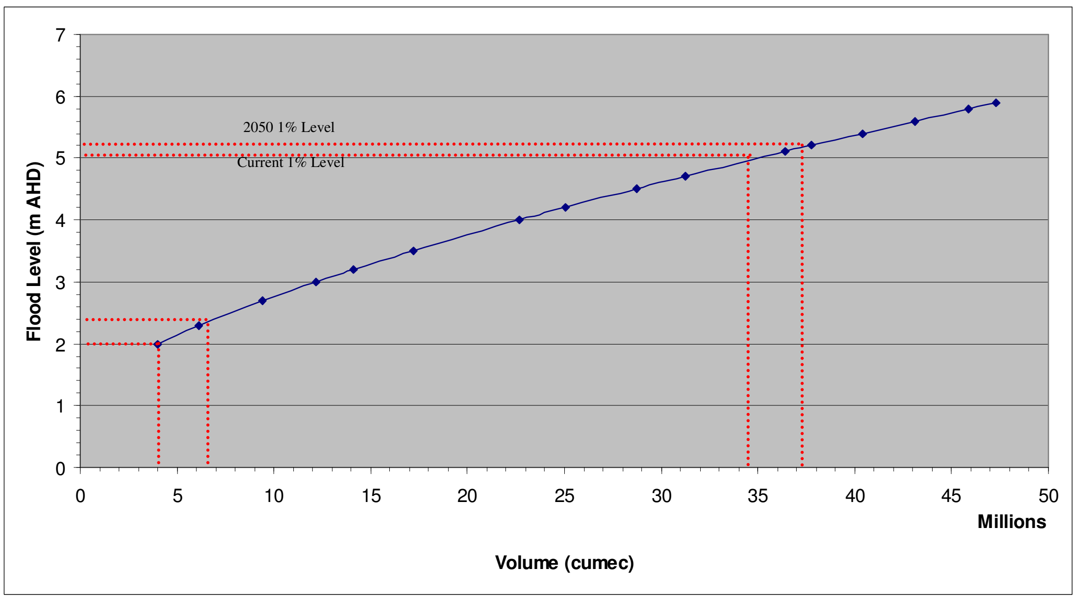


**Figure 7 Stage storage relationship for the area of the Moruya River floodplain behind the 'choke'**

Given that the approach outlined provided results which were within 2% of the Moruya Flood Study (DPWS, 1992) data at the Moruya Bridge, the methodology was then applied to estimated flood levels for a range of different recurrence events and both the predicted 2050 and 2100 sea level rise increases to then provide sufficient information to set flood planning levels for the development site. These levels were used to set conditions as part of the development approval.

## Comparison of stage storage results with updated Moruya Floodplain management data (Worley Parsons, 2011).

Since the flood level estimations were completed and the development approved, the Moruya Floodplain Management Study has been updated by Worley Parsons to incorporate sea level rise impacts using a 2 dimensional hydraulic model RMA2. This allowed us to compare our results from the stage storage approach with modeled results that incorporated sea level rise impacts.

Updating the hydraulic model from the original 1 dimension (1D) to 2 dimensions (2D) produced slightly different results. When like events were compared between the 1D model and 2D model, the 2D model calculated lower flood levels upstream of the Moruya Bridge and higher downstream of the bridge, particularly closer to the river entrance where the effects of the trained entrance had a significant impact. Flood levels at the Moruya Bridge itself hardly changed with the updated modeling approach (from 5.15 to 5.14m AHD).

When the results from the 2D model which incorporated climate change were compared to the stage storage approach it was clear that the simplified approach estimated higher flood levels. Table 6 compares the results for the 2 dimensional model and the stage storage model.

**Table 6 Comparison between updated flood study and stage storage approach**

| | 2D model (m AHD) | Stage storage model (m AHD) |
|---|---|---|
| 100 year flow and TW 2.40m AHD (2050 scenario) | 5.16 | 5.30 (+0.14) |
| 100 year flow and TW 2.90m AHD (2100 scenario) | 5.19 | 5.64 (+0.45) |

## Conclusion

Clearly, the stage storage approach for the Moruya floodplain above the ‘choke’ overestimates flood levels at the Moruya Bridge and surrounds. This is to be expected given that no allowance is made for the movement of the hydrograph through the system and it assumes coincidence of the freshwater hydrograph and ocean hydrograph over the entire area.

The results suggest, that for areas of limited hydraulic gradient the approach outlined in the present paper can provide a conservative estimation of the impacts of sea level rise for ocean influenced waterways at relatively low cost. This approach may be of use for estimating the impacts of future changes in sea level for the Moruya floodplain, and other low hydraulic gradient areas as a temporary, low cost solution until detailed modeling is undertaken.

## References

Department of Environment, Climate Change and Water 2009, *NSW Sea Level Rise Policy Statement*

Department of Public Works and Services1992, *Moruya River Flood Study*

Patterson Britton and Partners1996, *Moruya River Floodplain Management Study*

Worley Parsons 2011, *Moruya Flooding – Climate Change Assessment*