PRDW
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MUIZENBERG BEACHFRONT UPGRADE – SPECIALIST COASTAL MODELLING

Wave Overtopping and Reflection Modelling Report

REV.A

18 November 2022

City of Cape Town
Muizenberg, Cape Town, South Africa

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STAD KAAPSTAD

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1. INTRODUCTION

1.1 Project background

The Muizenberg beachfront is a Coastal Destination Place, having the highest recreational beach use in Cape Town and it is also one of the top 20 international attractions in Cape Town, attracting an estimated 90 000 foreign visitors per year, and many more local visitors daily. However, the public coastal infrastructure and services at Muizenberg are in decline.

To protect the public amenity, the City of Cape Town (CCT) is undertaking a project to rehabilitate and upgrade the coastal public infrastructure and services along the Muizenberg Beachfront. Due to the potential diverse development objectives of the eastern and the western areas, the beachfront will be upgraded in two separate initiatives. Phase 1 extends from the St James walkway to the parking area just west of the Pavilion, while Phase 2 is envisaged to extend from the Pavilion to the Zandvlei estuary mouth (Figure 1-1).

Figure 1-1: Muizenberg Beachfront location and aerial view.

The key project objectives are to:

- Retain and improve the recreational and amenity facilities along Muizenberg Beachfront to ensure a popular recreational and tourism destination is established over the long term;
- Construct a new coastal defense structure to protect the existing infrastructure and services, factoring in climate change and sea-level rise estimates;
- Ensure that such facilities are safe for public use and that such facilities optimize the use and enjoyment of the coastal environment by members of the public;
- Retreat infrastructure (excluding coastal defense infrastructure) to beyond the wave run up zone; and
- Ensure cost effective budget expenditure through ensuring the implementation of long-term sustainable coastal protection structures to protect infrastructure and services from present and future coastal hazards.

The City's Coastal Management Branch (CMB) have completed the feasibility design for Phase 1 (CCT, 2022a). The design framework for the Phase 1 development is shown in Figure 1-2, and includes the following key components:

- New stepped revetment coastal protection to replace the old wooden seawall and degraded stone steps. This is envisioned to provide continuous beach access, support and protect the promenade and other infrastructure, and preserve the sense of place and value of the beachfront;
- Refurbishment of hard and soft landscaping and amenities along the beachfront as well as an improved connection to the St James coastal walkway;
- Formalising and optimizing of the large informal parking area in the west of the site; and
- Reconfiguration of the parking area adjacent the Pavilion building (eastern boundary of the site).

Subsequent to the feasibility design for Phase 1 the designs have been updated as part of the Concept Design Stage, including a revised layout of the parking areas, landscaping, stepped revetment and promenade, notably including a 1 m setback of the stepped revetment (CCT, 2022b). The revised design framework for the Phase 1 development is shown in Figure 1-2.

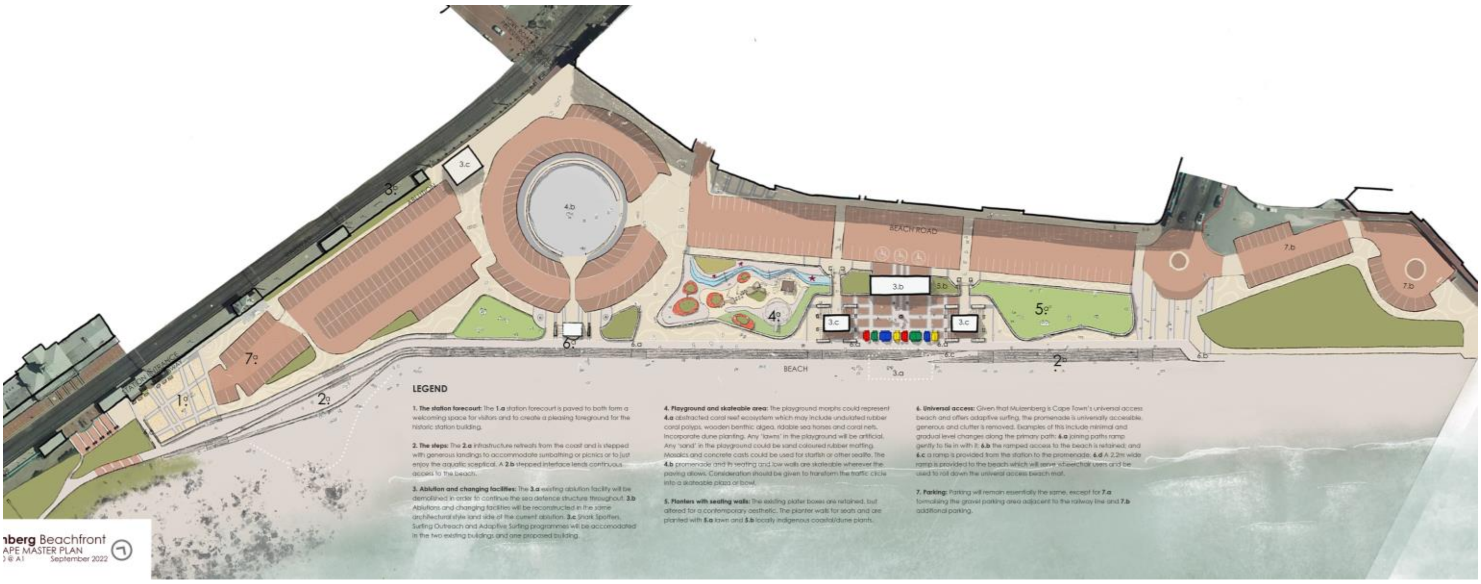


Figure 1-2: Muizenberg Beachfront design framework (CCT, 2022b).

1.2 Terms of reference

The CCT has appointed a panel of consultants for the Inception and Concept & Viability (preliminary design) stages of the Phase 1 project. PRDW has also been appointed directly by the CCT to provide coastal processes specialist studies required as input to the next design stage, which are the subject of this report.

The Terms of Reference for the specialist studies are:

- Bathymetric survey.
- Coastal hydrodynamics modelling study to determine design and construction water level and wave conditions, including climate change over the design life.
- Sediment dynamics study to determine minimum expected sediment levels in front of the seawall to design and optimise scour protection where required, and an assessment of the impact of the proposed revetment on the longshore and cross-shore sediment dynamics.
- Overtopping and flooding assessment and recommendations.
- Wave reflection analysis of the potential impact on surfing conditions.

The first three items in the Terms of Reference were addressed in the *Wave and Sediment Transport Modelling Report* (PRDW, 2022a). This report is a continuation of the previous study and covers the overtopping and resultant flooding for the project at the proposed levels and analyses the changes in wave reflection between the existing seawall and proposed revetment, and the potential impact on surfing conditions.

1.3 Scope of work

The Scope of Work includes a coastal hydrodynamics modelling study using the MIKE 3 Wave model comprising the following:

- Calibration of the wave model roughness height to resolve overtopping and reflection from the proposed revetment.
- Overtopping and flooding assessment:
  - Numerical flume (two-dimensional vertical, 2DV) simulations to quantify overtopping for the following cases:
    - 3x climate change horizons: 2026, 2046 and 2076
    - 3x cross-shore profile types (variability in beach levels): typical accreted and eroded profiles, and an extreme storm-eroded profile
    - 3x storm severities: 1-, 10-and 100-yr storm event.
  - The numerical flume simulations will test the sensitivity of overtopping and resulting flooding to:
    - climate change projection: SSP5-8.5 (the most conservative scenario) versus SSP1-2.6 (a low emissions scenario) for the proposed revetment;
    - design crest level: comparison of a 4.5 m MSL crest to the 3.5 m MSL proposed crest level; and
    - comparison of the existing seawall to the proposed revetment.
  - Full domain (three-dimensional, 3D) simulations to quantify overtopping of the proposed revetment and the resulting landside flooding for the following cases:
    - 1-y storm, 2026 (SSP5-8.5)
    - 1-y storm, 2076 (SSP5-8.5)
    - 100-y storm, 2046 (SSP5-8.5)

- 100-y storm, 2076 (SSP5-8.5)
- Analysis of changes in wave reflection for the proposed revetment:
  - Numerical flume simulations to quantify and compare the amount of reflected wave energy for existing seawall and the proposed revetment for the following cases:
    - 3x climate change scenarios: 2026 (SSP5-8.5), 2046 (SSP1-2.6 and SSP5-8.5)
    - 2x typical surfing conditions
  - Full domain simulations to quantify and compare the amount of reflected wave energy for existing seawall and the proposed revetment for two typical surfing conditions and a 2026 (SSP5-8.5) climate change scenario.

1.4 Report structure

Section 2 discusses the generic model inputs for this study, including the calibration of the wave model roughness height to parameterise the unresolved friction on the proposed revetment. The overtopping and flooding assessment for the proposed revetment is presented in Section 3. Section 4 analyses the changes in wave reflection between the existing seawall and proposed revetment, and the potential impact on surfing conditions. A summary of the outcomes is presented in Section 5 and a list of references is provided in Section 6.

2. MODEL SETUP

2.1 Introduction

This section describes the wave model used to simulate the overtopping, flooding and wave reflection assessments. An overview of the generic model inputs is summarised below, including the bathymetry, proposed revetment calibration and nearshore wave conditions.

2.2 Model description

The MIKE 3 Wave (M3W) Flexible Mesh model was used for the coastal flooding modelling. The application of the model is described in the User Manual (DHI, 2022a), while full details of the physical processes being simulated and the numerical solution techniques are described in the Scientific Documentation (DHI, 2022b).

The M3W is a phase-resolving wave model based on the 3D Navier-Stokes equations. An unstructured (flexible) mesh is used in the horizontal dimension with sigma layers in the vertical. The model includes the following processes:

- Wave refraction;
- Wave diffraction;
- Wave reflection;
- Bottom friction;
- Non-linear wave transformation;
- Surf and swash zone hydrodynamics;
- Wave breaking and run-up;
- Wave overtopping; and
- Coastal flooding.

The model is based on the numerical solution of the three-dimensional incompressible Reynolds-averaged Navier-Stokes equations. Thus, the model consists of continuity and momentum equations, and it is closed by a k-ε turbulence closure scheme in the vertical and horizontal. The free surface is taken into account using a sigma coordinate transformation approach. The spatial discretization of the governing equations in conserved form is performed using a cell-centred finite volume method. The time integration is performed using a semi-implicit scheme. The vertical convective and diffusion terms are discretized using an implicit scheme to remove the stability limitations associated with the vertical resolution. The remaining terms are discretized using a second-order explicit Runge-Kutta scheme. The projection method is employed for the non-hydrostatic pressure. The interface convective fluxes are calculated using a HLLC approximate Riemann solver. This shock-capturing scheme enables robust and stable simulation of flows involving shocks or discontinuities such as bores and hydraulic jumps. This is essential for modelling of waves in the breaking zone. The numerical dissipation accounts for the dissipation in the breaking waves.

2.3 Bathymetry and topography

2.3.1 Available datasets

The bathymetry and topography datasets used in this study have been combined from the following sources:

- MIKE by DHI CMAP Electronic Charts Database (DHI, 2022c)
- Multi-beam echo sounder survey by Tritan Survey (2022)
- LiDAR data from four surveys (between 2013 and 2021)
- Beach profile from measurements between 2004 and 2010 by CCT
- Spot height measurements taken for the Transport and Urban Development Authority (TDA) in 2018
- Beach profile surveys by Tritan Survey (2022)
- Landside design levels for the revised Muizenberg Beachfront design framework by HHO (HHO, 2022).

Three cross-shore configurations were considered for the upper beach profile in the numerical flume models. Typical eroded and accreted beach states were estimated by applying a ±5 m horizontal offset to the average beach profile from a set of beach profile measurements as shown in Figure 2-1. The location of the measurements is shown at Profile B in Figure 2-2. Where available, extreme storm-eroded profiles from SBEACH (Storm-induced BEAch Change) modelling were also considered for the upper beach profile. Further details of the cross-shore profiles are provided in the *Wave and Sediment Transport Modelling Report* (PRDW, 2022a).Due to the alongshore uniformity of the beach the upper profiles were assumed to be representative of the beach in front of the proposed revetment.

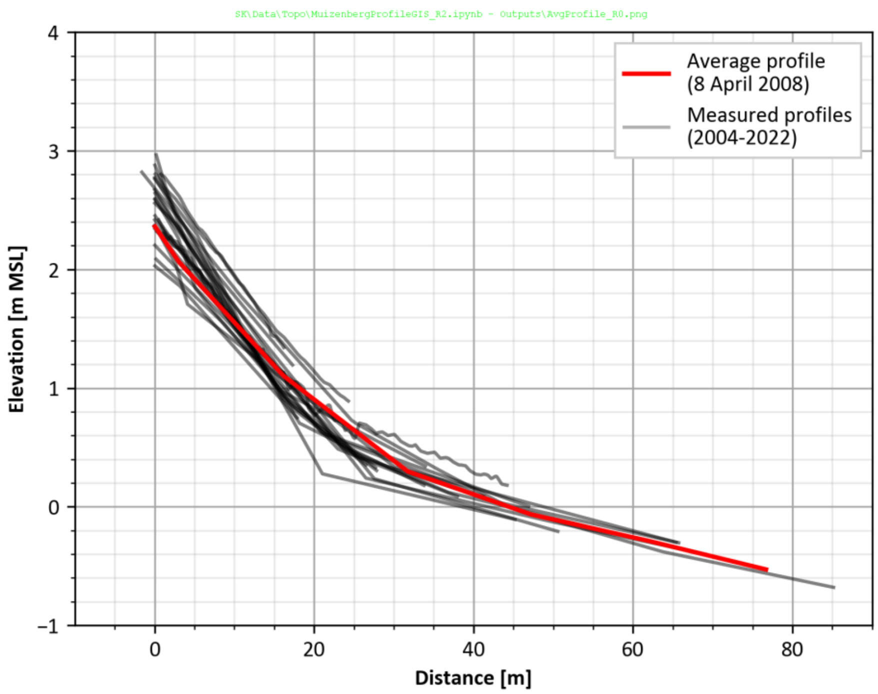


Figure 2-1: Average profile (measured on 8 April 2008) compared to the envelope of all measurements (2004 to 2022) along Profile B (refer to Figure 2-2).

Figure 2-2: Locations of Profiles A, B and C where historical beach profile data is available (2004-2010). Also shown is the proximity of the Tritan beach survey (coloured squares) relative to the profile locations.

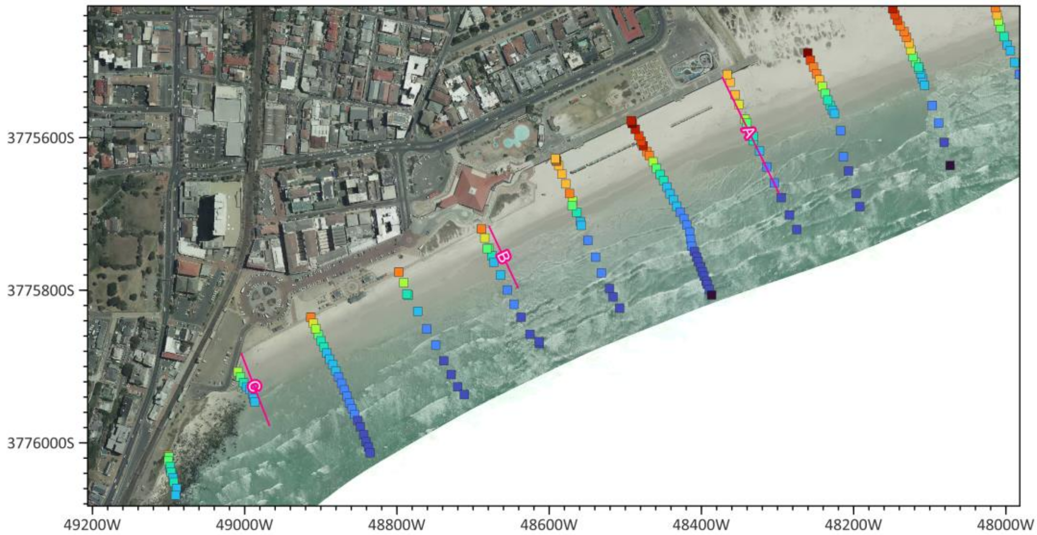


Figure 2-2: Locations of Profiles A, B and C where historical beach profile data is available (2004-2010). Also shown is the proximity of the Tritan beach survey (coloured squares) relative to the profile locations.

For the full domain model a single upper beach profile configuration based on a typical eroded beach was considered. The profile was applied along the full length of the beach.

2.3.2 Adjustments for long-term trends

The cross-shore profiles for the numerical flume models were adjusted for the long-term accretion trend due to longshore sand supply (+0.22 m/year) and coastline recession due to sea level rise (SLR) for 2026, 2046 and 2076. The median SLR projections for SSP5-8.5 (the most conservative scenario) and SSP1-2.6 (a low emissions scenario) were considered. For the full domain the combined bathymetry and topography dataset was divided into 500 cross-shore transects spaced at approximately 5 m intervals. The individual profiles were adjusted for long-term trends (2026, 2046 and 2076 – SSP5-8.5 only) and recombined to a single dataset. Details of the long-term trends and cross-shore adjustment methodology are described in the *Wave and Sediment Transport Modelling Report* (PRDW, 2022a).

Figure 2-3, Figure 2-4 and Figure 2-5 present the 2026, 2046 and 2076 bathymetries adjusted for long-term trends with the proposed revetment.

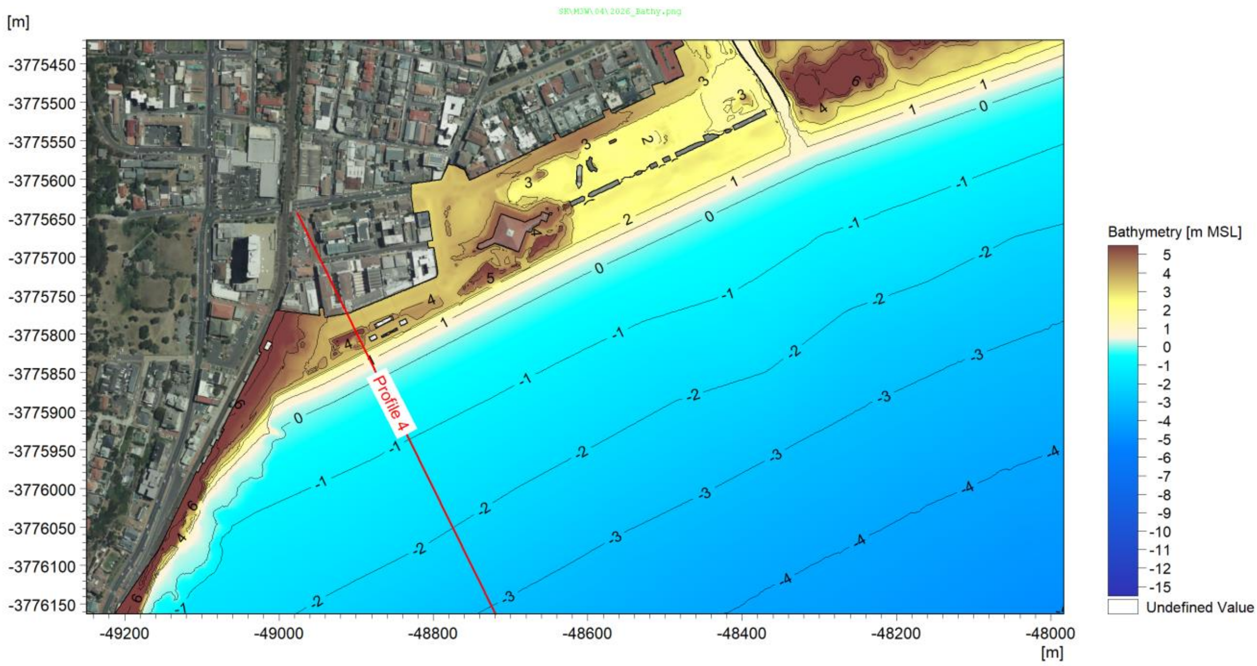


Figure 2-3: Bathymetry for a typical eroded beach level, adjusted for coastline recession due to sea level rise and accretion due to longshore sand supply to 2026. A cross-section at Profile 4 was used for the numerical flume models.

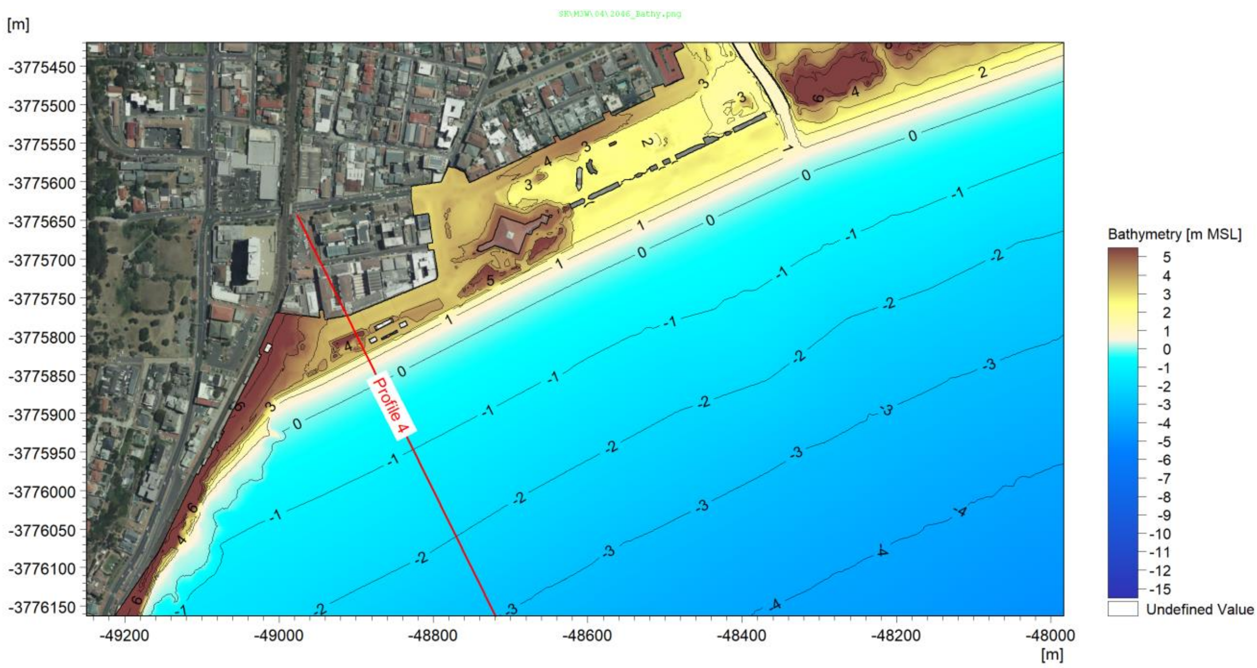


Figure 2-4: Bathymetry for a typical eroded beach level, adjusted for coastline recession due to sea level rise and accretion due to longshore sand supply to 2046. A cross-section at Profile 4 was used for the numerical flume models.

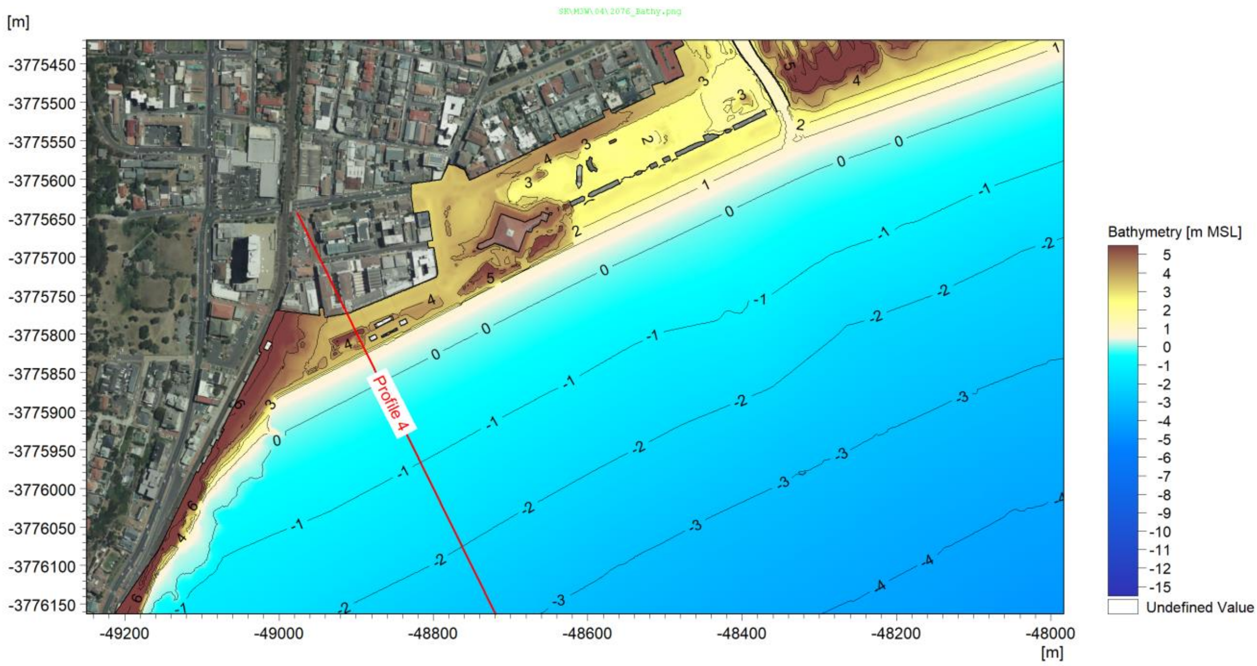


Figure 2-5: Bathymetry for a typical eroded beach level, adjusted for coastline recession due to sea level rise and accretion due to longshore sand supply to 2076. A cross-section at Profile 4 was used for the numerical flume models.

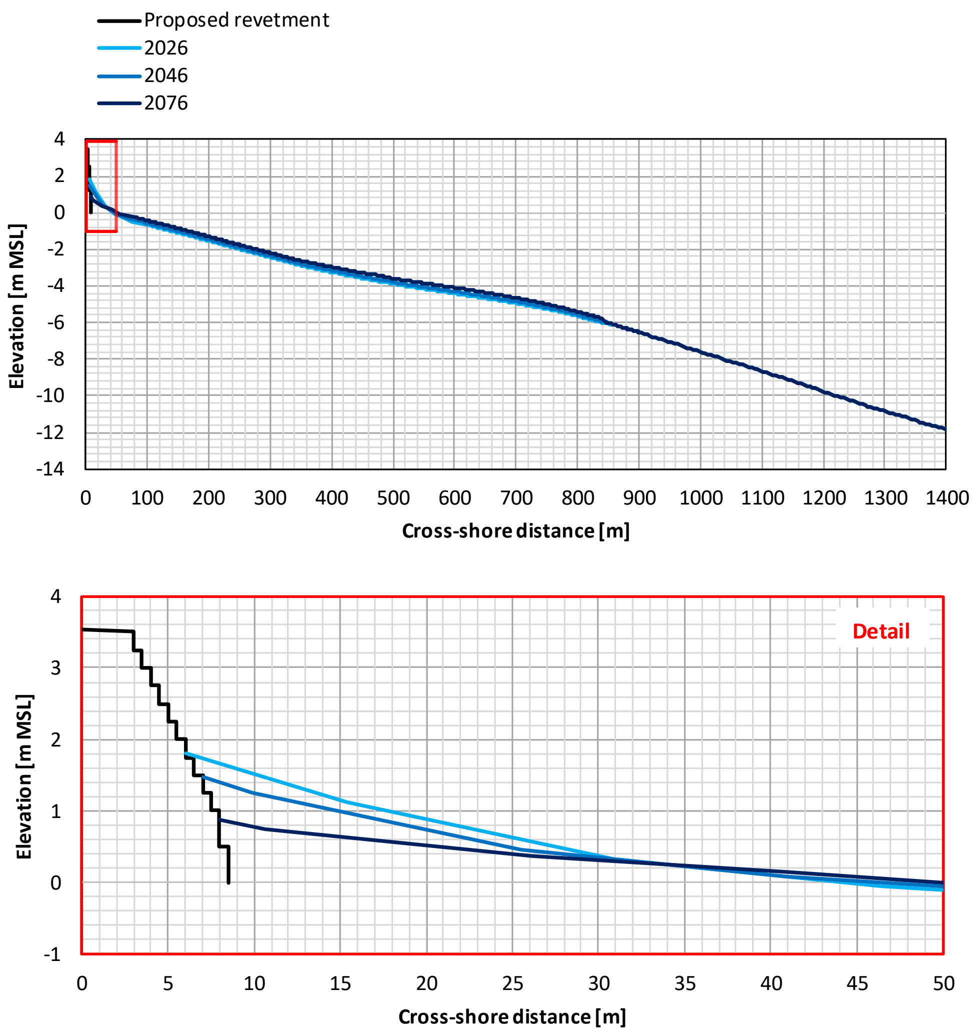


Figure 2-6: Cross-shore transect at Profile 4 for a typical eroded beach level, adjusted for coastline recession due to sea level rise and accretion due to longshore sand supply to 2026, 2046 and 2076.

2.4 Stepped revetment calibration

The main longitudinal promenade area will have 0.25 m x 0.5 m steps (typical cross-section shown in Figure 2-7) with 0.17 m easy climb step sections at regular intervals, while the step heights at Surfers Corner will increase to 0.5 m and include wider seating areas. A plan view of the step height configuration is shown in Figure 2-8.

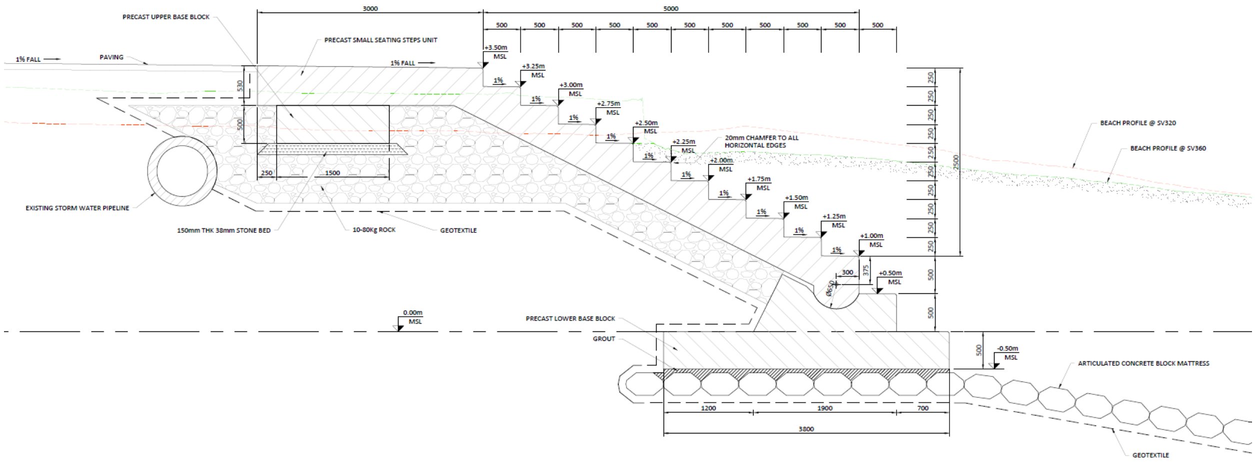


Figure 2-7: Cross-section of proposed revetment (PRDW, 2022b)

Figure 2-8: Plan view of step height configuration for the proposed revetment.

Considering the computational expense of running the M3W model at a sufficiently fine scale to resolve the individual steps of the proposed structure, the approach in this study was to resolve the average structure slope and to parameterise the additional friction on the slope caused by the individual steps. The applied friction was calibrated by comparing modelled overtopping discharge to results from physical model experiments.

Schoonees, et al. (2021) conducted full-scale flume experiments investigating wave overtopping discharge on two cross-sections with uniform step heights of 0.17 m and 0.50 m, respectively, and a slope of 1:3 each (refer to Figure 2-9). The purpose of the study was to estimate influence factors for roughness, γ_f, with which the wave overtopping discharge at stepped revetments can be estimated using the EurOtop overtopping formulae (EurOtop, 2018).

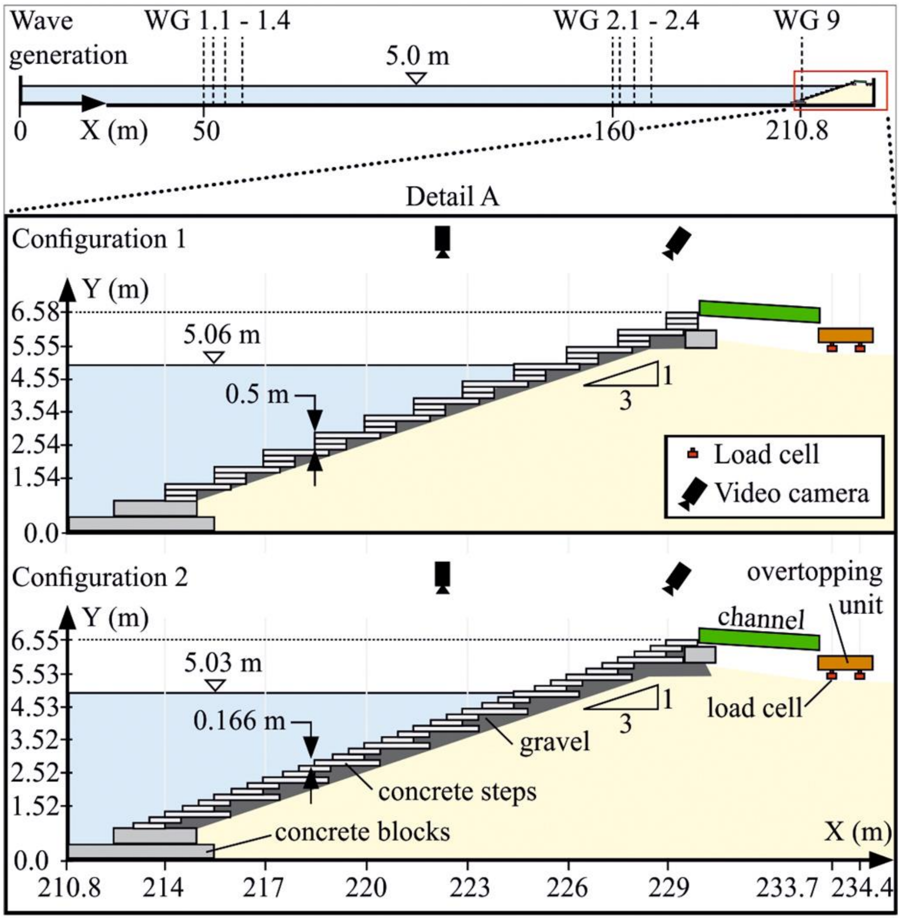


Figure 2-9: Physical model configuration to investigate overtopping rates for stepped revetments (Schoonees, et al., 2021).

The physical model setup was replicated in the MIKE Wave 3 model at physical model-scale. A numerical mesh representing the physical model configuration was set up for both step height configurations. The meshes comprised quadrangular elements one element wide with a resolution of 0.5 m. The stepped revetment was resolved with a 3:1 slope and a roughness height tuned to obtain the correct overtopping discharges.

Sixteen wave configurations were run in total covering the following range of parameters:

- H_{m0}: 0.77 m to 1.07 m
- $T_{m-1,0}$: 4.14 s to 6.52 s
- Wave steepness ($s_{m-1,0}$): 1.8% to 3.2%
- Crest freeboard: 1.52 m

The model output was the cumulative overtopped volume over the crest of the stepped revetment. The outputs from each run were processed to obtain overtopping discharge per unit width. The duration of the overtopping measurements excluded the warm-up period. Figure 2-10 presents an example of model outputs for a single case.

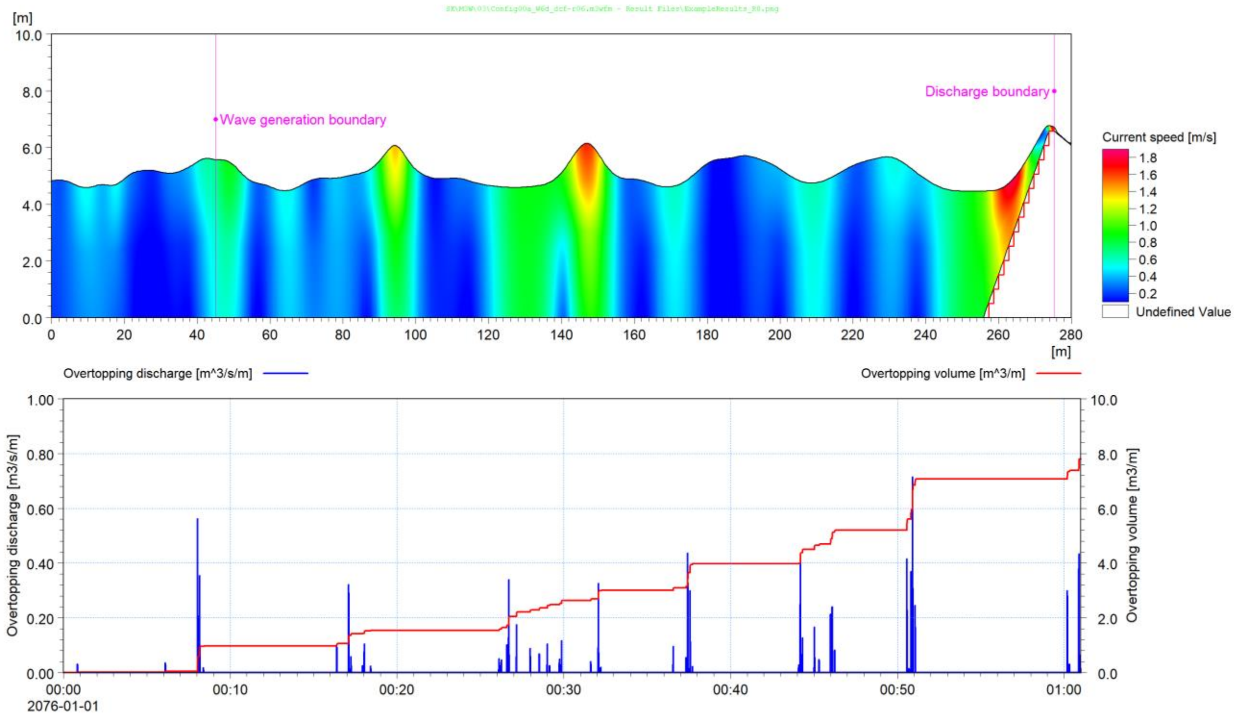


Figure 2-10: Example model outputs for a single overtopping case. Top: cross-section of 2DV model showing instantaneous surface elevation, orbital speed. The stepped revetment (red) is shown for visual reference only. Bottom: Time-series of overtopping discharge and accumulated volume over the crest of the stepped revetment.

The measured and modelled overtopping discharges were compared for a range of roughness heights, k_s, in order to find the most suitable value representative of each stepped revetment configuration. The roughness height only varies as a function of the step height and is therefore independent of the incident wave conditions. The modelled overtopping discharges were also compared to the theoretical EurOtop overtopping rates using a roughness influence factor prescribed by Schoonees, et al. (2021). The theoretical comparison was also used to determine the appropriate k_s for step heights of 0.25 m in lieu of physical results.

Figure 2-11 presents a comparison of modelled, measured and theoretical overtopping rates for step heights of 0.17 m, 0.25 m and 0.50 m. The results show the range of k_s tested and the most accurate value is outlined in red.

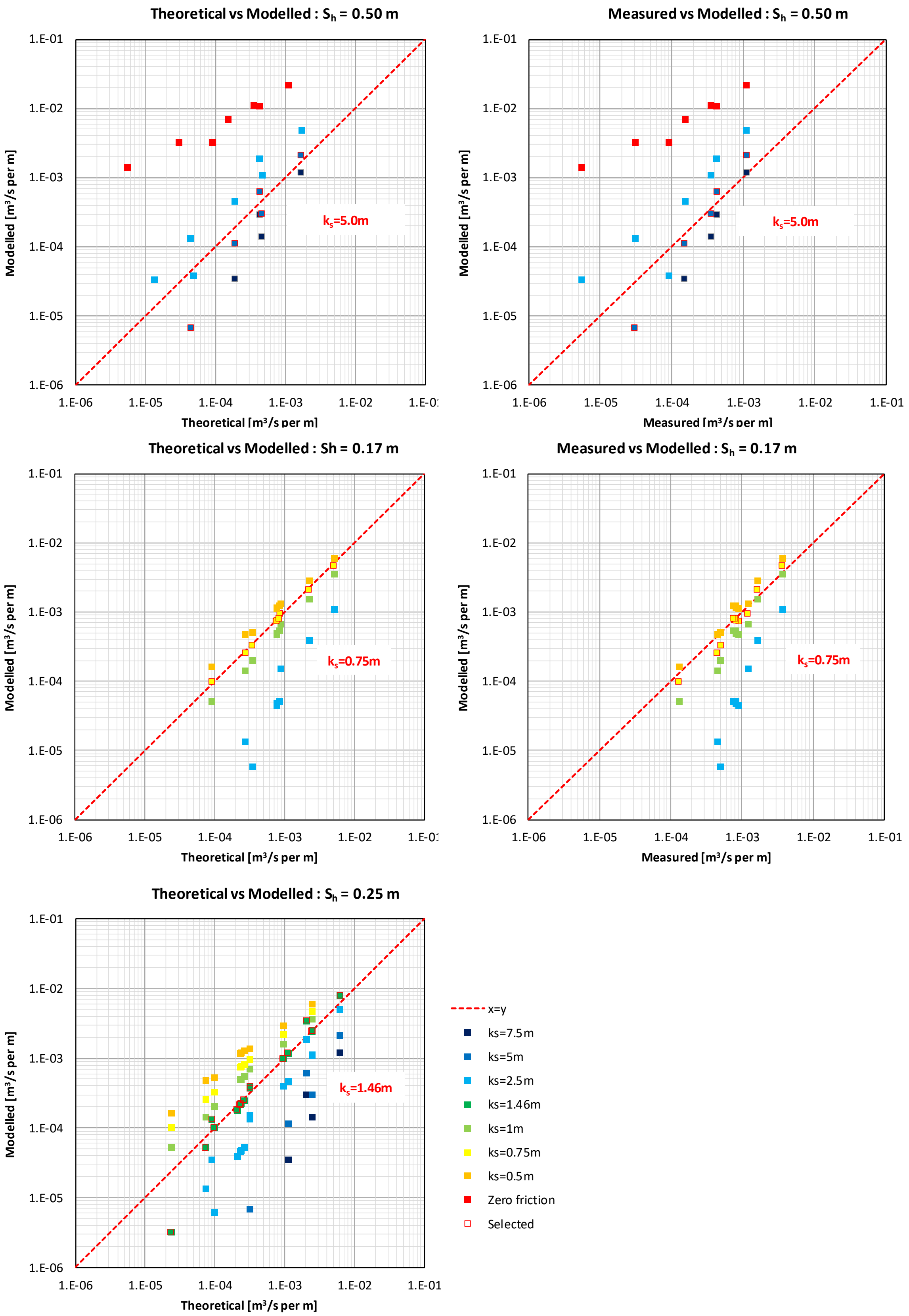


Figure 2-11: Comparison of modelled, measured and theoretical overtopping rates for step heights of 0.50 m (top), 0.17 m (middle) and 0.25 m (bottom) for a range of roughness heights. The most appropriate roughness height in each scenario is outlined in red.

The results show that the model can accurately predict the overtopping rates for a stepped revetment using the roughness height formulation on a smooth slope. The following roughness heights can be applied for a given step height:

- 0.17 m: k_s = 0.75m
- 0.25 m: k_s = 1.46m
- 0.50 m: k_s = 5.00m

2.5 Nearshore wave conditions

Over 42 years of available hindcast wave data were used to model the nearshore operational and extreme wave heights in the *Wave and Sediment Transport Modelling* study (PRDW, 2022a). The study showed two partitions of waves arriving at Muizenberg: larger wave heights with shorter wave periods driven by strong south-easterly winds over a long, unobstructed fetch, and smaller wave heights with longer wave periods typically associated with south-westerly swells refracting around Cape Point.

Figure 2-12 presents an example refraction plot showing contours of significant wave height (H_{m0}) and the output locations of two nearshore points: Point A (-15 m MSL) and Point B (-10 m MSL). The wave parameters at Point A were used to characterise the wave conditions near Muizenberg for this study.

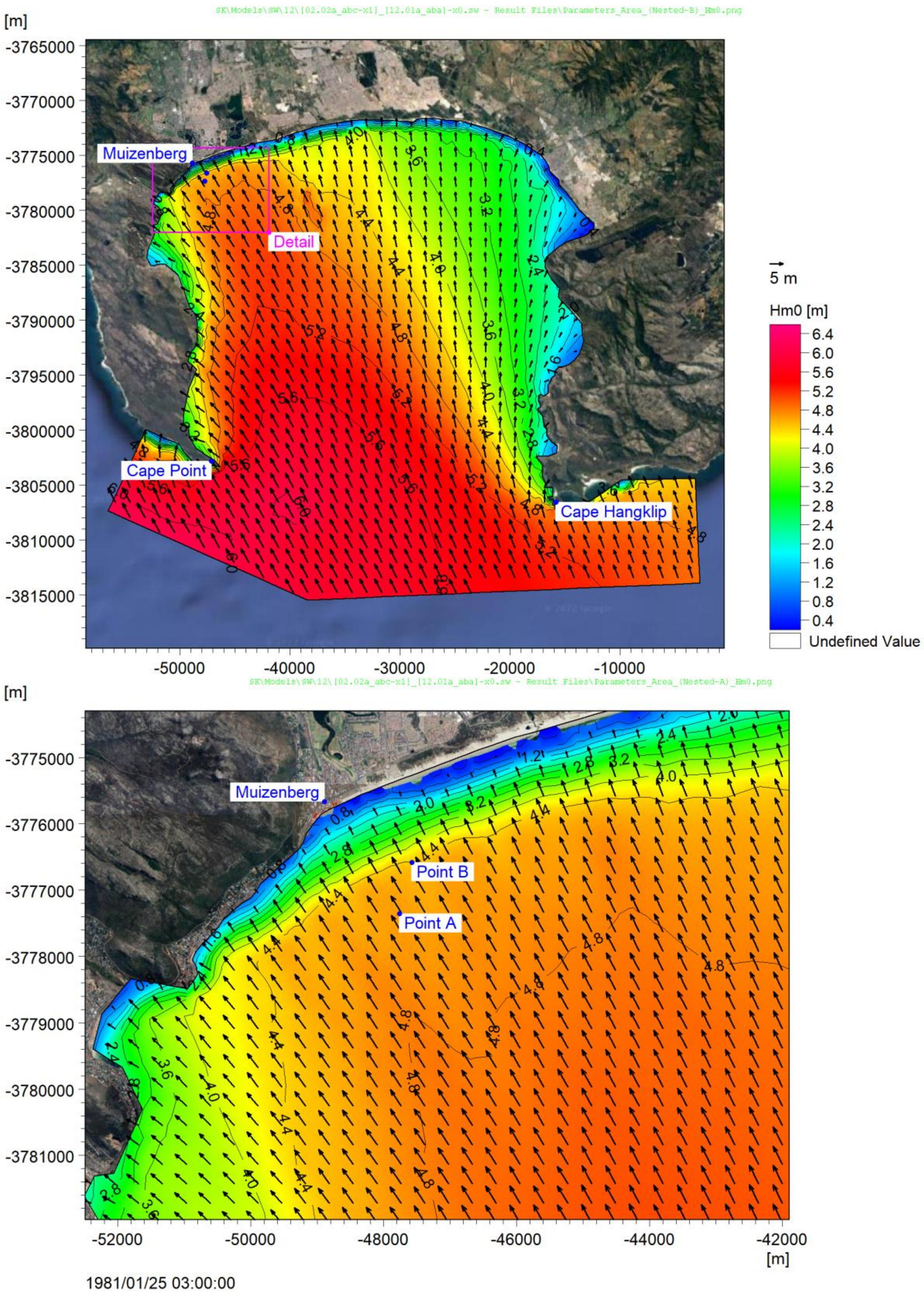


Figure 2-12: Example wave refraction plot for the 25 January 1981 storm event. The wave parameters at Point A (-15 m MSL) were used to characterise the wave conditions near Muizenberg for this study.

2.5.1 Extreme wave conditions

In the *Wave and Sediment Transport Modelling* study, extreme water levels were derived from measured storm surge, modelled wind set-up, SLR and predicted tides. The extreme nearshore design conditions were evaluated for the 1-, 10-, 100-, and 475-year return periods, analysed as a joint probability between the waves and the extreme water level. The effect of climate change on extreme wave heights, storm surge and SLR was included for three climate change horizons (2026, 2046 and 2076) for SSP5-8.5 (the most conservative scenario).

In addition to the previous study, extreme water levels are also provided using median SLR projections for SSP1-2.6 (a low emissions scenario). It is noted that the effects of climate change on extreme wave heights and storm surge were left unchanged (i.e., using SSP5-8.5), as quantified projections of these for SSP1-2.6 are not readily available. This results in only a small amount of conservatism, since the projected changes to these parameters are small compared to the effect of SLR.

The directional standard deviation (DSD) for the extreme wave conditions was estimated from H_{m0}-DSD trends for extreme waves to be approximately 16 deg. Initial flume tests showed that the steeper waves ($s > 1/65$) and wave dominated events resulted in larger overtopping events compared to flatter waves ($s < 1/65$) or storm surge dominant events. Thus, for this study only the steeper, wave-dominant cases are presented.

A summary of the extreme conditions at Point A is given in Table 2-1. Extreme conditions from 1-, 10-and 100-yr storm events were considered for the overtopping and flooding analyses in Section 3.

Table 2-1: Extreme conditions at Point A (-15 m MSL).

Extreme condition			Wave parameters[(a)]			Still water level	
Climate change		Return period	Return period	H_{m0}	T_p	Return period	Water level
Projection	Horizon	[y]	[y]	[m]	[s]	[y]	[m MSL]
SSP5-8.5	2026	1	1	3.59	9.7	1	1.41
		10	10	5.03	11.5	1	1.41
		100	100	6.48	13.0	1	1.41
	2046	1	1	3.62	9.7	1	1.54
		10	10	5.07	11.5	1	1.54
		100	100	6.53	13.1	1	1.54
	2076	1	1	3.66	9.8	1	1.83
		10	10	5.13	11.6	1	1.83
		100	100	6.60	13.2	1	1.83
SSP1-2.6	2026	1	1	3.59	9.7	1	1.40
		10	10	5.03	11.5	1	1.40
		100	100	6.48	13.0	1	1.40
	2046	1	1	3.62	9.7	1	1.51
		10	10	5.07	11.5	1	1.51
		100	100	6.53	13.1	1	1.51
	2076	1	1	3.66	9.8	1	1.68
		10	10	5.13	11.6	1	1.68
		100	100	6.60	13.2	1	1.68

Note:
(a): Only the steeper ($s > 1/65$) are presented in this study.

2.5.2 Typical surfing conditions

Typical surfing conditions were applied in the wave reflection assessment. Two typical surfing conditions, Case A and B, have been selected from the operational wave conditions at Point A.

With reference to Figure 2-13, the following wave heights (and associated wave periods) were selected:

- **Case A (smaller conditions):** the wave height for any given time and day is chosen, thus the median from the annual exceedance:

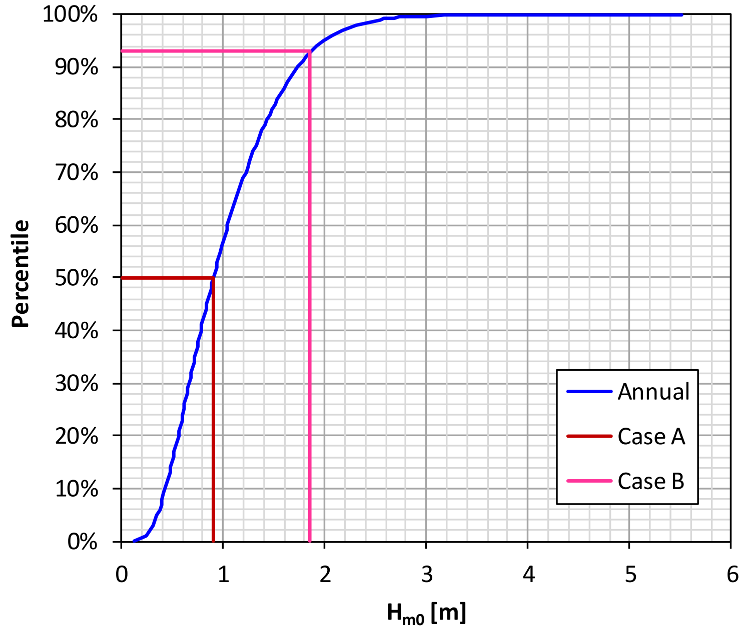
$$P(X < H_{m0,a}) = 50\%$$

$$\therefore H_{m0,b} = 0.91m \ (T_{p,a} = 12.5s).$$

- **Case B (larger conditions):** it is assumed the more experienced surfers would choose the largest wave conditions during one 6h daylight window per week of an available 12h per day; the probability for such a case would be:

$$P(X < H_{m0,b}) = 1 - (^{6}/_{12})/7 = 93\%$$

$$\therefore H_{m0,b} = 1.86m \ (T_{p,b} = 14s).$$



Percentile	Hm0 [m]
1%	0.24
2%	0.28
5%	0.35
10%	0.43
20%	0.56
30%	0.67
50%	$\boxed{0.91}$
70%	1.22
80%	1.43
90%	1.74
93%	$\boxed{1.86}$
95%	1.99
98%	2.31
99%	2.55

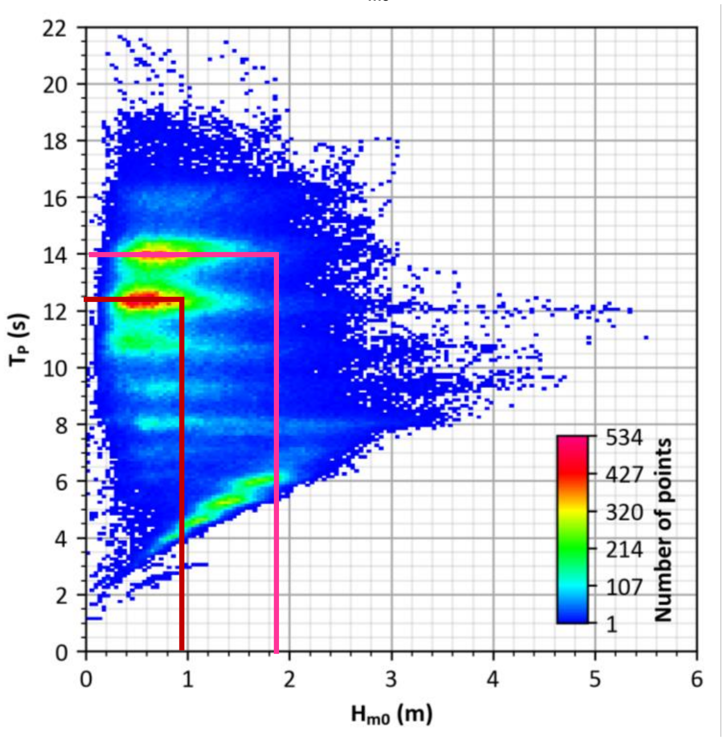


Figure 2-13: Percentile plot (top) and H_{m0}-T_p scatter (bottom) showing typical surfing conditions selected from over 42 years of operational wave conditions at Muizenberg.

A wave directional spreading of DSD = 13 (based on H_{m0}-DSD trends for operational waves) was assumed for both cases. Table 4-1 summarises the typical surfing conditions at Point A. Also shown are SWL combined from the 90th percentile high tide and SLR for 2026 (SSP5-8.5) and 2046 (SSP1-2.6 and SSP5-8.5). Storm surge was excluded from the typical surfing conditions.

Table 2-2: Typical surfing conditions at Point A (-15 m MSL).

Climate change		Wave Case	Wave parameters			
Projection	Horizon		H_{m0}	T_p	DSD	SWL
SSP5-8.5	2026	A	0.91	12.5	13	1.05
		B	1.86	14	13	1.05
	2046	A	0.91	12.5	13	1.19
		B	1.86	14	13	1.19
SSP1-2.6	2046	A	0.91	12.5	13	1.15
		B	1.86	14	13	1.15

The conditions summarised above were used in the wave reflection assessment in Section 4. The wave conditions were not adjusted for climate change as the main purpose of the climate change sensitivities was to analyse the relative impact on the beach levels and water levels on the reflections.

3. OVERTOPPING AND FLOODING ASSESSMENT

3.1 Introduction

This section covers overtopping and resultant flooding for the project at the proposed levels. Numerical flume (2DV) models have been used to test the sensitivity of the results to the climate change projection, a raised crest level for adaptive design and a comparison of the existing seawall to the proposed revetment. The results informed the selection of 4 scenarios for the full domain (3D) simulations to quantify the overtopping and resultant flooding along the full length of the proposed revetment.

3.2 Numerical flume

3.2.1 Bathymetry and mesh

A numerical flume was set up based on a cross-shore transect at Profile 4 (refer to Figure 2-3 to Figure 2-5). A unique mesh was configured for all combinations of beach profiles and structural configurations for the scenarios modelled (discussed further in Section 3.2.3).

Three structural configurations were modelled: the existing seawall, proposed revetment (crest level of 3.5 m MSL) and proposed revetment with an increased crest level (4.5 m MSL). For the existing seawall a 10:1 slope was used from the toe of the structure to a crest level of 3.1 m MSL, followed by a small step to 3.5 m MSL on the landward side of the main walkway. For the proposed revetment a simple 1:2 slope was used from the toe of the structure to a crest level of 3.5 m MSL. For the proposed stepped revetment with an increased crest level the slope was extended to 4.5 m MSL. For all structural configurations the landside features were resolved with simple slopes representing the lawns, walkways and parking.

Each configuration was set up as a one element wide numerical flume, i.e., a 2DV model. The wave generation line was placed at the -15 m MSL contour and a free outflow boundary was applied on the landward side to prevent the accumulation of water.

The model mesh comprises quadrangular elements with a depth-varying resolution varying between approximately 4.2 m offshore and approximately 1.2 m nearshore. This resulted in a resolution of approximately 25 points per wavelength across the whole domain for the shortest T_p modelled (9.7 s). The vertical mesh comprises three sigma layers with the bottom layer comprising 50%, the middle layer 30% and the surface layer 20% of the water column. Example model computational meshes are shown in Figure 3-1 and Figure 3-2 for the existing seawall and proposed revetment.

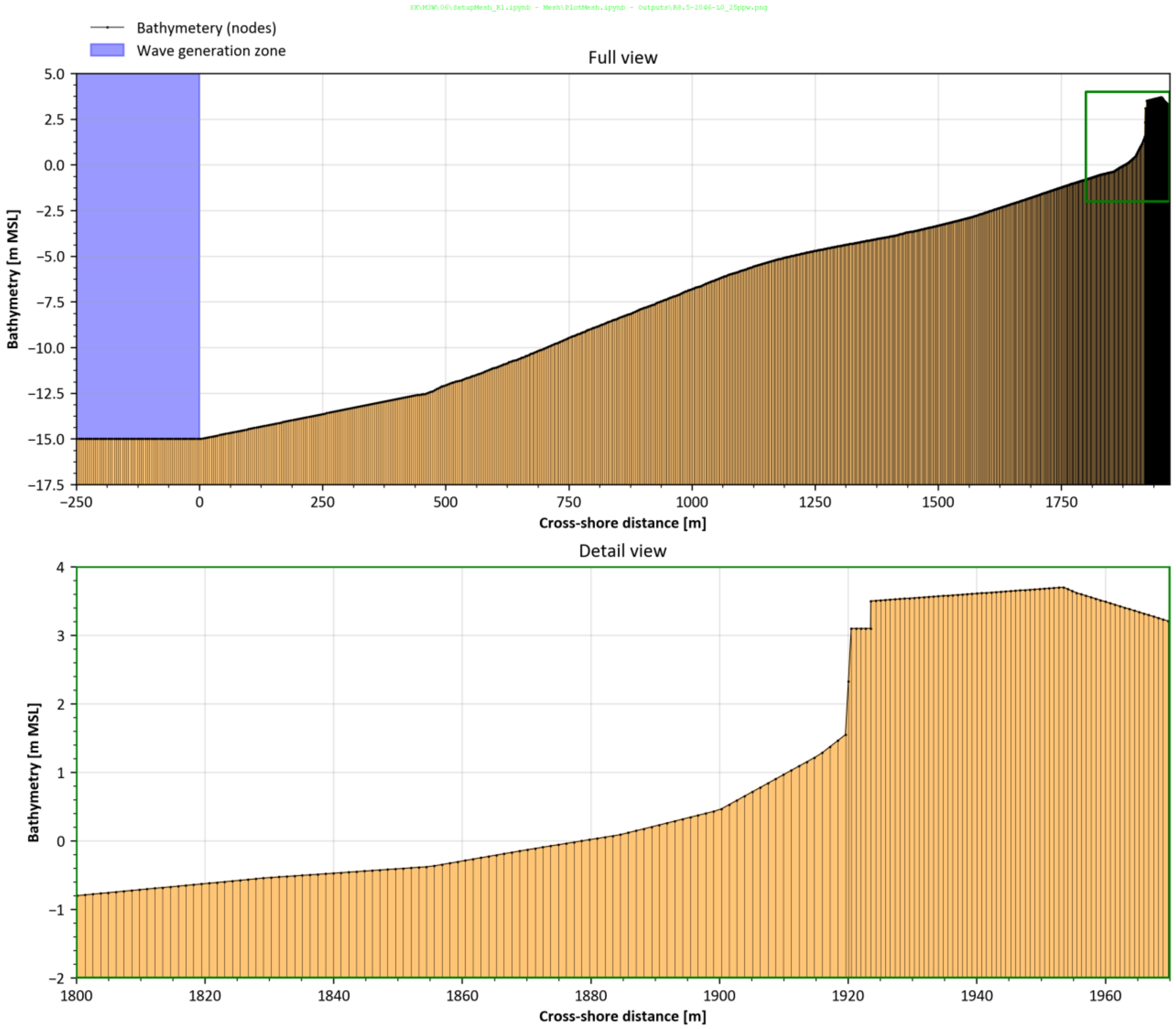


Figure 3-1: Profile 4; Example 2DV mesh for the existing seawall with a typical eroded beach level at 2046 (SSP5-8.5).

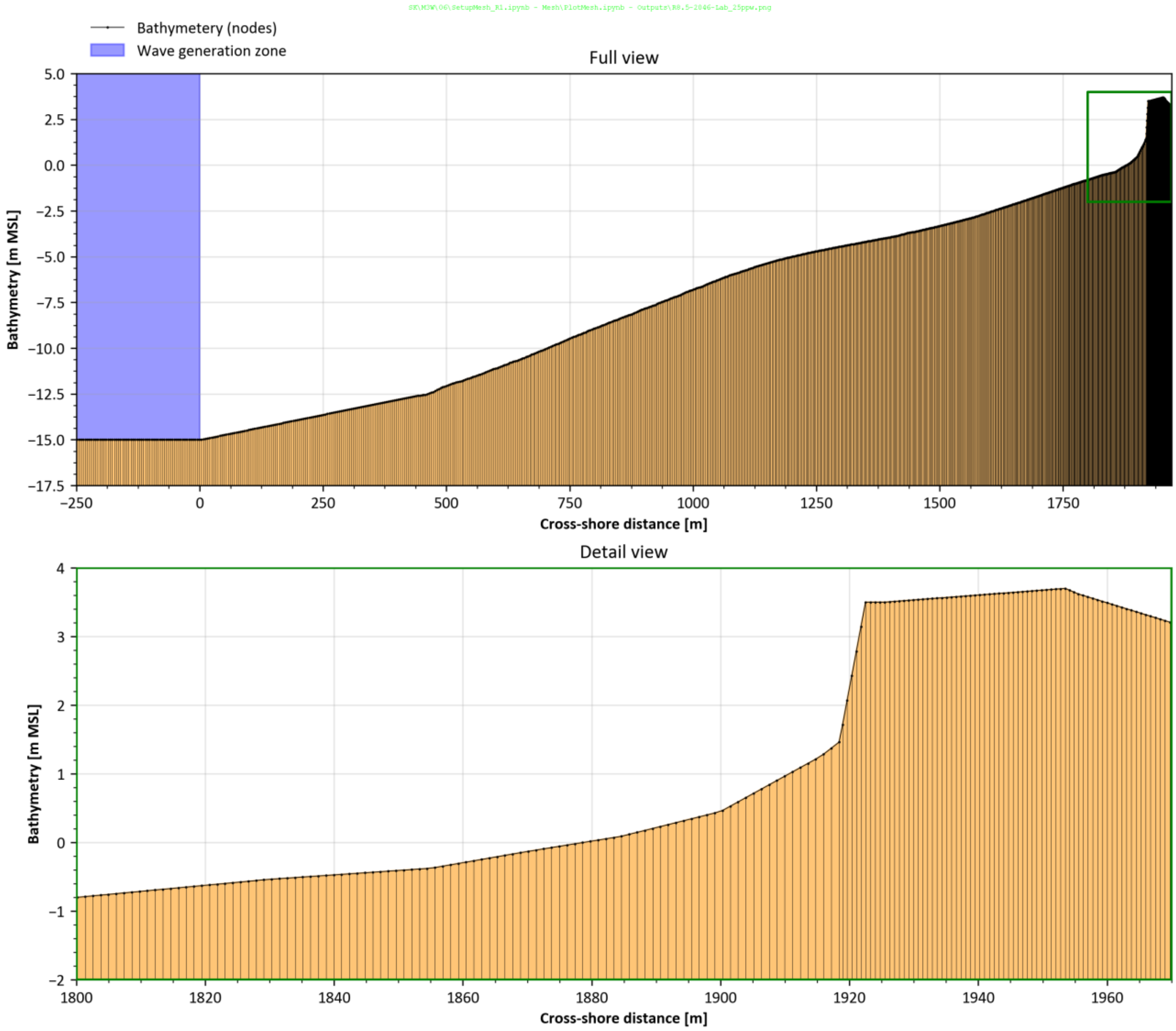


Figure 3-2: Profile 4; Example 2DV mesh for the proposed revetment with a typical eroded beach level at 2046 (SSP5-8.5).

3.2.2 Model parameters

Bottom friction was modelled using a roughness height formulation to include the following processes:

- Energy dissipation due to bottom friction;
- Wave overtopping of stepped revetments.

Bottom friction was modelled using a roughness height of k_s = 0.05 m across the whole domain, while the proposed revetment was set to k_s = 1.46 m to resolve the 0.25 m step height (refer to Section 2.4).

3.2.3 Scenarios modelled

The main modelled scenarios comprised combinations of the following:

- 3x climate change horizons: 2026, 2046 and 2076
- 3x upper beach profile types (i.e., variability in beach levels): typical accreted profile, typical eroded profile and a SBEACH storm-eroded profile
- 3x storm severities: 1-, 10-and 100-yr storm event.

The following sensitivities were tested in the numerical flume runs:

- Crest level: raised crest for the proposed revetment (from 3.5 m MSL to 4.5 m MSL)
- Climate change scenario: SSP5-8.5 (the most conservative scenario) versus SSP1-2.6 (a low emissions scenario) for the proposed revetment.
- Existing seawall versus proposed revetment

A summary of the extreme wave combinations modelled is provided in Table 2-1. A summary of the all the combinations and sensitivities modelled are provided in Table 3-1 below.

Table 3-1: Combinations of scenarios modelled for the numerical flume overtopping and flooding assessment.

Scenario	Baseline	Sensitivity 1: raised crest level	Sensitivity 2: climate change scenario	Sensitivity 3: existing seawall
Structure configuration				
Existing seawall				X
Proposed revetment	X		X	
Proposed revetment with raised crest level		X		
Upper beach profile				
Storm-eroded profile [(a)]	X	X		
Typical accreted	X	X	X	X
Typical eroded	X	X	X	X
Storm severity[(b)]				
1-yr	X	X	X	X
10-yr	X	X	X	X
100-yr	X	X	X	X
Climate change projection				
SSP5-8.5	X	X		X
SSP1-2.6			X	
Climate change horizon				
2026	X	X		X
2046	X	X	X	
2076	X	X	X	

Note:
(a) *SBEACH storm-eroded profiles were not available for the existing seawall or a SSP1-2.6 climate change projection*
(b) *Wave dominant storms with steepness of s > 1/65, refer to Table 2-1*

For each run a JONSWAP spectrum was used with a gamma of 2.0. The specified wave parameters were used by the model to generate a sea state by applying random phases. The duration of each simulation was set to three hours excluding spin-up time.

3.2.4 Model results

The model outputs included cumulative overtopped volume at the landward edge of the walkway and the maximum water depth and current speed landward of this boundary. The outputs from each run were processed to obtain overtopping discharge per unit width. The duration of the overtopping measurements excluded the warm-up period. Figure 3-3 presents an example of model outputs for single scenario.

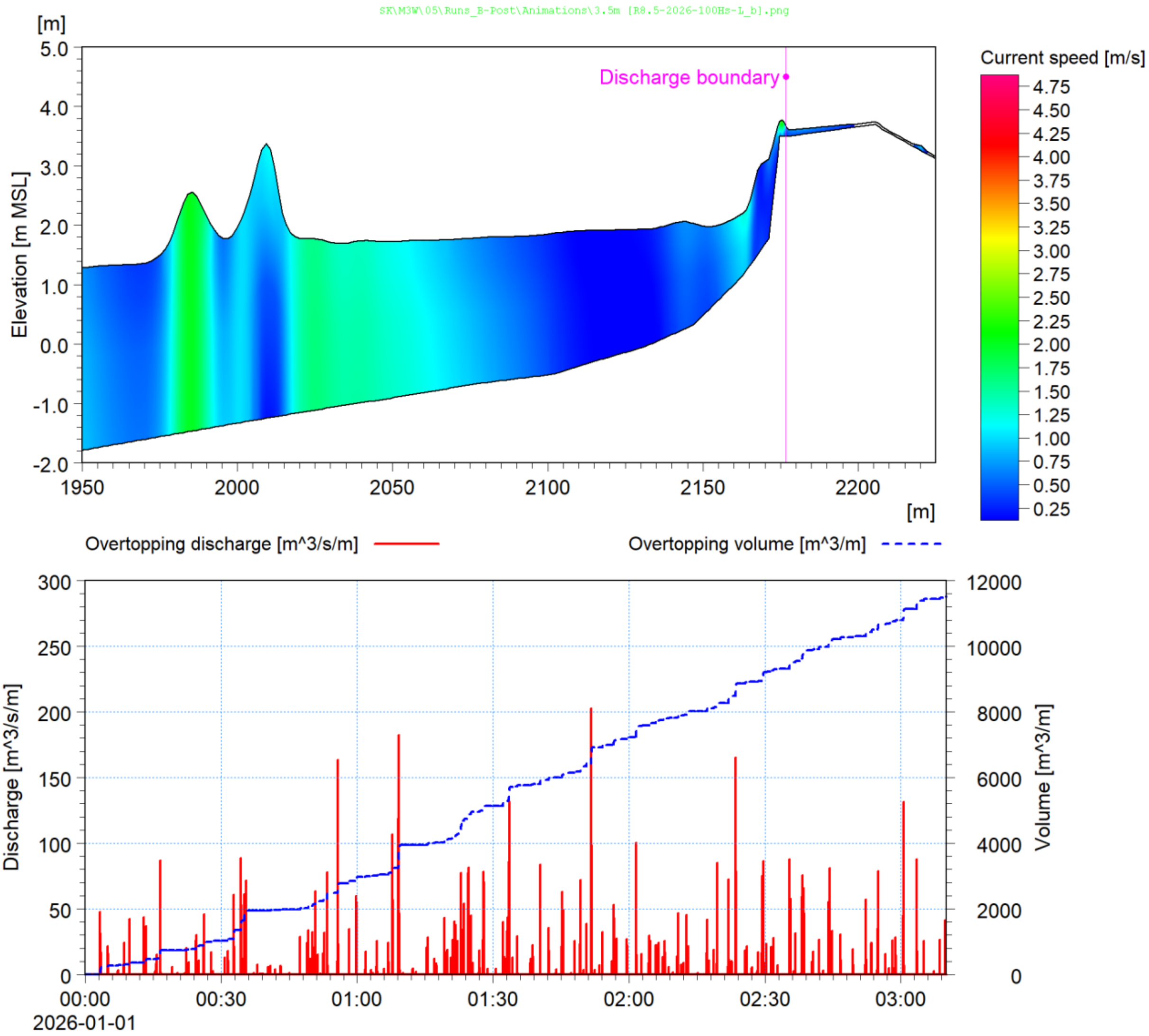


Figure 3-3: Example model outputs for the proposed revetment with a typical eroded beach profile during a 100-yr storm event (2026 climate change scenarios). Top: cross-section of 2DV model showing instantaneous surface elevation. Bottom: Time-series of overtopping discharge rate and accumulated volume over the crest of the proposed revetment.

Figure 3-4 presents a comparison of overtopping discharge for the baseline scenario and 3 sensitivity scenarios. Also shown are the overtopping limits extracted from EurOtop (2018) which have been used to classify the modelled overtopping discharge.

Figure 3-5 and Figure 3-6 similarly present the maximum depth and maximum current speed behind the walkway. Summary tables of overtopping discharge, maximum depth and maximum current speed follow the respective figures in Table 3-2, Table 3-3 and Table 3-4. Note that the tables only show the average from the three upper beach profiles for every combination of return period and climate change horizon.

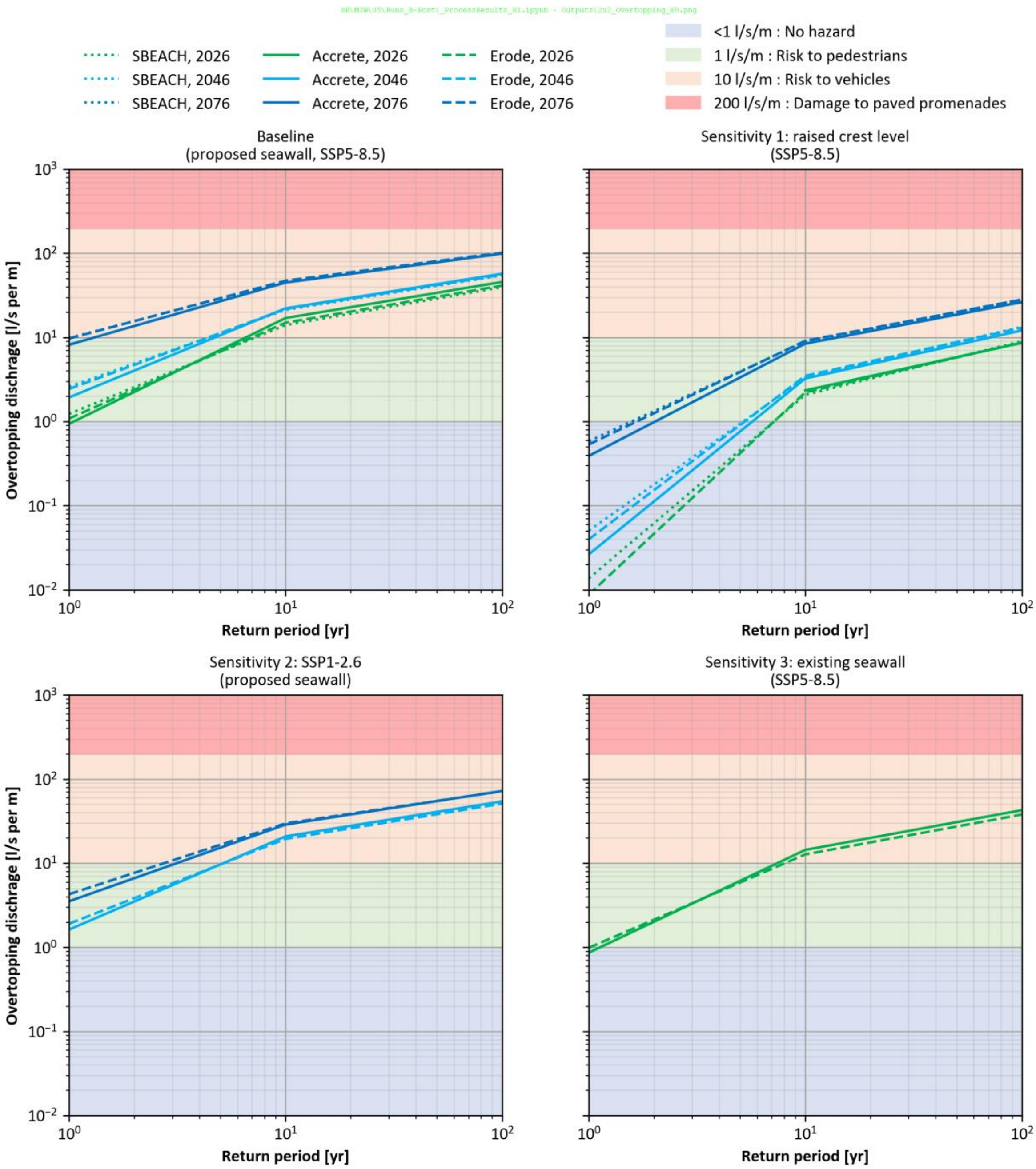


Figure 3-4: Comparison of overtopping discharge over the walkway.

Table 3-2: Summary of overtopping discharge over the walkway.

Return Period	Average overtopping discharge [l/s/m]								
	b: Baseline (proposed revetment, SSP5-8.5)			s_1: Raised crest level (SSP5-8.5)			s_2: SSP1-2.6 (proposed revetment)		s_3: existing seawall (SSP5-8.5)
	2026	2046	2076	2026	2046	2076	2046	2076	2026
1	1.1	2.3	9.2	0.0	0.0	0.5	1.8	3.9	0.9
10	15.3	21.7	46.1	2.2	3.4	8.8	20.1	29.2	13.6
100	42.2	55.8	100.5	8.8	12.8	27.6	52.8	72.0	40.3

Note: only the average overtopping rate from the three upper beach profiles is presented for every combination of return period and climate change horizon.

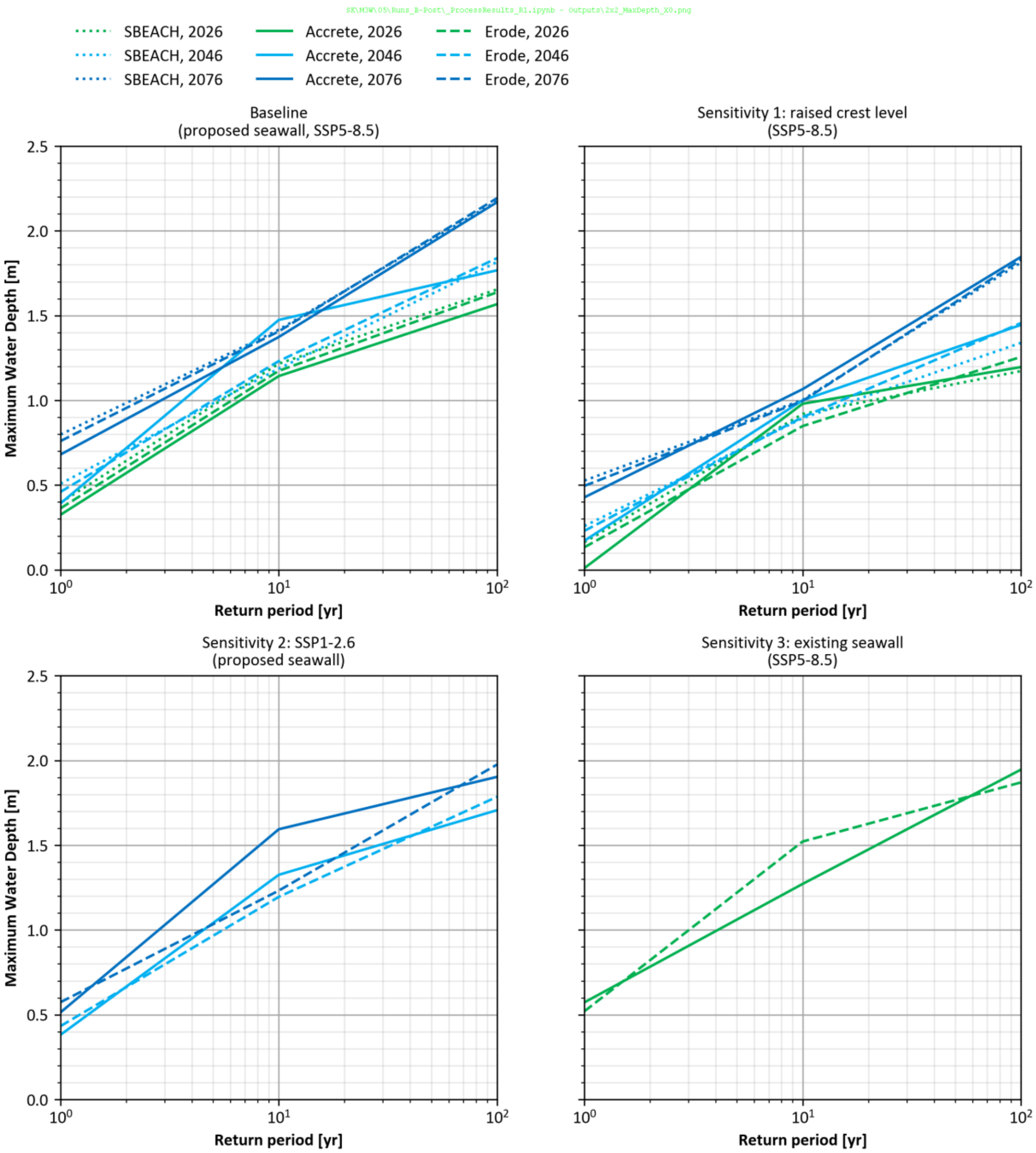


Figure 3-5: Comparison of maximum depth behind the walkway.

Table 3-3: Summary of maximum depths over the walkway.

Return Period	Maximum depth [m]								
	b: Baseline (proposed revetment, SSP5-8.5)			s_1: Raised crest level (SSP5-8.5)			s_2: SSP1-2.6 (proposed revetment)		s_3: existing seawall (SSP5-8.5)
	2026	2046	2076	2026	2046	2076	2046	2076	2026
1	0.36	0.45	0.75	0.10	0.22	0.48	0.27	0.36	0.55
10	1.18	1.30	1.40	0.92	0.93	1.02	0.84	0.94	1.40
100	1.62	1.81	2.18	1.21	1.41	1.83	1.16	1.29	1.91

Note: only the average maximum depth from the three upper beach profiles is presented for every combination of return period and climate change horizon.

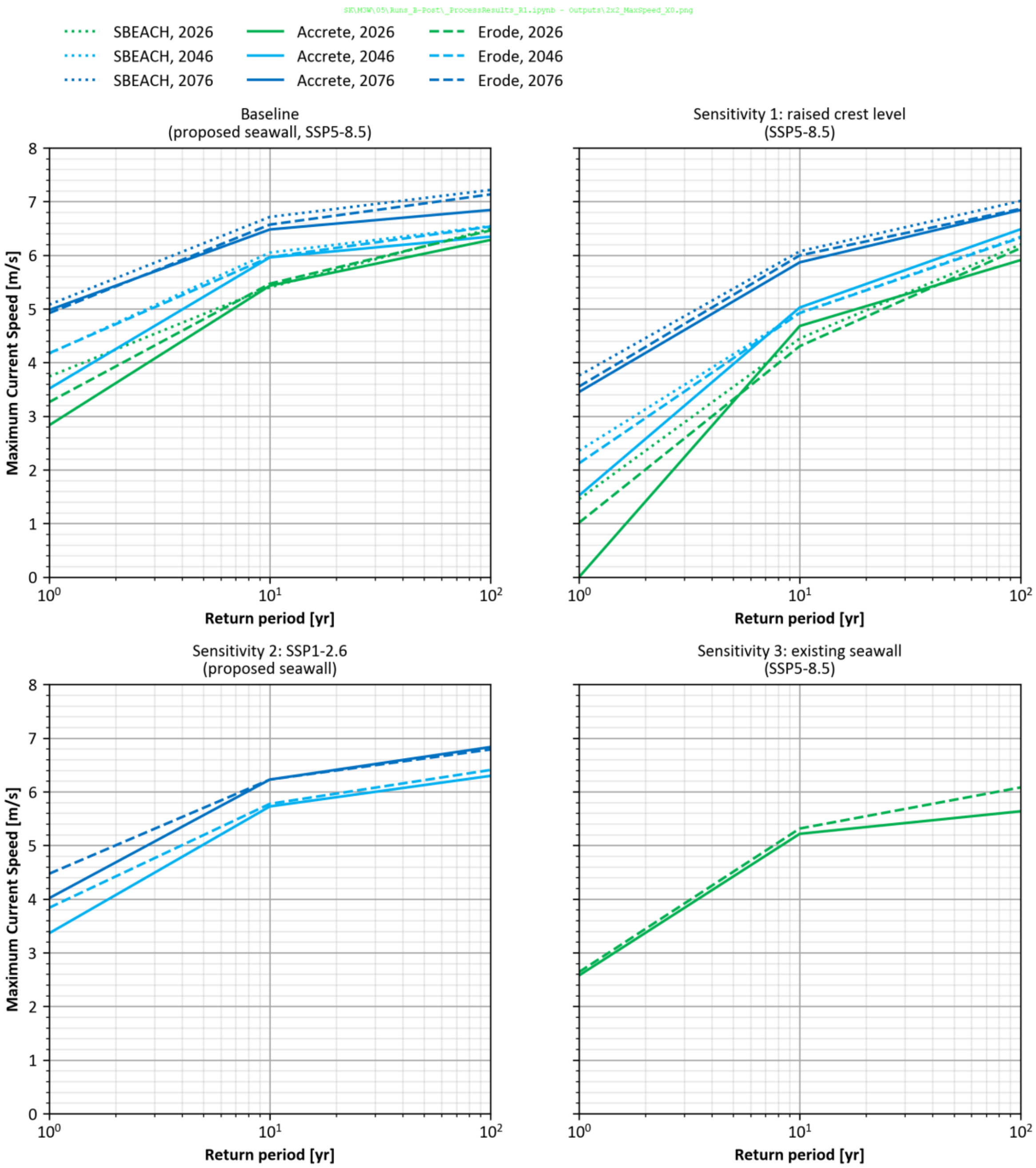


Figure 3-6: Comparison of maximum current speeds behind the walkway.

Table 3-4: Summary of maximum current speeds over the walkway.

Return Period	Maximum current speed [m/s]								
	b: Baseline (proposed revetment, SSP5-8.5)			s_1: Raised crest level (SSP5-8.5)			s_2: SSP1-2.6 (proposed revetment)		s_3: existing seawall (SSP5-8.5)
	2026	2046	2076	2026	2046	2076	2046	2076	2026
1	3.3	4.0	5.0	0.8	2.0	3.6	3.6	4.2	2.6
10	5.4	6.0	6.6	4.5	5.0	6.0	5.8	6.2	5.3
100	6.4	6.5	7.1	6.1	6.4	6.9	6.4	6.8	5.9

Note: only the average maximum current speed from the three upper beach profiles is presented for every combination of return period and climate change horizon.

Raising the crest level to 4.5 m reduces the overtopping by 80% for 10-year and 100-year events while overtopping for 1-year events are almost non-existent.

When comparing the SSP1-2.6 projection (a low emissions scenario) to the SSP5-8.5 projection (the most conservative scenario) the overtopping rates were modestly lower (on average 7% lower for 2046, and 32% lower for 2076) due to a lower still water depth and less eroded profile.

Relative to the existing seawall, the proposed revetment showed a small (< 7 %) increase in overtopping discharge.

Based on these results, it was agreed with the CCT to model the following scenarios in the full domain:

- 1-year storm wave with a 2026 climate change scenario
- 1-year storm wave with a 2076 climate change scenario
- 100-year storm wave with a 2046 climate change scenario
- 100-year storm wave with a 2076 climate change scenario

All of the above scenarios will be modelled with the proposed revetment (3.5 m MSL crest) combined with a typical eroded beach level and a SSP5-8.5 climate change projection.

3.3 Full domain

3.3.1 Bathymetry and mesh

The three sets of bathymetries adjusted for coastline recession due to sea level rise, longshore sand supply and a typical eroded beach level for 2026, 2046 and 2076 are presented in Section 2.2. The bathymetries were interpolated onto the computational mesh as described below.

The model mesh comprises triangles with a resolution varying between approximately 5.4 m offshore and approximately 2.7 m nearshore. This resulted in a resolution of approximately 20 points per wavelength for a peak wave period of T_p = 9.7 s. In the area of interest, the mesh was refined to an average resolution of 2 m and structures with steps or steep slopes were modelled with quadrangular elements with a resolution varying between 1 m and 2 m.

The wave generation line was placed along the -15 m MSL contour. Free outflow conditions were prescribed on the open boundaries on the landside (e.g., roads) to prevent excessive ponding of water during extreme overtopping events. A sponge layer was applied to the upstream boundary of the Zandvlei canal to absorb waves propagating up the estuary.

The vertical mesh comprises three sigma layers with the bottom layer comprising 50%, the middle layer 30% and the surface layer 20% of the water column. The model computational mesh is shown in Figure 3-7.

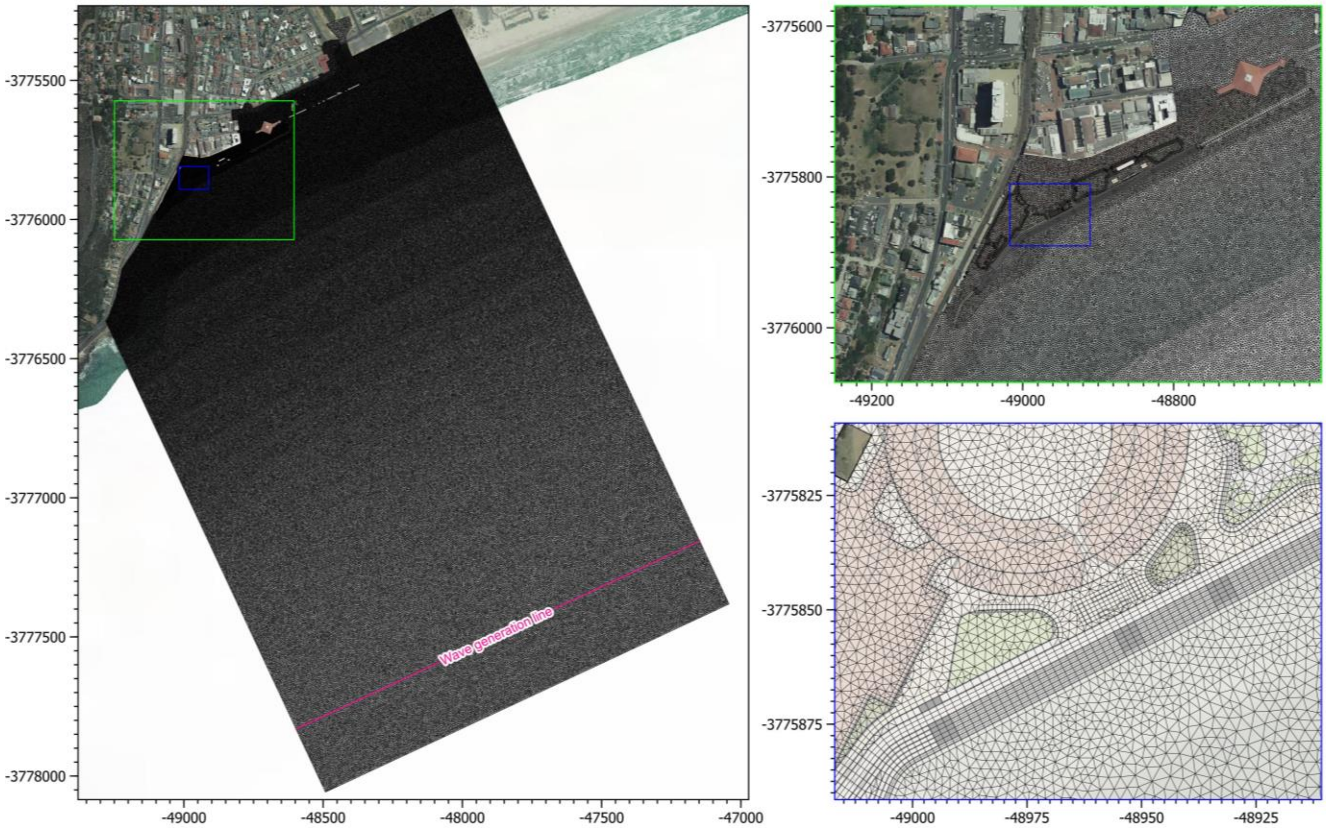


Figure 3-7: Computational mesh used in the overtopping and flooding model.

3.3.2 Model parameters

Bottom friction was modelled using a roughness height formulation to include the following processes:

- Energy dissipation due to bottom friction;
- Reduction in wave reflection from rocks; and
- Wave overtopping of stepped revetments.

The stepped revetment was modelled with a spatially varying bottom friction ranging between k_s = 0.75 m (step height = 0.17 m) and k_s = 5.0 m (step height = 0.5 m) to resolve the steps along the proposed revetment (refer to Section 2.4).

An increased friction was applied to rocky areas west of Surfers Corner to reduce wave reflection. A qualitative assessment of the reflections confirmed that the high friction is sufficient.

For the rest of the domain a roughness height of k_s = 0.05 m was applied.

3.3.3 Scenarios modelled

Table 3-7 summarises the four scenarios modelled in the full domain for the proposed revetment. All of the scenarios were modelled with a SSP5-8.5 climate change projection and a typical eroded upper beach profile.

Table 3-5: Extreme conditions modelled for the full domain overtopping and flooding assessment.

Extreme condition		Wave parameters			Still water level	
Climate change horizon	Return period	Return period	s > 1/65		Return period	Water level
			H_{m0}	T_p		
	[y]	[y]	[m]	[s]	[y]	[m MSL]
2026	1	1	3.59	9.7	1	1.41
2046	100	100	6.53	13.1	1	1.54
2076	1	1	3.66	9.8	1	1.83
2076	100	100	6.60	13.2	1	1.83

A shore-normal mean wave direction and a wave directional spreading of DSD = 16 deg was applied to all scenarios. A JONSWAP spectrum was used with a gamma of 2.0. The specified wave parameters were used by the model to generate a sea state by applying random phases and directions. The duration of each simulation was set to one hour excluding spin-up time.

3.3.4 Model results

Figure 3-8 shows an example of the instantaneous water surface elevation for a 2076 climate change scenario and a 100-year event.

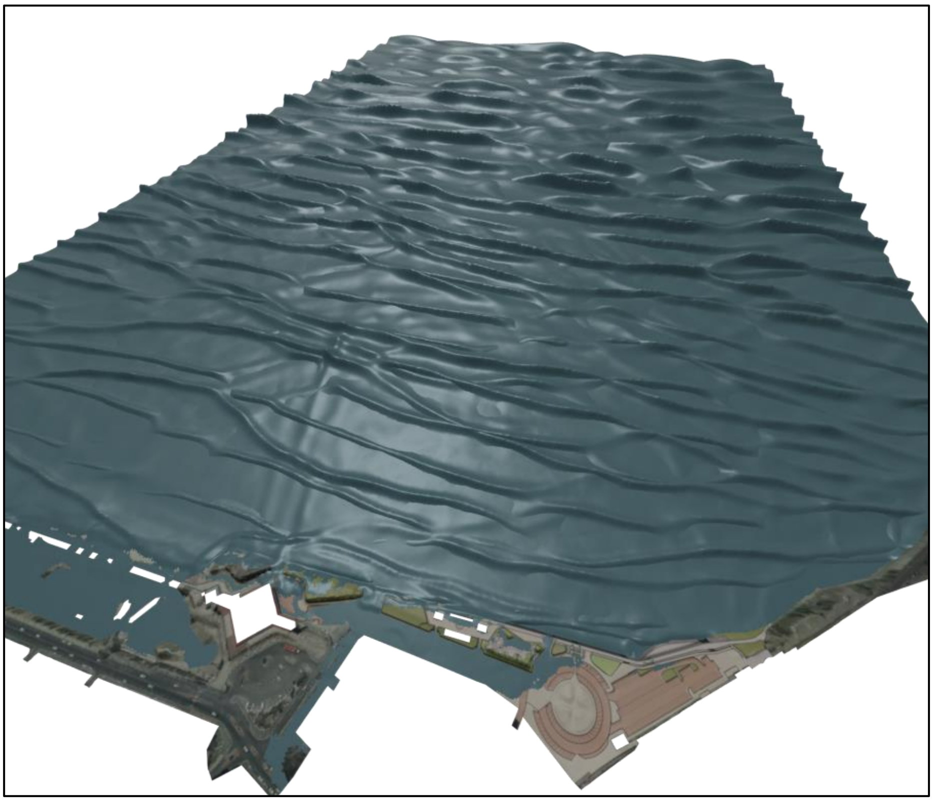


Figure 3-8: Snapshot of instantaneous water surface elevation (vertically exaggerated) for a 2076 climate change scenario and a 100-year event.

The model outputs included cumulative overtopped volume at the landward edge of the walkway and the maximum water depth and current speed landward of this boundary. The outputs from each run were processed to obtain overtopping discharge per unit width. The overtopping of the wall was divided into the six sections shown in Figure 3-9.

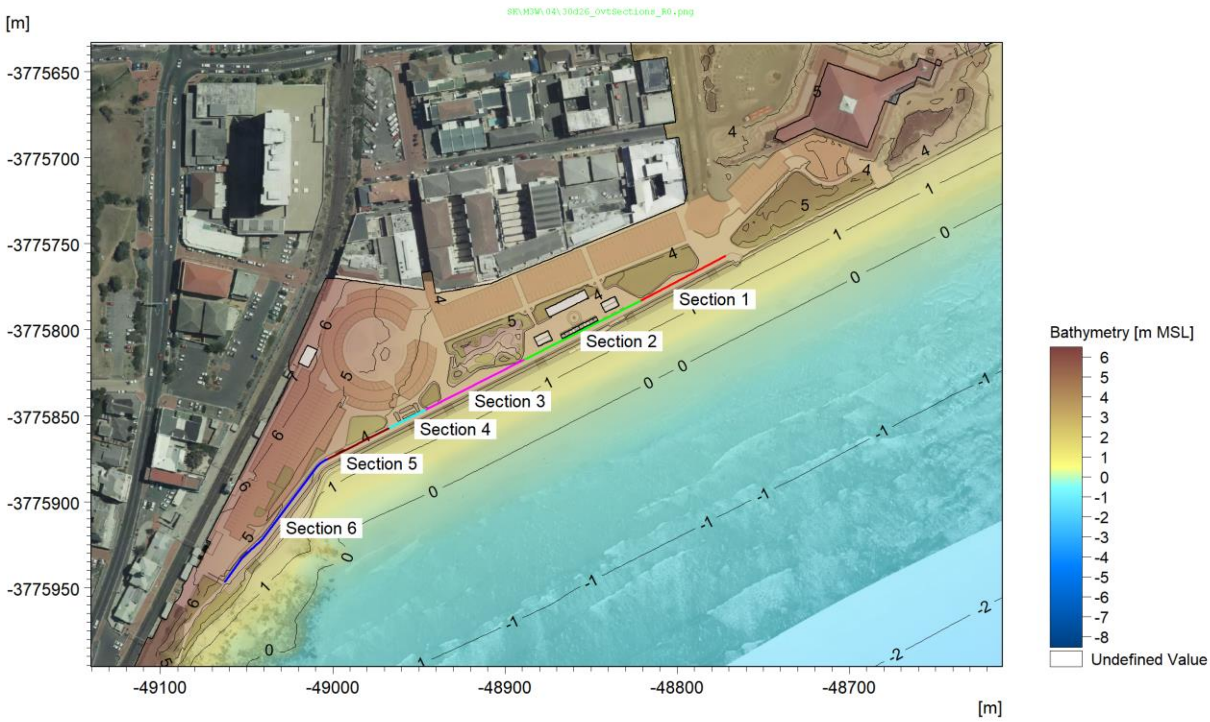


Figure 3-9: Overtopping sections and bathymetry for the 2026 climate change scenario. Sections 1 to 5 are approximately at 3.5 m MSL while the walkway at Section 6 is elevated to 4.0 m MSL.

The water depth and current speeds behind the wall are also of interest as these are hazardous to pedestrians, vehicles, and structures. Figure 3-10 presents an instantaneous plot of surface elevation showing overtopping of the seawall resulting in flooding landward of the wall.

Figure 3-10: Instantaneous plot of surface elevation for the 100-year return period (100-year wave with a 1-year water level) and a 2076 climate change scenario.

Table 3-6 presents the overtopping limits extracted from EurOtop (2018) which have been used to classify the modelled overtopping discharge. The mean overtopping discharge at each section and along the entire wall for each modelled condition for the proposed revetment is summarised in Table 3-7. The overtopping discharge is the average volume per second per meter of the wall, calculated from the total overtopped volume over the duration of the 60-minute sea state.

Table 3-6: Overtopping limits extracted from EurOtop (2018).

Hazard type	Limiting mean overtopping discharge [l/s/m]
No hazard	<1
Risk to pedestrians	1
Risk to vehicles	10
Damage to paved promenades	200

Table 3-7: Mean overtopping discharge for the proposed revetment (SSP5-8.5 climate change projection).

Extreme condition		Mean overtopping discharge [l/s/m]					
Climate change horizon	Return period [y]	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
2026	1	<0.1	-	-	-	-	-
2046	100	7.3	8.0	5.3	1.8	1.6	0.1
2076	1	0.5	0.8	0.7	0.3	0.1	-
2076	100	24.0	32.1	26.4	7.8	7.0	1.0

The results in Table 3-7 show that the mean overtopping discharge is about 3 to 5 times lower compared to the results from the numerical flume analyses, owing to the directional wave spreading and refraction processes in the 3D domain. Sections 1 to 3 consistently show higher overtopping rates, whereas the wave energy opposite Sections 4 to 6 is more affected by processes of refraction and reflection resulting in reduced overtopping. Section 6 is significantly better protected against overtopping due to the oblique angle against wave attack as well as the relative crest level (4.0 m MSL).

Comparing the modelled overtopping discharge to the limits in Table 3-6 shows that for the proposed revetment 1-year storm events present no hazard to vehicles or pedestrians along any of the sections. A 100-year event during 2046 is hazardous to pedestrians, while a 100-year event during 2076 becomes hazardous to vehicles along Sections 1 to 3.

The maximum water depths behind the proposed revetment are presented in the following figures:

- Figure 3-11: 1-year storm wave with a 2026 climate change scenario
- Figure 3-12: 100-year storm wave with a 2046 climate change scenario
- Figure 3-13: 1-year storm wave with a 2076 climate change scenario
- Figure 3-14: 100-year storm wave with a 2076 climate change scenario

A summary of maximum water depths behind the proposed revetment is provided in Table 3-8 for all modelled scenarios.

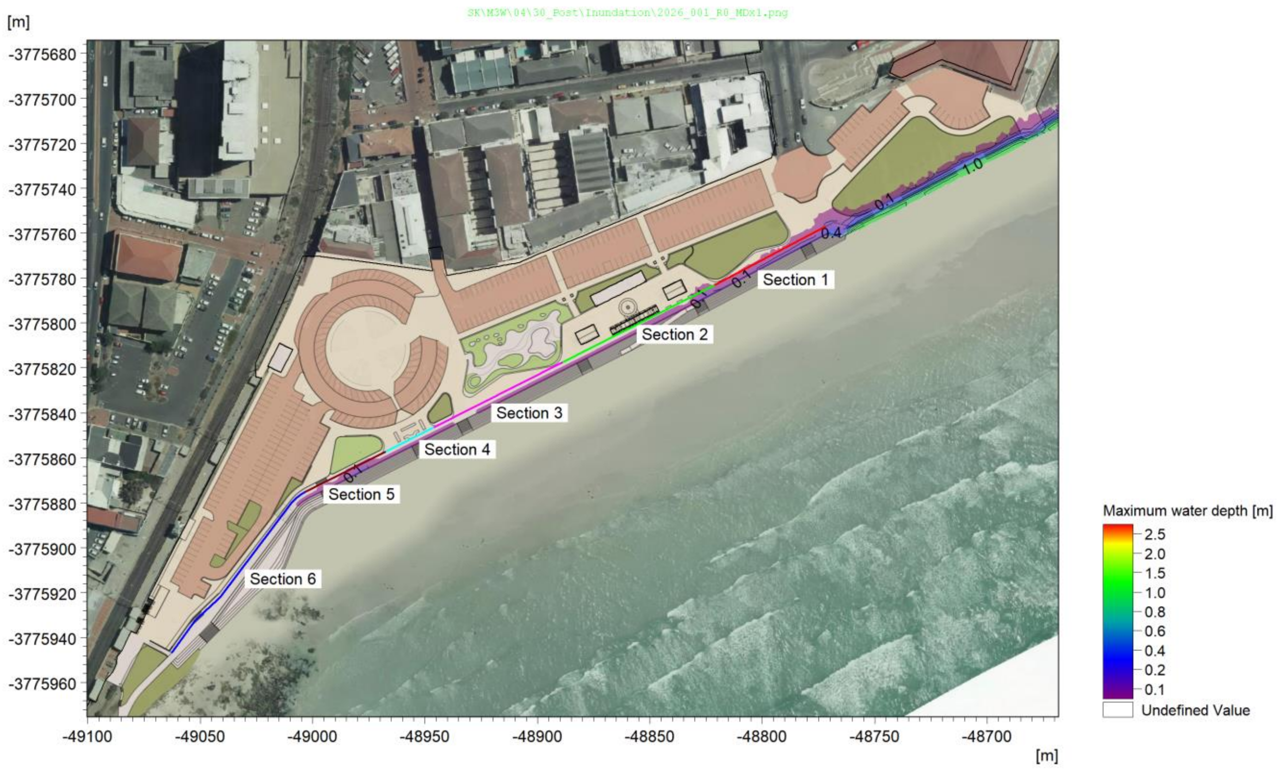


Figure 3-11: 2026; Maximum water depth behind the proposed revetment for the 1-year return period (1-year wave with a 1-year water level).

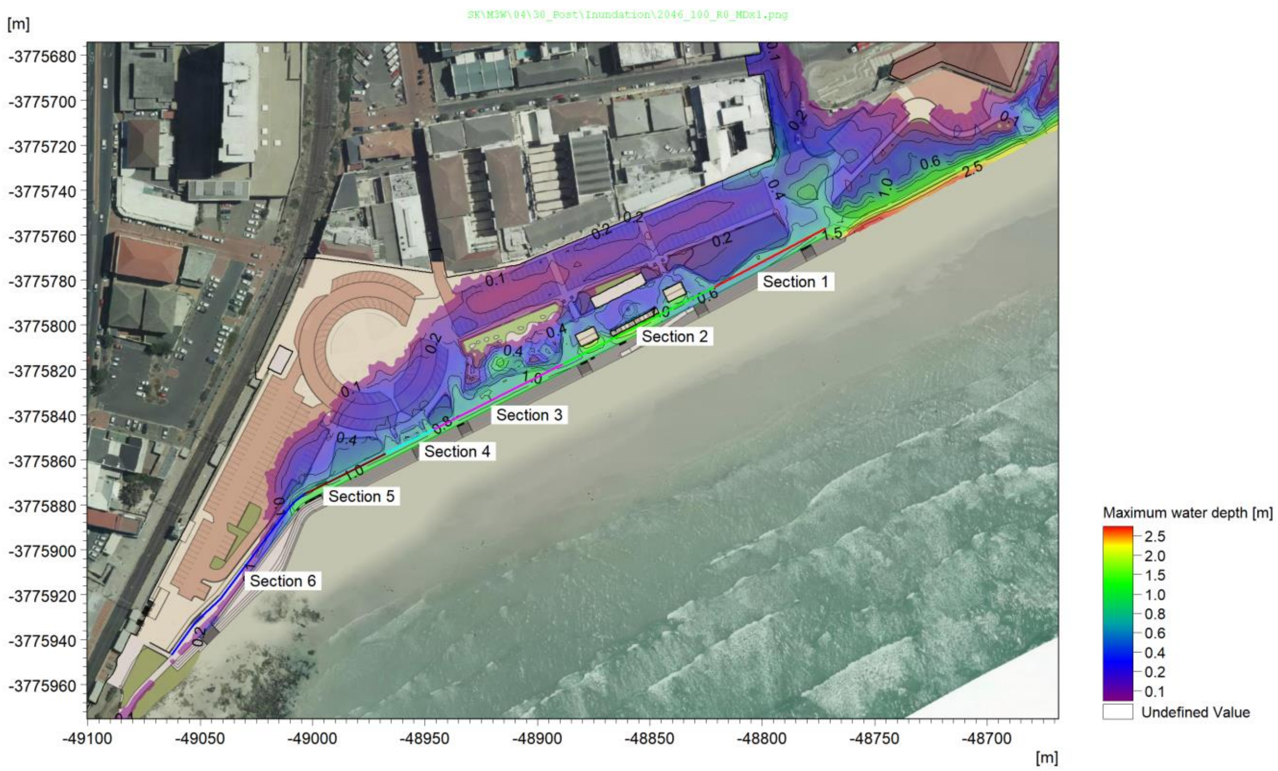


Figure 3-12: 2046; Maximum water depth behind the proposed revetment for the 100-year return period (100-year wave with a 1-year water level).

Figure 3-13: 2076; Maximum water depth behind the proposed revetment for the 1-year return period (1-year wave with a 1-year water level).

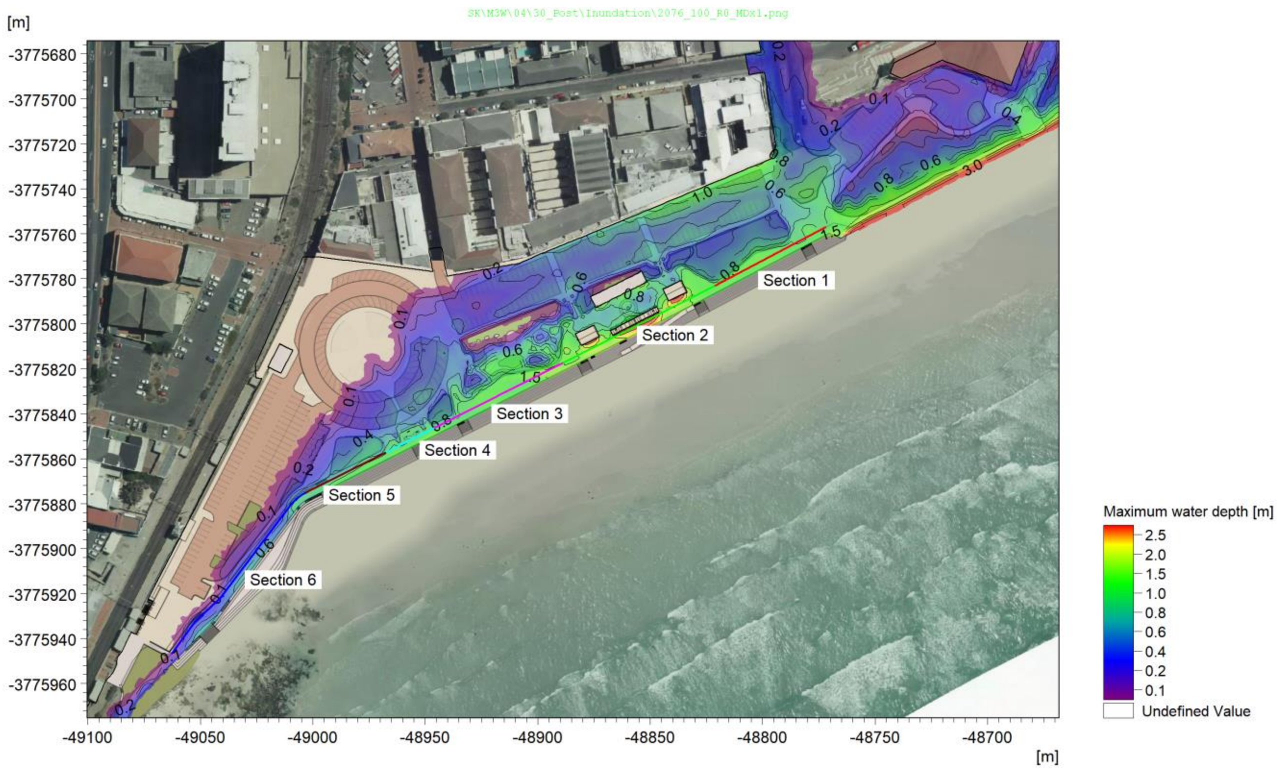


Figure 3-14: 2076; Maximum water depth behind the proposed revetment for the 100-year return period (100-year wave with a 1-year water level).

Table 3-8: Maximum water depth behind the proposed revetment.

Extreme condition		Maximum depth [m]						
Climate change horizon	Return period [y]	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Max
2026	1	0.09	-	-	-	-	-	0.09
2046	100	0.92	2.60	1.28	0.95	0.86	0.88	2.60
2076	1	0.47	0.59	0.47	0.44	0.32	-	0.59
2076	100	1.67	4.61	1.83	1.10	0.86	0.71	4.61

At the eastern end of the proposed revetment (Section 1), the relatively larger overtopping rates, combined with lower infrastructure levels and an unobstructed pathway, causes a weak spot resulting in increased flooding behind the promenade. The maximum water depths are typically associated with locations where waves run up against the seaward faces of structures (e.g., Section 2).

For a 1-year event in 2026 only the seaward edge of the promenade was inundated. For an increase in climate change horizon (2076) most of the promenade was overtopped, while the parking areas and most of the elevated lawns and vegetated areas remained dry.

For the 100-year events most of the parking areas were inundated, with only the more elevated western areas remaining dry. For an increased climate change horizon (2046 to 2076) the flooding extent and severity are generally worse, except at Surfers Corner where the contour plots show a slightly more landward flood line for the 2046 scenario, demonstrating the non-linearity of overtopping processes in a complex 3D environment.

The maximum current speeds behind the proposed revetment are presented in the following figures:

- Figure 3-15: 1-year storm wave with a 2026 climate change scenario
- Figure 3-16: 100-year storm wave with a 2046 climate change scenario
- Figure 3-17: 1-year storm wave with a 2076 climate change scenario
- Figure 3-18: 100-year storm wave with a 2076 climate change scenario

A summary of maximum current speeds behind the proposed revetment is provided in Table 3-9 for all modelled scenarios.

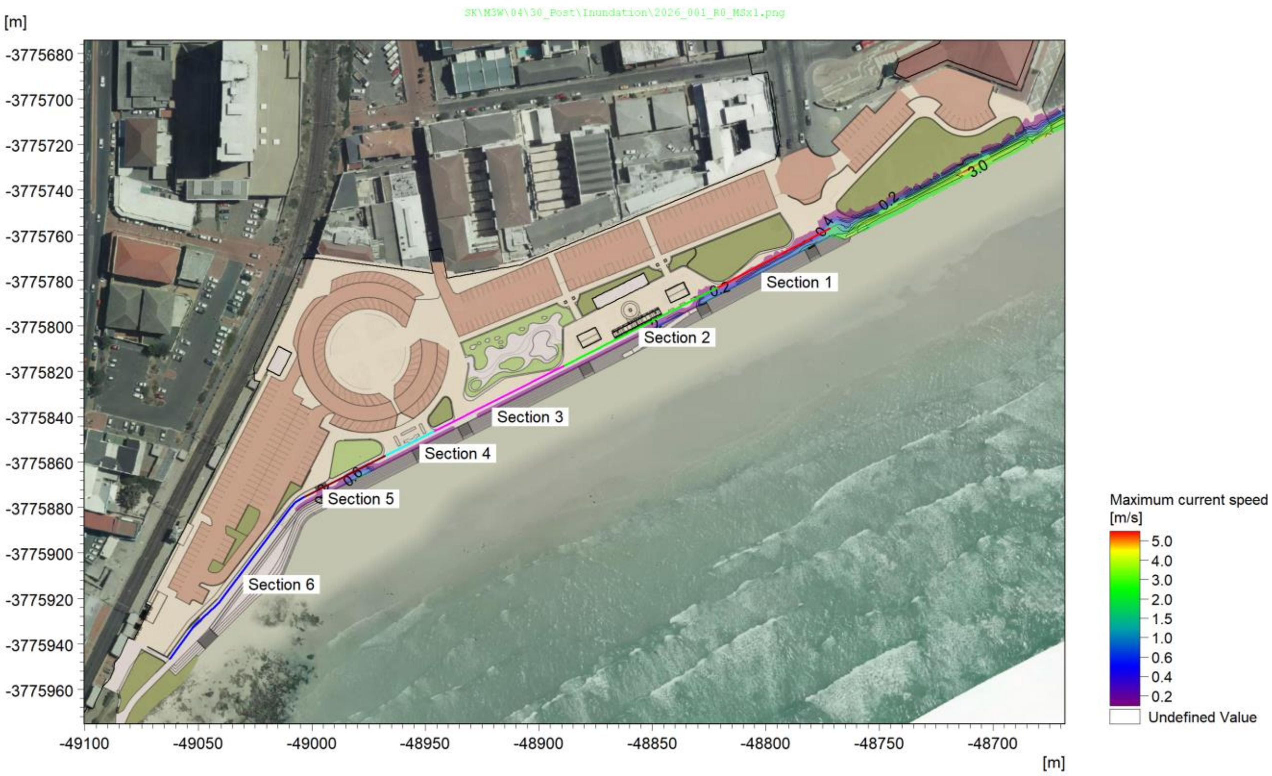


Figure 3-15: 2026; Maximum current speed behind the proposed revetment for the 1-year return period (1-year wave with a 1-year water level).

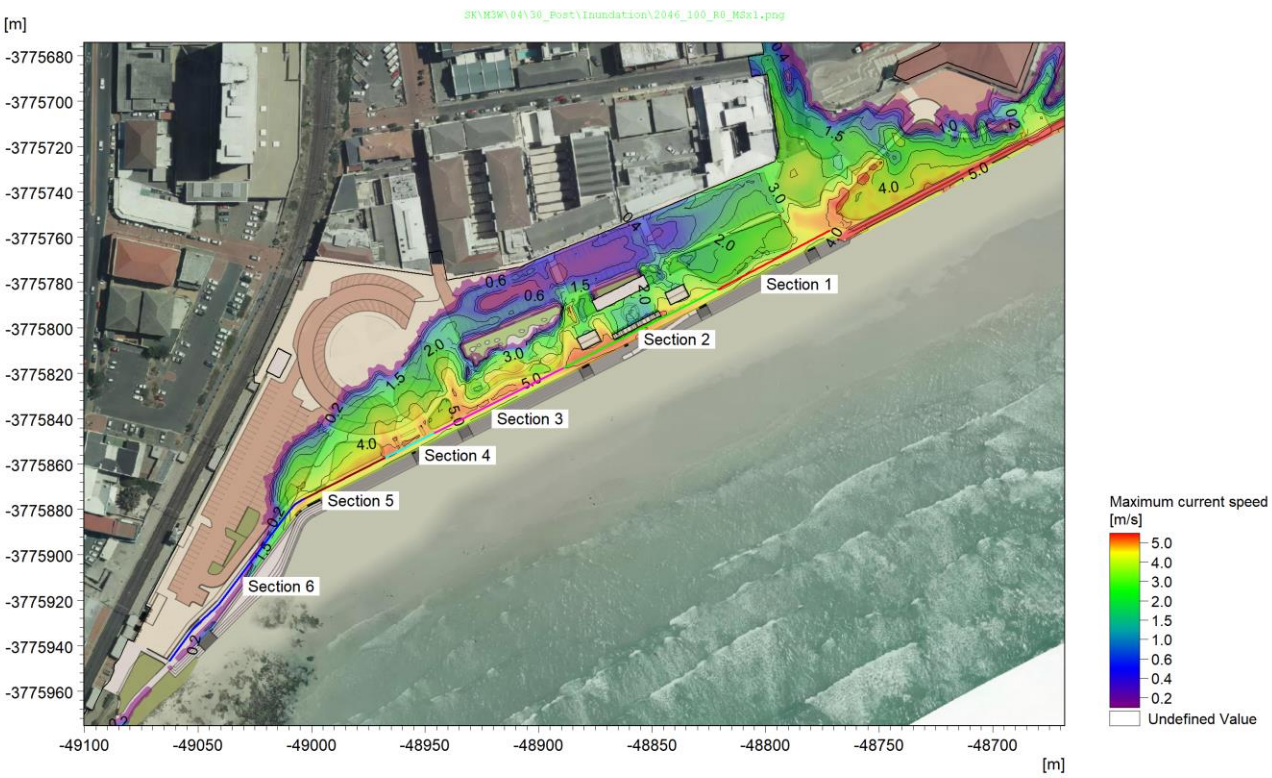


Figure 3-16: 2046; Maximum current speed behind the proposed revetment for the 100-year return period (100-year wave with a 1-year water level).

Figure 3-17: 2076; Maximum current speeds behind the proposed revetment for the 1-year return period (1-year wave with a 1-year water level).

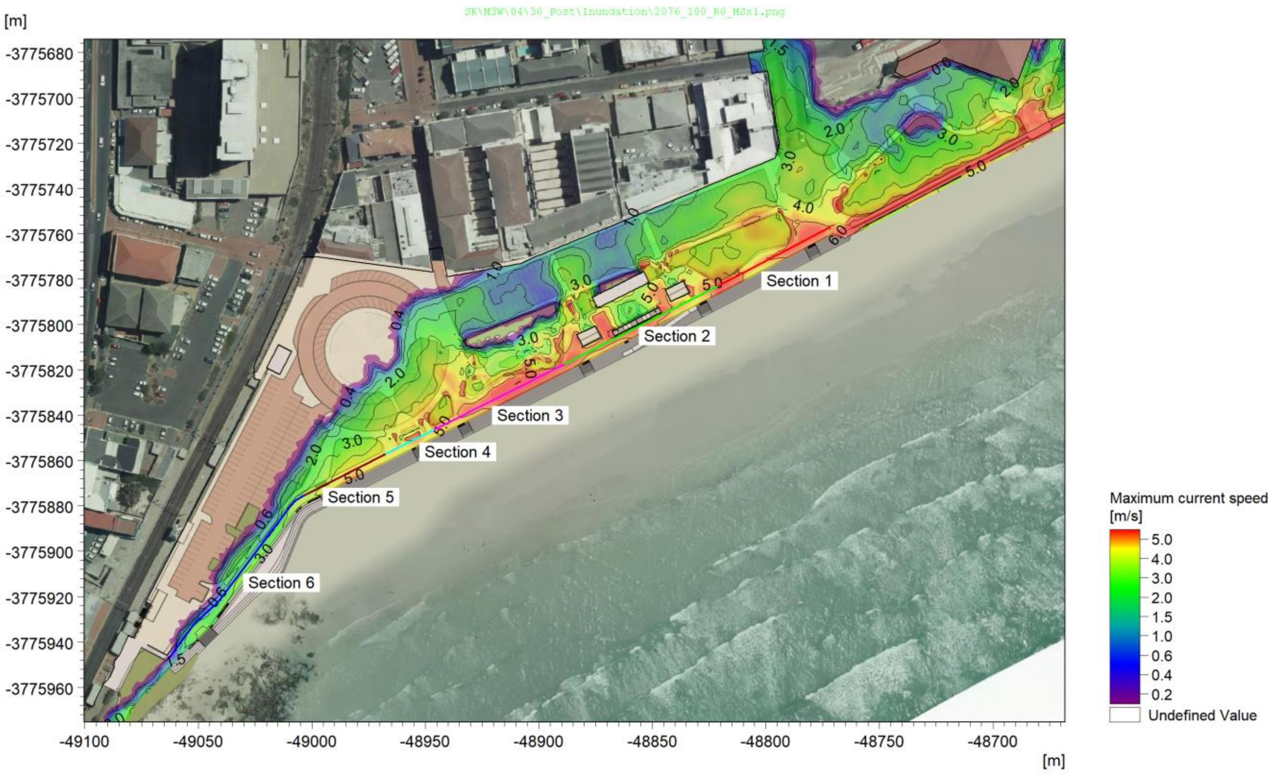


Figure 3-18: 2076; Maximum current speeds behind the proposed revetment for the 100-year return period (100-year wave with a 1-year water level).

Table 3-9: Maximum current speed behind the proposed revetment.

Extreme condition		Maximum current speed [m/s]						
Climate change horizon	Return period [y]	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Max
2026	1	1.17	-	-	-	-	-	1.17
2046	100	5.04	8.51	5.79	5.67	5.33	5.08	8.51
2076	1	2.60	3.30	3.04	2.79	2.70	-	3.30
2076	100	6.73	9.08	7.38	6.36	4.79	4.25	9.08

The current speeds were the strongest where the waves overtop the promenade without obstructions (e.g., buildings, steps or ramps). Similar to the overtopping and water depths, an increase in storm severity (1-year to 100-year) or climate change horizon (2026/2046 to 2076) typically resulted in increased current speeds, except for Surfers Corner where maximum current speeds reduced from 2046 to 2076 – analogous to the maximum water depths.

4. WAVE REFLECTION ASSESSMENT

4.1 Introduction

This section covers the analyses of changes in wave reflection between the existing seawall and proposed revetment, and the potential impact on surfing conditions. Numerical flume (2DV) models have been applied to compare the reflections between the existing and proposed structures for three climate change scenarios (2026 (SSP5-8.5), 2046 (SSP1-2.6 and SSP5-8.5)) and two typical surfing conditions. Full domain (3D) simulations were used to model a subset of the scenarios (2026, SSP5-8.5) to quantify and compare the amount of reflected wave energy for the existing seawall and proposed revetment.

4.2 Numerical flume

4.2.1 Bathymetry and mesh

The meshes used for the numerical flume wave reflection assessment are presented in Section 3.2.1. For this assessment only three climate change scenarios and two structural configurations with a typical eroded beach level were considered, thus 6 unique meshes (modelled scenarios are discussed further in 4.2.3).

4.2.2 Model parameters

Bottom friction was modelled using a roughness height formulation to include the following processes:

- Energy dissipation due to bottom friction;
- Wave overtopping of stepped revetments.

Bottom friction was modelled using a roughness height of k_s = 0.05 m across the whole domain, while the proposed revetment set to k_s = 1.46 m to resolve the 0.25 m step height (refer to Section 2.4).

4.2.3 Scenarios modelled

Table 4-1 summarises the 12 scenarios modelled in the numerical flume the existing seawall and proposed revetment. For all scenarios a typical eroded beach level was modelled (refer to Section 2.3.1).

Table 4-1: Typical surfing conditions modelled for the numerical flume wave reflection assessment.

Climate change		Wave Case	Seawall configuration	Wave Parameters			
Projection	Climate change			H_{M0} [m]	T_p [s]	DSD [deg]	SWL [m MSL]
SSP5-8.5	2026	A	Existing	0.91	12.5	13	1.05
		A	Proposed	0.91	12.5	13	1.05
		B	Existing	1.86	14	13	1.05
		B	Proposed	1.86	14	13	1.05
	2046	A	Existing	0.91	12.5	13	1.19
		A	Proposed	0.91	12.5	13	1.19
		B	Existing	1.86	14	13	1.19
		B	Proposed	1.86	14	13	1.19
SSP1-2.6	2046	A	Existing	0.91	12.5	13	1.15
		A	Proposed	0.91	12.5	13	1.15
		B	Existing	1.86	14	13	1.15
		B	Proposed	1.86	14	13	1.15

4.2.4 Reflections analysis

To separate wave reflections from the seawall, each modelled scenario was repeated with the structure replaced with an absorbing sponge layer to remove all reflections caused by the seawall. The reflected energy from the seawall can then be quantified in terms of total (H_{m0}), incident (H_{m0_i}) and reflected (H_{m0_r}) significant wave height, where:

$$H_{m0} = \sqrt{{H_{m0_i}}^2 + {H_{m0_r}}^2}$$
$$\therefore H_{m0_r} = \sqrt{{H_{m0}}^2 - {H_{m0_i}}^2},\ H_{m0_i} = \sqrt{{H_{m0}}^2 - {H_{m0_r}}^2} \quad \textbf{Eq 4-1}$$

The wave reflection coefficient, R, is the ratio of the reflected wave height relative to incident wave height, thus:

$$R = H_{m0_r}/H_{m0_i} \quad \textbf{Eq 4-2}$$

The incident waves in this case only exclude the reflections off the seawall but can include reflections from the beach in front of the wall. This method does not account for non-linear interactions between incident and reflected waves, but the results showed that these were relatively small.

4.2.5 Model results

Figure 4-1 presents an example comparison of instantaneous total, incident and reflected surface elevation for the existing seawall and proposed revetment for Wave Case B (H_{m0} = 1.86 m, T_p = 14.0 s) and a 2046 SSP5-8.5 median SLR scenario.

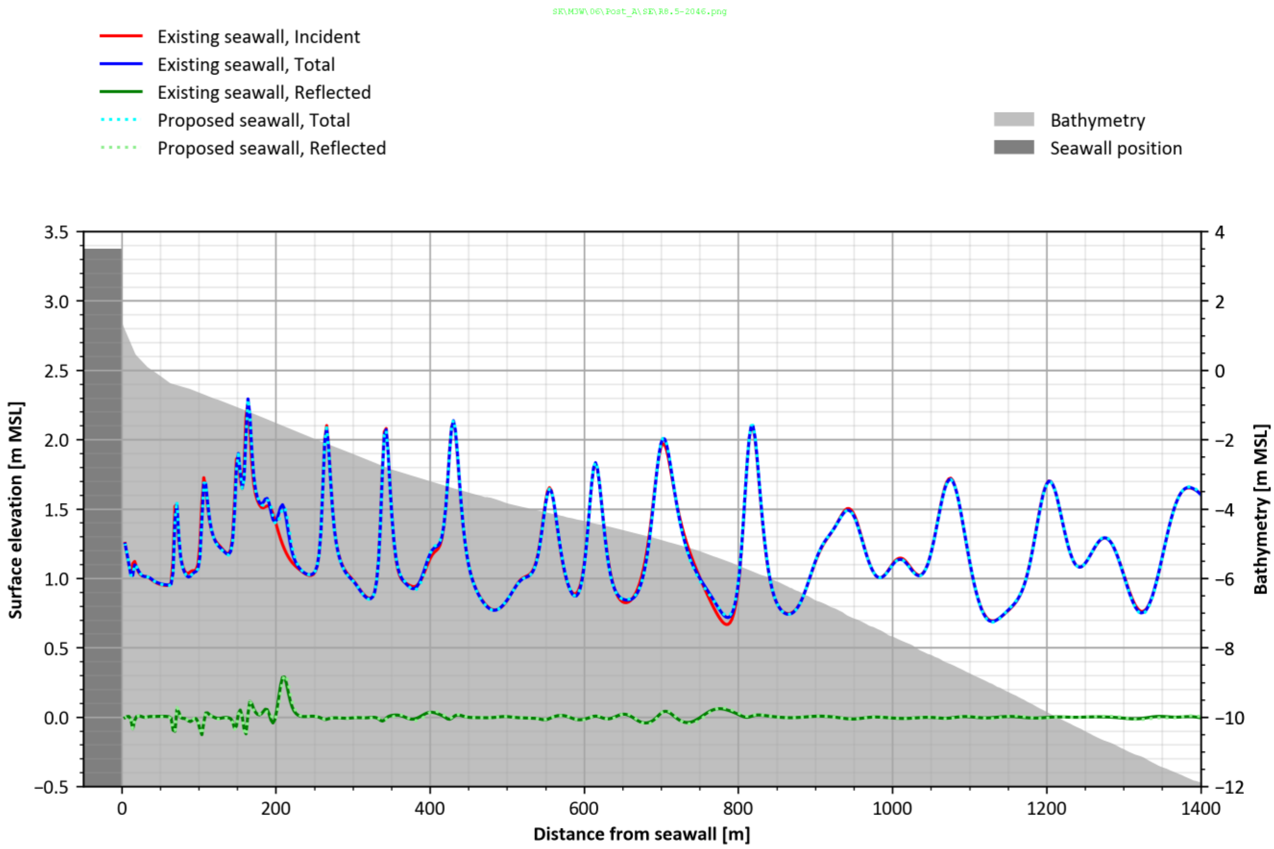


Figure 4-1: 2046, SSP5-8.5; Example comparison of instantaneous total, incident and reflected surface elevation for the existing seawall (solid line) and proposed revetment (dashed line). Note the seawall is plotted schematically to visualise the position of both seawalls.

For the timestep shown it can be seen that the reflections are relatively small and nearly identical for the existing seawall and proposed revetment.

The results have been processed using a time-domain analysis on the surface elevations to obtain H_{m0} for all the scenarios modelled. Incident and reflected wave heights have been separated from the total wave height using the methodology outlined in Section 4.2.4. The cross-shore distribution of heights for the existing seawall and proposed revetment during typical surfing conditions are presented in the following figures:

- Figure 4-2: 2026, SSP5-8.5 median SLR scenario
- Figure 4-3: 2046 SSP5-8.5 median SLR scenario
- Figure 4-4: 2046, SSP1-2.6 median SLR scenario

A summary of wave heights at two locations, 40 m, 125 m and 375 m from the existing seawall or proposed revetment (similar to output locations in full domain analysis in Section 4.3.4), are provided in Table 4-2.

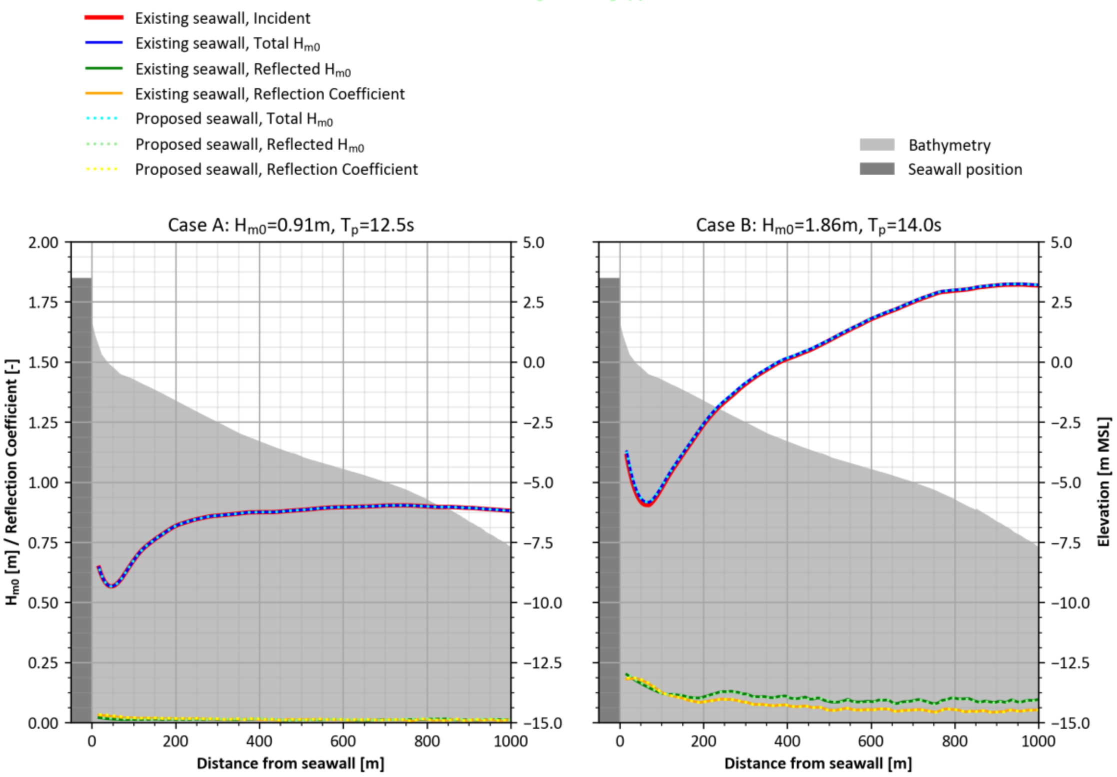


Figure 4-2: 2026, SSP5-8.5; cross-shore significant wave heights for the existing seawall and proposed revetment during typical surfing conditions.

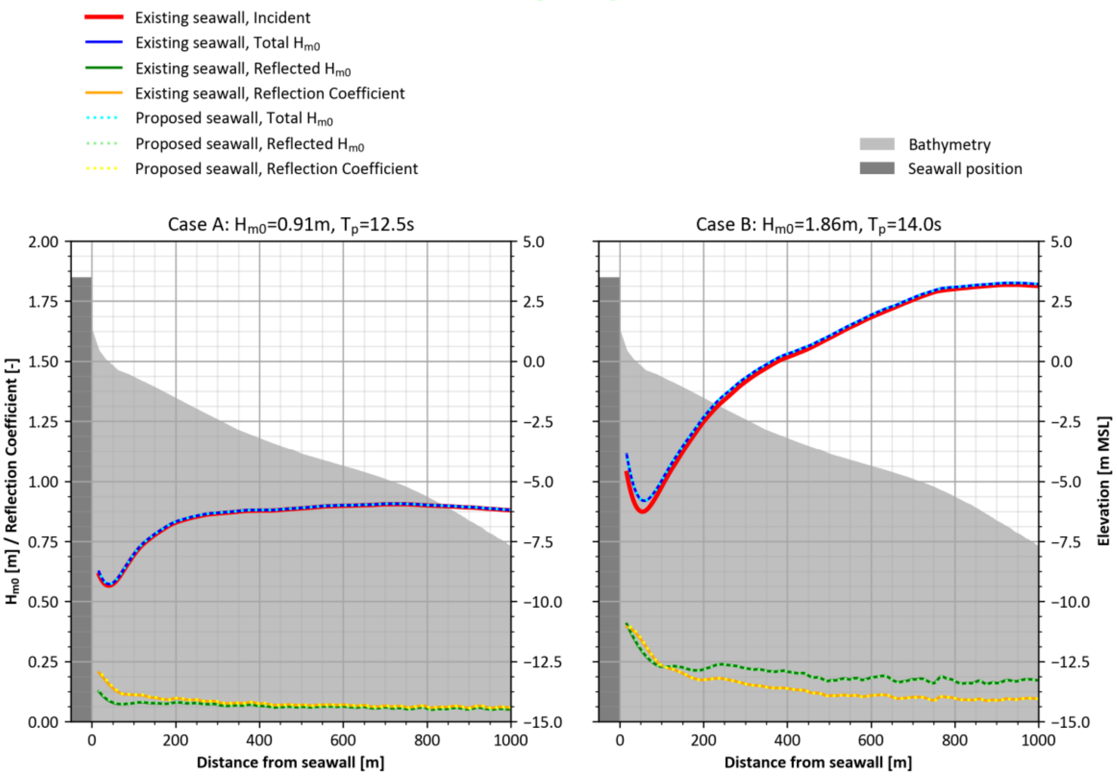


Figure 4-3: 2046, SSP5-8.5; cross-shore significant wave heights for the existing seawall and proposed revetment during typical surfing conditions.

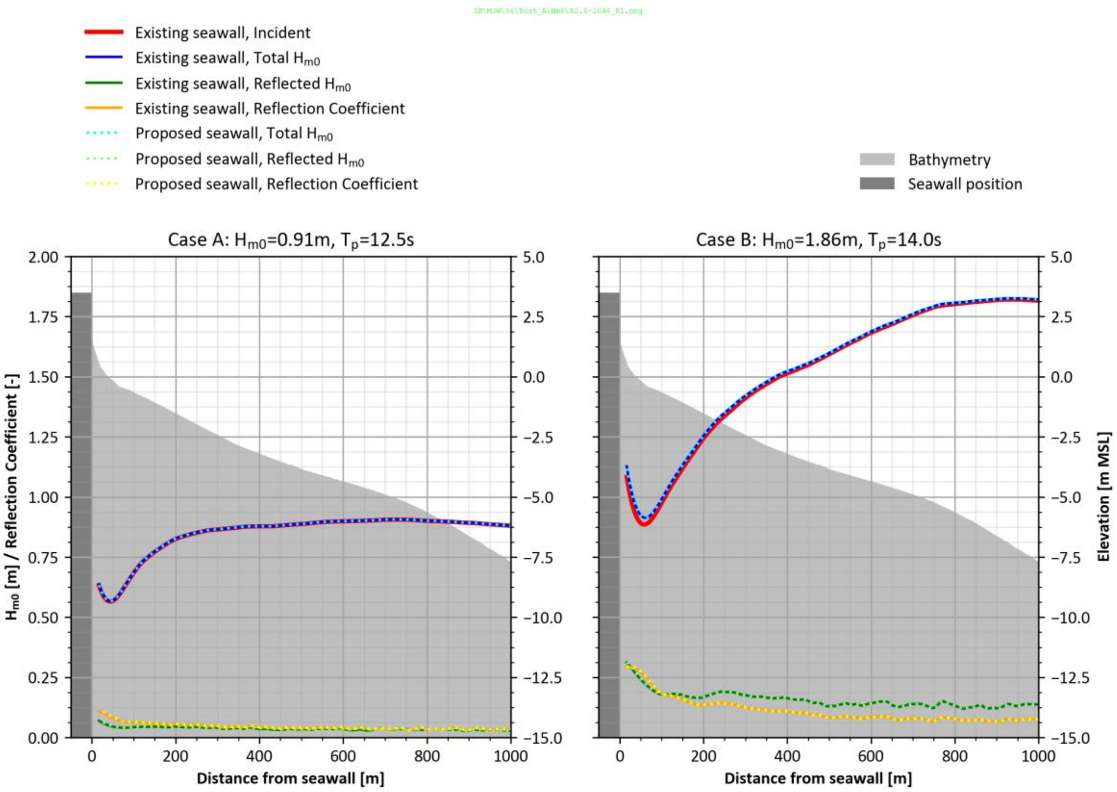


Figure 4-4: 2046, SSP1-2.6; cross-shore significant wave heights for the existing seawall and proposed revetment during typical surfing conditions.

Table 4-2: Total, incident and reflected significant wave heights for the existing seawall and proposed revetment during typical surfing conditions.

Climate change	Wave Case	Seawall	40 m from seawall				125 m from seawall				375 m from seawall			
			H_{m0} [m]	H_{m0i} [m]	H_{m0r} [m]	R [-]	H_{m0} [m]	H_{m0i} [m]	H_{m0r} [m]	R [-]	H_{m0} [m]	H_{m0i} [m]	H_{m0r} [m]	R [-]
2026, SSP5-8.5	A	Existing	0.57	0.57	0.02	0.03	0.73	0.73	0.01	0.02	0.87	0.87	0.01	0.02
		Proposed	0.57	0.57	0.02	0.03	0.73	0.73	0.01	0.02	0.87	0.87	0.02	0.01
	B	Existing	0.97	0.96	0.17	0.18	1.05	1.05	0.12	0.11	1.50	1.49	0.11	0.07
		Proposed	0.97	0.96	0.18	0.19	1.05	1.05	0.12	0.11	1.50	1.49	0.11	0.07
2046, SSP5-8.5	A	Existing	0.57	0.57	0.09	0.16	0.74	0.74	0.08	0.11	0.88	0.88	0.07	0.08
		Proposed	0.57	0.57	0.09	0.16	0.74	0.74	0.08	0.11	0.88	0.88	0.07	0.08
	B	Existing	0.95	0.90	0.33	0.37	1.07	1.05	0.23	0.22	1.51	1.50	0.21	0.14
		Proposed	0.96	0.90	0.34	0.37	1.07	1.05	0.23	0.22	1.51	1.50	0.21	0.14
2046, SSP1-2.6	A	Existing	0.57	0.57	0.05	0.09	0.74	0.73	0.04	0.06	0.88	0.88	0.04	0.05
		Proposed	0.57	0.57	0.05	0.09	0.74	0.73	0.05	0.06	0.88	0.88	0.04	0.05
	B	Existing	0.96	0.92	0.26	0.29	1.06	1.05	0.18	0.17	1.50	1.50	0.17	0.11
		Proposed	0.96	0.92	0.27	0.29	1.06	1.05	0.18	0.17	1.50	1.50	0.16	0.11

For all the scenarios modelled it can be seen that the wave reflection coefficients for the existing seawall and proposed revetment are nearly identical. This corresponds to the overtopping results (Section 3.2.4) which showed marginal differences between the two structures.

The reflected wave height is greater closer to the seawall, reducing over the first 100 m due to dissipation and shoaling and then remains fairly constant.

For the present day (2026) and the smaller wave case (A) the waves seldomly reach the seawall, thus resulting in a low reflection coefficient (R = 0.03 at 40 m from the seawall), while the larger waves do reflect more (R = 0.18 to 0.19).

For the more distant climate change horizon (2046) the increase in water level due to SLR and the resultant lower beach level in front of the structure increased reflection for both Case A (R = 0.16 at 40 m from the seawall) and Case B (R = 0.37). When comparing the SSP1-2.6 projection to the SSP5-8.5 projection the reduced severity of climate change and impact on the beach level reduces the reflections to R = 0.09 for Case A and R = 0.29 for Case B. Both projections show an increase in wave reflections from the proposed revetment in future when compared to present day conditions.

4.3 Full domain

4.3.1 Bathymetry and mesh

Bathymetries adjusted for climate change for 2026 were set up for both the existing and the proposed revetment. Only a typical eroded upper beach profile was considered, thus two unique meshes were set up. The 2026 bathymetry including the proposed revetment is presented in Figure 2-3.

The model mesh comprises triangles with a resolution varying between approximately 7.3 m offshore and approximately 3.4 m nearshore. This resulted in a resolution of approximately 20 points per wavelength for a peak wave period of T_p = 12.5 s. Structures with steps or steep slopes were modelled with quadrangular elements with a resolution of 2.8 m.

The wave generation line was placed along the -15 m MSL contour. Since the output of interest was wave reflection from the seawall or revetment, the landside detail was not resolved in the meshes. Free outflow conditions were prescribed on the open boundaries on the landside to prevent waves from reflecting from the boundaries.

The vertical mesh comprises three sigma layers with the bottom layer comprising 50%, the middle layer 30% and the surface layer 20% of the water column. The model computational mesh for the existing seawall and proposed revetment are shown in Figure 4-5 and Figure 4-6.

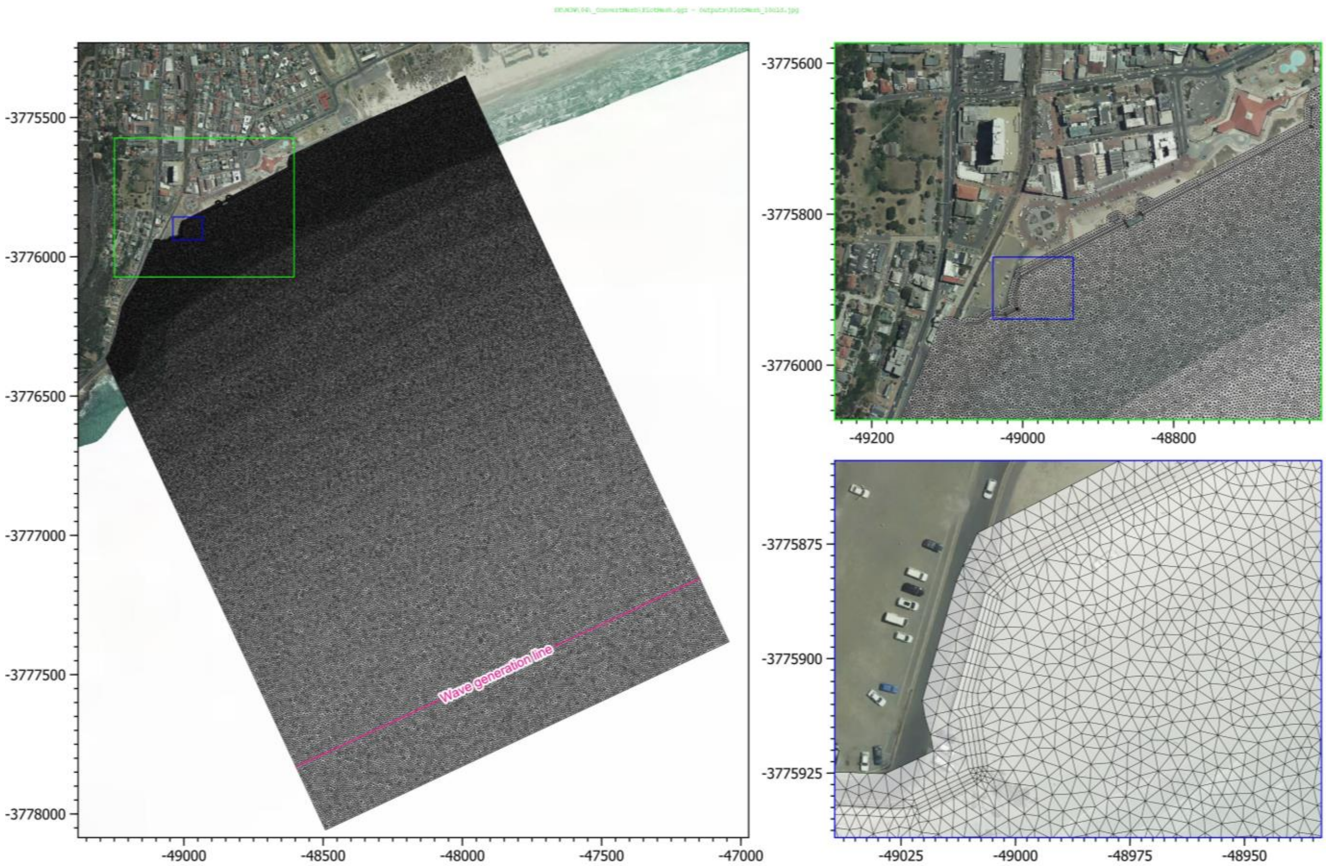


Figure 4-5: Computational mesh used in wave reflection analysis for the existing seawall.

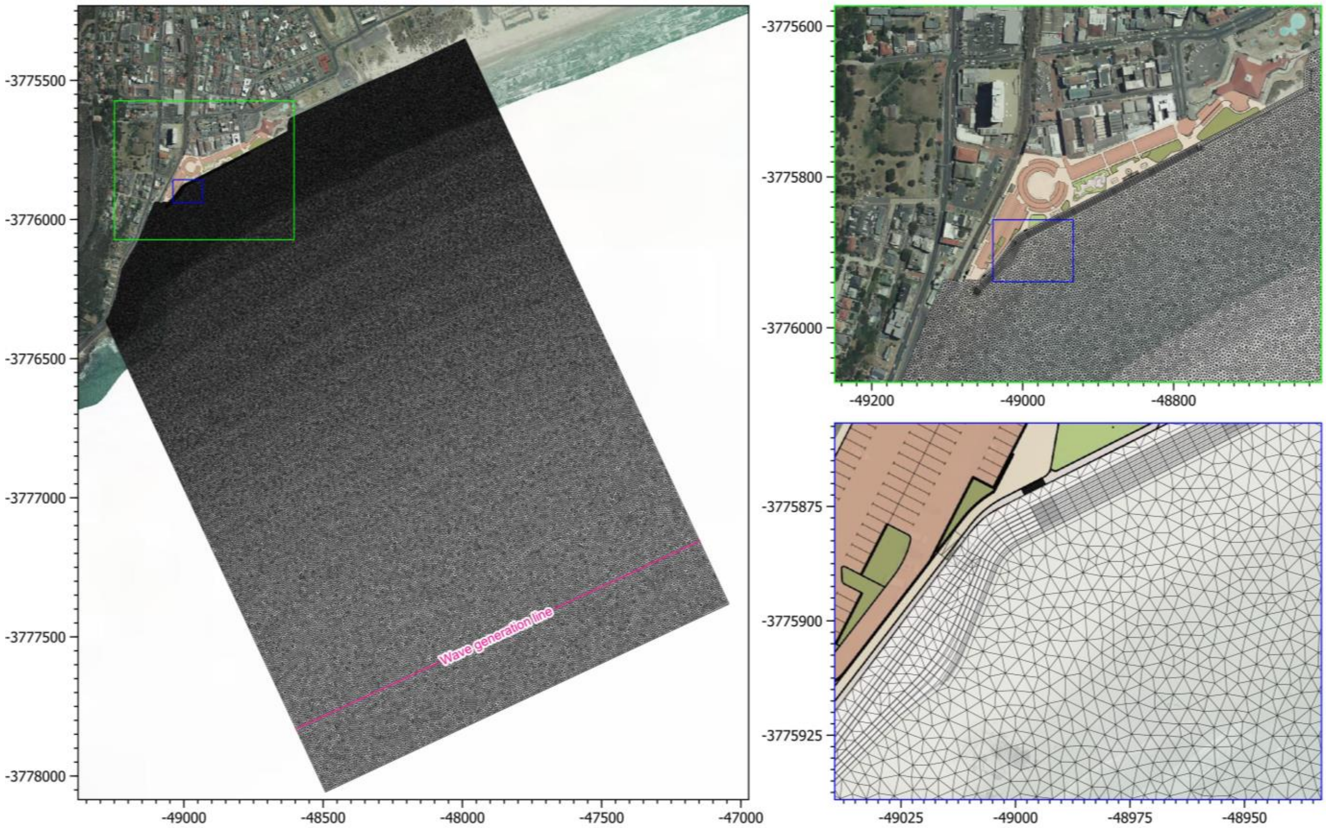


Figure 4-6: Computational mesh used in wave reflection analysis for the proposed revetment.

4.3.2 Model parameters

Bottom friction was modelled using a roughness height formulation identical to the setup described in Section 3.3.2

4.3.3 Scenarios modelled

Table 4-3 summarises the four scenarios modelled in the full domain for the wave refection analysis. For all scenarios the 2026 (SSP5-8.5) projection with a typical eroded beach level was modelled.

Table 4-3: Typical surfing conditions modelled for the full domain wave reflection assessment.

Climate change		Seawall configuration	Boundary conditions				
Projection	Climate change		Wave case	H_{m0} [m]	T_p [s]	DSD [deg]	SWL [m MSL]
SSP5-8.5	2026	Existing	A	0.91	12.5	13	1.05
	2026	Proposed	A	0.91	12.5	13	1.05
	2026	Existing	B	1.86	14	13	1.05
	2026	Proposed	B	1.86	14	13	1.05

4.3.4 Model results

Figure 4-7 presents a comparison of instantaneous surface elevation for the existing seawall and proposed revetment for wave Case B (H_{m0} = 1.86 m, T_p = 14.0 s).

SK\Models\M3W\04\10_Post\CompareB_2D_Area_R1.pptx

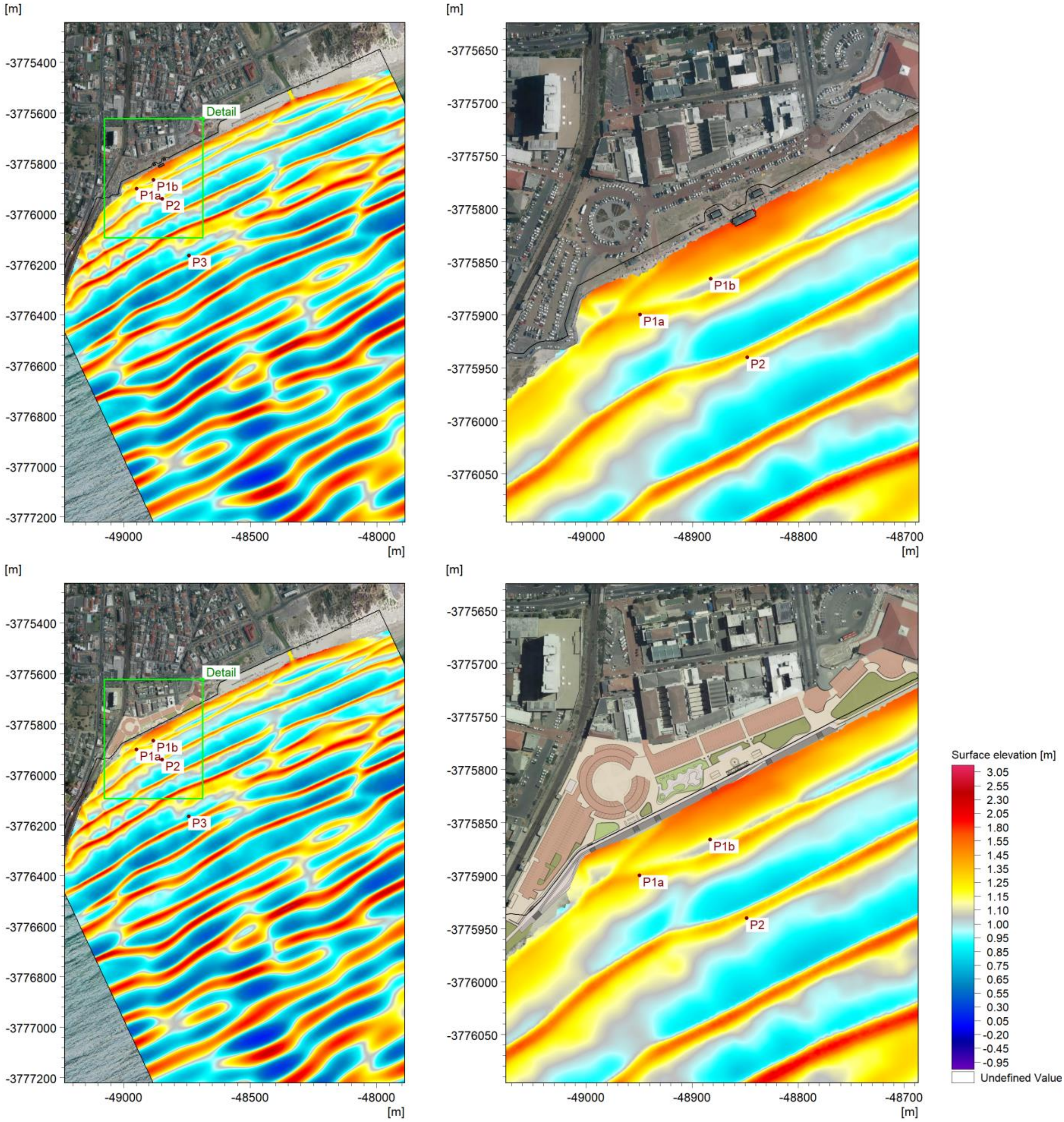


Figure 4-7: Instantaneous plot of surface elevation for the existing seawall (top) and the proposed revetment (bottom) for wave Case B.

Subtle differences between the existing seawall and proposed revetment can be observed for the timestep shown. Due to the new structural configuration the wave reflections at Surfers Corner are slightly reduced in height and reflect more energy offshore whereas the existing seawall tends to reflect waves shore-parallel (north-eastwards). Some reflections from the ablution facility were observed for the existing seawall, but the waves disperse within a short distance.

The changes in wave reflection between the existing seawall and proposed revetment were further assessed by comparing H_{m0} obtained from a 2D time-domain analysis on the surface elevations. Contours of H_{m0} are shown in Figure 4-8 and Figure 4-9 for the existing seawall and proposed revetment. H_{m0} difference plots ($H_{m0,proposed}$ – $H_{m0,existing}$) follow in Figure 4-10 and Figure 4-11 for wave Case A and B respectively.

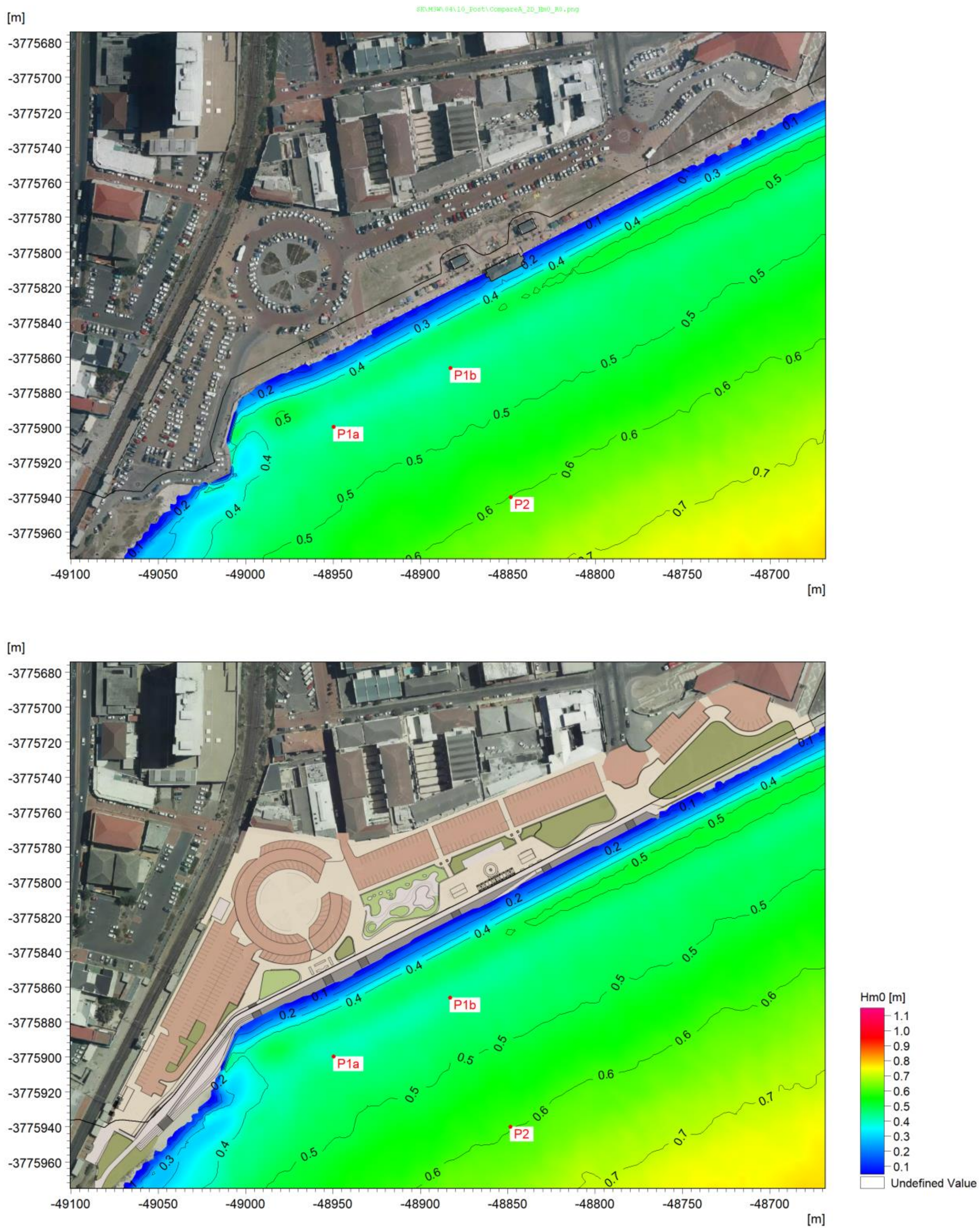


Figure 4-8: Contour plot of H_{m0} for the existing seawall (top) and the proposed revetment (bottom) for wave Case A.

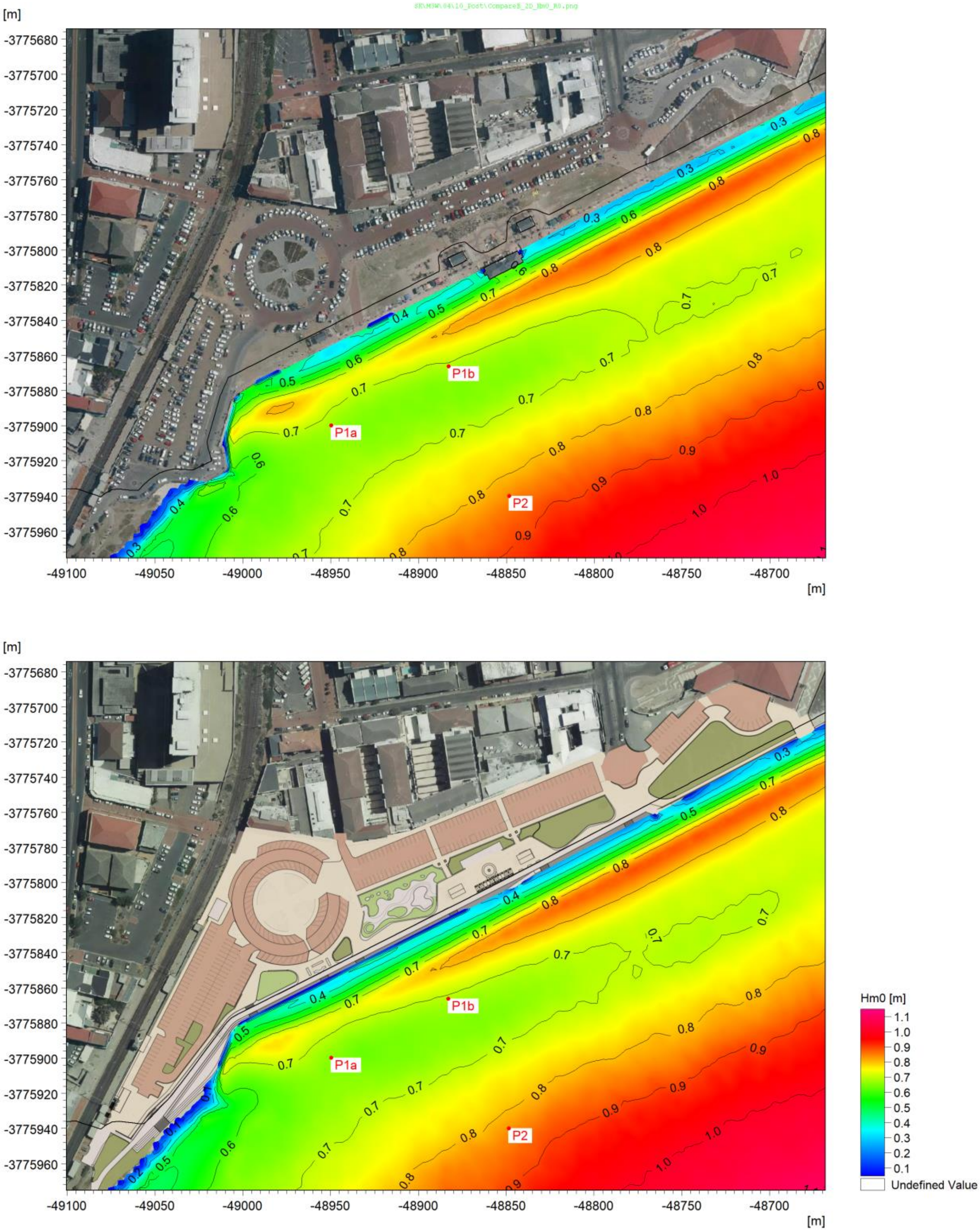


Figure 4-9: Contour plot of H_{m0} for the existing seawall (top) and the proposed revetment (bottom) for wave Case B.

Figure 4-10: H_{m0} difference plot showing the difference between the proposed and existing seawall for wave Case A. A positive change indicates an increase in wave height for the proposed revetment.

Figure 4-11: H_{m0} difference plot showing the difference between the proposed and existing seawall for wave Case B. A positive change indicates an increase in wave height for the proposed revetment.

The H_{m0} and H_{m0}-difference plots show a slight reduction (-0.13 m for Case A, -0.08 m for Case B) at Surfers Corner, which quickly dissipates offshore. This reduction supports the qualitative assessment in the *Wave and Sediment Transport Modelling* study (PRDW, 2022a), in which reflected wave energy in Surfers Corner was expected to be lower for the proposed structure. The relative impact of the existing ablution facility on the beach is also shown to be relatively small (-0.05 m for Case A, -0.13 m for Case B) and confined to a very small area. Differences in the order of ±0.05 m can be seen across the domain, likely caused by phase and directional differences in long-wave reflections, but the impact is considered negligible to surfing conditions.

A directional wave analysis was performed on outputs of surface elevation and orthogonal velocities at four locations (P1a, P1b, P2 and P3 – refer to Figure 4-7) to obtain 2D spectra (frequency and direction). The incident and reflected wave energy was estimated by separating the energy based on the shoreline orientation (roughly 65^{o}TN to 245^{o}TN). This differs from the method used in the numerical flume (as described in Section 4.2.4) as the reflected energy in the spectra accounts for all the energy reflected from the beach. Figure 4-12 and Figure 4-13 compare the spectra from the existing seawalls and proposed revetment at P1a, P1b, P2 and P3 for wave Case A and B respectively. A summary of total, incident, reflected wave heights and the resultant reflection coefficients, physically separated by the shoreline orientation, is provided in Table 4-4

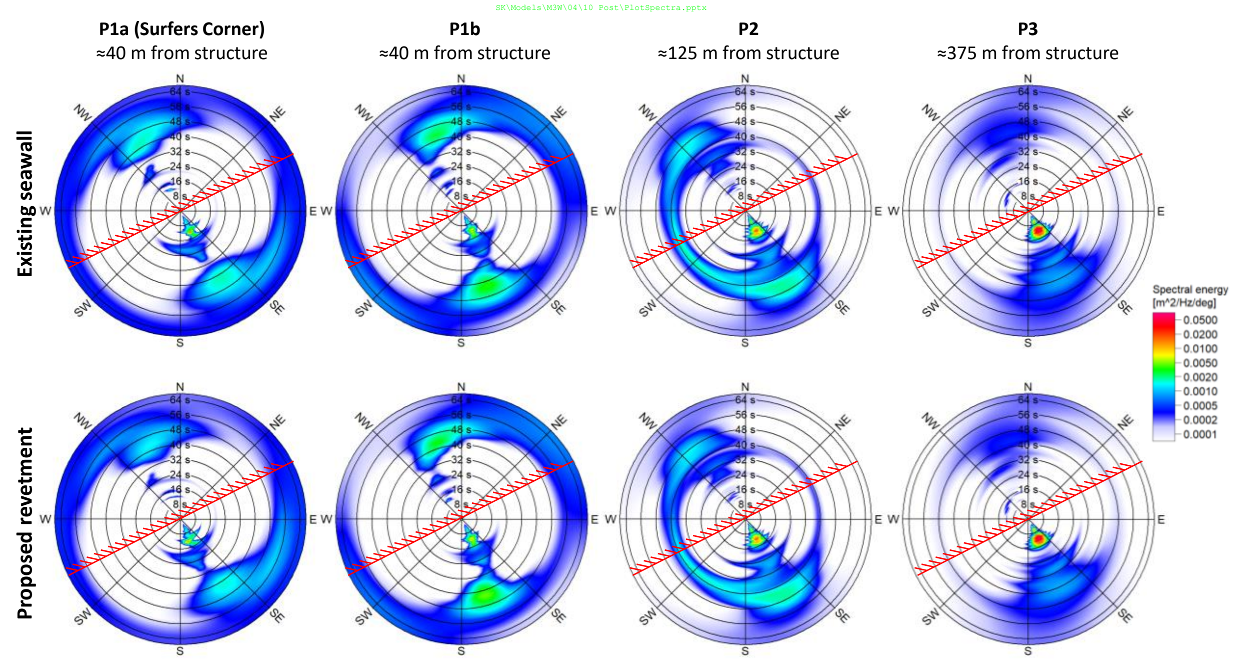


Figure 4-12: Wave Case A; Comparison of wave spectra at P1a, P1b,P2 and P3 for the existing seawall and proposed revetment. The beach orientation is visualised with a red line.

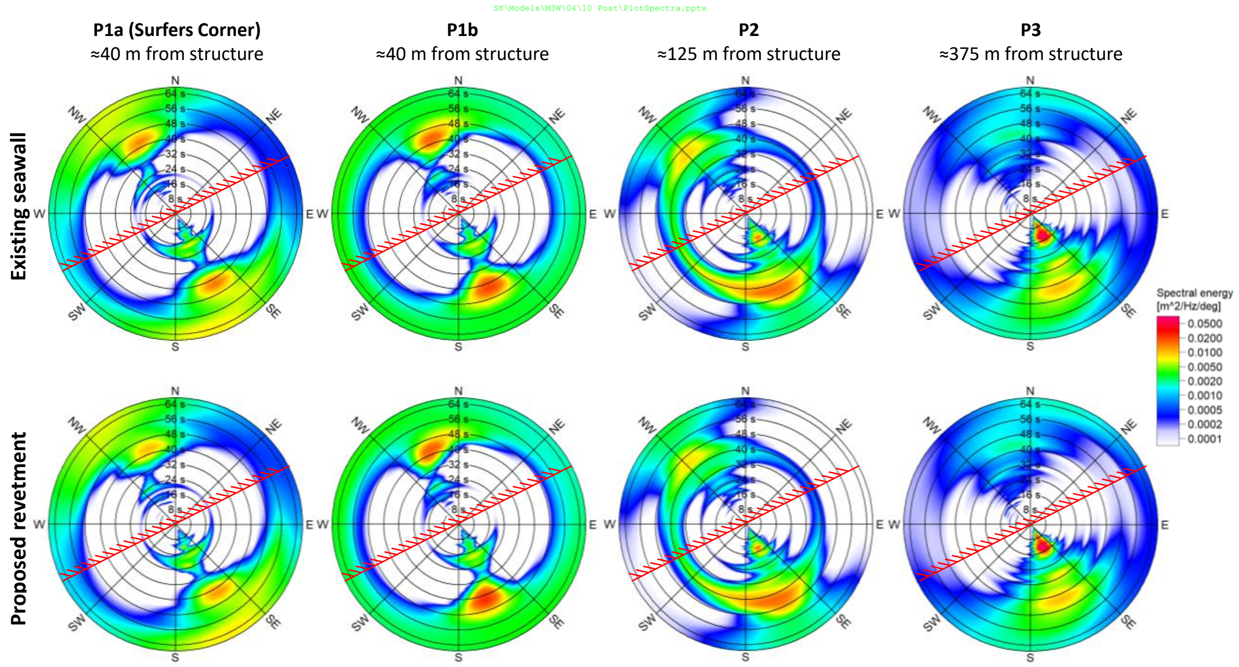


Figure 4-13: Wave Case B; Comparison of wave spectra at P1a, P1b,P2 and P3 for the existing seawall and proposed revetment. The beach orientation is visualised with a red line.

Table 4-4: Summary of spectral wave heights at P1a, P1b, P2 and P3 for the existing seawall and proposed revetment.

Wave Case	Structure	P1a (Surfers Corner) (≈40 m from structure)				P1b (≈40 m from structure)				P2 (≈125 m from structure)				P3 (≈375 m from structure)			
		H_{m0} [m]	$H_{m0,i}$ [m]	$H_{m0,r}$ [m]	R [-]	H_{m0} [m]	$H_{m0,i}$ [m]	$H_{m0,r}$ [m]	R [-]	H_{m0} [m]	$H_{m0,i}$ [m]	$H_{m0,r}$ [m]	R [-]	H_{m0} [m]	$H_{m0,i}$ [m]	$H_{m0,r}$ [m]	R [-]
A	Existing	0.43	0.40	0.16	0.40	0.42	0.39	0.16	0.40	0.60	0.58	0.15	0.26	0.83	0.82	0.12	0.15
A	Proposed	0.43	0.40	0.16	0.39	0.41	0.39	0.15	0.38	0.60	0.58	0.15	0.26	0.84	0.83	0.12	0.15
B	Existing	0.69	0.59	0.36	0.61	0.67	0.57	0.35	0.62	0.83	0.77	0.33	0.42	1.28	1.26	0.22	0.18
B	Proposed	0.68	0.58	0.35	0.60	0.66	0.56	0.34	0.60	0.82	0.76	0.32	0.42	1.29	1.27	0.22	0.18

The spectral analyses confirm that the wave reflection coefficients for the existing seawall and proposed revetment are nearly identical. At P1a and P1b, closer to the seawall, the reflections only showed a very minor decrease for the proposed revetment for both Case A and Case B. Although the incident wave heights may be expected to be the same, non-linear interaction with reflected waves could cause differences between the two structural configurations.

As the incident short waves start breaking in shallow water infragravity waves in the surf zone are generated. The reflections of the shorter waves dissipate and disperse relatively quickly while the reflected infragravity waves travel much further – as seen in the spectra plots. As a result of the short-wave dominant incident waves and infragravity dominant reflected waves, the reflection coefficients show a strong decrease from nearshore to offshore (e.g., ≈0.60 to ≈0.18 for Case B), while the actual change in reflected wave height is more gradual (e.g., ≈0.35 m to ≈0.22 m for Case B). These coefficients are also much larger relative to the numerical flume results as the reflection coefficients account for wave reflections off the structures and the beach.

Overall, the results agree that the changes in wave reflection between the existing seawall and proposed revetment are very minor, and the potential impact on surfing conditions would be negligible.

5. SUMMARY

The MIKE 3 Wave model was used to assess the overtopping and resultant flooding for the project at the proposed levels and analyse the changes in wave reflection between the existing seawall and proposed revetment, and the potential impact on surfing conditions.

Overtopping and flooding assessment

Numerical flume (2DV) models have been used to test the sensitivity of the results to the climate change projection, a raised crest level for adaptive design and a comparison of the existing seawall to the proposed revetment.

Raising the crest level to 4.5 m reduces the overtopping by 80% for 10-year and 100-year events while overtopping for 1-year events are almost non-existent.

When comparing the SSP1-2.6 projection (a low emissions scenario) to the SSP5-8.5 projection (the most conservative scenario) the overtopping rates were modestly lower (on average 7% lower for 2046, and 32% lower for 2076) due to a lower still water depth and less eroded profile.

Relative to the existing seawall, the proposed revetment showed a small (< 7 %) increase in overtopping discharge.

The results informed the selection of four scenarios for the full domain (3D) simulations to quantify the overtopping and resultant flooding along the full length of the proposed revetment under climate change projection SSP5-8.5.

The mean overtopping discharge from the full domain simulations are provided below:

Extreme condition		Mean overtopping discharge [l/s/m]					
Climate change horizon	Return period [y]	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
2026	1	<0.1	-	-	-	-	-
2046	100	7.3	8.0	5.3	1.8	1.6	0.1
2076	1	0.5	0.8	0.7	0.3	0.1	-
2076	100	24.0	32.1	26.4	7.8	7.0	1.0

Comparing the modelled overtopping rates to EurOtop limits showed that for the proposed revetment 1-year storm events present no hazard to vehicles or pedestrians along any of the sections. A 100-year event during 2046 is hazardous to pedestrians, while a 100-year event during 2076 becomes hazardous to vehicles along Sections 1 to 3.

The maximum depths and current speeds from the full domain simulations are summarised in the two tables below:

Extreme condition		Maximum depth [m]						
Climate change horizon	Return period [y]	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Max
2026	1	0.09	-	-	-	-	-	0.09
2046	100	0.92	2.60	1.28	0.95	0.86	0.88	2.60
2076	1	0.47	0.59	0.47	0.44	0.32	-	0.59
2076	100	1.67	4.61	1.83	1.10	0.86	0.71	4.61

Extreme condition		Maximum current speed [m/s]						
Climate change horizon	Return period [y]	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Max
2026	1	1.17	-	-	-	-	-	1.17
2046	100	5.04	8.51	5.79	5.67	5.33	5.08	8.51
2076	1	2.60	3.30	3.04	2.79	2.70	-	3.30
2076	100	6.73	9.08	7.38	6.36	4.79	4.25	9.08

At the eastern end of the proposed revetment (Section 1), the relatively larger overtopping rates, combined with lower infrastructure levels and an unobstructed pathway, causes a weak spot resulting in increased flooding behind the promenade. The maximum water depths are typically associated with locations where waves runup against the seaward faces of structures (e.g., Section 2).

For a 1-year event in 2026 only the seaward edge of the promenade was inundated. For an increase in climate change horizon (2046) most of the promenade was overtopped, while the parking areas and most of the elevated lawns and vegetated areas remained dry.

For the 100-year events most of the parking areas were inundated, with only the more elevated western areas remaining dry. For an increased climate change horizon (2046 to 2076) the flooding extent and severity is generally worse, except at Surfers Corner where the contour plots show a slightly more landward flood line for the 2046 scenario, demonstrating the non-linearity of overtopping processes in a complex 3D environment.

The current speeds were the strongest where the waves overtop the promenade without obstructions (e.g., buildings, steps or slopes). Similar to the overtopping and water depths, an increase in storm severity (1-year to 100-year) or climate change horizon (2026/2046 to 2076) typically resulted in increased current speeds, except for Surfers Corner where maximum current speeds reduced from 2046 to 2076 – analogous to the maximum water depths.

Wave reflection assessment

Numerical flume models have been applied to compare the reflections between the existing and proposed structures for three climate change scenarios (2026 (SSP5-8.5), 2046 (SSP1-2.6 and SSP5-8.5)) and two typical surfing conditions. Both projections show an increase in wave reflections from the proposed revetment in future when compared to present day conditions. Subsequently a subset of the scenarios (2026, SSP5-8.5) was modelled in full domain simulations to quantify and compare the amount of reflected wave energy for the existing seawall and proposed revetment.

Spatial comparisons of significant wave height (H_{m0}) changes due to the new structures showed a slight reduction (-0.08 m to -0.13 m) at Surfers Corner, which quickly dissipates offshore. This reduction supports the qualitative assessment in the *Wave and Sediment Transport Modelling* study (PRDW, 2022a), in which reflected wave energy in Surfers Corner was expected to be lower for the proposed structure. The relative impact of the existing ablution facility on the beach was also shown to be relatively small (-0.05 m to -0.13 m) and confined to a very small area.

A directional wave analyses was performed on outputs of surface elevation and orthogonal velocities at four locations to obtain 2D spectra and estimate reflected energy from a directional analysis of the spectral parameters. The results are provided below:

Wave Case	Structure	P1a (Surfers Corner) (≈40 m from structure)				P1b (≈40 m from structure)				P2 (≈125 m from structure)				P3 (≈375 m from structure)			
		H_{m0}	$H_{m0,i}$	$H_{m0,r}$	R	H_{m0}	$H_{m0,i}$	$H_{m0,r}$	R	H_{m0}	$H_{m0,i}$	$H_{m0,r}$	R	H_{m0}	$H_{m0,i}$	$H_{m0,r}$	R
		[m]	[m]	[m]	[-]	[m]	[m]	[m]	[-]	[m]	[m]	[m]	[-]	[m]	[m]	[m]	[-]
A	Existing	0.43	0.40	0.16	0.40	0.42	0.39	0.16	0.40	0.60	0.58	0.15	0.26	0.83	0.82	0.12	0.15
A	Proposed	0.43	0.40	0.16	0.39	0.41	0.39	0.15	0.38	0.60	0.58	0.15	0.26	0.84	0.83	0.12	0.15
B	Existing	0.69	0.59	0.36	0.61	0.67	0.57	0.35	0.62	0.83	0.77	0.33	0.42	1.28	1.26	0.22	0.18
B	Proposed	0.68	0.58	0.35	0.60	0.66	0.56	0.34	0.60	0.82	0.76	0.32	0.42	1.29	1.27	0.22	0.18

Overall, the results agree that the changes in wave reflection between the existing seawall and proposed revetment are very minor, and the potential impact on surfing conditions would be negligible.

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