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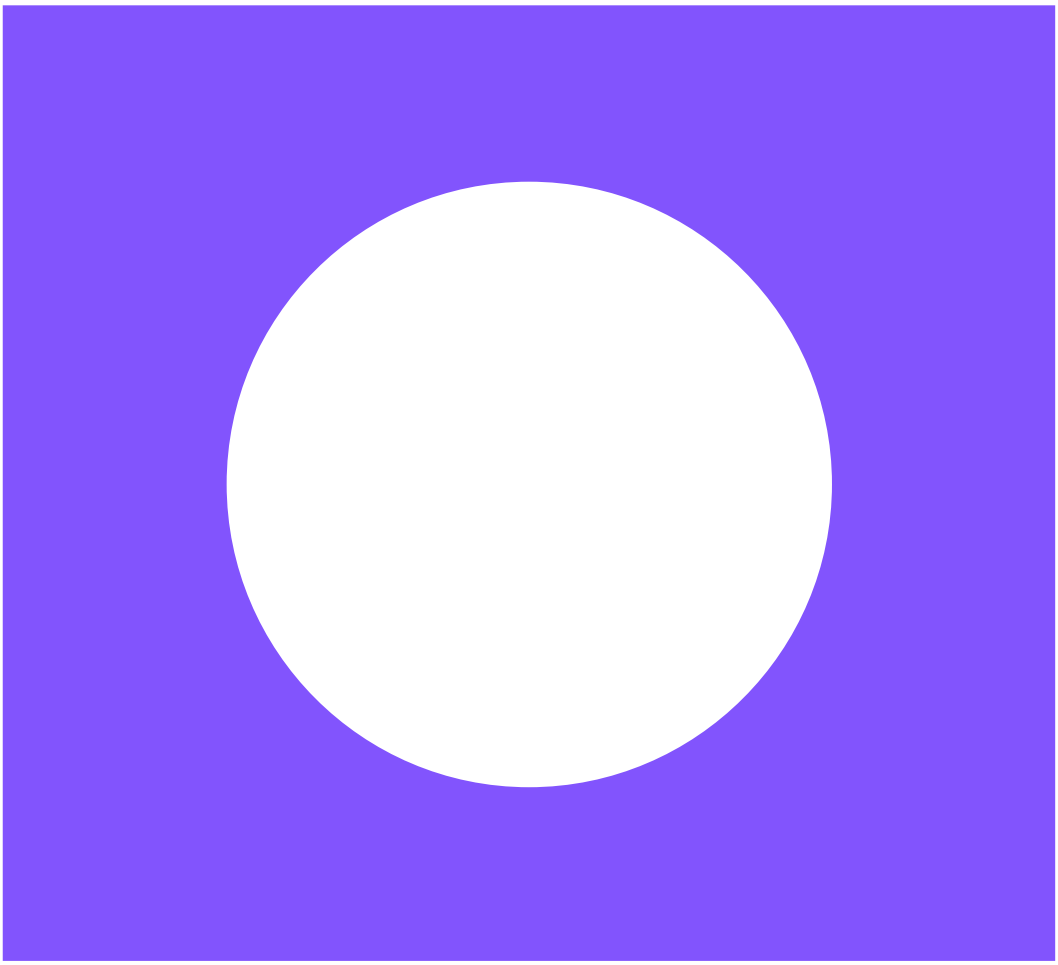

Orkney Islands CCAP

Kirkwall Modelling and Overtopping Assessment Studies

October 2026

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Orkney Islands CCAP

Kirkwall Modelling and Overtopping Assessment Studies

October 2026

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Glossary

1D	One-dimensional
2D	Two-dimensional
AEP	Annual Exceedance Probability
CD	Chart Datum
Cefas	Centre for Environment Fisheries and Aquaculture Science
CFB	Coastal Flood Boundary
DD	Delft Dashboard
DHI	Danish Hydraulic Institute
DSD	Directional Standard Deviation
Deg. N	Degrees from North
EA	Environment Agency
ECMWF	European Centre for Medium-Range Weather Forecast
EWL	Extreme Water Level
EVA	Extreme Value Analysis
FM	Flexible Mesh
HD	Hydrodynamic
H_s	Significant wave height
JPA	Joint Probability Analysis
km	Kilometres
m	Metre(s)
MM	Mott MacDonald
MSL	Mean Sea Level
MWD	Mean Wave Direction
N	North
ODN	UK Ordnance Datum Newlyn
OIC	Orkney Islands Council
OFM	Local Overland (hydrodynamic) Flow Model
PSGA	Public Sector Geospatial Agreement
RCP	Representative Concentration Pathway
RP	Return Period

SEPA	Scottish Environment Protection Agency
SLR	Sea-Level Rise
SOP	Standard of Protection
SW	Spectral Wave
T_p	Peak wave period
UKCP	UK Climate Projections
UKHO	United Kingdom Hydrographic Office

Executive summary

This report presents the development of hydrodynamic and spectral wave models, together with a detailed wave overtopping assessment, for a section of the Kirkwall (Orkney Islands) frontage to evaluate flooding under the baseline (2025) and future 100-year epoch (2125) conditions. An additional 45-year (2070) epoch was also analysed in the wave overtopping assessment. The assessment covers the 1-in-50, 1-in-200, and 1-in-1,000-year Return Period events, corresponding to Annual Exceedance Probabilities (AEPs) of 2%, 0.5%, and 0.1%, respectively. To support this assessment, regional and local hydrodynamic and wave models were established, accompanied by analyses of extreme water levels, wave conditions, and associated overtopping processes. A 2100 1-in-200-year event was also assessed to align with the planning horizon adopted by current SEPA flood modelling for comparison.

The approach taken can be summarised as follows:

- Suitable bathymetric, water level, current, wind, pressure and wave data were identified, quality assured and used to build the hydrodynamic and wave models.
- A regional MIKE21 FM Hydrodynamic (HD) model was successfully calibrated and validated using measured water levels and tidal diamond data. The model showed strong agreement with both datasets.
- The regional MIKE21 FM HD model was successfully further validated against a historical extreme water level at Kirkwall and Stromness in January 2005, demonstrating that the developed models are robust and suitably reliable for long-term simulations to analyse extreme water levels at Kirkwall.
- An extreme water level analysis showed that extreme water levels from publicly available Coastal Flood Boundary (CFB) data are underestimated when compared with the analysed extreme water levels adopted for this study.
- A regional MIKE21 FM Spectral Wave (SW) model was calibrated and validated using offshore wave buoy measurements. Model comparisons showed good agreement against measured offshore wave data, confirming that the model configuration is suitable for extended hindcast simulations and for deriving extreme wave conditions at Kirkwall. Local wave measurements were not available, and it is therefore recommended that for any future detailed studies, that this information is collected to help validate the wave modelling outcomes.
- Modelled extreme water level, wave and wind conditions were analysed and presented for the 1-in-50, 1-in-200 and 1-in-1,000-year return period events for the baseline (2025), mid-epoch (2070), and future (2125) epoch.
- Extreme water level and wave conditions were transformed to the toe of the Kirkwall coastal defence structures for use in subsequent wave overtopping assessments.
- For the 2100 epoch, additional joint probability analysis and wave transformation simulations were not undertaken. Instead, the required overtopping inputs for the 1-in-200-year return period event in 2100 were derived from the existing modelled datasets by
  - Selecting the 2070 joint-probability conditions as the most representative donor dataset, uplifting the associated water levels to 2100
  - Using closely matched transformed wave conditions from the 2125 1-in-200-year return period event.

  This approach made efficient use of existing results to proportionately support comparison with SEPA's 2100 flood mapping, which is a key dataset for development planning in Kirkwall.

- A local MIKE21 FM HD model was calibrated and validated using observed water levels at Kirkwall, including comparisons against the recorded extreme water level during the January 2005 event. The resulting model data agreement across tidal and surge conditions demonstrates that the model setup is appropriate for simulating local overland flooding (due to coastal events).
- The local overland HD flood model simulations were successfully validated against extreme water levels at Kirkwall. Overland flood model simulations were run for the 1-in-50, 1-in-200 and 1-in-1,000-year Return Period events for the baseline (2025) and future (2125) epoch scenarios. For the 2100 epoch, only the 1-in-200-year Return Period event was appraised.

The key findings of the assessment are summarised as follows:

- The climate change scenarios consistently produce higher water levels, larger inundation extents, and greater flow velocities. This indicates the importance of incorporating future climate allowances into flood hazard mapping and risk-based decision making.
- The comparison between the baseline and climate change epochs highlights a significant escalation in flood exposure and reinforces the need for future-proofing flood risk management and coastal defence planning at Kirkwall.
- Under baseline (2025) epoch conditions, there is minimal wave overtopping along the majority of the coastal defences to the west of Kirkwall Marina, with the exception of near the roundabout on Ayre Road.
- From 2070 (the mid-epoch), a significant increase in wave overtopping begins to occur for the 1-in-200-year Return Period event (understood to be the Standard of Protection of the defence).
- In the 2100 and 2125 epochs, the crest of the defence structures experience overflow (due to extreme sea-levels), contributing significantly more discharge to the flood extent than the wave overtopping.

1 Non-technical overview

1.1 Introduction

Orkney Islands Council (OIC) commissioned Mott MacDonald to undertake an overland flow modelling and wave overtopping assessment for the Kirkwall frontage. The aim of the modelling discussed in this report is to gain an understanding of the present day (2025) and future (2100 and 2125) coastal flood risk to Kirkwall.

1.2 Problem definition

Kirkwall is situated on the northern coast of the Orkney Mainland within the Orkney Islands archipelago and is exposed to hydrodynamic conditions driven by locally generated waves, tidal processes, and storm driven water level fluctuations (storm surges).

Kirkwall has a history of coastal flooding, particularly along its low-lying waterfront and harbour side areas where high water levels combine with wave overtopping action at the existing defences. Past events have demonstrated that the town is highly sensitive to relatively small changes in water levels, particularly when high tides coincide with storm conditions. These factors make the Kirkwall frontage a priority location for flood modelling and wave-overtopping assessments, enabling an appropriate evaluation of present-day flood mechanisms and future impacts due to projected climate change.

As the urban centre of the Orkney archipelago, Kirkwall is a focal point for much development on the islands. It is imperative that such development is resilient to flooding. Existing sources of flood risk information (e.g. the SEPA flood maps) are based on the Coastal Flood Boundary dataset, which is known to have low reliability in Orkney due to the hydrodynamic effects noted above.

1.3 Modelling process

The flood risk to Kirkwall was assessed for seven storm events of varying magnitude and probability. Future sea level rise projections from UKCP18[1] (Met Office, 2019) have been used to estimate future water levels. The storm events are given in Table 1.1.

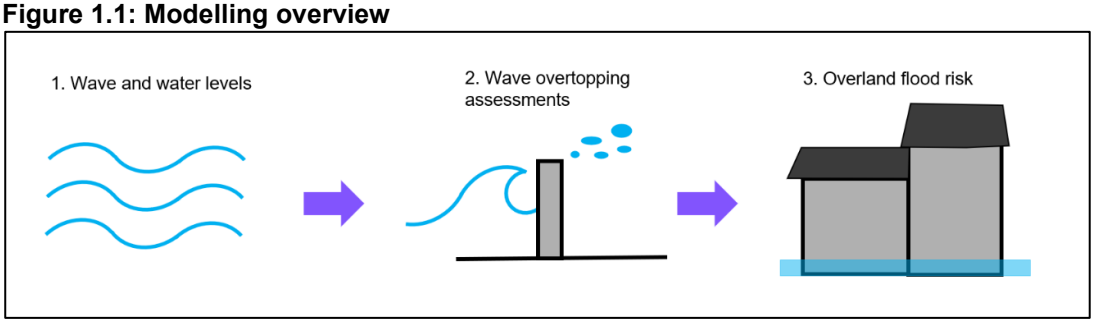
Table 1.1: Storm events

	Storm event	
Year	**Return period (1-in-x years)**	**Annual Exceedance Probability (x% AEP)**
2025	50	2
	200	0.5
	1000	0.1
2100	200	0.5
2125	50	2
	200	0.5
	1000	0.1

The modelling was undertaken in three key stages shown schematically in Figure 1.1.

[1] UK Climate Projections (UKCP18) guidance and reports - Met Office accessed on 6 January 2026

Figure 1.1: Modelling overview



Source: Mott MacDonald, 2026

1. **Waves and water levels** (offshore). The offshore coastal climate across the archipelago was modelled to understand present day and future water level and current speeds/directions. A spectral wave model study was also undertaken to understand extreme and typical wave conditions across the same area.
2. **Wave overtopping assessments.** An analysis was undertaken to calculate the mean flow rate of water into Kirkwall due to wave overtopping during storm events. The assessment was undertaken by routing the nearshore wave climate and water levels to calculate the estimated wave overtopping discharge at various points across the coastal frontage for three different storm events in two different time horizons.
3. **Overland flood risk.** The results of the coastal modelling and the wave overtopping assessments were used as inputs to an overland flow model. The overland flow model predicts the extent and depth of flooding within Kirkwall due to a coastal storm event. The model does not consider rainfall or any contributions from watercourses.

1.4 Results and interpretation

Figure 1.2, Figure 1.3, and Figure 1.4 below provide an overview of the change in flood risk between the different events for the baseline (2025) and future (2100 and 2125) epochs respectively.

1.4.1 Baseline (2025) results

The 2025 flood results suggest that the Kirkwall Harbour walls largely contain the coastal flooding in the present day. During the 1-in-50-year return period event, it is predicted that limited overtopping will occur at the defences around the Ayre roundabout (shown in figure). Coastal flooding is also predicted to impact the Ayre Road Ferry Terminal car park, as well the marina car park, both of which are seaward of the Kirkwall Harbour Flood Prevention Scheme. As the storm event increases in magnitude to a 1-in-200 year return period event, further flooding is experienced at the shorefront parking off Grainshore Road (shown in figure). As the storm magnitudes increase further the spatial extent of the flooding remains similar, however the depths increase. Under present day conditions, the flood water is largely contained by the sea walls. Where overtopping occurs, it impacts areas with few permanent built assets. It must be considered that the harbour walls performance as a flood defence is largely reliant on the effectiveness of the demountable barriers, as well as the flood resilience of the buildings along the wall. If the demountable barriers and flood resilience of the buildings are compromised, representative flood extents are much larger. All flood extent maps produced for the study are included in Appendix C.

1.4.2 Future (2100 and 2125) results

The results of the 2100 (1-in-200-year event only) and 2125 epoch analysis suggest widespread flooding to Kirkwall during all storm events assessed.

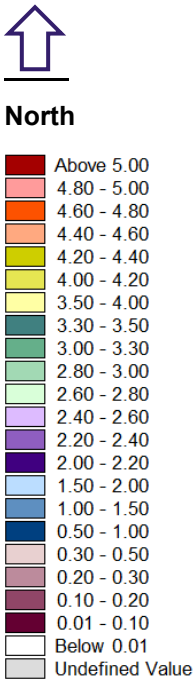
During the 2100 1-in-200-year return period event, shown in Figure 1.4, flooding of approximately 0.5 – 1.5 m depth occurs in central Kirkwall.

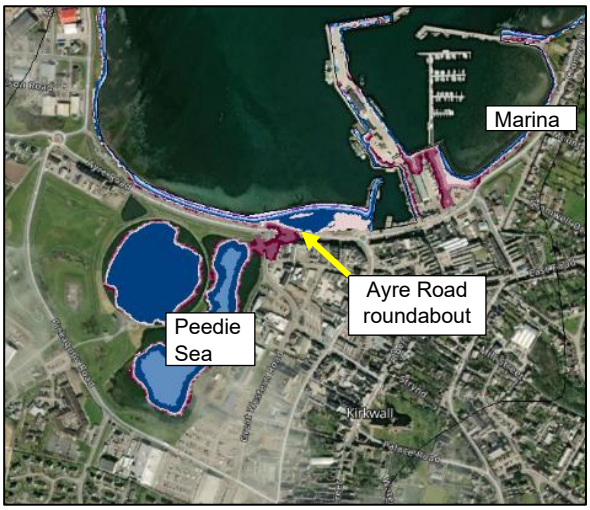
Flooding in the 2125 epoch shows slightly increased flood extents and increased flood depth when compared to the 2100 epoch. In the 1-in-50 event in 2125, there is widespread flooding of approximately 1 m in depth. The worst of the flooding is along Junction Road where depths are predicted to be in the range of 2.0-2.6 m. As the storm events increase in magnitude, the overall extent of the flooding remains the same, however, the depth of the flooding increases. The main contribution to the widespread flooding is due to water levels exceeding the crest levels of the harbour wall and resulting in an overflow event. In this scenario, sea water freely overflows the defence. The change in flood risk between 2025 and 2125 is due to the estimated sea level rise over the next 100 years. The increase in water levels results in relatively frequent surge events overflowing the harbour walls, leading to significant inundation of Kirkwall.

1.4.3 Kirkwall Harbour Flood Prevention Scheme standard of protection

The results of the study suggest that the existing Kirkwall harbour wall is an effective flood defence scheme in present day, however that effectiveness reduces significantly by 2100, with flood depths increasing further up to 2125. It is important to understand when in the future it is likely that the standard of protection of the walls will be compromised. To determine this, modelling was initially undertaken to determine when extreme water levels exceed the flood defence level, and then to determine when step changes in (wave) overtopping may occur. Increased overtopping or overflowing of the flood defences could result in increased inundation of the defended area, potentially flooding properties. Current sea level rise projections indicate that the level of the existing flood prevention scheme will be compromised from approximately 2070 in the 1-in-50-year return period storm. Due to the sea levels exceeding the harbour walls around 2070, there is a corresponding jump in wave overtopping rates.

Figure 1.2: Present day (2025) overland flood depths





1-in-50 year return period event

Flooding begins at the Ayre roundabout and flows towards the Peedie Sea. The Ayre and Burgh roads are most effected, with water depths up to approximately 0.3m.

Flooding also impacts parking at the marina and ferry terminal.



1-in-200 year return period event

Flood extents at the Ayre roundabout are similar to the 1-in-50 year return period events.

Flood depths and extents increase around ferry terminals and marina.

Flooding experienced at the seafront parking off Grainshore road on the western extent of Kirkwall Harbour.



1-in-1000 year return period event

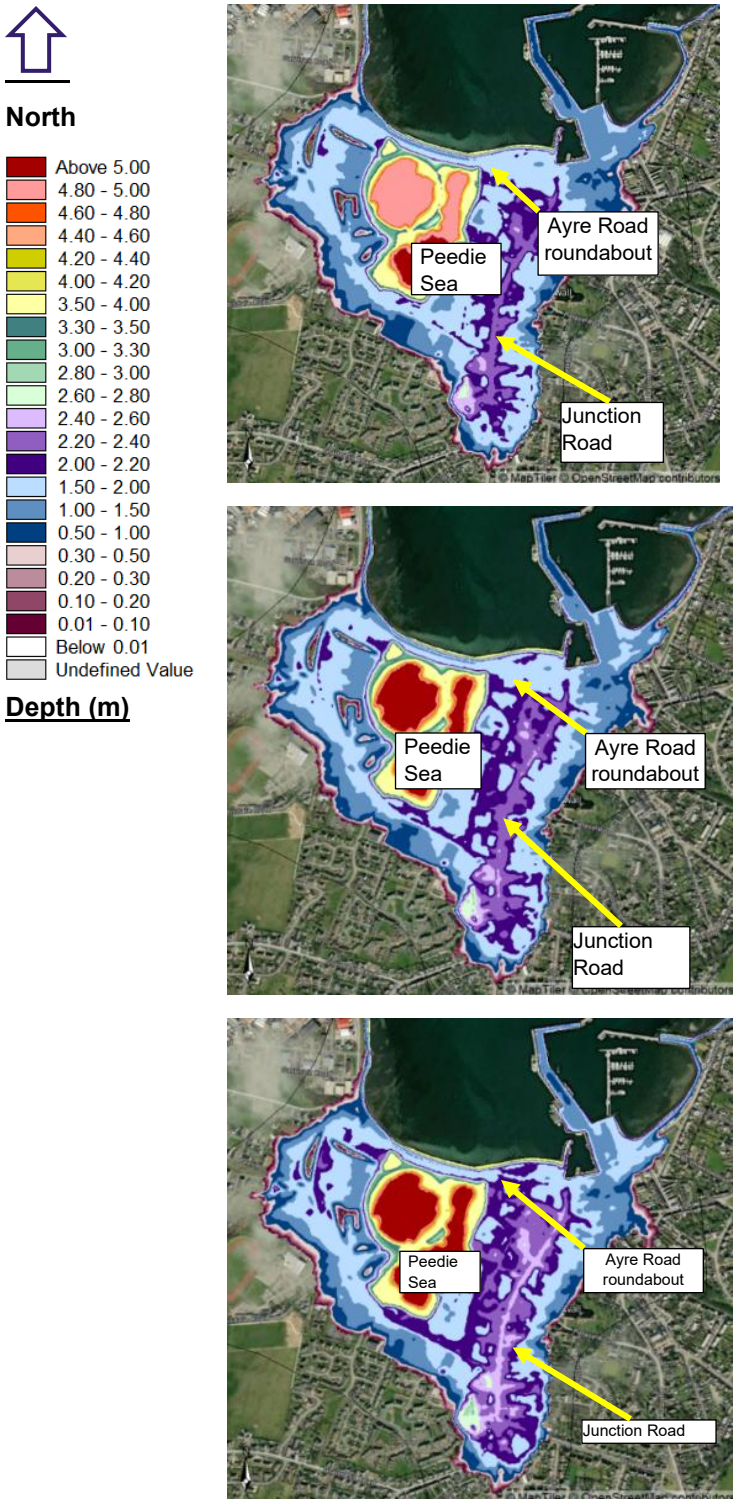
Flood extents at the Ayre roundabout increase but the depth remains approximately 0.3m

Flood depths and extents increase around ferry terminals and marina.

Flood extent increases at the seafront parking off Grainshore road.

Source: Mott MacDonald, 2026.

Figure 1.3: Future (2125) overland flood depths



1-in-50 year return period event

Widespread flooding across Kirkwall is predicted.

Flood depths largely greater than 1m. When discounting the Peedie Sea area, the worst flooding is experienced along Junction Road with flood depths predicted to be greater than 2m.

1-in-200 year return period event

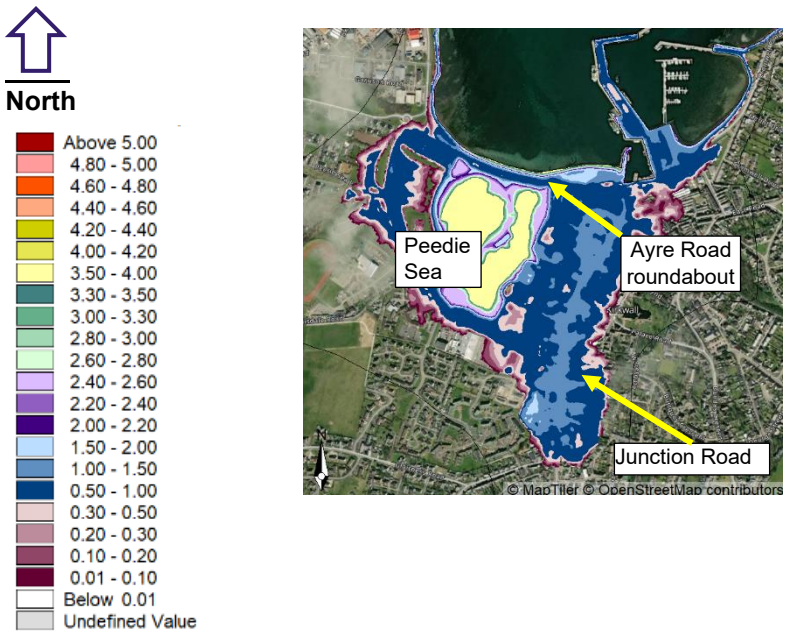
The flood extent is similar to the 1-in-50 year return period event, however the depths have increased.

1-in-1000 year return period event

Similar to the above, the flood extents remain similar, however the depths have increased between the 1-in-200 year event and the 1-in-1000 year return period event.

Source: Mott MacDonald, 2026

Figure 1.4: Future (2100) overland flood depths



Source: Mott MacDonald, 2026

1-in-200 year return period event

The flood extent shows significant inundation of the center of Kirkwall. Compared to the 2125 epoch 1-in-50 year, the 1-in-200 2100 event has slightly lesser extent and depth.

2 Introduction

Orkney Islands Council (OIC) commissioned Mott MacDonald to undertake an overland flow modelling and wave overtopping assessment for the Kirkwall frontage. Kirkwall (Figure 2.1) is situated on the northern coast of the Mainland within the Orkney Islands archipelago and is exposed to hydrodynamic conditions driven by locally generated waves, tidal processes, and storm driven water level fluctuations (storm surges). Kirkwall has a history of coastal flooding, particularly along its low-lying waterfront and harbour side areas where high water levels combine with wave overtopping action at the existing defences. Past events have demonstrated that the town is highly sensitive to relatively small changes in water levels, particularly when high tides coincide with storm conditions. These factors make the Kirkwall frontage a priority location for flood modelling and wave-overtopping assessments, enabling an appropriate evaluation of present-day flood mechanisms and future impacts due to projected climate change.

Figure 2.1: Study location.



Source: Mott MacDonald and Google Earth, 2026

2.1 Report structure

The report is structured as follows:

- Section 3 outlines the methodology adopted in the study;
- Section 4 summarises the datasets and sources of information used;
- Section 5 describes the setup and performance of the regional hydrodynamic model;
- Section 6 presents the development of the regional spectral wave model;
- Section 7 provides the extreme-value analyses derived from the regional model outputs;
- Section 8 introduces the local wave model used for the overtopping assessments;

- Section 9 presents the overtopping assessment methodology and results;
- Section 10 describes the configuration of the local hydrodynamic model;
- Section 11 presents the local overland flow model; and
- Section 12 summarises the overall findings of the report.

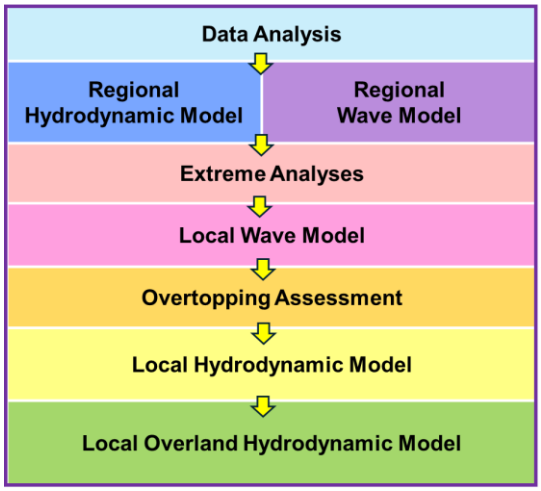
3 Methodology

A clear understanding of wave overtopping and the resulting flood extent is essential for the flooding assessment at Kirkwall. This requires a robust, high-resolution, and well calibrated numerical modelling framework capable of capturing the spatial and temporal variability of hydrodynamic and wave processes across the Orkney Islands, with focused refinement along the Kirkwall frontage to support the overtopping assessment. An overview of the methodology adopted for this study is presented in Figure 3.1.

The two-dimensional (2D) MIKE Flexible Mesh (FM) Hydrodynamic (HD) and Spectral Wave (SW) models, together with a one-dimensional (1D) wave transformation model, were developed for this study. The regional hydrodynamic model is described in Section 5, while the regional and local wave modelling approaches, including all relevant technical details, are presented in Sections 6 and 8, respectively. A sensitivity test was undertaken to assess the effects of fully coupling the hydrodynamic and wave models. However, the computational demand associated with the coupled configuration was significantly higher than required for the purposes of this study and therefore, the models were run separately. This approach did not affect the outcomes, as wave–current interaction is minimal at the regional scale, and both the hydrodynamic and wave models were calibrated against measured data where available.

The study also includes an Extreme Value Analysis (EVA) and a Joint Probability Analysis (JPA) of water levels and waves, based on the 46-year (1979–2024) outputs from the calibrated and validated regional models; these analyses are outlined in Section 7. The overtopping assessment is presented in Section 9, with overtopping inputs derived from the local wave model results (Section 8). A high-resolution local hydrodynamic model is described in Section 10, and the overland flow modelling, which assesses inundation under Baseline (2025) conditions, the 100-year epoch (2125) for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods (AEPs of 2%, 0.5%, and 0.1%, respectively), and the 2100 epoch for the 1-in-200-year return period (0.5% AEP) only, is detailed in Section 11. A separate JPA and additional wave transformation simulations were not undertaken for 2100, The 2100 overtopping inputs were derived from modelled wave conditions for other events and epochs using the proportionate approach described in Section 9.6;.

Figure 3.1: Study methodology.



Source: Mott MacDonald 2026

4 Data

The accuracy of any numerical model is highly dependent on the quality, resolution, and representativeness of the data used during model development, calibration, and validation. In hydrodynamic and wave modelling, the performance of the computational outcomes is particularly sensitive to the fidelity of bathymetric and topographic inputs, as well as the appropriateness and stability of the applied boundary conditions.

This study integrates a suite of quality-assured open-source datasets alongside site-specific measurements, as detailed below. All available data have been rigorously processed to ensure that boundary forcing, and domain conditions represent natural system dynamics as accurately as possible within the limitations imposed by the data availability for this study.

4.1 Coordinates and datums

Geographical data in the MIKE21 FM models use the British National Grid (OSGB36) horizontal datum. The vertical datum in the regional and local models is referenced to mean sea level and UK ordnance datum Newlyn (ODN), respectively.

4.2 Bathymetry

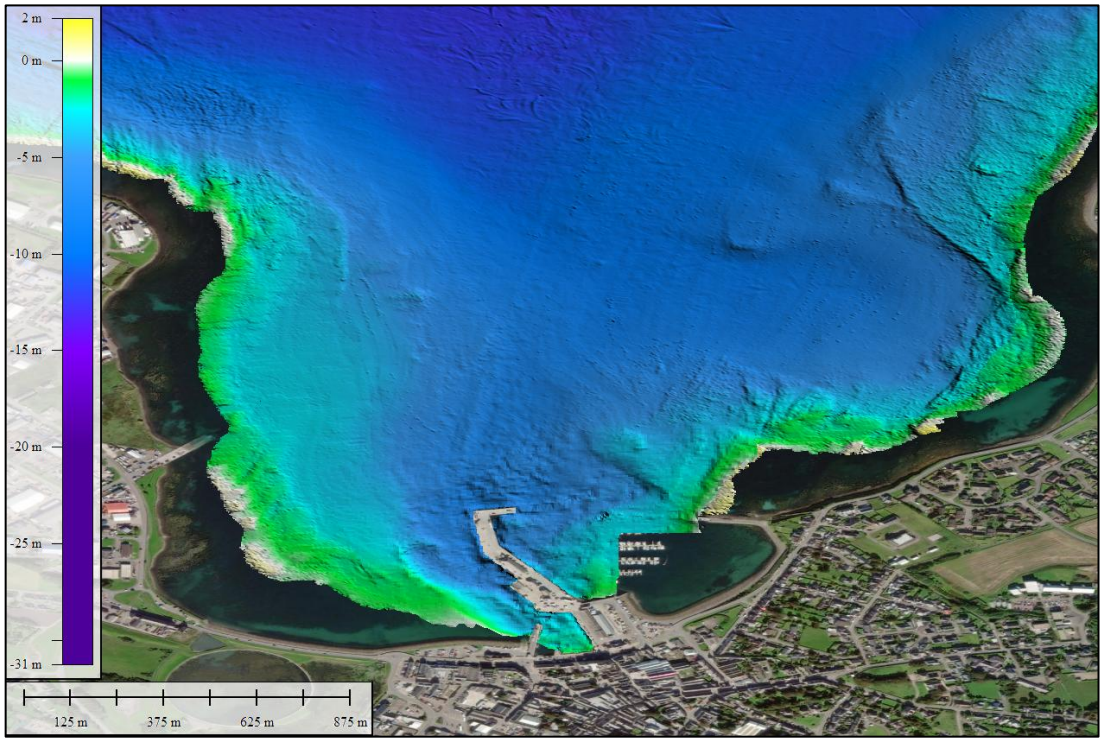
Bathymetric, intertidal and topographic datasets for the site and surrounding areas were sourced from both commercial providers and open-data platforms. Accurate bathymetry is fundamental to marine and coastal modelling, and high-quality datasets were selected to support the model development. The bathymetric information used across all models in this study is summarised in Table 4.1. Figure 4.1 presents the bathymetry for Kirkwall derived from UKHO data and the 2023 Triscom Ltd survey.

Table 4.1: Bathymetry data sources summary.

Source	Original Coordinates & Vertical Datum	Original Resolution	Remarks
UKHO[2] data	Lat, Long (WGS84) m CD	Variable and generally very fine (0.5 to 2 m)	Extremely useful measured data covering large areas around the islands & offshore
2022-2024 Geosight Land and Hydrographic Surveys and Triscom Ltd	UTM Zone 30N (ETRS89) m CD	Variable and generally very fine (0.5 to 2 m)	Very good resolution data for Kirkwall
2023 LiDAR - DTM	BNG - OSGB36 m ODN	0.5 m	Very good data for inland topography. Intertidal areas cropped to remove water reading.
2024 EMODnet[3]	Long Lat (WGS84) MSL	60 x 120 m	Only used for offshore areas
2025 GEBCO[4]	Long Lat (WGS84) MSL	300 x 300 m	Only used for offshore areas
CMAP	Long Lat (WGS84) MSL	Variable and generally coarse	Only used for offshore areas

Source: Mott MacDonald 2026

Figure 4.1: Kirkwall bathymetry data



Source: Mott MacDonald 2026. Contains UKHO and Triscom, 2023 data

[2] Seabed Mapping (admiralty.co.uk) accessed on 12 January 2026.

[3] EMODnet Map Viewer (europa.eu) accessed on 12 January 2026.

[4] Gridded Bathymetry Data | GEBCO accessed on 12 January 2026.

4.3 Water level

Water level data provide the boundary forcings for both the hydrodynamic and wave models. In addition, measured water-level time series were used in model calibration and validation. The water-level datasets applied in this study are summarised as follows:

- Measured water level data at Kirkwall and Stromness (Figure 4.2) supplied by the OIC (2009 to 2024).
- Predicted water levels at Offshore West Shetland (Figure 4.2) were extracted from the Delft Dashboard (DD) database[5] for 46-years (1979 to 2024).
- Predicted water levels at Kirkwall based on the extracted harmonic tidal constituents from the measured data using the MIKE21 Tide prediction toolbox[6].

Table 4.2 shows the tide levels that were extracted from the Hydrographic Office Total Tide[7] at Kirkwall in chart datum (CD), mean sea level (MSL) and Ordnance datum Newlyn (ODN).

Table 4.2: Tide levels (m) at Kirkwall.

	Kirkwall		
Parameter	**CD**	**MSL**	**ODN**
Highest astronomical tide (HAT)	3.5	1.72	2.1
Mean high water spring (MHWS)	3	1.22	1.6
Mean high water neap (MHWN)	2.4	0.62	1
Mean sea level (MSL)	1.78	0	0.38
Mean low water neap (MLWN)	1.3	-0.48	-0.1
Mean low water spring (MLWS)	0.6	-1.18	-0.8
Lowest astronomical tide (LAT)	-0.1	-1.88	-1.5

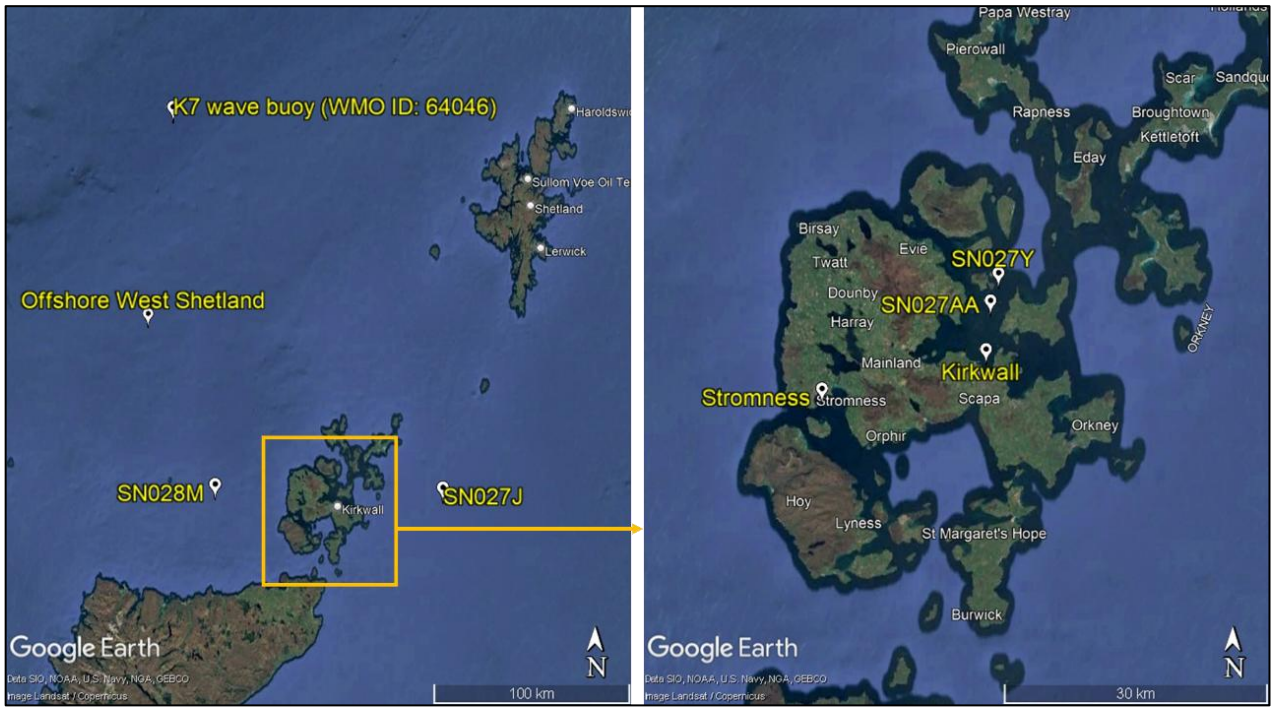
Source: Google Earth and Mott MacDonald 2026

[5] Maarten et al, 2020. Delft Dashboard: a quick set-up tool for hydrodynamic models | Journal of Hydroinformatics | IWA Publishing (iwaponline.com) accessed on 8 January 2026

[6] DHI, "MIKE 21 Tidal Analysis and Prediction Module Scientic Documentation," MIKE 21 - Tidal Analysis and Prediction Module, Horsholm, Denmark, 2017. on 2 January 2026

[7] Hydrographic Office (2026), Admiralty Total Tide accessed on 5 January 2026

Figure 4.2: Water level, current and wave data locations.



Source: Google Earth and Mott MacDonald 2026

4.4 Currents

In the absence of measured current speed and direction data, reference was made to Admiralty tidal diamond data from the TotalTide database to calibrate the hydrodynamic model. Please note, the UK Admiralty tidal diamond data is subject to the following uncertainty:

- The duration of the measurements used to derive the tidal diamond data is unknown. Often, the two tides in a day have slightly different peak speeds, with one being larger than the other due to the asymmetry in the water levels. This difference is not always evident in tidal diamond data.
- The actual date of the data recording is often unknown. It could be the case that significant bathymetric changes could have taken place since the data was recorded, both due to natural and anthropogenic changes.
- It is not known at what depth the measurements were made. They may be near the surface (for navigation reasons) or recorded at greater depths;
- Wind, wave or river flow effects can be measured in some coastal or estuarine regions, and it is not known if these have been removed from the tidal diamond current speeds and directions; and
- Location accuracy can mean there is some potential for a discrepancy in the location of the tidal diamond. This difference is especially important if it is located near an area with large current speed gradients, where a small horizontal distance can make a large difference in current speed.

Despite these uncertainties, tidal diamonds are still useful for model calibration without other data, especially if several sites show that the model agreement, in general, matches the data. Figure 4.2 shows the location of the tidal diamonds (SN028M, SN027J, SN027Y and SN027AA) used for the model calibration.

4.5 Waves

Wave data to support the study was identified as follows:

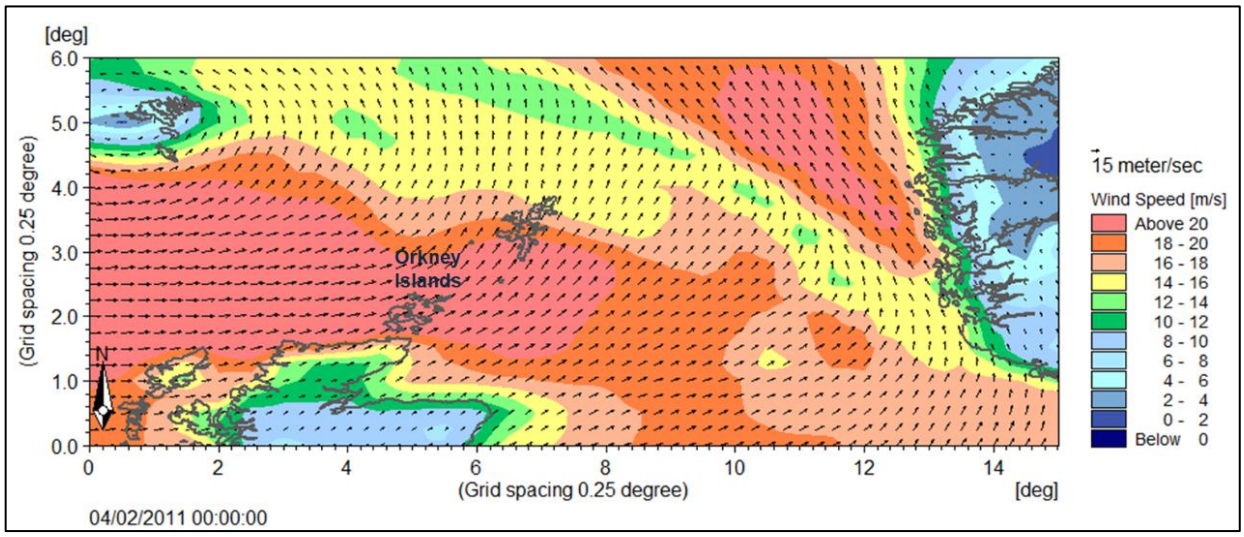
- Hindcast long-term hourly wave data from 1979 to 2024 (46 years) from the European Centre for Medium-Range Weather Forecast (ECMWF) fifth-generation reanalysis ERA5[8] database (0.5^0 spatial resolution, 1-hr intervals) used as the regional MIKE21 FM SW model boundaries (Section 6.4).
- Offshore measured data from the Centre for Environment Fisheries and Aquaculture Science (Cefas)[9] WaveNet at K7 wave buoy (WMO ID: 64046) (Figure 4.2, -4.5°E, 60.7°'N), in a water depth of approximately 900 m MSL from 2006 to 2011[10] for the regional wave model calibration (Section 6.6)

4.6 Wind

Wind is the primary forcing agent contributing to wave formation and therefore, using the best available wind data is essential. Hourly 10 m wind fields from the ECMWF ERA5 reanalysis (46-year hindcast, 0.25° spatial resolution, 1-hour intervals) were applied as wind forcing in both the regional hydrodynamic and wave models (Sections 5 and Section 6, respectively). It is noted that the ERA5 wind data have been calibrated against wind measurement worldwide and it is demonstrated that the ECMWF ERA5 data are suitable for developing a wave model (Hersbach et al. 2020).

Figure 4.3 presents the ECMWF ERA5 wind field on 4 April 2011, illustrating strong westerly winds with a magnitude reaching approximately 20 m/s propagating toward the Orkney Islands. The ECMWF ERA5 wind fields were also supplied in the hydrodynamic model to replicate meteorologically driven storm-surge events.

Figure 4.3: Overview of ECMWF ERA5 wind map on 4 February 2011.



Source: Mott MacDonald 2026

[8] ERA5 hourly data on pressure levels from 1940 to present accessed on 07 December 2025

[9] WaveNet - Download accessed on 20 November 2025.

[10] It is acknowledged that the dataset contains temporal gaps, and peak wave period (Tp) and mean wave direction (MWD) information was recorded.

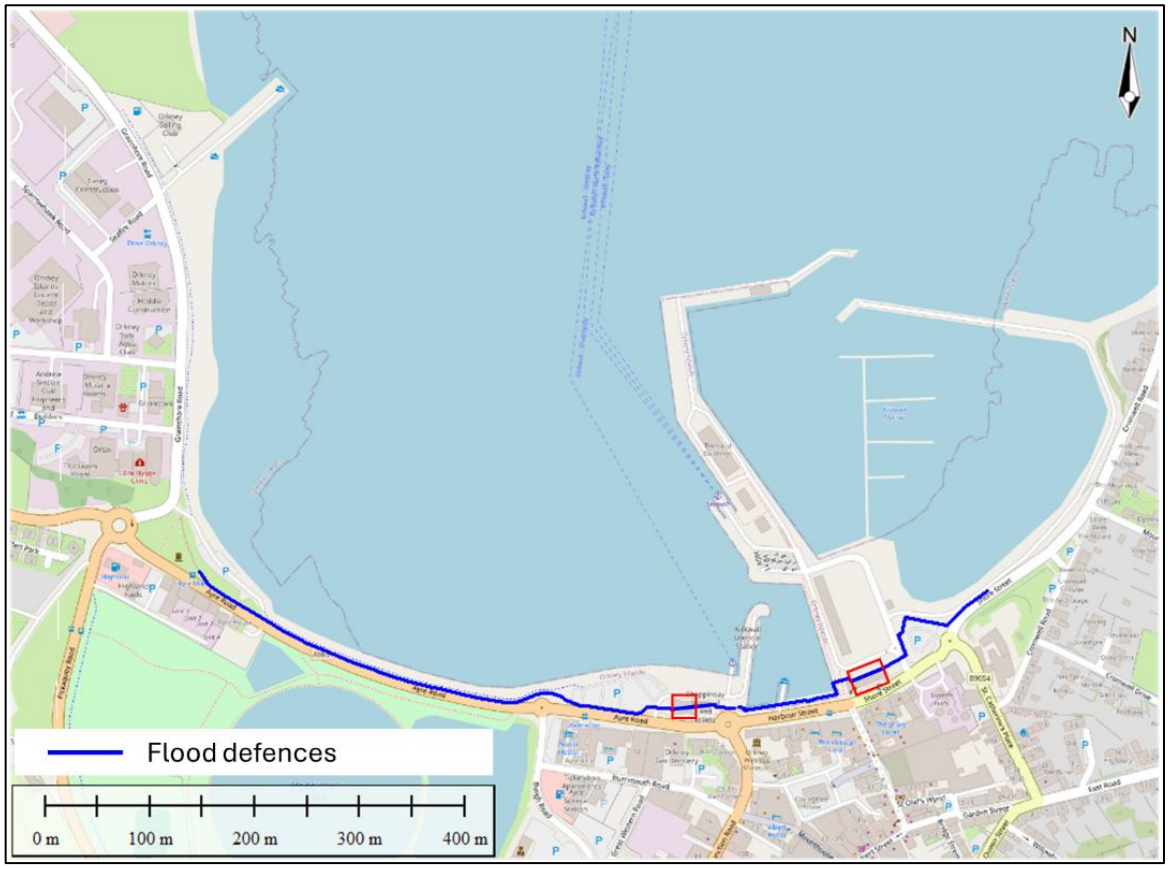
4.7 Air pressure

Hourly mean sea-level pressure fields from the ECMWF ERA5 reanalysis (46-year hindcast, 0.25° horizontal resolution) were applied as atmospheric boundary forcings in the hydrodynamic model to reproduce meteorologically driven storm-surge events.

4.8 Flood defences

Data on the flood defences (i.e. positions and crest levels) along the Kirkwall frontage were provided by the OIC. The defence structures were represented as dikes in the local overland hydrodynamic model (Section 11). Figure 4.4 shows the location of the flood defences that were incorporated in the model where the two buildings in front of the Kirkwall frontage (i.e. the red box in Figure 4.4) were treated as defences set at +3.9 m ODN, which is consistent to the defence heights on either side of the building structures.

Figure 4.4: Flood defences at Kirkwall frontage indicate as the blue line.



Source: OIC and Mott MacDonald 2026

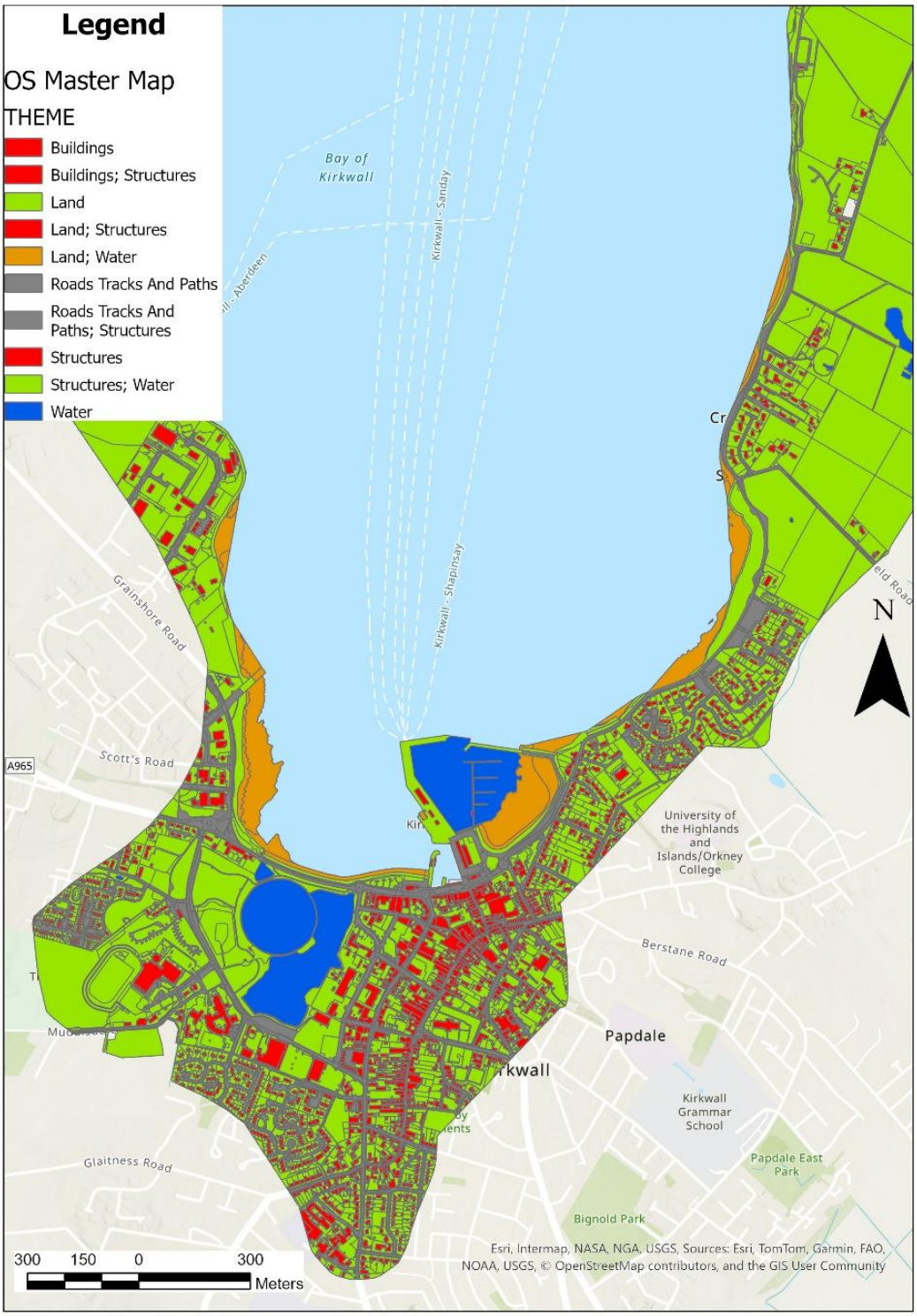
4.9 Ordnance Survey MasterMap

The OS MasterMap[11] topography layer is the most detailed and accurate view of Great Britain's landscape. It includes features representing the manmade and natural environment, such as land cover, buildings, water, rail, height, heritage, structures, administrative boundaries, and

[11] OS MasterMap Topography Layer | Data Products | OS License Number: PSGA Member License.

roads, tracks and paths. Figure 4.5 shows the overview of the OS MasterMap (Public Sector Geospatial Agreement (PSGA) Member License) that was used in this study.

Figure 4.5: Overview of OS MasterMap.



Source: Mott MacDonald 2026 (License Number: PSGA Member License)

4.10 Overland bed roughness

Table 4.3 presents the overland Manning's number that were used in the model based on Hick and Mason (1998) and Chow (1959) which aligned with the suggested values in the SEPA (2015) flood model guidance[12]. These values were supplied in the local hydrodynamic model as a bed roughness map (Section 10.2.5, based on OS MasterMap data).

Table 4.3: Manning's n and M roughness values based on OS MasterMap data.

Material	Manning's n	Manning's M
Land	0.07	14.29
Rail, roads, paths, heritage, antiquities	0.046	21.74
Marshes, reeds, intertidal	0.03	33.3
Inland water, floodplain	0.025	40
Buildings	0.02	50

Source: Mott MacDonald 2026

4.11 Climate change

4.11.1 Sea level rise

The sea level rise (SLR) projections were extracted from UKCP18[13] (Met Office, 2019) for inclusion in the simulations. The 8.5 Representative Concentration Pathway (RCP) projections along with the 95th percentile data were used in this study. The SLR values obtained for 2025 (i.e. Present day), 2070, 2100, and 2125 (i.e. 100-yr epoch) are shown in Table 4.4 where the calculation was performed based on the SEPA (2025)[14] guidance. An overview of the extracted grid from the UKCP18 database is shown in Figure 4.6.

The SEPA (2025) guidance indicates that both the magnitude and frequency of storm surges affecting Scotland remain uncertain and that their overall contribution to future coastal flood risk is expected to be considerably smaller than the influence of long-term sea-level rise. In line with this guidance, no climate-change adjustments to storm surge levels were applied in this study.

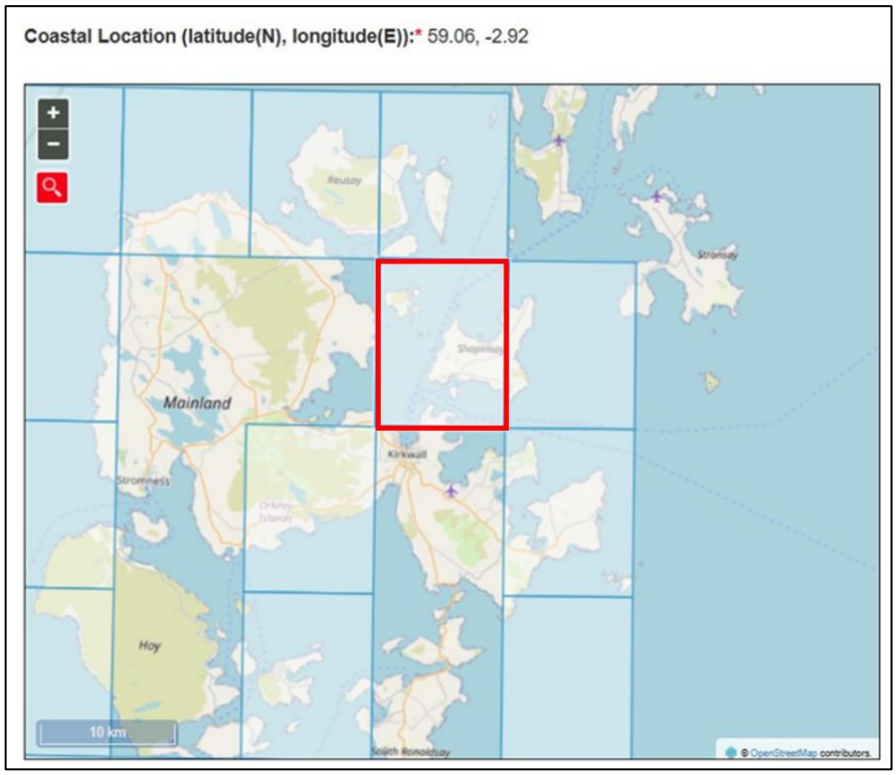
Table 4.4: Sea level rise based on UKCP18 RCP 8.5 at the 95th percentile.

Year	Estimated SLR (m)
2025	0.05
2070	0.50
2100	0.94
2125	1.31

Source: Mott MacDonald 2026

[12] Flood Modelling Guidance for Responsible Authorities, SEPA (2015) accessed on 5 March 2026.
[13] UK Climate Projections (UKCP18) guidance and reports - Met Office accessed on 6 January 2026
[14] Climate change allowances for flood risk assessment in land use planning version 6, SEPA (2025): climate-change-allowances-guidance_v6.pdf accessed on 6 January 2026

Figure 4.6: Sea level rise extracted grid from UKCP18 database.



Source: UKCP18 and Mott MacDonald 2026

4.11.2 Extreme water level

It has been noted by OIC Engineering that the Environment Agency Coastal Flood Boundary (CFB) [15] data appears to be underestimated. Consequently, extreme water levels (EWLs) used in this study were derived from an extreme value analysis (EVA) based on the calibrated regional hydrodynamic model (Section 5) and are presented in Section 7. Table 4.5 provides the extreme water levels obtained from the EVA results and the CFB data at node 82_20_A (Figure 3.4) for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods (i.e. AEP 2%, 0.5% and 0.1%, respectively) incorporating the projected sea level rise values listed in Table 4.4. Based on Table 4.5, it is evident that present day CFB values are underestimated by approximately 0.3 m when compared with the analysed extreme water levels adopted for this study.

[15] Coastal Design Sea Levels - Coastal Flood Boundary Extreme Sea Levels (2018) - data.gov.uk accessed on 10 February 2026.

Table 4.5: Extreme water levels (m ODN) including sea level rise (RCP 8.5, 95th percentile) for the selected epoch for CFB and Mott MacDonald (MM) predictions for 1-in-50, 1-in-200, and 1-in-1,000-year return periods.

Return period	CFB 2025	MM 2025	MM 2070	MM2100	MM 2125
50	2.76	3.11	3.56	4.00	4.37
200	2.88	3.20	3.65	4.09	4.46
1000	3.00	3.30	3.75	4.19	4.56

Source: Mott MacDonald 2026

Figure 4.7: CFB data point location.



Source: Mott MacDonald 2026

4.11.3 Wave and wind adjustments

Wave heights adjustments for the epoch year of 2070, 2100, and 2125 were based on SEPA, 2025 guidance. This involved increasing wave heights in all epochs by +20%, along with corresponding increases in wind speeds and wave periods, as defined by the relationship outlined in Section 7.

5 Regional HD model

5.1 Introduction

The regional HD model was developed using the MIKE21 FM HD[16] software, where the flexible mesh allows the resolution to be varied across the model domain. The regional HD model was built to transform hydrodynamic conditions from offshore deep-water areas to the shallow water nearshore areas around Orkney Islands. The model covers a large oceanic area of the North Atlantic Ocean, North Sea and Pentland Firth (Figure 5.1). The mesh gives higher resolution in the areas of interest (i.e. nearshore areas around Orkney islands) and reduced resolution further away or in areas with less bathymetric variability. This approach makes the model computationally more efficient.

5.2 Model Setup

5.2.1 Horizontal and vertical datums

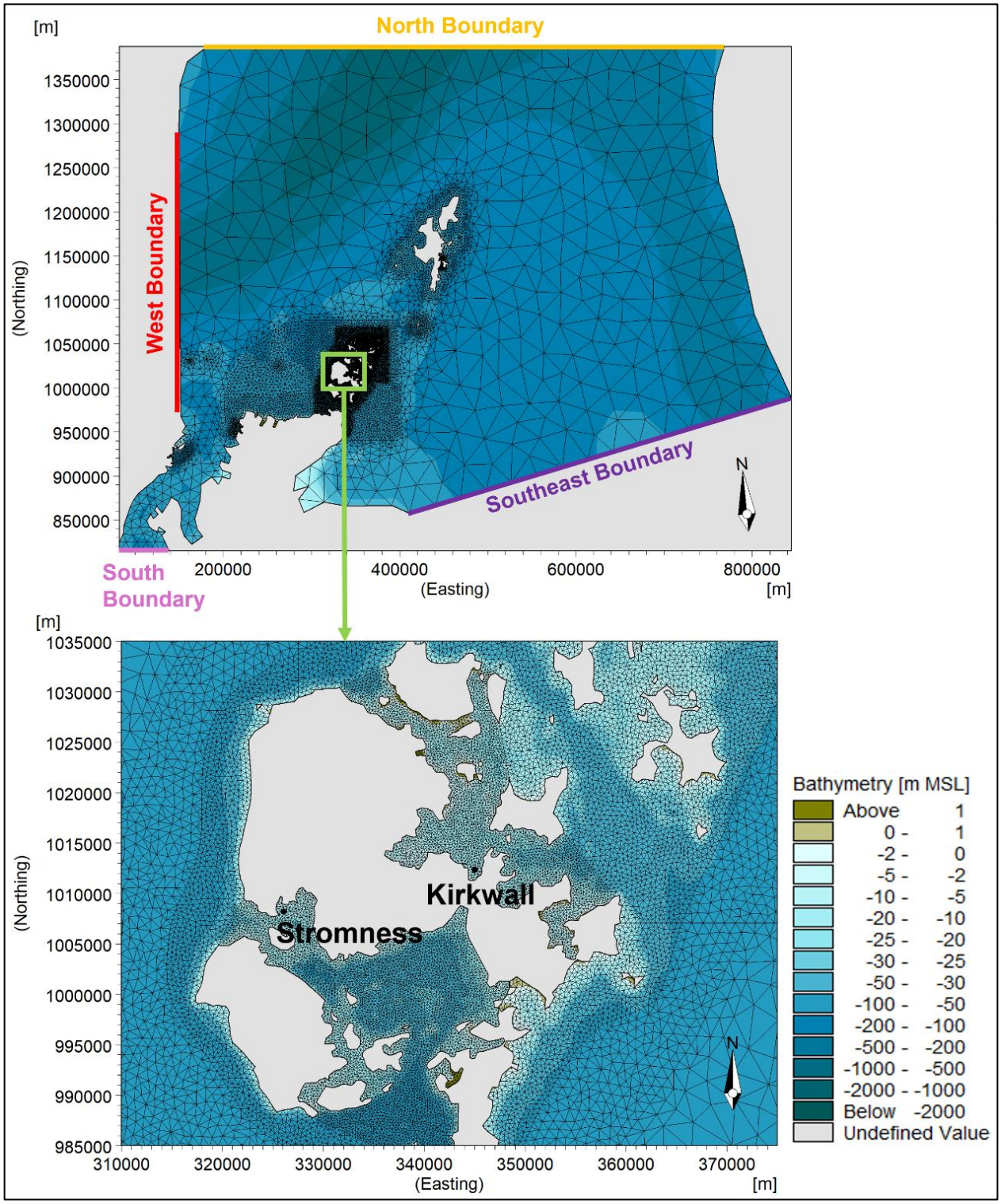
The model was setup using the horizontal and vertical datums as detailed in Section 4.1.

5.2.2 Regional model domain, bathymetry and mesh

Bathymetric data as described in Section 4.2 were used in the regional model. Figure 5.1 shows the wide-area model domain, bathymetry, mesh and an enlarged view around the extent of the study site. This mesh is coarser in the offshore region (1 to 30 km approximately), while at the area around Orkneys Islands and Kirkwall, the element sides of the mesh are between 200 to 800 m in length.

[16]DHI (2026). MIKE3 Flow FM User Guide. MIKE 3 Hydrodynamic Module FM accessed on 6 January 2026.

Figure 5.1: Regional model domain's bathymetry and mesh.



Source: Mott MacDonald 2026

5.2.3 Model density

A barotropic density function was applied in the model as it is sssumed that the stratification at a regional scale is negligible or not significant for a large scale area.

5.2.4 Model boundary conditions

The offshore water level and current velocity boundary of the regional model was defined between the north and east open boundaries of the regional model domain (Figure 5.1). The MIKE Zero global tidal model of heights[6] with a 0.125 x 0.125° grid resolution was used to create the water level time-series along the regional model boundaries. As the MIKE Zero global tide 0.125° model is not available for tidal model of currents, the MIKE Zero global tidal model of currents with a 0.25 x 0.25° grid resolution was used to create current velocity profiles along these boundaries. The MIKE Zero global tidal model considers ten major tidal constituents, including semi-diurnal M2, S2, K2, N2, the diurnal S1, K1, O1, P1 Q1 and the shallow water constituent M4.

5.2.5 Bed roughness

Iterative model sensitivity tests were undertaken in which the bed roughness of the Manning's number, M varied between 20 to 50 $m^{1/3}/s$. In the final simulation, a constant manning value of 32 $m^{1/3}/s$ was applied to the whole domain which gave a very good model performance.

5.2.6 Atmospheric forcings

The main wind forcing applied as surface boundary conditions was taken from the ECMWF ERA5 hindcast data (Section 4.6) as well as the mean sea-level pressure fields (Section 4.7).

5.2.7 Eddy viscosity

Eddy viscosity expresses the distribution of shear stress in a fluid and is related to the amount of flow turbulence. The present model selected horizontal eddy viscosity with a constant Smagorinsky (CS) formulation with the recommended CS value of 0.28 and eddy viscosity values of $1.8x10^{-6}$ and $1x10^{10}$ m^2/s for the minimum and maximum eddy viscosity, respectively.

5.3 Regional HD model calibration

Model calibration was undertaken during spring tides (20th to 27th March 2023) using measured water level data at Kirkwall and Stromness (Section 4.3) and UK Admiralty TotalTide database for currents (Section 4.4). During the calibration process, bed roughness, eddy viscosity, bathymetry, mesh resolution and boundary conditions were adjusted to obtain the best agreement between the calibration data and model results.

Figure 5.2 compares the measured, modelled, and predicted astronomical water levels at Kirkwall and Stromness. The modelled water levels reproduce the measured data sufficiently, with a slight overestimation that provides a conservative outcome. The predicted astronomical water levels are lower than both the measured and modelled levels, as expected, reflecting the exclusion of surge contributions from the predicted water levels. This highlights the importance of incorporating surge into hydrodynamic modelling where such data are available, as reliance on predicted (astronomical) water levels alone during model calibration would lead to an underestimation of actual water levels.

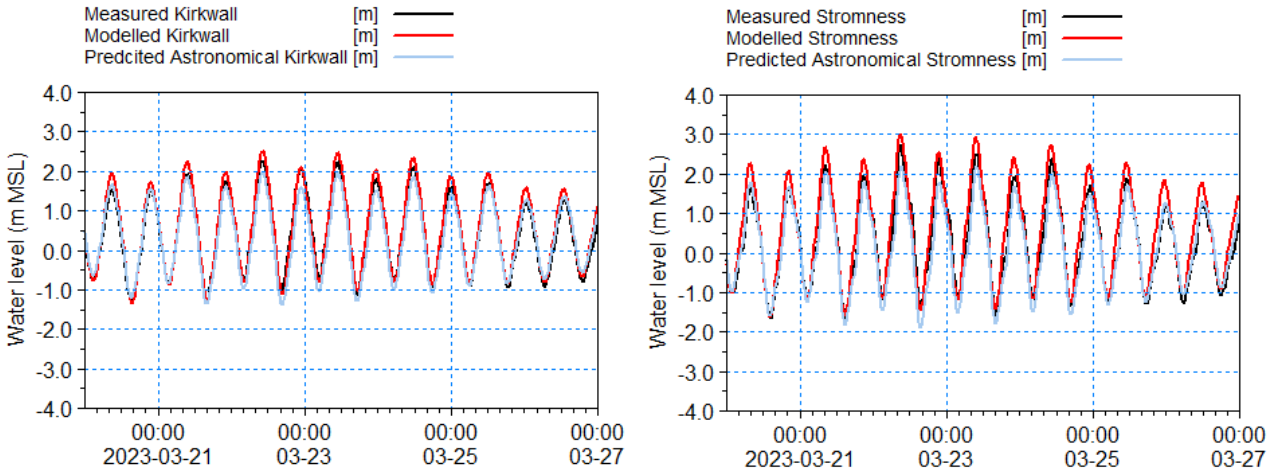
Table 5.1 shows the model performance statistics of root mean square error (RMSE), bias, coefficient of determination (R^2), standard deviation (SD) and index of agreement (IA) for water levels. It is observed that the modelled and measured water levels show a very good visual and statistical agreement with the R^2 >0.9 at all stations, indicating a very good model agreement

with the measured water level data. The model statistics meet the published model criteria performance in Williams & Esteves (2017).

Figure 5.3 shows the comparison of current speed and direction between modelled and tidal diamond data from the TotalTide database. The results show that the modelled current speed and direction align well with the Tidal Diamond data, indicating good model performance. Table 5.2 and Table 5.3 shows the model performance statistics of root mean square error (RMSE), bias, coefficient of determination (R^2), standard deviation (SD) and index of agreement (IA) for current speed and direction, respectively.

Based on Figure 5.2, Figure 5.3, Table 5.1, Table 5.2 and Table 5.3, the modelled results show a good visual and statistical agreement, indicating that the model compares well against the data used in model calibration. Overall, the model statistics meet the published model criteria performance in Williams & Esteves (2017)[17].

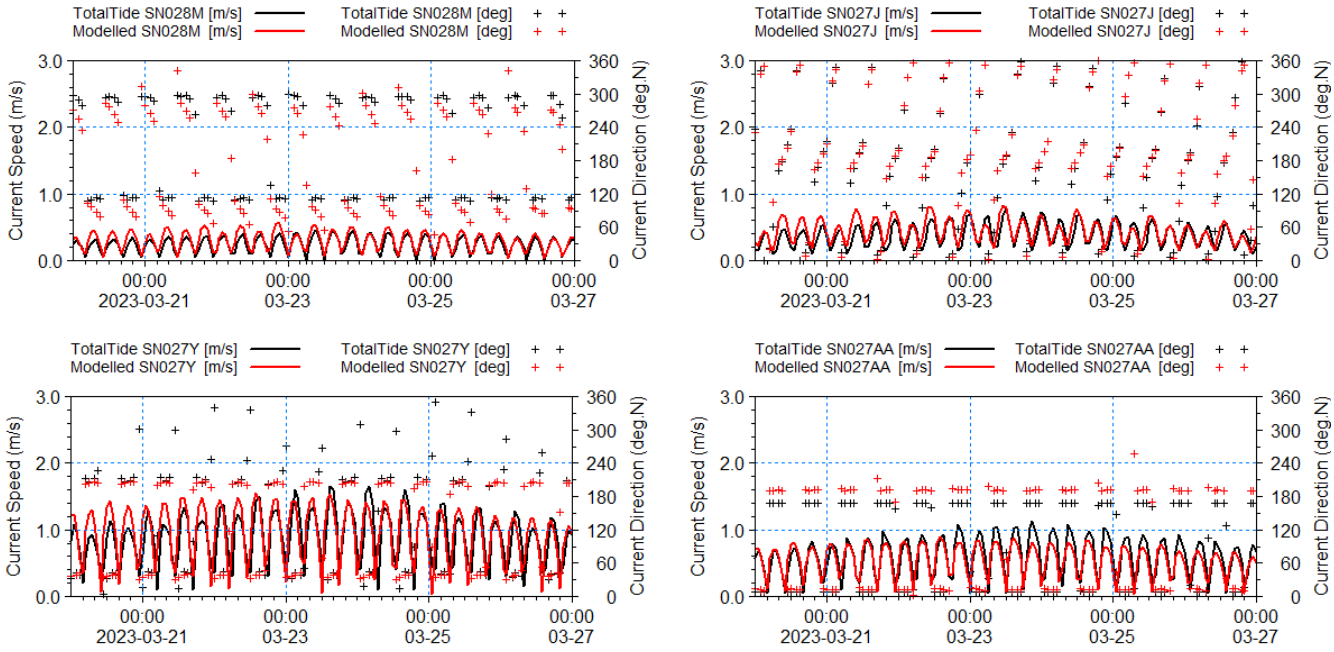
Figure 5.2: Model calibration for water levels.



Source: Mott MacDonald 2026

[17] Williams, J.J. & Esteves, L.S., 2017. Guidance on setup, calibration, and calibration of hydrodynamic, wave, and sediment models for shelf seas and estuaries. Advances in civil engineering, 2017.

Figure 5.3: Model calibration for current speeds and current directions.



Source: Mott MacDonald 2026

Table 5.1: Model calibration statistics for water level.

Station	RMSE (m)	Bias (m)	SD (m)	R^2 (m)	IA
Kirkwall	0.15	0.18	0.13	0.99	0.99
Stromness	0.33	0.42	0.30	0.94	0.97

Source: Mott MacDonald 2026

Table 5.2: Model calibration statistics for current speeds.

Station	RMSE (m)	Bias (m)	SD (m/s)	R^2 (m)	IA
SN028M	0.06	0.08	0.06	0.80	0.90
SN027J	0.10	0.12	0.11	0.65	0.87
SN027Y	0.22	0.26	0.26	0.62	0.88
SN027AA	0.12	0.14	0.14	0.77	0.92

Source: Mott MacDonald 2026

Table 5.3: Model calibration statistics for current directions.

Station	RMSE (^{0})	Bias (^{0})	SD (^{0})	R^2 (m)	IA
SN028M	32	43	36	0.86	0.95
SN027J	46	112	104	0.30	0.75
SN027Y	37	84	82	0.35	0.78
SN027AA	21	39	33	0.86	0.95

Source: Mott MacDonald 2026

5.4 Regional HD model validation

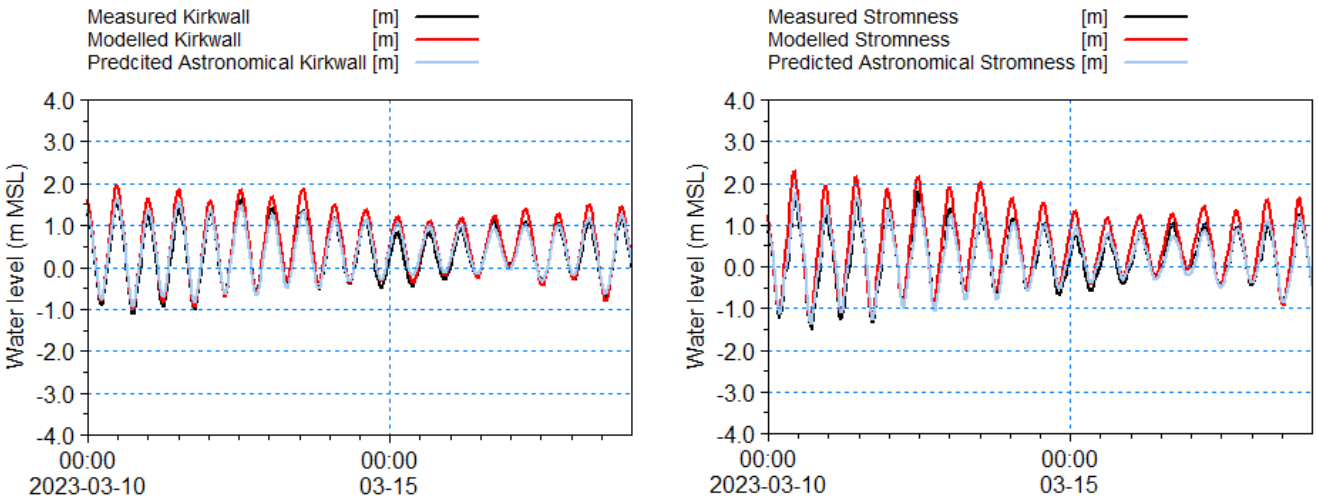
Model validation is usually undertaken after calibration using observed data from a different date or location within the model domain. In the present study, validation was undertaken for a neap tide period on 10th to 19th March 2023. Figure 5.4 shows the comparison between measured water, modelled and predicted water levels while Figure 5.5 provides the corresponding comparison for currents. Table 5.4, Table 5.5 and Table 5.6 shows the model performance statistics of RMSE, bias, R^2, SD and index of IA for water levels and currents, respectively.

Based on Figure 5.4, Figure 5.5, Table 5.4, Table 5.5 and Table 5.6, the model shows a good visual and statistical agreement, indicating that the model has been validated successfully. The model statistics meet the published model criteria performance in Williams & Esteves (2017).

The model was also run for January 2005 as part of the validation process, allowing comparison with the historical recorded high-water level at Kirkwall and Stromness provided by the OIC (Table 5.7). The results show that the model successfully reproduced the high-water levels observed at Kirkwall and slightly underpredicted at Stromness. Nonetheless, the resulting level of agreement is considered adequate for a regional-scale assessment.

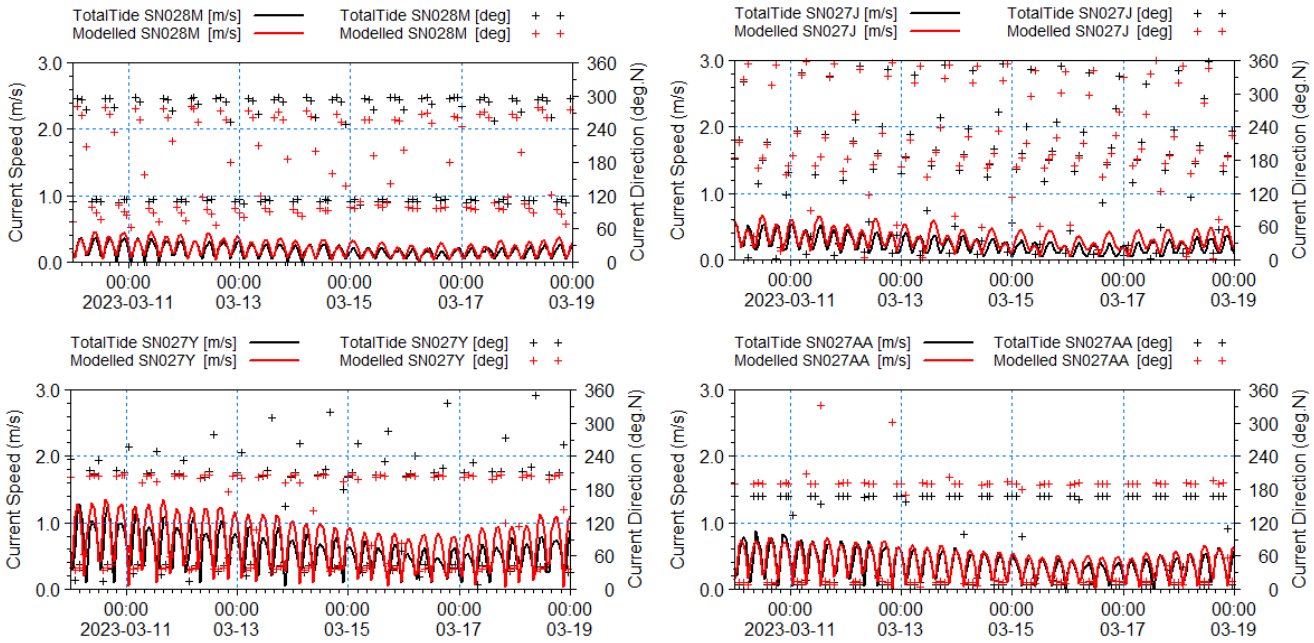
Based on the model calibration and validation, it was determined that the regional model demonstrates high reliability in predicting both water levels, surge events and currents. The model was then simulated over a 46-year period (1979–2024) to analyse long-term water levels at Kirkwall (Section 7) and to provide boundary conditions for the local hydrodynamic model (Section 10).

Figure 5.4: Model validation for water levels.



Source: Mott MacDonald 2026

Figure 5.5: Model validation for currents speeds and current directions.



Source: Mott MacDonald 2026

Table 5.4: Model validation statistics for water level.

Station	RMSE (m)	Bias (m)	R^2	SD (m)	IA
Kirkwall	0.20	0.24	0.14	0.98	0.97
Stromness	0.33	0.41	0.24	0.92	0.92

Source: Mott MacDonald 2026

Table 5.5: Model validation statistics for current speed.

Station	RMSE (m/s)	Bias (m/s)	R^2	SD (m/s)	IA
SN028M	0.05	0.07	0.05	0.77	0.89
SN027J	0.09	0.11	0.08	0.66	0.82
SN027Y	0.21	0.25	0.18	0.72	0.84
SN027AA	0.07	0.09	0.08	0.84	0.95

Source: Mott MacDonald 2026

Table 5.6: Model validation statistics for current direction.

Station	RMSE (^{0})	Bias (^{0})	R^2	SD (^{0})	IA
SN028M	33	48	41	0.81	0.93
SN027J	61	128	119	0.13	0.66
SN027Y	36	86	82	0.37	0.78
SN027AA	21	37	31	0.88	0.95

Source: Mott MacDonald 2026

Table 5.7: Maximum water level comparison (January 2005).

Station	OIC	Modelled
Kirkwall	2.83m ODN	2.80m ODN
Stromness	3.56m ODN	3.33m ODN

Source: Mott MacDonald 2026

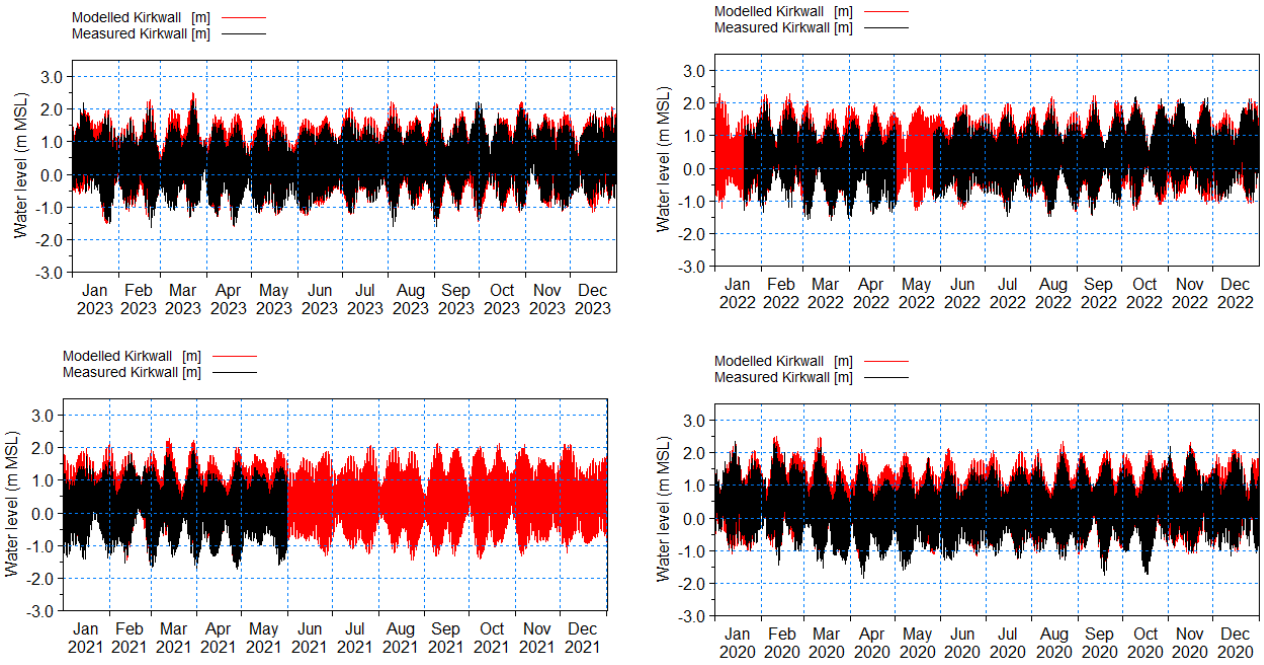
5.5 Regional HD results

The calibrated regional MIKE21 HD FM model was executed over a 46-year period (1979–2024) to characterise long-term water-level variability at Kirkwall. Figure 5.6 presents the comparison between modelled and measured water levels at Kirkwall for the years 2020 to 2023. The long-term modelled water levels show close agreement with the measured data, exhibiting a slight positive bias consistent with the earlier calibration and validation results, which provides a conservative representation. It is also noted that the measured dataset contains several gaps, limiting its suitability for long-term statistical analysis.

Consequently, the continuous 46-year modelled dataset offers a more robust basis for assessing water-level conditions at Kirkwall and across the wider Orkney Islands. The 46-year modelled water-level time series at Kirkwall, referenced to m ODN, is presented in Figure 5.7. Over the full simulation period, the statistical maximum, mean, and minimum water levels are +3.1 m ODN, +0.81 m ODN, and -1.3 m ODN, respectively.

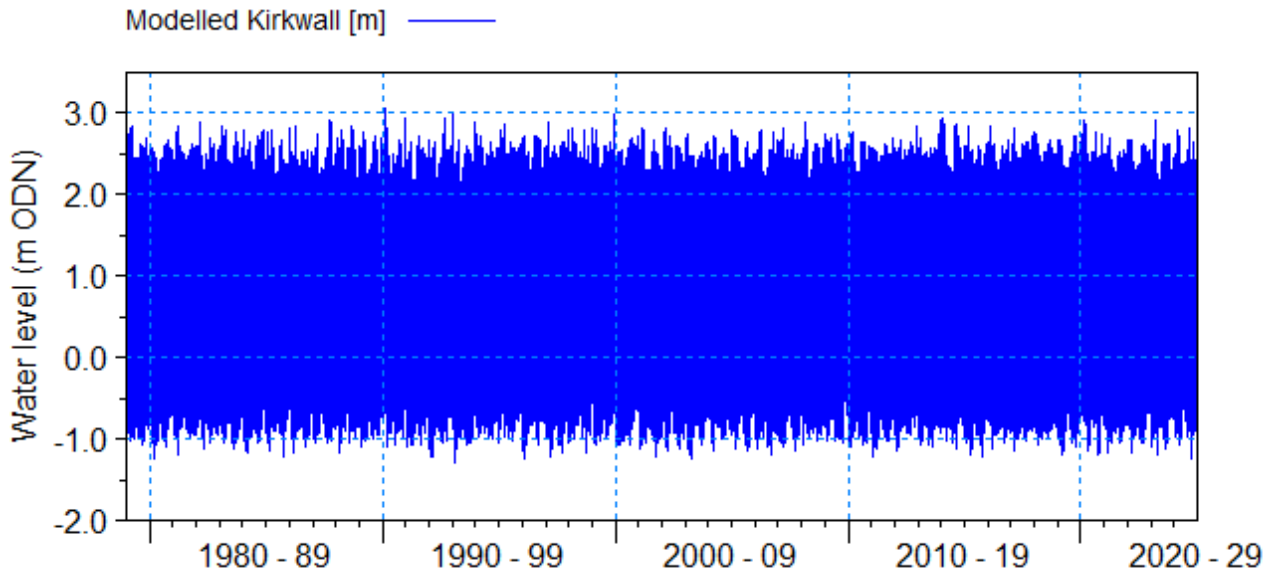
Figure 5.8 illustrates the statistically derived maximum water level obtained from the 46-year model simulation. The results indicate that the maximum water level at Kirkwall is up to +3.1 m ODN, while across the Orkney Islands the maximum water levels vary between +2.5 m ODN and +4.0 m ODN approximately. In addition, Figure 5.9 shows the statistically derived maximum current speed (46-year), indicating that peak current velocities around Kirkwall are relatively low, remaining below approximately 0.3 m/s.

Figure 5.6: Long term water level comparison.



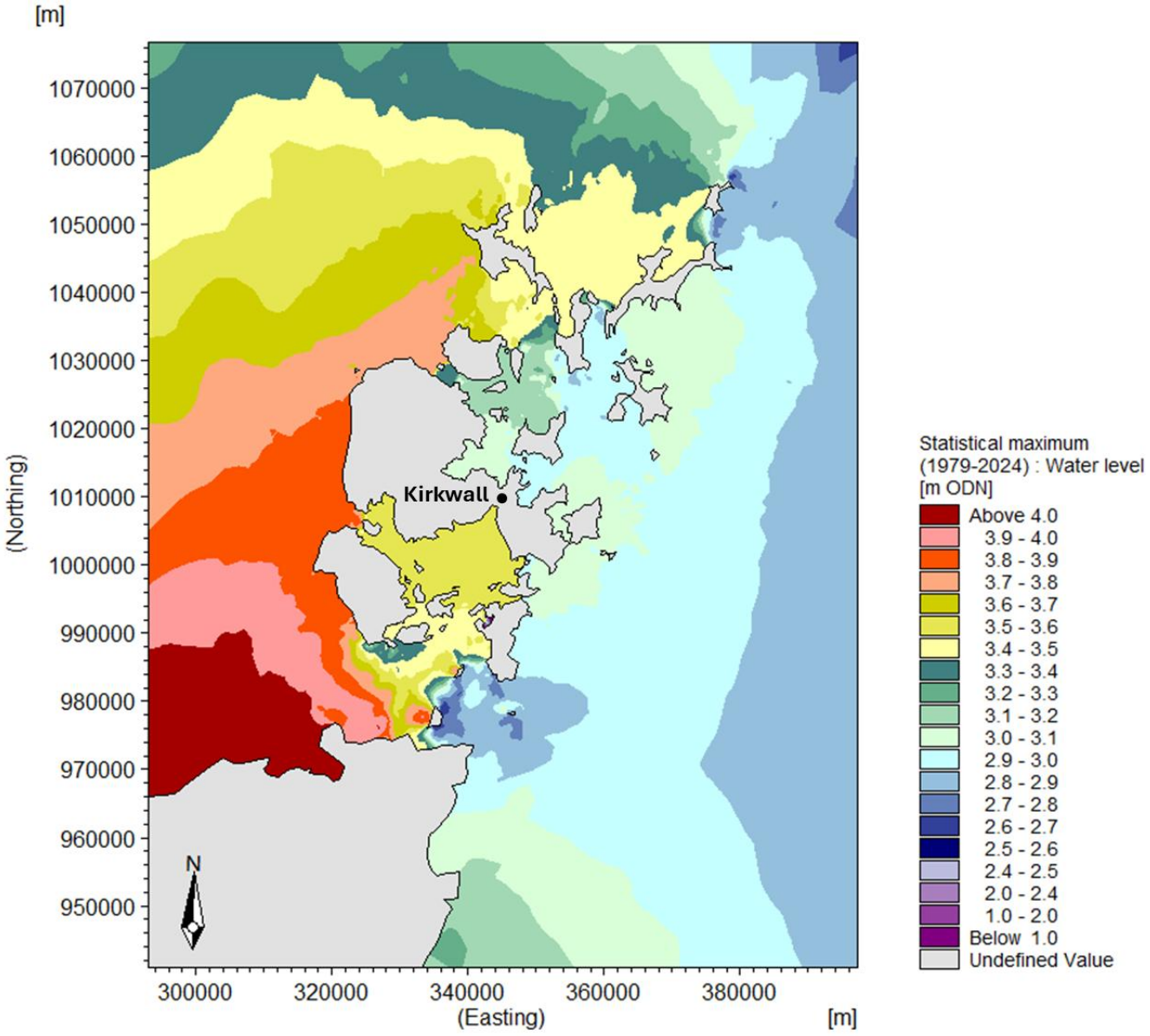
Source: Mott MacDonald 2026

Figure 5.7: Time series of 46-year modelled water level (m ODN) data at Kirkwall.



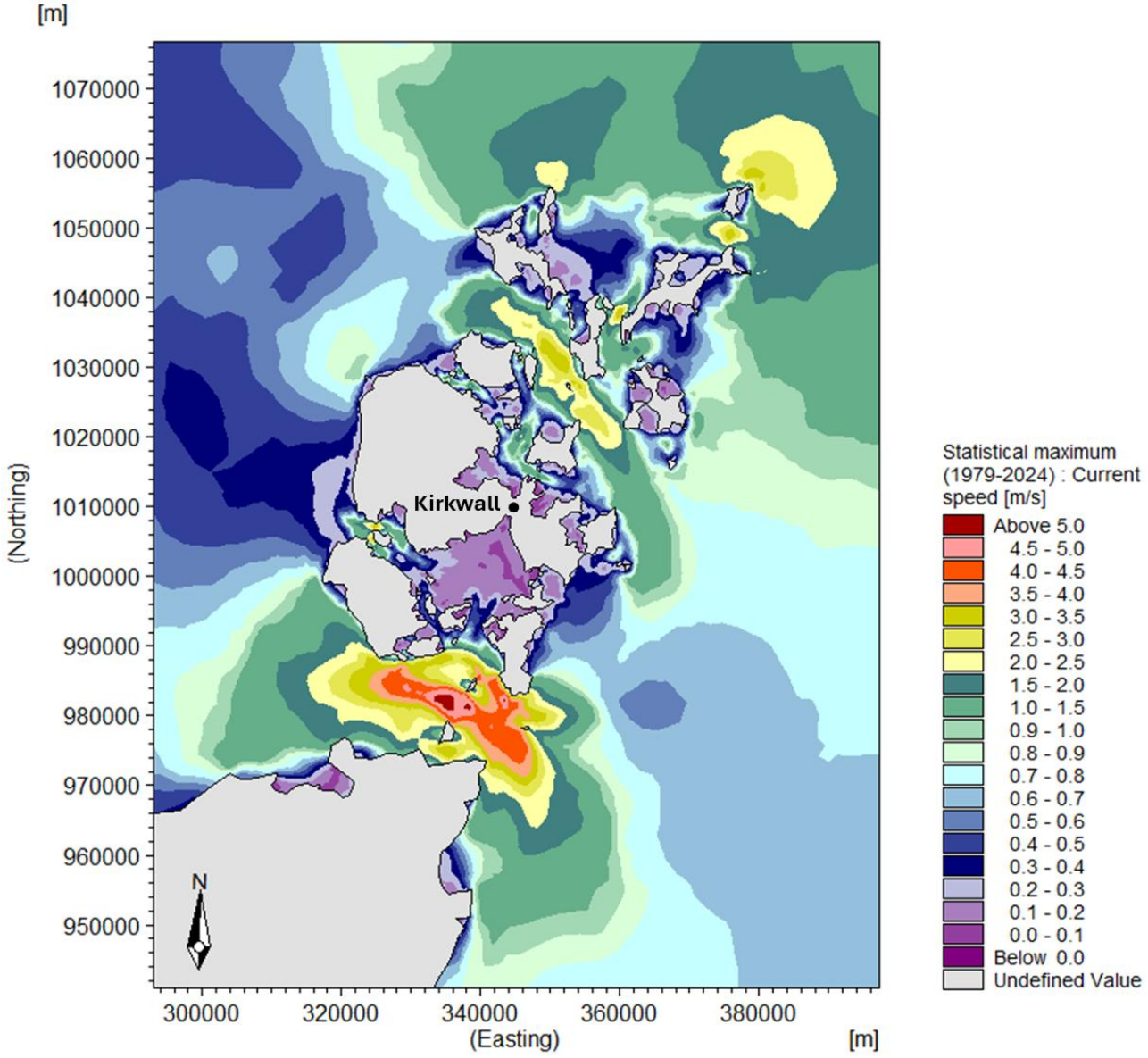
Source: Mott MacDonald 2026

Figure 5.8: Statistical maximum water level of 46-yr model results.



Source: Mott MacDonald 2026

Figure 5.9: Statistical maximum current speeds of 46-yr model results.



Source: Mott MacDonald 2026

6 Regional spectral wave model

6.1 Introduction

The regional spectral wave model was built using the MIKE21 FM SW software[18]. This third-generation state-of-the-art[19] model uses an unstructured mesh that enables full-time domain simulations based on the finite volume method. The model simulates wind-generated waves' growth, decay and transformation and swells in offshore and coastal areas. The model includes wave growth by the action of wind, nonlinear wave-wave interaction, dissipation due to white-capping, dissipation due to bottom friction, dissipation due to depth-induced wave breaking, refraction, diffraction and shoaling due to depth variations and the effect of time-varying water depth. Therefore, the model is deemed suitable to propagate waves from offshore to the nearshore area at Kirkwall.

6.2 Horizontal and vertical datum

The model was setup using the horizontal and vertical datums as detailed in Section 4.1.

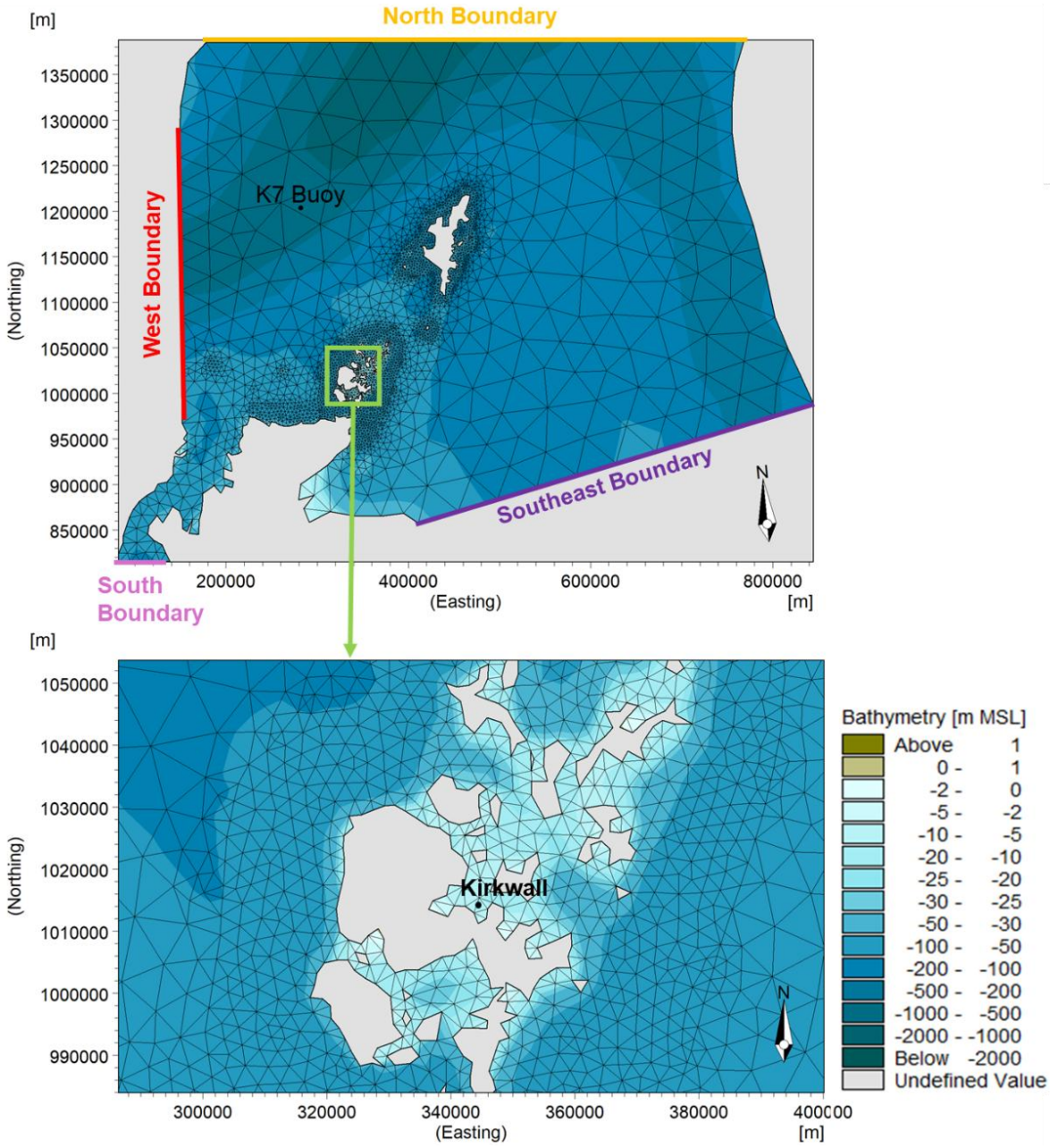
6.3 Regional model domain, mesh and bathymetry

The regional MIKE21 FM SW model was set up to cover a large oceanic extent of all offshore and nearshore areas that influence the wave conditions around Orkney Islands (Figure 6.1), consistent with the regional MIKE21 FM HD model extent (Figure 5.1). Bathymetric data as described in Section 4.2 were used in the regional model. The computational mesh employs a coarser resolution in offshore areas (on the order of 10–50 km), while in the nearshore region surrounding Kirkwall the mesh is refined to a resolution of approximately 1–3 km (Figure 6.1).

[18] DHI (2026). MIKE21 Spectral Waves overview. MIKE 21 Spectral Waves (mikepoweredbydhi.com) accessed on 13 January 2026.

[19] DHI (2017). Spectral Wave Module Scientific Documentation. MIKE 21 - Spectral Wave Module (mikepoweredbydhi.help) accessed on 13 January 2026.

Figure 6.1: Regional wave model domain, bathymetry, mesh and boundaries with an enlarged view within Kirkwall.



Source: Mott MacDonald, 2026.

6.4 Boundary conditions and forcings

The predicted water level conditions were imposed as a time series of tidal elevation at Offshore West Shetland, as detailed in Section 4.3. The wave parameters included, significant wave height (Hs), peak wave period (Tp) and mean wave direction (MWD), these were extracted from the ECMWF ERA5 data (Section 4.5) and applied at four open boundaries (South, Southeast,

North and West boundaries, Figure 6.1). A constant directional standard deviation (DSD) of 30° and 5° was used for the wind-waves and swell-waves, respectively at all open boundaries. The main wind forcing applied as surface boundary conditions was taken from the ECMWF ERA5 hindcast data (Section 4.6).

6.5 Model setup

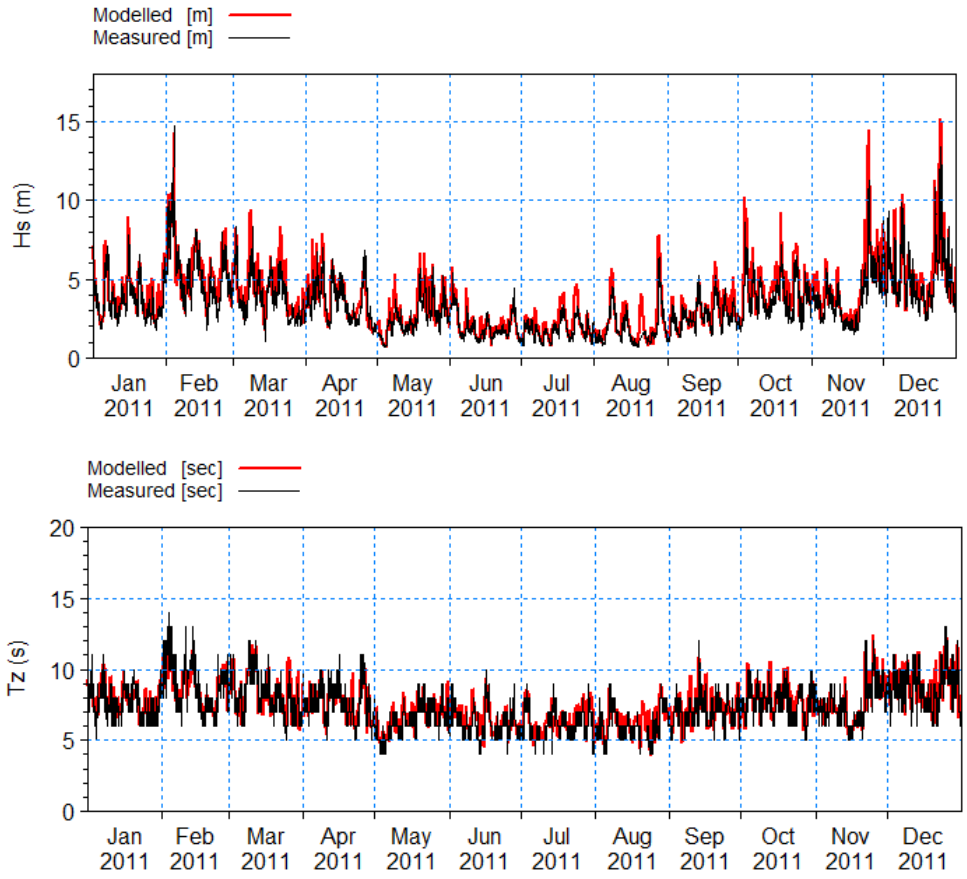
Following the standard modelling procedure, several iterative sensitivity tests were undertaken in which key wave parameters in the model setup were tuned to achieve the best model setup. The key parameters used in the regional wave model setup are summarised in Appendix A.

6.6 Regional SW model calibration

Regional wave model calibration was carried out using measured wave data from the K7 wave buoy (Section 4.5) as no local measured wave data is available. The calibration process involved adjusting model parameters including boundary conditions, bathymetry, wind forcing, wave breaking, and white-capping formulations. These parameters were refined iteratively to ensure that the modelled wave conditions were in close agreement with the measured data at key locations within the model domain. The year 2011 was selected as the calibration period, as it contained higher-wave events compared with other years for which measured wave data were available. Figure 6.2 presents the comparison between measured and modelled significant wave height (Hs) and zero-crossing wave period (Tz) at the K7 buoy. The figure shows that the model reproduces the measured wave conditions well. It is noted that peak period (Tp) and mean wave direction (MWD) were not recorded."

Model performance statistics are tabulated in Table 6.1, including bias, root-mean-square error (RMSE), standard deviation (SD) and index of agreement (IA). Table 6.1 shows that the model can reproduce the measured data with high accuracy for Hs, with both R^2 and IA values of 0.9. Similarly, Tz also show good agreement against measured data. The statistics values meet the wave model performance criteria recommended by Williams and Esteves (2017).

Figure 6.2: Model calibration - Comparison between modelled and measured data at K7 buoy for significant wave height (Hs) and zero-crossing wave period (Tz).



Source: Mott MacDonald, 2026

Table 6.1: Model calibration statistics for waves.

Parameter	Hs	Tz
Bias	-0.48m	-0.29s
RMSE	0.85m	0.83s
SD	0.70m	0.78s
R^2	0.90	0.80
IA	0.95	0.92

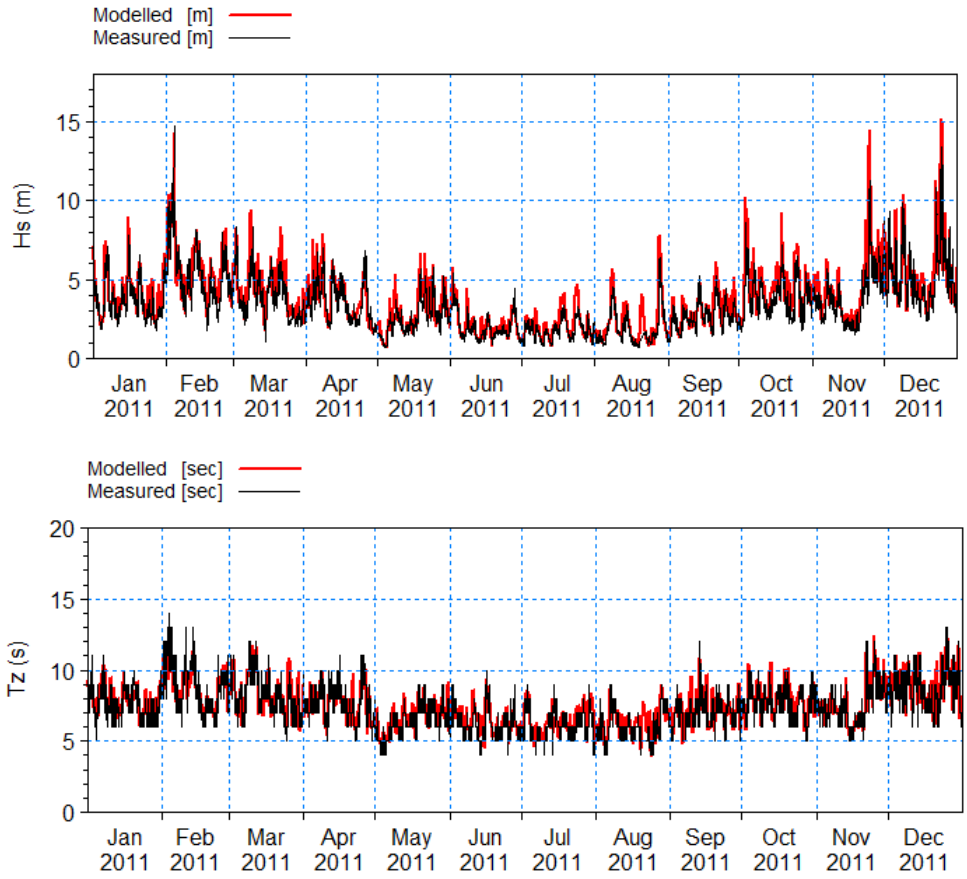
Source: Mott MacDonald 2026

6.7 Regional SW model validation

Validation of the regional wave model was undertaken by simulating a separate observational period. Figure 6.3 presents a comparison between measured and modelled Hs and Tz at the K7 buoy for the year 2009. The visual comparison indicates that the model reliably reproduces the measured wave conditions. Model-performance statistics summarised in Table 6.2 further confirm that the validation metrics meet the wave-model performance criteria recommended by Williams and Esteves (2017).

Based on model calibration and validation, it is determined that the model is sufficient for long-term simulations and for deriving extreme wave conditions (Section 7) to provide boundary inputs for the local wave model (Section 8).

Figure 6.3: Model validation - Comparison between modelled and measured data at K7 buoy for Hs and Tz.



Source: Mott MacDonald, 2026

Table 6.2: Model validation statistics for waves.

Parameter	Hs	Tz
Bias	-0.41m	-0.44s
RMSE	0.75m	1.05s
SD	0.63m	0.95s
R^2	0.86	0.62
IA	0.94	0.86

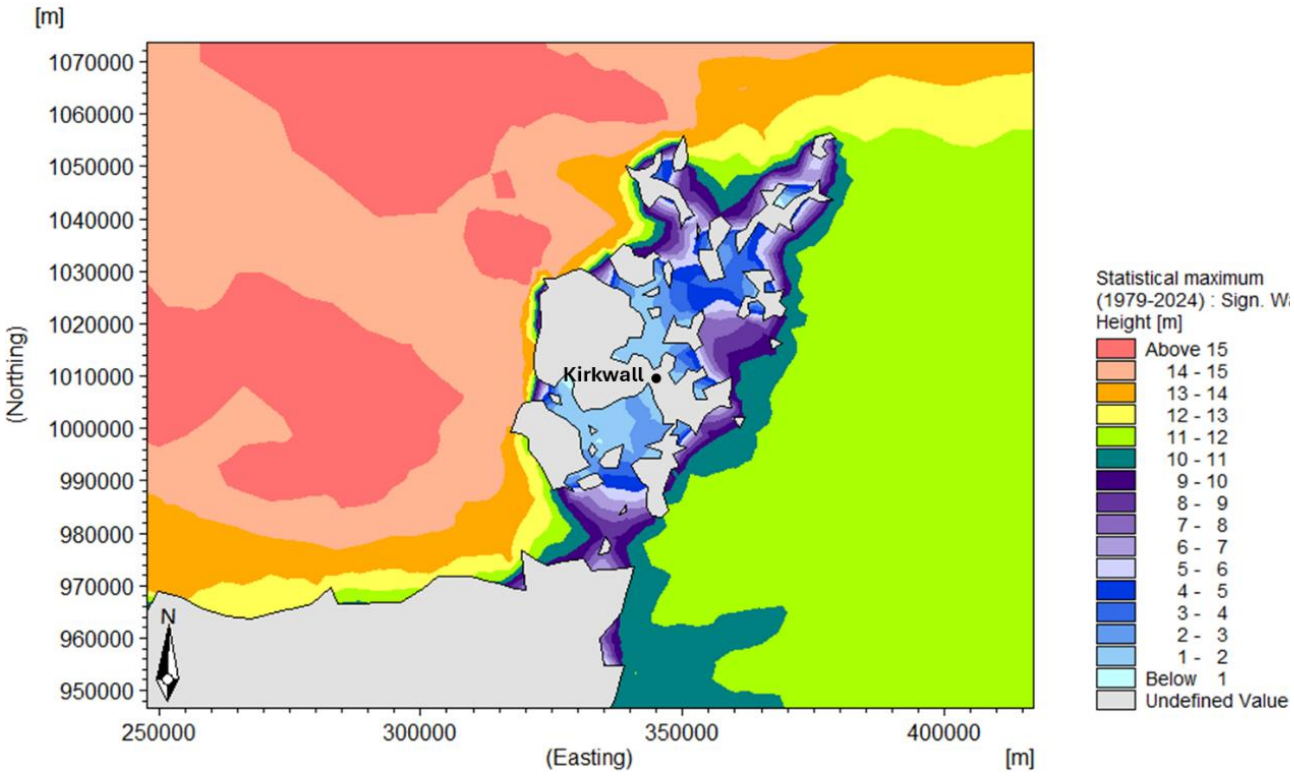
Source: Mott MacDonald 2026

6.8 Regional SW results

The calibrated regional MIKE21 SW FM model was executed over a 46-year period (1979–2024) to analyse extreme waves conditions (Section 7) at the study area. Figure 6.4 shows the statistically derived maximum Hs obtained from the 46-year model simulation. The results indicate that the maximum Hs at Kirkwall over the 46-year simulation period is approximately 1.5 m, consistent with the sheltered conditions within Kirkwall Bay. Across the wider Orkney Islands region, substantially higher maximum Hs values up to approximately 14 m are predicted in the fully exposed offshore sectors over this simulation period.

These elevated offshore values reflect the increased fetch and exposure to North Atlantic and North Seas wave energy, whereas the nearshore and semi-enclosed areas exhibit markedly reduced maximum Hs due to local sheltering, depth-induced wave transformation, and dissipation processes.

Figure 6.4: Statistical maximum Hs of 46-yr model results.



Source: Mott MacDonald 2025

7 Extreme analyses

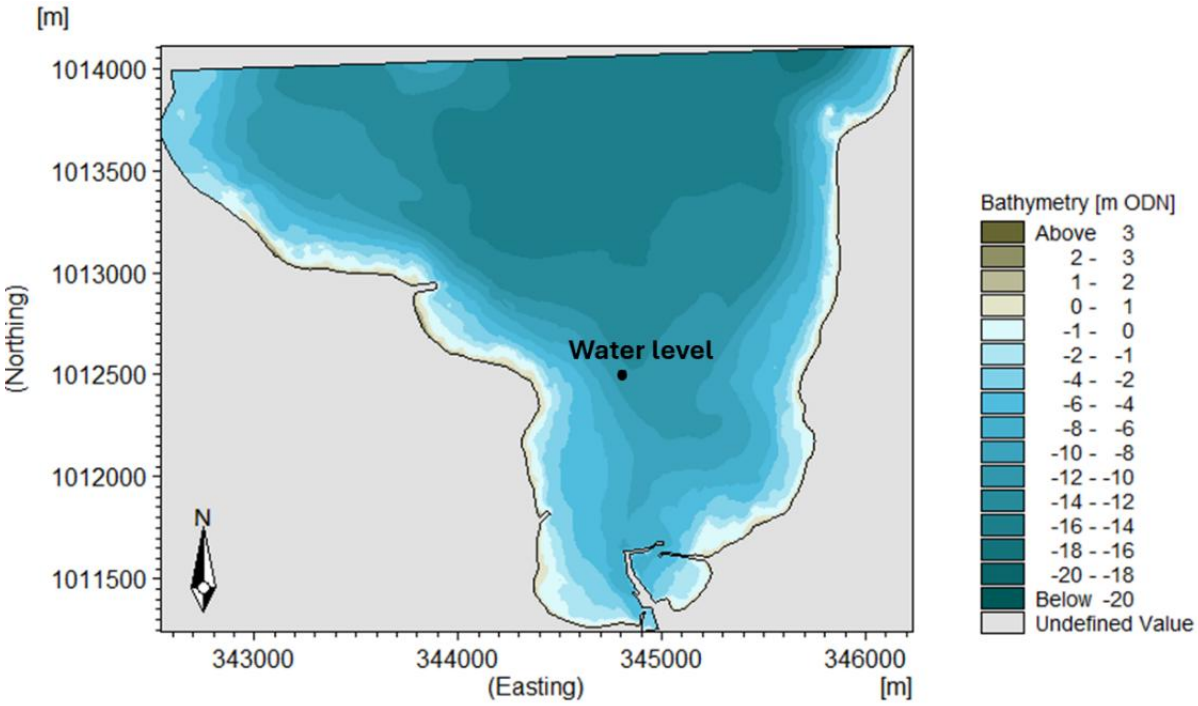
7.1 Introduction

Appropriate characterization of extreme water levels and wave conditions is essential for the Joint Probability Analysis (JPA) of water levels and waves. This was supported by an Extreme Values Analysis (EVA), which utilised 46 years of hindcast data from the regional MIKE21 HD FM (Section 5) and the regional MIKE21 SW FM (Section 6) models. The EVA was carried out using the Mike by DHI EVA tool[20].

7.2 Extreme water levels

The EVA of water levels was undertaken at the location shown in Figure 7.1, which corresponds to the CFB point (Figure 4.7). This location was selected to enable direct comparison against the CFB data. A comparison was also made using an extraction point located further inshore and this demonstrated that water level results are similar which indicates that water levels remain broadly consistent across the nearshore area of the bay.

Figure 7.1: Location of water level extraction for EVA



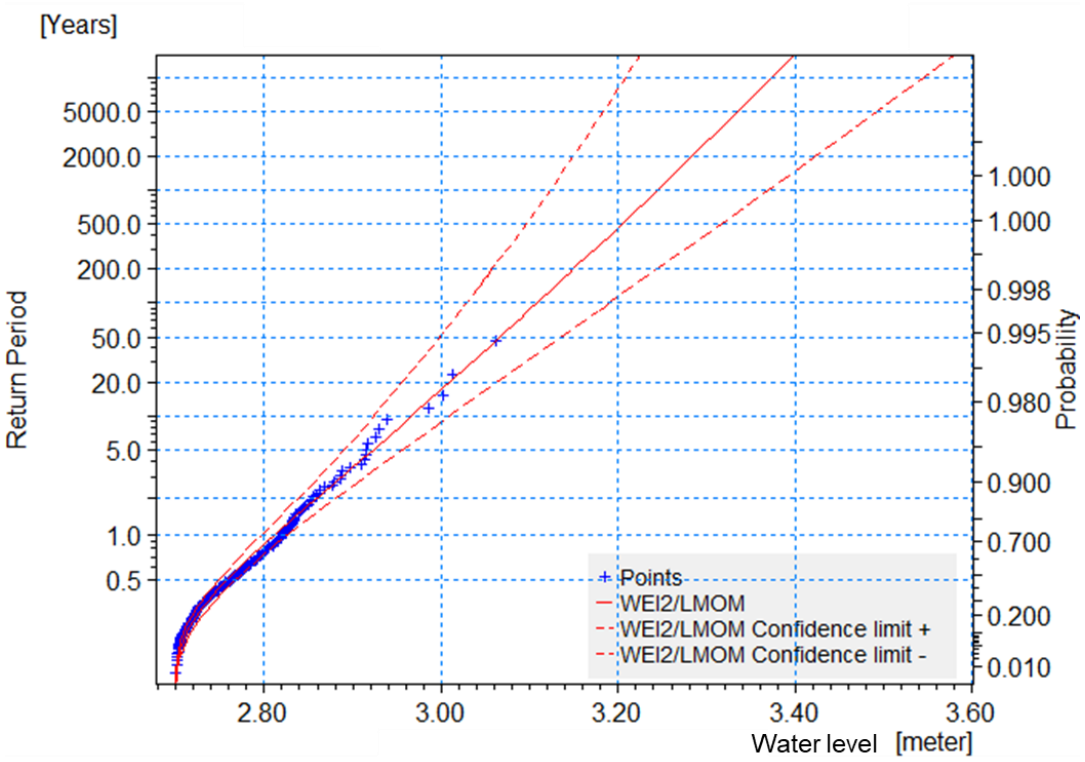
Source: Mott MacDonald, 2025.

The EVA of water levels considered a range of probability distributions and parameter estimation methods to determine extreme water levels for the JPA (Section 7.4). The best-fitted curve is shown in Figure 7.2, along with the associated 95% confidence intervals. The resulting

[20] Extreme Value Analysis User Guide, DHI (2021) accessed on 26 January 2026.

extreme water-level estimates for each epoch, including the applied SLR increments (Table 4.4), are summarised in Table 7.1.

Figure 7.2: EVA Probability Distribution Fit of water level.



Source: Mott MacDonald, 2026.

Table 7.1: Extreme water levels (m ODN) including sea level rise (RCP 8.5, 95th percentile) for the selected epoch between CFB and Mott MacDonald (MM) for 1-in-50, 1-in-200, and 1-in-1,000 year return periods.

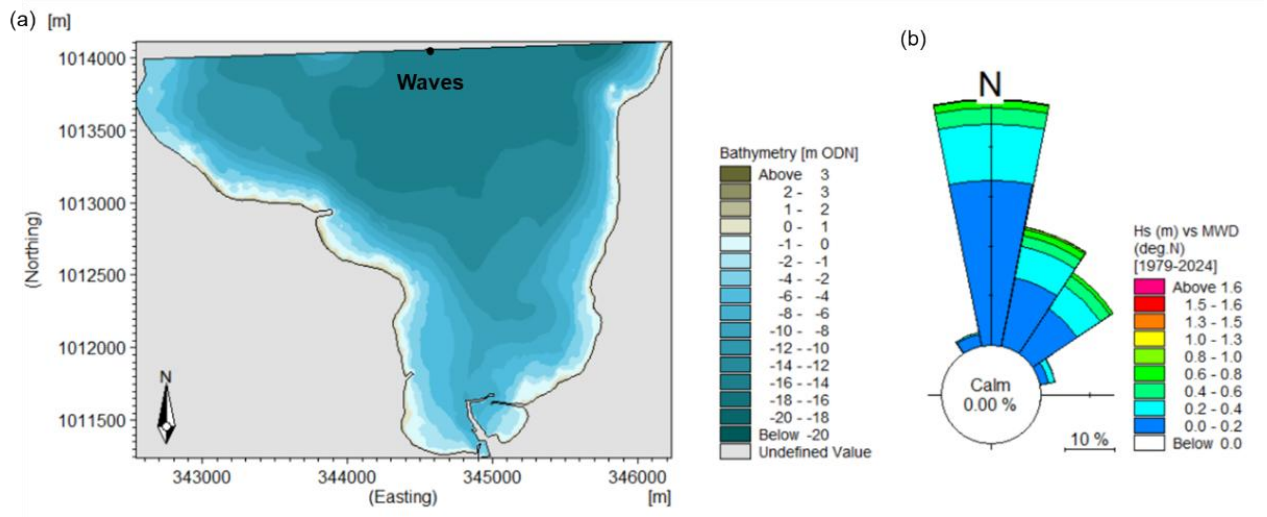
Return period	CFB 2025	MM 2025	MM 2070	MM 2100	MM 2125
50	2.76	3.11	3.56	4.00	4.37
200	2.88	3.20	3.65	4.09	4.46
1000	3.00	3.30	3.75	4.19	4.56

Source: Mott MacDonald, 2026

7.3 Extreme waves and winds

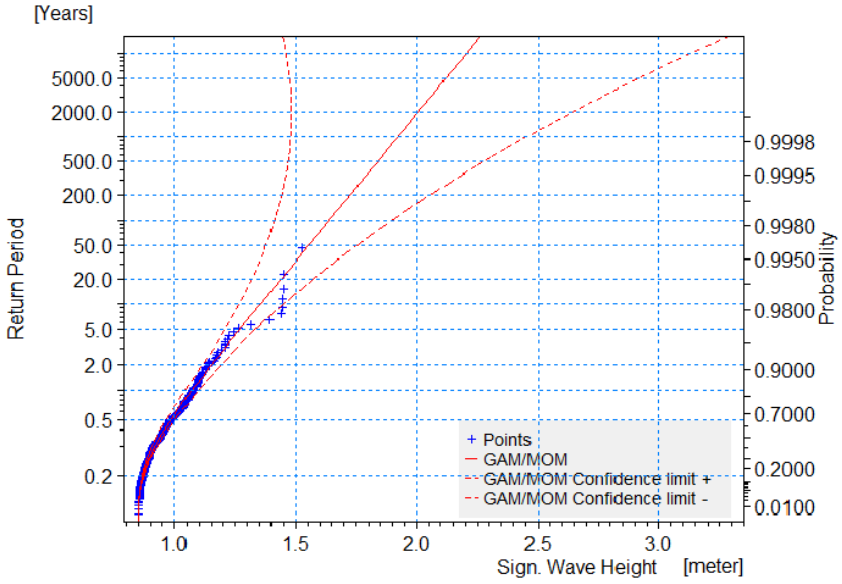
The EVA of Hs was undertaken at the location shown in Figure 7.3a, situated offshore at the boundary of the local spectral wave model domain. The EVA of Hs considered various probability distribution and estimation methods to calculate the extreme Hs for the JPA (Section 7.4). The best-fitted curve is shown in Figure 7.4, with lines showing the 95% confidence intervals. The estimated extreme Hs values for the present day 2025 are presented in Table 7.2. The 46-year wave rose (Figure 7.3b) indicates that the dominant mean wave direction ranges from the northwest to the northeast.

Figure 7.3: Location of Wave Data Extraction for EVA.



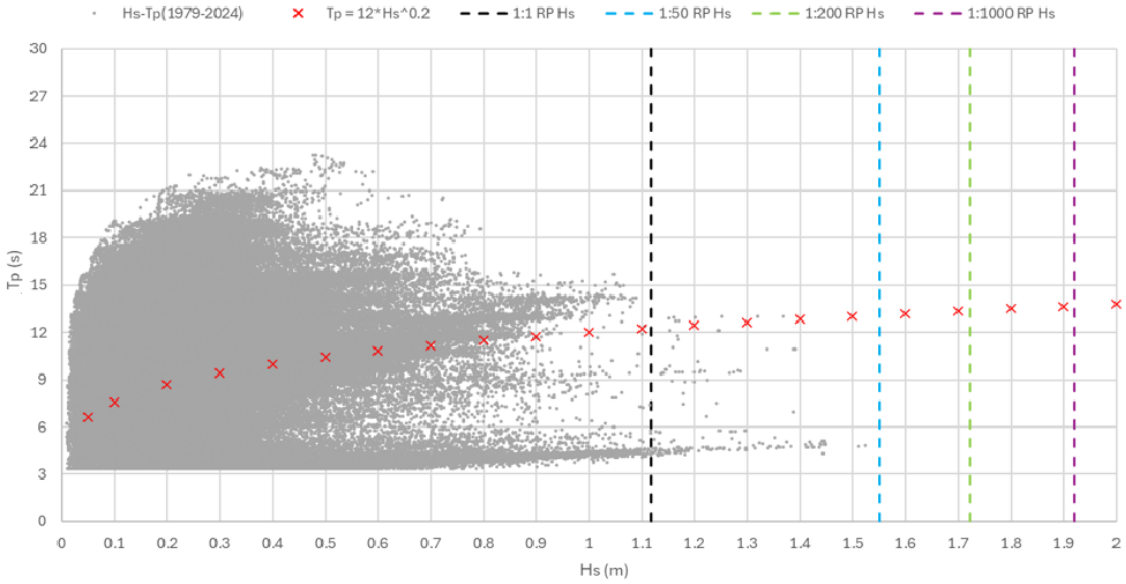
Source: Mott MacDonald, 2026.

Figure 7.4: EVA Probability Distribution Fit of H_S.



Extreme peak wave periods (T_P) were defined by the relationship between the 46-year predicted T_P and H_S values as shown in Figure 7.5. The calculated extreme T_P values are tabulated in Table 7.2 for the epoch year 2025.

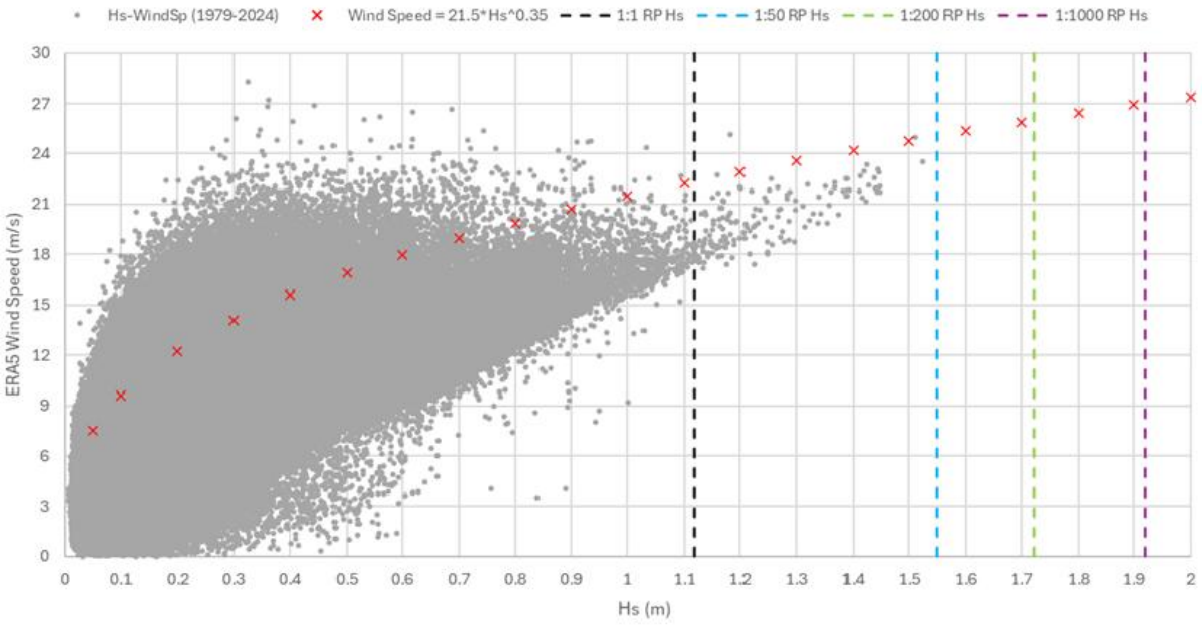
Figure 7.5: Hs and Tp relationship



Source: Mott MacDonald, 2026.

The extreme wind speeds were derived from the relationship between ECMWF ERA5 data and Hs. The relationship between wind speed and Hs over the 46-year period is shown in Figure 7.6. For consideration of the 45-yr and 100-yr epochs, the Hs is projected to increase by +20% as described in Section 4.11.3. Table 7.2 shows the extreme waves and wind speeds for epoch 2025, while Table 7.3 shows the extremes for epoch 2070 and 2125 of the 1-in-50, 200 and 1,000-yr return period.

Figure 7.6: Hs and Wind speed relationship.



Source: Mott MacDonald, 2025.

Table 7.2: Extreme H_s, T_p and Wind Speeds (for 2025).

Return period (year)	Hs (m)	Tp (s)	Wind Speed (m/s)
50	3.06	15.0	31.8
200	3.15	15.1	32.1
1000	3.24	15.2	32.5

Source: Mott MacDonald, 2026

Table 7.3: Extreme H_s, T_p and Wind Speeds (for 2070 and 2125).

Return period (year)	Hs (m)	Tp (s)	Wind Speed (m/s)
50	3.67	15.6	33.9
200	3.77	15.7	34.2
1000	3.89	15.8	34.6

Source: Mott MacDonald, 2026

7.4 Joint probability analysis

Joint Probability Analysis (JPA) is a statistical approach used to evaluate the combined likelihood of two or more interdependent variables occurring simultaneously. In this study, the JPA method was used to define the return period of combined extreme water levels and waves. The simplified JPA FD2308 tool from Defra and Environmental Agency Flood and Coastal Defence (Hawkes & Svensonn, 2005)[21] was used to perform the analysis which is aligned with the SEPA (2015) flood model guidance. The tool requires input of extreme wave heights as defined in Section 7.3 and extreme water levels (Section 7.2).

The results of the JPA for the years 2025, 2070, and 2125 are presented in Table 7.4, Table 7.5 and Table 7.6, respectively. It is noted in Section 4.11.3 that H_s is increased by +20% for epoch 2070 and 2125, along with corresponding increases in wave periods and wind speeds, as defined by the relationships in Figure 7.5 and Figure 7.6, respectively.

JPA and additional local wave transformation modelling were not undertaken for the 2100 epoch. Derivation of the 2100 wave conditions at the toe of the structure, and calculation of wave overtopping rates for this epoch are detailed in Section 9.6.

[21] Use of Joint Probability Methods in Flood Management: A guide to best practice. R&D Technical Report FD2308/TR2. Defra and Environmental Agency, March 2005.

Table 7.4: JPA of Hs and Water Level with corresponding wind speed and Tp (2025)

Wind Speed (m/s)	Tp (s)	Hs (m)	Joint exceedance return period (years)		
			50	200	1000
			Extreme Water Levels (Baseline 2025) m ODN		
19.82	11.45	0.79	3.06	3.15	3.25
20.33	11.62	0.85	3.06	3.15	3.25
20.73	11.75	0.90	3.06	3.15	3.25
21.45	11.98	0.99	3.02	3.13	3.25
22.01	12.16	1.07	2.96	3.08	3.21
22.44	12.30	1.13	2.92	3.03	3.17
22.88	12.43	1.19	2.88	2.99	3.12
23.47	12.62	1.29	2.82	2.94	3.07
23.93	12.76	1.36	2.78	2.89	3.03
25.04	13.09	1.55	2.68	2.80	2.93
25.53	13.24	1.63		2.75	2.89
26.03	13.38	1.73		2.71	2.84
26.70	13.58	1.86			2.79
27.23	13.73	1.96			2.75

Source: Mott MacDonald, 2026.

Table 7.5: JPA of Hs and Water Level with corresponding wind speed and Tp (2070)

Wind Speed (m/s)	Tp (s)	Hs (m)	Joint exceedance return period (years)		
			50	200	1000
			Extreme Water Levels (Epoch 2070) m ODN		
20.21	11.58	0.84	3.62	3.74	3.87
20.86	11.79	0.92	3.62	3.74	3.87
21.36	11.96	0.98	3.62	3.74	3.87
22.41	12.29	1.13	3.56	3.72	3.87
23.13	12.51	1.23	3.49	3.64	3.82
23.69	12.69	1.32	3.43	3.58	3.77
24.27	12.86	1.41	3.37	3.53	3.71
25.05	13.10	1.55	3.29	3.45	3.63
25.66	13.28	1.66	3.24	3.39	3.57
27.13	13.71	1.94	3.10	3.26	3.44
27.79	13.90	2.08		3.20	3.38
28.47	14.09	2.23		3.14	3.33
29.39	14.35	2.44			3.25
30.10	14.54	2.62			3.19

Source: Mott MacDonald, 2026.

Table 7.6: JPA of H_s and Water Level with corresponding wind speed and T_p (2125)

Wind Speed (m/s)	Tp (s)	Hs (m)	Joint exceedance return period (years)		
			50	200	1000
			Extreme Water Levels (Epoch 2125) m ODN		
20.21	11.58	0.84	4.43	4.54	4.67
20.86	11.79	0.92	4.43	4.54	4.67
21.36	11.96	0.98	4.43	4.54	4.67
22.41	12.29	1.13	4.37	4.52	4.67
23.13	12.51	1.23	4.30	4.45	4.62
23.69	12.69	1.32	4.24	4.39	4.57
24.27	12.86	1.41	4.19	4.34	4.51
25.05	13.10	1.55	4.11	4.27	4.44
25.66	13.28	1.66	4.06	4.21	4.38
27.13	13.71	1.94	3.93	4.08	4.26
27.79	13.90	2.08		4.03	4.20
28.47	14.09	2.23		3.97	4.15
29.39	14.35	2.44			4.07
30.10	14.54	2.62			4.02

8 Local wave model

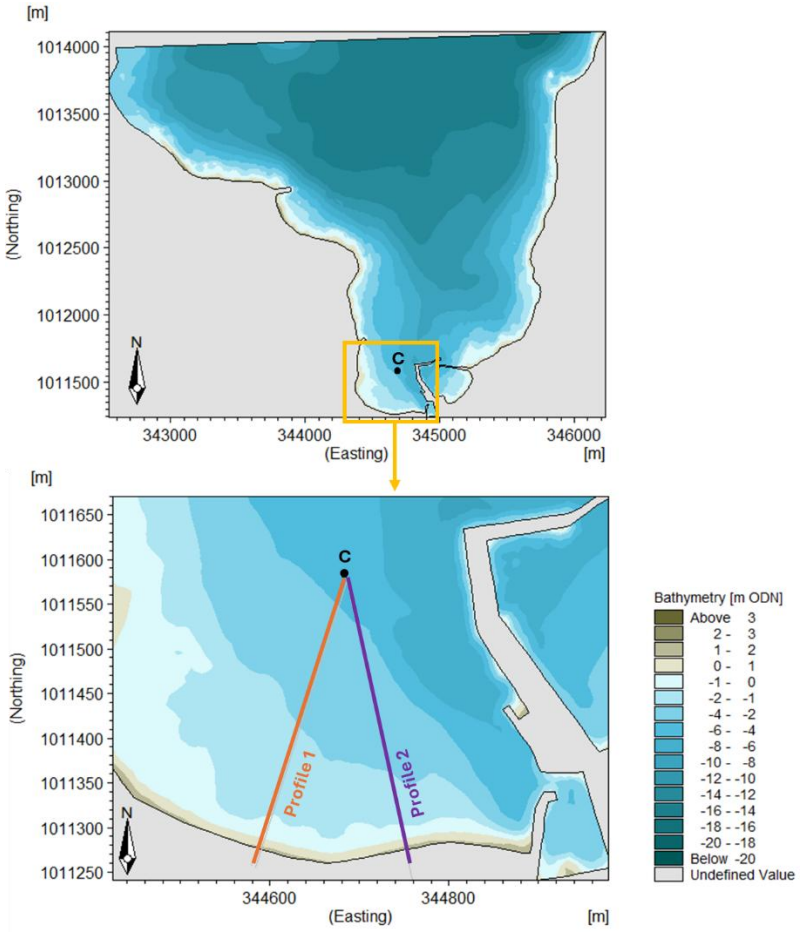
8.1 Introduction

Model calibration was not performed due to absence of local measured wave data. The local high-resolution MIKE21 FM SW model was configured using boundary conditions extracted from the calibrated regional model outputs employed in the JPA. The primary objective of the local SW model is to simulate extreme wave conditions for application as boundary conditions in one-dimensional (1D) wave transformation modelling (Section 8.6).

8.2 Local model domain, bathymetry and mesh

Bathymetric data as described in Section 4.2 were used in the local model. Figure 8.1 shows the model domain, bathymetry, and the location of extraction point which define the starting positions of the profiles used for 1D wave transformation (Section 8.6).

Figure 8.1: Local MIKE21 FM SW model domain, bathymetry with the location of extraction point (C) for 1D wave transformation

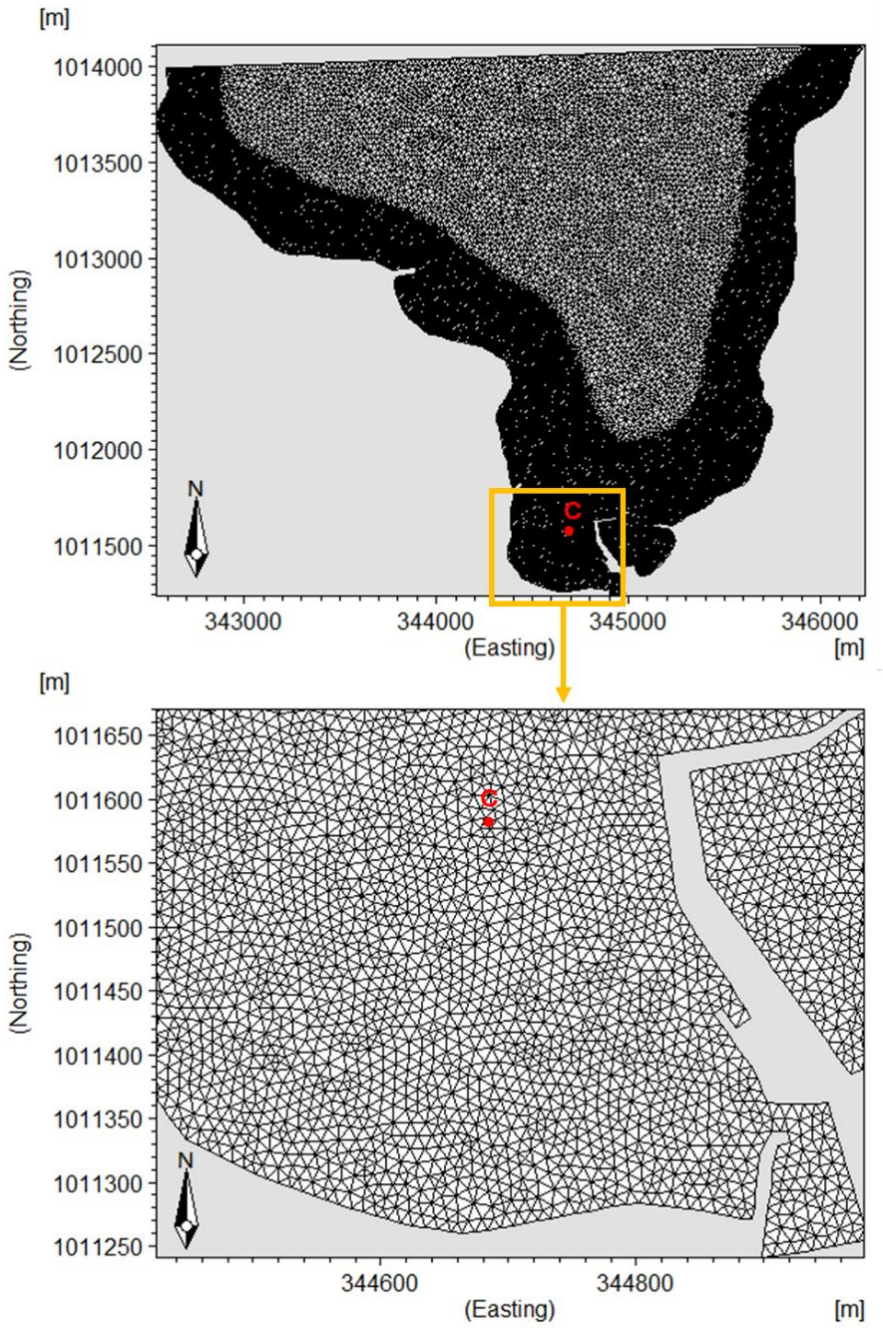


Source: Mott MacDonald 2026.

Figure 8.2 shows the model mesh for the whole domain with an enlarged view around the study area. The mesh is coarser in the offshore region (10 to 20 m approximately), while in the

nearshore areas within the study area the element sides of the mesh are approximately 5 to 10 m.

Figure 8.2: Local MIKE21 FM SW model mesh.



Source: Mott MacDonald 2026.

8.3 Local model boundary conditions and forcings

The wave parameters (Hs and Tp), wind speeds, and water levels were obtained from the JPA model outputs (Section 7.4). Wind direction and mean wave direction (MWD) were discretised at 22.5° intervals, covering the sector from northwest to northeast (i.e. four sectors; 340 deg.N to 47.5 deg.N).

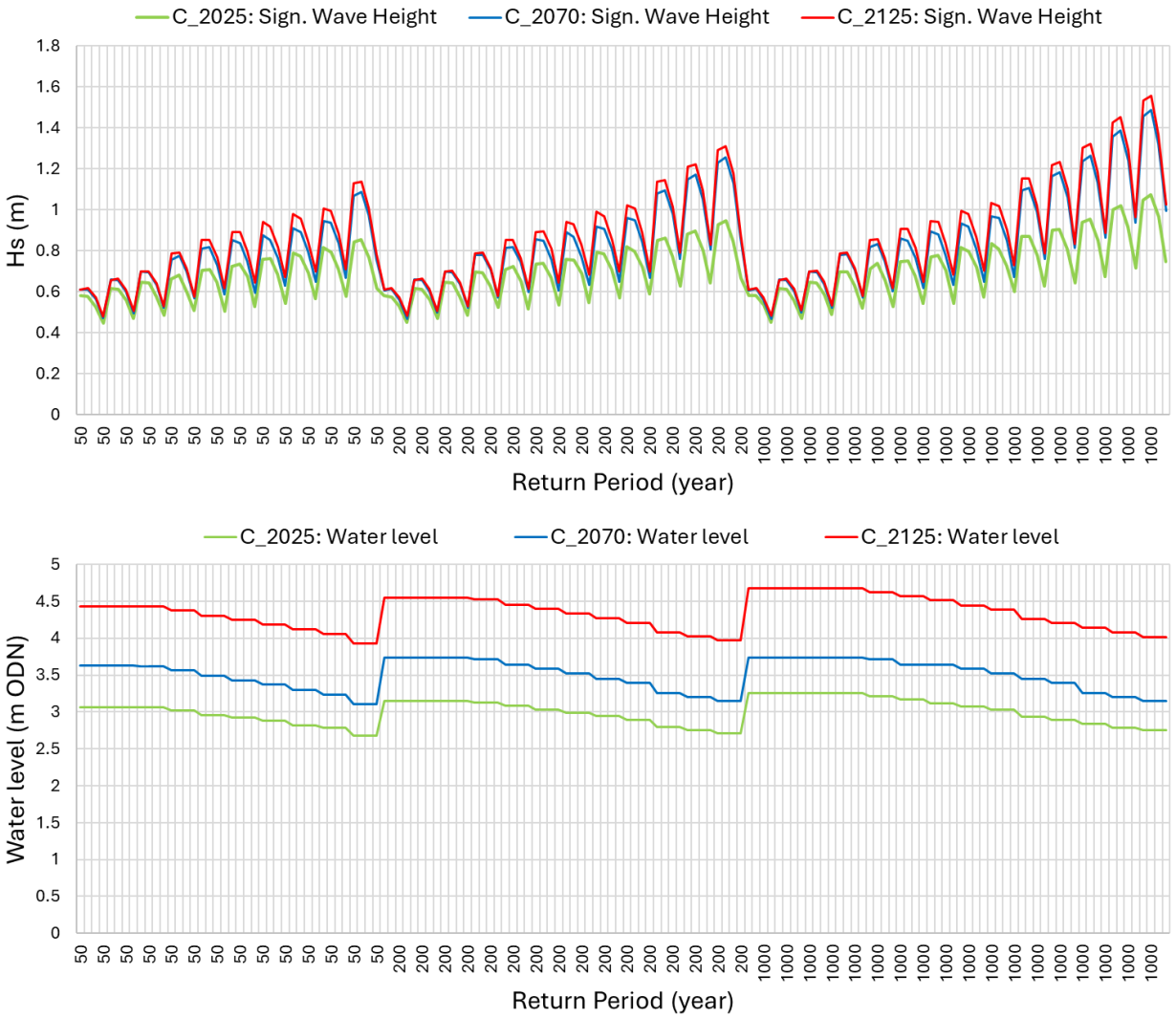
8.4 Model setup

Following the standard modelling procedure, several iterative sensitivity tests were undertaken in which key wave parameters in the model setup were tuned to achieve the best model setup. The key parameters used in the local wave model setup are summarised in Appendix B.

8.5 Model results

Figure 8.3 presents the Hs results at location C for 1-in-50, 1-in-200, and 1-in-1,000-year return periods for the years 2025, 2070, and 2125. The corresponding water level inputs from the JPA are also plotted. Results are shown for four discrete directional sectors, each simulated for every return period within each epoch year. The results demonstrate that Hs increases with both increasing return period and advancing epoch year, as anticipated. These findings confirm that the local SW models were correctly configured and simulated, providing reliable estimates of extreme wave conditions within Kirkwall Bay and ensuring their suitability as boundary conditions for 1D wave transformation modelling.

Figure 8.3: Local MIKE21 FM SW model results at location C, corresponding to 1-in-50, 200 and 1,000-year return periods, under various water level scenarios based on the JPA inputs for years 2025, 2070 and 2125.



Source: Mott MacDonald 2026.

8.6 1D wave transformation (using LITDRIFT)

The local MIKE21 SW model resolution of ~5 m was not considered adequate to resolve the nearshore seabed in sufficient detail. Therefore, the MIKE by DHI 1D LITDRIFT[22] wave model within the LITPACK software package was used to transform the nearshore local wave model results to the toe of the proposed new structure based on a much higher (1m) horizontal resolution profile. The LITDRIFT wave model calculates wave propagation towards the coast and includes the key physical wave processes such as shoaling, refraction, diffraction and wave-breaking[23]. It is noted that a 1D wave transformation scheme is considered appropriate

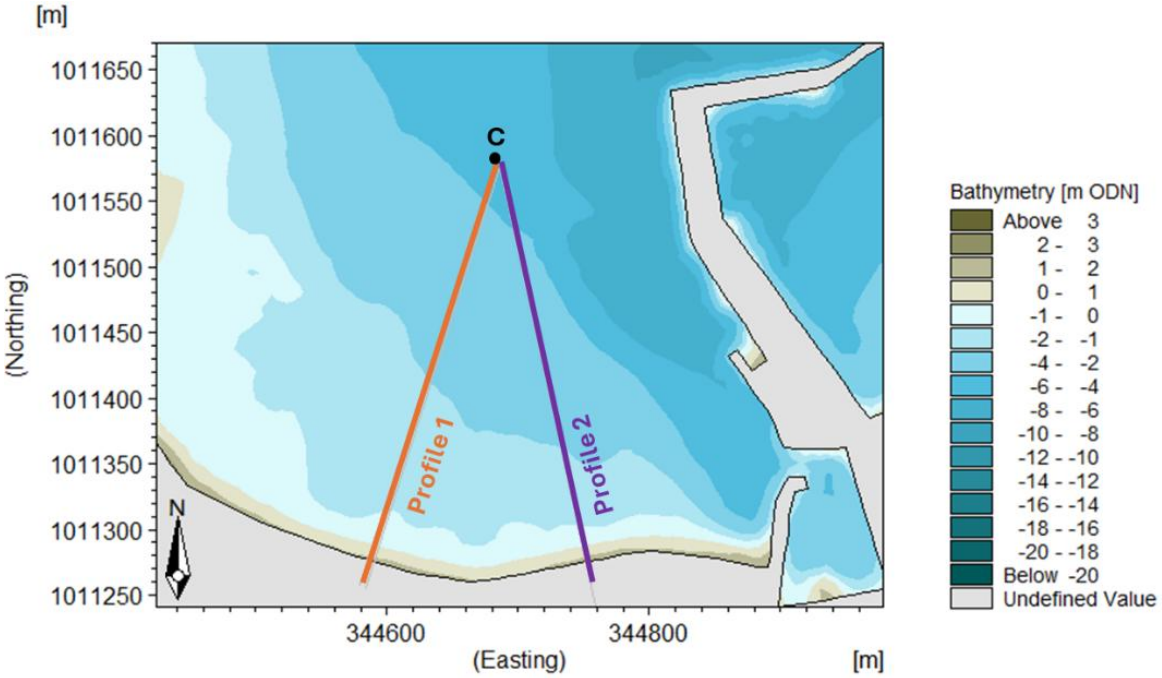
[22] DHI (2024), Littoral Processes FM: All Modules Scientific Documentation, Littoral Processes FM - All Modules (mikepoweredbydhi.help) accessed on 16 February 2026.

[23] DHI (2024), Littoral Processes Module, Littoral Processes FM (mikepoweredbydhi.help) accessed on 16 February 2026.

and sufficient for this purpose given the limited cross-shore extent and the relatively uniform bathymetric conditions.

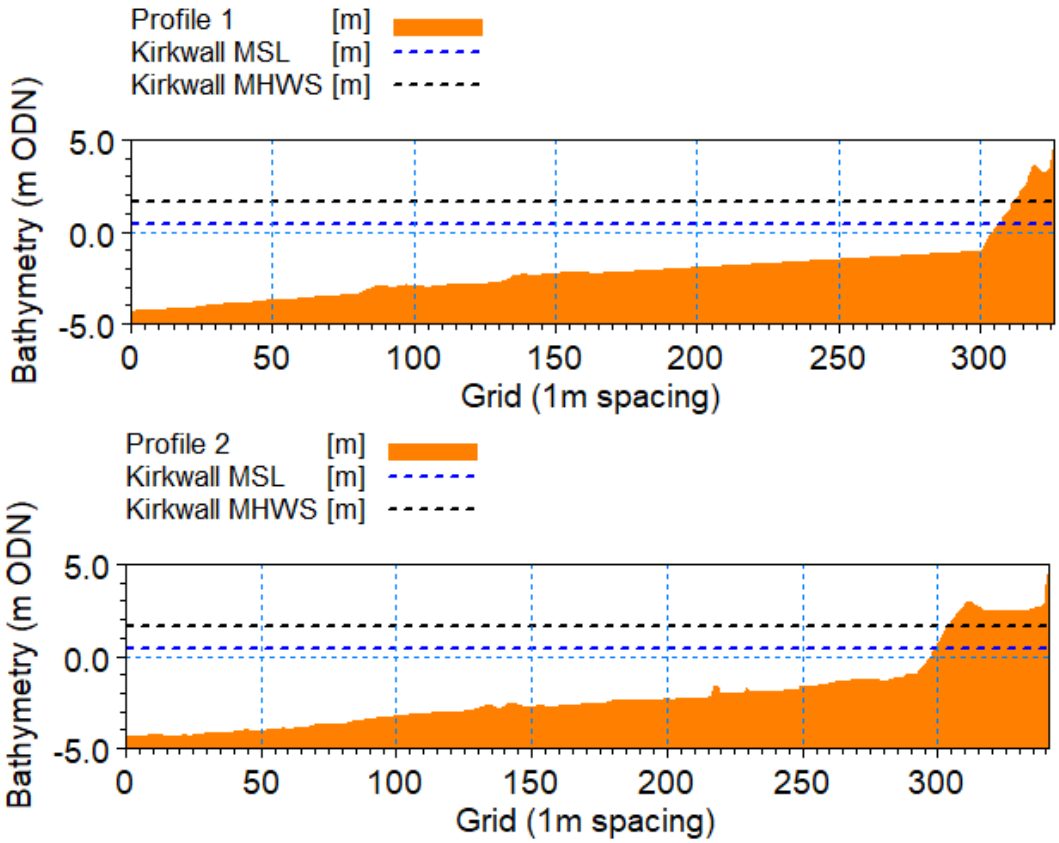
The LITDRIFT model requires nearshore waves and water level conditions from the nearshore transformed wave data produced from the local MIKE21 SW model results (Section 8.5). Two cross-shore profiles were established, extending landward to the existing flood defences (Section 4.8), and were constructed using the bathymetric data described in Section 4.2. Figure 8.4 shows the profile locations, and Figure 8.5 shows the profiles at each location.

Figure 8.4: Plan view of the profiles used in 1D wave transformation model.



Source: Mott MacDonald 2026.

Figure 8.5: Profiles used in 1D wave transformation model.



Source: Mott MacDonald 2026.

LITDRIFT was simulated for the baseline year, 2025 and the years 2070 and 2125 using Hs, peak wave Tp, water levels (WL) and mean wave direction (MWD) data from the local MIKE21 SW results of the JPA conditions (Section 8.5). Transformed wave data were then extracted at the toe of the structure for each profile for use in the subsequent overtopping calculations as detailed in Section 9.

Local wave and 1D transformation simulations for the 2100 epoch were not undertaken. Transformed wave inputs for the 2100 1-in-200-year return period overtopping assessment were derived from the simulations for other epochs as described in Section 9.6.

9 Overtopping assessment

9.1 Introduction

This section of the report details two investigations into the overtopping of the flood defences along the waterfront in Kirkwall.

The first part of this assessment is the development of overtopping predictions of the sea defences to feed into the second part of the local overland hydrodynamic flow model (OFM, detailed in chapter 11) for the Bay of Kirkwall. Overtopping calculations have been undertaken at several locations along the Kirkwall waterfront to generate time series of discharge rates for storm events with return periods of: 1-in-50, 1-in-200, and 1-in-1,000 years, in the present day, 2025, and at the end of the appraisal period, 2125. These overtopping rates were derived in accordance with the EurOtop 2018 manual[24]. The resulting time series form key inputs to the OFM Kirkwall which then aims to evaluate the flooding volumes landwards of the defences. For comparison with SEPA's 2100 flood mapping, an additional overtopping calculations were developed for the 2100 1-in-200-year return period event.

A separate overtopping assessment was also undertaken for a mid-epoch (2070) to evaluate the changing standard of protection of the waterfront defences over time. This check did not involve producing a time series and is not used in flood modelling. As per the scope, overtopping calculations were performed at three locations along the waterfront, taking profiles that sufficiently characterize the frontage.

This portion of the report is split into the following sections, which discuss the steps taken to achieve the required outputs:

1. **Purpose** (this section)
2. **Profile Selection**
    a. Overview of the coastal defence structures in place
    b. Selection of profile locations and characteristics of the defences at each of the three profiles
3. **Calculation of the overtopping time series**
    a. Overview of overtopping tool and approach to obtaining outputs to drive the OFM
    b. Topographic data used to determine inputs
    c. Profile inputs into the overtopping tool
    d. Hydraulic inputs into the tool for:
        i. Determining the pairing of extreme water and waves that result in the worst overtopping
        ii. Generating the overtopping time series from the worst-case pairs
4. **Inputs for the OFM**
    a. The worst-case wind and wave pairs that drive overtopping
    b. The output time series
5. **Mid-Epoch Assessment**
    a. The approach to determining a year for the mid-epoch, when the standard of protection of the defence changes
    b. Expected overtopping in the mid-epoch year
6. **2100 1-in-200-year return period overtopping inputs**

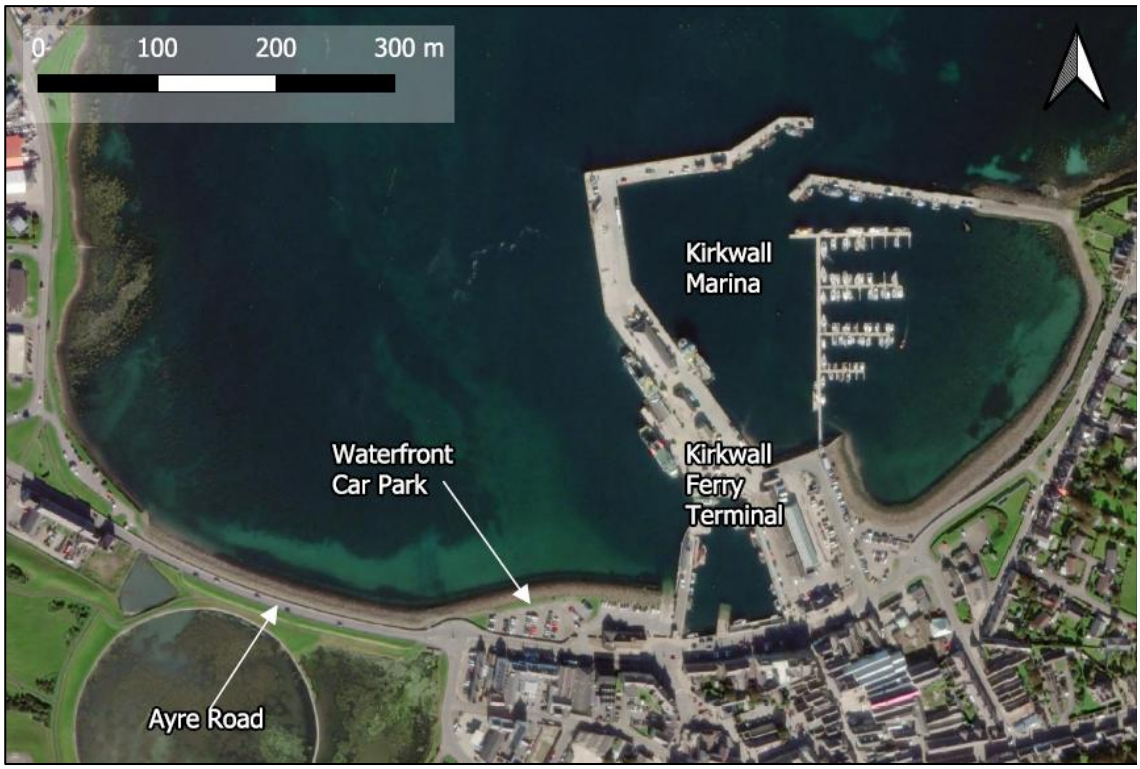
[24] Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B, EurOtop 2018.

a. Selection of the donor dataset
b. Development of the water-level and transformed wave time series
c. The output time series

9.2 Profile selection

The section of waterfront included in the analysis is shown in Figure 9.1 and spans approximately 1 km of coastline.

Figure 9.1: Overview of Kirkwall Seafront

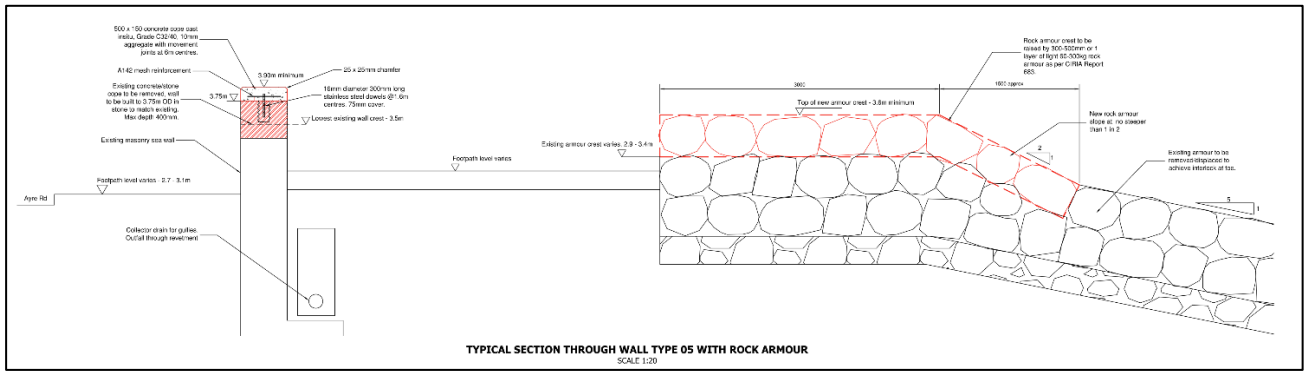


Source: Mott MacDonald, 2026. Esri, DigitalGlobe, GeoEye, i-cubed, USDA FSA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community.

Kirkwall Marina is likely to dissipate most of the energy from waves approaching the east of the frontage against the harbour arms. Therefore, the most significant overtopping is expected to occur to the west of the bay. Here, there is significant fetch length available for wave build-up approaching the defences running parallel to Ayre Road and the Waterfront Car Park. The three sections are therefore to be situated along this portion of the waterfront.

This length of frontage is defended by a revetment of rock armour, and a seawall set further back behind a promenade/car park. The defence has several breaks in it which are to be sealed with flood gates. For this assessment, the flood gates will be assumed to be in place and closed, so the standard of protection (SOP) of the defence is as designed. Improvements to the defence were carried out in 2016, designed to an SOP of a 1-in-200-year storm event. The crest of the seawall was raised along the entire waterfront to +3.9 mOD. The rock armour revetment was raised from +2.9-3.4 mOD to +3.6 mOD along the length of Ayre Road. The armour fronting the carpark was not raised during this intervention. Figure 9.2 shows the as-built for the section of the defence running along Ayre Road.

Figure 9.2: As-built section of defence along Ayre Road

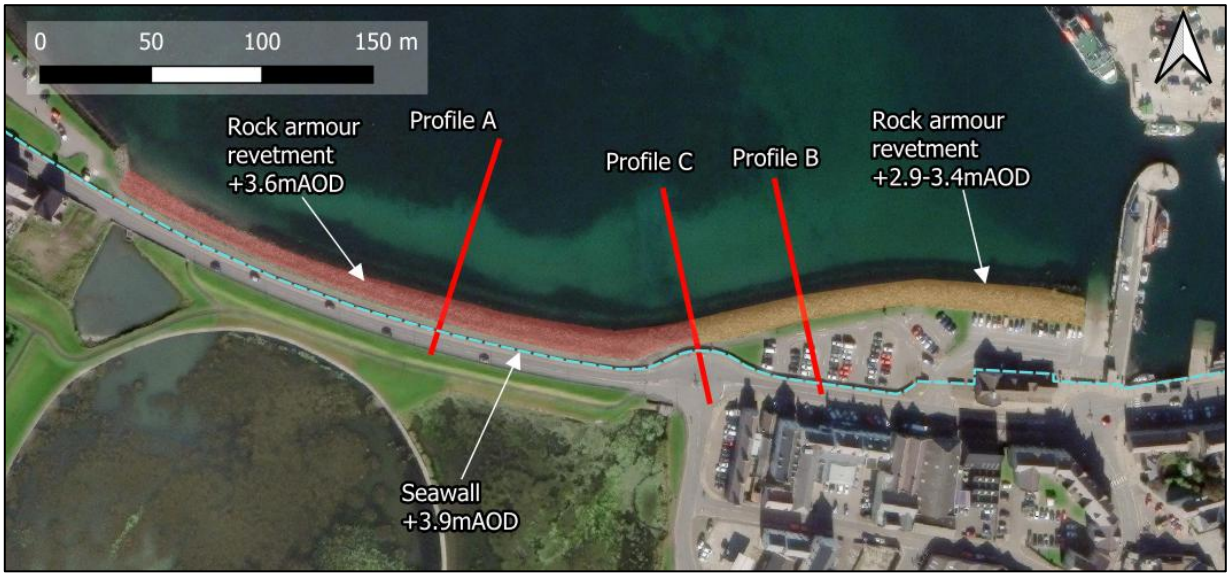


Source: Orkney Islands Council, 2016

To represent the differences in the promenade and armour crest level along Ayre Road and the car park, three locations were selected along the frontage, shown in Figure 9.3. Profile A has the raised armour crest of +3.6 mOD, and a promenade width of 7m. Profile B is taken across the car park with a lower armour crest of +2.9 mOD, but a significantly wider promenade, between 20 and 40m. These two profiles characterise the majority of the length of the defence.

A critical point with potential for high overtopping was identified at the region of transition between the original armour crest and the raised level, backed by a narrow promenade. This was taken to be Profile C. Whilst high overtopping was anticipated at this location, the length of defence with this combination of low armour crest and narrow promenade is relatively short; approximately 30m of the 400m run of defence shown in Figure 9.3.

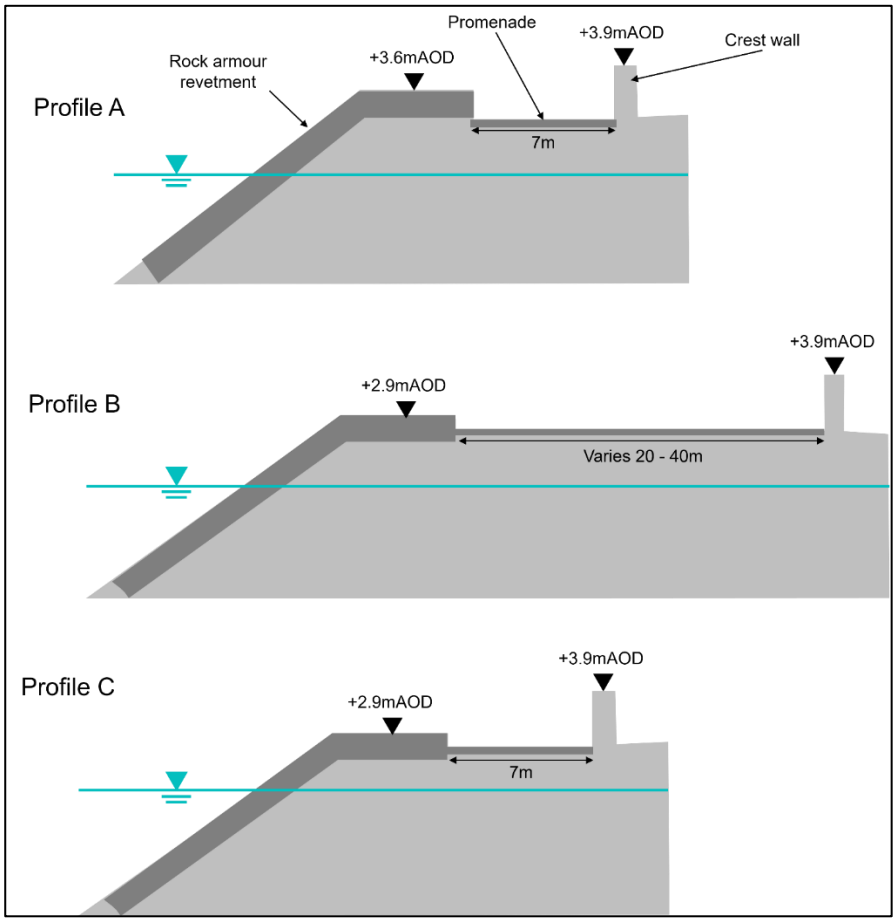
Figure 9.3: Location of sections along the frontage



Source: Mott MacDonald, 2026. Esri, DigitalGlobe, GeoEye, i-cubed, USDA FSA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Simplified geometry representing the key parameters of the three profiles are shown in Figure 9.4.

Figure 9.4: Comparison of defence section for each profile



Source: Mott MacDonald, 2026

9.3 Calculation of the overtopping time series

9.3.1 In-House Overtopping Tool

Mott MacDonald's in-house overtopping tool has been built in line with EurOtop 2018 with an internal selection tree that automatically identifies and applies the appropriate equations based on the input data provided. Generally, the rubble mound revetment methodology outlined in Chapter 6 of EurOtop was used for the assessments, though the tool selects the most appropriate methodology on a case-by-case basis, based on the design parameters input into the package.

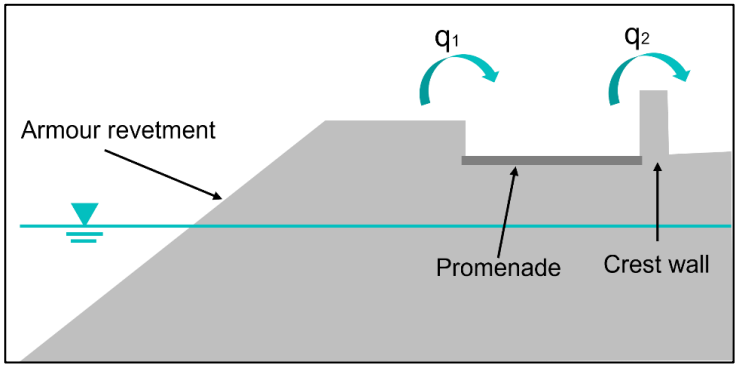
The tool takes both structural and hydraulic inputs. The structural inputs represent a profile of the coastal defence. The hydraulic inputs represent the wave and water conditions that the defence is subject to.

Because the purpose of the assessment is to understand residual inland flood risk from overtopping of the defences, the output taken from the tool is wave overtopping discharge, q, in litres per second per metre. No consideration is given to peak wave overtopping volume, which is a consideration only for damage to the defence or risk to people near the site of the wave overtopping.

Once the discharge time-series for each profile has been developed, this input is to be applied in the OFM to a length along the defence which the profile is considered to characterise.

In this assessment, overtopping discharge (q) has been calculated at two 'points of investigation' in each profile. The first is at the rear of the rock armour revetment (q_1) – this represents the discharge to the rear of the rock armour that flows over the area between the rock armour and the crest wall and will show any inundation seaward of the crest wall, which may have some consequence in the Waterfront car park. The second is the discharge over the crest of the sea wall (q_2). Capturing q_2 shows the discharge landward of the defence, which will be the primary contribution to the OFM. Figure 9.5 shows the position where the two discharges are calculated on an idealised profile of the defences. Figure 9.6 shows the two points of investigation, for the different profiles, that are to be applied within the OFM.

Figure 9.5: Flow outputs required for each profile



Source: Mott MacDonald, 2026

Figure 9.6: Flow Lines to be input into model



Source: Mott MacDonald, 2026. Esri, DigitalGlobe, GeoEye, i-cubed, USDA FSA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

For profiles A and C the inputs to the OFM are q_2, over the rear crest wall. Due to the promenade width of profile B, no waves are assumed to propagate to the rear wall of the car park. Hence, q_1 values are input to the OFM with the rear crest wall being included as a flood barrier in this model (the q_2 overtopping discharge is assumed to be zero).

Considering the proximity, comparable obliquity, and matching armour crest levels of profiles B and C, the hydraulic inputs for profile B were also used for profile C to simplify extraction of data from the regional hydraulic model. Additionally, C_q1 is represented by B_q1 as the geometry of the two profiles up to the crest of the armour is identical.

9.3.2 Available data

The input data used to develop the structural and hydraulic inputs were as follows:

9.3.2.1 As-builts

As-builts of the defences along the waterfront are available from works on the defences completed during 2016. These give the extents of the defence, crest levels, and slopes of revetments. The levels given within the as-builts are assumed to provide more accurate measurements than the topographic data.

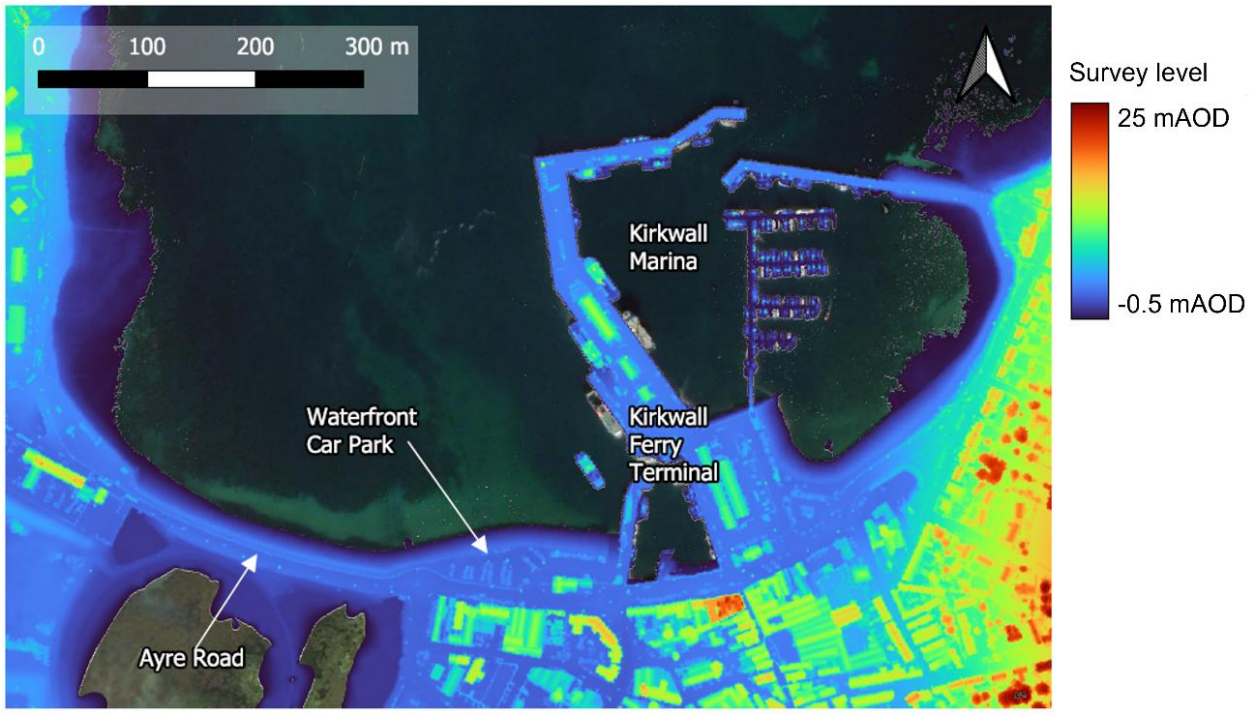
9.3.2.2 LiDAR

LiDAR from 2023, provided by Orkney Islands Council (OIC), covers the Orkney islands to a resolution of 0.5m. Two tiles – HY4411 and HY4511 – cover the entirety of the Kirkwall coastal frontage.

For input into the regional model, this was resampled to a resolution of 1m. The survey was performed at a water level of -0.5 mOD; as LiDAR does not penetrate the surface of the water, data below this level has been clipped. The defence extends below this level, so the position of the toe and slope of the foreshore cannot be determined from the LiDAR.

Profiles of the frontage were extracted from the digital surface model (DSM) as it contained finer details, such as the seawall running along the back of the promenade. From the DSM, the chainage, level, and coordinates of the profiles could be extracted and used for further calculation

Figure 9.7: Extent of LiDAR data



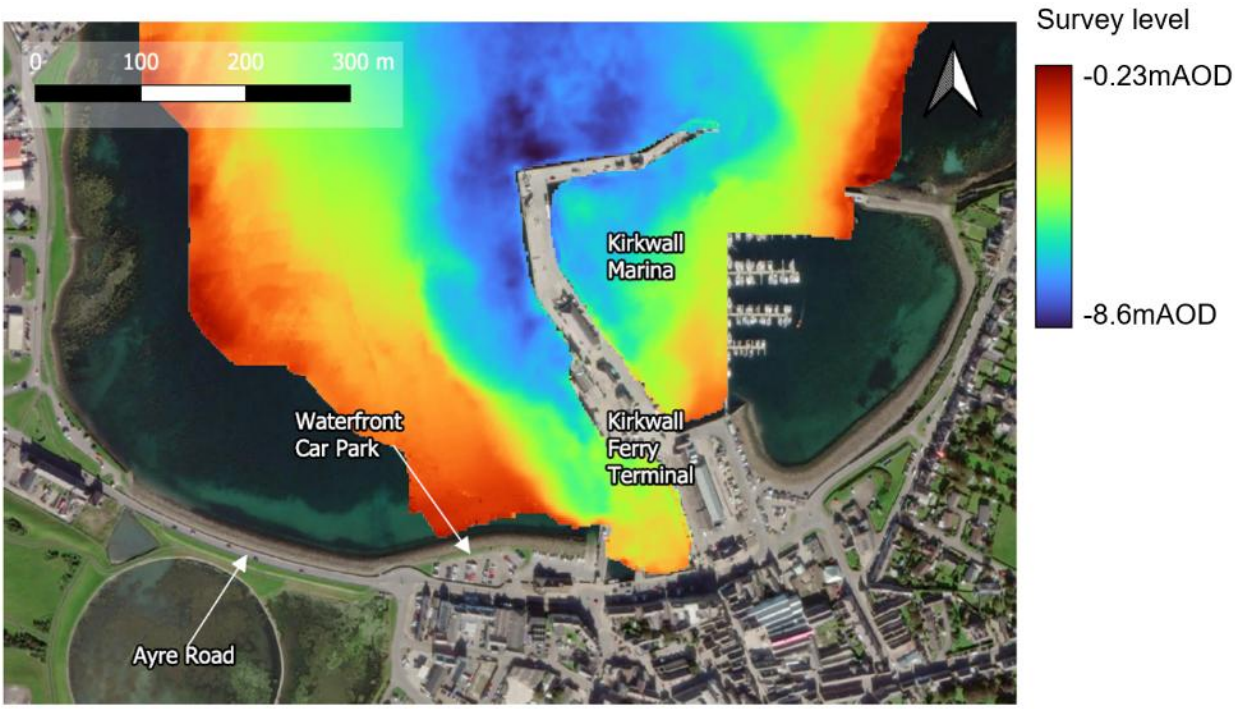
Source: Mott MacDonald, 2026. Orkney Islands Council, 2023. Esri, DigitalGlobe, GeoEye, i-cubed, USDA FSA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

9.3.2.3 Bathymetry

Bathymetry data was sourced from a combination of surveys produced by the UK Hydrographic Office, from 2009, 2020, and 2023. The levels of the survey data were converted from metres above Chart Datum (mCD) to metres above Ordnance Datum (mOD). These were combined to form a single piece of bathymetry as the levels on each of the surveys appeared to be in good agreement.

As these surveys were taken at relatively low tides, the bathymetry of the foreshore of profile A is not available, so assumptions will be made in the determination of foreshore profile, detailed in 9.3.3.1.

Figure 9.8: Extent of Bathymetry data

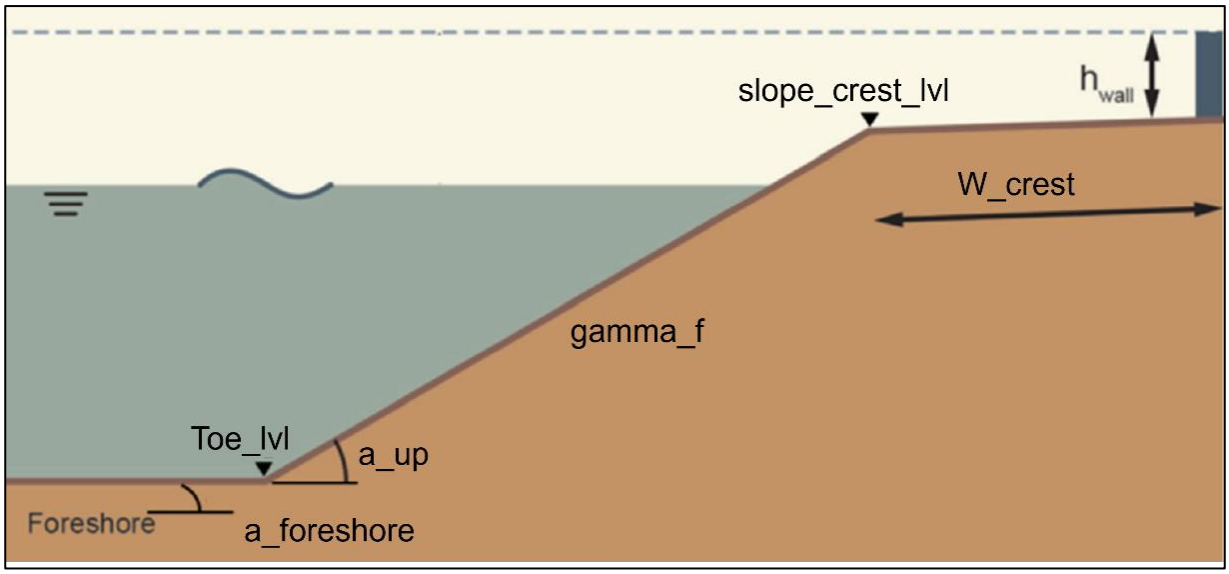


Source: Mott MacDonald, 2026. Esri, DigitalGlobe, GeoEye, i-cubed, USDA FSA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community. Contains United Kingdom Hydrographic Office data © Crown copyright and database right.

9.3.3 Profile inputs into the overtopping tool

The overtopping tool takes an idealised form of the defence - in this case consisting of a revetment, promenade, and seawall. The parameters required to discretise the defence for input into the tool are illustrated in Figure 9.9. and detailed in Table 9.1.

Figure 9.9: Overtopping tool structural inputs



Source: Overtopping Tool Documentation. Mott MacDonald, 2021.

Table 9.1: Overtopping tool structural inputs

Parameter	Description	Comment
cot_a_up	Slope angle of the revetment	Taken from as-builts
slope_crest_lvl [m]	Level at the crest of the upper slope	Taken from as-builts
crest_w [m]	Width of the crest of the defence	From as-builts. See Figure 9.10
toe_lvl [m]	Level of the toe of the structure	See 9.3.3.1
a_foreshore [deg]	Inclination of the foreshore	See 9.3.3.1
h_wall [m]	Height of the crest wall	Taken from as-builts
a_defence [deg]	Angle of the defence relative to north	Taken as the bearing of the section
h_bullnose [m]	Bullnose height	0 for all sections as no bullnose
a_bullnose [deg]	Inclination of bullnose	0 for all sections as no bullnose
gamma_f	Roughness reduction factor	See 9.3.3.2
battered_wall_angle [deg]	Batter wall slope angle	90 (vertical wall) for all sections
Rubble_mount_flag	If revetment is rubble mount or not	True for all sections

Source: Mott MacDonald, 2026

9.3.3.1 Foreshore slope and toe position

Due to the differing water levels at the time that the bathymetric and LiDAR surveys were taken, the position of the toe for profile A and the slope of the immediate foreshore could not be extracted from either dataset. The slope of the foreshore was assumed based on the available bathymetry further offshore. Using the slope of the revetment, detailed in the as-builts, the point of intersection of the armour and the foreshore slope was taken as the toe of the structure in this section. As the bathymetry for profile B had greater coverage, showing a linear foreshore and the elevation of the toe - similar to the level calculated for A - the assumptions made for profile A were considered reasonable.

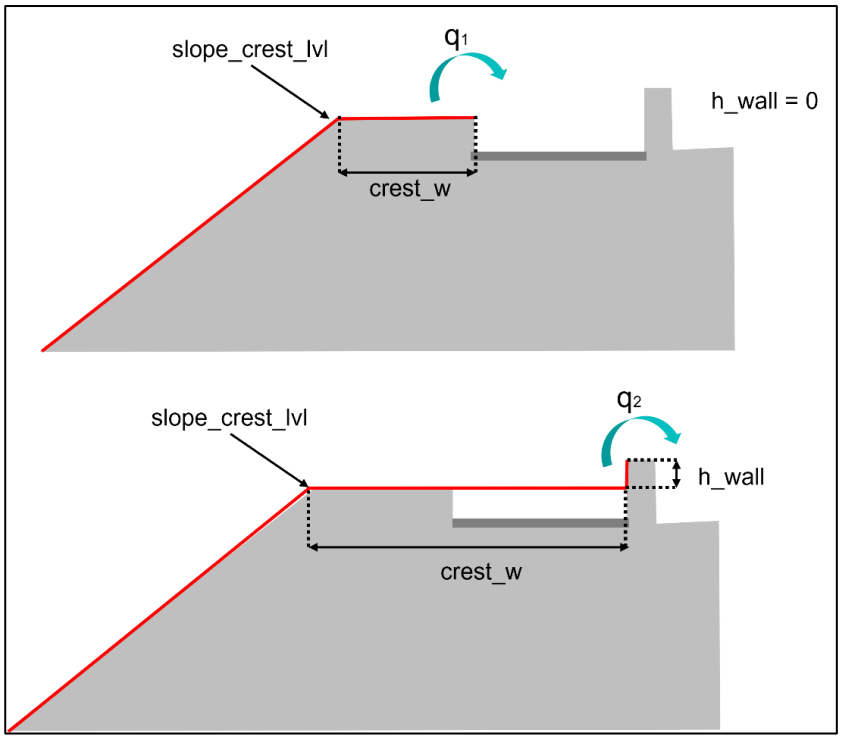
9.3.3.2 Roughness Reduction factor

The roughness reduction factor (γ_f) for rubble mound structures is given in Table 6.2 in EurOtop. For profile A, which had an additional layer of rock armour added when the crest level was raised in 2016, the value for γ_f was taken to be 0.55, corresponding to 2 layers of rock armour with an impermeable core. As the revetment at profiles B and C were not raised, the reduction factor was taken to be 0.6, corresponding to a single layer of rock armour with an impermeable core.

9.3.3.3 Calculation of q_1 and q_2

To calculate the discharges q_1 and q_2, two different model geometries need to be input into the tool, ending at the rear of the defence, and crest of the wall respectively. Figure 9.10 shows the geometries required to output the different flowrates, indicated by the red lines. The promenade width is conservatively assumed to be level with the crest of the armour.

Figure 9.10: Required geometry inputs to capture q_1 and q_2

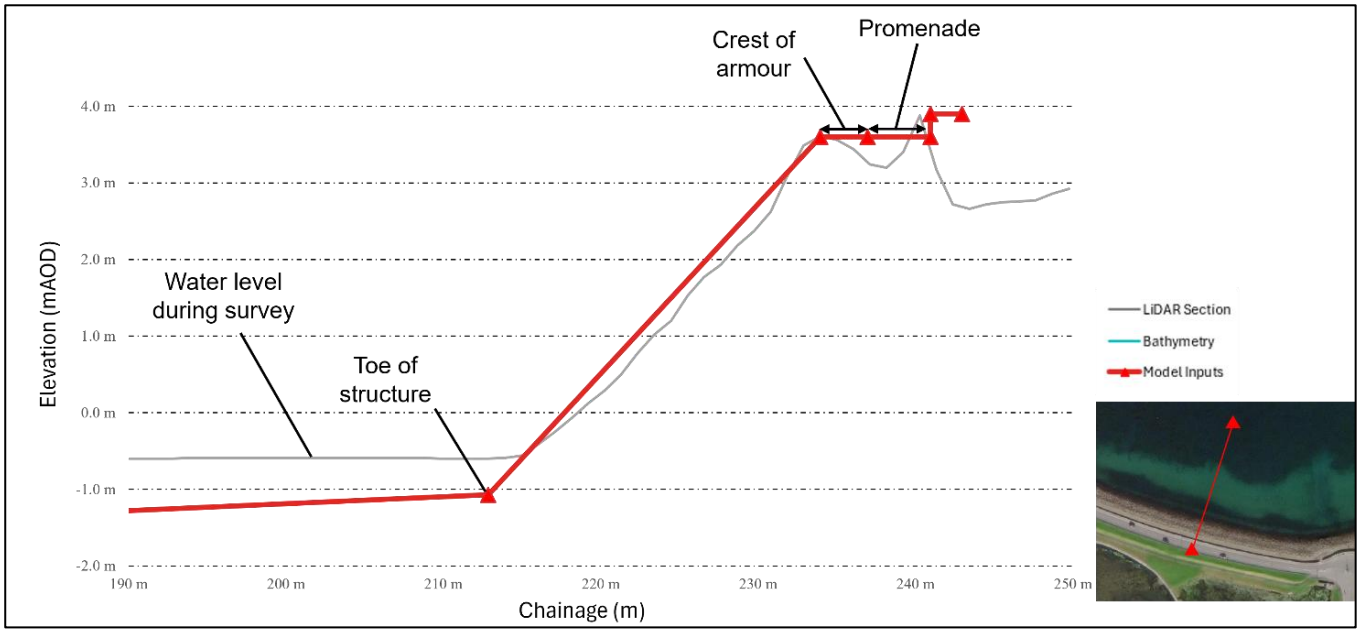


Source: Mott MacDonald, 2026

9.3.3.4 Discretised Sections

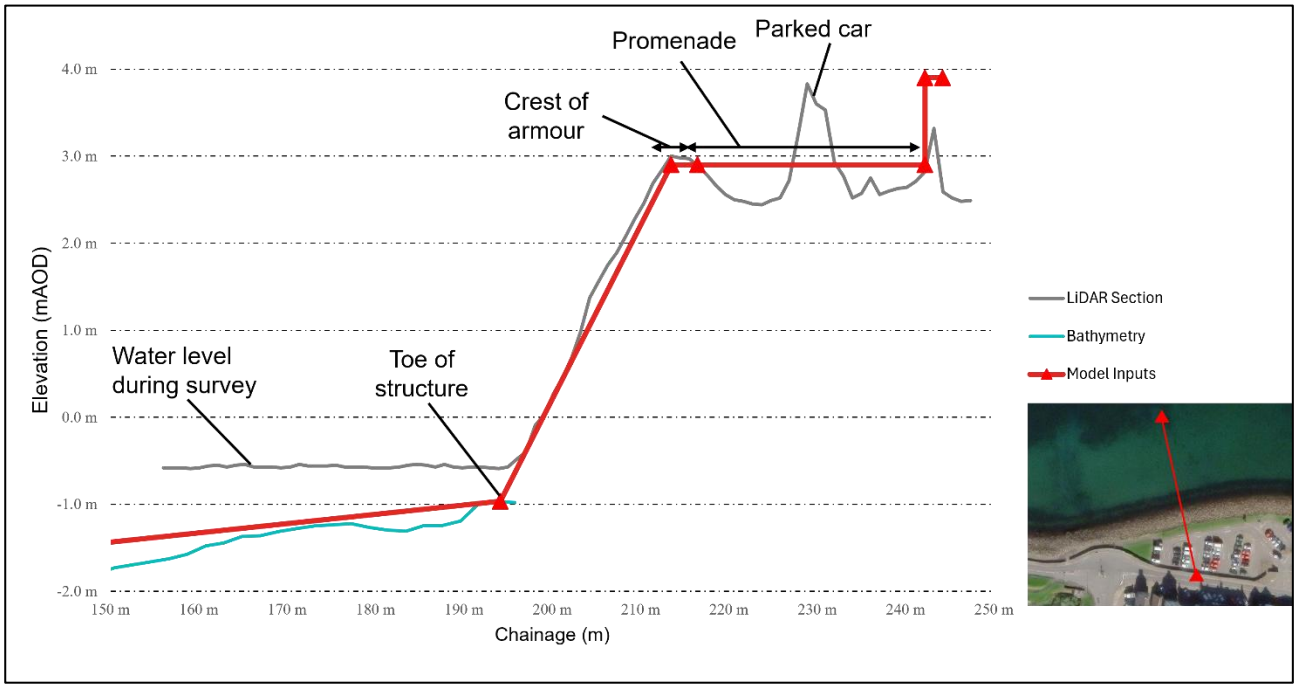
The discretised sections, overlaid with the topography from the DSM and available bathymetry data for profiles A, B, and C, can be seen in the below figures.

Figure 9.11: Profile A discretised geometry



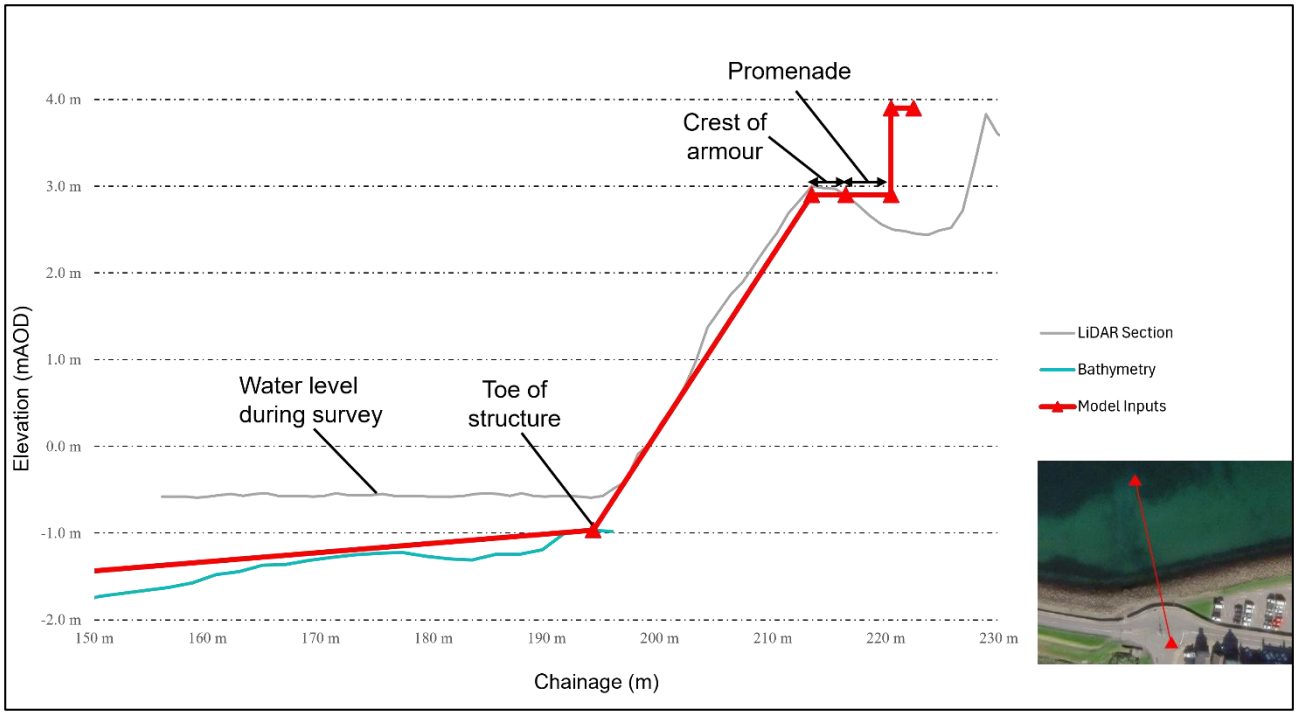
Source: Mott MacDonald, 2026

Figure 9.12: Profile B discretised geometry



Source: Mott MacDonald, 2026

Figure 9.13: Profile C discretised geometry



Source: Mot Macdonald, 2026

9.3.4 Hydraulic inputs

9.3.4.1 Hydraulic Inputs

Each hydraulic input into the tool represents either a single instantaneous set of water/wave conditions, or a step in a time series. This event is defined by the parameters detailed in Table 9.2. These wave conditions were extracted from the regional model.

Table 9.2: Overtopping tool wave and water level inputs

Parameter	Description
hm0_toe [m]	The spectral wave height extracted at the toe of the structure.
hm0_deep [m]	The spectral wave height determined offshore (in deep water). No refraction should be applied.
wl [m]	The (extreme) still water level.
Tm_1_0_deep [s]	Spectral wave period in deep water
wave_dir (deg)	Wave direction at the toe of the defence. This is in degrees from north (clockwise)
storm_dur [hrs]	Storm duration. Used to estimate the number of waves This parameter is redundant in this context.

9.3.4.2 1D wave transformation inputs

1D wave transformation outputs at the toe of the structures, extracted from the Joint-probability Analysis (discussed in 8.6) are used to determine the worst-case overtopping for each design case (the 50-, 200- and 1000-year return periods, for both the present and future epoch) which is used as the basis for developing the discharge time-series.

Each input into the overtopping tool represents an individual event with an offshore wave height and water level with the combined probability of the return event in each design case. For each

design case, there will be multiple pairs of water level and wave height, which all have the same return period.

For each pair, at each profile, hydraulic inputs for the tool were extracted both at an offshore point to get offshore wave height and wave period, and at the toe of the defence to get wave height at this point, as well as the associated wave direction, and water level. These inputs were dependent on the foreshore slope and toe elevation, so were produced for profiles A and B, with the inputs for B also used at profile C due to their bathymetric similarity.

Based on the flow inputs required for the OFM, shown in Figure 9.6, overtopping at A_q1, A_q2, B_q1 and C_q2 was calculated. From the output, the 'worst' pair of water level and wave height that resulted in the highest discharge could be identified for each design case, for these four points.

9.3.4.3 Time-series Inputs

The time-series input into the OFM represents a storm producing the worst overtopping. It would be unrealistic to have different pairs for each profile, as this would assume differing water levels across the model for each of the profiles at the same time. Therefore, for each design case, the pair from one of the four points (A_q1, A_q2, B_q1 and C_q2) with the greatest contribution to overtopping would be chosen as the 'driving' hydraulic conditions. These equivalent conditions were used for the time series inputs to all profiles.

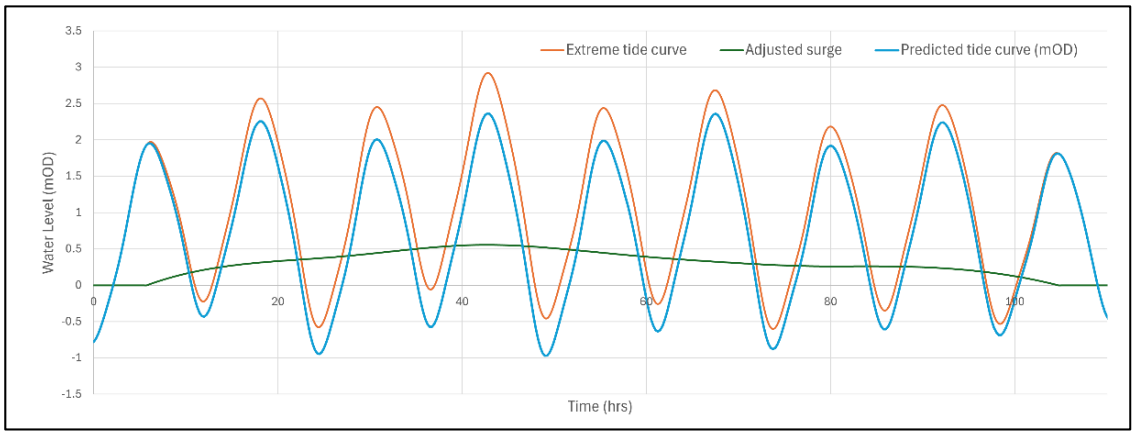
The method to generate a tidal time-series curve from this driving pair is detailed in the Coastal Flood Boundary Conditions guidance[25], produced by the Environment Agency.

The time series is a combination of:

- A tidal curve, generated from an astronomical tide prediction for Kirkwall. The selection corresponds to a high spring tide from 2023,
- A surge shape from Lerwick (considered comparable to tidal conditions in Kirkwall), aligned so that the peak of the surge and the tidal level occur at the same point. The magnitude of the surge is scaled so that the combination of the surge and the tidal profile result in a maximum water level equivalent to the extreme water level for the worst pair for that design case.

The time series is in 15-minute time steps. A plot of the components, plus the combined extreme tide curve is shown in Figure 9.14.

Figure 9.14: Tidal time series for a 1-in-50 return period event in the present epoch (2025)



[25] Coastal flood boundary conditions for the UK: update 2018. User guide © Environment Agency May 2019

Source: Mott MacDonald. 2026

Once the tidal time series was produced, the regional model was run to produce the wave conditions at the toe corresponding to the identified storm event (JPA pair). Hence, the inputs into the overtopping tool were the water level of the extreme tide curve over the time series, the same offshore wave height over the entire time series (and therefore same offshore mean period), with varying wave height at the toe. Wave direction remained constant.

For the purposes of the flood modelling, the tool was run over three consecutive tidal cycles. The central cycle captured the peak storm conditions, with an additional full tidal period on either side, from low tide to low tide, to represent a realistic duration of overtopping during a storm event. In total, this produced 37 hours of continuous data for use in the model.

9.4 Inputs for the OFM

9.4.1 Worst-case discharges

Table 9.3 summarises the outputs from the overtopping tool for the extracted 1D transformation pairs for each design case in the present and future epochs. Refer to Figure 9.5 for the structural designations (points of investigation in the tool).

Values in bold represent where overflow of the defence (the armour crest for A_q1 and B_q1, and wall crest for A_q2 and C_q2) was occurring, hence discharge would be significantly higher.

In the present epoch, it was identified that the driving case was decided as the pair that resulted in the worst overtopping at C_q2. Overtopping of the crest wall was much more significant at profile C compared to profile A and would have greater impact on the OFM than either of the inputs at the crest of the armour.

In the future epoch, in all design cases, there was overflow of the crest wall. Hence, the driving case was to be taken as the pair with both the highest water level and highest wave height.

Table 9.3: Discharge (in l/s/m) of the worst pair for each design case and point of investigation

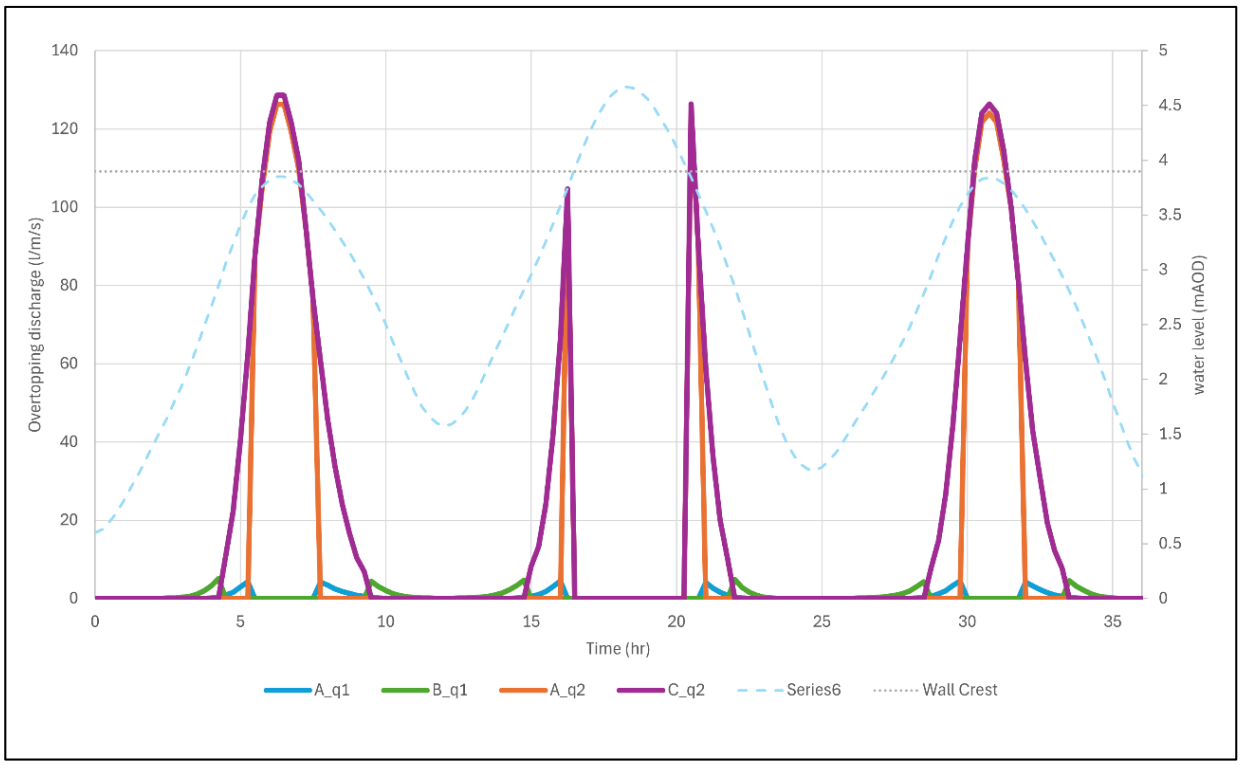
Epoch	Return	A_q1	A_q2	B_q1	C_q2
2025	50	3.0	0.01	***1278.9***	5.7
	200	7.1	0.03	***1541.4***	11.6
	1000	10.4	0.06	***1872.0***	18.3
2125	50	**1278.9**	**652.6**	**3200.9**	**652.6**
	200	**1541.4**	**866.0**	**3552.2**	**866.0**
	1000	**1872.0**	**1142.8**	**3982.8**	**1142.8**

Source: Mott MacDonald, 2026

9.4.2 Time series outputs

Figure 9.15 shows the time-series results for the future epoch 1-in-1000 event. This plot represents the typical profile of overtopping discharge over the times-series for all design cases that is fed into the OFM. As water level rises and falls, discharge also varies in a similar manner. As the water level approaches the crest of the defence, discharge increases exponentially. In cases where the water level overflows the crest of the wall (+3.9 mOD), the discharge at that time step was set to zero to avoid double counting, as overflow would be calculated separately within the OFM.

Figure 9.15: Time-series of water level and overtopping discharges for points of investigation, for the future epoch (2125) 1-in-1000 event



Source: Mott MacDonald, 2026

9.5 Mid-epoch analysis

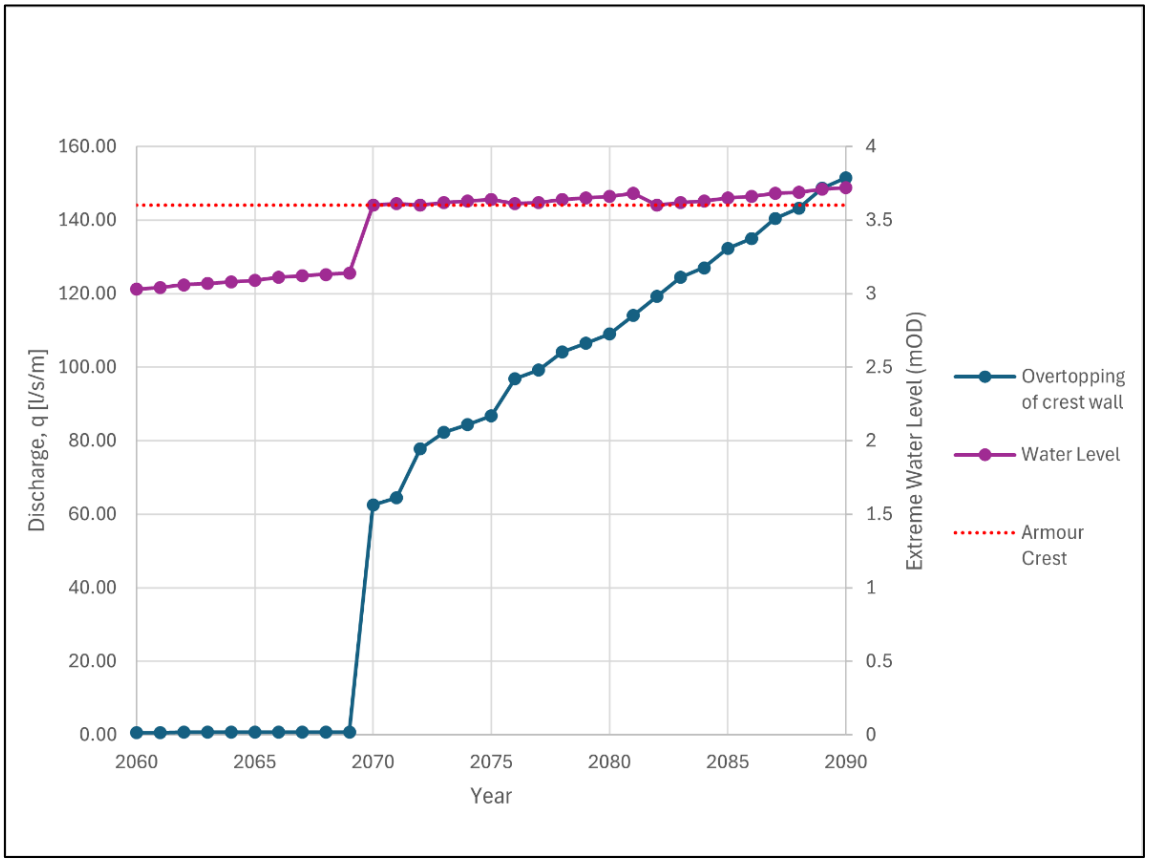
9.5.1 Identifying mid-epoch scenario

The present and future epochs (2025 and 2125 respectively) were defined in the scope; however, the mid-epoch was to be determined separately. This was to be set to a year when the standard of protection of the defence was compromised.

The approach used was to identify the year in which overtopping of the crest wall of profile A (the longest reach of the scheme) becomes significant.

To estimate overtopping on a year-by-year basis in the future, the extracted 1D transformation pairs produced for the present epoch (2025) were uplifted. Wave heights were uplifted by 20% (as per 4.11.3) for all years, and the extreme water levels were increased year on year based on SLR from 2025. The worst pair for each year was then found. Figure 9.14 shows the discharge over the crest wall (A_q2) of the worst pair from 2060-2090, as well as the extreme water level (q2_wl) for that pair. This is for a 1-in-200 return event (the SOP of the defence).

Figure 9.16: Discharge and water level of the worst pair in each future year for a 1-in-200 return event at profile A



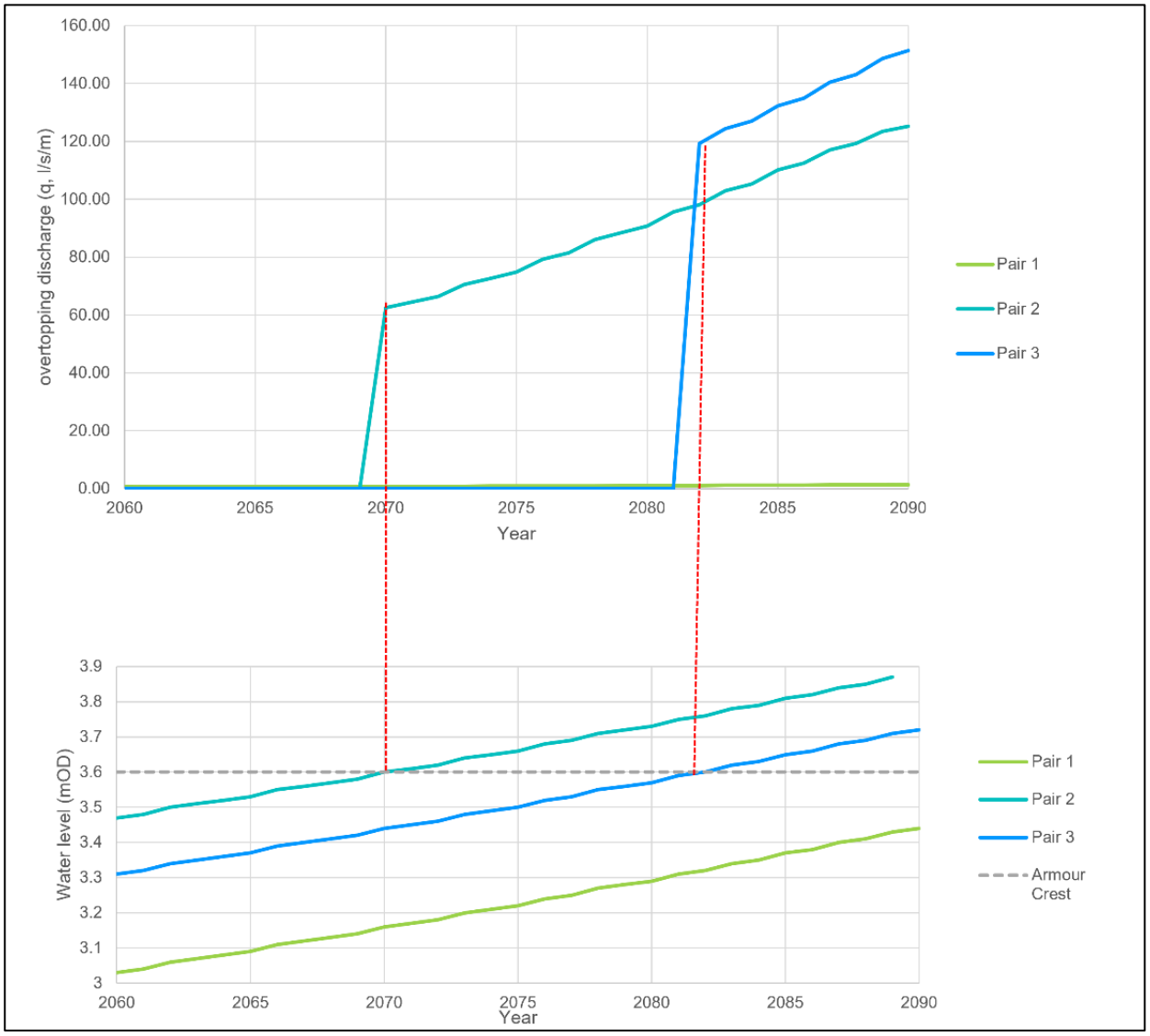
Source: Mott MacDonald, 2026.

The discharge over the crest wall (A_q2) begins to increase significantly from 2070 onwards. This increase is a product of the water level overflowing the crest level of the armour in section A (3.6mOD) - i.e. 2070 is the first year for the 1-in-200 event where there is a pair with an extreme water level above the armour crest, so the armour is no longer as effective, and waves will propagate to the crest wall. Within the tool this represents a changing calculation pathway - from a rubble mound approach (chapter 6 of the EurOtop manual) to the vertical wall methodology (chapter 7), resulting in a significant spike in discharge compared to the year prior.

The plot shows a non-linear increase in water level and overtopping beyond 2070 – this is the result of the worst pair of wind and wave conditions changing. For each pair year-on-year, the wave height remains constant, but the water level is uplifted by SLR to that year. As the extreme water level increases, overtopping discharge also increases linearly, with a significant jump in the year that the water level is first uplifted above 3.9mOD. Considering the different water levels of each pair in the present epoch, the number of years to uplift to above 3.9mOD differs. The first pair to reach this level occurs in 2070. In later years, pairs with lower initial water levels but higher waves (there is an inverse relationship between water level and wave height) are uplifted to a water level above the armour crest. At this point, the larger waves result in greater overtopping than pairs with a lower wave but higher initial water level, causing a non-linear increase in the discharge of the worst pair. This behaviour is displayed in Figure 9.17. Pairs 1, 2, and 3 have wave heights of 1.4m, 0.8m and 1.0m respectively, that remain constant across the appraisal period, with a water level that is uplifted yearly by SLR. In the first portion of the appraisal period, Pair 1 has the largest wave height which results in the most overtopping. Pair 2 has the highest 2025 water level so reaches +3.9mOD first, resulting in a significant jump in

overtopping. Pair 3 has a lower starting water level but higher wave – once the water level reaches the armour crest level it jumps to having the greatest overtopping of the three pairs. The change between Pair 1 and Pair 2 as the pair causing the greatest overtopping results in the greatest change in the SOP provided by the defence. Hence, 2070 was taken as the mid-epoch year – representing the point at which there is a significant change in the way the defence functions, and consequently a change in the standard of protection it offers.

Figure 9.17: Change in overtopping and water level of 3 JPA pairs over the time period



Source: Mott Macdonald, 2026.

There are other potential years in which the behaviour of the defence changes (e.g. 2098, in which the crest wall first overflows) that could be taken as the mid epoch year, but it was noted that 2070 provides useful insight (overtopping with the crest wall as the only defence) in comparison to the behaviour of the defence in the present epoch (overtopping over both armour and crest wall) and the future epoch (overflow of crest wall in all cases).

Additionally, it was identified that a failure to close the flood gates would be most likely in locations away from the centre of Kirkwall; notably the access routes through the crest wall

along Ayre Road. In this scenario, as 2070 is the first year in which the armour of section A overflows, this would also be the first year that flood water could enter Kirkwall unimpeded.

9.5.2 Mid-epoch outputs

Table 9.4 shows the overtopping discharge for the worst 1D wave transformation pair at each point of investigation in the mid epoch year.

As this epoch is not to be included in the OFM, a time-series will not be developed from these results. The overtopping discharge for A_q1 and B_q1 should be taken tentatively in these results as these values are calculated as overflow of the defence which is subject to significant uncertainty in the tool.

The significant increase in overtopping of A_q2 in 2070, compared to the present epoch, indicates a significant reduction in the standard of protection of the defence along this reach. At this point the function of the defence may need to be reassessed. The actual impact of significant overtopping along section A is uncertain; it will affect traffic along Ayre Road, but it is unclear how much of this additional overtopped water would enter Kirkwall – much of this discharge may flow into the Peedie Sea.

Additionally, considering the difference in discharge between A_q1 and A_q2 in the present epoch, and the sharp increase in A_q2 once the crest of the armour overflows in the mid-epoch, the benefit provided by the combined defence of both rock revetment and crest wall is apparent. In cases where the rock armour is overflowing but the crest wall is not, the defence is significantly more vulnerable to overtopping.

Table 9.4: Discharge (in l/s/m) of the worst pair for the mid-epoch

Epoch	Return	A_q1	A_q2	B_q1	C_q2
2070	50	**12.3**	55.82	**1033.3**	55.5
	200	**90.5**	95.11	**1302.1**	90.9
	1000	**93.2**	110.63	**1302.1**	101.7

Source: Mott MacDonald, 2026

9.6 2100 1-in-200-year return period event derivation

To enable direct comparison with SEPA's 2100 flood mapping for Kirkwall and to support future planning decisions, an overland flood model representing the 2100 1-in-200-year return period conditions (including climate change) has been prepared.

Given that significant modelling had been undertaken for the 2025, 2125 and 2075 assessments, it was important to maintain a proportionate level of assessment. On this basis, new overtopping calculations were undertaken for the 1-in-200-year return period event only. The inputs to this new overtopping calculation were derived using the existing 2025, 2070 and 2125 joint probability datasets. The existing data were compared to identify the most representative basis for deriving the 2100 conditions. The approach is described in more detail below.

An overtopping time series was developed for the rear of the seawall as input for the OFM. As Section B is considered to have no overtopping of the seawall (as per 9.3.1), time series were prepared for sections A_q2 and C_q2 only.

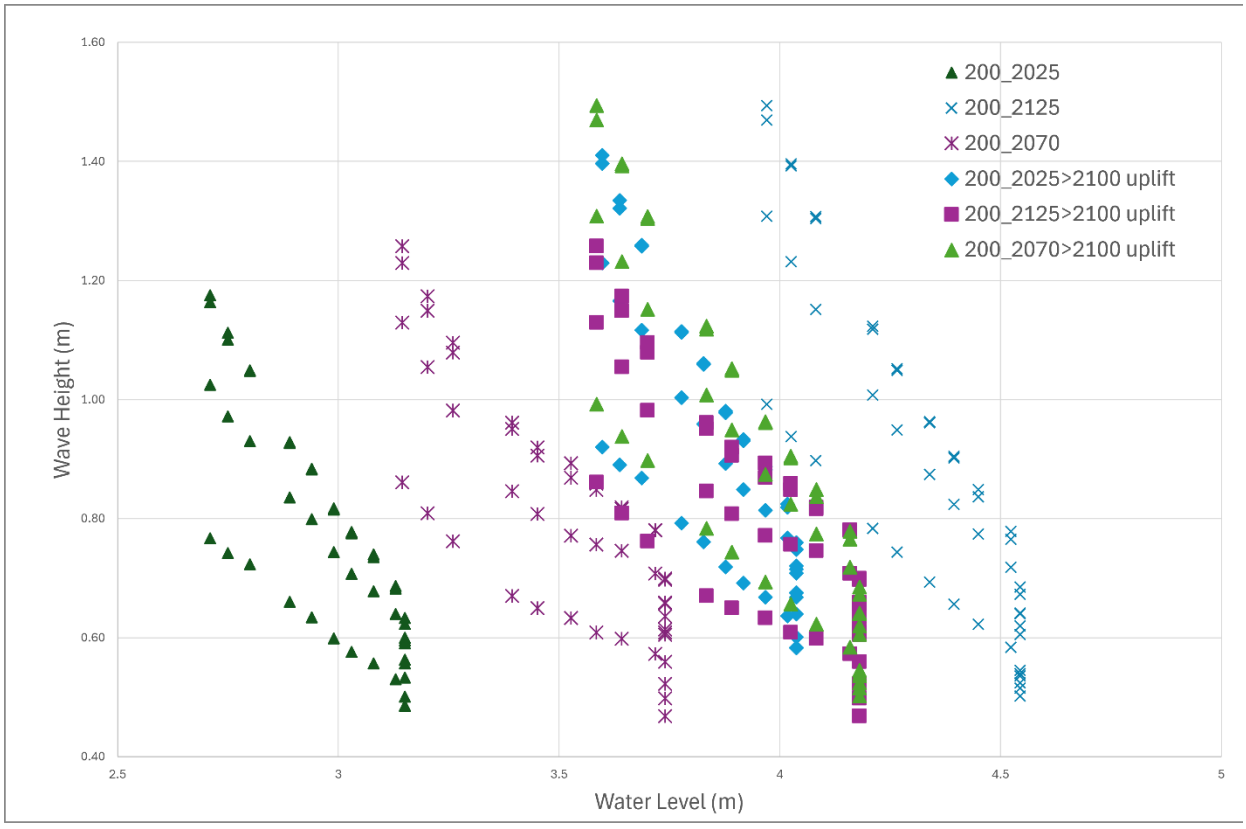
9.6.1 Selection of donor dataset

To assess the best fit for translating wave parameters, the existing 2025, 2070 and 2125 joint-probability datasets were compared to identify the most representative basis for deriving the 2100 conditions. For this comparison, the water levels associated with each dataset were adjusted to the 2100 epoch using the applicable sea-level-rise differences. The original and adjusted wave and water-level pairs are shown in Figure 9.18. The comparison showed that:

1. The 2025 dataset uplifted to 2100 predicted lower wave heights and water levels than the adjusted 2070 or 2125 data, because the 2025 input conditions (notably the waves) used to generate the joint probability pairs had not been uplifted by the climate change allowances. As a consequence, results from this case would drive lower overtopping and significantly less overflow. As a more conservative assumption, noting that accurate representation of climate change effects is essential, the uplifted 2025 pairs were excluded from further assessment.
2. Applying sea-level-rise projections from 2070 to 2100 to the 2070 dataset allowed the 2100 water levels to be derived – within the UKCP18 time mean sea level anomaly projections dataset (rather than extrapolating beyond the dataset between 2100 and 2125).
3. The 2125 dataset, when adjusted back to 2100, produced extreme water level-wave combinations broadly comparable to those derived from 2070, whilst being marginally less conservative. The difference is due to the different approaches to sea level rise beyond 2100 in UKCP18.

On this basis, the 2070 joint-probability dataset was selected as the donor dataset for constructing the 2100 offshore wave and water-level pairs.

Figure 9.18: Original wind/wave pairs for 2025, 2070 and 2125 epochs, as well as uplifted/downlifted pairs to 2100



Source: Mott Macdonald, 2026.

9.6.2 Development of the 2100 time series

The time series was developed as follows:

1. The offshore wave height and water level pairs produced from the joint probability model for the 2070 epoch were uplifted by SLR from 2070 to 2100. Wave height was not uplifted, as the 2070 dataset already includes a 20% wave height uplift, as discussed in Section 4.11.3.
2. The time series inputs to the inundation model are required for the peak water level/offshore wave height combination generating the greatest combined overflow and wave-driven overtopping discharge (the controlling condition).
3. In several joint probability pairs in 2100, water levels exceeded the +3.9 m ODN seawall crest, so the controlling condition was governed principally by the greatest direct overflow. The most severe water level will generate the most overflow. Away from the peak of the tide, the most severe overtopping would occur based on the highest wave height in a joint probability pair with the most severe water level. For both Sections A and C, the selected pair comprised a water level of 4.18 m ODN and an offshore significant wave height of 0.70 m.
4. Using the same methodology as detailed in Section 9.4.2, a 37-hour time series of water levels was developed, taking the worst pair as the peak water level.
5. To derive variation in wave height at the toe of the structure through the tidal cycle in the controlling condition, joint probability pairs with similar wave heights in other epochs were compared. Variation across the tidal cycle could then be determined by transforming (stretching) the time series from the donor event to the new event.

   The previously modelled 2125 1-in-200-year return period event was selected to provide the corresponding variation in transformed wave height at the structure toe. Its offshore significant wave height of 0.69 m closely matches the 0.70 m offshore wave height of the selected 2100 pair. For each 2100 time step:.

   a. The water level was matched to the nearest water level in the 2125 1-in-200 year return period time series.
   b. The corresponding wave height at the toe of the structure from the 2125 event was then assigned to the 2100 timestep.
   c. The difference between matched water levels was small, with an average deviation of ±0.01 m and a range of approximately 0.00 to ±0.05 m. Given these small differences, the approach was considered sufficiently accurate for development of the overtopping time series.

   This approach is considered reasonable as the 1D wave transformation process is empirical. For a given offshore wave height and water level, the transformed wave height at the toe should be the same regardless of the wider storm event context.

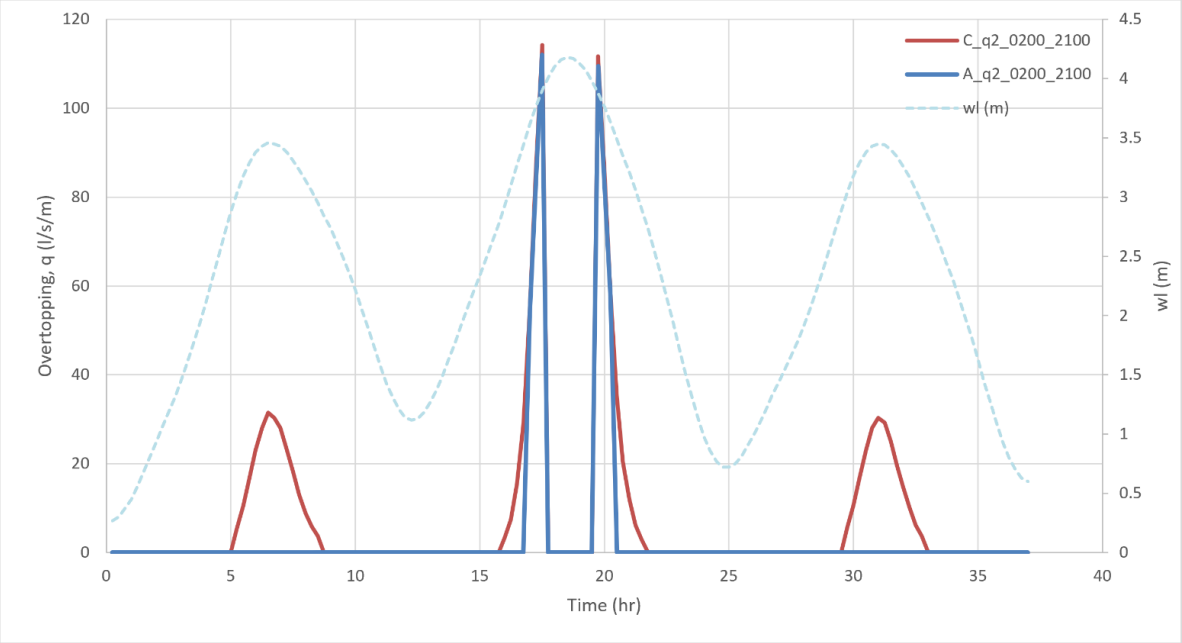
The completed time series was run using the in-house EurOtop tool to determine overtopping discharge at each timestep. As in Section 9.4.2, when the wall is overflowing, overtopping discharge is set to 0 to avoid double counting with overflow predicted in the OFM.

9.6.3 Overtopping behaviour

Figure 9.19 shows overtopping discharge output against time from the EurOtop tool for the 2100 time series. This time series covers three tidal cycles, with a maximum water level at 18.5 hours, and local maxima at approximately hours 7 and 31. Between hours 17.5 and 19.5, the water level exceeds the crest elevation of the wall. As the resulting overflow is captured within the OFM, overtopping is set to zero during this period. The local maxima at hours 7 and 31,

reach water levels of approximately +3.5 mOD. At this water level, the water surface is above the crest of the rock armour at Section C but remains below the crest elevation at Section A. Consequently, a local maxima in overtopping discharge is observed for C_q2, while A_q2 shows only a minimal increase at this point.

Figure 9.19: Overtopping time series for the 2100 epoch



Source: Mott Macdonald, 2026.

10 Local hydrodynamic model

10.1 Introduction

The primary objective of the local MIKE21 FM HD model is to simulate detailed hydrodynamic conditions in the study area required for the local overland hydrodynamic model (Section 11). The local model setup followed a process similar to that described for the regional model in Section 5. However, several changes described below were deemed necessary to achieve the required model performance.

10.2 Model setup

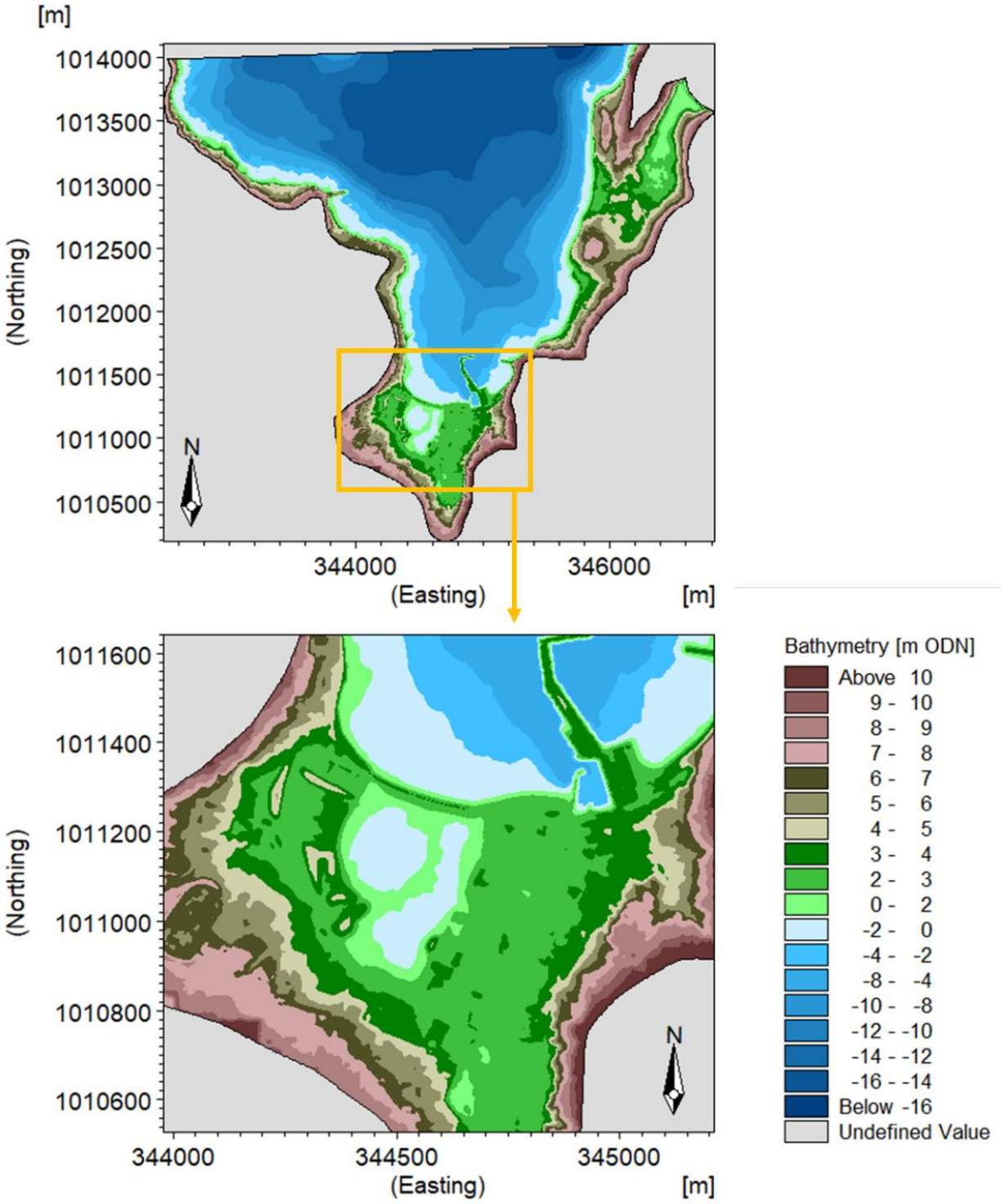
10.2.1 Horizontal and vertical datums

The model was setup using the horizontal and vertical datums as detailed in Section 4.1.

10.2.2 Local model bathymetry and mesh

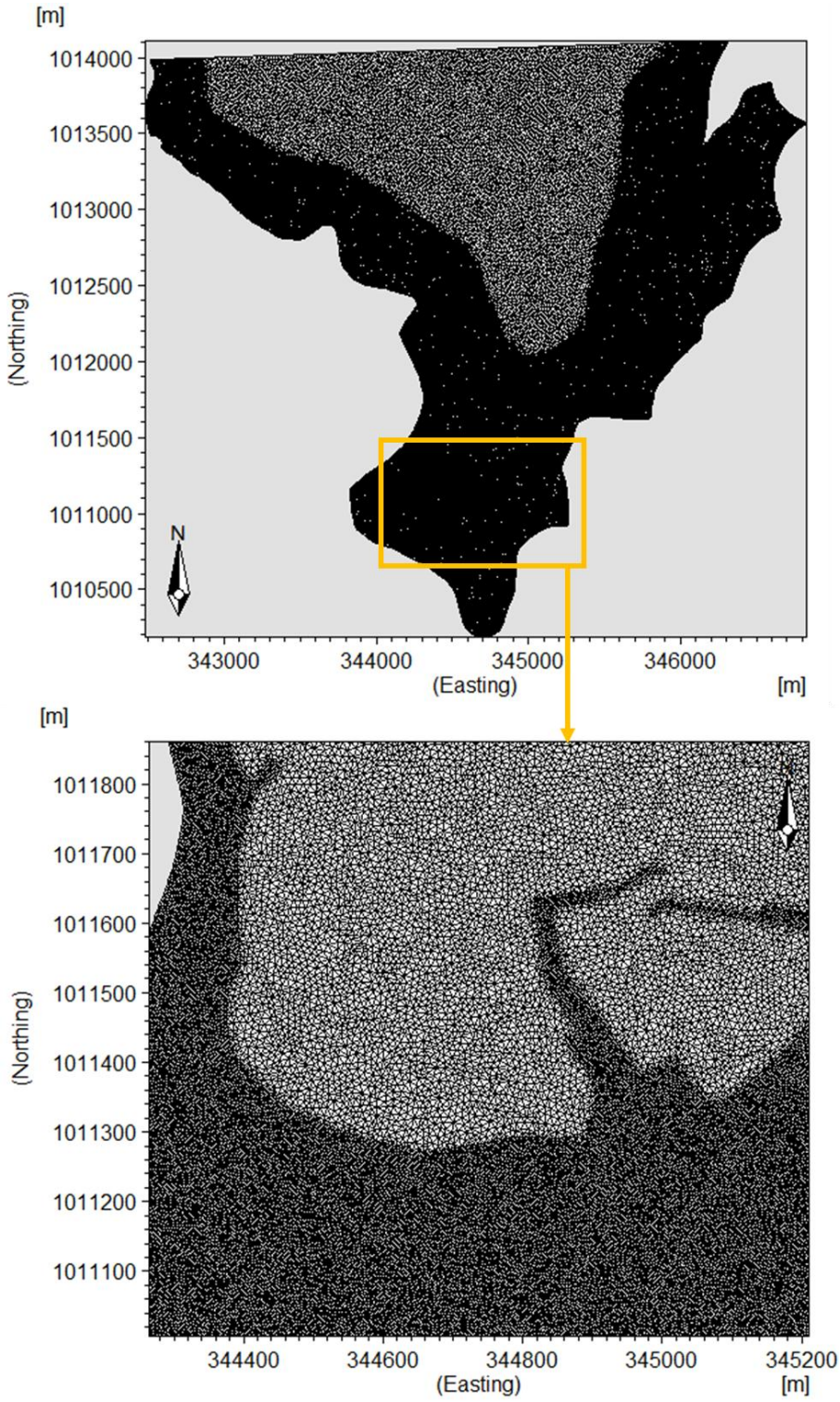
Bathymetric data as described in Section 4.2 were used in the local model. Figure 10.1 shows the wide-area model domain, bathymetry, and an enlarged view around the study area. Figure 10.2 shows the model mesh for the whole domain with an enlarged view around the study area. A range of model sensitivity tests using different mesh resolutions were undertaken to establish the appropriate balance between model run times (mesh resolution) and resulting hydrodynamic conditions. In the final version of the model, the mesh is coarser in the offshore region (10 to 20m approximately), while in the nearshore areas within the study area the element sides of the mesh are approximately 5 m.

Figure 10.1: Local MIKE21 FM HD model domain and bathymetry.



Source: Mott MacDonald 2026.

Figure 10.2: Local MIKE3 FMHD model mesh.



Source: Mott MacDonald 2026.

10.2.3 Local model density

A barotropic density function was applied in the model as the stratification at the study area is not significant based on Scottish Marine Assessment (2020)[26] on salinity.

10.2.4 Local model boundary conditions

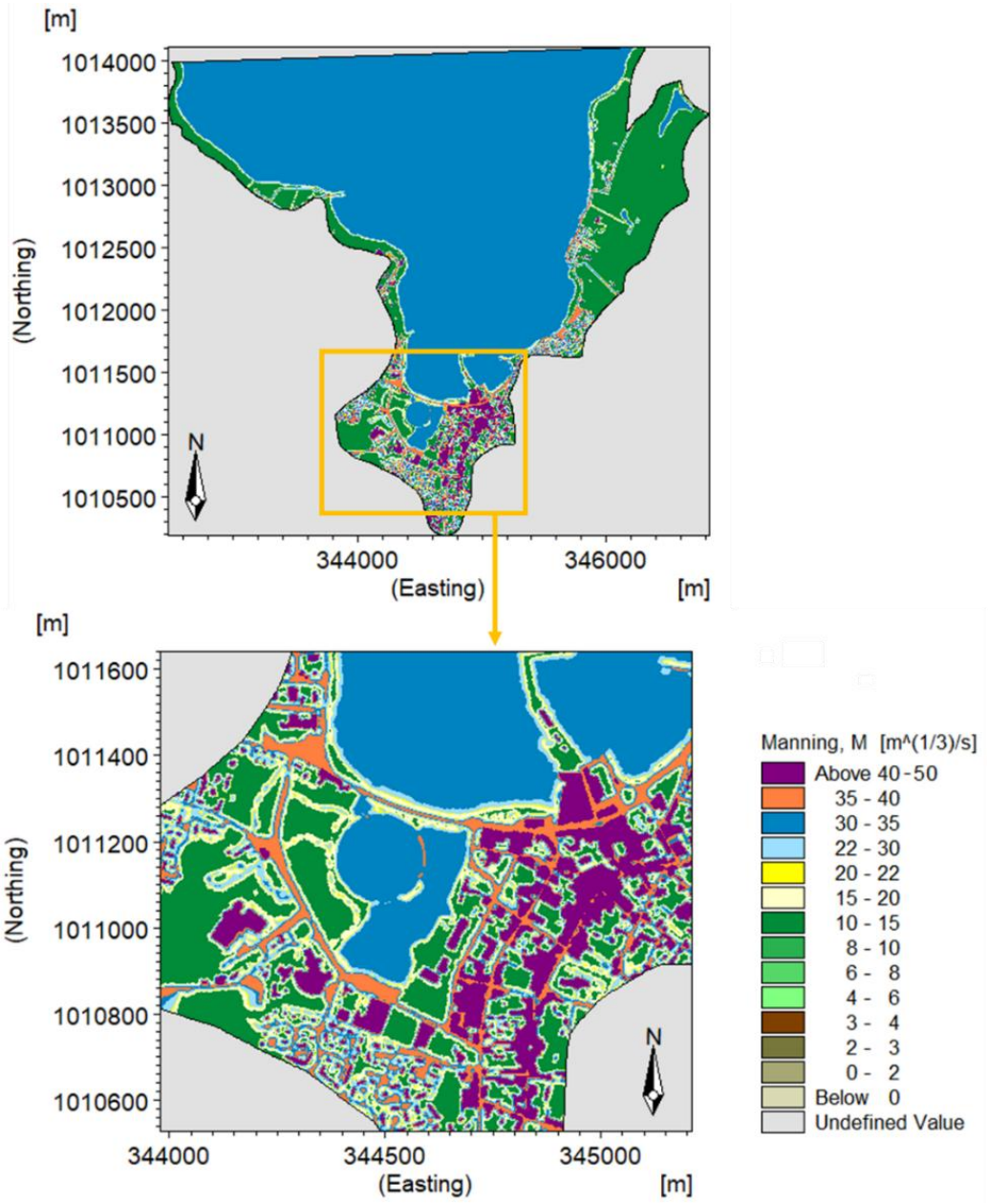
The water level and tidal flows (u-velocity and v-velocity currents) from the calibrated regional model results were applied to ensure that the local model's tidal flows and water levels were correctly represented.

10.2.5 Local bed roughness

Similar to the calibrated regional model, a series of iterative sensitivity tests were conducted for Manning's number (M). In the final simulation, a Manning roughness map was implemented in the model (Figure 10.3). A uniform Manning, M of 32 $m^{1/3}/s$ was assigned to all wet cells within the bay, while spatially variable Manning values were applied across the overland area as described in Section 4.10. The overland manning values are consistent with those recommended in the SEPA (2015) flood model guidance[12].

[26] Marine Scotland Assessment (2020), Scottish Government. Salinity | Scotland's Marine Assessment 2020 accessed on 12 January 2026.

Figure 10.3: Overview of bed roughness map (i.e Manning, M).



Source: Mott MacDonald 2026.

10.2.6 Atmospheric forcings

The same setup as applied in the calibrated and validated regional model (Section 5.2.6).

10.2.7 Local eddy viscosity

The same setup as applied in the calibrated regional model (Section 5.2.7).

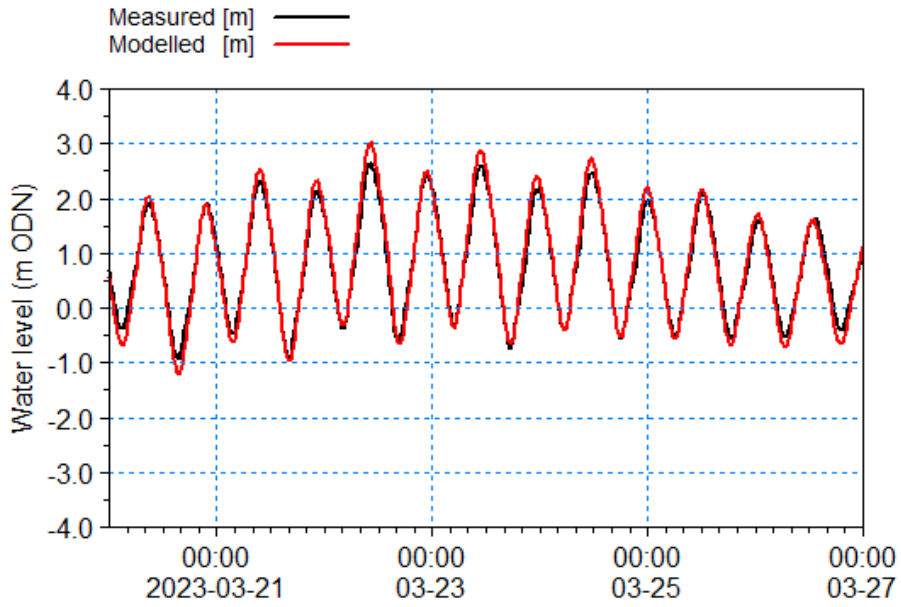
10.3 Local HD model calibration

Local model calibration was undertaken using measured water level data at Kirkwall as described in Section 4.3 for spring tides between 20th to 27th March 2023. It is noted that no

current data are available for local model calibration. During the calibration process, bed roughness, eddy viscosity, bathymetry, mesh resolution, number of layers and boundary conditions were adjusted to obtain the best agreement between predicted water levels and model results. Figure 10.4 compares measured water levels against model results at Kirkwall while Table 10.1 shows the error statistics of root mean square error (RMSE), bias, coefficient of determination (R^2), standard deviation (SD) and index of agreement (IA) between the predicted and modelled water levels.

Based on Figure 10.4 and Table 10.1, the modelled and predicted water levels show very good visual agreement, with only a slight overestimation that provides a conservative outcome. The statistical performance is also strong, with a bias of less than 0.13 m and an $R^2 > 0.9$, indicating excellent correspondence with the measured water-level data. Overall, the model skill metrics meet the performance criteria published by Williams & Esteves (2017).

Figure 10.4: Local model calibration comparison between measured and modelled water levels at Kirkwall.



Source: Mott MacDonald 2025.

Table 10.1: Summary of local MIKE21 FMHD model calibration statistics for water levels.

Station	Bias (m)	RMSE (m)	SD (m)	R^2	IA
Kirkwall	0.13	0.16	0.16	0.99	0.99

Source: Mott MacDonald 2026

10.4 Local HD model validation

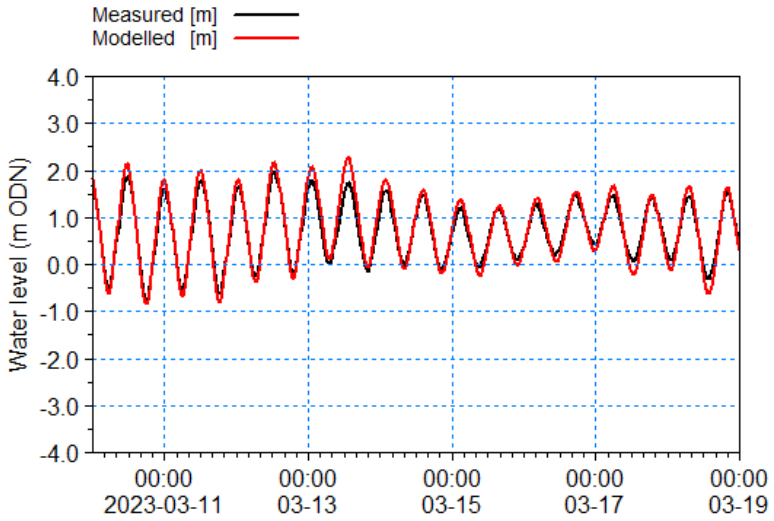
Model validation was undertaken during neap tides (3rd to 7th March 2024). Figure 10.5 shows measured water levels against model results at Kirkwall while Table 10.2 shows the error statistics of RMSE, bias, R^2, SD and IA between the predicted and modelled water levels.

The model was also simulated for January 2005 as part of the validation process, allowing comparison with the historical recorded extreme water level at Kirkwall provided by the OIC

(Table 10.3). The results show that the model successfully reproduced the extreme water level observed at Kirkwall.

Based on Figure 10.5, Table 10.2 and Table 10.3, the measured and modelled water levels show a very good visual and statistical agreement with the bias less than 0.13m with R^2 >0.9 at Kirkwall, indicating a very good model agreement with the predicted water level data. Overall, the model meets the performance criteria published by Williams & Esteves (2017), confirming its suitability for the overland flow modelling (Section 11).

Figure 10.5: Local model validation comparison between measured and modelled water levels at Kirkwall.



Source: Mott MacDonald 2026.

Table 10.2: Summary of local MIKE21 FMHD model validation statistics for water levels.

Station	Bias (m)	RMSE (m)	SD (m)	R^2	IA
Kirkwall	0.13	0.16	0.16	0.97	0.99

Source: Mott MacDonald 2026

Table 10.3: Maximum water level comparison at Kirkwall (January 2005 flooding event).

Station	OIC	Modelled
Kirkwall	2.83m ODN	2.85m ODN

Source: Mott MacDonald 2026

11 Local overland hydrodynamic model

11.1 Introduction

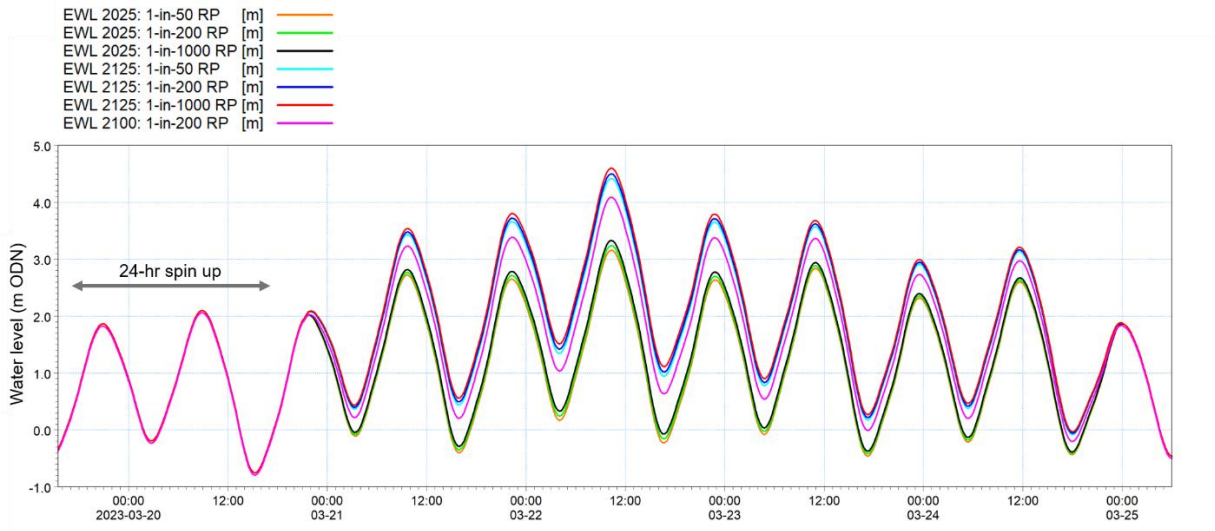
The main objective of the local overland MIKE21 FM HD model is to simulate detailed hydrodynamic conditions within the study area for flooding assessment, including water levels, flood extents, and current speeds. It should be noted that a comprehensive flood risk assessment was not discussed, as it was not part of this study.

11.2 Model setup

The local overland MIKE21 FM HD model used the same computational domain as the calibrated local hydrodynamic model, as illustrated in Figure 10.1. Building representation followed the SEPA (2015) flood model guidance[12], in which buildings were grouped and simplified as stubby building blocks at 5m resolution. The building elevations were taken directly from the DTM LiDAR dataset (Section 4.2), and these elevations were increased by 0.3 m within the model domain as recommended in the SEPA flood model guidance. Simplification of building geometry was necessary as many building footprints within the study area are smaller than the model resolution (5 m, Figure 10.2), which prevents the model from resolving individual structures accurately. Furthermore, full retention of detailed building geometry leads to a substantial increase in computational time without delivering a proportional improvement in predicted flood extent. Consequently, the simplified building representation offers a practical and robust solution for the overland flow modelling undertaken in this study. Spatially varying bed roughness values were applied using the roughness map shown in Figure 9.3 and detailed in Section 10.2.5. A sensitivity test was also undertaken for the building-representation bed roughness using lower Manning values and the results show no significant change in flood extent.

The extreme water levels (EWLs) at Kirkwall (Section 4.11.2) were applied to the Lerwick surge profile from the CFB[15] dataset as shown in Figure 11.1 and used as boundary conditions for the model runs during high Spring tides in March 2023. This period was selected to maintain consistency with the local MIKE21 FM HD model calibration and to provide a conservative assessment. The boundary conditions were further refined through sensitivity tests to certify alignment between the model outputs and the EWL values observed at Kirkwall (Table 4.5). In addition, all simulations were simulated for a duration exceeding 110.5 hours which included an initial 24-hour spin-up phase to allow the model to achieve stable hydrodynamic conditions and maintain overall model stability.

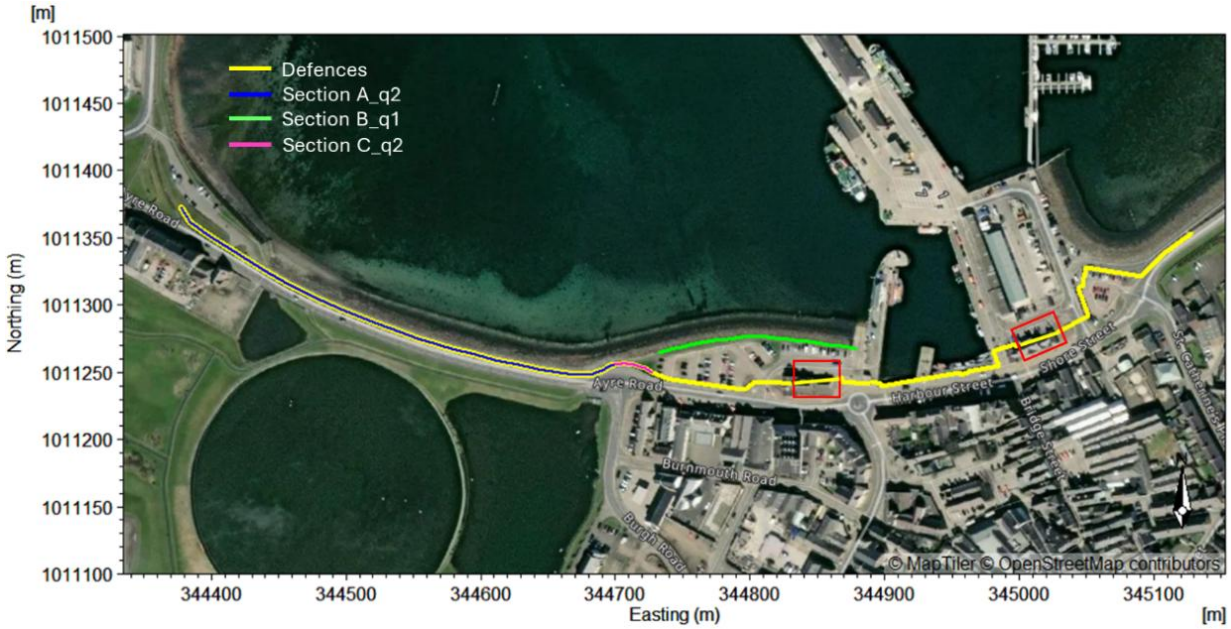
Figure 11.1: Time series of extreme water levels with surge for the model runs and predicted water levels at Kirkwall.



Source: Mott MacDonald 2025

The flood defence structures (Section 4.8) were represented as dikes in the model and the overtopping discharges were applied at three designated sections as shown in Figure 11.2, for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods under both the Baseline 2025, and future 2125 scenarios. For the 2100 scenario, only the 1-in-200-year return period was run. These discharge values were supplied based on the overtopping assessment in Section 9. As outlined in Section 4.8, the two buildings located in front of the Kirkwall frontage, highlighted by the red box in Figure 11.2, represented as defence structures within the model. These structures were assigned a crest level of +3.9 m ODN, consistent with the defence heights on either side of the buildings. In addition, an alternative model scenario was simulated in which building elevations were taken directly from the DTM LiDAR dataset and increased by 0.3 m across the model domain, in accordance with SEPA flood modelling guidance. The results of this additional scenario are presented in Appendix C.

Figure 11.2: The location of the overtopping discharges along the frontage.



Source: Mott MacDonald 2026

Table 11.1 shows a summary of model scenarios that were simulated for the local overland HD model. The model results of the Baseline 2025, Epoch 2100 and 2125scenarios are presented in the next section accordingly.

Table 11.1: Summary of the model scenarios.

Scenarios	Description
Baseline 2025 (1-in-50 RP)	• Baseline scenario for the 1-in-50 year RP (i.e AEP 2%) • Surge extreme water level of 1-in-50 year RP with SLR • Overtopping discharges for 1-in-50 year RP at designated sections
Baseline 2025 (1-in-200 RP)	• Baseline scenario for the 1-in-200 year RP (i.e AEP 0.5%) • Surge extreme water level of 1-in-200 year RP with SLR • Overtopping discharges for 1-in-200 year RP at designated sections
Baseline 2025 (1-in-1,000 RP)	• Baseline scenario for the 1-in-1,000 year RP (i.e AEP 0.1%) • Surge extreme water level of 1-in-1,000 year RP with SLR • Overtopping discharges for 1-in-1,000 year RP at designated sections
Epoch 2100 (1-in-200 RP)[27]	• 75-year epoch scenario for the 1-in-200 year RP (i.e AEP 0.5%) • Surge extreme water level of 1-in-200 year RP with SLR • Overtopping discharges for 1-in-200 year RP at designated sections
Epoch 2125 (1-in-50 RP)	• 100-year epoch scenario for the 1-in-50 year RP (i.e AEP 2%) • Surge extreme water level of 1-in-50 year RP with SLR • Overtopping discharges for 1-in-50 year RP at designated sections
Epoch 2125 (1-in-200 RP)	• 100-year epoch scenario for the 1-in-200 year RP (i.e AEP 0.5%) • Surge extreme water level of 1-in-200 year RP with SLR • Overtopping discharges for 1-in-200 year RP at designated sections
Epoch 2125 (1-in-1,000 RP)	• 100-year epoch scenario for the 1-in-1,000 year RP (i.e AEP 0.1%) • Surge extreme water level of 1-in-1,000 year RP with SLR • Overtopping discharges for 1-in-1,000 year RP at designated sections

Source: Mott MacDonald, 2026

[27] A future 75-year epoch (2100) was only assessed for the 1-in-200-year return period event to align with the planning horizon adopted in SEPA flood modelling.

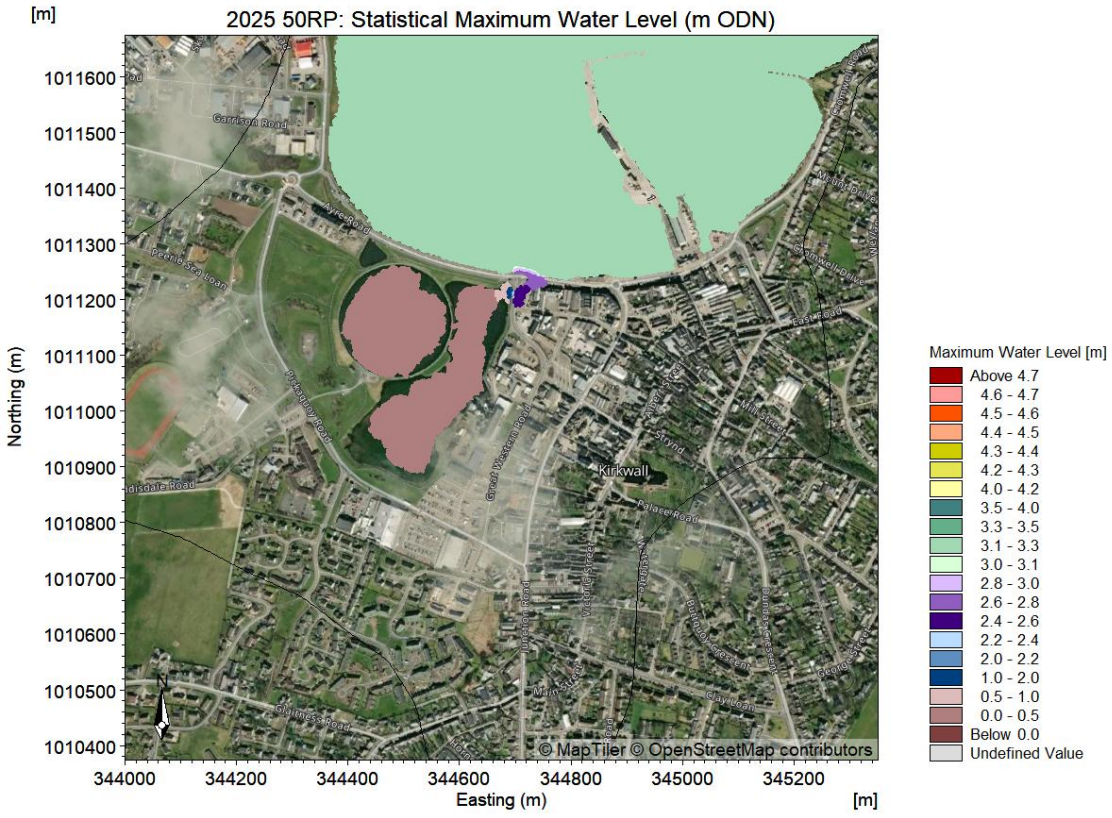
11.3 Model results

This section presents the model results for the Baseline 2025, Epoch 2100 and Epoch 2125 scenarios outlined in Table 11.1

11.3.1 Flood water level

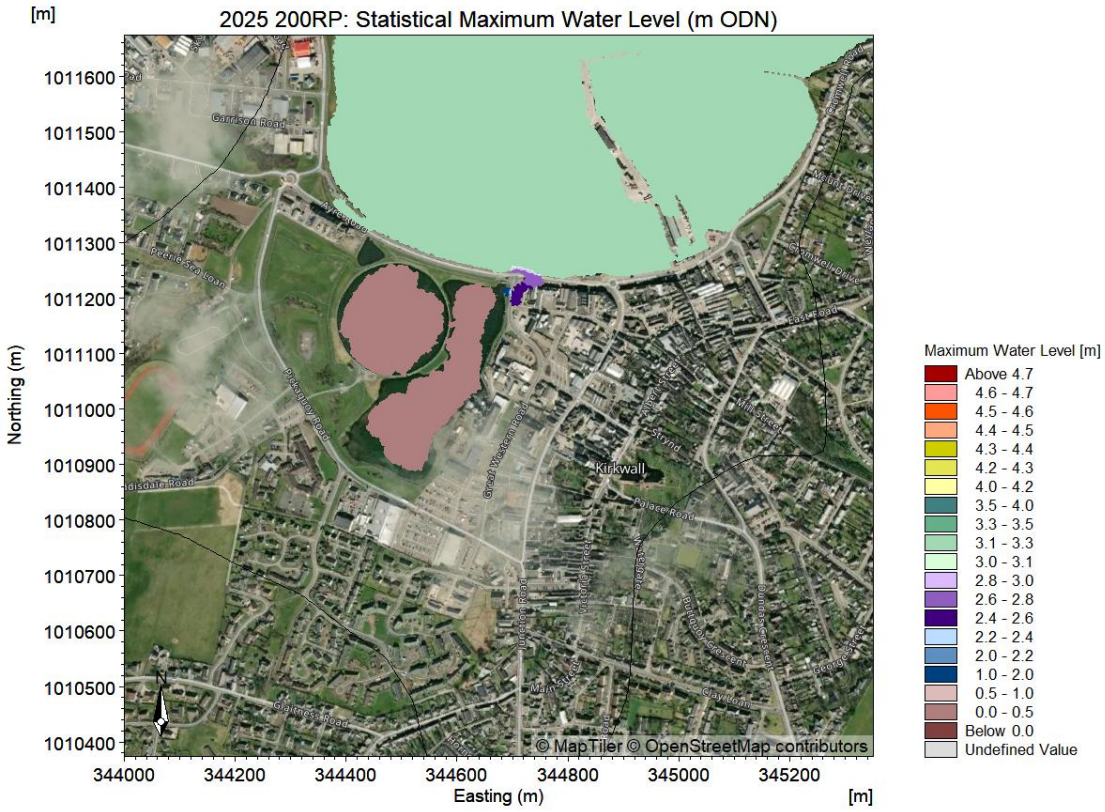
Figure 11.3 to Figure 11.5, and Figure 11.6 to Figure 11.8 show the statistical maximum flood water level results for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods of the Baseline 2025 and Epoch 2125, respectively. and presents the summary of maximum water level values at Kirkwall against the extreme water levels at Kirkwall for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods for the Baseline 2025 and Epoch 2125, respectively. Figure 11.9 and present the statistical maximum flood water level results for the 1-in-200-year return period for the 2100 Epoch. It is observed that the maximum water levels at Kirkwall are matched with the EWL at Kirkwall in Section 4.11.2. This demonstrates that the model successfully replicated the EWLs and was appropriately configured for the simulations. The results show that water levels increase with return period, and the Epoch 2100 and Epoch 2125 simulations produce higher water levels than the Baseline 2025 scenarios, as expected.

Figure 11.3: Statistical maximum water level of the baseline 2025 1-in-50 year RP.



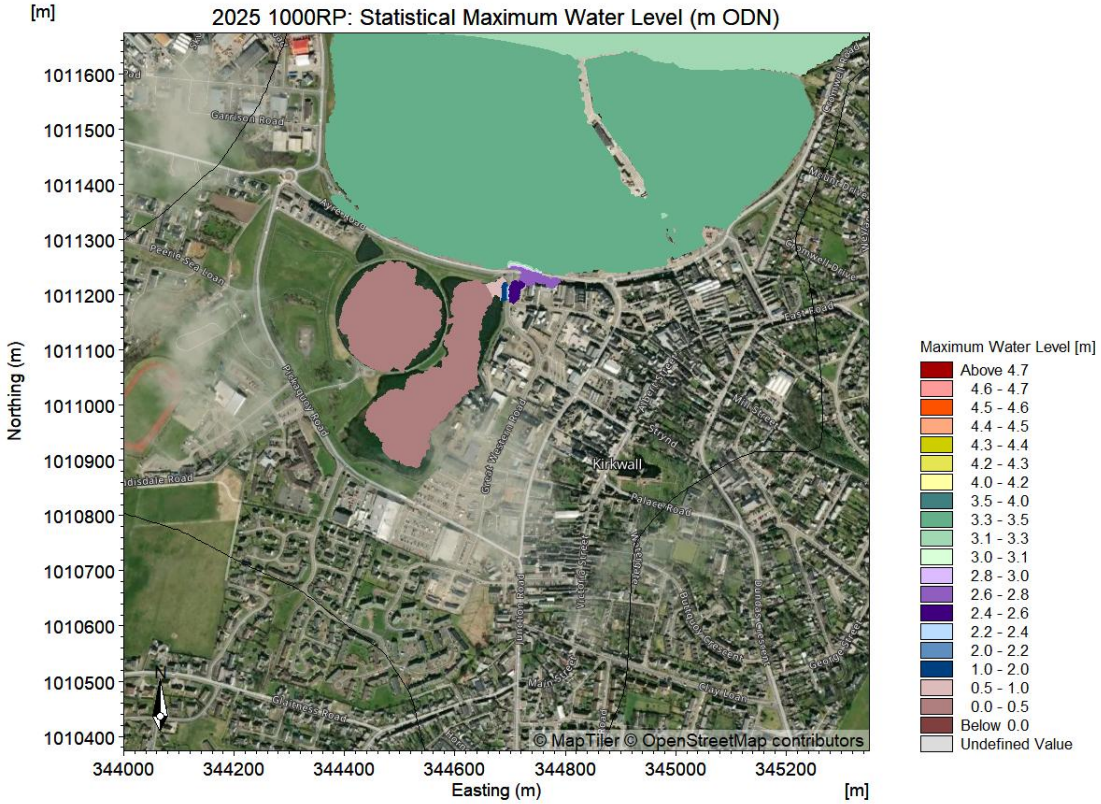
Source: Mott MacDonald 2026

Figure 11.4: Statistical maximum water level of the baseline 2025 1-in-200 year RP.



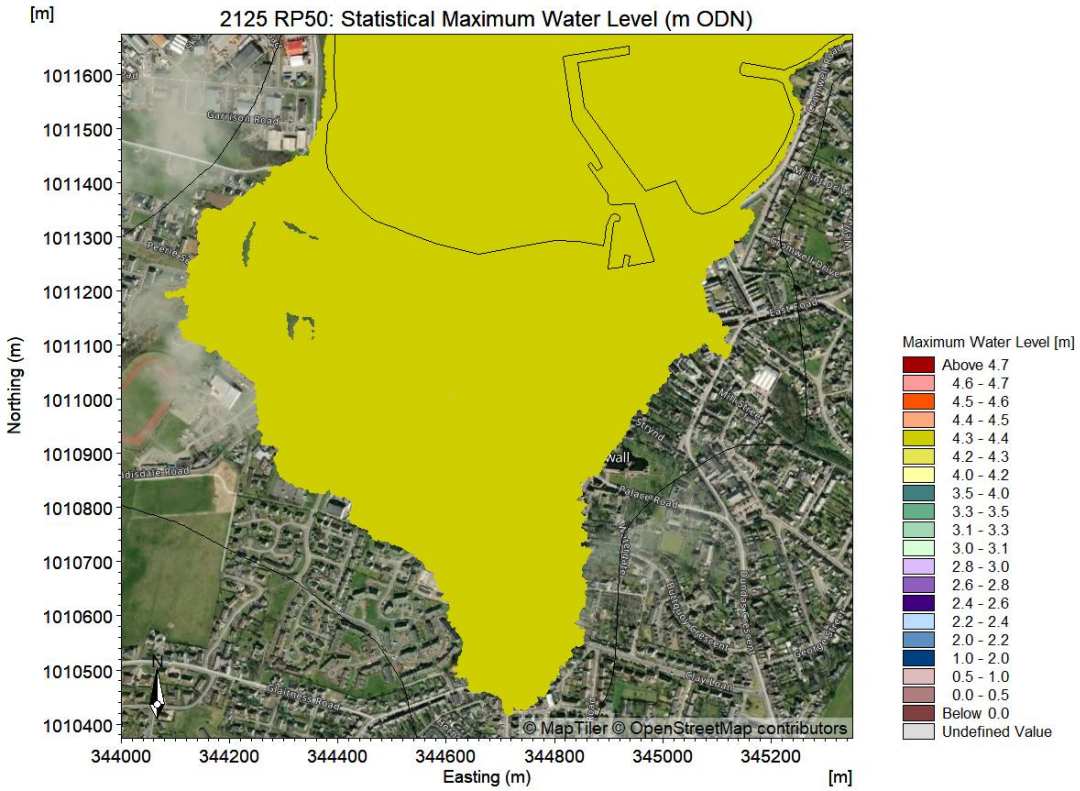
Source: Mott MacDonald 2026

Figure 11.5: Statistical maximum water level of the baseline 2025 1-in-1000 year RP.



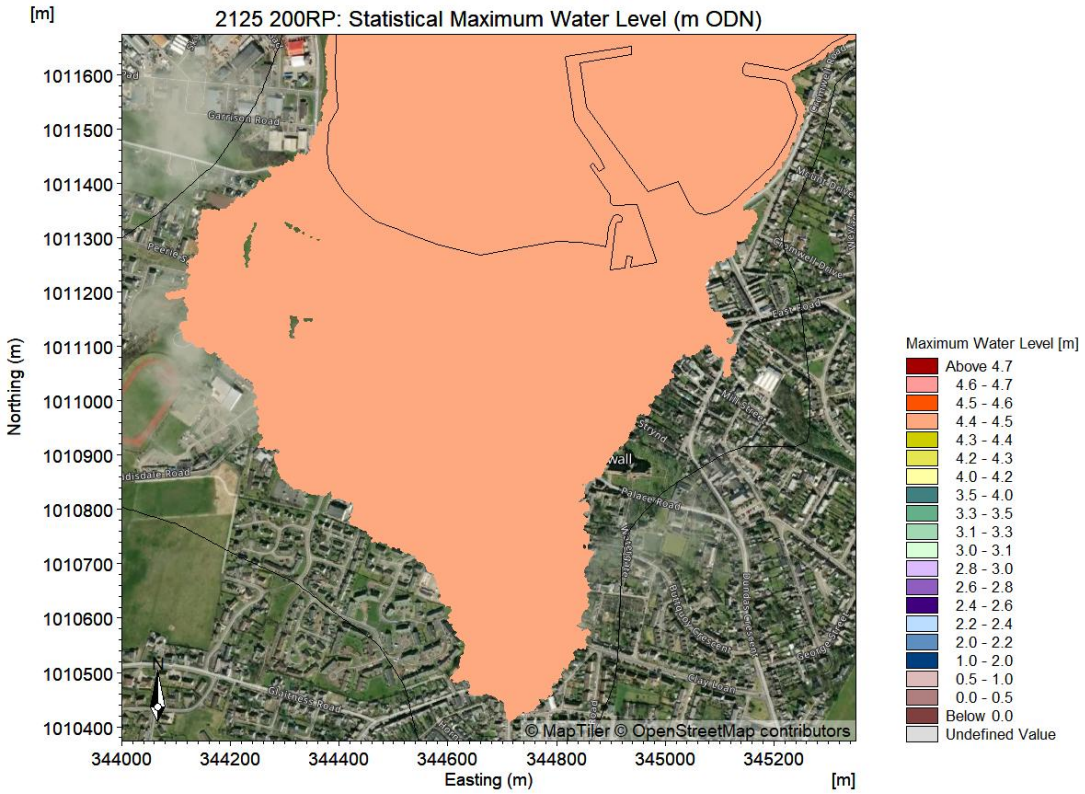
Source: Mott MacDonald 2026

Figure 11.6: Statistical maximum water level of the 2125 1-in-50 year RP.



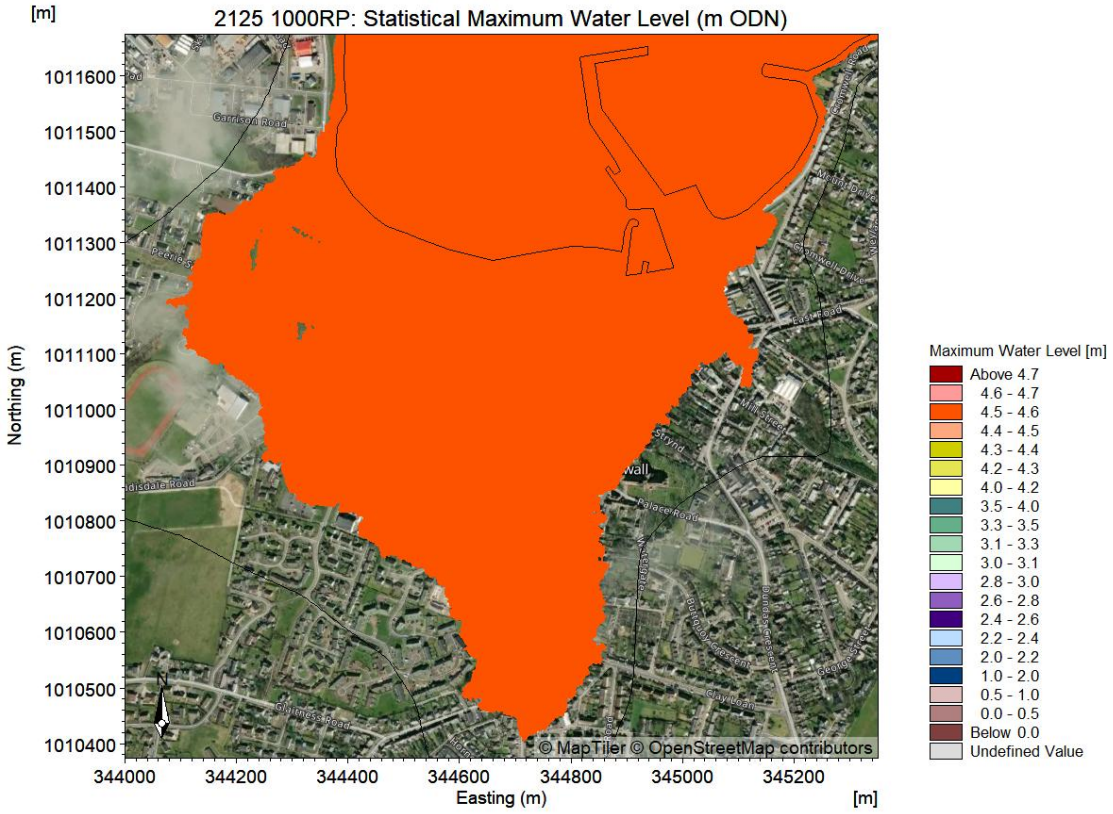
Source: Mott MacDonald 2026

Figure 11.7: Statistical maximum water level of the 2125 1-in-200 year RP.



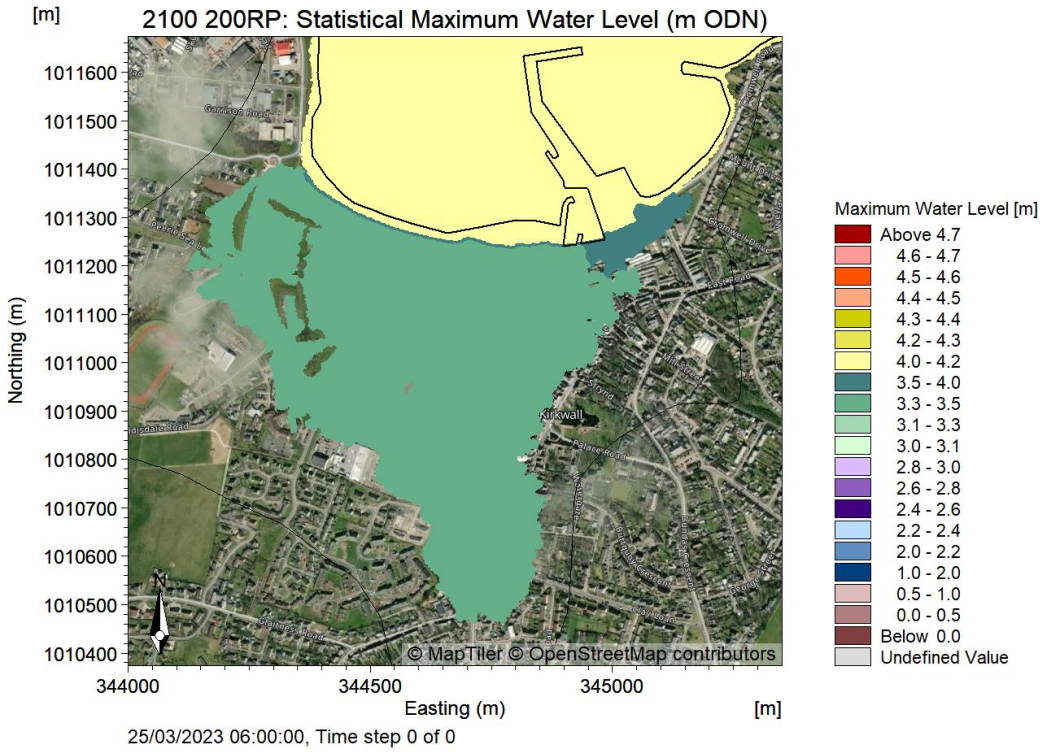
Source: Mott MacDonald 2026

Figure 11.8: Statistical maximum water level of the 2125 1-in-1000 year RP.



Source: Mott MacDonald 2026

Figure 11.9: Statistical maximum water level of the 2100 1-in-200 year RP.



Source: Mott MacDonald 2026

Table 11.2: Extreme water levels (m ODN) including sea level rise (RCP 8.5, 95th percentile) for the baseline 2025 vs maximum water levels of the local overland flow model.

Return period	EWL 2025	Overland flow model
50	3.11	3.13
200	3.20	3.21
1000	3.30	3.30

Source: Mott MacDonald 2026

Table 11.3: Extreme water levels (m ODN) including sea level rise (RCP 8.5, 95th percentile) for the epoch 2125 and maximum water levels of the local overland flow model.

Return period	EWL 2125	Overland flow model
50	4.37	4.38
200	4.46	4.46
1000	4.56	4.56

Source: Mott MacDonald 2026

Table 11.4: Extreme water levels (m ODN) including sea level rise (RCP 8.5, 95th percentile) for the Epoch 2100 vs maximum water levels of the local overland flow model.

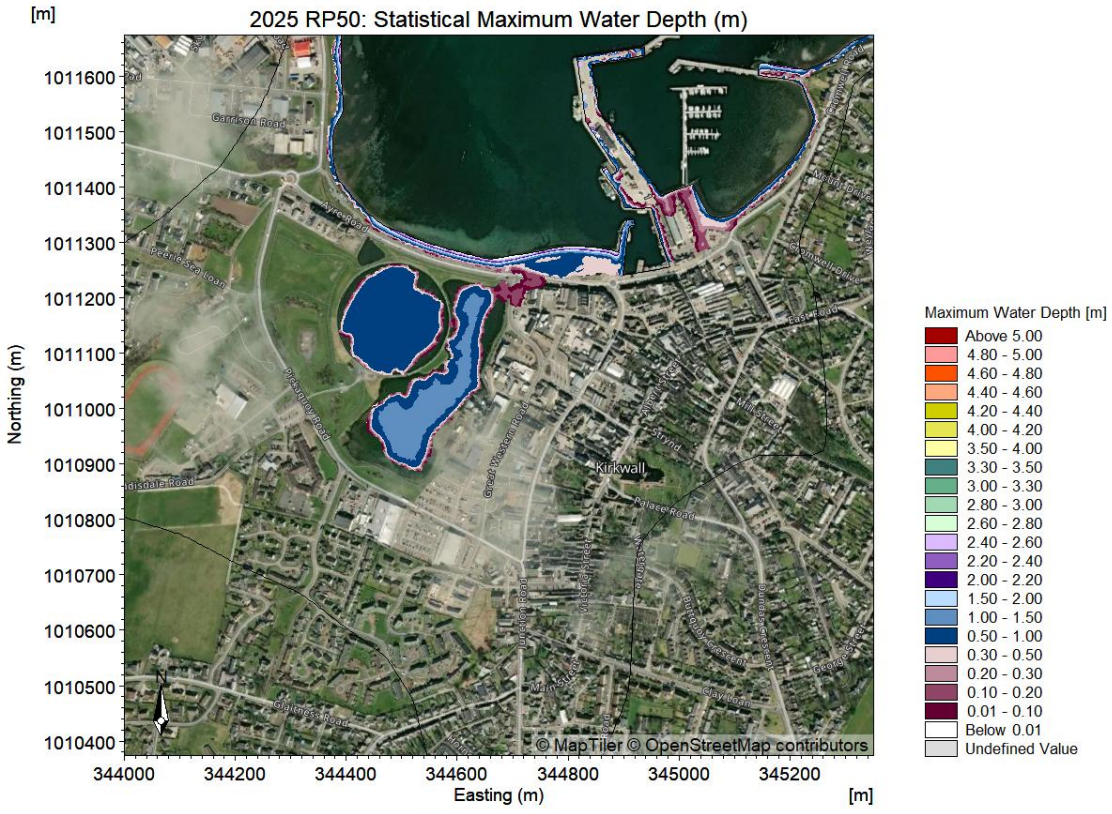
Return period	EWL 2100	Overland flow model
200	4.09	4.09

Source: Mott MacDonald 2026

11.3.2 Water depth

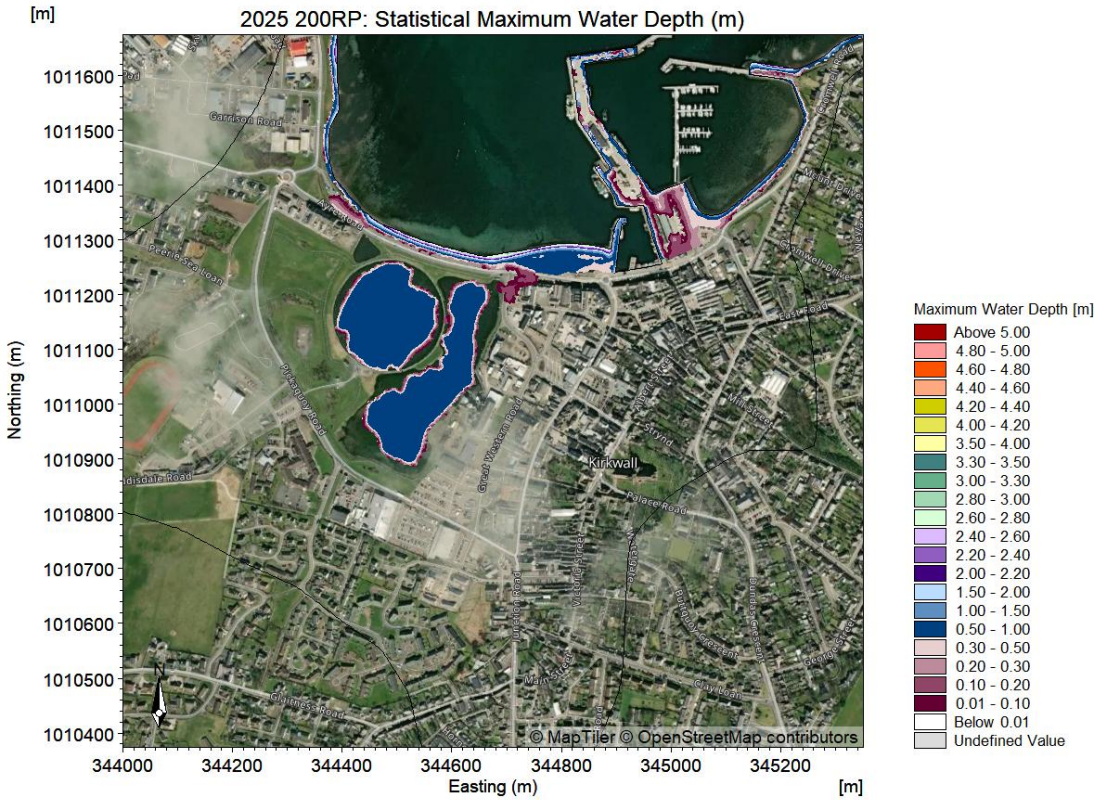
Figure 11.10 to Figure 11.12, and Figure 11.13 to Figure 11.5 shows an overview of the statistical maximum water depth results on the overland area only for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods of the 2025 and 2125 epochs, respectively. Figure 11.16 presents the statistical maximum water depth results on the overland area for the 2100 1-in-200-year return period event.. The results show that flood depth and inundation extent increase with return period, and the Epoch 2100 and 2125simulations produce higher water depths and more widespread flooding than the Baseline 2025 scenarios, as anticipated.

Figure 11.10: Statistical maximum water depth of the baseline 2025 1-in-50 year RP.



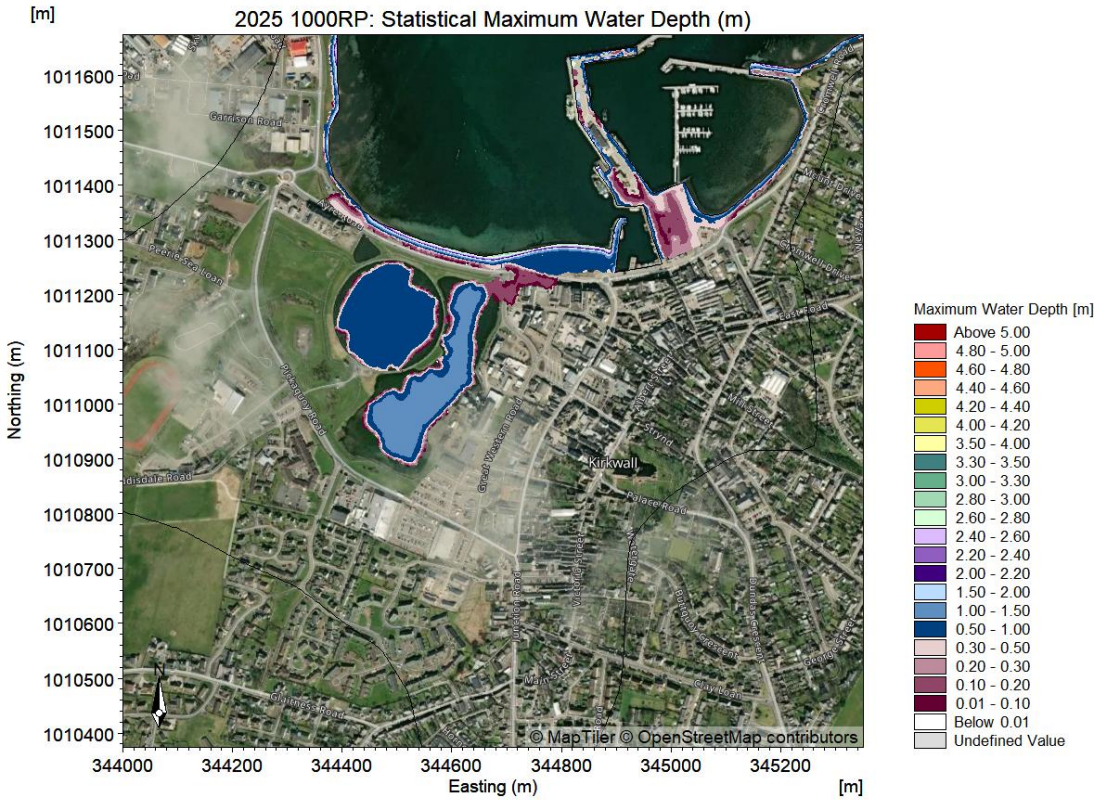
Source: Mott MacDonald 2026

Figure 11.11: Statistical maximum water depth of the baseline 2025 1-in-200 year RP.



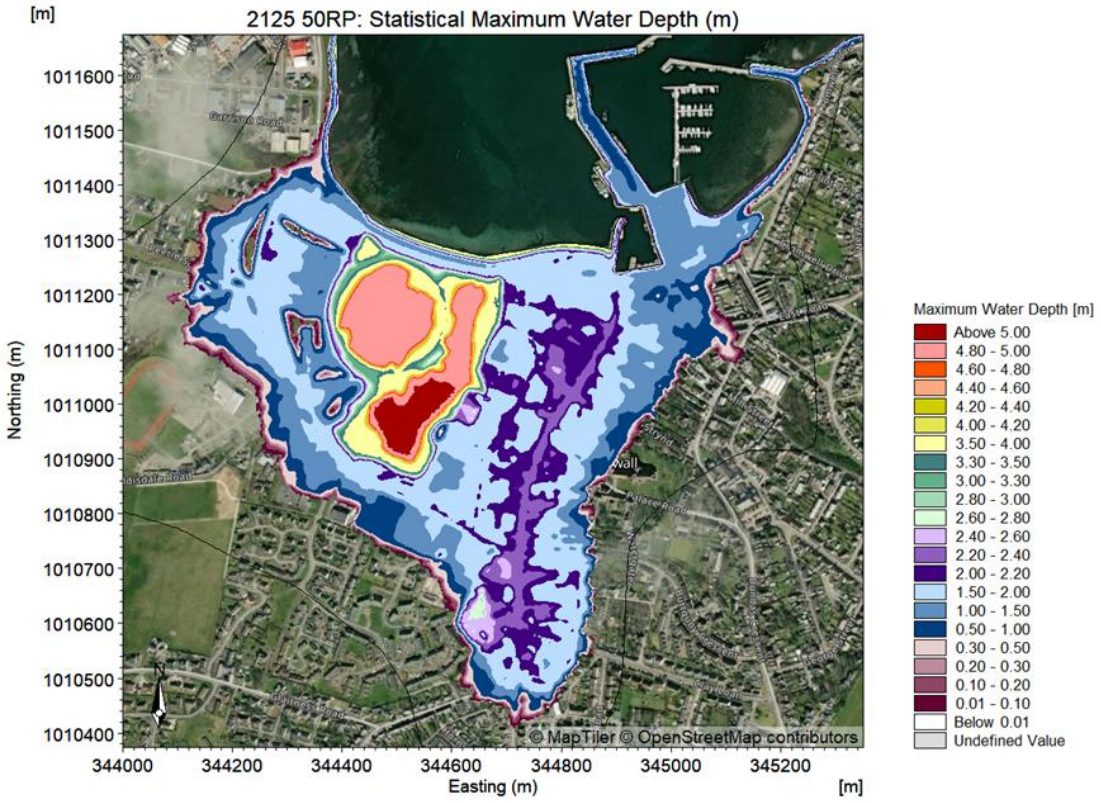
Source: Mott MacDonald 2026

Figure 11.12: Statistical maximum water depth of the baseline 2025 1-in-1000 year RP.



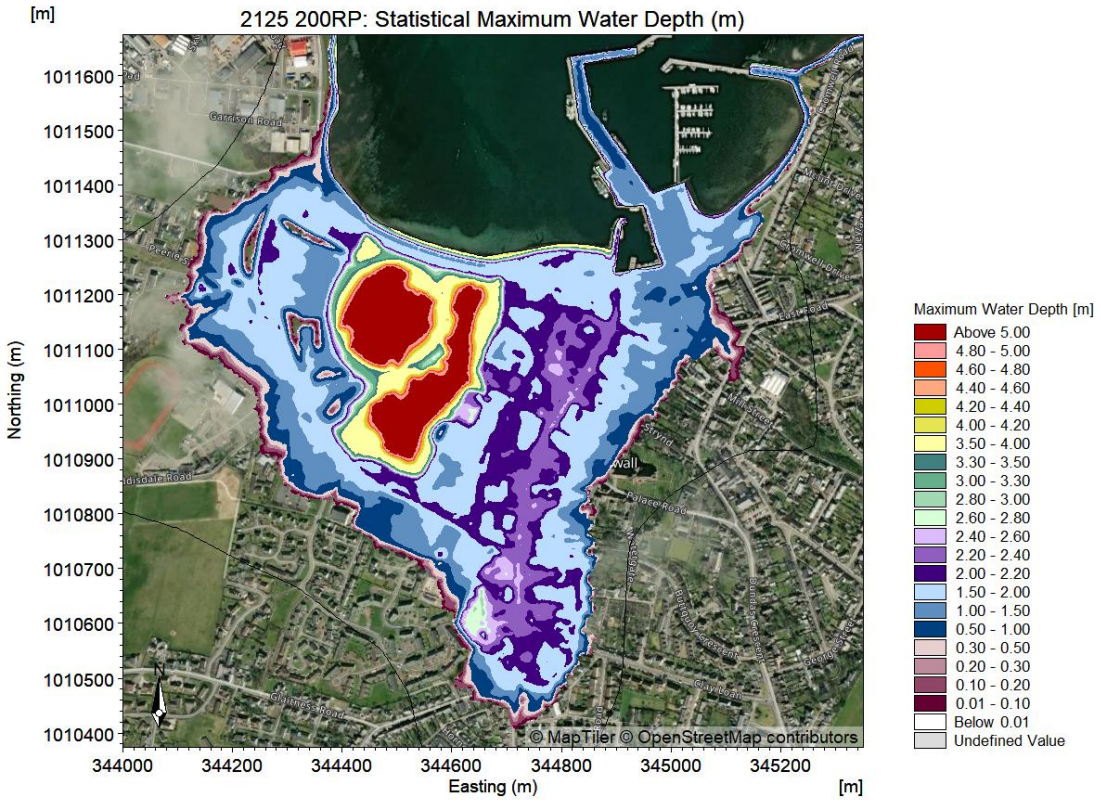
Source: Mott MacDonald 2026

Figure 11.13: Statistical maximum water depth of the baseline 2125 1-in-50 year RP.



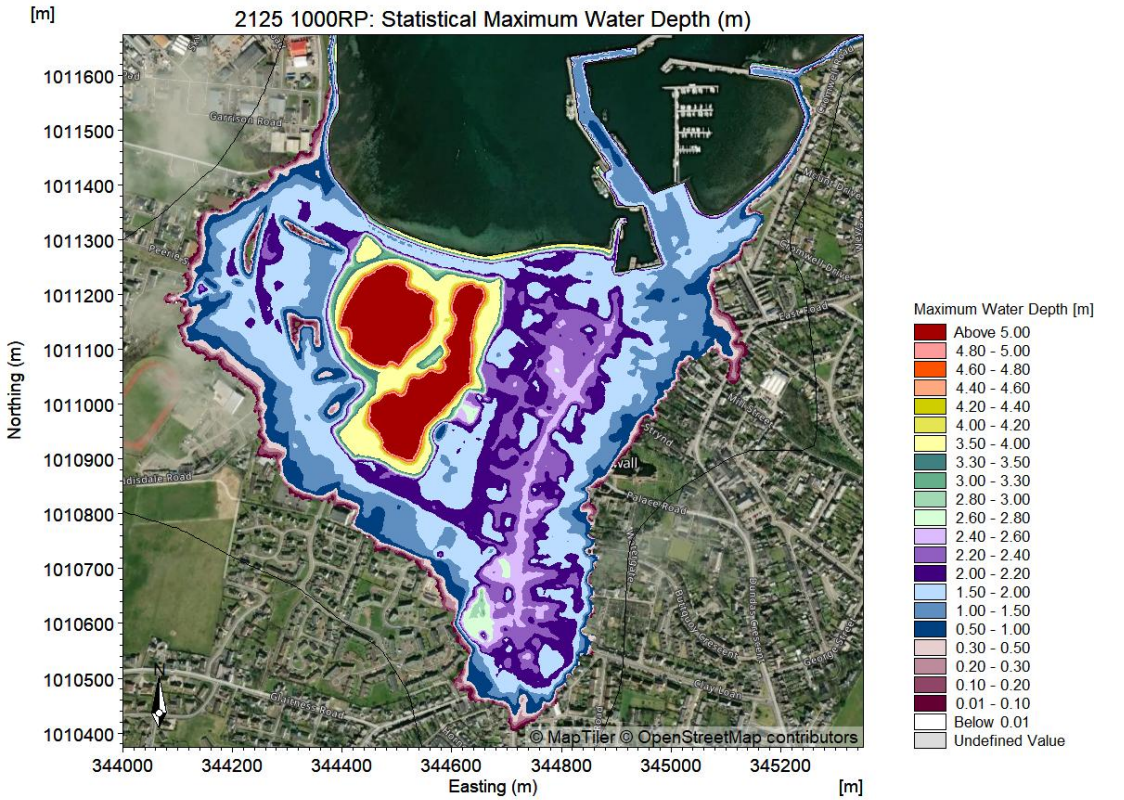
Source: Mott MacDonald 2026

Figure 11.14: Statistical maximum water depth of the baseline 2125 1-in-200 year RP.



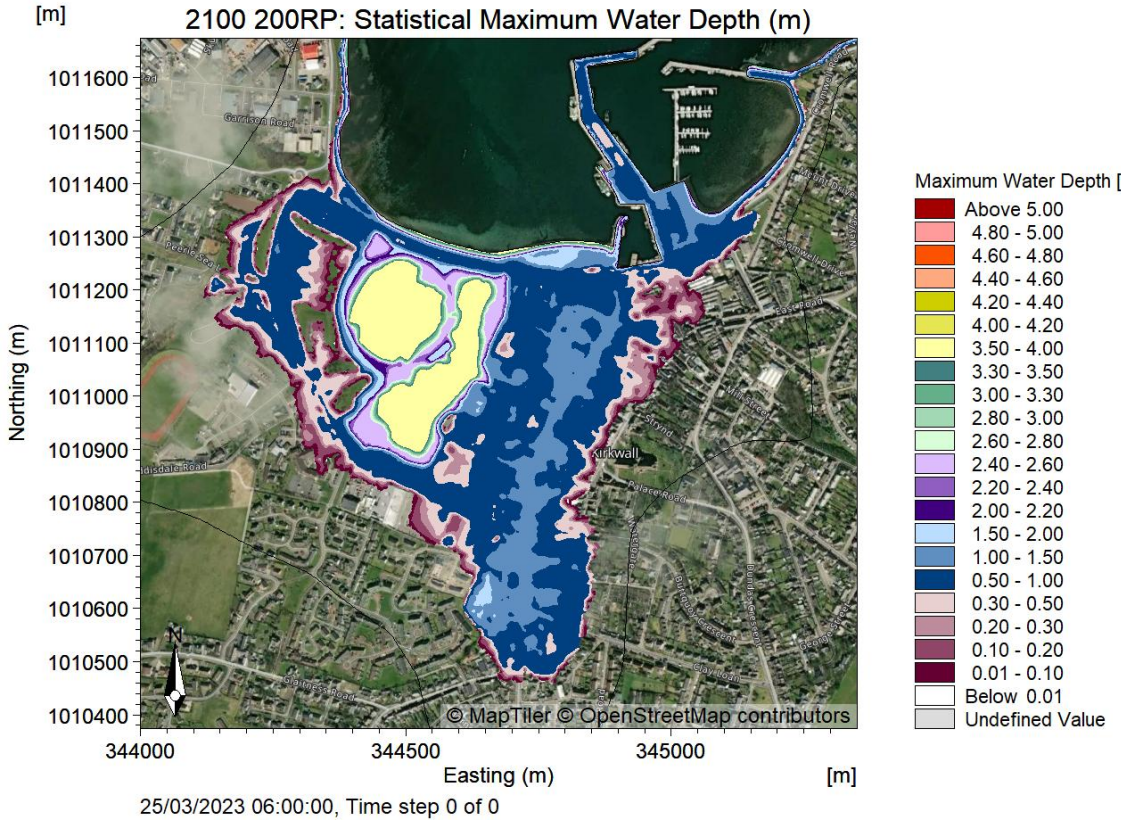
Source: Mott MacDonald 2026

Figure 11.15: Statistical maximum water depth of the baseline 2125 1-in-1000 year RP.



Source: Mott MacDonald 2026

Figure 11.16: Statistical maximum water depth of the baseline 2100 1-in-200 year RP.

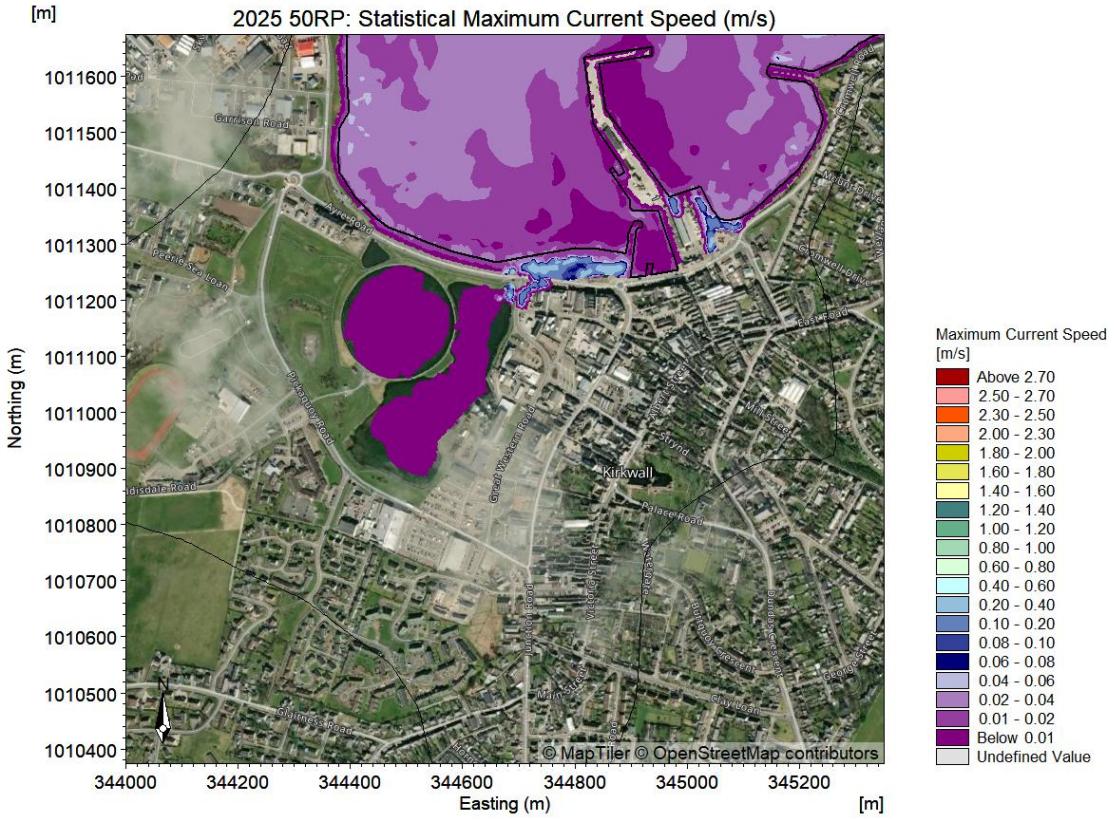


Source: Mott MacDonald 2026

11.3.3 Current speed

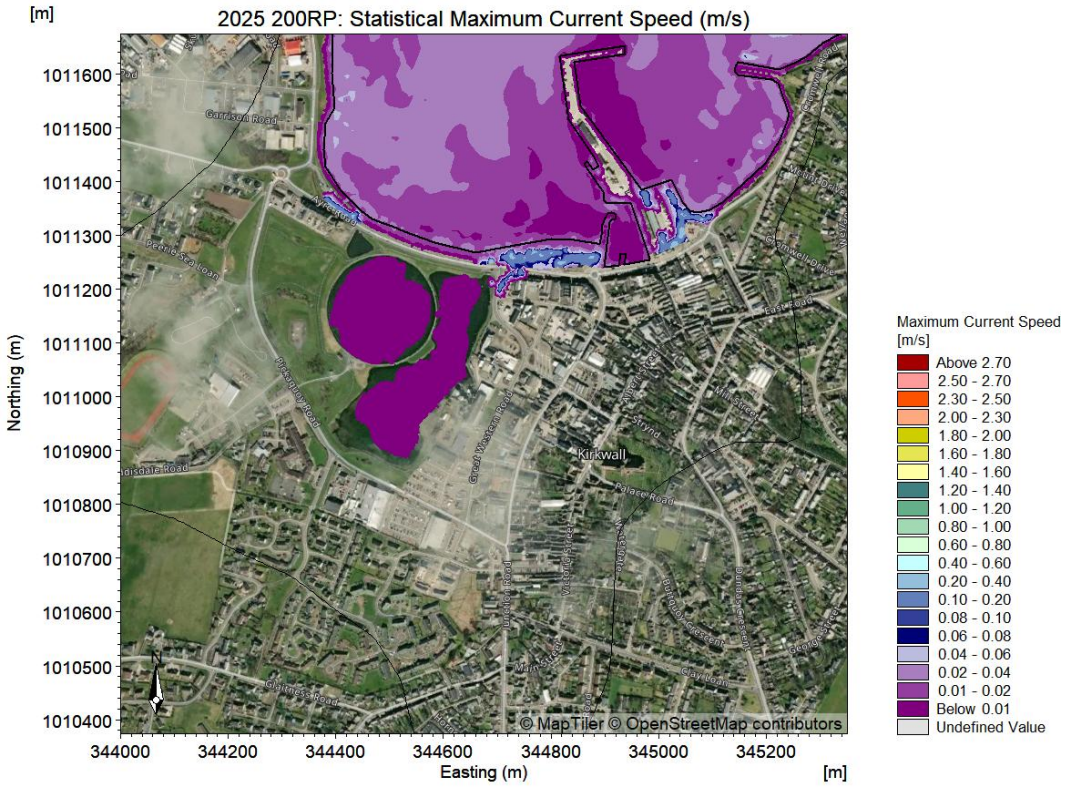
Figure 11.17 to Figure 11.19, and Figure 11.20 to Figure 11.22 shows an overview of the statistical maximum current speed results for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods of the Baseline 2025 and Epoch 2125, respectively. Figure 11.23 presents the statistical maximum current speed results for the 1-in-200-year return periods of the 2100 Epoch. The results show that the Epoch 2125 simulations produce higher current speeds and more widespread on the overland area than the Baseline 2025 scenarios, as expected.

Figure 11.17: Statistical maximum current speed of the baseline 2025 1-in-50 year RP.



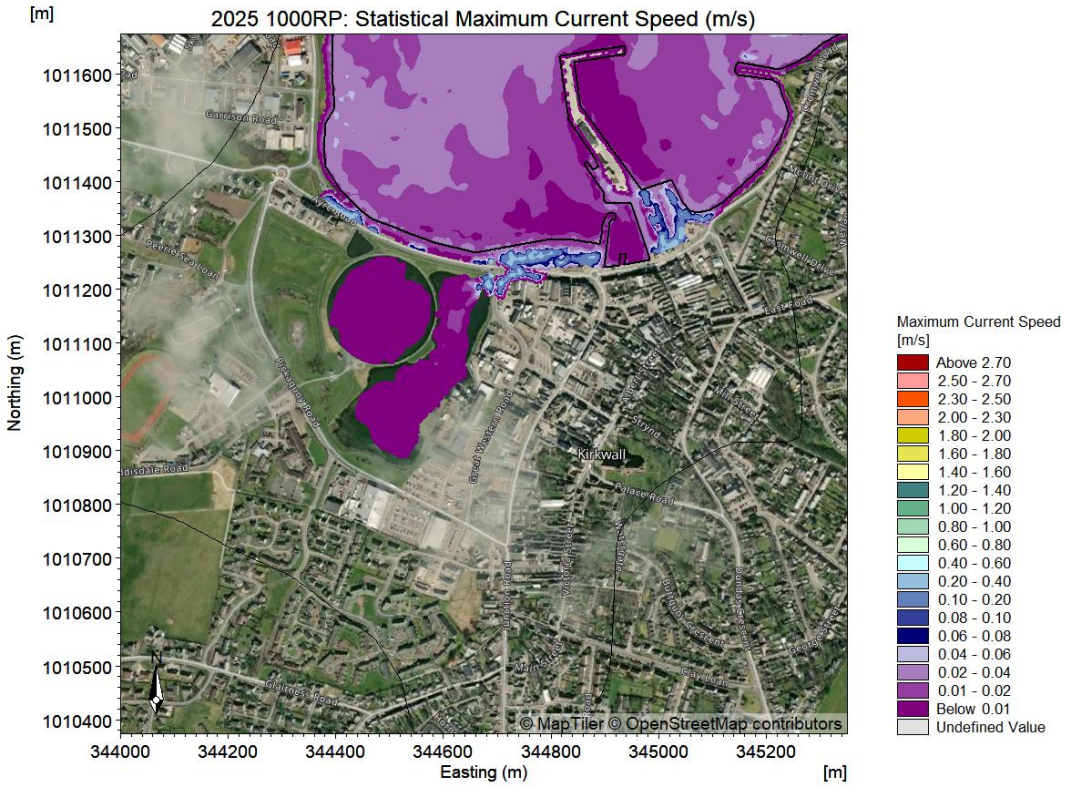
Source: Mott MacDonald 2026

Figure 11.18: Statistical maximum current speed of the baseline 2025 1-in-200 year RP.



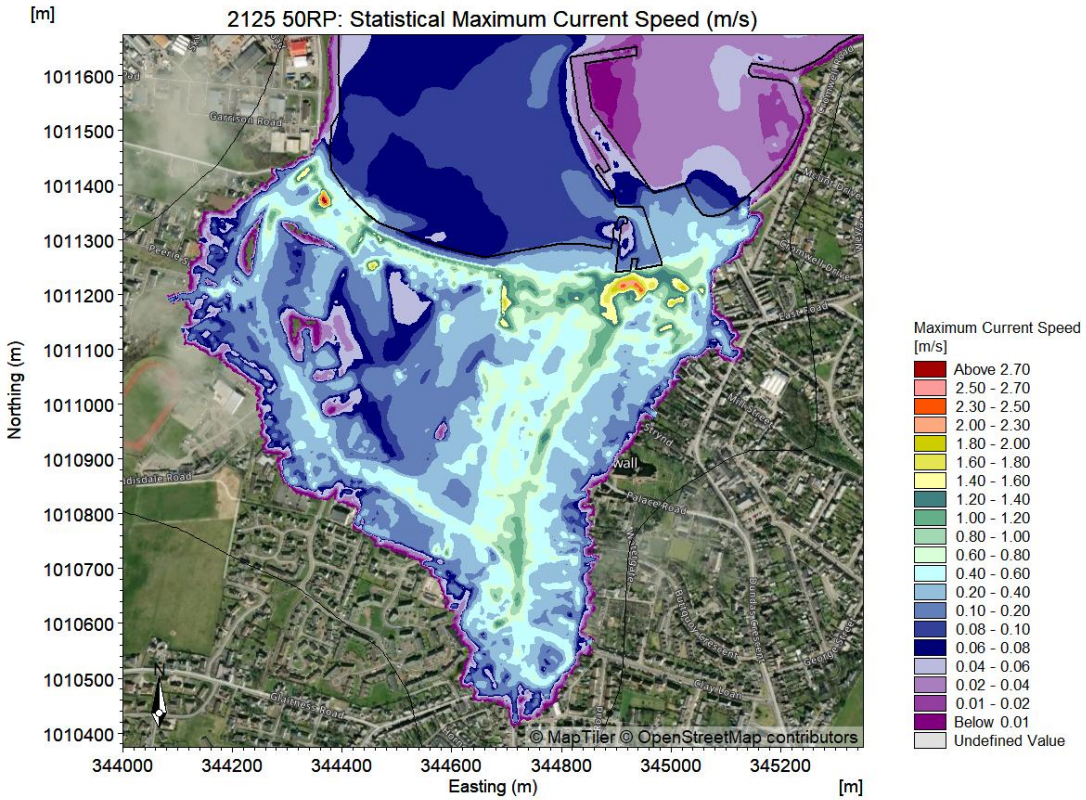
Source: Mott MacDonald 2026

Figure 11.19: Statistical maximum current speed of the baseline 2025 1-in-1000 year RP.



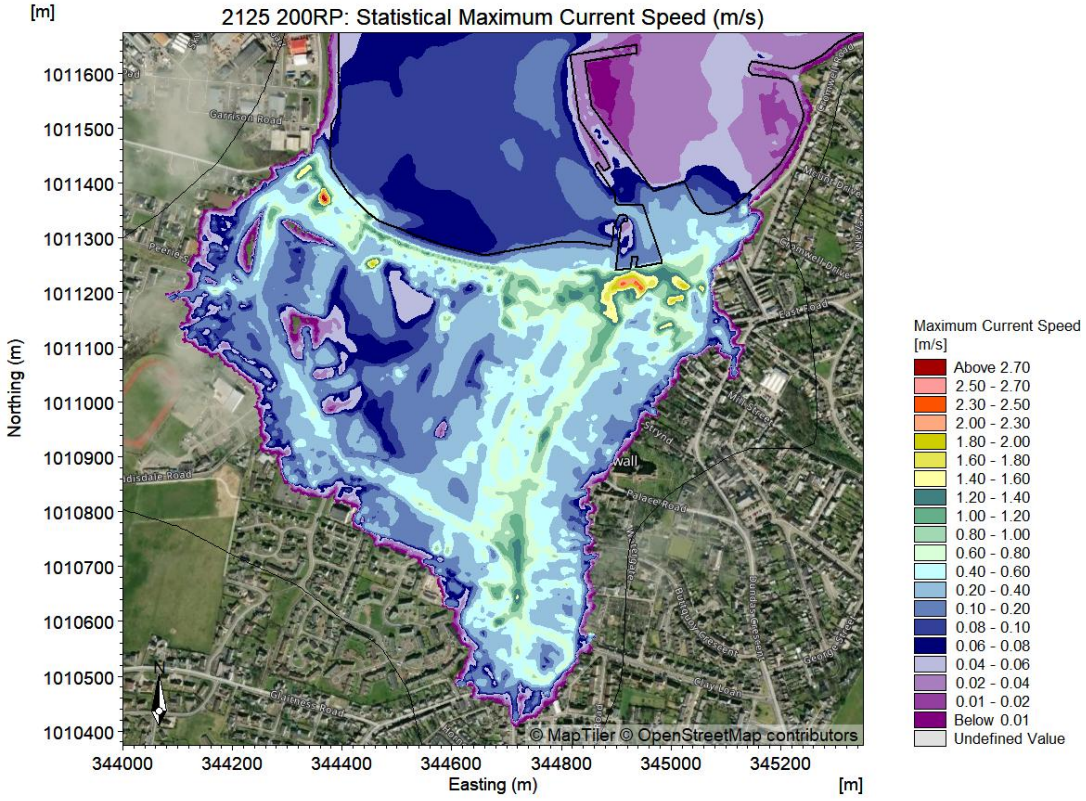
Source: Mott MacDonald 2026

Figure 11.20: Statistical maximum current speed of the baseline 2125 1-in-50 year RP.



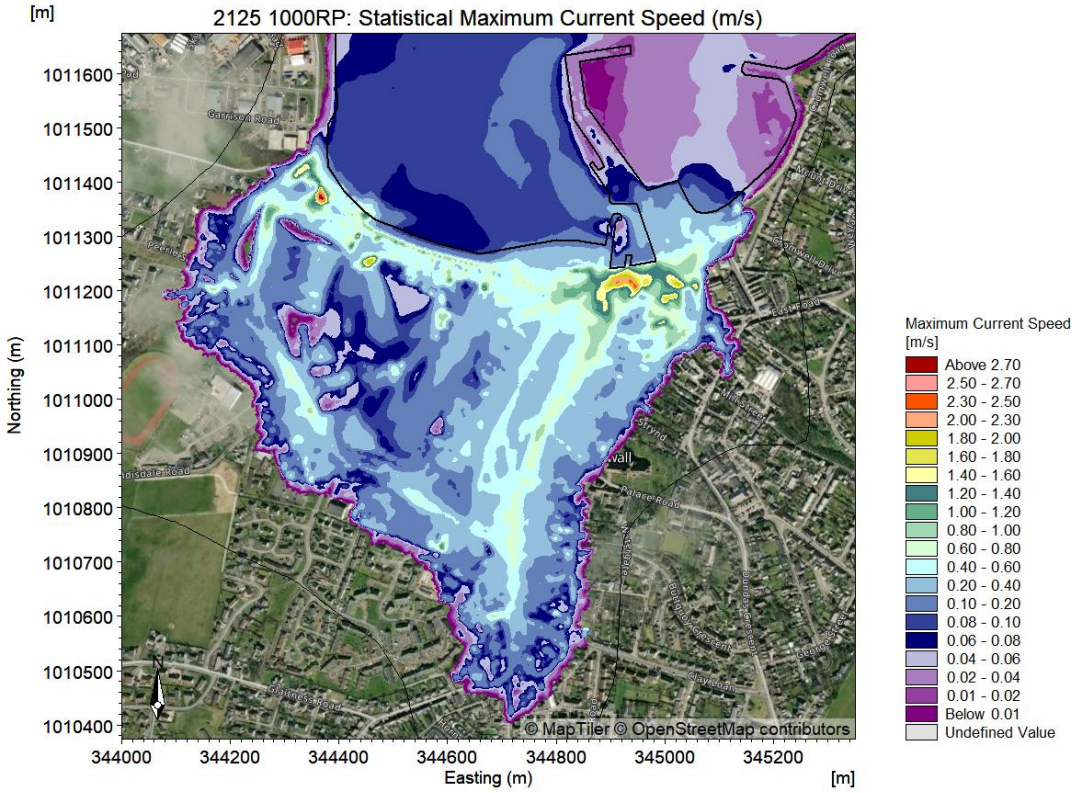
Source: Mott MacDonald 2026

Figure 11.21: Statistical maximum current speed of the baseline 2125 1-in-200 year RP.



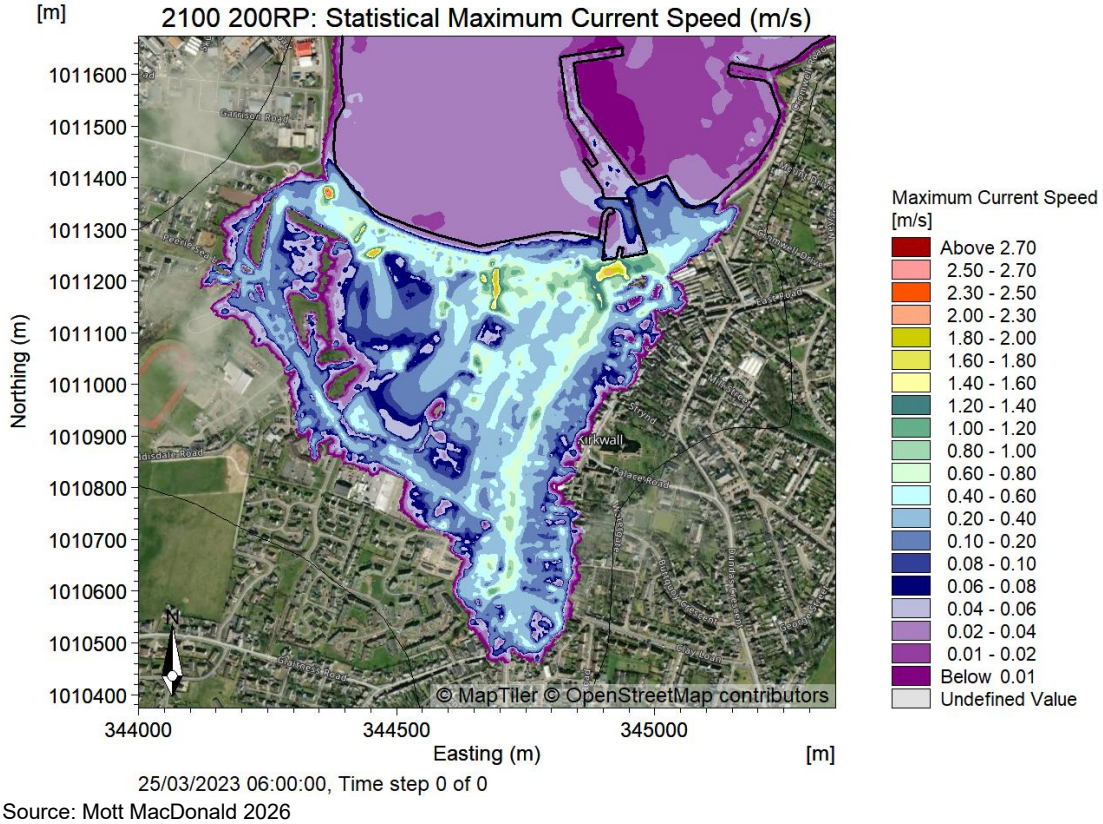
Source: Mott MacDonald 2026

Figure 11.22: Statistical maximum current speed of the baseline 2125 1-in-1000 year RP.



Source: Mott MacDonald 2026

Figure 11.23: Statistical maximum current speed of the baseline 2100 1-in-200 year RP.



Source: Mott MacDonald 2026

12 Summary

This report presents the development of hydrodynamic and spectral wave models, together with a detailed overtopping assessment, for the Kirkwall frontage to evaluate flooding under the baseline (2025), 75-year epoch (2100), and 100-year epoch (2125) conditions. An additional 45-year (2070) epoch scenario was also analysed in the overtopping assessment. The assessment covers the 1-in-50, 1-in-200, and 1-in-1,000-year return periods for 2025 and 2125, and the 1-in-200-year return period for 2100. To support this assessment, regional and local hydrodynamic and wave models were established, accompanied by analyses of extreme water levels, wave conditions, and associated overtopping processes.

The study has shown the following:

- Suitable bathymetric, water level, current, wind, pressure and wave data were identified, quality assured and used to build the hydrodynamic and wave model for the simulations;
- The regional MIKE21 FM HD model was successfully calibrated and validated using measured water levels and tidal diamond data. The model showed strong agreement with both datasets, confirming its suitability for the simulations;
- The regional MIKE21 FM HD models were successfully validated against the historical extreme water level at Kirkwall and Stromness in January 2005, demonstrating that the developed models are robust and suitably reliable for long term simulations to analyse extreme water levels at Kirkwall;
- The extreme water level analysis shows that the extreme water levels from the CFB data are underestimated when compared with the analysed extreme water levels adopted for this study;
- The regional MIKE21 FM SW model was calibrated and validated using offshore wave buoy observations due to the absence of local wave measurements. Model comparisons showed good agreement against measured offshore wave data, confirming that the model configuration is suitable for extended hindcast simulations and for deriving extreme wave conditions at Kirkwall; Local wave measurements were not available, it is recommended that for any future detailed studies that this information is collected to help validate the wave modelling outcomes;
- The extreme water levels, waves and wind conditions were analysed and presented for the 1-in-50, 1-in-200 and 1-in-1,000 year return periods for both the baseline and future epoch scenarios;
- Extreme water levels and waves have been successfully transformed to the structure’s toe for use in the overtopping calculation and assessment;
- For the 2100 1-in-200 year return period event, a separate JPA and additional 1D wave transformations were not undertaken. The overtopping inputs were instead derived using the 2070 dataset uplifted to 2100 and closely matched transformed wave conditions from the 2125 1-in-200 year return period event. This proportionate approach was adopted to support comparison with SEPA’s 2100 flood mapping
- The local MIKE21 FM HD model was calibrated and validated using observed water levels at Kirkwall, including comparisons against the recorded extreme water level during the January 2005 event. The resulting model data agreement across tidal and surge conditions demonstrates that the model setup is appropriate for simulating local overland HD model to support flooding assessments;

- The local overland HD model simulations were successfully validated against extreme water levels at Kirkwall;
- The maximum water level and water depth in the 100-yr epoch scenarios is greater than the baseline scenarios as anticipated;
- The Epoch 2125 simulations produce higher current speeds and were more widespread on the overland area than the Baseline 2025 scenarios;
- The simulated flood extent increases with return period where the 1,000-year event producing a substantially larger inundation footprint than the 50-year and 200-year event.
- The comparison between the baseline and climate change scenarios highlights a significant escalation in flood exposure and reinforces the need for future-proofing flood risk management and coastal defence planning at Kirkwall.
- The climate change scenarios consistently produce higher water levels, larger inundation extents, and greater flow velocities. This indicates the importance of incorporating future climate allowances into flood hazard mapping and risk-based decision making.

The main outputs of the overtopping analysis are:

- An overtopping discharge time-series for the present and future epochs, for 50-, 200-, and 1000-year return events;
- There is anticipated to be no overtopping at the rear of the Waterfront Car Park, due to the significant promenade width;
- In the present epoch, there is minimal overtopping along the majority of the crest wall, with the exception of profile C (near the roundabout on Ayre Road);
- The combined effect of the revetment and crest wall is significant; there is significant discharge over the crest of the armour in the present epoch, but negligible discharge over the crest wall;
- Once the crest of the revetment of the defence overflows, overtopping of the crest wall increases dramatically;
- The year that this significant increase in overtopping begins to occur for the 1-in-200-year event is 2070 (the mid-epoch);
- In the future epoch (2125) the crest of the defence overflows, contributing significantly more discharge to the OFM than overtopping.
- Under the 2100 1-in-200 year return period scenario, the seawall is overtopped by direct overflow around the central storm peak. Consequently, direct overflow is the principal contribution to the predicted inundation, with wave overtopping contributing during the surrounding tidal peaks when water levels remain below the seawall crest.

12.1 Assumptions and limitations

- The results from the modelling study have made the best use of available data. This should be reviewed and updated as and when new information becomes available.
- The coupled configuration required significantly greater computational time, thus the regional hydrodynamic and wave models were operated separately. This approach did not influence the results, as wave-current interaction at the regional scale is minimal in the enclosed setting of Kirkwall Bay and both models were independently calibrated against available measurements.
- The JPA was performed using the JPA FD2308 tool from Defra and Environmental Agency.

- The climate change scenarios were agreed with the OIC and followed the SEPA (2025) Climate change allowances for flood risk assessment.
- The best available bathymetry and LiDAR datasets were merged and interpolated across the model domain, filling areas where data gaps were present.
- The overland flow model simulation period was selected to coincide with the high spring tide in March 2023 to provide a conservative assessment which is the same as the hydrodynamic model calibration window to maintain consistency across this study.
- The surge shape profile was taken at Lerwick from the CFB data.
- The overland domain was configured using a 5 m grid resolution, which provides an efficient balance between computational cost and spatial detail, allowing key surface features such as buildings and road networks to be adequately represented within the model
- The buildings and overland bed roughness were setup in the model based on the SEPA (2015) flood model guidance.
- Buildings were aggregated and represented as stubby building blocks at 5 m resolution, reflecting the fact that many structures are smaller than the grid size and cannot be resolved individually. Full retention of detailed building geometry would substantially increase computational time without providing a corresponding improvement in predicted flood extents.
- The overland flow modelling considered coastal flooding only; rainfall, drainage, and fluvial sources were not included in the overland flow model.
- The existing flood defence crest levels applied in the model were provided by the OIC.
- The flood defence structures were represented as dikes in the local overland hydrodynamic model.
- The two buildings in front of the Kirkwall frontage were treated as defences set at +3.9 m ODN, which is consistent to the defence heights on either side of the building structures.
- No calibration was undertaken for the overland flow model; however, the model outputs were compared against SEPA's published Flood Risk Management mapping for the 2080 high-emissions climate-change scenario (1-in-200-year return period) to provide a benchmarking reference.
- No joint probability analysis or one-dimensional transformation was performed to produce the 1-in-200-year return overland flow model for 2100 as interpolation between data from 2070 and 2125 was considered sufficient to produce the overtopping time series. This was proportional approach was appropriate due the minimal contribution of overtopping to the overland flow model in this epoch
- The overtopping discharge values applied in the overland flow model were derived from the overtopping assessment.
- No calibration was performed for the overtopping tool, however overtopping discharges were within the range expected, and the tool has been validated in prior projects.
- Overtopping was calculated using EurOtop 2018 methods.
- Foreshore slope and toe of defence were estimated for one defence section as LiDAR and bathymetry lacked data – assumptions were backed with available data.
- Time-series inputs to overland hydrological flow model assume the worst overtopping occurs for a storm event with the peak of the tide and surge aligned, based on EA (2019) guidance

- It is assumed no wave overtopping occurs over the rear wall of the Waterfront Car Park when water levels are below the level of the promenade wall due to the large car park width and limited depth of water in the car park.

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A. Regional MIKE21 FM SW model setup

Table A.1: Summary of the regional MIKE21 FMSW model setup.

	Parameters	Description
	Basic Equations	Fully Spectral, Instationary formulation
Spectral Discretisation	No. of frequency	25
	Min Frequency	0.03 Hz
	Frequency factor	1.1
	Number of directions	360 degrees rose (36 directions)
	Separation of wind sea and swell	Wage age approach; Coefficient = 0.83; Power = 1
Solution Technique	Geographical space discretisation	Low-order, fast algorithm
	Max. no of levels in transport calculation	32
	No of steps in source calculation	1
	Min time step	0.01 sec
	Max time step	3600 seconds
Water Level Conditions		Time series of the predicted water levels at the Offshore West Shetland station.
Wind Forcing	Wind data	Varying in time and domain used wind data from the ECMWF ERA5 (wind velocities at 10m elevation)
	Soft start	0
	Growth parameter	1.42
	Wave age turning parameter	0.015
	Type of air-sea	Uncoupled
	Type of drag	Version 1
	Background Charnock parameter	0.01
Energy Transfer		Include quadruplet-wave interaction
Wave Breaking	Model	Wave breaking
	Type of gamma	Specified gamma
	Gamma data	Constant: 0.8
	Alpha	Constant: 1
Bottom Friction	Model	No Bottom friction
White Capping	Model	White Capping Included
	Dissipation Coefficient, C dis	Constant: 1
	Dissipation Coefficient, DELTA dis	Constant: 1
Initial Conditions		Spectra from empirical formula from JONSWAP fetch growth expression
Boundary Conditions		Wave parameters the ERA5 data.

Source: Mott MacDonald, 2026

B. Regional MIKE21 FM SW model setup

Table B.1: Summary of the local MIKE21 FMSW model setup.

Parameters		Description
Equation		Directional-decoupled, quasi-stationary
Directional discretisation	No. of direction	360 degree rose (36 directions)
Solution technique		Quasi-stationary
Quasi-stationary formulation	Geographical space discretisation	Low-order, fast algorithm
	Method	New-Raphson iteration
	Max. number of iterations	200
	Tolerance (RMS-norm. of residual)	0.001
	Tolerance (Max-norm of change in Hs)	0.025
	Relaxation factor	0.1
Water level conditions		JPA (Section 7.4)
Current conditions		No current variation included
Wind forcing	Wind data	JPA (Section 7.4); Wind direction = 22.5° intervals, covering the sector from northwest to northeast.
	Soft start	0
	Wind generation formula	SPM84
Diffraction	Smoothing factor	1
	Number of smoothing steps	5
Wave breaking	Model	Wave breaking
	Type of gamma	Specified gamma
	Gamma data	0.8
	Alpha	1
	Gamma (wave steepness)	5
Bottom friction	Model	0
Initial conditions		Spectra from empirical formula from JONSWAP fetch growth expression
Boundary conditions		JPA (Section 7.4)); MWD = 22.5° intervals, covering the sector from northwest to northeast.

Source: Mott MacDonald, 2026

C. Local Overland Hydrodynamic model updates

MOTT MACDONALD

Orkney Islands CCAP

Project:	Kirkwall Modelling and Overtopping Assessment Studies		
Our reference:	100122478-MMD-ZZ-ZZ-TN-MD-0001	**Revision:**	S3 P01
Prepared by:	Raydo Atan	**Date:**	02/04/2026
Approved by:	Simon Burchett	**Checked by:**	Adrian Wright
Subject:	Local Overland Hydrodynamic Model Updates		

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1 Introduction

A local overland MIKE21 FM HD model was simulated for two cases where the buildings in front of the Kirkwall frontage were treated as follows:

- Case 1: Building acts as a flood defence (crest level = +3.9 m ODN).
- Case 2: Building does not act as a flood defence (effective crest level = ground level +0.3 m).

For Case 1, the local overland MIKE21 FM HD model was simulated to incorporate defences at the positions of the two buildings located along the Kirkwall frontage, as indicated by the red box in . The buildings were schematised as defences set at +3.9 m ODN, which is consistent with the defence heights on either side of the buildings. Wave overtopping discharges were applied along three designated lengths, as shown in Figure 1.1.

For Case 2, the buildings were not considered to act as flood defences (at least to the same height as in Case 1). The schematisation involved assuming the building footprints had the same ground elevation as the surrounding areas but raised by an additional 0.3m allowance (SEPA flood model guidance), e.g. to represent building threshold elevations. Wave overtopping discharges were applied along the same three designated lengths, as shown in Figure 1.2.

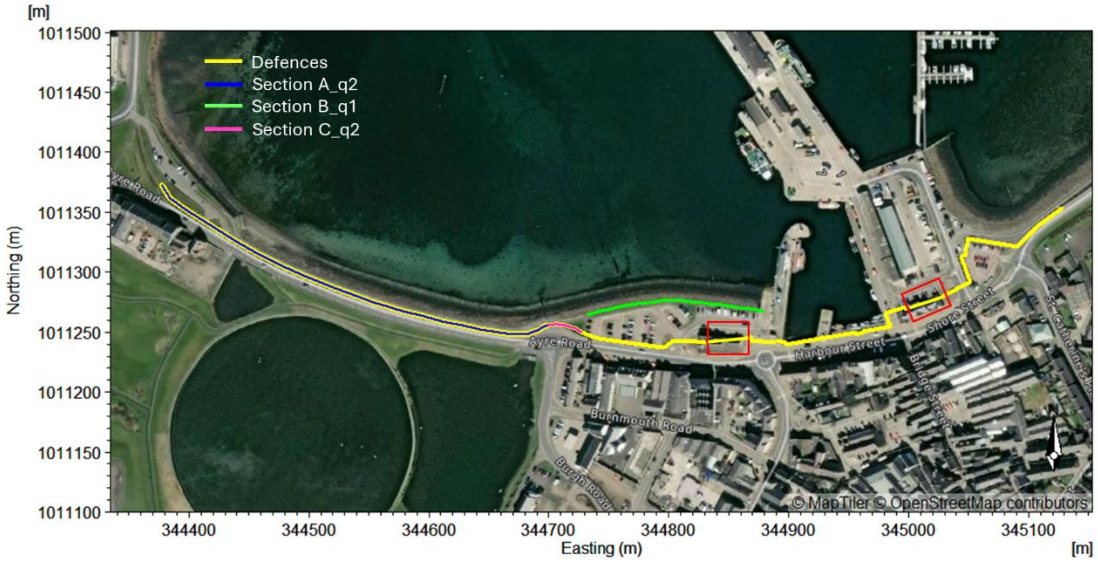
Table 1.1 shows a summary of model scenarios for both cases that were simulated for the local overland flood model. The model results for the present-day 2025 and 2125 epoch scenarios are presented in the next section accordingly for each case.

Table 1.1: Model Scenarios

Scenarios	Description
Present-day 2025 (1-in-50 RP)	• Present-day scenario for the 1-in-50-year RP (i.e. AEP 2%) • Surge extreme water level of 1-in-50-year RP with SLR • Overtopping discharges for 1-in-50-year RP at designated sections
Present-day 2025 (1-in-200 RP)	• Present-day scenario for the 1-in-200-year RP (i.e. AEP 0.5%) • Surge extreme water level of 1-in-200-year RP with SLR • Overtopping discharges for 1-in-200-year RP at designated sections
Present-day 2025 (1-in-1,000 RP)	• Present-day scenario for the 1-in-1,000-year RP (i.e., AEP 0.1%) • Surge extreme water level of 1-in-1,000-year RP with SLR • Overtopping discharges for 1-in-1,000-year RP at designated sections
Epoch 2125 (1-in-50 RP)	• 100-year epoch scenario for the 1-in-50-year RP (i.e. AEP 2%) • Surge extreme water level of 1-in-50-year RP with SLR • Overtopping discharges for 1-in-50 year-RP at designated sections
Epoch 2125 (1-in-200 RP)	• 100-year epoch scenario for the 1-in-200-year RP (i.e. AEP 0.5%) • Surge extreme water level of 1-in-200-year RP with SLR • Overtopping discharges for 1-in-200-year RP at designated sections
Epoch 2125 (1-in-1,000 RP)	• 100-year epoch scenario for the 1-in-1,000-year RP (i.e. AEP 0.1%) • Surge extreme water level of 1-in-1,000-year RP with SLR • Overtopping discharges for 1-in-1,000-year RP at designated sections

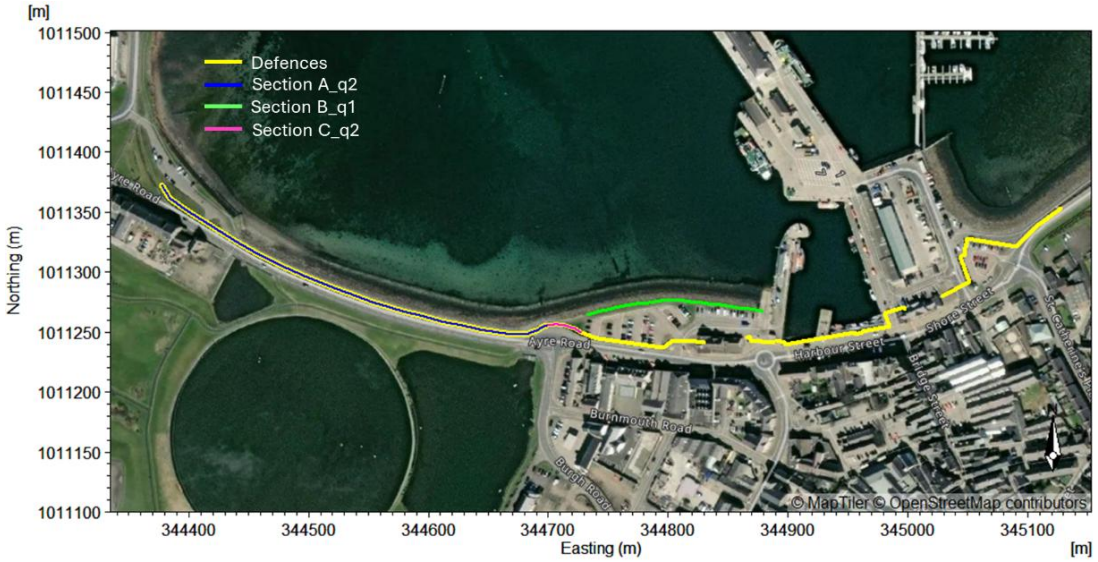
Source: Mott MacDonald, 2026

Figure 1.1: Flood Defences and Wave Overtopping Boundaries (Case 1)



Source: Mott MacDonald 2026. Imagery from Google Earth.

Figure 1.2: Flood Defences and Wave Overtopping Boundaries (Case 2)



Source: Mott MacDonald 2026. Imagery from Google Earth.

1.1 Summary

The present-day 2025 Case 1 results show that representing the two buildings as flood defences produces a significant reduction in tidal inundation and overland flow pathways. In contrast, the present-day 2025 Case 2, where building elevations are taken directly from the Digital Terrain Model LiDAR dataset (and increased by 0.3 m), results in substantially greater flood extents.

The results also show that the 2125 epoch simulations produce higher water levels, water depths and current speeds than the present-day 2025 scenarios, as expected.

Review of existing as built information for the Kirkwall Harbour Flood Defence scheme indicates that the resilience level of the buildings in the flood defence line is likely higher than 0.3 m, though robustness to the defended level is less likely as that would rely on doors and other non-flood defence assets.

On this basis Case 2 could be considered overly conservative/ pessimistic given there are some PFR measures incorporated in the buildings.

We recommend that the Case 1 results to OIC and SEPA as a reasonable representation of the likely behaviour if demountables are deployed correctly. We also recommend that OIC explore additional measures on the buildings identified in this note to improve the robustness up to the level of the defences, and that the installation of the demountables on the defences (especially the gates on the car park and the ferry terminal) be trialled and the robustness of the approach confirmed.

2 Case 1 results: Buildings as defences

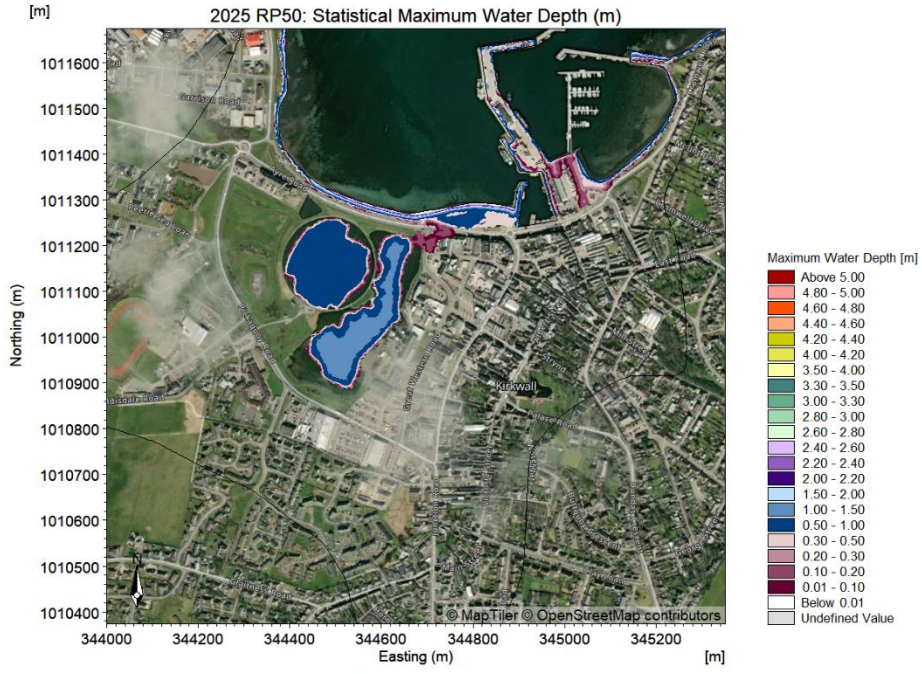
This section presents the model results for water level, water depth and current speed for the present-day 2025 and 2125 epoch scenarios as outlined in for Case 1.

3 Water depth

Figure 3.1 to Figure 3.12 show an overview of the statistical maximum water depth results on the overland area only for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods for present-day 2025 and 2125 epochs respectively. As expected, the results show that flood depths and inundation extent increase with return period, and the 2125 epoch simulations lead to greater water depths and more widespread flooding than the present-day 2025 scenarios.

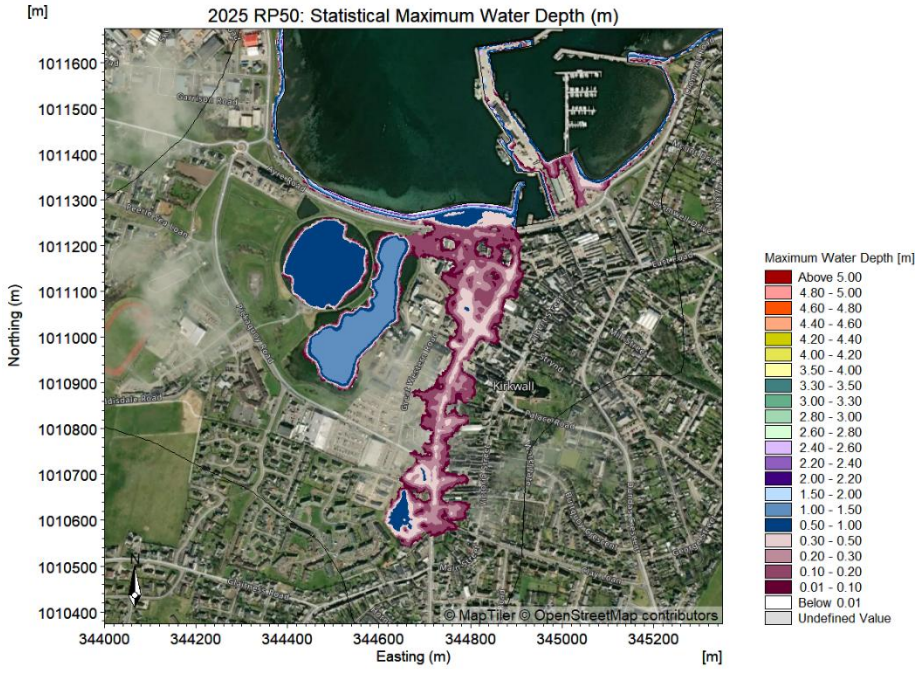
3.1 Present day (2025)

Figure 3.1: Case 1 - Statistical maximum water depth (2025, 1-in-50-year RP)



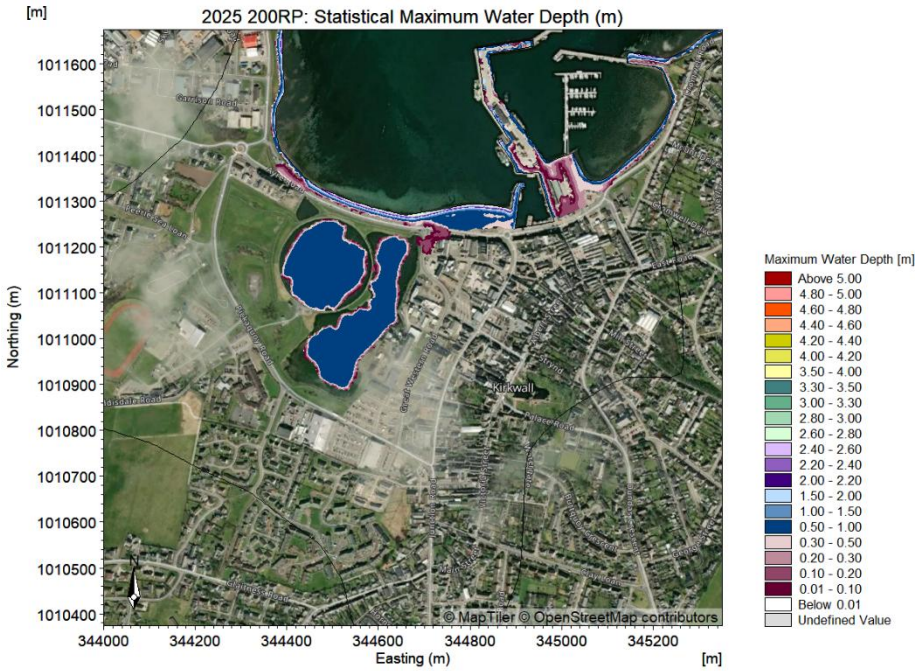
Source: Mott MacDonald 2026

Figure 3.2: Case 2 - Statistical maximum water depth (2025, 1-in-50-year RP)



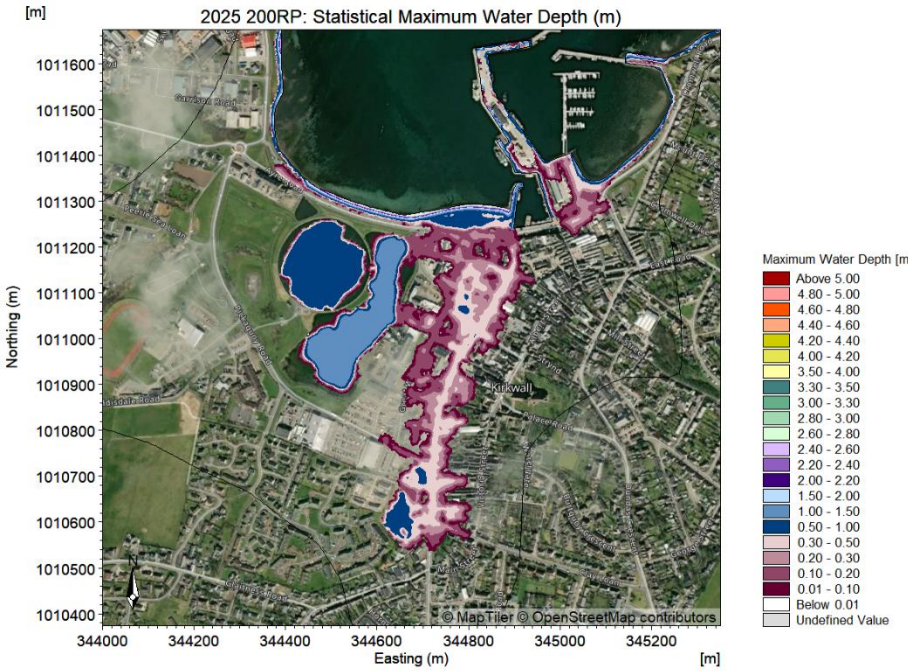
Source: Mott MacDonald 2026

Figure 3.3: Case 1 - Statistical maximum water depth (2025, 1-in-200-year RP)



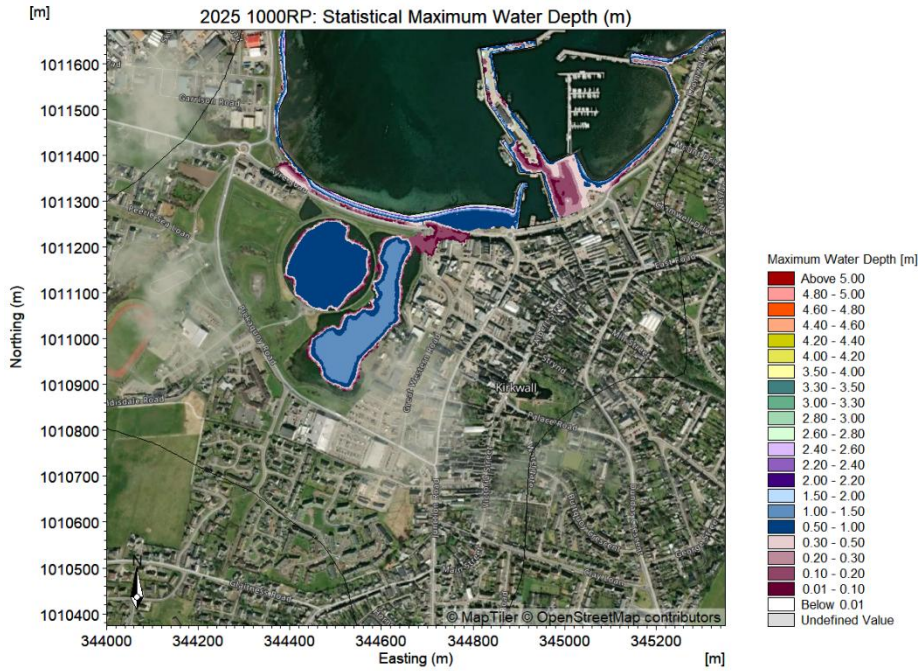
Source: Mott MacDonald 2026

Figure 3.4: Case 2 - Statistical maximum water depth (2025, 1-in-200-year RP)



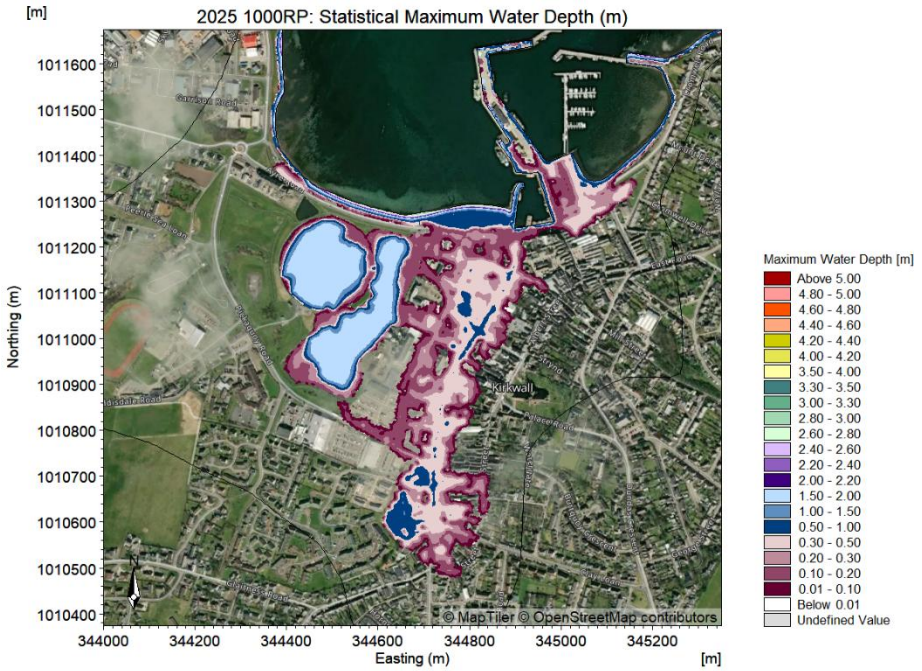
Source: Mott MacDonald 2026

Figure 3.5: Case 1 - Statistical maximum water depth (2025, 1-in-1000-year RP)



Source: Mott MacDonald 2026

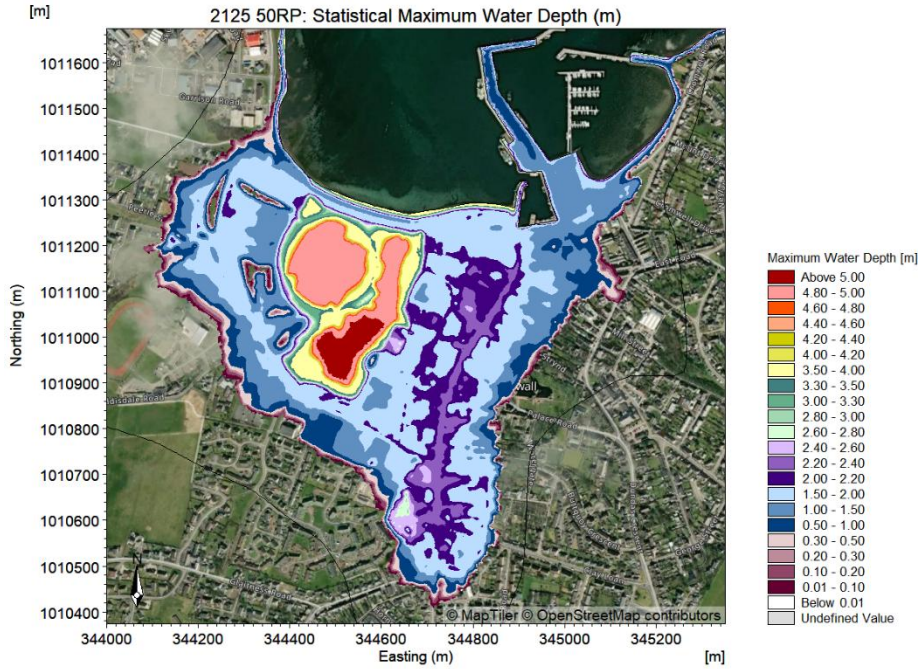
Figure 3.6: Case 2 - Statistical maximum water depth (2025, 1-in-1000-year RP)



Source: Mott MacDonald 2026

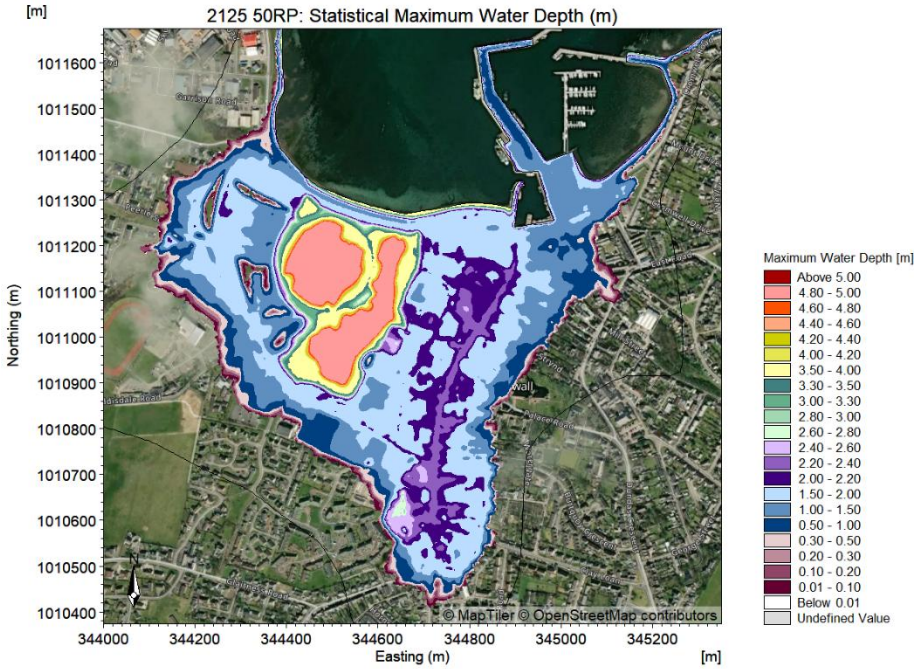
3.2 2125

Figure 3.7: Case 1 -Statistical maximum water depth (2125, 1-in-50-year RP)



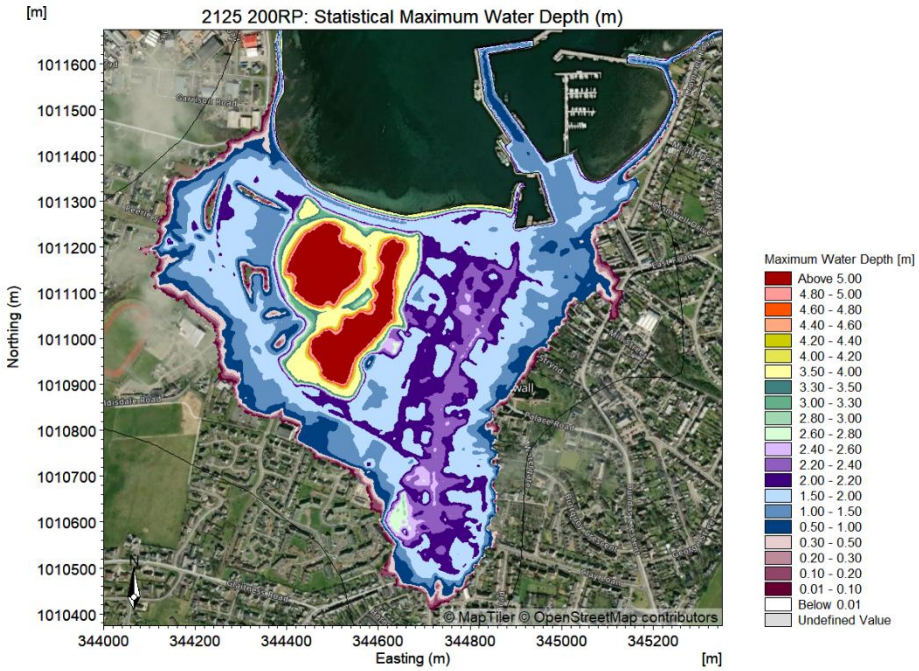
Source: Mott MacDonald 2026

Figure 3.8: Case 2 - Statistical maximum water depth (2125, 1-in-50-year-RP)



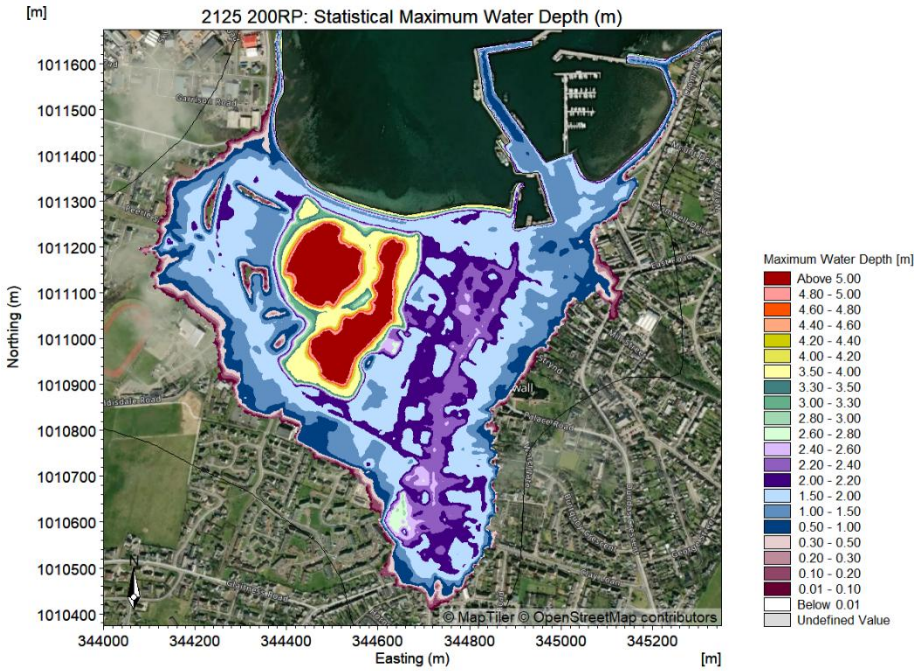
Source: Mott MacDonald 2026

Figure 3.9: Case 1 - Statistical maximum water depth (2125, 1-in-200-year RP)



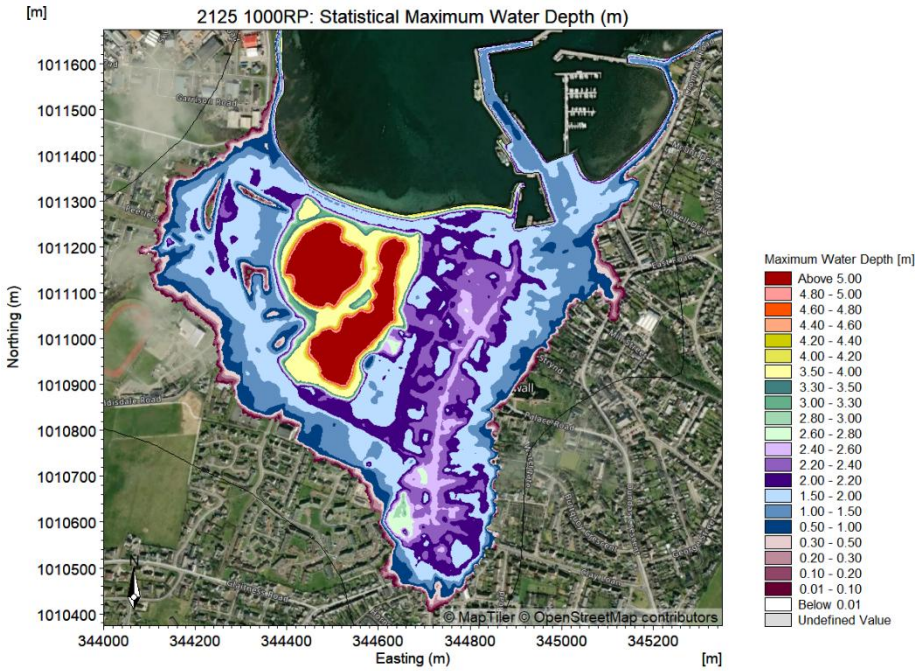
Source: Mott MacDonald 2026

Figure 3.10: Case 2 - Statistical maximum water depth (2125, 1-in-200-year RP)



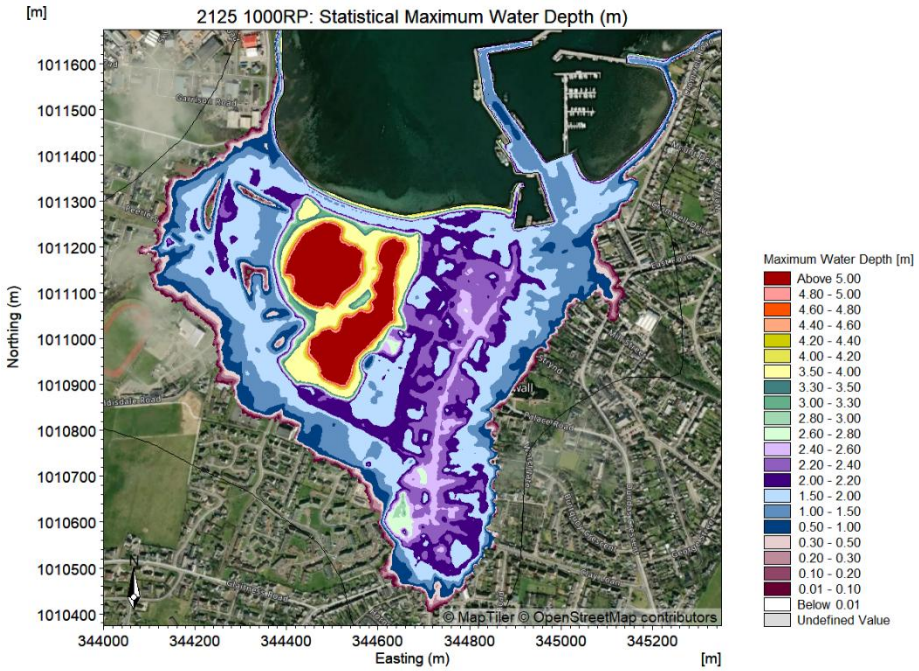
Source: Mott MacDonald 2026

Figure 3.11: Case 1 - Statistical maximum water depth (2125, 1-in-1000-year RP)



Source: Mott MacDonald 2026

Figure 3.12: Case 2 - Statistical maximum water depth (2125, 1-in-1000-year RP)



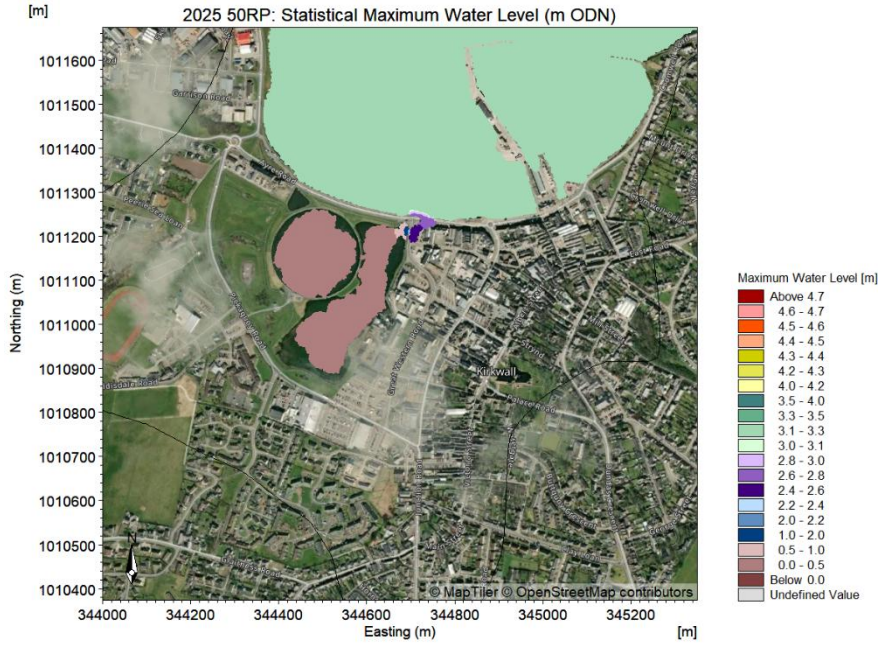
Source: Mott MacDonald 2026

4 Water level

Figure 4.1 to Figure 4.12 show an overview of the statistical maximum water level results for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods, for the present-day 2025 and 2125 epochs respectively. As expected, the results show that water levels increase with return period, and the 2125 epoch simulations produce higher water levels than the present-day 2025 scenarios .

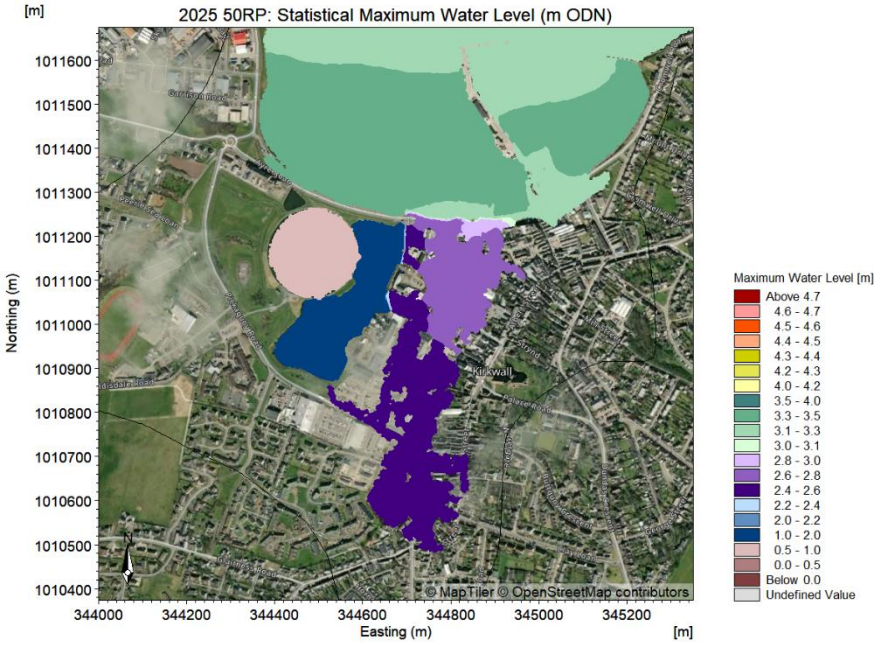
4.1 Present day (2025)

Figure 4.1: Case 1 - Statistical maximum water level (2025, 1-in-50-year RP)



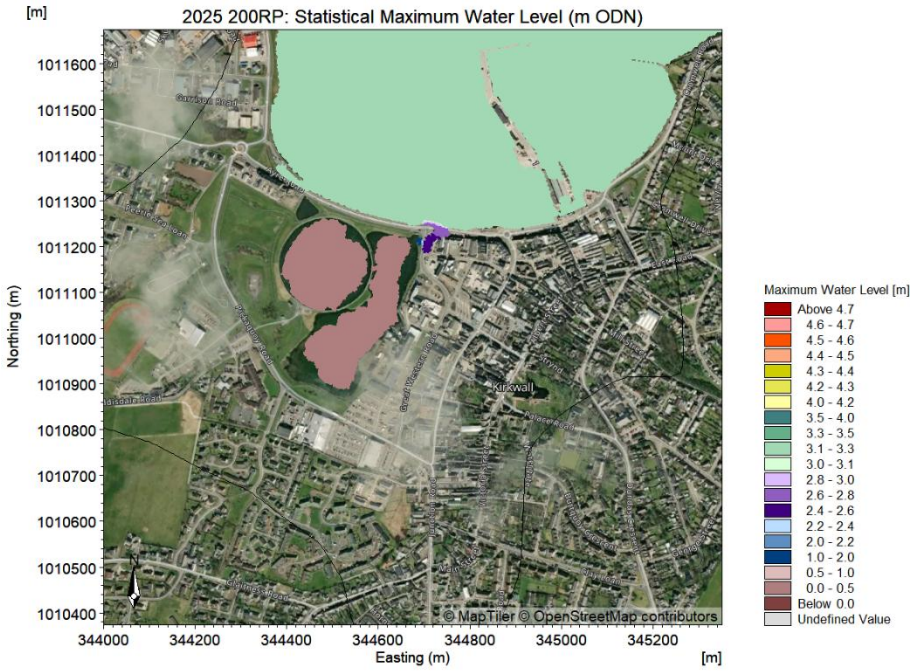
Source: Mott MacDonald 2026

Figure 4.2: Case 2 - Statistical maximum water level (2025, 1-in-50-year RP)



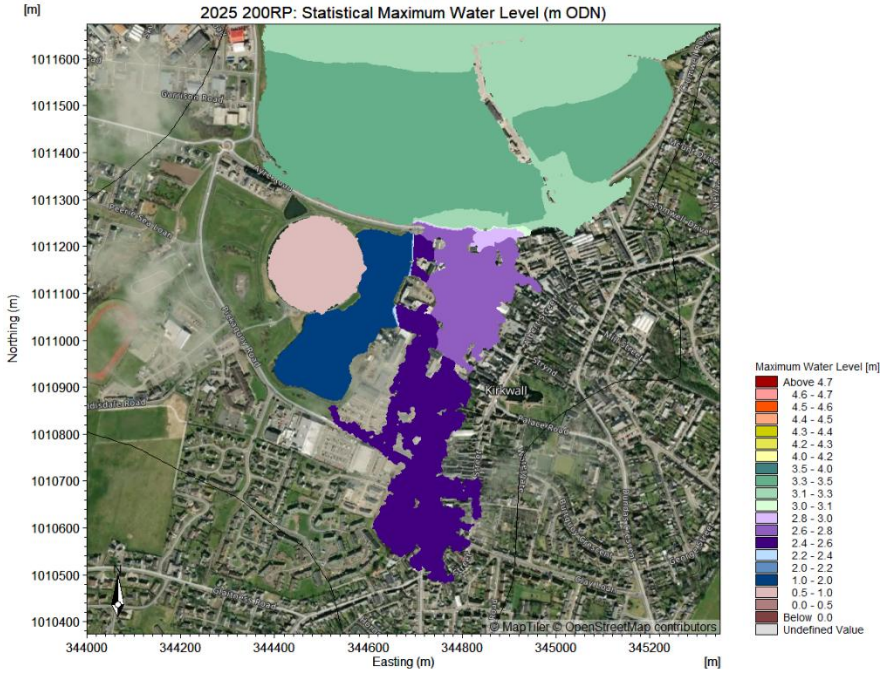
Source: Mott MacDonald 2026

Figure 4.3: Case 1 - Statistical maximum water level (2025, 1-in-200-year RP)



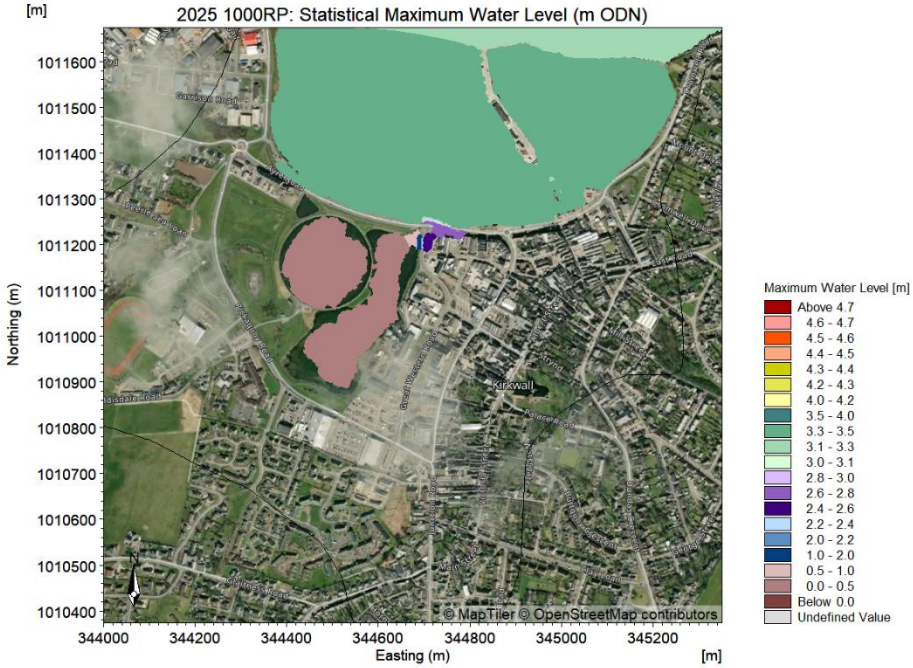
Source: Mott MacDonald 2026

Figure 4.4: Case 2 -Statistical maximum water level (2025, 1-in-200 year RP)



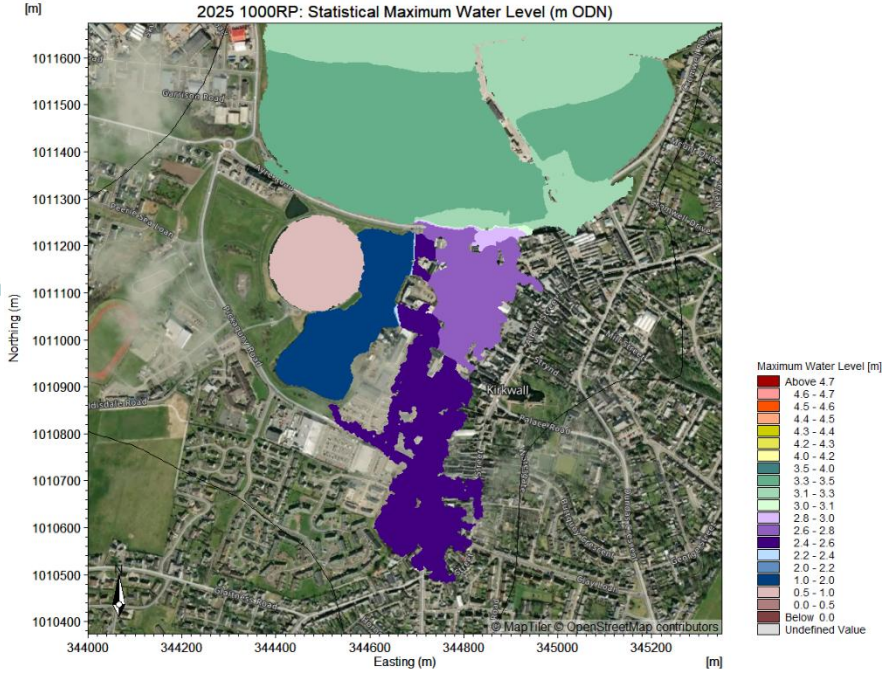
Source: Mott MacDonald 2026

Figure 4.5: Case 1 - Statistical maximum water level (2025, 1-in-1000-year RP)



Source: Mott MacDonald 2026

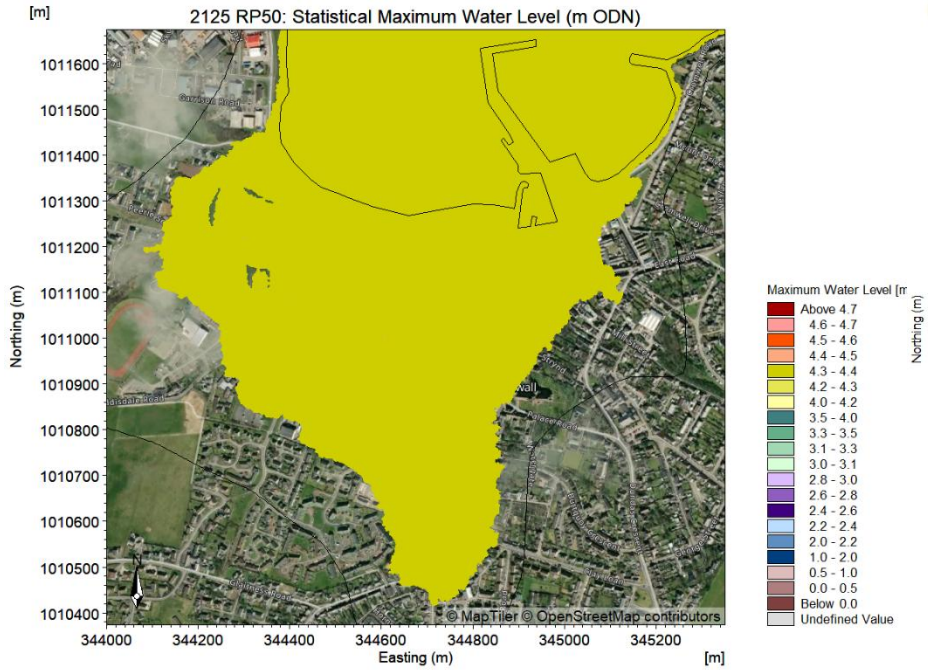
Figure 4.6: Case 2 -Statistical maximum water level (2025, 1-in-1000 year RP)



Source: Mott MacDonald 2026

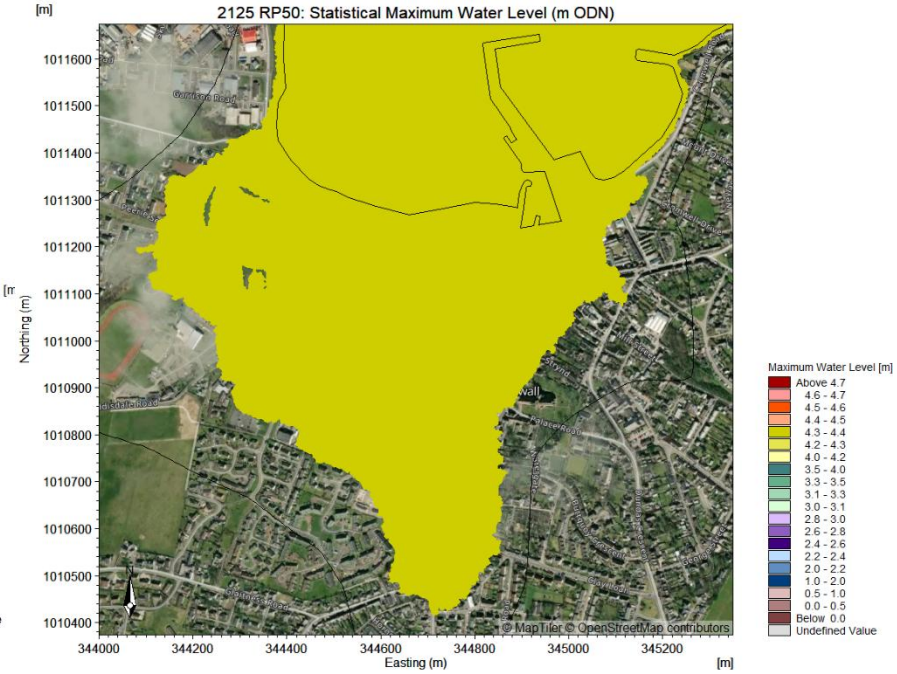
4.2 2125

Figure 4.7: Case 1 - Statistical maximum water level (2125 1-in-50-year RP)



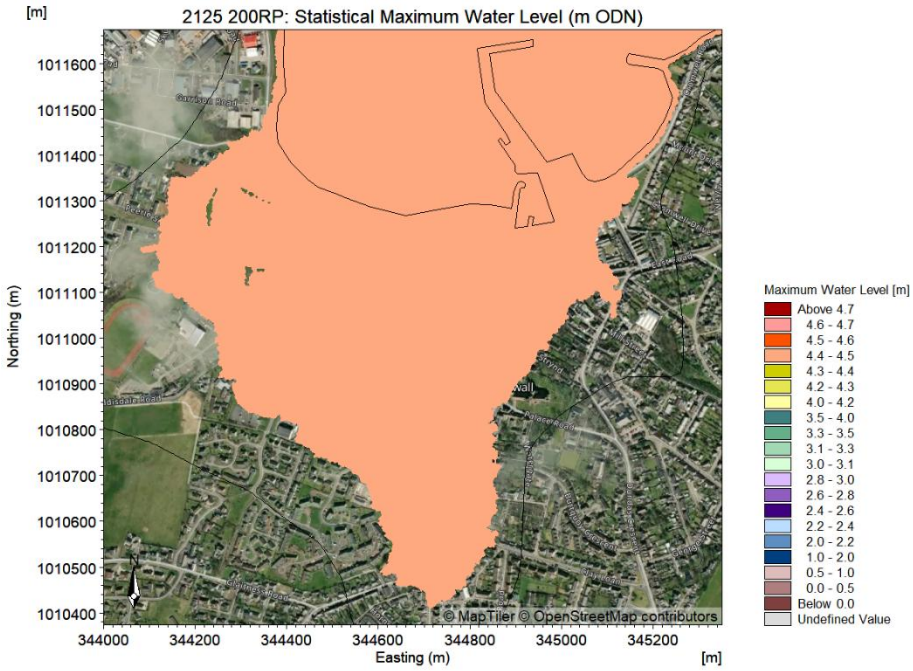
Source: Mott MacDonald 2026

Figure 4.8: Case 2 - Statistical maximum water level (2125, 1-in-50-year RP)



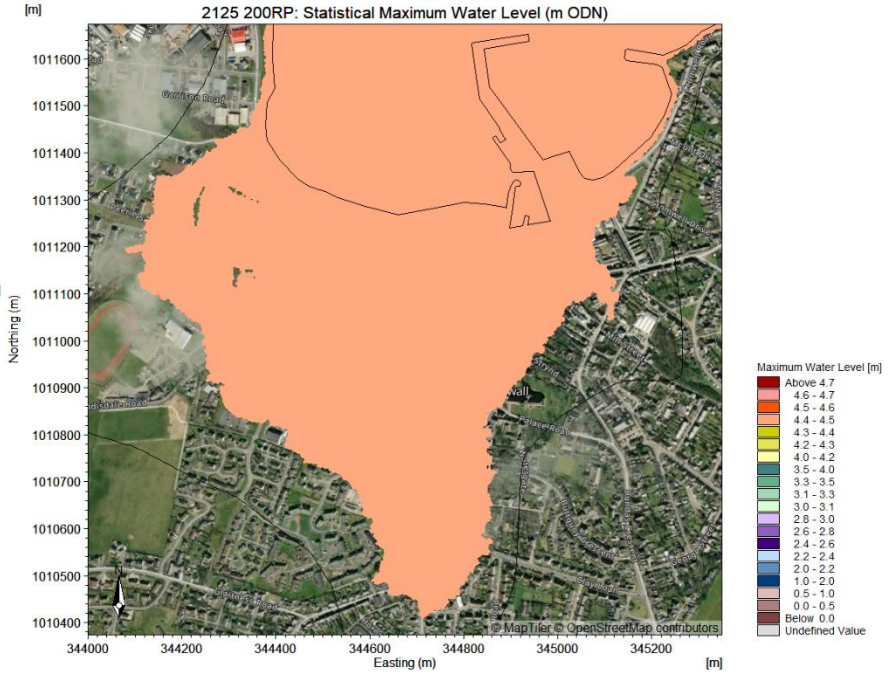
Source: Mott MacDonald 2026

Figure 4.9: Case 1 - Statistical maximum water level (2125, 1-in-200-year RP)



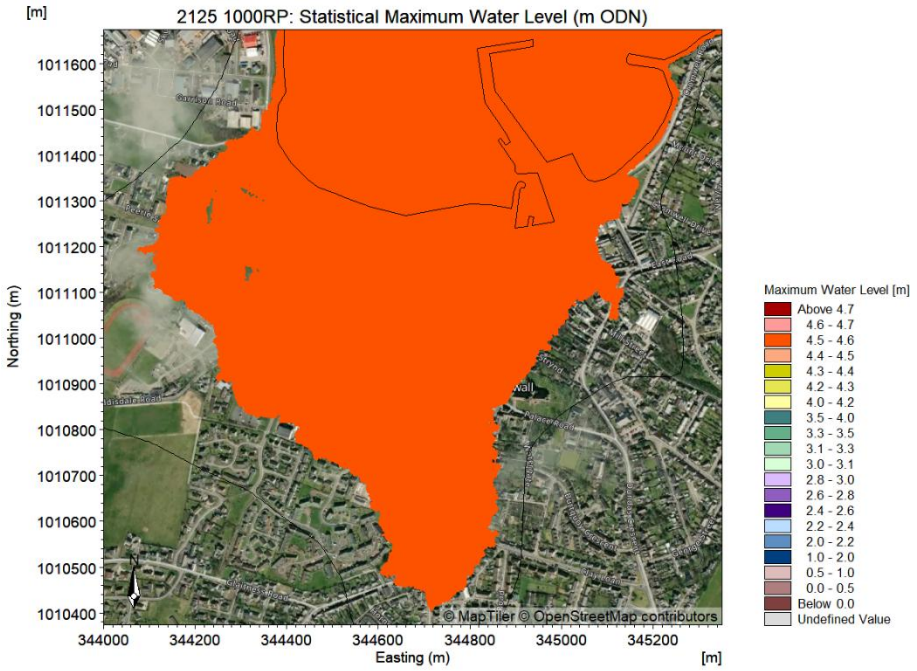
Source: Mott MacDonald 2026

Figure 4.10: Case 2 - Statistical maximum water level (2125, 1-in-200-year RP)



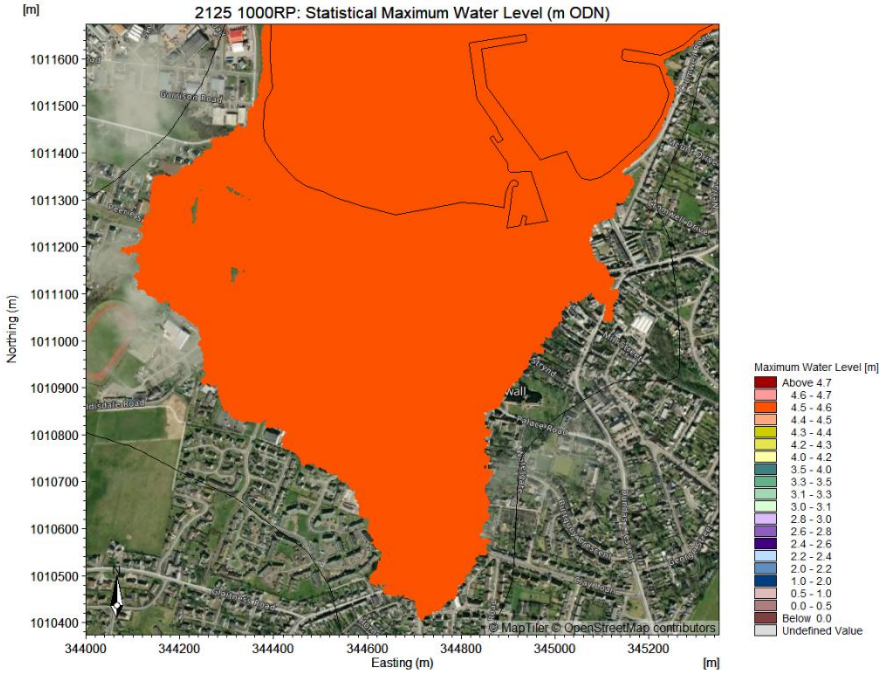
Source: Mott MacDonald 2026

Figure 4.11: Case 1 - Statistical maximum water level (2125, 1-in-1000-year RP)



Source: Mott MacDonald 2026

Figure 4.12: Case 2 - Statistical maximum water level (2125, 1-in-1000 year RP)



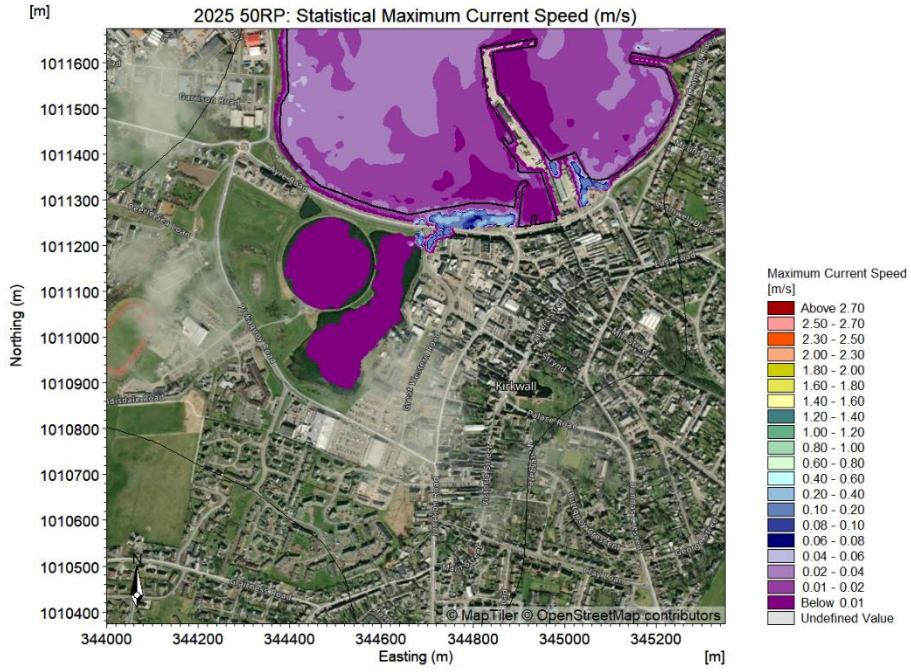
Source: Mott MacDonald 2026

5 Current speed

Figure 5.1 to Figure 5.12 show the statistical maximum current speed results for the 1-in-50, 1-in-200, and 1-in-1,000-year return periods for present-day 2025 and 2125 epochs respectively. The results show that the 2125 epoch simulations produce higher current speeds (over a more widespread area) than the present-day 2025 scenarios.

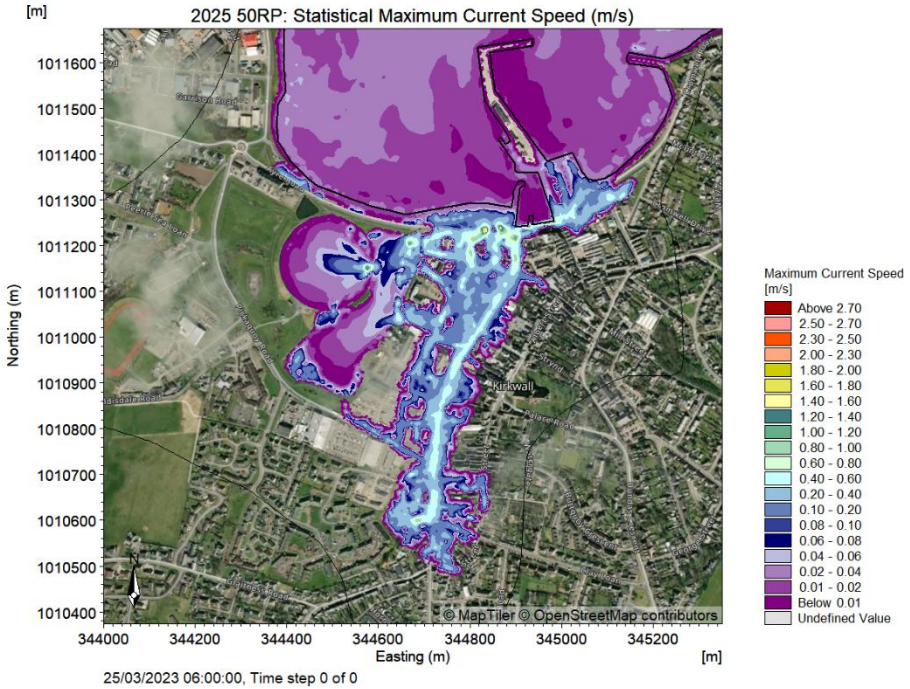
5.1 Present day (2025)

Figure 5.1: Case 1 - Statistical maximum current speed (2025, 1-in-50-year RP)



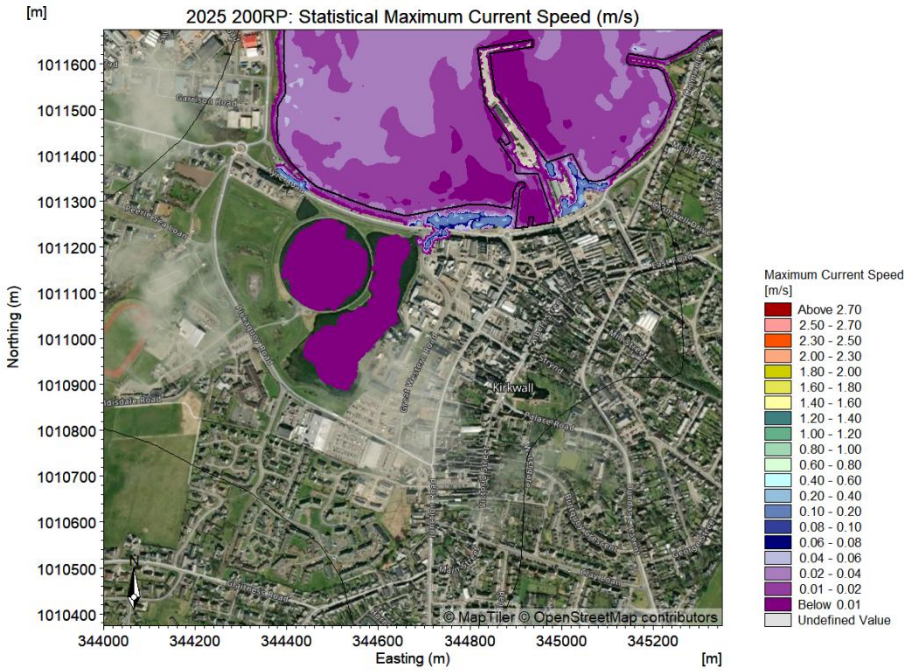
Source: Mott MacDonald 2026

Figure 5.2: Case 2 - Statistical maximum current speed (2025, 1-in-50-year RP)



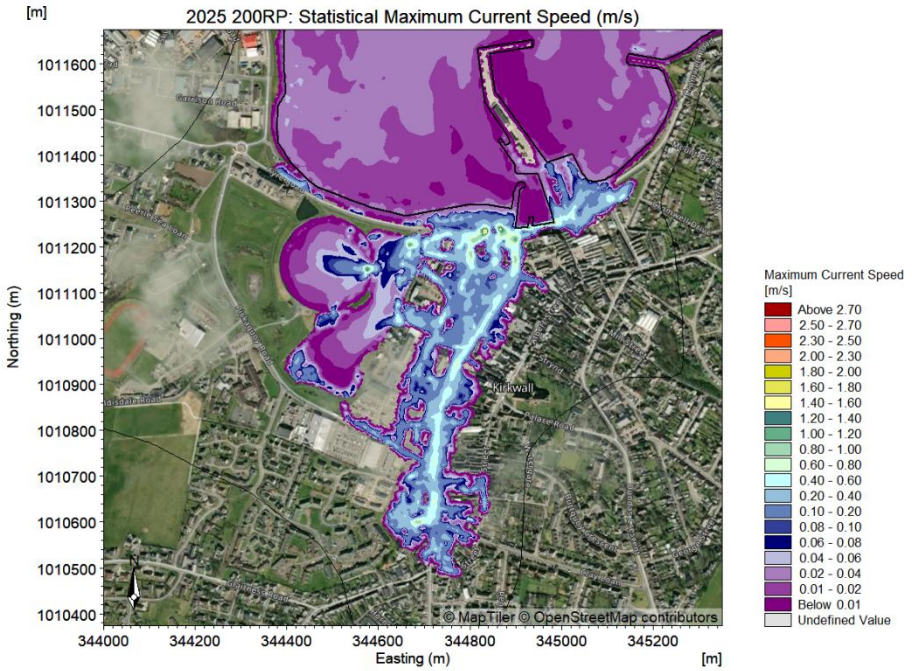
Source: Mott MacDonald 2026

Figure 5.3: Case 1 - Statistical maximum current speed (2025, 1-in-200-year RP)



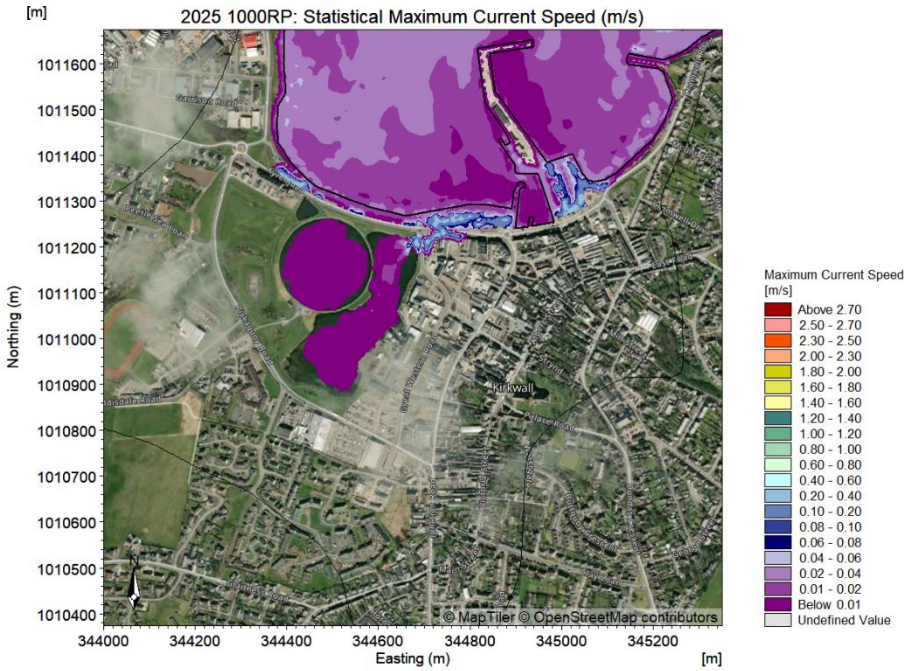
Source: Mott MacDonald 2026

Figure 5.4: Case 2 - Statistical maximum current speed (2025, 1-in-200-year RP)



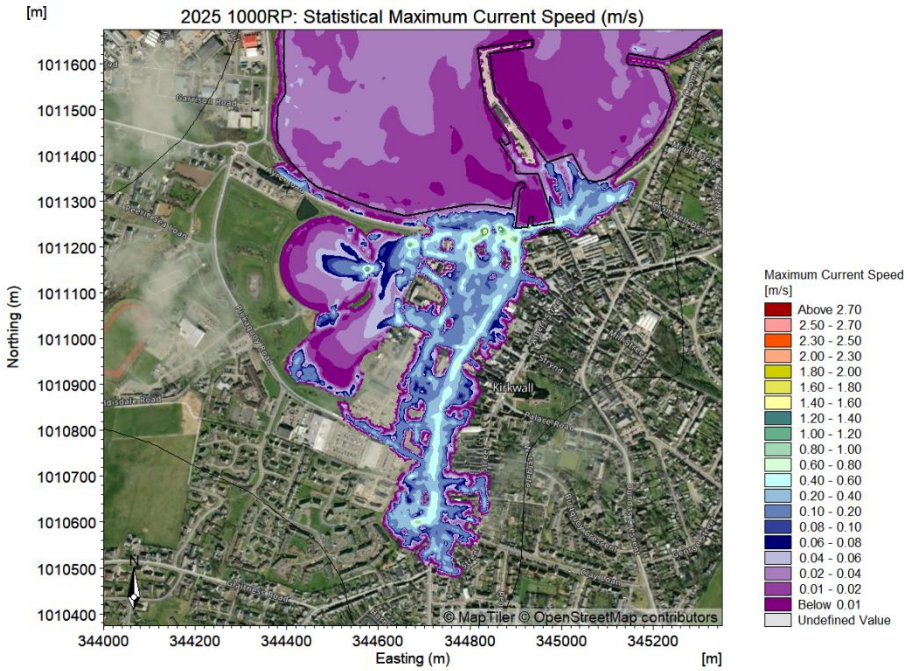
Source: Mott MacDonald 2026

Figure 5.5: Case 1 - Statistical maximum current speed (2025, 1-in-1000-year RP)



Source: Mott MacDonald 2026

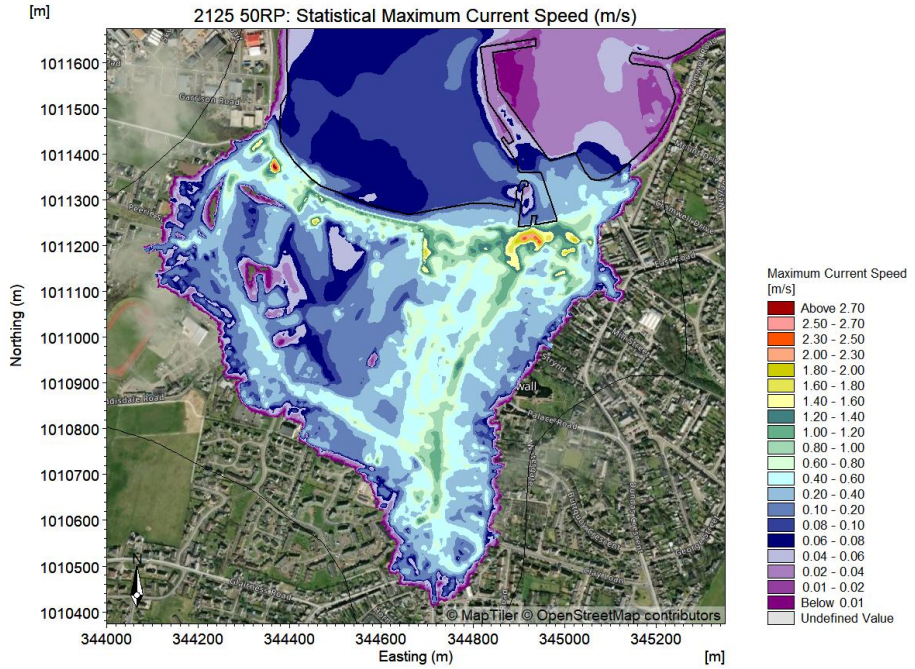
Figure 5.6: Case 2 - Statistical maximum current speed (2025, 1-in-1000-year RP)



Source: Mott MacDonald 2026

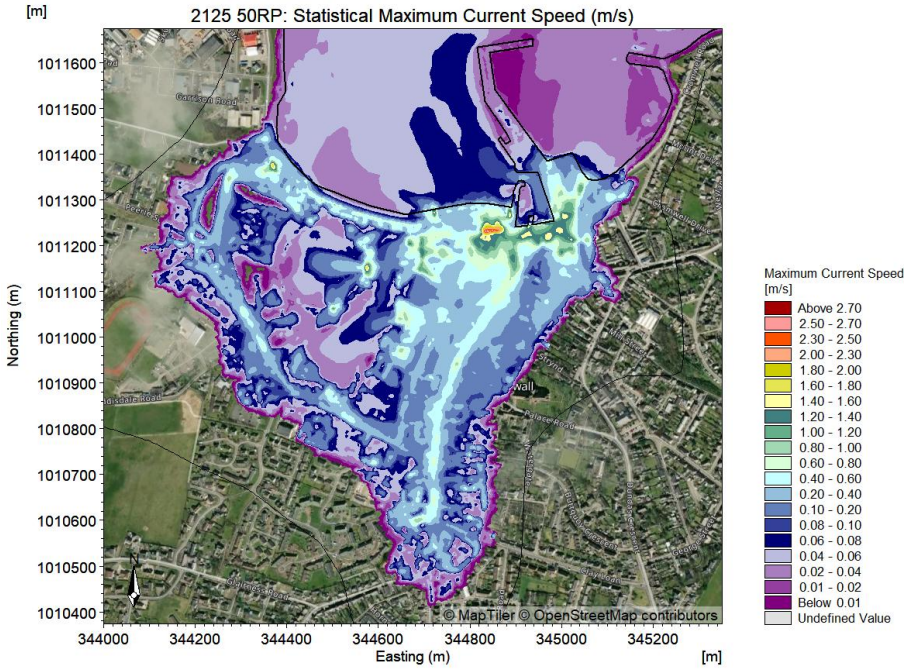
5.2 2125

Figure 5.7: Case 1 - Statistical maximum current speed (2125, 1-in-50-year RP)



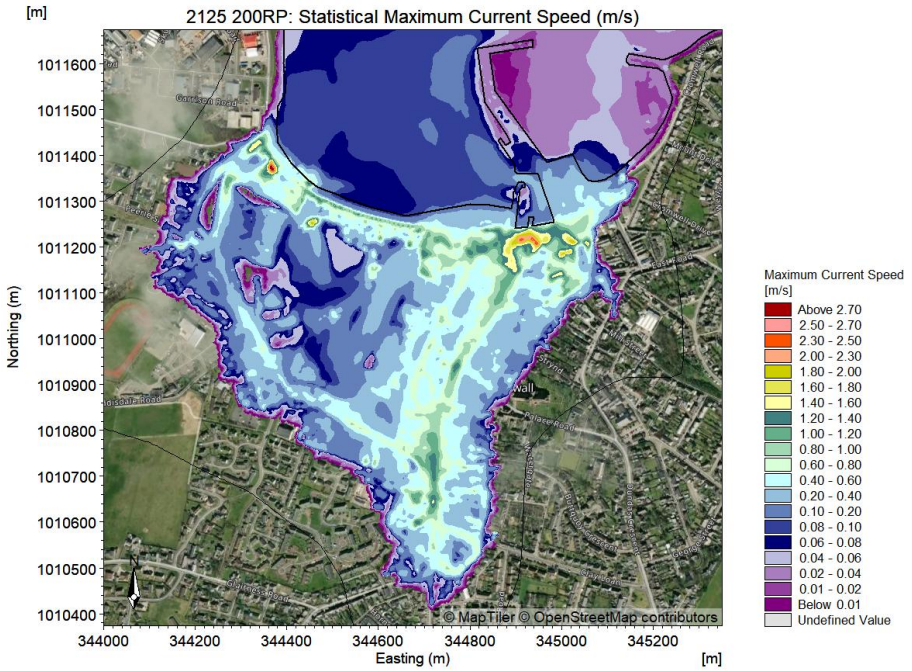
Source: Mott MacDonald 2026

Figure 5.8: Case 2 - Statistical maximum current speed (2125, 1-in-50-year RP)



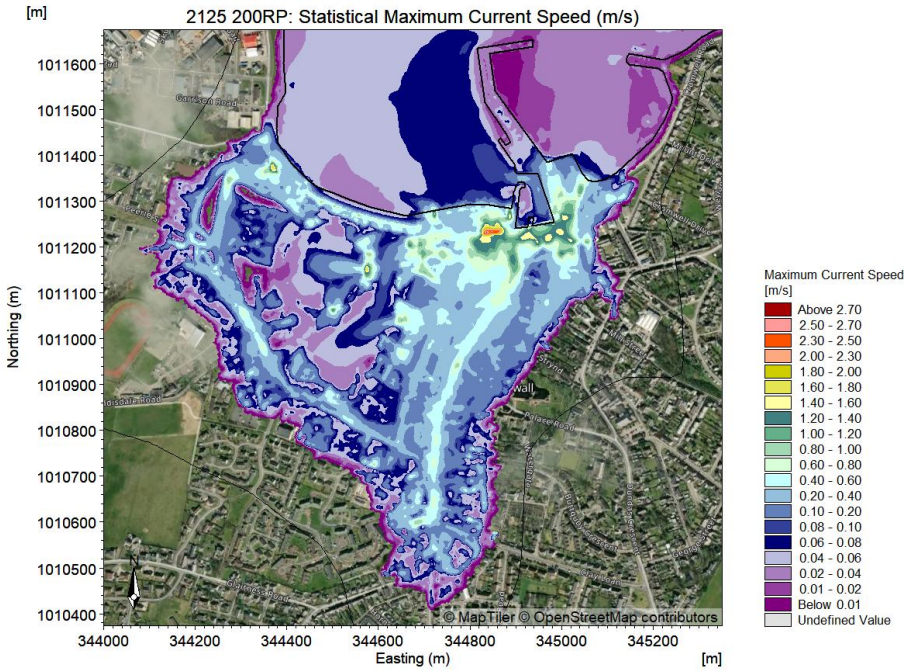
Source: Mott MacDonald 2026

Figure 5.9: Case 1 - Statistical maximum current speed (2125, 1-in-200-year RP)



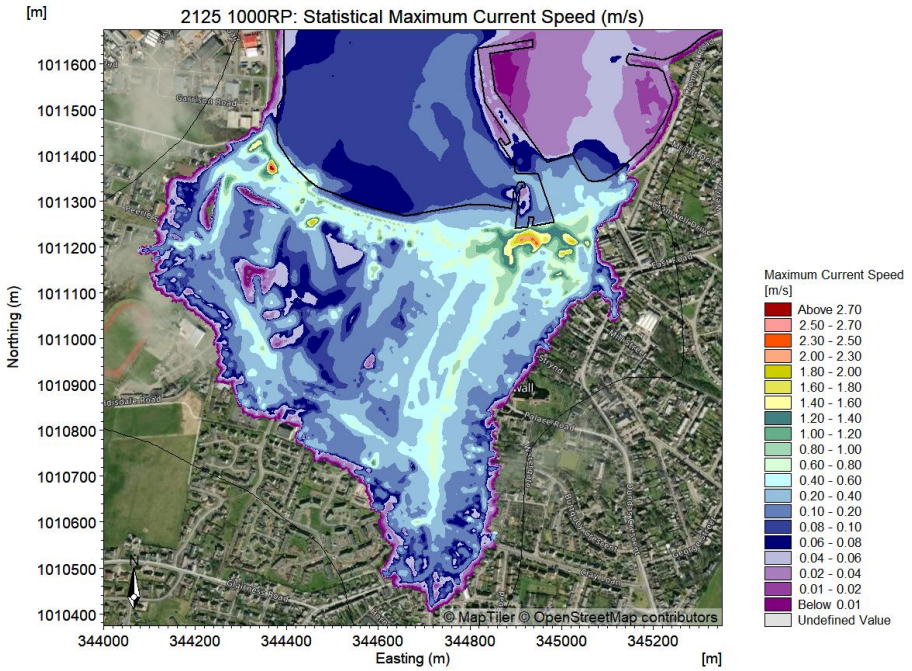
Source: Mott MacDonald 2026

Figure 5.10: Case 2 - Statistical maximum current speed (2125, 1-in-200-year RP)



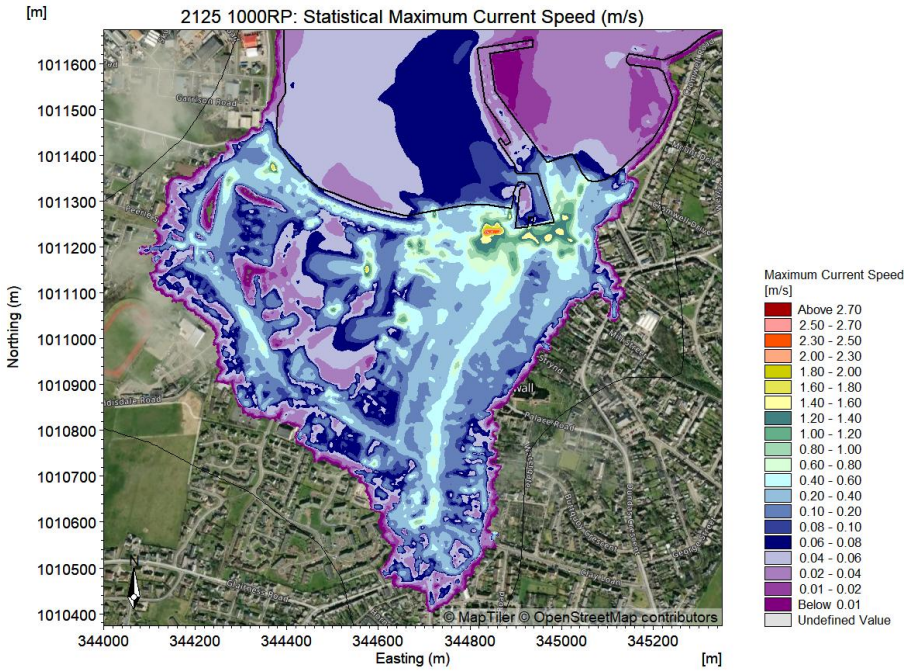
Source: Mott MacDonald 2026

Figure 5.11: Case 1 - Statistical maximum current speed (2125, 1-in-1000-year RP)



Source: Mott MacDonald 2026

Figure 5.12: Case 2 - Statistical maximum current speed (2125, 1-in-1000-year RP)



Source: Mott MacDonald 2026

