

GRANGER BAY VISION

Wave and Hydrodynamic Modelling Study

REV.D

07 July 2026



V&A WATERFRONT
Cape Town, South Africa



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V&A WATERFRONT
Cape Town, South Africa



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1. INTRODUCTION

1.1 Project background

The proposed Granger Bay development is located between the Western dolos revetment and the Granger Bay Small Craft Harbour, also known as the Water Club Marina, as shown in Figure 1-1.

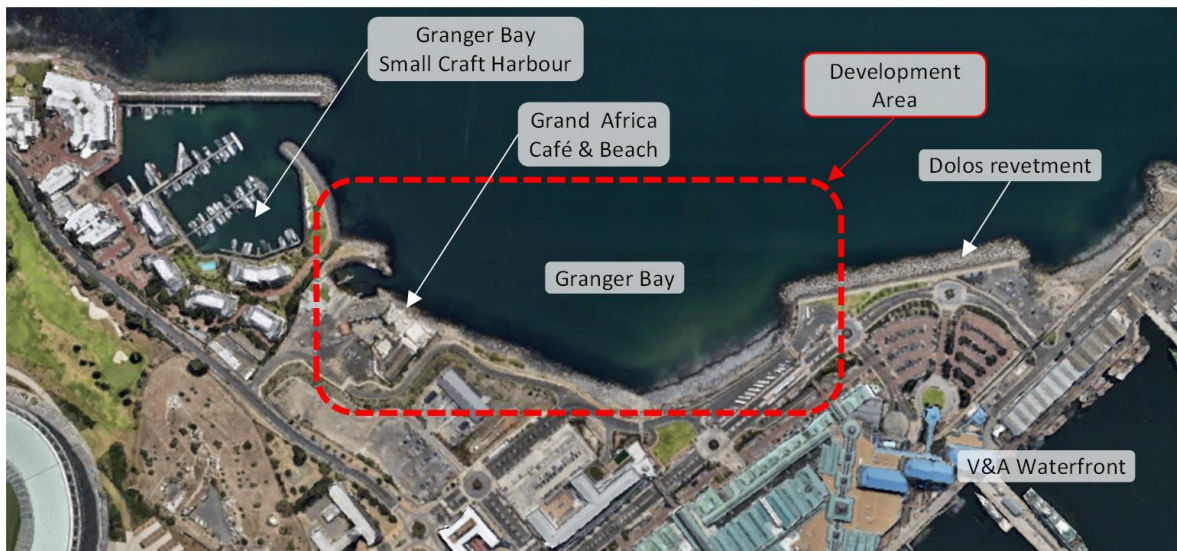


Figure 1-1: Site Location.

PRDW have been appointed by the V&A Waterfront to undertake a numerical modelling study of the proposed development to determine the changes that the proposed development will have on the marine environment.

1.2 Objectives

The objectives of this study are to estimate the changes that the proposed development will have on the following physical parameters:

- Current speed and direction;
- Flushing of water and seawater temperatures within the new basin;
- Wave heights and wave-induced currents;
- Accumulation of mud on the seabed (the existing seabed in Granger Bay is generally sandy because during large wave events any mud that is deposited during calm periods is resuspended, but it is possible that the new development will reduce the waves sufficiently to reduce the resuspension of



mud and thus increase the percentage of mud on the seabed, with associated changes in marine ecology).

- Longshore sediment transport was not considered to be impacted due to the reasons discussed in Section 2.

These results will provide inputs to the marine ecology study.

1.3 Layouts

Two layouts will be assessed:

1. The existing situation as shown in Figure 1-2, referred to as the baseline layout.
2. The new “cascade” layout of the development as shown in Figure 1-2, referred to in this report as the development layout. The cascade layout consists of the construction of two new breakwaters extending off the Oceana breakwater and the existing dolos revetment, reclaiming land and constructing a new revetment between the existing dolos revetment and the breakwater. The crest level of the proposed development is +5.5 m MSL. A 3D impression of the cascade layout can be seen in Figure 1-3.

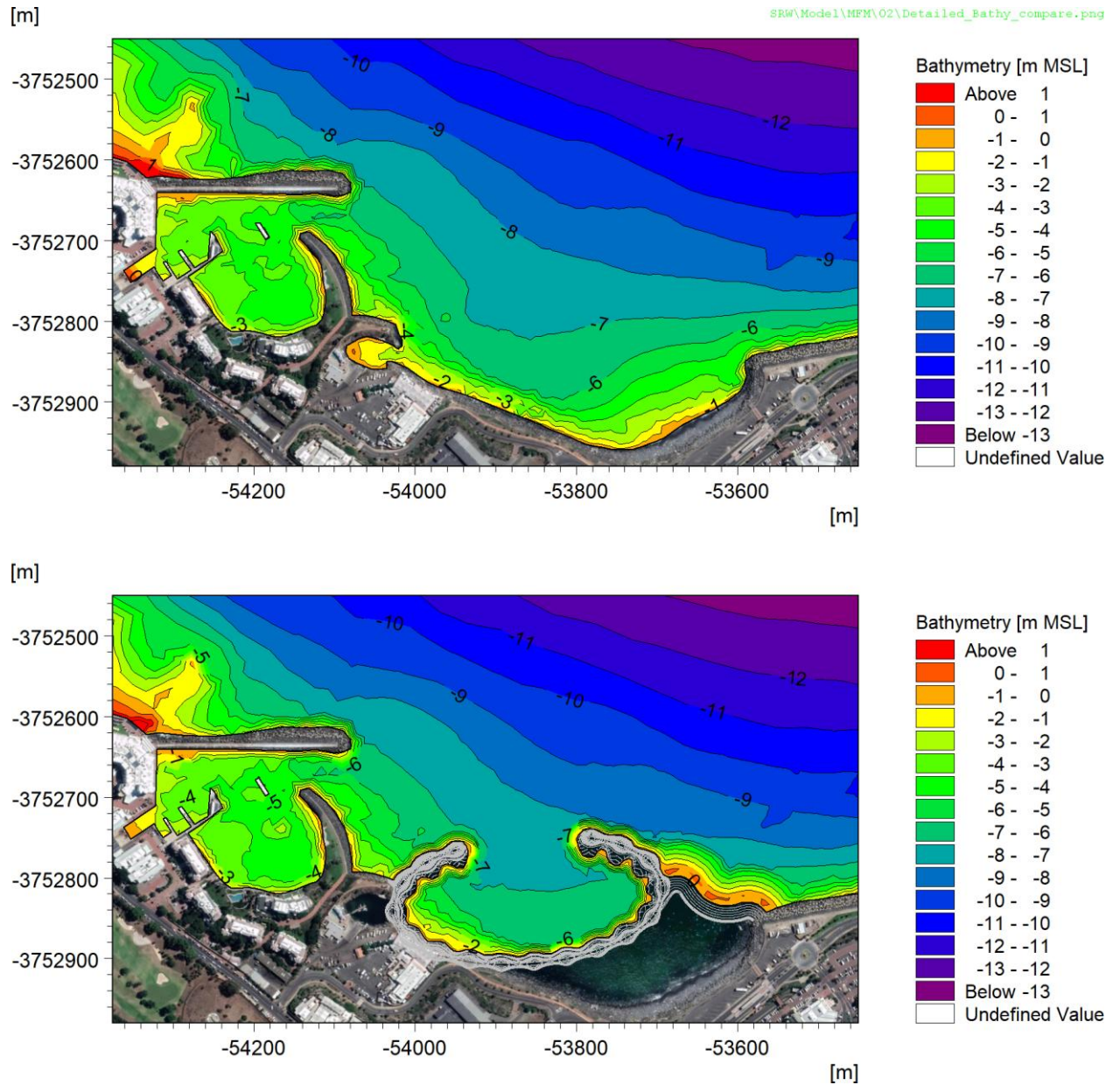


Figure 1-2: Baseline layout (top) and the new “cascade” development layout (bottom).

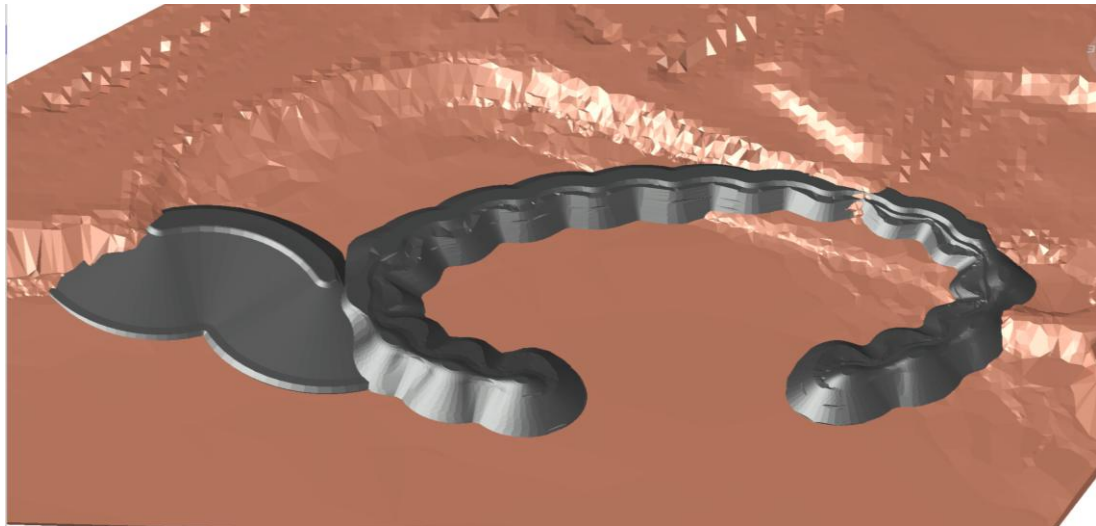


Figure 1-3: Concept of the “cascade” development layout.

1.4 Methodology

A 3D hydrodynamic numerical model will be used to simulate the currents, water levels, seawater temperature and salinity including forcing due to wind, tides, upwelling and regional currents. The model will be run for 6 weeks in summer/autumn and 6 weeks in winter/spring. The model has previously been calibrated against measurements. The outputs will be the following for both layouts for both summer/autumn and winter/spring:

- The maximum depth-averaged current speed for each layout.
- The residual depth-averaged current speed for each layout (this is the vector averaged current and indicates the flushing of the basin).
- The maximum surface seawater temperature for each layout.

A wave refraction numerical model will be used to simulate the wave transformation from deepwater to a depth of 20 m offshore of Granger Bay for one year. A 3D phase-resolving wave numerical model will then be used to simulate the combined short and long waves for each layout for a number of wave conditions, i.e. the 1-month return period storm in summer, 1-month return period storm in winter and 1-year return period storm. The outputs will be the following:

- Wave height for each layout for each wave condition.
- Combined short and long wave induced bed shear stress for each layout. These shear stresses will be compared to critical shear stresses for mud resuspension, which will allow the spatial and temporal extent of possible mud deposition inside the development to be estimated.

In addition to the above, results from wave modelling of extreme conditions (475-year return period) conducted for the design of the marine infrastructure as described in (PRDW, 2025) are further analysed and presented in this Section 4.3 to indicate potential changes in wave heights and wave-induced currents during extreme wave storms.

1.5 Limitations

The model outputs will need to be interpreted by the marine ecologists to assess the impact on the marine ecology. Should the marine ecologists assess the impacts due to any of these parameters to be significant, then additional modelling may be required. This may include testing mitigation options such as permeable or slotted breakwaters to improve flushing, redirecting stormwater discharges to outside the development (if



applicable), more detailed assessment of mud accumulation, etc. This additional modelling is excluded from the current scope.

2. LONGSHORE SEDIMENT TRANSPORT

The transport of fine to coarse-grained sand takes place under strong currents or under the combined actions of currents and waves which contribute by stirring up bed sediments. Such sediment movement typically occurs during large wave conditions which also generate currents.

Figure 2-1 shows offshore and inshore wave scatter diagrams for hindcast storms covering more than 40 years of data. From the spread of wave directions in comparison with the coastline orientation, it is clear that if there were any wave-driven longshore transport its general direction would be towards the east. Due to the sand-starved, rocky nature of the Mouille Point and Sea Point coastlines, there is a very limited supply of sand for eastward transport, suggesting that such transport would be very low.

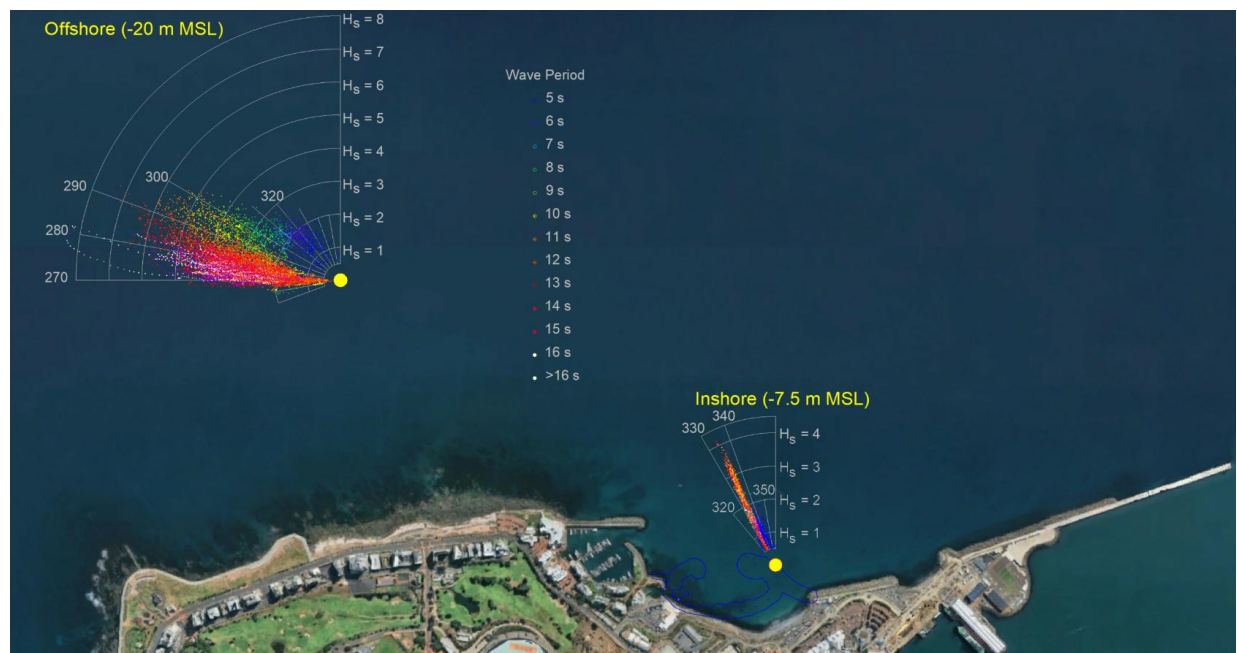


Figure 2-1: Offshore and inshore wave scatter diagrams.

Notwithstanding the lack of updrift supply to Granger Bay, there is also a lack of evidence of supply from Granger Bay towards the east. If significant eastward transport did take place, sand would deposit in the entrance channel of the Port of Cape Town (see Figure 2-2) which would require maintenance dredging to ensure sufficient channel depth for navigation. No such dredging has taken place along the approach channel for the Port of Cape Town, as confirmed by the previous harbour master (S Bentley, pers comm). Water depths increase towards the east and reach levels deeper than -17 m CD at the eastern extent of the main breakwater. The advertised entrance channel depth is -15.8 m CD.

Considering the above, no significant longshore sediment transport currently exists at the site and therefore the development will not change the longshore transport. The potential impact of the development on the accumulation of mud on the seabed is discussed in Section 4.4.

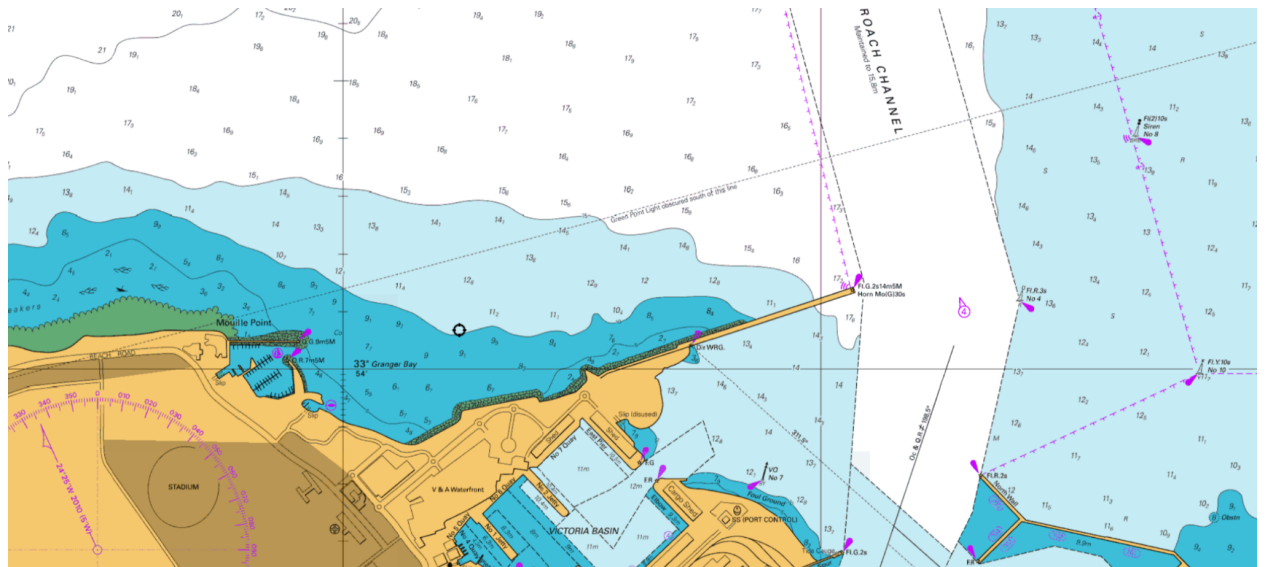


Figure 2-2: Naval chart of Granger Bay and Port of Cape Town approach channel.



3. MODELLING

This section contains technical details of the setup and calibration of the numerical models.

3.1 Hydrodynamic model

3.1.1 Description

The three-dimensional MIKE 3 Flow Flexible Mesh Model was used to model the currents and seawater temperatures. The application of the model is described in the User Manual (DHI, 2023a), while full details of the physical processes being simulated and the numerical solution techniques are described in the Scientific Documentation (DHI, 2023b).

The model is based on the numerical solution of the three-dimensional incompressible Reynolds averaged Navier-Stokes equations invoking the assumptions of Boussinesq and of hydrostatic pressure. The model consists of the continuity, momentum, temperature, salinity and density equations and is closed by a k- ϵ vertical turbulence closure scheme. Horizontal eddy viscosity is modelled with the Smagorinsky formulation.

The time integration of the shallow water equations and the transport equations is performed using a semi-implicit scheme, where the horizontal terms are treated explicitly and the vertical terms are treated implicitly. In the vertical direction a structured mesh, based on a sigma-coordinate transformation is used, while the geometrical flexibility of the unstructured flexible mesh comprising triangles or quadrangles is utilised in the horizontal plane.

The model includes the following physical phenomena:

- Currents due to tides;
- Currents due to wind stress on the water surface;
- Currents due to density gradients caused by seawater temperature and salinity;
- Coriolis forcing;
- Bottom friction;
- Flooding and drying;
- Heat exchange.

3.1.2 Setup

3.1.2.1 Mesh and bathymetry

The WG19 horizontal coordinate system was used for this study. All spatial plots include x and y axes showing the x and y coordinates in metres in the WG19 system. True north is always pointing upwards. The vertical datum adopted in this study is Mean Sea Level (MSL).

The model mesh comprises triangles with a resolution varying from about 3 km at the offshore boundary to 10 m in the vicinity of the site. The vertical mesh is comprised of twelve sigma layers with equal layer thicknesses.

The model bathymetry has been obtained from the following sources:

- CMAP electronic hydrographic charts (DHI, 2023c),
- Available bathymetric surveys in Table Bay,
- Granger Bay hydrographic survey (Underwater Surveys, 2022).

Figure 3-1 and Figure 3-2 present the model bathymetry and mesh, respectively, for the baseline layout while Figure 3-3 and Figure 3-4 present detail around the site for baseline and development layout.

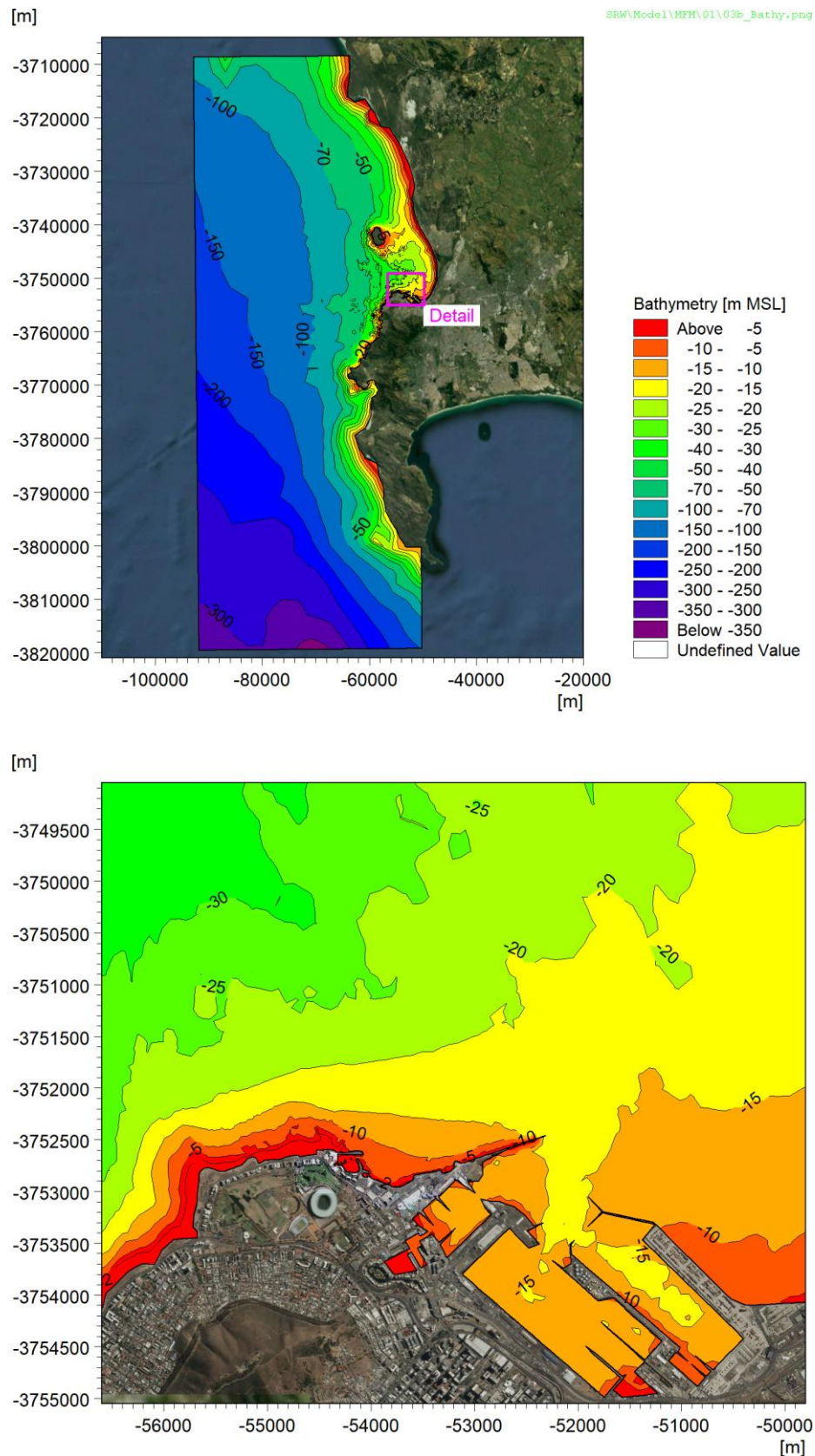


Figure 3-1: Model bathymetry for the baseline layout.

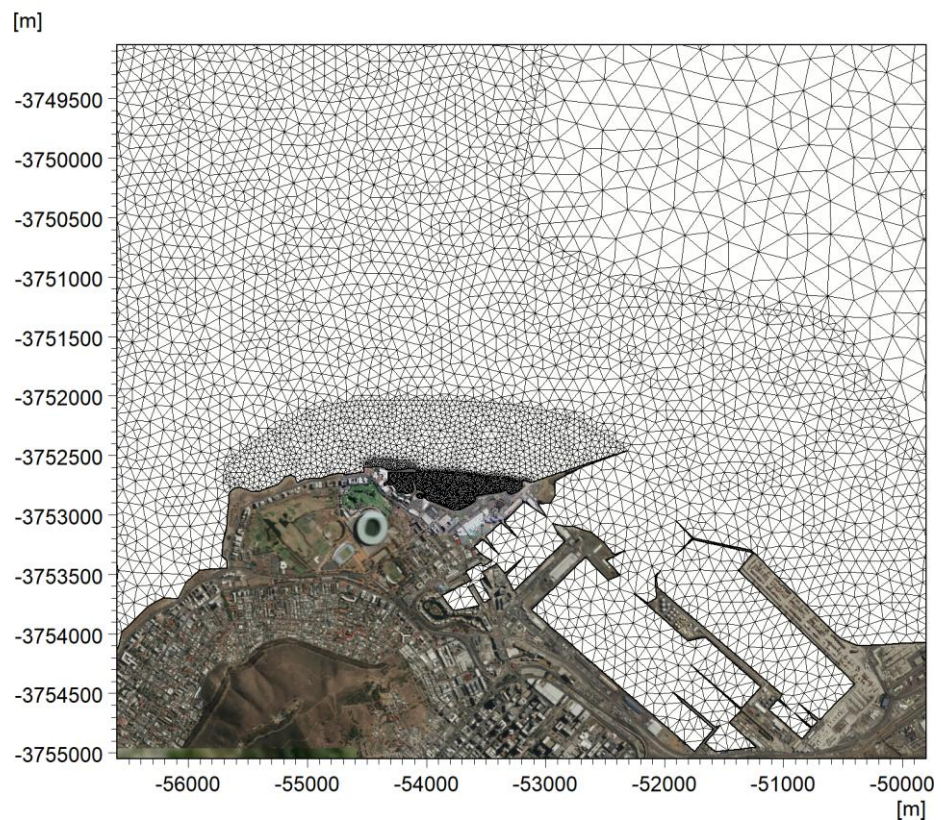
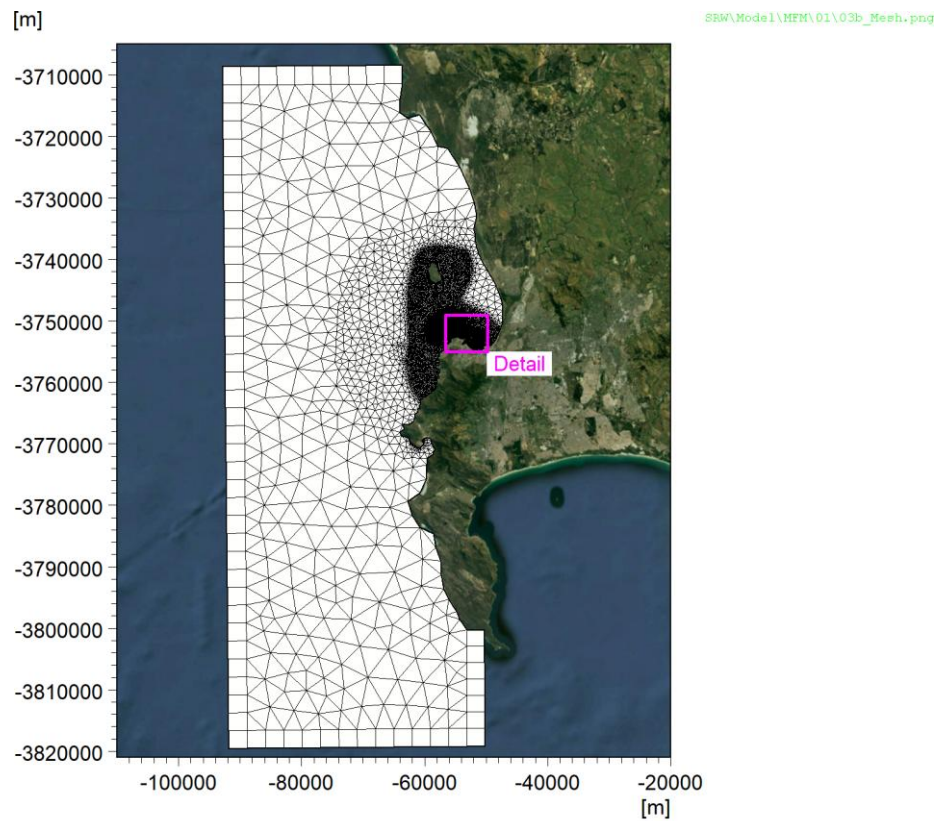


Figure 3-2: Model mesh for the baseline layout.

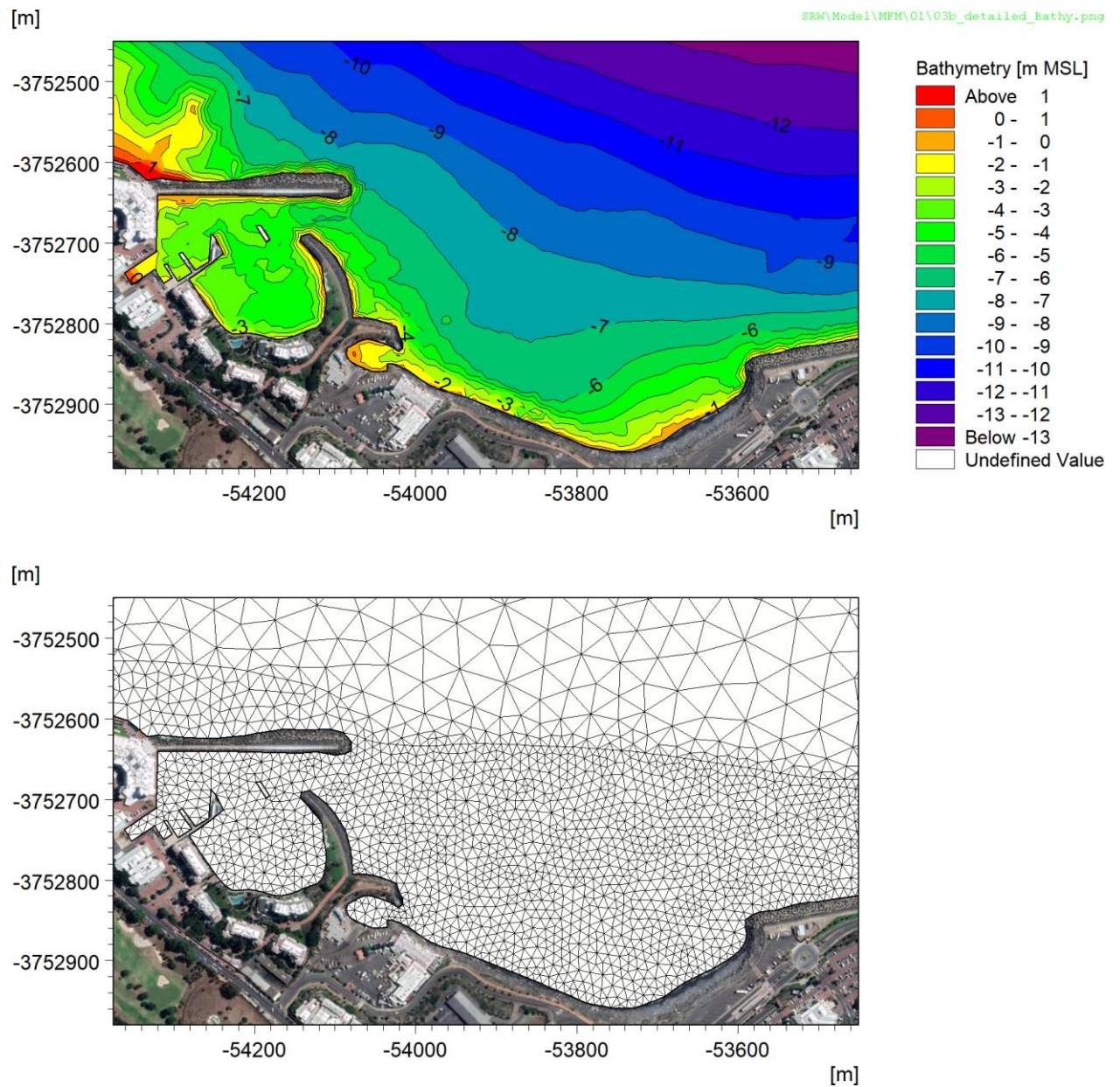


Figure 3-3: Detailed bathymetry and mesh for the baseline layout.

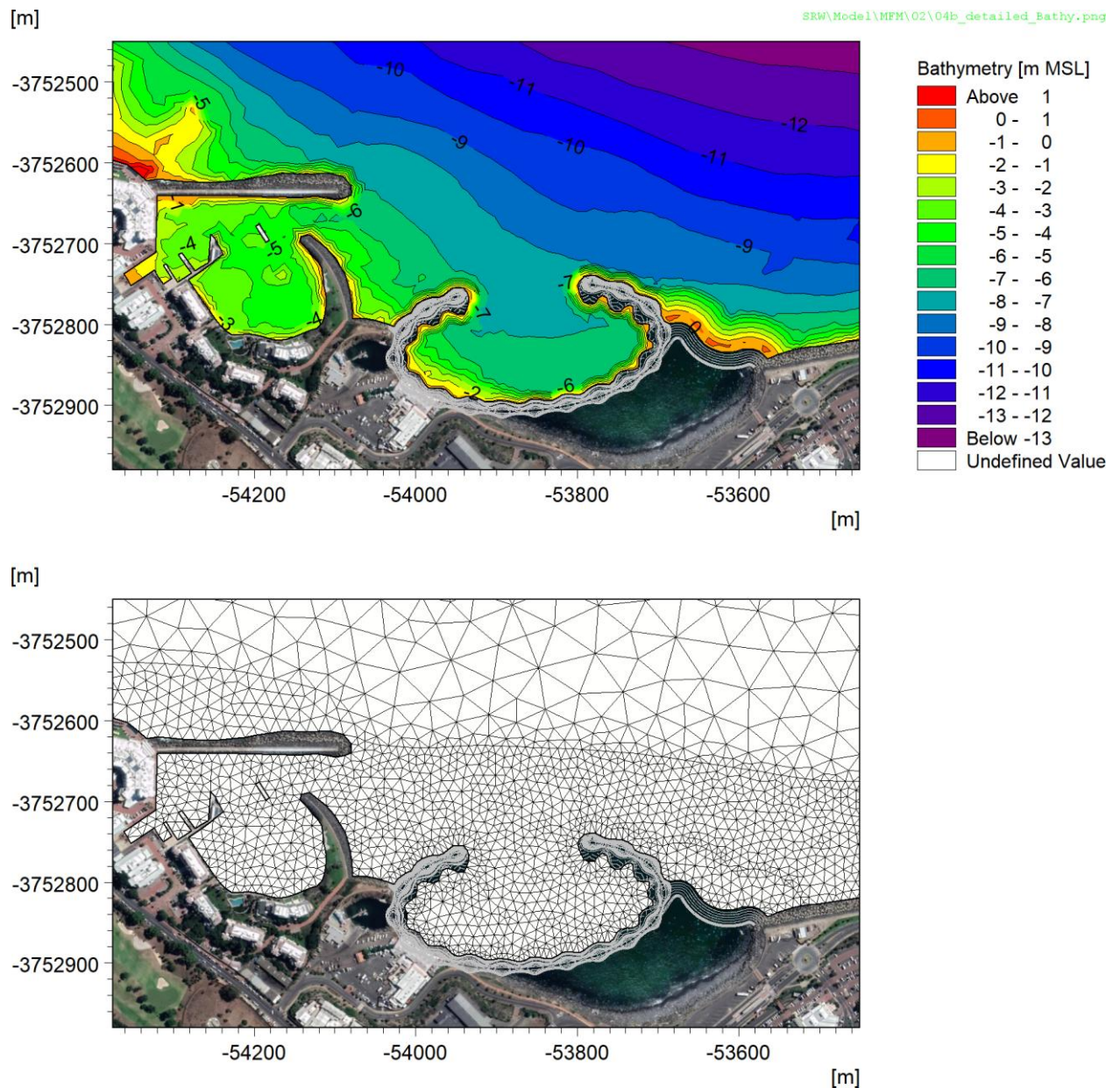


Figure 3-4: Detailed bathymetry and mesh for the development layout.

3.1.2.2 Wind

Wind is an important forcing mechanism for currents, vertical mixing and upwelling. The wind fields near the outfalls are influenced by the surrounding mountains and thus require a high spatial resolution. The Climate System Analysis Group at the University of Cape Town has kindly provided wind data from the Weather Research and Forecasting (WRF) model with a spatial resolution of 4 km and a temporal resolution of 1 hour.

Figure 3-5 shows two snapshots in time illustrating the high spatial variability in the 4 km resolution modelled winds around Cape Town. Figure 3-6 shows a time series comparison between the modelled winds and measured winds at three locations around Cape Town (Cape Town International Airport, Cape Point and Koeberg) for the winter/spring and summer/autumn modelling periods. These results indicate that the WRF modelled winds compare well to the measured winds at three different locations across Cape Town and will thus provide reliable forcing to the hydrodynamic model.

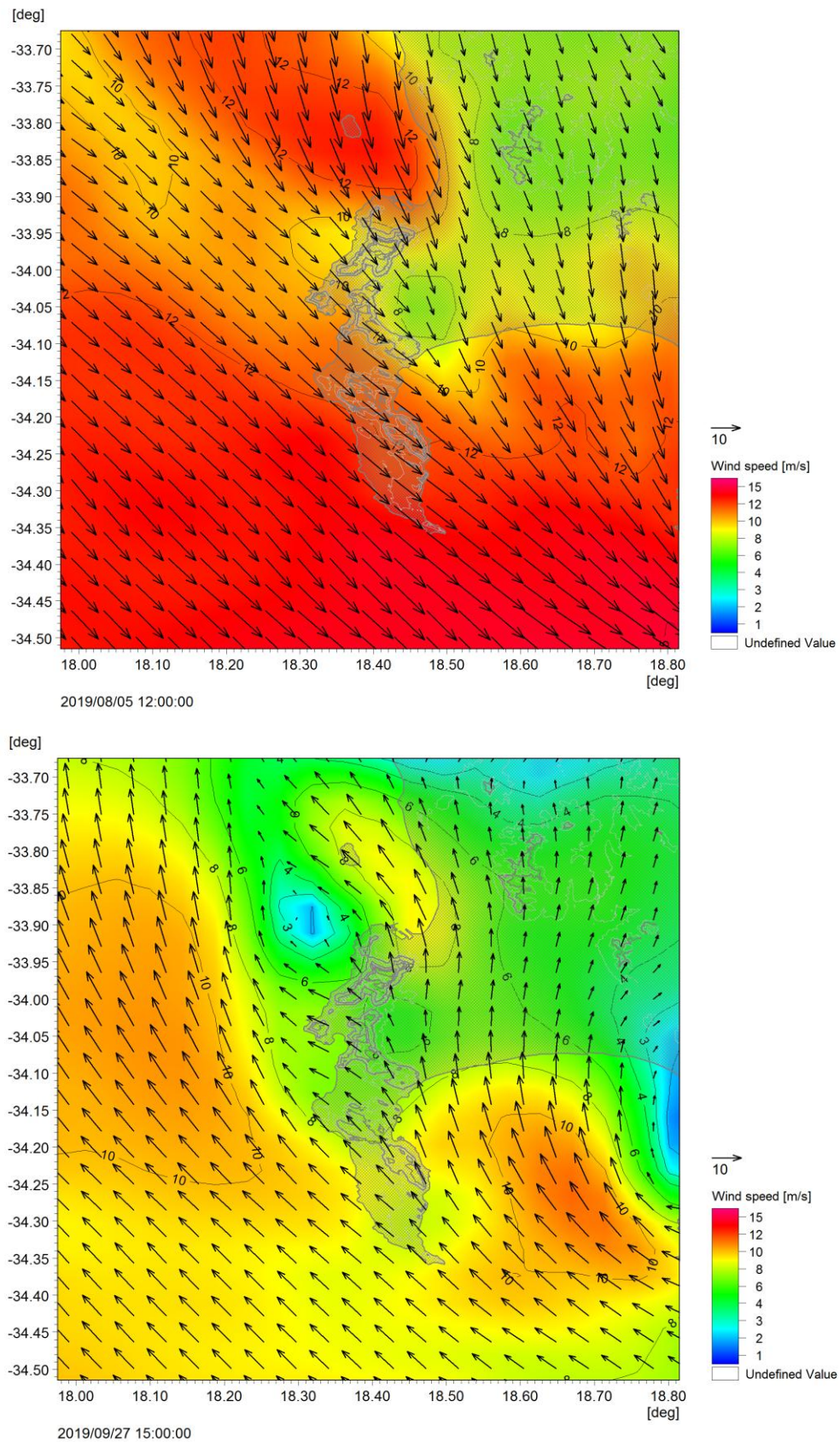


Figure 3-5: Snapshots illustrating the high spatial variability in the 4 km resolution modelled winds around Cape Town for two wind directions (grey lines show topography at 100 m intervals).

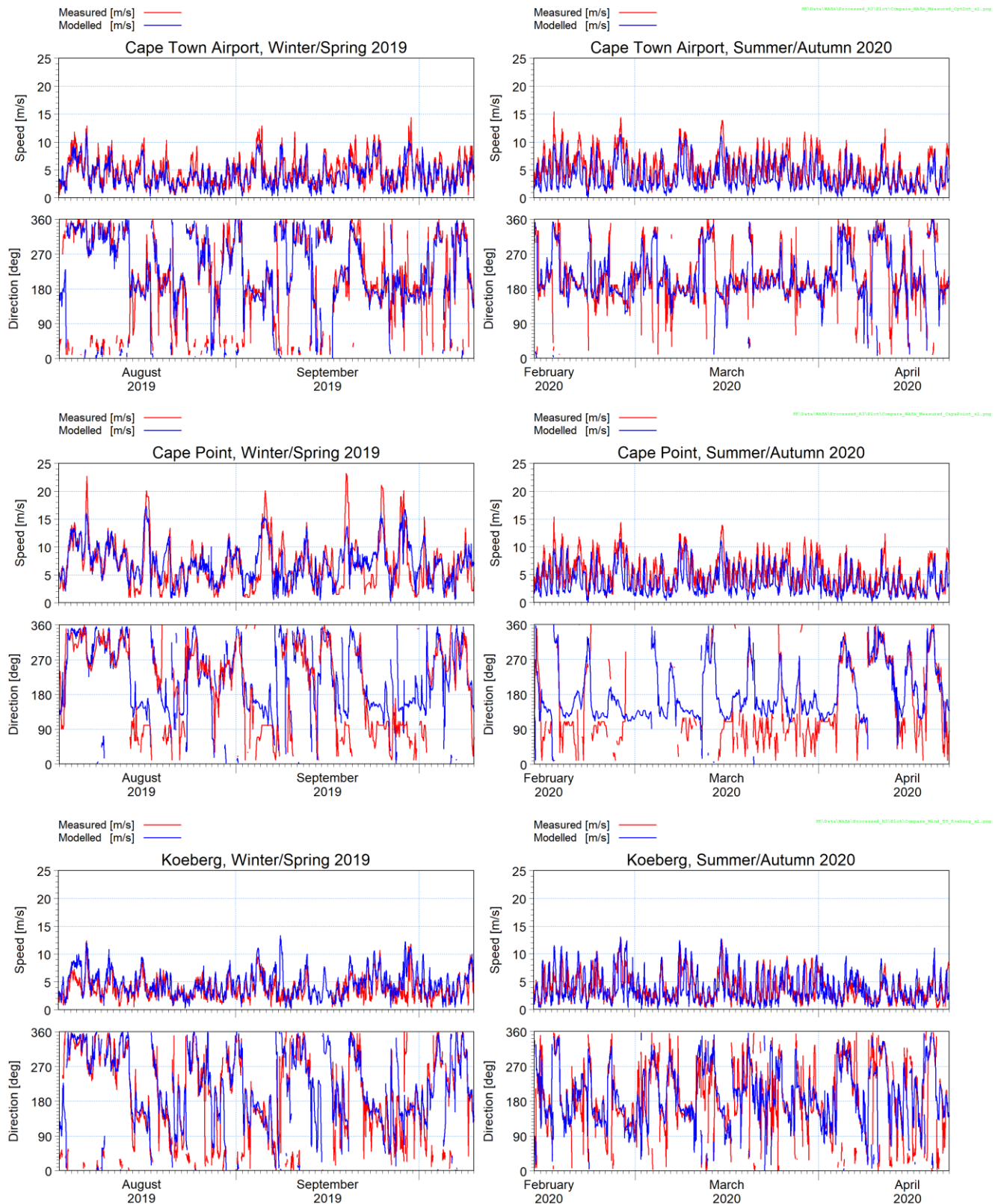


Figure 3-6: Time series of modelled compared to measured winds at Cape Town International, Cape Point and Koeberg.

3.1.2.3 Boundary conditions

The model boundary conditions were extracted from global hindcast models as described in Table 3-1:

Table 3-1: Global hindcast models used for the model boundary conditions.

Model	Parameter	Spatial resolution [°]	Temporal resolution [hrs]
HYbrid Coordinate Ocean Model (HYCOM) global ocean circulation model (HYCOM, 2020).	Non-tidal water levels	0.08	3
	Non-tidal ocean currents		
	Seawater temperature		
	Seawater salinity		
DTU10 (Technical University of Denmark) global tidal model (DTU, 2010).	Predicted tides	0.125	Predicted from constituents.
	Predicted tidal currents	0.25	
European Centre for Medium-Range Weather Forecasts (EMCWF) ERA5 global atmospheric model (ECMWF, 2020).	Air temperature	0.25	1
	Humidity		
	Solar radiation		

3.1.2.4 Model settings

The vertical eddy viscosity was computed using the $k-\epsilon$ vertical turbulence closure scheme, while the vertical eddy dispersion was set to 0.1 times the vertical eddy viscosity. This scaling factor was applied to compensate for additional vertical mixing caused by the use of only 12 vertical layers and the associated smoothing of the vertical density gradients.

The wind friction coefficient and seabed roughness were tuned to physically realistic values which yielded the best agreement between modelled and measured currents at Green Point.

3.1.3 Calibration

The model was calibrated to measured water level, current and seawater temperatures, as described below.

3.1.3.1 Water levels

The predicted tidal levels at the Port of Cape Town were extracted from the MIKE by DHI CMAP Electronic Charts Database (DHI, 2023c). Figure 3-7 presents a time series comparison of modelled water levels to predicted tides at the Port of Cape Town. The comparison shows an excellent comparison in both tidal phase and amplitude.

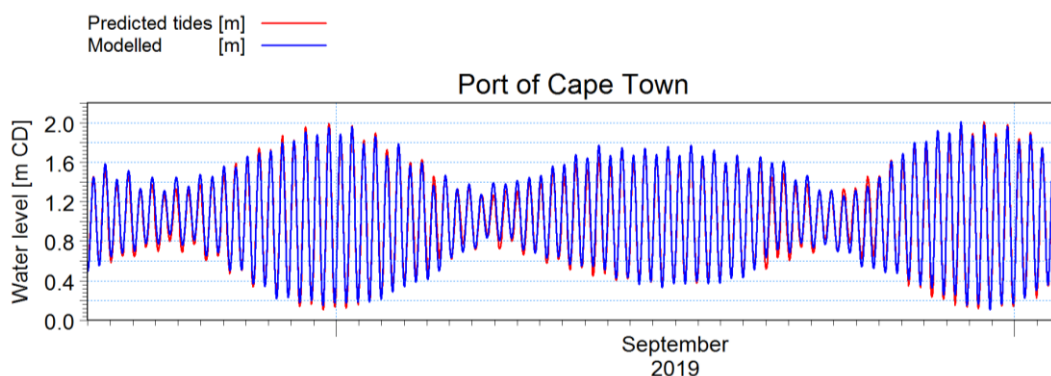


Figure 3-7: Time series comparison of modelled water levels to predicted tides at the Port of Cape Town.

3.1.3.2 Currents

Figure 3-8 presents a time series comparison of the modelled and measured currents near the end of the Green Point outfall. Note that current direction is by convention the direction towards which the current is flowing. Overall, the model reproduces the main features of both the surface and seabed currents and is considered acceptably accurate for the purposes of this study.

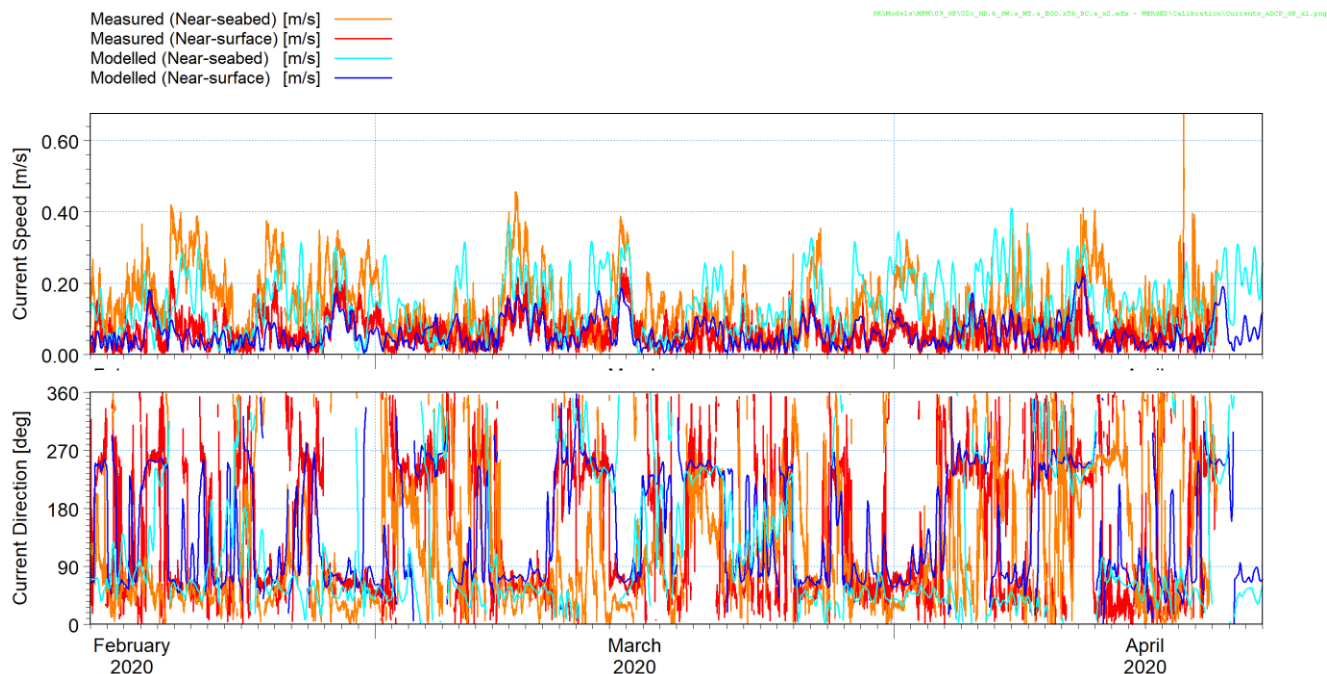


Figure 3-8: Time series comparison of measured and modelled current speed and direction near the Green Point outfall.

3.1.3.3 Seawater temperature

The modelled seawater temperatures were compared to measurements near the end of the Green Point outfall as shown in Figure 3-9. The measured time series shows that the seawater temperatures are complex and vary over a range of time-scales due to upwelling, solar radiation and other dynamic processes. The model is able to predict the mean trend in seabed seawater temperatures including some of the discrete dynamic events. The modelled sub-surface temperatures tend to be overpredicted while the surface temperatures tend to be underpredicted. Despite these limitations the model is considered adequately calibrated for seawater temperature for the purposes of this study.

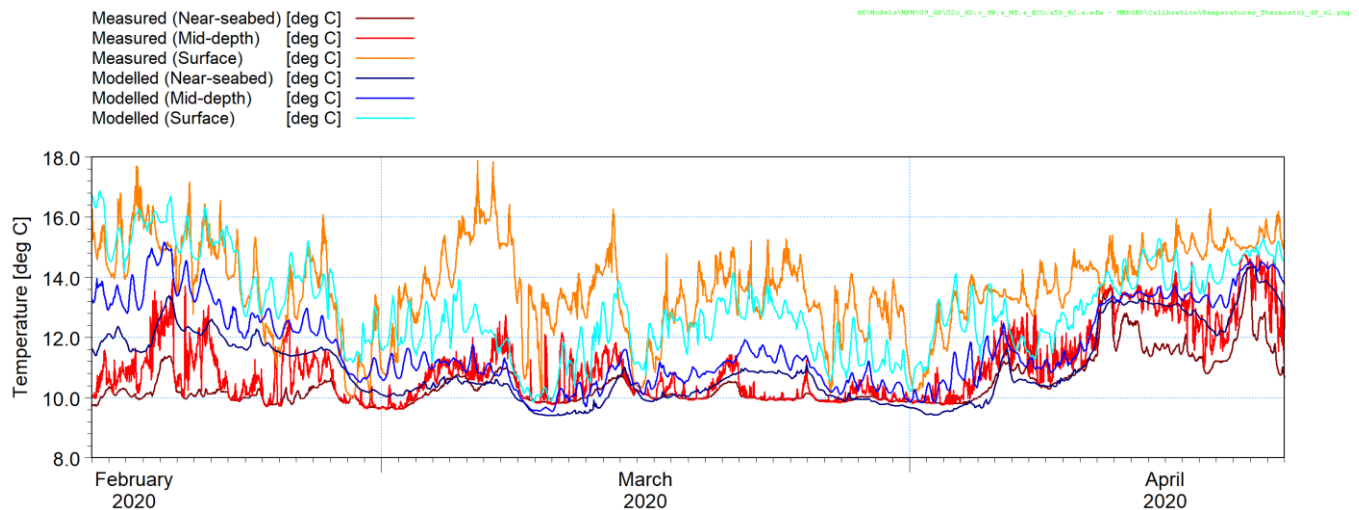


Figure 3-9: Summer/autumn: Time series of measured and modelled seawater temperatures.

3.1.4 Cases modelled

Two cases of six weeks each were modelled for the hydrodynamics. These periods were selected based on the availability of calibration data.

- A summer/autumn period from 14/02/2020 to 27/03/2020.
- A winter/spring period from 01/08/2019 to 12/09/2019.

3.2 Phase-resolving wave model

3.2.1 Description

The MIKE 3 Wave Model FM (M3WFM) was used to simulate the wave heights and wave-induced bed shear stresses.

M3WFM is a phase-resolving wave model based on the 3D Navier-Stokes equations and with the free surface described by a height function. The numerical techniques applied are based on an unstructured (flexible) mesh approach. The application of M3WFM is described in the User Manual (DHI, 2023d), while full details of the physical processes being simulated and the numerical solution techniques are described in the Scientific Documentation (DHI, 2023e). The model validation by DHI is described in the Validation Report (DHI, 2023f). PRDW have also validated the model for wave run-up and overtopping using measured data.

The M3WFM model includes the following processes:

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

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 $k-\varepsilon$ 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 or porous structures. The numerical dissipation accounts for the dissipation in the breaking waves.

3.2.2 Setup

3.2.2.1 Bathymetry and mesh

The model bathymetry has been obtained from the following sources:

- CMAP electronic hydrographic charts (DHI, 2023c),
- Available bathymetric surveys in Table Bay,
- Granger Bay hydrographic survey (Underwater Surveys, 2022).

The flexible mesh comprises triangles with a resolution varying between approximately 6.5 m offshore and approximately 3 m around the study area in Granger Bay.

The model bathymetry and mesh are presented in Figure 3-10 and Figure 3-11 for the whole domain for the baseline layout. Figure 3-12 to Figure 3-15 present detailed bathymetries and meshes for the study area for the baseline layout and the new development layout.

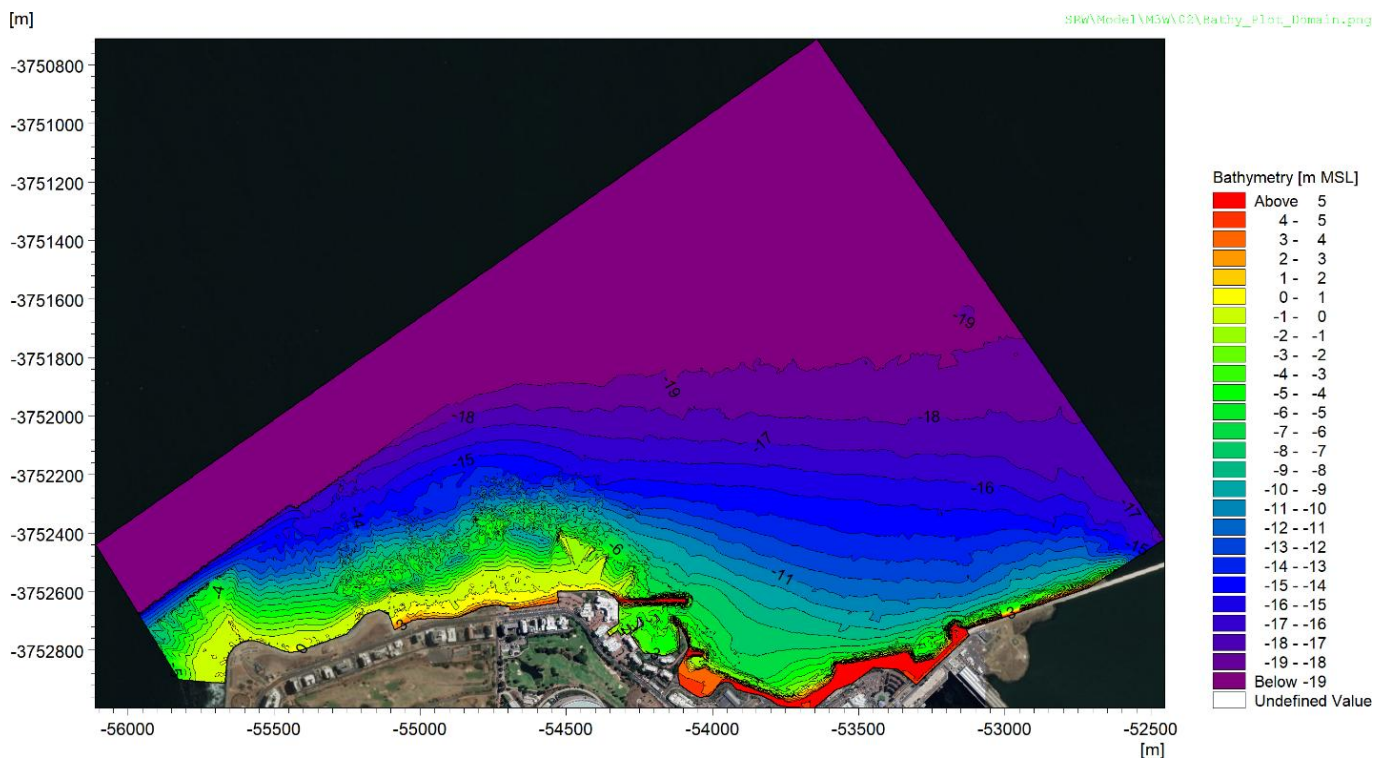
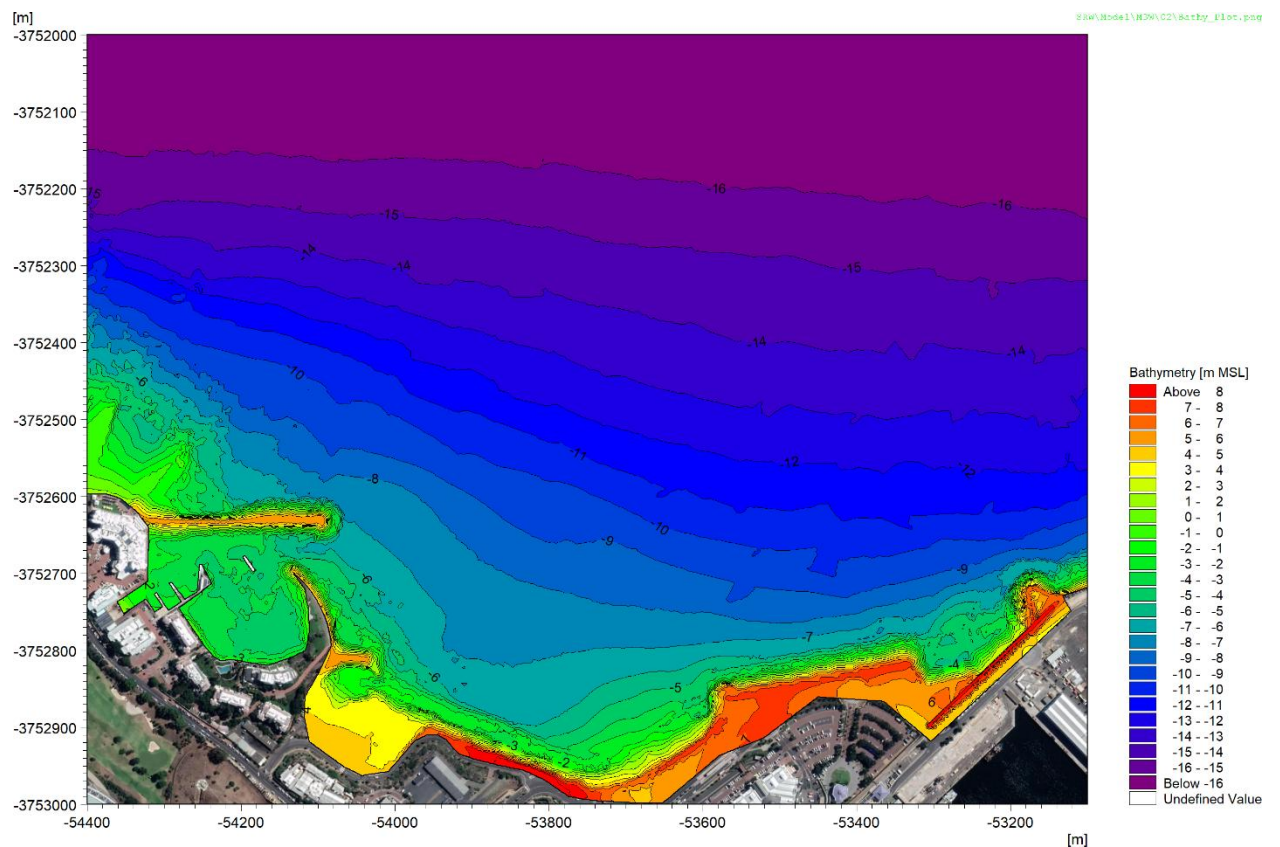
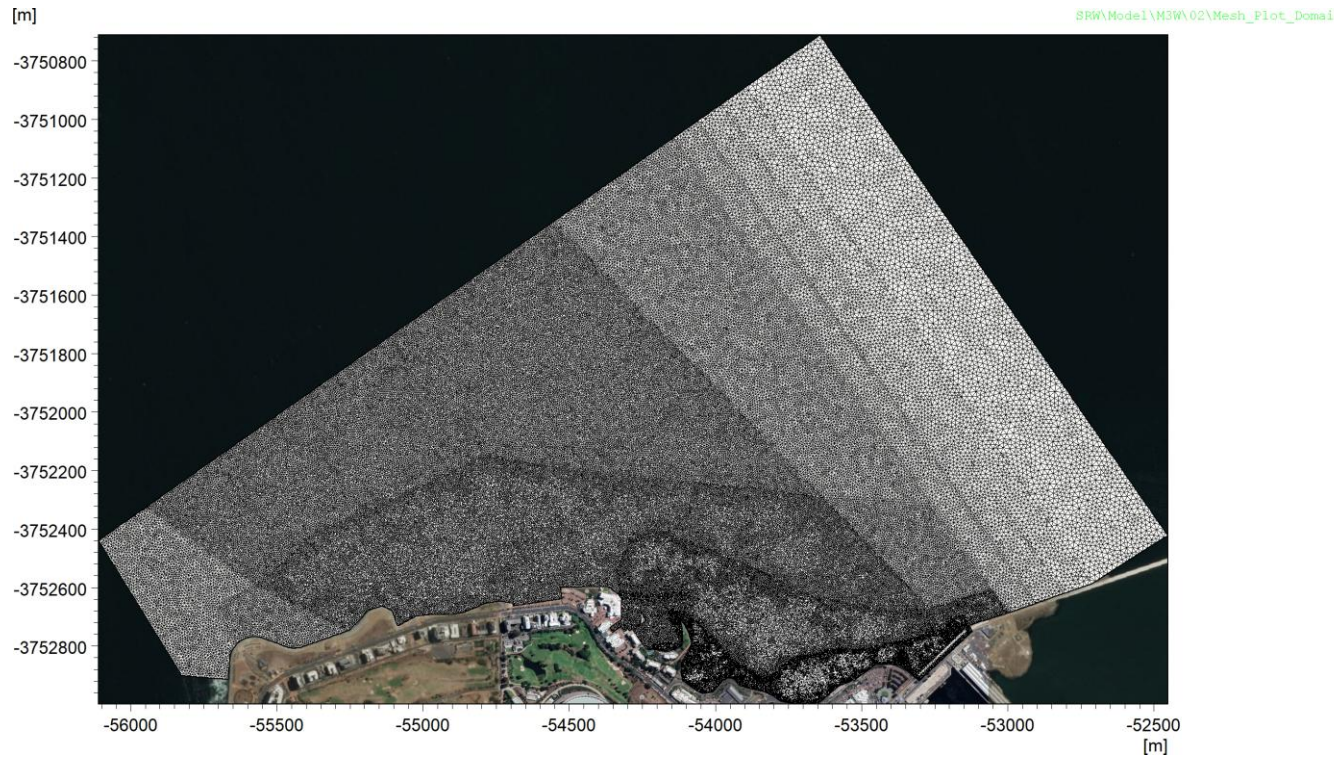
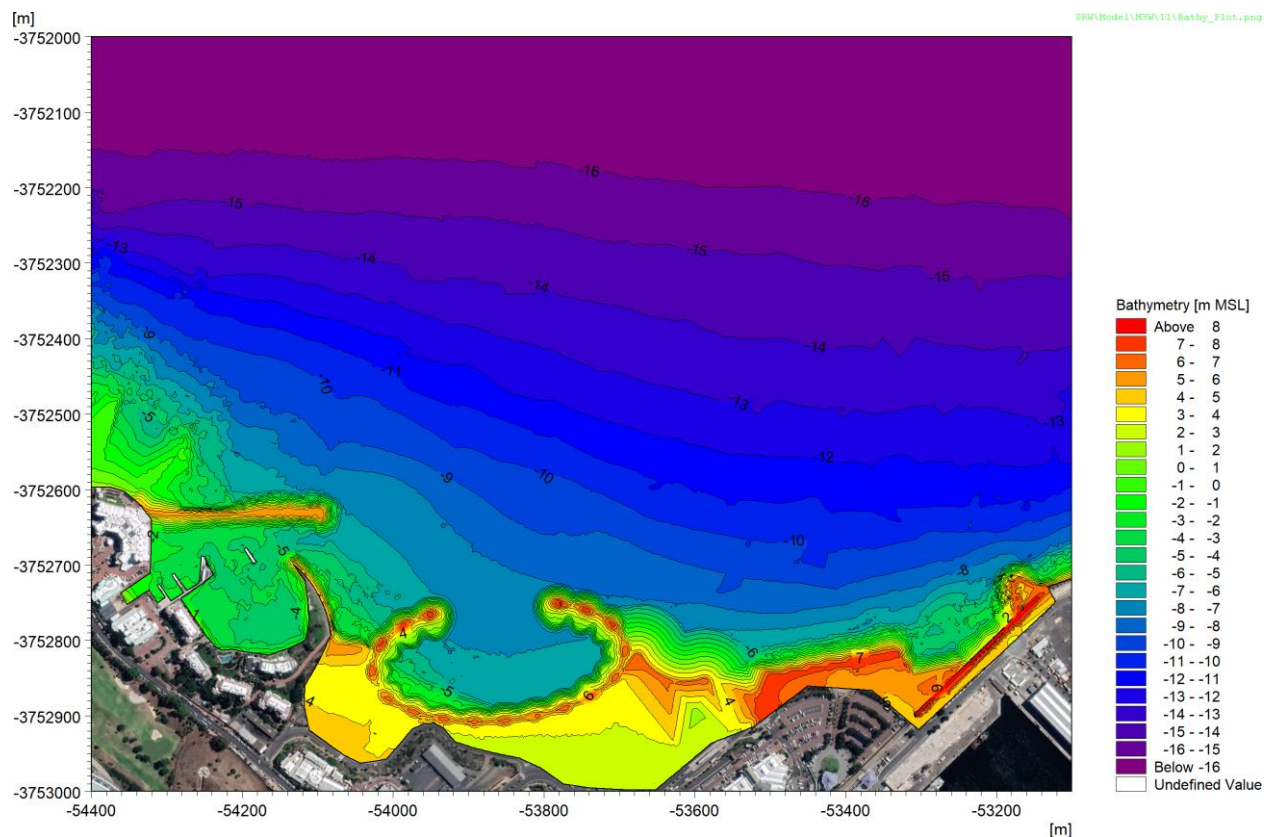
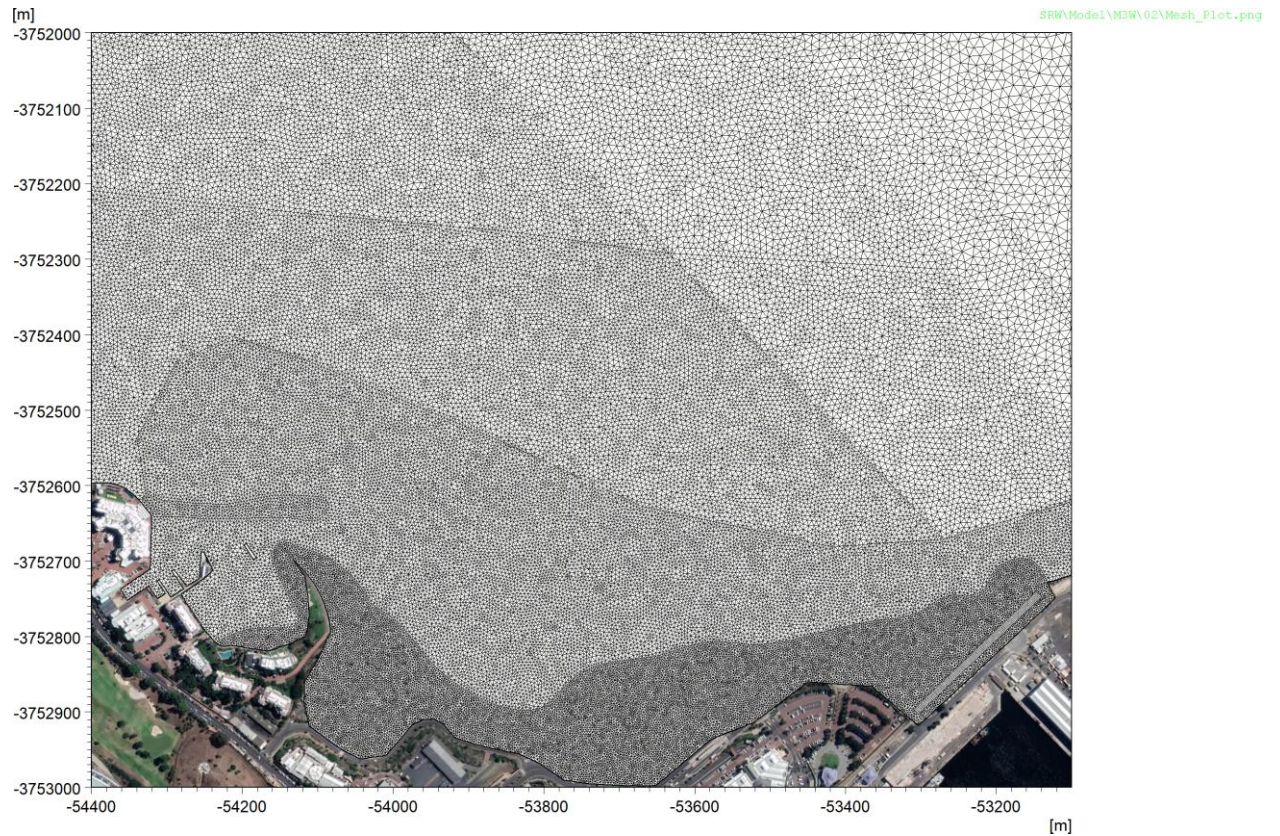


Figure 3-10: Overview of bathymetry for whole domain (baseline layout).





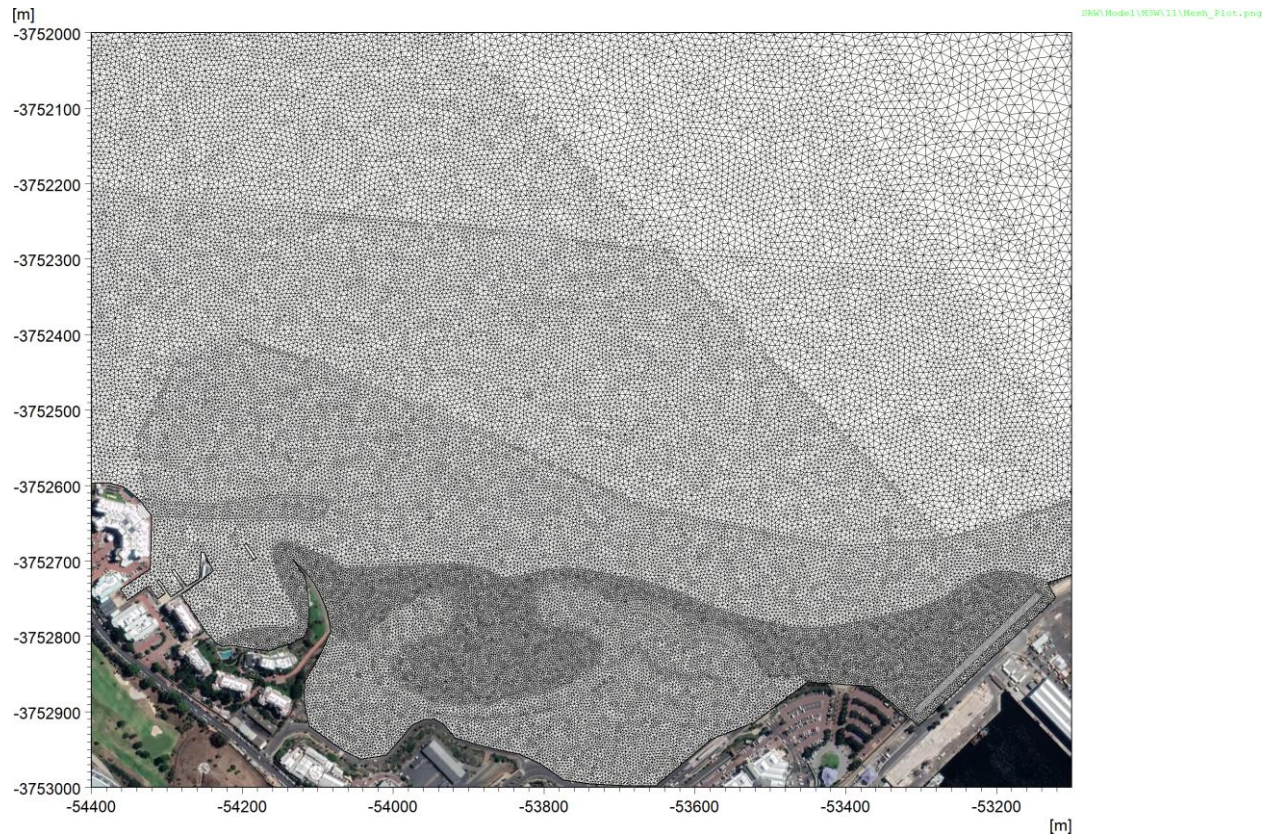


Figure 3-15: Detailed mesh for new development layout.

3.2.2.2 Model inputs

Bottom friction was modelled using the quadratic drag coefficient formulation. The friction coefficient was calibrated to each armouring and structure type in the model (rock slope, dolos slope and rock berm breakwater) by comparing to a detailed, porosity-based flume model. The porosity-based model can simulate the resistance of porous structures and more realistically reproduce wave reflection and run-up, however it is computationally intensive to do this at a full 3D scale. Therefore, flume models were used to obtain a calibrated friction value for each type of porous structure.

Each run consisted of 5 minutes of model spin-up and a further 60 minutes of the modelled sea state. The spin-up was ignored in all the results. Waves in the model were generated along the -20 m MSL contour on the north-western boundary.

3.2.3 Calibration

In the absence of measured water level or wave data in Granger Bay, a qualitative model calibration was undertaken by simulating a specific storm event and comparing the model results to photographs of wave run-up and overtopping taken at the corresponding date and time.

The storm of 13 July 2020 was used for the model calibration. A 1-hour sea state at 15:00 (UTC+2) was modelled using the offshore wave conditions at -20 m MSL obtained from the wave refraction model and measured water levels from Cape Town, as described in PRDW (2022). The model input conditions are presented in Table 3-2.

Table 3-2: Model inputs at 15:00 for the 13 July 2020 storm.

H_{m0} [m]	T_p [s]	MWD [deg]	DSD [deg]	Water Level [m MSL]
8.0	16.9	276	16	0.1

Figure 3-16 presents a snapshot of the instantaneous water surface elevations during the 13 July 2020 storm. This can be compared to Figure 3-17 and Figure 3-18 which show photographs of overtopping along the rock revetment and in front of the Grand Africa Café, respectively.

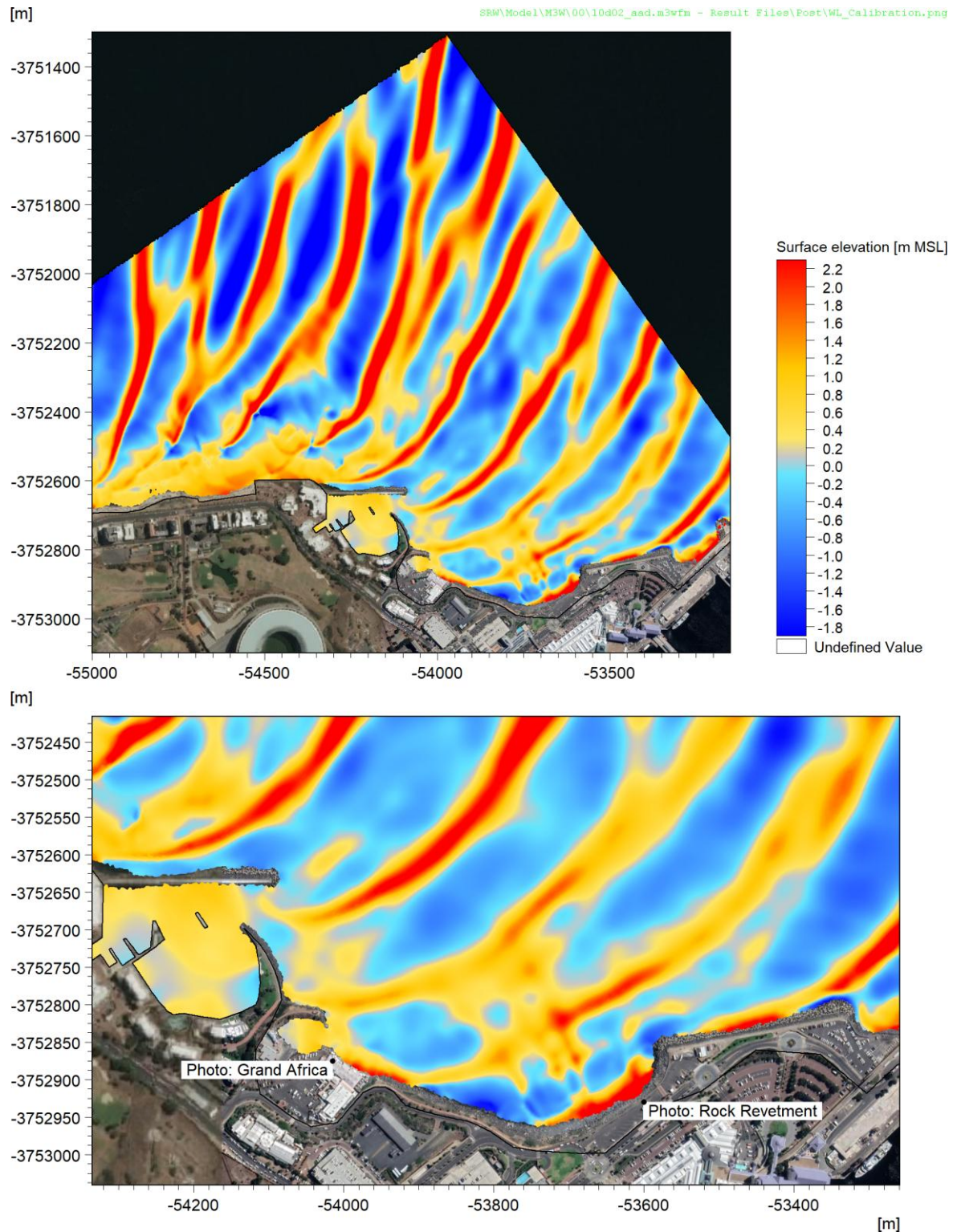


Figure 3-16: Instantaneous modelled surface elevation of an overtopping event during the 13 July 2020 storm.



Figure 3-17: Overtopping of rock revetment at approximately 15:00 on 13 July 2020. Photo credit: Stephen Luger.

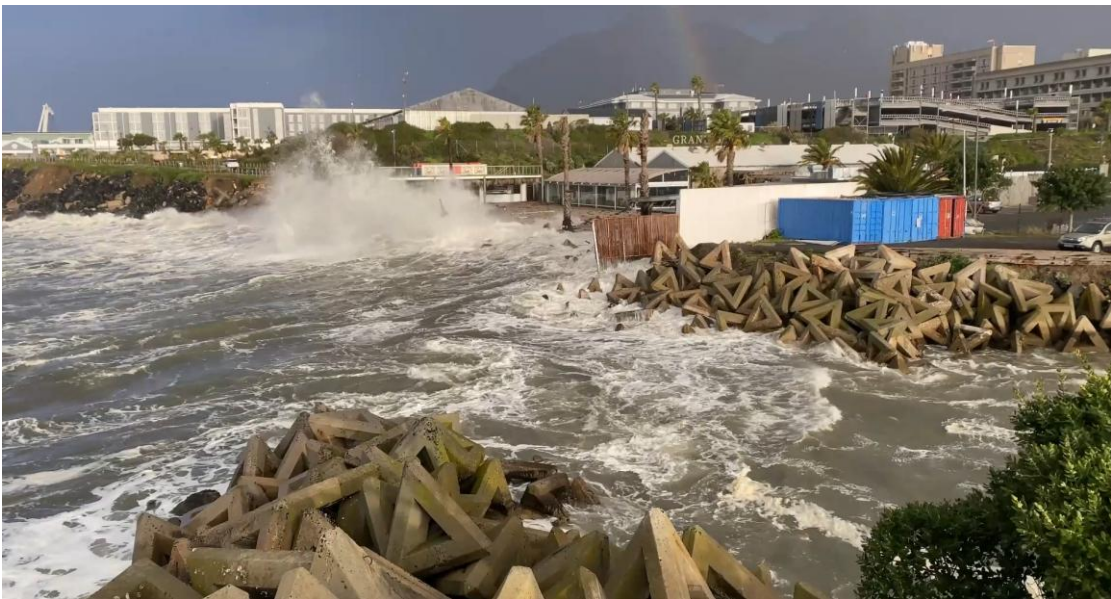


Figure 3-18: Overtopping in front of Grand Africa Café on 13 July 2020 (exact time unknown). Photo credit: Anton Holtzhausen.

The snapshot presented in Figure 3-16 shows similar overtopping to the photos taken during the storm at both locations. Although this is not a quantitative comparison, this suggests that the model is accurately reproducing all the processes and as such can be considered to be sufficiently accurate for the purposes of this study.

3.2.4 Critical bed shear stress

Figure 3-19 shows the percentage of fines (sediment grain size $<63 \mu\text{m}$) obtained from a number of historical sediment sampling campaigns in Table Bay. The fines percentages in shallow areas exposed to waves are low. In deeper water offshore of Robben Island, the analyses shows fines percentages of approximately 10% to 15%. This suggests the temporary deposition of fine particulate matter, i.e. most of the time the bed shear

stresses are sufficiently low for deposition to occur, but resuspension occurs during storm events with large waves. The permanent depo-centres were identified by a percentage fines >20% and include the Koeberg intake basin, the Port of Cape Town and Murray's Harbour (on Robben Island).

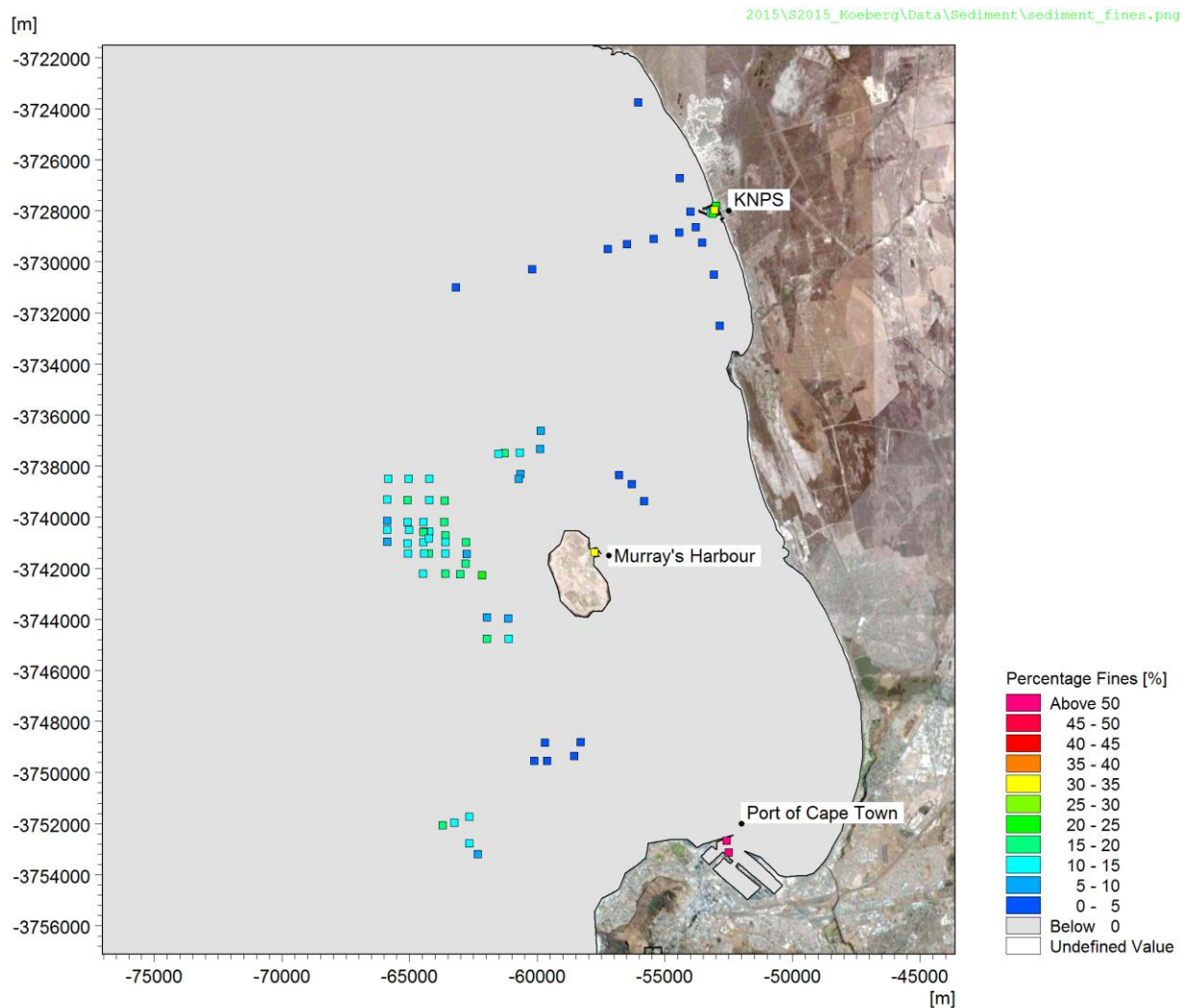


Figure 3-19: Percentage fines obtained from a number of historical sediment sampling campaigns.

Figure 3-20 shows the bed shear stresses modelled by PRDW at each location plotted against the measured percentage fines at the corresponding location. The expected behaviour is that areas with low bed shear stresses will have a higher fines percentage (muddy seabed), since the stresses during storm events are too low to resuspend the fines that are deposited during calm periods. As shown in the figure, this trend is supported. The distinction between permanent depo-centres (fines >20%, muddy) and areas with temporary or little deposition (fines <20%, sandy) is at a bed shear stress of approximately 0.2 N/m^2 .

Based on these results, areas where the bed shear stress exceeds 0.2 N/m^2 during storm events will tend to be sandy, whilst areas where the bed shear stress remains below 0.2 N/m^2 will tend to be muddy.

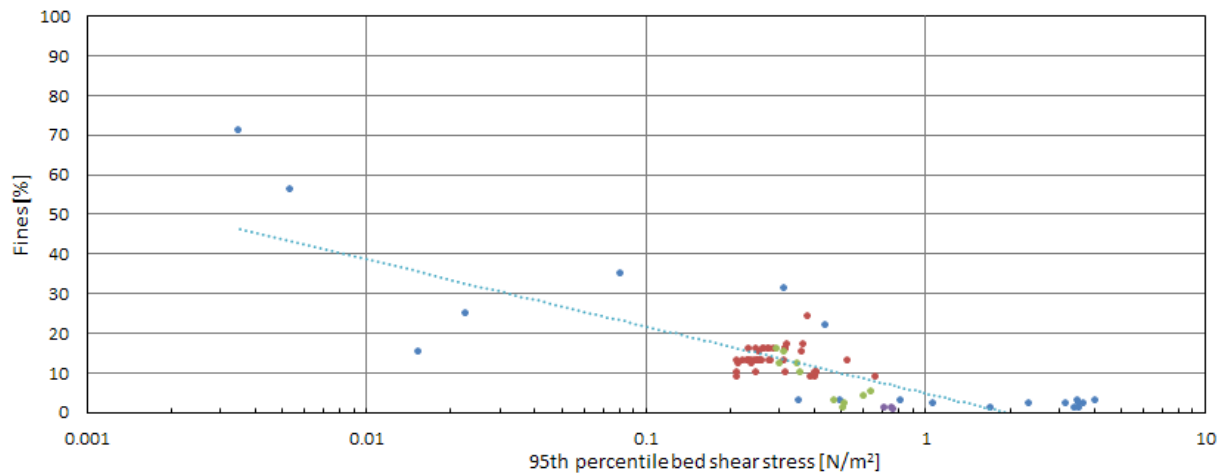


Figure 3-20: Correlation of the percentage fines measured in grab samples of the seabed sediment and the 95th percentile modelled bed shear stress.

In Granger Bay, the hydrodynamic model showed that the current speeds due to wind, tides and ocean currents are low and thus did not contribute significantly to the bed shear stresses, which are primarily driven by waves.

The phase-resolving wave model (as described in Sections 3.2.1 to 3.2.3) was used to calculate the wave-induced bed shear stresses using a bed roughness of 0.001 m and the wave-induced currents at the seabed time-averaged over the modelled sea-state duration of 45 minutes. These bed shear stresses include contributions from both swell waves (period <25 s) and longer period waves caused by bound waves, surf beat and harbour resonance.

Figure 3-21 shows the modelled wave-induced bed shear stresses in Granger Bay during a 1-year return period storm event. Areas where the bed shear stress exceeds 0.2 N/m² during storm events will tend to be sandy and are indicated in shades of blue, whilst areas where the bed shear stress remains below 0.2 N/m² will tend to be muddy and are indicated in shaded of brown. The modelled wave-induced bed shear stresses were compared to the high-resolution bathymetric surveys of the Water Club Marina and Granger Bay shown in Figure 3-23 and Figure 3-22.

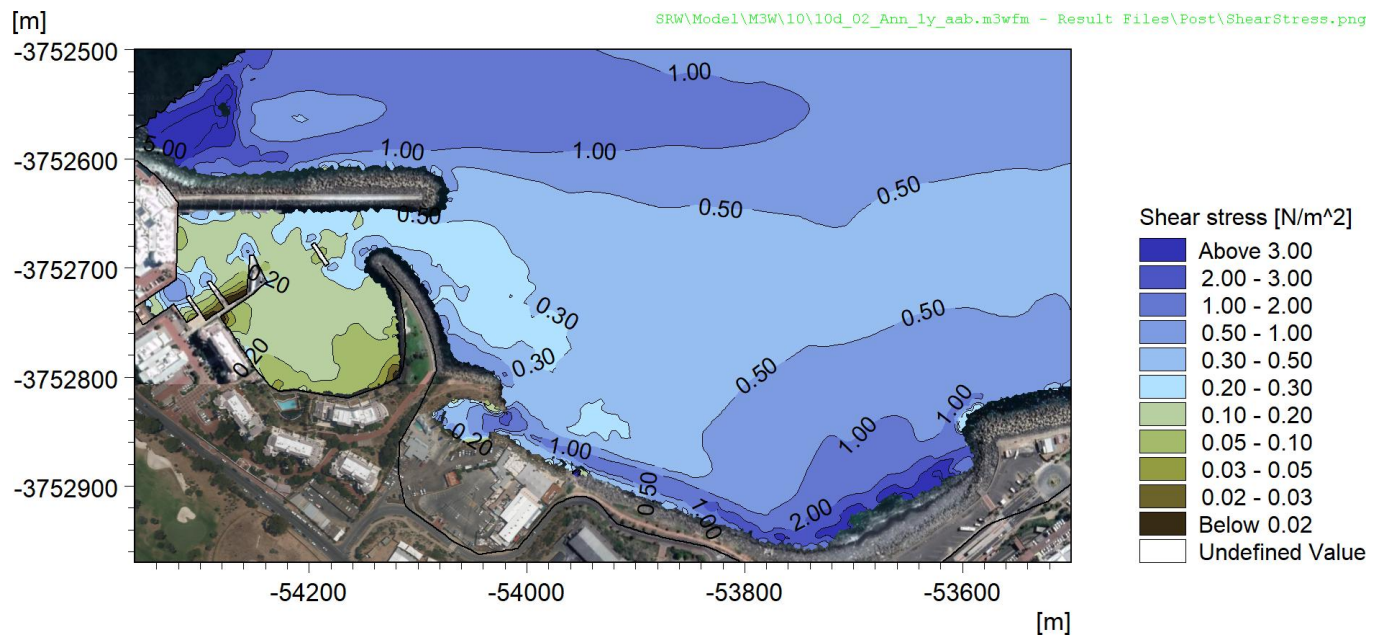


Figure 3-21: Modelled bed shear stresses during 1-year return period storm.

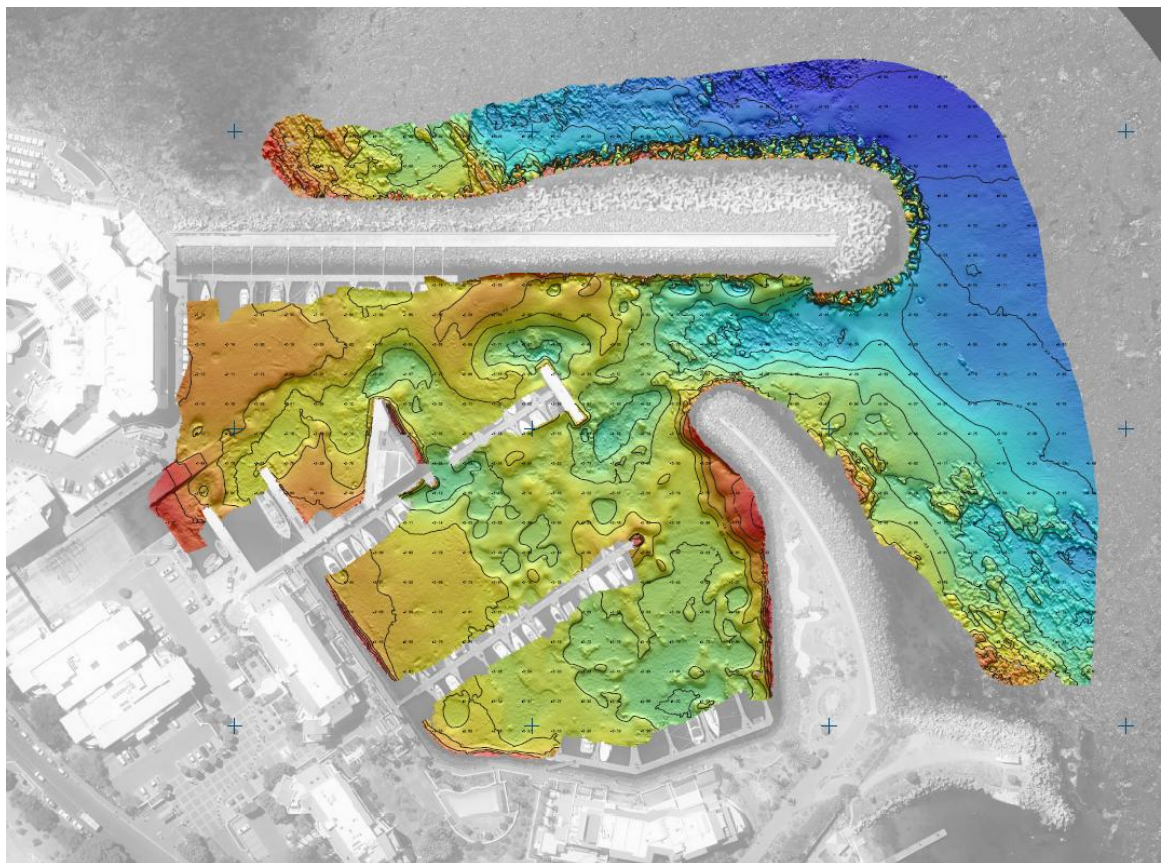


Figure 3-22: High-resolution bathymetric survey of the Water Club Marina (Tritan Survey, 2021)

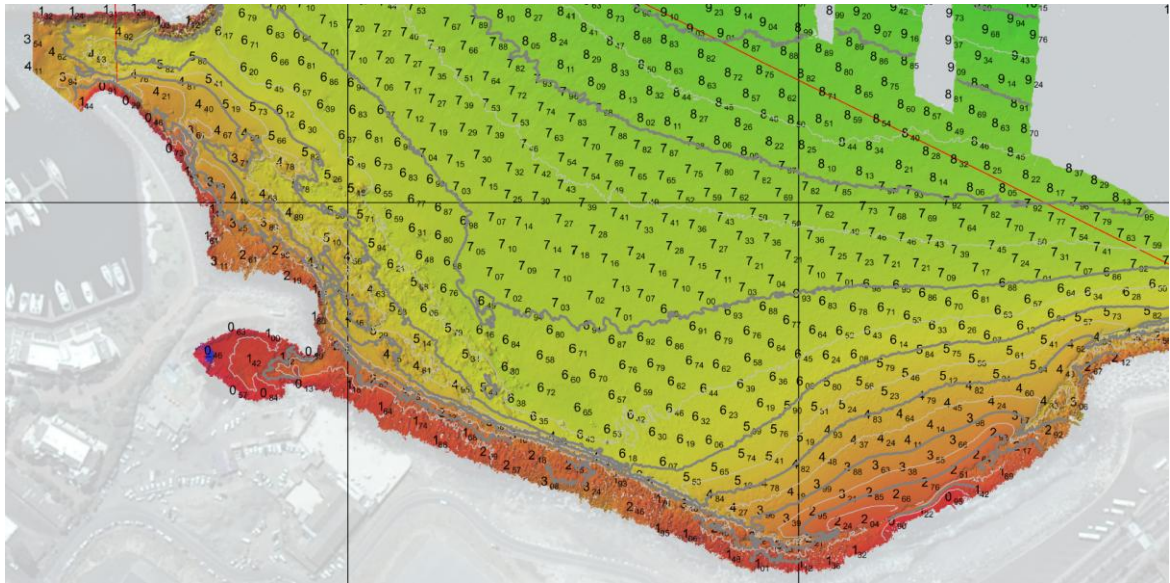


Figure 3-23: High-resolution bathymetric survey of Granger Bay (Underwater Surveys, 2022).

The Water Club Marina survey indicates a muddy seabed inside the marina, except at the entrance and adjacent to the non-floating quay wall west of the entrance where there are scour holes induced by strong currents due to harbour resonance. This is consistent with the model results which predict mud inside the marina except at the scour holes mentioned above.

The Granger Bay survey shows rocky areas along the shoreline north and east of Oceana, with what appears to be a sandy bottom over the remainder of the bay. This is consistent with the model results which predict sand everywhere except inside the Water Club Marina.

This comparison suggests that the model well represents the wave-induced bed shear stresses around the site and confirms that the use of 0.2 N/m^2 as the critical shear stress for long-term mud accumulation.

3.2.5 Cases modelled

The wave model needed to be representative of the storm wave climate over the year. Fines are expected to be deposited during all calm periods. To check whether these fines will accumulate for long periods or will be resuspended during storms, the average 1-month return period storm during summer, the average 1-month return period storm during winter as well as the average 1-year return period storm were modelled.

The 1-year return period was determined from the extreme value analysis (EVA) described in PRDW (2022). The 1-month return periods for summer and winter were determined from a year of modelled waves. A typical year of waves was simulated at the wave model boundary at a depth of -20 m MSL using the same spectral wave model described in PRDW (2022). A time series of these waves is presented in Figure 3-24 with the 1-month summer, 1-month winter and 1-year significant storm wave heights (H_{m0}) indicated.

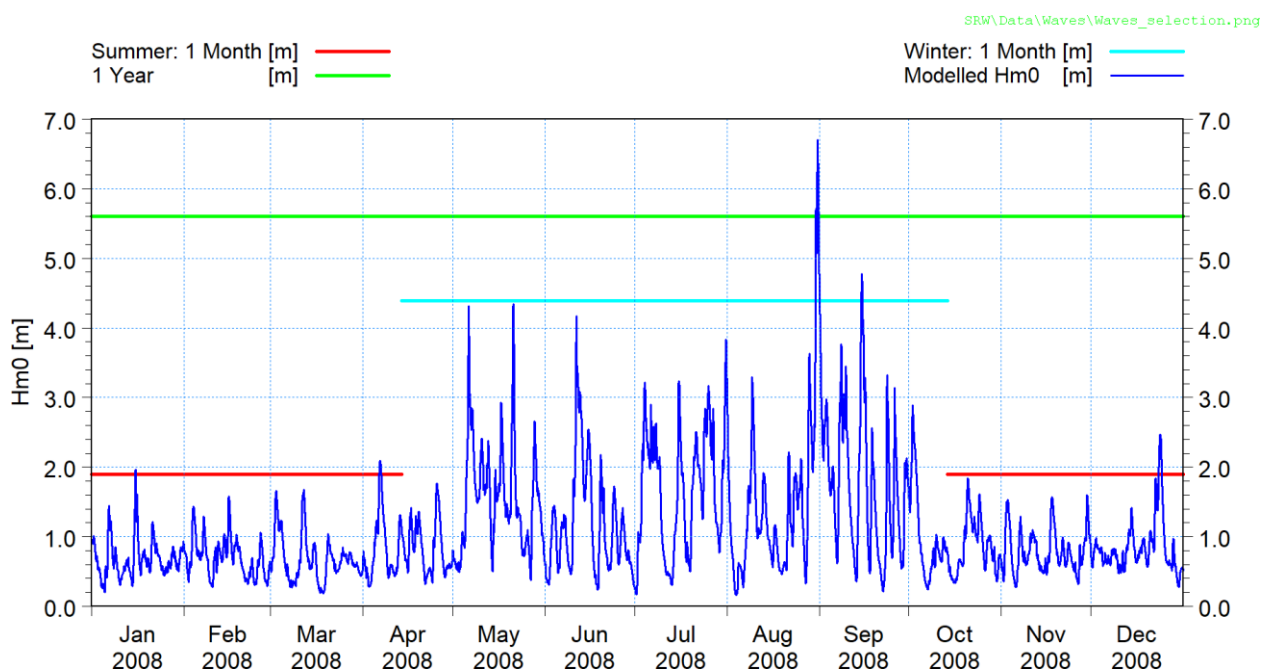


Figure 3-24: Time series of H_{m0} at -20 m MSL for a representative year with 1-month summer, 1-month winter and 1-year return period significant storm wave heights indicated.

After selecting the H_{m0} , typical peak wave periods (T_p), mean direction at T_p (D_{mTp}) and directional standard deviations (DSD) were assigned to each condition based on the data from the modelled year. The wave conditions are presented in Table 3-3 below.

Table 3-3: Wave parameters in -20 m MSL.

Season	Return Period	H_{m0} [m]	T_p [s]	D_{mTp} [deg]	DSD [deg]
Summer	1 month	1.9	13	268	16
Winter	1 month	4.4	14	280	16
Annual	1 year	5.6	14	280	16

In addition to the cases described above, three extreme cases modelled for the design of the marine infrastructure for the new development were analysed to assess the impact of the development on extreme waves within the Water Club Marina. These included a selection of the 475-year extreme cases detailed in the Wave Modelling Report (PRDW, 2025). These cases are presented in Table 3-4 below.

Table 3-4: Extreme wave parameters in -20 m MSL.

Return Period [y]		H_{m0} [m]	T_p [s]	D_{mTp} [deg]	DSD [deg]	Water Level [m MSL]
Waves	Water Level					
475	47.5	10.49	17	282.5	20	2.15
475	47.5	9.03	15	287.5	20	2.15
475	47.5	4.98	10	302.5	20	2.15

It is noted that the additional 475-year extreme events were modelled with the final design layout which differs slightly from the layout modelled originally for the 1-month and 1-year events for this study. The layouts are substantially similar and the very minor variations don't affect outcomes of the study.

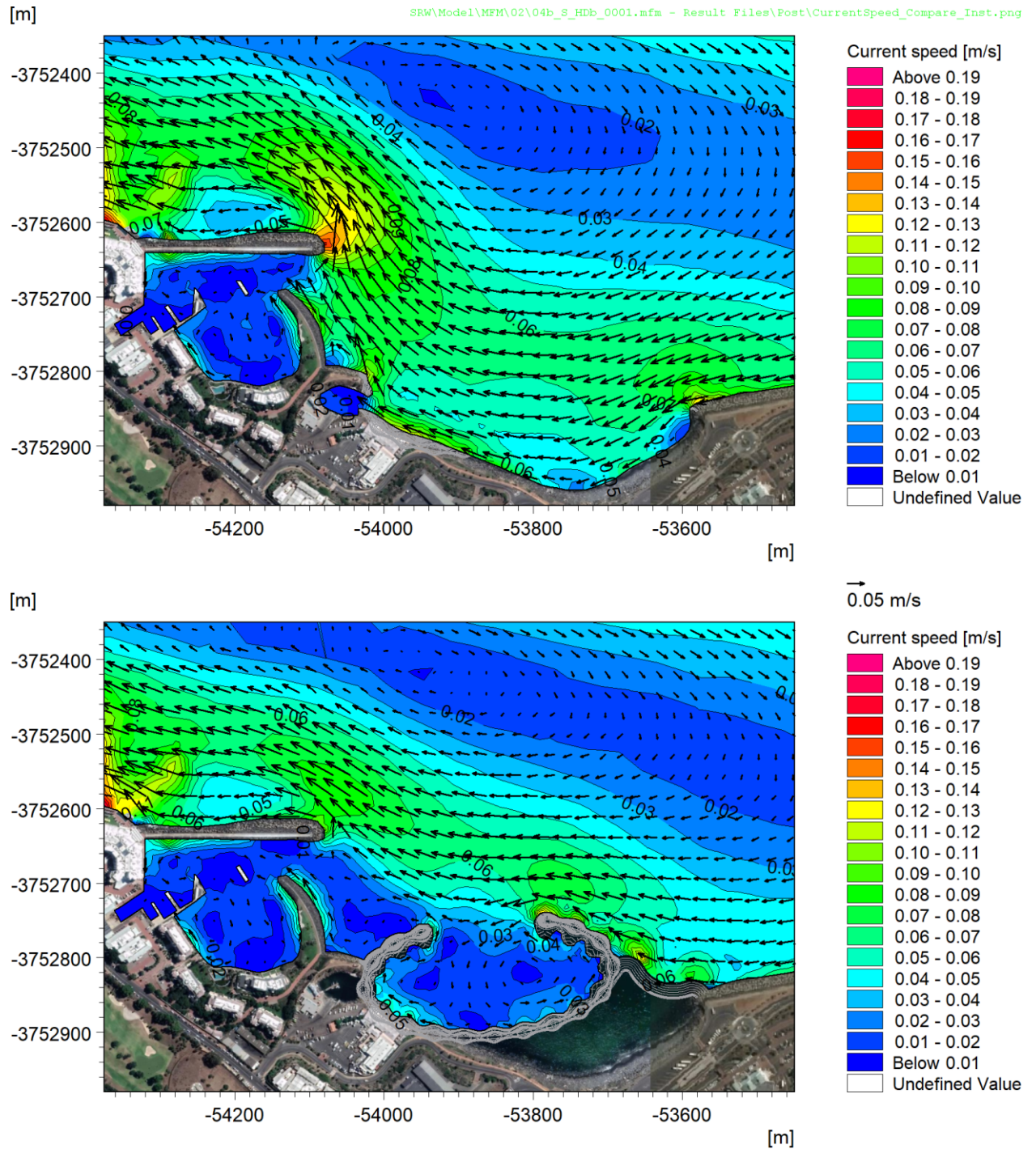


4. RESULTS

4.1 Current speed and direction

This section presents the results of the current speed due to wind, tides and ocean currents only, excluding wave-driven currents which are included in the wave-induced bed shear stresses described in Section 4.4.

Instantaneous depth-averaged current speeds and vectors during a south-easterly wind event are presented in Figure 4-1, while Figure 4-2 presents the instantaneous current speeds and vectors during a north-westerly event. The maximum (99th percentile) current speeds during the 6-week simulation for summer/autumn are presented in Figure 4-3 and for winter/spring in Figure 4-4. As the current speeds around the study site are very low, only the maximum currents (defined for robustness as the 99th percentile current speed over the 6-week simulation) are presented.



27/02/2020 15:00:00

Figure 4-1: Instantaneous depth-averaged current speed during a south-easterly wind event for the baseline (top) and development (bottom) layouts.

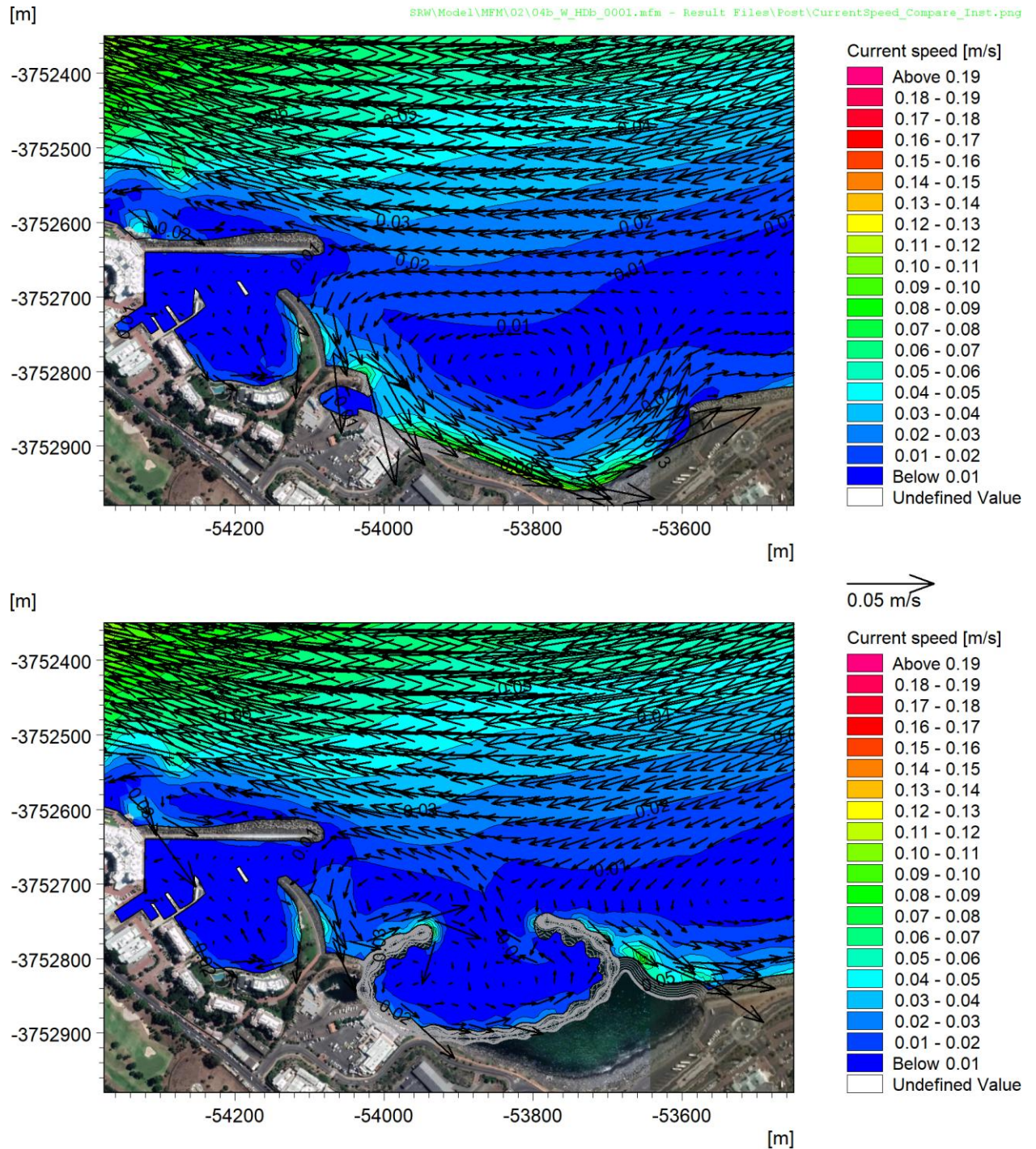


Figure 4-2: Instantaneous depth-averaged current speed during a north-westerly wind event for the baseline (top) and development (bottom) layouts.

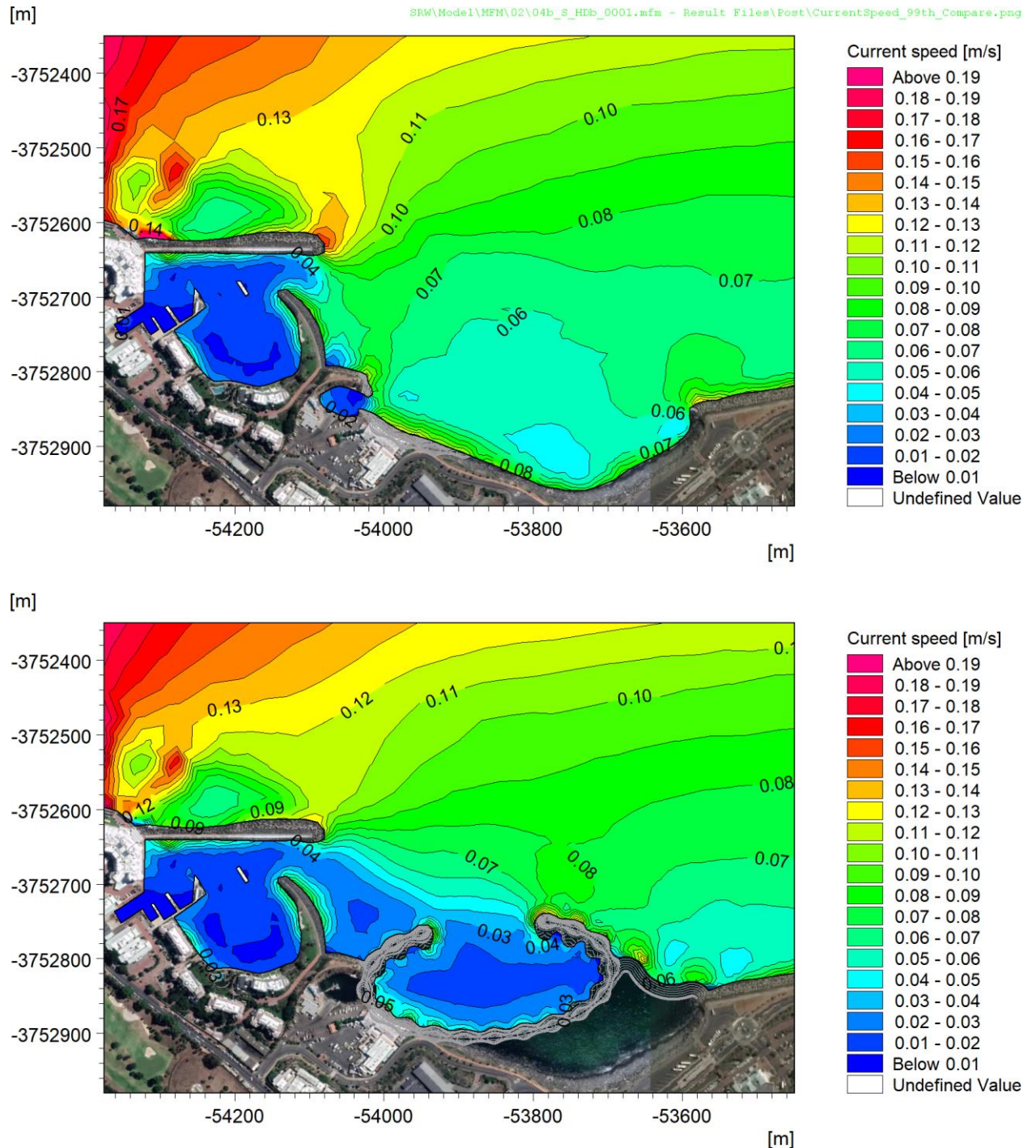


Figure 4-3: Maximum (99th percentile) depth-averaged current speed for the baseline (top) and development (bottom) layouts for the summer/autumn case.

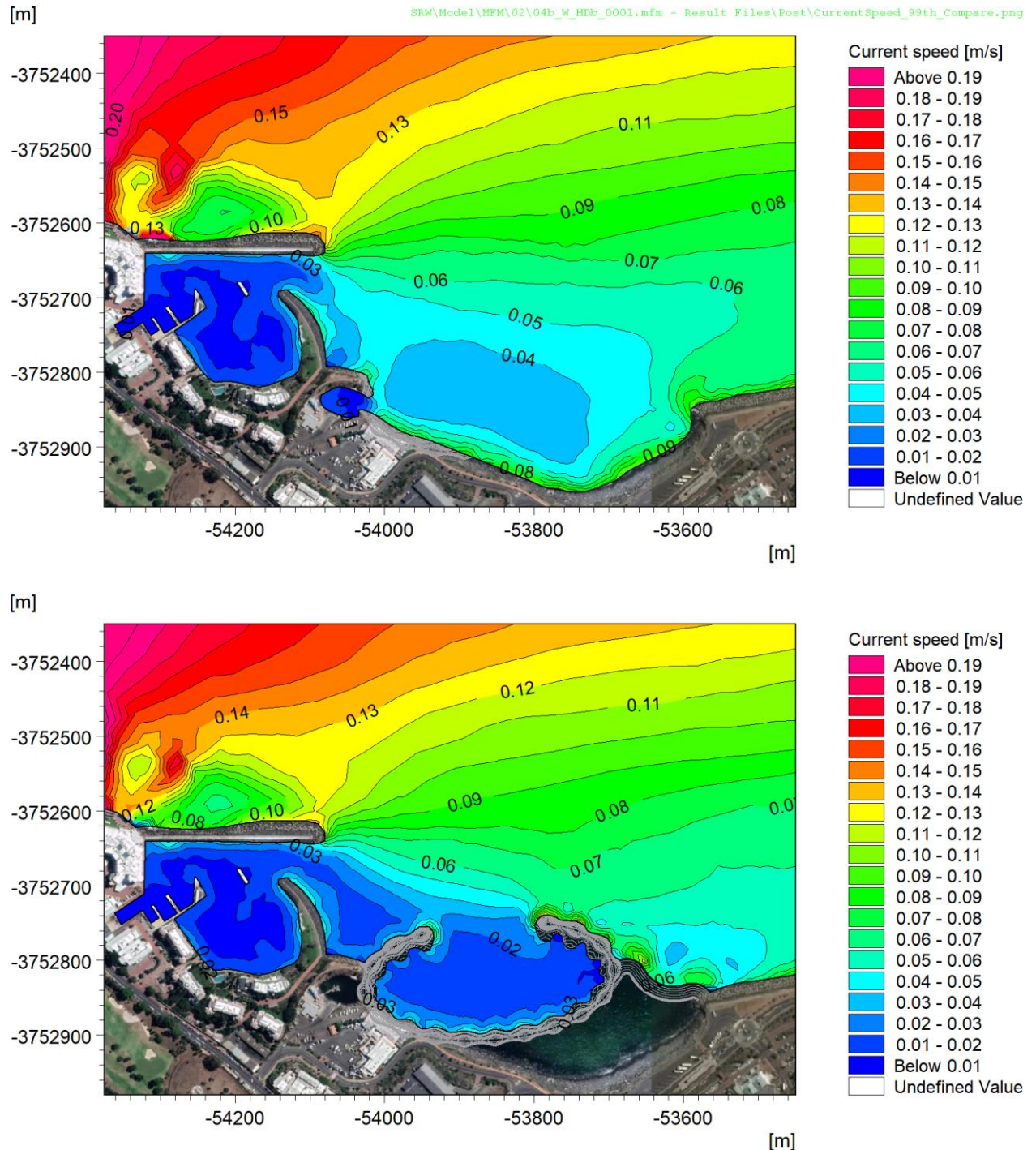


Figure 4-4: Maximum (99th percentile) depth-averaged current speed for the baseline (top) and development (bottom) layouts for the winter/spring case.

The model results inside the development layout are compared to the baseline layout. In addition, the results inside the development layout are compared to those inside the existing Water Club Marina, since this may assist with the marine ecological assessment.

The figures above show that:

- The current speeds in Granger Bay due to wind, tides, and ocean currents are very low with speeds generally not exceeding 0.1 m/s.



- The addition of the development reduces the maximum current speed inside the development from 0.06 m/s in summer and 0.04 m/s in winter to 0.02 m/s.
- The maximum current speeds inside the new development are higher (0.02 m/s) than those inside the Water Club Marina (0.01 m/s).
- The development does not change the currents significantly beyond 300 m from the development.

4.2 Flushing of water and seawater temperature

The depth-averaged residual current was calculated to provide an estimate of circulation patterns with and without the development. These residual currents are calculated as the vector average of the depth-averaged velocities over the 6-week simulation for summer/autumn and winter/spring and are presented in Figure 4-5 and Figure 4-6, respectively.

The maximum (99th percentile) surface seawater temperature is presented to show the effect of the new development for summer/autumn and winter/spring in Figure 4-7 and Figure 4-8. Note that these are only the maximum temperatures during the 6-week simulation periods and not an estimate of the absolute maximum temperatures that can be expected. They do however provide a robust estimate of the increase in temperature inside the development.

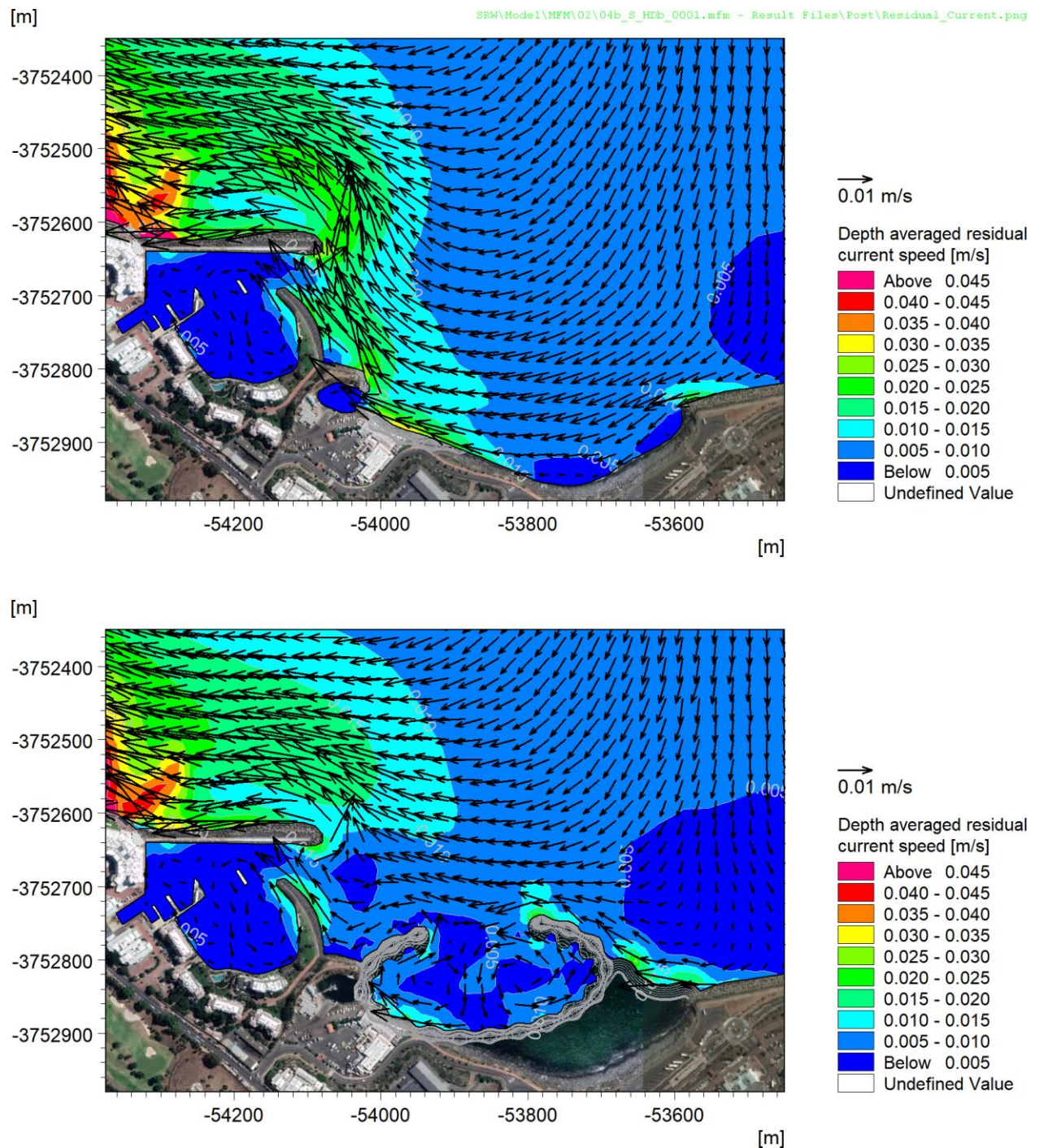


Figure 4-5: Depth-averaged residual currents for the baseline (top) and development (bottom) layouts for the summer/autumn case.

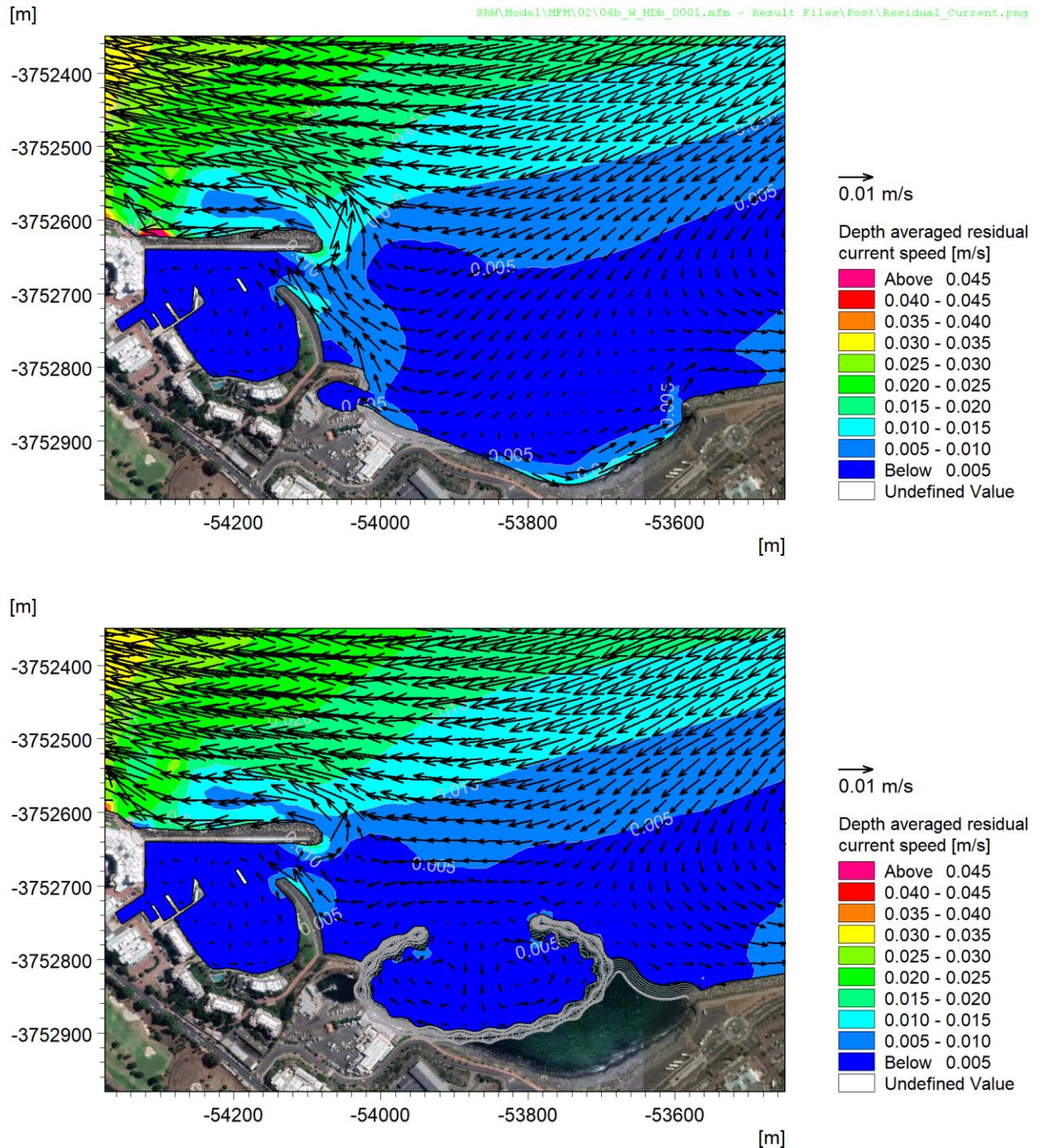


Figure 4-6: Depth-averaged residual currents for the baseline (top) and development (bottom) layouts for the winter/spring case.

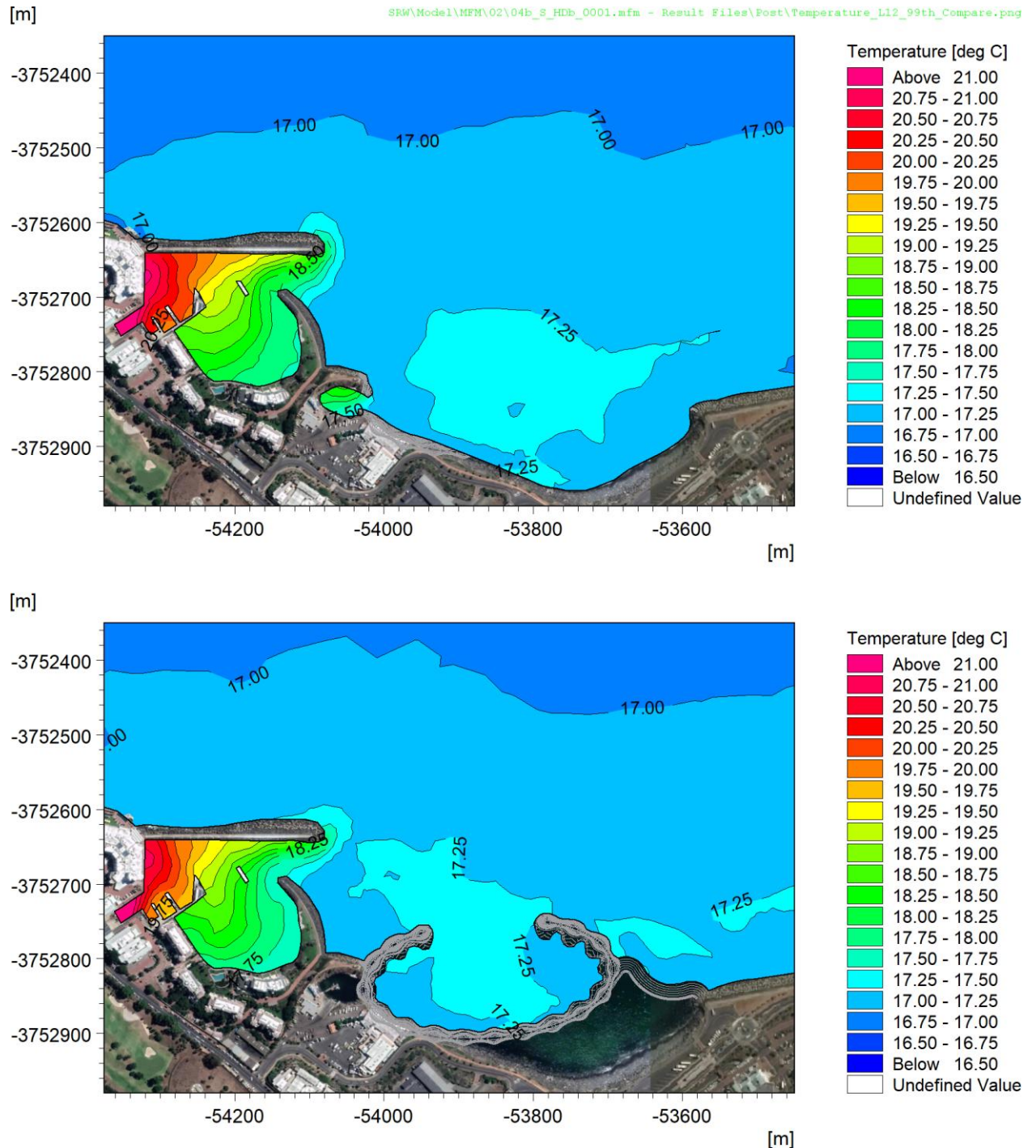
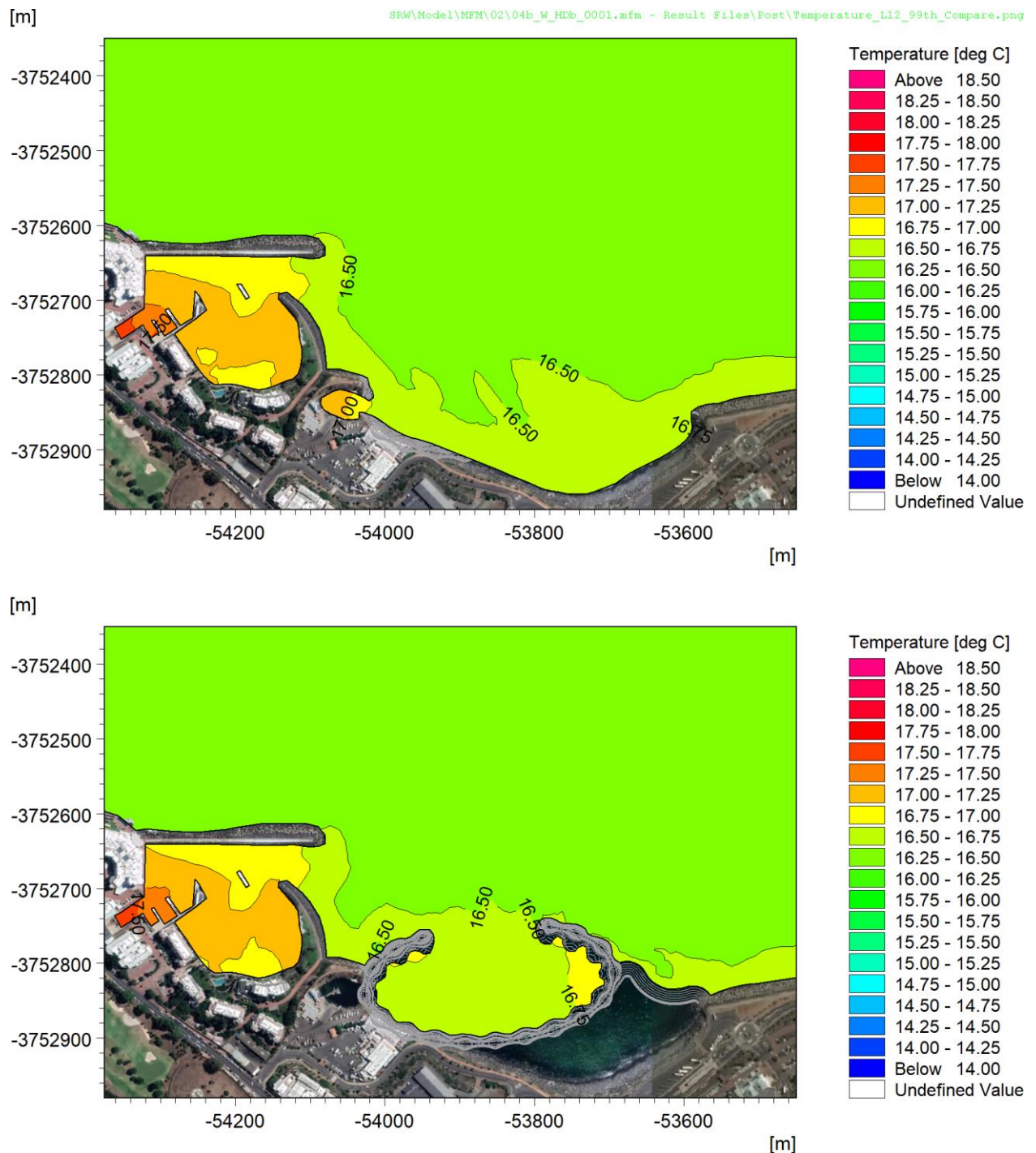


Figure 4-7: Maximum (99th percentile) surface seawater temperature for the baseline (top) and development (bottom) layouts for the summer/autumn case.



The figures above show that:

- The residual circulation inside the new development comprises twin eddies which will induce flushing of water out of the development. This circulation should also assist in limiting the accumulation of floating pollutants in the proposed new embayment, although direct wind forces on floating debris may overcome the relatively weak currents, leading to accumulation during north-westerly events.
- The residual circulation is stronger in summer/autumn due to the strong south-east winds.
- The residual circulation within the development is slightly weaker than the baseline circulation, but stronger than the circulation within the Water Club Marina.



- The maximum surface seawater temperature inside the development area increases slightly (by about 0.25°C) in the winter/spring case, but remains the same in summer/autumn. This is explained by the stronger residual circulation in summer/autumn which does not allow a temperature build-up despite a net input of solar radiation in summer.
- The weaker residual circulation within the Water Club Marina results in a build-up in seawater temperature inside the Water Club Marina, particularly in summer.

4.3 Waves

4.3.1 Example results

The total significant wave heights (H_{m0}) and the 99th percentile depth-averaged wave-induced current speeds for each wave condition are presented in this section for the baseline and development layouts. The modelled cases are presented in Table 4-1.

Table 4-1: Modelled wave cases.

Return Period [y]		H_{m0} [m]	T_p [s]	D_{mTp} [deg]	DSD [deg]	Water Level [m MSL]
Waves	Water levels					
1 month in summer	-	1.9	13	268	16	0.201
1 month in winter	-	4.4	14	280	16	0.201
1	-	5.6	14	280	16	0.201
475	47.5	10.5	17	282.5	20	2.15
475	47.5	9.0	15	287.5	20	2.15
475	47.5	5.0	10	302.5	20	2.15

Figure 4-9 presents an example of the instantaneous water surface elevations for the 1-year return period storm, which shows the individual wave crests (warm colours) and troughs (cool colours) as well as the processes of wave refraction, diffraction, reflection and harbour resonance.

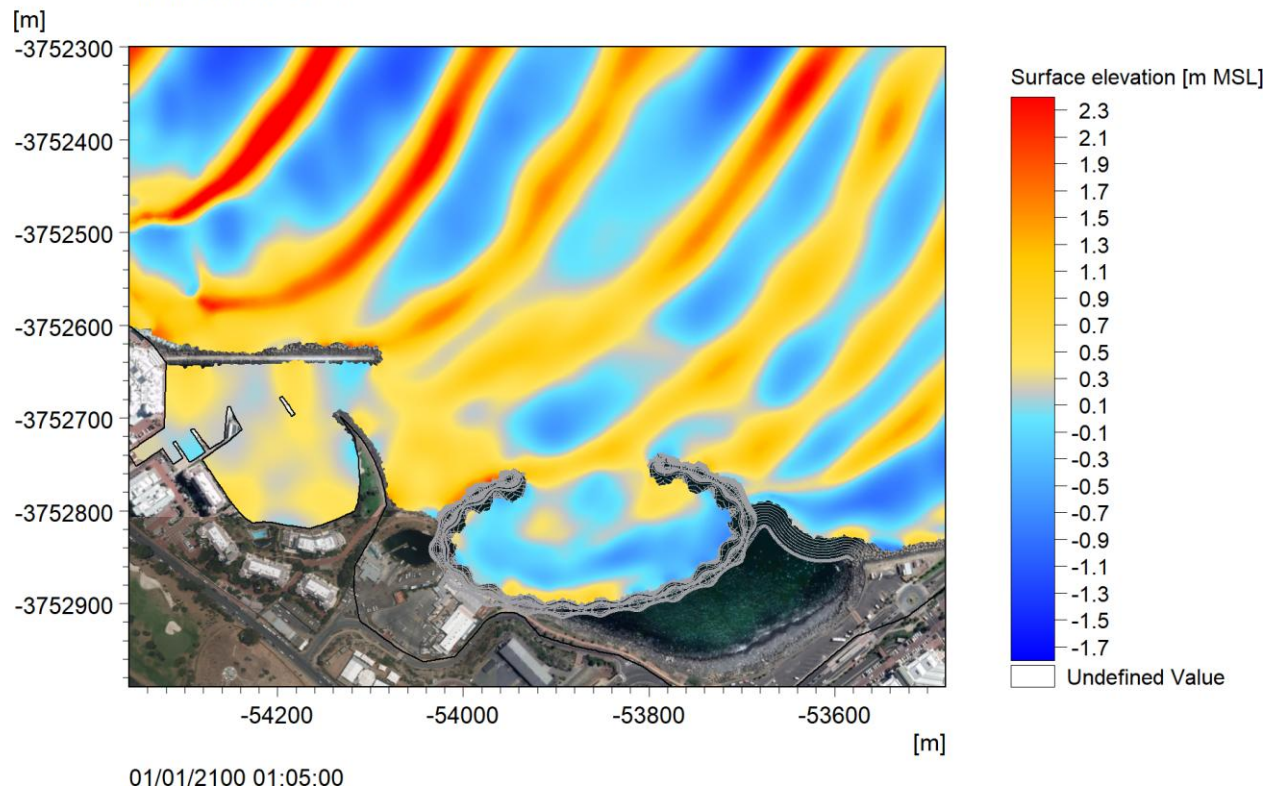
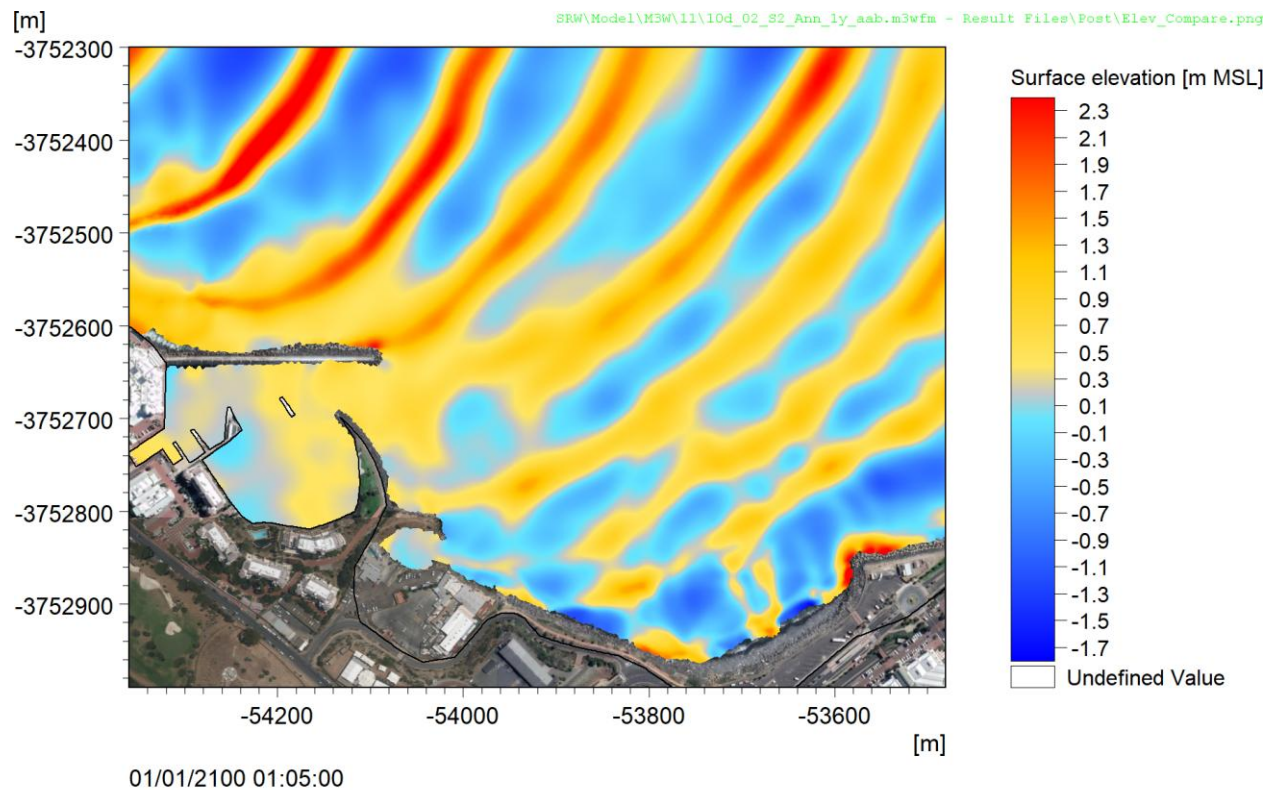


Figure 4-9: Instantaneous water surface elevations for baseline (top) and development (bottom) layouts for the 1-year return period.

4.3.2 Changes in wave heights

These total significant wave heights include both swell waves (period <25 s) and longer period waves caused by bound waves, surf beat and harbour resonance. The H_{m0} results are shown in Figure 4-10 to Figure 4-15.

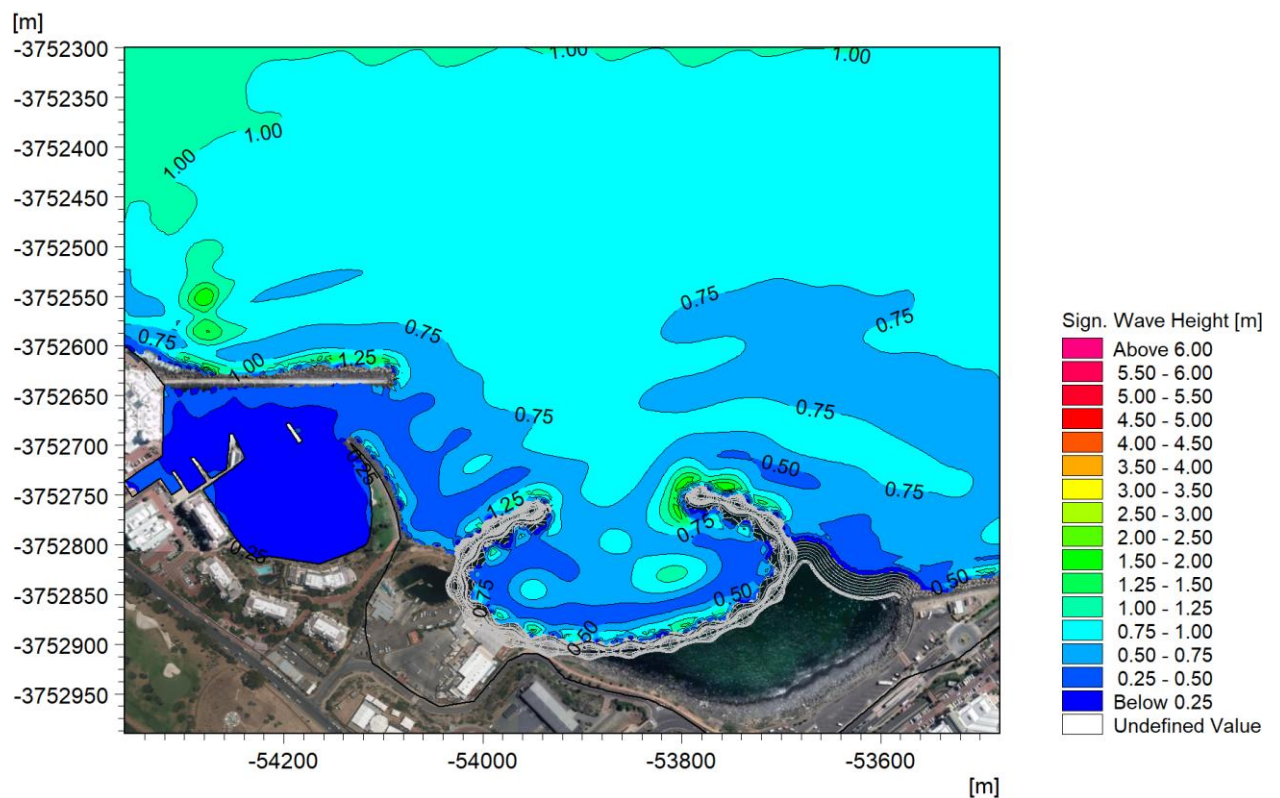
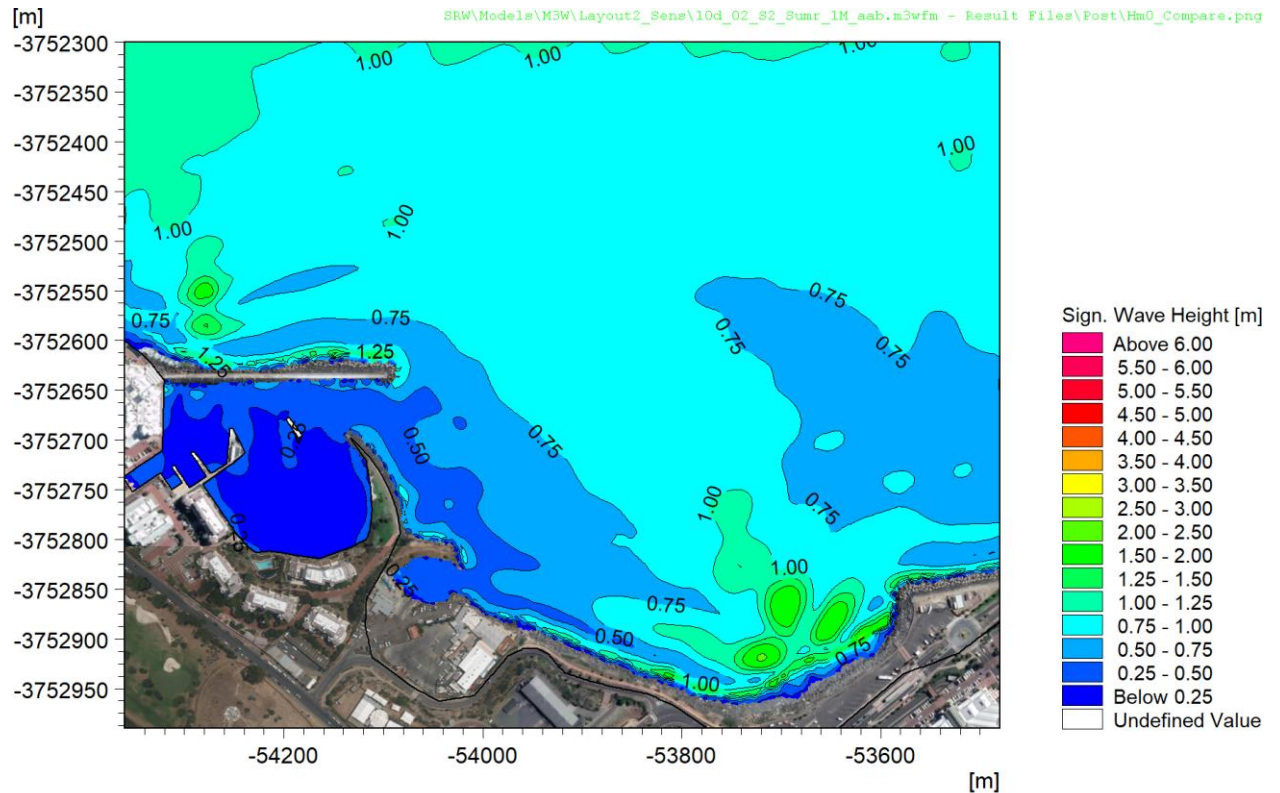
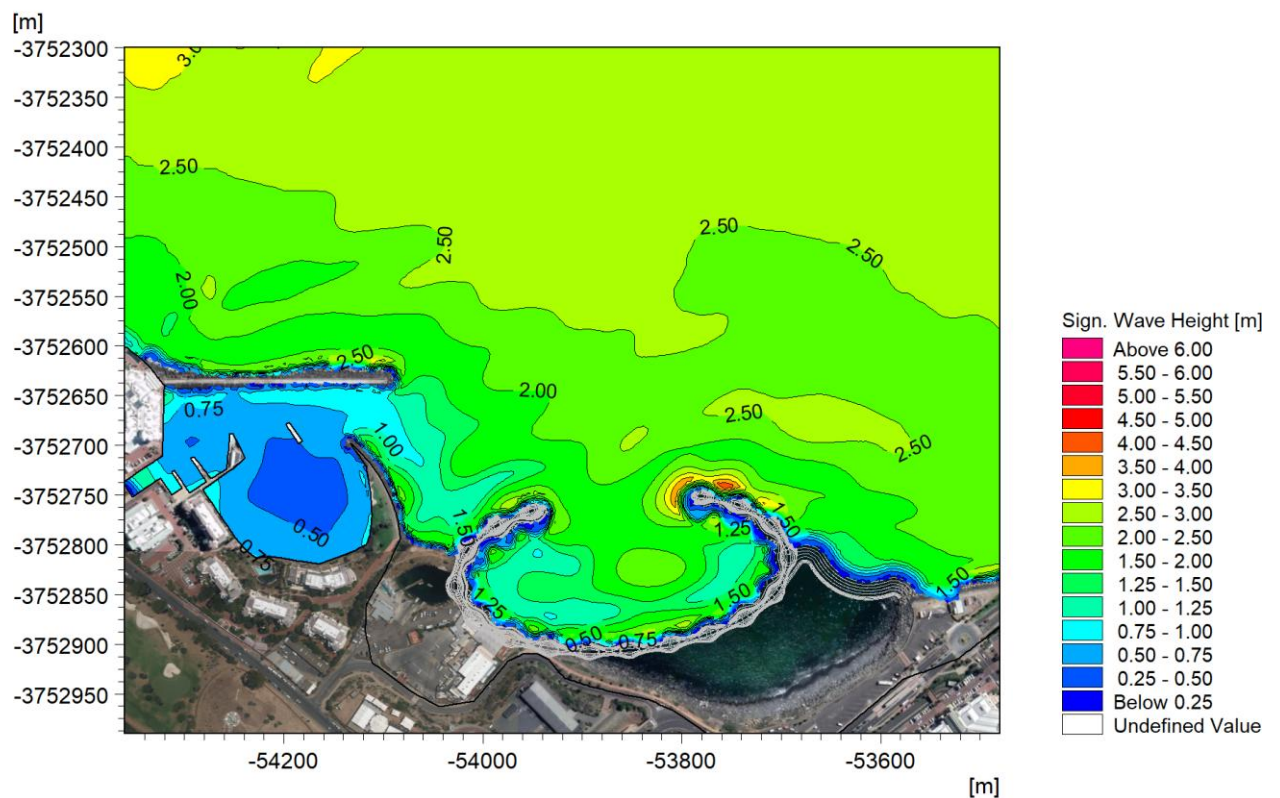
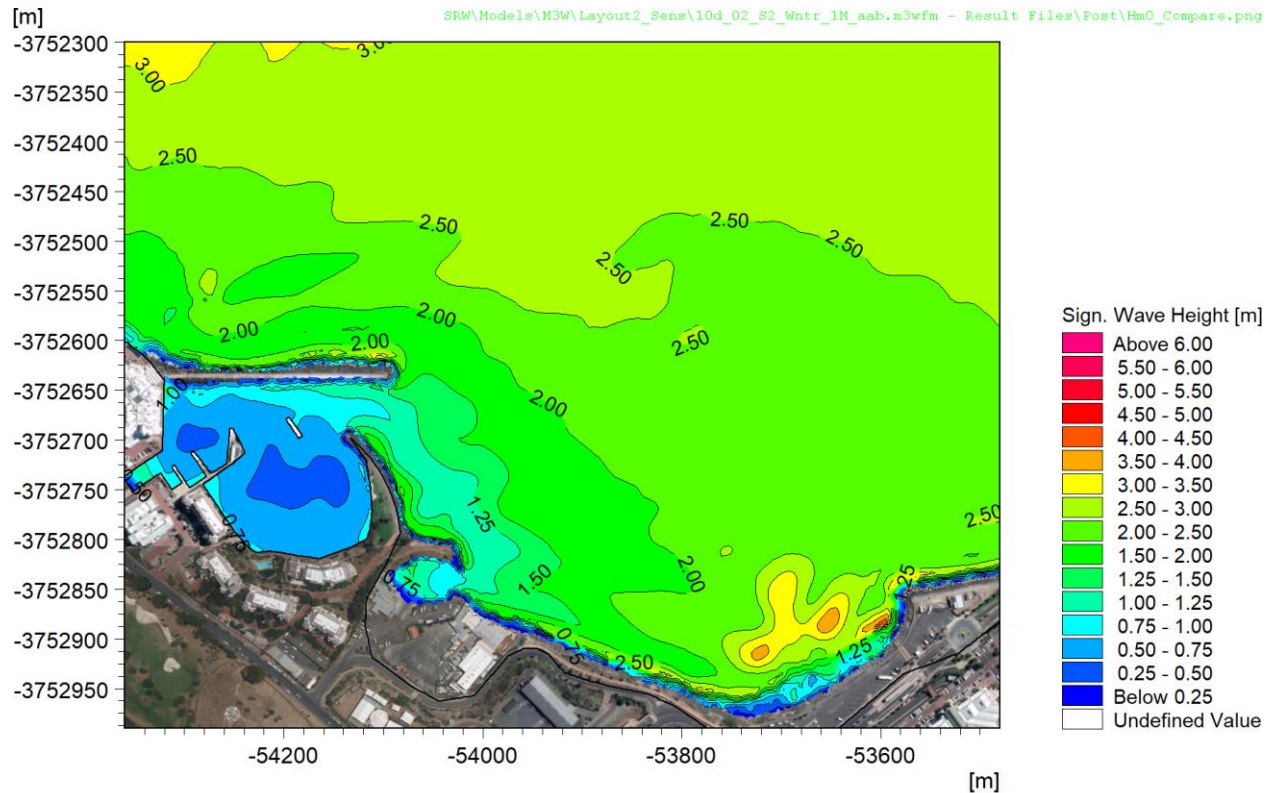
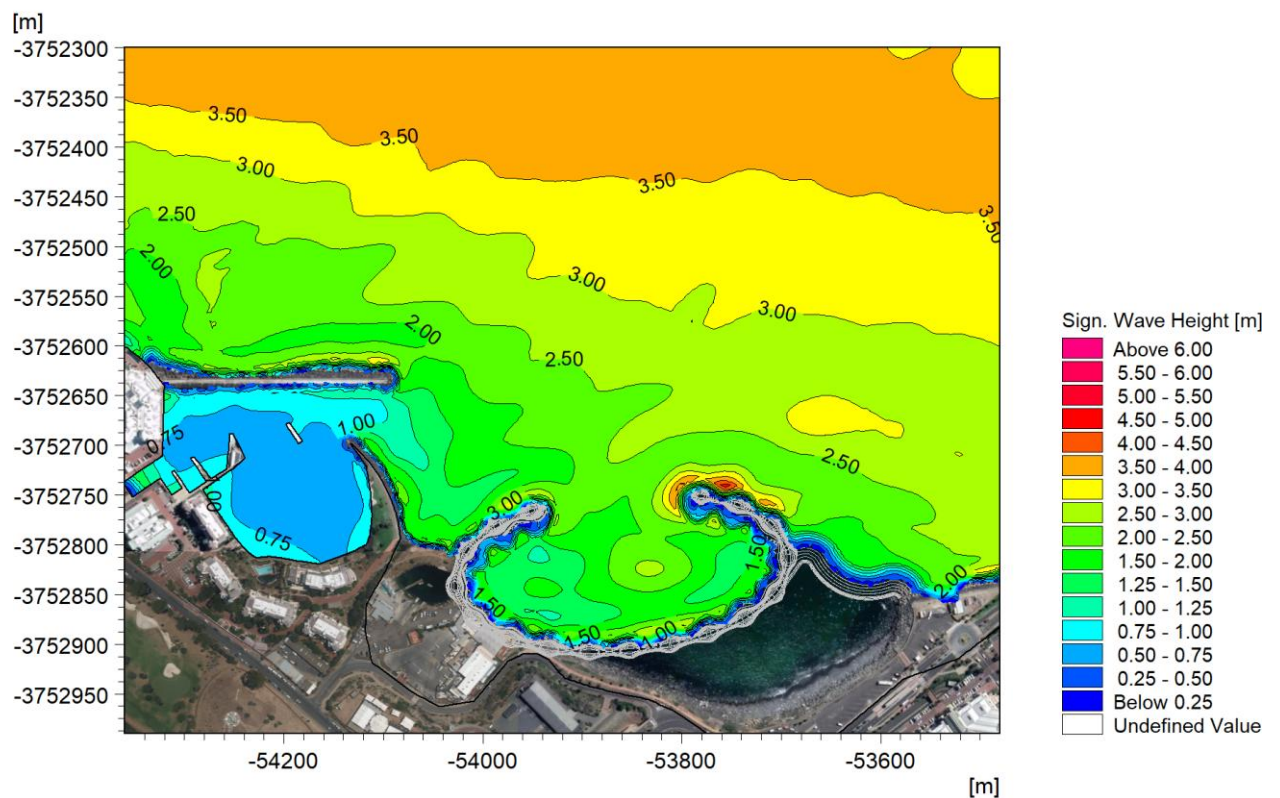
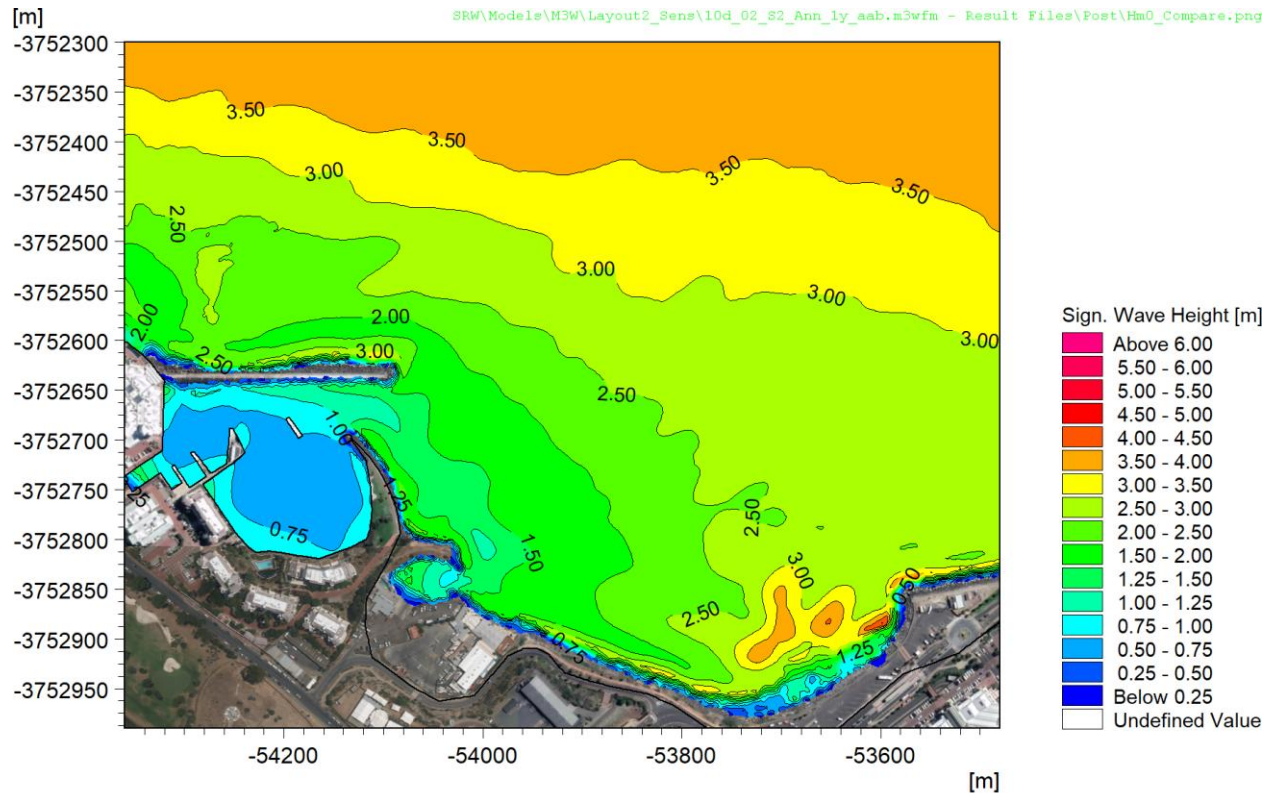


Figure 4-10: Total H_{m0} for baseline (top) and development (bottom) layouts for the 1-month return period in summer.



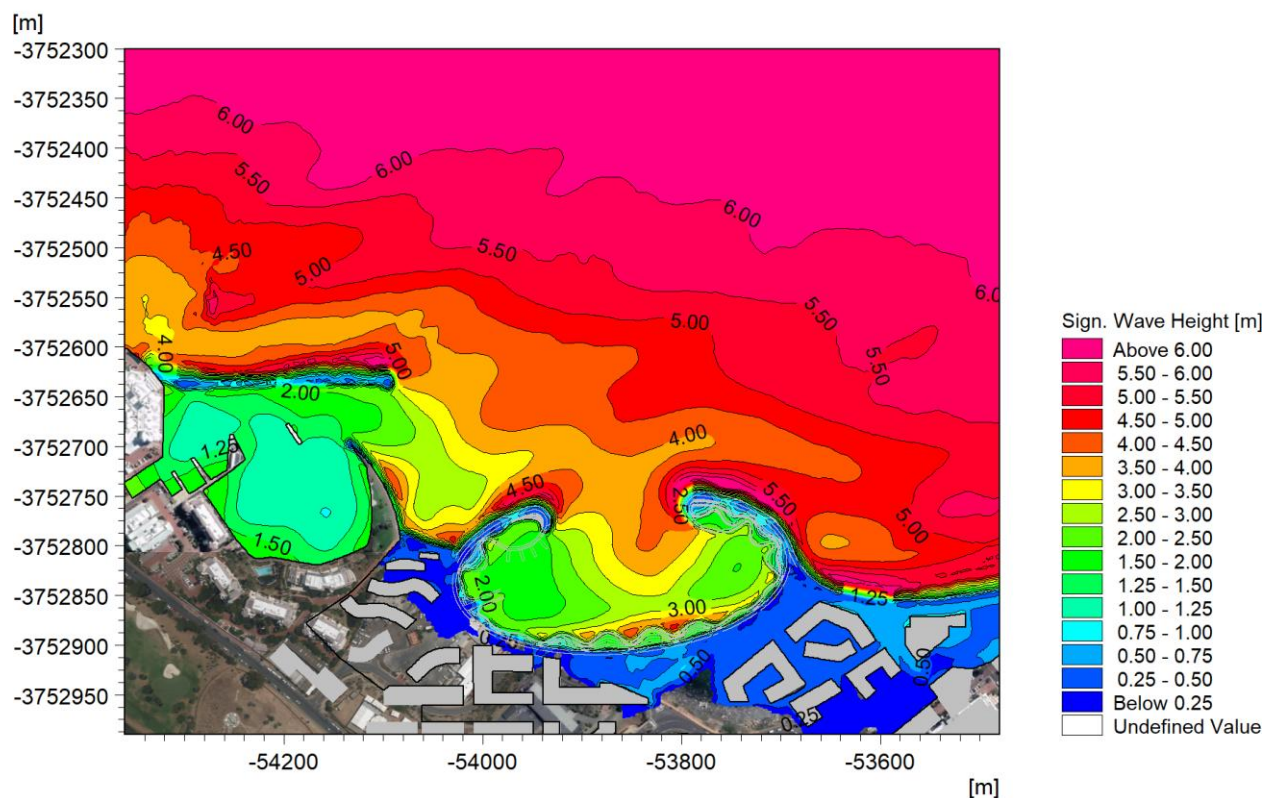
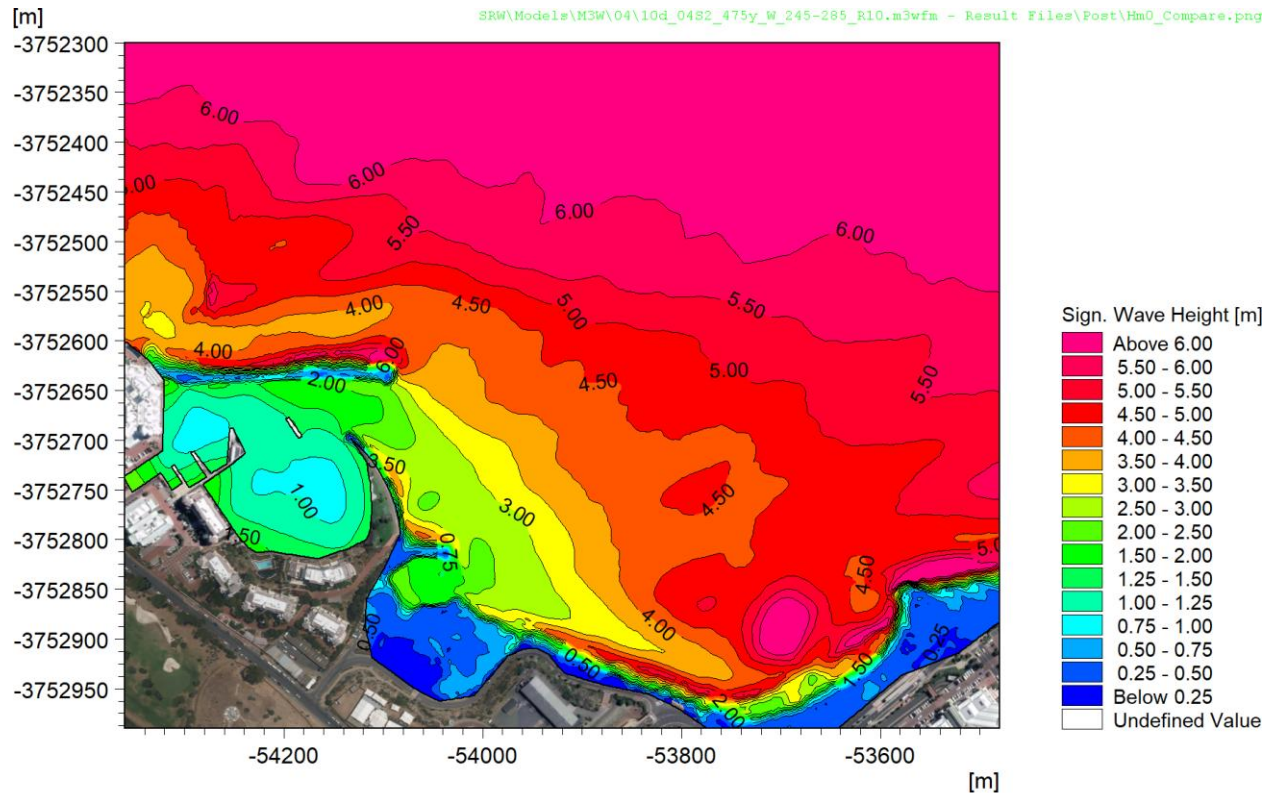
2100/01/01 01:05:00

Figure 4-11: Total H_{m0} for baseline (top) and development (bottom) layouts for the 1-month return period in winter.



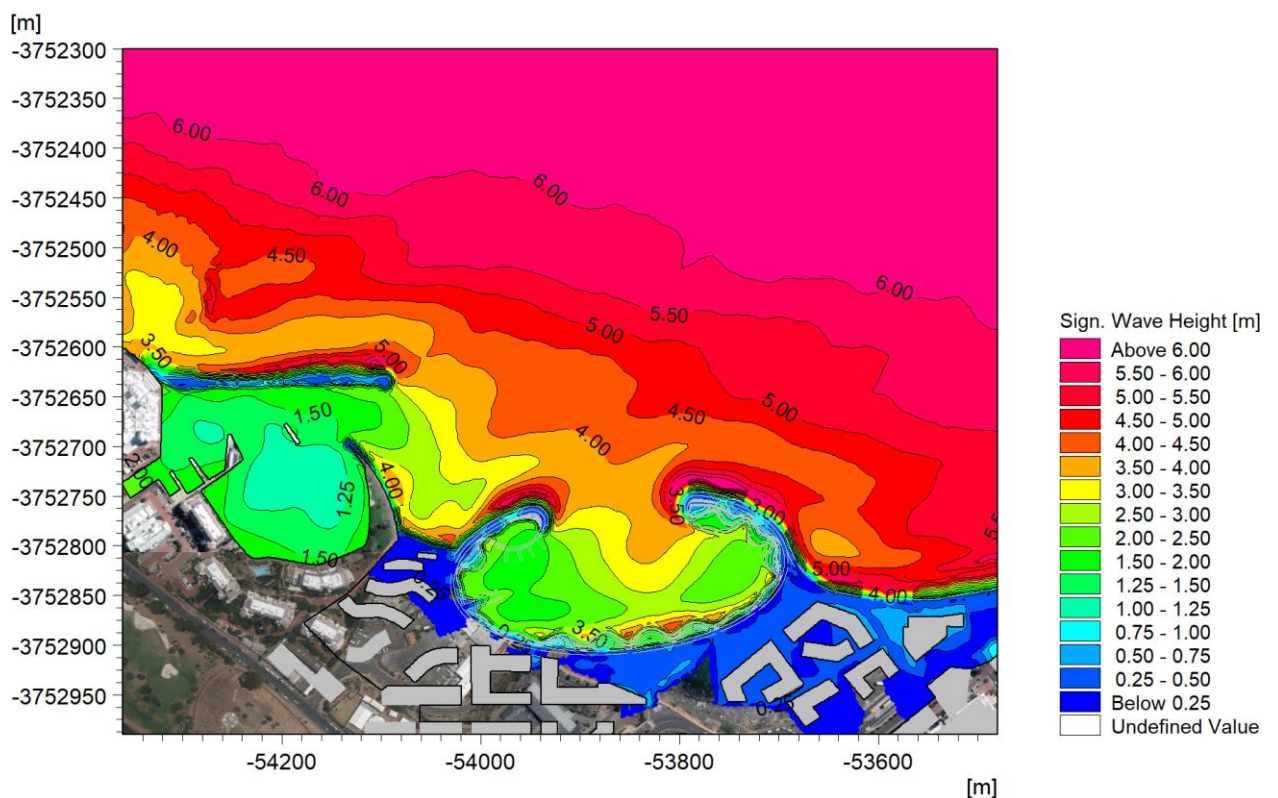
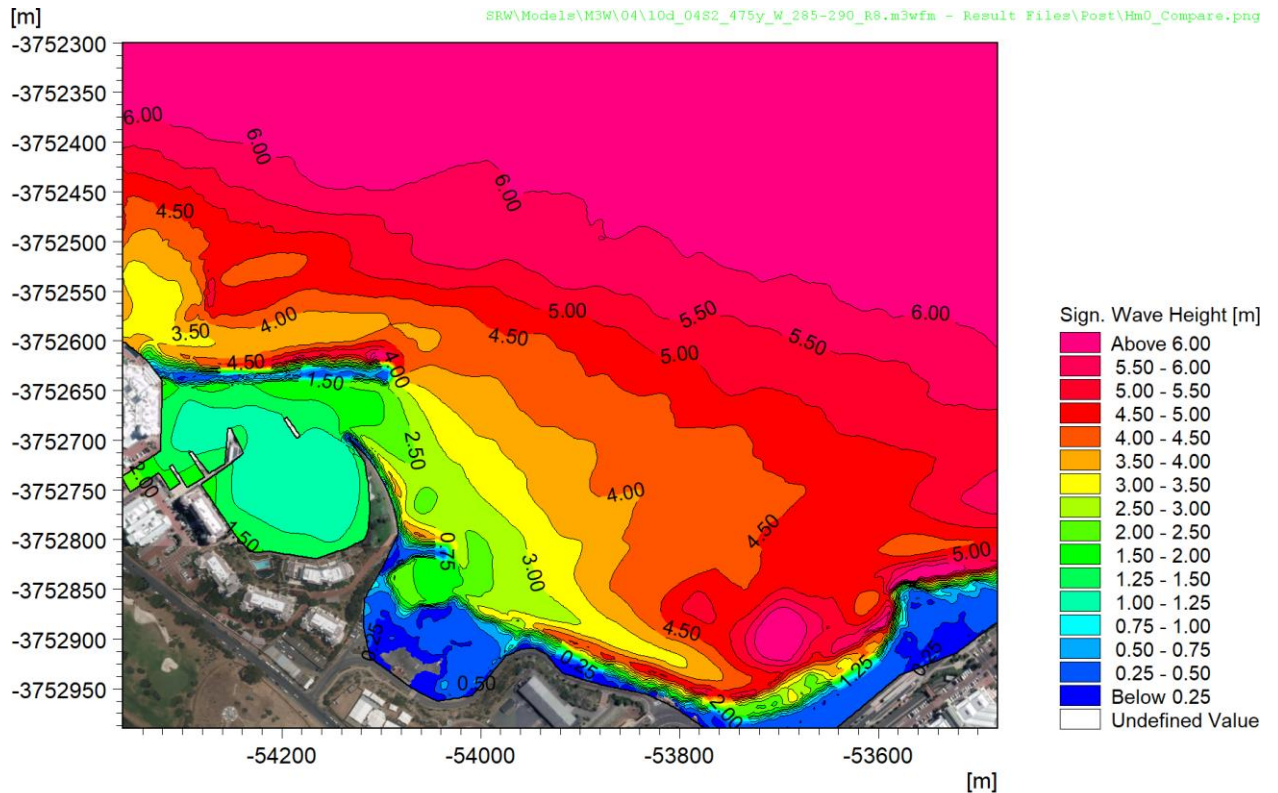
2100/01/01 01:05:00

Figure 4-12: Total H_{m0} for baseline (top) and development (bottom) layouts for the 1-year return period.



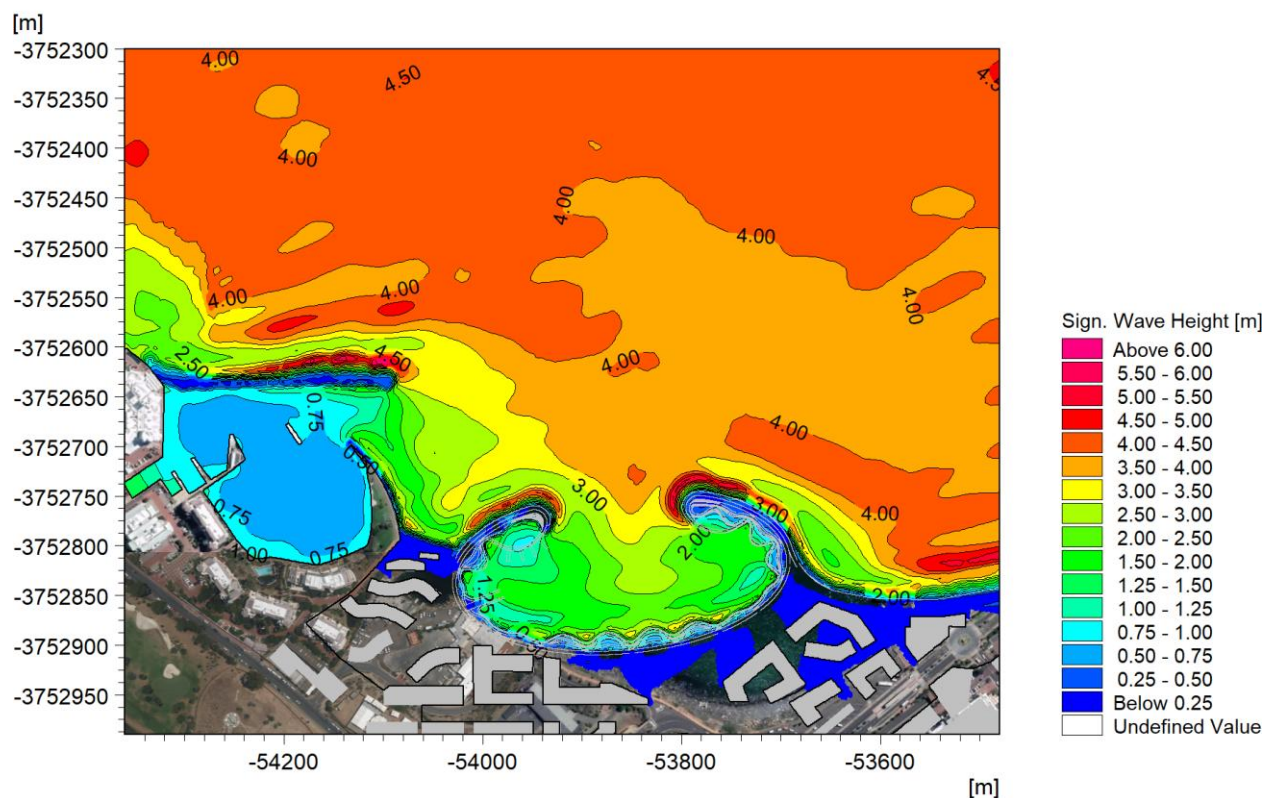
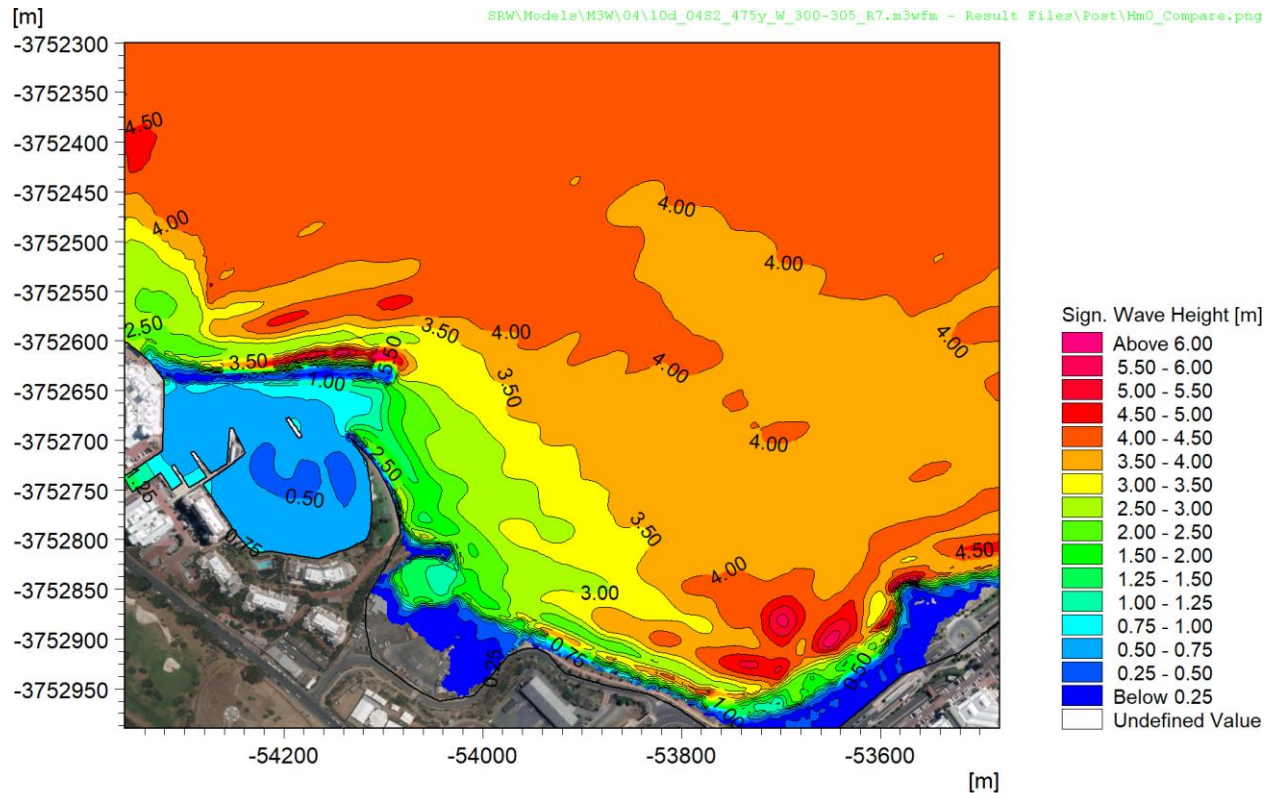
2100/01/01 01:05:00

Figure 4-13: Total H_{m0} for baseline (top) and development (bottom) layouts for the 475-year return period from 282.5 degrees.



2100/01/01 01:05:00

Figure 4-14: Total H_{m0} for baseline (top) and development (bottom) layouts for the 475-year return period from 287.5 degrees.



2100/01/01 01:05:00

Figure 4-15: Total H_{m0} for baseline (top) and development (bottom) layouts for the 475-year return period from 302.5 degrees.

Table 4-2 presents the basin-averaged total H_{m0} for each case and layout and shows the difference in total H_{m0} .

Table 4-2: Basin-averaged total H_{m0} in the Water Club Marina for each case.

Case	Total H_{m0} [m]		Difference [m]
	Baseline	Development	
1-month in summer	0.22	0.19	-0.03
1-month in winter	0.61	0.58	-0.03
1-year	0.72	0.69	-0.03
475-year from 282.5 degrees	1.20	1.31	0.11
475-year from 287.5 degrees	1.26	1.38	0.12
475-year from 302.5 degrees	0.61	0.73	0.11

The results show that for the 1-month summer and winter cases, as well as the 1-year return period case, there is a general reduction of 0.03 m in the total significant wave height within the Water Club Marina with the addition of the development. For the extreme design cases (475-year return period), there is a slight increase in total wave height within the Water Club Marina with the addition of the proposed development. This corresponds to an increase in the total wave height of approximately 0.12 m for the largest case.

The wave height results presented above were used to evaluate the change in waves along the inside of the main breakwater and the outside of the lee breakwater for the Water Club Marina. This was done by extracting the maximum significant wave height (H_{m0}) occurring anywhere along the structure toes for any of the 475-year return period cases and comparing this result between the baseline and the development layouts. There is very little to no change to the H_{m0} on the outside of the main breakwater of the Water Club Marina. Changes in wave height behind the main breakwater are due to reflections off the development's western breakwater that run parallel to the Water Club Marina's lee breakwater and approximately normal to the main breakwater. For breakwater stability an increase in wave height for directions normal to the breakwater is more significant than for directions close to parallel to a breakwater.

The results along the inside toe of the main breakwater saw a maximum increase in H_{m0} (approximately normal to the breakwater) of 0.04 m from 2.09 m to 2.13 m (2%). In terms of breakwater stability an increase in design wave height of two percent is not considered significant and the main breakwater is therefore not significantly affected by reflections off the western breakwater of the development.

The outside of the lee breakwater saw an increase in H_{m0} (approximately parallel to the breakwater) of 0.53 m from 3.18 m to 3.71 m (17 %). Since this increase in wave height is for a direction approximately parallel to the lee breakwater it is expected that this will have less of an implication for breakwater stability compared to waves normal to the structure.

Overall, the development is not expected to have a significant impact on the long-term stability of the existing Water Club Marina breakwaters.

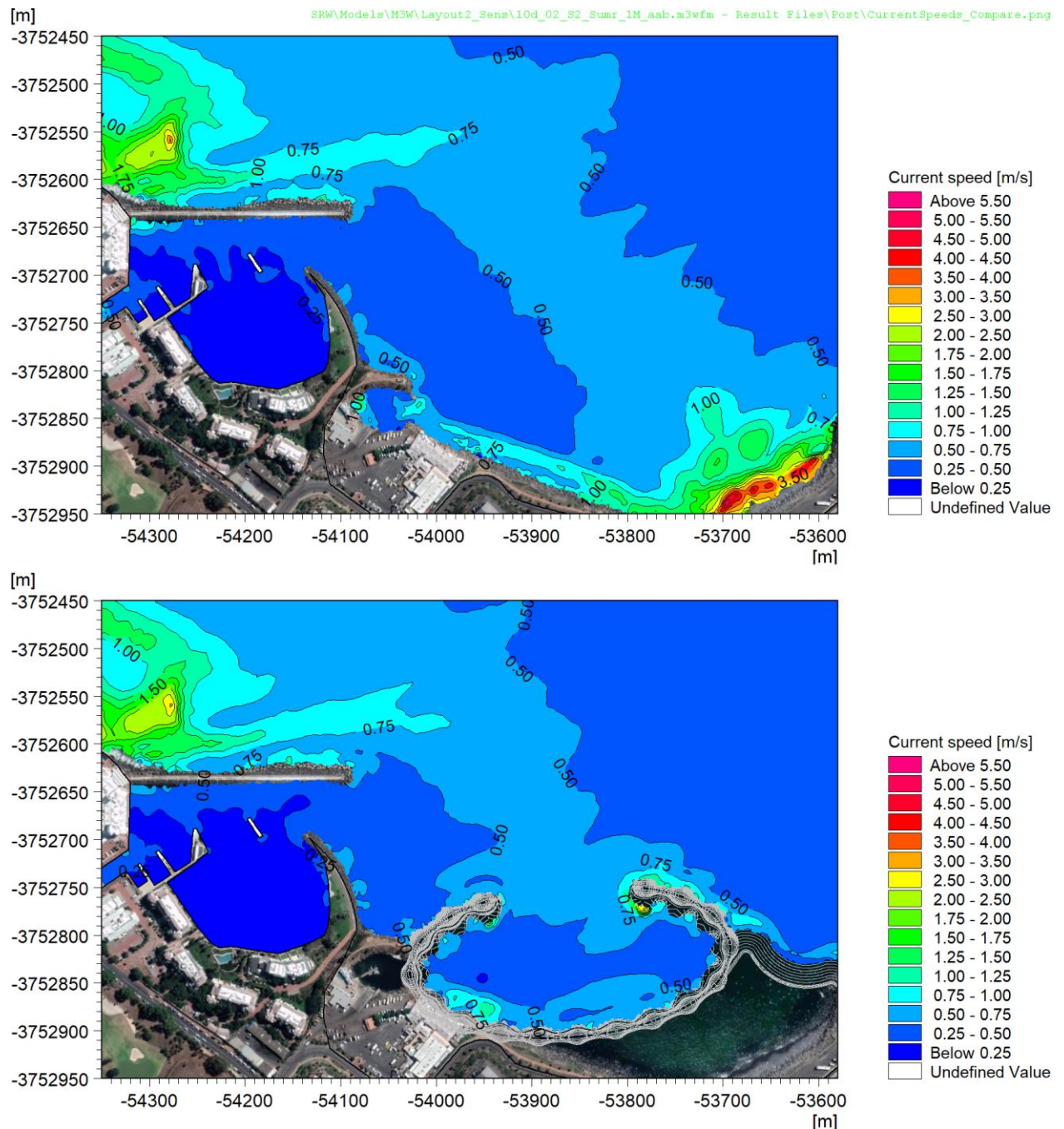
The wave height inside the proposed development is generally reduced compared to the baseline layout, however there is significant amplification in the centre of the development due to harbour resonance. Boats can avoid this area of amplification by staying in the lee of the western breakwater. The wave heights inside the proposed development are significantly larger than those inside the Water Club Marina. However, wave heights in the lee of the western breakwater and on the slipway are similar to those within the current Oceana facility. Downtime for launching from the slipway is also expected to be similar to the current Oceana slipway.

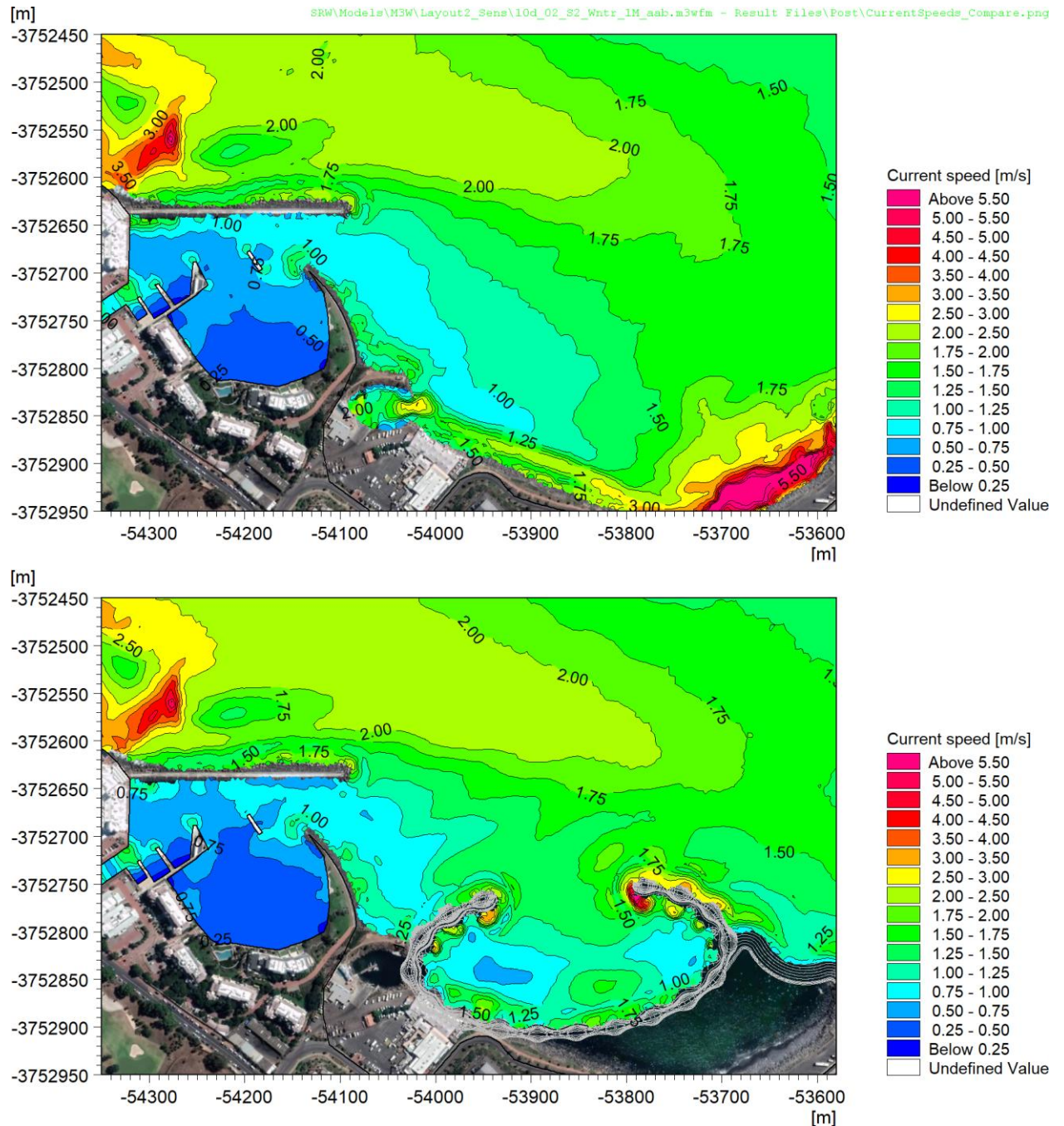
The reflections off the eastern breakwater of the proposed development generate nodes and anti-nodes, changing the wave height to the north-east of the development by up to 0.5 m for up to 300 m away from the structure. However, the development does not change the wave heights significantly beyond 500 m from the development.

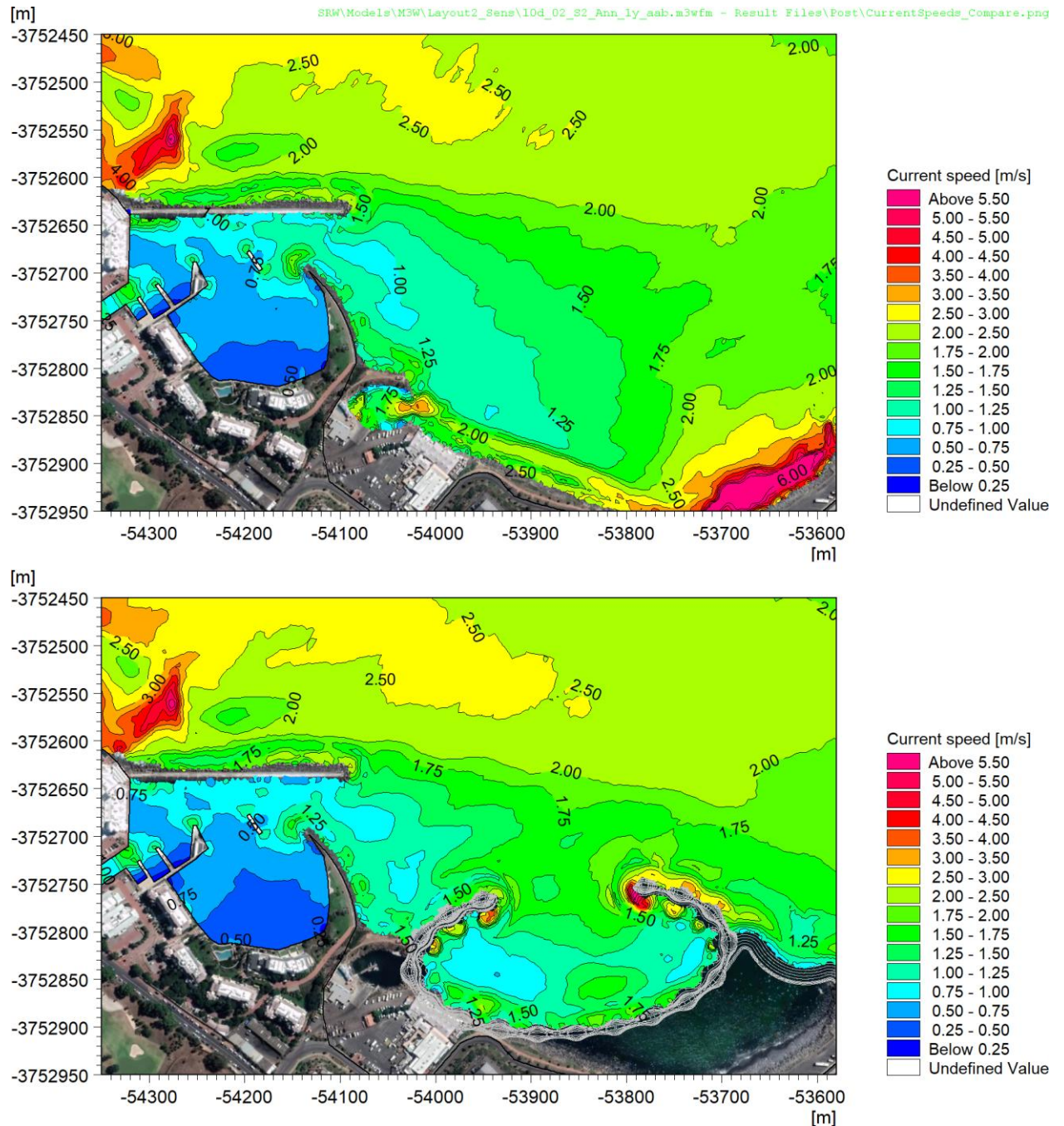
4.3.3 Changes in wave-induced currents

In addition to wave heights, changes in the wave-induced currents within the Water Club Marina have the potential to affect forces on moored vessels. Given the relatively shallow depths inside the marina and typical wave periods modelled, the waves within the Water Club Marina typically respond as shallow water waves. Therefore, a change in the wave-induced depth-averaged current speeds along with wave heights provide an indication on changes in forcings on the vessels.

Figure 4-16 to Figure 4-21 present the 99th percentile depth-averaged wave-induced current speed inside the development for the 1-month, 1-year and 475-year cases.







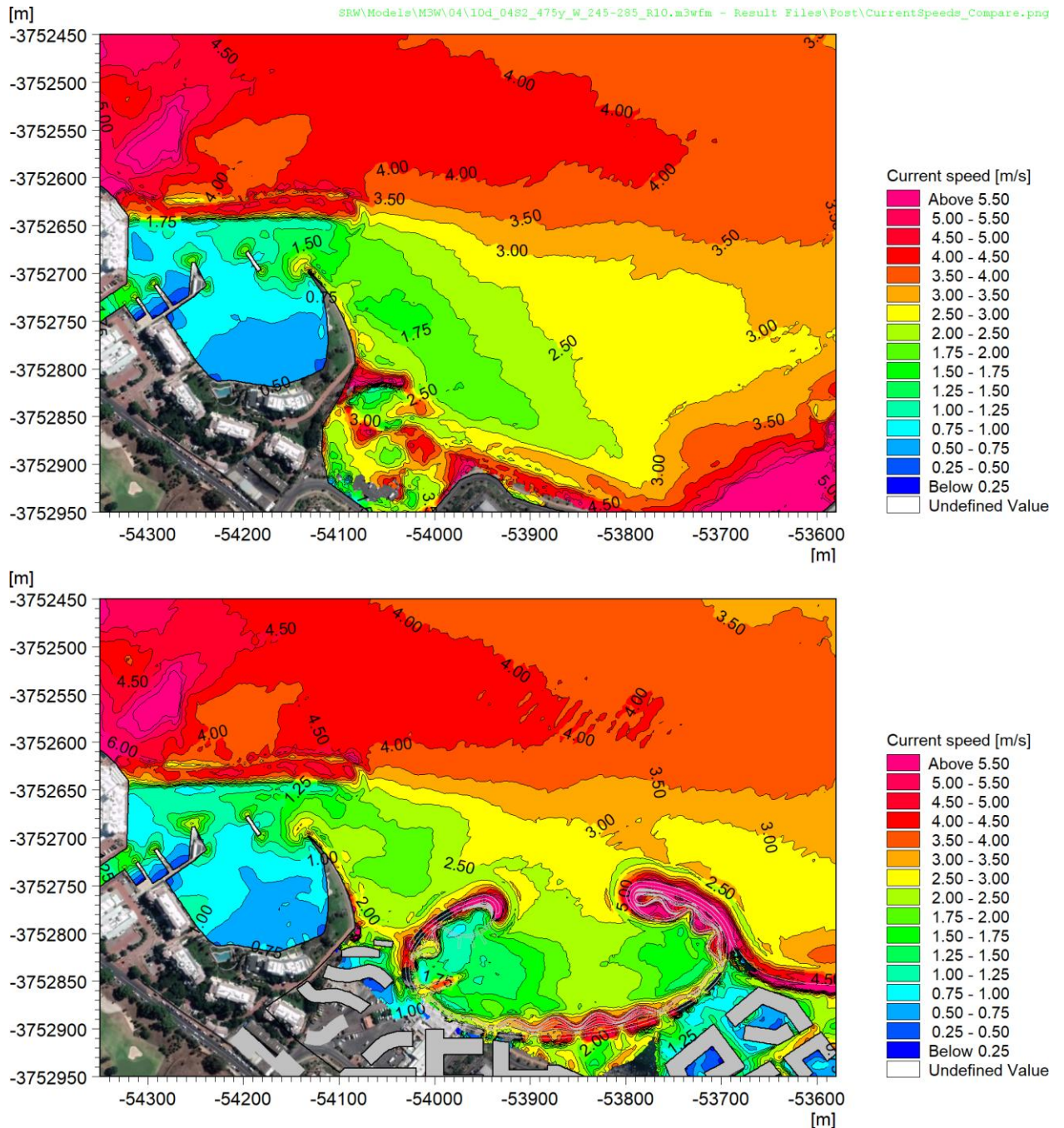


Figure 4-19: 99th percentile depth-averaged current speed for baseline (top) and development (bottom) layouts for the 475-year return period wave from 282.5 degrees.

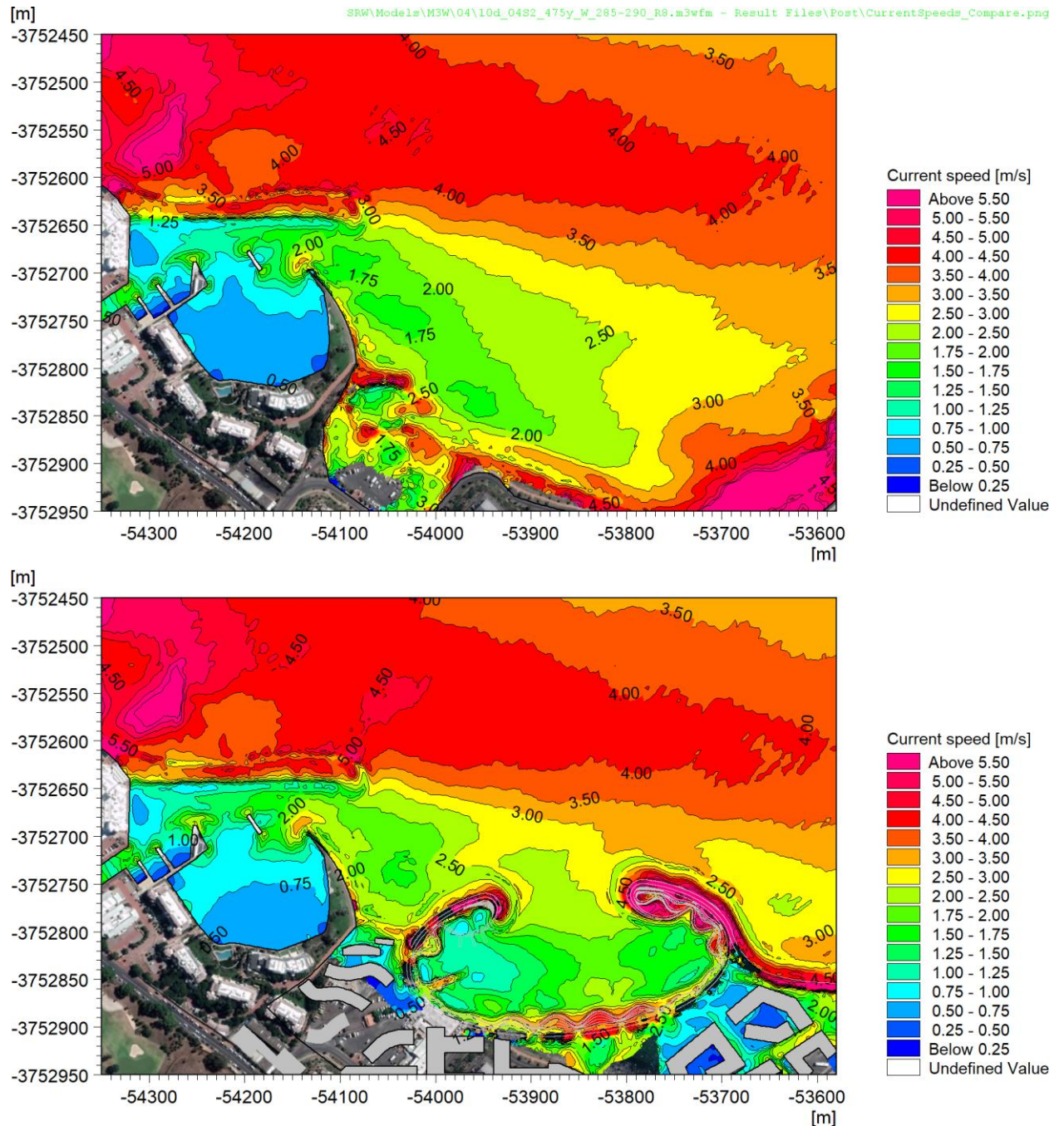


Figure 4-20: 99th percentile depth-averaged current speed for baseline (top) and development (bottom) layouts for the 475-year return period wave from 287.5 degrees.

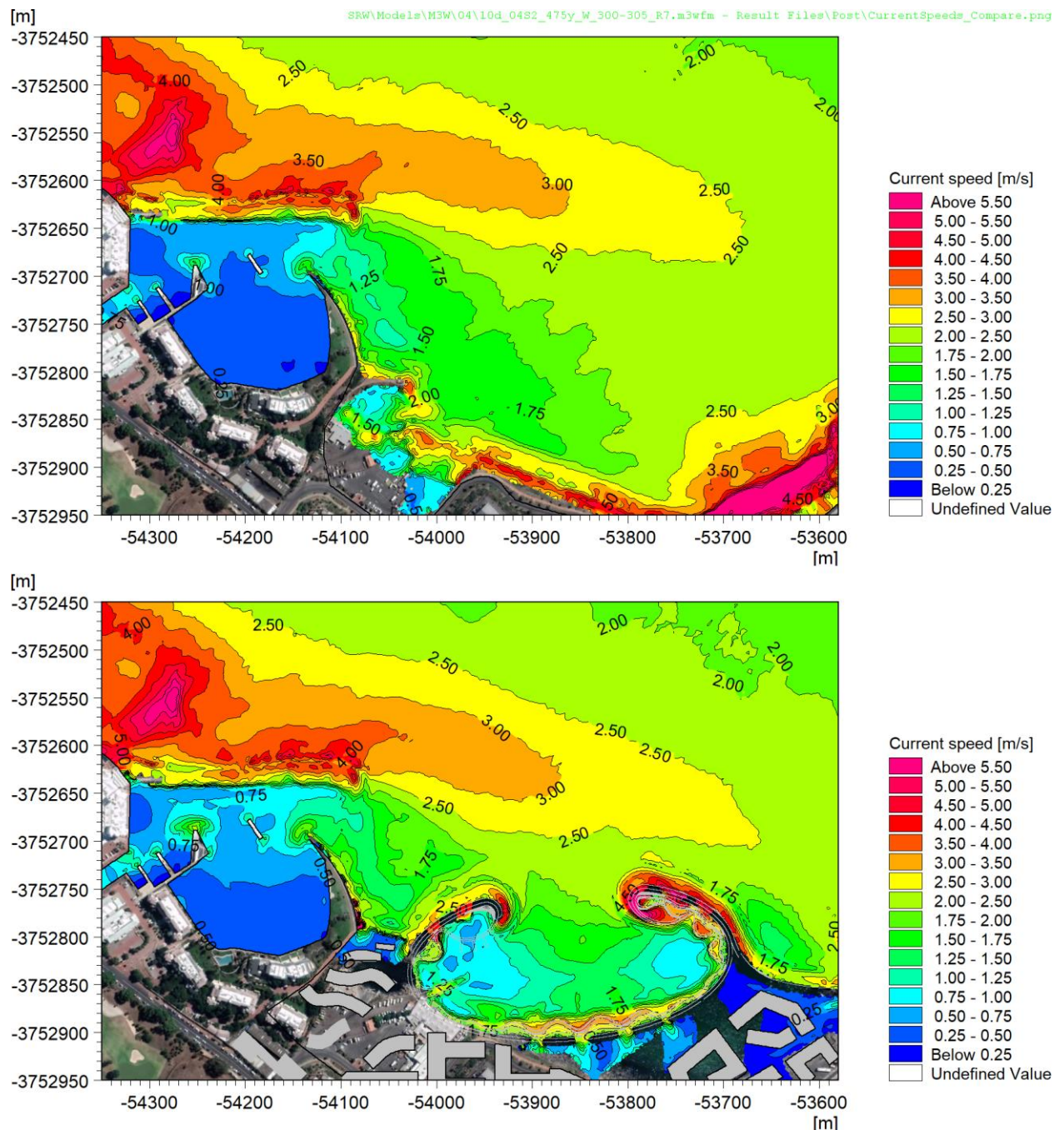


Figure 4-21: 99th percentile depth-averaged current speed for baseline (top) and development (bottom) layouts for the 475-year return period wave from 302.5 degrees.

The changes in the current speeds are not significant for the 1-month wave in summer with a slight reduction for the 1-month in winter and the 1-year cases. For the 475-year conditions there is slight increase in the depth-averaged wave-induced current speeds observed over the whole Water Club Marina, with a few spots of larger current speed increase. These mainly occur on the corners of the hard structures due to local flow acceleration around these structures. Generally, the areas predicted to experience high wave-induced flow velocities are similar between the baseline and development cases. Therefore, the results do not highlight any new areas at risk of high current-induced loading on vessel moorings during extreme storms, and the overall observed changes in wave-induced current speeds are generally small.

4.3.4 Sensitivity to simulation duration

As all wave cases presented here were modelled for a duration of one hour, a sensitivity test to simulation duration was conducted to confirm that the simulation time was sufficient to adequately characterize the heights of long-period waves, which are typically defined as waves with periods between 25 and 300 seconds. This consisted of running the 1-year return period case both with and without the development for three hours and for one hour. The model results were post-processed to separate the long and short wave heights in the Water Club Marina. Figure 4-22 presents the long wave height (H_{m0}) with the development for both a one-hour and three-hour run duration.

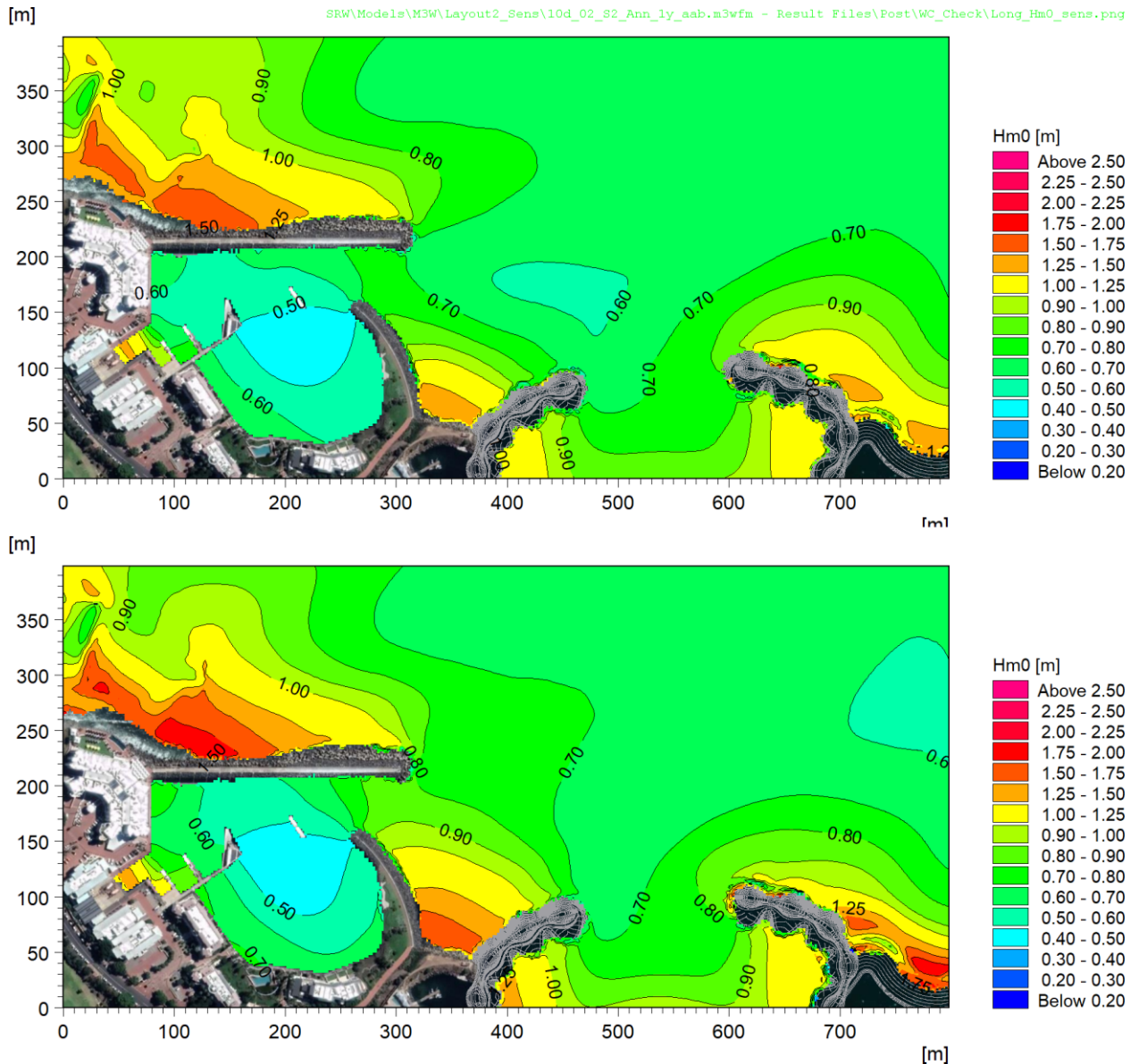


Figure 4-22: Long wave H_{m0} for the development layout for a 1-hour runtime (top) and a 3-hour runtime (bottom) for the 1-year case.

The sensitivity test showed that the use of a one-hour duration had a minor impact (compared to the three-hour runtime) on the significant wave height (H_{m0}) for the long waves. When averaged over the marina basin, the baseline layout showed an average reduction in long wave height of 0.02 m (3.8%) for the three-hour duration while the development layout showed an increase of 0.02 m (3.8%) for the three-hour duration. This



result indicates that the one-hour run duration is sufficient to characterize the significant wave heights (H_{m0}) of the long waves in the Water Club Marina.

The robustness of the results presented in this study is further supported by the fact that an identical wave train was modelled for both with and without the development. This ensures that the same set of waves are compared making the differences only a function of the layout and not of the wave train modelled.

4.4 Accumulation of mud

The existing seabed in Granger Bay is generally sandy because during large wave events any mud that is deposited during calm periods is resuspended, but it is possible that the new development will reduce the waves sufficiently to reduce the resuspension of mud and thus increase the percentage of mud on the seabed, with associated changes in marine ecology.

As described in Section 3.2.4, the wave model results have been processed to obtain the wave-induced bed shear stresses. The results for the three wave storm cases considered are presented in Figure 4-23 to Figure 4-25. Areas where the bed shear stress exceeds 0.2 N/m^2 during storm events will tend to be sandy and are indicated in shades of blue, whilst areas where the bed shear stress remains below 0.2 N/m^2 will tend to be muddy and are indicated in shaded of brown.

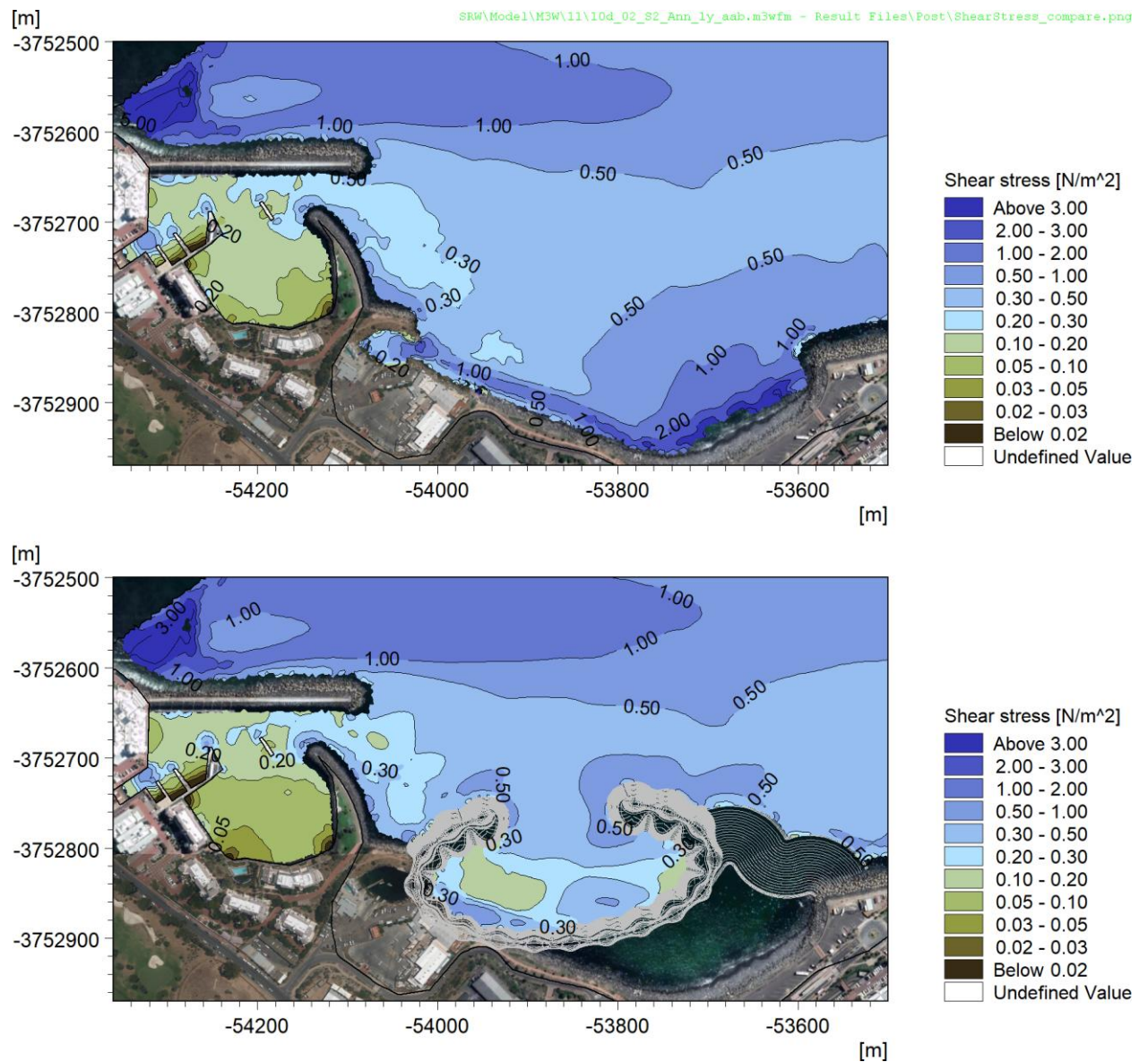


Figure 4-23: Wave-induced bed shear stress for baseline (top) and development (bottom) layouts for the 1-year return period storm event.

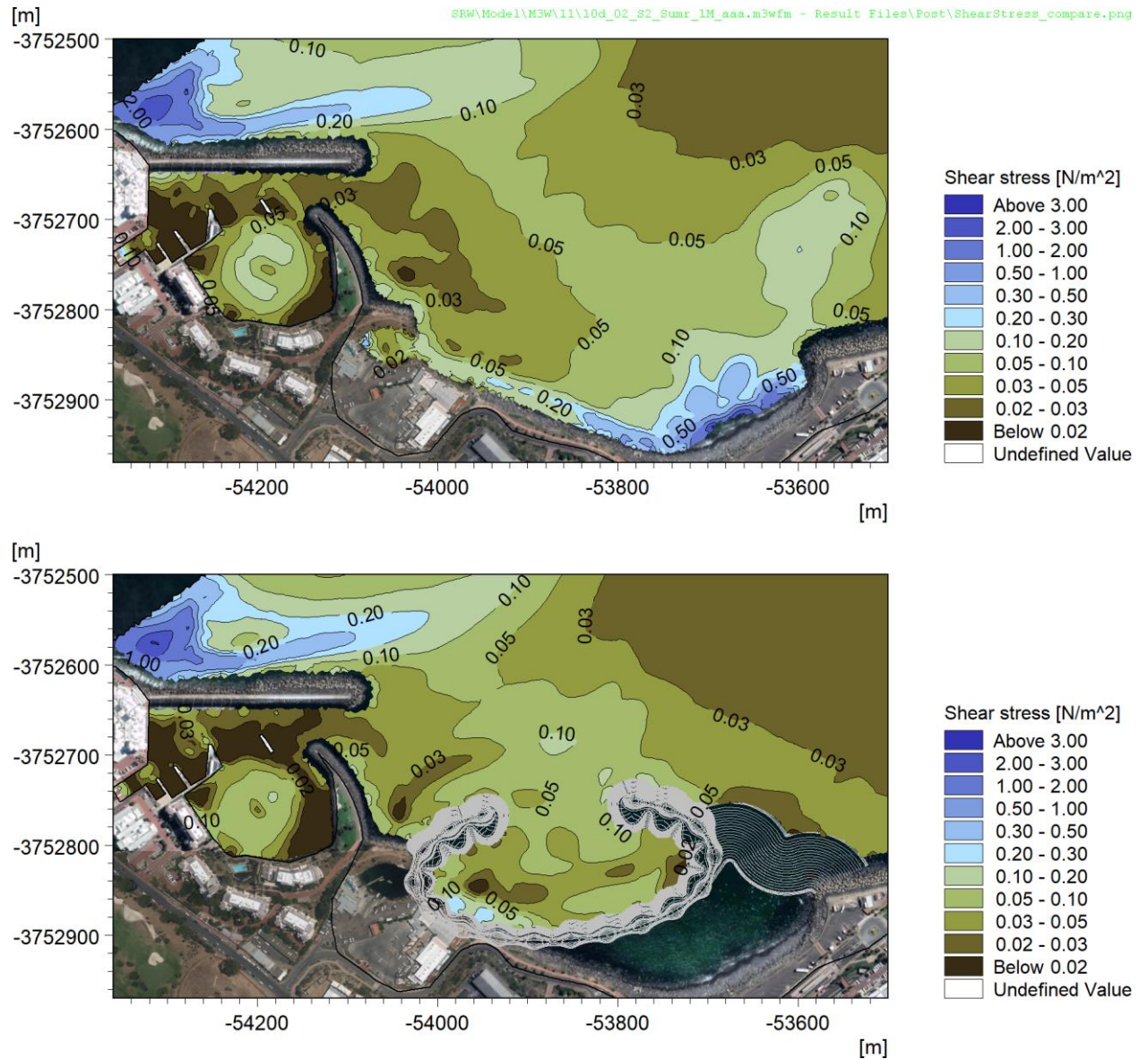


Figure 4-24: Wave-induced bed shear stress for baseline (top) and development (bottom) layouts for the 1-month return period storm event in summer.

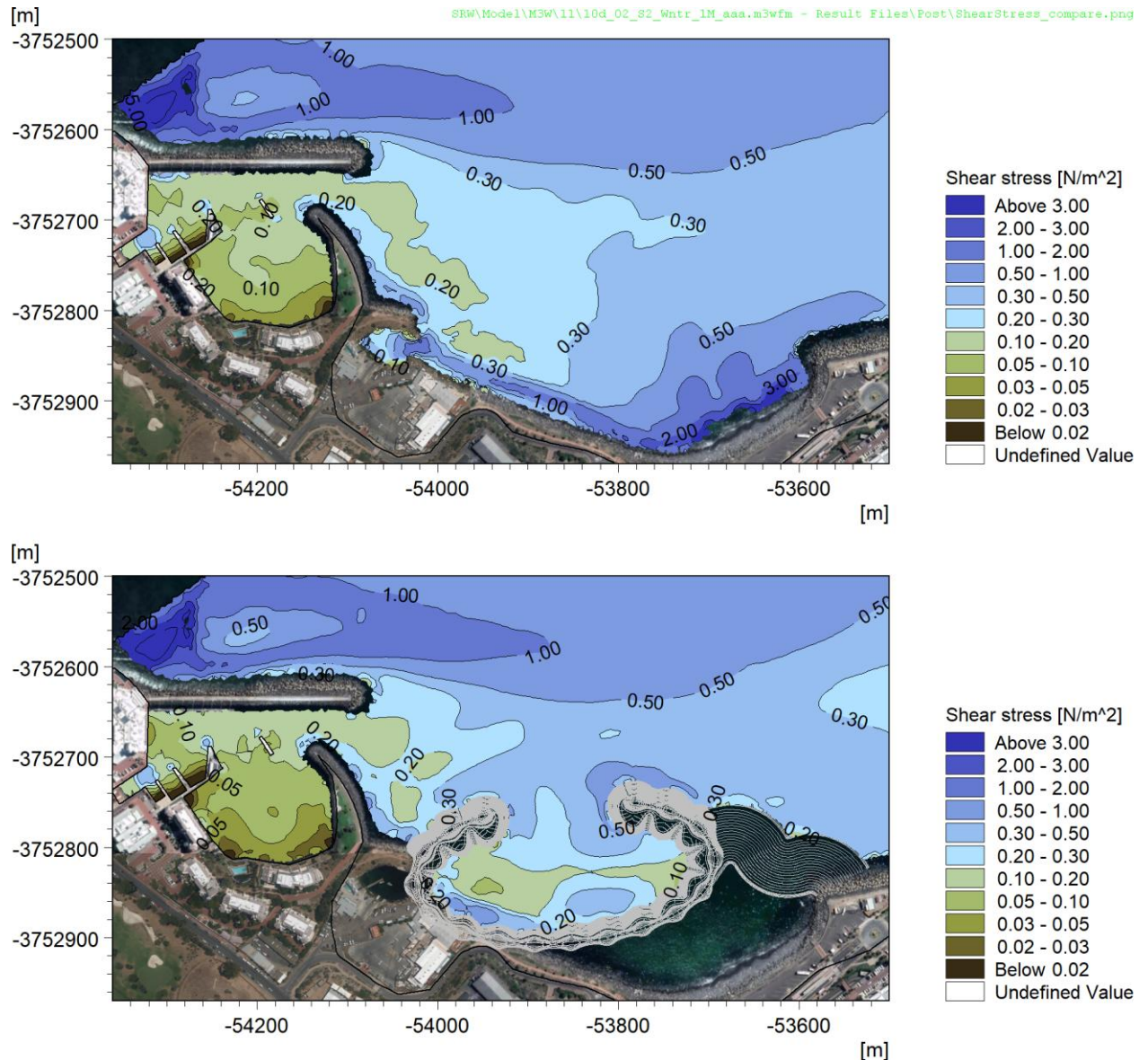


Figure 4-25: Wave-induced bed shear stress for baseline (top) and development (bottom) layouts for the 1-month return period storm event in winter.

The figures above show that:

- For the baseline case, mud will accumulate on the seabed in the area of the development during summer, but this will mostly be resuspended in winter and completely resuspended once a year, thus no longer-term accumulation of mud will occur on the seabed.
- For the development case, mud will accumulate on the seabed within the development during summer, with approximately half of the development seabed area remaining muddy after winter and two small areas remaining muddy after one year. There is thus some risk of longer-term mud accumulation within the development.
- For the baseline case, longer-term mud accumulation is predicted within the entire inner area of the Water Club Marina, except for the entrance and some localised areas of scour induced by strong currents due to harbour resonance. This result is consistent with bathymetric surveys of the Water Club Marina (see Figure 3-22) and historical dredge records, which indicate that periodic maintenance dredging of the Water Club Marina has been required.



- The development does not result in significant changes in mud accumulation beyond 300 m north of the development. The development has no significant impact on the bed shear stresses in the Water Club Marina, thus no additional mud accumulation and no additional volume of maintenance dredging is predicted for the Water Club Marina.



5. SUMMARY

The study used the existing situation as a baseline and compared the changes in currents, waves, sediment resuspension and temperature to assess impacts of the new development. The model results indicate the following:

- No significant longshore sediment transport currently exists at the site and therefore the development will not change the longshore transport regime in Table Bay.
- The addition of the development reduces the maximum wind-driven current speed inside the development from 0.06 m/s in summer and 0.04 m/s in winter to 0.02 m/s.
- The maximum wind-driven current speeds inside the new development are higher (0.02 m/s) than those inside the Water Club Marina (0.01 m/s).
- The residual circulation inside the new development comprises twin eddies which will induce flushing of water out of the development. This circulation should also assist in limiting floating pollutants remaining in the bay.
- The residual circulation is stronger in summer/autumn due to the strong south-east winds.
- The residual circulation within the development is slightly weaker than the baseline circulation, but stronger than the circulation within the Water Club Marina.
- The maximum surface seawater temperature inside the development area increases slightly (by about 0.25°C) in the winter/spring case, but remains the same in summer/autumn.
- The weaker residual circulation within the Water Club Marina results in a build-up in seawater temperature inside the Water Club Marina, particularly in summer.
- The wave height inside the proposed development is generally reduced compared to the baseline layout, however there is significant amplification in the centre of the development due to harbour resonance.
- The wave heights inside the development are significantly larger than those inside the Water Club Marina.
- Within the Water Club Marina, for the 1-month summer and winter cases, as well as the 1-year return period case, there is a general reduction of 0.03 m in the total significant wave height within the Water Club Marina with the addition of the proposed development. For the extreme design cases (475-year return period), there is a slight increase in total wave height within the Water Club Marina with the addition of the proposed development. This corresponds to an increase in the total wave height of approximately 0.12 m for the largest case.
- For the Water Club Marina main breakwater the maximum increase in wave height on its lee side (approximately normal to the breakwater) of approximately two percent was found. This is not considered significant in terms of breakwater stability.
- For the Water Club Marina lee breakwater the maximum increase in wave height (approximately parallel to the breakwater) of approximately seventeen percent was found. It is expected that this increase in wave height (approximately parallel to the breakwater) will have less of an implication for breakwater stability compared to waves normal to the structure.
- Wave-induced current speeds (depth-averaged) were used to assess the impact on vessels and moorings within the Water Club Marina. The changes in the current speeds are not significant for the 1-month wave in summer with a slight reduction for the 1-month in winter and the 1-year cases. For the extreme conditions there is slight increase in the depth-averaged wave driven current speeds observed over the whole marina, with a few spots of larger current speed increase. The largest



increases in current speed occur on the corners of the hard structures due to accelerations around these structures. The results do not highlight any new areas at risk of high current-induced loading on vessel moorings during extreme storms, and the overall observed changes in wave-induced current speeds are generally small.

- For the baseline case, mud will accumulate on the seabed in the area of the development during summer, but this will mostly be resuspended in winter and completely resuspended once a year, thus no longer-term accumulation of mud will occur on the seabed.
- For the development case, mud will accumulate on the seabed within the development during summer, with approximately half of the development seabed area remaining muddy after winter and two small areas remaining muddy after one year. There is thus some risk of longer-term mud accumulation within the development.
- For the baseline case, longer-term mud accumulation is predicted within the entire inner area of the Water Club Marina, except for the entrance and some localised areas of scour induced by strong currents due to harbour resonance. This result is consistent with bathymetric surveys of the Water Club Marina and historical dredge records, which indicate that periodic maintenance dredging of the Water Club Marina has been required.
- The development does not result in significant changes in mud accumulation beyond 300 m north of the development. The development has no significant impact on the bed shear stresses in the Water Club Marina, thus no additional mud accumulation and no additional volume of maintenance dredging is predicted for the Water Club Marina.
- The development does not result in significant changes to currents or temperature beyond 300 m of the development, and beyond 500 m in the case of waves.

These model results will need to be interpreted by the marine ecologists to assess the impact on the marine ecology.



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