Conwy County Borough Council

Penrhyn Bay Coastal Defence Improvements

Coastal Processes Impact Assessment

282129-ARP-XX-RP-MAR-001

P02 | 24 October 2021

This report takes into account the particular instructions and requirements of our client.

It is not intended for and should not be relied upon by any third party and no responsibility is undertaken to any third party.

Job number 282129

Ove Arup & Partners Ltd
8 Fitzroy Street
London
W1T 4BJ
United Kingdom
www.arup.com

ARUP

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1 Introduction

Arup submitted a formal Environmental Impact Assessment (EIA) screening request for Penrhyn Bay Coastal Scheme proposal on behalf of Conwy County Borough Council (CCBC) in December 2020.

NRW has issued a Screening Opinion under the Marine Works (Environmental Impact Assessment) Regulations 2007 (as amended) ref SC2007 on 29th March 2021. In Section 4.3, NRW TE provided several comments reproduced below and this Report aims at addressing them.

4.3 Coastal Processes

NRW TE provide the following comments relating to coastal processes:

Comment 1.
There is no discussion on the impacts that the new structure will have on the hydrodynamics at the site. It does though remark that the coastal processes are primarily now controlled by the two groynes at northern and southern end which already affects the natural variability of the coast. I.e. interruption of long shore sediment transport from the north. And an accelerated movement of sediment alongshore towards the south where the sediment tends to pile up and bypass the southern groyne. No interpretation has been given on the offshore flow conditions particularly during storms. There is evidence from the beach profiles of an intertidal berm which moves with the seasons at the southern end. There is a transport potential of sediment towards the west but this is much smaller as the wind fetch from the east is limited and the waves are smaller.

We advise that a Coastal Processes Impact Assessment should be carried out to determine the alteration to the current/wave hydrodynamics and the movement of sediment both alongshore and offshore in the presence of the new groyne which will act to split the frontage up into sub cells and will create more offshore driven flows around the structure on both phases of the tide. There will also potentially be significant draw down of sediment during storm events particularly from the North west and north. A more detailed understanding on the alteration to hydrodynamics in the presence of the central groyne will inform beach management planning and the requirement for future beach nourishment. A more detailed understanding on the impact that alteration to hydrodynamics and sediment transport offshore will also inform the assessment on the Sabellaria reef's recovery/rejuvenation following any habitat destruction caused by the construction of the groyne.

Comment 2

The section of coastline is already modified by the presence of groynes and as a result the addition of another groyne will act to modify the flow and wave patterns further. It is not envisaged that the new structure will have a far-reaching impact on protected sites outside the locality of Penhryn Bay.

Comment 3
There is no detail in the screening opinion on where the 68500m^3 of nourished beach material will be sourced from. Any temporary impacts arising from sediment disturbance in the water column arising from these activities should be factored into the coastal processes' assessment.

Comment 4
Alteration of physical processes and sediment transport locally within Penhryn Bay resulting from the proposed works will have a direct impact to the *Sabellaria alveolata* (S7 Habitat of Principle Importance to Wales, Environment Wales Act 2016) and potentially to the WFD coastal water body status and these must be considered.

This Report is subdivided into 8 Sections:

- Project background (Section 2)

- Outline of proposed works (Section 3)
- Supporting numerical modelling studies (Section 4) – addresses NRW TE Comment 1, 2
- Impact on hydrodynamics from the proposed works (Section 5) – addresses NRW TE Comment 1 and 2
- Impact on sediment transport from the proposed works (Section 6) – addresses NRW TE Comment 1, 2 and 3
- Residual risks (Section 7)
- Impact of the proposed works on WFD and Sabellaria Alveolata (Section 8) – addresses NRW TE Comment 1 and 4

2 Project background and scope

Penrhyn Bay is a coastal embayment in the county of Conwy, North Wales. The Shoreline Management Plan states that the policy for Penrhyn Bay is to 'Hold the Line'; whilst the Conwy Local Development Plan (LDP) emphasises the importance of preserving the county's unique natural beauty, safeguarding public rights of way and allowing tourism to continue to make a vital contribution to the economy.

The existing defences provide coastal erosion and tidal flooding protection to over 300 residential and commercial properties, utilities infrastructure (including a Dŵr Cymru Welsh Water (DCWW) pumping station, a Wastewater Treatment Works (WwTW) sea outfall and SP Energy Networks assets), the adjacent promenade, A547 highway, Wales Coast Path, National Cycle Route, and the Rhos-on-Sea Golf Club. The present beach and promenade also form a key part of the local community's well-being and recreation space, whilst adding to the area's wider tourist attractions.

In the absence of the continuing beach management works, regularly undertaken by CCBC, the sea wall and revetment are highly likely to fail through the on-going drawdown of the foreshore exposing the corroded and structurally weak sheet piled toe foundations.

The present regime of maintenance activities is unsustainable and is likely to increase in frequency over time, as the mobility of the beach increases due to the beach material particle attrition and the increasing severity of storm events and sea level rise associated with climate change.

The Objectives for the project are to:

1. Manage the effects of flood risk and coastal erosion to people and property.

2. Reduce the on-going capital maintenance costs experienced as part of the present beach maintenance regime to safeguard the public backshore assets.

3. Meet the objectives of the Shoreline Management Plan (SMP2) for Penrhyn Bay, which include:

    • Hold the Line

    • Maintaining and widening the beach.

    • Acting in an environmentally acceptable way.

4. Incorporate wider benefits and added amenity value into project design and construction (including consideration of Welsh Government's Community Benefits Policy and the Well-Being of Future Generations (Wales) Act 2015). To maintain and, where possible, enhance the natural environment.

5. Maintain and improve the local area – with an awareness of the amenity value presently provided by Colwyn Bay and Llandudno (neighbouring towns).

3 Proposed Works

To achieve the above objectives, a T-shape rock groyne and associated beach nourishment are proposed.

The rock groyne is approx. 100m long by 90m in width. The rock groyne is suited to the local area, which already comprises a number of rock groynes and the beach nourishment will comprise similar material to existing beach deposits.

The plan shape of the capital beach nourishment was determined from numerical modelling as the equilibrium position resulting from the incident wave direction and wave diffraction behind the groyne structure.

The minimum beach width is 10m, the crest level is between +5mODN and +6mODN, to be confirmed, and the beach nourishment slope is proposed at (V:H) 1:8. These design parameters are discussed further in the following sections of the report.

Figure 1 shows a plan of the proposed works in Penrhyn Bay at a high tide (left) and outline at low tide (right).



Figure 1 – Plan of proposed works

4 Supporting numerical modelling studies

Significant work has been undertaken by Arup and sub-consultant ABPmer since 2017 to study different options and understand the coastal processes. The outcome of these studies is documented in the following reports (also provided attached).

[1] Penrhyn Bay Coastal Defence Improvements, Baseline review and option development, ABPmer Report No. R2776. A report produced by ABPmer for Ove Arup and Partners Limited, April 2017 (referred herein R2776); see Appendix A

[2] Design Development Modelling, Penrhyn Bay, ABPmer Report No. R3471 Revision 2. A report produced by ABPmer for Ove Arup and Partners Limited, September 2020 (referred herein R3471); see Appendix B

[3] Design Development Modelling, Beach Management Strategy, ABPmer Report No. R3504 Revision 3. A report produced by ABPmer for Ove Arup and Partners Limited, October 2020 (referred herein R3504); see Appendix C

The numerical modelling study carried out at the OBC stage in 2017 supported the coastal processes assessment and the examination of the performance of alternative options (i.e. equilibrium bay shape using MepBay) and beach profile variation (using Shingle-B).

In 2020 additional numerical modelling of the preferred OBC option (T-shape rock groyne + beach nourishment) focused on the hydrodynamics and coastal processes assessment to support the design development and predict the scheme performance. Alternative scheme layouts were examined using Mike21 software from DHI, widely known and used in industry.

Based on the numerical modelling, a Beach Management Strategy was produced to set out the design criteria for the beach focusing on beach width and potential trigger points for intervention.

For ease of reference, a summary of the coastal processes that addresses NRW's comments from the EIA Screening Opinion is presented below.

5 Impact of proposed works on hydrodynamics

5.1 Offshore to nearshore wave conditions

ABPmer set up a regional wave hindcast model where the domain was based on the bathymetry, prevailing wind and wave directions (Figure 2) and any bathymetric features that could potentially influence the local flow and wave climate.

Waves in this area of North Wales are predominantly coming from NW to N sectors. NE sector is of interest for the scheme but not prevailing. A typical model output for one of the 1-in-1-year wave events (the NNW event) is shown in Figure 3. This figure shows the incident wave heights and scaled wave vectors representing the incident wave direction.

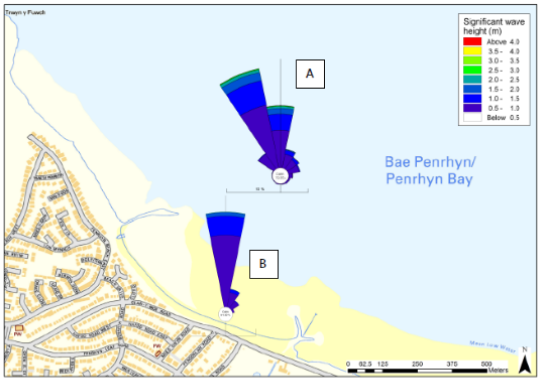


Figure 6. Nearshore (A) and inshore (B) wave roses using a combination of observed water levels and offshore SEASTATES hindcast

Figure 2 – Wave roses offshore based on SEASTATES hindcast (Figure 6 extracted by R3504)

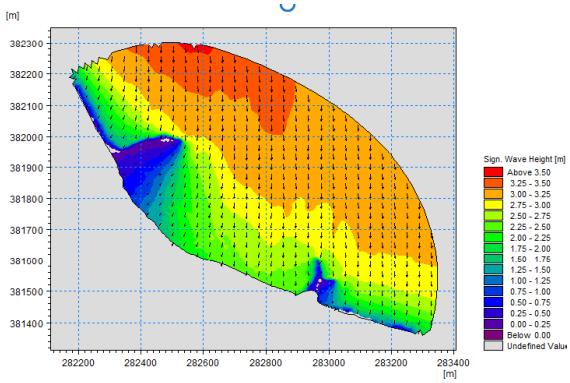


Figure 4. Example wave vectors from 1-in-1-year NNW wave event, under the present-day scenario with deepened nearshore bathymetry

Figure 3 – 1:1yrs NNW wave event (Figure 4 extracted by R3471)

To derive extreme design wave and water level conditions for the proposed structure, offshore hindcasts of waves and of tide and surge water levels, have been subject to a Joint Probability Analysis (JPA). Key Joint Probability (JP) pairs have then been transformed through numerical modelling inshore and wave conditions have been extracted at the toe of the proposed structure. The derived Sea Level Rise values from the Welsh Government (2017) guidance were used to uplift the water levels in JPA for the future years. Detailed results are provided in Appendix M of R3471.

A local MIKE21 Spectral Wave (SW) numerical model was run for each of the JP wave/ water level pairs and transferred into Penrhyn Bay. The model included the present-day bathymetry nearshore within the bay (no additional structures have been included).

The nearshore wave transformation results were processed in QGIS to show local wave crest orientations, which were then assessed to derive the predicted beach crest positions (see Figure 4).



Figure 5. Wave crests representative of a 1-in-1-year wave event from WNW under the present-day scenario



Figure 6. Wave crests representative of a 1-in-1-year wave event from NNW under the present-day scenario

Figure 4 – Wave processes under the existing scenario (Figures 5 and 6 extracted from R3471)

From Figure 5 to Figure 8 of R3471, the beach shape estimated from the wave crests are shown to replicate well the observed shoreline. The variability of the estimated beach crest positions also aligns closely to the modelling MepBay results (carried out previously in 2017). Therefore, it is considered that the numerical model is validated for the assessment of the coastal processes and the prediction of the expected beach shapes for the proposed scheme.

Wave data were extracted for all JPA wave and water level pairs at five specific points located within Penrhyn Bay, along the general line of the proposed structure. Detailed results are provided in Appendix M of R3471.

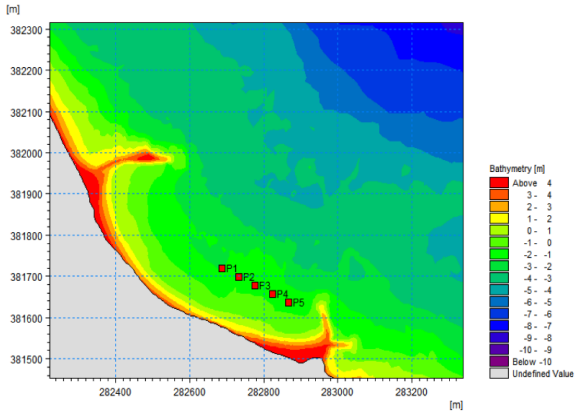


Figure 21. Nearshore data extraction locations

Hs is shown to increase from epoch to epoch, which is as expected given the increase in climate change allowances as the epochs progress. Additionally, both direction sectors (WNW and N) show all five extraction locations to have similar values of Hs within each run scenario, with wave height differences ranging by 0.1 to 0.2 m between extraction locations. Both direction sectors also show that the central extraction locations (P2–P4) generally have slightly larger wave height values than P1 and P5.

Figure 5 - Nearshore data locations (Figure 21 extracted from R3471)

5.2 Wave and current modelling results

The impact of the proposed scheme on the local current regime is predicted through numerical modelling. The approach, setup and results of the coupled wave and current modelling are described in Chapter 5 of R3471.

The hydrodynamic model was used to simulate storm conditions with high recurrence interval (1 in 1 year) and low recurrence intervals (1 in 200 years) and different directions offshore.

- Present-day tide-only condition (mean spring tide);
- Present-day 1 in 200-year joint condition with waves from WNW;
- Present-day 1 in 200-year joint condition with waves from NNE; and
- Present-day 1 in 1-year joint condition with waves from WNW.

A full model results and output plots from the coupled wave/current modelling is provided in R3471, Chapter 5.2 and Appendix N, for each of the directional sectors assessed.

Figure 6 compares the current speeds from combined wave and tide within the bay under the baseline (existing) scenario and under a 1 in 200-year wave event from WNW.

Key findings are extensively discussed in R3471 Chapter 5.2.2 and summarized below:

- Currents within the bay are dominated by wave driven flows, when there are moderate to large waves.
- Under the baseline conditions (without scheme) both the WNW and NNE waves develop and drive a longshore current that is directed out and around the western groyne which then crosses the bay and then passes across the end of the fishtail groyne to the east. The redirection of the flow is also seen to develop a counter current along the beach in the western half of the bay.

- With the presence of the 'T' groyne, flow is pushed offshore thus increasing the flow speed directly off the 'T' groyne. Some wave breaking on the 'T' groyne also induces locally stronger flow directly adjacent to the structure.
- A negligible difference in current speed is observed across the outer extent of the western groyne.
- Wave breaking on the structure result in a small onshore directed flow. This is beneficial as the modelling is not predicting currents which would cause the nourished material to be directed offshore and lost.
- No significant impacts are expected on the adjacent coast because there is a rock revetment protecting the coastline to the East of Penrhyn Bay and the bay to the West will not be impacted due to the predominant easterly current circulation in this area of the Irish Sea.

Figure 7 compares the current speeds from wave and tide within the bay under the baseline (existing) scenario and under a 1 in 1-year wave event from WNW.

Key findings are extensively discussed in R3471 Chapter 5.2.2 and summarized below:

- The longshore wave driven current is directed across the bay in a west to east direction. The 'T' groyne forces the flow to be more offshore and stronger.
- There is negligible impact on the peak flow speed observed over the Dŵr Cymru Welsh Water outfalls
- The results from the baseline conditions also show a focused west – east current responsible for sediment transport. Under the proposed scheme, the flows in the western embayment are now weaker with more localised circulation patterns, as they are more aligned to the incident waves.
- A small eddy is also observed directly behind the western arm of 'T' groyne, with offshore directed flow close to the structure. The strength of this circulation pattern is considered typical to what could be expected on the open coast.

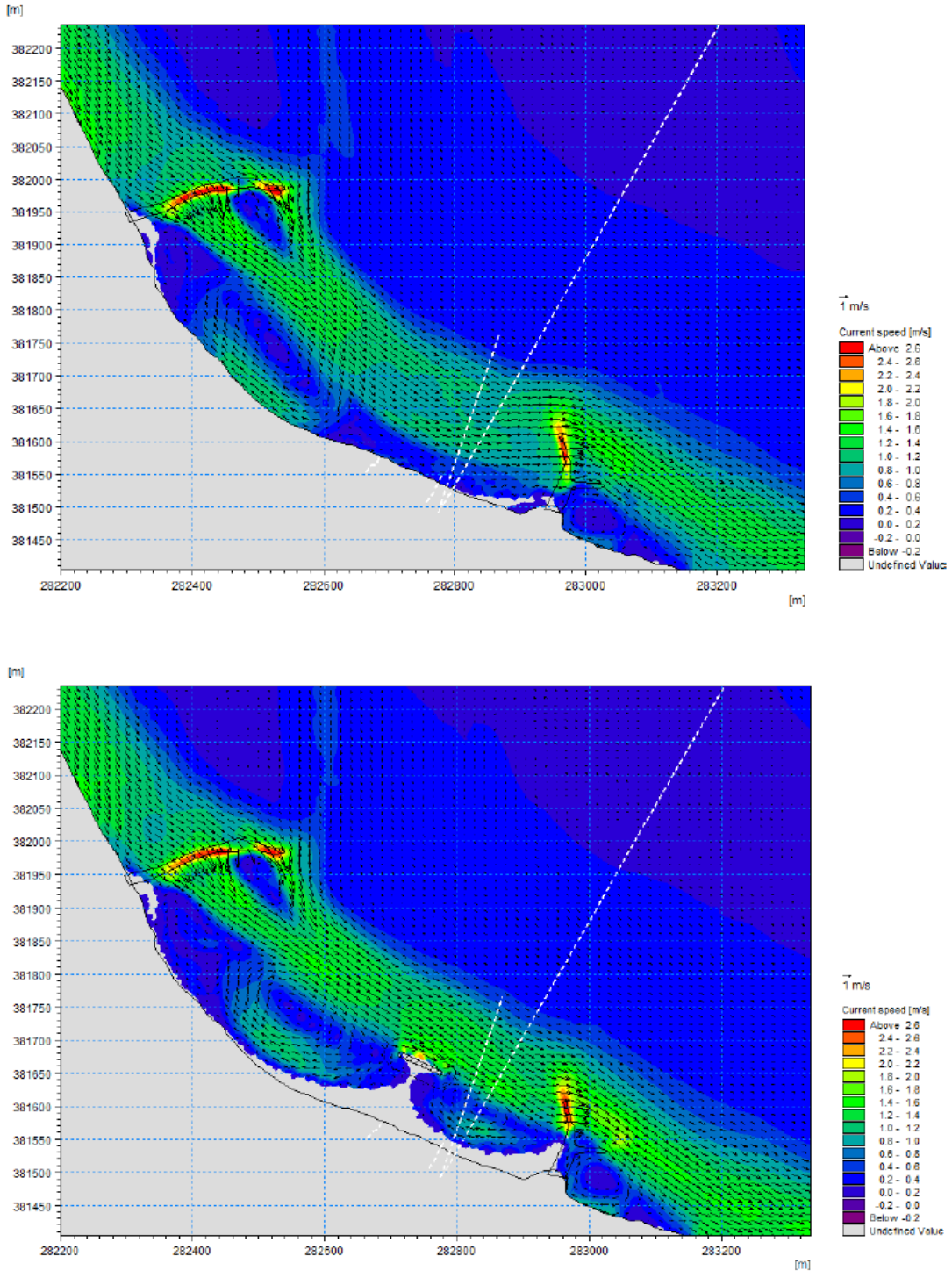


Figure 24. **Peak combined WNW 1 in 200-year wave and tide current flow: Baseline (top)and Scheme (bottom)**

Figure 6 – Wave and tide current flow example for WNW 1:200 yrs (Figure 24 extracted from R3471)

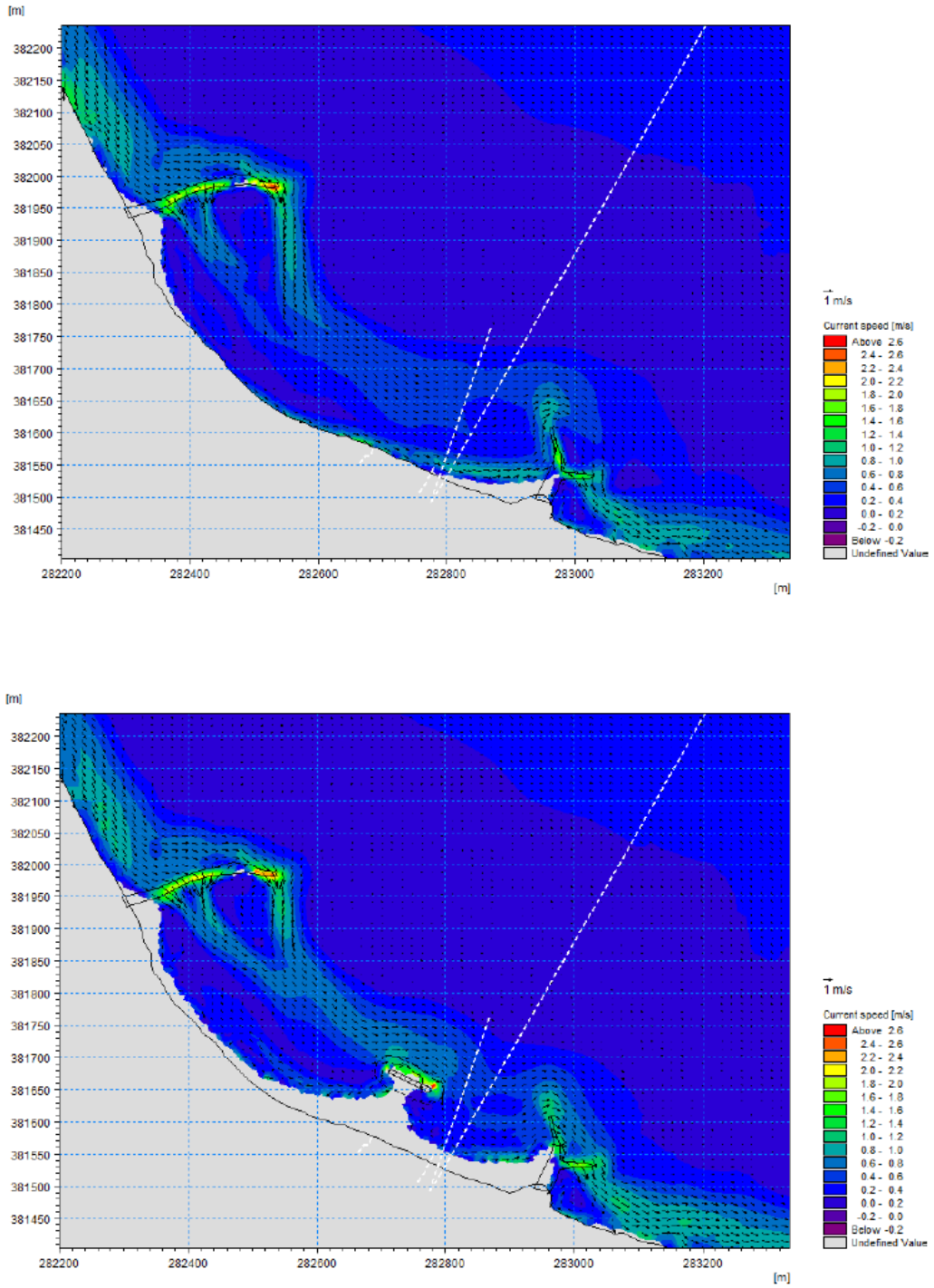


Figure 30. **Peak combined 1 in 1-year wave and tide current flow: Baseline (top)and Scheme (bottom)**

Figure 7 – Wave and tide current flow example for WNW 1:1 yrs (Figure 30 extracted from R3471)

6 Impact of proposed works on sediment transport

6.1 Baseline sediment transport processes

The baseline coastal processes are presented in the simple scheme of Figure 8 and described below.

Predominant offshore waves approach the beach from NW to N sectors after diffraction around the west groyne and refraction. Waves from the NE are less frequent and be subject to less refraction as they are transformed towards the shore.

Under the existing coastal situation beach material is transported by the wave and current action away from the centre of the bay to the shelter of the groyne, mainly the eastern groyne. Ongoing attrition and abrasion of the material results in a reduction of material size and therefore it becomes more mobile.

As a result of significant volume accumulation, a small amount of material currently passes over the east groyne crest and is lost from the bay. However, historically, the overall volume of the upper beach has remained relatively stable.



Figure 8 – Schematic description of baseline coastal processes

6.2 Projected sediment transport processes

The presence of the new 'T' shaped groyne within the bay results in localised wave diffraction around the ends of the 'T' arms, which in turn adjust the natural alignment of the beach in the lee of the structure, i.e. the diffraction determines the shape of the beach crest.

The new 'T' shaped groyne has been designed to hold a nourished upper beach within the central section of Penrhyn Bay, thus providing protection to the sea wall and additional amenity benefit.

Beach nourishment will provide for a beach width that allows for natural change and storm response with expected minimal maintenance over the years. The lower sediment transport rates will decrease the attrition and abrasion which is the main cause of the reduction in material size which makes it more mobile. Details of the nourishment scheme are provided in R3405 and summarized below.

The beach will be initially nourished using the envelope of the beach crest alignments derived from the modelling of different wave directions. A buffer width of 10 m is adopted which is close to the upper beach crest width of 8 m that has been built recently when reinstating the beach as part of the current maintenance programme. The resultant plan-shapes for the three main incident wave directions are shown in Figure 9.

The proposed beach crest level is between +5mODN and +6mODN, to be confirmed, based on the crest level from previous nourishment schemes of +4.9mODN and the natural berm level at the eastern end of the bay of approx. +5.8mODN due to the accumulation of material over time. The beach berm will cover and protect most of the stepped revetment section.

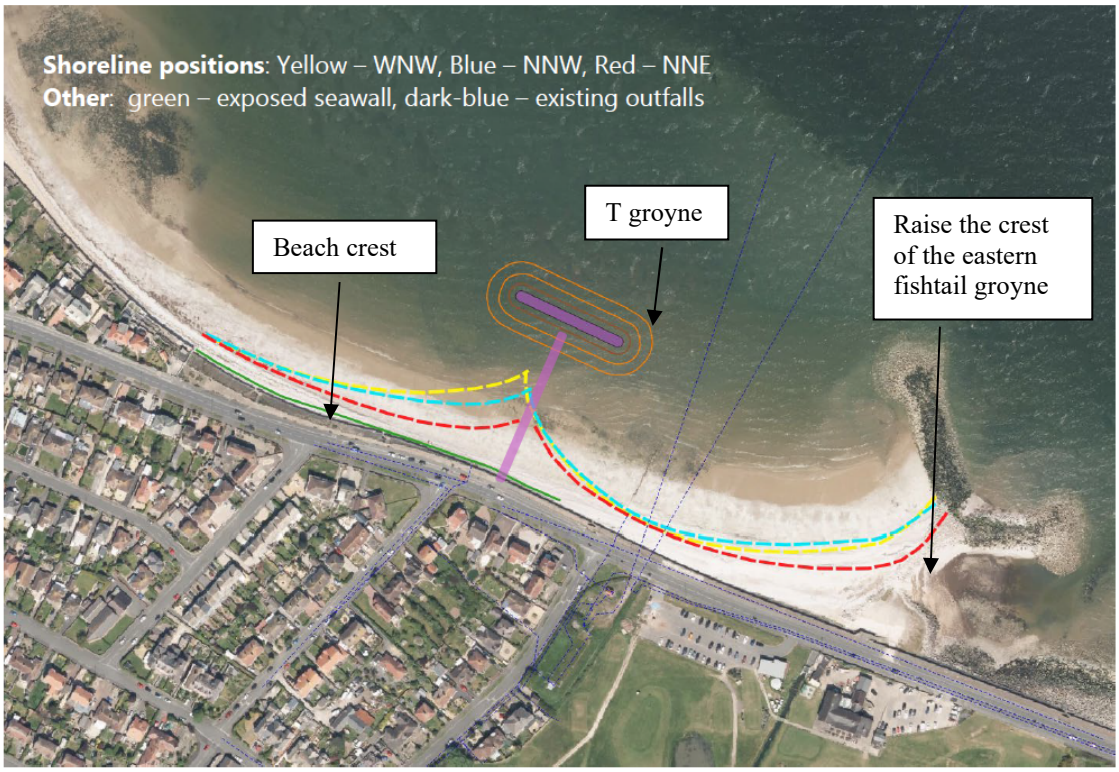


Figure 10. 'T' shaped groyne and beach crest positions with 10 m minimum buffer width

Figure 9 – 'T' shaped groyne and beach (Figure 10 extracted from R3504)

The beach crest positions from Figure 9 show that:

- A tombolo is predicted to form with the beach crests attaching to the shore connecting structure, but the beach crest is not expected to attach to the shore parallel part of the groyne, thus a fully developed salient is not expected.
- With the nourished beach having a typical slope of 1 in 8, the nourished material should generally be contained landward of the shore parallel component of the 'T' groyne, thus minimising the likelihood of being lost from the bay through longshore sediment transport.
- The beach crest alignments for the WNW (yellow) and NNW (blue) are close due to the wave transformation from offshore to nearshore which rotates them to almost the same incident direction. Large waves from the NNE are much less common and they are less likely to persist for period required to fully realign the beach to the position in red. Therefore, it is expected that the beach crest will be close to the yellow/blue most of the time.
- Any seaward migration of the beach crest at this location could also lead to material being transported around the fishtail groyne. The present schemes show that the location of the beach crest is similar to that presently observed.

The 'T' groyne connects the shore parallel component of the structure to the land, so that there will be limited natural transfer of material from the western embayment to the eastern embayment and *vice versa*.

The proposed works also include raising locally the level of the eastern fishtail groyne to help limit the loss of upper beach material from the bay.

6.3 Trigger level for intervention

Under very large events the cross-shore profile can be expected to vary, and the beach berm will be pushed back towards the sea wall, and adjustment to the plan shape will occur as the beach responds to the different incident wave conditions.

To manage this a trigger condition needs to be set to identify when the beach levels are such that their performance in protecting the seawall are compromised and therefore intervention works should be undertaken to reinstate the beach.

Overtime and with longer-term monitoring, it will be possible to set this trigger level with a greater understanding of the conditions at the site with the scheme in place, but initially it is suggested the trigger level should be defined as a minimum 2 m beach crest width, with a level above the initial nourishment level of between +5mODN and +6mODN.

The rationale for this criteria is discussed in Section 3.5 of R3504.

6.4 Material sourcing and construction impact

Rock armour (including all the underlayers and core) to form the T-shape groyne will be selected based on the following criteria:

- Size and properties to provide long term stability with acceptable damage such that low maintenance is required
- Proximity of source to reduce carbon footprint from transportation

Beach nourishment material will be selected based on the following criteria:

- Similar size to what is observed in the upper beach in the eastern side of the bay
- Size and properties to provide long term stability of beach with low degradation over time and short- and long-term amenity for beach users Angular stone will become rounder over time due to abrasion and weathering, however the amenity value of angular stone is low, therefore rounded material is preferred. High density and high abrasion resistance are preferred.
- Proximity of source to reduce carbon footprint from transportation. We are aware that Conwy Council has sourced adequate beach nourishment material from a nearby quarry. Further details will be communicated to NRW as soon as available.

During construction there is a risk that the imported material for the core of the rock groyne and the beach nourishment may cause turbidity in the water column. However, the risk is minimized because:

- At all low tides (2x per day over 24 hours) the whole footprint of proposed works is above the water line therefore the need to place material underwater will be reduced, subject to Contractor methodology.
- All material imported to site will be washed before placement.
- Material for both rock groyne and beach nourishment consists of mainly large particles, therefore suspension and dispersion will be reduced comparing to fine sediments.
- Any impacts will be localized in space and in time.

Furthermore, before construction commences, the Contractor will be required to produce a method statement and risk assessment (including temporary construction measures, if required) to be approved by NRW.

7 Residual risks and uncertainties in hydrodynamics and sediment transport

7.1 Climate change considerations

Climate change over the next 100 years will likely influence the scheme through:

- Increased water levels. Higher water levels cause an increase the natural beach berm level that is shaped under low frequency storms. Since little material is expected to be lost from the bay during storms, the beach is expected to return to its equilibrium shape over time.
- Potential increased wave heights. With increased water levels the waves nearshore may also increase slightly, but the effect of this is expected to be less significant.

7.2 Need for additional numerical and/or physical modelling

It is not possible to predict with certainty beach evolution over time, due to interaction of several non-linear physical processes and stochastic nature of weather conditions.

Additional numerical modelling and physical modelling have been considered as potential alternatives to refine the design further. However, both have been discarded for the following reasons:

- Physical modelling of sediment transport is very complex by nature due to the required downscaling of sediment and the change in properties and characteristics, it requires at least 2-3 months which is not compatible with the timescales of the project and the need to meet the March 2022 deadline to allow Conwy Council to receive funding from Welsh Government.
- Based on historical coastal behaviour and extensive numerical modelling done to date, the coastal processes (hydrodynamics and sediment transport) are well understood. The numerical models were validated and the pragmatic approach to defining beach shape for nourishment is robust. More layouts could be tested, e.g. shifting the structure further west but it increases the beach nourishment volume and associated cost significantly.

7.3 Long term monitoring

Arup has already engaged with the Wales Coastal Monitoring Centre to recommend changes to the monitoring programme to provide more useful information for the management of the bay.

Results from the surveys over the next years will allow a greater understanding of how the implemented scheme is behaving and will inform any changes to the Beach Management Strategy document.

Although the maintenance requirements of the rock armour forming the ‘T’ groyne depend on the rock sizing (i.e. the larger the rocks, the more stable the structure), the overall functionality and efficiency in promoting wave diffraction and retaining sediment transport should not be significantly affected by minor damage and dislodgment of a small percentage of rocks.

The rock groyne and beach nourishment will provide protection to the primary flood defence which is the seawall. However, seawall inspections should continue to be undertaken.

8 Impact of proposed works on WFD and Sabellaria Alveolata

As part of the Detailed Design, the following reports will be produced in due course:

- Non-statutory Environmental Report
- Coastal Water Framework Directive: Screening
- HRA Habitat Regulation Assessment Stage 1: Screening
- Ecological Impact Assessment.

- CEMP Construction environmental management plan

In particular, the impact of the proposed works on WFD coastal water body status will be assessed separately and at a later stage and NRW will be consulted.

An intertidal biotope survey has been carried out in June and the main conclusions were:

- A significant area of Penrhyn Bay in the middle of the bay is colonized by Sabellaria Alveolata which will be impacted directly by the construction activity and will be buried if within the footprint of the rock groyne and beach nourishment. The Sabellaria reef may be impacted indirectly but to a much lower degree by changes in coastal processes because:
    - Changes in current velocities are relatively small
    - The inflow of sediments to the bay from the east is unaffected.
    - Apart from the direct destruction of the reef located within the proposed works footprint, no changes are expected to the surrounding seabed substrate from altered coastal processes that can impact the settlement/colonisation of the Sabellaria.
    - The Sabellaria requires input of sediment to support tube development. No change in input of sediment to the bay is expected due to the works. The Sabellaria area that is not covered by the proposed works may actually benefit from lower shingle mobility which can destroy its tube structures. The higher localized turbulence around the rock groyne can also benefit growth since such conditions can transport suspended particles it requires.
- There are areas of the existing rock groynes with colonisation of Sabellaria Alveolata.
- A shift in the rock groyne west up to 30-40m was considered to reduce the direct impact on the central area of Sabellaria, however the beach nourishment would still bury it, therefore no benefit is observable. Moving the rock groyne beyond 30-40m west has significant impacts in the additional volume and cost of beach nourishment material required.
- The numerical modelling reported in R3471 included an iterative approach which ended with the proposed groyne location. Other locations for the T-shape groyne closer to shore and further east showed worse performance from a coastal processes point of view. Therefore, shifting the proposed works either east or west has significant disadvantages.
- The Detailed Design project timescale is not compatible with long term monitoring of the Sabellaria Alveolata which would allow a greater understanding of the its ephemeral nature.
- In general terms, currently it is considered that the Sabellaria is not in prime condition and although in some areas there are some biodiversity hotspots it is not a generalized throughout the bay.

As per NRW's EIA Screening Opinion response, we will continue to consider measures to avoid, reduce and / or offset effects on the Sabellaria Alveolata and will engage with NRW TE, separately from this Coastal Processes Impact Assessment, to present more thoroughly the findings of the intertidal biotope survey and agree any mitigation measures.

Indicatively, mitigation measures can be:

- Analysing colonisation of Sabellaria in the rock armour of the 2 existing groynes to replicate any identified characteristics of the environment in the new groyne design.
- Acknowledgement that the rock groyne is a porous structure with material size from sand in the core to 6t rocks in the armour and can provide new habitat.
- Habitat enhancements not directly related to the Sabellaria such as rock pools, textured surfaces, holes in the rock armour.
- Translocation of the Sabellaria reef, although we are not aware of any successful trial in the UK.

Appendix A

Penrhyn Bay Coastal Defence Improvements, Baseline review and option development, ABPmer Report No. R2776, April 2017

A1

Penrhyn Bay
Coastal Defence Improvements

Baseline review and option development

Ove Arup and Partners Limited

February 2017

Creating sustainable solutions for the marine environment







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Penrhyn Bay
Coastal Defence Improvements

Baseline review and option development

February 2017



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Heidi Roberts	David Finch	Peter Whitehead
		

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ABPmer
Quayside Suite, Medina Chambers, Town Quay, Southampton, Hampshire SO14 2AQ
T: +44 (0) 2380 711844 W: http://www.abpmer.co.uk/

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1 Introduction

ABPmer has been commissioned by Ove Arup and Partners Limited (Arup) to support a Project Appraisal Report (PAR) being presented to Conwy County Borough Council (CCBC) aimed at identifying a preferred option for future coastal defence at Penrhyn Bay. The two main objectives of the options being considered are: a) to manage coastal flooding; and b) control beach erosion and sediment transport.

At present significant on-going maintenance cost are being incurred through the recycling of shingle/cobble within the bay. A key objective of any proposed scheme will be to reduce future maintenance requirements.

A coastal defence assessment was undertaken in 2014, on behalf of CCBC by Coastal Engineering UK Ltd (CEUK). This assessment examined the present coastal defence afforded to Penrhyn Bay, the coastal processes that occur and identified seven possible Future Management Options. Of these initial seven options, four options have been selected as having the greatest merit and are being considered further within the PAR.

The present assessment:

- Provides a review of the existing baseline conditions at Penrhyn Bay, including wave climate, sediment transport processes and historical trends in beach levels. This review considers the information included in the earlier Coastal Defence Review prepared by CEUK (CEUK, 2014);
- Develops four selected options into workable conceptual schemes, taking account of the coastal processes within the bay. These schemes have been developed through an assessment of cross-shore profile response and a requirement to obtain stable beach orientations in order to reduce future maintenance requirements.

2 Site Overview

2.1 General description

Penrhyn Bay is a northeast facing coastal embayment located around 4 km east of Llandudno and approximately 3 km northwest of Colwyn Bay. The study frontage itself is approximately 800 m in width, contained within two groyne structures facing northeast and north-northwest respectively (Figure 1).

At the southern extent of the site and behind the southern groyne of *circa* 200 m in length, Glan-Y-Mor Road lies immediately behind the promenade at approximately 2 m below the defence crest level. Landward of the road is Dormy House Hotel and Rhos-on-Sea Golf Club, both of which are located on a low-lying alluvial plain. The heavily modified Afon Ganol drains through the golf club and exits through at outflow pipe through the southern groyne. To the south (and east) of the southern groyne the promenade is fronted by rock armouring.

Within the central area of the bay the road steepens uphill, eventually ending at a matching elevation to that of the promenade; of which widens before ending at a row of private housing immediately on top of the beach face. Behind the road is primarily domestic housing associated with the town of Penrhyn Bay.

The northern end of the site contains around 300 m of privately defended properties. A mixture of grasses and scrub have established upon the upper beach material that has accumulated in the lee of the northern groyne. This is a *circa* 350 m structure to the very north of the study extent, with the beach north of this leading up onto the carboniferous limestone headland of Little Orme.



Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 1. Aerial imagery of study extent, including local SMP2 boundary

2.2 Shoreline management policy

The study area lies within the North West England and North Wales Shoreline Management Plan (SMP2, Halcrow 2011) Area 11a. The SMP2 long term plan for Unit 11a is to 'continue to manage the risk of coastal erosion and flooding to industrial, residential, non-residential, commercial and amenity assets along the coast'. The SMP2 recommends that 'beach management will become increasingly important to sustain beaches which are important for coastal defence, amenity, tourism and environmental conservation' (Halcrow, 2011). The specific unit policies are as follows:

- Unit 11a Policy Unit 2.1: Little Orme to Rhos-on-Sea (Penrhyn Bay). The policy is Hold The Line (HTL) for all three epochs by maintaining, improving and raising the defences in response to future climate change/ sea level rise. This includes investigating improving the beach control structures to widen the beach. (Halcrow, 2011). A HTL approach is likely to be the most economically viable and the most effective option in the long term, therefore successfully managing the risk to the surrounding communities of Penrhyn Bay and Rhos-on-Sea.

2.3 Existing defences

The defences at Penrhyn Bay have been described below using a combination of previous reports (CEUK 2014), 2015 engineering surveys, 2013/14 LiDAR datasets, historical engineering drawings and a site survey in 2016.

- Two groyne structures are constructed of quarried rock (Figure 2). The northern groyne extends eastward from the upper beach and is approximately 350 m long, intercepting waves directly impacting on the defences in the centre and south of the Bay. The southern groyne is fishtailed composed on an inner section and two outer arms between 100 and 120 m long, designed to block longshore drift and shelter the defences from easterly wave conditions. Maximum heights of the structures range between 3 and 5.5 m above Ordnance Datum Newlyn (mODN). Both of these structures were constructed in 1990, at which time additional beach nourishment was also undertaken.

- Defences throughout the southern extent of the study frontage (Defence 1, Figure 3) consist of a 0.5-2 m high concrete wall faced with masonry from the base road level to the promenade level at around 6.5 mODN. On the seaward side of the defence a 2 m high recurve wall, around 1 m higher (7.5 mODN) than the promenade level, sits above a 1:2 gradient stepped revetment. This revetment goes down to the underlying beach level at a toe level of around 3.20 mODN, where a sheet pile toe exists.

- Defences through the central section of the site consist of a mass concrete wall faced with masonry from road level to crest height of the promenade at around 9.3 mODN (Defence 2, Figure 3). A 1:2 gradient rock armour slope sits on top of a 2 m concrete recurve wall of crest 6.81 mODN. Below this a 1:2 gradient stepped revetment falls down to a toe level of around 3.5 mODN, where a sheet pile to exists.

- The northern side of the Bay contains a series of private defence structures (Defence 3, Figure 3). These are mainly in the form of 1-3 m high vertical walls irregular in shape, of which were constructed around the 1970's. Crest heights vary between 4 and 6 mODN.

All three defences are fronted by varying levels of shingle and cobble material. This material primarily resides in the northern and southern extent of the frontage, but regular maintenance is undertaken to recycle this material into the central section of the bay.

The current condition of the public defences within central and southern sections of the site is understood to be in 'fair to poor' (CEUK 2014). The rock armour slope, concrete stepped revetment and recurve wall were originally constructed in the 1950's, and are now displaying evidence of abrasion (with some areas exposing the underlying steel reinforcement), and sealant between joints being removed. Exposure of the sheet piling at the toe of the defences has been noted prior to initial beach management works, and this was also assessed to be in a 'poor' condition with signs of decay (CEUK 2014).

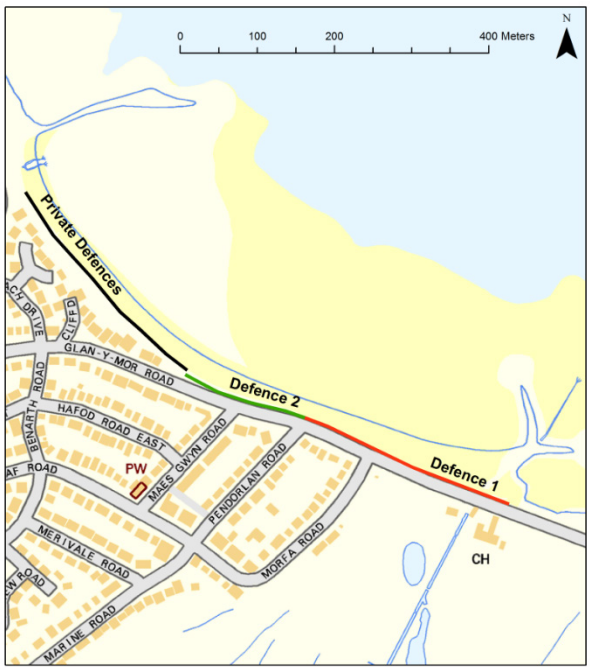


Northern groyne



Southern groyne

Figure 2. Photographs of groyne structures





Defence 1



Defence 2, including rock armouring



Defence 3, private defences

Figure 3. Location of defence sections and photographs of typical sections

2.4 Present beach management practices

Current beach management within Penrhyn Bay originated in the 1980's as a result of a previous failure to retain beach levels at the toe of the pre-existing 1950's public defences. Following consultation the northern and southern groynes were constructed in 1990 to control the movement of sediment. In addition to this a beach recharge of mixed material was carried out, this included the addition of:

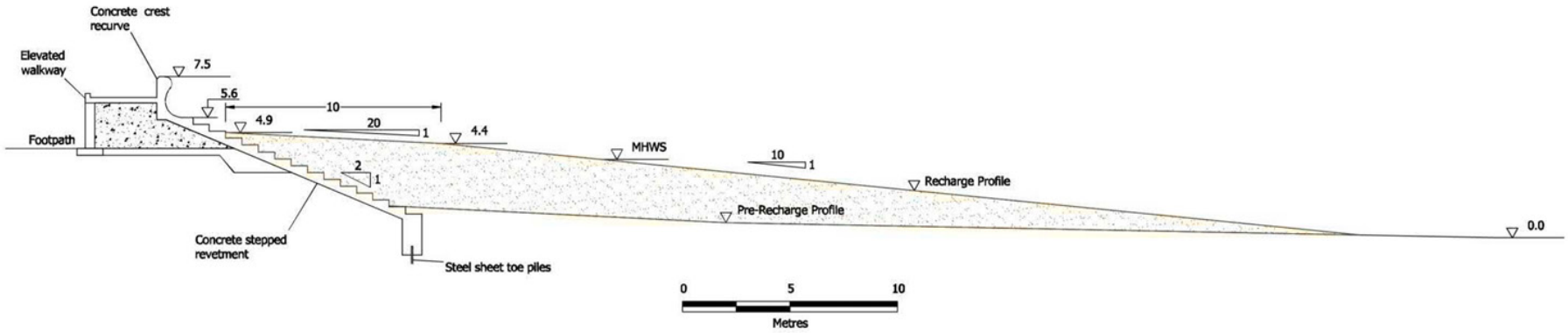
- Sand (D_{50} 0.5-2 mm) throughout the privately defended western section of the Bay;
- Shingle (D_{50} 20-40 mm) throughout the central section; and
- Cobble material (D_{50} 120-130 mm) throughout the eastern section of the Bay.

The recharge raised the upper beach profile to a slope of 1:10 up to MHWS level, with a 1:20 gradient from MHWS up to a level of 4.9 mODN (CEUK 2014). These beach profiles are reproduced from CEUK, 2014 in Figure 4.

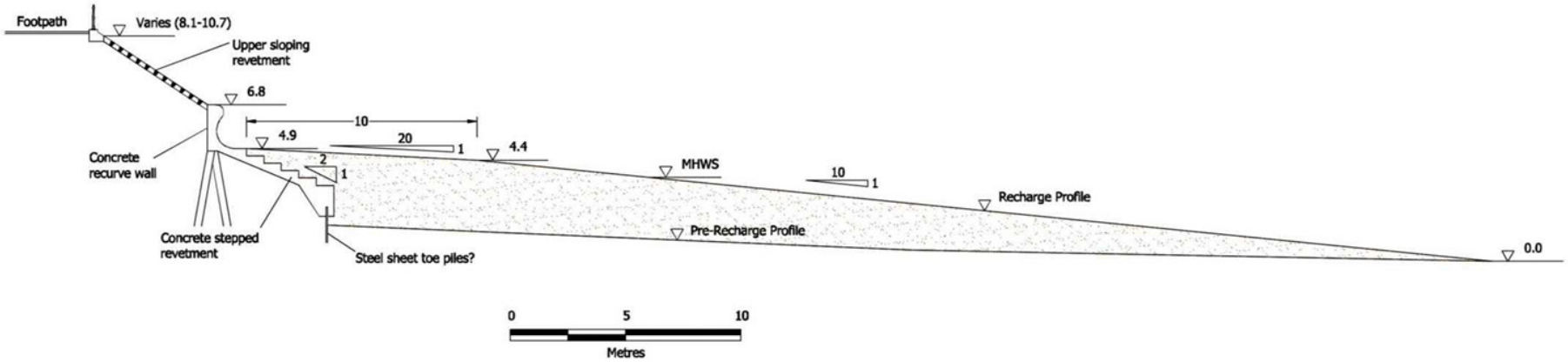
Since this recharge the sheet piling at the toe of the defences has not been consistently exposed as a result of beach drawdown and the realignment of the beach. Overall, the recharge has been considered to have provided an improvement to the pre-existing defences. However as time progresses ongoing attrition and abrasion of the recharge material has resulted in a fining of material size by around 50%, and is therefore more mobile than desirable (HR Wallingford, 2000).

A series of reinstatement schemes have been undertaken since 2003 (Figure 5), with the natural profile of the upper beach artificially altered to protect the existing defences from toe exposure. Such activities are required every 12-18 months (CEUK 2014) and consist of the re-cycling of material from the accreting eastern section of the Bay (where overspill of the southern groyne is also apparent) to the more central sections of the bay which experiences the greatest beach erosion.

CEUK 2014, examined the variation in beach in beach levels throughout the bay from 1997 to 2010, by comparing the numerous levels data sets available. This concluded that the volumes in the western side of the beach have remained stable, although material reinstated within the central portion of the beach is quickly observed to migrate to the eastern side of the bay. However in general the report suggests that no significant change in overall volume of the upper beach has occurred across the Bay when considering reinstatement activities.



Recharge profile in Defence 1



Recharge profile in Defence 2

Source: CEUK, 2014

Figure 4. Profile of the 1990 recharge throughout eastern (top) and western (bottom) extents of the Bay



Southern beach accumulation 10/02/2005



Reinstatement 15/03/2005



Southern groyne overspill 10/02/2005



Reinstatement 15/03/2005



Southern beach accumulation 15/03/2007



Reinstatement 28/03/2007



Southern groyne overspill 15/03/2007



Reinstatement 28/03/2007

Figure 5. **Accumulation of sediment in the southern section of the beach and sediment overspill of the southern groyne (left) requiring frequent reinstatement works (right)**

3 Baseline Conditions

This section provides a description of marine and coastal physical processes within the study area. This baseline characterisation forms the basis from which assessments of potential defence options can be carried out. The focus of the baseline review has been tailored towards providing sufficient detail to support the options development and appraisal. Of particular interest are the waves and water levels which will directly interact with the coastal defences and the beach material fronting the defence structures. This baseline understanding also has a critical role to play in informing assessments of potential changes to the defence structures.

3.1 Water levels

3.1.1 Tidal water levels

The coastline surrounding the study site has a hypertidal range, with a spring tide range in excess of 7 m. This is due to the tidal wave propagating via the St Georges channel to the south and an additional channel between Ulster and the Mull of Galloway to the north. This propagation is virtually simultaneous (Myres 1993), leading to an amplification effect on the tidal wave. The tidal cycle at Penrhyn Bay is semi-diurnal, i.e. two tides per day.

Table 1 provides the astronomical tides at Colwyn Bay, the nearest location for which values are available from Admiralty Tide Tables (ATT). These levels are based on corrections from the nearest Standard Port.

Table 1. Tidal characteristics at Liverpool, adjusted locally for Colwyn Bay

Location	Reference Still Water Level							Mean Range (m)	
	HAT	MHWS	MHWN	MSL	MLWN	MLWS	LAT	Spring	Neap
Liverpool (mCD)	10.3	9.4	7.5	5.3	3.2	1.1	0.0	8.3	4.3
Liverpool (mODN)	5.37	4.47	2.57	0.37	-1.73	-3.83	-4.93		
Colwyn Bay (mCD)	8.7	7.8	6.1	4.20*	2.4*	0.78*	-0.1*	7.0	3.7
Colwyn Bay (mODN)	4.6	3.7	2.0	0.10*	-1.8*	-3.42*	-4.30*		
* Indicates no available data in 2016 ATT, therefore values from CEUK (2014) referenced									

Source: UKHO, 2016.

Datum conversion for Colwyn Bay and Liverpool from metres above chart datum (mCD) and mODN are -4.10 m and -4.93 m respectively (UKHO, 2016).

3.1.2 Extreme water levels

The predicted water levels in 2.1.1 do not take into consideration atmospheric conditions, including atmospheric pressure and wind, which can apply an additional variation in the level of the observed tides.

The Environment Agency coastal flood boundary dataset (CFBD) provides an assessment of extreme water levels based on tidal and non-tidal surge components for the base year of 2008 (Environment Agency CFBD, 2011).

The extreme levels reported in the CFBD in proximity to Penrhyn Bay (Environment Agency Chainage 1116) are summarised in Table 2. These values have also been adjusted to a base year of 2017 by adding a sea level rise value of 0.042 m between 2008 and 2017 as per the original UKCP09 dataset (the original CFBD dataset does not extend to this date).

Table 2. Extreme water levels for base year of 2008 and adjusted to include sea level rise for 2017 epoch – (Environment Agency Location Chainage 1116)

Return Period (Years)	Epoch 2008 (mODN)	Epoch 2017 (mODN)
1:1	4.8	4.84
1:2	4.9	4.94
1:5	5.01	5.05
1:10	5.09	5.13
1:20	5.18	5.22
1:50	5.29	5.33
1:100	5.38	5.42
1:200	5.47	5.51
1:1000	5.67	5.71

3.1.3 Relative sea level

The Northeast Irish Sea has seen relatively stable fluctuation in mean sea level, varying less than 1 m around present day levels in the last 5,000 years (Halcrow 2012). Future changes in relative sea level (RSL) are quoted as the net effect of geological adjustments in land levels and the projected absolute changes in mean sea level. A summary of the UKCP09 relative sea level rise projections up to 2100 and extrapolated up to 2117 for this project (Grid Cell 18959 at Penrhyn Bay) are presented in Table 3. A linear interpolation of UKCP09 predictions has then been carried out between 2100 and 2117 in order to provide return periods from the present day. These projections are based on the medium emission scenario for the 95 percentile confidence interval, as per Welsh Government Adapting to Climate Change Guidance (2011[1]). For this scenario the estimated RSL between 2017 and 2117 is calculated to be 0.705 m.

Table 3. Sea level rise predictions (UKCP09)

Year	Medium Emissions 95%ile (m)
1990	0.000
2000	0.038
2010	0.082
2017	0.115
2020	0.129
2030	0.181
2040	0.237
2050	0.298
2060	0.363
2070	0.433
2080	0.506
2090	0.584
2100	0.667
2110	0.757
2117	0.820
Note: Epoch values have been calculated using a linear extrapolation of the time series beyond the published data.	

Source: UKCP09 Grid Cell 18959

[1] http://gov.wales/docs/desh/publications/111231floodingclimatechangeen.pdf

3.2 Tidal currents

Across nearshore regions of Liverpool Bay tidal flows are generally east to west in orientation, with the flood tide in an easterly direction, and the ebb in a westerly. Flow speeds are generally higher on the flood than the ebb, with values around 1 m/s and 0.74 m/s respectively (HR Wallingford 2005). This leads to the potential for sediment transport in an onshore direction (CEUK 2014).

The Little Orme Head is also understood to create a clockwise eddy during benign conditions, resulting in a small localised current within Penrhyn Bay (Halcrow 2012).

Further inshore tidal streams are approximately parallel in orientation to the coastline, rotating more south-easterly in direction on the flood, and north-westerly on the ebb. Flow speeds prior to the construction of the groynes were found to be significantly lower in magnitude to those in the wider Bay (British Maritime Technology, 1987), with flows generally eastward on the flood and westward on the ebb. With the construction of the northern and southern groynes in 1990, this flow inshore has also been intercepted such that tidal flows within bay will have been further weakened and thus sediment transport processes in the bay will be dominated by wave induced currents.

3.3 Waves

3.3.1 Local wave climate

Little Orme Head provides a degree of wave sheltering for Penrhyn Bay from offshore high magnitude north-westerly wave conditions, which are partially refracted and diffracted from their initial generation in the Irish Sea. Easterly and north-easterly conditions are restricted in fetch length by parts of northeast Wales and northwest England. Similarly, the Constable Bank (located approximately 7 km offshore) is likely to limit the effect of northerly storm waves and provide a degree of wave dissipation (Halcrow 2012), particularly at low tide.

In order to accurately represent wave conditions in the nearshore of the site ABPmer has used the SEASTATES database of hindcast offshore data (ABPmer 2013). SEASTATES is a spectral wave model which covers the North Atlantic and European shelf seas, and is driven by the spatially and temporally varying CFSR (Climate Forecast System Reanalysis) wind fields developed by NOAA (National [American] Oceanic and Atmospheric Administration). The SEASTATES hindcast has been previously validated against 28 buoys within UK and European waters.

The standard SEASTATES hindcast does not allow for water level variations. However, water levels variations at the site are significant and will affect the nearshore wave climate. Therefore a local 30 years hindcast of nearshore wave conditions was developed utilising varying water levels recorded at Llandudno combined with offshore SEASTATES wave data (ABPmer 2012). This hindcast data set covers the 30 year period between 1980 and 2010 and wave data was extracted at a nearshore location, just offshore of the Bay in a depth of -4.5 mODN, and an inshore location, located at the toe of the defence structures. The wave rose from the point outside the Bay set is shown in Figure 6

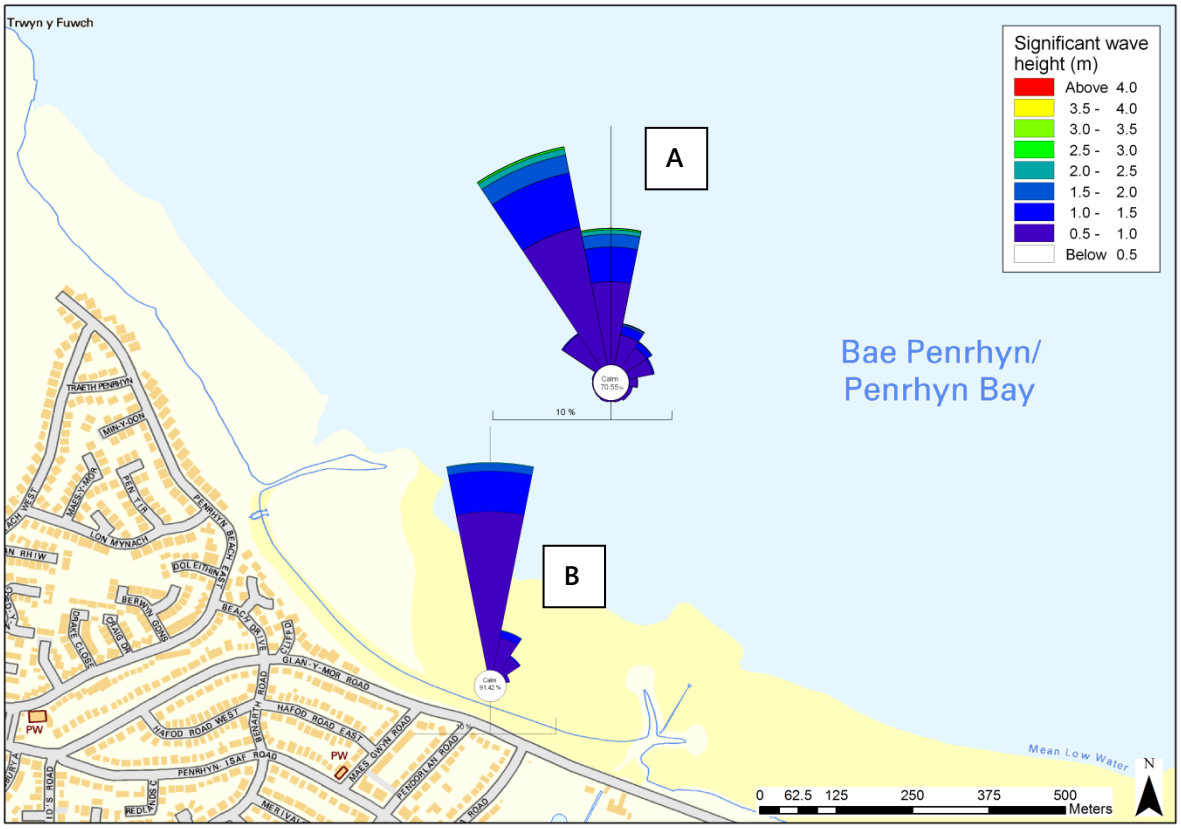


Figure 6. **Nearshore (A) and inshore (B) wave roses using a combination of observed water levels and offshore SEASTATES hindcast**

Predominant wave conditions just offshore of the bay occur from north-north-westerly or northerly directions, with over 90% of significant wave heights less than 1.5 m (Figure 6, Table 4). As these waves reach the bay, and location of the groynes (in maximum depths of approximately -2.5 (northern) and -2 (southern) mODN respectively), they will be afforded further refraction. Waves from the northeast are less frequent and typically have a significant wave height less than 1 m. These waves, however, will be afforded less refraction as they are transformed towards the shore.

Table 4. **Frequency table of significant wave heights and directions for the nearshore (Figure 6, A)**

Hs (m)	Blank Value	Direction (°)								Grand Total (%)
		0-45	45-90	90-135	135-180	180-225	225-270	270-315	315-360	
0-1	0.94	12.64	15.58	8.51	1.66	0.86	0.60	1.37	49.01	**91**
1-2	0.00	2.16	0.22	0.01	0.00	0.00	0.00	0.01	5.58	**8**
2-3	0.00	0.17	0.00	0.00	0.00	0.00	0.00	0.00	0.63	**1**
3-4	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.03	**0**
Total (%)	**1**	**15**	**16**	**9**	**2**	**1**	**1**	**1**	**55**	**100**
Values relate to percentage occurrence										

As these waves reach the two groynes, the groynes provide a varying level of sheltering and waves will be further modified by the effect of diffraction. The northern groyne will defract waves entering the bay from northerly, north-westerly and north-easterly directions, whilst the southern groyne only has a significant effect on waves entering from more north-easterly directions. Its effect on the waves is also more restricted to the southern end of the bay.

3.3.2 Extreme waves

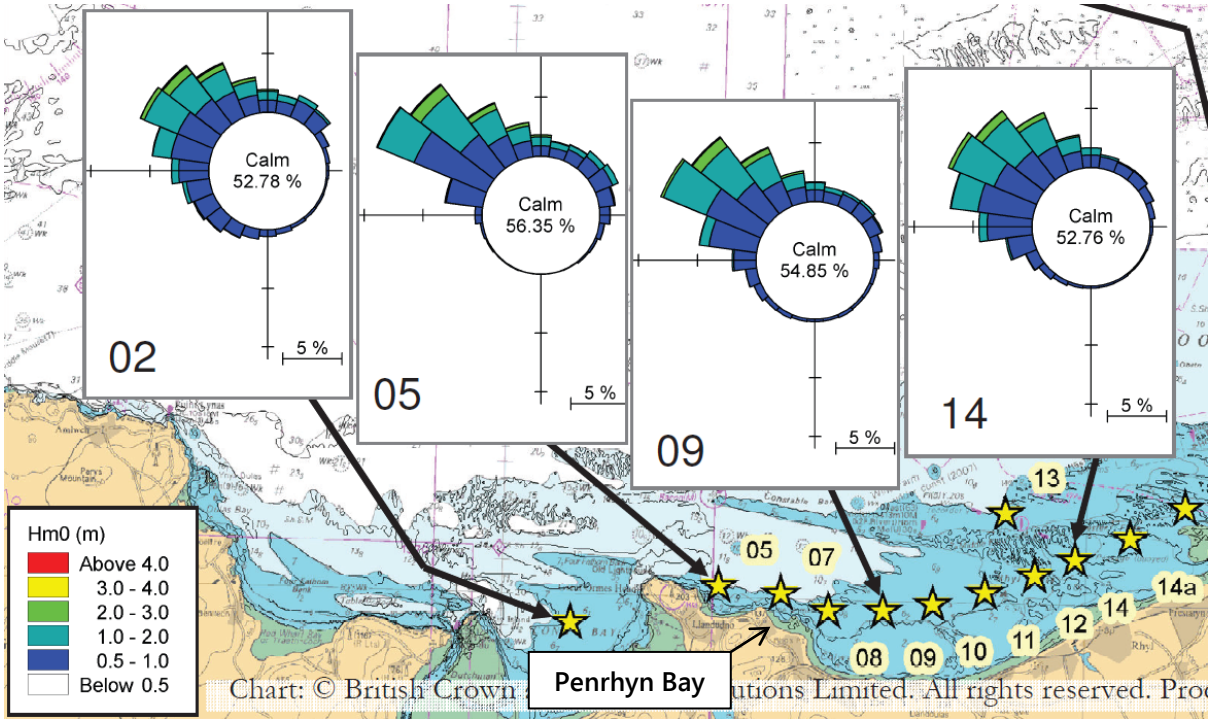
The location of Penrhyn Bay in relation to the semi-enclosed basin of the eastern Irish Sea means that offshore extreme wave modelling is likely to over-represent the conditions experienced in the Bay. In order to truly represent extreme conditions in the nearshore, a Joint Wave and Water Level Probability Analysis (JPA) for the North West and North Wales was completed by Halcrow (2012) as part of the SMP2. This provides combinations of wave heights and water levels for an inshore location relevant to Penrhyn Bay for return periods of 1:1 and 1:100 years (Table 5). A full table with varying return periods between 1:1 and 1:200 is provided in Appendix A. The seabed depth at which this data was obtained was -14 mODN.

Table 5. Extreme wave conditions within the nearshore from the JPA

1:1 Return Period		1:100 Return Period	
W. Level (mODN)	H_{m0} (m)	W. Level (mODN)	H_{m0} (m)
0.00	3.39	0.00	5.46
2.00	3.37	2.00	5.39
2.60	3.25	2.60	5.31
2.73	3.20	3.10	5.08
3.10	3.05	3.46	4.82
3.40	2.95	3.68	4.63
3.49	2.90	4.15	4.00
3.75	2.73	4.40	3.69
3.96	2.50	4.54	3.50
4.10	2.34	4.71	3.20
4.27	2.10	4.88	2.90
4.40	1.83	5.05	2.50
4.50	1.60	5.17	2.10
4.72	1.00	5.26	1.60
4.82	0.00	5.35	1.00
-	-	5.39	0.00

Source: Halcrow, 2012

It is noted that the extreme 1:1, 1:50 and 1:100 year significant wave heights in this assessment are 3.39, 5.15 and 5.46 respectively. These are significantly greater than expected, based on the largest wave height condition from the SEASTATES data which was 3.78 m, based on a 30 year record. However, it also noted that the SEASTATES extraction point is located significant further inshore, in a seabed depth of -4.5 mODN compared to -14 mODN for the JPA. Thus this discrepancy is expected to be due to the additional refraction that will have been undergone by these waves as they travel inshore. This is further supported through a consideration of the wave roses from the two data sets, where the JPA identifies a far greater proportion of waves from westerly and north-westerly directions (Figure 7). As such, the extreme wave heights included in Table 5 are considered to over-estimate the waves entering the Bay.



Source: Halcrow, 2012

Figure 7. **Wave roses at coastal locations in close proximity to Penrhyn Bay from Halcrow 2012, JPA Study**

3.4 Sediment transport regime

Penrhyn Bay is largely placed upon bedrock of carboniferous limestone, which can be seen to outcrop at Little Orme Head. Immediately on top of this is a layer of sandstone which is likely to provide a source of sediment. Sporadic Quarternary deposits of boulder clay are also apparent, along with estuarine alluvium linked to the last glacial period.

Currently the natural intertidal consists of a mixture of sand and shingle, with higher sand proportions in the shallower graded lower beach and shingle/cobble material in the upper steeper beach. This coarser material is suggested to be a combination of the beach recharge material, and a product of earlier shoreline erosion exposing glacial deposits.

Topographic profiles have been extracted from two Light Detection and Ranging (LiDAR) datasets at five sites around the bay (Figure 8). The profiles at the very southern extent of the Bay show a pronounced berm with heights of around 5 and 5.8 mODN (profile 1, Figure 9). These berms lie approximately 20-30 m seaward of the defences. These berm heights are significantly higher than the beach level directly against the defences, where the beach levels are approximately 4.5 mODN. The original nourished level was around 4.9 m against the seawall when the 1990 beach was originally nourished.

Towards the more central section of the bay (Profiles 2–4, Figure 9), the upper beach pushes up against the defence with no noticeable berm.

In the northern section of the bay a beach berm again becomes more pronounced (Profile 5, Figure 9), where an establishment of grasses and scrub can be seen on the top of the berm. Here, the maximum berm height is around 4.5-5 mODN. The berm is generally much flatter than that observed at the

southern end of the bay and is only marginally higher, than the beach level at the toe of the cliff (4.2-4.5 mODN). At this location the beach is more sheltered by the larger more north-north-westerly and northerly waves which would act to build up the berm under extreme wave conditions.

The overall littoral drift is reported to be from west to east along the frontage containing Penrhyn Bay (Halcrow 2012). An overall sediment transport potential (not considering restricting structures) of 22,000 m^3 was calculated by HR Wallingford (HR Wallingford 1991), although this is understood to relate to the movement of sand sized fraction.

The Little Orme headland restricts the majority of sediment drift from beaches to the north of Penrhyn Bay, but a small amount of fine material by-passing is suggested to occur (Liverpool Bay Coastal Group, 1999). Tidal currents across Penrhyn Bay are small, and therefore the majority of sediment transport is expected to be due to wave action, which can also result in both the onshore and offshore transport of finer sediment material.

The supply of sediment to Penrhyn Bay has been considerably modified as a result of the construction of the two groyne structures (CEUK 2014). The bay has become compartmentalised with no ability for coarser shingle and cobble material to enter the bay from the north. The southern groynes also limits the loss of this material from the bay, although some small losses are observed when the berm builds up at the southern end of the beach and becomes transported over the base of the southern groyne.

The influx, export and transport of finer material on the lower foreshore has also become more complex as a result of the groyne structures, since the natural littoral drift has been blocked and more complex currents are setup in the bay by the structures. The difference in the relative lengths of the two structures and the orientation of the bay to the prevailing waves, may suggest that there could be a greater loss from the bay than an influx of material. However, the bed level change calculation in CEUK, 2014 do not suggest a significant long-term loss or gain of material in the lower foreshore. It is also noted that underlying boulder clay is generally resistant to erosion but vulnerable if exposed to wave action. Thus overtime is could still be expected that there will be some overall beach level lowering in the lower foreshore.

As identified in CEUK, 2014, the primary movement of sediment in the bay relates to the upper shingle/cobble beach in the central and southern end of the bay. This movement was noted immediately after the 1990 beach recharge (CEUK 2014). In the southern half of the bay the drift is west to east and results in overall lowering of beach levels in the central section of the bay and an accretion of material in the southern section. Several reinstatement campaigns have been undertaken to recycle material from the southern section to the central section, however, in each case this material is returned to the southern end of the bay. As opposed to this the northern half of the bay has remained relatively stable.

The primary reason for these observations is that the upper beach is now directly controlled by the two groyne structures and it now wishes to adopt an alignment which it not commensurate with the existing defences. In particular, the defence in the central section of the bay, protrudes seawards of the natural alignment the beach would wish to adopt. Therefore, material placed in front of this central section during the reinstatement campaigns, is relatively quickly transported to the southern end of the frontage as the beach attempts to recover to a more natural equilibrium position. This principle is examined further in Section 5.

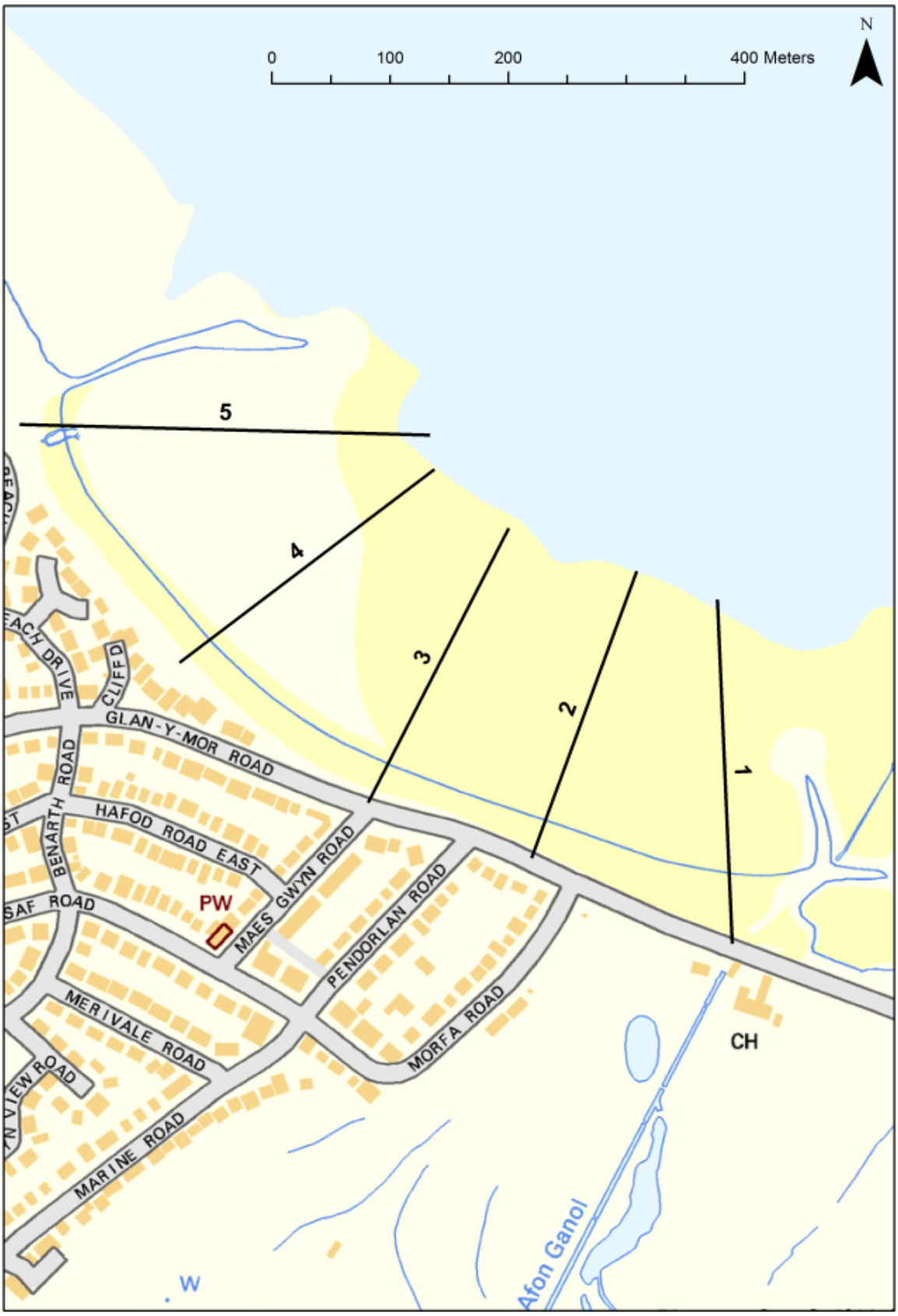


Figure 8. Topographic profile locations throughout Penrhyn Bay

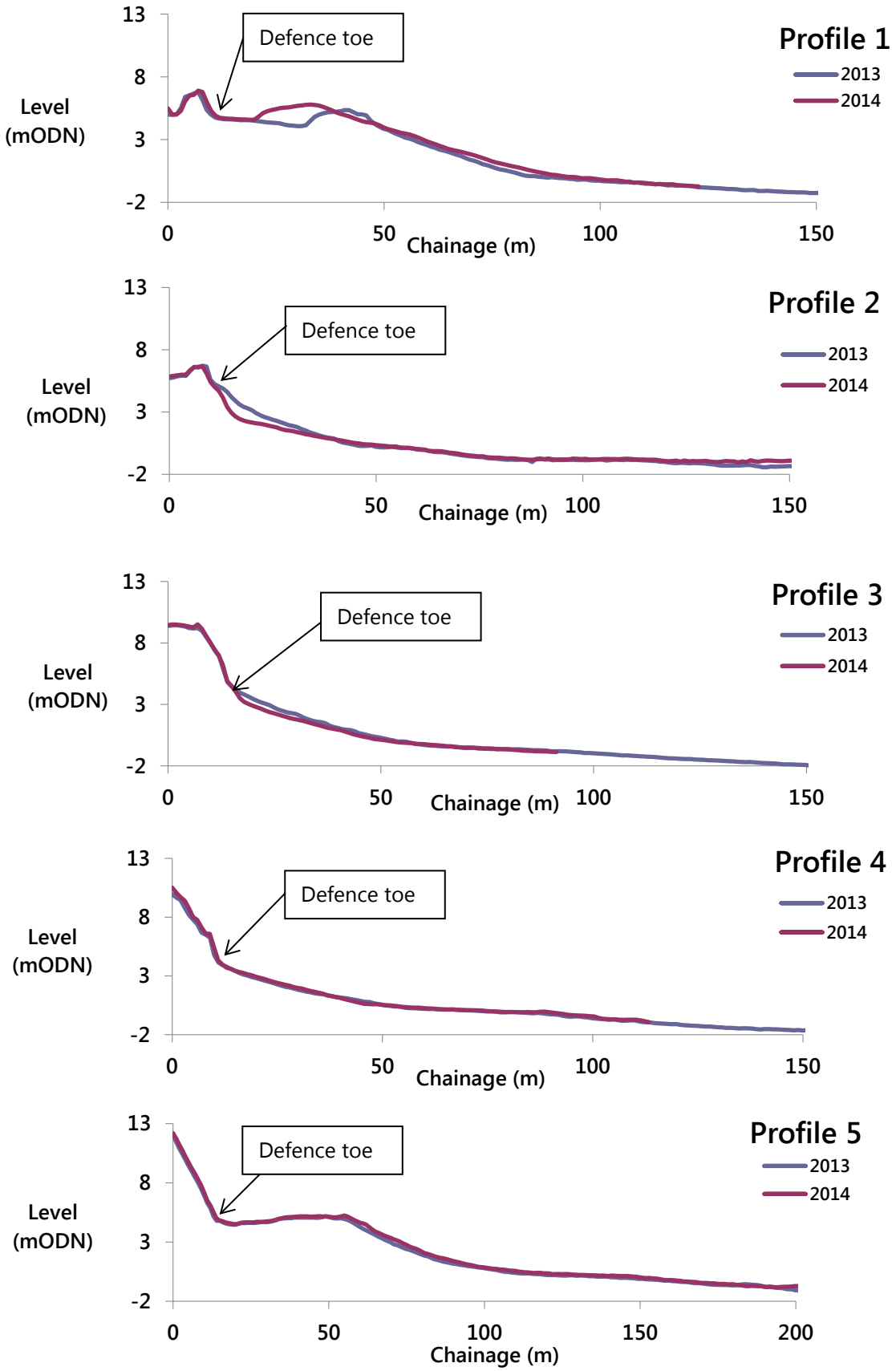


Figure 9. **Topographic profiles extracted from 2013 and 2014 LiDAR datasets. Locations shown in Figure 8**

4 Beach Profile Variability (Shingle B)

4.1 Overview

In order to assess future management options that incorporate a beach it is important to understand the natural variability that may occur to the beach. This natural variability occurs due to both cross-shore process (considered here) and longshore process (considered in Section 5).

The natural variability of the upper shingle beach profiles for Penrhyn Bay has therefore been explored via the application of the two dimensional parametric model (Shingle-B[2]). This model predicts the response of a predominantly shingle beach under waves including bimodal sea states (HR Wallingford 2016). The parametric model is based upon a mobile bed flume study undertaken by HR Wallingford to examine the profile responses to gravel beaches to bimodal wave spectra, characterised by swell and wind wave periods in various combinations. Further information on the model can be found at the Channel Coastal Observatory website including the associated user manual.

The model predicts the likely beach profile that will result under varying sea states. This then provides an indication of the amount of material required within the profile in order for the beach to provide the protection required. It also provides information on the width of the profile, and can therefore be used to assess what length a structure may need to be if it is to hold the toe of the beach and prevent bypassing.

The model input parameters are listed below and described in the following sections:

- Beach profile
- Wave height and water levels (JPA)
- Wave period; and
- Wave height and water levels (SEASTATES time series).

4.2 Model inputs

4.2.1 Profiles

The general profile of the beach within Penrhyn Bay required schematisation prior to being inputted to the model. This has been achieved via a combination of LiDAR extraction and engineering drawings. In order to represent the maximum volatility of the beach the profile in the southern end of the beach was used (Profile 1, Figure 9). This section of the bay is subject to the greatest amount of material movement and consists of a more natural beach section, and one that is not significantly influenced by the defences.

The profile is split into three main sections, the berm crest, the upper gradient and lower gradient. The 5 mODN berm crest represents the recharged beach profile in the centre of the bay. The 6 mODN berm crest represents the observed berm at the northern and southern sections (Figure 10).

The upper beach gradient was fixed at 1:8 to a chainage of 80 m, in line with the measured profile. This is also the same starting gradient used throughout the model development (HR Wallingford 2016). The lower beach shallows to a gradient of 1:50.

[2] Additional Note: Shingle-B was funded by the Environment Agency as a flood and coastal erosion risk management grant in aid project, Grant No. LDW 41231.

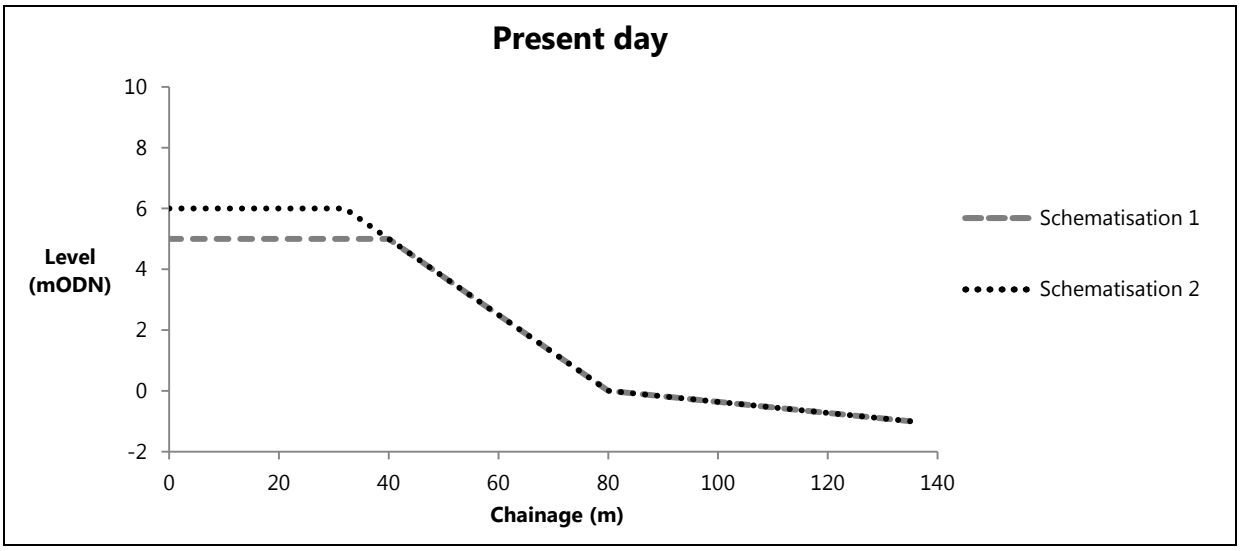


Figure 10. Present day scenarios

Shingle-B can be used to examine future design conditions, but it requires the berm height to be higher than the still water levels. Consequently with increased water levels as a result of climate change predicted Shingle-B cannot be used with the berm schematised to the present day level. To represent a beach profile under a future climate change scenario the lower and upper beach gradients were fixed at the same values to the present day, but the berm crest increased further to 7 mODN. This profile is shown in (Figure 11).

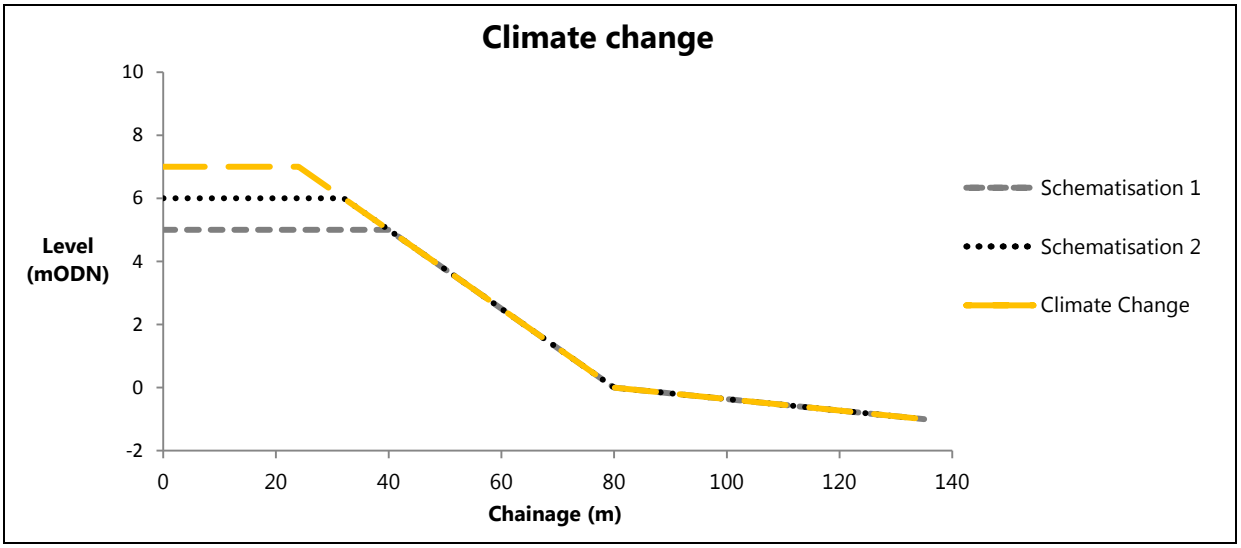


Figure 11. Climate change profile schematisation

4.2.2 Wave and water levels (JPA)

A number of significant wave height/water levels pairings were initially selected from the earlier JPA analysis (Halcrow 2012) for both the 1:1 and 1:100 year return periods. Five pairings for the annual (1:1 year) and extreme (1:100 year) events were chosen to give a representative sample for the different wave and water level combinations, e.g. a more extreme wave height occurring at a low water level or a lower wave height occurring at a more extreme water level. The selected JPA combinations with assigned wave periods are shown in Table 6.

Table 6. Selected JPA combinations with assigned wave periods

Shingle-B Run Number	1:1 Return Period			1:100 Return Period		
	Water Level (mODN)	Wave Height Hm_0 (m)	Peak Wave Period (s)	Water Level (mODN)	Wave Height Hm_0 (m)	Peak Wave Period (s)
1	2.00	3.37	8.5	2.00	5.39	12.0
2	3.10	3.05	8.0	3.46	4.82	11.0
3	3.96	2.50	8.0	4.54	3.50	10.0
4	4.40	1.83	7.5	5.05	2.50	8.0
5	4.72	1.00	6.5	5.35	1.00	6.5

Whilst the model can use a bi-modal sea state, the sheltered location of Penrhyn Bay in relation to open Atlantic waves means that waves are generated locally within the Irish Sea. Therefore a more standard, non-bimodal sea state was adopted.

The JPA does not include wave period and therefore SEASTATES hindcast data was used to develop a relationship between significant wave height (Hs) and peak wave period (Tp) for waves coming from 270° to 45° (Figure 12). Wave periods were then extrapolated manually for the extreme waves in the JPA and provided in Table 6.

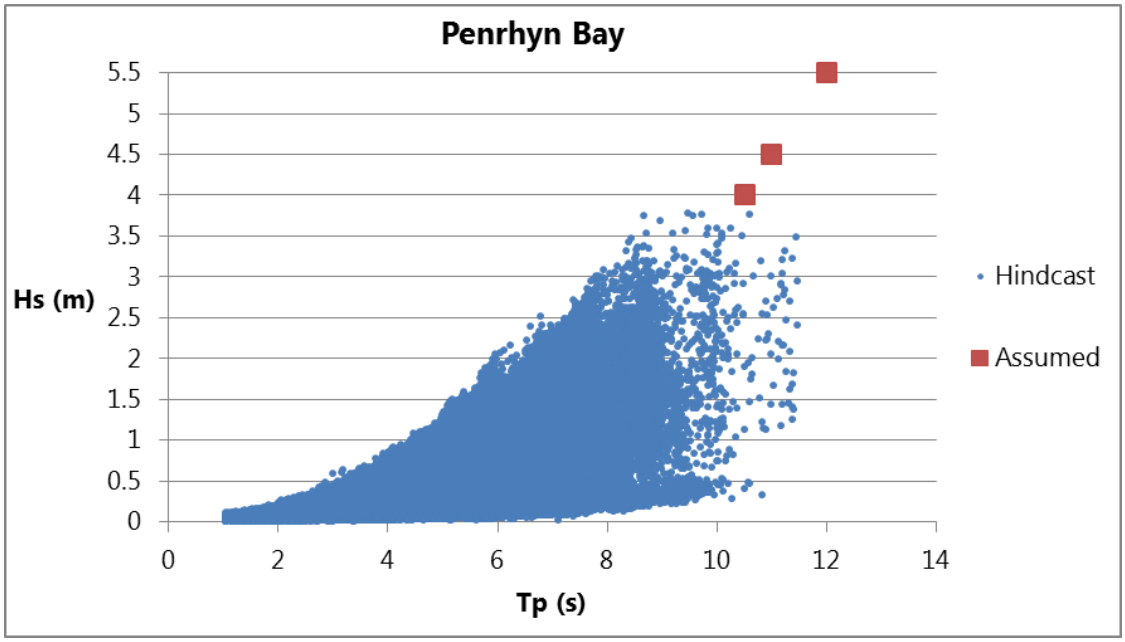


Figure 12. Significant wave height against peak wave period from the SEASTATES hindcast

4.2.3 Waves and water levels (SEASTATES time series)

The extraction point used within the JPA is -14 mODN and the larger north westerly and northerly waves are thought to be afforded further refraction before they reach the site. The largest wave recorded at the SEASTATES location in a seabed depth of -4.5 mODN was 3.78 m. This data record covers 30 years so this event can be roughly considered to have a 1:30 years return period. But this is significantly than that expected from the JPA analysis where the 1:1, 1:50 and 1:100 year significant wave heights were 3.39, 5.15 and 5.73 respectively.

The location of the wave climate for use in Shingle-B it not prescribed, but it represents a nearshore point at which depth limited breaking has not taken place and where waves can be expected to be transformed to the coast without further significant refraction. For these reasons it was felt that the initial JPA pairings were significantly over predicting the beach response.

The 1:100 year JPA pairing that resulted in the greatest profile response (Run 2), was therefore modified such that the wave height was reduced, based on the knowledge that the 1:50 year wave height is more likely to be around that observed in the SEASTATES record. The full adjustment made is described in the equation below:

$$\left(\left(\frac{\text{JPA 1:100 max Hm}_0}{\text{JPA 1:50 max Hm}_0}\right) \times (\text{SEASTATES 1: 37 max H}_s)\right) \times \left(\frac{\text{JPA Run 2 max H}_s}{\text{JPA 1:100 max Hm}_0}\right) \quad \text{Equation 1}$$

The adjusted 1:100 year return event pairing is provided in Table 7, for the case with and without sea level rise.

Table 7. Adjusted SEASTATES maximum wave height for a 1:100 return period

Run Number	Water Level (mODN)	Significant Wave Height (m)	Peak Wave Period (s)
2 (Present Day)	3.46	3.53	9.0
2 (Epoch 2117)	4.21	3.53	9.0

4.3 Shingle-B results

The model was initially run for the five selected JPA representations set out in Table 6. It is important to note that the higher water levels for both the 1:1 and 1:100 (model Run 5) resulted inundation of the berm, which resulted in the output profile being invalid. Run 5 results have therefore not been reproduced here.

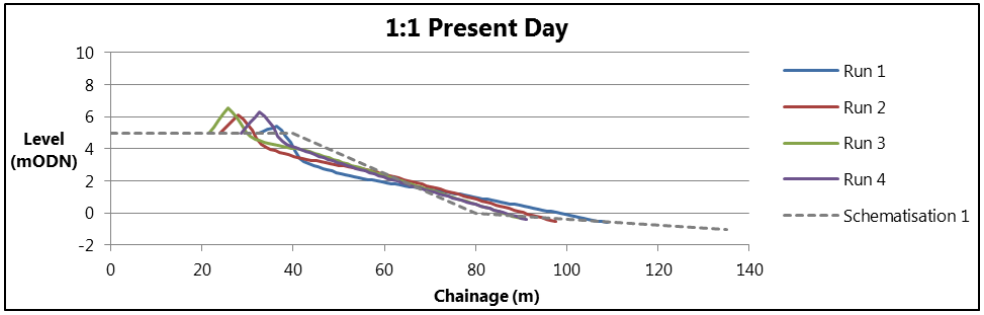
The result of Shingle-B for the three beach schematisations are shown in Figure 13 below. As noted in Section 4.2.1, beach schematisation 1 most closely represent the present beach, and from this it can be seen that the beach berm is built up in a very similar way to that observed in nature (Profile 1, Figure 9), with a maximum berm height closer to 6 mODN. The berm is also set back some 20 m from the original berm and around 20 m in front of the defences, similar seen in the southern section of the beach.

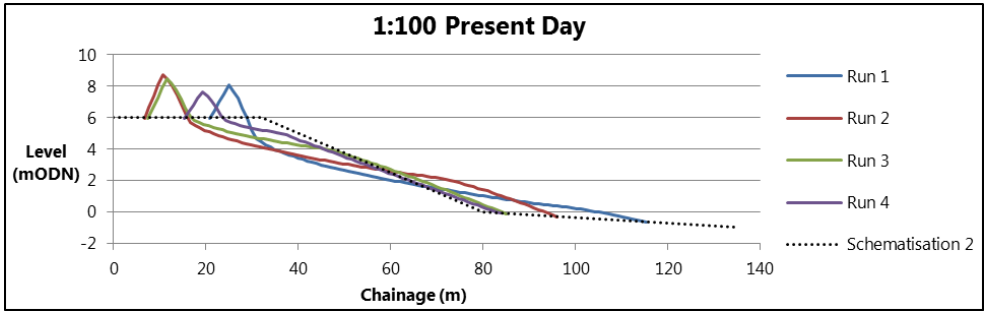
The profile response under the 1:100 years events, however, is much more dramatic and indicates that event in the southern section of the bay the berm could be up against the defence under some wave/water level pairings. This response is greater than initially anticipated and therefore further consideration is given to the use of JPA data set (Halcrow 2012) in comparison to wave conditions derived from the time series data.

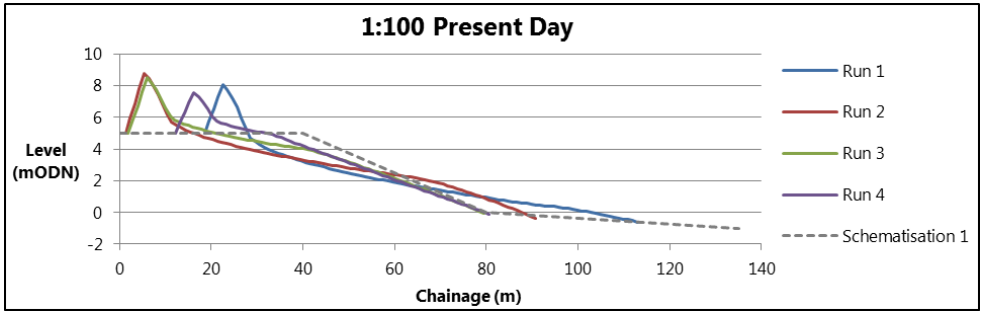
Model results using adjusted time series wave conditions are presented in Figure 14. This figure also includes the results from unadjusted Run 2 JPA conditions.

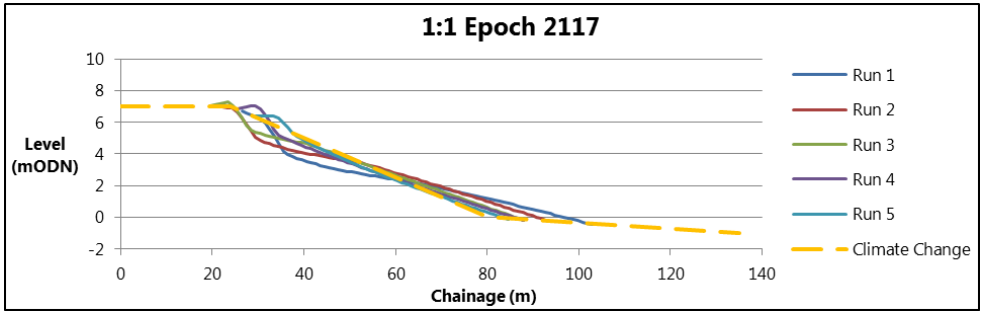
The time series results are considered to be much more realistic, but still highlight that under the present day an extreme wave event could potential raise the crest of the beach berm to a height of 7.5 mODN (the height of the existing defences). Under the climate change scenario the beach berm is predicted to reach a level around 8 mODN.

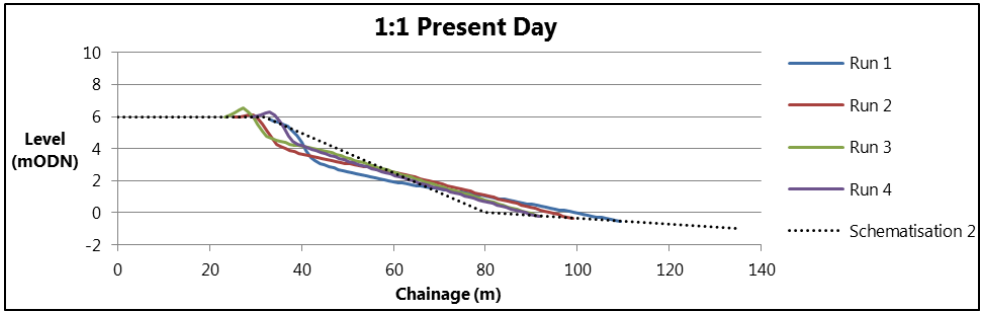
The results also show how beach material has been drawn down the profiles and the toe of the shingle beach has extended around 15 m seawards of its starting position.











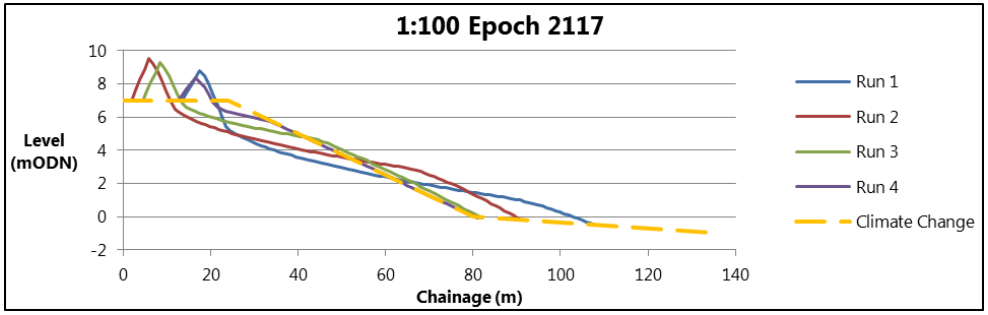


Figure 13. Shingle-B results for JPA wave and water levels

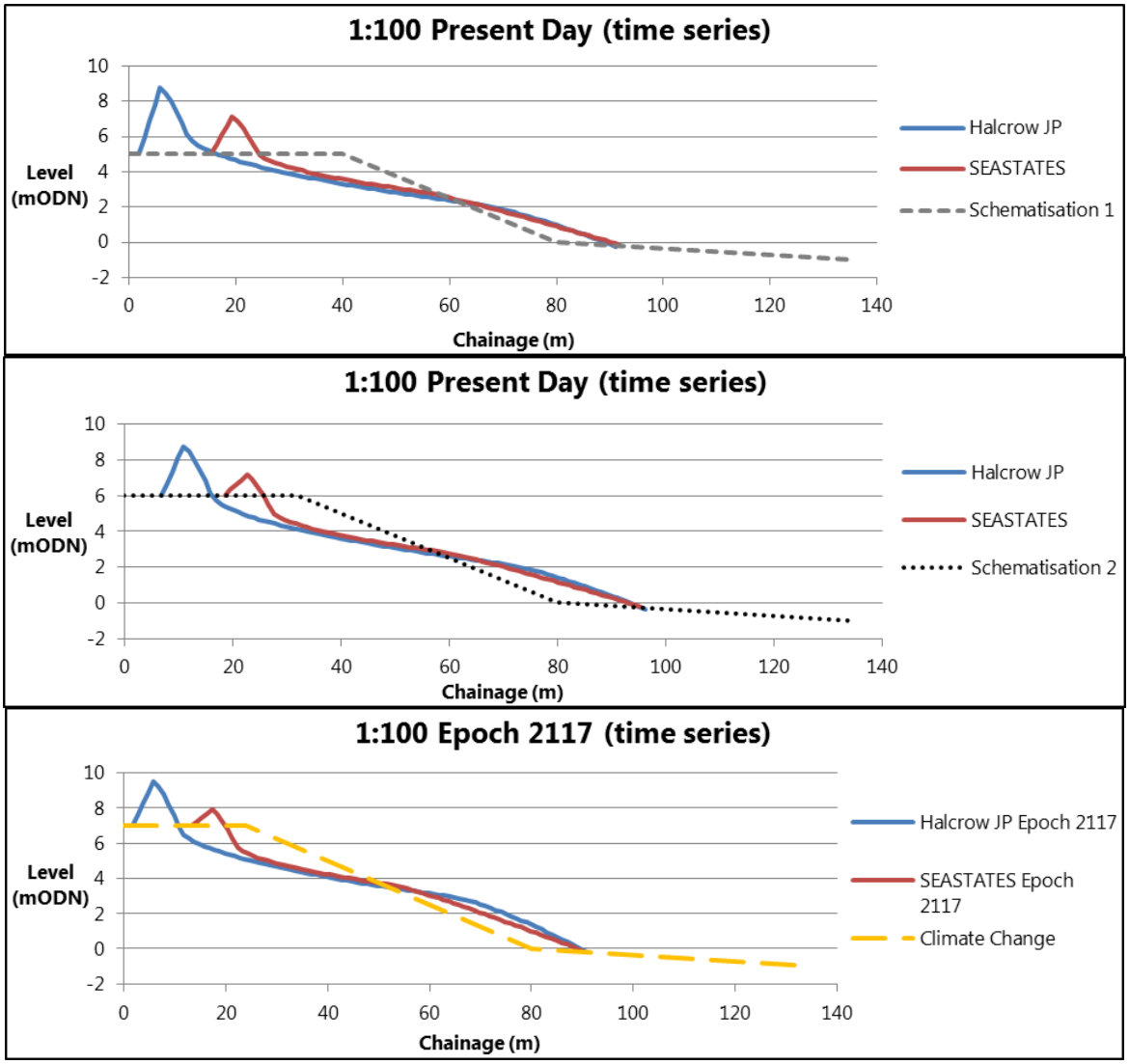


Figure 14. Shingle-B profile comparison for SEASTATES time series adjustment and Run 2 JPA wave and water level inputs

Photographic evidence of past berm positons are provided in (Figure 15) which clearly show how the berm has been raised up above the general beach crest height in the past.



Source: CCBC

Figure 15. Photographic evidence of past berm heights and positions

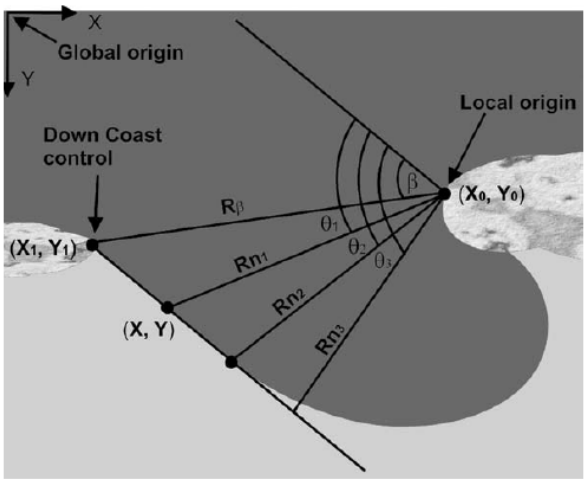
5 Equilibrium Bay Shape (MepBay)

5.1 Model overview

A parabolic bay shape model for equilibrium planform of bay beaches (MepBay) has been applied to the site in order to examine the equilibrium shape of the bay during normal conditions. The full model methodology is explained in more detail by Hsu & Evans (1989) and Fontoura Klein *et al*. (2003), but is briefly summarised below. By applying this model to Penrhyn Bay the natural bay shape as a result of the existing groynes can be determined. This information and the tool can be further used to examine potential future defence options where a stable beach is required as part of the coastal defences.

5.2 Model inputs

MepBay operates around two primary parameters, the control line length R_β and the wave obliquity angle β (Figure 16, left). R_β is determined using a Graphical User Interface (GUI) via two control points of locations (X_0, Y_0) and (X_1, Y_1). β is controlled by the tangent of the downcoast shoreline position. This is the natural 0 net drift beach alignment that would be achieved away from the influence of the structure. Although aerial imagery close to Mean High Water Springs (MHWS) has been used throughout the analysis in the GUI (Figure 16, right), comparison to LiDAR was initially made in order to determine whether the diffraction point needed to be modified due to the extent of the submerged part of the groynes. As the submerged section of the northern groyne is significant, the location of the associated diffraction point for this structure was modified slightly from that visible in the aerial imagery.



Source: Hsu & Evans, 1989



Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 16. Sketch of model workings (left) and an example application to Penrhyn Bay (right)

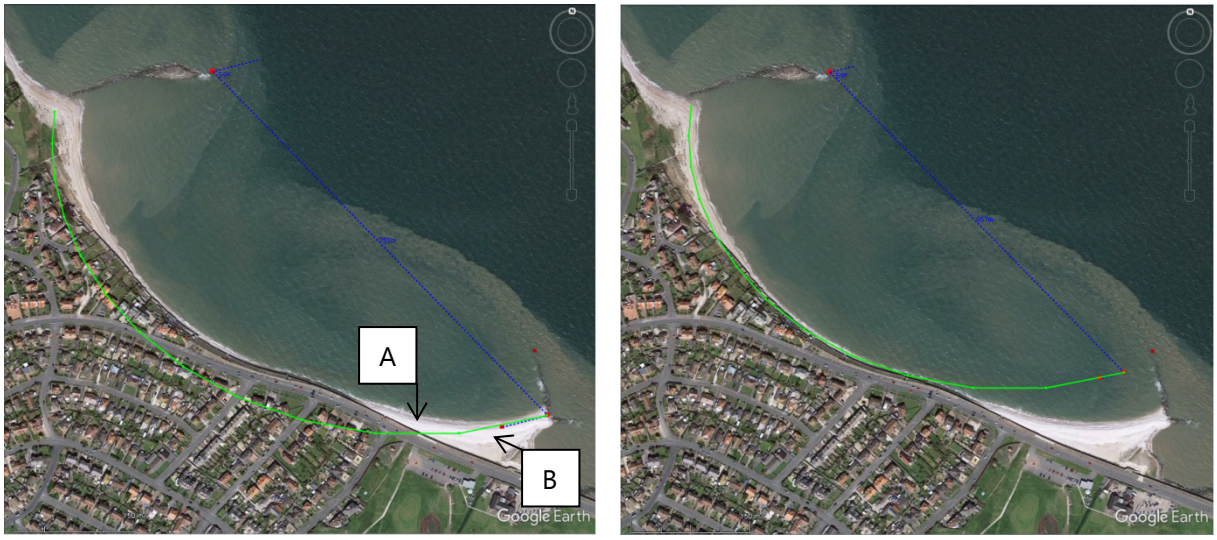
5.3 Estimated shoreline orientations for different wave directions

5.3.1 North-westerly waves

Considering the location of the predominant wave direction within the nearshore (Figure 6), further wave refraction is likely to occur prior to waves interacting with the structures within the Bay. Predominant north westerly waves will experience a clockwise rotation in direction, becoming more north-north-westerly as they reach the embayment. From this direction it is not expected that the southern groyne has any significant sheltering effect on the southern end of the Bay, and therefore a single control point (northern groyne) has been used in this scenario.

Figure 17 (left image) shows the equilibrium bay shape with a downcoast control location and transect line parallel to the berm within the southern section. In this image, the beach at the southern end of the bay is pushed up against the southern groyne, and the equilibrium bay shape through the remainder of bay lies landward of the bay's current position. Whilst this is appropriate for the southern section of the bay it does not represent what is happens in the northern section of the bay. To adjust this for the northern half of the bay, a positional adjustment of the downcoast control point by around 75 m in a northwest direction (Figure 17, right) is required. Having made such an adjustment the equilibrium shape in the northern bay closely matches that of the current berm in the northern half of the bay.

In the adjusted condition (Figure 17, right) the beach in the northern section of the bay is effectively controlled by two features, the diffraction around the tip of the northern groyne and the hard coastal defences in the centre of the bay, thus effectively creating a 'pocket' beach between the central hard defences and the base of the northern groyne. The alignment of this bay is then controlled by the diffraction point at the end of the northern groyne.



Equilibrium line (southern berm) for NW waves

Equilibrium line (northern berm) for NW waves

Figure 17. Single control point application for north-westerly conditions

5.3.2 Northerly waves

As with waves from a north-westerly direction, is it suspected that waves that are northerly at the SEASTATES extraction point, will be refracted further before they reach the embayment, arriving in a more north-north-easterly direction. From this direction the northern groyne has a reduced sheltering effect on the southern section of the Bay (Figure 18), and the southern groyne becomes the main controlling structure at the most southern extent of the beach. Consequently an additional control point has been added for this structure (Figure 18, C).

Figure 18 shows the resultant equilibrium bay shape from these more northerly waves across the whole bay. The two lines do not meet as hard defences in the central section of the beach are also having a controlling influence on the beach. Very little change is noted in the northern half of the beach, but the beach orientation in the southern section is observed to rotate clockwise from that during the north-westerly condition.



Equilibrium lines for N waves (two control points)

Figure 18. Two control point application for northerly conditions

5.3.3 North-easterly waves

The natural orientation of the Bay and nearshore bathymetry results in little wave refraction from north-easterly directions, with waves entering the bay normal to the main orientation of the bay. From this direction both groyne structures have a sheltering effect, and therefore two control points have been retained (Figure 19).

Figure 19 shows the resultant equilibrium bay shape from these more north-easterly waves across the whole bay. Again the two lines do not meet as the hard defences in the central section of the beach are also having a controlling influence. Within the northern section, it initially appears that the beach is rotating into the shelter of the groyne, however, over the length of the beach berm there is still very little change in the beach alignment from that seen for the north-westerly and northerly waves.

In the southern end of the bay the beach, however, the beach orientation is generally observed to be rotated more clockwise than for the other conditions, with a seaward advancement of the berm position closer to the central section of the bay (Figure 19, D), and a small landward recession in close proximity to the southern groyne (Figure 19, E).



Equilibrium lines for NE waves (two control points)

Figure 19. Two control point application for north-easterly conditions

5.3.4 Comparison of equilibrium bay shape

Figure 20 overlays the equilibrium profiles from the three directions considered above. The figure suggests three main points in relation to the existing shoreline:

- In general the northern sections of the bay remains stable, due the existence of the northern groyne, the present of the central hard defences and the volume of material contained in the upper beach over this section of the bay. The groyne structure provides sufficient shelter to the northern section of the beach from the variance in wave climate experienced at the site. As a result, the equilibrium shoreline position within this section has a predicted variance of less than 5 m, which is understood to be consistent with actual variations experienced.

- The defence structures within the central Bay section are held in front of the equilibrium Bay position. This explains why the upper beach in this section experience frequent reinstatement works.

- The southern section of the bay is more volatile, with the beach rotating as result of the incident wave direction. It has not been possible to confirm whether the level of variability predicted, directly reflect that observed, and such a comparison would be complicated by past reinstatement programmes, but these results clearly support the observations that material is relatively rapidly transported to the southern end of the frontage after a reinstatement campaign. But the results also indicate that under north-easterly waves transport can occur in the other direction, moving material back towards the central defences. However, this transport is likely to be significantly slower as the waves from this direction are smaller. Furthermore, the volume of material in the section is limited, and there would be insufficient material to protect the central section defences, even if the beach were to align more to the north-easterly waves.



Figure 20. Comparison of equilibrium berm position for varying wave directions

6 Option Development

Four options have been selected for further consideration from the seven schemes originally proposed in the previous coastal defence assessment (CEUK 2014). These four options have then been developed further into workable conceptual schemes using the results of the process review and the tools in Sections 4 and 5. The four selected options for further consideration are described below; the numbers are as per those in CEUK, 2014:

- **Option 2**: Ongoing beach reinstatement alongside a re-nourishment of the central upper beach with cobble material re-cycled from the southern section of the beach as well as additionally sourced material.

- **Option 3**: A single promontory structure within the central section of the Bay, effectively dividing the Bay into two sub sections, and restricting west-to-east sediment transport. This option potentially provides an opportunity for improved amenity such as better access to the beach from the structure, or a flat promenade area for visitors.

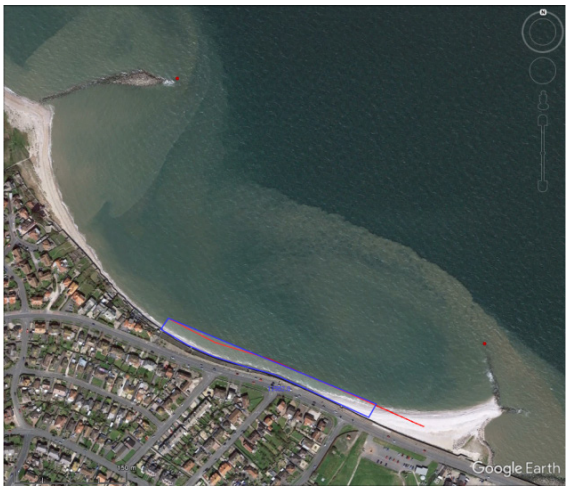
- **Option 4**: A series of rock groynes perpendicular to the existing shoreline position, restricting the west-to-east sediment transport. Longshore access arrangements would need to be incorporated to allow movement of plant machinery used for reinstatement.

- **Option 5**: A detached nearshore breakwater located in the central section of the Bay. This would dissipate wave action prior to entering the defences, reducing the potential for sediment transport within the central section.

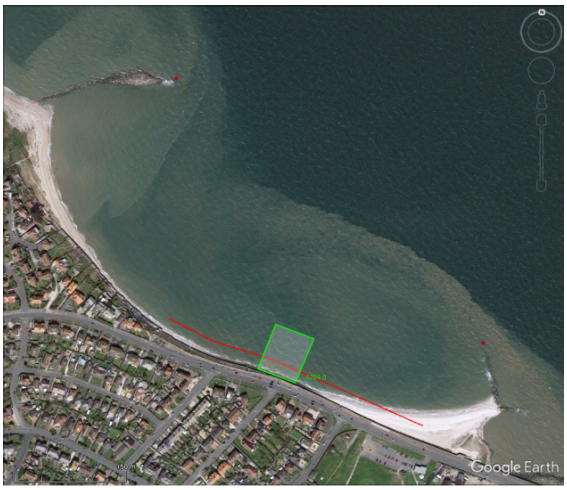
All four options also include the local raising of the landward segment of the southern groyne crest height, to prevent the overspill and loss of beach material from the embayment. Which is considered to be a sensible proposal.

These four schemes, as initially schematised in CEUK 2014, are presented in Figure 21. Included within Figure 21, is a red line, which represent a minimum beach buffer width of 20 m, from the public defence to the proposed initial placement of the beach berm. This minimum berm width has been identified from the assessment undertaken in Section 4, and represents the beach berm width that is likely to be required to allow for the beach berm to regress landwards during extreme storm events based on cross-shore process. Initially it is assumed that this beach berm height would be placed at a level of 6 mODN, beyond which it would slope down at slope of 1:8 until the material meets with the existing seabed. In the future with sea levels rise it is anticipated that this berm height would need to be increased.

For the identified options, this minimum beach width has been used as the starting point when developing the schemes with equilibrium beach positions being examined beyond this minimum beach width. The development of the four options is described in the subsequent sections.



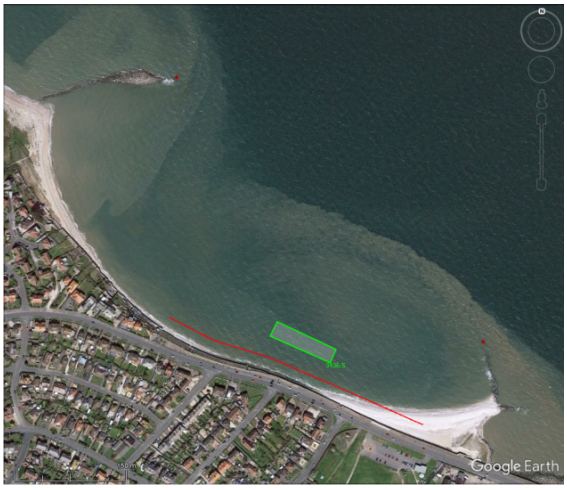
Option 2: Beach re-cycling



Option 3: Promontory structure



Option 4: Rock groynes



Option 5: Nearshore detached breakwater

Figure 21. Overview of the four selected options. A 20 m buffer in beach width displayed as a red line

6.1 Option 2: Ongoing beach reinstatement (utilising larger size material from the southern beach)

Figure 22 shows the location of 20 m minimum beach width along the public defences, together with equilibrium bay shapes that would accommodate this beach width, under both north-westerly waves (left) and north north-easterly waves (right). These bay shapes are shown for information only and are not considered viable as they would require a very significant nourishment campaign and an extension of the southern groyne, both of which are not proposed under this option.

The bay shapes, however, to help to show how the centre of the bay is significantly seaward of the natural alignment of the bay, and therefore that west-east transport of material will be expected across the centre of the bay under the larger more northerly wave conditions. Under more north-easterly waves it is also identified that some east-west transport may occur with material being moved into the lee of the northern breakwater. Here, it may become trapped and not transported back under more northerly waves. However north-easterly waves are smaller and less frequent so the potential for such transport is significantly less.



North-westerly waves North-North-easterly waves

Figure 22. Required berm position to maintain buffer width using Option 1. The area in nourishment and cobbles would be placed is shown within the blue shaded rectangle

The proposed option is therefore very similar to the existing situation and regular beach reinstatement will be required. The additional capital nourishment that is proposed under this option is unlikely to effect the requirement and frequency of the reinstatement works. The use of larger cobble material in the centre of the bay will reduce the west- east transport to an extent, but since the bay alignment is a reasonable distance off the natural bay alignment and significant waves do occur at the site, it is expected that these cobbles will still be transported to the southern end, leaving the centre again exposed.

6.2 Option 3: Promontory structure

For this option a square promontory structure has been analysed for simplicity and it is the typical width of the structure offshore, and the distance of the structure offshore that are important when determining the control the structure will have on the adjacent beaches. For example the trunk section leading from the head of the promontory to the promenade could be significantly narrower.

In terms of maintaining an adequate beach width in front of the solid defences, a promontory structure of similar size to that suggested within the previous coastal defence assessment (CEUK 2014) is likely to block west-to-east sediment transport under more north-westerly and northerly waves, and it will therefore hold a beach of adequate volume to the west of the structure (Figure 23, left). However, to obtain a stable equilibrium bay to the east of the structure, which would protect the solid defences, a large quantity of nourishment would be required for the entire southern section of the beach (Figure 23, left).

Moving the structure eastwards (Figure 23, right) significantly reduces the amount of additional material required in the southern beach section, as the beach crest line moves landwards. In this location the beach to the west of the promontory is still relatively well aligned to the minimum beach width alignment and does not requiring significant amount of additional material to meet the equilibrium stable alignment.

However, having moved the structure eastward, its length will no longer contain the amount of material required in the beach profile and therefore west-to-east bypassing of material around the structure head is likely unless the length of the structure is increased.



As located in CEUK, 2014



Located further east

Figure 23. Promontory structure option development

Figure 24, left, shows and alternate scheme with the head of the promontory now further offshore. This option should be more than sufficient to hold the beach to the west, with limited bypassing, and it also provides greater sheltering to the east so the volume of nourishment required for the southern section of the beach reduced further. Ultimately it is likely that an optimal distance offshore would lie somewhere between the two and a further scheme (Figure 24, right), has been developed with the promontory distance offshore being 80 m in length.

Figure 23 and Figure 24, include the equilibrium bay shapes under north north-easterly waves. These are the more dominant waves at the site, both in respect to frequency and size, and it is these waves that generate the west – east transport that is experienced in the bay. Figure 25, however show the equilibrium beach alignment for the extended offshore promontory structure for north-easterly waves. To the east of the promontory the beach is nicely contained, however, to the west, significantly more beach nourishment would be required to acquire a stable equilibrium bay shape that would provide the minimum beach width along the public defences. Thus under north-easterly waves there is potential for east-west transport that will take material from the centre of the bay to the lee of the

northern breakwater. Here, it may be effectively trapped and not then transported back under more northerly waves. However, waves from the northeast are significantly less frequent and smaller in size and will have significantly less potential to transport material.

From this assessment the concept layout for this scheme, in respect to location and extent, for the promontory, is that shown in Figure 24, right. In the layout the offshore width of the promontory is around 65 m, and lies around 80 m seaward of the existing seawall. Under this option some beach nourishment will be required for the southern beach section in order to build out to the equilibrium beach position, but then on-going reinstatement/recycling work within this section is likely to be minimal.

To the north of the promontory it is proposed that the beach material is initially built out as shown in Figure 24, right. Under north-easterly waves it is recognised that some east-west transport could occur removing beach material from the central section, however, the rate of this transport is expected to be relatively low and the beach is expected to remain relatively stable, being held by the promontory under northerly waves. Some limited – on-going reinstatement/recycling works, however, should be allowed for, particularly to deal with possible losses to the north.



Offshore distance increased



Offshore distance 80 m

Figure 24. Promontory structure option development – increase distance offshore



Offshore distance increased

Figure 25. Promontory structure under north-easterly waves

6.3 Option 4: Rock groynes

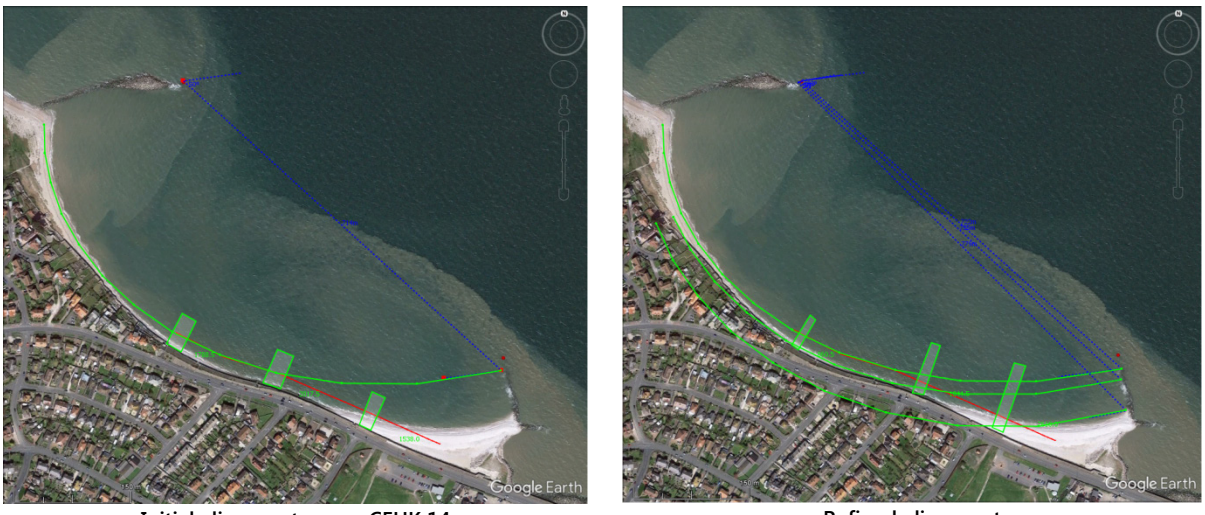
The initial coastal defence assessment (CEUK, 2014), schematised a scheme with three rock groynes located, equally spaced, within the central section of the bay and around 50 m in length. The aim of these groynes is to restrict sediment transport and retain material within the central section of the Bay.

An initial attempt to align beaches within this schematised groyne field, (Figure 26, left), identified a number of issues:

- The most western groyne may not be adding significant value;
- The most eastern groyne would need to be significantly longer;
- Large volumes of additional nourishment may be required in the central groyne bay and southern beach section to meet the equilibrium bay shapes; and
- All groyne lengths are likely to have to be longer to hold the toe of the beach.

A modified groyne field was therefore developed in order to try to optimise the length of the structures and the volumes of material required. This layout is shown in Figure 26 (right). By adjusting the position and relative spacing of the groyne the minimum beach width is achieved using significantly less beach material, and the beach movements within the groyne bays will be relatively well controlled. The most western groyne bay is included within this option, but for the reasons set out in Section 6.2, it may not be adding significant benefit. Indeed, to make the scheme compatible to the concept layout for the promontory option is could be removed. This may result in some additional reinstatement requirements, but these are expected to be small and will be of the same order of that for the promontory option.

The length of the three groynes in Figure 26 (right), from west to east, are 50 m, 65 m and 90 m respectively. It is assumed that height of the groynes along it length with be variable, reducing in height offshore in-line with the renourished beach profile, but always around 1-2 m above the height of the renourished beach.



Initial alignment as per CEUK 14 Refined alignment

Figure 26. Groyne layout within Option 4

6.4 Option 5: Detached breakwater

The proposed breakwater option, as schematised in CEUK 2014, was initially examined using the MepBay tool. The length of this structure is approximately 100 m, and it is located 55 m offshore. The result of this assessment is shown in Figure 27 for both north north-westerly (left) and north-easterly waves (right). This initial schematisation is observed to work reasonably well, but would require significant additional nourishment in the southern section of the beach and in the lee of the breakwater to achieve the stable planform. Other modifications to this layout were examined, including moving the breakwater further east and reducing its length to try and reduce the requirement for nourishment. However, these are not considered to work effectively as the toe of the beach to the east of the breakwater starts to extend beyond the structure.

As with the promontory and groyne options, structures toward the centre of the bay hold material from traveling west-east under northerly waves and the required nourishment has been considered purely on this basis. However, under the less frequent and smaller north-easterly waves, there is still a tendency for east – west transport of the nourished material in the centre of the bay. Whilst this transport is expected to be small, it may result in the occasional requirement for reinstatement/recycling campaigns to replenish material into the centre of the bay.

In this assessment it is assumed that the breakwater is surface piercing at all tidal states and would not be significantly overtopped. It this is not the case significant retreat of the beach in the lea of the breakwater could occur.



North north-westerly waves



North-easterly waves

Figure 27. Proposed nearshore breakwater

A further feature of the detached breakwater option, is that it provides areas of water that appear calm in the lee of the structure. However, under wave conditions these areas are subject to significant currents, which are directed offshore. This can have significant implications for bather/public safety and needs to be considered.

7 Summary and Recommendations

A review of coastal processes within Penrhyn Bay has confirmed the general understanding set out in the Coastal Defence Review undertaken by CEUK (CEUK, 2014). The four preferred options from this review have been further refined into concept schemes, using information on expected cross-shore mobility and longshore variability. The later has been examined by the consideration of equilibrium bay shapes.

The four concept schemes and their derivation are described in Section 6, which also provides an indication of the likely on-going beach reinstatement work that may be required for each option. For each option a significant level of beach nourishment has been identified to ensure that minimum beach width is maintained in front of the defences.

For which every scheme is taken forward, consideration will need to be given to public safety and beach access and the added amenity values that may be added by the scheme. Further detailed studies should also be undertaken during detailed design to refine the concept schemes developed during this assessment.

8 References

ABPmer, 2012. Colwyn Bay Waterfront Development Phase 2. Beach Modelling. ABP Marine Environmental Research Ltd, Report No. R.2031.

ABPmer, 2013. SEASTATES Wave Hindcast Model: Calibration and Validation Report. ABP Marine Environmental Research Ltd, Report No. R.2145.

British Maritime Technology, 1987. Aberconwy Coastal Study Parts I-V. Report No. 4627702

CEUK, 2014. Penrhyn Bay Coastal Defence Assessment. Coastal Engineering UK Ltd

Environment Agency CFBD, 2011. Coastal flood boundary conditions for UK mainland and islands.Project: SC060064/TR4: Practical guidance design sea level

Fontoura Klein, A.H., Vargas, A., Alice Raabe, A.L. & Hsu, J.R.C. 2003. Visual assessment of bayed beach stability with computer software. Computers & Geosciences. 29. 1249-1257 pp.

Halcrow, 2011. North West England and North Wales Shoreline Management Plan SMP2

Halcrow, 2012. North West England and North Wales Shoreline Management Plan SMP2 Supporting Studies- Joint Probability Study: Extreme Wave Heights and JOIN_SEA results

HR Wallingford, 1991. Colwyn Borough Sea Defence Review. Report No. EX2367.

HR Wallingford, 2000. Conwy County Borough Council. Post Project Appraisals. Performance Evaluation: Penrhyn Bay Coastal Works

HR Wallingford, 2005. Rhos-on-Sea Breakwater Siltation Study, Colwyn Bay. Advice on managing sediment accumulation. Report EX 4992.

HR Wallingford, 2016. Modelling shingle beaches in bimodal seas. Development and application of Shingle-B. Report RT002.

Hsu, J.R.C. & Evans, C. 1989. Parabolic bay shapes and applications. Proceedings of the Institute of Civil Engineers Part 2. 87. 557-570 pp.

Liverpool Bay Coastal Group, 1999. Liverpool Bay Shoreline Management Plan, subcell 11a. Great Ormes Head to Formby Point.

Myers, J.A.L., 1993. West Coasts of England and Wales Pilot: Wests Coasts of England and Wales and South Coast of Scotland from Cape Cornwall to Mull of Galloway including Isle of Man. 12th ed. Taunton: Hydrographic Office. 323 pp.

United Kingdom Hydrographic Office, 2016. Admiralty Tide Tables.

Welsh Government Adapting to Climate Change Guidance, 2011.
http://gov.wales/docs/desh/publications/111231floodingclimatechangeen.pdf

9 Abbreviations/Acronyms

ABPmer	ABP Marine Environmental Research Ltd
Arup	Ove Arup and Partners Limited
ATT	Admiralty Tide Tables
CCBC	Conwy County Borough Council
CEUK	Coastal Engineering UK Ltd
CFBD	Coastal Flood Boundary Conditions
CFSR	Climate Forecast System Reanalysis
GUI	Graphical User Interface
HAT	Highest Astronomical Tide
Hm_0	Significant wave height (integral analysis)
Hs	Significant wave height (wave by wave analysis)
HTL	Hold the Line
JPA	Joint Probability Analysis
LAT	Lowest Astronomical Tide
LiDAR	Light Detection and Ranging
mCD	metres above Chart Datum
MHWN	Mean High Water Neaps
MHWS	Mean High Water Spring
MHWS	Mean High Water Springs
MLWN	Mean Low Water Neaps
MLWS	Mean Low Water Springs
mODN	metres above Ordnance Datum Newlyn
MSL	Mean Sea Level
NOAA	National [American] Oceanic and Atmospheric Administration
PAR	Project Appraisal Report
RSL	Relative Sea Level
SMP2	Shoreline Management Plan
Tp	Peak Wave Period
UK	United Kingdom
UKCP09	UK Climate Projections - UKCP09

Cardinal points/directions are used unless otherwise stated.

SI units are used unless otherwise stated.

Appendix



A Joint Probability Analysis

Extreme wave Joint Probability Analysis (JPA) for extraction Point 7 close to Penrhyn Bay at a depth of -14.5 mODN. Location Point 282858E, 383715N (British National Grid).

1-year Joint RP		2-year Joint RP		5-year Joint RP	
W. Level (mODN)	H_{m0} (m)	W. Level (mODN)	H_{m0} (m)	W. Level (mODN)	H_{m0} (m)
0.00	3.39	0.00	3.71	0.00	4.15
2.00	3.37	2.00	3.68	2.00	4.12
2.60	3.25	2.60	3.59	2.60	4.02
2.73	3.20	2.87	3.50	3.10	3.81
3.10	3.05	3.10	3.40	3.40	3.63
3.40	2.95	3.40	3.20	3.56	3.50
3.49	2.90	3.40	3.20	3.75	3.34
3.75	2.73	3.75	2.95	3.88	3.20
3.96	2.50	3.82	2.90	4.10	2.94
4.10	2.34	4.10	2.63	4.15	2.90
4.27	2.10	4.20	2.50	4.37	2.64
4.40	1.83	4.40	2.17	4.47	2.50
4.50	1.60	4.44	2.10	4.66	2.10
4.72	1.00	4.65	1.60	4.80	1.60
4.82	0.00	4.81	1.00	4.93	1.00
10-year Joint RP		20-year Joint RP		50-year Joint RP	
W. Level (mODN)	H_{m0} (m)	W. Level (mODN)	H_{m0} (m)	W. Level (mODN)	H_{m0} (m)
0.00	4.41	0.00	4.71	0.00	5.15
2.00	4.37	2.00	4.68	2.00	5.13
2.60	4.29	2.60	4.58	2.60	4.97
3.10	4.13	3.10	4.35	3.10	4.74
3.40	3.95	3.40	4.23	3.40	4.56
3.75	3.62	3.75	3.94	3.71	4.33
3.82	3.50	4.00	3.56	4.14	3.77
4.09	3.20	4.13	3.40	4.34	3.50
4.19	3.07	4.26	3.24	4.40	3.41
4.35	2.90	4.40	3.11	4.54	3.20
4.37	2.85	4.53	2.90	4.73	2.90
4.62	2.50	4.77	2.50	4.92	2.50
4.79	2.10	4.89	2.10	5.05	2.10
4.91	1.60	5.01	1.60	5.14	1.60
5.03	1.00	5.13	1.00	5.27	1.00
5.11	0.00	5.21	0.00	5.32	0.00
100-year Joint RP		200-year Joint RP			
W. Level (mODN)	H_{m0} (m)	W. Level (mODN)	H_{m0} (m)		
0.00	5.46	0.00	5.73		
2.00	5.39	2.00	5.62		
2.60	5.31	2.60	5.54		
3.10	5.08	3.10	5.39		
3.46	4.82	3.47	5.09		
3.68	4.63	3.75	4.84		
4.15	4.00	4.10	4.35		
4.40	3.69	4.43	3.90		
4.54	3.50	4.71	3.50		
4.71	3.20	4.86	3.20		
4.88	2.90	5.02	2.90		
5.05	2.50	5.16	2.50		
5.17	2.10	5.26	2.10		
5.26	1.60	5.35	1.60		
5.35	1.00	5.40	1.00		
5.39	0.00	5.47	0.00		

Source: Halcrow, 2012

ABPmer
Quayside Suite, Medina Chambers, Town Quay, Southampton SO14 2AQ

T +44 (0)23 80 711840
F +44 (0)23 80 711841
E enquiries@abpmer.co.uk

www.abpmer.co.uk



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Appendix B

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B1

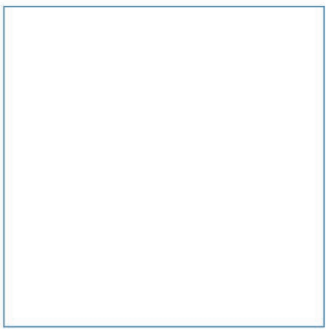
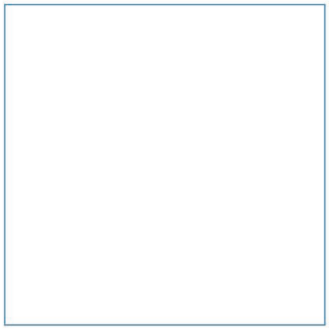
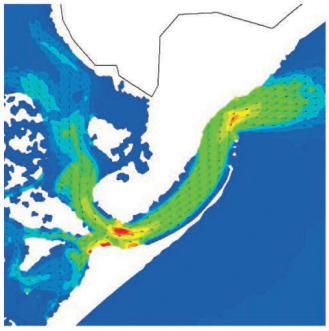
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Contributing Authors

David Finch, Tom Finch

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ABPmer
Quayside Suite, Medina Chambers, Town Quay, Southampton, Hampshire SO14 2AQ
T: +44 (0) 2380 711844 W: http://www.abpmer.co.uk/

Summary

ABPmer has been commissioned by Arup to support the design development of new coastal defences at Penrhyn Bay. The work is being commissioned on behalf of Conwy County Borough Council (CCBC).

A local MIKE21 Spectral Wave (SW) model has been built for this study. Primarily, the local model has been used to examine and refine a proposed scheme layout as recommended by the OBC. The local SW model has also been used to transform key Joint Probability (JP) wave and water level pairings inshore; wave conditions have subsequently been extracted at the toe of the proposed structure to help inform design considerations.

The scheme refinement consists of the testing of 4 scheme layouts using waves from three approach directions. The results of have been processed to allow beach shapes of 10 m and 20 m beach buffers to be interpreted. This part of the study provides model predictions of how each scheme layout will impact the shape and volume of the beach, with the exposed seawall within Penrhyn Bay being the focal point for protection.

From the assessment, and through discussion with ARUP and CCBC, Layout 3 with the shore-connected structure is identified as the preferred layout. It is further considered that a minimum beach buffer width of around 10 m will be sufficient to provide protection to the seawall. This layout is considered to perform well against the study performance criteria and will create relatively stable beach plan shapes in both the eastern and western embayments.

The Joint Probability Analysis (JPA) has been carried out in two stages. Firstly, determination of the extreme water levels and wave heights as univariate extremes; and secondly, the JPA itself. The analyses were completed for the present-day epoch (2020); for future epochs (2070 and 2120); water level, wave height and wind speed allowances were uplifted to account for agreed climate change projections using relevant Welsh Government guidance. From the nearshore data extraction of the JP model runs, significant wave height (Hs) is shown to increase from epoch to epoch, which is as expected given the increase in climate change allowances as the epochs progress. Both direction sectors (N and WNW) show all five extraction locations to have similar values of Hs within each run, with differences in Hs ranging by 0.1 to 0.2 m between extraction locations in each run scenario.

Tide and wave driven flow modelling has also been undertaken under a series of extreme wave and water level events to examine the flow regime with the proposed scheme in place. These model results are also compared against the present-day baseline case.

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1 Introduction

ABPmer has been commissioned by Arup to support the design development of new coastal defences at Penrhyn Bay. The work is being commissioned on behalf of Conwy County Borough Council (CCBC). To provide overarching context to the study, the Shoreline Management Plan 22 policy for this stretch of coastline is hold the line for short- to long-term.

The Outline Business Case (OBC) concluded there was a need to maintain beach levels in the Bay to protect the existing defences. The existing defences provide coastal erosion and tidal flood protection and failure would affect a significant number of residential and commercial properties and utilities infrastructure. The beach is currently maintained by beach management on an annual to 18-month basis and the OBC recommended the construction of a new rock beach control structure in the centre of the Bay to reduce the frequency of beach management activities.

The OBC was supported by a coastal processes assessment which included an examination of the equilibrium bay shape using MepBay and beach profile variation using Shingle-B. The present assessment builds upon the previous studies and uses a local MIKE21 Spectral Wave (SW) model to examine and refine the proposed structure layout. This SW model is also coupled to a hydrodynamic (HD) model, to assess the combined wave/current flow speeds across the study area.

In addition, a Joint Probability Analysis (JPA) has been undertaken, to derive offshore extreme wave and water level conditions relevant to the site. The MIKE21 SW model has subsequently been used to transform these conditions inshore to the toe of the proposed beach control structure to support the design of the structure.

A separate Beach Management Strategy sets out the design criteria for the beach focusing on beach width and trigger points for intervention.

To provide the outputs of the Penrhyn Design Development study, this report is structured, as follows:

Section 2; Describes the set-up of the numerical model mesh for the SW and coupled HD+SW model runs;

Section 3; Details the work undertaken to test a range of layout options attempting to refine the proposed scheme design;

Section 4; Provides the outputs for the extreme wave and water level assessments, along with the Joint Probability Analysis and the subsequent wave transformation modelling; and

Section 5; Explains the approach taken to couple the HD and SW models, along with an assessment of the influence of the scheme on combined wave/current flow speeds.

A full range of associated model outputs, along with other supporting data, are provided in extensive appendices.

2 Model Mesh Construction

2.1 Model domain

To determine the extent and orientation of the model domain, an assessment of ABPmer's regional wave hindcast model was completed. This assessment took into consideration: bathymetry, prevailing wind and wave directions (Figure 1) and any bathymetric features that could potentially influence the local flow and wave climate. The resulting model domain extends offshore, to beyond the -20 m(ODN) contour into relatively deep water where the incident waves will be relatively uniform.

The model domain has also been orientated slightly to a NW direction to suit the prevailing wave conditions (Figure 1), and extends laterally to allow space for waves propagating from the range of directional sectors that are of importance for this study (WNW – NNE).

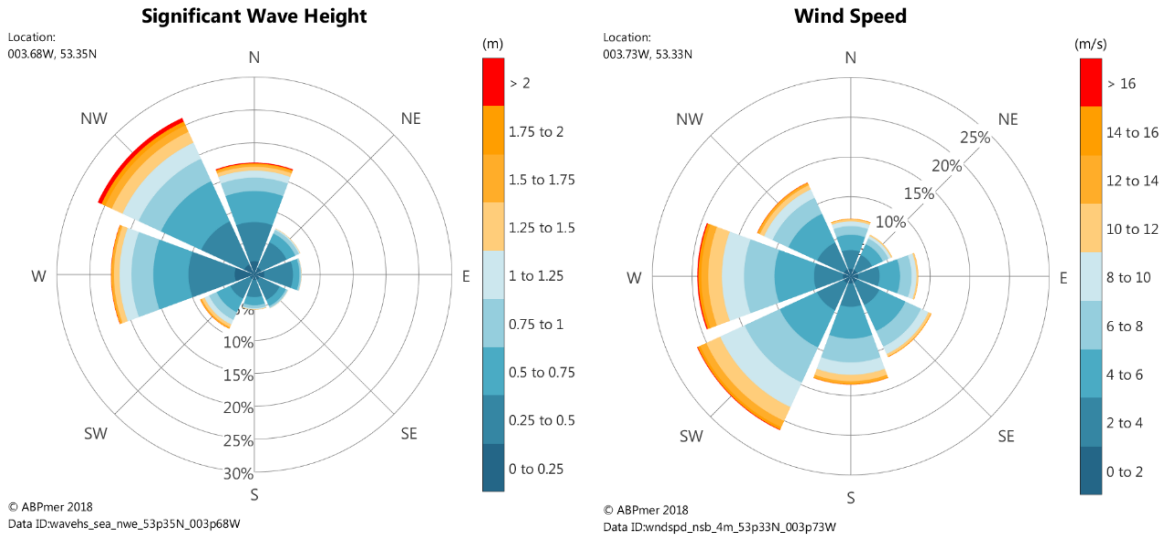


Figure 1. Wave and wind roses extracted from SEASTATES offshore of site

The full model domain and detail of nearshore model mesh is shown in Figure 2. Penrhyn Bay has the finest mesh resolution of approximately 5 m, with the grid coarsening out to the model extents where the grid resolution is approximately 950 m.

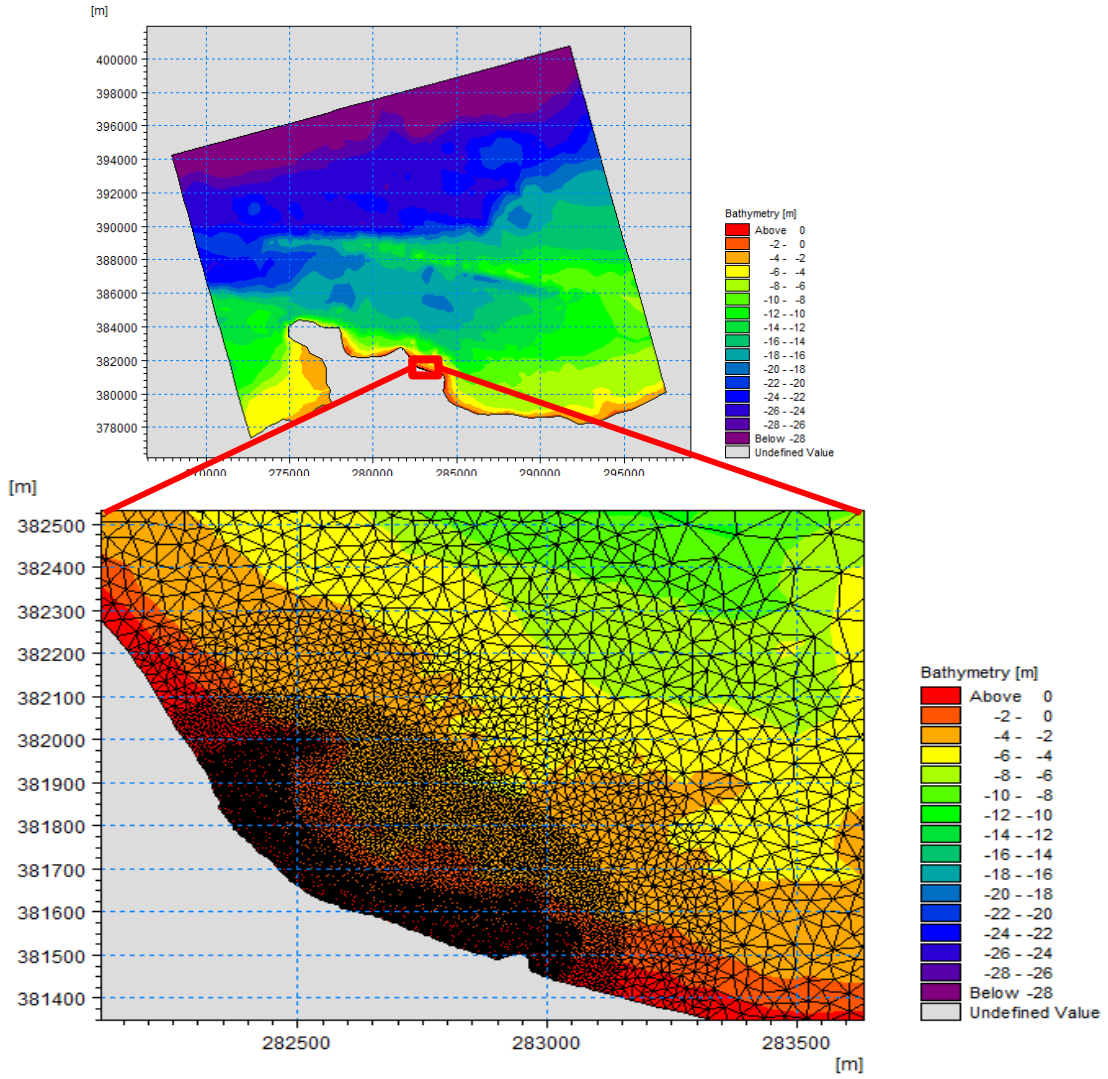


Figure 2. MIKE21 SW Model domain, Full extent and detail of site (showing model mesh)

2.2 Model bathymetry

Once the model domain was constructed, the bathymetry / topographic data was collated and interpolated onto the grid, once all datasets were converted to consistent projections (OSGB and mODN are used here). The data sources used to populate the model domain consist of:

- Composite and historic LiDAR data through the Welsh Government Lle data portal;
- Detailed 2019 Multibeam data from UKHO Admiralty Marine Data Portal; and
- Digital Admiralty Chart data from C-MAP (through ABPmer's DHI MIKE21 License).

Data used was prioritised in the order identified above.

3 Scheme Refinement

3.1 Modelling approach

For the scheme refinement assessment, the MIKE21 SW model has been used to examine the beach plan shape that would be induced as a result of diffraction around the existing layout and following implementation of the proposed offshore breakwater structure. As the seabed in the lee of the structures, and the nourished material to be placed as part of the scheme, is mobile, this material will wish to realign behind any new structure until a new equilibrium position is obtained. This equilibrium position is affected by the incident wave direction and the resultant wave diffraction behind the structure.

The initial step was to create a copy of the model bathymetry and deepen/ flatten Penrhyn Bay from the seawall, down and out to the -1.4 mODN contour. In doing so, it allowed the model to show the diffracted wave patterns caused solely by the existing structures, since all other nearshore bathymetric features and processes, (including refraction), had been removed.

Wave events representative of a 1-in-1-year event, from three directional sectors (WNW, NNW and NNE) were then modelled for the existing condition, with the deepened inshore bathymetry, to show the diffraction patterns and bay shapes that would be expected based purely on the existing structures at the site. This validation exercise was undertaken to prove the validity of the approach and model setup and is detailed in Section 3.2.

Following the model validation, four scheme layouts were tested iteratively, meaning each consecutive layout was an amendment or refinement on the preceding scheme design(s). This modelling is detailed in Section 3.3. Structures within Penrhyn Bay, representing both 'existing layout' and 'with scheme refinements', were incorporated into the model within the bathymetry and as 'structure' features within the model set-up. The 'structures' feature is used to restrict waves from being able to cross the 'structure', even where it could possibly be overtopped at some states of the tide. Further details are provided in Sections 3.2 and 3.3.

Within the assessment, wave model results from each model run were processed into a QGIS layer where the incident wave direction could be rotated to show the orientation of the local wave crests. These wave crests were then interpreted to define the beach shape that is likely to result from the structures under the incident wave direction being considered. For each scheme layout, beach crest positions were then identified allowing for both a 10 and 20 m beach buffer along the section of seawall to be protected. In this assessment the buffer width of 20 m was initially chosen, as this was identified with in the OBC assessment as the indicative buffer width that may be required if the beach is to provide the primary defence for the frontage. However, as the beach is not required to provide the key flood protection function, but instead is being included to provide protection to the sea wall, it was considered that a smaller buffer width may be acceptable. Hence, a buffer width of 10 m was adopted. This is close to the upper beach crest width of 8 m that has been used recently when reinstating the beach as part on the regular on-going maintenance programme.

The results of each scheme layout are discussed in Section 3.3.

When presenting the results from the plan-shape modelling, the following symbology is used:

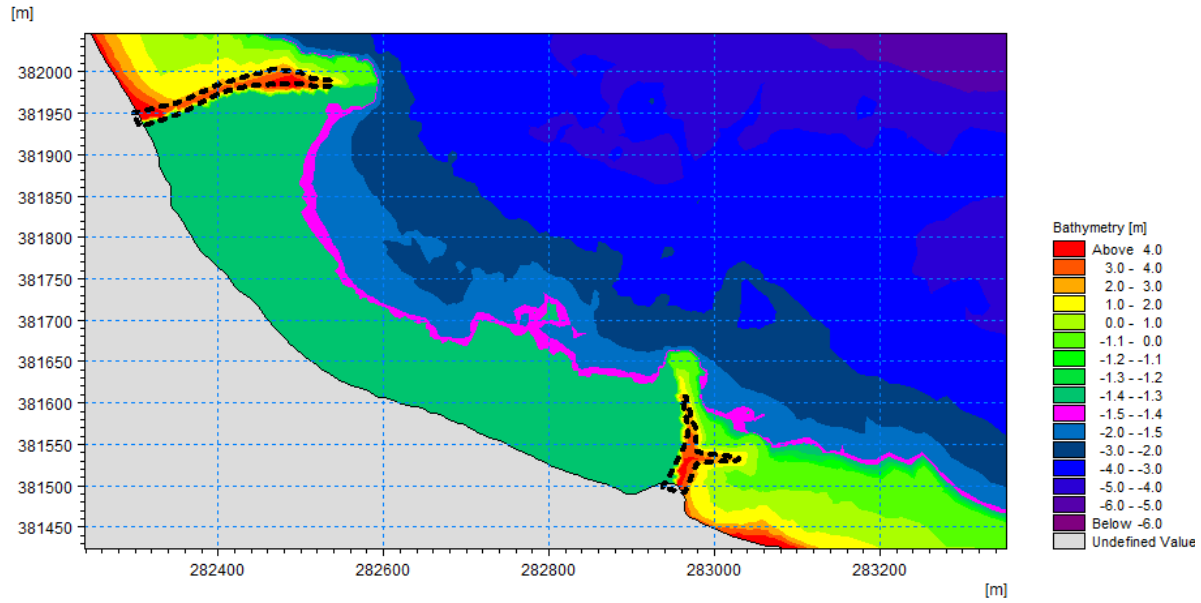
- Yellow: beach crest position, relates to waves from WNW.
- Blue: beach crest position relates to waves from NNW.
- Red: beach crest position relates to waves from NNE.

For the scheme assessment solid beach contours represent the beach crest position for a 20 m wide minimum buffer and dashed or dotted lines are used for the beach crest position for a 10 m wide minimum buffer width.

3.2 Model validation

3.2.1 Model setup

The model validation runs all adopt an amended present-day bathymetry, which consists of the inshore bed deepened to -1.4 mODN, shown in Figure 3.



(Black dashed lines represent the 'structure' features defined in the model)

Figure 3. Flattened nearshore bathymetry of the present-day scenario

The validation model runs were undertaken for three incident wave directional sectors, WNW, NNW and NNE, commensurate with the MepBay study at OBC.

The wave events used are representative of the 1-in-1-year wave event derived from an extreme value analysis of 2 directional sectors: N and WNW. The characterisation of the wave sectors used in the extremal analysis is described in Section 4.2.1. For the NNW and NNE direction sectors, the 1-in-1-year wave condition was taken from the analysis of the northerly sector waves. For the WNW direction sector, the 1-in-1-year wave condition was taken from the analysis of the WNW sector waves. The wave events, and their corresponding wind speeds (Ws), are outlined in Table 1.

Table 1. 1-in-1-year wave events used for the model validation.

Directional Sector	Hs (m)	Tp (s)	Ws (m/s)	Hs and Ws Direction (degrees)
WNW	4.64	9.0	18.7	300
NNW	3.83	8.1	16.5	337
NNE	3.83	8.1	16.5	11

Model runs were undertaken using a constant water level (WL) of 3.74 m(ODN) which is representative of Mean High-Water Spring (MHWS) at Penrhyn Bay.

Although the present-day structures are well represented in the model bathymetry/ topography, they have also been included as a 'structure' in the model set-up. Whilst the crest heights of the west and east groynes are approximately 4.7 m and 4.3 m(ODN) respectively, some sections of the structures are lower than this and, at high water levels, waves can sometimes pass over the structure. With waves passing over the structures this will affect the wave crest in the lee of the structure. Whilst this is a real feature, for most of the time the structures are surface piercing and waves are prevented from passing over them. As the bay shape/ shoreline position in the bay is more a function of conditions when waves are not overtopping the structures, 'structures' have been defined in the model set-up to prevent any wave energy passing over the groynes. Along both groynes, a 'structure' is defined in the model as a polyline across which no wave transmission is allowed. The 'structures' included in the baseline mode are shown in Figure 3 as the black dotted lines.

3.2.2 Validation results

A typical model output for one of the 1-in-1-year wave events (the NNW event) is shown in Figure 4. This figure shows the incident wave heights and scaled wave vectors representing the incident wave direction. These results were processed in QGIS to show local wave crest orientations, which were then assessed to derive the predicted beach crest positions.

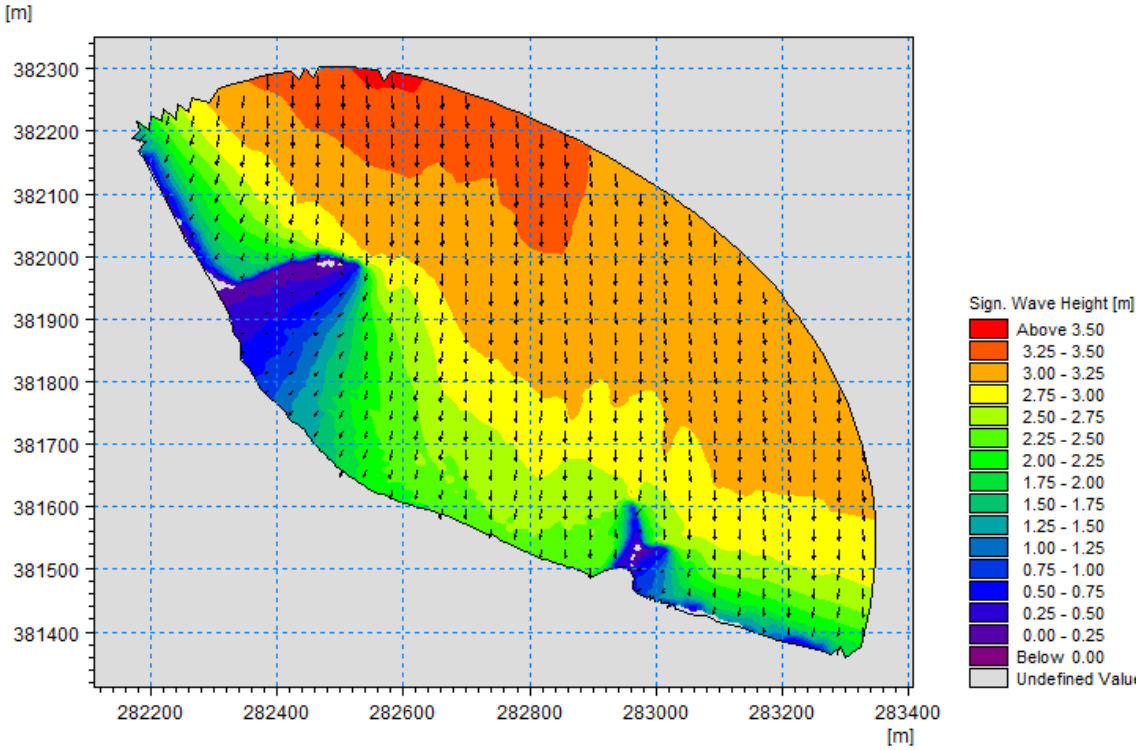


Figure 4. Example wave vectors from 1-in-1-year NNW wave event, under the present-day scenario with deepened nearshore bathymetry

The wave crests, and interpreted beach crest positions for each of the three directions, are shown in Figure 5 to Figure 7. These results are overlaid with an aerial image that shows the beach crest position at the time the image was taken. The results are also compared against one another and the beach-shape contours derived from the earlier MepBay study (ABPmer, 2017a) in Figure 8.

From Figure 5 to Figure 8, the beach crest positions estimated from the wave crests are shown to replicate the observed shoreline well. The variability of the estimated beach crest positions also aligns closely to the MepBay results. The study approach and model setup are therefore considered appropriate for assessing the expected beach plan shapes for the proposed scheme variants.



Figure 5. **Wave crests representative of a 1-in-1-year wave event from WNW under the present-day scenario**



Figure 6. **Wave crests representative of a 1-in-1-year wave event from NNW under the present-day scenario**



Figure 7. **Wave crests representative of a 1-in-1-year wave event from NNE under the present-day scenario**



Figure 8. **Comparison of the beach crest positions alongside the MepBay results (insert)**

3.3 Scheme assessment

Following the validation exercise described in Section 3.2, four alternative schemes have been assessed. The schemes have been assessed iteratively for the same three waves conditions used in the validation exercise. The water level used was the present day (2020), MHWS level of 3.74 m ODN as used in the validation exercise.

Each scheme was examined in turn, with the results from the scheme being used to inform the layout of the next scheme to be assessed. The structure layout details and summary of the results from each layout are described in Sections 3.3.1 to 3.3.4. The modelled wave crests and derived beach crest position for each layout and incident wave event are presented Appendix A to D.

3.3.1 Layout 1

Layout details

Layout 1 was developed from CAD drawing 252256-ARP-DRG-00001.dwg, provided by Arup on 7 May 2020. The scheme consisted of a shore-connected, shore-parallel breakwater, the general dimension and position of which was very similar to the promontory scheme identified at OBC. The crest of the shore-parallel breakwater section is approximately 70 m long with an elevation of 5.41 mODN. The breakwater section is situated around 75 m from the existing shoreline.

The outer shore-parallel breakwater was digitised and included within the model bathymetry as shown in Figure 9. Whilst the crest of the structure is higher than the modelled water level, a 'structure' was also defined along its length ensuring no waves could pass through it.

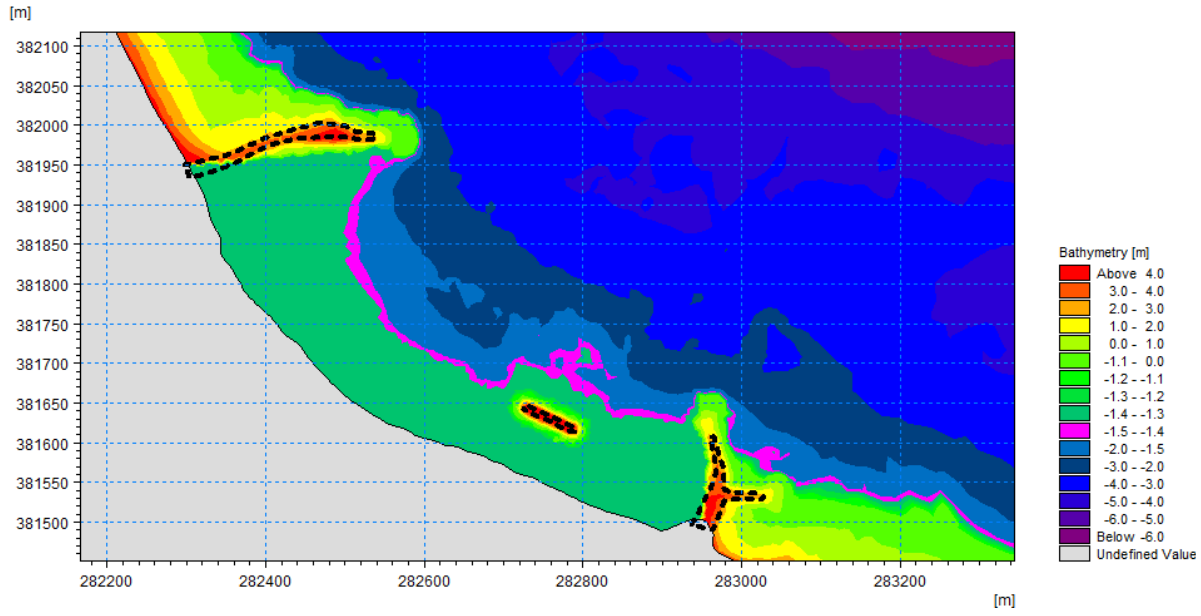


Figure 9. Model mesh for Layout 1 with shore-parallel breakwater included (black dashed lines represent the 'structure' features defined in the model)

Whilst a shore connection was included in the proposed layout (Drawing 252256-ARP-DRG-00001.dwg), this was excluded from the model run, so the diffraction pattern could be examined without it present. In reality, a tombolo or salient will form naturally behind the breakwater, which would provide some connection to the breakwater via the beach. For this initial simulation, the test was undertaken without it present to help ascertain if this would be the case.

Derived beach crest positions

The wave crests developed from the modelling and the derived beach crest positions are shown in Appendix A for each of the three direction sectors. For each direction sector, beach crest positions were derived assuming a minimum beach buffer requirement of 20 m and 10 m along the frontage.

The resultant beach crest positions are summarised in Figure 10.



Figure 10. **Layout 1 beach crest positions, for 10 m minimum buffer (top) and 20 m minimum buffer (bottom)**

The upper plot shows the beach crest position with a 10 m beach buffer and the bottom plot with a 20 m buffer. To help develop the beach crest position, these plots also show the extent of the critical section of seawall that required protection (green line), and an associated 20 m offset line (black dashed line). This latter line defines the minimum beach buffer width that is required for the 20 m buffer case.

Key observations

Key observations from the Layout 1 model results are identified below.

- The salient wishes to attach to the breakwater for both buffer widths and for all direction sectors.
- A greater nourishment volume would be required for the 20 m buffer width scenario over the 10 m buffer width scenario.
- At the western end of the area to be protected, the beach crest position lies seaward of the defence and, therefore, additional nourishment material would be required to enable the shoreline positions to tie in with the beach to the west. Again, a larger quantity of material would be required here for the 20 m buffer width scenario over that for the 10 m buffer width scenario.
- The toe of the shingle beach adjacent to the breakwater is likely to extend beyond the main slope of the breakwater for many of the beach crest positions. This is because the distance between the nourished beach crest and beach toe will be around 50 m as identified at OBC. If material extends beyond the structure it is much more likely to be lost from the bay.
- Under NNE waves, the beach crest position against the trunk of the eastern fishtail groyne is very narrow. It is noted that shingle already overtops this groyne at this location under extreme waves, which means material is being lost from the bay.
- The toe of the breakwater lies very close to one of the Welsh Water outfall pipes.
- The 'pinch-point '(the point at which the minimal buffer width is achieved) in the eastern embayment lies slightly to the east of the short Welsh Water outfall.

3.3.2 Layout 2

Model layout

Based on the results of Layout 1, Layout 2 was developed by locating the shore-parallel breakwater section a further 25 m offshore. This was undertaken to help ensure that the beach material would be contained landward of the breakwater. To prevent the breakwater toe from getting closer to the Welsh Water outfall, it was also migrated west slightly. The dimension of the breakwater remained unaltered and was, therefore, effectively repositioned 5 m west and 25 m north. The breakwater was then located approximately 90 m offshore from the existing shoreline.

It should be noted that the seabed is relatively shallow sloping at this location; therefore, the overall height of the breakwater above the seabed may not be significantly greater that than of Layout 1. However, it is further offshore and any shore-connected structure would therefore need to be longer.

The modified model bathymetry is shown in (Figure 11). For this layout, a 'structure' was also defined in the model to represent the shore-connected structure (Figure 11). This was done as it was more likely that a tombolo would not be formed, and the inclusion of the structure would allow the diffraction associated with both the eastern and western ends the shore parallel breakwater to be more clearly defined.

Layout 2 was again examined for the same three wave conditions.

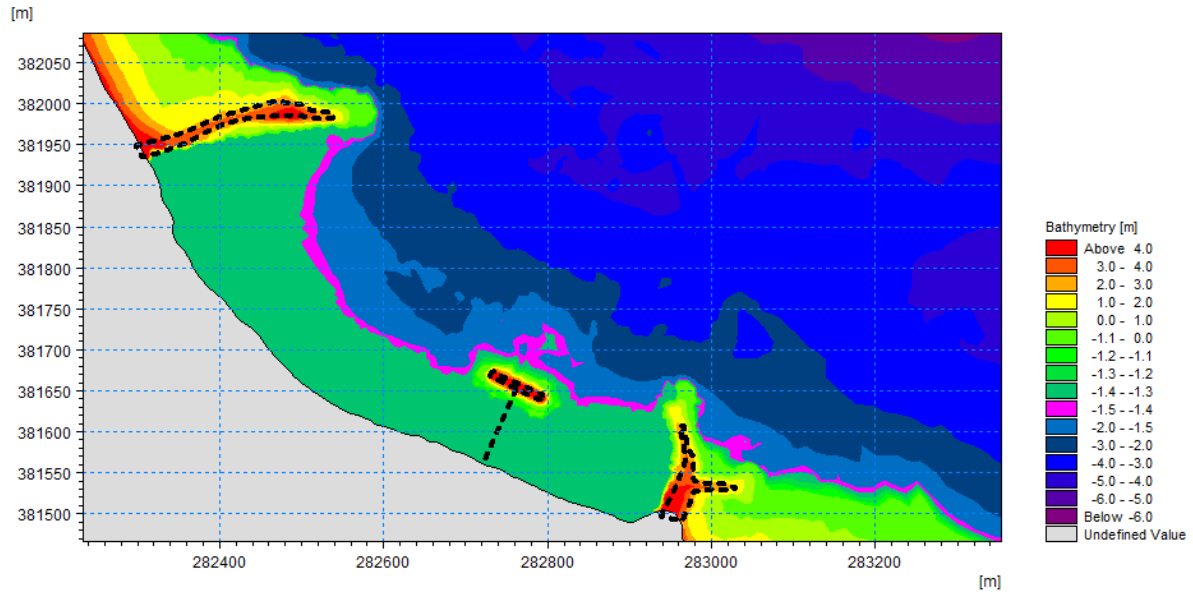


Figure 11. Model mesh for Layout 2 with included model structures (dashed black lines)

Derived beach crest positions

The wave crests developed from the modelling, and the derived beach crest positions, are shown in Appendix B for each of the three direction sectors. The resultant beach crest positions are summarised in Figure 12.



Figure 12. **Layout 2 beach crest positions, for 10 m minimum buffer (top) and 20 m minimum buffer (bottom)**

Key observations

Key observations specific to the Layout 2 model results are identified below (observations that are similar to those for Layout 1 are not repeated):

- For the WNW and NNW directions, the more prevailing wave conditions, a tombolo is still predicted to form with the 20 m wide buffer, but the width of the attachment is narrower. Under NNE waves the salient is not predicted to attach with a 20 m wide buffer.
- With a 10 m wide buffer, the salient is predicted to get close to the breakwater for the WNW and NNW cases, but does not attach.
- The nourished material should generally be contained landward of the breakwater, especially with the case of a 10 m wide buffer.
- Under NNE waves, the 'pinch point' to the west of the breakwater, i.e. the location where buffer is at its minimal width, lies in-line with the western end of the breakwater, rather than at the western end of the frontage to be protected. This is because the shoreline extends seawards as this location.

3.3.3 Layout 3

Model layout

Based on the previous model results, Layout 3 was developed by applying a 20 m westward shift to the shore-parallel breakwater in Layout 2. The shift was applied along the axis of the breakwater, so it was maintained at a similar distance from the shoreline.

The shift was primarily undertaken to try and relocate the pinch point in the western embayment, and to provide a greater distance between the breakwater and the Welsh Waster outfalls.

The modified model bathymetry is shown in Figure 13. As per Layout 2, a 'structure' was also defined in the model to represent the shore-connected structure.

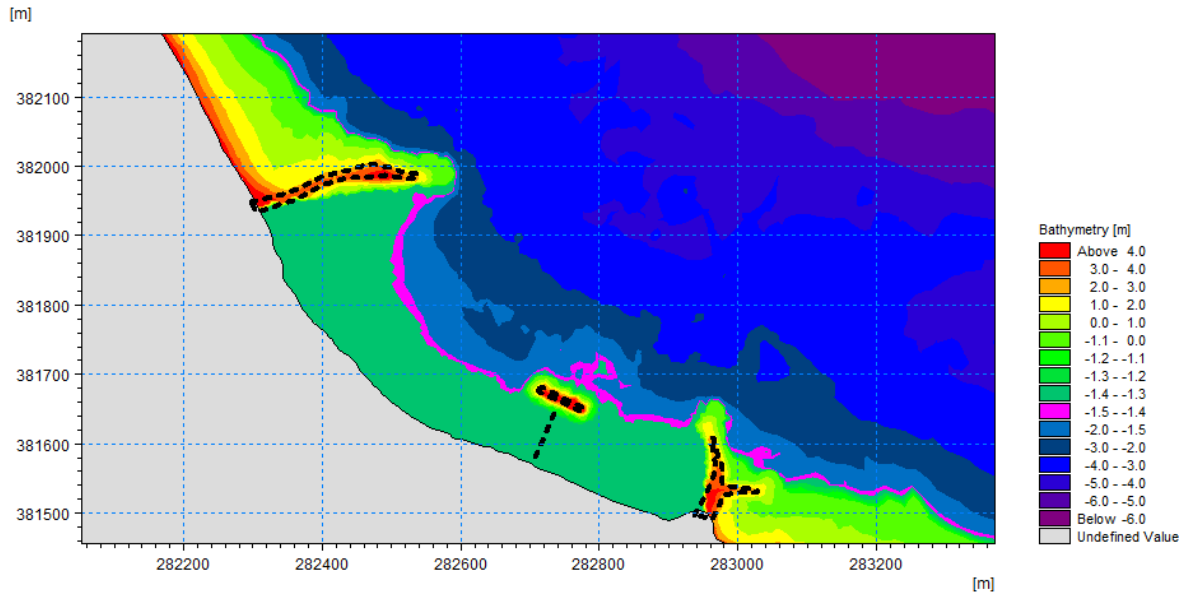


Figure 13. Model mesh for Layout 3 with included model structures (dashed black lines)

Derived beach crest positions

The wave crests developed from the modelling, and the derived beach crest positions, are shown in Appendix C for each of the three direction sectors. The resultant beach crest positions are summarised in Figure 14.



Figure 14. **Layout 3 beach crest positions, for 10 m minimum buffer (top) and 20 m minimum buffer (bottom)**

Key observations

Key observations specific to the Layout 3 model results are identified below (observation that are similar to those for layout 2 are not repeated):

- Under the NNE wave, the beach crest position in the western embayment now closely mirrors that of the existing shoreline; i.e. there is no specific 'pinch point' where the shoreline protrudes seawards in the lee of the breakwater.
- In the eastern embayment, the short Welsh Water outfall is slightly less protected by the breakwater and the 'pinch point' appears to have moved very slightly westward to align with it.

3.3.4 Layout 4

Model layout

Layout 4 was undertaken to examine the implication of possibly removing the shore connection structure. The model bathymetry is therefore identical to that for Layout 3, however the 'structure' that was defined to represent the shore connection structure has been removed. The model bathymetry, with the defined 'structures' annotated, is shown in Figure 15.

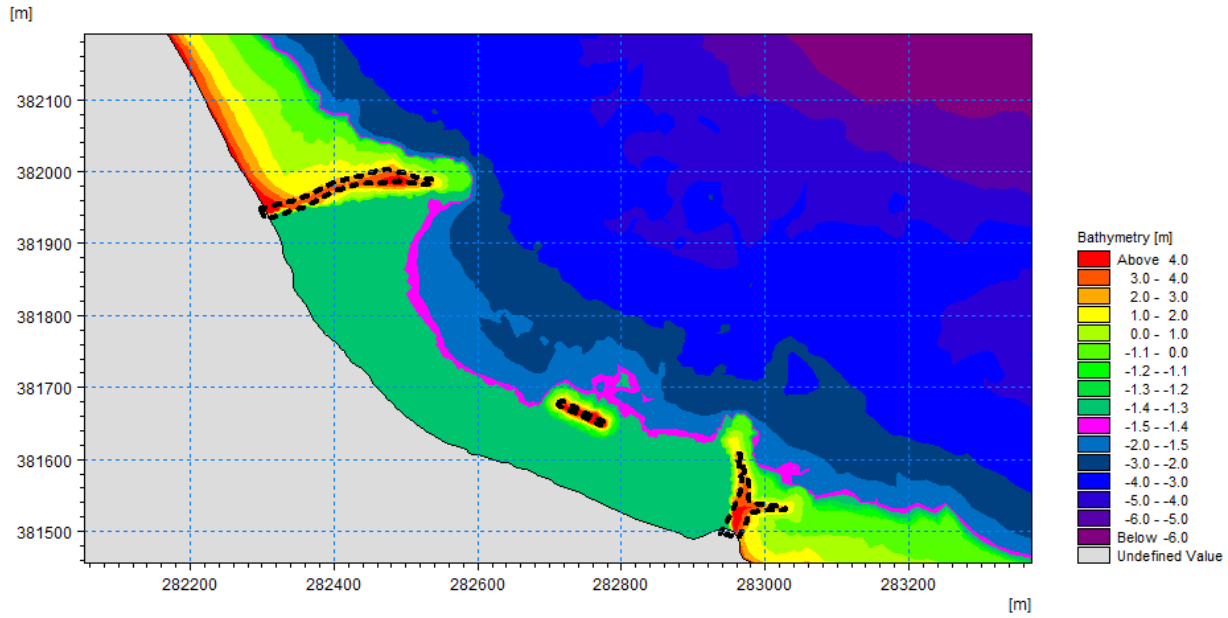


Figure 15. Model mesh for Layout 4 with included model structures (dashed black lines)

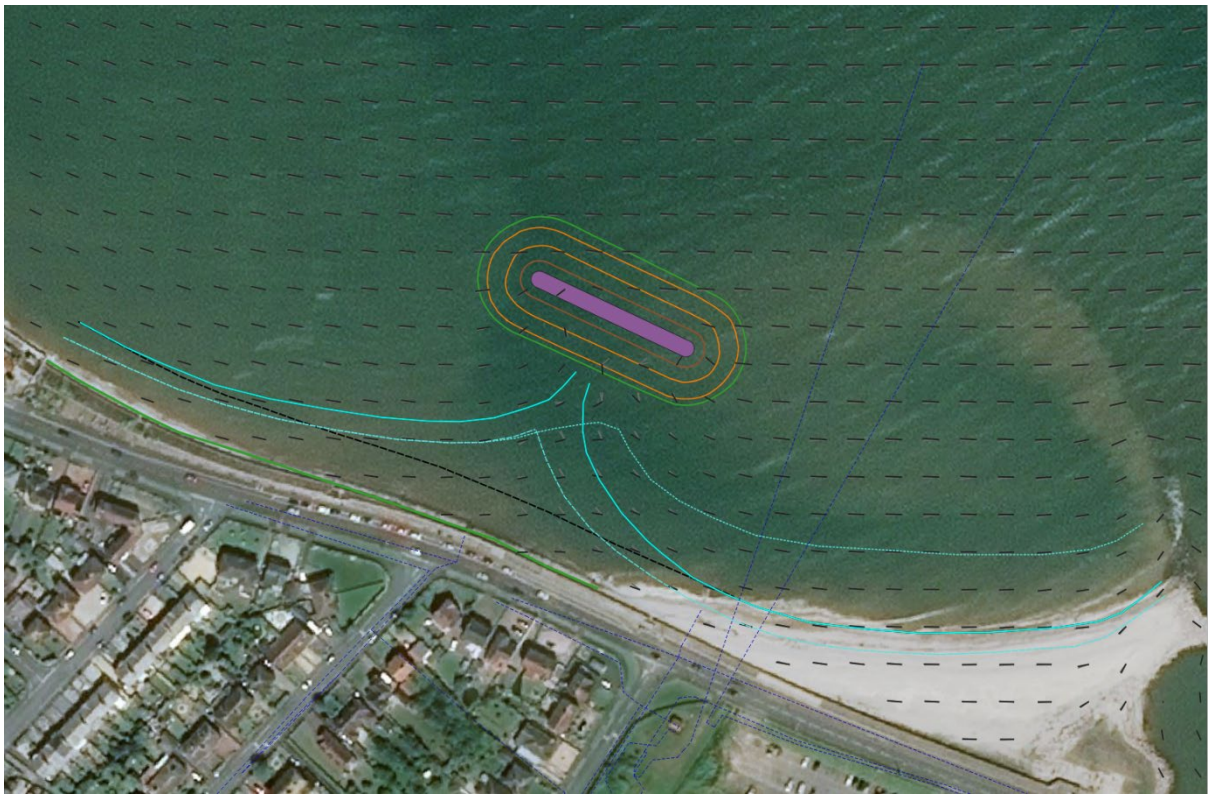
Derived beach crest positions

With the shore connected structure removed in this layout, this enabled waves to propagate behind the breakwater, from the western embayment to the eastern embayment and *vice versa*. Under the NNE wave conditions, where the waves are aligned relatively normal to the breakwater, the waves merge behind the breakwater defining a clear salient form for both the 10 m and 20 m buffer widths (Figure D3, Appendix D).

However, for the WNW and NNW wave conditions, which are incident to the breakwater from a slightly oblique direction, the situation is more complex as waves to the west of the breakwater penetrate behind the breakwater into the eastern embayment. In the model there is nothing to stop this as the model bathymetry has been depended.

However, the salient or tombolo developed in the lee of the breakwater would restrict the transformation of waves behind the breakwater from one embayment to the other.

Therefore, for those cases where it is clear that the salient would attach to the breakwater, e.g. the 20 m buffer width scenarios from WNW and NNW, then it is assumed that the breakwater is effectively attached and results from Layout 3 are used to define the beach crest positions. For this purpose, the wave crest plots in Appendix D, also have the wave crests from Layout 3 superimposed in grey. An example of this is shown in Figure 16, where the 20 m wide buffer beach crest (solid line) is defined based on the grey (Layout 3), wave crests in the eastern bay.



(Reproduced from Appendix D)

Figure 16. Wave crests and beach crest position for Layout 4 with NNW waves, wave crest from Layout 3 superimposed in grey

Where it was not initially clear if the salient would attach, beach crest positions for both the eastern and western embayment were initially developed by first examining the 'pinch point' locations in each embayment and then using the wave crest orientation to define beach crest position. The process used for previous layouts. However, due to the complex wave interaction pattern behind the breakwater this was not effective, as indicated by the dashed lines in Figure 16. The salient form developed would also not be stable under such a wave condition. Therefore, the approach adopted was to develop the salient/ tombolo form from the more dominant wave direction side (the westerly embayment in this case), and only once the salient had attached or turned, use the appropriate wave crests from Layout 3 or Layout 4 to define the beach crest position in the eastern embayment (as shown by dotted line in Figure 16).

A key result of this, is that the beach crest position lies seaward of the minimum buffer width.

The wave crests, and derivation of the beach crest position, are provided in Appendix D, and the resultant beach crest positions are summarised in Figure 17. For the WNW and NNE wave direction sectors, the resultant beach crest positions are considered reasonable, however, for the NNW case, the

developed shoreline for the 10 m buffer width is considered less reliable. Nevertheless, it is clear that without an increased volume of material within the eastern embayment, it is expected that some material from the western bay would be transported into the eastern embayment under WNW and NNW waves.



Figure 17. Layout 4 beach crest positions, for 10 m minimum buffer (top) and 20 m minimum buffer (bottom)

Key observations

Key observations specific to the Layout 4 model results are identified below (observations that are similar to those for Layout 2 are not repeated):

- With the larger 20 m buffer width, the beach is expected to attach for the prevailing WNW and NNW wave condition and would therefore perform as per Layout 3 under the same conditions.
- For the smaller 10 m buffer width scenario, the beach crest positions are much more complex to discern, particularly with more oblique waves from the WNW and NNW.
- For the 10 m buffer width scenario, it is expected that the minimum buffer width in the eastern embayment would need to be more than 10 m. If not, it is expected that material would migrate from the western bay to the eastern bay.
- Layout 4 could create greater uncertainty in the Beach Management Strategy because the beach profiles from predominant wave directions tend to be more unstable.
- Layout 4 may require greater beach maintenance than Layout 3 (with associated costs) because shingle will still be transported west to east under certain conditions.

3.4 Comparative assessment

To summarise the scheme refinement, each layout has been compared with its successive layout. Through discussion with ARUP and CCBC, because the beach is not required to provide the primary flood defence protection (but instead is required to protect the existing seawall) a buffer width of 20 m is unlikely to be required; therefore, the comparison compares the 10 m beach buffer width scenarios.

3.4.1 Layout 1 vs Layout 2

Figure 18 compares the results from Layout 1 and 2. With the breakwater extended further offshore, the salient no longer attached to the breakwater. Importantly the toe of the nourished beach would then be contained landward of the offshore breakwater. With Layout 1, the toe of the beach will just extend outside of the new breakwater.

The beach shapes and position of the beach crest, in both embayments, is generally similar for both layouts, although moving the breakwater offshore in Layout 2, makes the eastern embayment slightly more curved. Therefore, slightly less material is required in the eastern embayment, where the beach already has an ample volume of material. Layout 2 effectively helps to redistribute this material slightly better.

From a safety perspective, with the beach crest reaching the breakwater, this would allow the public to walk out to the breakwaters during high water extreme events which could be potentially dangerous if waves overtop the breakwater. A similar concern exists if the shore connected structure in Layout 2 were to allow/encourage the public to reach the breakwater under the same conditions.

The close proximity of the beach crest to the breakwater in Layout 1 is also likely to lead to steeper local beach profiles and stronger currents directly in the lee of the breakwater, than for Layout 2. Therefore, Layout 1 is less favourable from a bathing safety perspective.

As the seabed is relatively flat between Layout 1 and Layout 2, the height of the breakwater above the seabed (and thus rock volume required) is unlikely to be significantly greater for Layout 2 over Layout 1; however, the shore connected structure will need to be longer. Since the salient also attaches as a tombolo for Layout 1, it may not be necessary to include a shore-connected structure for this layout, as the beach will attach to the breakwater naturally
It is highlighted that under extreme high water and wave events, waves overtopping the breakwater in both layouts will cause some temporary local lowering / scouring of the beach immediately in the lee of the breakwater.



Figure 18. Comparison of Layout 1 (top) and Layout 2 (bottom) for the 10 m beach buffers

3.4.2 Layout 2 vs Layout 3

The results of Layout 2 and Layout 3 are compared in Figure 19. The two schemes perform very similarly, though the curvature of the eastern embayment becoming a little more drawn out. This very slightly amends the 'pinch point' in the eastern embayment to coincide slightly closer to the Welsh Water outfall. This may be of benefit if this helps to prevent an increase in beach build up at the outfall.



Figure 19. Comparison of Layout 2 (top) and Layout 3 (bottom) for the 10 m beach buffers

In the western embayment, Layout 2 included a 'pinch point' under the NNE waves that was in line with the western end of the breakwater, where the land protrudes seaward. By adjusting the breakwater west, there is no specific 'pinch point' under NNE waves as the beach crest now follows the line of the defence very closely. However, it should be noted the NNE wave events are less frequent, so the occasions when the beach may fully align to the NNE beach crest location are likely to be limited.

The other key benefit of Layout 3 over layout 2 is that there is now greater distance between the toe of the breakwater and the Welsh Water outfall to the east.

3.4.3 Layout 3 vs Layout 4

The results of Layout 3 and Layout 4 are compared in Figure 20.



Figure 20. Comparison of Layout 3 (top) and Layout 4 (bottom) for the 10 m beach buffers

As described in Section 3.3.4, without the shore-connected structure the interpretation of the model results was more difficult for the more oblique WNW and NNW waves. Nevertheless, it is clear that there would be much greater variability in the location of the salient behind the breakwater, with the likelihood that it would also attach under the more oblique WNW waves.

As a result of this, greater material is considered to be required within the eastern embayment, else the material would be lost from the western embayment to the eastern embayment. This transfer of material would be significantly less and slower than the loss that occurs under the present management/ recycling works, but it would nevertheless occur.

Associated with this, under significant wave and water level events, a strong current flow may be developed going west to east directly in the lee of the breakwater.

3.5 Conclusions

From the assessment and through discussion with ARUP and CCBC, Layout 3, with the shore-connected structure is the preferred layout. It is further considered that a minimum beach buffer of around 10 m will be enough to provide protection to the seawall. This layout is considered to perform well and will create relatively stable beach plan shapes in both the eastern and western embayments.

It is further identified that:

- Overtopping of the breakwater will occur under extreme wave and water level conditions, which will lead to some temporary local scouring in the lee.
- Shingle will still overtop the trunk of the eastern breakwater, as occurs presently. This overtopping of shingle will increase with climate change and may also increase as a result the breakwater. It is recommended that the trunk of the eastern breakwater is raised to prevent this loss of shingle from the bay.
- If the shore-connected structure permits the public to reach the breakwater during significant wave and water level events, this could put them in danger of water overtopping the breakwater
- As the salient will form out from the mainland, there may be potential to optimise the shore-connected structure; for example, it may not be necessary for it to actually join the existing sea wall.

4 Extremes Wave and Water Levels

4.1 Modelling approach

To derive extreme design wave and water level conditions for the proposed structure, offshore hindcasts of waves and of tide and surge water levels, have been subject to a Joint Probability Analysis (JPA). Key Joint Probability (JP) pairs have then been transformed inshore and wave conditions have been extracted at the toe of the proposed structure. The analysis is described in further detail below.

4.2 Joint probability analysis

To undertake the JPA, a 40-year hindcast timeseries of wave heights has been extracted at the offshore wave model boundary from the ABPmer SEASTATES European Shelf Hindcast Wave Model (www.seastates.net), together with a coincident timeseries of water levels from the ABPmer SEASTATES North West European Shelf Tide and Surge Model. Extraction locations are shown Appendix E and Appendix F.

The JPA has been carried out in two stages. Firstly, determination of the extreme water levels and wave heights as univariate extremes; and secondly, the JPA itself. The analyses were completed for the present-day epoch (2020). For future epochs (2070 and 2120); water level, wave height and wind speed allowances were uplifted to account for climate change projections.

4.2.1 Wave conditions

Extreme wave heights have been determined based on the long-term wave hindcast data extracted offshore of Penrhyn Bay.

The review of the hindcast data (scatter plot in Appendix G) identified that the largest waves at the offshore boundary of the local model arrive from a relatively oblique wave angle, with waves from 258.75° – 303.75°. Waves from the more northerly sector are smaller, however, these would reach the site having undergone less refraction. Therefore, the JPA has been completed for both WNW and northerly direction sectors. For the WNW sector (which includes the WNW wave direction modelled in the scheme assessment study (Section 3)), the analysis is based on waves from 258.75° – 303.75°. For the northerly sector (which includes the NNW and NNE wave directions modelled in the scheme assessment study (Section 3)), the analysis is based on waves from 303.75° to 11.75°.

The extremes analysis was undertaken on de-clustered independent events. This was done to ensure that each wave 'event' in the extremes analysis is statistically independent. In this exercise, a single event was defined by wave height being above the threshold for a minimum of three hours, and a 24-hour interval was selected before a wave height reaching this threshold could be considered as another (independent) event.

The extremes analysis was then undertaken by fitting a Generalized Pareto Distribution (GPD) to the peak events (the fits are provided in Appendix H). During this process, the storm events are further analysed using mean residual life plots, which provide a suitable range of thresholds for the analysis. Shape and scale parameters for this range of thresholds are plotted to identify the most suitable threshold value to be used in the final analysis. This standard checking process in GPD fitting helps to ensure that the GPD only utilises appropriate events that describe the extreme event population.

To suit the Welsh Government Policy Advice, Welsh Government (2017) (Appendix I) suggests that sensitivity to increased wind speed and wave height should be considered. Although it does not propose how the results from this sensitivity assessment should be considered, Table 2 outlines the wave height and wind speed climate change allowances that have been applied to the different JP epochs.

Table 2. Wave and wind climate change adjustments from Welsh Government (2017)

Epoch	Wave Height	Wind Speed
2020	0%	0%
2070	+5%	+5%
2170	+10%	+10%

4.2.2 Water levels

Whilst long-term hindcast waters levels have been extracted for the site for use in the JPA (to examine the dependency between waves and water level), univariate extreme water levels have been derived from Environment Agency (2019) Coastal Flood Boundary Dataset, with a base year of 2017 (Table 3). The location at which EA CFBD data has been extracted is shown in Appendix J .

As per Welsh Government Policy Advice, Welsh Government (2017)[1] guidance will be applied to future epochs in order to define projected sea-level rise (SLR). These values will then be used to uplift the water level values in the JP pairs for future years. The derived SLR values from the Welsh Government (2017) guidance, for each epoch under consideration, are provided in Table 4.

Table 3. CFBD extreme water levels.

Return Period (RP)	EA CFBD mODN (2017 base year)
20	5.09
50	5.20
100	5.28
200	5.36

Table 4. Climate Change uplift to CFBD 2017 base year derived from Welsh Government 2017 Guidance

Epoch	Uplift from CFBD 2017 base year (m)
2020	0.0105
2070	0.4405
2120	1.1205

[1] Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities in Wales" (Welsh Government, 2017).

4.2.3 JPA

The aim of the JPA is to analyse the hindcast wave dataset created by the wave modelling, in conjunction with long-term water levels, to determine the correlation between large wave events and high-water levels.

This correlation value is then used to predict pairings of wave and water level conditions of equal annual exceedance probability (AEP). In this assessment, water levels immediately fronting the site have been utilised from ABPmer's 40-year hindcast tide and surge model. This model provides a consistent set of water level data at the site over the same period as the wave hindcast. The tide and surge model have been subject to extensive calibration (ABPmer 2017b).

The JPA was undertaken using the JOINSEA approach (Defra, 2005) and the resultant Joint Probability curves are provided in Appendix H. Separate JPA's have been undertaken for the WNW and northerly direction sectors. In the JPA, the extreme water levels are adjusted to those determined from the CFBD 2018 levels (Section 4.2.2)

Key JP pairs were selected for the; 20, 50, 100 and 200-year return periods from the full list of JP pairs for each direction sector (as in Appendix K) for use in the wave modelling. The pairs selected are provided in Table 5 for the WNW sector and Table 6 for the northerly sector. These tables also include associated wave periods and wind speeds, which were derived by examining the relationship between Hs and Tp and then Hs and Wind speed (and which are shown in Appendix L).

Table 5. Selected JP pairs representative of the WNW directional sector

Return Period (RP)	AEP (%)	Water Level (mODN)	Significant Wave Height (m) (258.75° to 303.75°)	Peak Wave Period (s)	Wind Speed (m/s @10 m)
2020					
T20	**5**	2.0	4.6	8.9	18.6
		3.5	4.0	8.3	17.0
		4.6	2.0	6.7	10.7
T50	**2**	3.0	6.4	10.5	22.3
		4.0	5.3	9.6	20.2
		4.8	3.0	7.2	14.1
T100	**1**	3.0	7.9	11.7	24.5
		4.7	6.0	10.2	21.6
		5.1	3.0	7.2	14.1
T200	**0.5**	3.0	8.3	12.0	24.9
		4.8	6.0	10.2	21.6
		5.2	3.0	7.2	14.1
2070					
T20	**5**	2.4	4.8	9.1	19.1
		3.9	4.2	8.5	17.5
		5.0	2.1	6.9	11.1
T50	**2**	3.4	6.7	10.8	22.9
		4.4	5.6	9.8	20.8
		5.2	3.2	7.4	14.5
T100	**1**	3.4	8.3	12.0	24.9
		5.1	6.3	10.4	22.2
		5.5	3.2	7.4	14.5
T200	**0.5**	3.4	8.7	12.3	25.3
		5.2	6.3	10.4	22.2
		5.6	3.2	7.4	14.5
2120					
T20	**5**	3.1	5.1	9.4	19.7
		4.6	4.4	8.7	18.1
		5.7	2.2	7.0	11.4
T50	**2**	4.1	7.0	11.0	23.4
		5.1	5.8	10.0	21.3
		5.9	3.3	7.6	15.0
T100	**1**	4.1	8.7	12.3	25.2
		5.8	6.6	10.7	22.7
		6.2	3.3	7.6	15.0
T200	**0.5**	4.1	9.1	12.6	25.5
		5.9	6.6	10.7	22.7
		6.3	3.3	7.6	15.0

Table 6. Selected JP pairs representative of the N directional sector

Return Period	AEP (%)	Water Level (mODN)	Significant Wave Height (m) (303.75° to 11.75°)	Peak Wave Period (s)	Wind Speed (m/s @10 m)
2020					
T20	**5**	3.0	3.8	8.1	16.4
		3.8	3.0	7.2	14.1
		4.5	1.5	5.8	8.9
T50	**2**	3.0	4.7	9.0	18.8
		4.0	4.0	8.3	17.0
		4.7	3.0	7.2	14.1
T100	**1**	3.0	5.6	9.8	20.8
		4.5	4.8	9.1	19.0
		4.9	3.5	7.8	15.6
T200	**0.5**	3.5	5.6	9.8	20.8
		4.5	5.1	9.4	19.7
		5.0	3.6	7.9	15.9
2070					
T20	**5**	3.4	4.0	8.3	17.0
		4.2	3.2	7.4	14.5
		4.9	1.6	6.0	9.2
T50	**2**	3.4	4.9	9.2	19.4
		4.4	4.2	8.5	17.5
		5.1	3.2	7.4	14.5
T100	**1**	3.4	5.9	10.1	21.4
		4.9	5.0	9.3	19.6
		5.3	3.7	8.0	16.1
T200	**0.5**	3.9	5.9	10.1	21.4
		4.9	5.4	9.6	20.3
		5.4	3.8	8.1	16.4
2120					
T20	**5**	4.1	4.2	8.5	17.5
		4.9	3.3	7.6	15.0
		5.6	1.7	6.1	9.4
T50	**2**	4.1	5.2	9.5	19.9
		5.1	4.4	8.7	18.1
		5.8	3.3	7.6	15.0
T100	**1**	4.1	6.2	10.3	21.9
		5.6	5.3	9.6	20.1
		6.0	3.9	8.2	16.6
T200	**0.5**	4.6	6.2	10.3	21.9
		5.6	5.6	9.8	20.8
		6.1	4.0	8.3	16.9

4.3 Wave transformation modelling

4.3.1 Model parameters

The SW model used to run the JP pairs into Penrhyn Bay uses the model domain and mesh as described in Section 2 and shown in Figure 2. It includes the present-day bathymetry nearshore (i.e. it has not been artificially deepened as per the scheme assessment test) and no additional structures have been included.

Model parameters generally followed those of the SEASTATES hindcast model with the key exception being bed friction, for which a lower value (smoother bed) was applied. For this, a Nikuradse Roughness, value of 0.004 was applied. A lower value is recommended by DHI for shallow water, and the value adopted was based on experience of ABPmer for similar sites where nearshore data has been available to aid model calibration.

Boundary wave conditions are applied uniformly along the outer boundary (or boundaries), which are typically aligned close to the incident wave direction. For the WNW sector, waves were applied at the model from 303° and for the northerly sector from 348°. These directions were considered representative of the wider direction sectors being examined. Wind conditions were applied in the same directions as the wave conditions.

The model was then run for each of the JP wave/ water level pairs identified in Section 4.2.3.

4.4 Inshore wave conditions

The wave data were extracted at five specific points located within Penrhyn Bay, along the general line of the proposed structure. The extraction locations are tabulated below in Table 7 and shown in Figure 21.

The full set of tabulated results for the modelled JP pairs, including the associated parameters of Tp and Ws, are presented in Appendix M. Results are also provided in the accompanying data file: "Penrhyn_JPA_extreme_Nearshore_v2_20200729.xlsx".

Table 7. Penrhyn Bay nearshore extraction locations.

Location	Easting	Northing	Bed level (mODN)
P1	282687	381720	-2
P2	282732	381699	-1.4
P3	282777	381678	-1.3
P4	282823	381657	-1.4
P5	282868	381636	-1.4

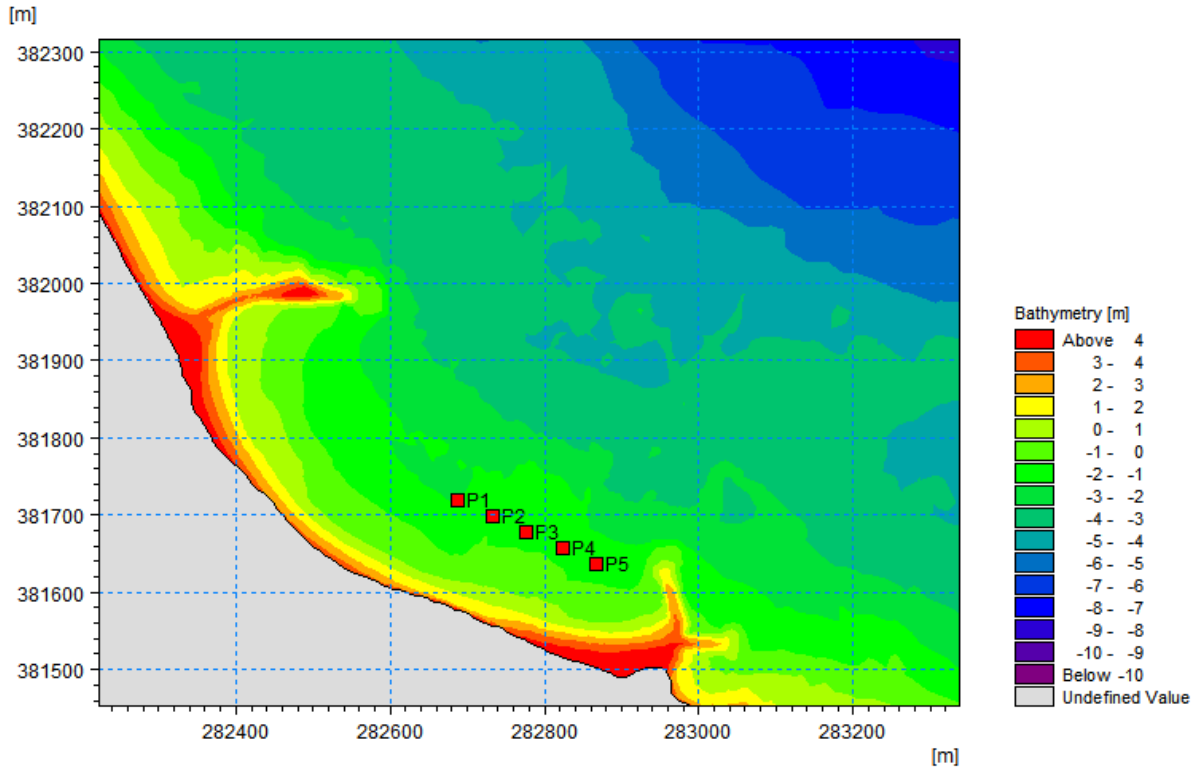


Figure 21. Nearshore data extraction locations

4.5 Conclusions

As shown by the extracted wave data, Hs is shown to increase from epoch to epoch, which is as expected given the increase in climate change allowances as the epochs progress. Additionally, both direction sectors (WNW and N) show all five extraction locations to have similar values of Hs within each run scenario, with wave height differences ranging by 0.1 to 0.2 m between extraction locations. Both direction sectors also show that the central extraction locations (P2–P4) generally have slightly larger wave height values than P1 and P5.

These small variations are likely to derive from local bathymetric features defined within the fine resolution model mesh.

5 Flow Modelling

The following sections describe the approach, setup and results of the coupled HD and SW modelling, which predicts the influence of the proposed scheme on the local current regime.

5.1 Coupled model setup

The coupled HD/SW model uses the same model extent and mesh described in Section 2, and subsequently applied to the stand-alone wave modelling described in Sections 3 and 4.

Hydrodynamic boundary forcing conditions have been extracted from the ABPmer SEASTATES Tide and Surge hindcast database and applied to the external boundaries illustrated in Figure 2. Temporal and spatially varying flow conditions are applied to the north and east boundaries, with a time-varying water level series applied to the west boundary. The run period, which extends over approximately 2.5 days, includes a mean spring tide period.

Coupled HD/SW conditions relating to the central joint pairing at the present-day 1 in 200-year event (as described in Section 4.2 and shown in Appendix K) have also been applied to the coupled model. Offshore boundary wave conditions, coupled with a mean spring tide (with water level uplift to match the desired peak elevation), are applied, as detailed in Table 8.

Table 8. Coupled HD/SW conditions applied to the modelling, derived from the JPA process described previously

Direction sector	Joint RP	Coupled model inputs			
		Water level (m)	Hs (m)	Tp (s)	WS (m/s)
WNW	1 in 200-yr	4.8	6.0	10.2	21.6
NNE	1 in 200-yr	4.5	5.1	9.4	19.7

Source: Selected JP pairs highlighted in Appendix K

The coupled HD/SW model has subsequently been run for the following conditions:

- Present-day tide-only condition (mean spring tide);
- Present-day 1 in 200-year joint condition with waves from WNW;
- Present-day 1 in 200-year joint condition with waves from NNE; and
- Present-day 1 in 1-year joint condition with waves from WNW.

The tide-only condition is run to allow comparison between with and without wave cases. Extreme joint conditions (200-year RP) have been assessed to consider the predicted impact of the proposed structure on the Dŵr Cymru Welsh Water outfall, which are aligned just to the east of the planned breakwater. The more frequent (1-year RP) extreme joint condition is run to assess the potential impact of the structure on nearshore currents and inform consideration of bather safety.

5.2 Coupled model results

The following sections detail the results of the coupled modelling assessment, including selected outputs to illustrate the key findings. A full set of model output plots from the coupled HD/SW modelling is provided in Appendix N, for each of the directional sectors assessed.

5.2.1 Influence of wave-induced currents

Whilst peak currents offshore of Penrhyn Bay on an ebb and flood tide can reach speed over 1 m/s, due to the shoreline orientation and location of adjacent headlands the tidal current in the bays are weak, as shown in Figure 22. Therefore, the currents within the bay are dominated by the wave driven flows, when there are moderate to large waves.

Figure 24 compares the modelled current speeds within the bay 'tide only' and 'wave + tide' scenarios with the 1 in 1-year wave / water level event from WNW around high water. It is clearly evident from the comparison of the model outputs that the wave-induced current exerts a significant influence on the local flow regime.

Key observations from these models runs are noted below.

- With the inclusion of waves the alongshore driven flow is directed around the western and eastern breakwater with peak flows speeds observed off the tip of the structures.
- At high tide some wave driven flow occurs over the western and eastern groynes which are partially submerged.
- At lower tide levels, (thus without flow occurring over the western structure) flows under the combined WNW wave and tide conditions form eddies in the lee of the existing western groyne structure, with shoreward flows in an east to west direction (building beach material to the western side of Penrhyn Bay, Conversely, shoreward flows along the eastern side of the bay are shown to be west to east, helping to maintain the beach in the lee of the existing eastern breakwater.

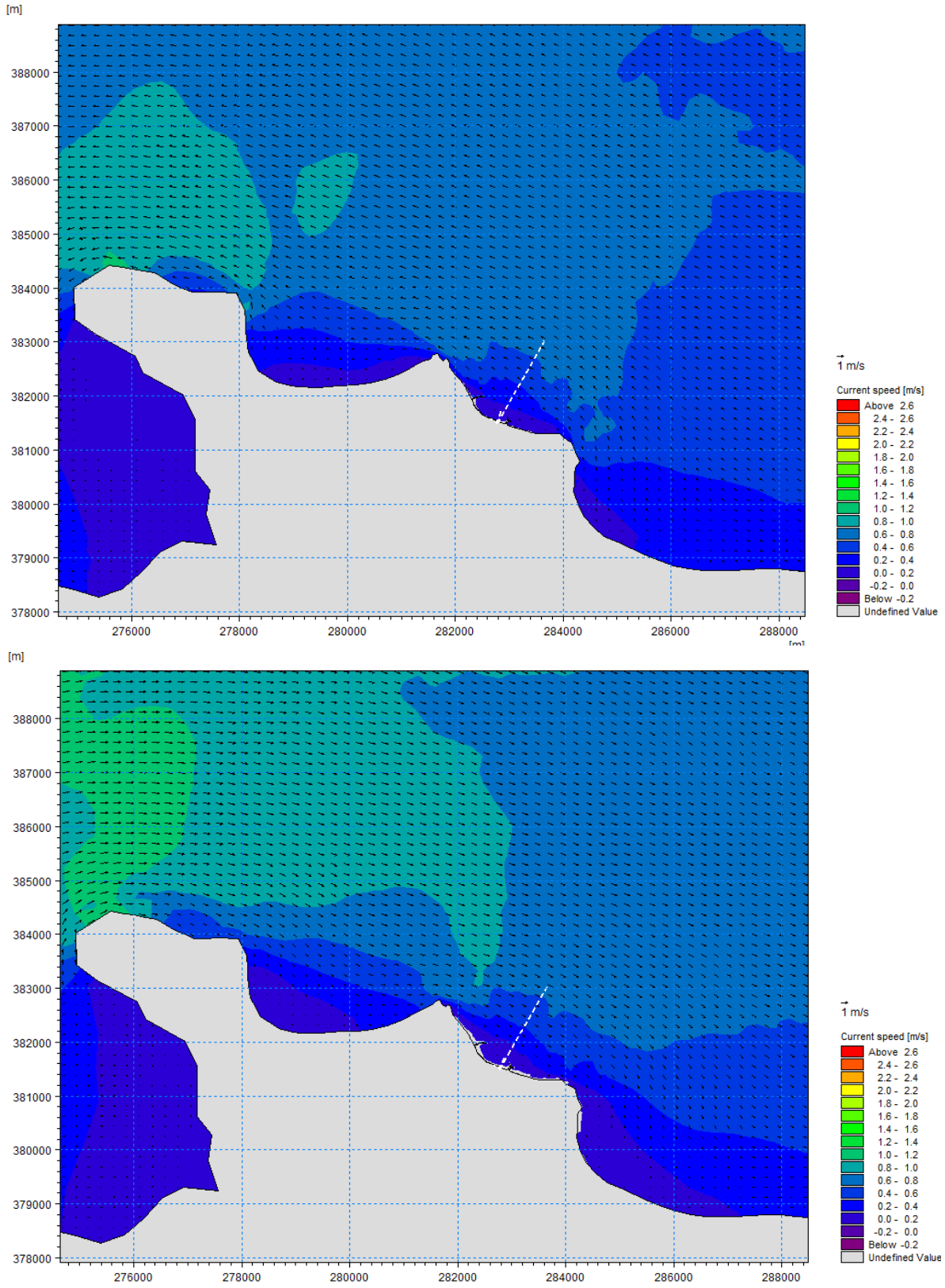


Figure 22. Peak ebb (top) and flood (bottom) tidal currents over spring tide (excluding waves)

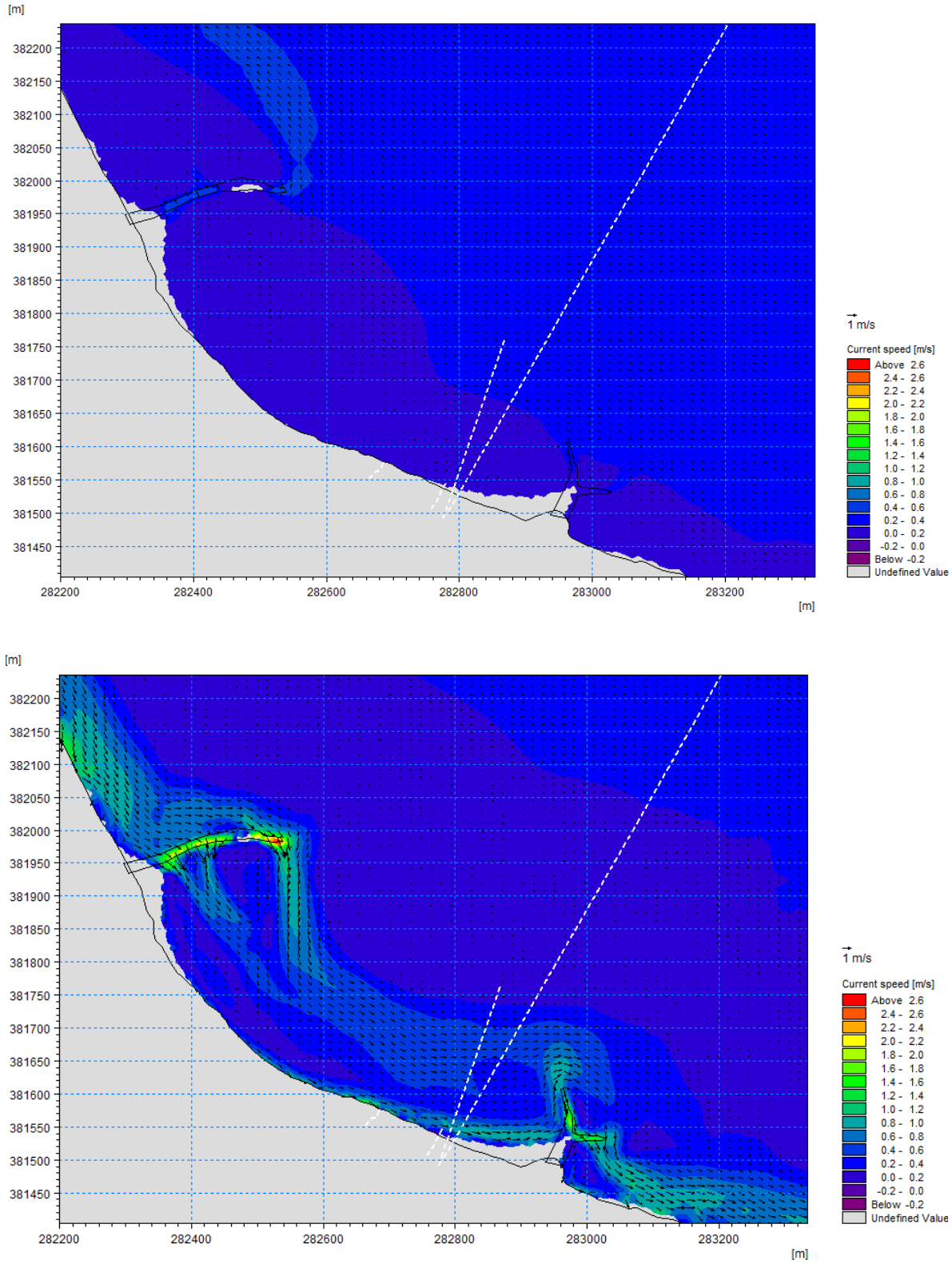


Figure 23. **Comparisons of tide only and wave and tide driven flows for 1 in 1-yr event from WNW**

5.2.2 Predicted impact of breakwater on combined wave/current flow conditions

A series of model runs have been undertaken to consider the impact of the proposed scheme on the local combined wave-current flow regime. This assessment includes running extreme 200-year return period paired wave and water levels, and the more frequent 1-year extreme joint event.

200-year return period events

Figure 24 and Figure 27 show the modelled flow regime under existing (baseline) and proposed scheme scenarios for WNW and NNE 1 in 200 year wave conditions, respectively. Detail of the modelled flow regime around the proposed scheme is also shown, for each wave approach direction, in Figure 25 and Figure 28.

The plots show the model output around the peak water level (in line with the paired return period events); model outputs at hourly intervals throughout the modelled tidal cycle are provided in Appendix N. The modelled difference in flow speeds, for each of the assessed wave directions, is provided in Figure 26 and Figure 29..

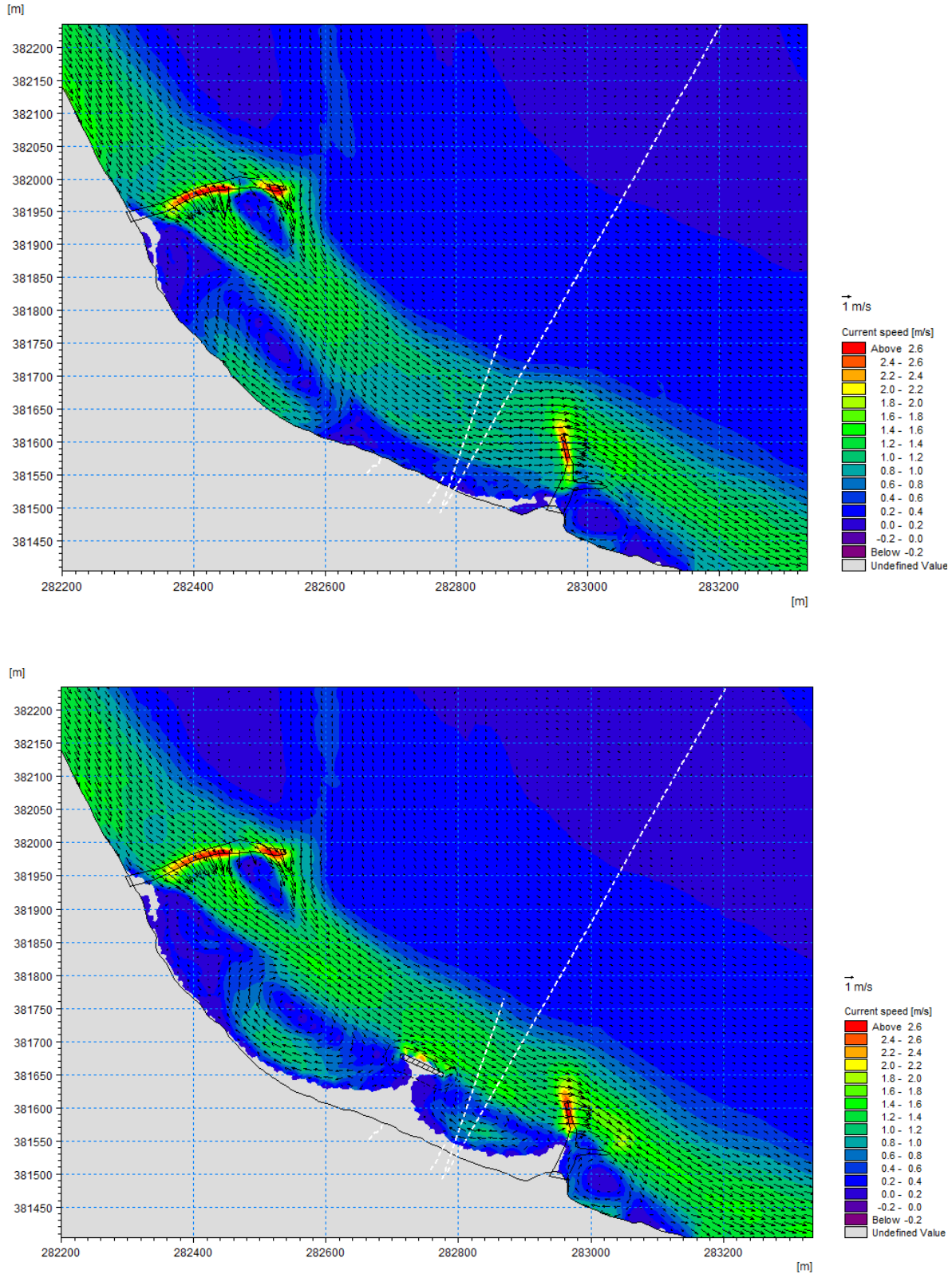


Figure 24. **Peak combined WNW 1 in 200-year wave and tide current flow: Baseline (top)and Scheme (bottom)**

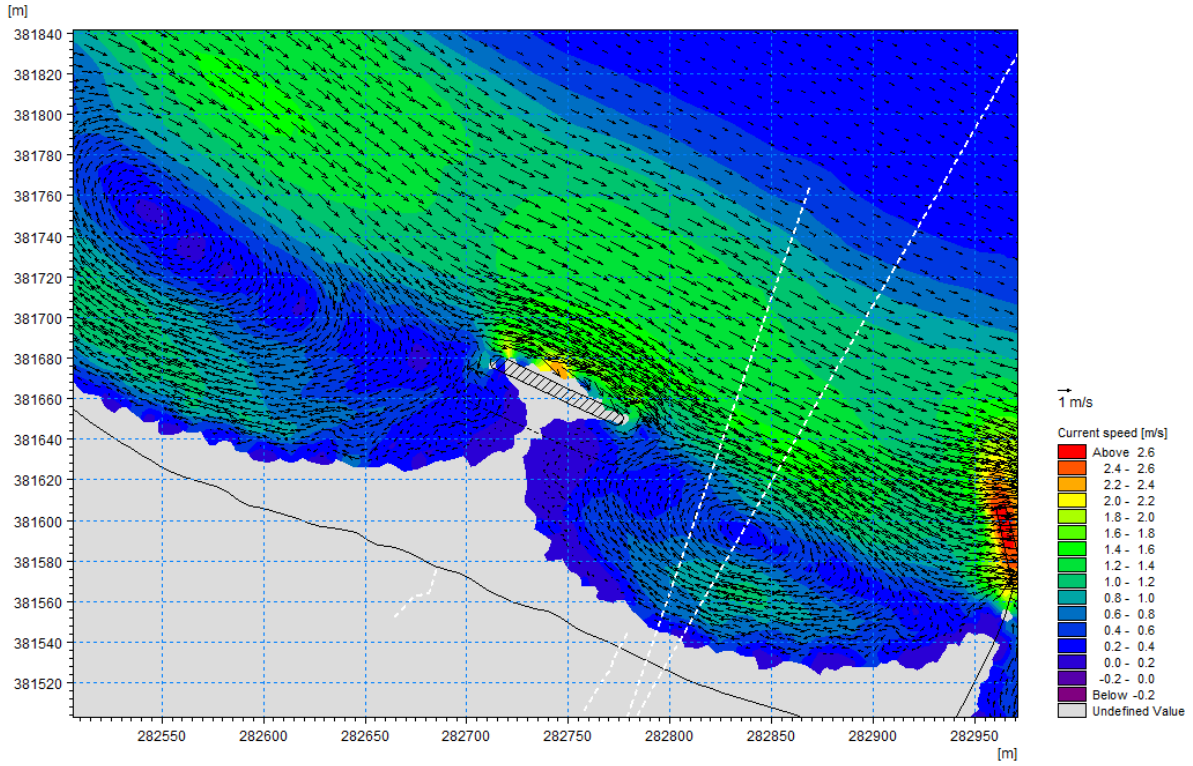


Figure 25. **Flow detail around breakwater for peak combined WNW 1 in 200-year wave and tide current flow**

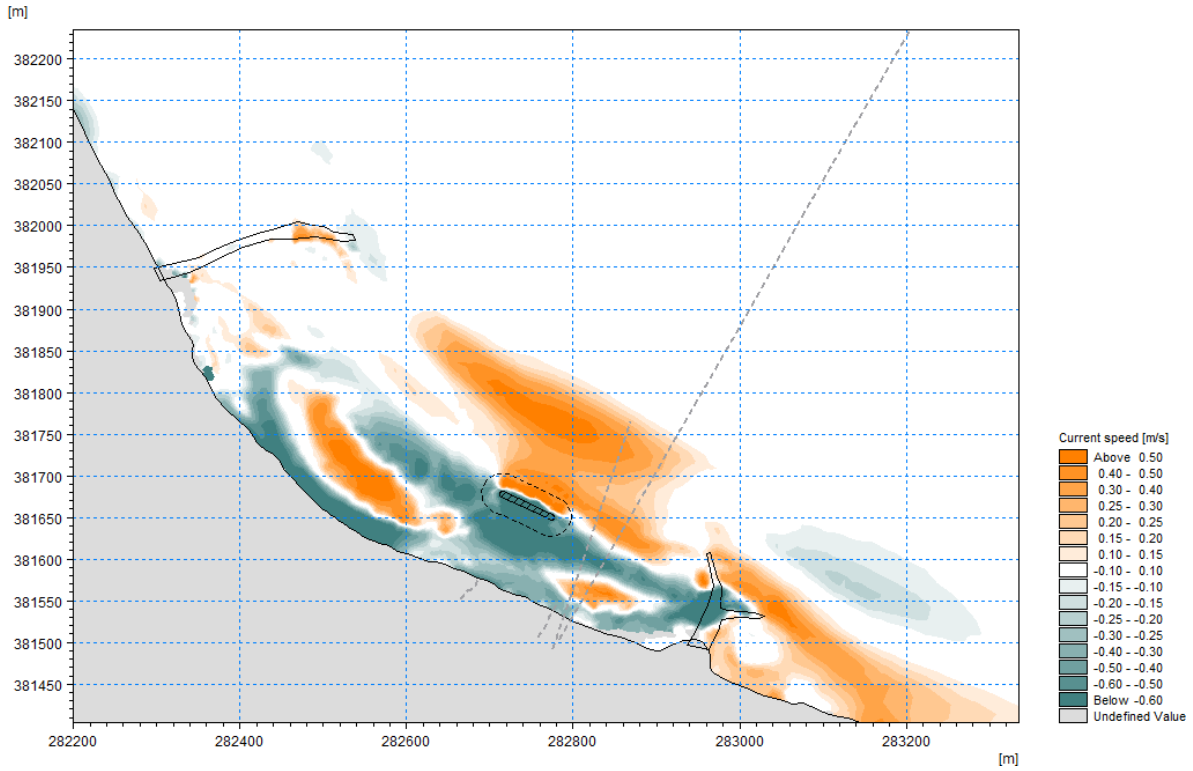


Figure 26. **Difference in peak combined WNW 1 in 200-year wave and tide current flow**

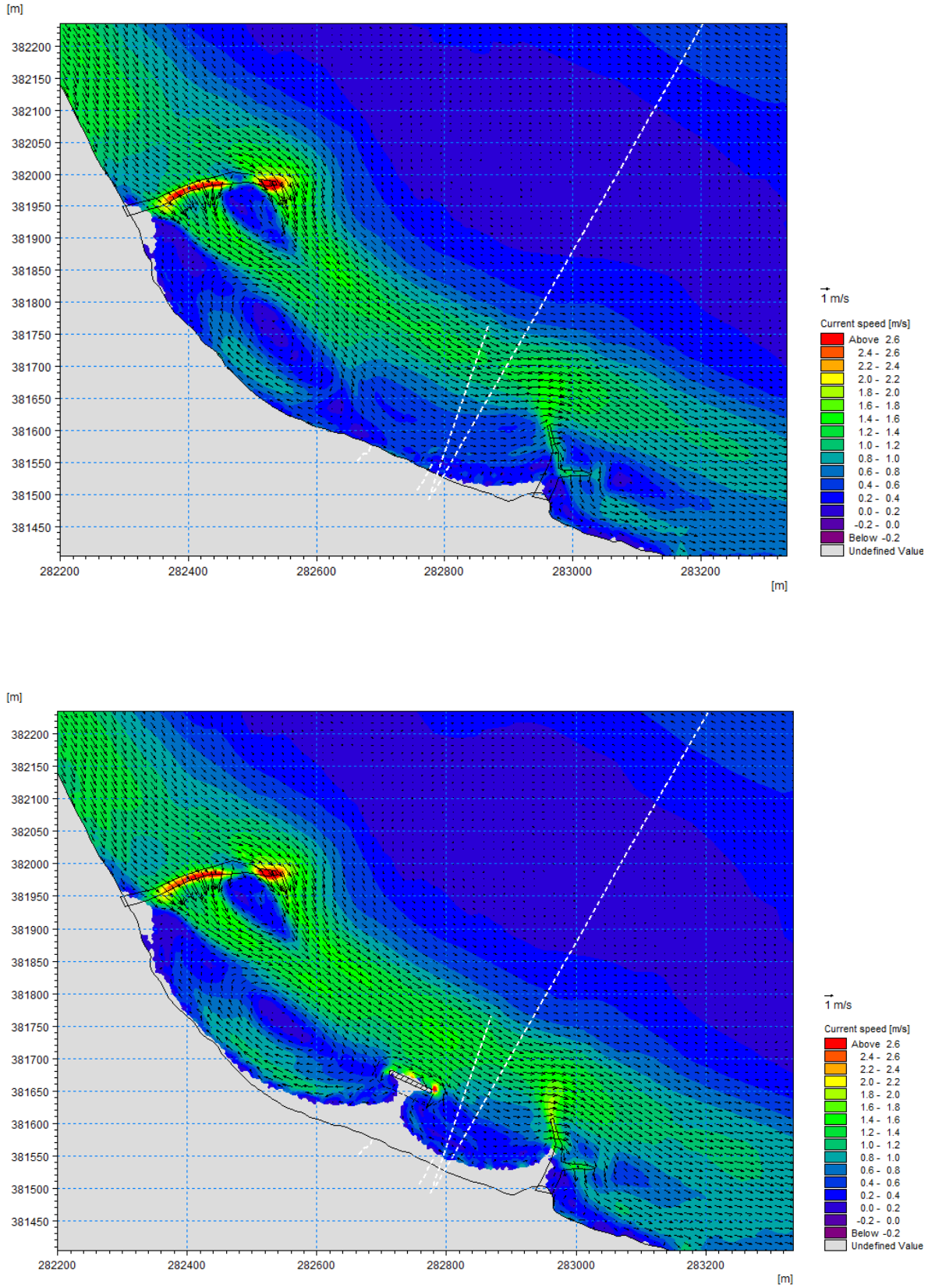


Figure 27. **Peak combined NNE 1 in 200-year wave and tide current flow: Baseline (top)and Scheme (bottom)**

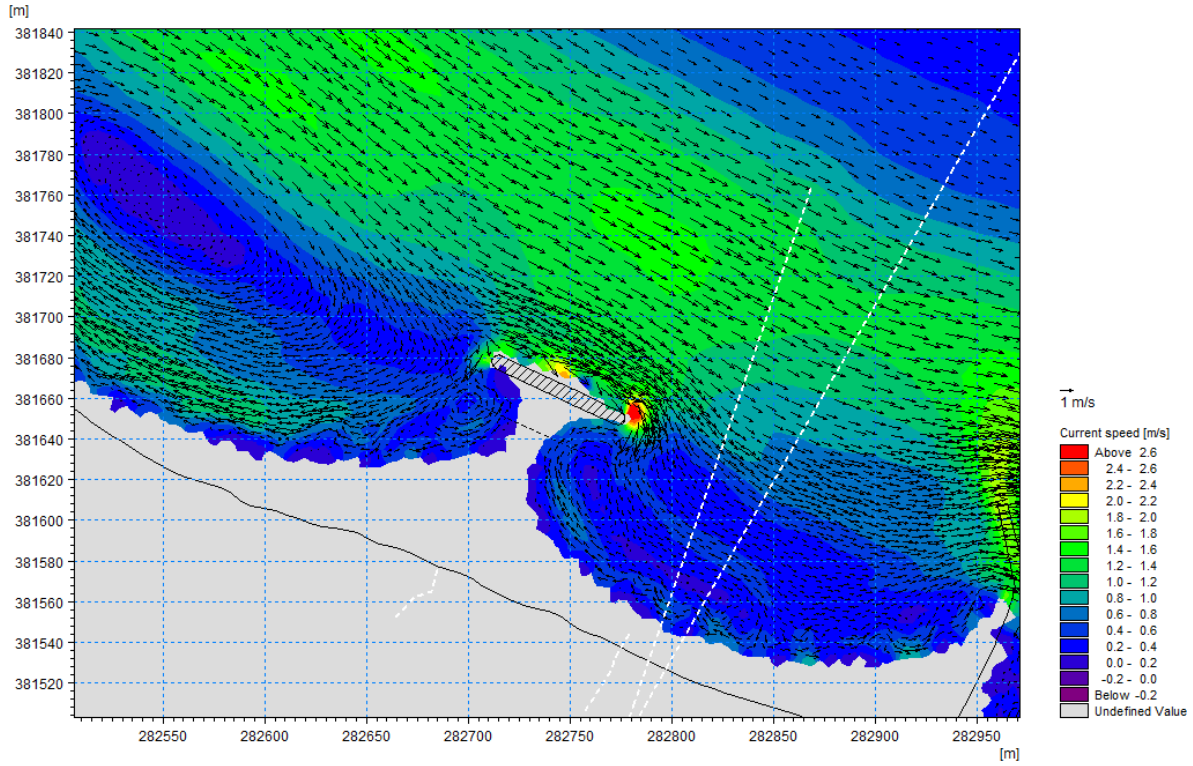


Figure 28. **Flow detail around breakwater for peak combined NNE 1 in 200-year wave and tide current flow**

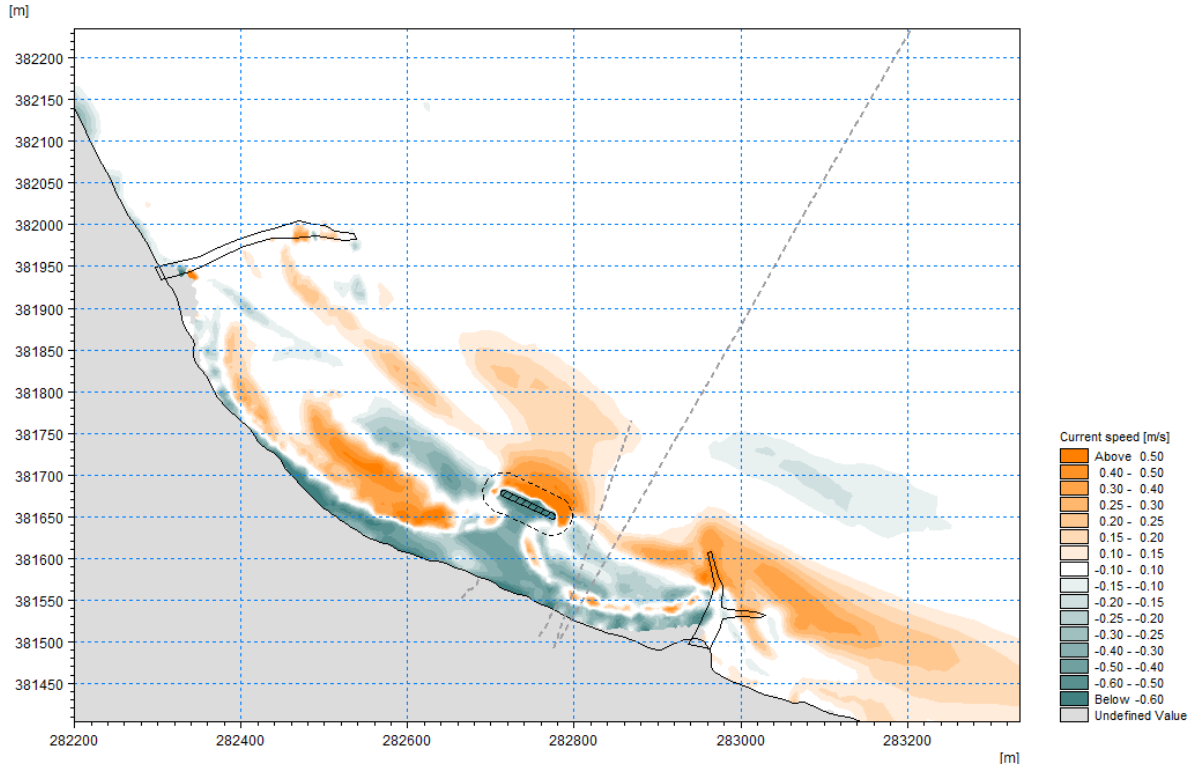


Figure 29. **Difference in peak combined NNE 1 in 200-year wave and tide current flow**

Under the baseline conditions (without scheme) both the WNW and NNE waves develop and drive a longshore current that is directed out and around the western groyne which then crosses the bay and then passes across the end of the fishtail groyne to the east. The redirection of the flow is also seen to develop a counter current along the beach in the western half of the bay.

At high water levels (used in these simulations) some water is also observed to pass over the western breakwater. This induces a west - east current across the outer bay, particularly to the west.

With the presence of the 'T' groyne this bypassing flow, is pushed offshore of the 'T' groyne causing the flow to be funnelled offshore of the structure, thus increasing the flow speed directly off the 'T' groyne. Some wave breaking on the 'T' groyne also induces locally stronger flow directly adjacent to the structure. This also causes a slight increase in the current speeds across, and to the east, of the fishtail groyne. Over the fishtail groyne speeds under the WNW event, typically increase by around 0.5 m/s to 2.6 m/s and increase of around 20%. It is important, therefore, that the beach does not extend beyond or over this groyne else this could result in increased material loss.

A small difference in current speed is also observed across the outer extent of the western groyne, But this difference is considered negligible.

The realignment of the bypassing flow, together with the inclusion of the nourished beach, locally changes the eddy that is formed in the western side of the beach. Under WNW, the flow strengths associated with this eddy in the western half of the bay are relatively similar with and without the scheme, but in the eastern embayment a counter current is also developed with the present schemes, which did not occur previously. For the NNE wave events, the counter eddy in the western embayment is enhanced with the scheme, however, it is identified that the beach nourishment within the model was based on WNW thus this would be expected as the beach reoriented. In the eastern embayment there is no significant counter current, with smaller scale local circulation apparent in the bay.

From the detailed area plots, whilst these is some small localised eddied directly in the lea of the 'T' groyne arms, especially under the NNE wave which are more normal to the structure, there is no clear directed flow offshore. This is partly because the wave breaking on the structure also results in a small onshore directed flow. This is beneficial as the modelling is not predicting currents which would cause the nourished material to be directed offshore.

The increased flow offshore of the 'T' groyne, due to the funnelling of the longshore wave driven flow extends to the location of the two longer outfalls. The increase is greatest over the western of the two outfalls under the WNW waves when the bypassing longshore currents are at their greatest (Figure 24 and Figure 26). Across this outfall current speeds increase by around 0.4 m/s to around 1.8 m/s, under the 1 in 200-yr conditions simulated. This is an increase of around 30%. However, it is noted, that over this region and as the tide is rising and falling, direct wave breaking would be occurring over the outfalls. This is a highly energetic process, and this wave breaking will not be significantly affected by the scheme.

1-year return period event

Figure 30 shows the modelled flow regime under existing (baseline) and proposed scheme scenarios. Details of the modelled flow regime around the proposed scheme are also shown in Figure 31, and the resultant difference in flow speeds is provided in Figure 32. The plots show the model output around the peak water level (in line with the paired return period events); model outputs at hourly intervals throughout the modelled tidal cycle are provided in Appendix N.

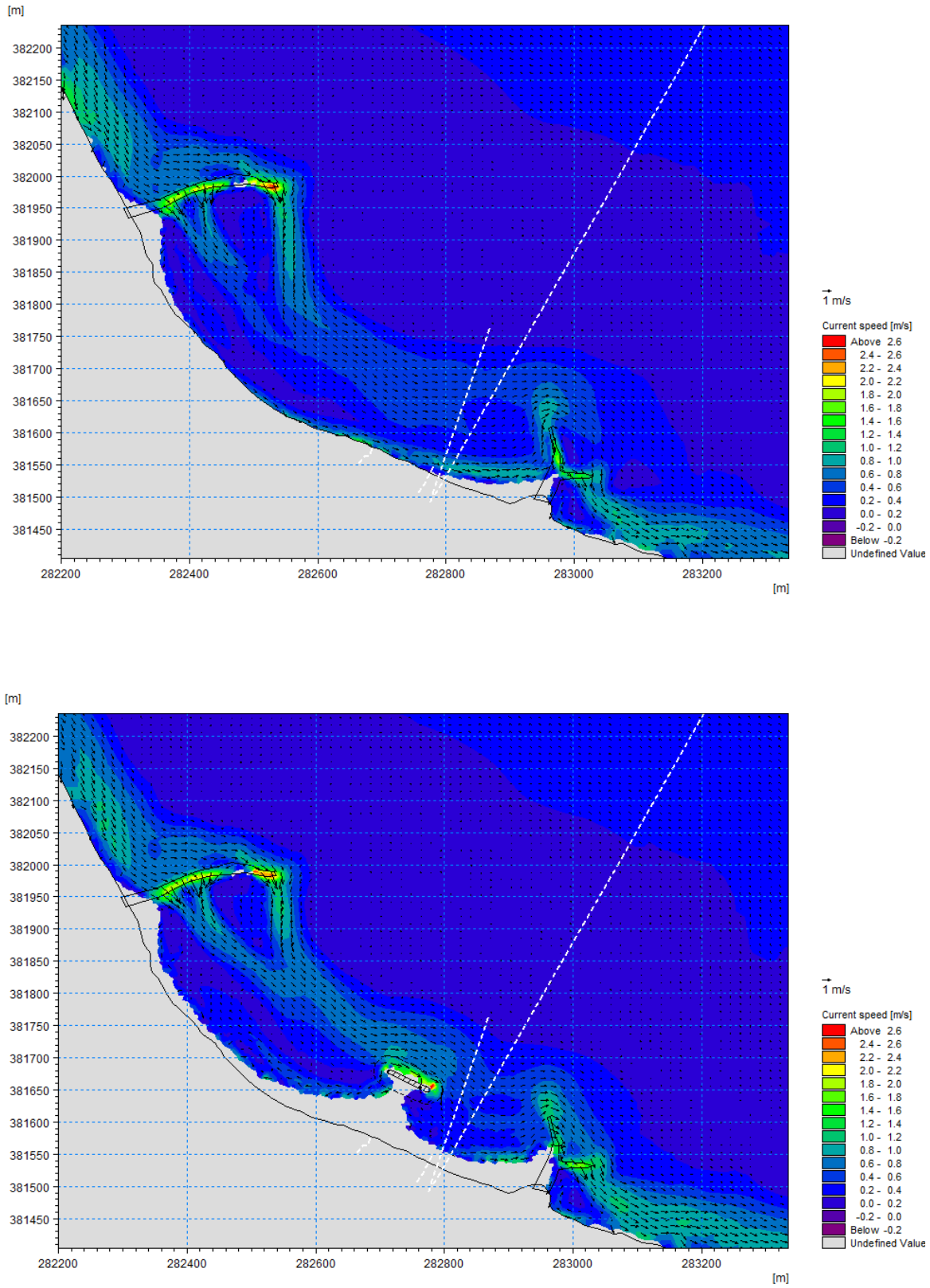


Figure 30. **Peak combined 1 in 1-year wave and tide current flow: Baseline (top)and Scheme (bottom)**

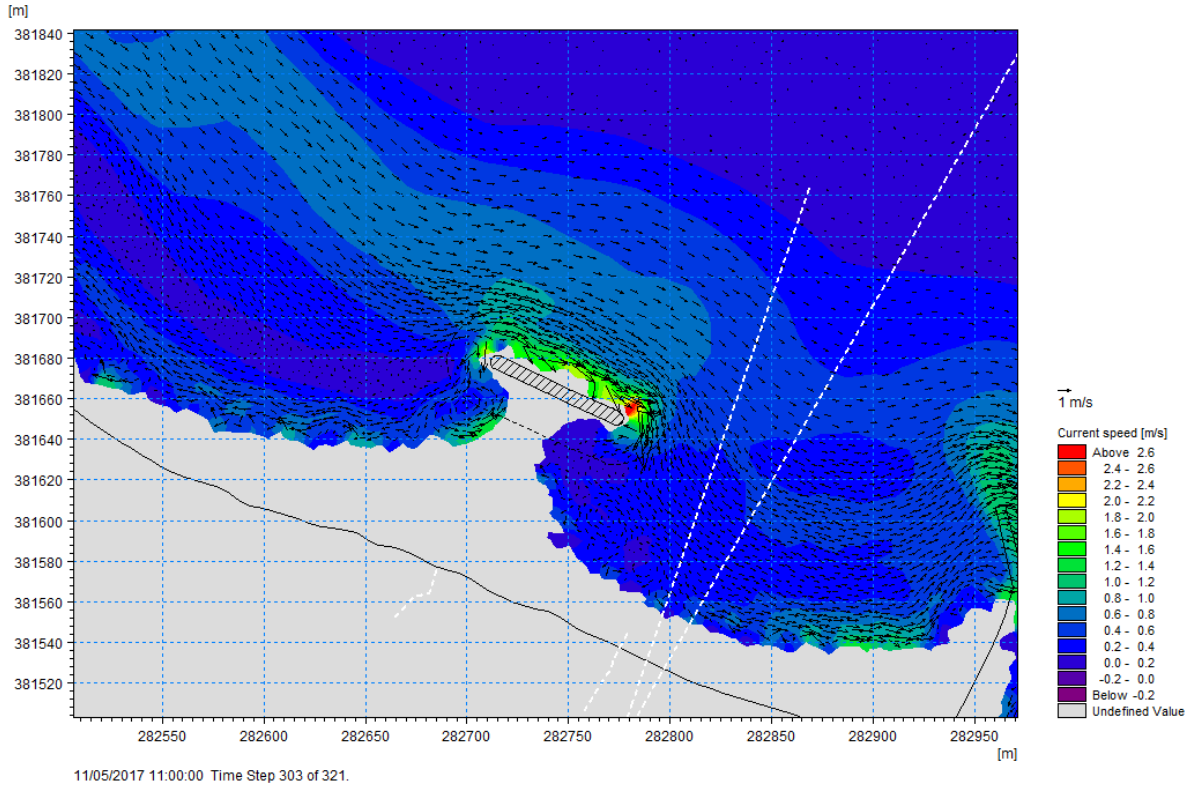


Figure 31. **Flow detail around breakwater for peak combined WNW 1 in 1-year wave and tide current flow**

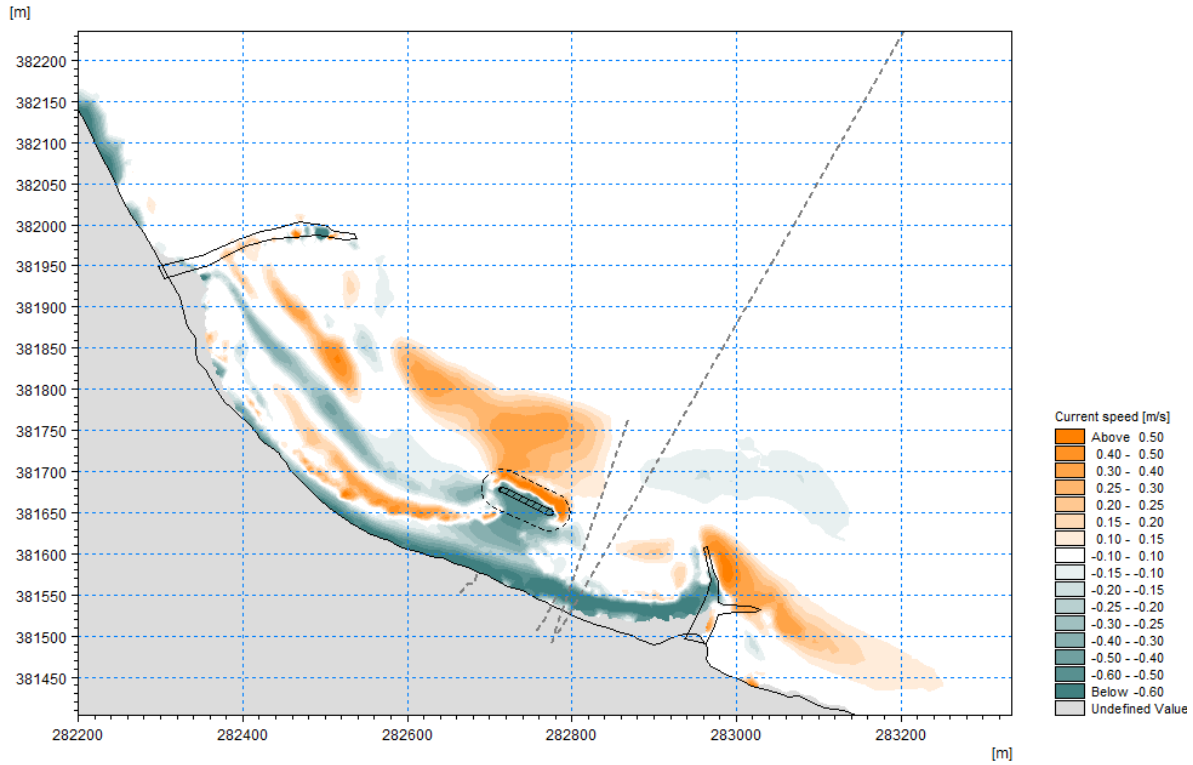


Figure 32. **Difference in peak combined 1 in 1-year wave and tide current flow**

As observed for the more extreme wave conditions the longshore wave driven current is directed across the bay in a west to east direction. The 'T' groyne again interrupts this forcing the flow to be more focused and stronger, just offshore of the 'T' groyne. However, under the 1 in 1-yr event modelled there is negligible impact on the peak flow speed observed over the Dŵr Cymru Welsh Water outfalls.

The results from the baseline conditions also show a focused west – east current, along the shoreline at the location of the previous beach reinstate campaigns. It is this wave driven current that has result in the material at this location being transport to the eastern side of the bay. Under the proposed scheme, the flows in the western embayment are now weaker with more localised circulation patterns, as they are more aligned to the incident waves.

A small eddy is also observed directly behind the western arm of 'T' groyne, with offshore directed flow close to the structure. Such flows circulations are to be expected with shore parallel structures and these can pose a risk to bathers where bathers can be taken offshore by these current. The size strength of this circulation pattern is, however, considered typical to that is predicted and could be expected on the open coast. Nevertheless, as the presence of such flow circulations directly in the lee of the breakwater are to be expected under wave conditions, it is advisable that the public are made aware of such features.

6 References

ABPmer - SEASTATES: http://www.seastates.net/

ABPmer, (2017a). Penrhyn Bay Coastal Defence Improvements, Baseline review and option development, ABPmer Report No. R.2776. A report produced by ABPmer for Ove Arup and Partners Limited, April 2017.

ABPmer (2017b). SEASTATES North West European Continental Shelf Tide and Surge Hindcast Database Model Validation Report. ABPmer, Report No. R.2784.

Defra, 2005. Use of Joint Probability Methods in Flood Management – A Guide to Best Practice. R&D Technical Report FD2308/TR2 (PB 11209). Joint Department for Environment, Food and Rural Affairs (Defra)/ Environment Agency: Flood and Coastal Defence R&D Programme. March 2005.

Environment Agency (2016), Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities

Environment Agency (2019) Coastal flood boundary conditions for the UK: update 2018. Project: SC060064/TR7: User guide.

Welsh Government (2017). Adapting to Climate Change: Guidance for Flood and Coastal Erosion Risk Management Authorities in Wales

7 Abbreviations/Acronyms

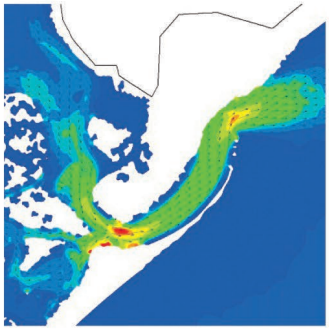
AEP	Annual Exceedance Probability
CAD	Computer-Aided Design
CCBC	Conwy County Borough Council
CFBD	Coastal Flood Boundary Database
C-MAP	Commercial marine bathymetric data source
Defra	Department for Environment, Food & Rural Affairs
DHI	Danish Hydraulic Institute
dwg	Autocad Drawing file
EA	Environment Agency
GPD	Generalized Pareto Distribution
HD	Hydrodynamic
Hs	Significant Wave Height
HW	High Water
JOINSEA	Joint Probability of Waves and Water Levels
JP	Joint Probability
JPA	Joint Probability Analysis
LiDAR	Light Detection And Ranging
Lle	Lle Geo-Portal (Welsh Government and Natural Resources Wales)
MepBay	Model of Equilibrium of Bay Beaches
MHWS	Mean High Water Springs
MIKE21	Modelling Software - hydrodynamics, waves, sediment dynamics, water quality and ecology (DHI)
OBC	Outline Business Case
ODN	Ordnance Datum Newlyn
OSGB	Ordnance Survey Great Britain
QGIS	Open-Source Cross-Platform Desktop Geographic Information System
QGIS	Open Source Geographic Information System
RMA	Risk Management Authorities
RP	Return Period
SLR	Sea Level Rise
SW	Spectral Wave
Tp	Peak Wave Period
UK	United Kingdom
UKCP09	UK Climate Projections 2009
UKHO	UK Hydrographic Office
WL	Water Level
Ws	Wind Speeds

Cardinal points/directions are used unless otherwise stated.

SI units are used unless otherwise stated.

Appendices




Innovative Thinking - Sustainable Solutions

ABP mer

A Layout 1

A.1 Layout 1 WNW wave crests



A.2 Layout 1 NNW wave crests

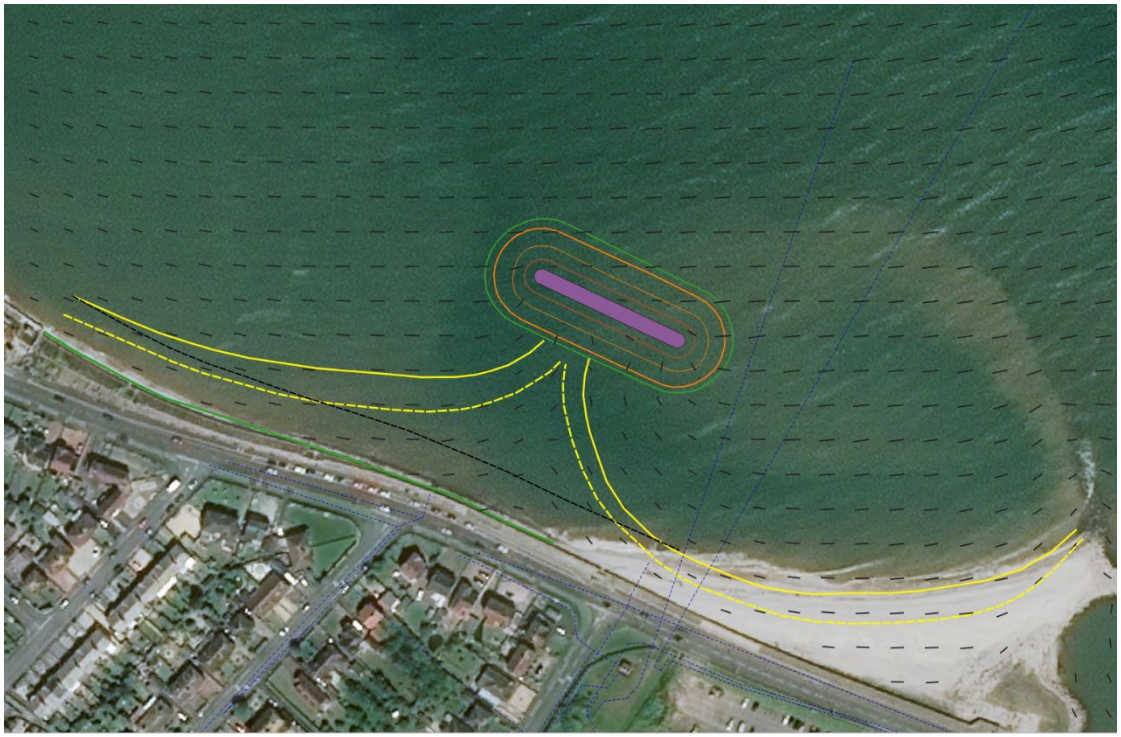


A.3 Layout 1 NNE wave crests

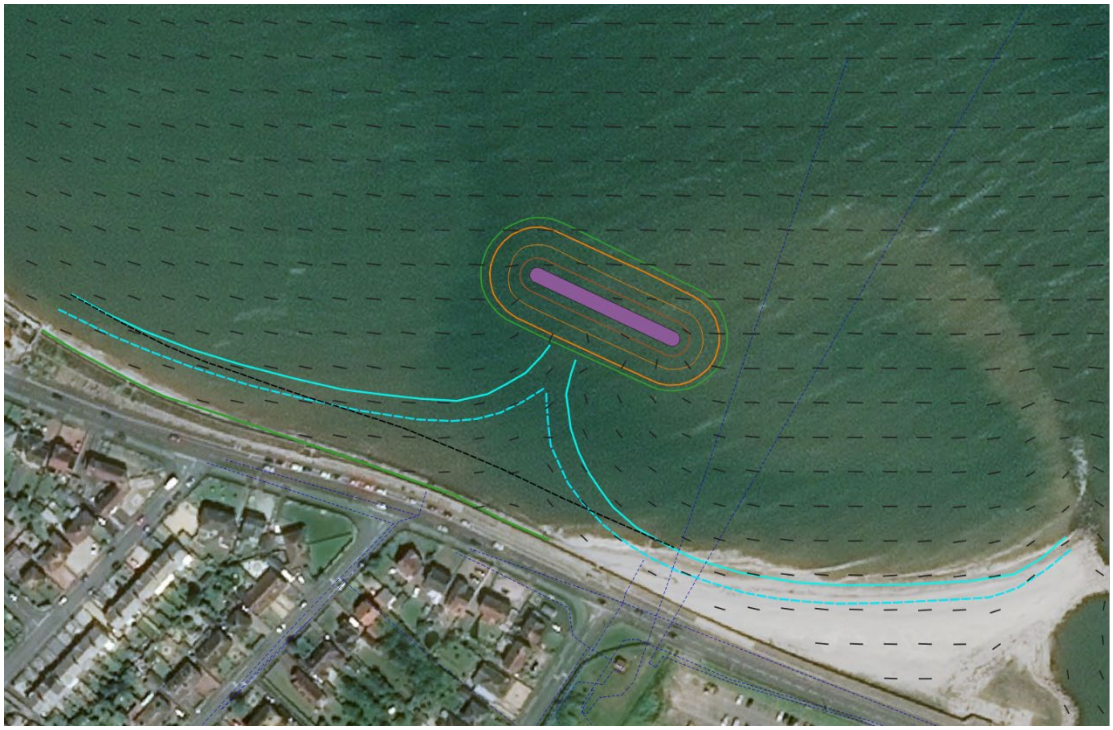


B Layout 2

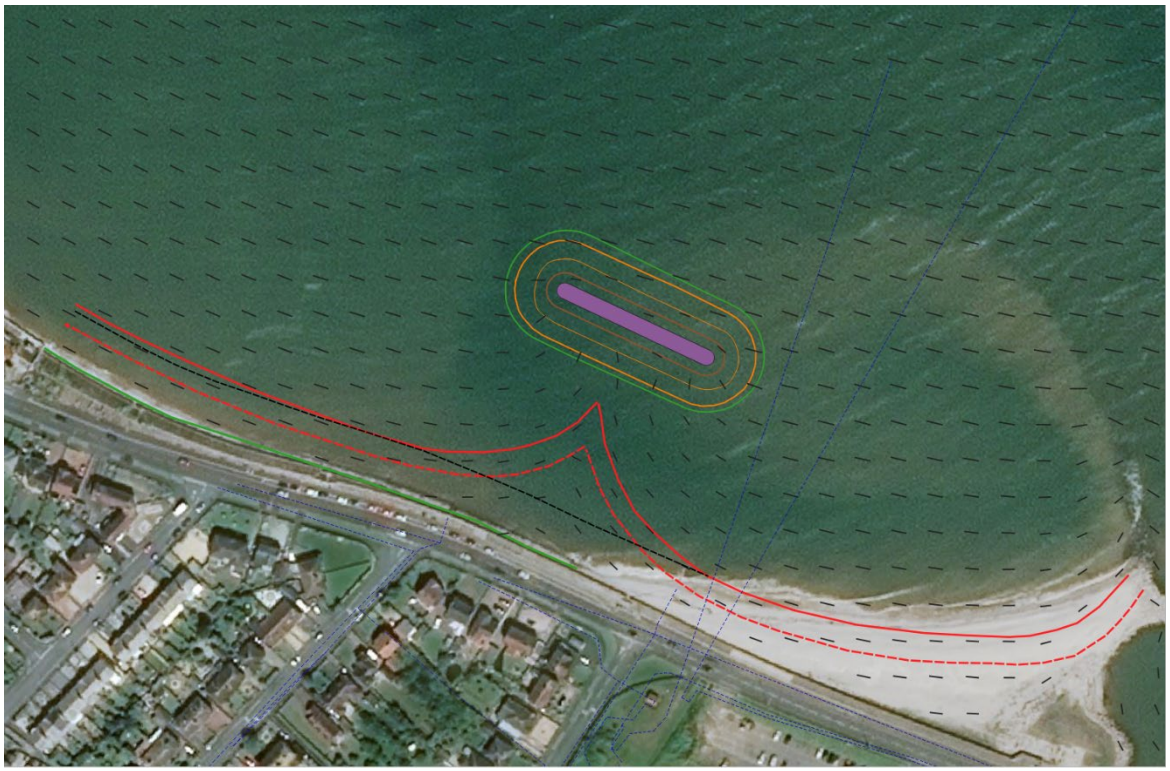
B.1 Layout 2 WNW wave crests



B.2 Layout 2 NNW wave crests

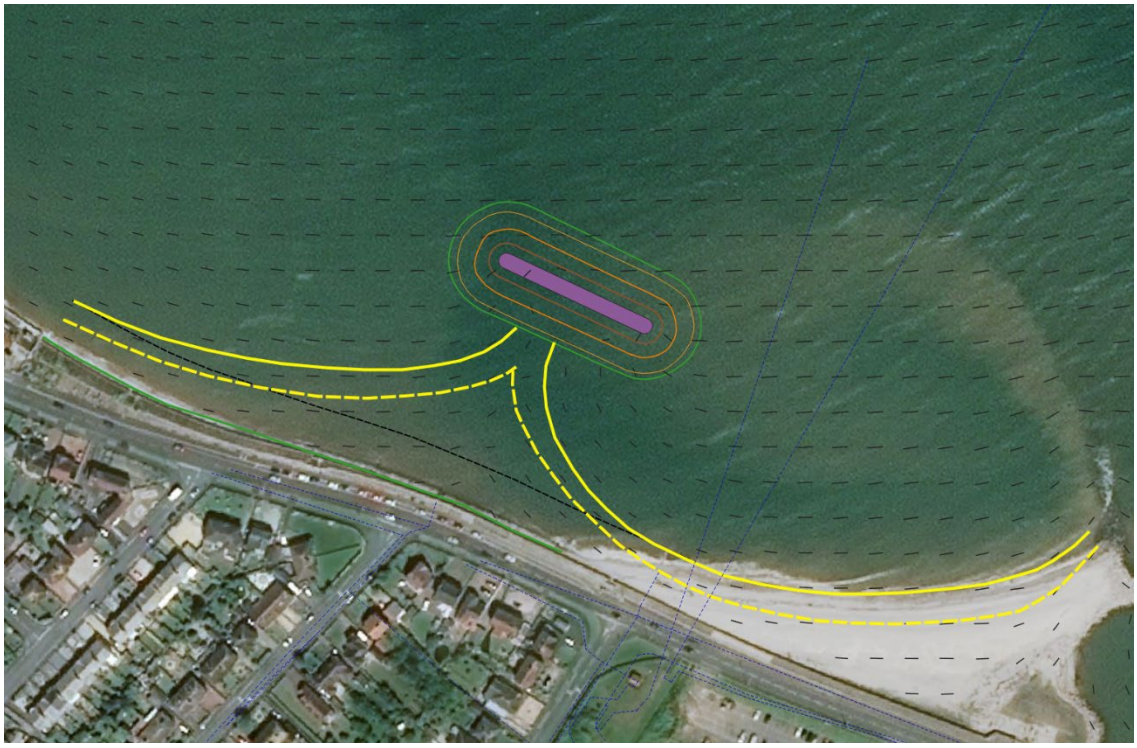


B.3 Layout 2 NNE wave crests

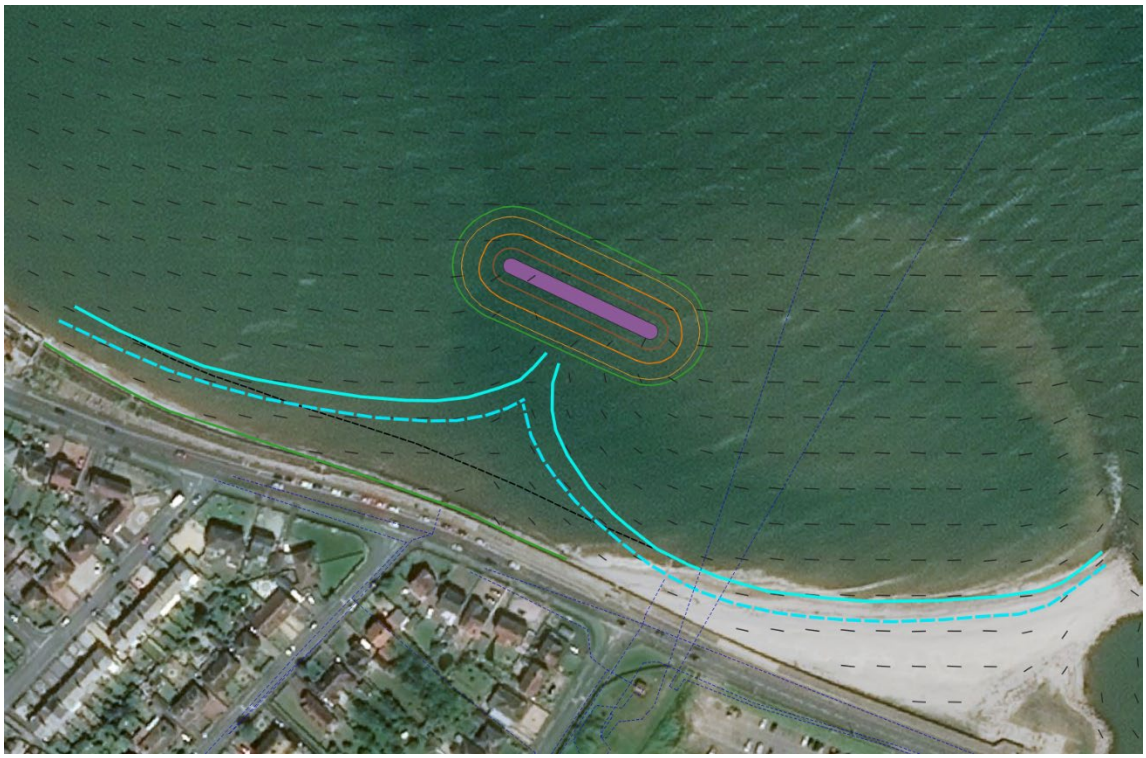


C Layout 3

C.1 Layout 3 WNW wave crests



C.2 Layout 3 NNW wave crests

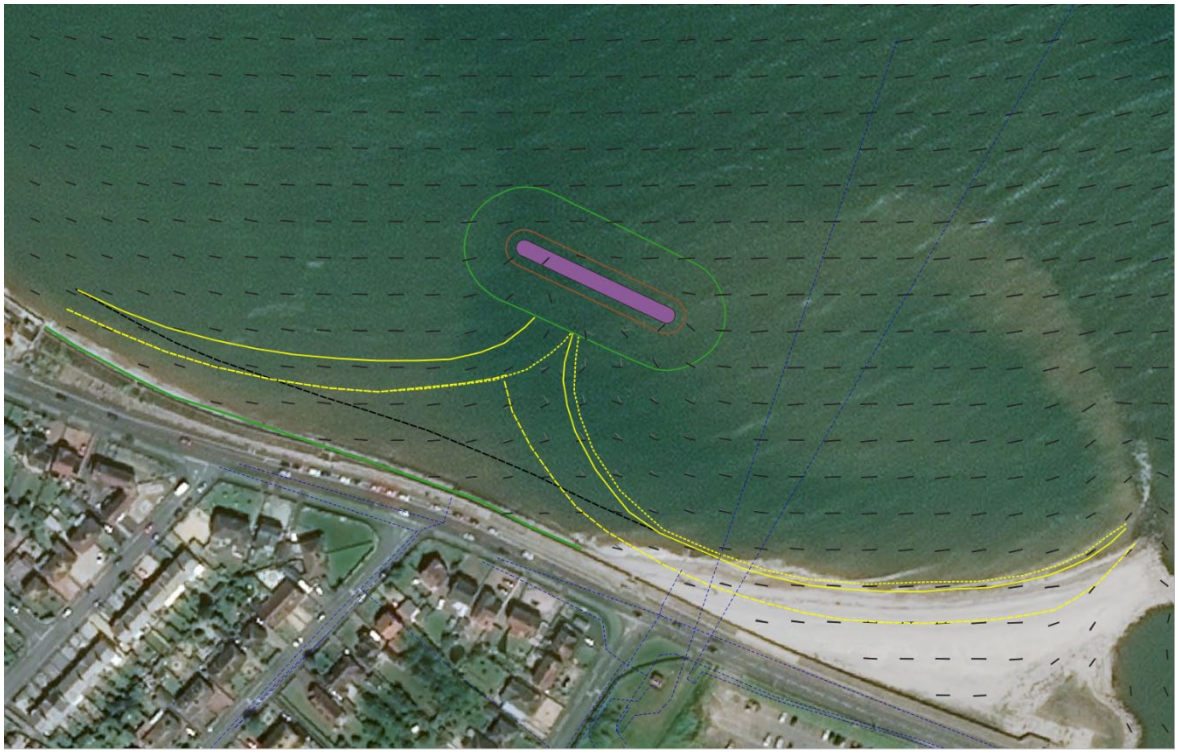


C.3 Layout 3 NNE wave crests

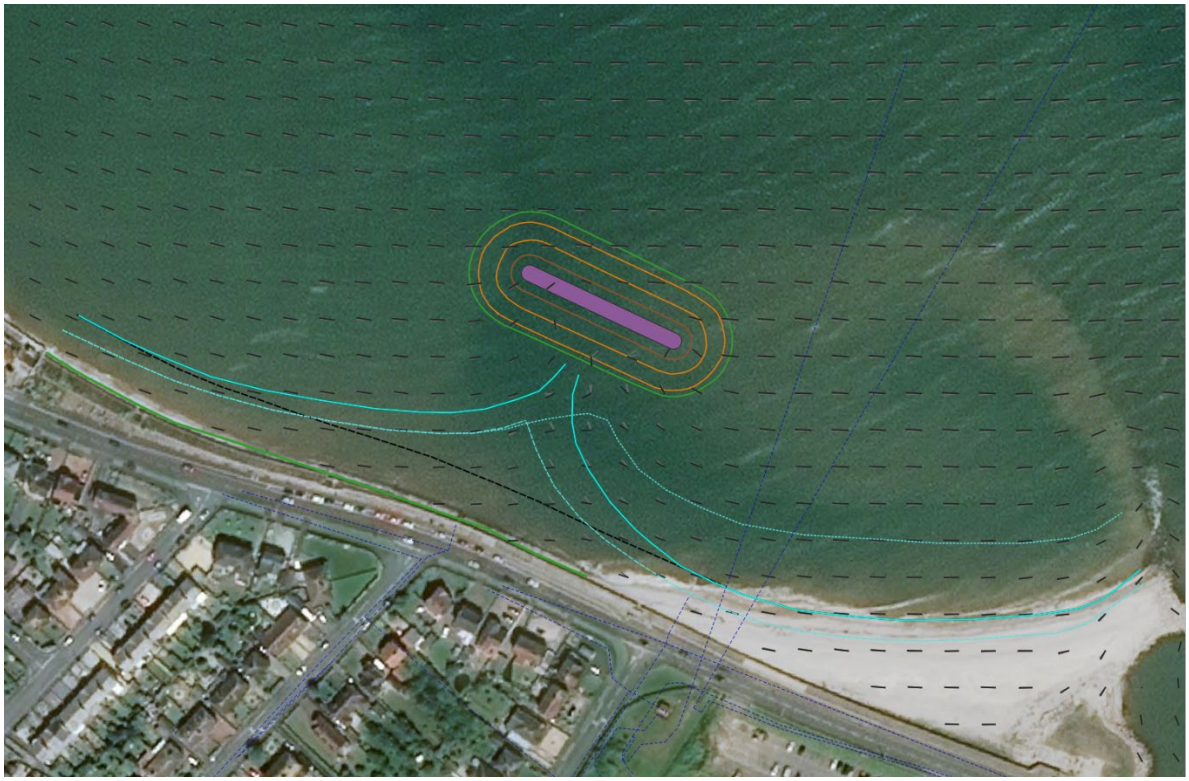


D Layout 4

D.1 Layout 4 WNW wave crests



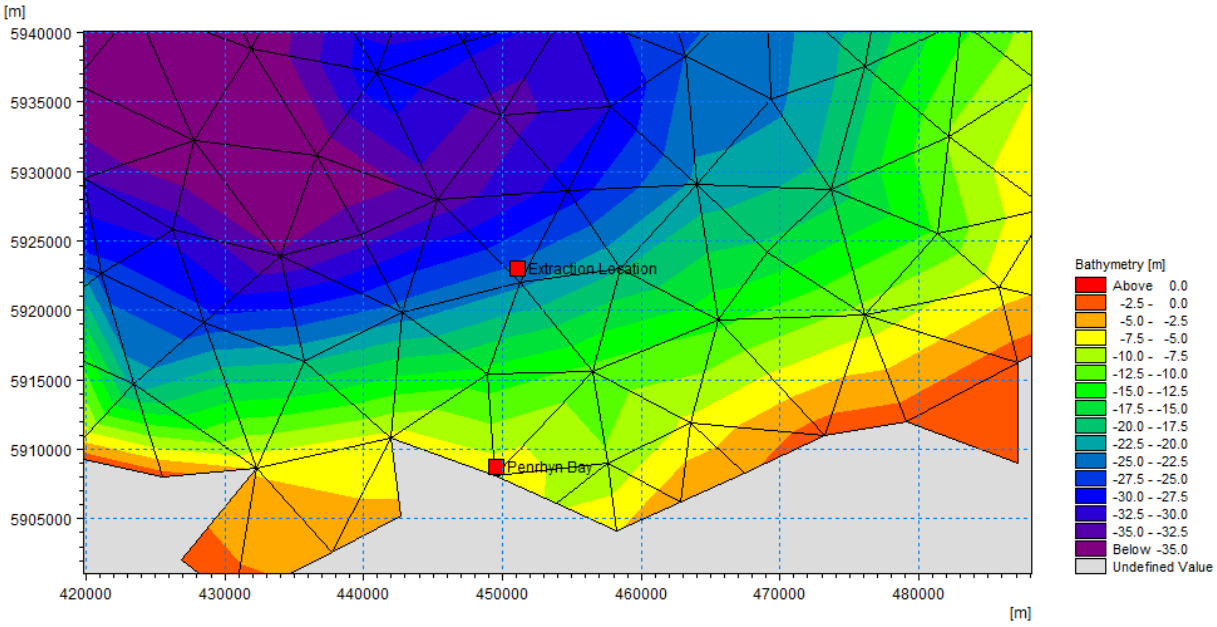
D.2 Layout 4 NNW wave crests



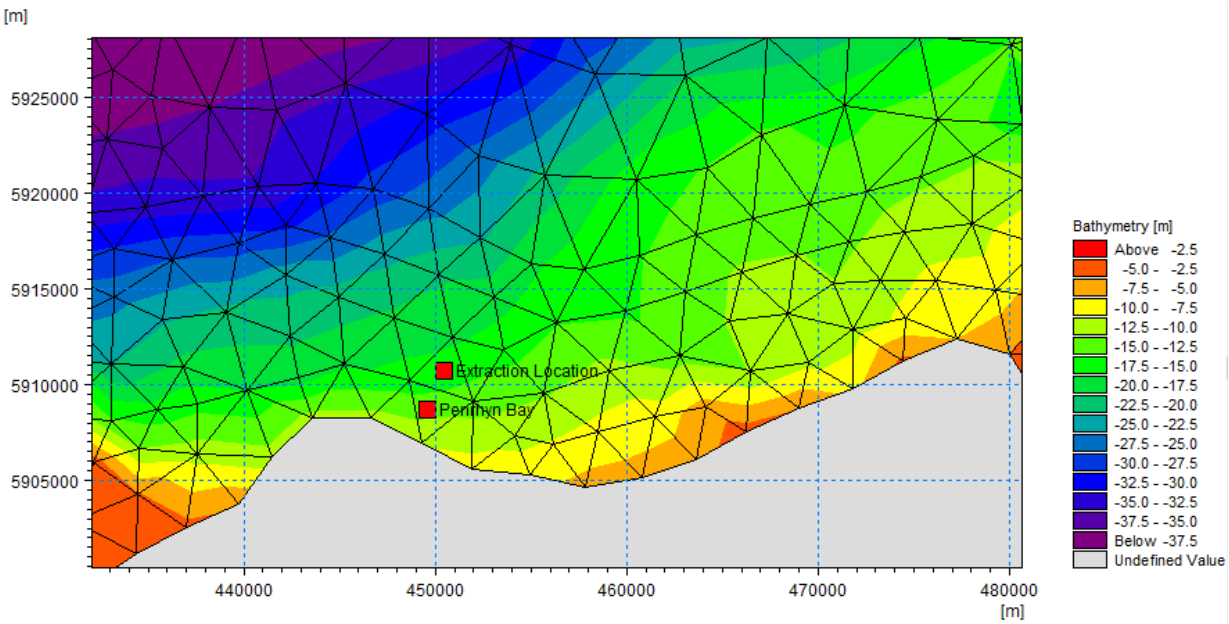
D.3 Layout 4 NNE wave crests



E SEASTATES Hindcast wave extraction location



F SEASTATES Tide and Surge extraction location



G Hindcast Wave Scatter Data

G.1 Hs vs Dir – SEASTATES winds and waves, Joint frequency distribution (All year)

			Wave Direction [FROM]																
Significant Wave Height (m)	Lower (>=)		348.75	11.25	33.75	56.25	78.75	101.25	123.75	146.25	168.75	191.25	213.75	236.25	258.75	281.25	303.75	326.25	
		Upper (<)	11.25	33.75	56.25	78.75	101.25	123.75	146.25	168.75	191.25	213.75	236.25	258.75	281.25	303.75	326.25	348.75	Sum
	7.5	8.0																	
	7.0	7.5													0.00	0.00			0.00
	6.5	7.0													0.00	0.00			0.00
	6.0	6.5													0.00	0.00			0.01
	5.5	6.0	0.00												0.00	0.01	0.00		0.01
	5.0	5.5	0.00											0.00	0.00	0.01	0.00		0.02
	4.5	5.0	0.00										0.00	0.00	0.01	0.03	0.00	0.00	0.05
	4.0	4.5	0.01	0.00									0.00	0.01	0.03	0.06	0.01	0.01	0.13
	3.5	4.0	0.01	0.00								0.00	0.02	0.02	0.06	0.16	0.03	0.02	0.32
	3.0	3.5	0.03	0.01	0.00		0.00					0.01	0.07	0.07	0.14	0.29	0.09	0.05	0.76
	2.5	3.0	0.07	0.03	0.01	0.00	0.00	0.00		0.00	0.01	0.04	0.18	0.20	0.34	0.53	0.16	0.13	1.70
	2.0	2.5	0.15	0.06	0.03	0.02	0.01	0.01	0.00	0.01	0.02	0.12	0.42	0.51	0.74	0.94	0.34	0.23	3.61
	1.5	2.0	0.33	0.14	0.12	0.07	0.03	0.03	0.03	0.04	0.08	0.28	0.80	1.12	1.52	1.55	0.66	0.49	7.30
	1.0	1.5	0.62	0.36	0.36	0.33	0.19	0.18	0.14	0.17	0.32	0.80	1.73	2.16	2.53	2.31	1.24	1.05	14.49
	0.5	1.0	1.45	1.10	1.09	1.28	1.44	1.13	0.96	0.82	1.13	2.16	3.72	4.17	4.55	4.22	2.93	2.77	34.91
	0.0	0.5	2.17	1.63	1.65	1.99	2.71	2.57	1.74	1.41	1.41	1.95	2.95	3.11	2.94	2.97	2.72	2.79	36.70
	Sum		4.85	3.33	3.26	3.69	4.39	3.91	2.86	2.44	2.97	5.37	9.88	11.36	12.87	13.08	8.19	7.55	100.00

This table is based on 40 years of data (01/01/1979 to 31/12/2018) at 1 hour intervals.

G.2 Hs vs Tp – SEASTATES winds and waves, Joint frequency distribution (All year)

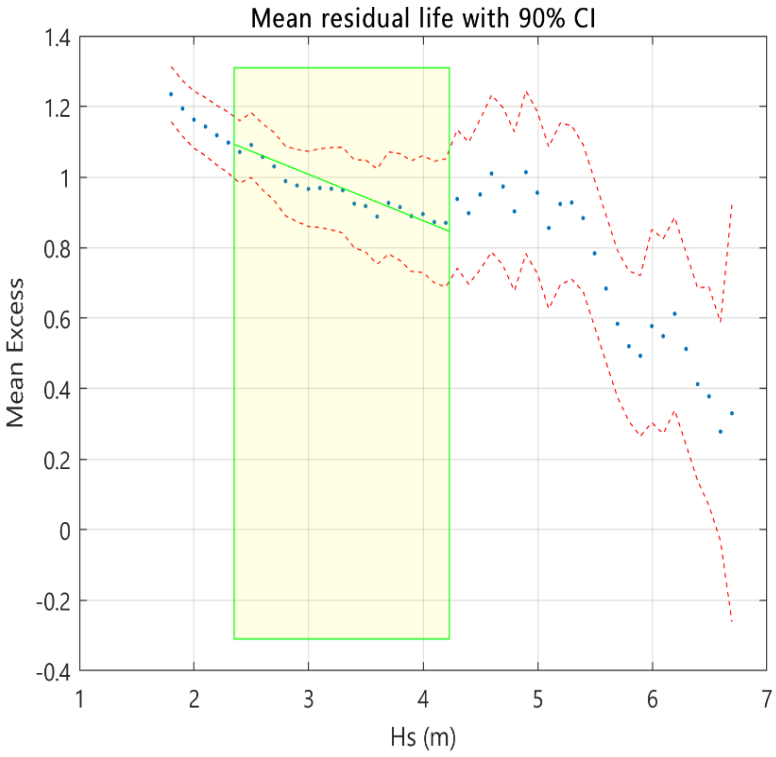
	Peak Wave Period (s)																	
Significant Wave Height (m)	Lower (>=)		0	2	4	6	8	10	12	14	16	18	20	22	24		Cum.	
		Upper (<)	2	4	6	8	10	12	14	16	18	20	22	24	26	Sum	Sum	Exced.
	7.5	8.0															100.00	0.00
	7.0	7.5						0.00								0.00	100.00	0.00
	6.5	7.0					0.00	0.00								0.00	100.00	0.00
	6.0	6.5					0.00	0.00								0.01	100.00	0.00
	5.5	6.0					0.01	0.00								0.01	99.99	0.01
	5.0	5.5				0.00	0.02	0.00								0.02	99.98	0.02
	4.5	5.0				0.00	0.05	0.00								0.05	99.96	0.04
	4.0	4.5				0.03	0.11									0.13	99.91	0.09
	3.5	4.0				0.21	0.11									0.32	99.78	0.22
	3.0	3.5				0.73	0.03									0.76	99.45	0.55
	2.5	3.0			0.05	1.64	0.00									1.70	98.69	1.31
	2.0	2.5			0.93	2.68	0.00									3.61	96.99	3.01
	1.5	2.0			5.11	2.18										7.30	93.39	6.61
	1.0	1.5		0.16	14.06	0.27	0.00									14.49	86.09	13.91
	0.5	1.0	0.00	16.60	18.29	0.02				0.00						34.91	71.61	28.39
	0.0	0.5	0.78	34.98	0.57	0.12	0.07	0.05	0.05	0.04	0.02	0.01	0.00	0.00		36.70	36.70	63.30
	Sum		0.78	51.74	39.01	7.89	0.41	0.06	0.05	0.04	0.02	0.01	0.00	0.00		100.00		
	Cumulative Sum		0.78	52.52	91.52	99.41	99.82	99.88	99.93	99.97	99.99	100.00	100.00	100.00	100.00			
	Exceedence		99.22	47.48	8.48	0.59	0.18	0.12	0.07	0.03	0.01	0.00	0.00	0.00	0.00			

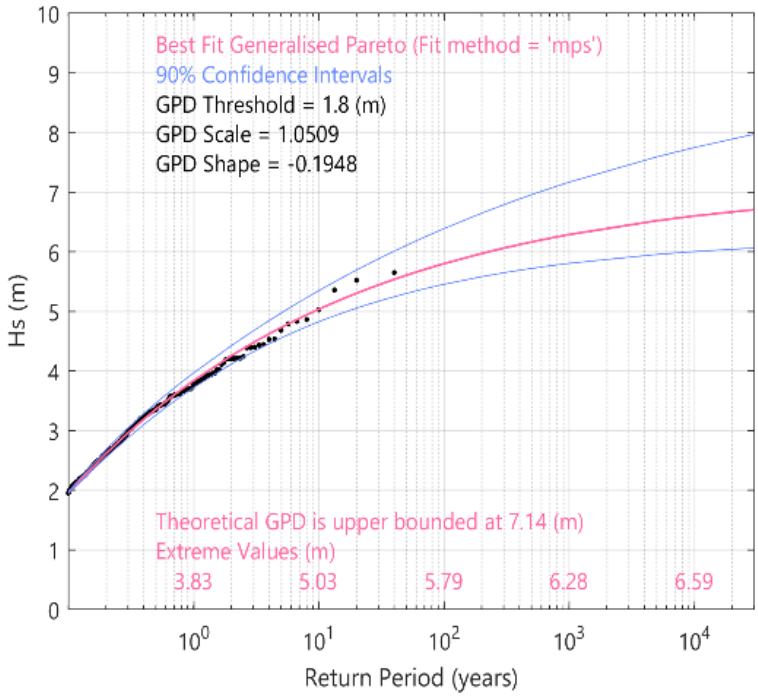
This table is based on 40 years of data (01/01/1979 to 31/12/2018) at 1 hour intervals.

H Univariate Extremes and JP Curves

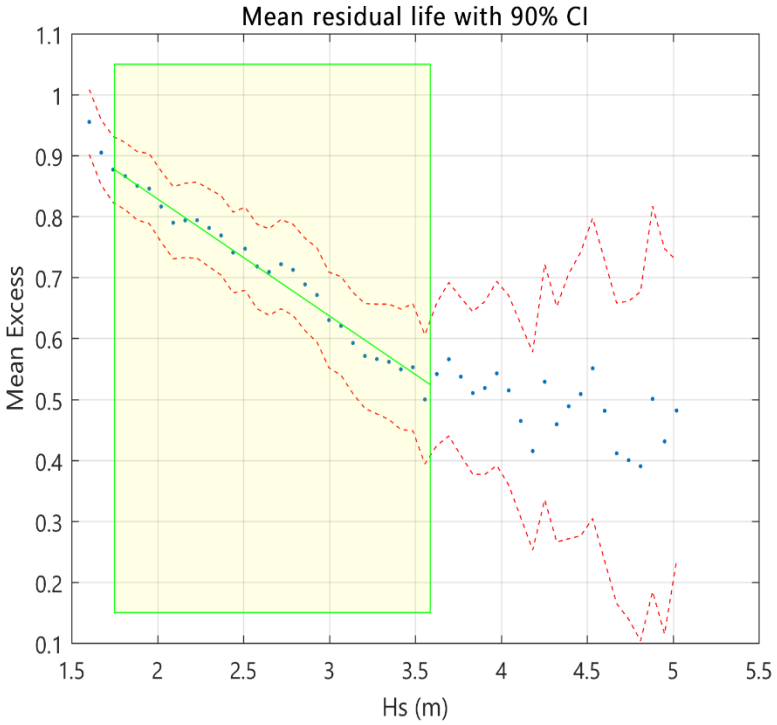
H.1 Univariate Extremes

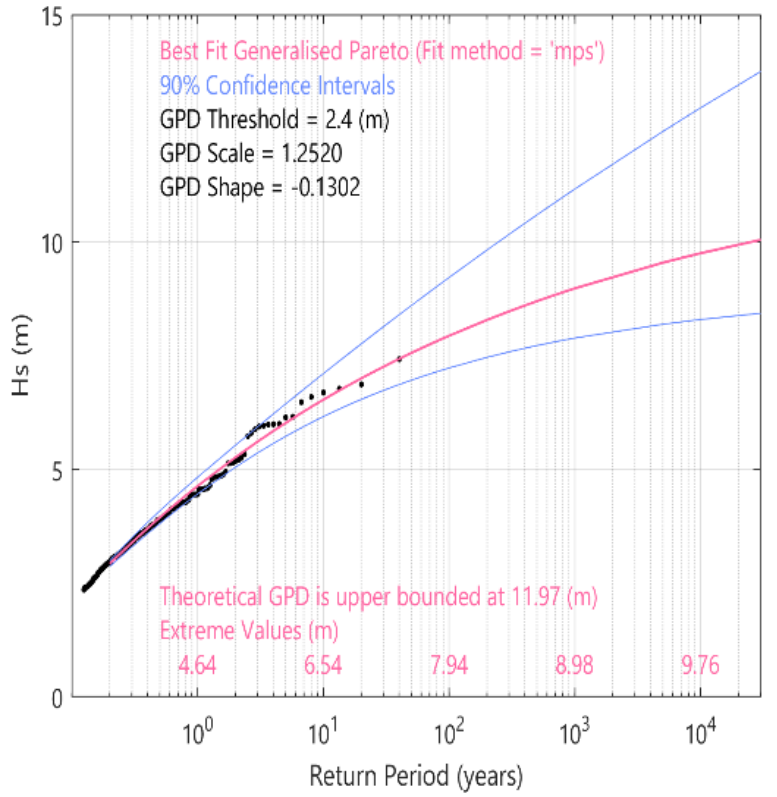
Northerly directional sector





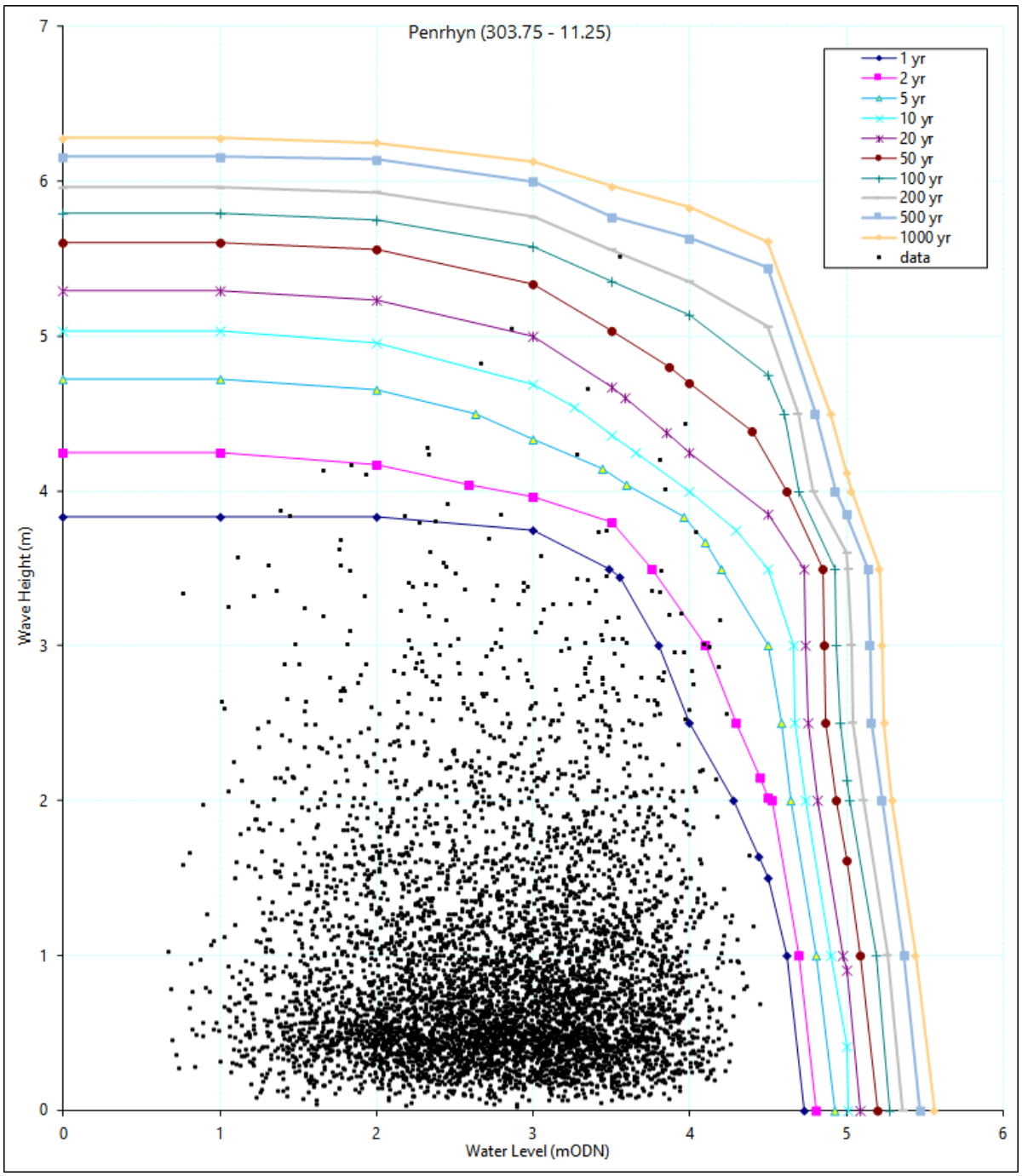
WNW directional sector



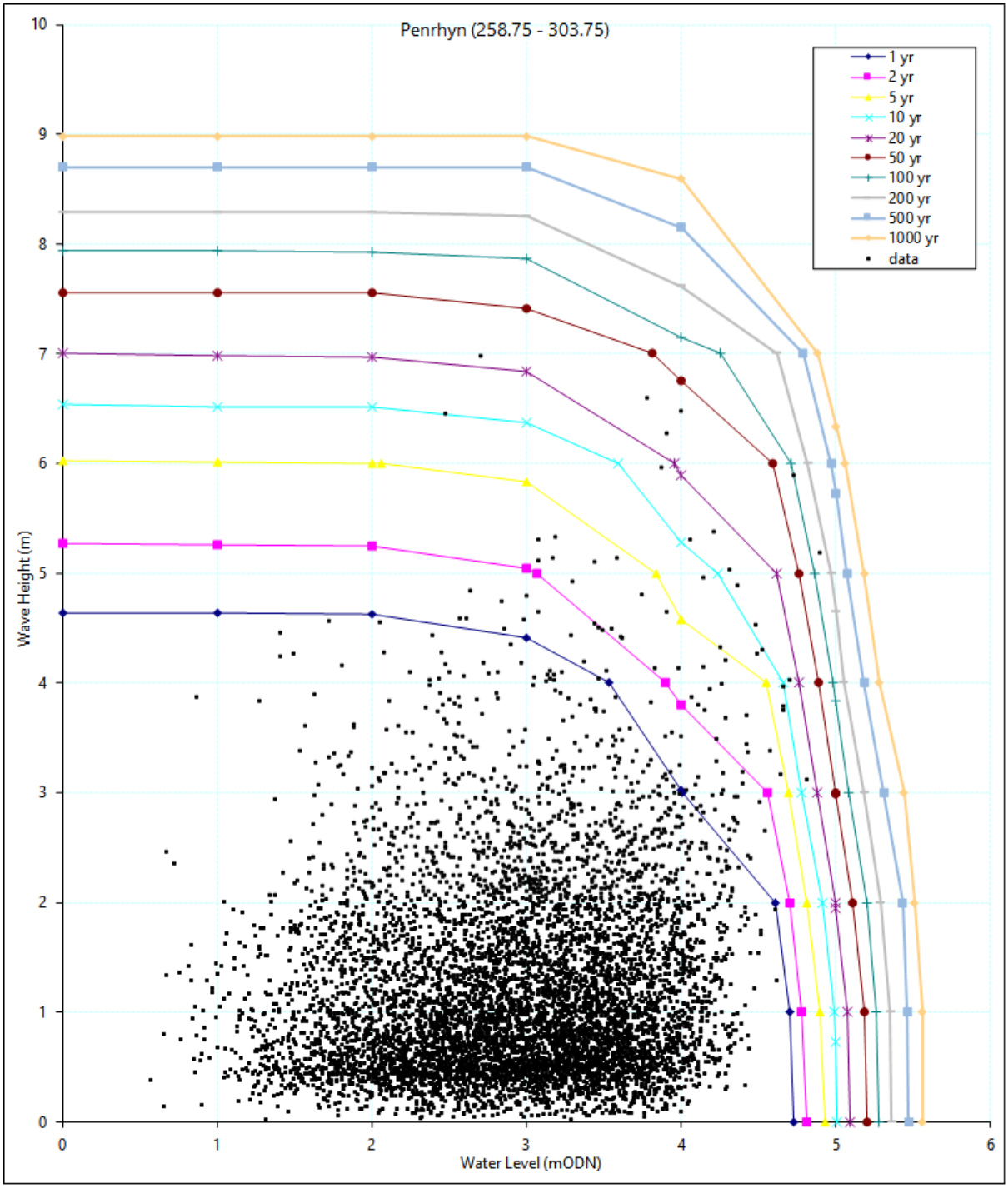


H.2 JP Curves

Northerly directional sector



WNW directional sector



I Welsh Government 2017 Climate Change Advice

5.1.1 Mean Relative Sea Level Change

The current industry guidelines in relation to climate change are those provided in 'Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities in Wales' (Welsh Government, 2017). This guidance is to be used by risk management authorities when making investment decisions. The guidance is shown in Figure 1.

3. Change to relative mean sea levels

A single regional allowance for the annual change for the Welsh coastline is presented in **Table 4**. It is recommended that RMAs use these figures for their investment decisions. These projections are consistent with the latest global predictions for sea level rise. The rate of change is projected to increase in each epoch.

Projections beyond 2115 should be derived by extrapolating beyond 2115.

	Sea level rise mm/yr up to 2025	**Sea level rise mm/yr 2026 to 2055**	**Sea level rise mm/yr 2056 to 2085**	**Sea level rise mm/yr 2086 to 2116**
H++ scenario	6	12.5	24	33
Annual change (mm/yr)	**3.5**	**8.0**	**11.5**	**14.5**

Table 4 - Mean sea level allowance (compared to 1990 baseline, includes land movements)

Table 4 also provides the H++ scenario as presented within the UKCP09 marine report to provide users with estimates of sea level rise increase beyond the likely range but within physical plausibility.

It is envisaged that only where the consequences of flooding or erosion could be extreme would the H++ scenario need to be considered within assessments for sensitivity purposes covering the period to 2115.

Figure 1 - Extract from 'Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities in Wales' (Welsh Government, 2017)

5.2.2 Wind and waves

Welsh Government (2017) guidance states:

"Change to wave climate is presented within UKCP09. There are large uncertainties especially with the projected extreme values.

Projections of changes to wave climate in UKCP09 are given for return periods up to 50 years. Projections of longer return period wave heights will reflect the same pattern of change but with larger error bars. Changes in wave period and direction are small and more difficult to interpret.

Due to the significant uncertainty both to the future position of the storm track over the UK and the projections of wave climate within UKCP09, it is recommended that RMAs employ a sensitivity analysis to understand the impact on flood risk and coastal change, and the form of any feasible options. Recommended national precautionary sensitivity ranges for offshore wind speed and wave height are presented in Table [2]."

Table 2 - Recommended national precautionary sensitivity ranges for offshore wind speed and wave height

Parameter	1990 to 2025	2025 to 2055	2055 to 2085	2085 to 2115
Offshore wind speed	+5%		+10%	
Extreme wave height	+5%		+10%	

The same guidance is also provided in Environment Agency (2016). However, no guidance is provided as to how the sensitivity should be considered.

J CFBD Extraction

J.1 CFBD location used is in Yellow (1116)



K Full JP Pairs

Highlighted values indicate the wave/water level pairs selected for wave transformation modelling.

K.1 WNW sector JP Pairs

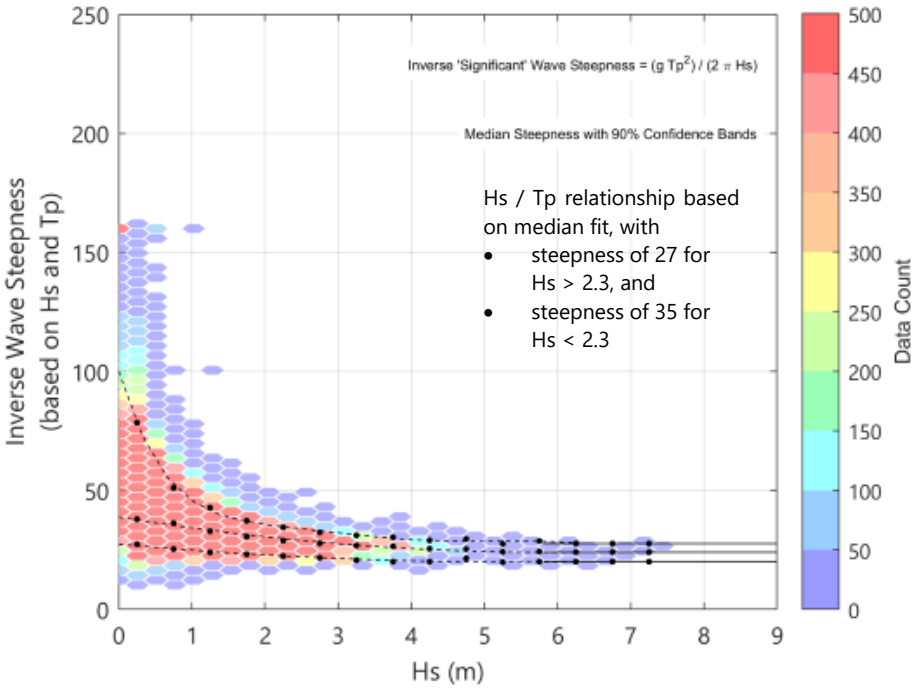
1 in 1		1 in 10		1 in 50		1 in 100		1 in 200	
WL (mODN)	Hs (m)	WL (mODN)	Hs (m)	WL (mODN)	Hs (m)	WL (mODN)	Hs (m)	WL (mODN)	Hs (m)
0.0	4.6	0.0	6.5	0.0	7.6	0.0	7.9	0.0	8.3
1.0	4.6	1.0	6.5	1.0	7.6	1.0	7.9	1.0	8.3
2.0	4.6	2.0	6.5	2.0	7.6	2.0	7.9	2.0	8.3
3.0	4.4	3.0	6.4	3.0	7.4	3.0	7.9	3.0	8.3
3.5	4.0	3.6	6.0	3.8	7.0	4.0	7.2	4.0	7.6
4.0	3.0	4.0	5.3	4.0	6.8	4.3	7.0	4.6	7.0
4.0	3.0	4.2	5.0	4.6	6.0	4.7	6.0	4.8	6.0
4.6	2.0	4.7	4.0	4.8	5.0	4.9	5.0	5.0	5.0
4.7	1.0	4.8	3.0	4.9	4.0	5.0	4.0	5.0	4.7
4.7	0.0	4.9	2.0	5.0	3.0	5.0	3.8	5.1	4.0
		5.0	1.0	5.0	3.0	5.1	3.0	5.2	3.0
		5.0	0.7	5.1	2.0	5.2	2.0	5.3	2.0
		5.0	0.0	5.2	1.0	5.3	1.0	5.4	1.0
				5.2	0.0	5.3	0.0	5.4	0.0

K.2 Northerly sector JP Pairs

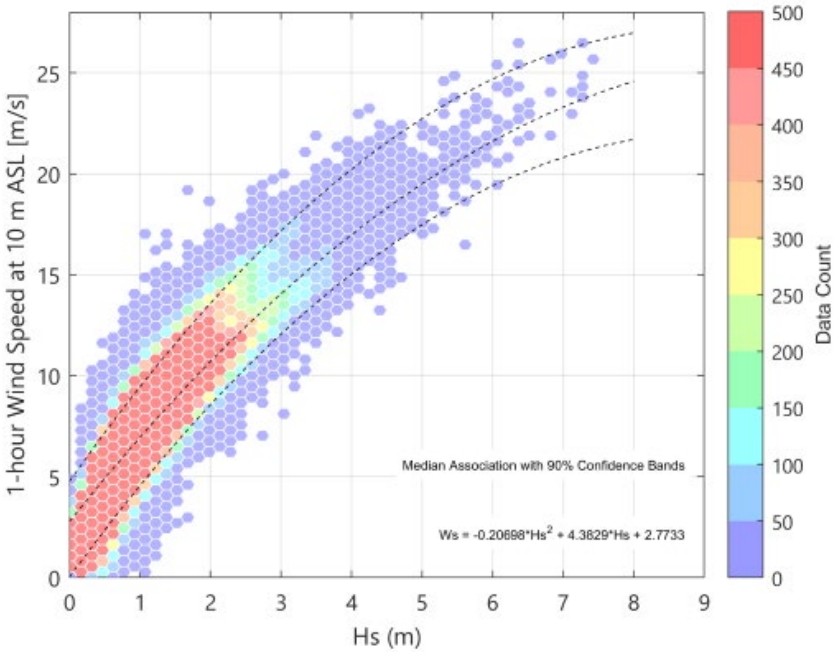
1 in 1		1 in 10		1 in 50		1 in 100		1 in 200	
WL (mODN)	Hs (m)	WL (mODN)	Hs (m)	WL (mODN)	Hs (m)	WL (mODN)	Hs (m)	WL (mODN)	Hs (m)
0.0	3.8	0.0	5.0	0.0	5.6	0.0	5.8	0.0	6.0
1.0	3.8	1.0	5.0	1.0	5.6	1.0	5.8	1.0	6.0
2.0	3.8	2.0	5.0	2.0	5.6	2.0	5.8	2.0	5.9
3.0	3.8	3.0	4.7	3.0	5.3	3.0	5.6	3.0	5.8
3.5	3.5	3.3	4.5	3.5	5.0	3.5	5.4	3.5	5.6
3.6	3.4	3.5	4.4	3.9	4.8	4.0	5.1	4.0	5.4
3.8	3.0	3.7	4.3	4.0	4.7	4.5	4.8	4.5	5.1
4.0	2.5	4.0	4.0	4.4	4.4	4.6	4.5	4.7	4.5
4.3	2.0	4.3	3.8	4.6	4.0	4.7	4.0	4.8	4.0
4.4	1.6	4.5	3.5	4.9	3.5	4.9	3.5	5.0	3.6
4.5	1.5	4.7	3.0	4.9	3.0	4.9	3.0	5.0	3.5
4.6	1.0	4.7	2.5	4.9	2.5	5.0	2.5	5.0	3.0
4.7	0.0	4.7	2.0	4.9	2.0	5.0	2.1	5.0	2.5
		4.9	1.0	5.0	1.6	5.0	2.0	5.1	2.0
		5.0	0.4	5.1	1.0	5.2	1.0	5.3	1.0
		5.0	0.0	5.2	0.0	5.3	0.0	5.4	0.0

L Omnidirectional Hs Relationships

L.1 Hs / Tp relationship



L.2 Hs / Ws relationship



M JP Nearshore Wave Results

M.1 WNW Sector JP Results

Return Period	AEP (%)	P1_Old Colwyn: Sign. Wave Height (m)	P2_Old Colwyn: Sign. Wave Height (m)	P3_Old Colwyn: Sign. Wave Height (m)	P4_Old Colwyn: Sign. Wave Height (m)	P5_Old Colwyn: Sign. Wave Height (m)
2020						
T20	**5**	2.0	2.0	2.0	2.0	1.9
		2.3	2.4	2.5	2.5	2.4
		1.1	1.2	1.2	1.3	1.3
T50	**2**	2.5	2.6	2.6	2.7	2.7
		2.7	2.9	2.9	3.0	2.9
		1.6	1.7	1.8	1.9	1.9
T100	**1**	2.6	2.7	2.7	2.7	2.8
		3.1	3.2	3.3	3.4	3.3
		1.6	1.7	1.8	1.9	1.9
T200	**0.5**	2.6	2.7	2.7	2.7	2.7
		3.1	3.3	3.3	3.4	3.4
		1.6	1.7	1.8	1.9	1.9
2070						
T20	**5**	2.1	2.2	2.2	2.2	2.2
		2.4	2.5	2.6	2.7	2.6
		1.1	1.2	1.2	1.3	1.3
T50	**2**	2.7	2.8	2.8	2.9	2.9
		2.9	3.0	3.1	3.2	3.3
		1.7	1.8	1.9	2.1	2.0
T100	**1**	2.8	2.9	2.9	3.0	3.1
		3.3	3.4	3.5	3.6	3.7
		1.7	1.8	1.9	2.0	2.0
T200	**0.5**	2.8	2.9	2.9	3.0	3.1
		3.3	3.5	3.5	3.6	3.7
		1.8	1.8	1.9	2.0	2.0
2120						
T20	**5**	2.4	2.5	2.5	2.6	2.5
		2.6	2.7	2.8	3.0	2.8
		1.2	1.3	1.3	1.4	1.4
T50	**2**	3.0	3.1	3.1	3.2	3.2
		3.2	3.3	3.4	3.5	3.4
		1.9	2.0	2.0	2.2	2.1
T100	**1**	3.1	3.2	3.2	3.4	3.4
		3.6	3.7	3.8	3.9	3.9
		1.9	2.0	2.0	2.2	2.1
T200	**0.5**	3.1	3.2	3.2	3.3	3.4
		3.6	3.7	3.8	3.9	3.8
		1.9	2.0	2.0	2.2	2.1

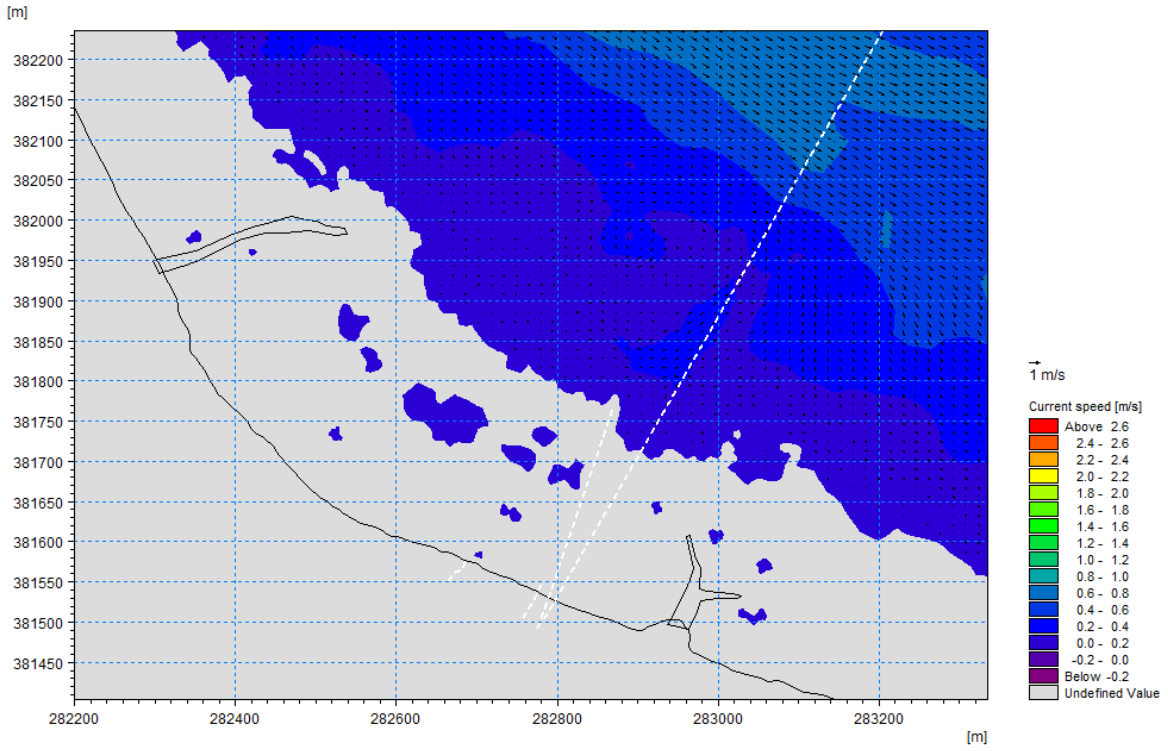
M.2 Northerly Sector JP Results

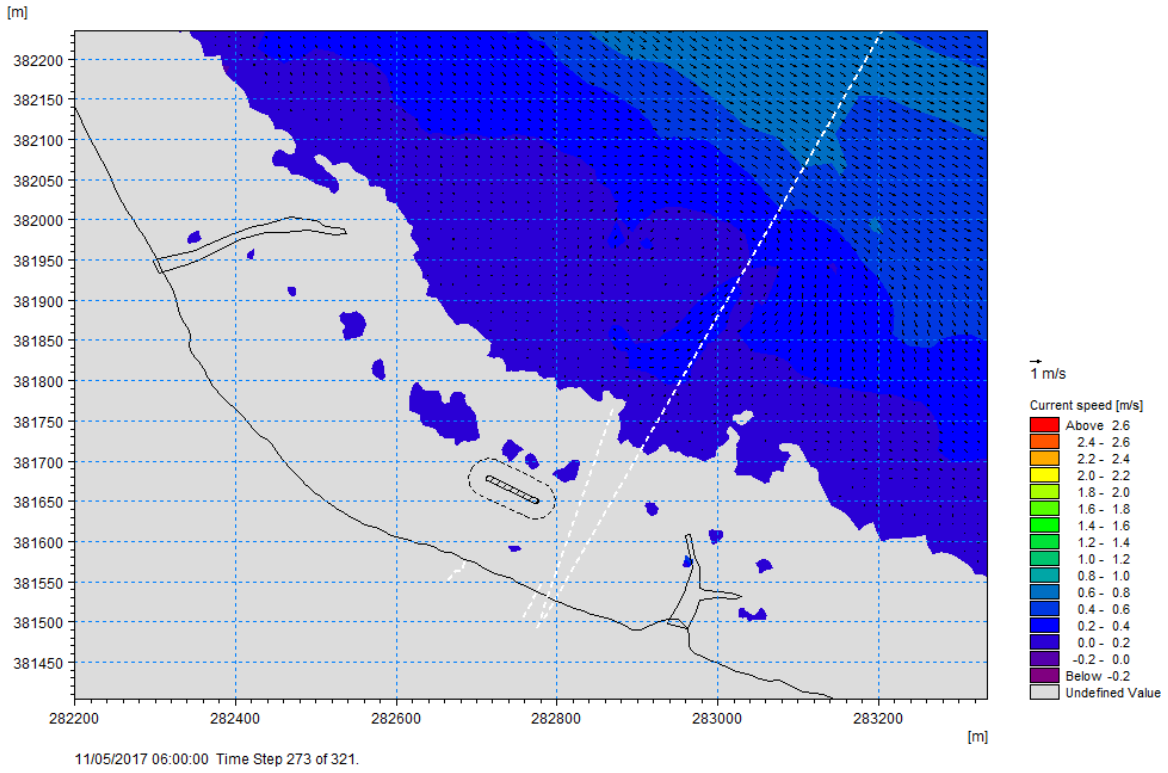
Return Period	AEP (%)	P1_Old Colwyn: Sign. Wave Height (m)	P2_Old Colwyn: Sign. Wave Height (m)	P3_Old Colwyn: Sign. Wave Height (m)	P4_Old Colwyn: Sign. Wave Height (m)	P5_Old Colwyn: Sign. Wave Height (m)
2020						
T20	**5**	2.3	2.4	2.4	2.5	2.4
		2.3	2.4	2.4	2.5	2.4
		1.1	1.1	1.2	1.3	1.2
T50	**2**	2.5	2.6	2.6	2.7	2.6
		2.8	2.9	2.9	3.0	3.0
		1.7	1.8	1.9	2.0	1.9
T100	**1**	2.6	2.7	2.7	2.8	2.8
		3.1	3.2	3.2	3.3	3.2
		1.9	2.0	2.1	2.2	2.1
T200	**0.5**	2.9	3.0	3.0	3.0	3.1
		3.1	3.2	3.2	3.3	3.2
		2.0	2.0	2.1	2.2	2.1
2070						
T20	**5**	2.5	2.6	2.6	2.7	2.6
		2.4	2.5	2.6	2.7	2.6
		1.2	1.2	1.3	1.3	1.3
T50	**2**	2.7	2.8	2.8	2.9	2.8
		3.0	3.1	3.1	3.2	3.1
		1.9	2.0	2.0	2.2	2.1
T100	**1**	2.8	2.9	3.0	3.0	3.0
		3.3	3.4	3.5	3.5	3.4
		2.1	2.1	2.2	2.3	2.3
T200	**0.5**	3.1	3.2	3.2	3.3	3.3
		3.3	3.4	3.4	3.5	3.5
		2.1	2.2	2.3	2.4	2.3
2120						
T20	**5**	2.8	2.9	2.9	3.0	2.9
		2.6	2.7	2.8	2.9	2.8
		1.2	1.3	1.3	1.4	1.4
T50	**2**	3.1	3.1	3.2	3.3	3.5
		3.2	3.3	3.4	3.5	3.5
		2.0	2.1	2.2	2.3	2.2
T100	**1**	3.2	3.2	3.3	3.4	3.4
		3.6	3.7	3.8	3.8	3.8
		2.2	2.3	2.4	2.5	2.4
T200	**0.5**	3.4	3.5	3.5	3.6	3.8
		3.6	3.7	3.8	3.9	3.8
		2.3	2.4	2.4	2.5	2.5

N Coupled Wave and Tide Model Outputs

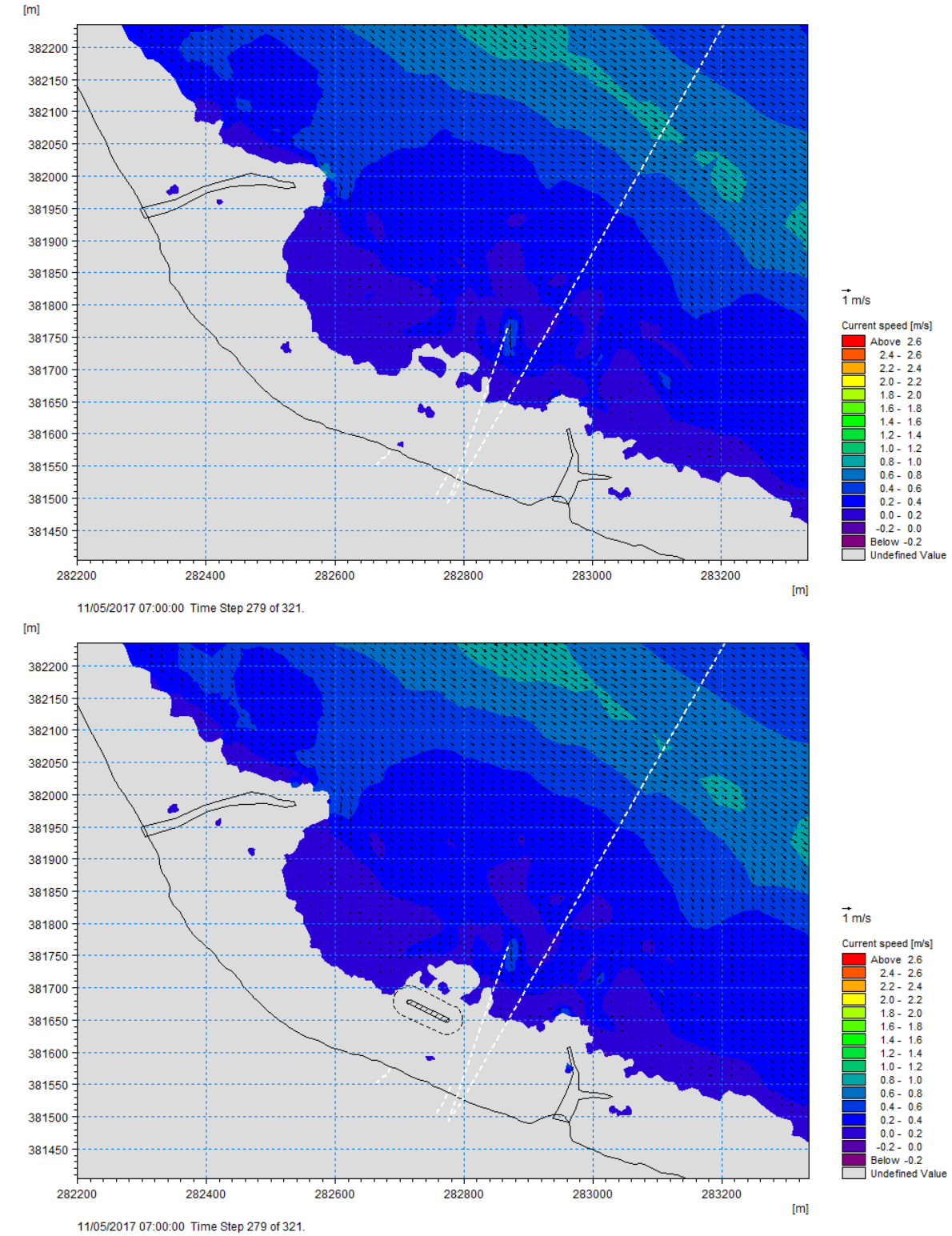
N.1 1 in 1-year event

HW -5 hr

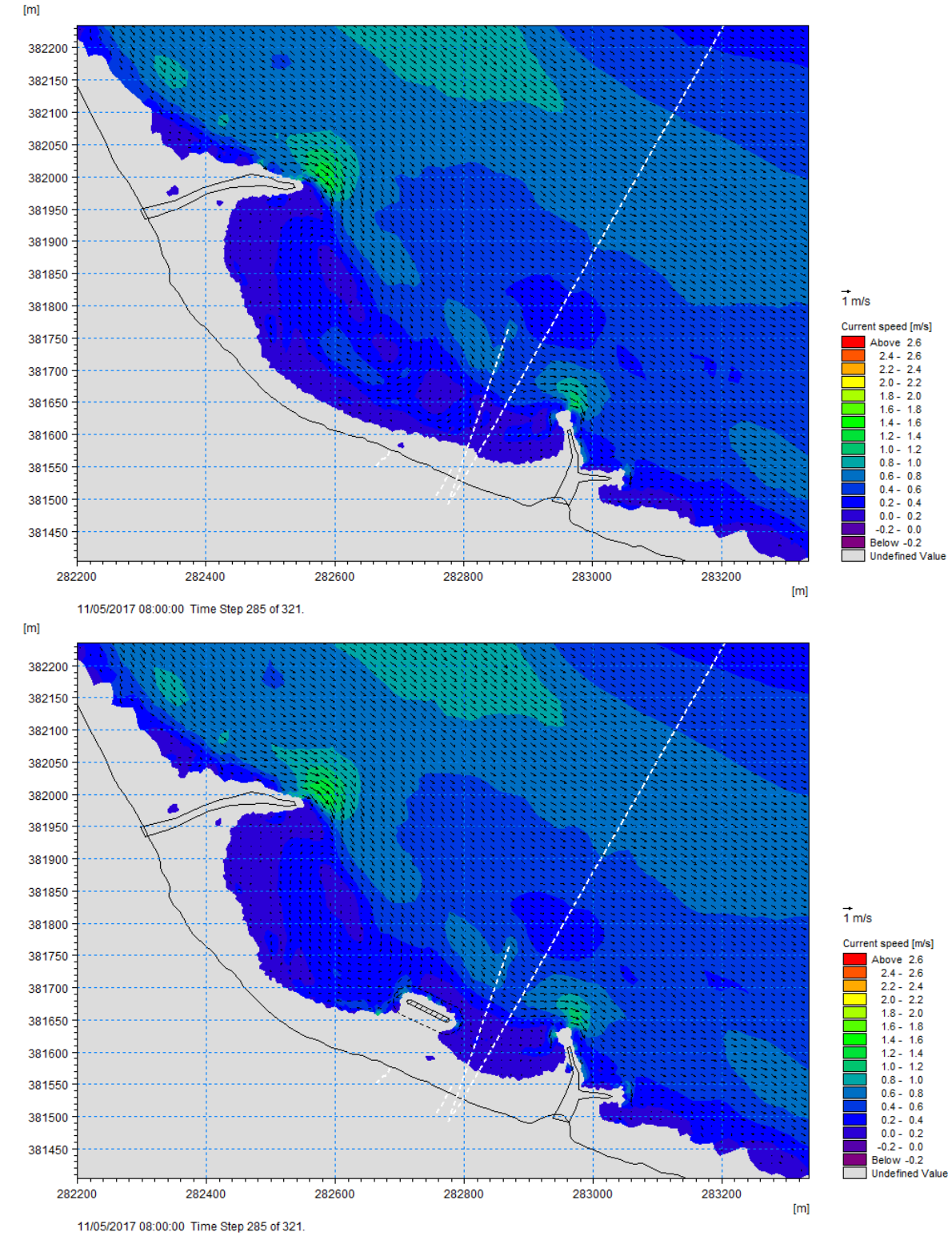




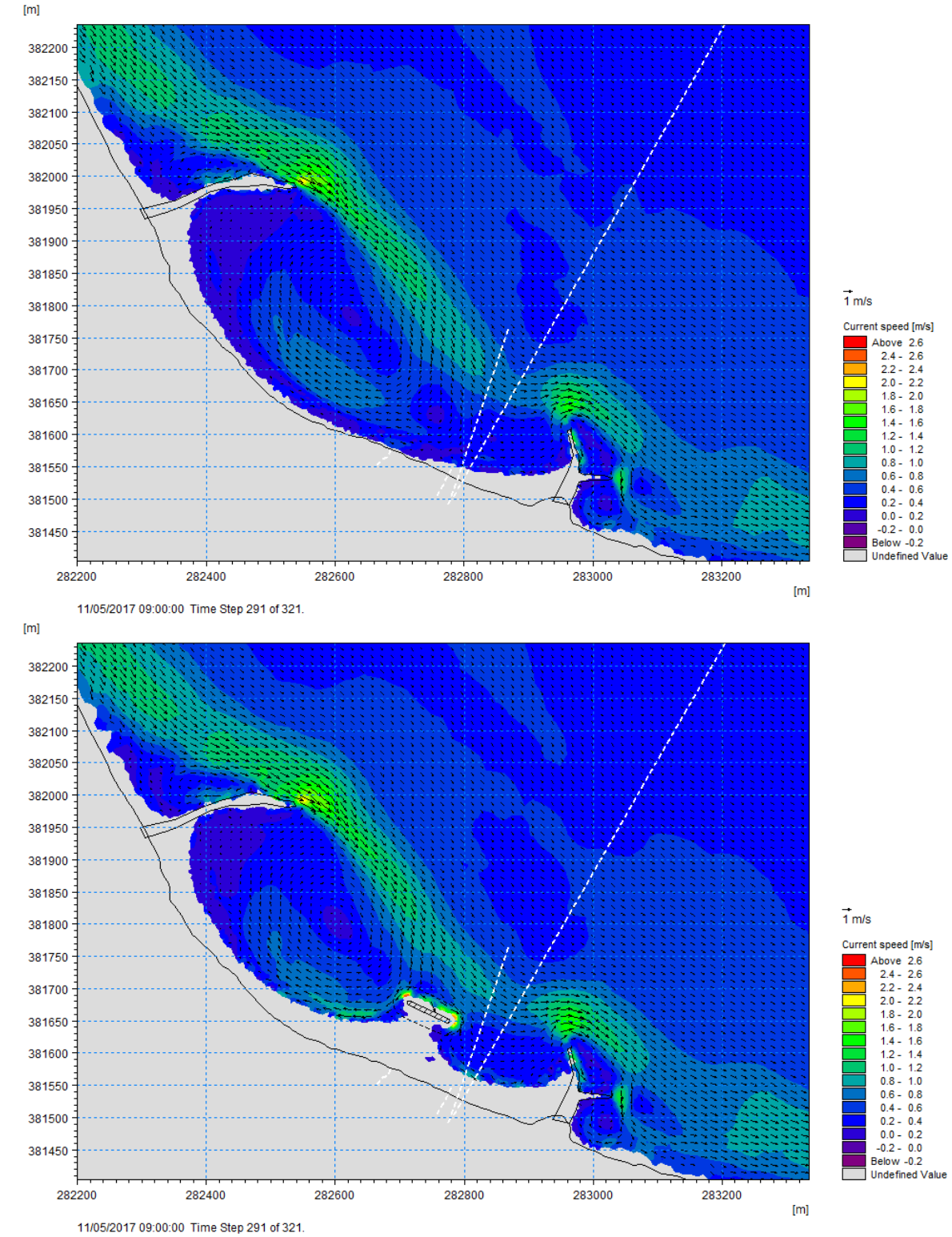
HW -4 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 07:00:00 Time Step 279 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

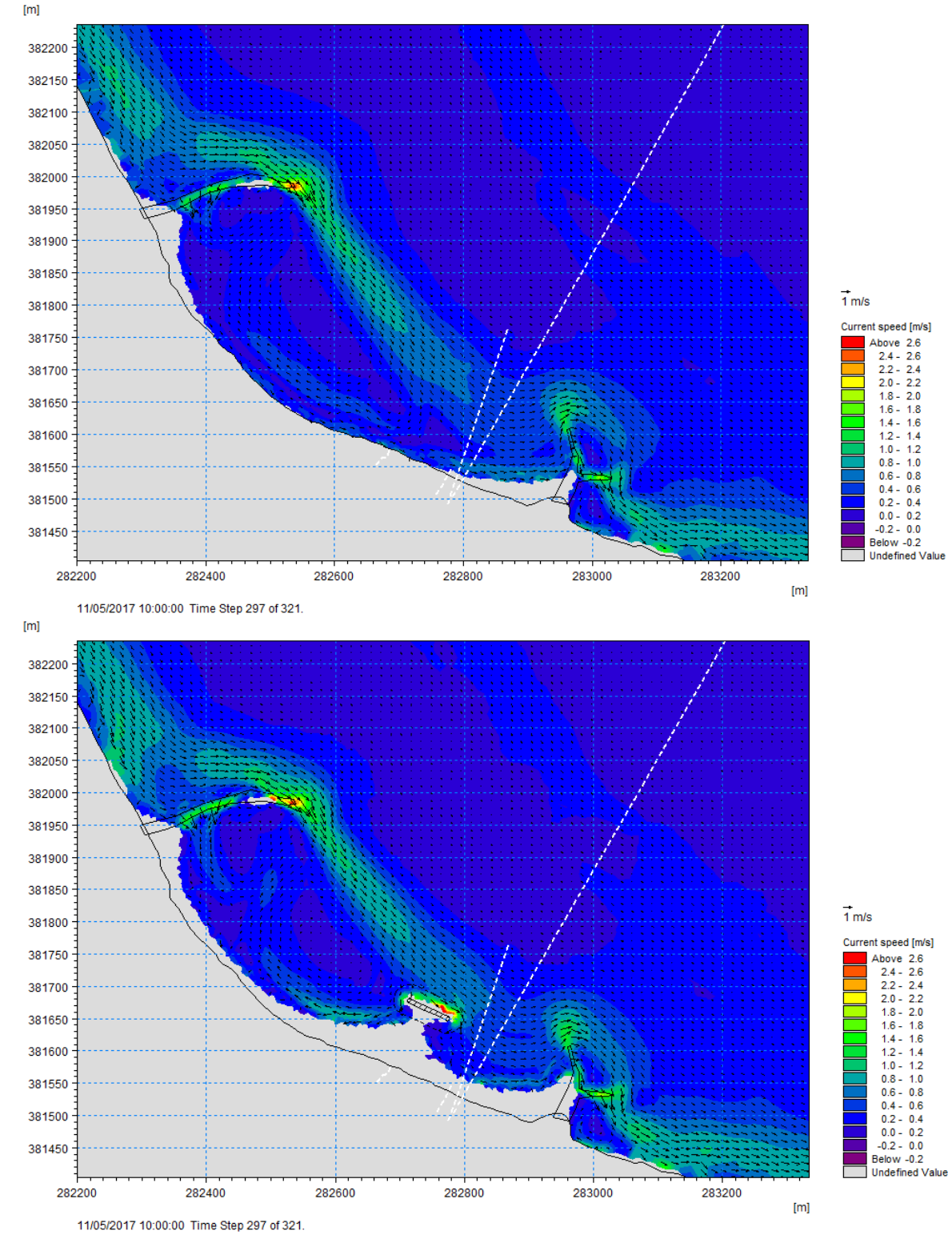
HW -3 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 08:00:00 Time Step 285 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

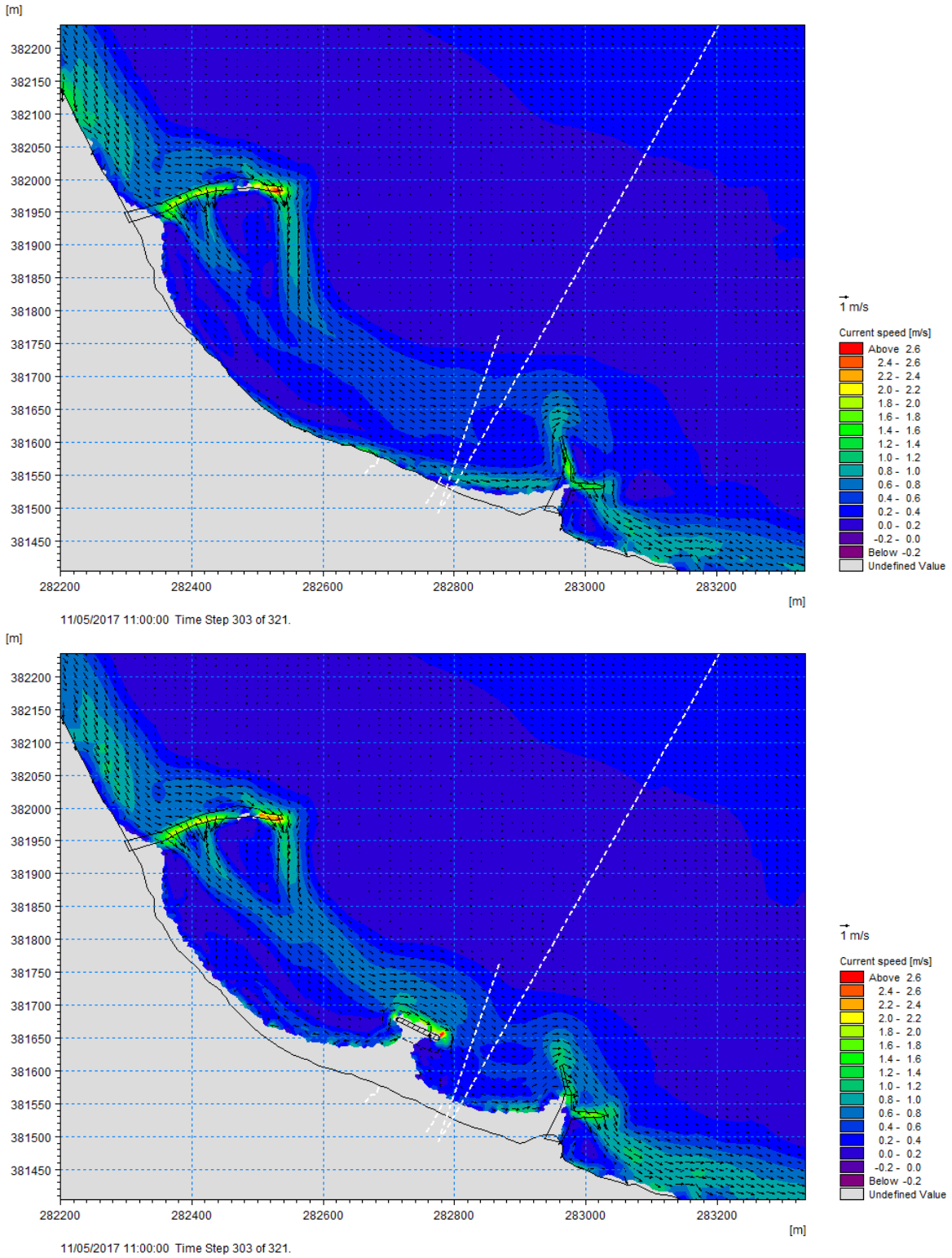
HW -2 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 09:00:00 Time Step 291 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

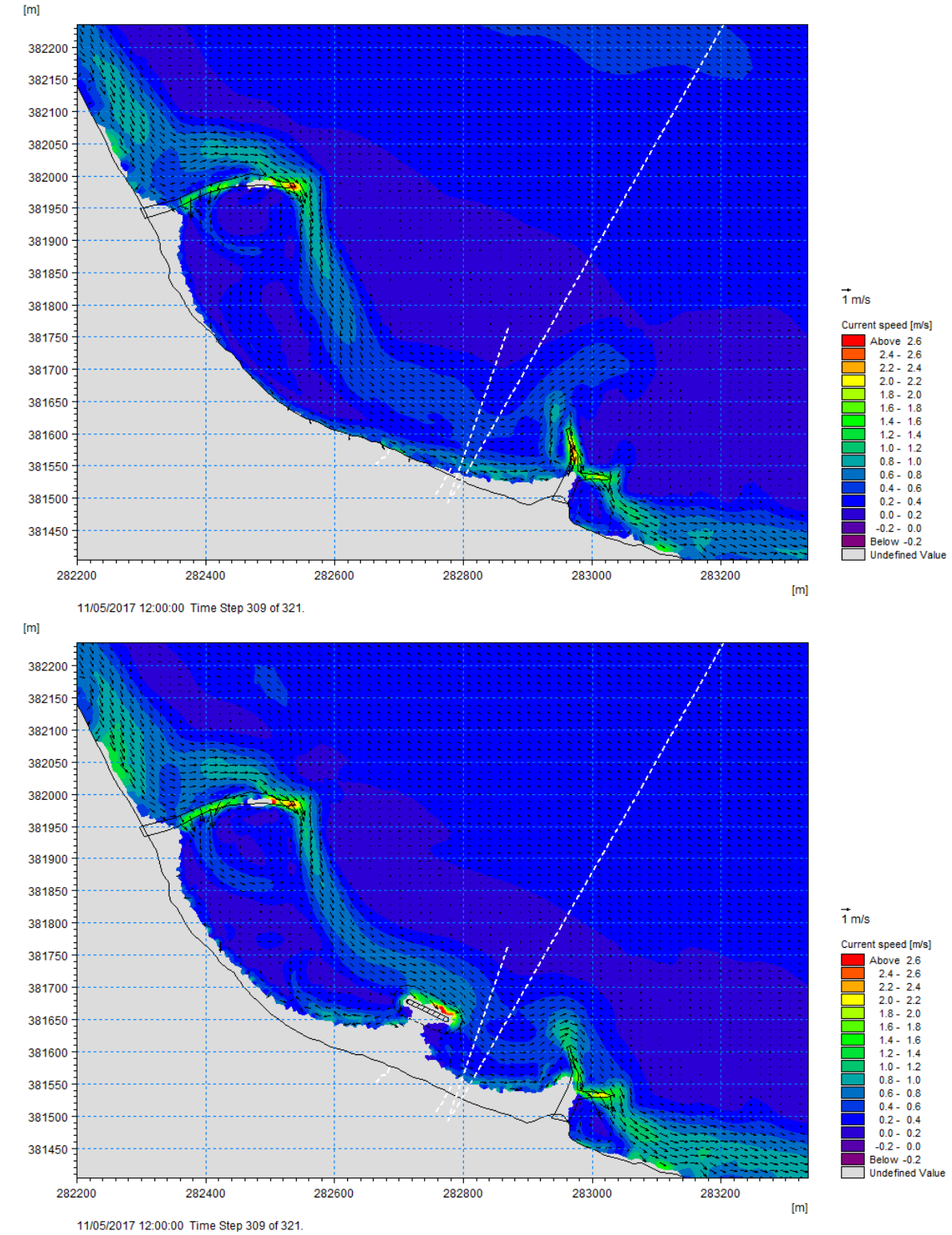
HW -1 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 10:00:00 Time Step 297 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

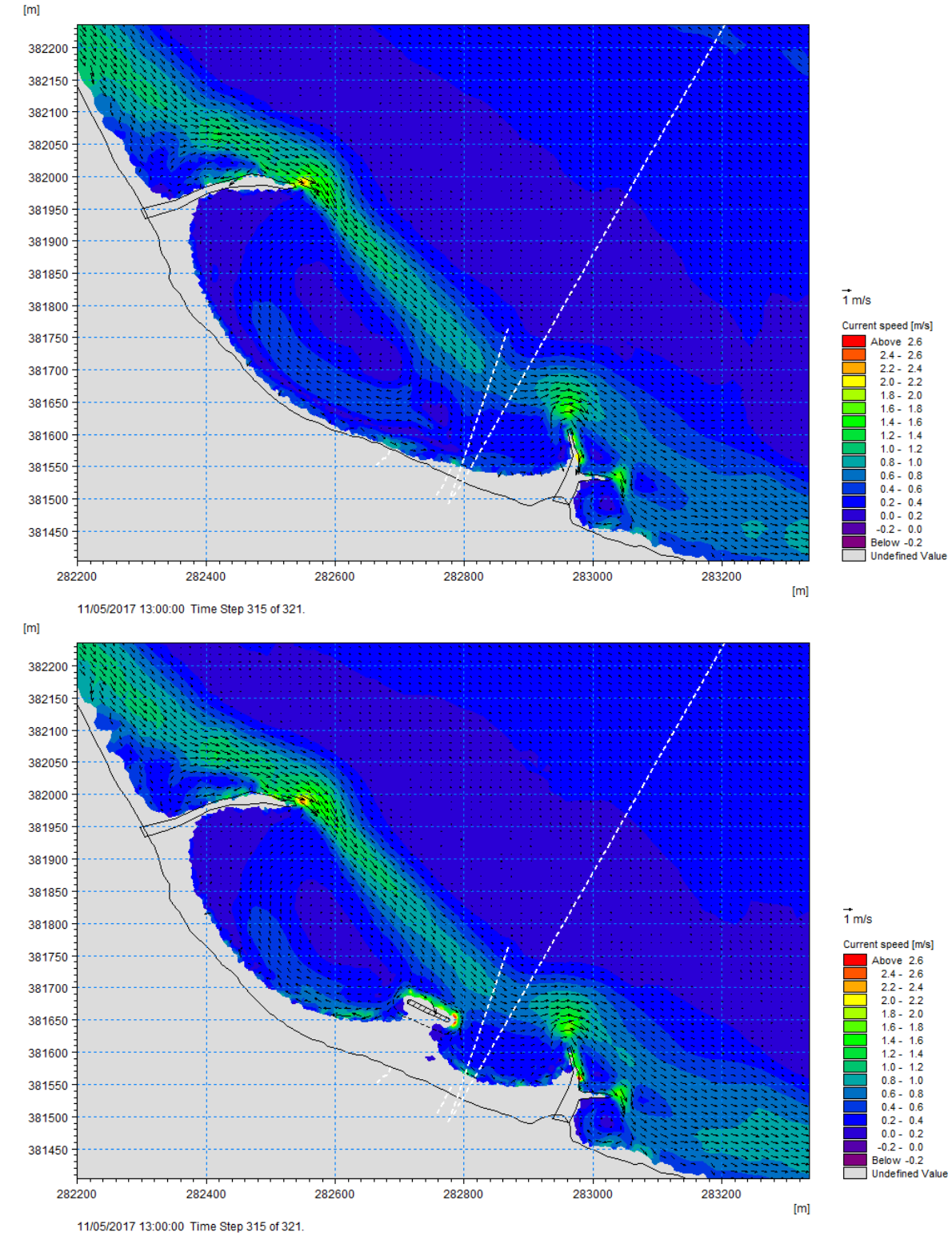
HW


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 11:00:00 Time Step 303 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

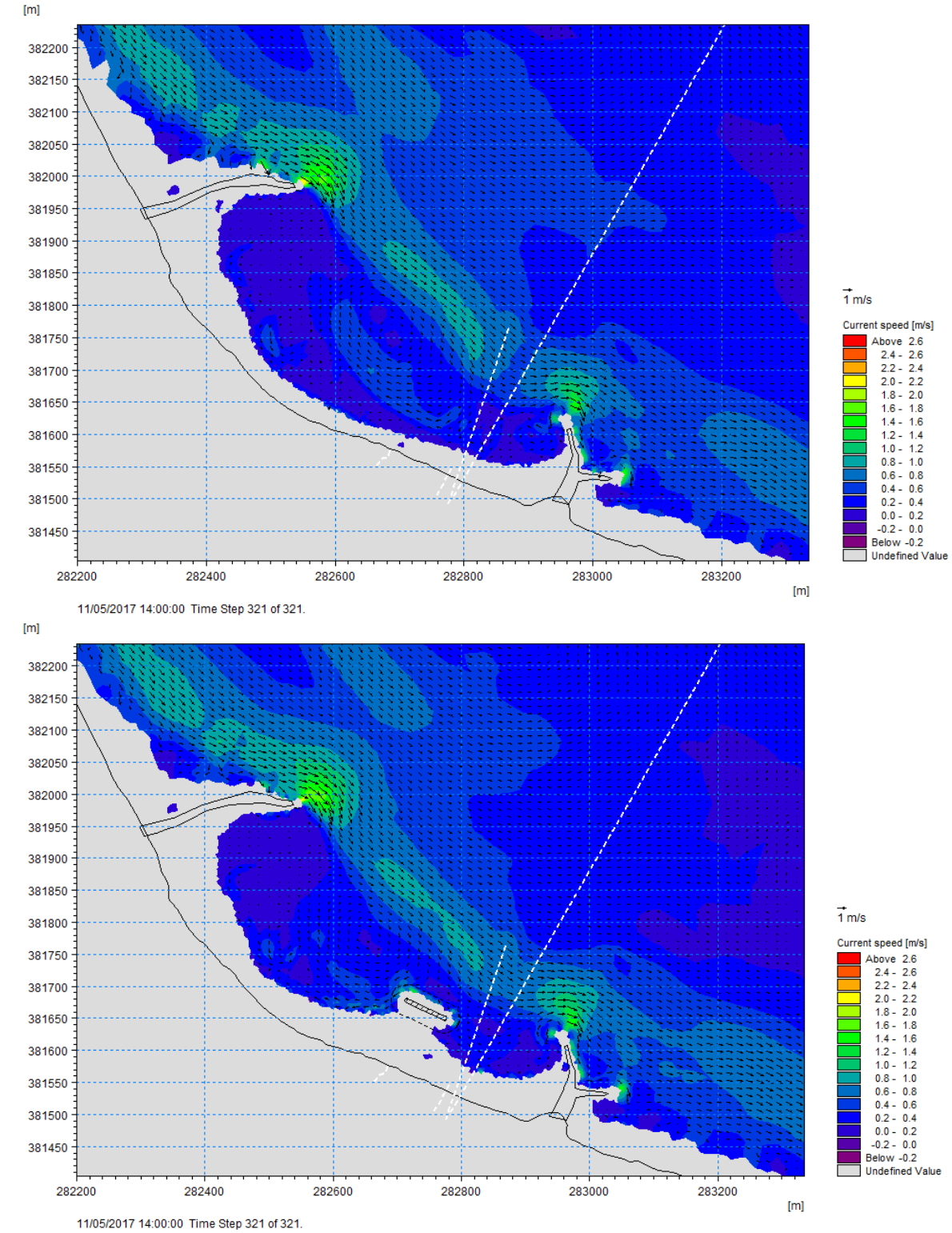
HW +1 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 12:00:00 Time Step 309 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW +2 hr

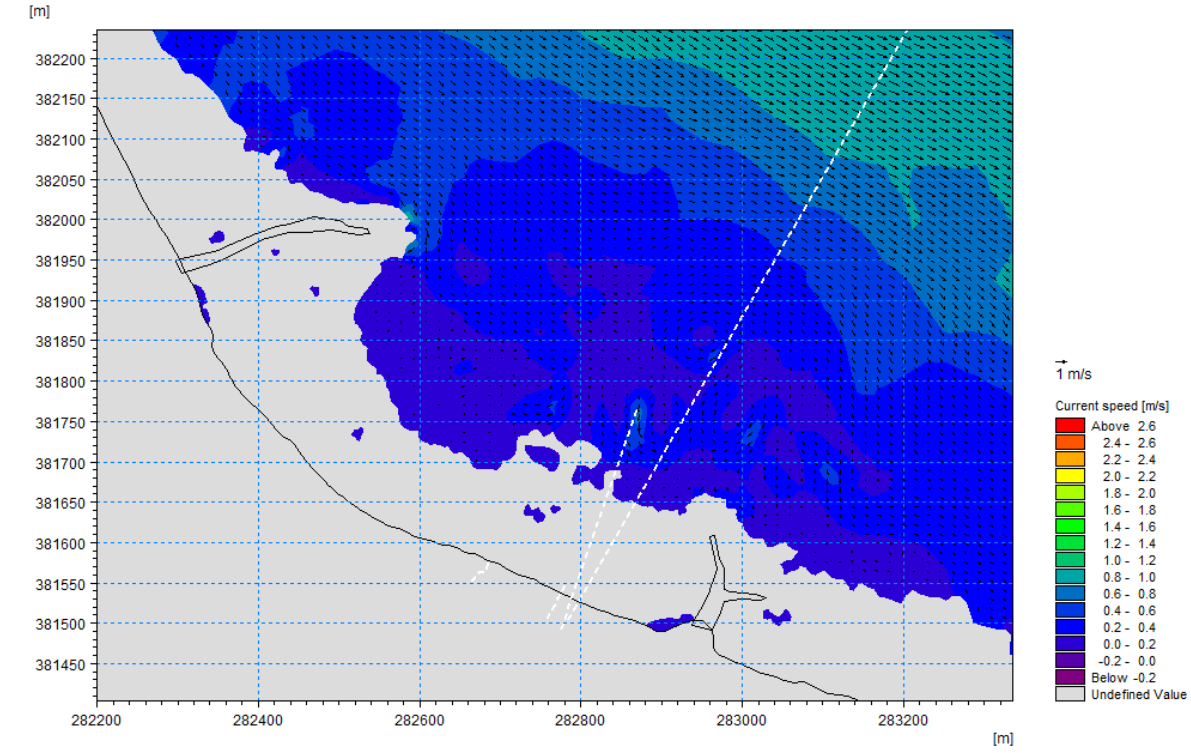

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 13:00:00 Time Step 315 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

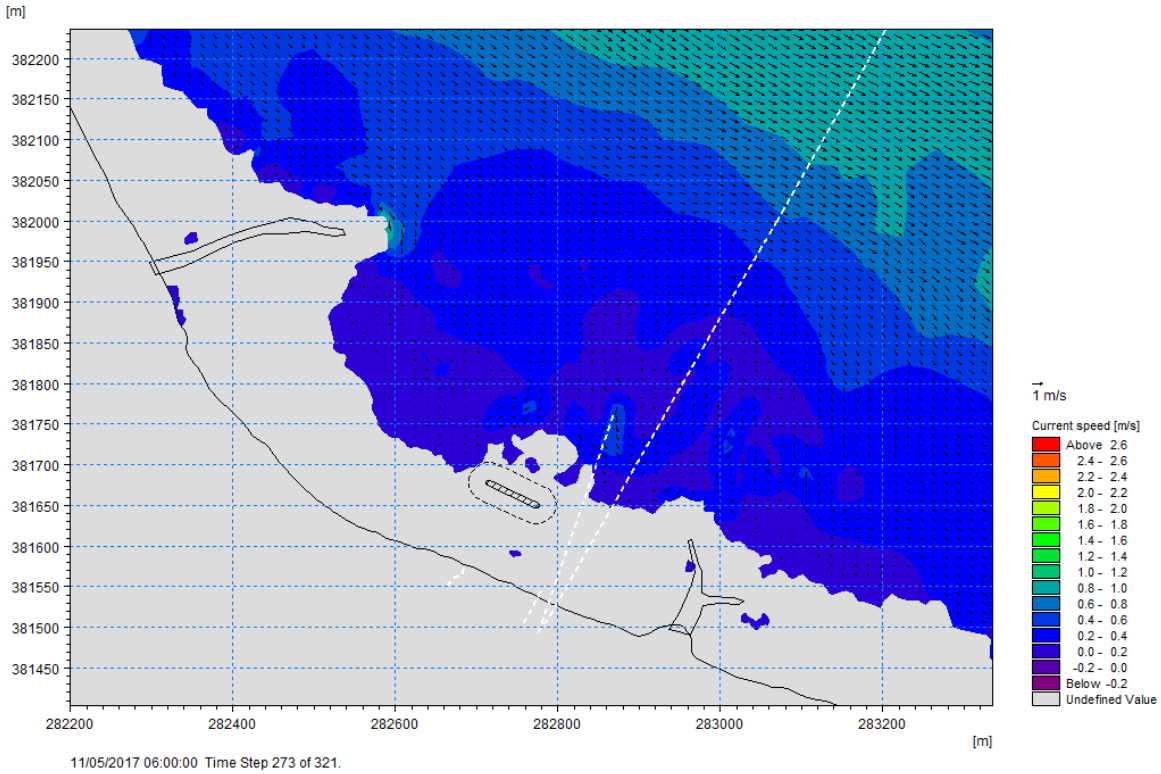
HW +3 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 14:00:00 Time Step 321 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

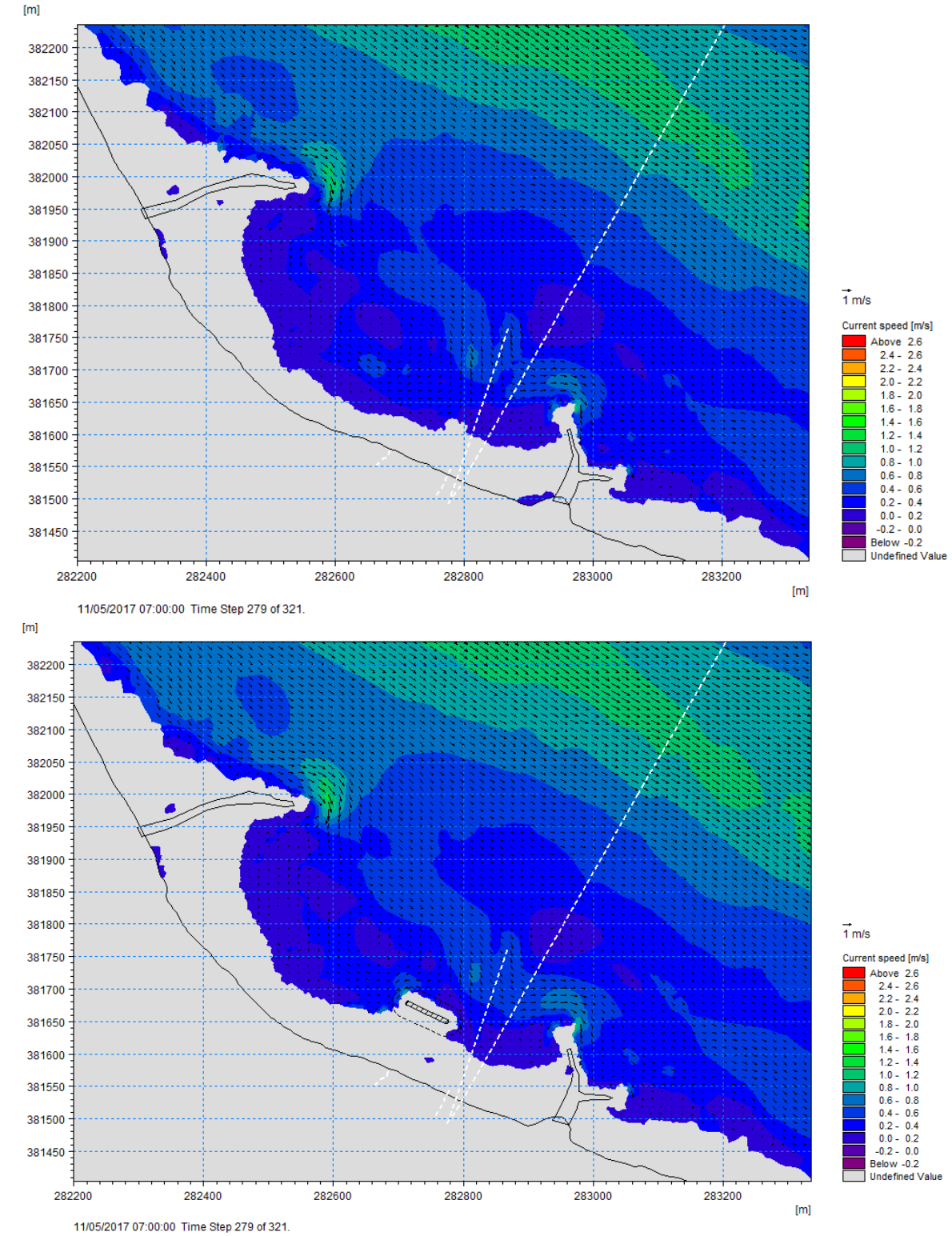
N.2 1 in 200-year event WNW

HW -5 hr

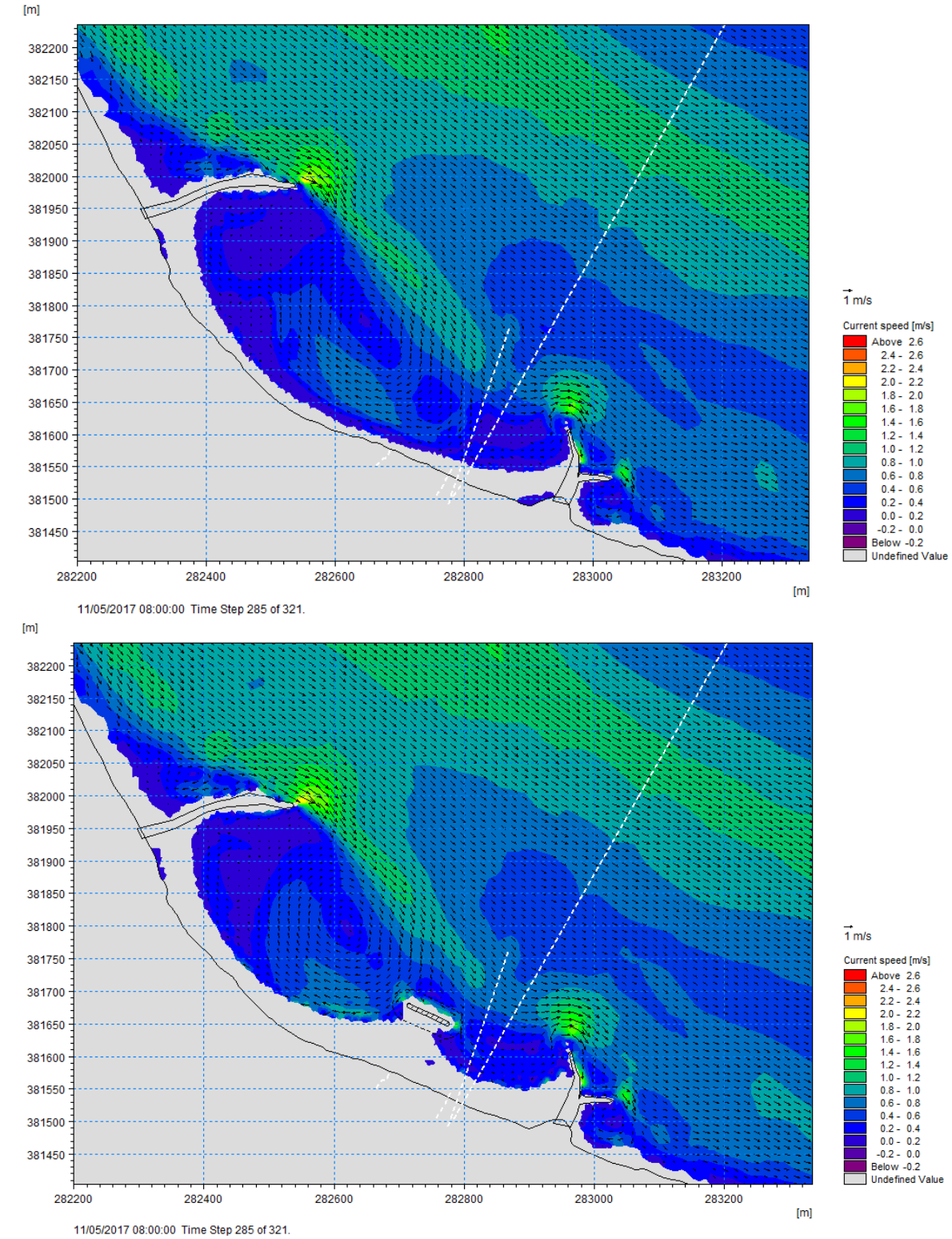




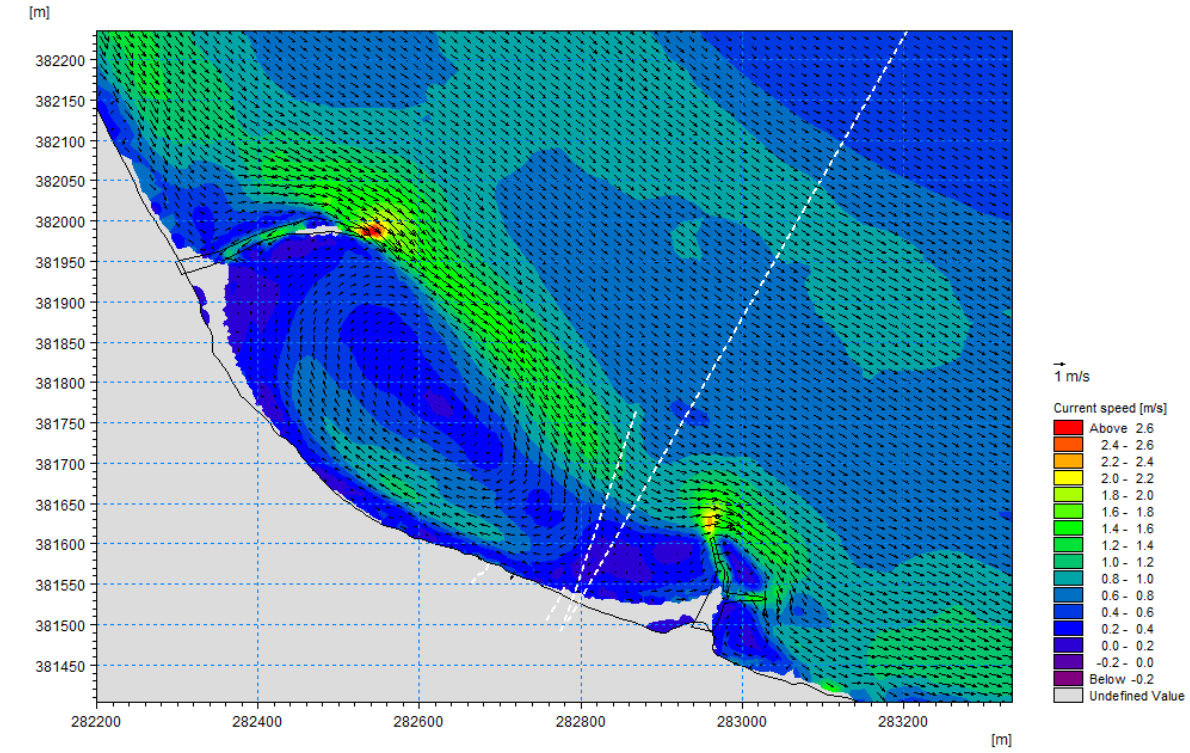
HW -4 hr

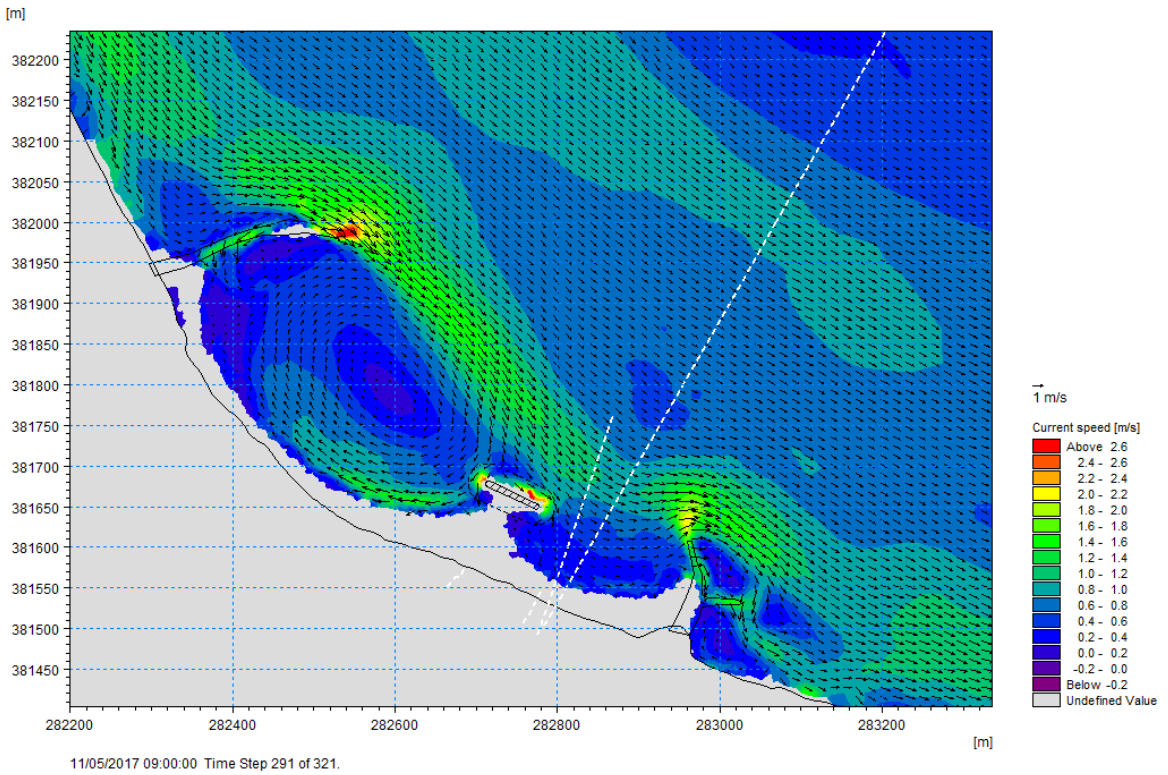

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 07:00:00 Time Step 279 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW -3 hr

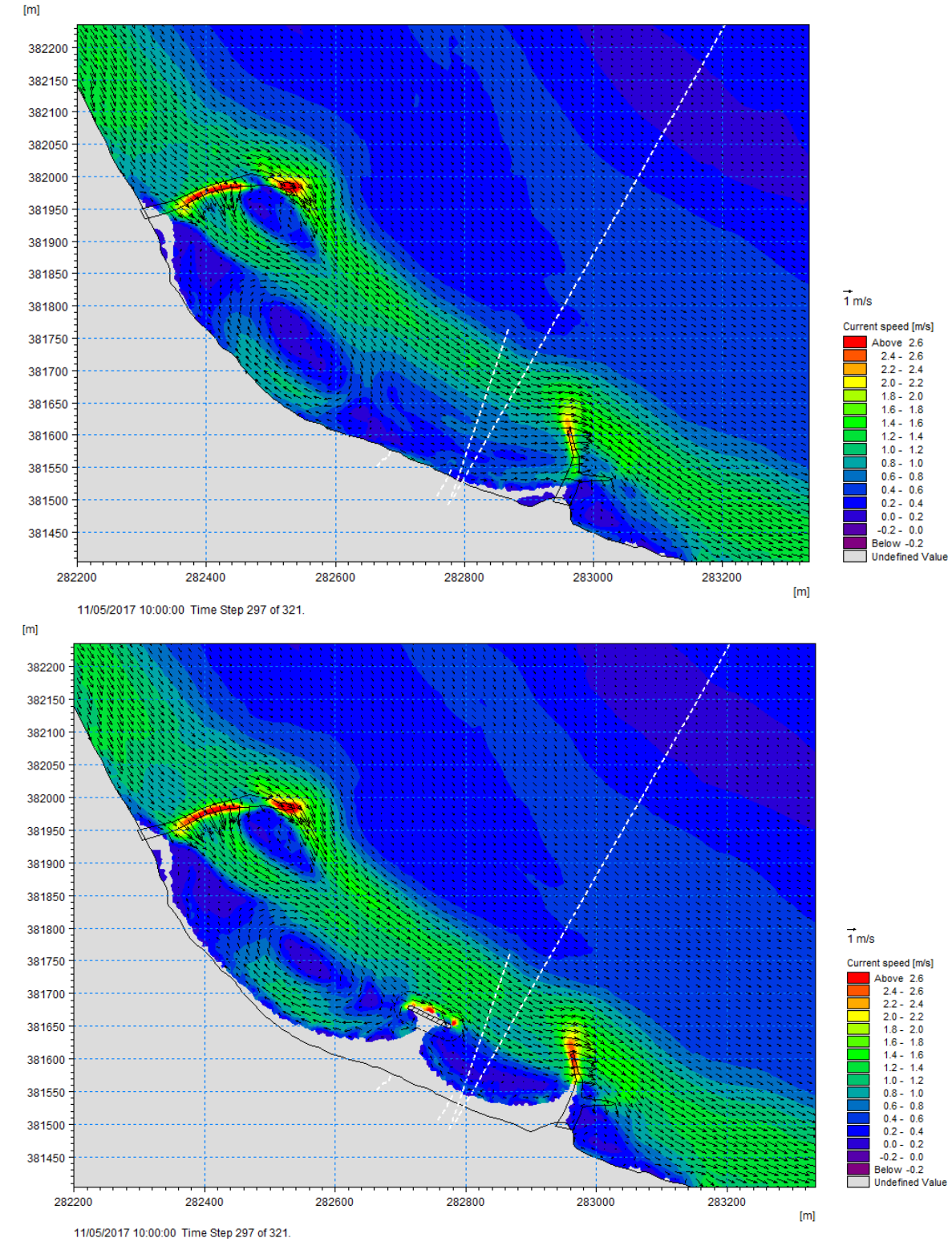

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 08:00:00 Time Step 285 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW -2 hr

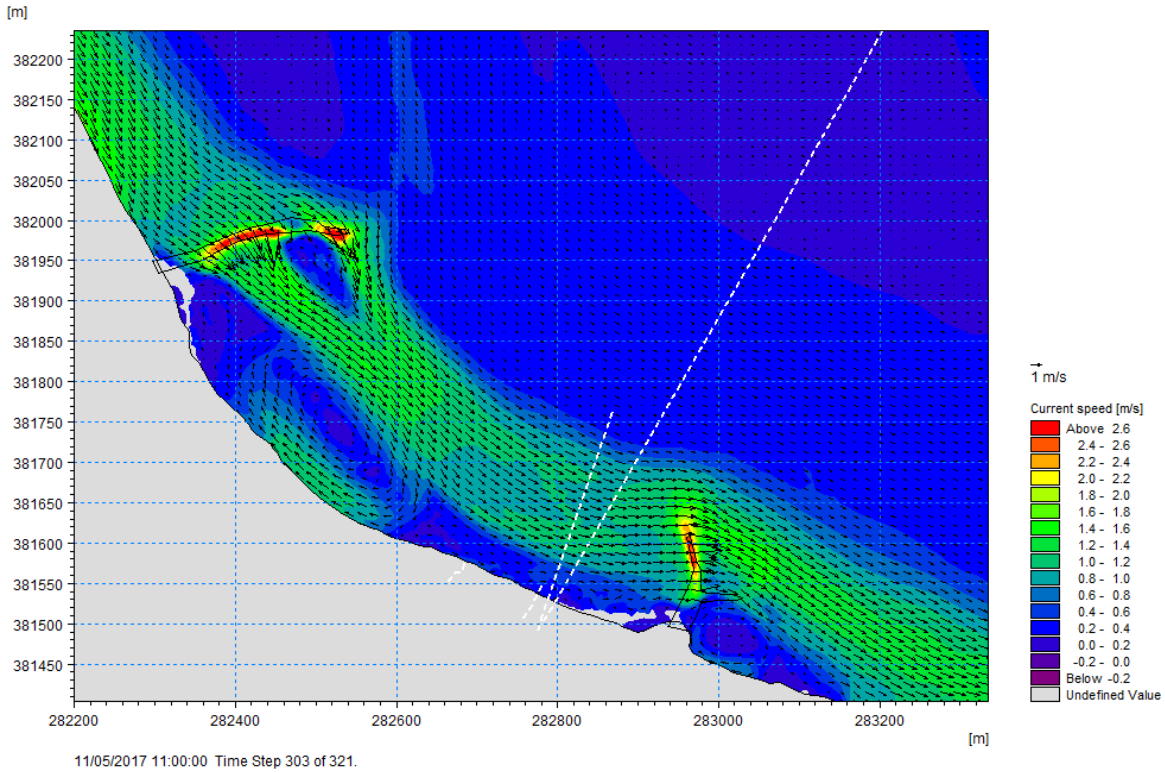

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 09:00:00 Time Step 291 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

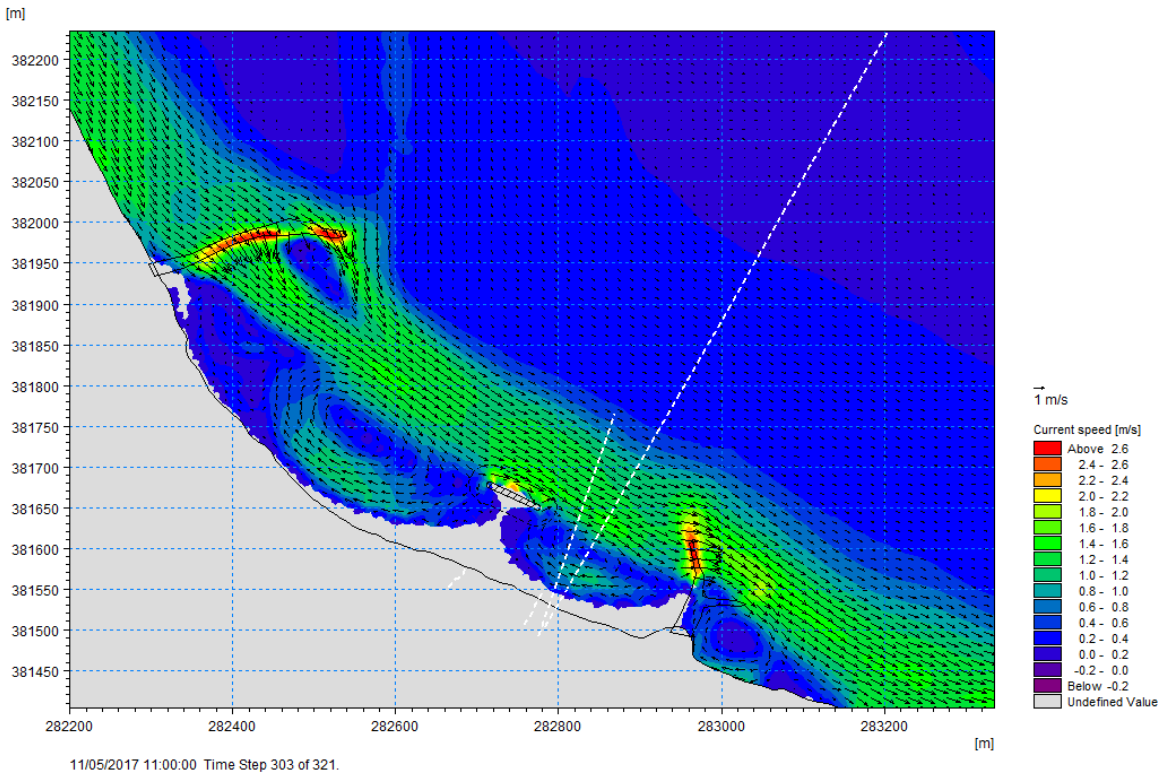

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 09:00:00 Time Step 291 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW -1 hr

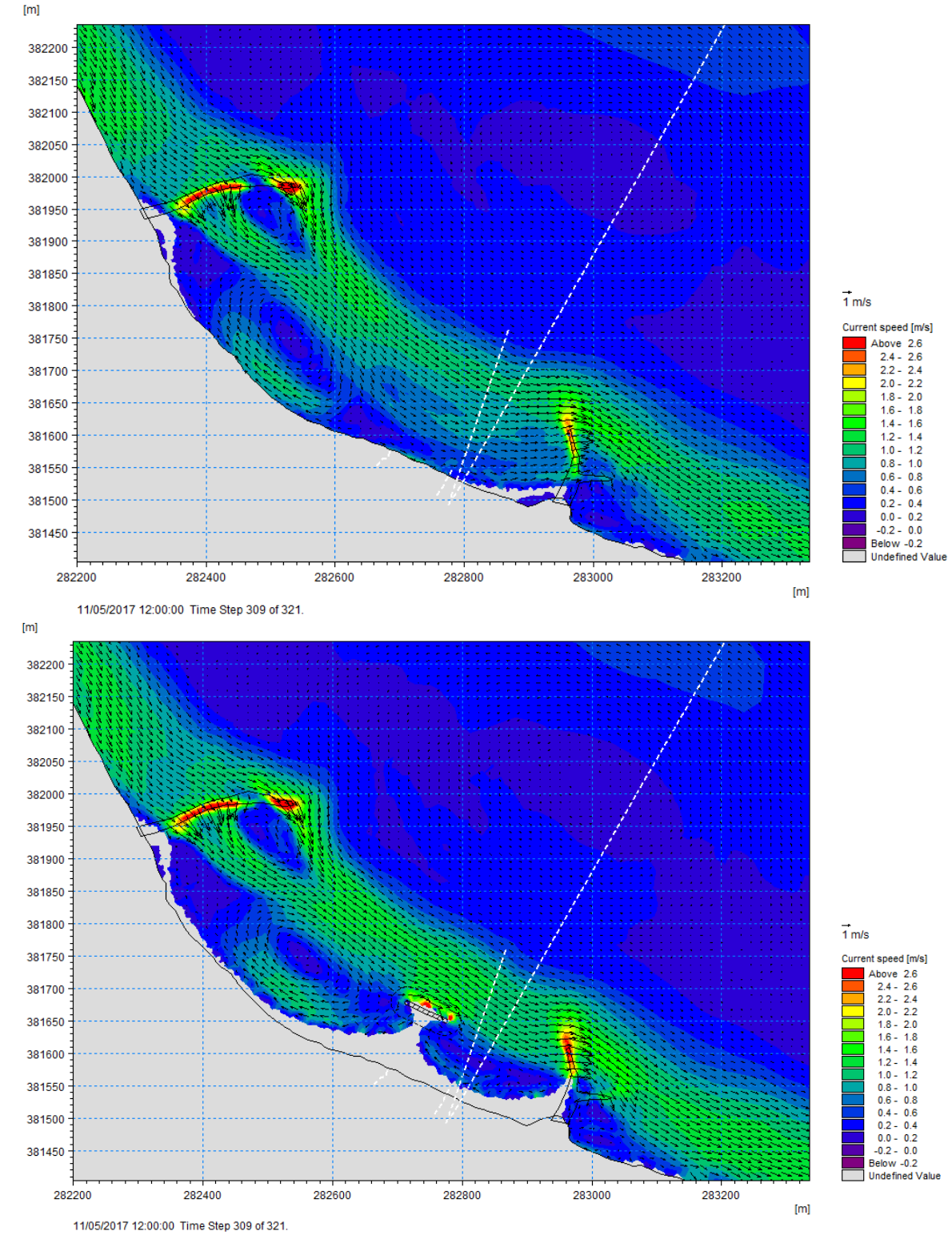

[m]
382200
382150
382100
382050
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381900
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381750
381700
381650
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381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 10:00:00 Time Step 297 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW

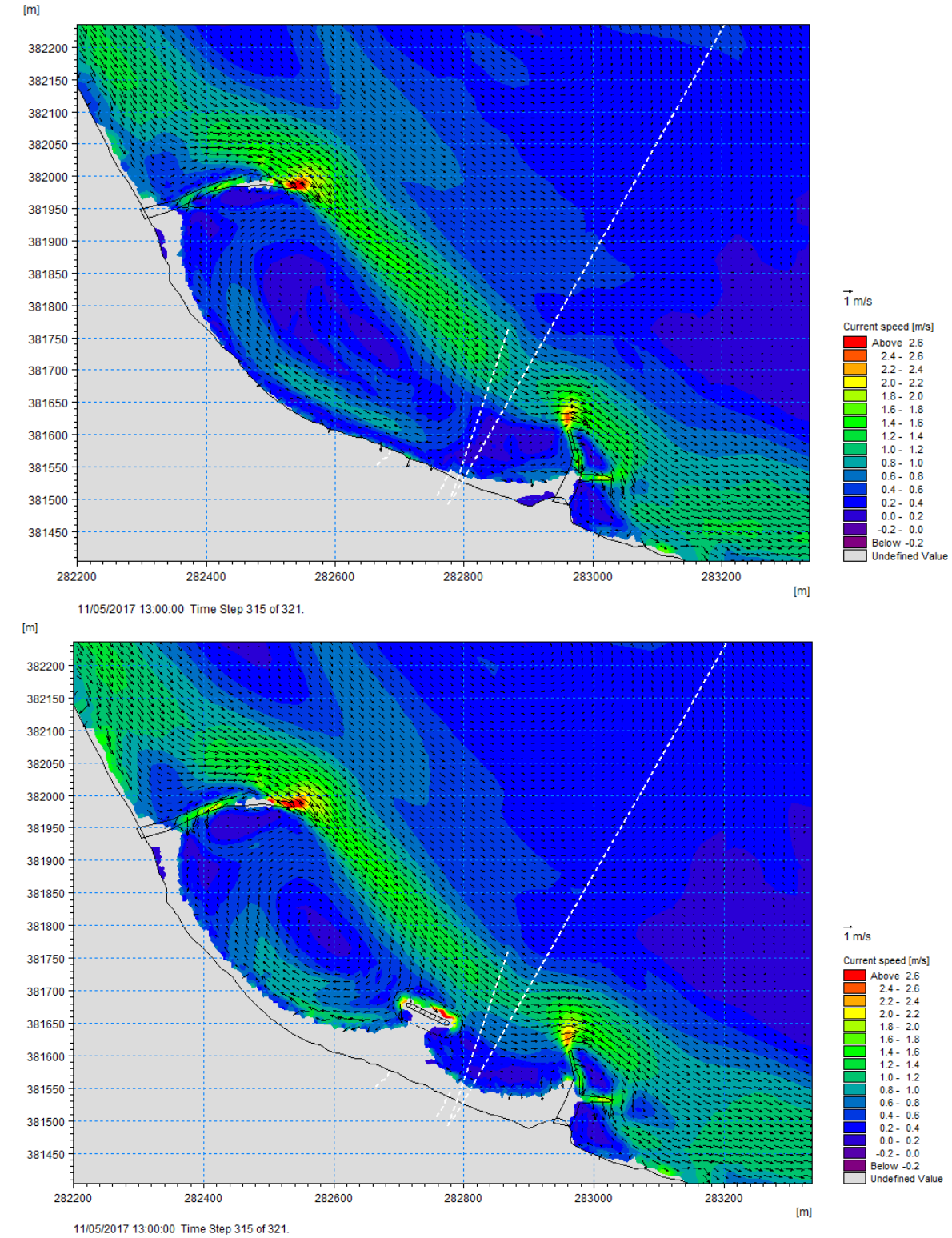

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 11:00:00 Time Step 303 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 11:00:00 Time Step 303 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

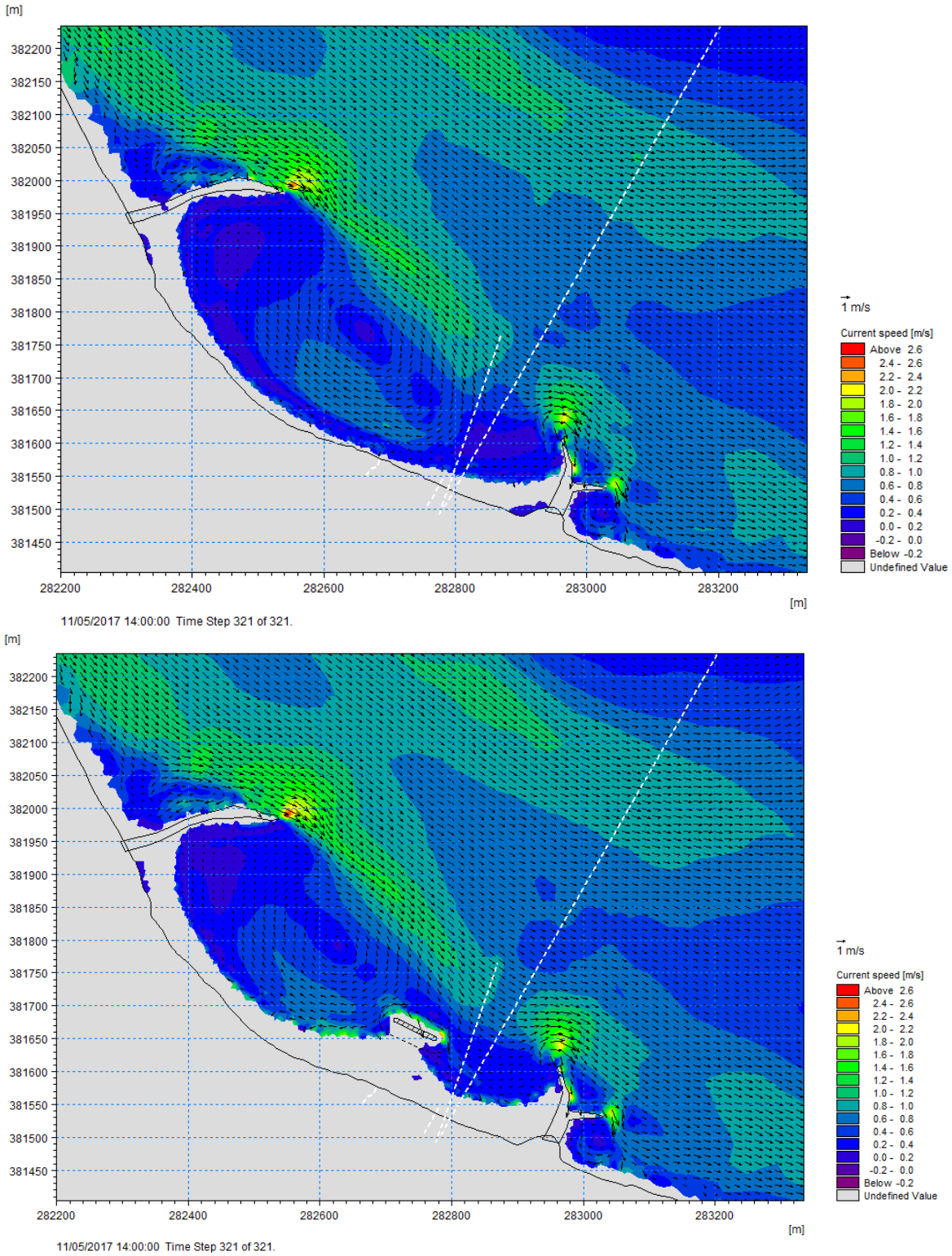
HW +1 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 12:00:00 Time Step 309 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW +2 hr

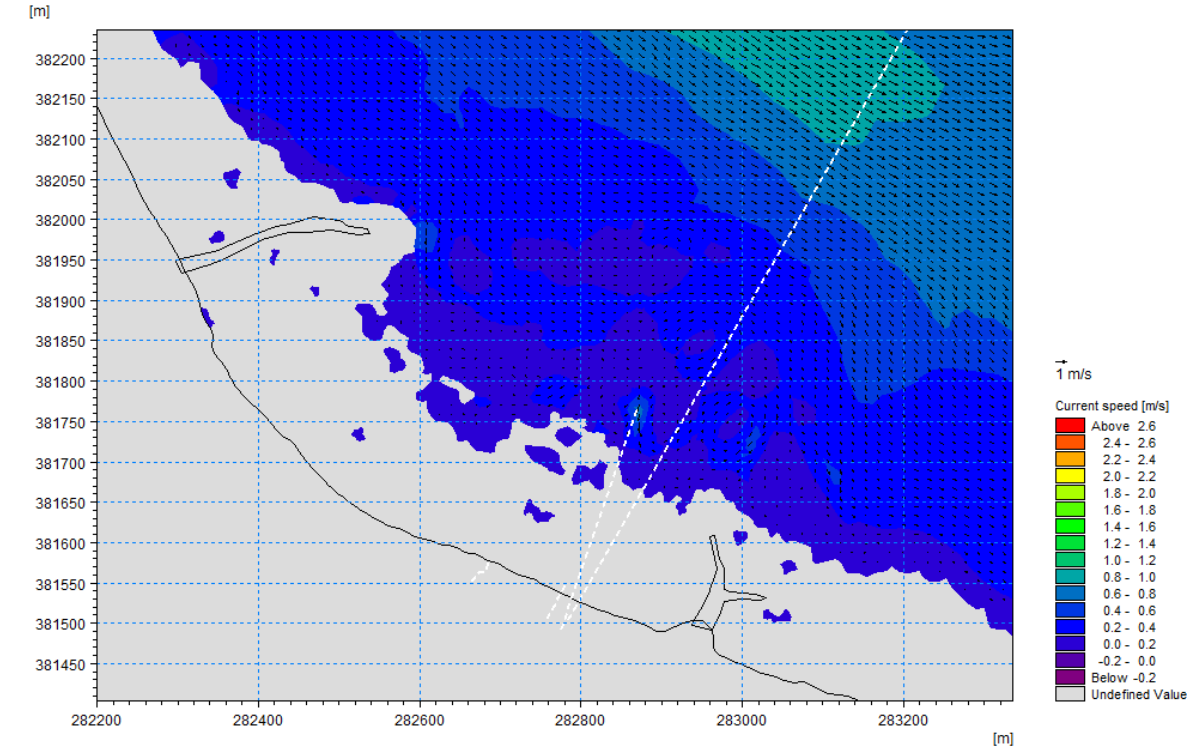

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 13:00:00 Time Step 315 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

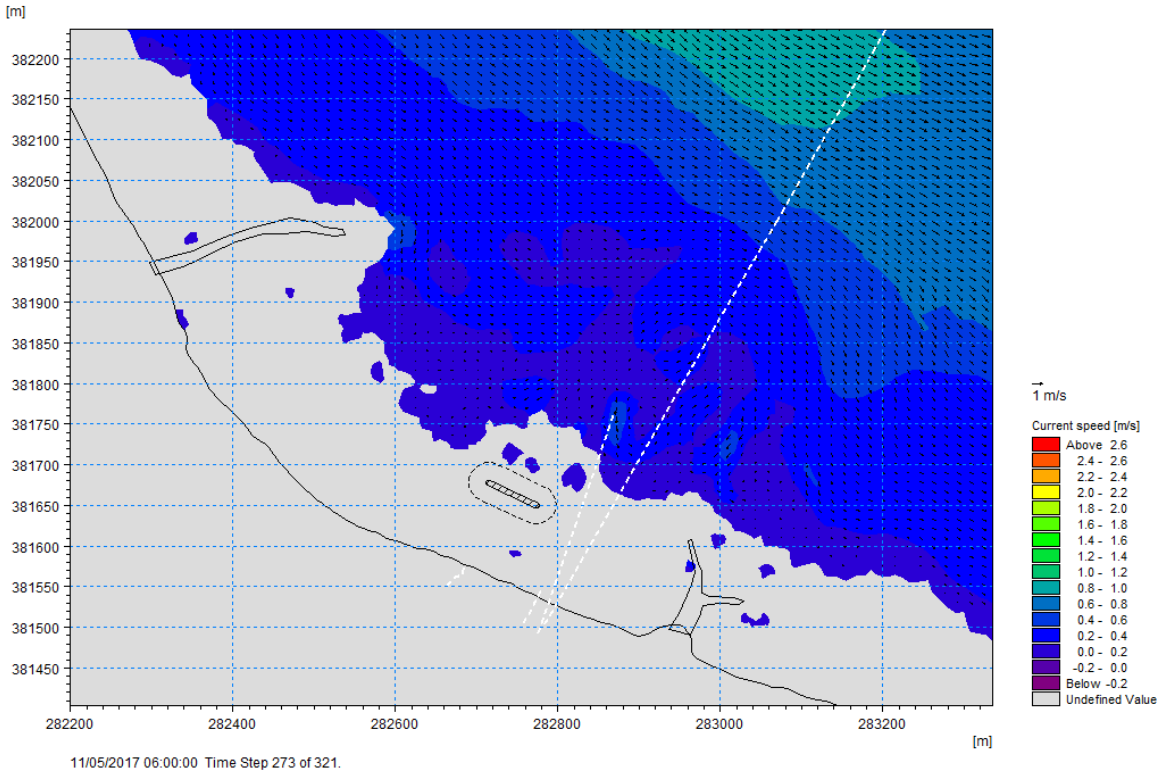
HW +3 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 14:00:00 Time Step 321 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

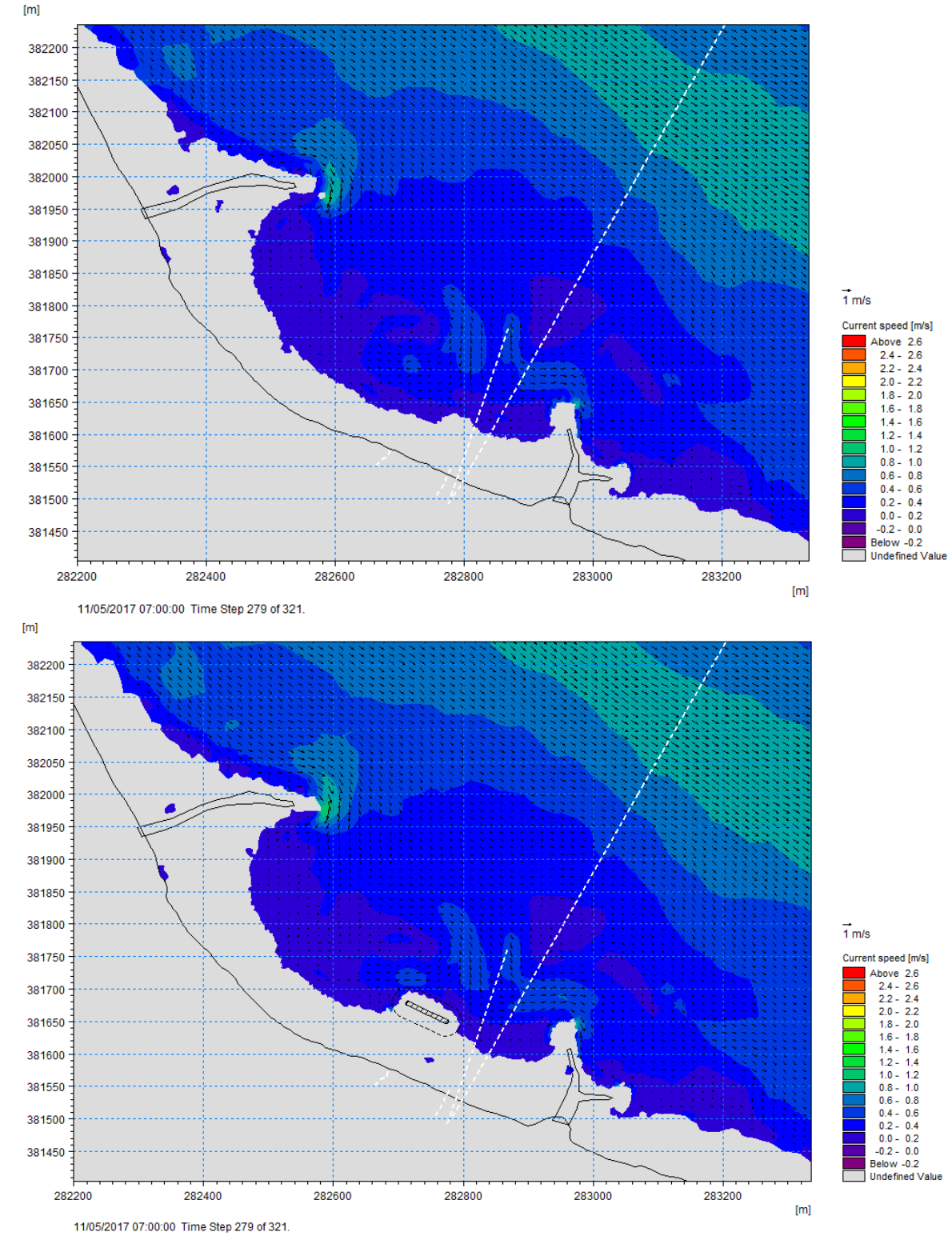
N.3 1 in 200-year event NNE

HW -5 hr



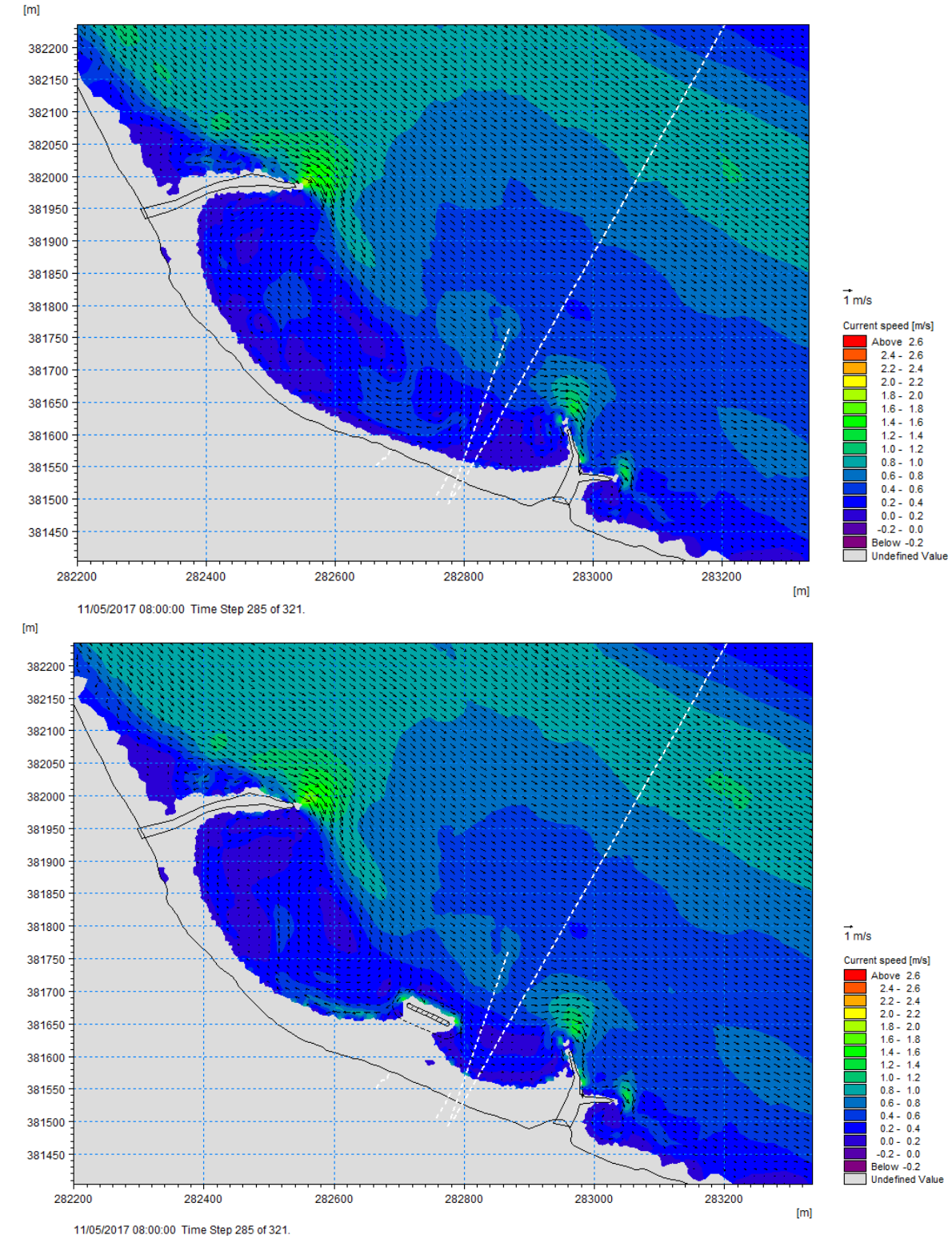


HW -4 hr

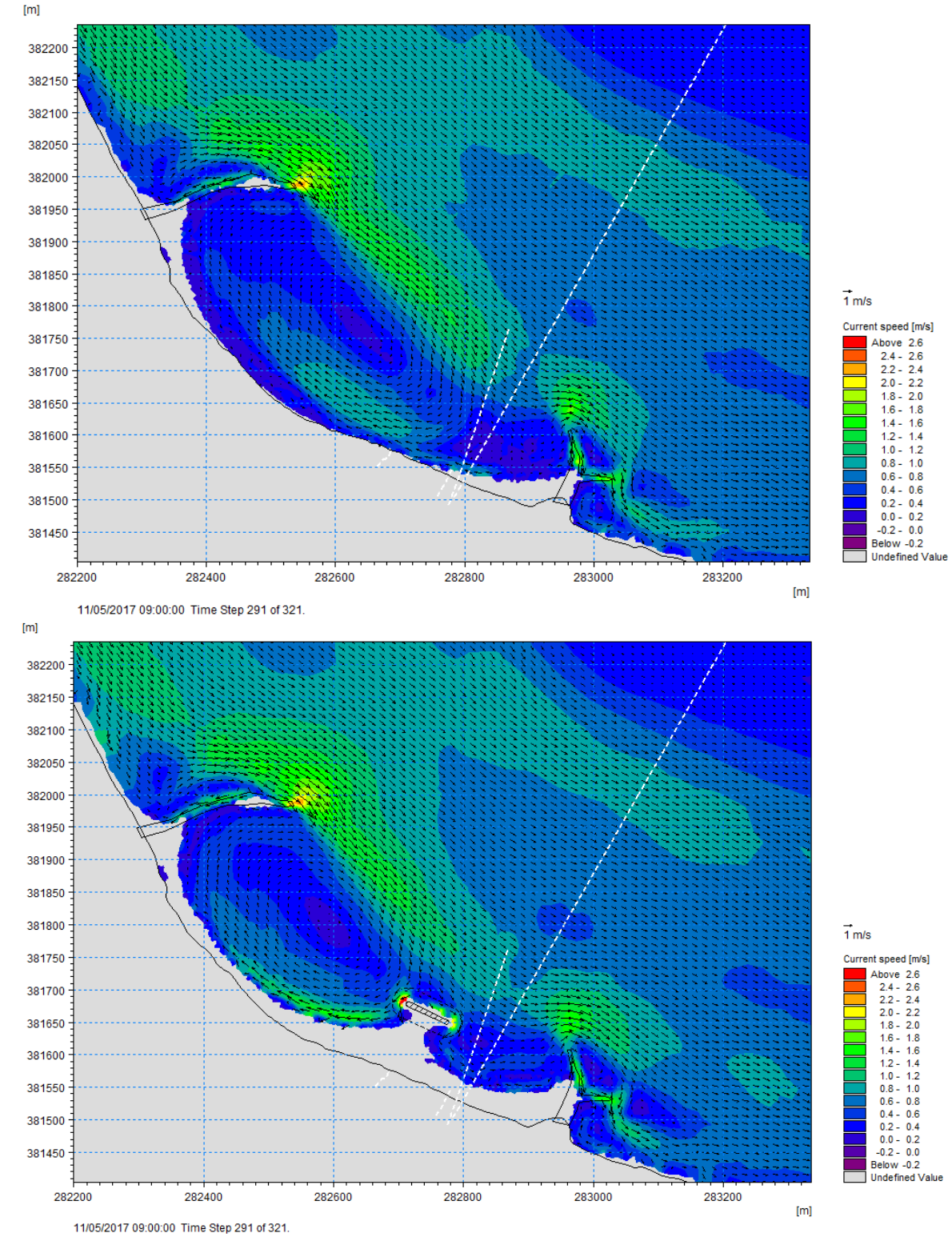


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 07:00:00 Time Step 279 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

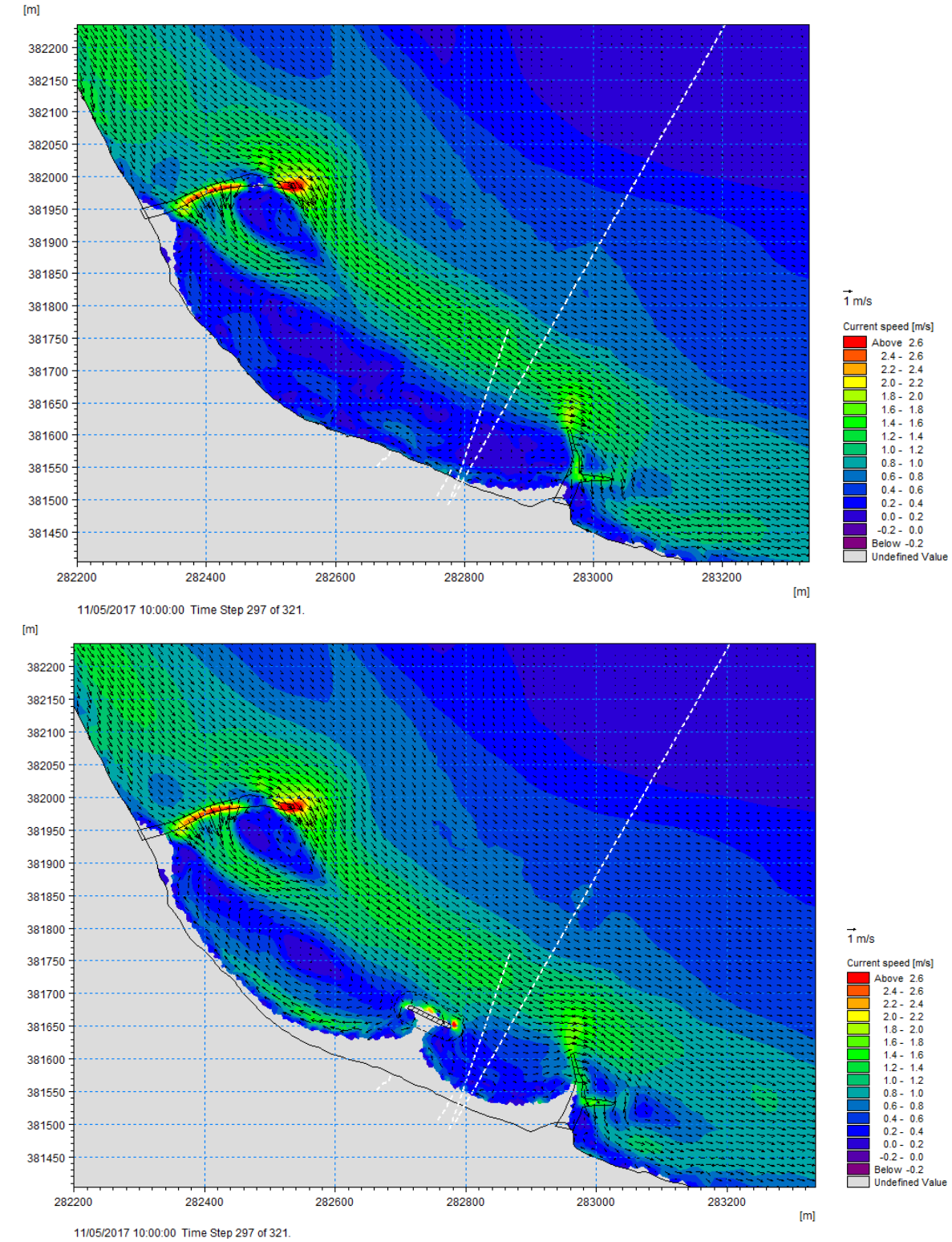
HW -3 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 08:00:00 Time Step 285 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

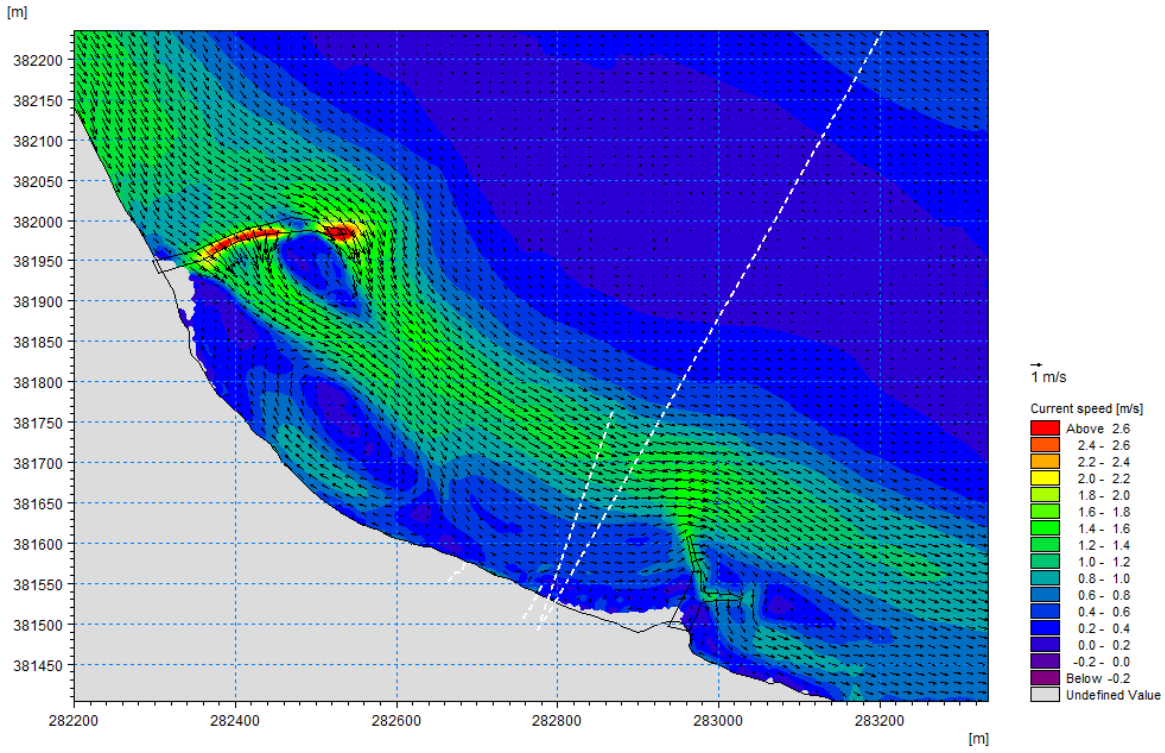
HW -2 hr

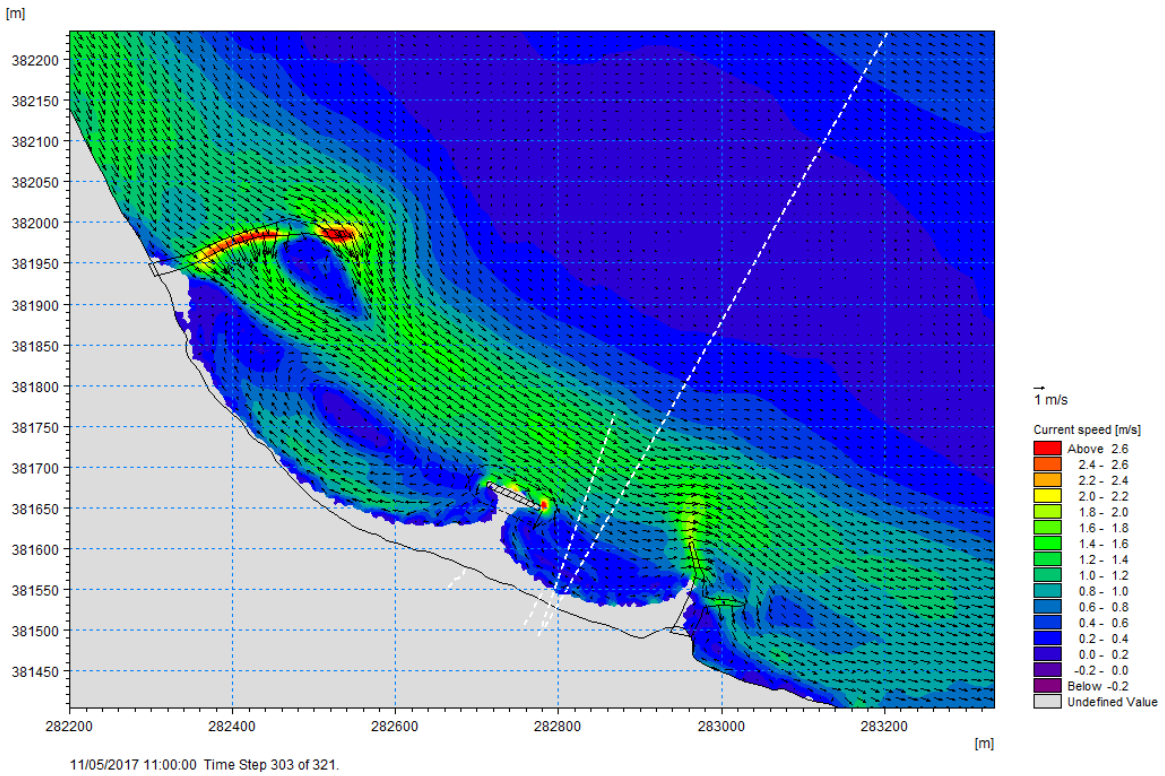

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 09:00:00 Time Step 291 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW -1 hr

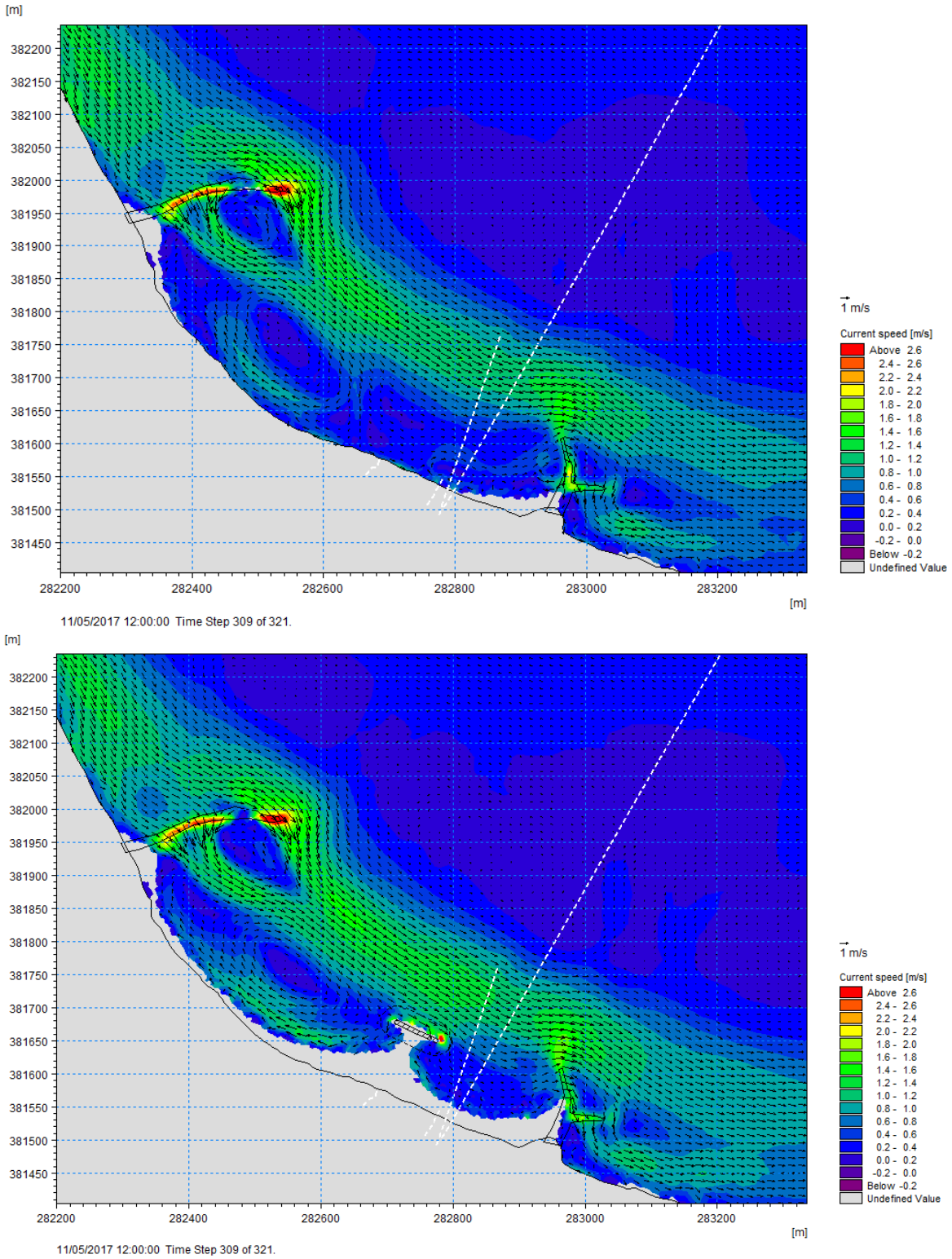

[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 10:00:00 Time Step 297 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW

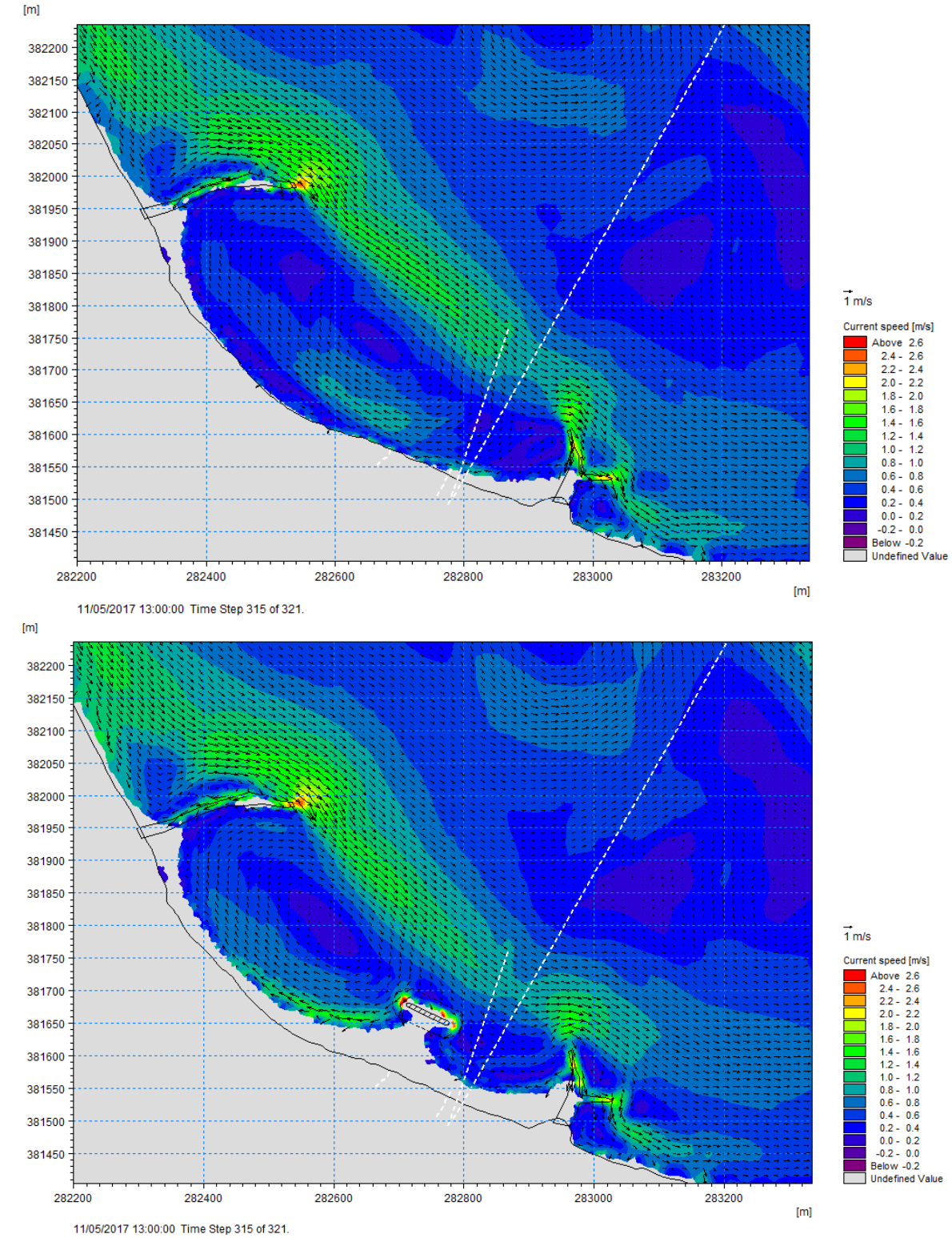

[m]
382200
382150
382100
382050
382000
381950
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381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 11:00:00 Time Step 303 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 11:00:00 Time Step 303 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

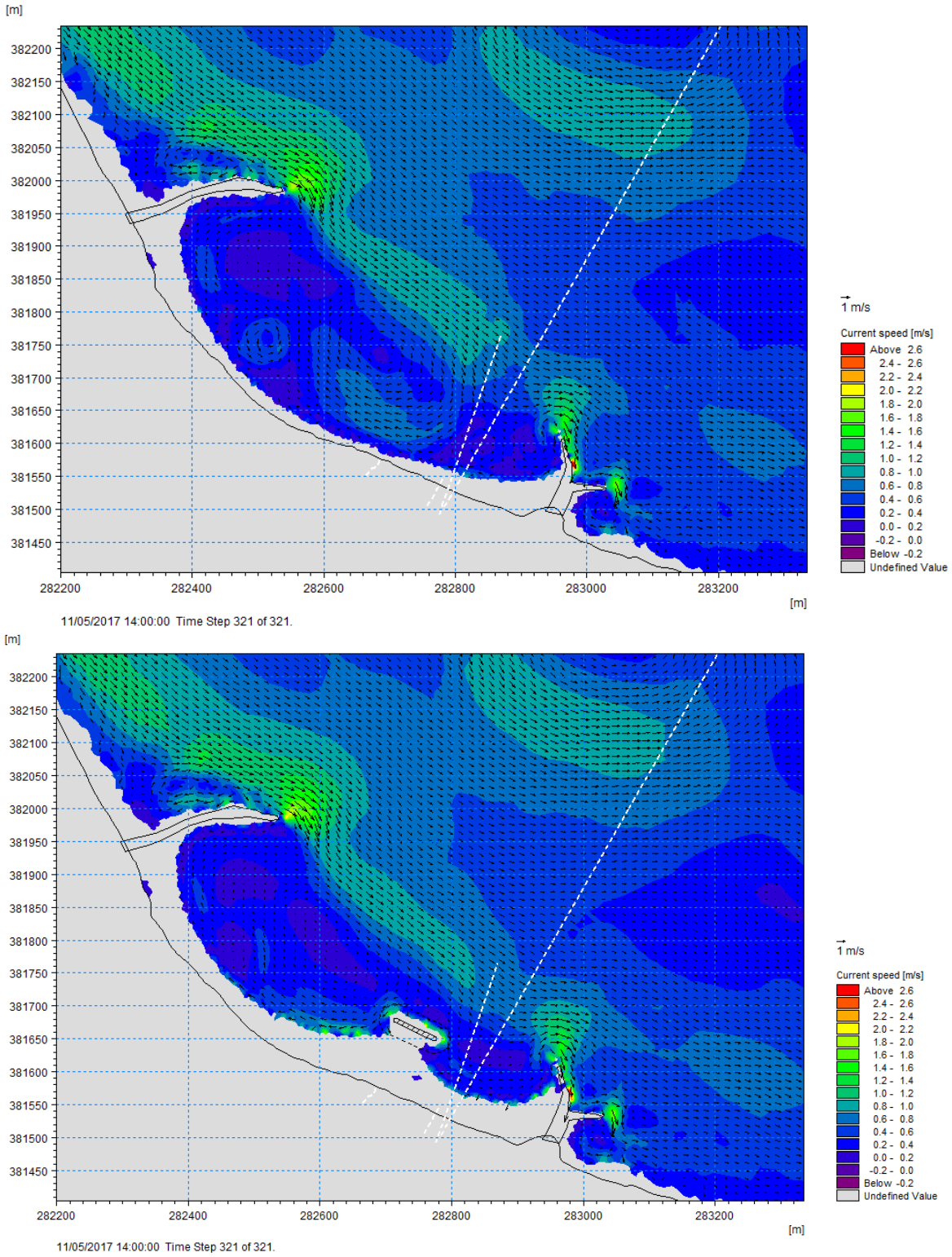
HW +1 hr


[m]
382200
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382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 12:00:00 Time Step 309 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW +2 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 13:00:00 Time Step 315 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value
[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 13:00:00 Time Step 315 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

HW +3 hr


[m]
382200
382150
382100
382050
382000
381950
381900
381850
381800
381750
381700
381650
381600
381550
381500
381450
282200
282400
282600
282800
283000
283200
[m]
11/05/2017 14:00:00 Time Step 321 of 321.
1 m/s
Current speed [m/s]
Above 2.6
2.4 - 2.6
2.2 - 2.4
2.0 - 2.2
1.8 - 2.0
1.6 - 1.8
1.4 - 1.6
1.2 - 1.4
1.0 - 1.2
0.8 - 1.0
0.6 - 0.8
0.4 - 0.6
0.2 - 0.4
0.0 - 0.2
-0.2 - 0.0
Below -0.2
Undefined Value

Contact Us

ABPmer

Quayside Suite,
Medina Chambers
Town Quay, Southampton
SO14 2AQ

T +44 (0) 23 8071 1840
F +44 (0) 23 8071 1841
E enquiries@abpmer.co.uk

www.abpmer.co.uk

Appendix C

Design Development Modelling,
Beach Management Strategy,
ABPmer Report No. R3504
Revision 3, October 2020

C1

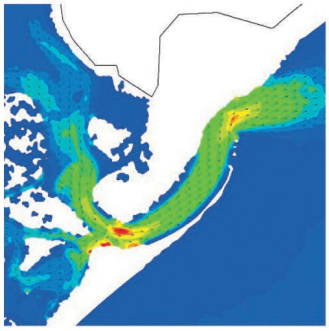
Ove Arup and Partners Limited

Penrhyn Coastal Defence: Design Development

Beach Management Strategy

October 2020




Innovative Thinking - Sustainable Solutions

ABP mer

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Penrhyn Coastal Defence: Design Development

Beach Management Strategy

October 2020



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	Beach Management Strategy	
Commissioned by	Ove Arup and Partners Limited	
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Document ref	R.3504	
Project no	R/4871/03	
Date	**Version**	**Revision Details**
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25/09/2020	2	Issued for Client Use
02/10/2020	3	Issued for Client Use – Additional figure included

Prepared (PM)	Approved (QM)	Authorised (PD)
David Finch	Adam Fulford	Heidi Roberts
		

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ABPmer
Quayside Suite, Medina Chambers, Town Quay, Southampton, Hampshire SO14 2AQ
T: +44 (0) 2380 711844 W: http://www.abpmer.co.uk/

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1 Introduction

ABPmer has been commissioned by Arup to undertake coastal process and modelling studies in support of the design development of a coastal defence scheme at Penrhyn Bay. The work is being commissioned on behalf of Conwy County Borough Council (CCBC).

The beach is currently maintained by beach management on an annual to 18-month basis and the Outline Business Case (OBC) has recommended the construction of a new rock beach control structure in the centre of the Bay to reduce the frequency of beach management activities.

The overall objective of the scheme is to maintain beach levels in the Bay to protect the existing defences. The existing defences provide coastal erosion and tidal flood protection and failure would affect a significant number of residential and commercial properties and utilities infrastructure. The beach is not required to manage the risk of overtopping.

The OBC concluded that "in the absence of the present works undertaken by CCBC, the sea wall and revetment are at risk of failure through the on-going draw-down of the foreshore exposing the corroded and structurally weak sheet piled toe. However, the present regime is considered unsustainable and likely to increase in frequency over time due to increased beach mobility due to the beach material particle attrition and due to climate change and sea level rise. CCBC would like to implement a solution looking to stabilise the beach and reduce the ongoing cost of maintenance, whilst improving the amenity value of this section of coastline.

This Beach Management Strategy (BMS) documents the potential beach management activities which will be required to maintain design beach levels. The BMS assumes the prior construction of a 'T' shaped groyne as illustrated in Figure 1.



Figure 1. Coastal Defence Scheme, including 'T' shaped groyne, nourished beach crest

The BMS is based upon the beach being managed using beach width as the prime metric of beach condition, and the trigger for intervention. The BMS sets out:

- The current status of the beach;
- What is known about its behaviour over recent years;
- Present understanding of the sediment transport regime and beach behaviour;
- The requirements for the beach capital scheme in terms of beach width, crest level and slope;
- Minimum beach width, which will act as a trigger for intervention;
- Requirement for beach management; and
- How residual risk and uncertainty has been addressed and should be managed in the future.

It is not possible to fully predict all elements of beach evolution, due to interaction of a number of non-linear physical processes. Whilst the BMS acknowledges these risks and provides approaches to mitigation, residual risks remain that the beach will perform significantly differently to expectations, especially in localised areas and over short time periods.

The coastal process, wave and flow modelling studies provide the evidence base for the BMS. The OBC was supported by a coastal processes assessment which included an examination of the equilibrium bay shape using MepBay and beach profile variation using Shingle-B. Studies supporting the design development built upon the previous study and used a local MIKE21 Spectral Wave (SW) model to examine and refine the proposed structure layout.

A Joint Probability Analysis (JPA) was also undertaken, to derive offshore extreme wave and water level conditions relevant to the site. The MIKE21 SW model was subsequently used to transform these conditions inshore to the toe of the proposed beach control structure to support the design of the structure. Flow modelling was also undertaken to investigate the effect of increased flow speeds on the Dŵr Cymru Welsh Water (DCWW) outfalls located nearby.

These studies were documented in the following:

- ABPmer, (2017). Penrhyn Bay Coastal Defence Improvements, Baseline review and option development, ABPmer Report No. R.2776. A report produced by ABPmer for Ove Arup and Partners Limited, April 2017; and
- ABPmer, (2020). Design Development Modelling, Penrhyn Bay, ABPmer Report No. R.3471 Revision 2. A report produced by ABPmer for Ove Arup and Partners Limited, September 2020.

The results of these studies have been reviewed in combination and interpreted with respect to future beach mobility, providing the evidence base for the BMS. Given the inherent uncertainties in beach response associated with natural changes in physical processes and climate change, the BMS should be subject to further investigation as part of Detailed Design along with checking and validation through regular beach monitoring.

The BMS has been developed based on the rock structure design developed by Arup and sets out the proposed capital beach and maintenance requirement for the beach. Providing the beach is effectively managed, the proposed beach and beach control structure is considered to be typical for such a scheme and therefore should be capable of being constructed, operated and maintained without any undue or unforeseeable health and safety risks. It is understood that Arup, as Principal Designer, will ensure that requirements for Health and Safety under Construction (Design and Management) Regulations (CDM) 2015 are fully addressed in respect to the BMS. The Principal Contractor and Client is also directed to consider their duties under CDM 2015.

2 Current Status of the Beach

2.1 Introduction

Penrhyn Bay is a northeast facing coastal embayment located around 4 km east of Llandudno and approximately 3 km northwest of Colwyn Bay. The bay is approximately 800 m in width, contained within two groyne structures facing northeast and north-northwest respectively. Aerial photography of the Bay is shown in Figure 2 and Figure 3.



Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 2. Aerial imagery of Penrhyn Bay

At the eastern extent of the site and landward of the fishtail groyne of *circa* 200 m in length, Glan-Y-Môr Road lies immediately behind the promenade, approximately 2 m below the defence crest level. Landward of the road is Dormy House Hotel and Rhos-on-Sea Golf Club, both of which are located on a low-lying alluvial plain. The heavily modified Afon Ganol drains through the golf club and exits through at outflow pipe through the eastern groyne. Three further outfalls (two long and 1 short) also exist through the eastern half of the bay and further culvert discharges through the seawall in the central section of the bay. The location of the outfalls is shown on Figure 1 (blue dashed lines).



CEUK, 2014

Figure 3. Aerial Photograph of Penrhyn Bay taken in 2002

Within the central area of the bay the road steepens uphill, eventually ending at a matching elevation to that of the promenade; of which widens before ending at a row of private housing immediately on top of the beach face. Behind the road is primarily domestic housing associated with the town of Penrhyn Bay.

The northern end of the site contains around 300 m of privately defended properties. A mixture of grasses and scrub have established upon the upper beach material that has accumulated in the lee of the western groyne. This is a *circa* 350 m structure to the very north of the study extent, with the beach north of this leading up onto the carboniferous limestone headland of Little Orme.

The study area lies within the North West England and North Wales Shoreline Management Plan (SMP2, Halcrow 2011) Area 11a. The SMP2 long term plan for Unit 11a is to 'continue to manage the risk of coastal erosion and flooding to industrial, residential, non-residential, commercial and amenity assets along the coast'.

The SMP2 recommends that 'beach management will become increasingly important to sustain beaches which are important for coastal defence, amenity, tourism and environmental conservation' (Halcrow, 2011).

2.2 Hydrodynamic Environment

2.2.1 Water levels

The coastline surrounding the study site has a spring tide range in excess of 7 m. This is due to the tidal wave propagating into Liverpool Bay via the St Georges Channel to the south and an additional channel between Ulster and the Mull of Galloway to the north. This propagation is virtually simultaneous (Myres, 1993), leading to an amplification effect on the tidal wave. The tidal cycle at Penrhyn Bay is semi-diurnal, i.e. two tides per day.

Table 1 provides the astronomical tides at Colwyn Bay, the nearest location for which values are available from Admiralty Tide Tables (ATT). These levels are based on corrections from the nearest Standard Port.

Table 1. Tidal characteristics at Liverpool, adjusted locally for Colwyn Bay

Location	Reference Still Water Level							Mean Range (m)	
	HAT	MHWS	MHWN	MSL	MLWN	MLWS	LAT	Spring	Neap
Liverpool (mCD)	10.3	9.4	7.5	5.3	3.2	1.1	0.0	8.3	4.3
Liverpool (mODN)	5.37	4.47	2.57	0.37	-1.73	-3.83	-4.93		
Colwyn Bay (mCD)	8.7	7.8	6.1	4.20*	2.4*	0.78*	-0.1*	7.0	3.7
Colwyn Bay (mODN)	4.6	3.7	2.0	0.10*	-1.8*	-3.42*	-4.30*		
* Indicates no available data in 2016 ATT, therefore values from CEUK (2014) referenced									

Source: UKHO, 2016.

Datum conversion for Colwyn Bay and Liverpool from metres above chart datum (mCD) and mODN are -4.10 m and -4.93 m respectively (UKHO, 2016).

Extreme water levels off Penrhyn Bay are provided in Table 2 below which have been derived from Environment Agency (2019) Coastal Flood Boundary Dataset (Location 1116).

Table 2. CFBD extreme water levels.

Return Period	EA CFBD mODN (2017 base year)
20	5.09
50	5.20
100	5.28
200	5.36

2.2.2 Flow regime

Across nearshore regions of Liverpool Bay tidal flows are generally east to west in orientation, with the flood tide in an easterly direction, and the ebb in a westerly. Flow speeds off the coast are generally higher on the flood than the ebb, with values around 1 m/s and 0.74 m/s respectively (HR Wallingford 2005).

The Little Orme Head is also understood to create a clockwise eddy during benign conditions, resulting in a small localised current within Penrhyn Bay (Halcrow 2012).

Further inshore tidal streams are approximately parallel in orientation to the coastline, rotating more south-easterly in direction on the flood, and north-westerly on the ebb. Flow speeds prior to the construction of the groynes were found to be significantly lower in magnitude to those in the wider Bay (British Maritime Technology, 1987), with flows generally eastward on the flood and westward on the ebb. With the construction of the western and eastern groynes in 1990, this flow inshore has also been intercepted such that tidal flows within bay will have been further weakened and thus sediment transport processes in the bay will be dominated by wave induced currents.

The predicted flood and ebb tidal flows over the wider region are presented in Figure 4 (reproduced from ABPmer, 2020), which confirm the conditions described above, and further highlight the reduction in tidal flow velocities close to the coast and within Penrhyn Bay.

The predicted wave and tide driven currents over high water, under a 1 in 1-yr WNW wave event, are presented in Figure 5, for both the baseline and proposed scheme. From Figure 5, the wave driven longshore current can be seen to be directed around and over the western groyne, where it then crosses the bay and heads around and over the eastern fishtail groyne. With the 'T' groyne and nourished beach in place this flow is funnelled, across the seaward side of the 'T' groyne increasing flow speed just offshore of the structure. Other small local changes can also be observed, but these are not considered to affect the general variability of the scheme. Further discussion on the predicted flow regime is provided in ABPmer, 2020.

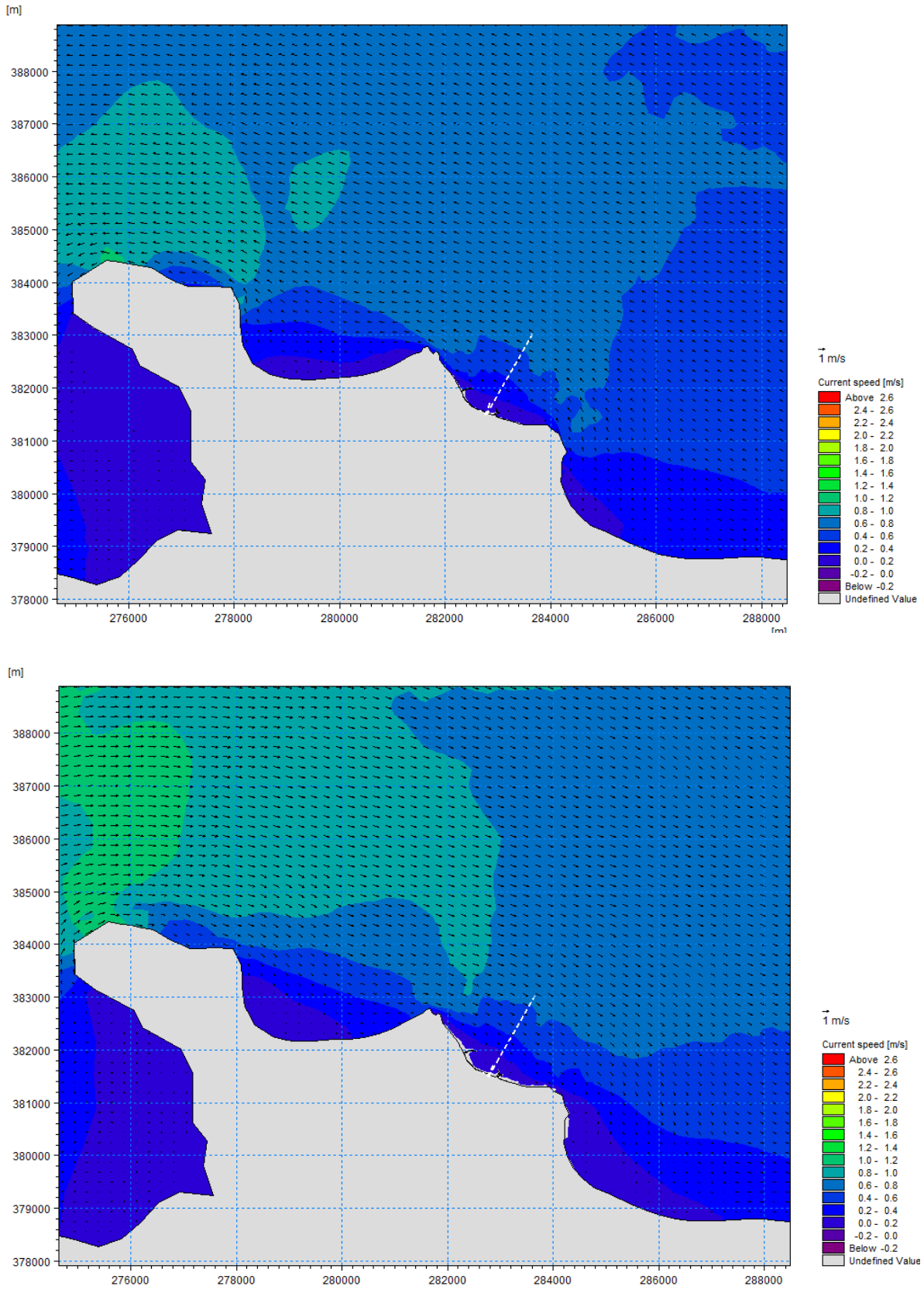


Figure 4. **Peak ebb (top) and flood (bottom) tidal currents over spring tide (excluding waves)**

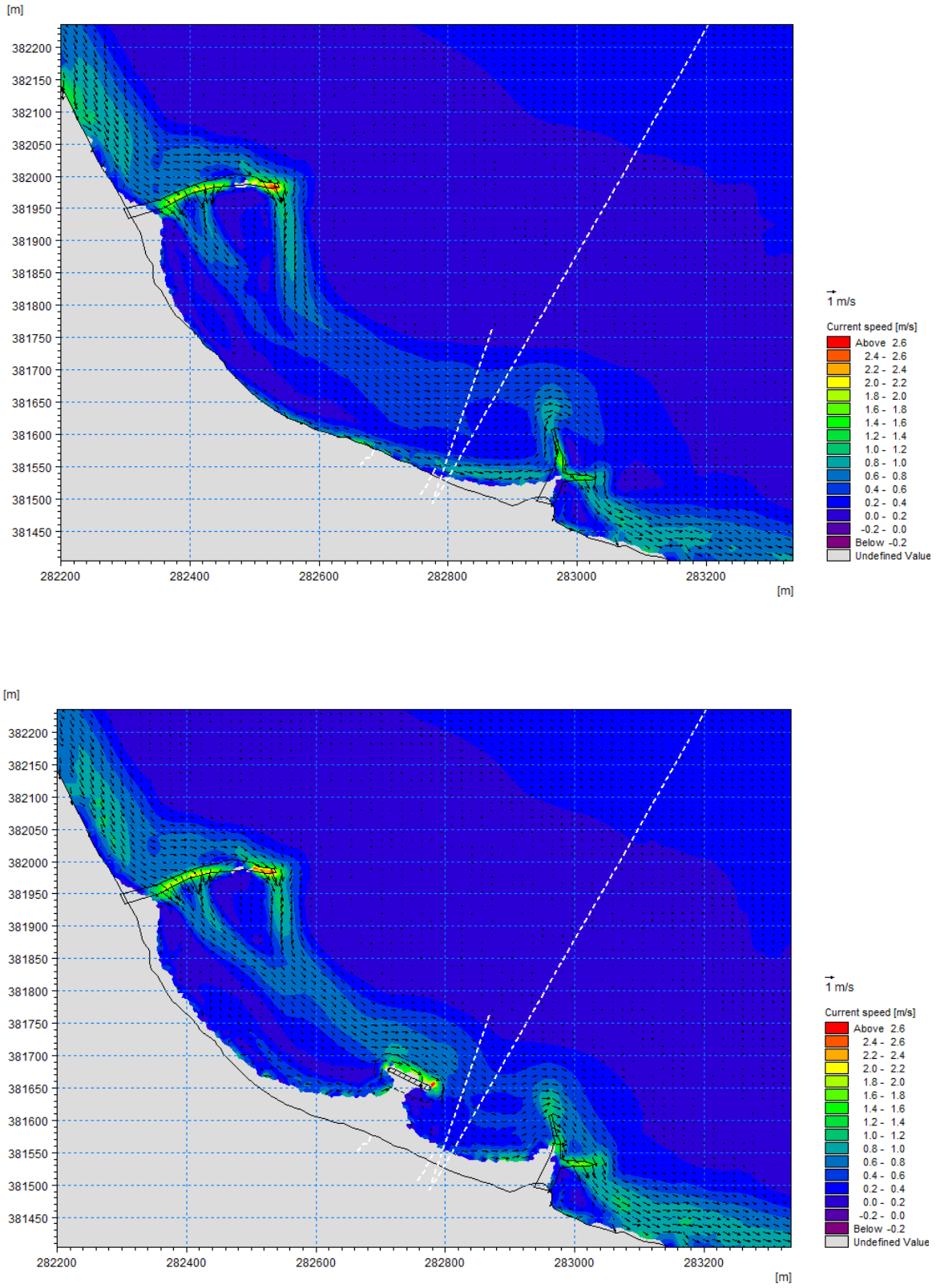


Figure 5. **Peak combined 1 in 1-year wave and tide current flow: Baseline (top) and Scheme (bottom)**

2.2.3 Wave climate

Little Orme Head provides a degree of wave sheltering for Penrhyn Bay from offshore high magnitude north-westerly wave conditions, which are partially refracted and diffracted from their initial generation in the Irish Sea. Easterly and north-easterly conditions are restricted in fetch length by parts of northeast Wales and northwest England. Similarly, the Constable Bank (located approximately 7 km offshore) is likely to limit the effect of northerly storm waves and provide a degree of wave dissipation (Halcrow 2012), particularly at low tide.

Predominant wave conditions just offshore of the bay occur from north-north-westerly or northerly directions, with over 90% of significant wave heights less than 1.5 m (Figure 6). As these waves reach the bay, and location of the groynes (in maximum depths of approximately -2.5 (western) and -2 (eastern) mODN respectively), they will be afforded further refraction.

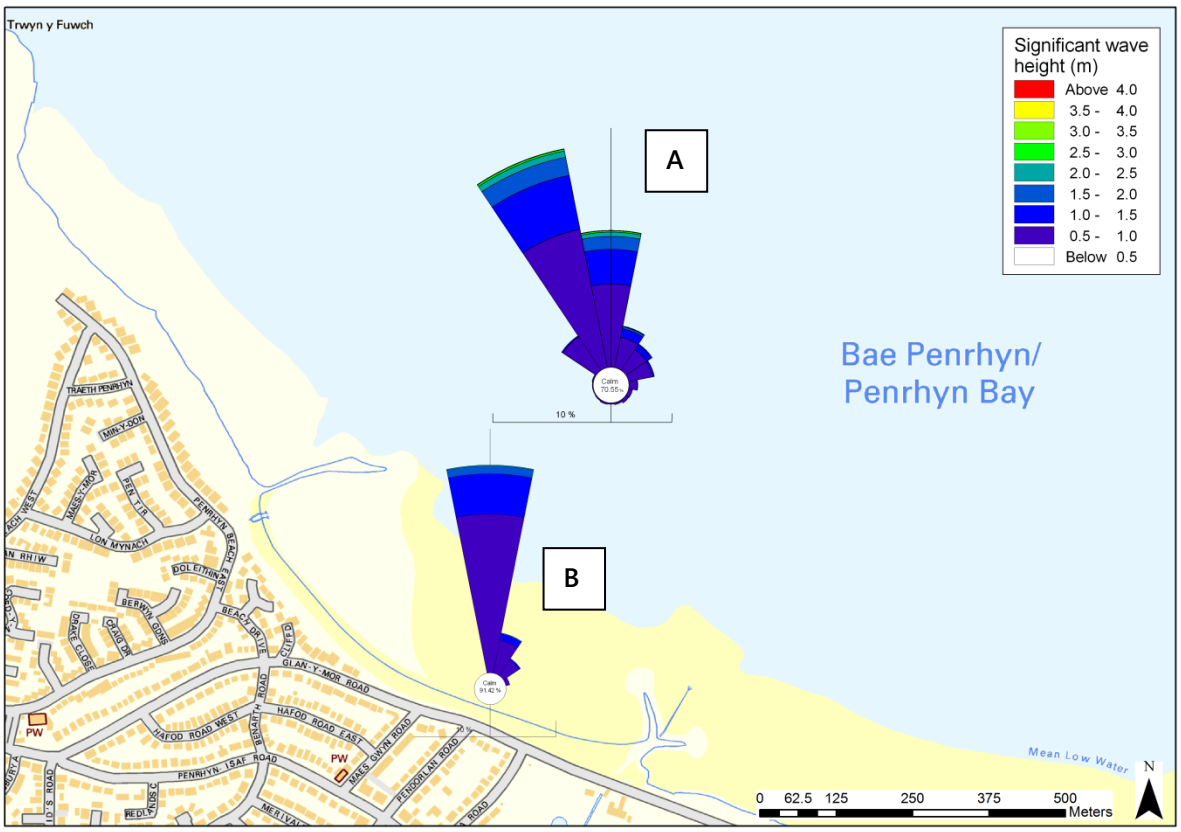


Figure 6. Nearshore (A) and inshore (B) wave roses using a combination of observed water levels and offshore SEASTATES hindcast

Waves from the northeast are less frequent and typically have a significant wave height less than 1 m. These waves, however, will be afforded less refraction as they are transformed towards the shore.

The inclusion of the 'T' shaped groyne within the bay results in localised wave diffraction around the ends of the 'T' arms, which in turn adjust the natural alignment of the beach in the lea of the structure.

2.3 Sediment transport regime

Penrhyn Bay is largely placed upon bedrock of carboniferous limestone, which can be seen to outcrop at Little Orme Head. Immediately on top of this is a layer of sandstone which is likely to provide a source of sediment. Sporadic Quaternary deposits of boulder clay are also apparent, along with estuarine alluvium linked to the last glacial period.

The overall littoral drift is reported to be from west to east along the frontage containing Penrhyn Bay (Halcrow 2012). An overall sediment transport potential (not considering restricting structures) of 22,000 m^3 was calculated by HR Wallingford (HR Wallingford 1991), although this is understood to relate to the movement of sand sized fraction.

The Little Orme headland restricts most of the sediment drift from beaches to the north of Penrhyn Bay, but a small amount of fine material by-passing is suggested to occur (Liverpool Bay Coastal Group, 1999). Tidal currents across Penrhyn Bay are small, and therefore most sediment transport is expected to be due to wave action, which can also result in both the onshore and offshore transport of finer sediment material.

The supply of sediment to Penrhyn Bay has been considerably modified as a result of the construction of the two groyne structures (CEUK 2014). The bay has become compartmentalised with no ability for coarser shingle and cobble material to enter the bay from the west. The eastern groynes also limits the loss of this material from the bay, although some small losses are observed when the berm builds up at the eastern end of the beach and becomes transported over the base of the eastern fishtail groyne.

The influx, export and transport of finer material on the lower foreshore has also become more complex as a result of the groyne structures, since the natural littoral drift has been blocked and more complex currents are setup in the bay by the structures. The difference in the relative lengths of the two structures and the orientation of the bay to the prevailing waves, may suggest that there could be a greater loss from the bay than an influx of material. However, the bed level change calculation in CEUK, 2014 do not suggest a significant long-term loss or gain of material in the lower foreshore. It is also noted that underlying boulder clay is generally resistant to erosion but vulnerable if exposed to wave action. Thus, over time, it could still be expected that there will be some overall beach level lowering in the lower foreshore.

The natural intertidal consists of a mixture of sand and shingle, with higher sand proportions in the shallower graded lower beach and shingle/cobble material in the upper steeper beach. This coarser material is suggested to be a combination of the beach recharge material, and a product of earlier shoreline erosion exposing glacial deposits.

The primary movement of sediment in the bay relates to the upper shingle/cobble beach in the central and eastern end of the bay (CEUK, 2014). This movement was noted immediately after the 1990 beach recharge (CEUK 2014). In the eastern half of the bay the drift is west to east and results in overall lowering of beach levels in the central section of the bay and an accretion of material in the eastern section. Several reinstatement campaigns have been undertaken to recycle material from the eastern section to the central section, however, in each case this material is returned to the eastern end of the bay. As opposed to this the western half of the bay has remained relatively stable.

The primary reason for these observations is that the upper beach is now directly controlled by the two groyne structures and it now wishes to adopt an alignment which it not commensurate with the existing defences. In particular, the defence in the central section of the bay, protrudes seawards of the natural alignment the beach would wish to adopt. Therefore, material placed in front of this central section

during the reinstatement campaigns, is relatively quickly transported to the eastern end of the frontage as the beach attempts to recover to a more natural equilibrium position.

2.4 Upper beach behaviour

At the eastern extent of the Bay, a pronounced beach berm exists with crest heights of up to around 5.8 mODN. This berm lies approximately 30 m seaward of the defences, at its widest point. The berm height is also higher than the beach level directly against the defences, where the beach levels are approximately 4.5 mODN. The original nourished level of the beach, when originally instated in the central section of the bay in 1990, was around 4.9 m.

Towards the more central section of the bay, the upper beach pushes up against the defence with no noticeable berm.

In the western section of the bay a beach berm again becomes more pronounced, where an establishment of grasses and scrub can be seen on the top of the berm. Here, the maximum berm height is around 4.5-5 mODN. The berm is generally much flatter than that observed at the southern end of the bay and is only marginally higher, than the beach level at the toe of the cliff (4.2-4.5 mODN). At this location the beach is more sheltered from the larger, longer period, north-westerly waves which would act to build up the berm under extreme wave conditions.

Current beach management within Penrhyn Bay originated in the 1980's as a result of a previous failure to retain beach levels at the toe of the pre-existing 1950's public defences. Following consultation, the western and eastern groynes were constructed in 1990 to control the movement of sediment. In addition to this a beach recharge of mixed material was carried out, this included the addition of:

- Sand (D_{50} 0.5-2 mm) throughout the privately defended western section of the Bay;
- Shingle (D_{50} 20-40 mm) throughout the central section; and
- Cobble material (D_{50} 120-130 mm) throughout the eastern section of the Bay.

The recharge raised the upper beach profile to a slope of 1:10 up to MHWS level, with a 1:20 gradient from MHWS up to a level of 4.9 mODN (CEUK 2014). This nourishment profile is shown in Figure 7.

Since this recharge the sheet piling at the toe of the defences has not been consistently exposed as a result of beach drawdown and the realignment of the beach. Overall, the recharge has been considered to have provided an improvement to the pre-existing defences. However, as time progresses ongoing attrition and abrasion of the recharge material has resulted in a fining of material size by around 50%, and is therefore more mobile than desirable (HR Wallingford, 2000).

A series of reinstatement schemes have been undertaken since 2003, with the natural profile of the upper beach artificially altered to protect the existing defences from toe exposure. Such activities are required every 12-18 months (CEUK 2014) and consist of the re-cycling of material from the accreting eastern section of the Bay (where overspill of the eastern groyne is also apparent) to the more central sections of the bay which experiences the greatest beach erosion.

CEUK 2014, examined the variation in beach in beach levels throughout the bay from 1997 to 2010, by comparing the numerous levels data sets available. This concluded that the volumes in the western side of the beach have remained stable, although material reinstated within the central portion of the beach is quickly observed to migrate to the eastern side of the bay. However, in general the report suggests that no significant change in overall volume of the upper beach has occurred across the Bay when considering reinstatement activities.

2.5 Historic interventions

The two rock armour breakwaters were designed to modify the wave propagation inshore and encourage the natural accretion of material between them - the aspiration was a stable beach with self-sustaining sand nourishment. However, the imported beach has been found to be mobile under the action of waves and tides, particularly encouraging transportation from the centre of the bay towards the eastern breakwater. Hence, regular beach maintenance activities to recycle material back to the centre of the bay to protect the existing back-shore assets are required every 18-24 months, at significant cost to CCBC's capital maintenance funds.

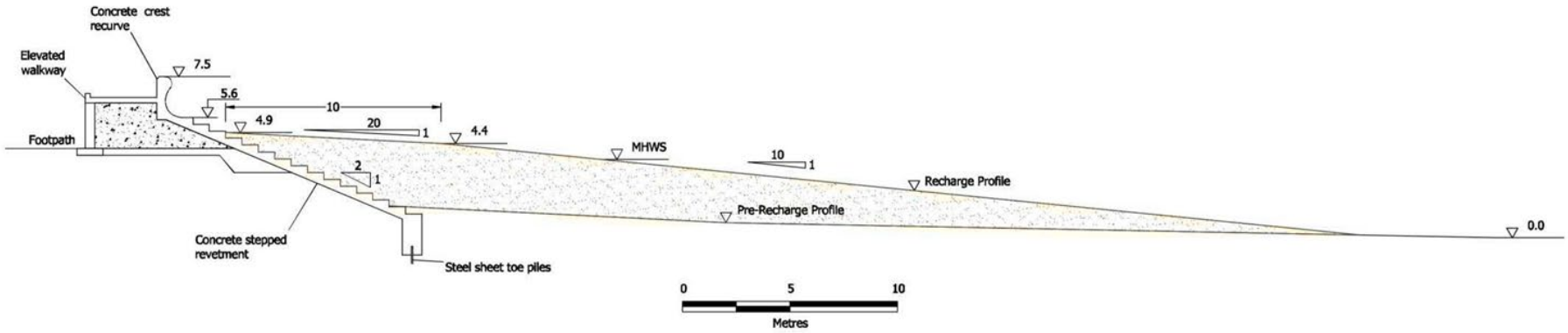
Since 2003, the following maintenance practices have been undertaken within the bay:

- Recycling of upper beach material and re-profiling activities to move transported material back to the central section of the frontage - every 12-18 months depending on requirements.
- General maintenance and repairs to the stepped revetment - every 2 years
- Removal of shingle from the Afon Ganol outfall - every 12-18 months

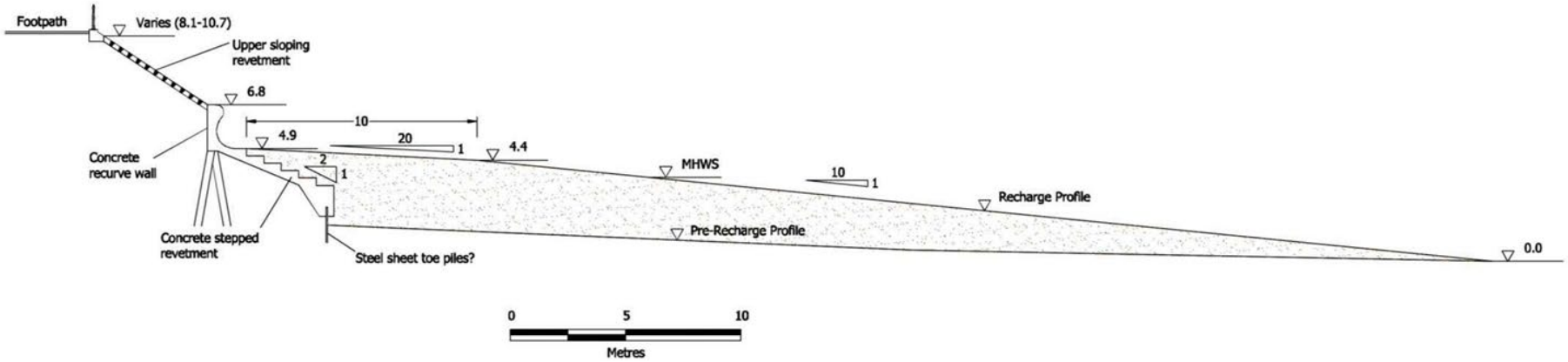
A typical work plan for the recycling/profiling work has been provided by CCBC and is presented in Figure 8. From Figure 8 the reinstated profile has a relatively flat upper berm with a crest height of 5 m and width of 8 m, then a uniform beach slope of 1 in 6.

Overall, the recharge/recycling has been considered to have provided an improvement to the pre-existing defences. However, the material that has been placed within the central section of the beach has be transported back toward the eastern end of the bay relatively quickly.

Example images, showing the natural accretion of material in the eastern section of the bay, overspill of material over the eastern groyne and the reinstate activities are provided in Figure 9,



Recharge profile in Defence 1



Recharge profile in Defence 2

Source: CEUK, 2014

Figure 7. Profile of the 1990 recharge throughout eastern (top) and western (bottom) extents of the Bay

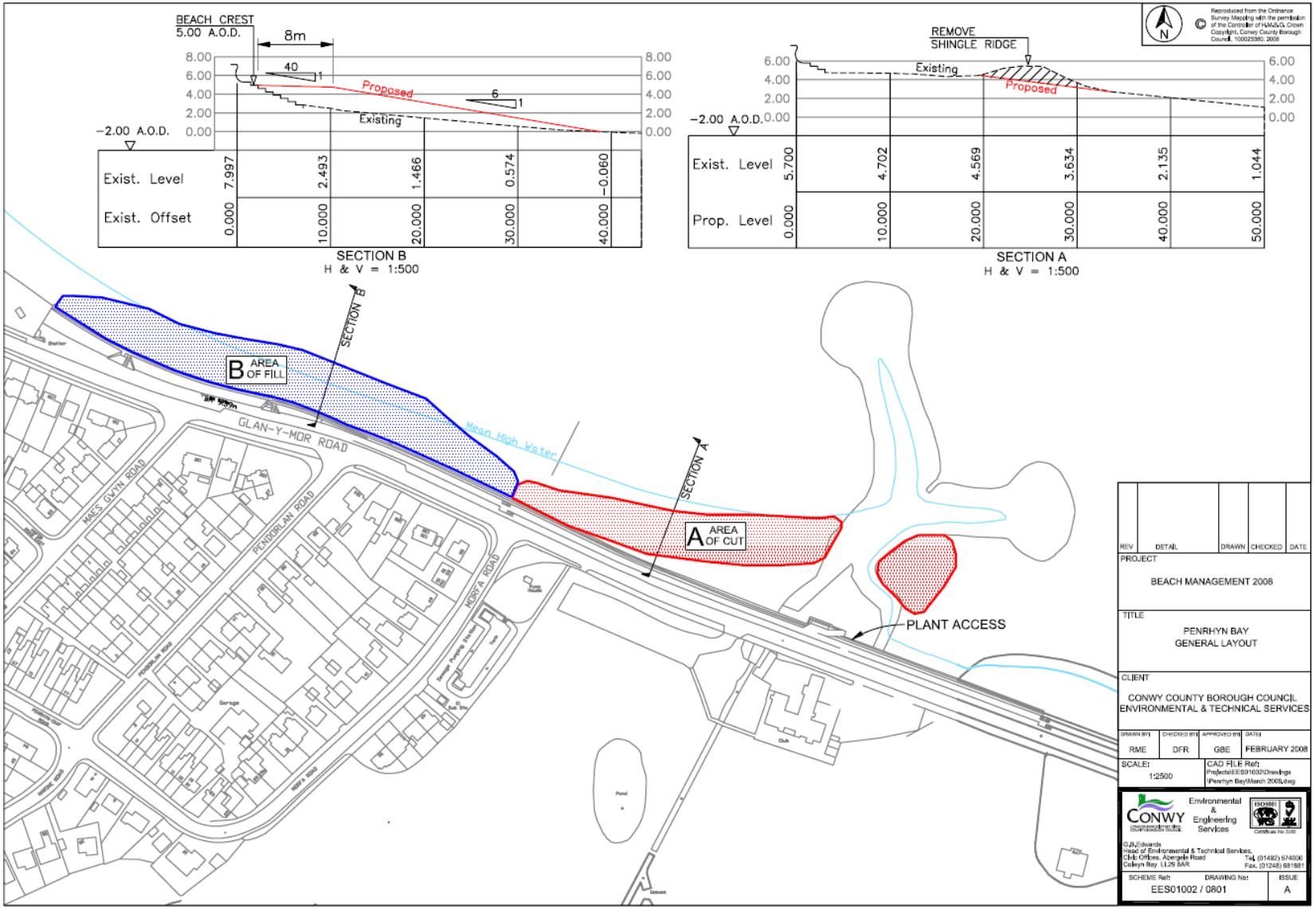


Figure 8. Typical beach management recycling/reprofiling activity



Eastern beach accumulation 10/02/2005



Reinstatement 15/03/2005



Eastern groyne overspill 10/02/2005



Reinstatement 15/03/2005



Eastern beach accumulation 15/03/2007



Reinstatement 28/03/2007



Eastern groyne overspill 15/03/2007



Reinstatement 28/03/2007

Figure 9. **Accumulation of sediment in the eastern section of the beach and sediment overspill of the eastern groyne (left) requiring frequent reinstatement works (right)**

3 Capital Nourishment Scheme

Analysis of the historic beach behaviour (CEUK 2014), has shown that there has been no significant change in overall volume of the upper beach. However, regular recycling and beach maintenance has been required as the layout of the present beach control structures does hold material within the central section of the bay.

The new 'T' shaped groyne has therefore been designed to hold a nourished upper beach within the central section of the bay, thus providing protection to the sea wall and additional amenity benefit.

The development of the 'T' shaped groyne, and the location, size and extent of the structure have been considered and refined to provide a relatively stable upper beach which will help to minimise the requirement for future beach management. The scheme has also been developed together with recommendations to raise the level of the eastern fishtail groyne to help limit the loss of upper beach material from the bay.

As part of the scheme, capital beach nourishment will provide for a beach width that allows for natural change and storm response without the need for intervention. Details of the nourishment scheme are provided below.

The scheme has been developed using the desk based coastal process investigation produced in support of the OBC (ABPmer 2017), and wave modelling (ABPmer 2020). Sediment transport modelling has not been undertaken and the scheme's parameters have been determined based upon expert geomorphological judgement. Further investigation and assessment could be undertaken to refine and examined the scheme further at detailed design.

The scheme should be supported by regular beach monitoring to determine the sensitivity of the beach to certain wave conditions and the effects of climate change.

3.1 Beach width

The beach width represents the beach berm width that is likely to be required to allow for the beach berm to regress landwards during extreme storm events based on cross-shore process. It also provides a buffer for longshore migration, and the readjustment of the bay shape to different incident wave conditions. Initial investigations in support of the OBC identified a preferred design beach berm width of 20 m. This was determined from an assessment of existing cross shore variability using parametric modelling of shingle movement under different sea states including bimodal seas (ABPmer, 2017).

Spectral wave modelling was undertaken to support the rock groyne design (ABPmer, 2020) and used to examine the beach plan shape that would be induced as a result of diffraction around the structures. As the seabed in the lee of the structures, and the nourished material to be placed as part of the scheme, is mobile, this material will wish to realign behind any new structure until a new equilibrium position is obtained. This equilibrium position is affected by the incident wave direction and the resultant wave diffraction behind the structure.

Wave events representative of a 1-in-1-year event, from three directional sectors (WNW, NNW and NNE) were modelled with a deepened inshore bathymetry, to show the diffraction patterns and bay shapes that would be expected based on the three groyne structures (western and eastern groynes and new central structure). The wave crests were interpreted to define the beach shape that is likely to result from the structures under the incident wave direction being considered.

In this assessment a minimum beach width of 20 m was initially chosen, as this was identified within the OBC assessment as the indicative buffer width that may be required if the beach is to provide the primary defence for the frontage. However, as the beach is not required to provide the key flood protection function, but instead is being included to provide protection to the sea wall, it was considered that a smaller buffer width may be acceptable.

A reduced buffer width of 10 m was assessed which is close to the upper beach crest width of 8 m that has been used recently when reinstating the beach as part of the current maintenance programme. The resultant plan-shapes for the three incident wave conditions are shown in Figure 10 for the preferred 'T' shaped groyne alignment.

The beach crest positions show that:

- A tombolo is predicted to form with the beach crests attaching to the shore connecting structure, but the beach crest is not expected to attach to the shore parallel part of the structure, thus a fully developed salient is not expected;
- With the nourished beach having a typical slope of 1 in 8 (see section below), the nourished material should generally be contained landward of the shore parallel component of the 'T' groyne.
- The pinch point in the western embayment (to west of 'T' groyne), is at the western end of the seawall do be defended, for the WNW and NNW wave conditions, for the less frequent NNE waves the beach crest more closely follows the alignment of the seawall, with no specific pinch point location;
- In the eastern embayment (to the east of the 'T' groyne), the pinch point, is similarly located close to the three outfalls for all three wave conditions.
- The beach alignments for the WNW and NNW are closely aligned, with the greatest variability observed for the NNE waves. However, persistent large waves from the NNE are much less common and they are less likely to persist for period required to fully realign the beach to this position;
- As the eastern fish tail groyne the beach crest lies in a similar position to that observed at present, although there is some variability in this position between the WNW/NNW and the NNE wave alignments. At present material is observed to wash over the trunk of this structure and is lost from the embayment. It is therefore recommended that the crest level of the structure is raised to prevent this; and
- Any seaward migration of the beach crest at this location could also lead to material being transported around the fishtail groyne. The present schemes show that the location of the beach crest is similar to that presently observed, but this will be an area that will require monitoring.

As a result of the study, the 'T' groyne connects the shore parallel component of the structure to the land, so that there will be limited natural transfer of material from the western embayment to the eastern embayment and *vice versa*.

A **buffer width of 10 m** is proposed, and it is important to note that beach monitoring will give further understanding of beach response to wave conditions and this buffer width could be reduced in the future.

It is proposed that beach is initially nourished using the **beach crest alignments derived from the WNW and NNW wave conditions**, taking the most seaward alignment from each.



Figure 10. 'T' shaped groyne and beach crest positions with 10 m minimum buffer width

3.2 Crest height

It is important that the nourished beach crest height, beach width and beach slope (plan-shape) are considered together, as this effectively defines the upper beach volume available within the bay.

The last major nourishment scheme used a crest level of 4.9 mODN, and a similar level has been adopted when reinstating the beach in subsequent maintenance campaigns. However, the natural berm height at the eastern end of the beach has typically been around 5.8 m ODN (ABPmer 2017). This is also indicated in Figure 8. The natural berm height results from the incident waves and water levels under more extreme events pushing up the material at the back of the beach (assuming sufficient beach material is available).

The cross-shore modelling undertaken for OBC (ABPmer 2017), also estimated berm levels of approximately 6 m for annual events rising to *circa* 7.5 mODN for extreme events. Under the climate change scenario, the beach berm is predicted to reach a level around 8 mODN for a 1:100-year event. However, from other studies it has also been identified that the Shingle-B model utilised in these studies may overestimate the actual height the crest the berm may achieve under the more extreme conditions.

From, these assessments, it is considered appropriate to design a present-day beach crest that is relatively stable under non-extreme conditions and a **design beach crest height of 6 mODN is proposed**. This berm height allows for coverage of the stepped revetment and the lower section of the wave return wall.

As sea level rise occurs, the beach crest would natural increase, therefore, is it proposed that additional nourishment material should be obtained to raise the beach crest level in the future in line with sea level rise.

The design beach berm height is therefore anticipated to be 6 mODN for present day conditions.

3.3 Beach slope

A **design slope seaward of the berm of 1:8** is proposed. This is the typical slope for the (shingle) sediment grain size proposed. This is the approximate slope of the upper eastern beach shown in the profiles assessed at OBC (ABPmer, 2017).

The appropriate slope should be reviewed once the source of sediment has been identified.

3.4 Sediment grain size

The objective of the scheme is to provide a shingle berm which will protect the structure from exposure and undermining, by creating a relatively stable beach plan-shape.

Ideally the source material will have similar size characteristics including size and grading to the upper beach in the eastern side of the bay. No present grading curve has been available for this material, but from photographic imagery it appears to contain a **mix of shingle and cobbles**.

It has been previously identified, that ongoing attrition and abrasion of the previous recharge material has resulted in a fining of material size by around 50%, and is therefore more mobile than desirable (HR Wallingford, 2000). I.e. the material was lost more rapidly from the central section of the bay following reinstatement that desired. With the new 'T' shaped groyne, the upper coarse beach material will be held more naturally within the central section of the beach. Therefore, the key design characteristic is for it to sit at a relatively steep slope, i.e. around 1 in 8 or steeper, so the toe of the upper beach does not extend beyond the 'T' groyne or existing fishtail groyne to the east.

This should be the case, if the nourished beach grading is similar or coarser to the existing beach material. As identified from the earlier nourishment campaign, fine material will be winnowed out from the beach and 'softer' material will be more prone to abrasion. Such factors should be considered when selecting the nourishment material and the design of the beach adjusted accordingly.

3.5 Trigger level for Intervention

The proposed scheme has been developed to help maintain an upper beach in the central section of bay which then provide additional protection to the existing seawall. The nourished beach is not required to meet any other performance criteria, e.g. in respect to overtopping. Therefore, there is no defined performance criteria for the beach.

The 'T' shaped groyne and nourished beach has been developed to provide a relatively stable beach, and the proposed present-day beach crest is similar to that naturally observed at the eastern beach. Therefore, a significant reduction in the beach crest width below the 10 m minimum width used to define the plan-shape should be limited. It is also likely that this reduction will occur close to the identified pinch points (Section 3.1).

Nevertheless, under very large events the cross-shore profile can be expected to vary, and the beach berm will be pushed back towards the sea wall, and adjustment to the plans shape will occur as the

beach responds to the different incident wave conditions. Furthermore, over the longer-term some loss of material may also occur from the wider bay.

To manage this a trigger condition needs to be set to identify when the beach levels are such that their performance in protecting the seawall are compromised and therefore intervention works should be undertaken to reinstate the beach.

Overtime and with longer-term monitoring, it will be possible to set this trigger level with a greater understanding of the conditions at the site with the scheme in place, but initially it is suggested the trigger level should be defined as:

- **A minimum 2 m beach crest width, with a level above 6 mODN**

This minimum beach crest width should be maintained along the entire length of the seawall to be protected.

The rationale for this is set out below:

- The predicted beach alignments at the key pinch point locations in both the western and eastern embayments after the scheme is constructed for the three incident conditions examined. Furthermore, at these locations the predicted beach alignment is fairly similar to the seawall alignment. Therefore, at the locations where the minimum beach width is anticipated to occur there should be a relatively limited variation in the beach width due to changes in the incident wave direction.
- The beach crest height of 6 m is slightly greater to that observed on the natural beach, therefore at this trigger point there would be 2 m of material to help absorb wave energy under relatively extreme (annual events).
- If a more extreme event were to occur with the beach at this trigger condition, it is possible that, the wave could still reach the seawall, which could prevent the berm from increasing higher, and it could possible also result in some local drawdown of the upper beach profile as the wave reflect off the wall. However, it is expected that there would still be sufficient material in the beach profile to reduce much of the wave energy that would reach the wall, limiting the potential for damage of the seawall.

The trigger level should be regularly reviewed as part of the beach management strategy, as the performance of the beach is better understood, and greater monitoring data is available. It is also expected to be amended in response to climate changes and sea level rise.

4 Monitoring and Maintenance Requirements

4.1 Existing monitoring

Beach monitoring is presently being undertaken as part of the shoreline management process and is collated by the Wales Coastal Monitoring Centre (WCMC). Penrhyn Bay is categorised as Category 1 site by WCMC and subject to the present monitoring programme:

- A beach topography survey - consisting of beach profiles at 50 m spacing and cross lines every three years; and
- Bi-annual profiles beach profiles at 500 m centres.

The programme is presently funded for five years and the first topographic survey was completed in June/July of 2020. The data from this survey is not yet available. The initial beach profiles from the bi-annual profile monitoring survey are however available and these are shown in Figure 11. Unfortunately, only one of the bi-annual profile locations is in Penrhyn Bay. This profile lies 9 m beyond the western end of the public defences (seawall). The western end of the seawall is the primary pinch point in the western embayment. The WCMC profile does lie very close to this, but ideally it would be located at the western end of the seawall.

It is also identified that a series of LiDAR survey have been undertaken at the site in the past, although we do not know the present plan for further surveys to be flown in the future.



Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 11. Location of bi-annual profiles undertaken by WCMC (lines are surveyed at 500 m intervals)

4.2 Future monitoring requirements

Following the construction of the scheme and the initial beach management nourishment it is proposed that an extended monitoring campaign is undertaken for a three-year period. The data from this campaign will enable the performance of the beach to be better understood and will help to refine the monitoring programme going forward. It will also provide valuable information against which the trigger level can be reviewed.

During this initial three-year period, it is important to obtain information on the beach plan shape variability, including location of pinch points and the beach profile response. It is therefore important that the surveys are undertaken, bi-annually and after significant storm events (e.g. those with an AEP greater than 1 in 1 year).

Alternate approaches are available to collect data during these surveys, and the data to be obtained will typically need to be more detailed than that collected by the WCMC bi-annual surveys. However, it is important to link this post scheme/BMS monitoring with the activities of the WCMC.

One option could be for the BMS monitoring to survey the upper shingle beach along the profiles used in the WCMC topographic surveys, if the locations meet the objectives of both monitoring programmes (WCMC topographic profile locations unknown). Furthermore, since the WCMC survey programme has only recently been initiated it would be timely to explore whether the profiles being monitored could be adjusted or appended slightly, to provide profiles that are better aligned with the new features within the bay and BMS monitoring. For example, it would be preferable for the long-term bi-annual surveys to be taken in line with the pinch point locations in both the western and eastern embayments, as indicated on Figure 12.

As LiDAR data is obtained, this should also be reviewed, as this will provide further information on beach response and behaviour.

Volumetric checks should be undertaken on the LiDAR data to identify potential material loss using LiDAR data. If LiDAR data is not available, volumetric check should be undertaken using the topographic data. Although this will be less reliable than LiDAR data it will still provide valuable information on volumetric change.

It is recommended that long-term beach monitoring is reviewed after the initial three-year period at least every 5 years after this.



Map data ©2016 Google. Image © 2016 Terrametrics. Data © SIO, NOAA, U.S. Navy, NGA, GEBCO

Figure 12. Proposed location of amended or additional bi-annual profile lines (orange dashed lines) – existing WCMC profiles overlaid

5 Climate Change Considerations

Climate change over the next 100 years will likely influence the frontage through:

- Increased water levels;
- Increased wave heights within the general climate;
- Changes in mean wave direction from the long-term mean, and greater annual variation; and
- Increased storminess - increased magnitude of storms for a given return period.

Changes to incident wave directions and the dominance of waves from different direction sectors are likely to have limited impact on the schemes, which creates a relatively stable bay shape from the range of incident wave conditions examined.

The key effect of climate change will be an increase water levels which will increase the natural berm height that will be reached under extreme wave and water levels conditions. With increased water levels the waves nearshore will also increase slightly, but the effect of this is expected to be less significant in respected to beach mobility/performance than the actual increase in water level.

As the natural beach berm height will increase in line with sea level rise, the design beach berm should be raised through future nourishment campaigns.

6 Residual Risks and Uncertainties

It is not possible to fully predict all elements of beach evolution, due to interaction of a number of non-linear physical processes. The effects of sea level rise and changing wave climate as a result of climate change are also uncertain.

To provide further confidence in the scheme performance, further investigation could be undertaken as part of the detailed design, such as the assessment of the scheme using mobile bed physical modelling.

Regular beach monitoring will also be required to provide information on sediment mobility following implementation of the scheme and the BMS should be reviewed on a regular basis. The first review is recommended following an initial three-year period over which intensive monitoring should be undertaken and then on a five-yearly basis after this. The period can be revised once there is greater understanding of post scheme sediment mobility.

The scheme has been developed to provide protection to the seawall and the trigger level for intervention has been set, accepting that some wave energy could reach the seawall under extreme wave and water level conditions. There will still, therefore, be a requirement for seawall inspections. The scope for this is not covered here but should be considered separately.

Similarly, this document does not address inspection and maintenance requirements for the new 'T' groyne. However, it is noted, that whilst the maintenance requirements of the rock armour forming the 'T' groyne will depend on the rock sizing to be defined at Detailed Design, the overall functionality and efficiency in promoting wave diffraction and retaining sediment transport should not be significantly affected by minor damage and dislodgment of a small percentage of rocks.

7 References

ABPmer, 2017. Penrhyn Bay Coastal Defence Improvements, Baseline review and option development, ABPmer Report No. R.2776. A report for Ove Arup and Partners Limited, April 2017.

ABPmer, 2020. Design Development Modelling, Penrhyn Bay, ABPmer Report No. R.3471 Revision 2. A report for Ove Arup and Partners Limited, September 2020.

ABPmer - SEASTATES: http://www.seastates.net/

British Maritime Technology, 1987. Aberconwy Coastal Study Parts I-V. Report No. 4627702

CEUK, 2014. Penrhyn Bay Coastal Defence Assessment. Coastal Engineering UK Ltd

Environment Agency (2019) Coastal flood boundary conditions for the UK: update 2018. Project: SC060064/TR7: User guide.

Halcrow, 2011. North West England and North Wales Shoreline Management Plan SMP2

Halcrow, 2012. North West England and North Wales Shoreline Management Plan SMP2 Supporting Studies- Joint Probability Study: Extreme Wave Heights and JOINSEA results

HR Wallingford 1991, Colwyn Borough Sea Defence Review. Report No. EX2367.

HR Wallingford, 2000. Conwy County Borough Council. Post Project Appraisals. Performance Evaluation: Penrhyn Bay Coastal Works.

HR Wallingford 2005, Rhos-on-Sea Breakwater Siltation Study, Colwyn Bay. Advice on managing sediment accumulation. Report EX 4992.

Liverpool Bay Coastal Group, 1999, Liverpool Bay Shoreline Management Plan, Subcell 11a. Great Ormes Head to Formby Point.

Myres, 1993, West Coasts of England and Wales Pilot: Wests Coasts of England and Wales and South Coast of Scotland from Cape Cornwall to Mull of Galloway including Isle of Man. 12th ed. Taunton: Hydrographic Office. 323 pp.

UKHO, 2016, Admiralty Tide Tables. UK Hydrographic Office.

8 Abbreviations/Acronyms

AEP	Annual Exceedance Probability
ATT	Admiralty Tide Tables
BMS	Beach Management Strategy
CCBC	Conwy County Borough Council
CD	Chart Datum
CDM	Construction (Design and Management) Regulation
CEUK	Coastal Engineering UK Ltd
CFBD	Coastal Flood Boundary Dataset
DCWW	Dŵr Cymru Welsh Water
DHI	Danish Hydrological Institute
EA	Environment Agency
GEBCO	General Bathymetric Chart of the Oceans
HAT	Highest Astronomical Tide
JPA	Joint Probability Analysis
LAT	Lowest Astronomical Tide
LiDAR	Light Detection and Ranging
MepBay	Model of Equilibrium of Bay Beaches
MHWN	Mean High Water Neaps
MHWS	Mean High Water Springs
MIKE21	Modelling Software - hydrodynamics, waves, sediment dynamics, water quality and ecology (DHI)
MLWN	Mean Low Water Neaps
MLWS	Mean Low Water Springs
MSL	Mean Sea Level
NGA	National Geospatial-Intelligence Agency
NOAA	National Oceanic and Atmospheric Administration
OBC	Outline Business Case
ODN	Ordnance Datum Newlyn
SIO	Scripps Institution of Oceanography
SMP2	Shoreline Management Plan (Round 2)
SW	Spectral Wave
U.S.	United States (America)
UK	United Kingdom
UKHO	UK Hydrographic Office
WCMC	Wales Coastal Monitoring Centre

Cardinal points/directions are used unless otherwise stated.

SI units are used unless otherwise stated.

Contact Us

ABPmer

Quayside Suite,
Medina Chambers
Town Quay, Southampton
SO14 2AQ

T +44 (0) 23 8071 1840
F +44 (0) 23 8071 1841
E enquiries@abpmer.co.uk

www.abpmer.co.uk