Lake Illawarra entrance channel: management options assessment

Volume 2: appendices

WRL TR 2024/07, November 2024

By I R Coghlan, B M Miller, A J Harrison, M Mason and T A Tucker

FINAL DRAFT

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Appendix A Numerical model development

A1 Preamble

To assess the hydraulic impacts of implementing various management options within the entrance channel, WRL developed an updated hydrodynamic model of the Lake Illawarra estuary using the modelling package RMA. This appendix provides an overview of the development and calibration of this updated model used in the Stage 2 options assessment.

A2 Modelling package

RMA-2 (version 9.0H; King, 2023) is a two-dimensional, depth-averaged finite element hydrodynamic model that solves the shallow water equations for water surface elevations and horizontal velocity components. It is suitable for simulation of flow in vertically well-mixed water bodies such as complex riverine environments (including the Lake Illawarra estuary). RMA-2 calculates a finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. The main internal model parameters applied to the model are eddy viscosity, bed friction and turbulent mixing. RMA-2 simulates the process of bank wetting and drying as the water level changes through the use of marshing elements. Marshing simulates drying by approximating elements with a smaller width and higher friction for water transfer thereby effectively preventing flow in those elements while conserving mass.

A3 Existing models

WRL (2019) had a previous two-dimensional RMA-2 model of Lake Illawarra which was developed in 2019 to assess the impact of proposed rock protection works beneath the northern end of Windang Bridge. This model was calibrated against tidal flow and water level data captured in September and October 2018. This model was used in Stage 1 of the options assessment (Coghlan et al., 2023).

MHL (2020) subsequently developed another two-dimensional RMA-2 model of Lake Illawarra in 2020 to further assess the impact of proposed rock protection works beneath Windang Bridge. This model was calibrated against tidal flow and water level data captured in February and March 2020. A copy of this model was provided to WRL for review to inform WRL's updated model of the estuary.

The mesh, boundary condition and friction parameters from these two models was considered in the development of WRL's updated model for Stage 2 of the options assessment.

A4 Model extent and mesh generation

The hydrodynamic model extends from an ocean boundary and includes the entire surface area of the lake. Catchment inflow channels (Macquarie Rivulet, Duck Creek, Brooks Creek and Mullet Creek) were built out to the tidal extent. The model extent and mesh are shown in Figure A-1. As velocities in the lake are very small, the majority of the lake is represented by a coarse numerical grid. However, in the entrance channel the model resolution is significantly increased to represent the complex bathymetry and tidal flow dynamics. This irregular model mesh was based on the model used by WRL in the Lake Illawarra Preliminary Management Options Assessment by Coghlan et al. (2023), with significant

refinements in the channel to accurately represent the 2021 bathymetry without unnecessary resolution (to optimise model run time).

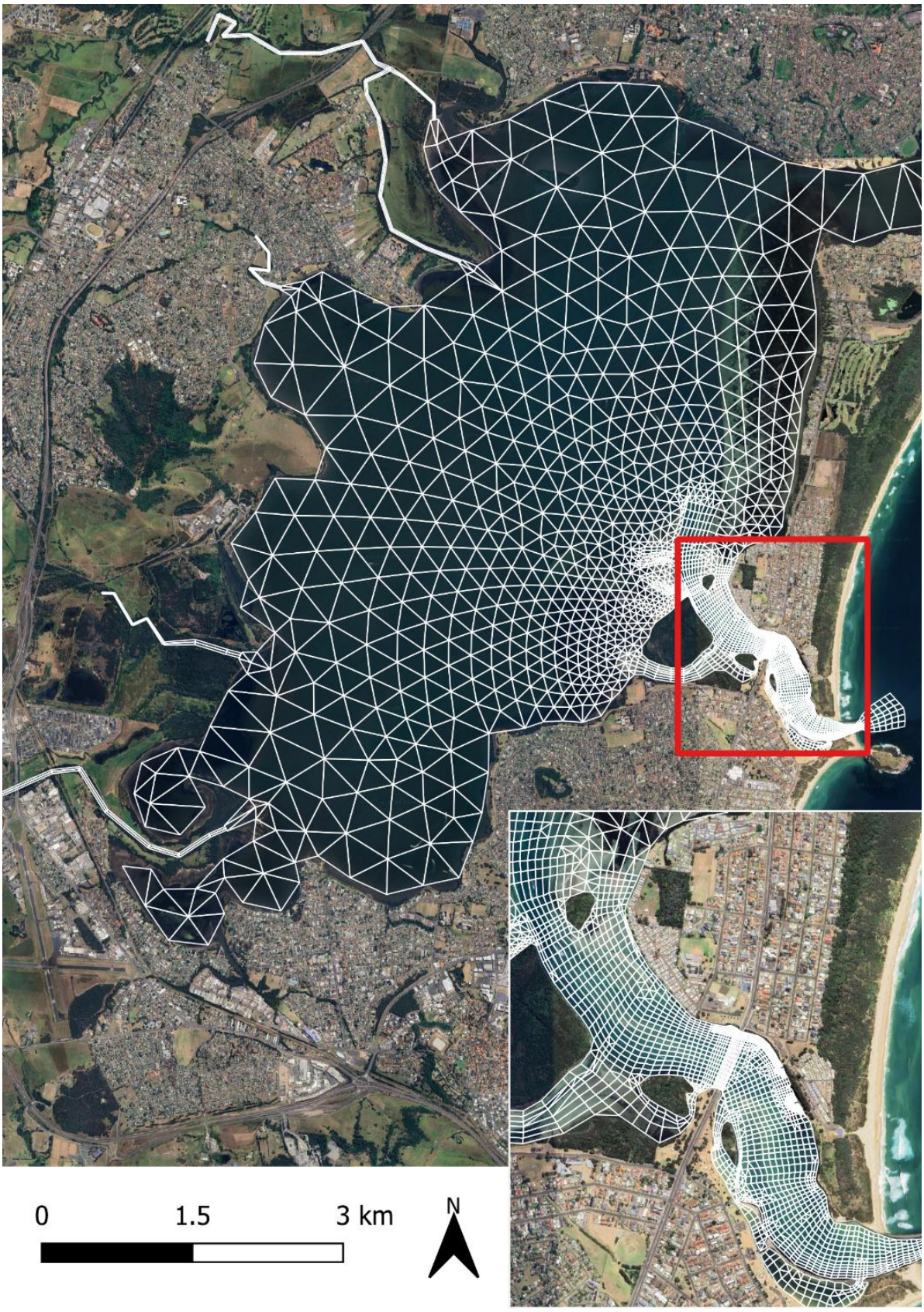


Figure A-1 Numerical model extent and mesh

A5 Bathymetry

Three sets of bathymetric surveys were used in constructing this model:

- Bathymetry collected for DCCEEW (formerly) DPE in the main lake and tributaries between 29 August 2022 and 2 June 2023, shown in Figure A-2
- Bathymetry collected by WRL in between 29 November 2021 and 3 December 2021 covering the entrance channel and flood and ebb deltas, shown in Figure A-3
- Bathymetry collected by Transport for NSW on 16 December 2021 covering the area underneath and adjacent to the bridge, shown in Figure A-3 (inset)

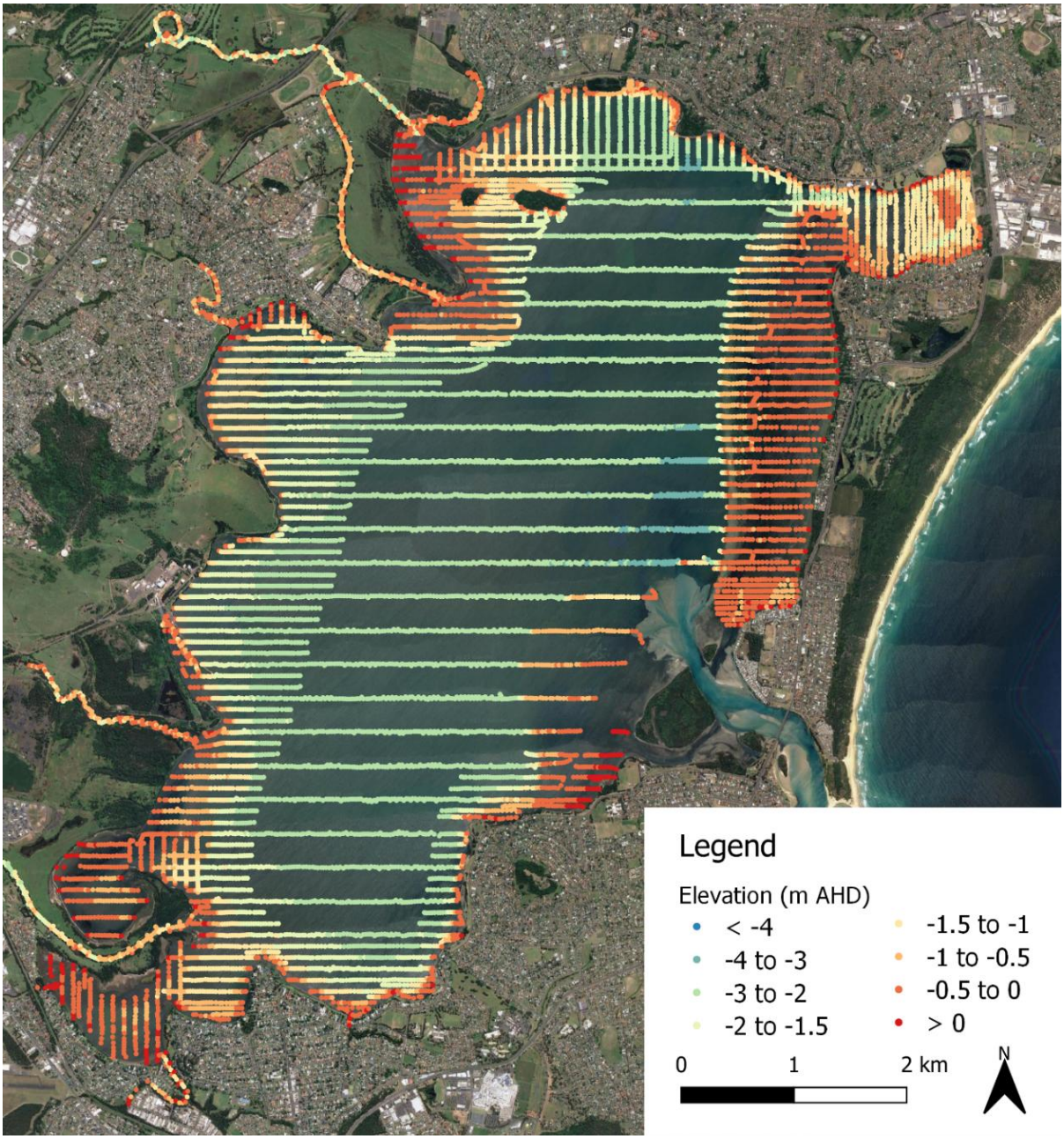


Figure A-2 Main lake bathymetry collected for DPE between 29 August 2022 and 2 June 2023

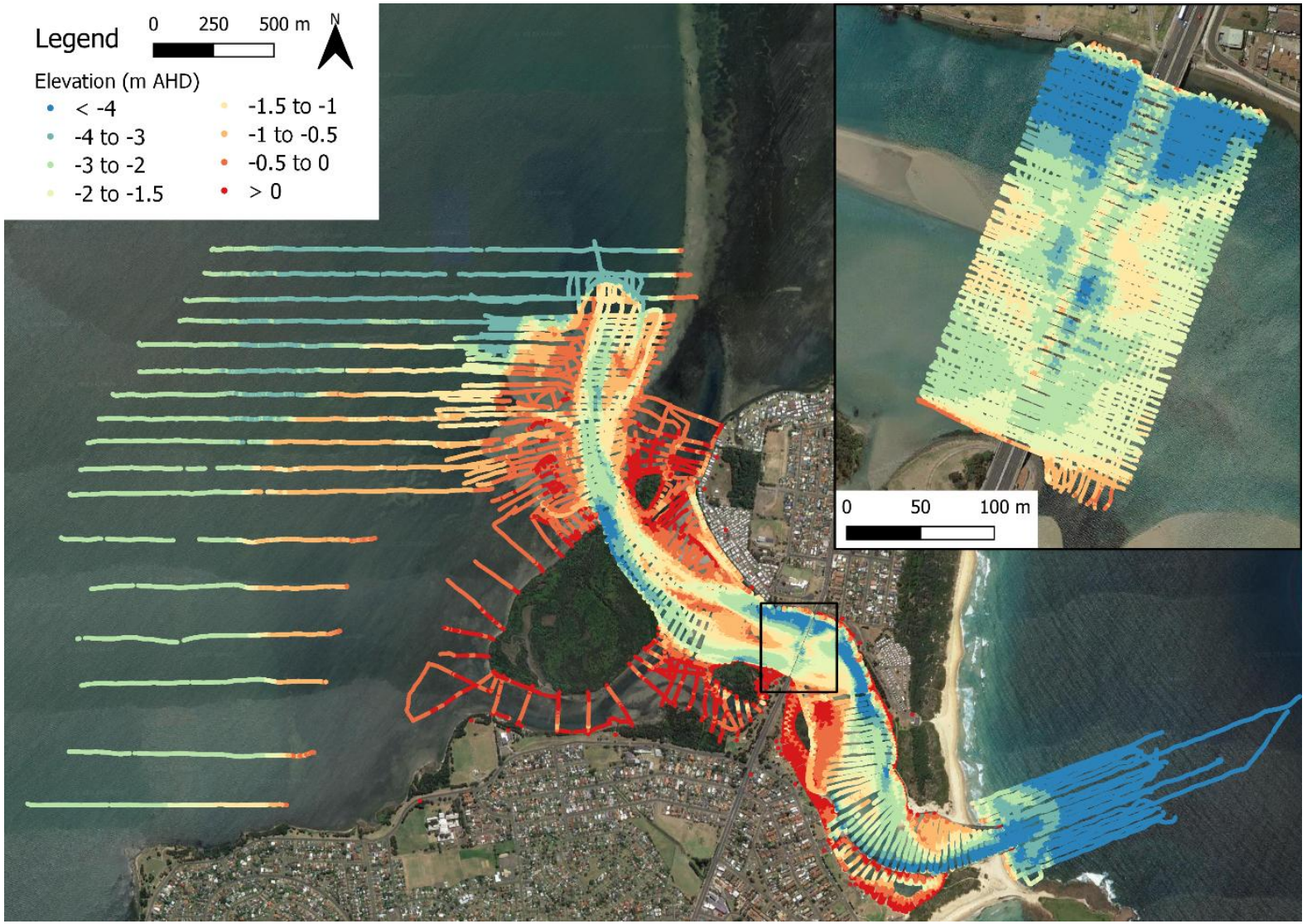


Figure A-3 Entrance channel bathymetry collected by WRL between 29 November 2021 and 3 December 2021 (main map) and under bridge bathymetry collected by Transport for NSW on 16 December 2021 (inset)

A6 Model simulation period

The model had a 14 day warm up period (17 November 2021 to 30 November 2021) and a 90 day calibration period (1 December 2021 to 28 February 2022). The calibration period was selected as it included larger spring tide conditions and corresponded with co-incident bathymetry collection by WRL and tidal gauging and water level data collection by MHL (described in Tucker et al., 2023a and MHL, 2023). The model was run on a 15 minute timestep.

A7 Boundary conditions

A7.1 Offshore tide

The downstream boundary of the model is the full ocean tide immediately offshore of the training walls. These boundary conditions are the primary driver of the model, determining the tidal flows through the entrance channel. There are several coastal tidal level monitoring stations that could be used as the oceanic boundary condition at Lake Illawarra including Port Kembla (operated by National Tidal Centre [NTC] / Bureau of Meteorology [BoM] station number 068253; approximately 9 km away), Jervis Bay (operated by MHL; approximately 66 km away) and Bundeena (operated by MHL; approximately 58 km away).

Atmospheric pressure, wind and wave setup, latitude and longitude and the geometry of the coast all have effects on the tide, and create the differences between observations at these stations. Due to its close proximity to the Lake Illawarra entrance, atmospheric pressure and latitude should not be an issue when using the Port Kembla tide gauge, however as it is located inside the port, there may still be effects from energy losses at the entrance of Port Kembla, or wave and wind setup from certain directions, which do not affect the tides driving the Lake Illawarra entrance in the same way. Both the Jervis Bay and Bundeena tidal gauges also offer a reasonable fit to the observed data, and would be satisfactory driving tides for the model, however, there are noticeable anomalies on occasional tides which differ from Lake Illawarra and Port Kembla. For example, a comparison between the model driven by Port Kembla and Bundeena boundary conditions and the observed data is shown in Figure A-4. On the basis that, of the three monitoring stations, the Port Kembla tidal gauge offered the best fit with observed water levels it was adopted for all subsequent tidal simulations.

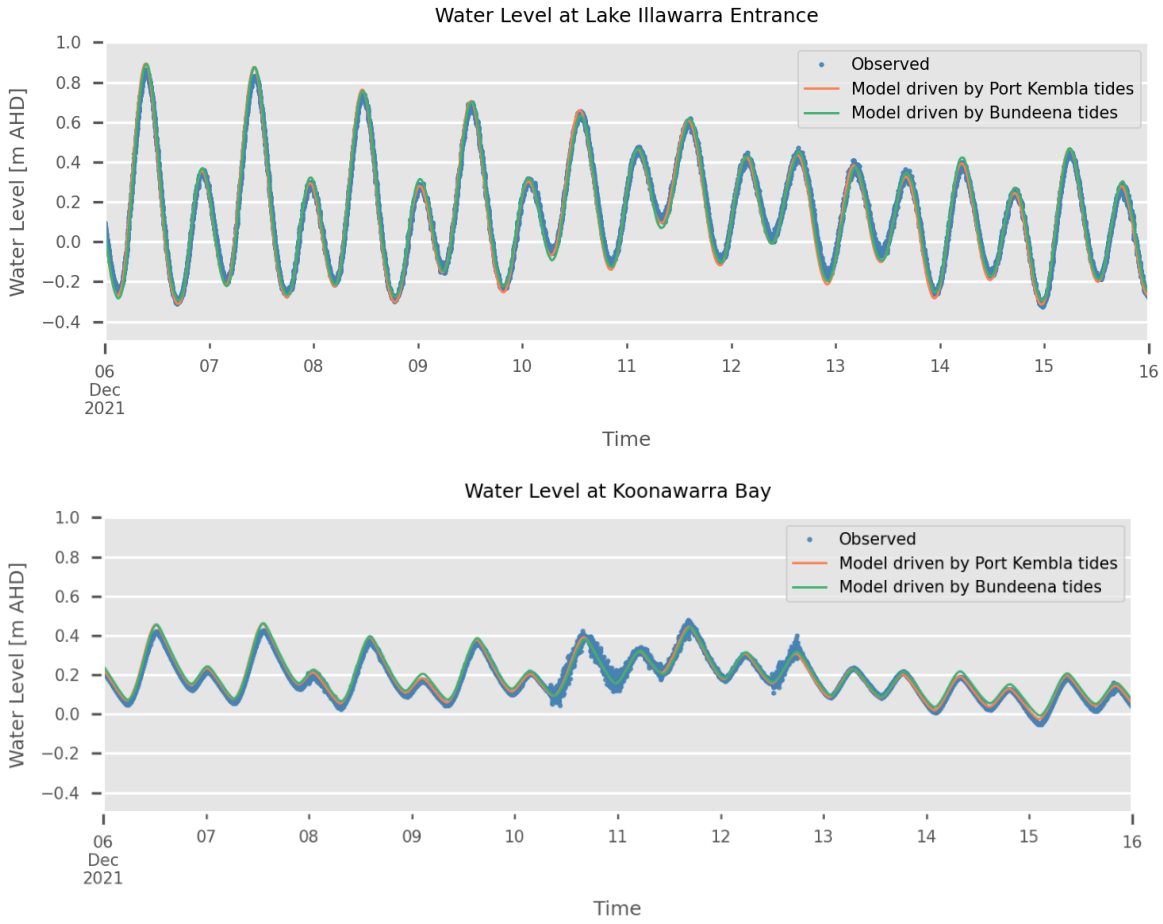


Figure A-4 Modelled results at Lake Illawarra Entrance (upper) and Koonawarra Bay, in the lake, (lower) using Port Kembla gauge as the driving tides (orange) and Bundeena (green)

A7.2 Other boundary conditions

There were no catchment inflows or rainfall included in the model simulations. Review of the daily rainfall at Albion Park (Figure A-5) indicates that this was a reasonable assumption for the model warm up and calibration period except for a large rainfall event at the end of February 2022.

Since the RMA-2 numerical model is a fixed bed type model, the bathymetry in the model does not evolve throughout the simulations; that is, there are no morphological changes (e.g. sediment erosion, sediment deposition/accretion or the supply of marine sediment into the entrance).

There were also no waves (wind waves, swell waves or tsunami) included in the model simulations.

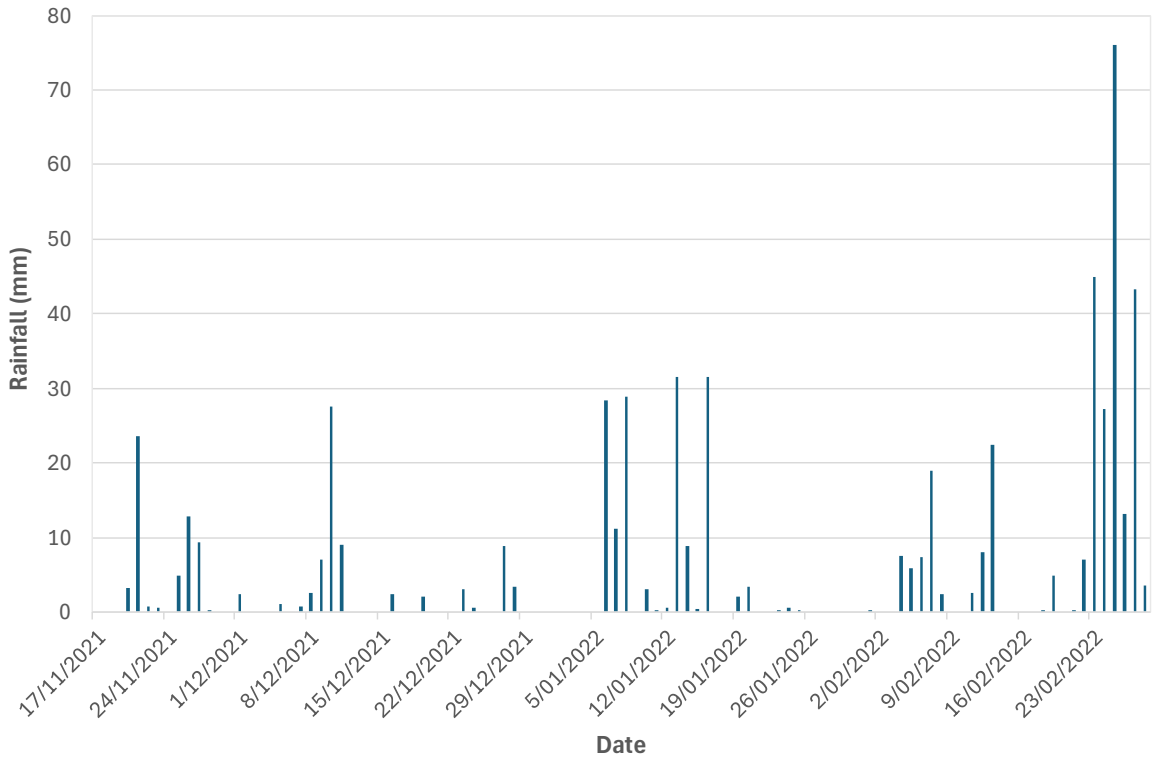


Figure A-5 Rainfall at Albion Park (Shellharbour Airport) BOM gauge 068241 for 14 day warm up period (17/11/2021 – 30/11/2021) and 90 day calibration period (01/12/2021 – 28/02/2022)

A8 Hydrodynamic calibration (December 2021 – February 2022)

A8.1 Parameters

Calibration involved refining the model geometry (i.e. mesh resolution and shape) and parameters (predominantly bed friction) to achieve a good fit to the observed data. The model calibration used a Manning's roughness (n) of 0.025 in the lake and entrance channel with an area approximately 65 m wide under the bridge having a higher roughness of 0.08. This higher roughness simulated greater losses under the bridge due to roughness caused by friction of the rock armouring, and losses caused by the bridge pylons. For comparison, the previous WRL model (WRL, 2019) used a Manning's roughness of 0.02 to 0.022 in the main channel, 0.04 in shallow side channels and 0.08 under the bridge. The previous MHL model (MHL, 2020) used a roughness of 0.02 in the entrance channel (including under the bridge) and 0.025 in the lake.

A8.2 Gauging locations

Model calibration was conducted using the flow data collected over one full spring tidal cycle (ebb and flood) on 6 December 2021 and water level data available from four temporary water level gauges from 1 December 2021 to 28 February 2022, as well as four permanent water level gauges operated by MHL (described in Tucker et al., 2023a and MHL, 2023). These locations are shown in Figure A-6 and are tabulated in Table A-1.

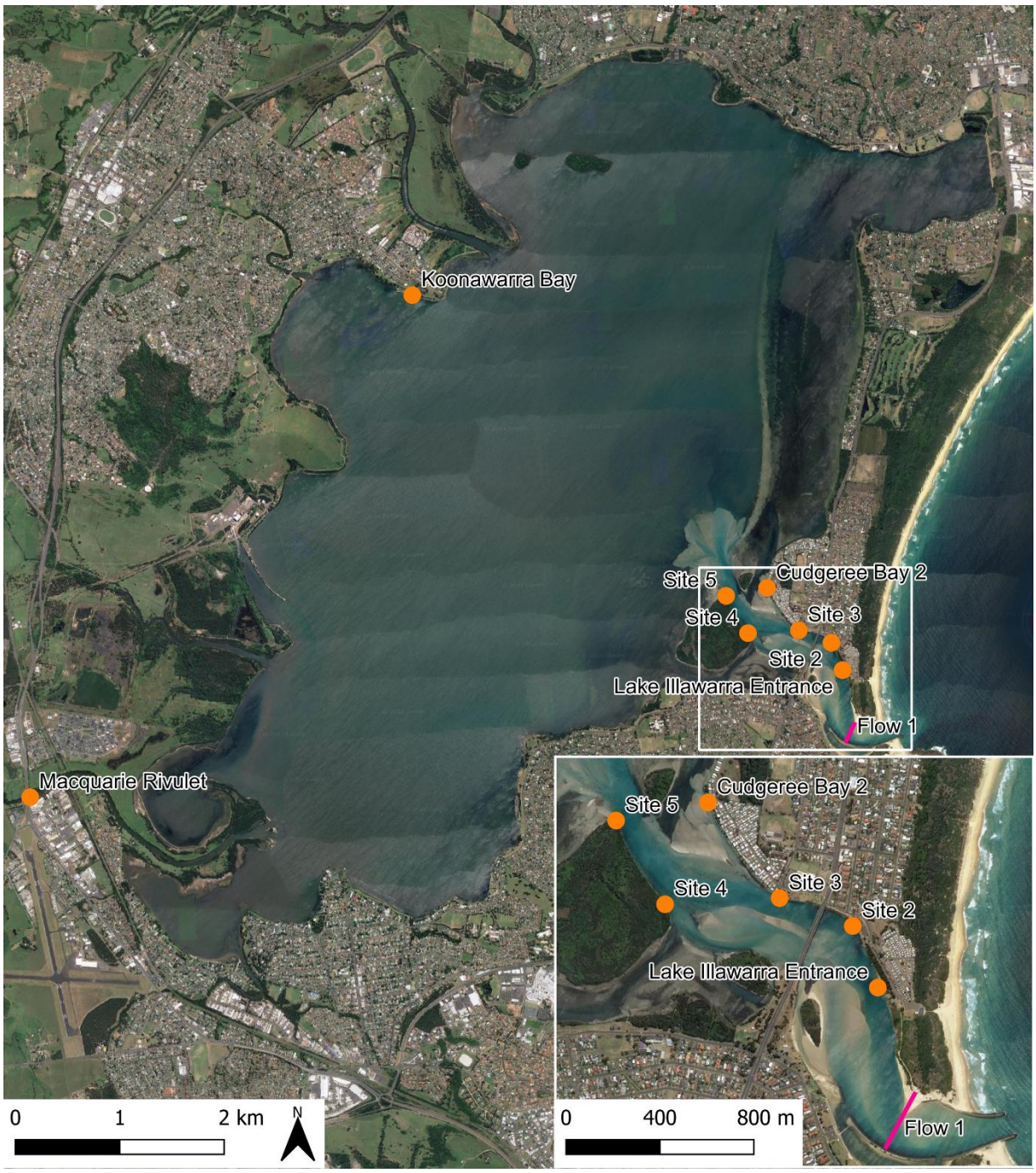


Figure A-6 Water level and flow gauging locations for the 1 December 2021 to 28 February 2022 model calibration period

Table A-1 Summary of flow gauging and water level data used for model calibration

Name	Type	Time Period
Flow 1	MHL flow gauging campaign	6/12/2021
Lake Illawarra Entrance	MHL permanent water level gauge	1/12/2021 to 28/02/2022
Cudgeree Bay 2	MHL permanent water level gauge	1/12/2021 to 28/02/2022
Koonawarra Bay	MHL permanent water level gauge	1/12/2021 to 28/02/2022
Macquarie Rivulet	MHL permanent water level gauge	1/12/2021 to 28/02/2022
Site 2	MHL temporary water level gauge	1/12/2021 to 28/02/2022
Site 3	MHL temporary water level gauge	1/12/2021 to 28/02/2022
Site 4	MHL temporary water level gauge	2/12/2021 to 28/02/2022
Site 5	MHL temporary water level gauge	1/12/2021 to 28/02/2022

A8.3 Results

Figures A-7 to A-15 show a 3 day view of the calibration period (tidal flow and water levels) during the period of tidal flow gauging, while Figures A-16 to A-23 show the full 3 month water level calibration period. As can be seen from these results, a good fit is achieved, although the tidal range at most sites is slightly too large, especially at the peak of large high tides. This likely is due to an underestimation of losses in the vicinity of Site 2, possibly due to inaccuracies estimating the losses caused by bridge pylons at higher water levels using bed friction. Nevertheless, the model has achieved a more accurate fit to observed levels than previous models and is fit for the purpose of assessing the hydraulic impacts of implementing of management options within the entrance channel.

The tsunami generated by the Hunga Ha-apai (Tonga) eruption on 15 January 2022 can be seen in the water levels observed in mid-January (Figures A-16 to A-23), however the model does not include this.

The peak that can be seen in the Macquarie Rivulet observed data at the end of February (Figure A-23) was due to a large rainfall event near the end of the month (see Figure A-5). The model fails to represent this peak as it does not include catchment inflows, since the majority of the calibration period was dry enough that inflows did not affect lake water levels.

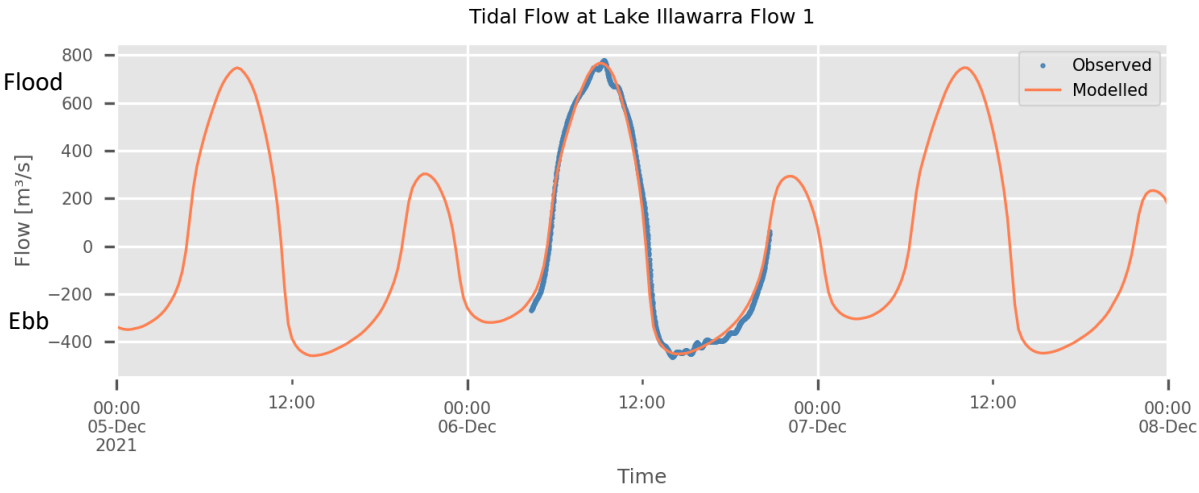


Figure A-7 Tidal flow calibration results at Flow location 1 for 3 day subset – upstream flow (flood tide is positive)

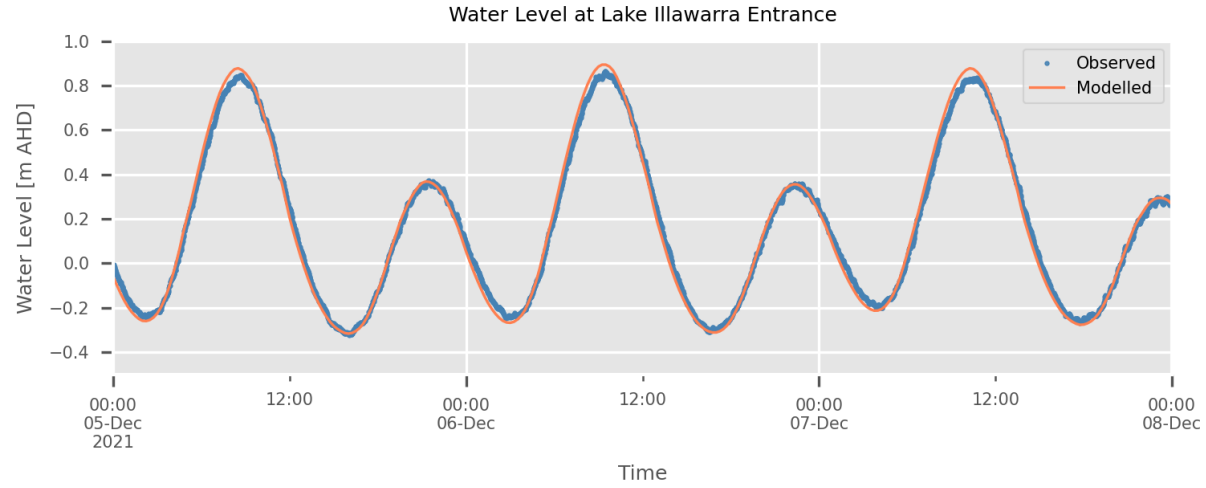


Figure A-8 Water level calibration results at Lake Illawarra Entrance for 3 day subset

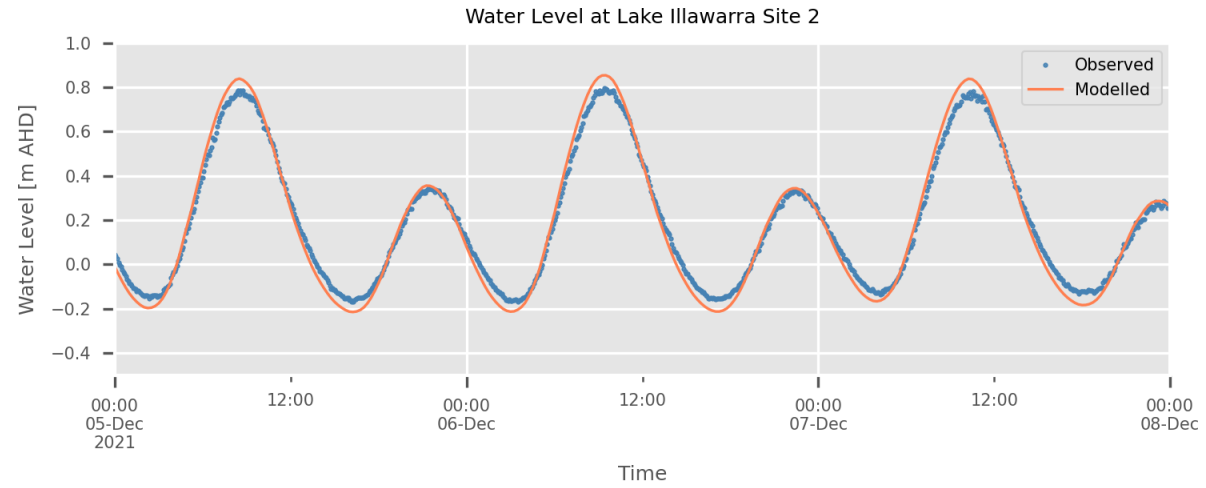


Figure A-9 Water level calibration results at Site 2 for 3 day subset

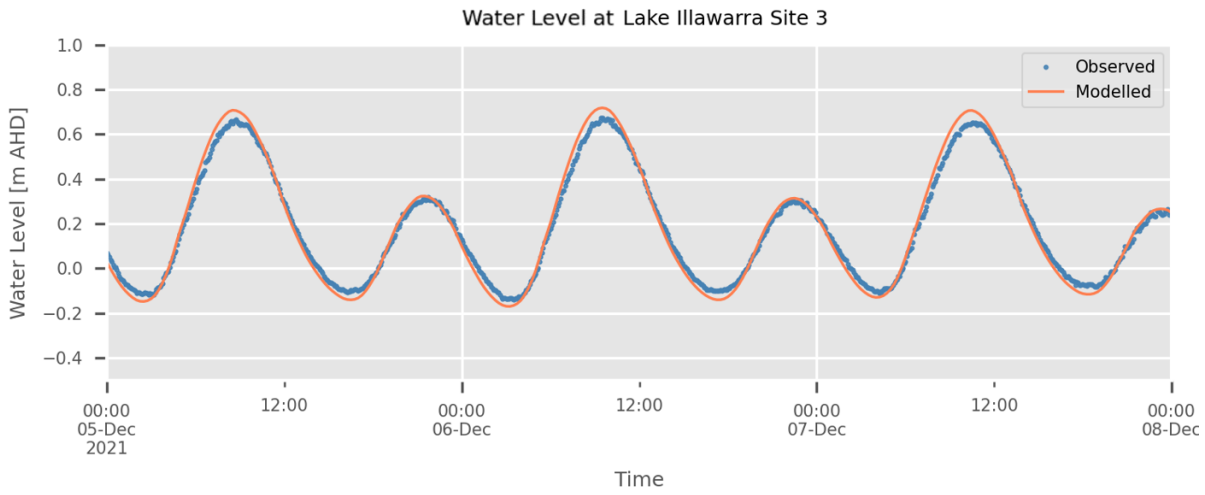


Figure A-10 Water level calibration results at Site 3 for 3 day subset

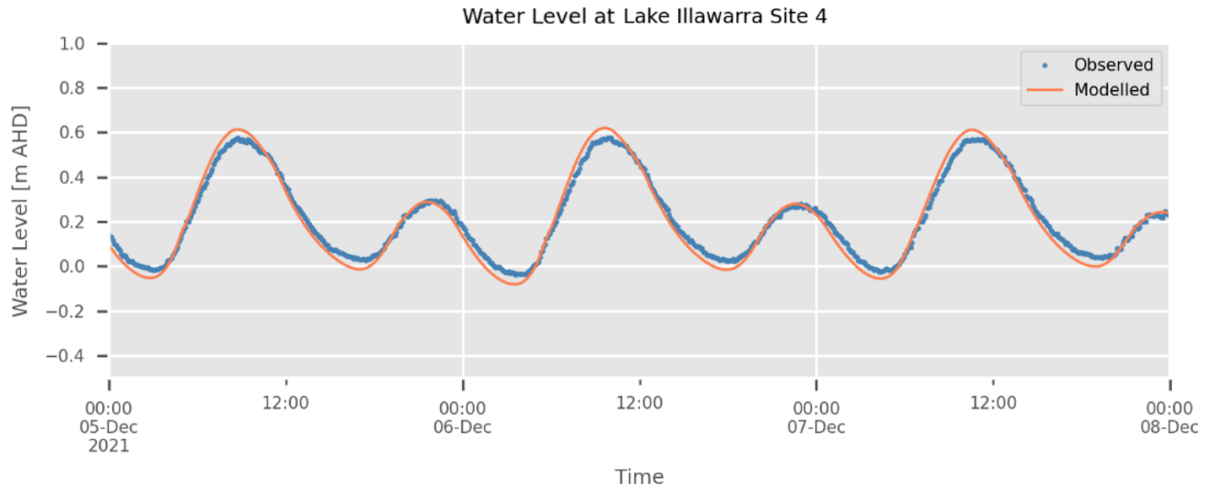


Figure A-11 Water level calibration results at Site 4 for 3 day subset

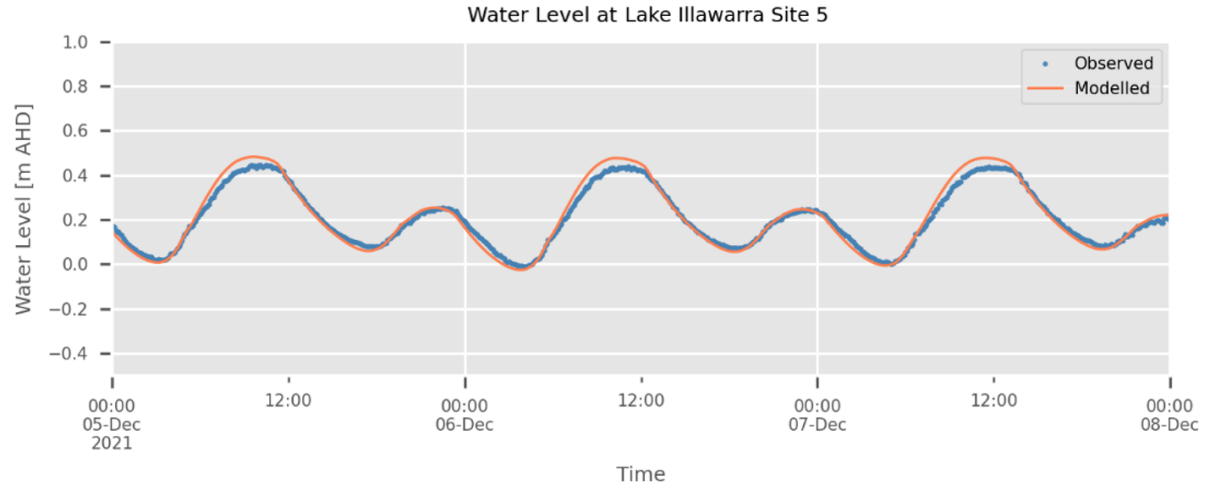


Figure A-12 Water level calibration results at Site 5 for 3 day subset

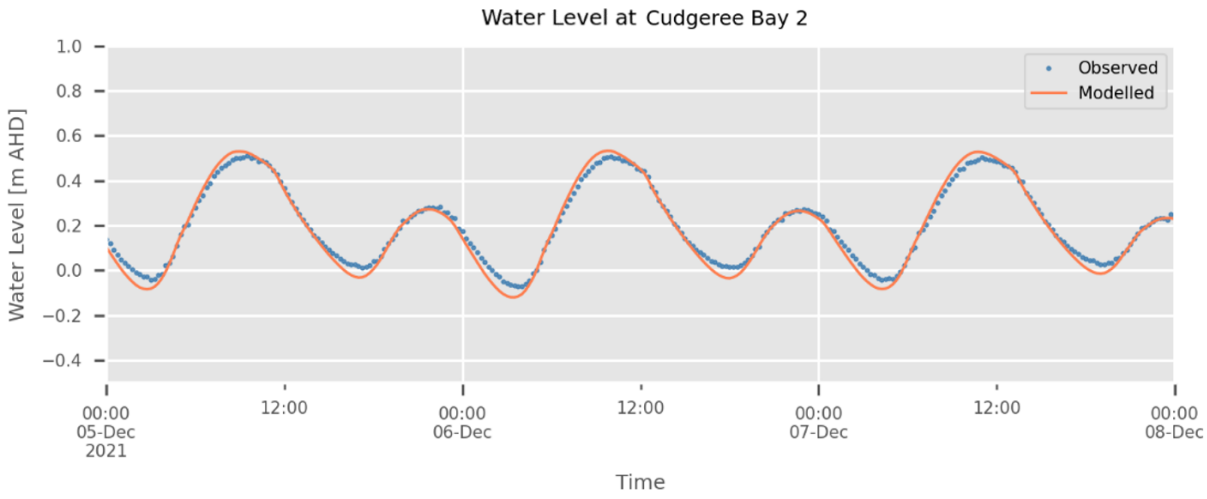


Figure A-13 Water level calibration results at Cudgeree Bay for 3 day subset

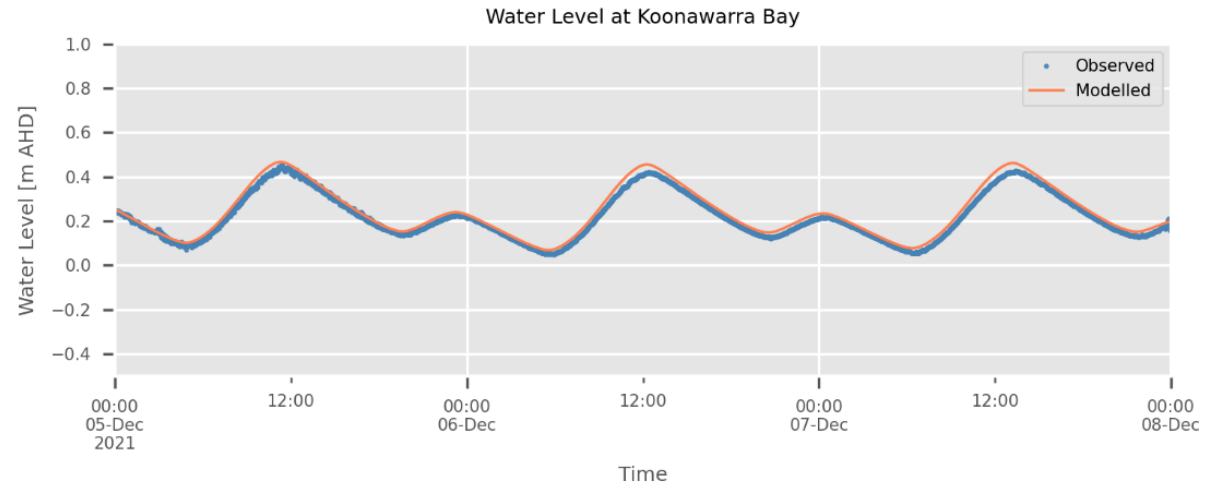


Figure A-14 Water level calibration results at Koonawarra Bay for 3 day subset

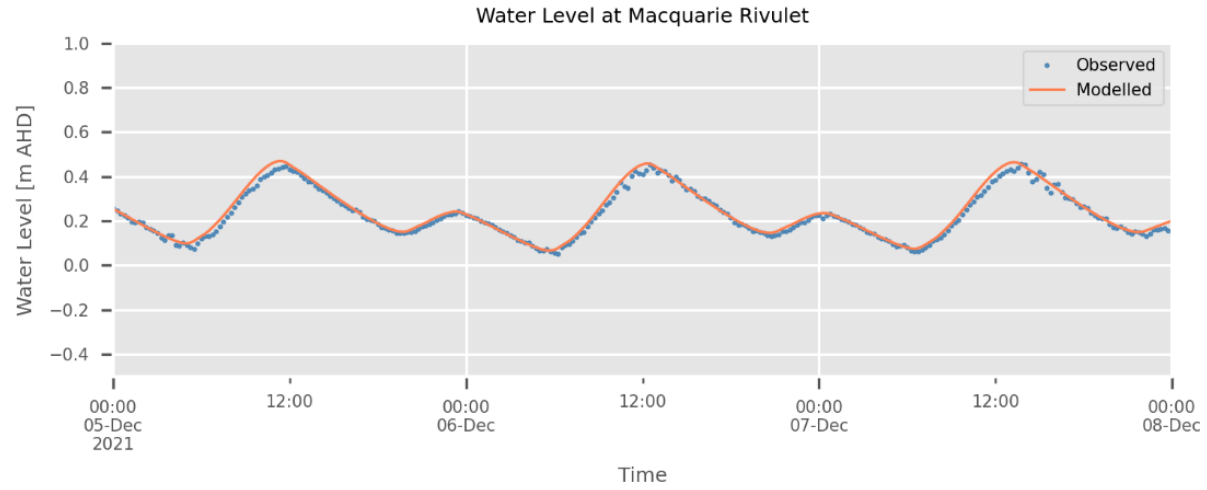


Figure A-15 Water level calibration results at Macquarie Rivulet for 3 day subset

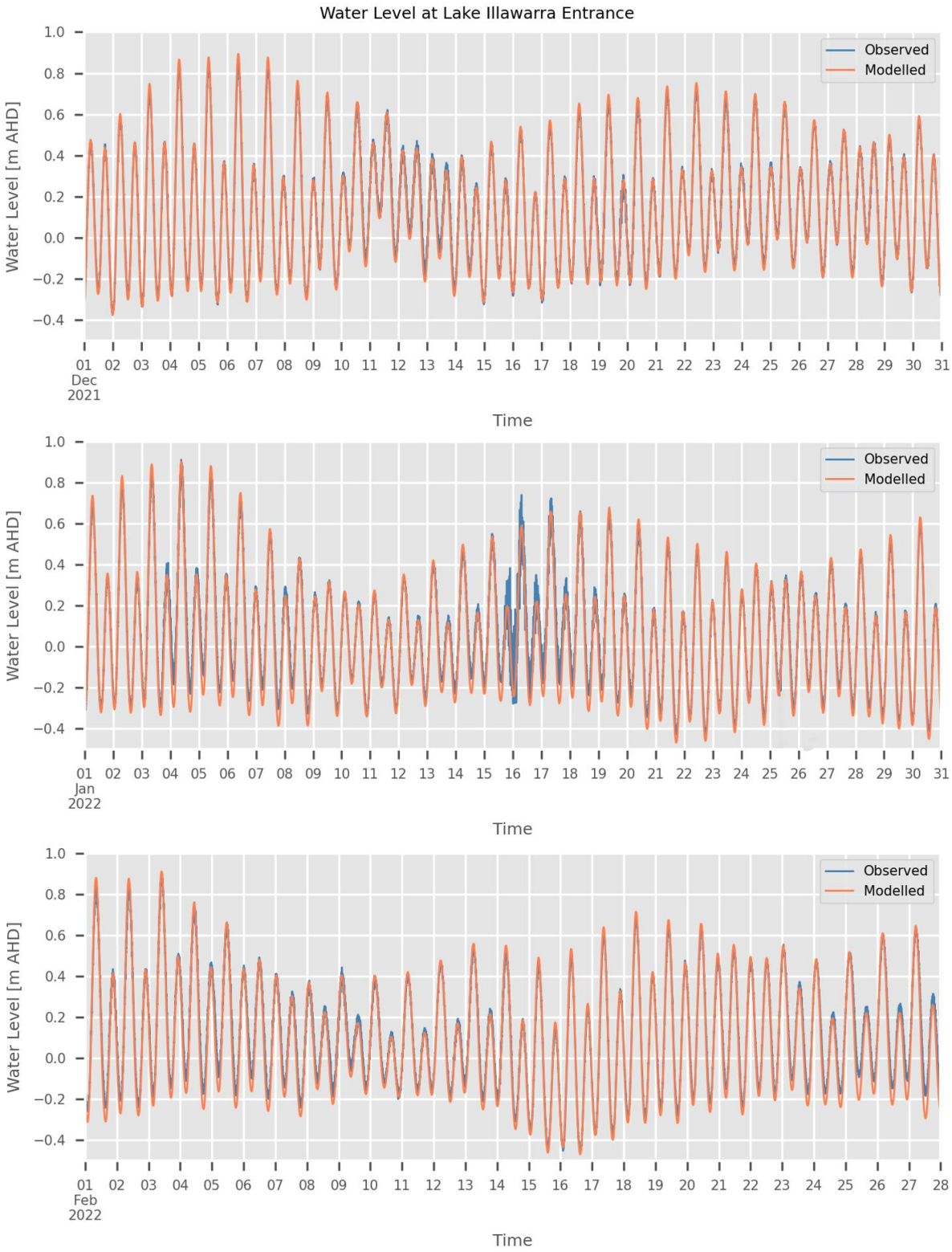


Figure A-16 Water level calibration results at Lake Illawarra Entrance for full 3 month calibration period

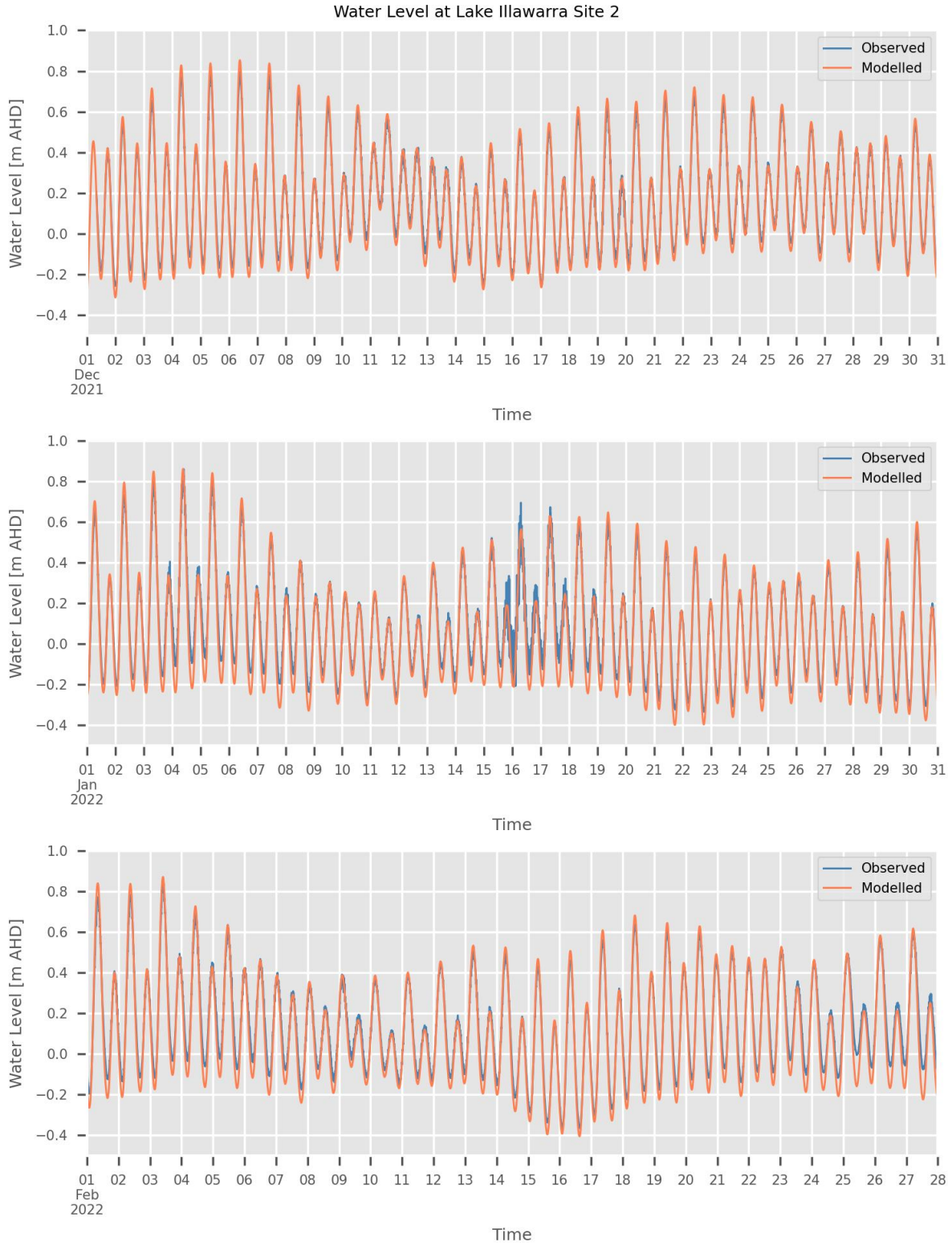


Figure A-17 Water level calibration results at Site 2 for full 3 month calibration period

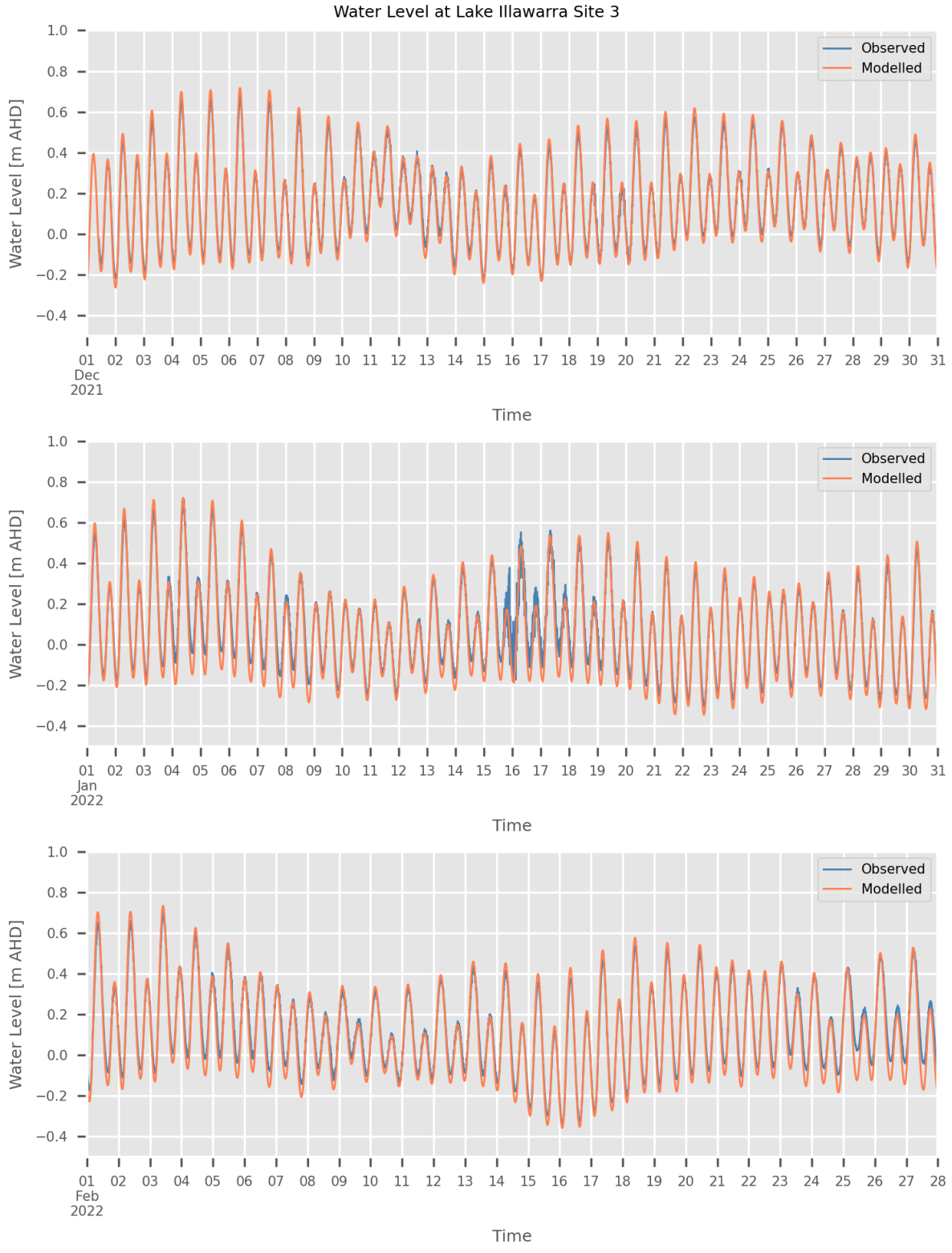


Figure A-18 Water level calibration results at Site 3 for full 3 month calibration period

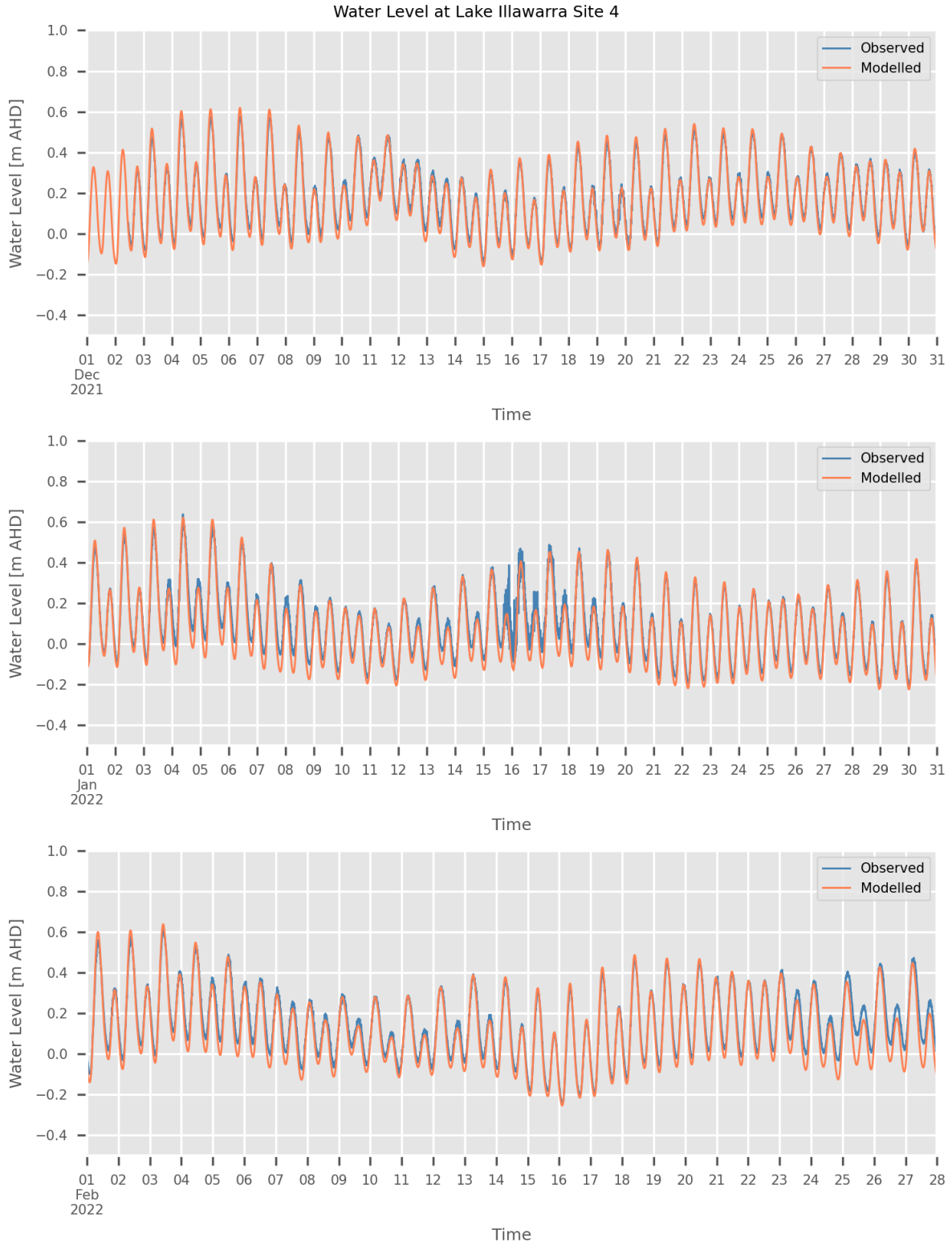


Figure A-19 Water level calibration results at Site 4 for full 3 month calibration period

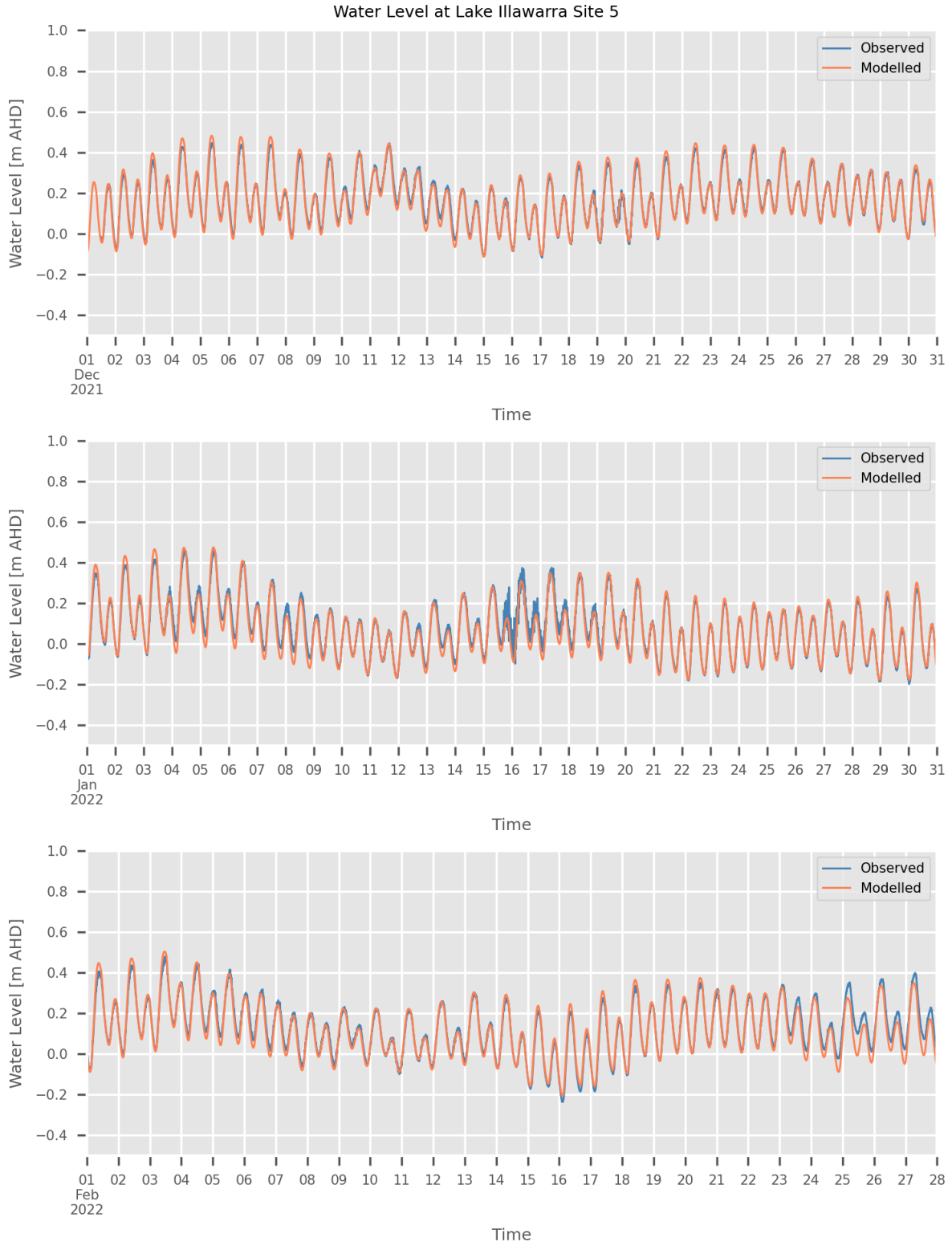


Figure A-20 Water level calibration results at Site 5 for full 3 month calibration period

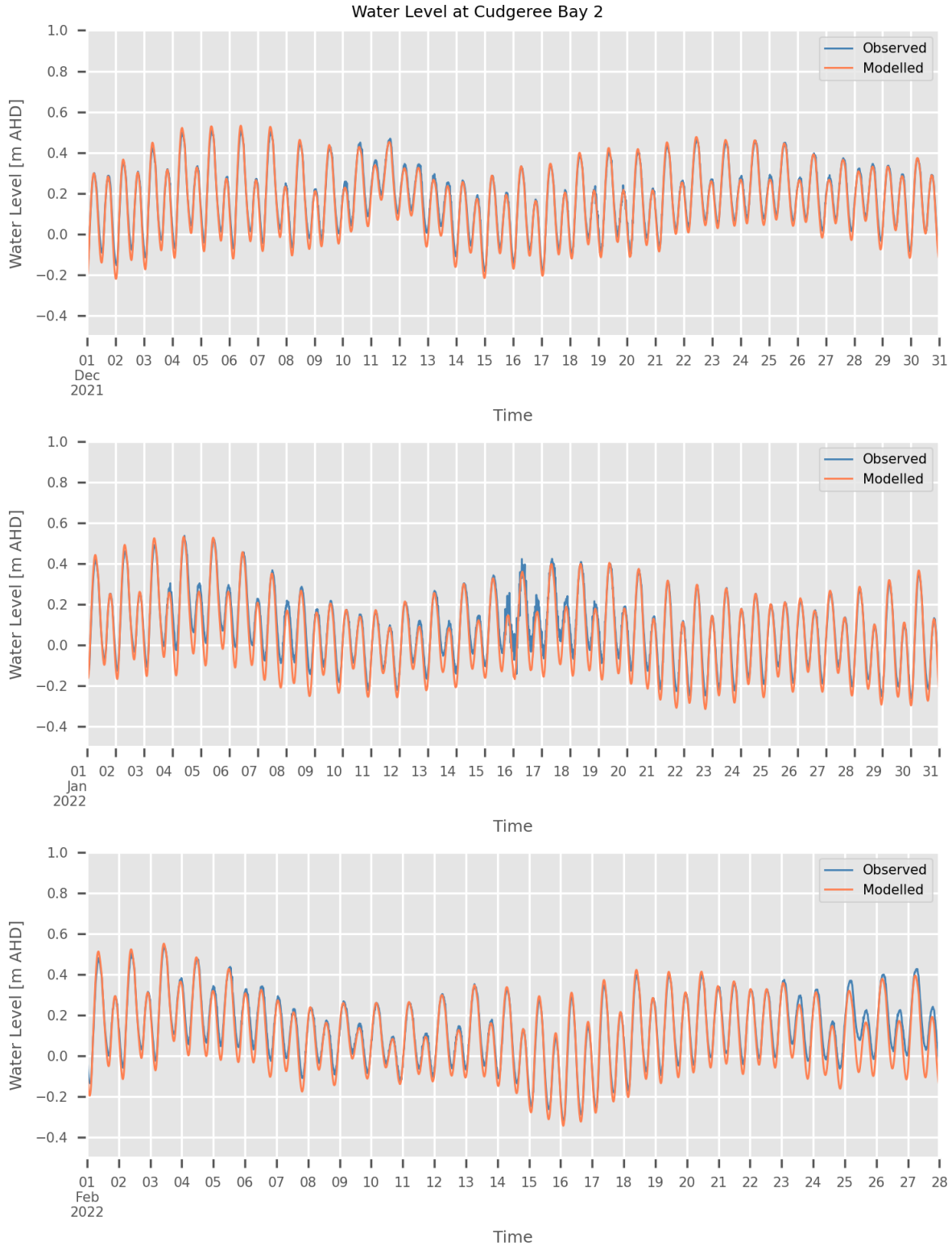


Figure A-21 Water level calibration results at Cudgeree Bay for full 3 month calibration period

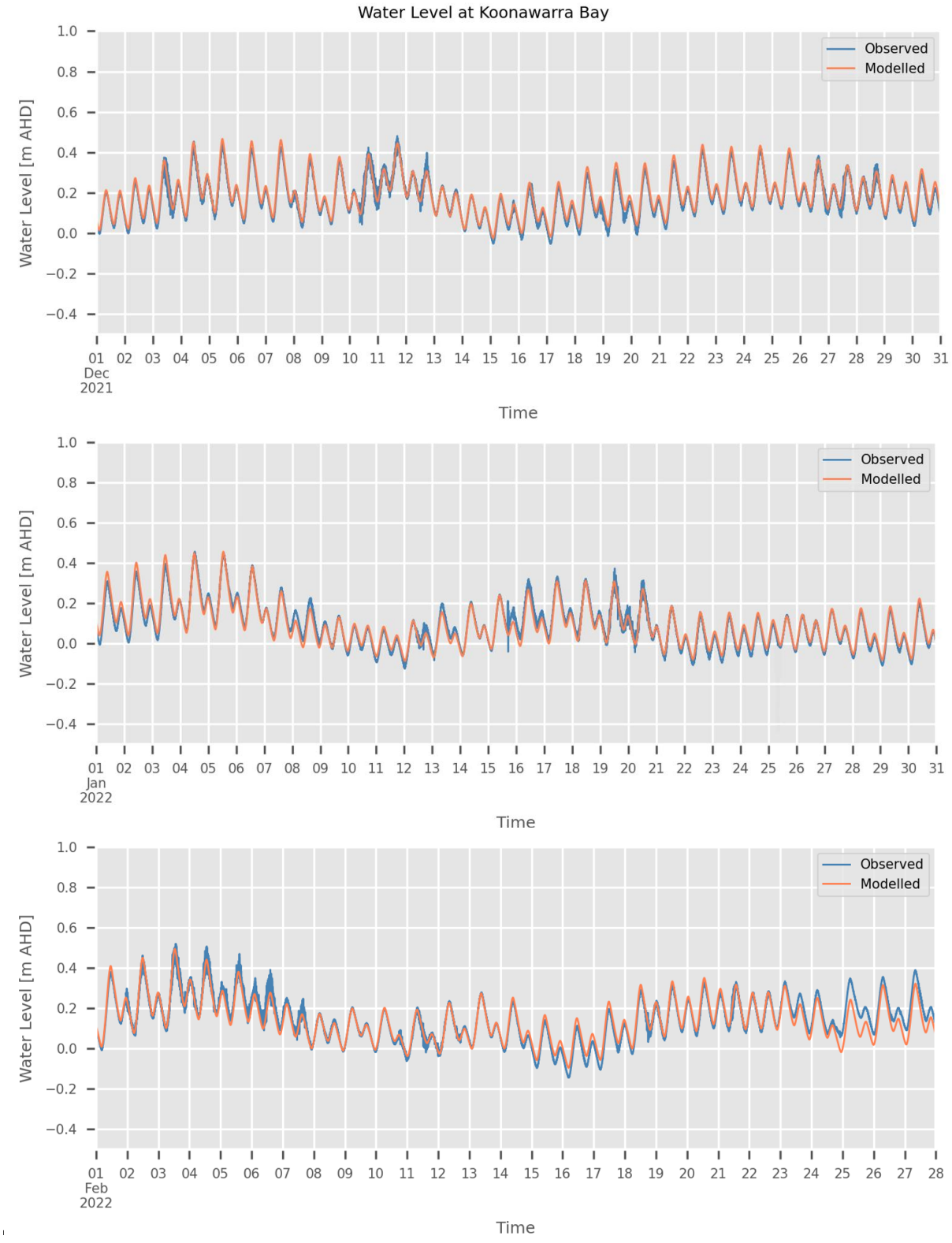


Figure A-22 Water level calibration results at Koonawarra Bay for full 3 month calibration period

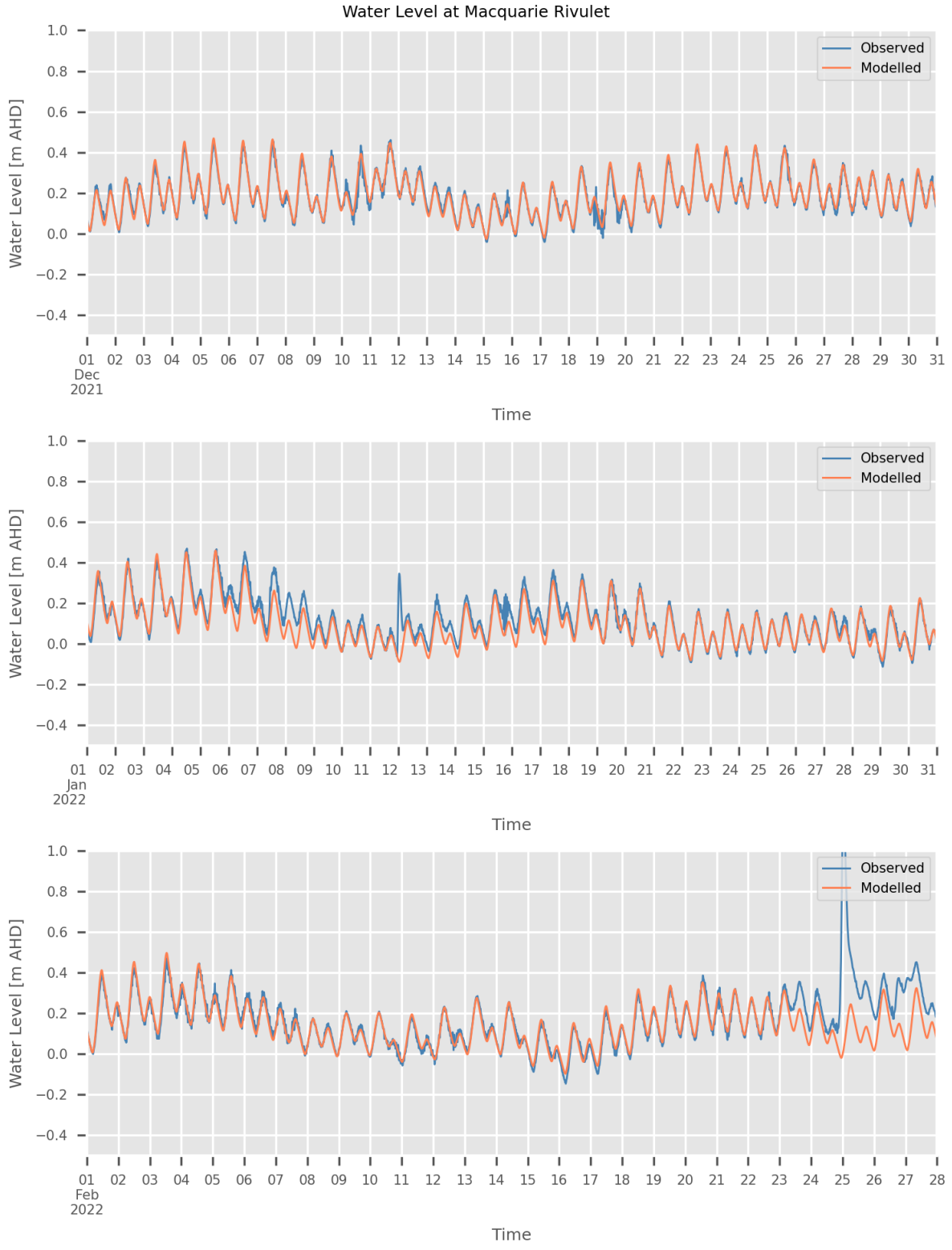


Figure A-23 Water level calibration results at Macquarie Rivulet for full 3 month calibration period

Appendix B Modelled management option cross-sections

B1 Option 1: Rock weir beneath Windang Bridge

Option 1 consists of a rock weir located beneath Windang Bridge as shown in Figure 3.1. Ten different geometries for the weir were modelled, as follows:

- Option 1A: -0.5 m AHD weir with one 100 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes (Figure B-1)
- Option 1B: -0.5 m AHD weir with two 50 m wide channels (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes (Figure B-2)
- Option 1C: -0.5 m AHD weir with one 50 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes (Figure B-3)
- Option 1D: -0.5 m AHD weir with one 25 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes (Figure B-4)
- Option 1E: 0 m AHD weir with one 50 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes (Figure B-5)
- Option 1F: -0.5 m AHD weir all the way across (Figure B-6)
- Option 1G: 0 m AHD weir all the way across (Figure B-7)
- Option 1H: 0.1 m AHD weir all the way across (Figure B-8)
- Option 1I: 0.25 m AHD weir all the way across (Figure B-9)
- Option 1J: 0.5 m AHD weir all the way across (Figure B-10)

All Option 1 weirs had an adopted crest length (i.e. the dimension in the along stream direction) of 5 m.

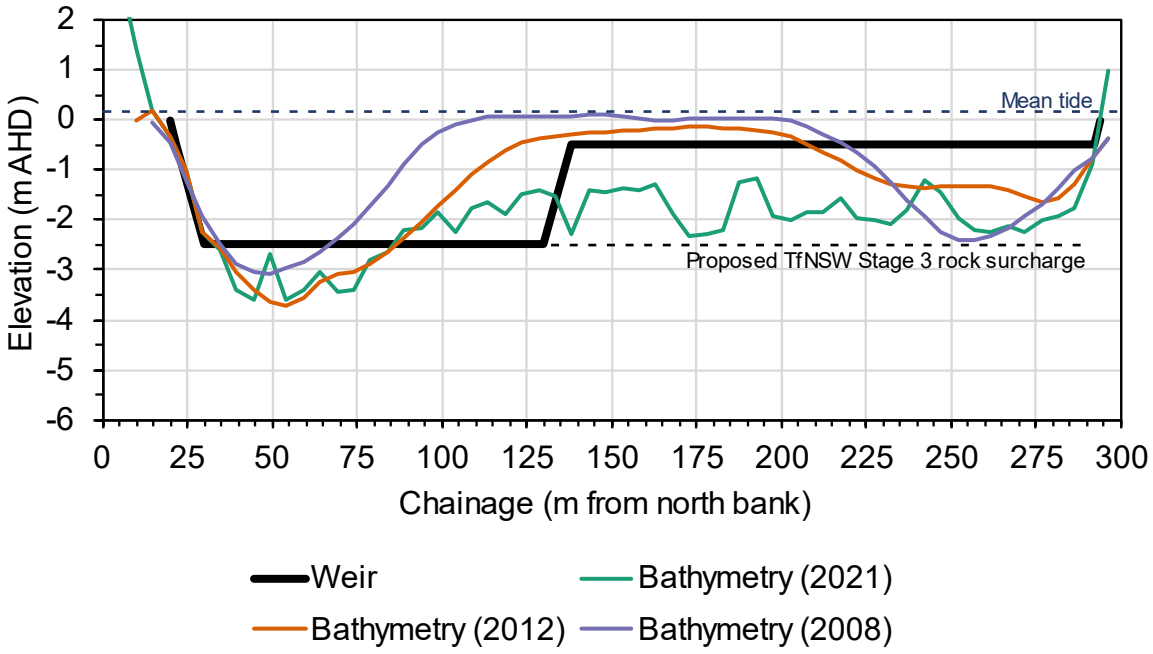


Figure B-1 Option 1A: -0.5 m AHD weir with one 100 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes

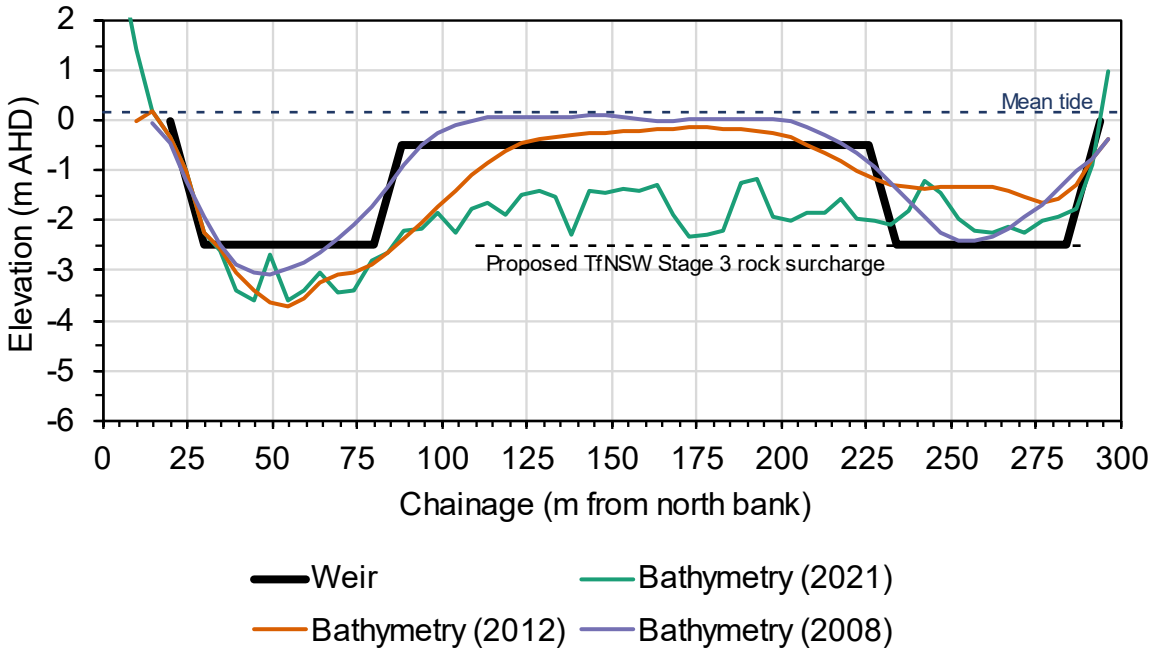


Figure B-2 Option 1B: -0.5 m AHD weir with two 50 m wide channels (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes

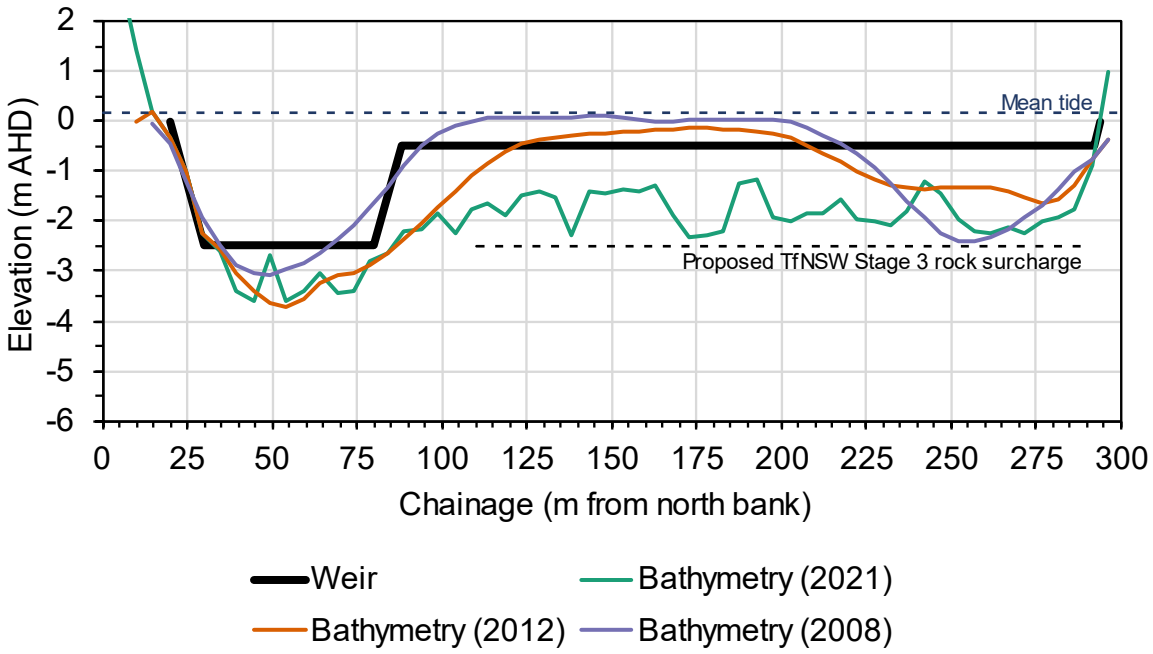


Figure B-3 Option 1C: -0.5 m AHD weir with one 50 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes

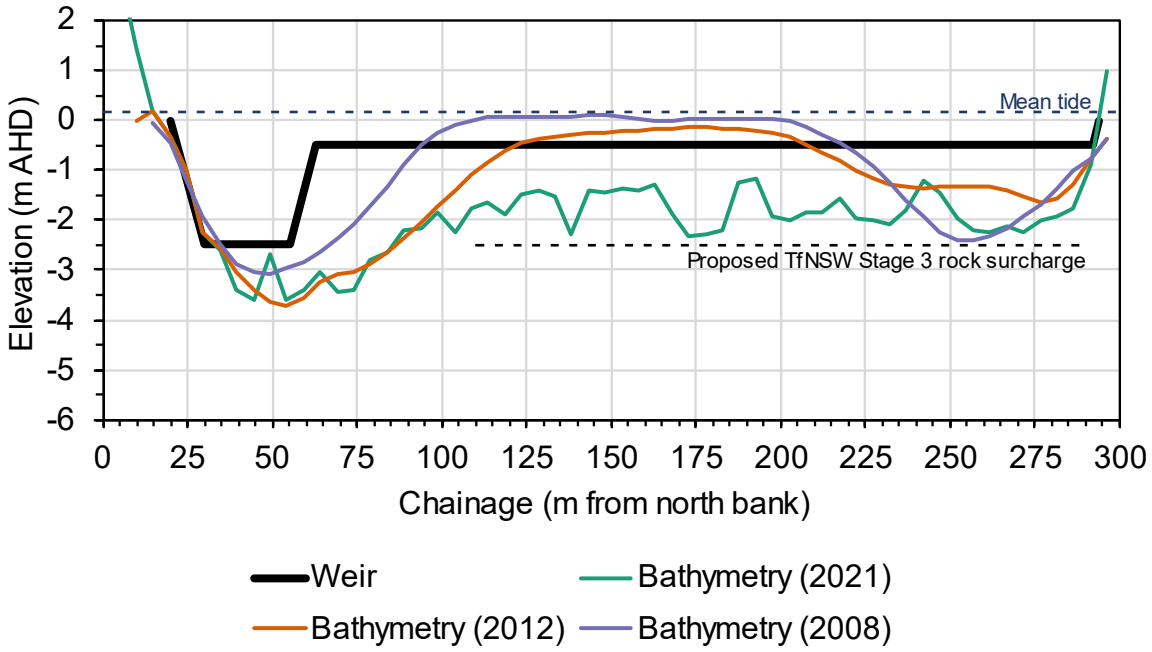


Figure B-4 Option 1D: -0.5 m AHD weir with one 25 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes

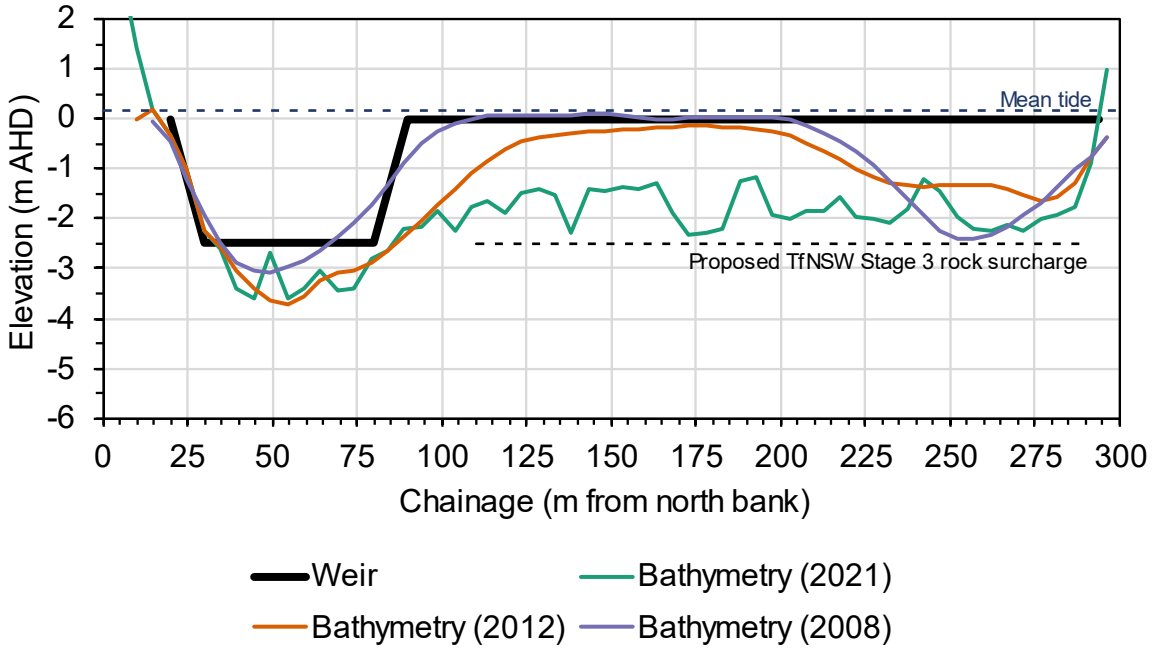


Figure B-5 Option 1E: 0 m AHD weir with one 50 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes

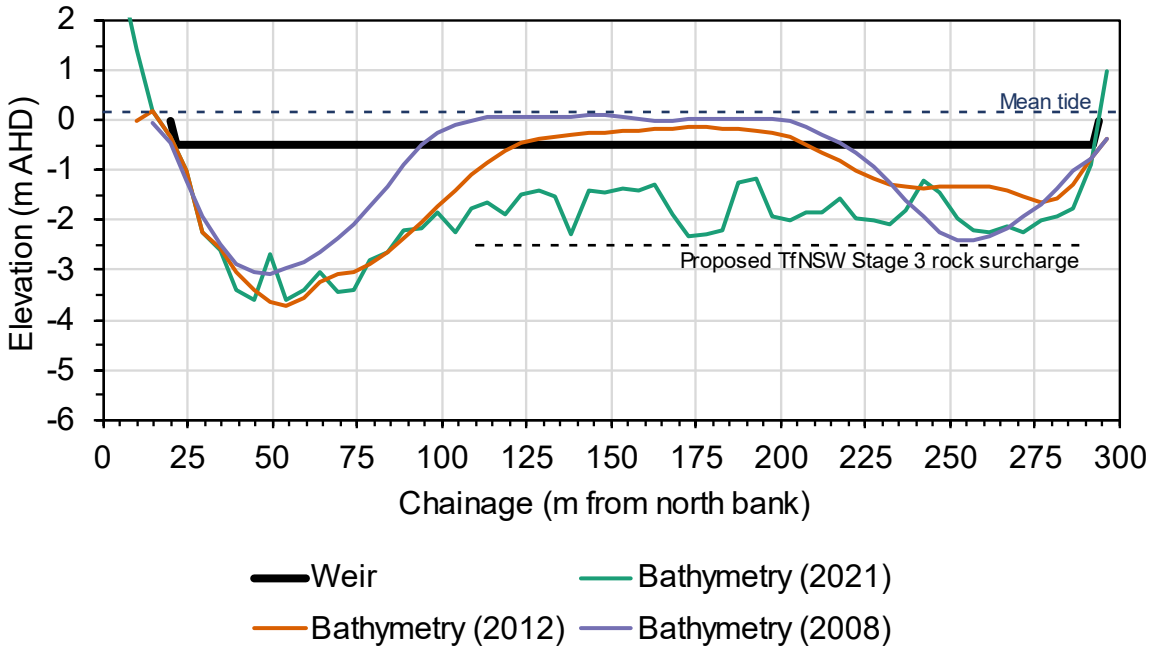


Figure B-6 Option 1F: -0.5 m AHD weir all the way across

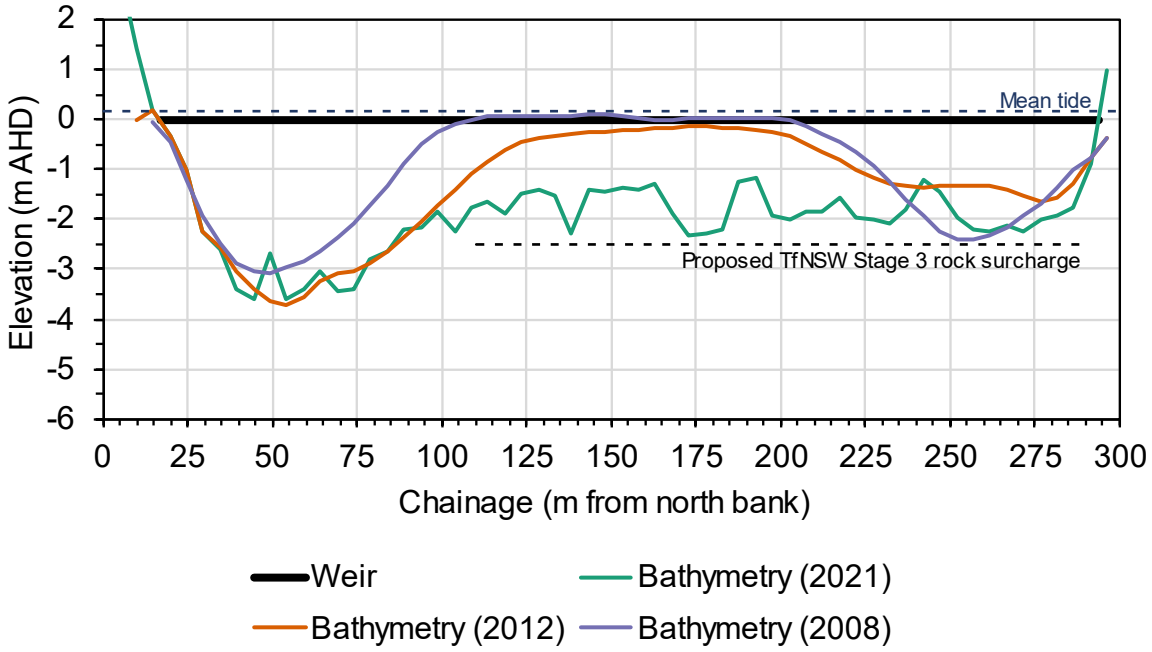


Figure B-7 Option 1G: 0 m AHD weir all the way across

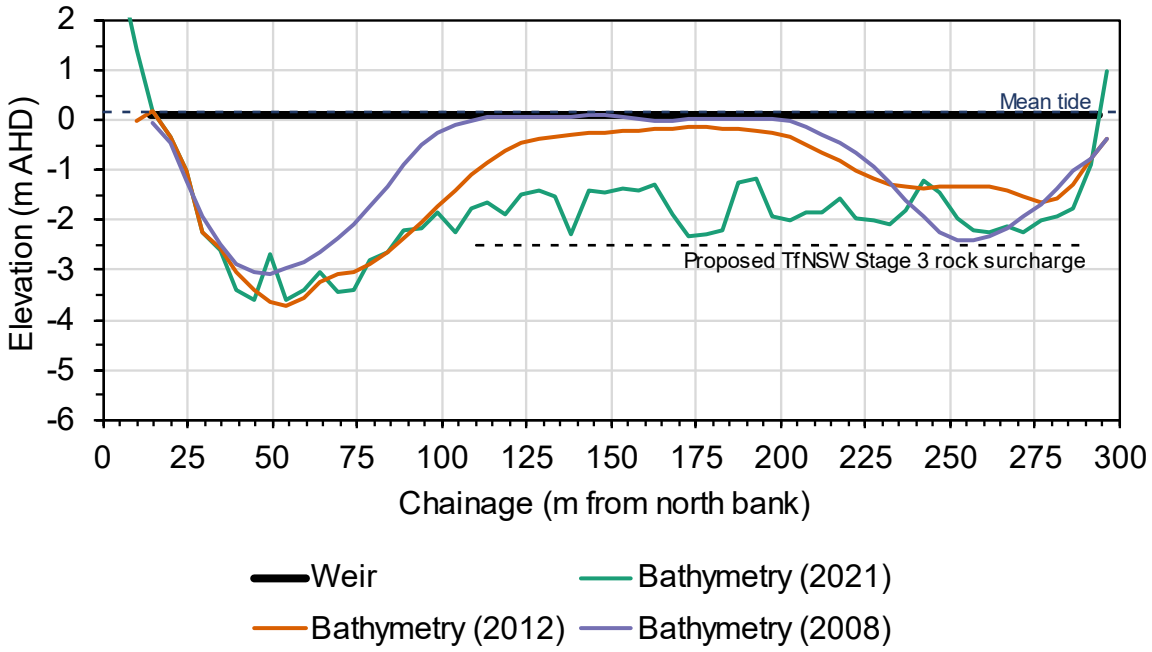


Figure B-8 Option 1H: 0.1 m AHD weir all the way across

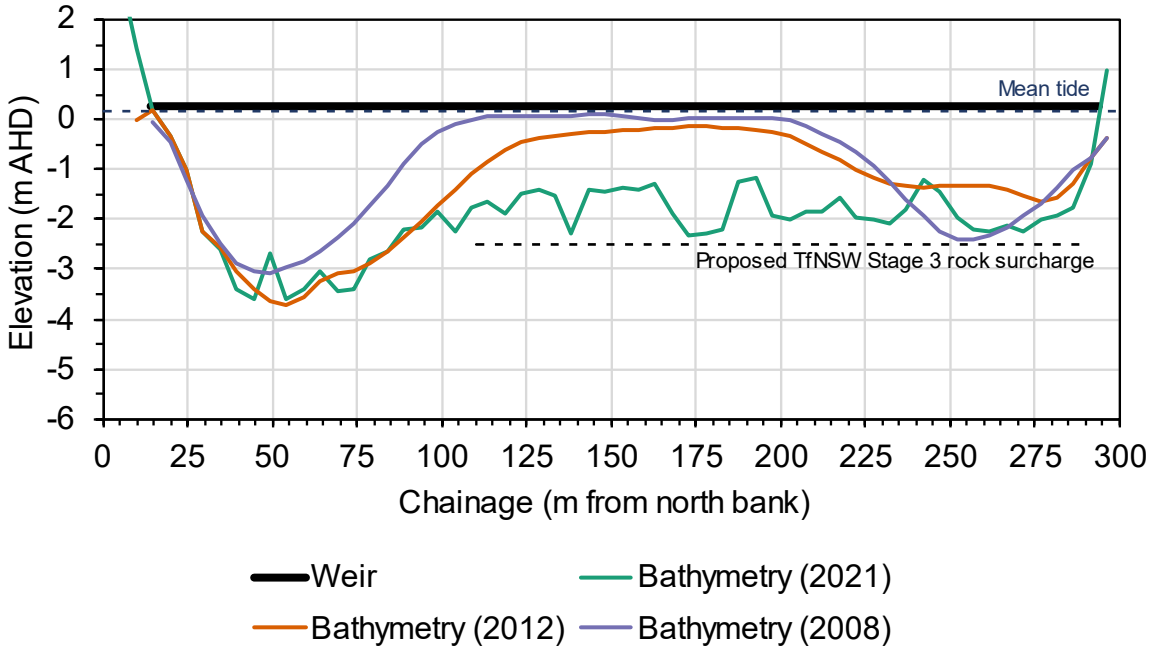


Figure B-9 Option 1I: 0.25 m AHD weir all the way across

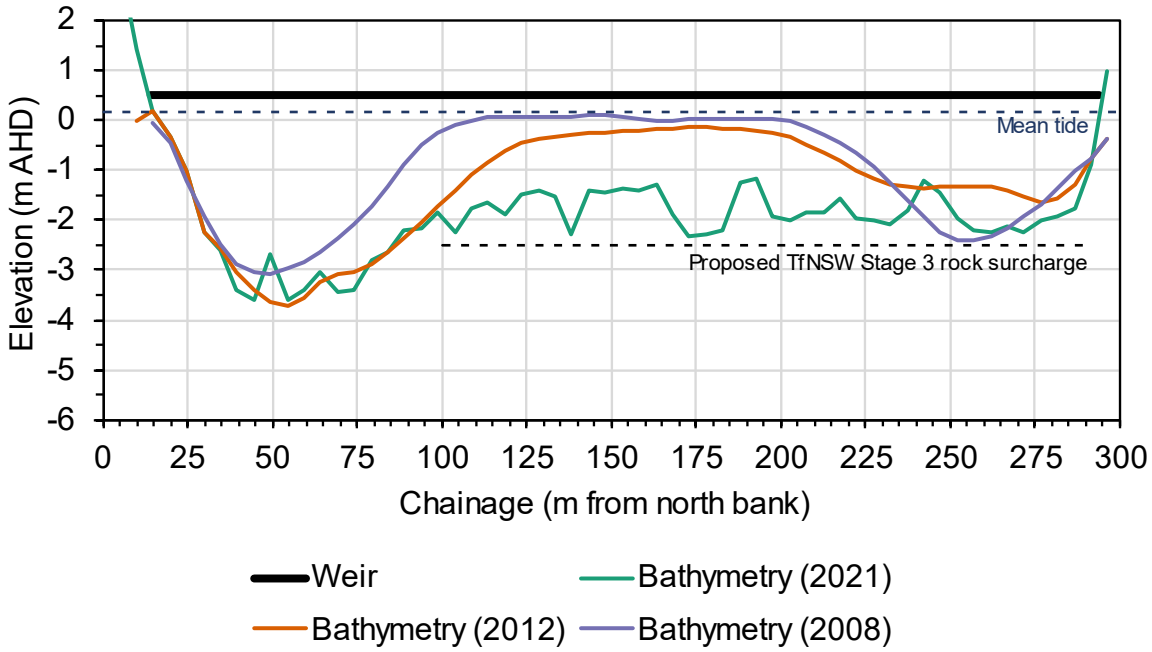


Figure B-10 Option 1J: 0.5 m AHD weir all the way across

B2 Option 3: Rock weirs at the western end of the entrance across all three channels

Option 3 consists of three rock weirs located at the western end of the entrance channel as shown in Figure 3.2. Two different geometries for the weir in the central channel were modelled, as follows:

- Option 3A: -0.5 m AHD weir with a 50 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes (Figure B-11)
- Option 3B: -0.5 m AHD weir all the way across (Figure B-12)

The weirs located on the northern and southern channels were the same for both scenarios with a constant crest elevation of -0.5 m AHD (see Figure B-13 and Figure B-14, respectively). Since these are located below the 2021 bathymetry level, these weirs did not change the results compared to the base case.

All Option 3 weirs had an adopted crest length (i.e. the dimension in the along stream direction) of 5 m.

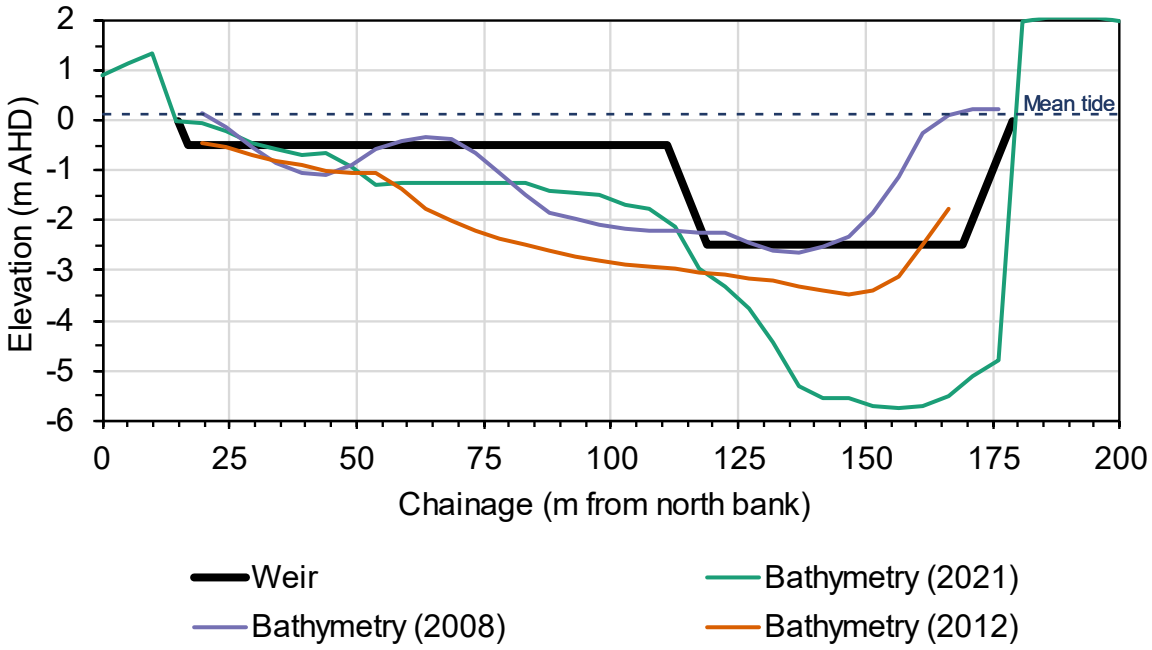


Figure B-11 Option 3A: -0.5 m AHD weir across the central channel with a 50 m wide channel (base width) at -2.5 m AHD with 1 vertical : 4 horizontal batter slopes

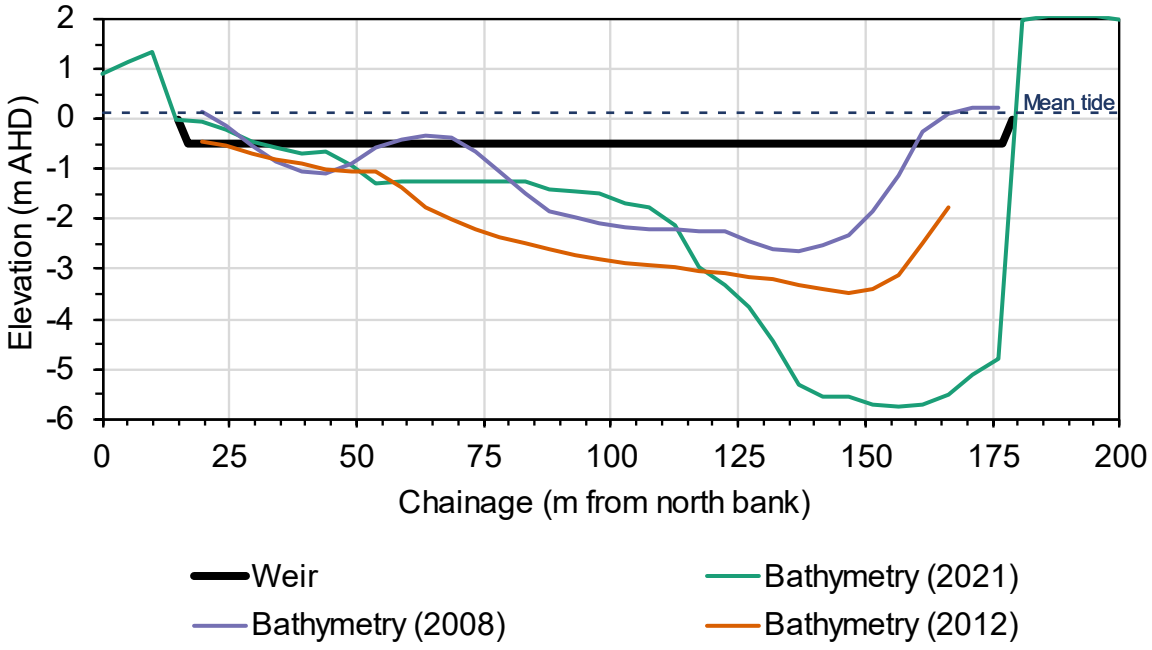


Figure B-12 Option 3B: -0.5 m AHD weir located across the central channel

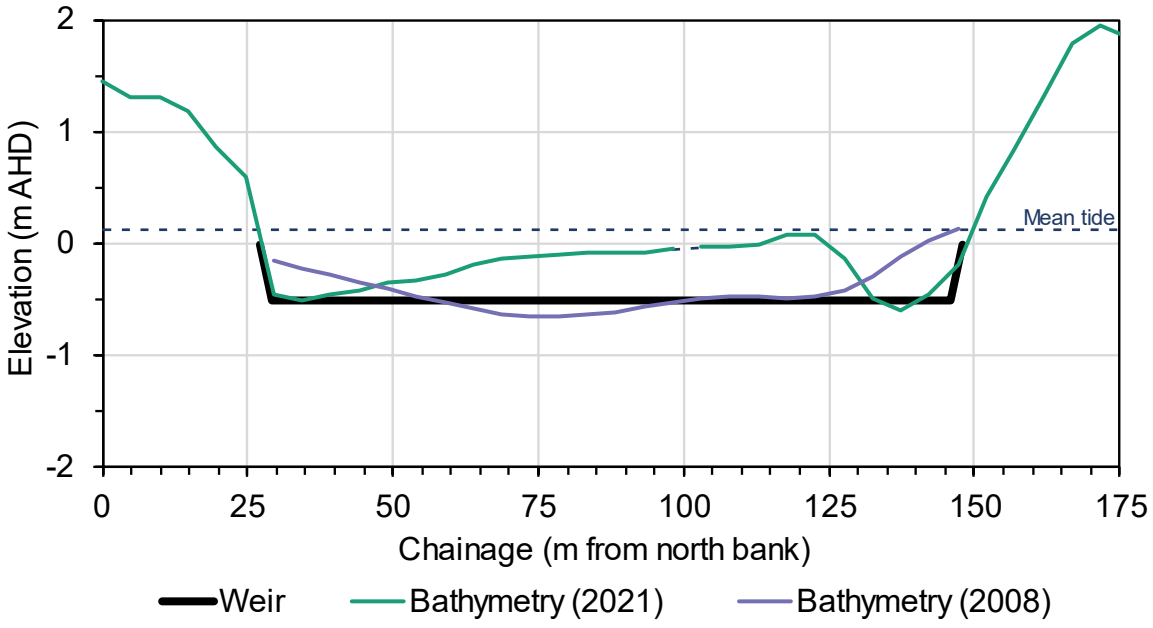


Figure B-13 Option 3A and 3B: -0.5 m AHD weir located across the northern channel

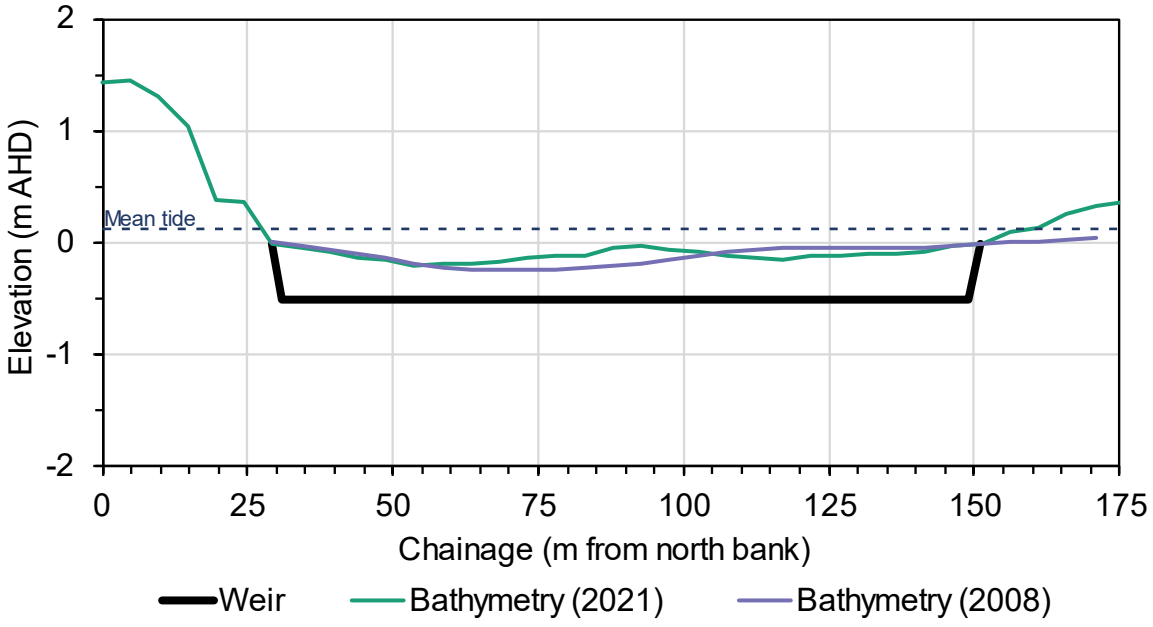


Figure B-14 Option 3A and 3B: -0.5 m AHD weir located across the southern channel

B3 Option 4: Staggered rock groyne field

Option 4A consists of three groynes located near the entrance to the ocean as shown in Figure 3.3. The adopted geometry included a crest width of 5 m at an elevation of 2 m AHD and side slopes of 1V:1.5H. Cross-sections for each Option 4A groyne are shown in Figures B-15 to B-17.

Option 4B is identical to Option 4A except that the bed within and around the groyne field is armoured with rock of 0.8 m thickness as shown in Figure 3.4. Cross-sections for each Option 4B groyne are shown in Figures B-18 to B-20 and include only the 2021 bathymetry and the crest level of the bed armouring.

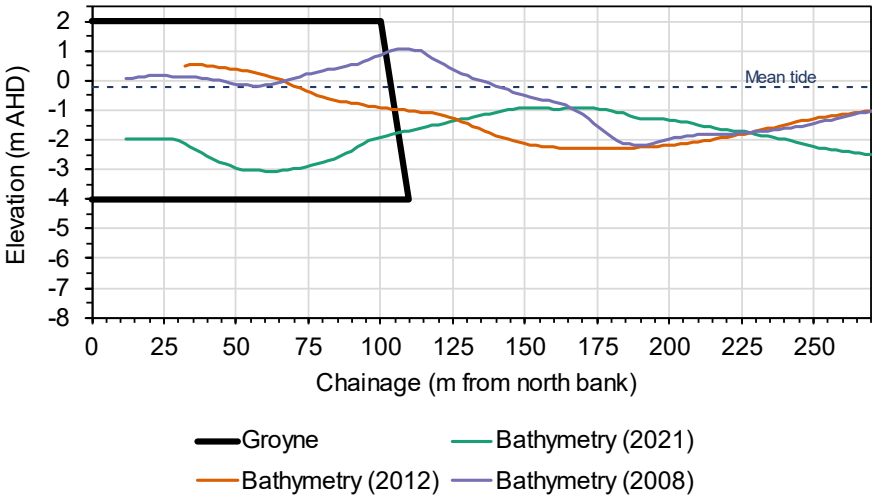


Figure B-15 Option 4A: 100 m long rock groyne 1 (see also Figure 3.3)

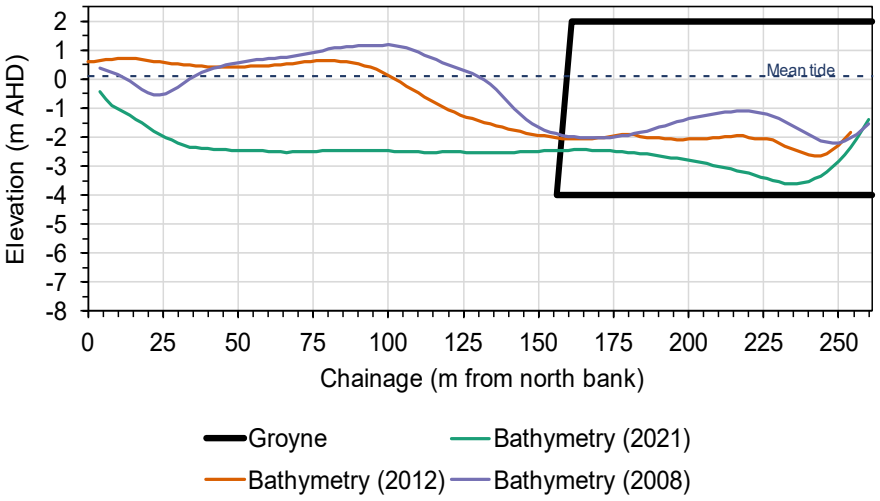


Figure B-16 Option 4A: 100 m long rock groyne 2 (see also Figure 3.3)

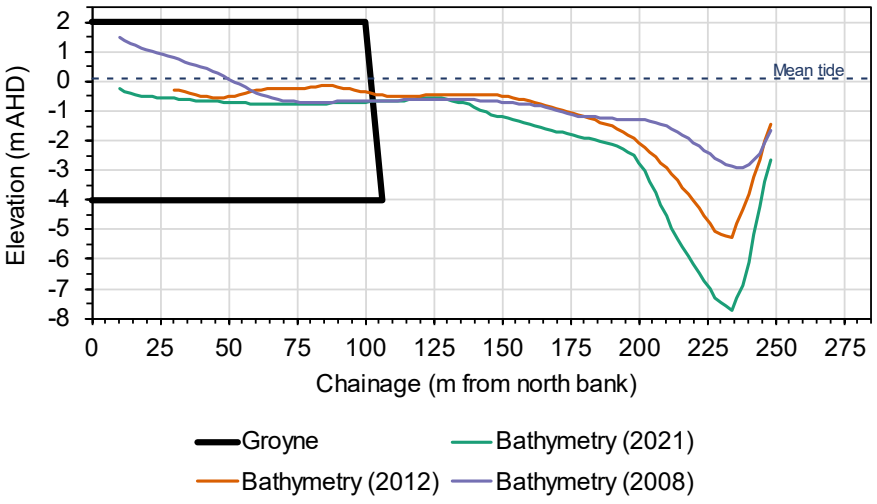


Figure B-17 Option 4A: 100 m long rock groyne 3 (see also Figure 3.3)

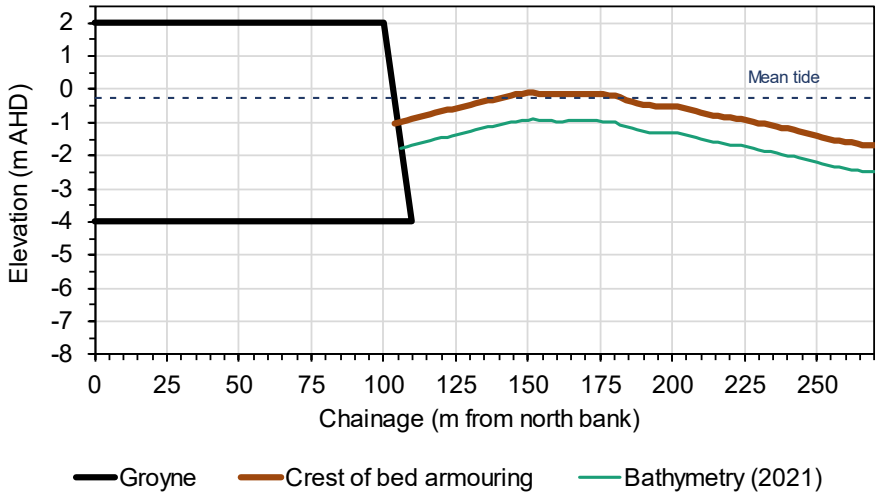


Figure B-18 Option 4B: 100 m long rock groyne 1 with armoured bed (see also Figure 3.4)

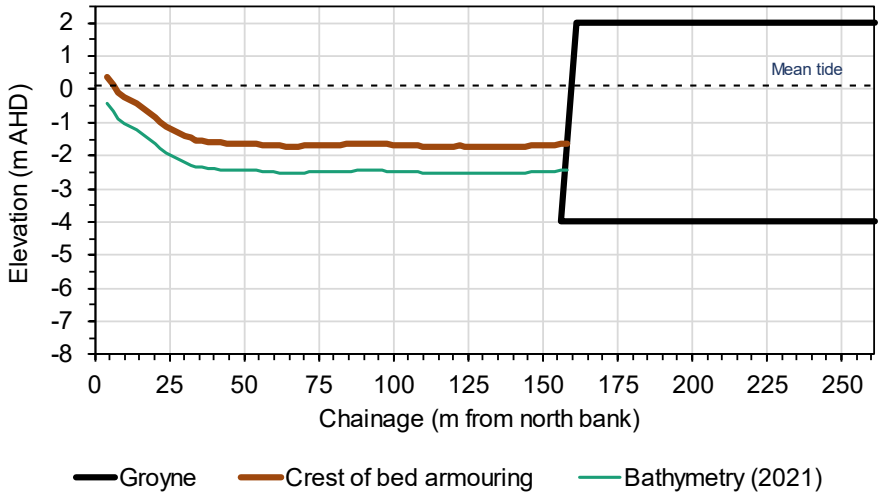


Figure B-19 Option 4B: 100 m long rock groyne 2 with armoured bed (see also Figure 3.4)

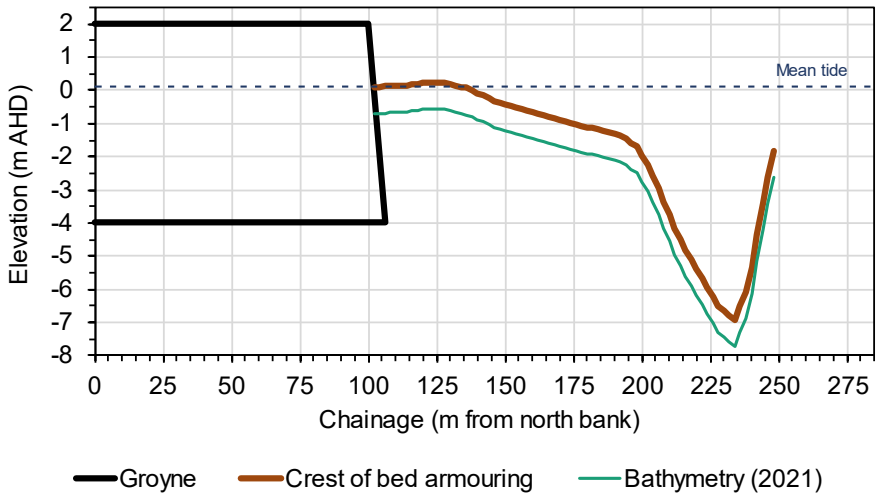


Figure B-20 Option 4B: 100 m long rock groyne 3 with armoured bed (see also Figure 3.4)

Appendix C Modelled tidal regime results

As discussed in Section 3.5.1, to assess whether each option geometry would satisfy the 2008 target tidal range in the lake, model outputs for water level between the entrance and Lake Illawarra were extracted along the transect shown in Figure 3.7. These locations were chosen to coincide with:

- MHL permanent water level gauge: “Lake Illawarra Entrance”
- MHL temporary water level gauge: “Lake Illawarra Site 2”
- MHL temporary water level gauge: “Lake Illawarra Site 3”
- MHL permanent water level gauge: “Cudgeree Bay”
- MHL permanent water level gauge: “Koonawarra Bay”

Water levels at the Koonawarra Bay gauge were found to be representative of water levels across the entire lake. Model output at Koonawarra Bay was used to assess whether each option geometry satisfied the 2008 target tidal range in the lake.

Measurements between 30 June 2008 to 27 July 2008 were obtained from the following water level gauges:

- NTC/BoM permanent water level gauge: “Port Kembla” (representative at the channel entrance)
- MHL permanent water level gauge: “Lake Illawarra Entrance”
- MHL permanent water level gauge: “Cudgeree Bay”
- MHL permanent water level gauge: “Koonawarra Bay”

It is acknowledged that between 2008 (target tidal range period) and 2021 (model calibration period), the water level gauges at “Lake Illawarra Entrance” and “Cudgeree Bay” were moved to avoid shoaling impacts. However, the small distances they were moved (Table C-1) would have a negligible impact on the water level transects along the entrance channel.

Table C-1 Relocation details of MHL permanent water level gauges

MHL water level gauge name	Relocation date	Old coordinates (GDA94/MGA56)		New coordinates (GDA94/MGA56)	
		Easting	Northing	Easting	Northing
Lake Illawarra Entrance	June 2013	304467	6176073	304579	6176195
Cudgeree Bay*	November 2020	303934	6177233	303859	6176981

* Note that the MHL water level gauge “Cudgeree Bay” was renamed “Cudgeree Bay 2” when relocated in November 2020

The average high and low tide water levels between the ocean and Lake Illawarra for each structure geometry modelled (including the same 2008 observations and modelled base case) are shown in Figures C-1 to C-14.

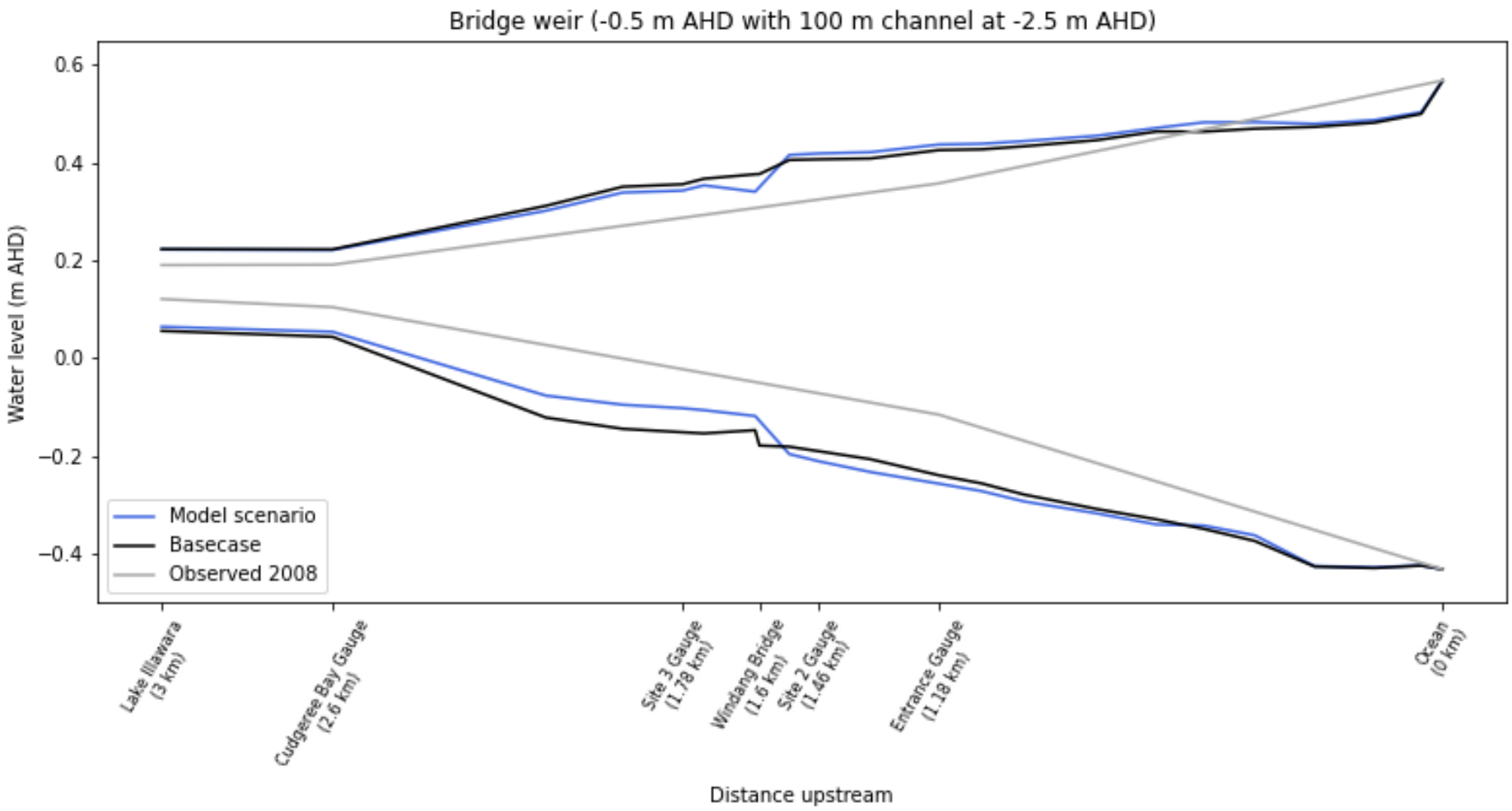


Figure C-1 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1A (bridge weir, blue): -0.5 m AHD weir with 1×100 m wide channel at -2.5 m AHD

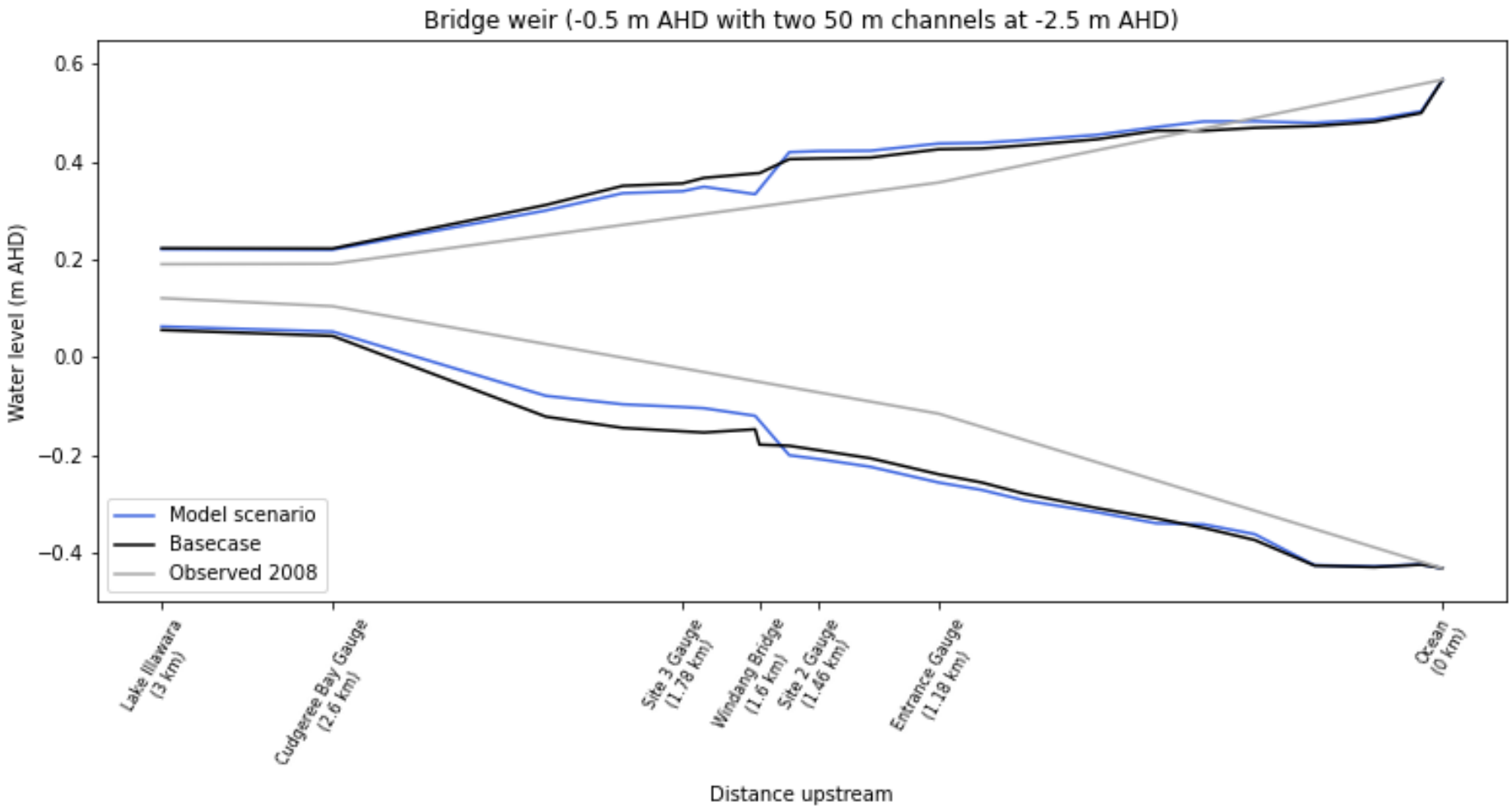


Figure C-2 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1B (bridge weir, blue): -0.5 m AHD weir with 2×50 m wide channels at -2.5 m AHD

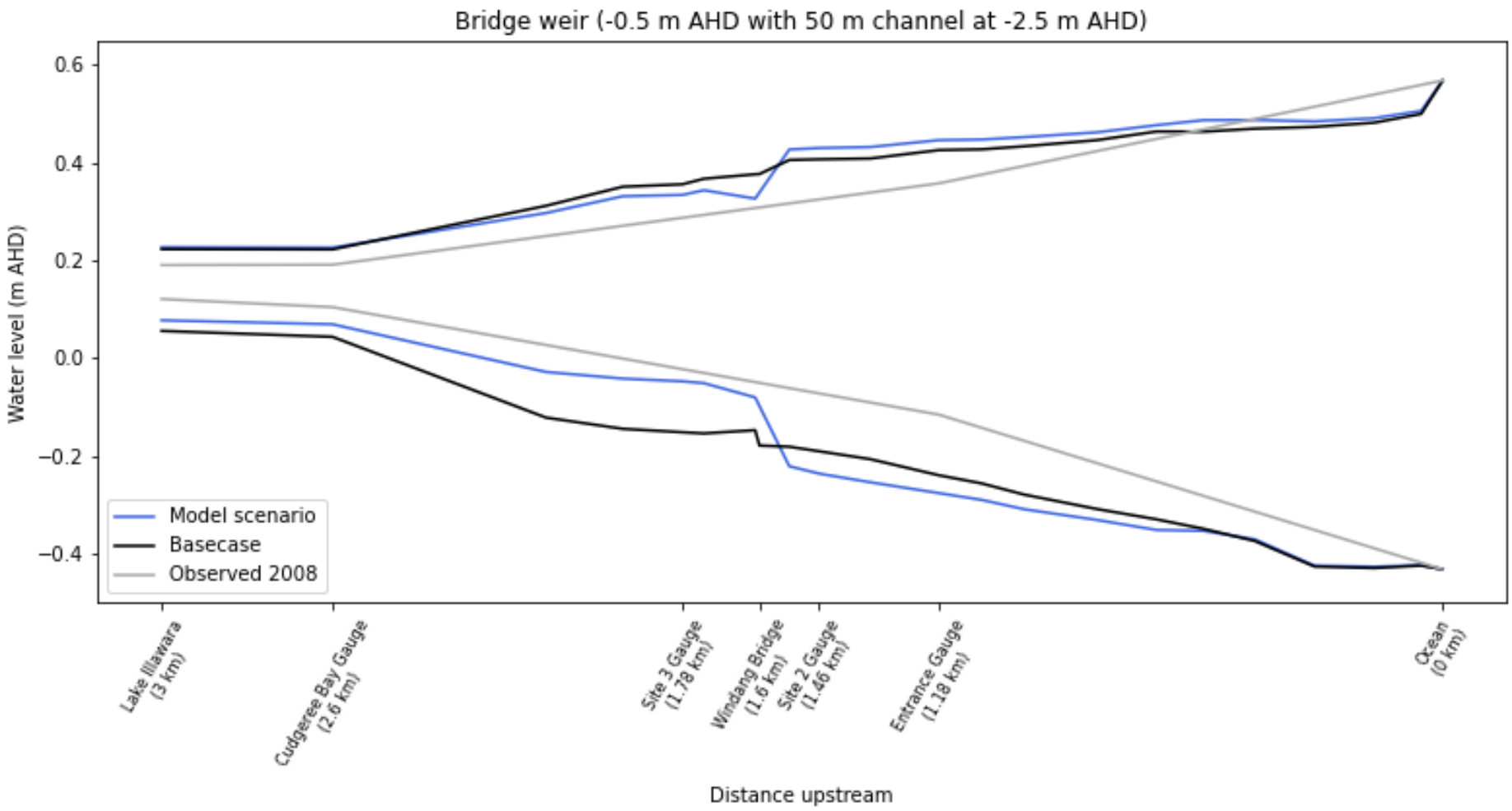


Figure C-3 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1C (bridge weir, blue): -0.5 m AHD weir with 1×50 m wide channel at -2.5 m AHD

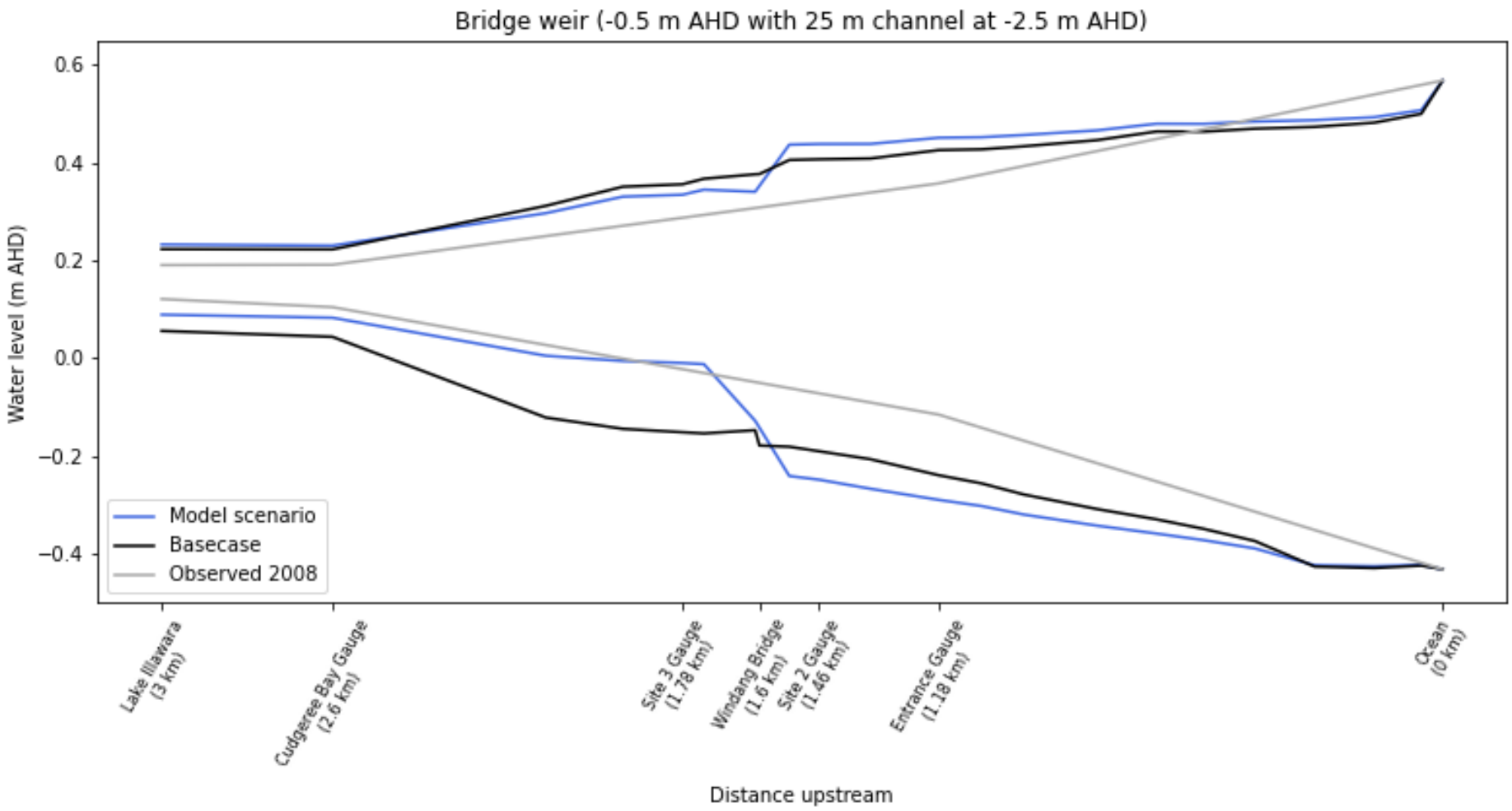


Figure C-4 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1D (bridge weir, blue): -0.5 m AHD weir with 1×25 m wide channel at -2.5 m AHD

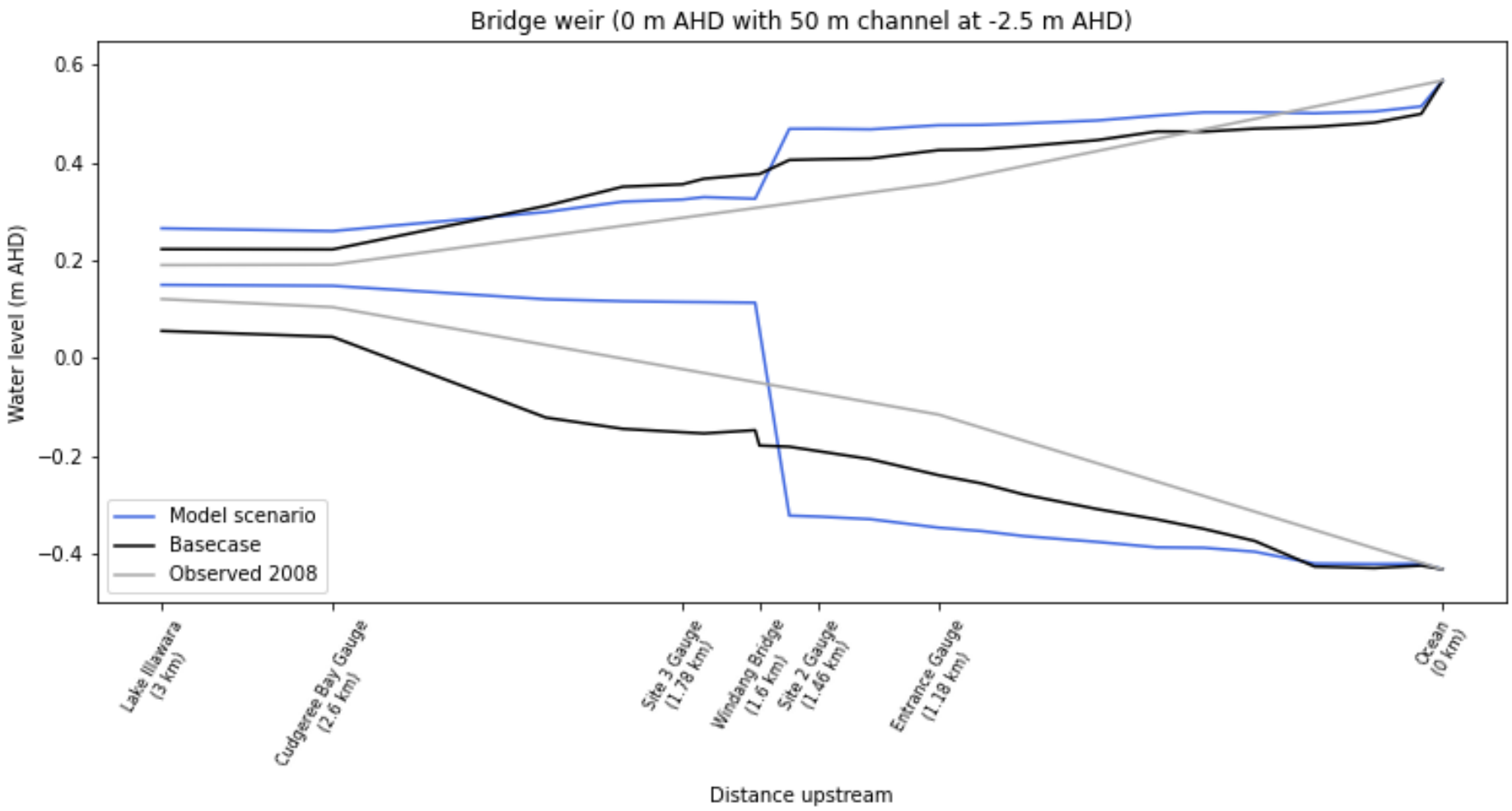


Figure C-5 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1E (bridge weir, blue): 0 m AHD weir with 1×50 m wide channel at -2.5 m AHD

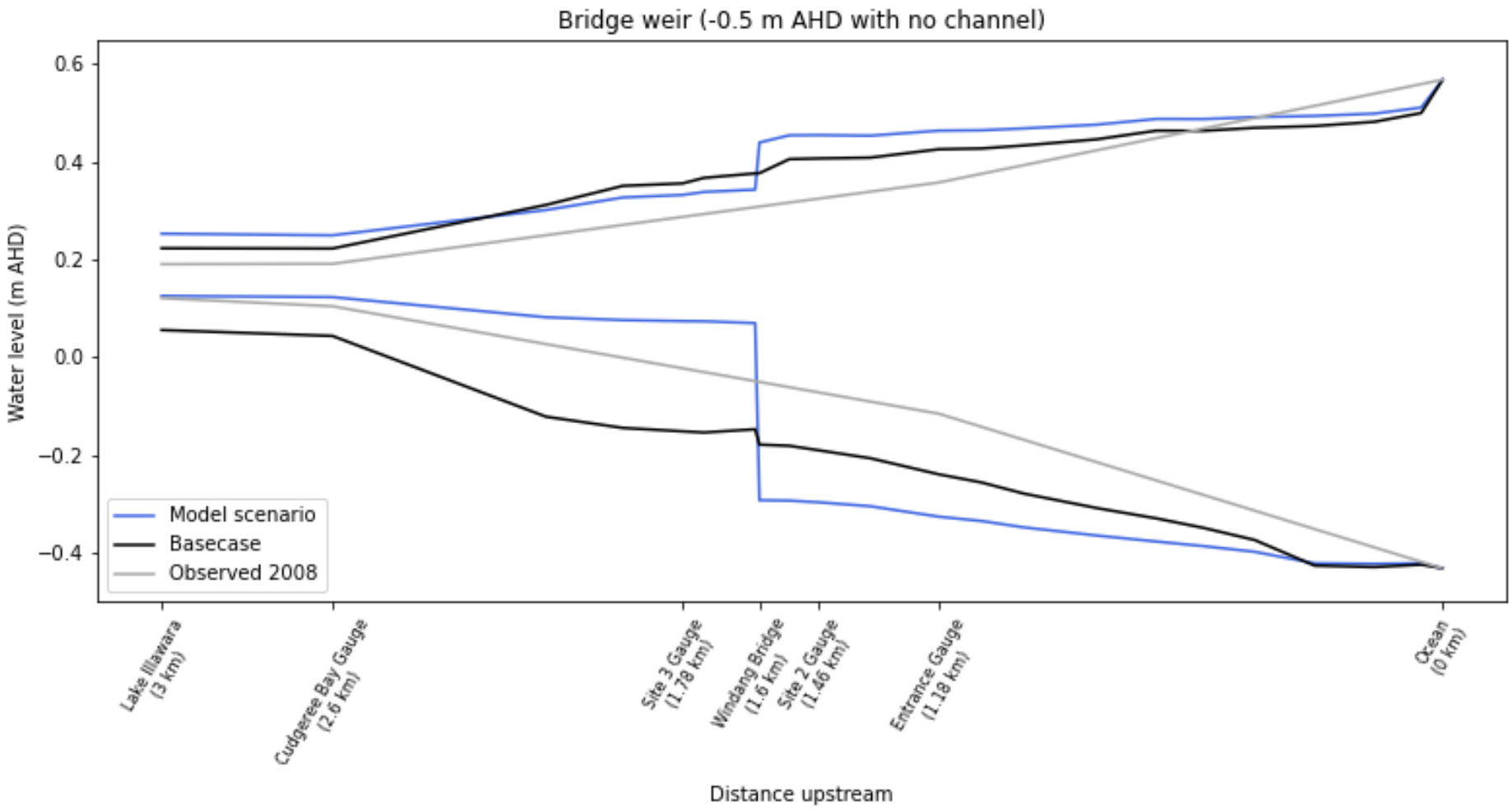


Figure C-6 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1F (bridge weir, blue): -0.5 m AHD weir all the way across (no channel)

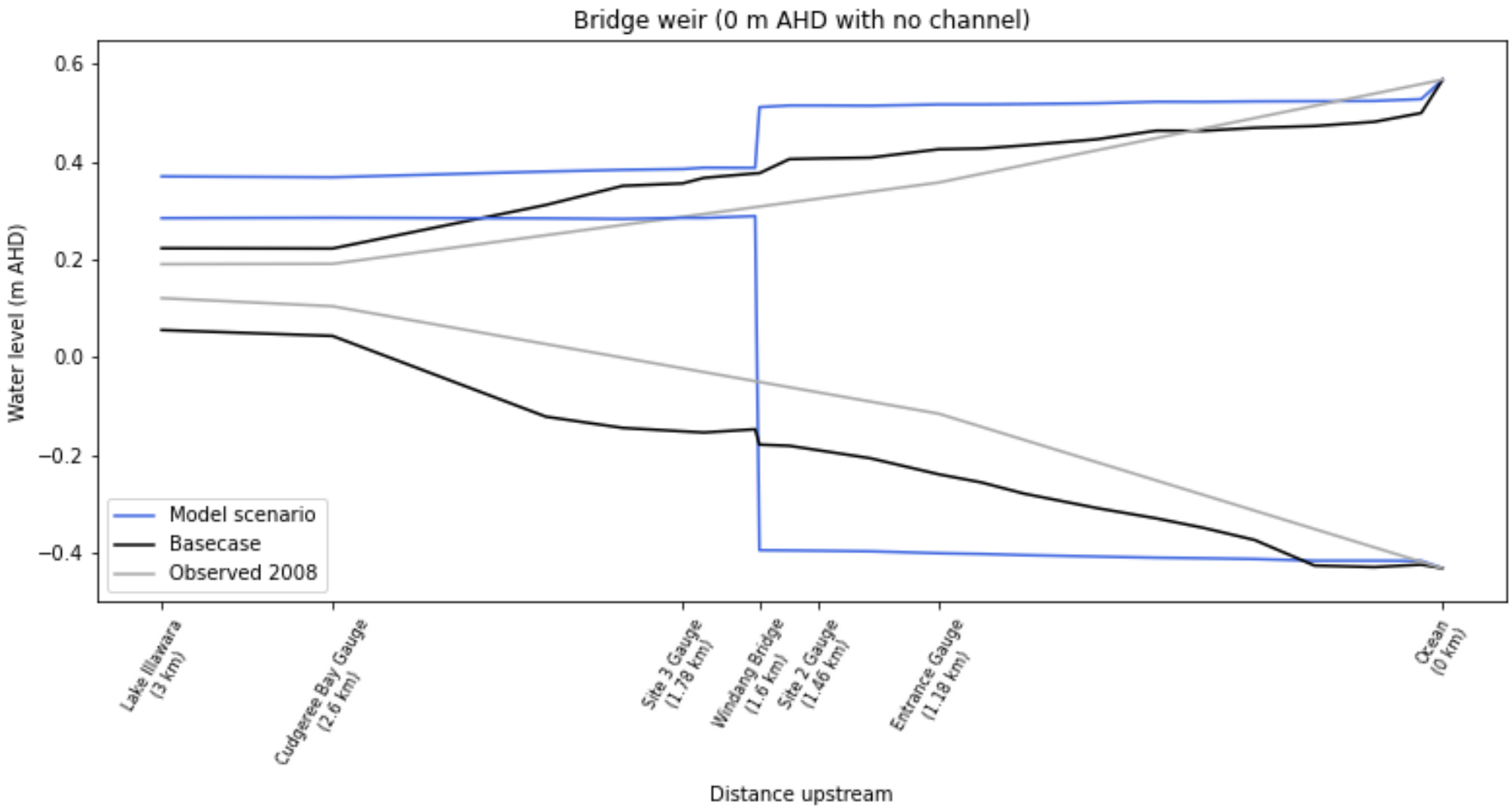


Figure C-7 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1G (bridge weir, blue): 0 m AHD weir all the way across (no channel)

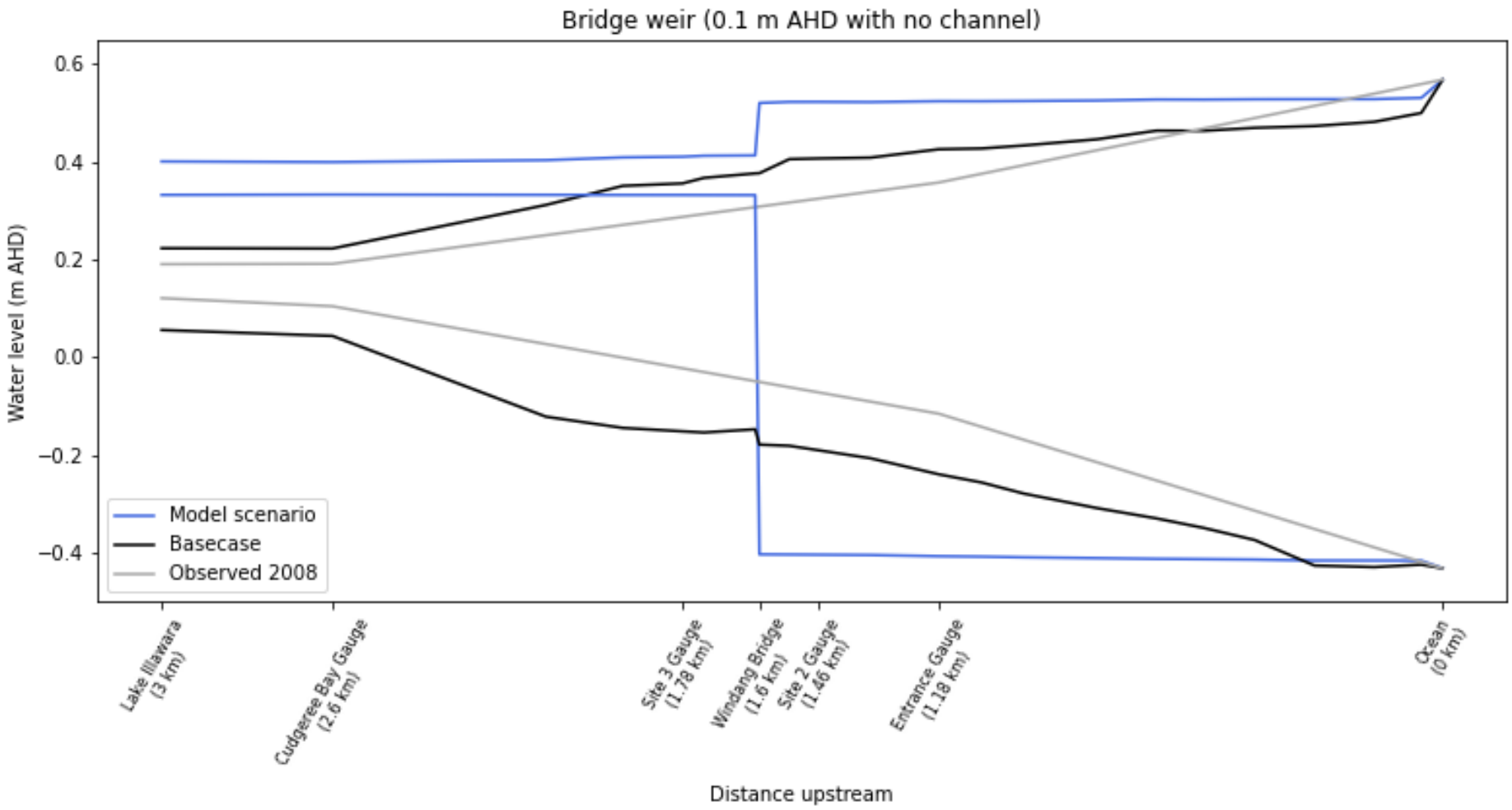


Figure C-8 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1H (bridge weir, blue): 0.1 m AHD weir all the way across (no channel)

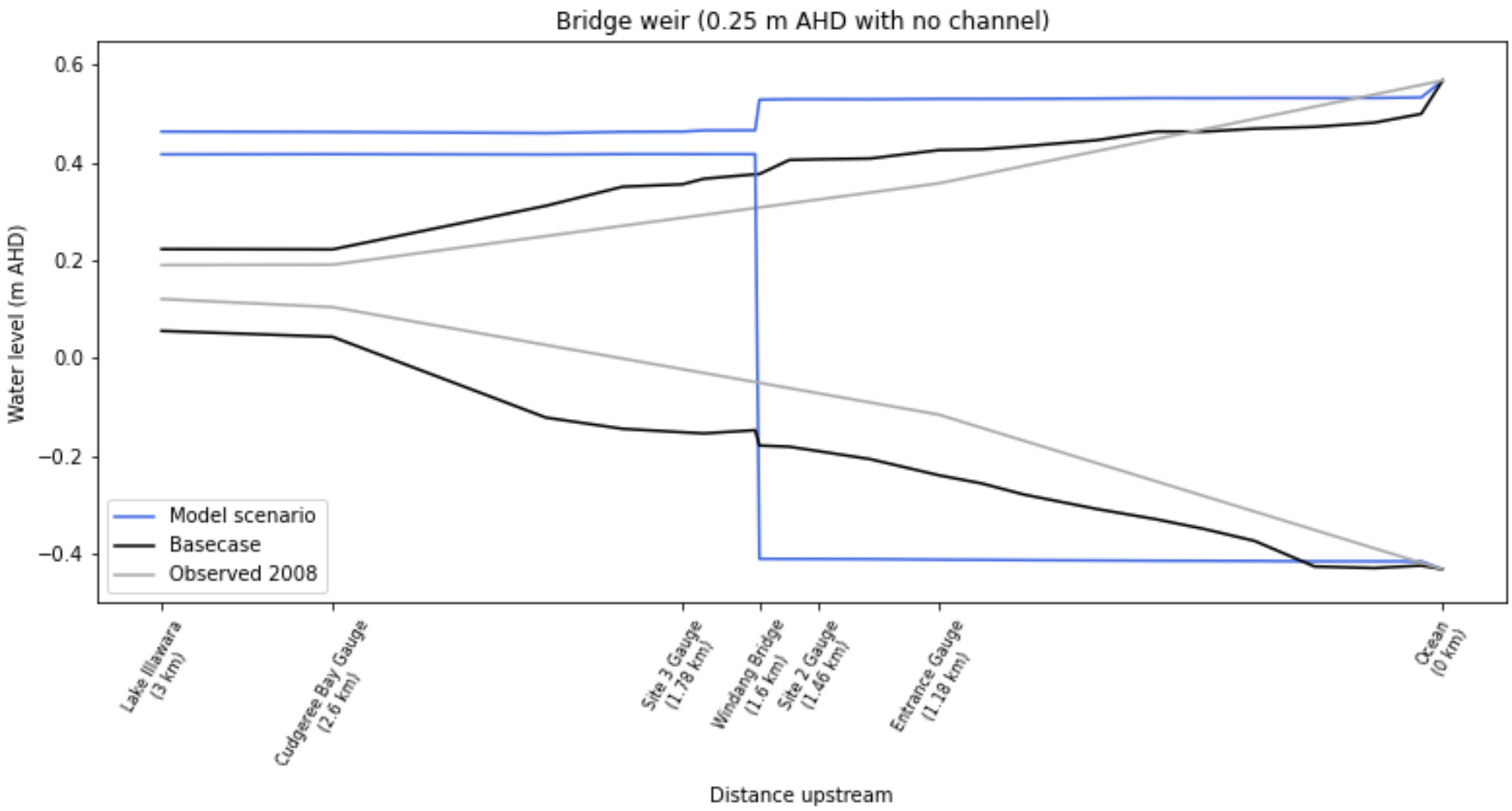


Figure C-9 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1l (bridge weir, blue): 0.25 m AHD weir all the way across (no channel)

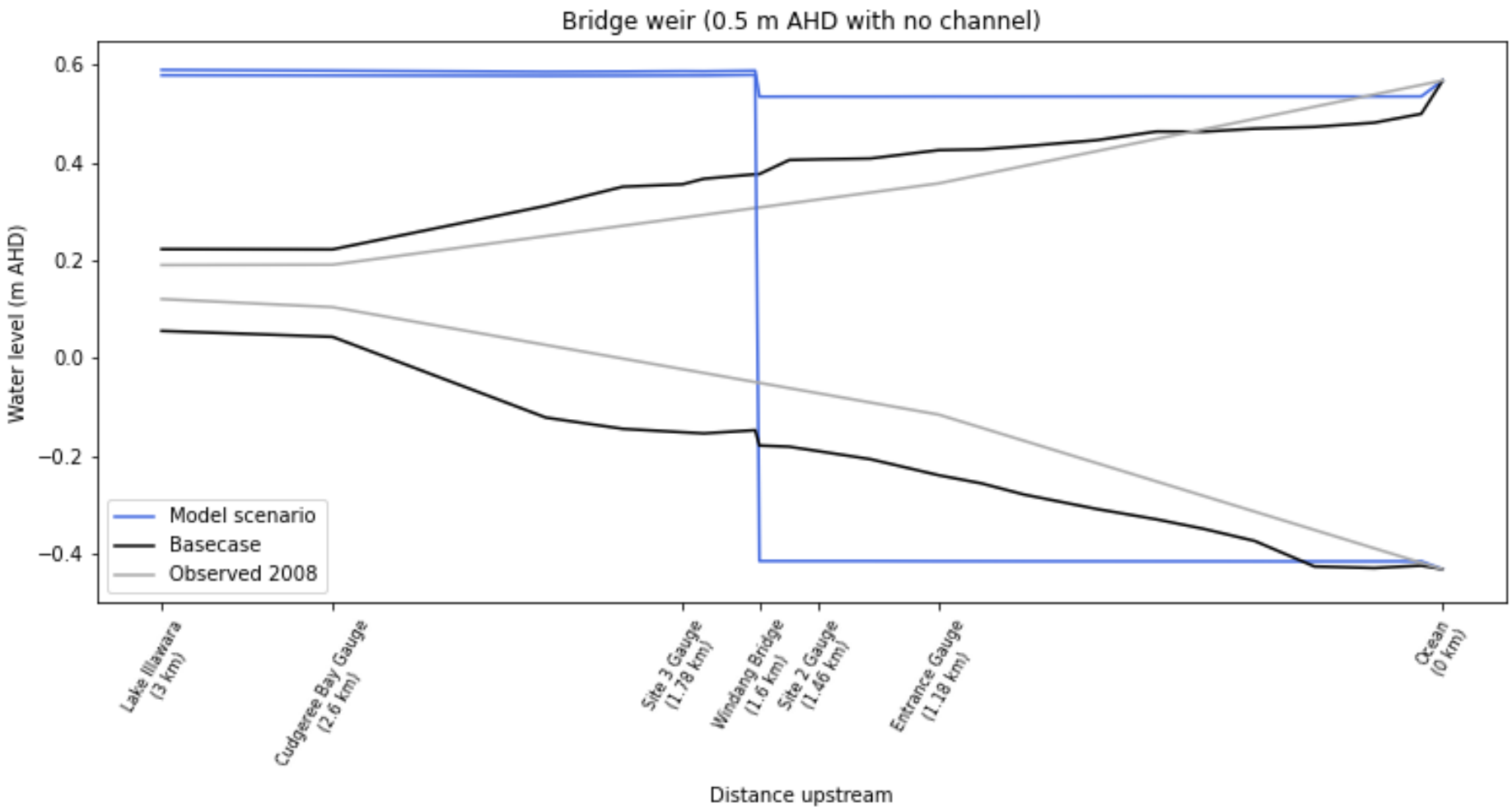


Figure C-10 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 1J (bridge weir, blue): 0.5 m AHD weir all the way across (no channel)

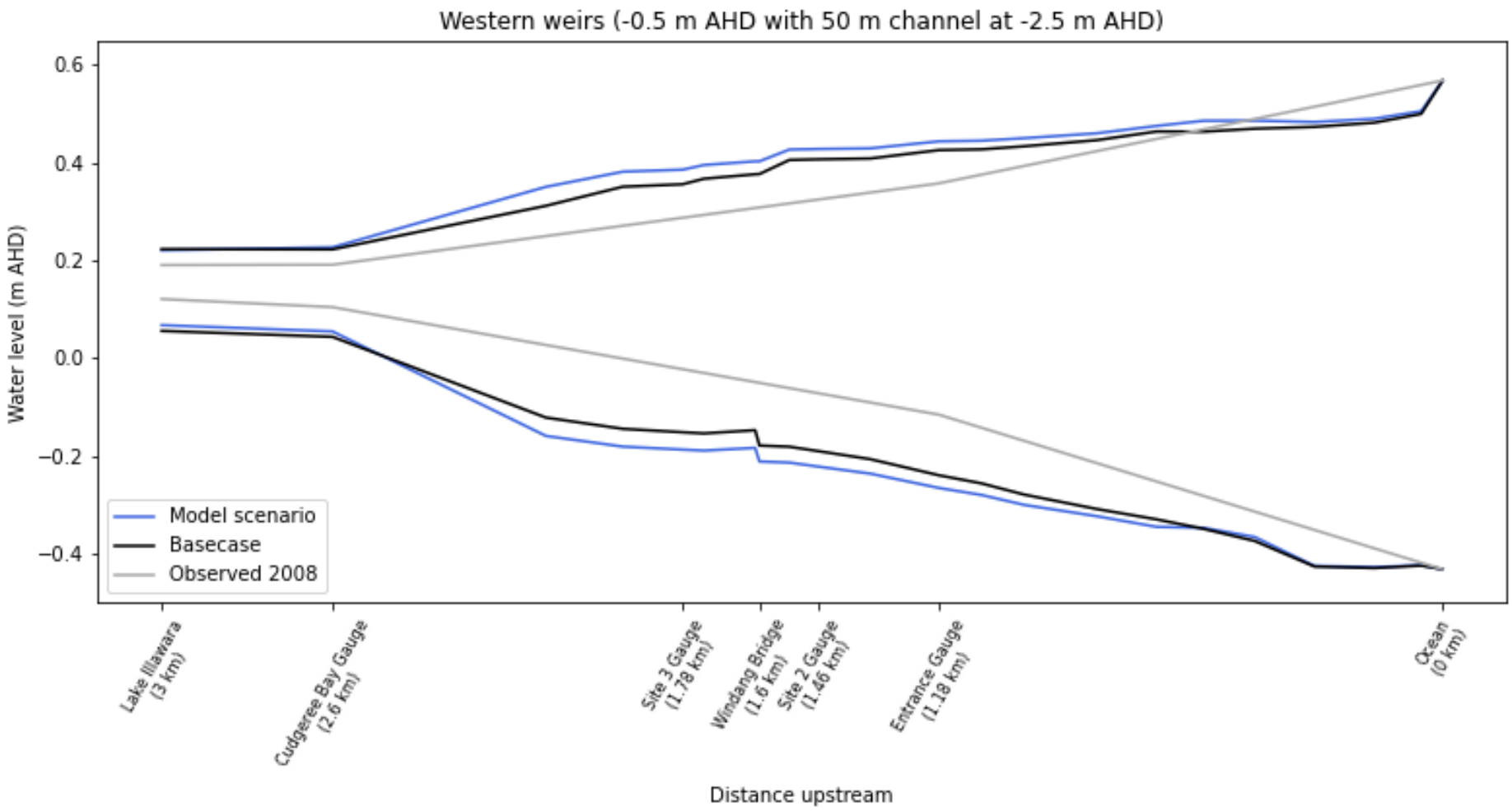


Figure C-11 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 3A (western weirs, blue): -0.5 m AHD weir with 50 m wide channel at -2.5 m AHD (central channel); -0.5 m AHD weirs (northern and southern channels)

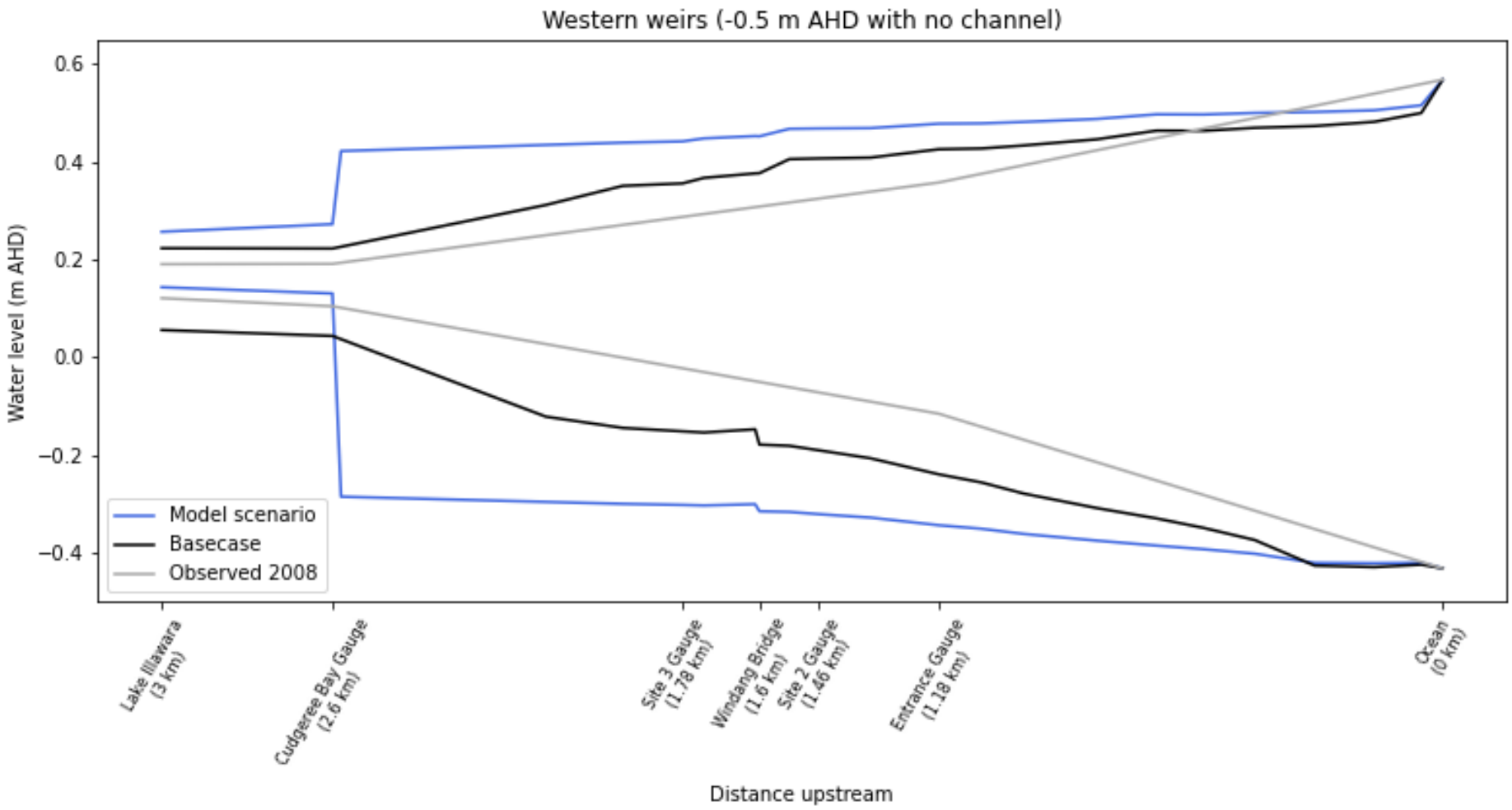


Figure C-12 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 3B (western weirs, blue): -0.5 m AHD weirs across all three channels

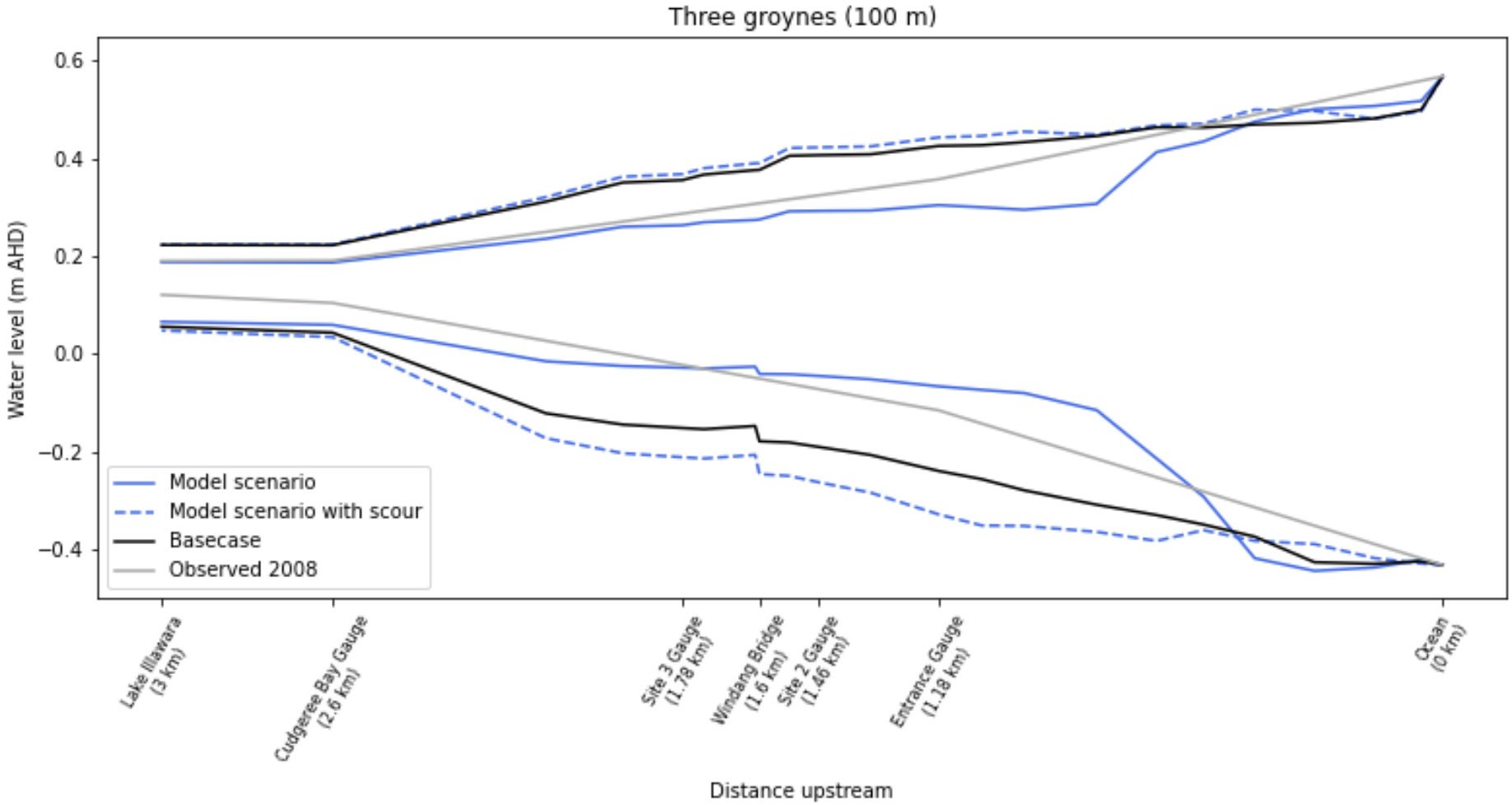


Figure C-13 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 4A (groyne field, blue; groyne field with local bed scour, dashed blue): Three 100 m long groynes

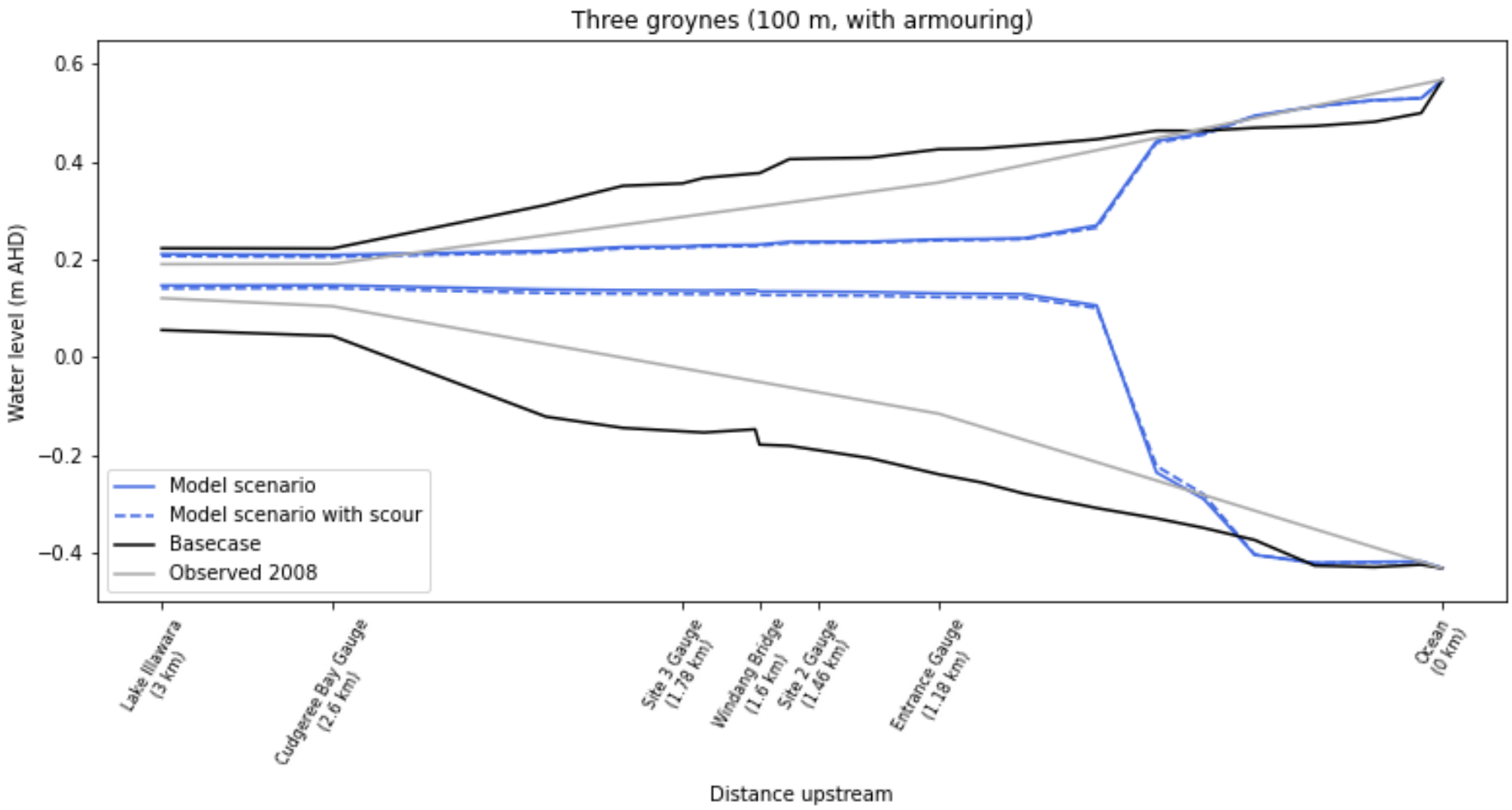


Figure C-14 Average high and low tide water levels (30/06/2008 – 27/08/2008) along the channel for the observed water levels in 2008 (grey), the modelled base case (2021 entrance geometry, black) and modelled Option 4B (groyne field, blue; groyne field with local bed scour, dashed blue): Three 100 m long groynes (with armoured bed)

Appendix D Modelled bed shear results

Sediment samples collected by WRL in 2018 (Tucker et al., 2023a) found that the majority of sediment in the entrance channel was medium sand (0.25 to 0.50 mm diameter) with some coarse sand/gravel (>1 mm diameter). Berenbrock and Tranmer (2008) identified the following critical bed shear values:

- Medium sand: 0.2 to 0.3 N/m^2
- Coarse sand: 0.3 to 0.5 N/m^2
- Very coarse sand: 0.5 to 1.3 N/m^2

Subsequently, a bed shear greater than approximately 1 N/m^2 is likely to result in sediment mobilisation and potential erosion (e.g. higher bed shear can be inferred to equal a higher potential rate of erosion). Bed shear (τ_b) throughout the model domain was calculated using the approach outlined by Chow (1959) shown in Equation D-1.

$$\tau_b = \frac{gn^2\rho u^2}{R_h^{\frac{1}{3}}} \quad \text{(D-1)}$$

where:
g = Acceleration due to gravity (9.81 m/s^2)
n = Manning's 'n' friction (0.025 as per the model calibration; see Appendix A Section A8.1)
ρ = Density of fluid (~1,024 kg/m^3 for seawater)
u = Depth averaged velocity (m/s)
R_h = Hydraulic Radius (approximated as depth (m))

Velocity and depth were taken as outputs from the numerical model to calculate bed shear over time across the model domain. Figure D-1 shows the approximate maximum bed shear stress over the 30 June 2008 to 27 July 2008 for the base case. Any coloured areas in the figures indicate likely sediment mobilisation whenever this maximum bed shear time step (15 minute duration) occurs during the 28 day simulation. Darker coloured areas can be inferred to have higher erosion rates.

Figures showing the approximate maximum bed shear stress for each structure geometry modelled are shown in Figure D-2 to Figure D-17.

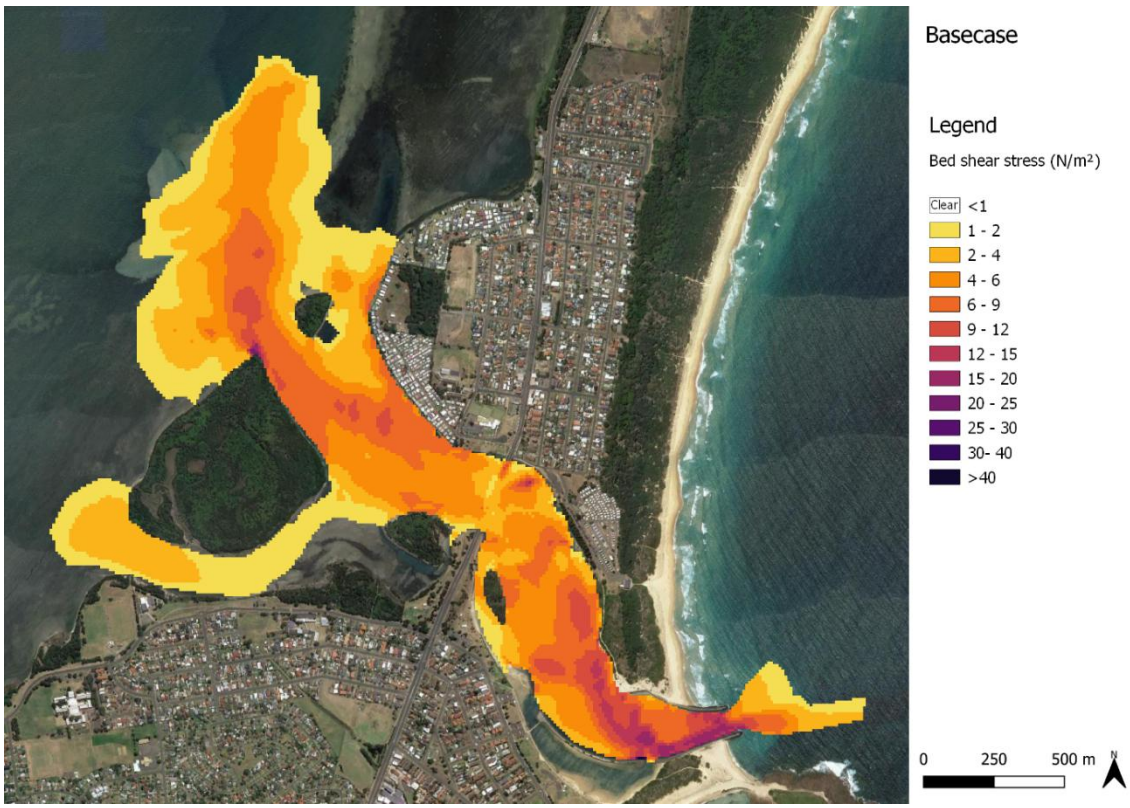


Figure D-1 Approximate maximum bed shear for modelled base case scenario (2021 channel geometry)

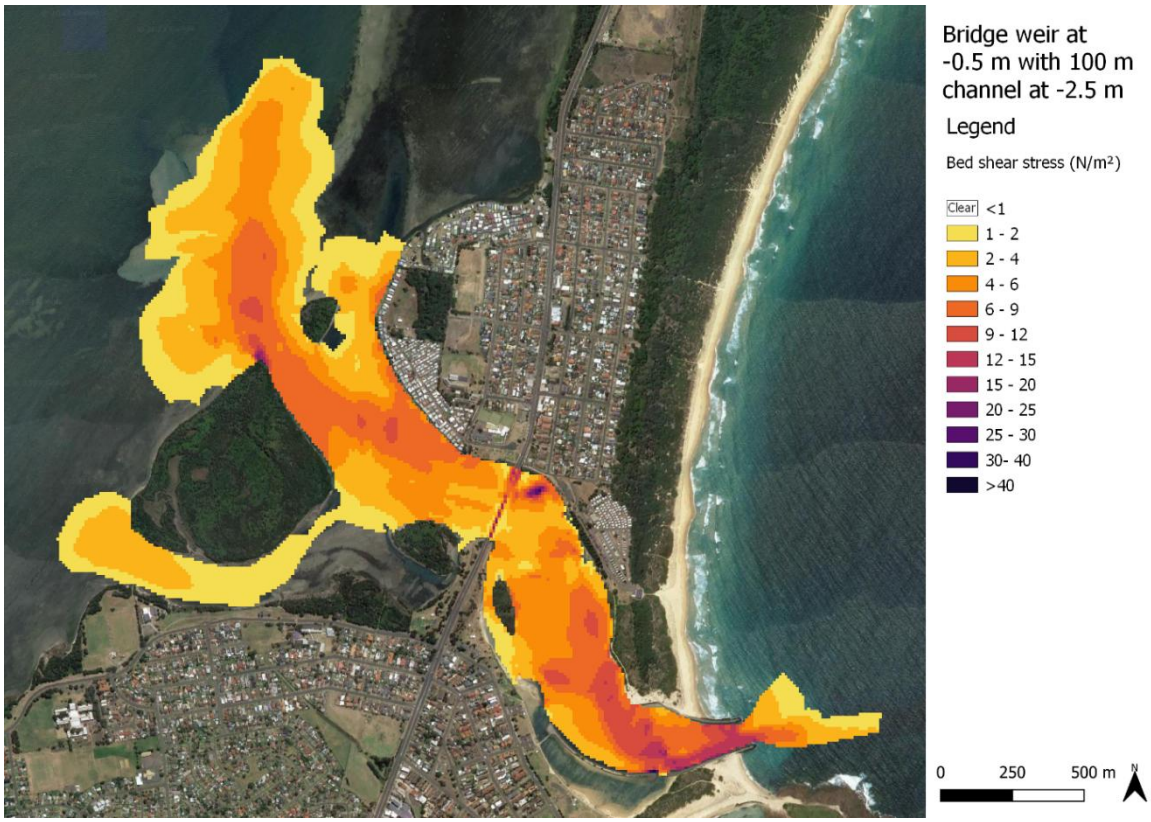


Figure D-2 Approximate maximum bed shear for modelled Option 1A (bridge weir): -0.5 m AHD weir with 1×100 m wide channel at -2.5 m AHD

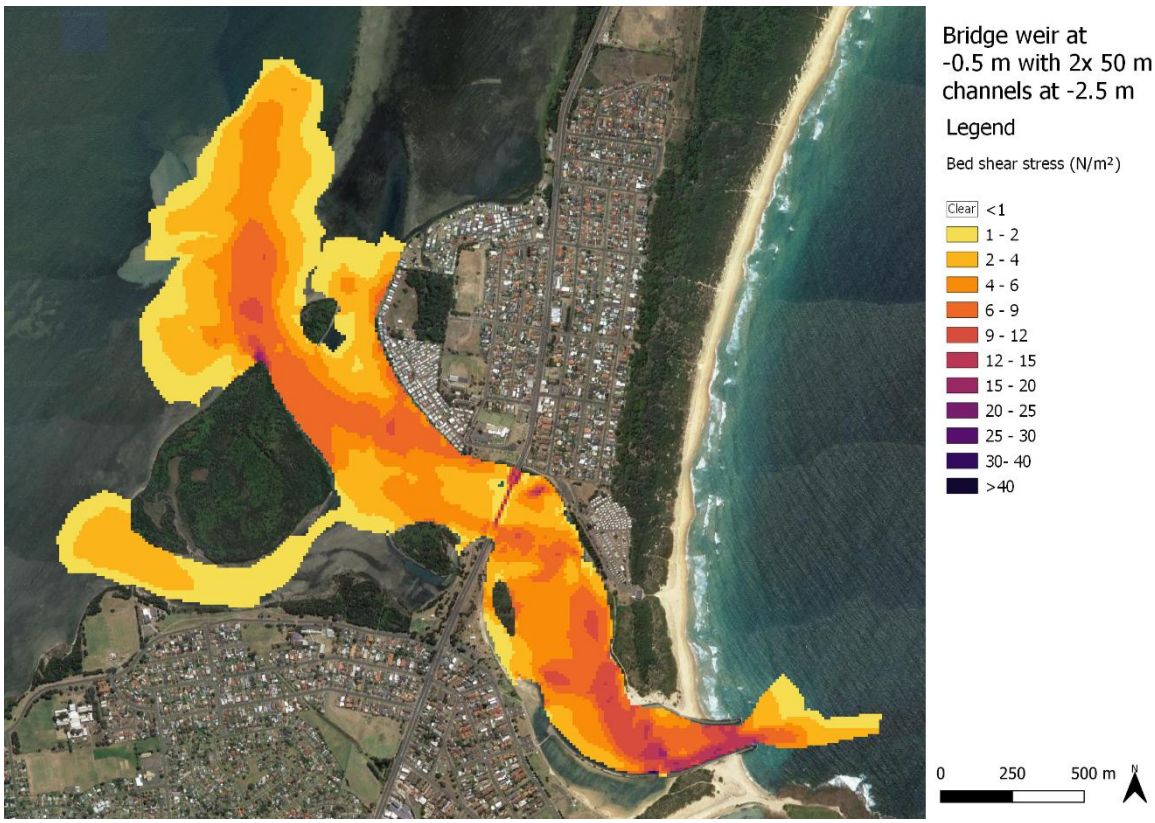


Figure D-3 Approximate maximum bed shear for modelled Option 1B (bridge weir): -0.5 m AHD weir with 2×50 m wide channels at -2.5 m AHD

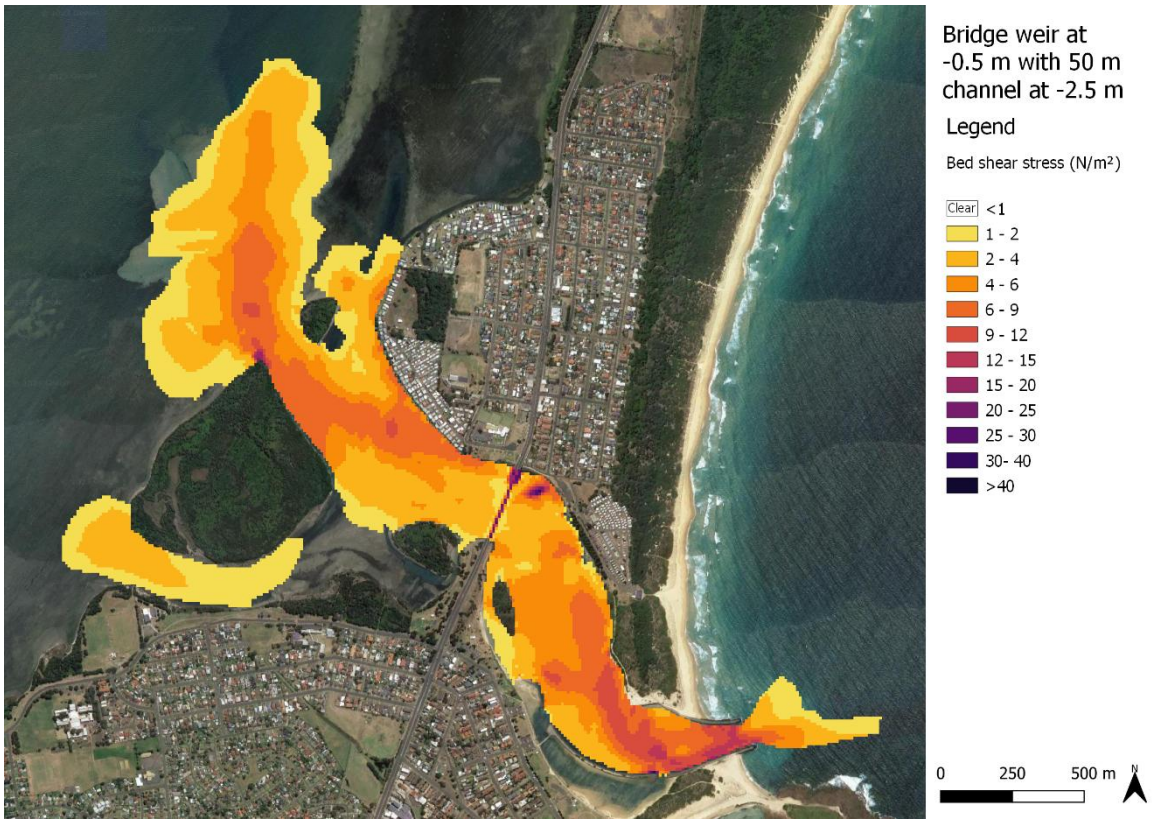


Figure D-4 Approximate maximum bed shear for modelled Option 1C (bridge weir): -0.5 m AHD weir with 1×50 m wide channel at -2.5 m AHD

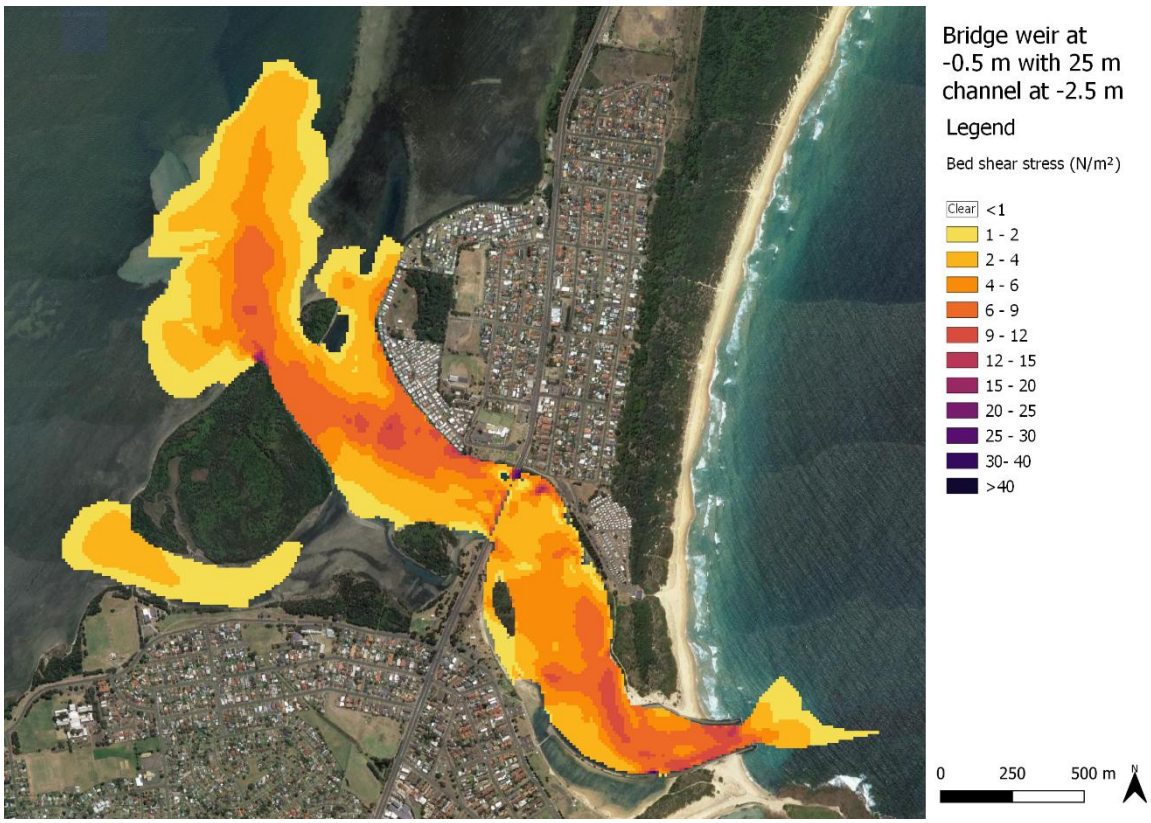


Figure D-5 Approximate maximum bed shear for modelled Option 1D (bridge weir): -0.5 m AHD weir with 1×25 m wide channel at -2.5 m AHD

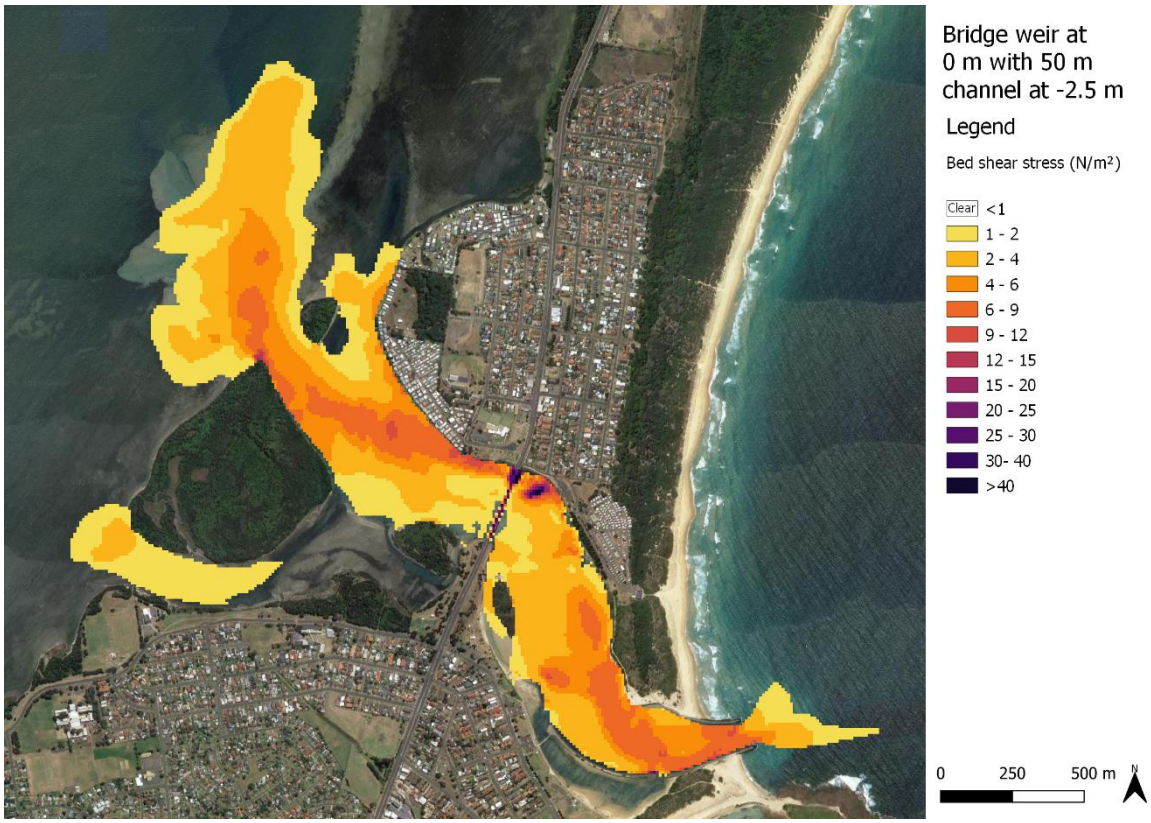


Figure D-6 Approximate maximum bed shear for modelled Option 1E (bridge weir): 0 m AHD weir with 1×50 m wide channel at -2.5 m AHD

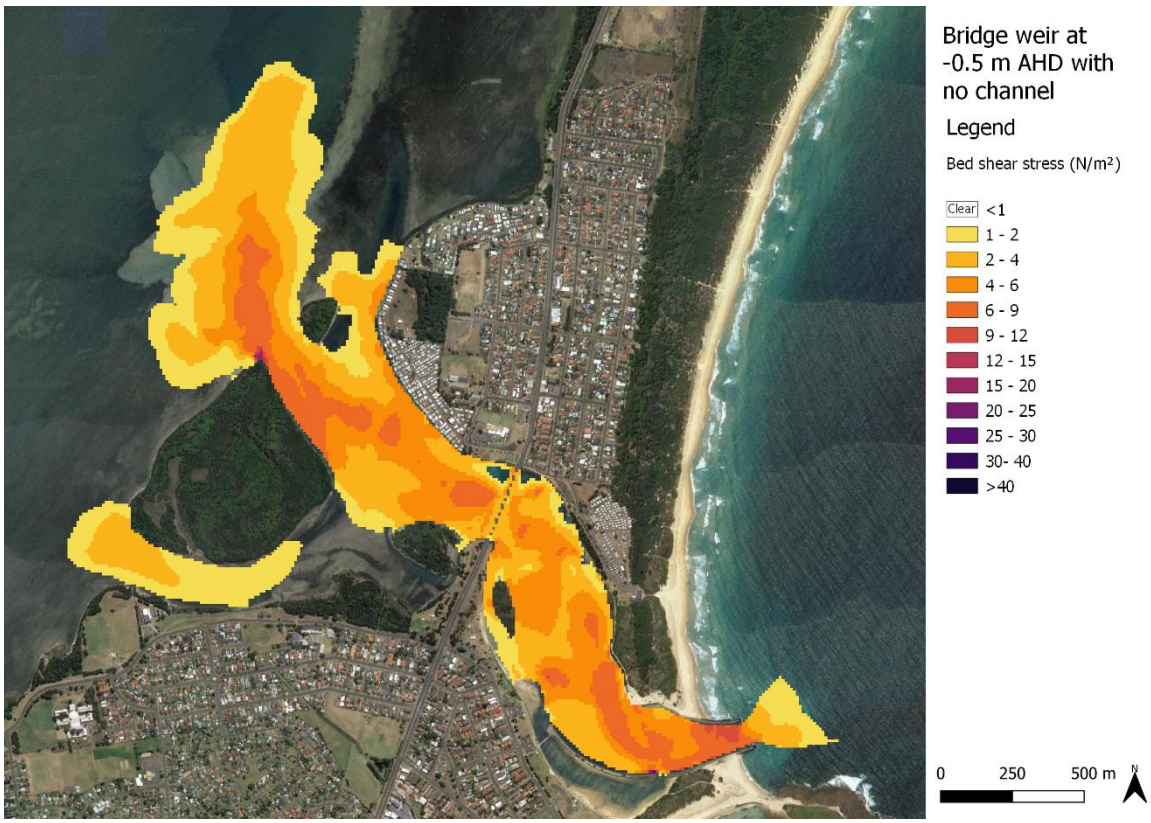


Figure D-7 Approximate maximum bed shear for modelled Option 1F (bridge weir): -0.5 m AHD weir all the way across (no channel)

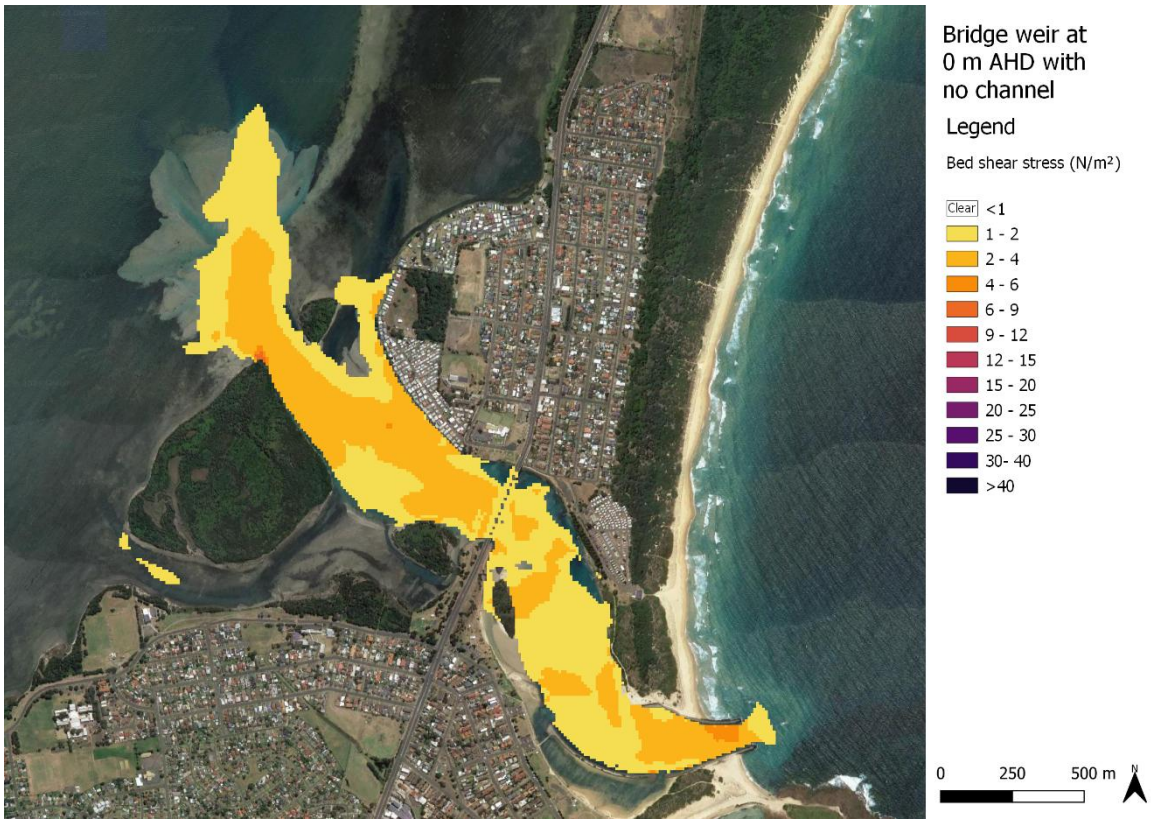


Figure D-8 Approximate maximum bed shear for modelled Option 1G (bridge weir): 0 m AHD weir all the way across (no channel)

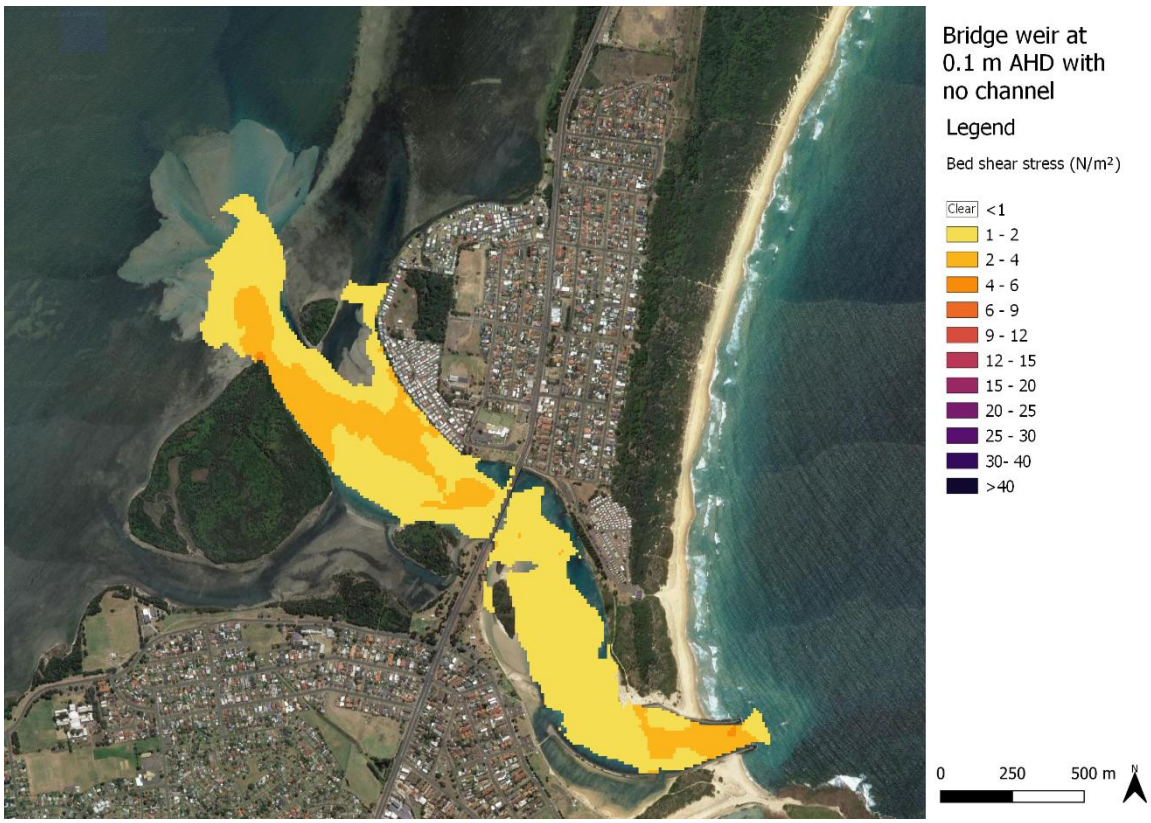


Figure D-9 Approximate maximum bed shear for modelled Option 1H (bridge weir): 0.1 m AHD weir all the way across (no channel)

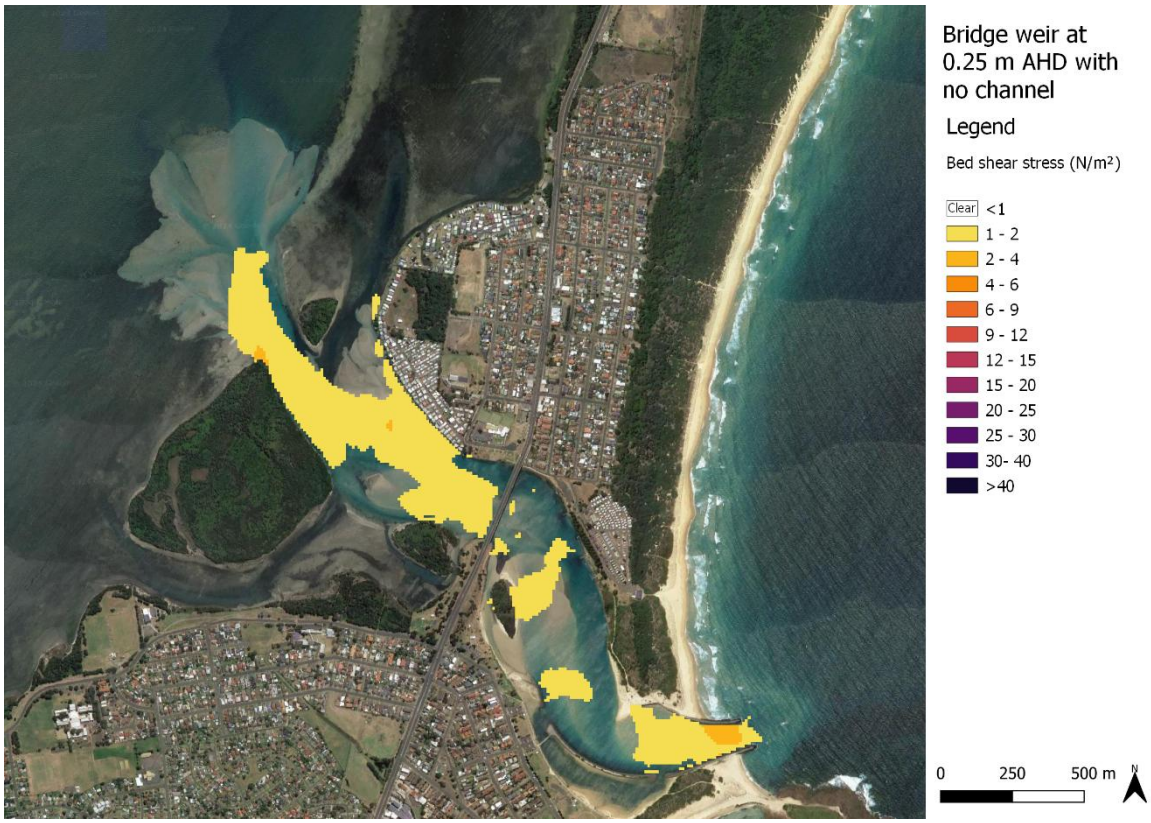


Figure D-10 Approximate maximum bed shear for modelled Option 1I (bridge weir): 0.25 m AHD weir all the way across (no channel)

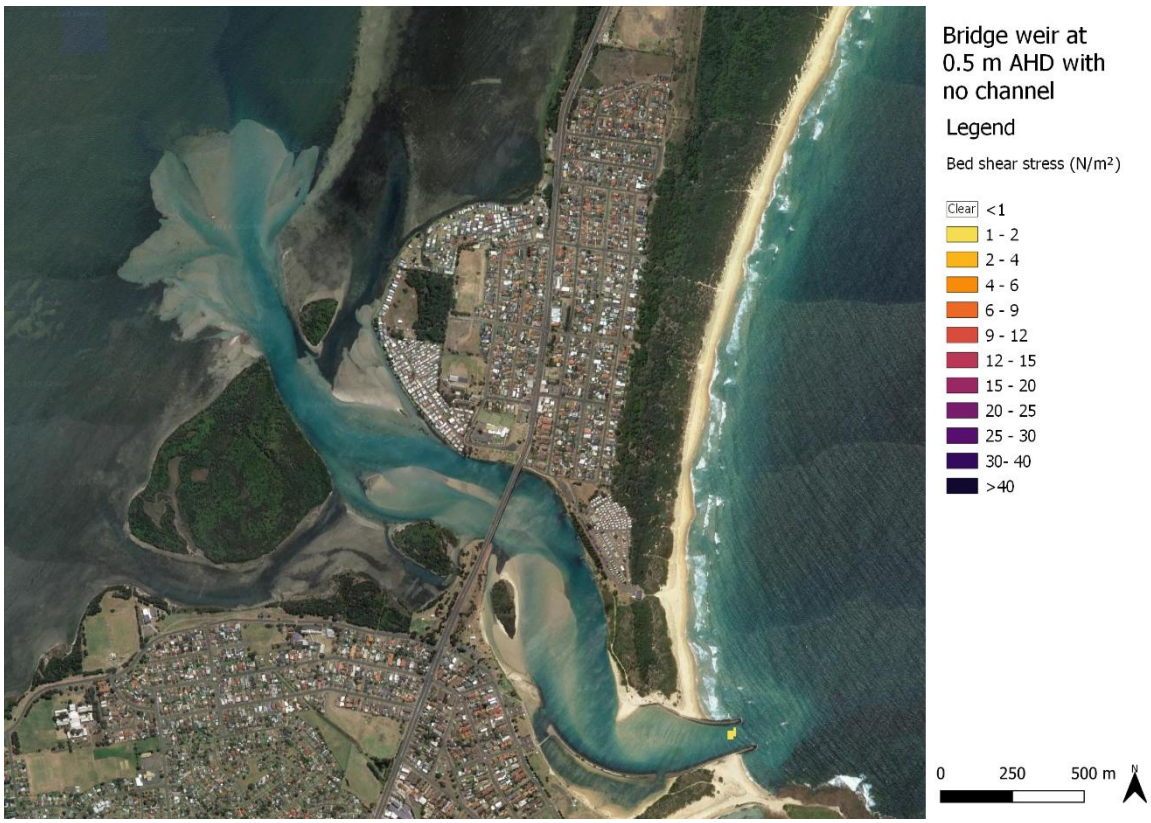


Figure D-11 Approximate maximum bed shear for modelled Option 1J (bridge weir): 0.5 m AHD weir all the way across (no channel)

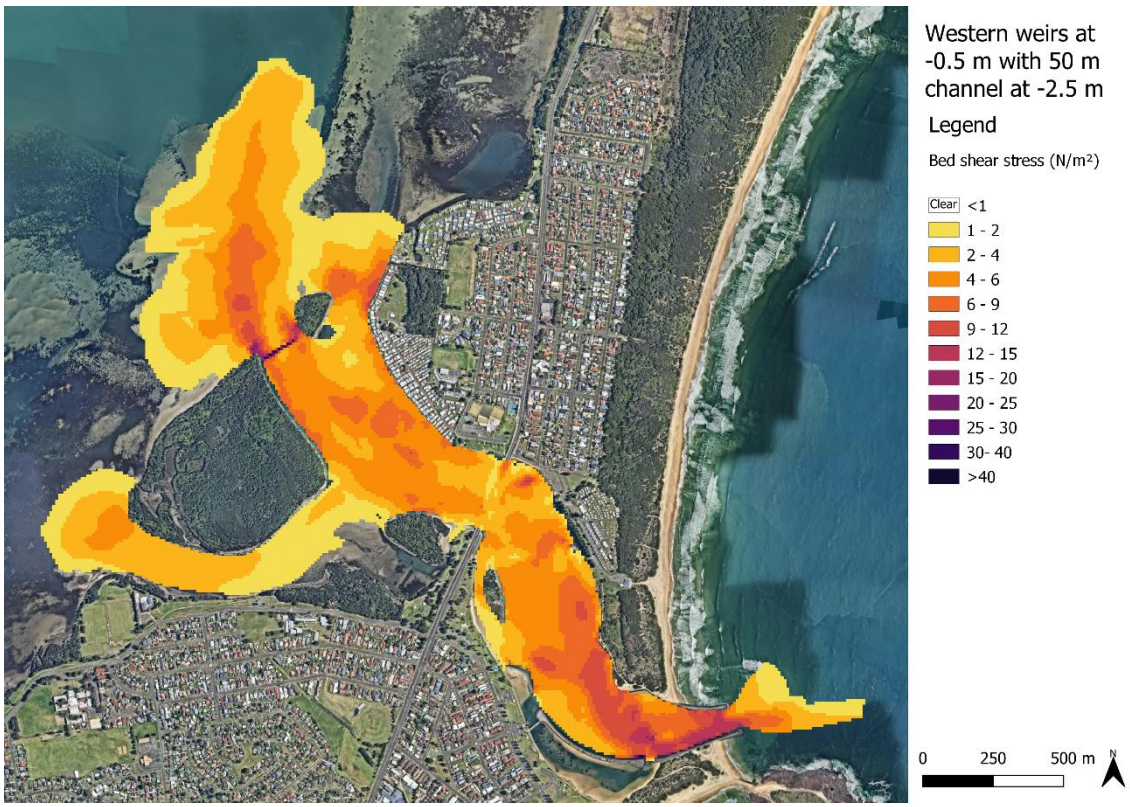


Figure D-12 Approximate maximum bed shear for modelled Option 3A (western weirs): -0.5 m AHD weir with 50 m wide channel at -2.5 m AHD (central channel); -0.5 m AHD weirs (northern and southern channels)

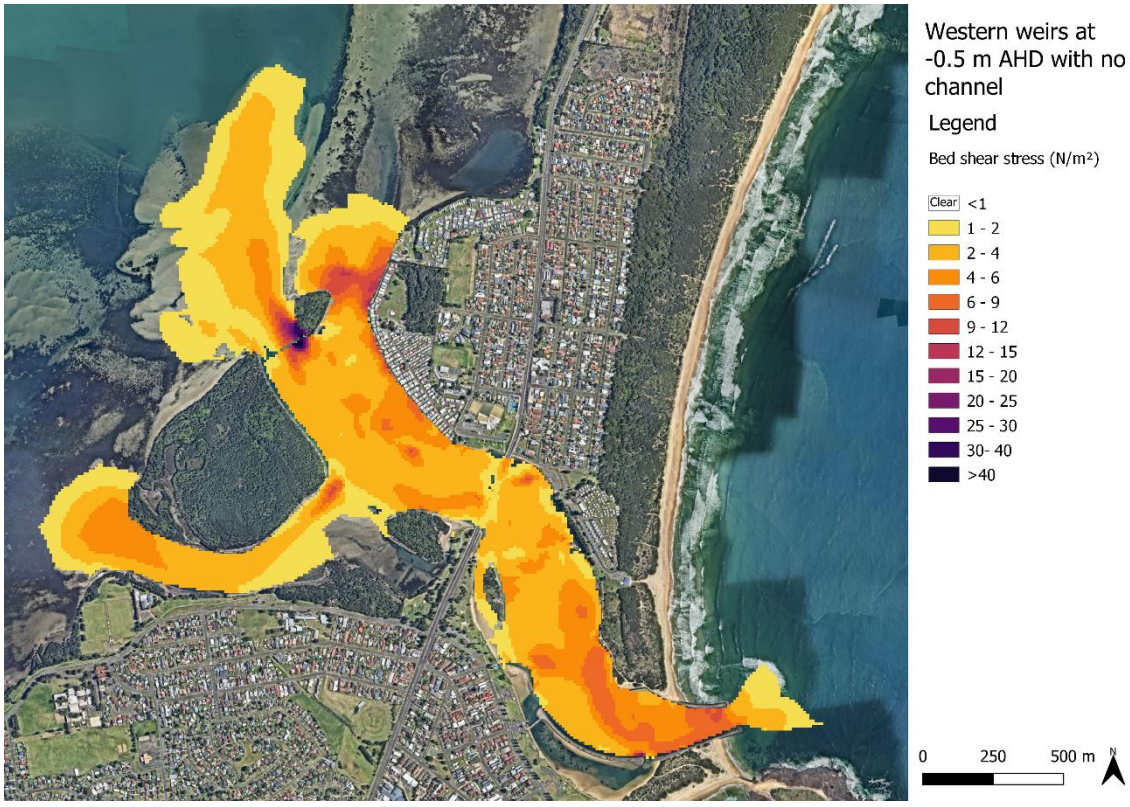


Figure D-13 Approximate maximum bed shear for modelled Option 3B (western weirs): -0.5 m AHD weirs across all three channels

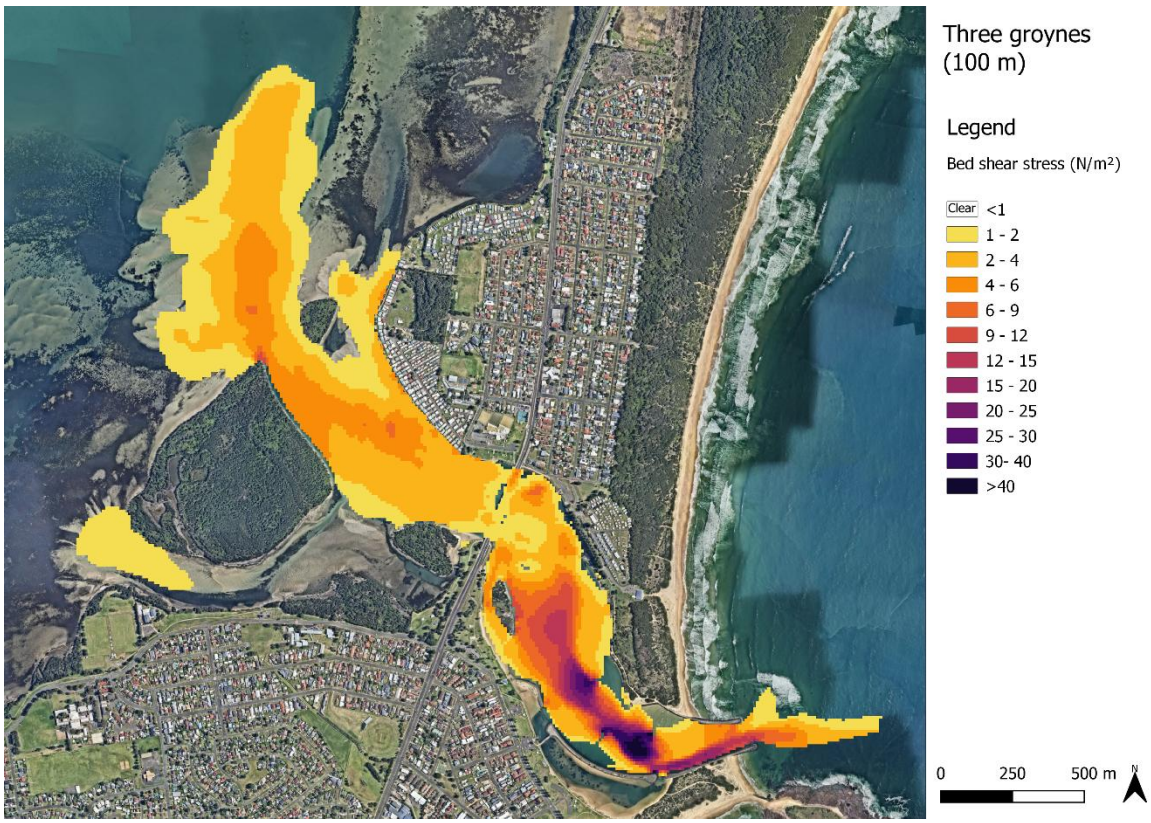


Figure D-14 Approximate maximum bed shear for modelled Option 4A (groyne field): Three 100 m long groynes

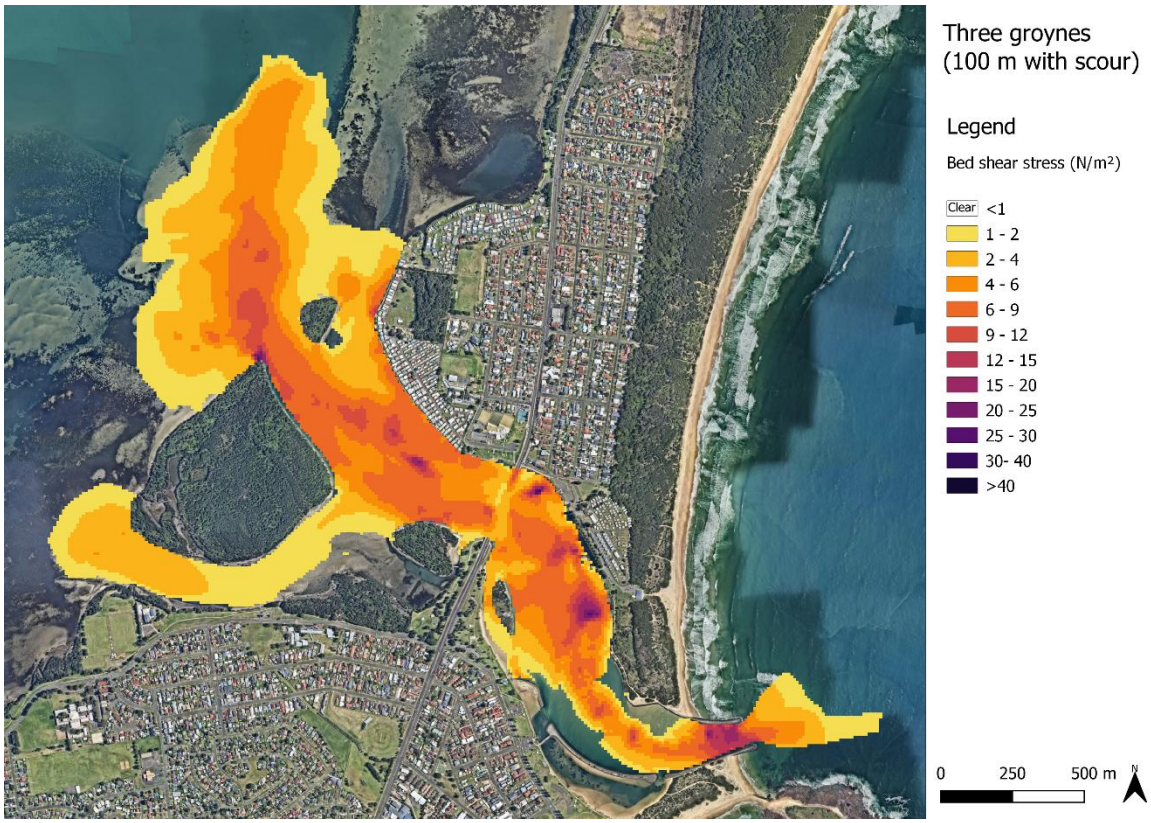


Figure D-15 Approximate maximum bed shear for modelled Option 4A (groyne field): Three 100 m long groynes (with local bed scour)

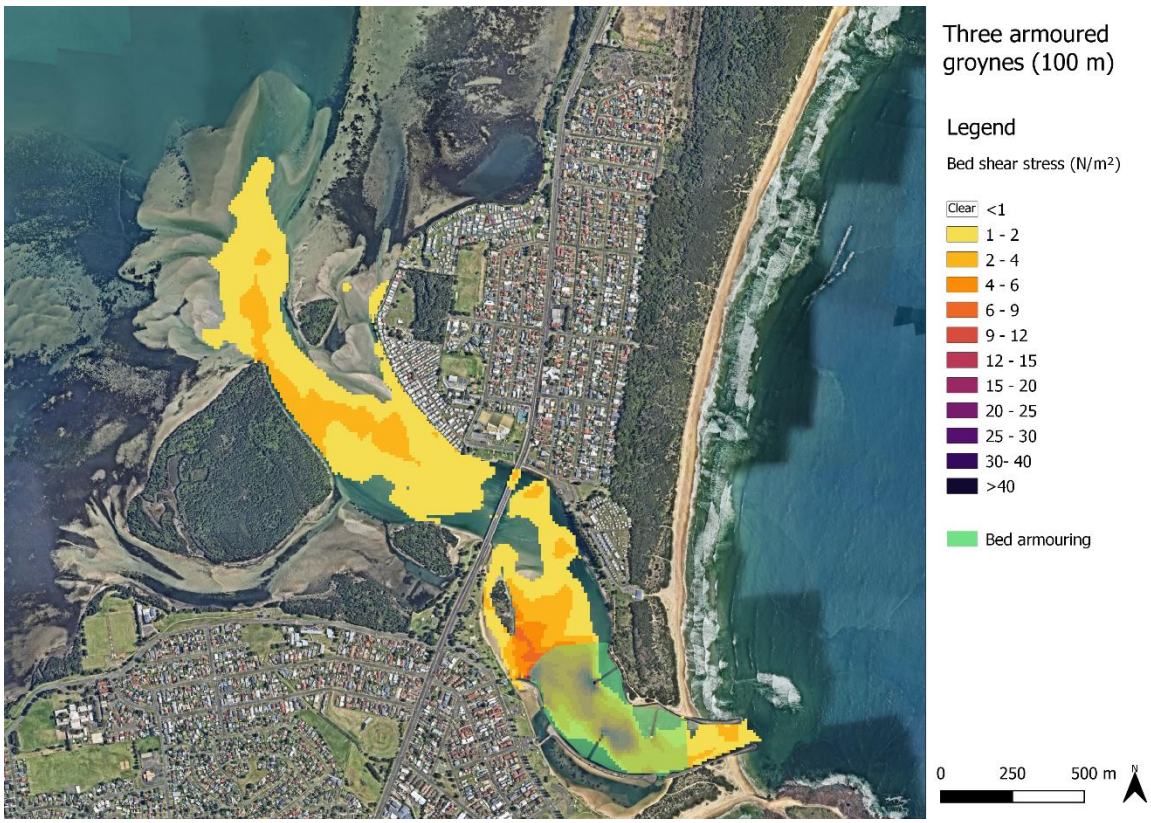


Figure D-16 Approximate maximum bed shear for modelled Option 4B (groyne field): Three 100 m long groynes (with armoured bed)

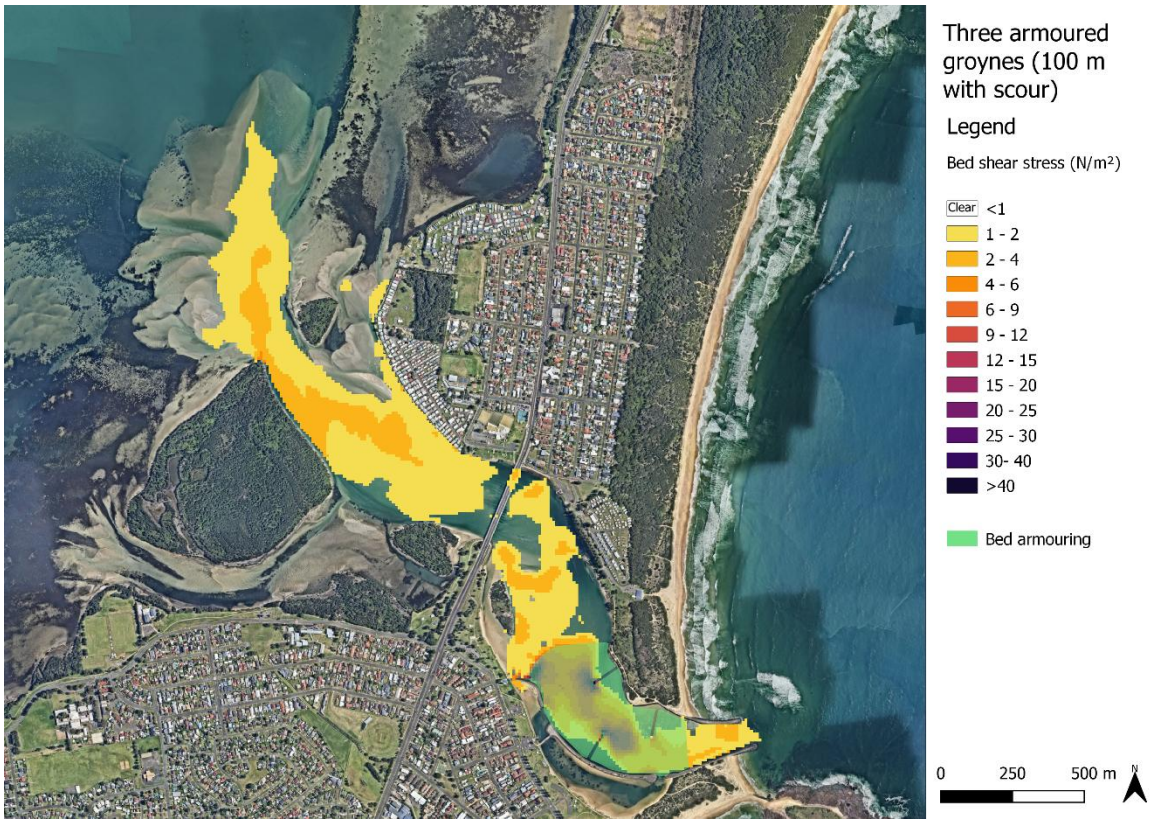


Figure D-17 Approximate maximum bed shear for modelled Option 4B (groyne field): Three 100 m long groynes (with armoured bed and local scour)

Appendix E Assumptions for preliminary cost estimates

E1 General

The cost estimates developed for the long-term management options should be considered preliminary only. It is noted that the cost estimates were intended for comparison between the options and were not prepared to the level of detail of a tender submission. WRL has not had any engagement with contractors and materials suppliers (e.g. quarries) in this area. A design working life of 50 years (contingent on ongoing maintenance) is typically adopted for types of coastal/estuarine options considered in this report, subject to subsequent detailed design and community requirements. Depreciation of assets over the design life has not been considered in the reported ongoing costs.

The costs in this assessment only explicitly consider capital construction cost and ongoing maintenance costs. WRL did not examine economic benefits (such as the value of the protected assets). The assessment also did not consider changes to ecosystem services or environmental offset costs.

Material quantities for the options were derived by WRL from the geometry included in the numerical model. Unit rates for materials were determined from information available to WRL from similar projects in Australia.

A range of costs have been provided for each option. Further information is provided below; however, the upper range typically includes increased contingency costs (from 15 to 30%) and an increase in rock costs by 50% to demonstrate the uncertainty in these estimates.

Note that these costs are quoted exclusive of Goods and Services Tax (GST) in 2024 AUD$.

It is acknowledged that the cost estimates prepared in this Stage 2 report are higher than those for similar options in the Stage 1 report (Coghlan et al., 2023). This is primarily due to revised unit rates, refined structure geometries and recommended complementary works (e.g. bed armouring with the staggered groyne field and large scale sand nourishment with removal of the training walls).

E2 Capital cost

E2.1 Structural components

The cost for supply (including delivery) and placement of rock armour (assumed density 2.65 t/m^3 and 30% void fraction) with land-based construction for groynes has been assumed to be $150 per tonne. For the associated bed armouring, supply and water-based construction from a barge/punt has been assumed to be $500 per tonne.

For weirs at the western end of the entrance channel, the cost for supply and placement of rock from a barge/punt has been assumed to be $700 per tonne. A higher rock cost for water-based construction has been adopted for the western weirs compared to bed armouring around the groynes due to more difficult site access.

For weirs comprising rock armour near Windang Bridge (options 1 and 2), the cost for supply and placement of rock from a barge/punt has been assumed to be $900 per tonne. This highest rock cost for water-based construction has been adopted due to the constraints associated with construction beneath Windang Bridge. Note that it has been assumed that the Stage 3 scour protection works under Windang Bridge, which are currently being installed at the time of writing by Transport for NSW, are complete. That is, the rock armour for the Stage 3 works is assumed to be in place and its cost is not included in the capital cost estimate for options 1 and 2. As a sensitivity test, if the weir is not constructed beneath Windang Bridge but slightly upstream or downstream of it, the cost for supply and placement of rock from a barge/punt has been assumed to be $700 per tonne. This lower rock cost has been considered because the constraints of construction beneath Windang Bridge do not apply. Note that, based on the November and December 2021 bathymetry surveys, the required rock volume is approximately the same if the weir is located within 150 m of Windang Bridge.

For option 2, the adjustable gate structures beneath Windang Bridge have an assumed construction cost of $400,000 per metre length. An upper limit fee increasing this cost by 50% has been included for sensitivity.

An upper limit fee increasing all rocks cost by 50% has been included for sensitivity.

E2.2 Large scale sand nourishment

For the large scale sand nourishment associated with option 5, the cost for supply and distribution of the sand has been given as a range of $20 to $50 per m^3 (excluding contractor profit and contingency).

E2.3 Detailed design and approvals

The cost for detailed design, environmental assessments, heritage assessments and approvals has been assumed to be $500,000 for each option.

E2.4 Training walls removal

The construction of the training walls completed in 2007 was estimated to cost $11.6M (LIA, 2011). Based on a Consumer Price Index (CPI) rise of 48.8% between March 2008 and June 2024 (ABS, 2024), the replacement cost of the breakwaters in 2024 AUD$ is estimated to be $17.3M (including contingency and contractor profit).

There is limited literature available regarding the cost to remove coastal structures. However, an artificial surfing reef (“Pratte’s Reef”), comprising sand filled geotextile containers, in California (USA) began construction in 2000 and was removed in 2010. The total cost to remove this reef was nearly identical to the construction cost (Leidersdorf, et al., 2011).

Cognisant of this example and considering that armour rock for the Lake Illawarra entrance training walls is understood to have been supplied as a by-product from nearby road construction at a reduced rate, the estimated cost for removing both training walls has been assumed to be 100% of the 2024 replacement cost (excluding detailed design of the removal process and approvals). An upper limit fee increasing this cost by 50% has been included for sensitivity.

E2.5 Contractor profit

A profit of 10% for the primary contractor has been allowed for in addition to all other costs.

E2.6 Contingency

A contingency sum of 15% has been added to the sub-total costs. This is based on standard industry practice for preliminary design estimates of this nature. An upper limit contingency cost of 30% has been included for sensitivity.

E2.7 Rounding

The preliminary total capital cost estimate reported for each option was rounded to the nearest $5M.

E3 Ongoing costs

E3.1 Structural components

The average annual cost for maintenance of rock armour in groynes and weirs has been estimated to be 1% of the initial capital cost to supply and place the rock (Gordon, 1989).

For option 2, ongoing maintenance and management costs for the adjustable gate structures beneath Windang Bridge have been assumed to be 1.5% of the initial construction cost for this component.

For option 4, the average annual cost for maintenance of the bed armouring has been assumed to be 0.5% of the initial construction cost for this component.

E3.2 ICOLL management

For option 5 (i.e. return to an ICOLL state), the average annual cost for management of the entrance (e.g. managing the beach berm to facilitate opening at lower than natural levels at a desired location, artificial opening) has been estimated to be $100,000 to $200,000 per year.