Kyleakin Pier Development
Hydraulic Modelling
Final Report

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Consulting Engineers

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1 INTRODUCTION

Marine Harvest Scotland Ltd have appointed RPS to undertake computational modelling of the hydraulic regime for a proposed scheme which will involve developing an existing Pier site located immediately to the west of Kyleakin, see Figure 1.1 below. The proposed scheme includes developing the existing Pier to create a 160m long berth and using a series of concrete caissons to create a smaller 79m long berth at the end of the existing Pier. The proposed development also includes the capital dredging of *c*. 190,000m^3 to achieve a minimum operating depth of -8.5m CD.

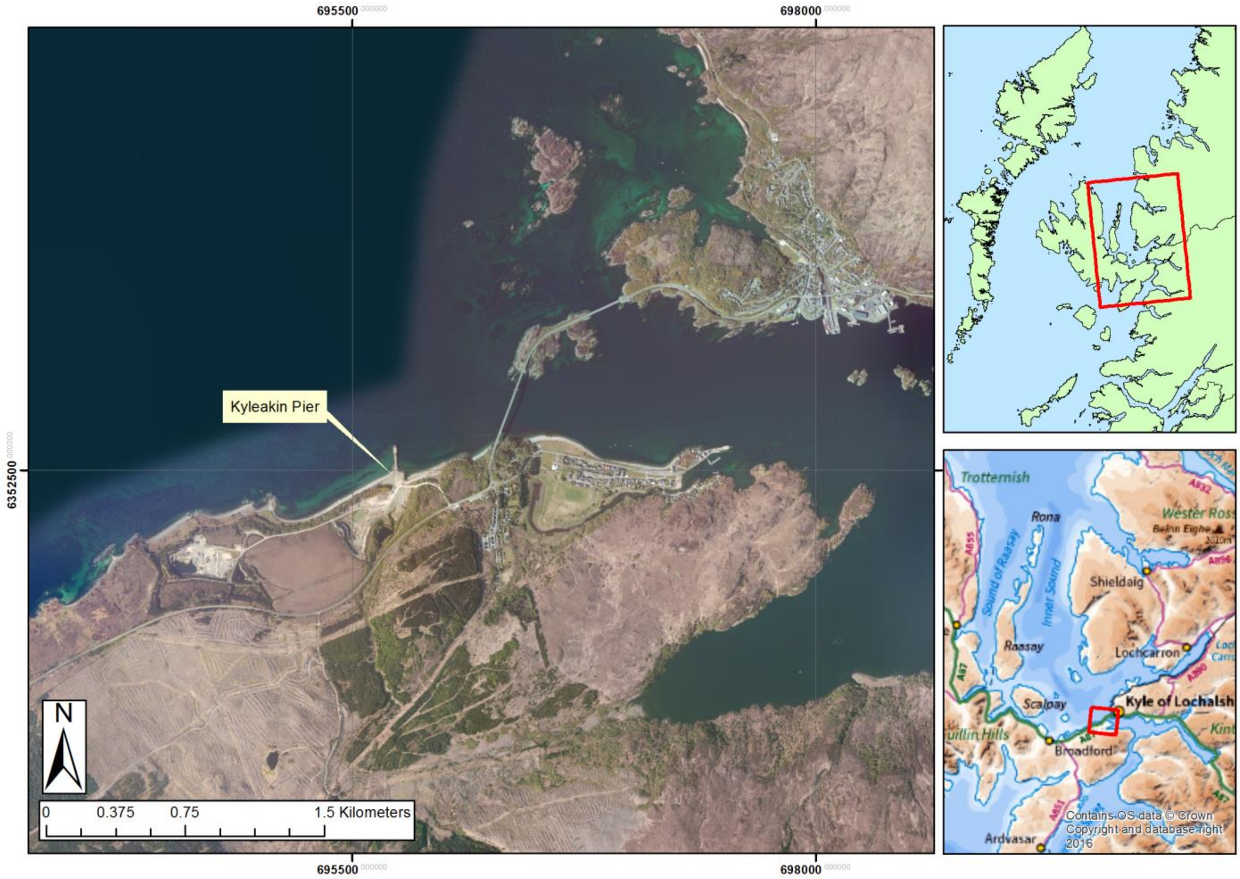


Figure 1.1: Location of Kyleakin in relation to the Isle of Skye and Outer Hebrides.

The area surrounding the existing Kyleakin Pier has several environmental designations, including one Special Area of Conservation (SAC) and one Special Protection Area (SPA) amongst others. Thus advanced computational modelling is required to investigate the possible impacts of the proposed Pier development on the adjoining designated areas.

The objectives of this study are to:

1. Briefly describe existing site conditions and the development scheme proposed for the existing Pier at Kyleakin
2. Present the field data collected in support of this study.
3. Develop and calibrate numerical models of the Kyleakin area.
4. Identify and summarise baseline conditions in the study area.
5. To understand and quantify the potential effects of the proposed scheme on the existing coastal processes including the:
    - Tidal regime;
    - Wave climate;
    - Sediment transport regime;
    - The impact of the associated capital dredging programme.
6. Detail measures that could be implemented to stabilise the proposed dredged side slopes, if necessary.
7. Analyse the impact of propeller induced scour protection at the berths.
8. Assess the likely future dredge requirement of the proposed development.

The computational modelling used to determine baseline conditions and assess the impact of the proposed scheme was undertaken using RPS' in house suite of MIKE coastal process modelling software developed by the Danish Hydraulic Institute. This software has been described in more detail in Appendix 1.

2 EXISTING AND PROPOSED KYLEAKIN PIER

2.1 EXISTING PIER

The existing Kyleakin Pier is located just south west of the Isle of Skye Bridge that connects the Isle of Skye and Kyle of Lochalsh. The existing Pier, which is shown in Figure 2.1 below, is a solid sheet piled structure and is exposed to wind and swell waves propagating across the Inner Sound from the northerly sectors and also wind waves generated over shorter fetches from the west and easterly sectors.

Figure 2.1: The existing solid sheet piled Pier at the study site at Kyleakin[1].

As with much of coastline in this area, Kyleakin is subject to semi-diurnal tides, meaning that there are generally two high waters and two low waters each day. According to the Admiralty Chart issued by the United Kingdom, Hydrography Office (UKHO) the Mean High Water Spring (MHWS) and Mean Low Water Spring (MLWS) levels are +5.30m and +0.60m above Chart Datum at the Kyle of Lochalsh which is *c.*2.5km from the study site. The Highest Astronomical Tide at the Kyle of Lochalsh is +5.90m CD.

[1:] Image Copyright: John Allan. Image licensed under the Creative Commons Attribution Share Alike Licence

Tidal currents at Kyleakin are relatively complex due to the large volume of water that is forced in and out of Loch Alsh during each tidal cycle. The solid nature of Kyleakin Pier contributes to the complexity of the tidal regime in this area by generating notable eddying effects on either side of the Pier depending on the phase of the tidal cycle. The existing structure also creates a littoral barrier along the shoreline and interrupts the littoral drift of marine sediment along the nearshore area.

Tidal stream information published by the UKHO indicates that current speeds can approach *c.* 1.5m/s near the Isle of Skye Bridge during spring tidal conditions. Peak current speeds during typical neap tidal cycles are substantially lower and reach *c.*0.60 m/s.

As can be seen from Figure 2.2 there are numerous environmental designations in close proximity to Kyleakin Pier, these designations include:

- Special Areas of Conservation (SAC)
- Candidate Special Areas of Conservation (cSAC)
- Special Protection Areas (SPA)
- Marine Protected Areas (MPA)
- Sites of Special Scientific Interest (SSSI)
- Ramsar Wetlands of International Importance (RAMSAR)

Of particular importance to this study is the MPA at Lochs Duich, Long and Alsh which has been selected due to the internationally important numbers of flame shell molluscs. The flame shell bed covers an area of 0.93 km^2 from the shallow tide-washed waters of Kyleakin, through the mouth of Loch Alsh and out into the inner sound. Much of the MPA overlaps with the Lochs Duich, Long and Alsh SAC which has been designated for extensive areas of tide-swept reefs, extremely sheltered rock reefs and horse mussel beds.

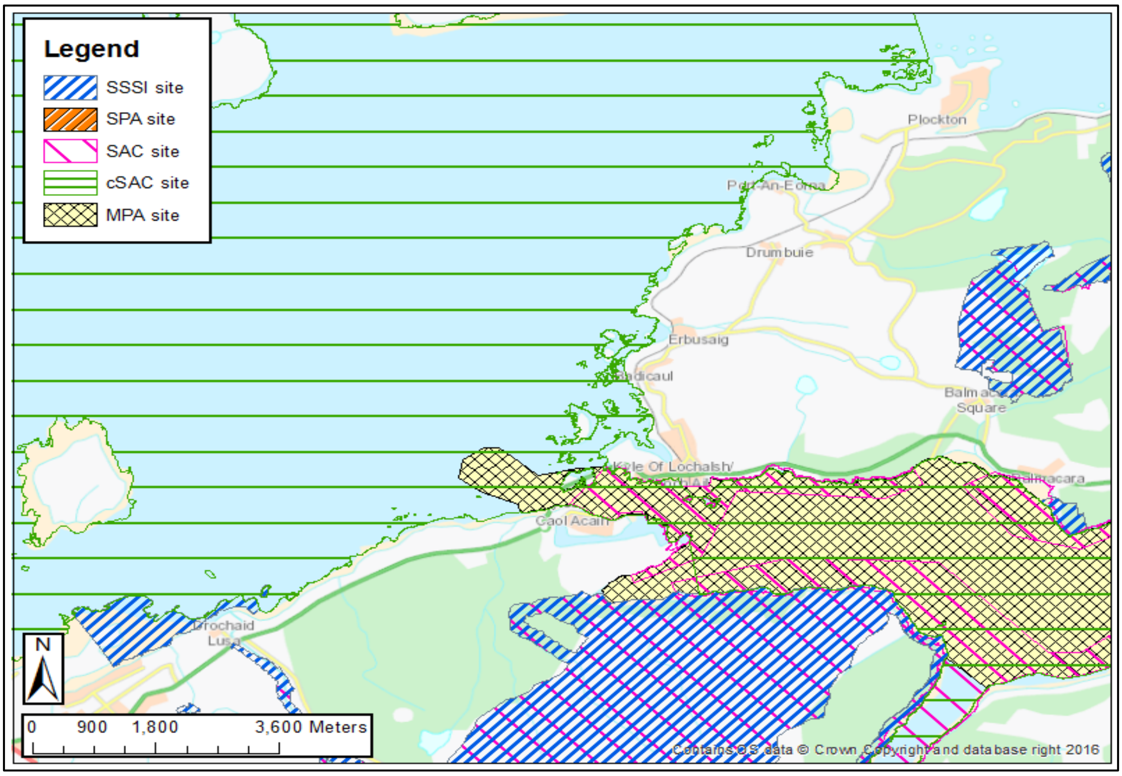


Figure 2.2: Environmental designations near Kyleakin.

2.2 PROPOSED PIER

Marine Harvest Scotland Ltd propose to develop the existing solid sheet piled Pier which will form part of a feed mill site on the Isle of Skye. The proposed development works comprise the following elements; full details of which are included on the planning drawings:

- Dredging an area of the seabed in the immediate vicinity of the proposed Pier to -8.5m CD;
- Developing the existing structure to create a 160m long quay and berthing facility
- Creating a new 79m long quay and berthing facility at the end of the existing structure using concrete caissons
- Constructing a new 48m long quay wall and a new 75.44m x 15m slipway at the coastline.
- Reclaiming three areas of land to the south of Kyleakin Pier.
- Protecting two areas of reclaimed land by constructing rock armour revetments.

The extent and configuration of the two proposed quays are outlined in Figure 2.3 whilst Figure 2.4 illustrates the area to be dredged as specified by Wallace Stone LLP. For comparison purposes the existing and proposed Pier configurations have been illustrated in Figure 2.5 overleaf.

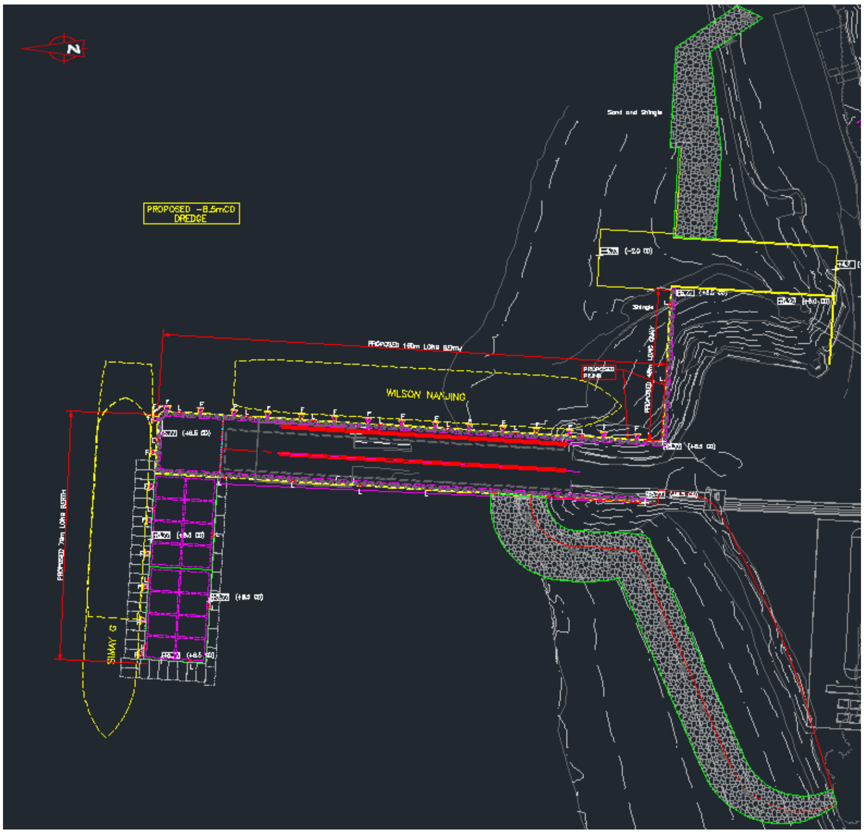


Figure 2.3: Schematic outline of the proposed development at Kyleakin Pier.

Figure 2.4: Extent of the capital dredging works required as part of the proposed development.

Figure 2.5: Existing (upper) and proposed (lower) Kyleakin Pier configurations.

2.2.1 Amendment to proposed Pier development

It should be noted that after RPS issued the first draft of this report in November 2016, Wallace Stone made several modifications to the proposed Pier development. These amendments included:

1. A minor adjustment to the orientation of the rock armour on the western side of the proposed Pier structure
2. Replacing the vertical wall on the eastern side of the proposed slipway with rock armour.
3. Introducing two 2.4m gaps between the proposed caissons to allow for future settlement.
4. Repositioning the northern boundary of the proposed dredge extent by *c.*35m to the south.

The proposed changes have been illustrated in Figure 2.6 below. As these changes were made after the first issue of this report they have not been accounted for in the numerical modelling. The effects of the amendments on the coastal processes have been expertly assessed and it is concluded that these changes are non-material for the following reasons:

- Amendment 1 and 2 relate to a minor adjustment to rock armour which is located close to the High Water Mark (HWM) where minimal changes would only persist for a short duration of time during the High Water phase of a typical tidal cycle and the circulation patterns in this area are not affected by the change.
- Amendment 3 extends the overall length of the proposed 79m quay by introducing two 2.4m gaps between the caissons will not produce a notable change in the coastal processes and is indeed within the 5m grid spacing used in the models.
- Amendment 4 reduces the overall footprint of the proposed dredging; therefore the modelling that has been presented in this report will provide a conservative assessment.

As these changes are non-material, the models that are presented in this report which have been used to quantify and assess the potential effects of the proposed scheme on the existing coastal processes are considered to be fit for purpose and suitably reflective of the proposed development.

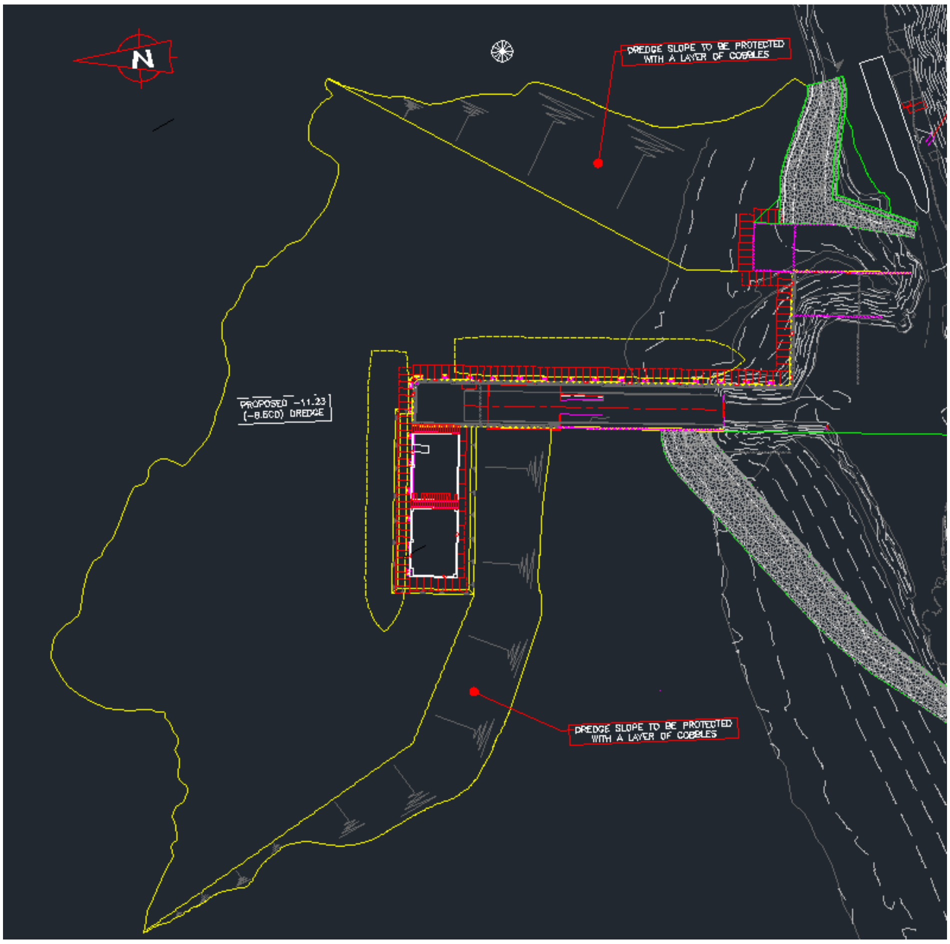


Figure 2.6: Schematic outline of the updated proposed development at Kyleakin Pier.

3 SURVEYS AND INVESTIGATIONS

In July 2016 Aspect Land & Hydrographic Surveys Ltd (herein ALHS) were contracted by Wallace Stone LLP on behalf of Marine Harvest to carry out a range of bathymetry, flow monitoring and sediment sampling surveys. The data recorded during of these field investigations were used to assist with elements of the design stage of the project and to inform the range of numerical models presented in this study. The data collected during these surveys and investigations are summarised in the following sections of this chapter.

3.1 BATHYMETRIC SURVEY AND FLOW MONITORING

An overview of the extent and resolution of the high resolution multi bean survey that was undertaken by ALHS is presented in Figure 3.1 below. The data collected during this survey was digitised into a 2m grid dataset which was then used to develop the range of numerical models that employed throughout this study. An overview of the extent and resolution of the survey data is presented in Figure 3.1.

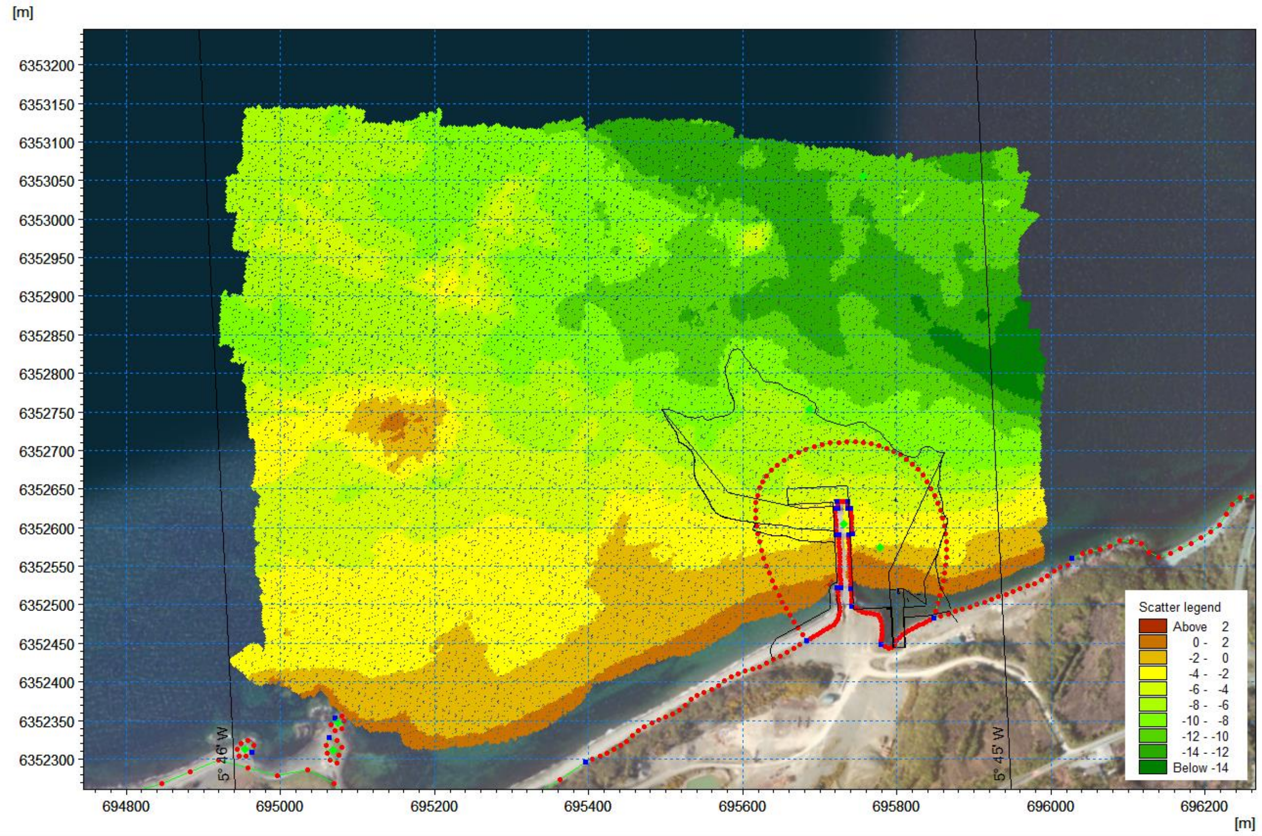


Figure 3.1: Extent and resolution of the 2016 bathymetric survey undertaken by ALHS.

Two Acoustic Doppler Current Profiler (ADCP) devices were deployed over the course of a 6 week period to record tidal current speeds and directions at two different locations. One device was deployed to the north west of the existing Pier whilst the second device was deployed to the north east of the existing Pier; both devices were deployed near the -9.0mCD contour. Both ADCP devices were set up to record information at 0.5 m intervals.

The deployment location of the two devices in relation to Kyleakin Pier is presented in Figure 3.2 below.

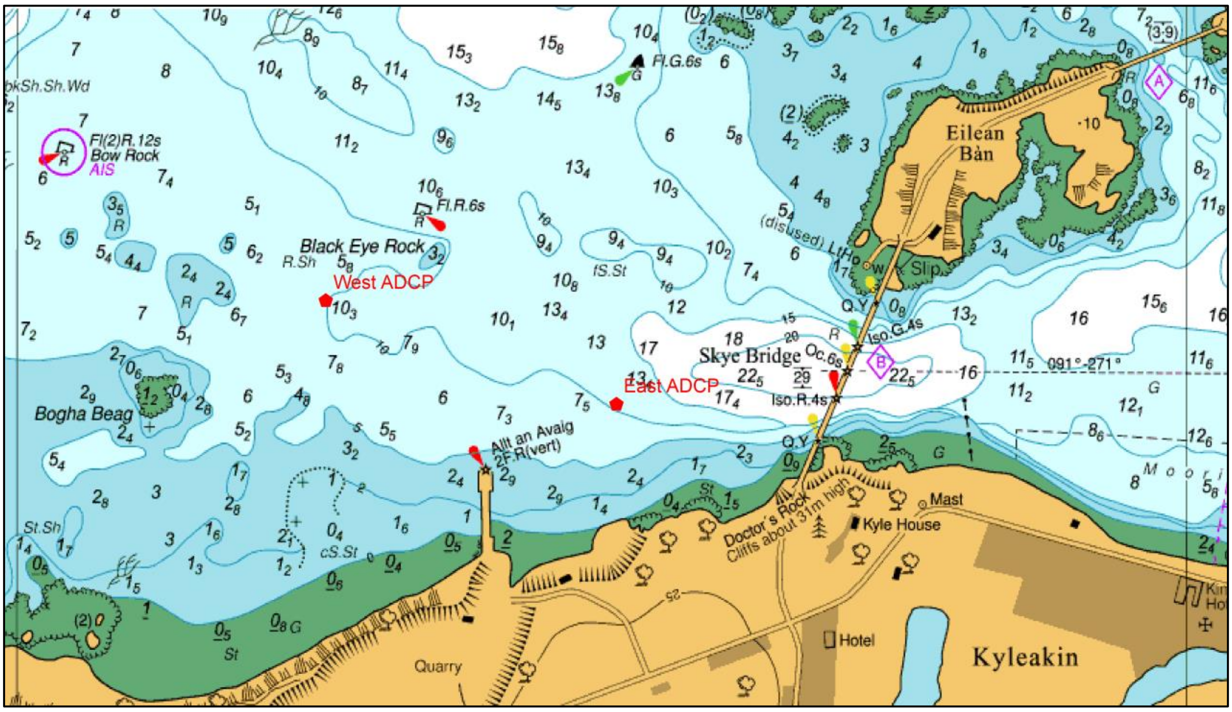


Figure 3.2: Location of the ADCP surveys in relation to Kyleakin Pier.

The data recorded by both ADCP devices across the entire month confirmed the complexity of the tidal current regime at Kyleakin. Figure 3.3 overleaf illustrates the variability of the tidal current speeds and directions across three consecutive layers of the water column (i.e. 1.5m). It will be seen that the current speeds can be as low as 0.02m/s in one layer but as high as 0.60m/s in another. Similar variability can be observed in the current direction recordings throughout the majority of the 6 week deployment period.

Weather records that covered the deployment date confirmed that the variability amongst the water column could not be attributed to particularly bad weather. Instead, the extreme variability has been attributed to both the complex nature of the tides in this region and the relatively small bin sizes to which both devices were set up to record data to (i.e. 0.50m) which could have affected the accuracy of the recorded data.

In order to transform this data into useable time-series data, RPS calculated the depth averaged current speed and directions across the bottom, mid and top layers of the water column. This to an extent, reduced the noise of both ADCP datasets and enabled the data to be used for calibration and verification purposed (as detailed in Appendix 2), however given the limitations of this dataset the recorded data should be assessed with caution.

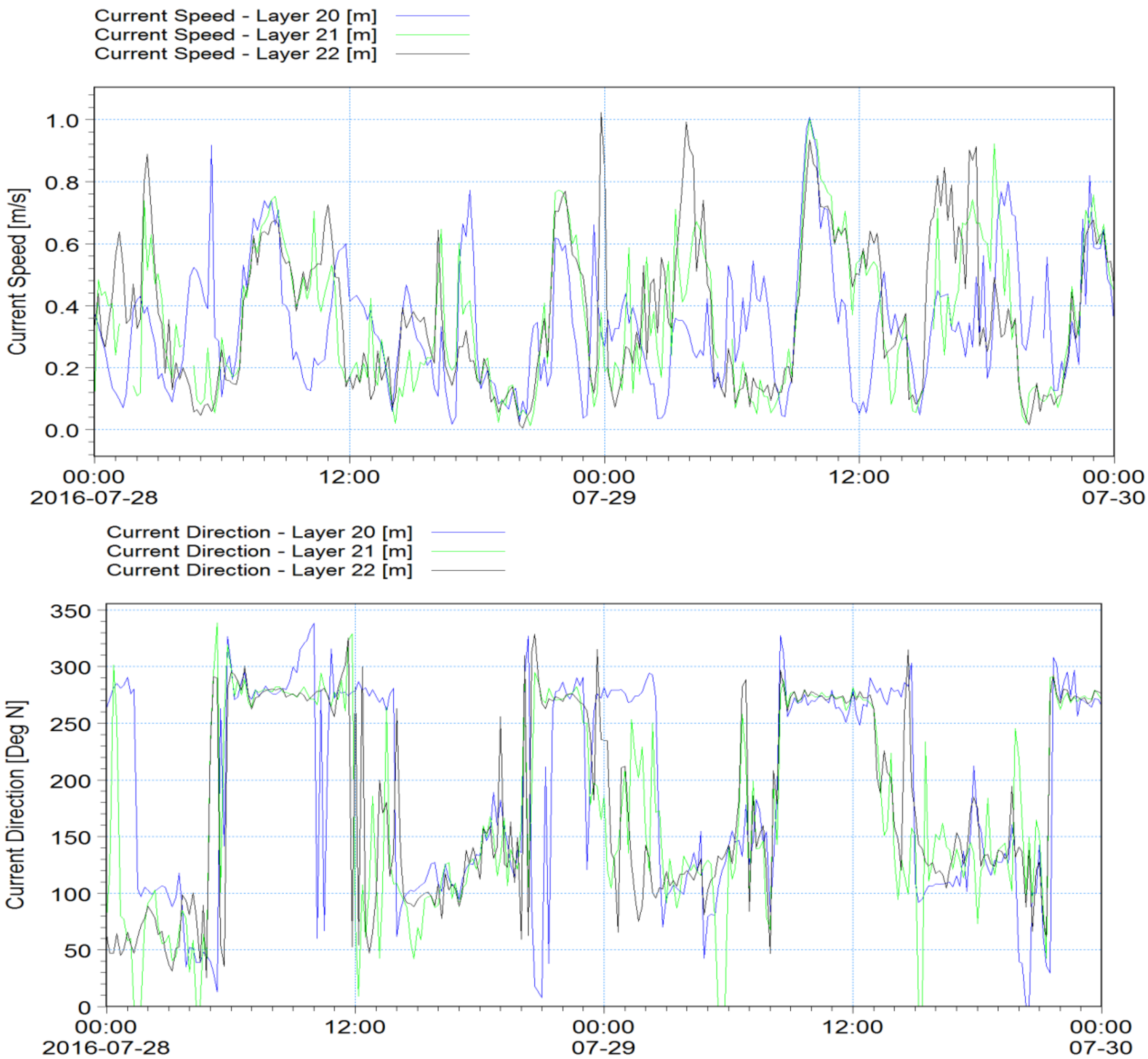


Figure 3.3: Extreme variability of current speeds and directions across three consecutive 0.5m bins – West ADCP.

3.2 SEDIMENT SAMPLING AND ANALYSIS

ALHS were also contracted to carry out an extensive sediment sampling programme using vibrocore and surface grab techniques to support the sediment transport modelling detailed in Section 7 of this report.

Whilst the sediment samples were collected in August 2016 by ALHS, the laboratory analysis of the sediments was carried out by Environmental Scientific Group (ESG) in Burton on Trent. Each sample was analysed for Particle Size, Metals and Chemicals. The location of samples taken at Kyleakin and the Dn50 particle grain size which is used as an indicator of particular size distribution is shown in Figure 3.4 below.

In brief, the sediment sampling programme found that much of the offshore area was dominated by a significant surface layer of cobbles and shingle. Closer inshore there was a wider distribution of very fine to coarse sand material with localised regions capped with gravel material.

A full description of the geotechnical survey is detailed in the corresponding report issued by ALHS (see Appendix 3).

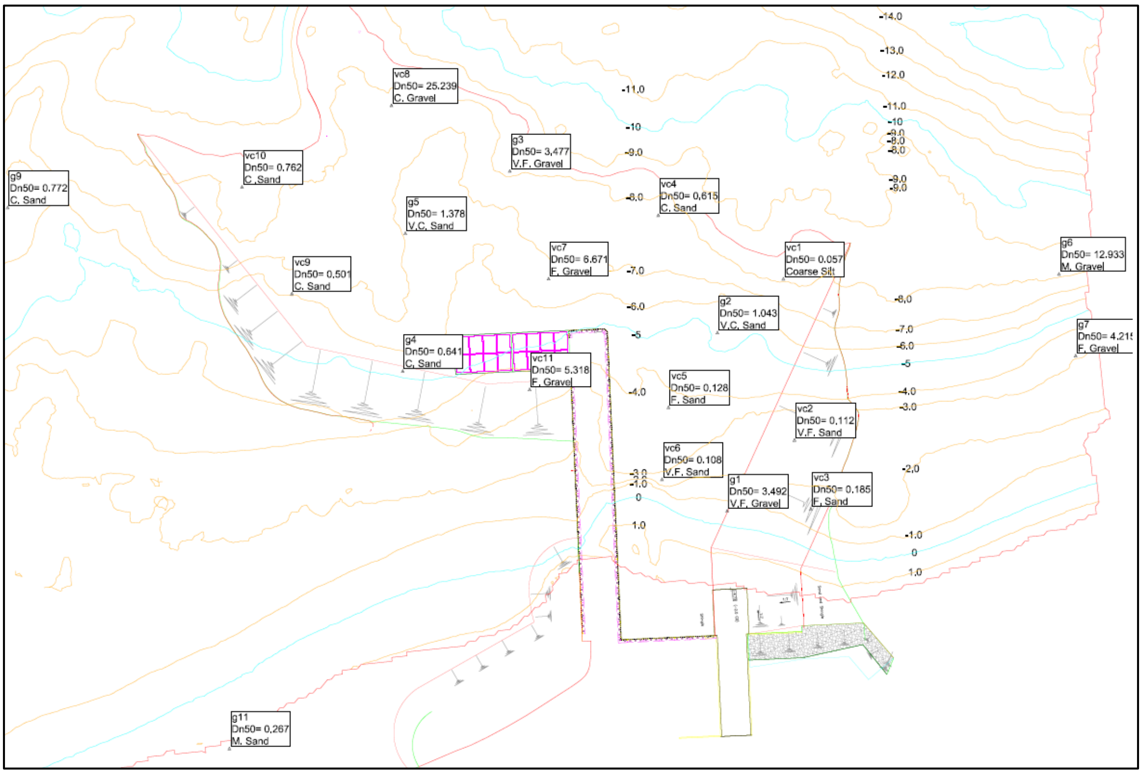


Figure 3.4: Location of sediment sampling stations at Kyleakin and sediment classification at each point.

4 METHODOLOGY FOR MODELLING COASTAL PROCESSES

4.1 OVERVIEW

Due to the numerous environmental designations within close proximity of Kyleakin advanced computational modelling was undertaken to investigate the possible impacts of the proposed Pier development on the existing coastal processes (tides, waves and sediment transport).

The RPS Coastal Processes Team has undertaken various modelling tasks in order to ascertain the potential impacts of the proposed scheme on the coastal processes at Kyleakin. The modelling tasks were divided into three main areas as follows:

- Flow regime modelling;
- Wave climate modelling
- Sediment transport modelling

The computational modelling was undertaken using RPS' in house suite of MIKE coastal process modelling software which was developed by the Danish Hydraulic Institute. Details of the modelling software used in this study are described in Appendix 1.

4.2 COASTAL PROCESS MODELS

As this study was interested in quantifying a range of coastal processes that occur at the site and adjoining areas (i.e. the flow of tidal current, the propagation of waves and the transport of sediment) it was necessary to develop and utilise three different numerical models. Each of the three models used for this study and a description of the data used to develop each model variation is described in the following sections.

The models presented in Sections 4.2.2 and 4.2.3 which were primarily used to simulate the hydrodynamic tidal regime at the study site were both developed and calibrated using several different datasets, including data recorded by the UK Hydrographic Office and also detailed hydrographic data that was specifically recorded for this study. For the purposes of brevity, the calibration process has been presented in Appendix 2 however it can be noted that the models were found to give a good representation of tidal flow patterns throughout the model domain and were considered adequate for modelling the coastal processes in the Kyleakin area.

4.2.1 West coast of Scotland model

In order to maximise computational efficiency RPS reduced the extent of an existing model Scottish waters model that was developed for Marine Harvest so that only the northern section of the west coast of Scotland model was included. For the purposes of this study, this reduced model domain is referred to as the West coast of Scotland model.

The extent and mesh structure of this model has been illustrated in Figure 4.1 below. The model was developed using flexible mesh technology and had cell sizes ranging from $50x10^6m^2$ to $50,000m^2$. The bathymetry for this model was obtained from a range of sources including bathymetric data supplied by the UK Hydrographic Office (UKHO) under the IMAGINE project and from detailed hydrographic surveys undertaken for previous projects.

This particular model was used to transform offshore waves from the North Atlantic into the Inner Sound and Isle of Raasay area whereby boundary conditions could be derived for the Outer Isle of Raasay and Kyleakin model that has been described in Section 4.2.2.

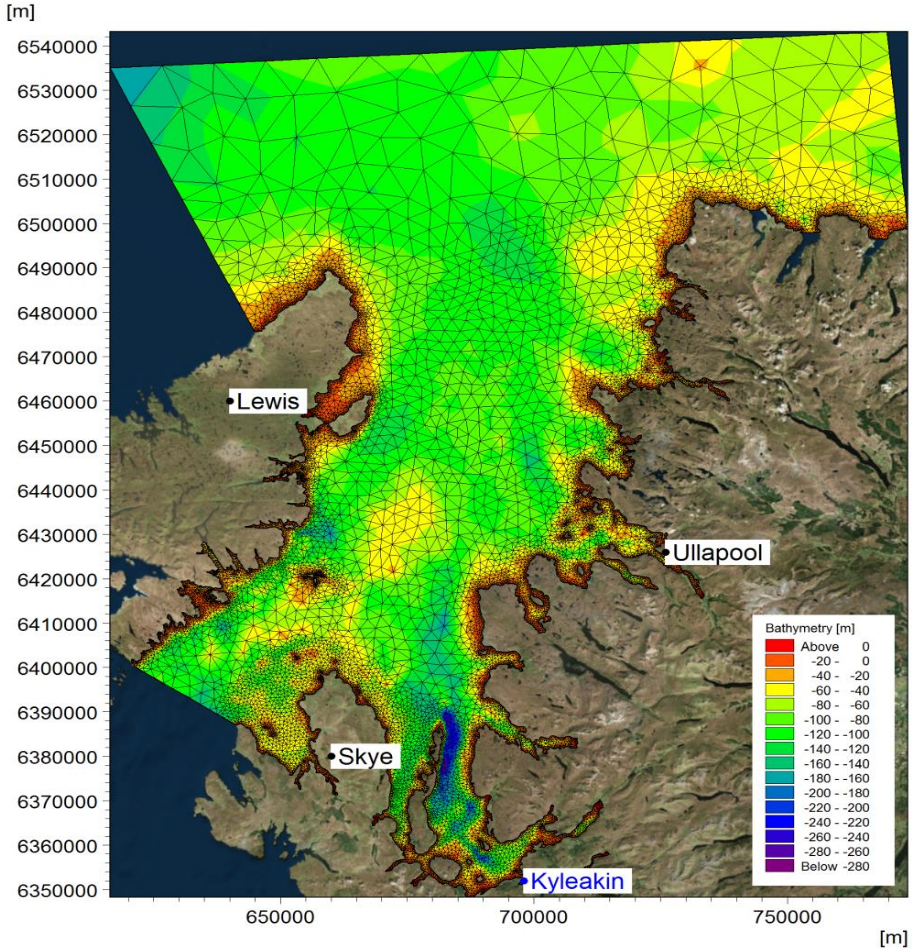


Figure 4.1: Extent and structure of the West Coast of Scotland model (to CD).

4.2.2 Outer Isle of Raasay and Kyleakin model

The Outer Isle of Raasay and Kyleakin model extends from beyond the Isle of Rona to the north and the Sound of Sleat to the south as shown in Figure 4.2. The bathymetry for this model was defined relative to MSL and used a collection of in-house bathymetric survey data including a recent survey of the Kyleakin area which was undertaken in 2016 and also high resolution data collected by the UKHO as part of the government funded INSPIRE initiative. As can be seen from Figure 4.2 the model resolution varied across the domain with coarser cells at the model boundaries and finer cells in the order of 5m in the vicinity of Kyleakin.

The boundaries of the Outer Isle of Raasay and Kyleakin model were specifically chosen so that tidal flows were accurately simulated throughout the domain. This model was primarily used to simulate tidal conditions at Kyleakin with the aim of deriving tidal boundary conditions for the model presented in Section 4.2.3.

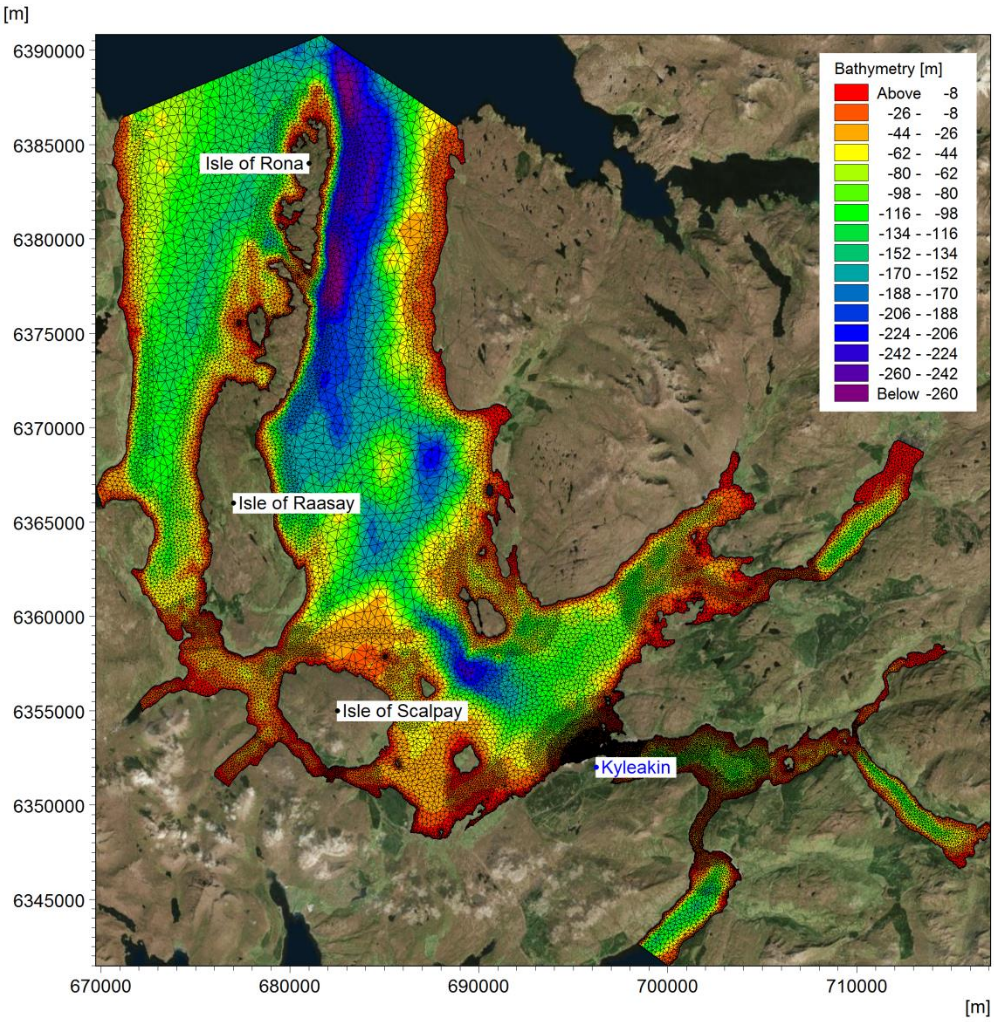


Figure 4.2: Extent and bathymetry of the Outer Isle of Raasay and Kyleakin model (to MSL).

4.2.3 Inner Isle of Raasay and Kyleakin model

To improve computational efficiency and decrease model run times a third model was developed. This model which defines the extent of the study area was based on Outer Isle of Raasay and Kyleakin model but had a reduced domain that maintained the same high resolution detail in the Kyleakin area. Again, this model was defined to MSL.

This model utilised tidal boundary conditions derived from the model detailed in Section 4.2.2 and was ultimately used to run coupled hydrodynamic, spectral wave and sediment transport simulations Two different versions of this particular model were developed to represent the existing and proposed Pier at Kyleakin.

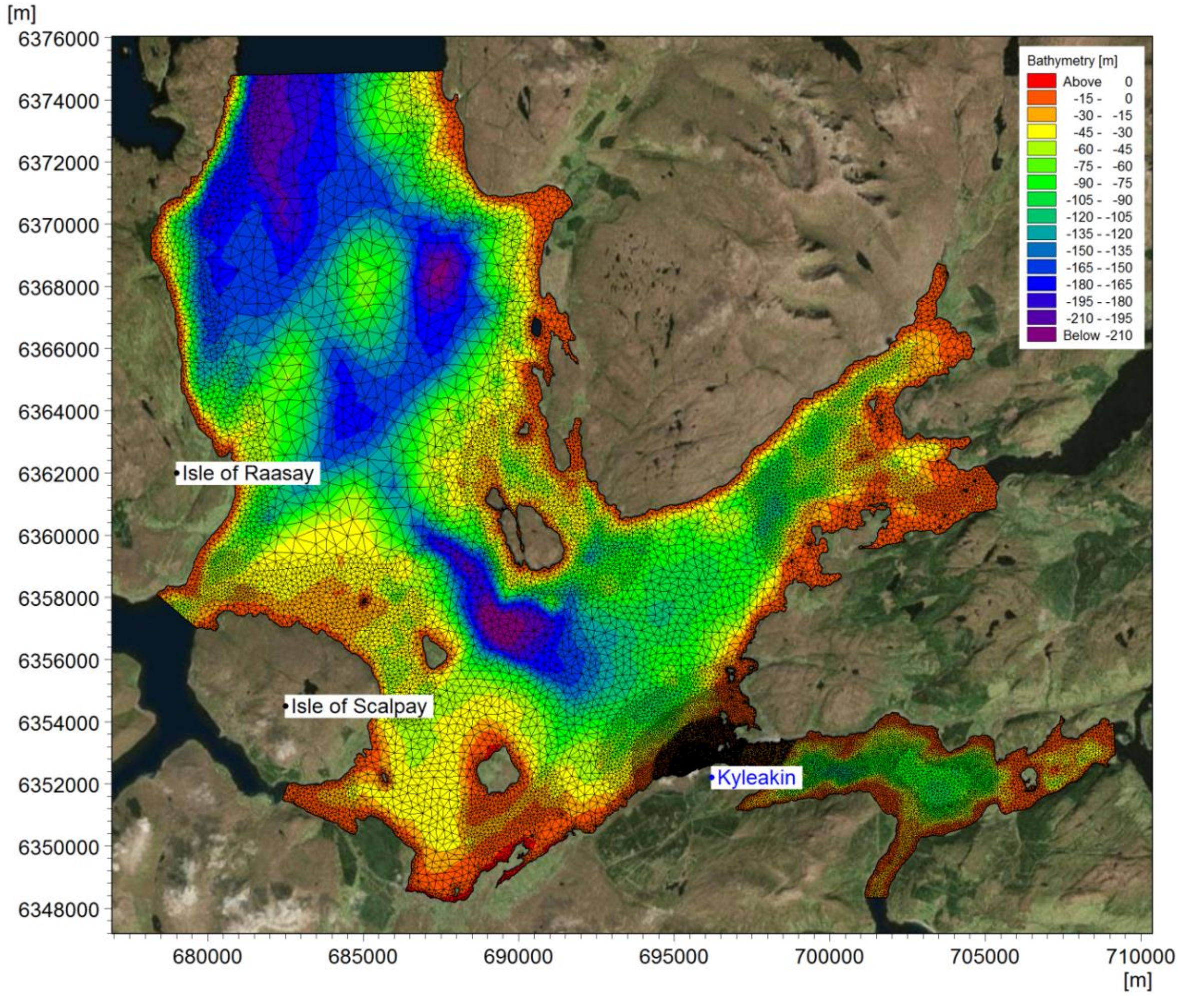


Figure 4.3: Extent and bathymetry of the Inner Isle of Raasay and Kyleakin model (to MSL).

Figure 4.4 below illustrates the bathymetry of the existing Pier configuration. By comparing this to Figure 4.5 which illustrates the bathymetry of the proposed development and the associated dredging works it can be seen that both schemes have been represented by the numerical models to a high degree of accuracy.

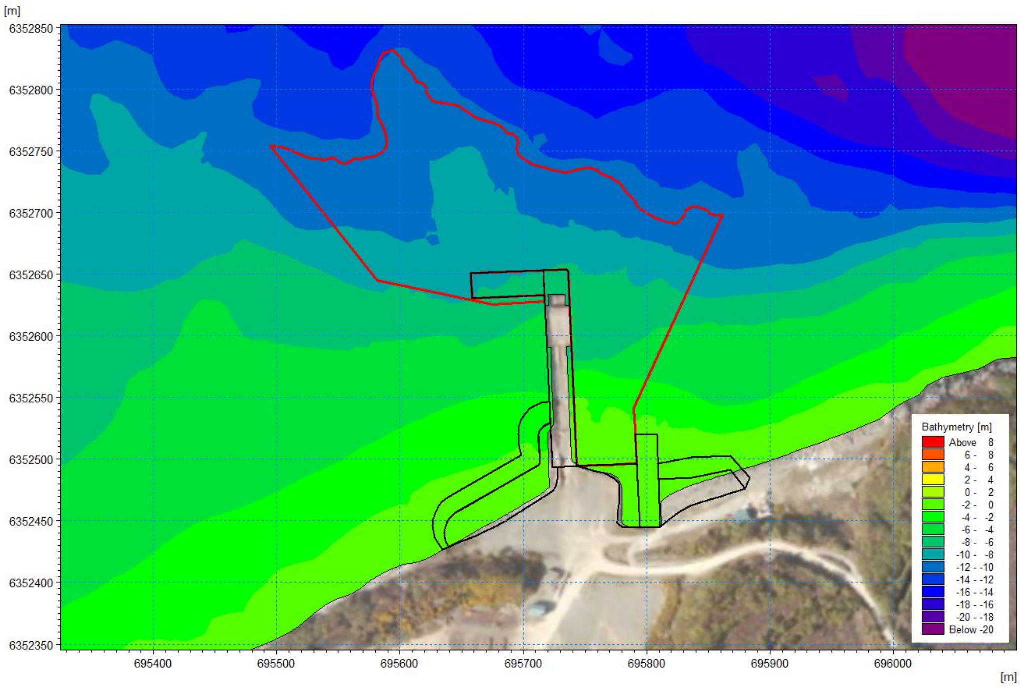


Figure 4.4: Bathymetry of the Existing Kyleakin model to MSL with the proposed development outlined in black and proposed dredge area outlined in red.

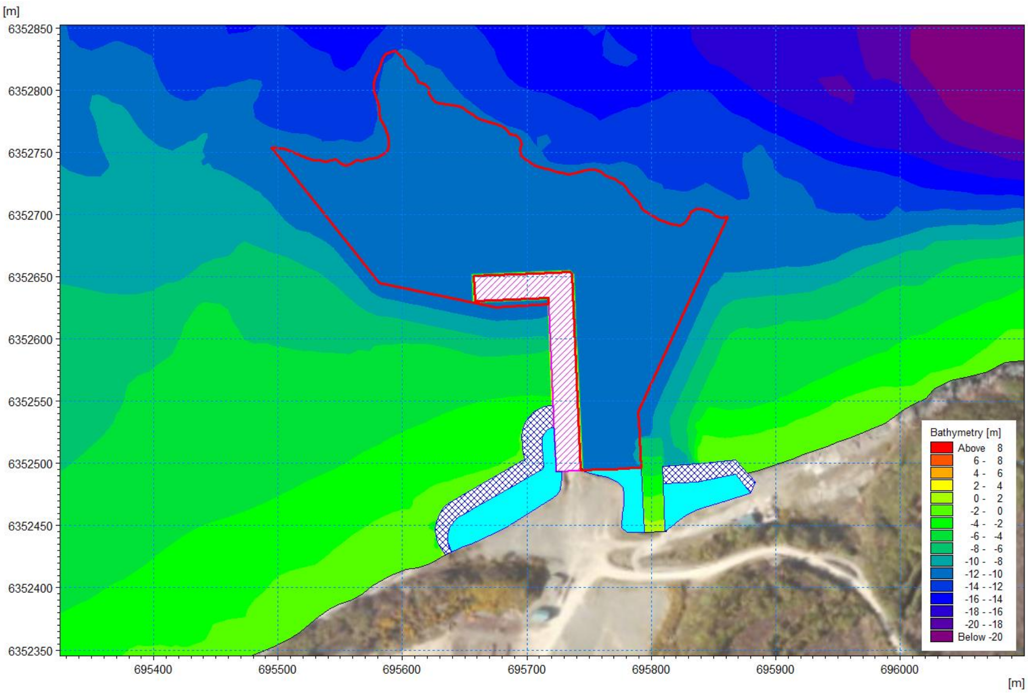


Figure 4.5: Bathymetry of the Proposed Kyleakin model to MSL with the proposed dredge area outlined in red.

4.3 FLOW REGIME MODELLING

The Outer Isle of Raasay and Kyleakin model described in Section 4.2.2 was used to simulate tidal flows within the model domain for a full 29 day lunar tidal cycle so that the hydrodynamics used for the dredge plume and dispersion model were representative of the full range of tidal states that will be experienced during construction phase. The hydrodynamic models were verified by comparison of recorded current meter readings collected specifically for this study (see Appendix 2).

The bathymetry in the finest area of the models presented in Section 4.2.2 and 4.2.3 were modified to take account of the proposed developments as can be seen in Figure 4.4 and Figure 4.5 which illustrates the existing and the proposed Pier model configurations respectively. The tidal flow models were then re-run for the same 29 day lunar month as was previously simulated for existing conditions. This enabled a comparison to be made of the flow conditions both before and after the proposed development and provided data for the modelling of sediment processes.

4.4 WAVE CLIMATE MODELLING

For storm directions with long fetches, (directions 315^o to 45^o) the modelling was undertaken using a two stage computational model simulation procedure. This involved first transforming waves from the North Atlantic into the Inner sound using the West Coast of Scotland model (Section 4.2.1) and then using the Outer Isle of Raasay and Kyleakin model (Section 4.2.2) to transform these waves from the Inner Sound into the study area.

For storm directions with shorter fetches, (directions 60^o to 300^o) the modelling was undertaken using the more detailed Inner Isle of Raasay and Kyleakin model described in Section 4.2.3.

The model simulations were run for each relevant 15^o sector at high tide levels. As storms from the south west to the west north west directions are frequently accompanied by storm surges, all the wave simulations were undertaken including the appropriate level of storm surge.

As with the flow modelling, all wave modelling was undertaken using the bathymetry for both the existing and proposed Pier configurations. Radiation stresses derived from the spectral wave analysis were also used to inform an assessment of the impact of the proposed Pier on the littoral currents regime, this assessment is detailed in Section 5.

4.5 SEDIMENT TRANSPORT MODELLING

Sediment transport model simulations were undertaken to assess the impact of the proposed Pier and capital dredging requirements on the sediment transport regime. The simulations were undertaken using the MIKE 21 coupled Hydrodynamic, Spectral and Sediment Transport modules which took account of tides, waves and sediment transport including seabed level changes. A full description of the modules used can be found in Appendix 1.

As the Pier at Kyleakin is exposed to the most arduous wave climate during storm events from the north west (see Section 6 for the wave climate analysis), sediment transport simulations were undertaken for a typical 4 day long 1 in 1 year storm event from the north west.

All sediment transport modelling was initially undertaken using the existing model domain and then repeated using the proposed model domain. This enabled a direct comparison to be made of the sediment transport both before and after the proposed development.

Specific bank stability simulations were undertaken using the same wave and current data from the sediment transport models as well as specific wave disturbance study data which was input into the 3D LitSTP program which numerically resolves the movement of sediments under wave and currents action.

4.6 BOUNDARY CONDITIONS

The tidal boundary data used for the Kyleakin model was generated by RPS' Western UK and Irish Waters Storm Surge model. This model stretches from the North-western end of France into the Atlantic to 16° west, including the Porcupine Bank and Rockall. In the other direction it stretches from the Northern part of the Bay of Biscay to just south of the Faeroes Bank. Overall, the model covers the Northern Atlantic Ocean and UK continental shelf up to a distance of 600km from the Irish Coast as illustrated in Figure 4.6.

The RPS Tidal and Storm Surge model was constructed using flexible mesh technology; along the Atlantic boundary the model features a mesh size of 13.125' (24km). The cell size of this model varied between 200m along various coastlines to 3.5km at the model boundaries.

The Irish Atlantic coast has been described using cells of on average 3km size while in the Irish Sea the maximum cell size is limited to 3.5 km decreasing to 200m along the Irish coastline. The bathymetry of this model was generated from a number of different sources including digital chart data and surveys of several banks and coastal areas. This model is driven by astronomic tides generated using a global tidal model designed by a team at the Danish National Survey and Cadastre Department (KMS).

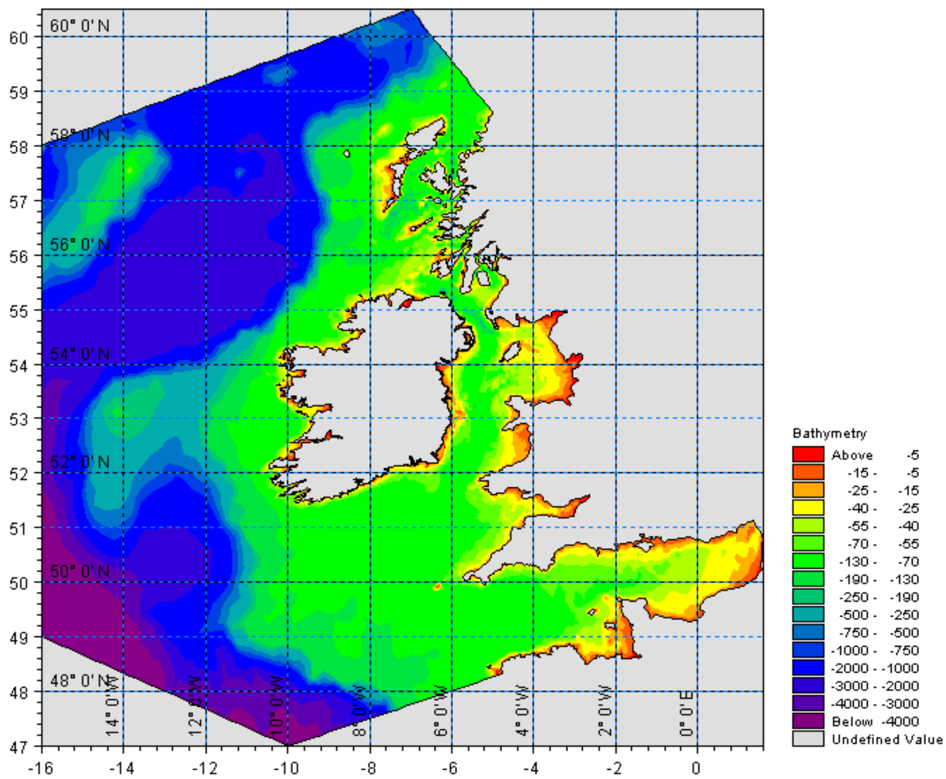


Figure 4.6: Extent of the RPS Irish Seas Tidal Surge model.

4.7 MODELLING SOFTWARE

The coastal processes at Kyleakin were simulated using the coupled MIKE 21 Flow Model FM. The MIKE 21 Flow Model FM is a state of the art modelling system based on a flexible mesh approach. The modelling system was developed by the Danish Hydraulics Institute (DHI) for applications within oceanographic, coastal and estuarine environments. The MIKE modelling software package has been approved by numerous leading institutions and authorities including the US Federal Emergency Management Agency (FEMA).

The Hydrodynamic Module is the basic computational component of the entire MIKE 21 Flow Model FM modelling system providing the hydrodynamic basis for the advection/dispersion Module, ECO Lab Module, Mud Transport Module and Sand Transport Module. For this study RPS utilised the following modules within the MIKE software package:

- Hydrodynamic module;
- Spectral Wave module;
- Particle Tracking module; and
- Mud Transport module;

A full description of these modules and the key parameters governing the coastal processes within the simulations can be found in Appendix 1.

5 TIDAL REGIME AT KYLEAKIN PIER

5.1 TIDAL REGIME UNDER EXISTING CONDITIONS

The hydrodynamic model illustrated in Figure 4.4 that represented the existing Kyleakin Pier configuration was used to simulate a month of tidal conditions across the entire model domain. Results of the numerical simulations indicated that at Kyleakin there is a distinct phase difference between peak current velocities and the surface elevation as illustrated in Figure 5.1. As a consequence of this phase difference, peak current velocities do not coincide with the mid-ebb and mid-flood tidal regime but are instead observed *c.*1.5 hours before mid-flood and mid-ebb tides.

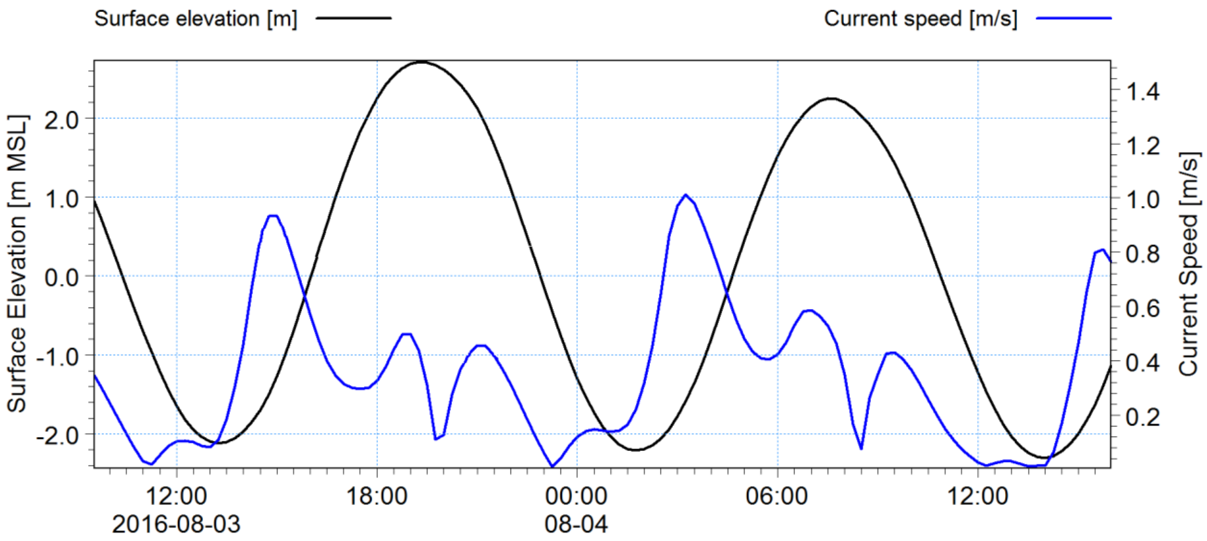


Figure 5.1: Phase difference between the surface elevation and current speeds at Kyleakin during a typical spring tidal cycle.

The current field at different phases of a spring tidal cycle is illustrated overleaf in Figure 5.2. From this figure it will be seen that the highest current velocities are observed during peak-flood tidal cycles when current velocities and approach 0.9m/s at the end of the existing Pier. On closer inspection of the results eddies can be seen to shed of either side of the existing Pier depending on the phase of the tidal cycle.

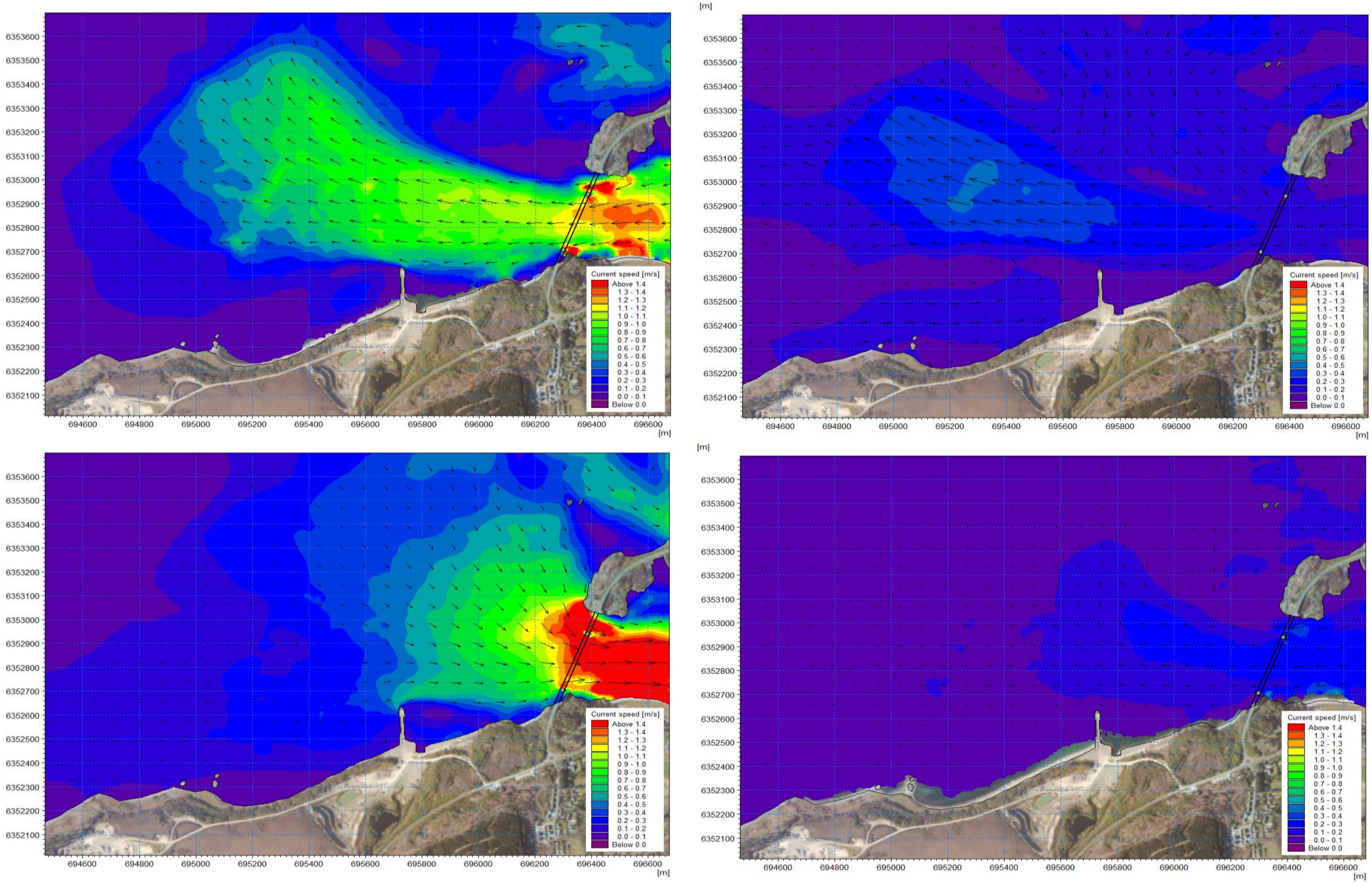


Figure 5.2: Typical peak flood (top left), high water (top right), peak ebb (bottom left) and low water (bottom right) spring tidal regimes – Existing Layout.

Residual currents were also used to assess the hydrodynamic regimes. The residual current is the average current over a full tidal phase, i.e. 12.44 hours and provides an indication of the direction of the long term sediment transport regime, the period over which the spring residual tidal currents were analysed is illustrated in Figure 5.3.

The residual tidal current regime at Kyleakin under the existing Pier layout is illustrated in Figure 5.4. It will be seen that the residual current within the immediate vicinity of the Pier is generally low at *c.* 0.1 – 0.2m/s and flows in a westerly direction. The residual current velocities can be seen to increase to as much as 0.6 - 0.7m/s as the tide becomes restricted as it passes under the Isle of Skye Bridge.

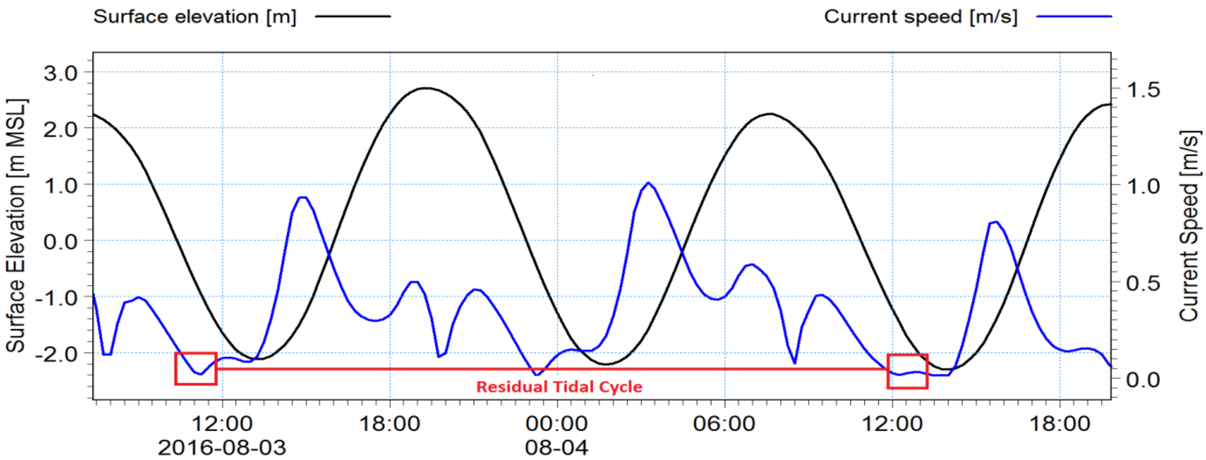


Figure 5.3: Typical 12.44hr duration used to calculate the residual current tidal regime.

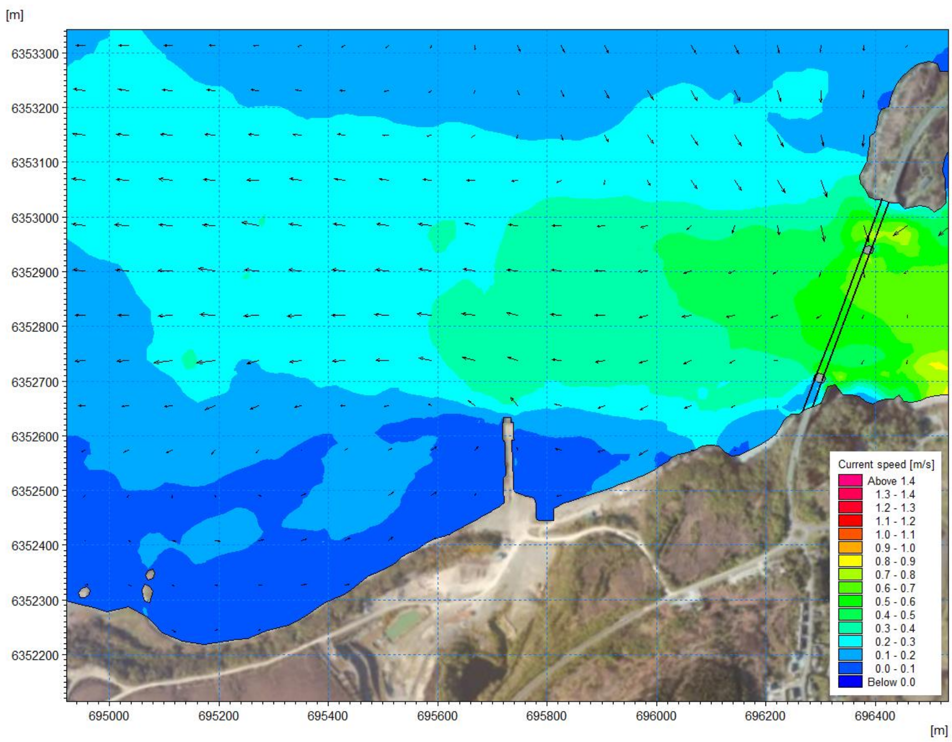


Figure 5.4: Residual spring tidal current speeds at Kyleakin – Existing Layout.

5.2 LITTORAL CURRENT REGIME UNDER EXISTING CONDITIONS

Littoral transport can be defined as the movement of material in the nearshore area and is governed primarily by the prevailing littoral current regime, i.e. the combination of tidal, wave and wind generated currents. The littoral current regime is an important coastal process that can become highly modified by any coastal engineering works, particularly during the construction or development of structures connected to the shoreline.

As littoral transport rates are greatest during storm conditions model simulations were undertaken to assess and quantify the littoral current regime at Kyleakin during a moderate north westerly storm event which could typically be expected once every year (1 in 1 year return period). These simulations used the same flexible mesh model presented in Figure 4.3. The 1 in 1 year wave boundaries were derived as part of the wave analysis presented in Section 6.

Figure 5.5 illustrates the significant wave heights and mean wave directions of the waves approaching the existing Kyleakin Pier structure during a 1 in 1 year return period storm event at the peak-flood phase of a spring tide. It will be seen that during this event waves at the end of the existing Pier have a significant wave height of between 1.3 – 1.4m.

The littoral current field at peak-flood around the existing Pier structure is illustrated in Figure 5.6. It will be seen that the littoral current regime is very similar to the tidal current regime presented in Figure 5.2, but due to the wind and wave driven currents there is a notable easterly littoral current flow in the nearshore area to the west of the existing Pier. There is also an easterly littoral current flow in the nearshore area to the east of the Pier, but the magnitude of these currents is much less due to strong tidal currents passing under the Skye Bridge.

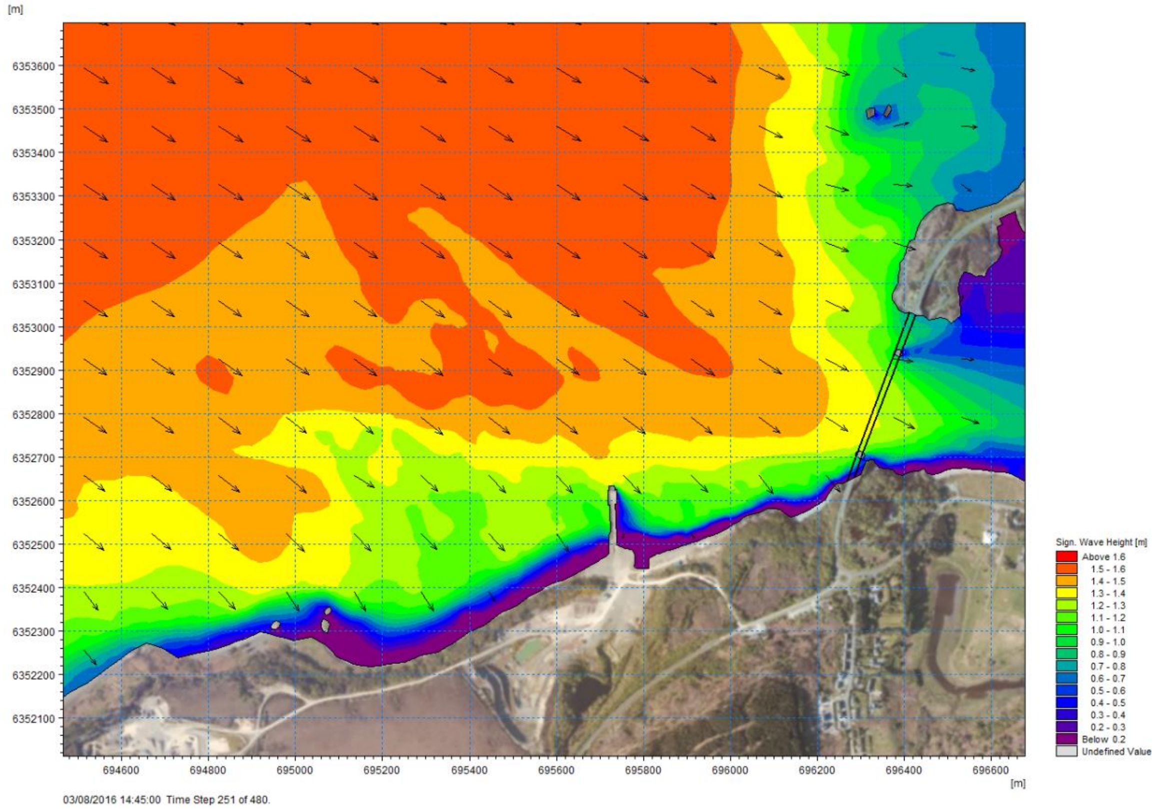


Figure 5.5: Significant wave height and mean wave direction during a typical North Westerly storm event at spring Peak-Flood – Existing Layout.

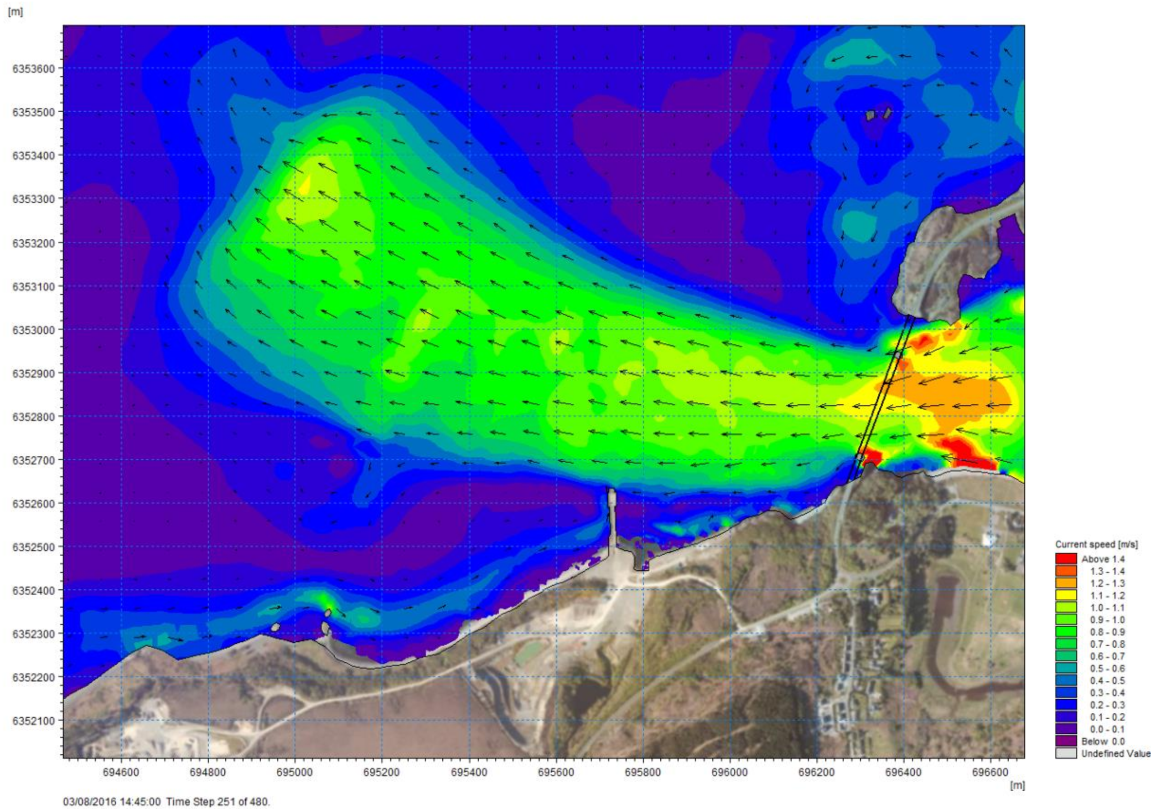


Figure 5.6: Littoral currents during a typical North Westerly storm event at spring Peak-Flood – Existing Layout.

As residual currents are important in the sediment transport regime of the area, the residual littoral flow was also determined by analysis of the U and V velocity components over a complete tidal cycle. As would be expected, the residual littoral current regime illustrated in Figure 5.7 is very similar to the residual tidal current regime illustrated in Figure 5.4. However there are significant differences between the two; It can be seen in Figure 5.7 that the prevailing wind and waves are driving a notable nearshore littoral current to the east.

On the west side of the existing structure the residual littoral current flows can be seen to reach 0.30m/s; under these conditions the resultant littoral drift would transport sediment along the shore towards the Pier. On the east side of the Pier, the wind and wave driven nearshore currents are not as strong as the residual tidal currents that flow in the opposite direction.

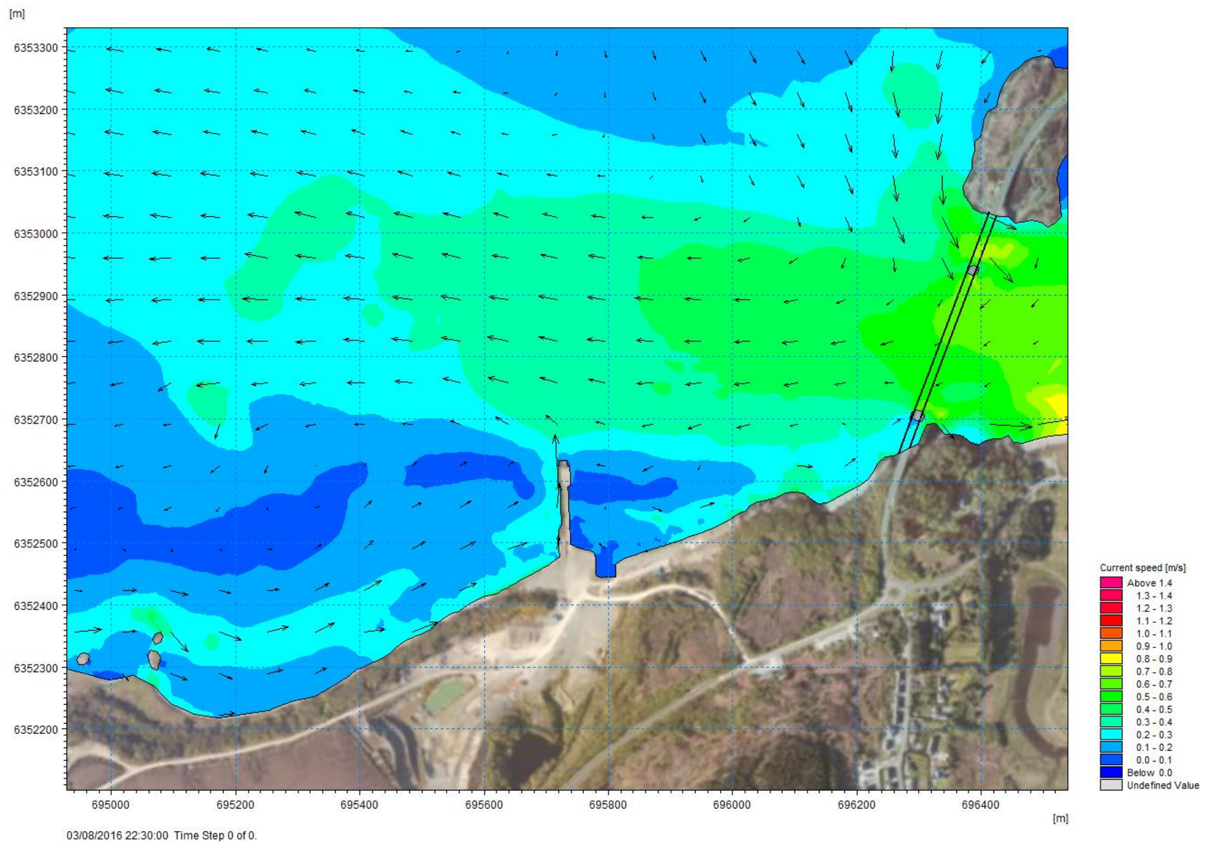


Figure 5.7: Residual Littoral currents during a typical North Westerly storm– Existing Layout.

5.3 TIDAL REGIME UNDER PROPOSED CONDITIONS

The hydrodynamic model was then re-run for the same time period as the existing scenario modelled in Section 5.1 of this report using an updated model to reflect the implementation of the proposed scheme; this updated model was previously illustrated in Figure 4.5.

The current field at different phases of a spring tidal cycle with the proposed scheme *in situ* is illustrated in Figure 5.9 overleaf. As can be seen from this figure there is still a dominant bi-directional flow at Kyleakin that flows in a west – easterly direction. Again the greatest tidal velocities in the Kyleakin area are observed during the peak-flood tidal cycle just beyond the end of the proposed Pier structure. The overall tidal flow pattern of the proposed Pier is very similar to that of the existing Pier with both configurations generating notable eddies on either side of the Pier depending on the phase of the tidal cycle.

The residual tidal current regime at Kyleakin under the proposed Pier layout is illustrated in Figure 5.8. It will be seen that the proposed Pier does have a notable effect on the residual tidal current regime, particularly on the western side of the proposed Pier where there is a decrease in velocities and a slight displacement of the eddy in this area. The actual differences between the proposed and existing residual tidal current regimes are described in further detail in Section 5.4 overleaf.

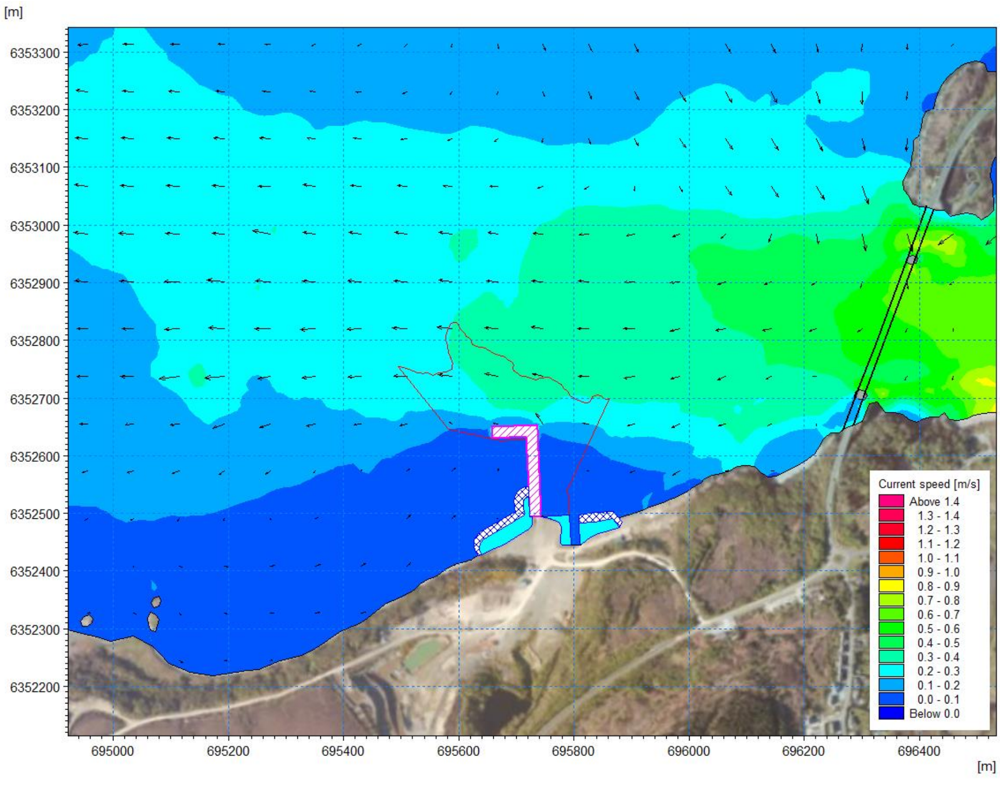


Figure 5.8: Residual spring tidal current speeds at Kyleakin – Proposed Layout.

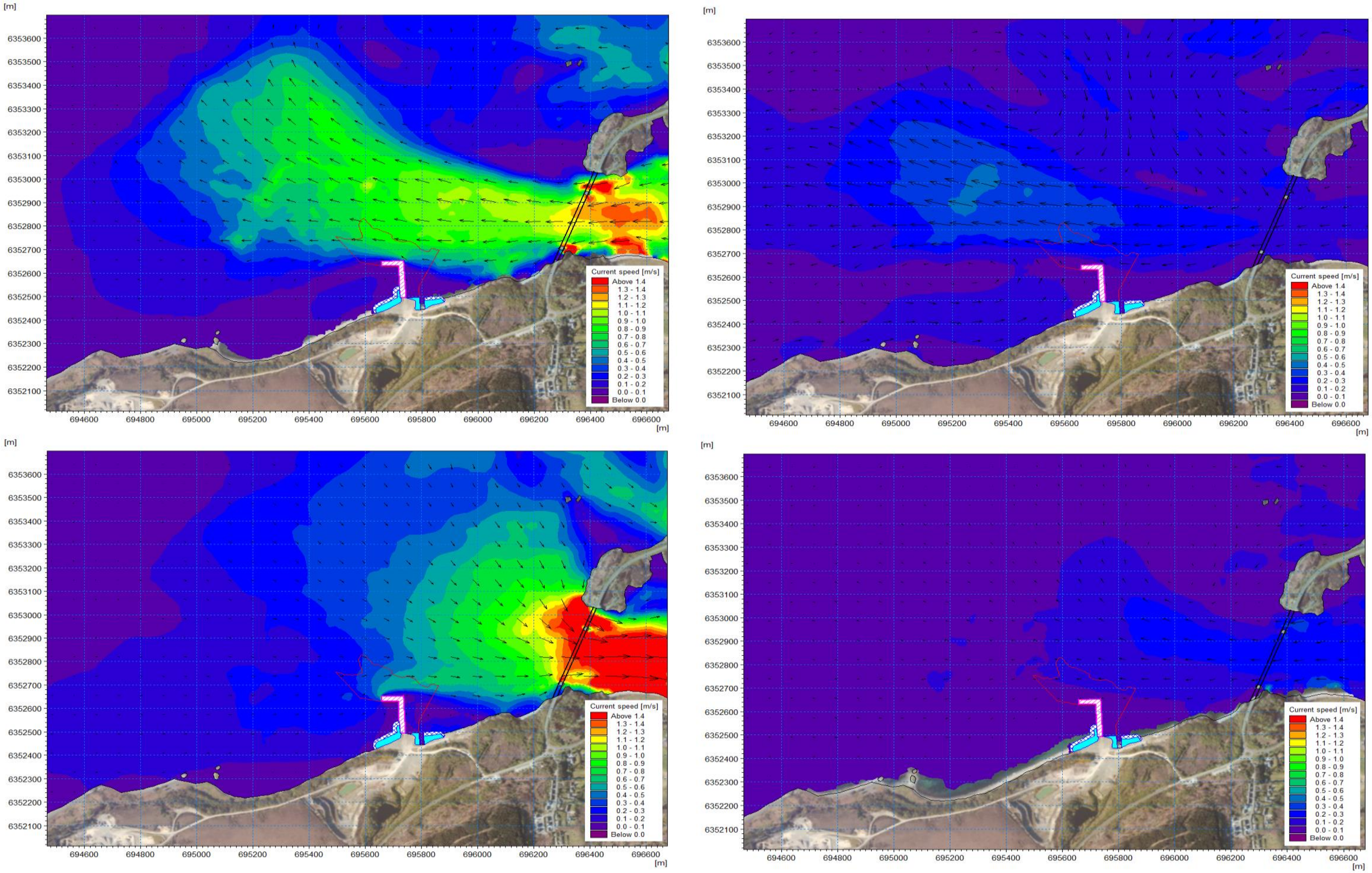


Figure 5.9: Typical peak flood (top left), high water (top right), peak ebb (bottom left) and low water (bottom right) spring tidal regimes – Proposed Layout.

5.4 LITTORAL CURRENT REGIME UNDER PROPOSED CONDITIONS

The littoral current model simulations described in Section 5.2 were re-run for the same time period and with the same 1 in 1 year storm event from the north west but with the updated model domain to reflect the implementation of the proposed scheme.

For completeness, the wave climate for the same storm event at the peak-flood phase of a spring tidal cycle but with the proposed development *in situ* has been illustrated in Figure 5.10. It will be seen that the wave climate is generally similar but that there are minor differences in significant heights in close proximity to the proposed Pier structure. The impact of the proposed development on the existing wave climate has been discussed in more detail in Section 6.

The littoral currents during a typical north westerly storm event at a spring peak-flood tidal cycle under proposed conditions are illustrated in Figure 5.11. In general it is difficult to distinguish any notable differences between the existing and proposed littoral current regimes at this phase of the tidal cycle, however there are minor differences caused primarily by the eddies shedding from the end of the proposed 79m quay structure.

The residual littoral current regime with the proposed development in situ is presented in Figure 5.12. It will be seen that even under proposed conditions, the residual littoral current still flows in a north westerly direction except in the nearshore area where the wind and wave currents drive the littoral currents in an easterly direction towards the Pier. Figure 5.12 illustrates how the proposed 79m quay forces the littoral current to turn and flow in a westerly direction as opposed to following perpendicular to the existing Pier structure as it does under existing conditions.

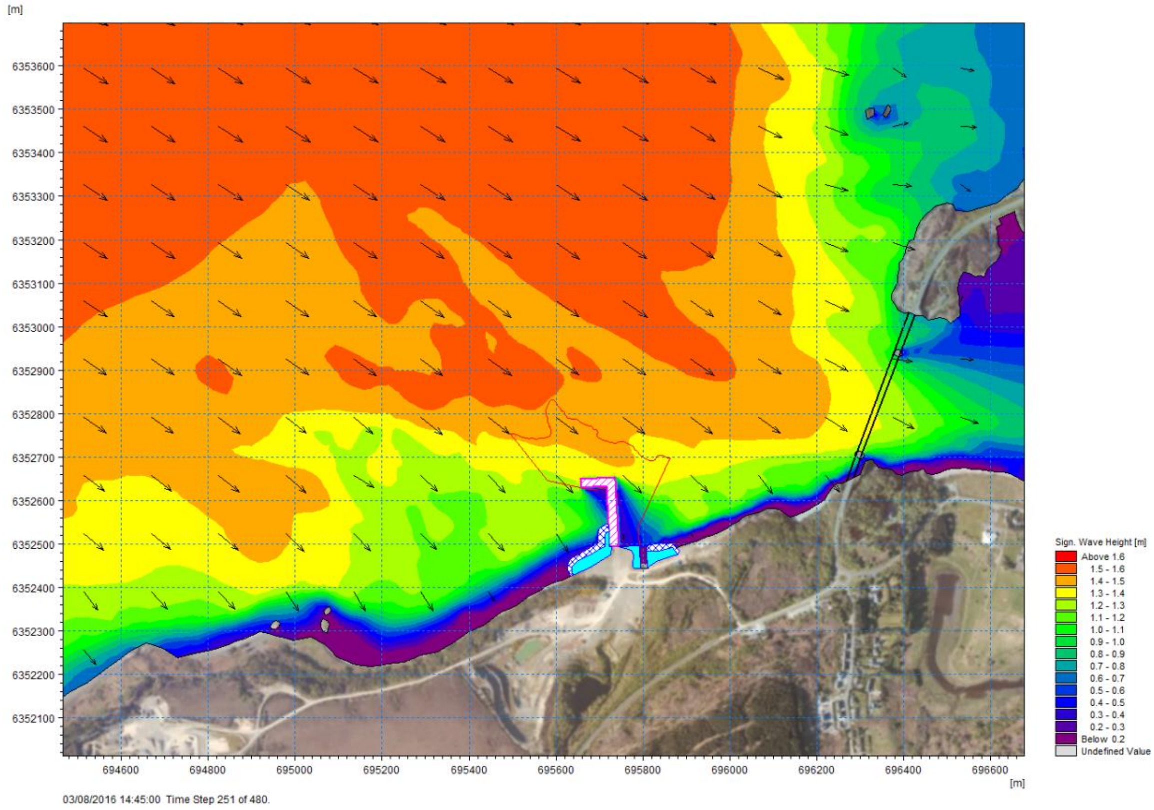


Figure 5.10: Significant wave height and mean wave direction during a typical North Westerly storm event at spring Peak-Flood – Proposed Layout.

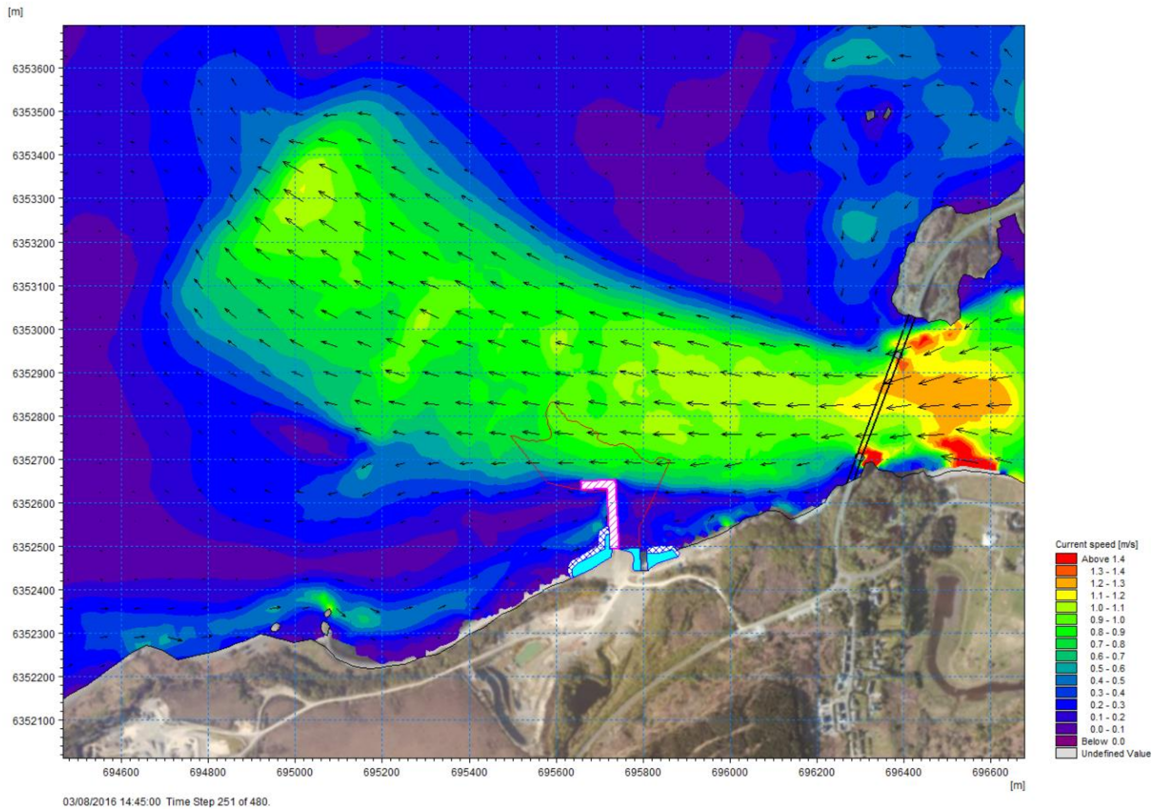


Figure 5.11: Littoral currents during a typical North Westerly storm event at spring Peak-Flood – Proposed Layout.

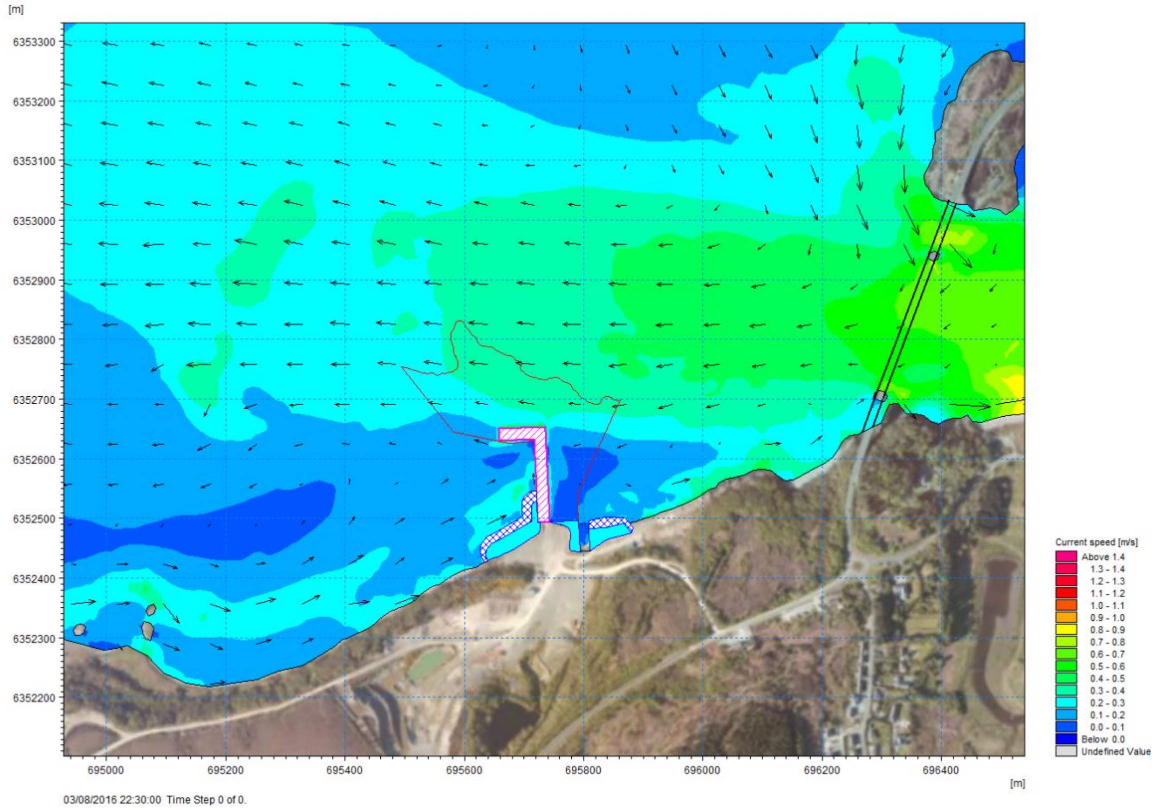


Figure 5.12: Residual Littoral currents during a typical North Westerly storm– Proposed Layout.

5.5 IMPACT OF THE PROPOSED SCHEME ON THE FLOW REGIME

Impact on Tidal Flows

Figure 5.13 illustrates the resulting differences in the flow regime between the proposed scheme and the existing scheme at various phases of a spring tidal cycle. The difference figures presented represent the current speed for the proposed development minus existing current speeds, therefore the increases in current speed will be positive and decreases negative. These scalar calculations do not take account of directional changes; however an assessment of the results indicated that there were only minimal directional changes to the existing flow regime beyond the immediate vicinity of the works as a result of the proposed development.

It will be seen from the difference plots that under typical tidal conditions the changes in current speeds as a result of the proposed development do not generally exceed ±0.10m/s within the nearshore area proximal to the development. As expected, the greatest changes in the current regime are observed at peak-flood and peak-ebb flows. During the peak flood phase, velocities are decreased by 0.06 – 0.08 m/s; this equates to a difference of *c.* 26%. However, these changes are dissipated within approximately a 200m radius of the study site

During the peak-ebb phase of the tidal cycle, aside from a similar decrease in the existing current velocities as observed during the peak-flood phase, there is an increase in current velocities of *c.*0.15m/s at the western extent of the proposed 79m quay. This phenomenon can be attributed to a small eddy shedding from the corner of the new 79 quay.

It will be seen from Figure 5.13 that the proposed scheme does result in a slightly more persistent change to the current velocities at high water. This change of 0.04-0.06m/s to the west of the existing Pier structure can be seen to dissipate within *c.*600m and is a result of a minor difference in how the eddies are shed off the Pier structure. For reference, the position of the -9.5m CD contour which reflects the approximate location of the environmentally sensitive flame shell beds has been illustrated in this figure and others throughout the report.

The difference in the proposed and existing residual tidal current regime has been illustrated in Figure 5.14 overleaf. As can be seen from this figure the greatest change in the residual spring tidal current regime does not exceed 0.06m/s. Minor changes can be observed outside of the proposed dredging area however these changes do not exceed ±0.04m/s and are not expected to result in any major changes to the tidal regime.

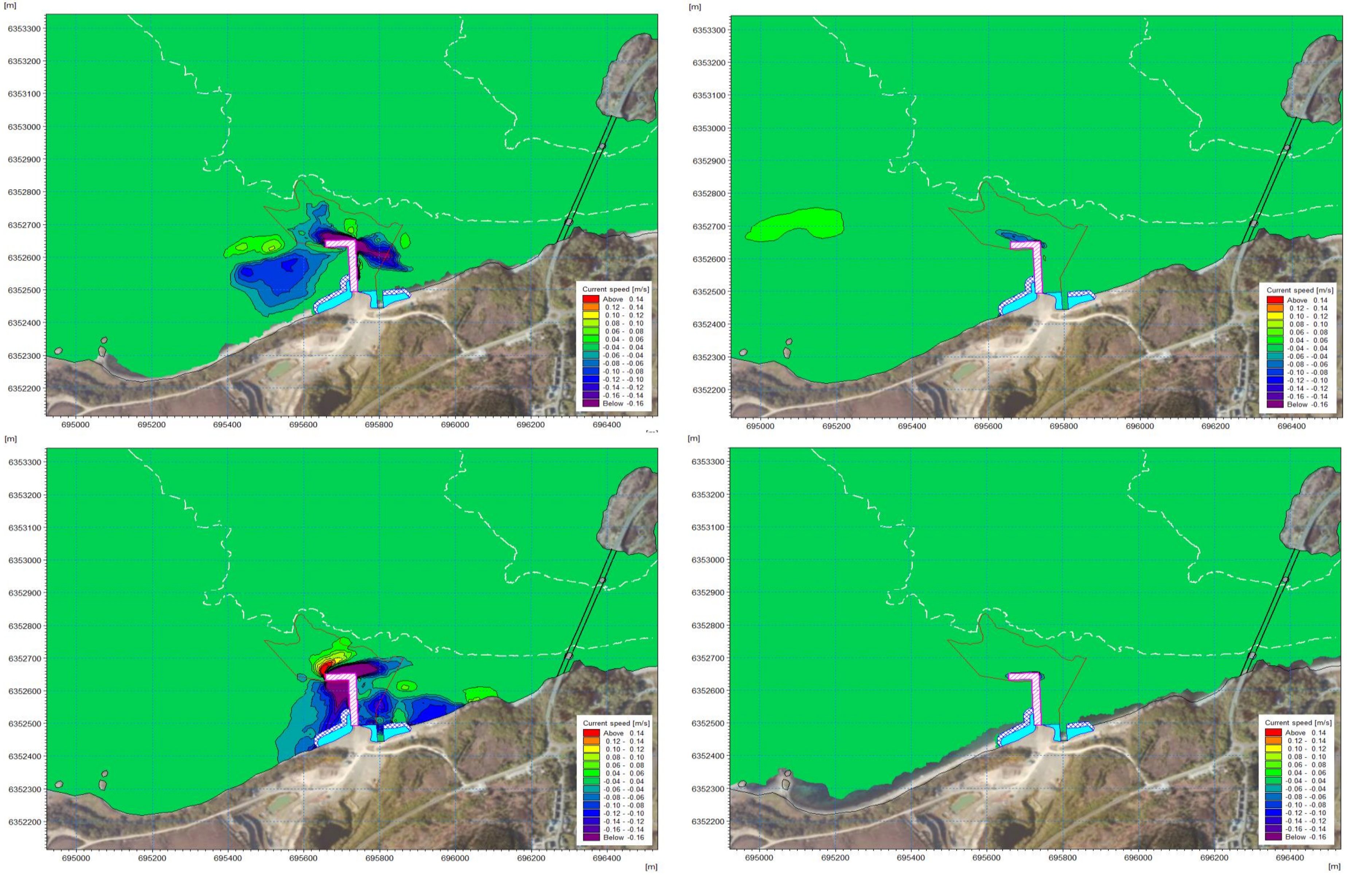


Figure 5.13: Differences in typical peak flood (top left), high water (top right), peak ebb (bottom left) and low water (bottom right) spring tidal regimes – Proposed minus existing.

Impact on Littoral Flows

It will be seen from Figure 5.15 which illustrates the difference in the proposed and existing 1 in 1 year residual littoral current regime that the proposed development appears to show some change to the west of the existing Pier structure. These changes are due to interaction of the littoral current flow that is deflected 90^{o} to the west and eddies shedding from the corner of the proposed 79m quay structure. The greatest change as a result of this combined effect does not exceed 0.082m/s. All changes to the residual littoral currents can be seen to dissipate within 1km of the proposed Pier structure and therefore do not result in any changes to the wider littoral currents.

A series of minor changes can also be identified to the east of the existing Pier structure; however these differences can be attributed almost exclusively to the change in the prevailing wave climate in this area. The increased depths in this area as a result of the proposed dredging programme means that waves begin to break much closer to the shoreline; this in turn affects the nearshore residual littoral current flows as can be seen in Figure 5.15.

5.5.1 Summary of the impact of the Proposed Pier development on the existing flow regime.

Based on the findings presented in Section 5 it can be concluded that under typical tidal conditions the proposed Kyleakin Pier configuration will result in minimal changes to the tidal regime within the immediate vicinity of the proposed works. Results from numerical simulations demonstrate that the proposed Pier could change existing tidal velocities by ±0.10m/s and that the greatest changes are observed during peak-flood and peak-ebb flows. It is important to note that under normal tidal conditions, these changes are generally limited to within the nearshore area proximal to the development.

The proposed Pier was also found to result in minor changes to the existing residual tidal and residual littoral current regimes; however changes in current velocities of less than 0.10m/s were found to persist for *c.*1.0 hour at peak ebb and flood spring tidal cycles.

Therefore, based on the results presented in this Section of study it can be concluded that the proposed Pier structure will not result in any major changes to the existing tidal or littoral current regime beyond the nearshore area proximal to the development.

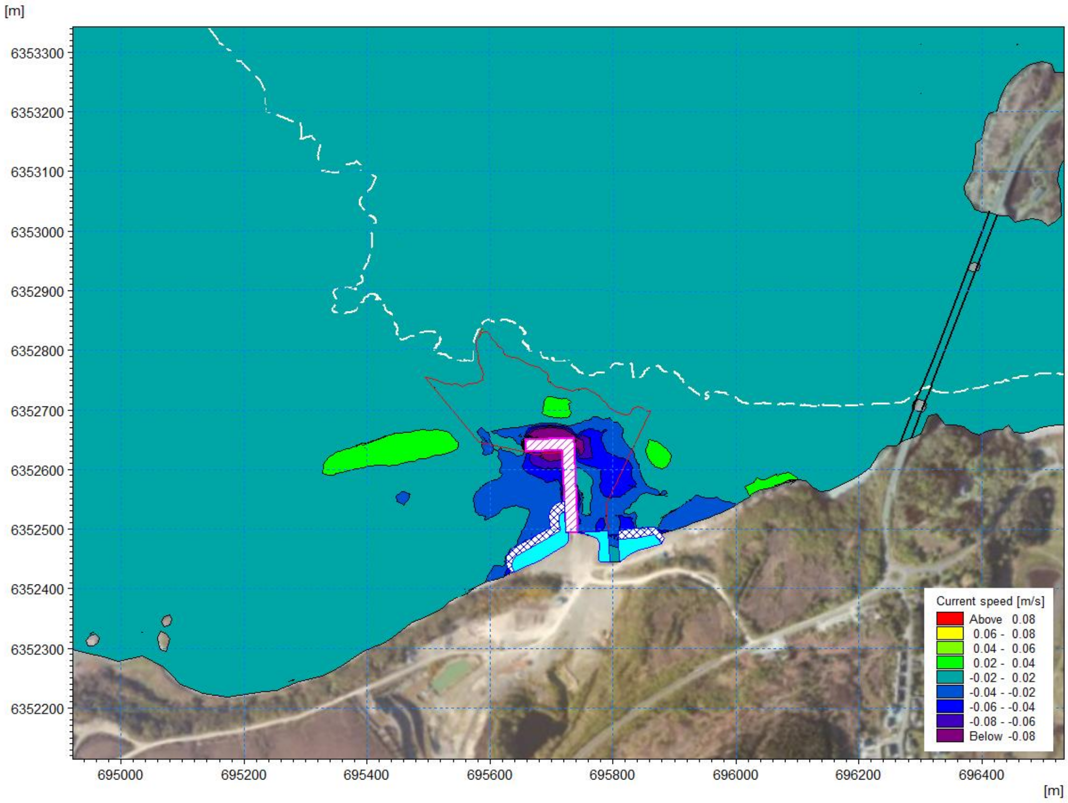


Figure 5.14: Difference in residual spring tidal current speeds at Kyleakin – Proposed minus existing.

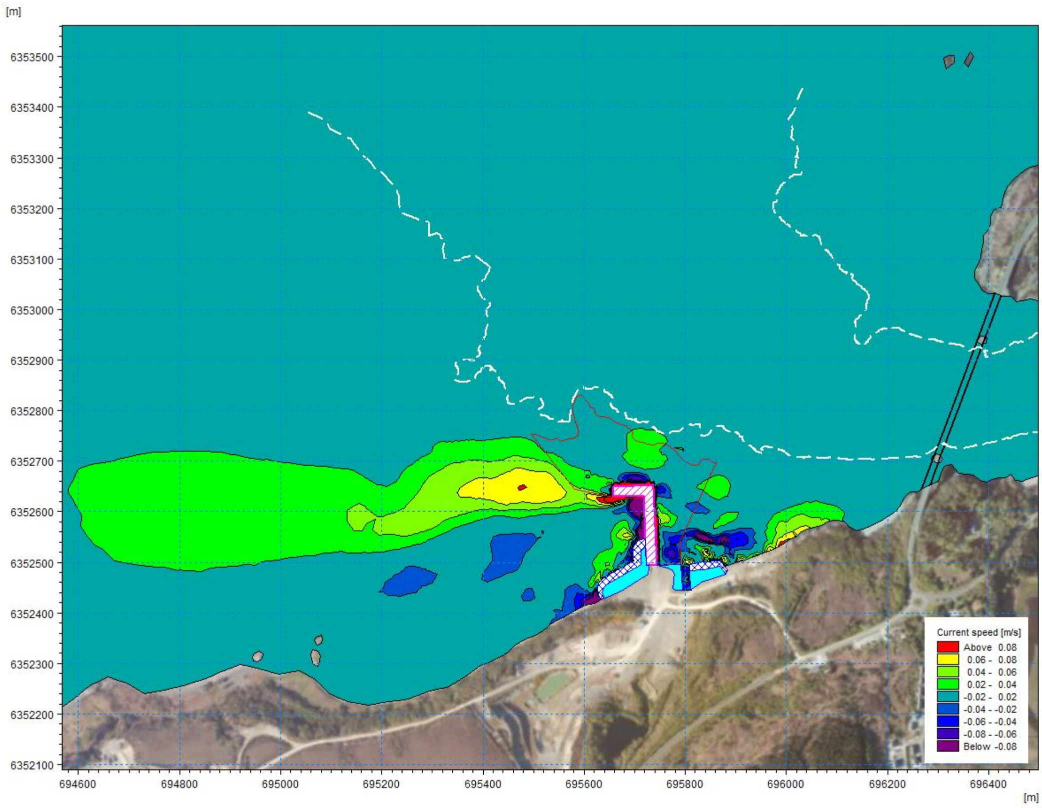


Figure 5.15: Difference in residual Littoral currents during a typical North Westerly storm event – Proposed minus existing.

6 WAVE CLIMATE AT KYLEAKIN PIER

6.1 WIND DATA

Wind data for wave generation was taken from two sources. For the average wave climate the data used was based on 25 years of wind speed data from the ECMWF atmospheric model for a point at 57.5°N, 6.0°W. Due to the coarse nature of the ECMWF atmospheric model and the location of the data point it is known that the wind speeds are under calculated and thus the wind speeds have been increased by 17%.

The wind rose generated from the adjusted ECMWF data set for the period 1991 to 2016 is shown in Figure 6.1. As expected the rose shows that the most frequent winds come from the south to west sector with the most frequent strong winds coming from the south west to north west sector.

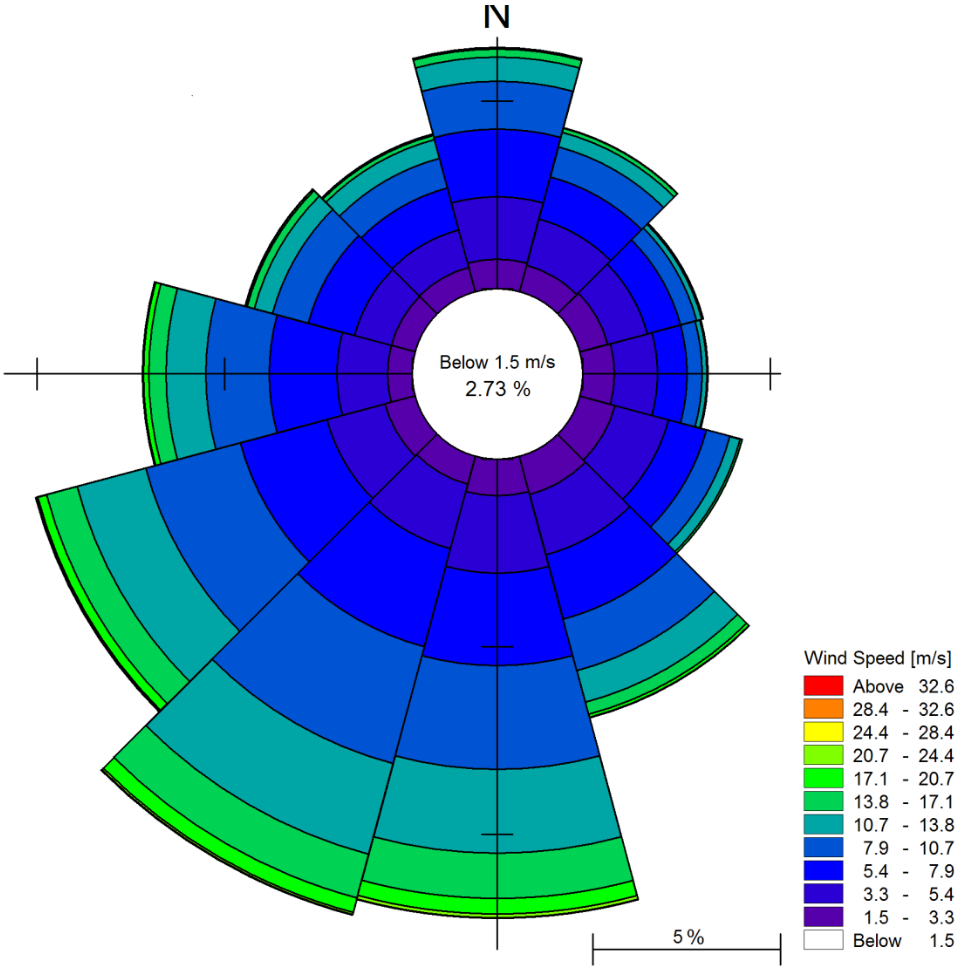


Figure 6.1: Wind rose for the period 1991 – 2016 based on ECMWF's atmospheric model.

The wind data prepared by the Met Office for BS EN 1991-1-4:2005 for extreme wind speeds throughout the British Isles has been used as the base data set for winds influencing storm wave generation over the fetches approaching the Pier site at Kyleakin with the wind speeds adjusted for overwater values where appropriate.

The wind speeds have been adjusted for the length of time required to fully develop the waves over the fetches. The values used in the storm simulations for the local fetches are given in Table 5.1.

Table 5.6.1: Wind speeds for storm simulations over local fetches relative to the proposed Pier.

Storm Direction deg N	1 in 1 year return period [m/s]	1 in 50 year return period [m/s]	1 in 100 year return period [m/s]	1 in 200 year return period [m/s]	1 in 500 year return period [m/s]
15	19.8	28	29.54	31.38	33.08
30	18.8	26.65	28.18	30.03	31.65
45	17.97	25.34	26.65	28.21	29.73
60	17.31	24.41	25.65	27.14	28.61
75	17.31	24.41	25.65	27.14	28.61
90	19.02	26.91	28.38	30.14	31.77
105	19.02	26.91	28.38	30.14	31.77
120	19.14	27.08	28.57	30.36	32
225	24.6	34.81	36.74	39.06	41.17
240	24.6	34.81	36.74	39.06	41.17
255	24.6	34.81	36.74	39.06	41.17
270	24.6	34.81	36.74	39.06	41.17
285	24.6	34.81	36.74	39.06	41.17
300	24.02	34	35.83	38.02	40.07
315	22.37	31.6	33.28	35.29	37.2
330	20.67	29.25	30.88	32.84	34.61
345	21.02	29.73	31.36	33.32	35.12
360	20.62	29.18	30.81	32.75	34.52

6.2 OFFSHORE WAVE DATA

North Atlantic storm waves can enter the Minch from the northerly sector. Wave data for these storms was taken from statistical analysis of 17 years of 3 hourly data derived from the ECMWF European waters wave model. The point chosen for the analysis was at 5.5^{o} W, 59^{o}N.

The 1 in 500 year storm from the north north west to north sector was found to have a significant wave height[1] of 14.5 metres with a mean wave period of 15.2 seconds. The 1 in 500 year storm for the north north east sector had significant wave heights of 9.61 metres with a mean wave period of 12.25 seconds.

The equivalent 1 in 1 year return period storms were found to produce waves with a significant wave height of 6.81 metres and a mean wave period of 11.10 seconds for the north north west to north sector. For a 1 in 1 year event from the north north east sector waves were found to have a significant wave height of 4.14 metres with a corresponding mean wave period of 8.62 seconds.

[1]In physical oceanography, the significant wave height (Hs) is defined as the average height of the highest one-third of waves in a wave spectrum.

6.3 AVERAGE WAVE CLIMATE MODELLING PROCEDURE

The average wave climate approaching the Pier at Kyleakin was simulated for each 30° sector using the Mike21 SW wave model. The simulations were undertaken for wind speeds equivalent from Beaufort Force 2 through to Beaufort Force 11.

The simulations were undertaken at high tide, however it should be noted that due to the water depth around the proposed Pier the wave climate will be similar at other tidal levels. The average wave rose for a point just north-west of the Pier based on the 25 years of wind records is shown in Figure 6.2. It will be seen from this figure that whilst the most frequent waves come from the westerly sector, the largest waves approach the Pier site from 300° and 330° sectors.

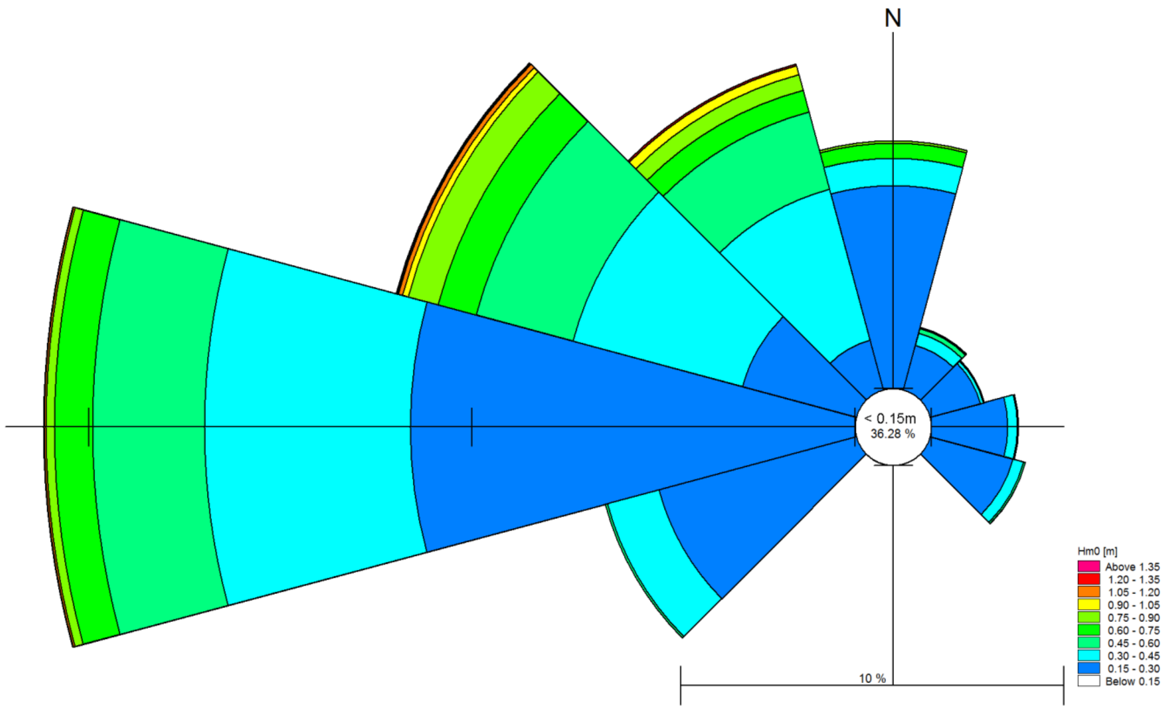


Figure 6.2: Average wave climate rose at a point just NW of Pier site.

6.4 STORM WAVE MODELLING PROCEDURE

Aside from analysing the average wave climate at the study site, a series of spectral wave simulations were undertaken to establish the wave climate at the Pier during a series of specific return period events.

Previous wave climate studies of this area have demonstrated that the waves generated in the Atlantic which propagate over long fetches into the inner sound from 315^o to 45^o are highly modified by the relatively narrow section of the Inner Sound to the extent that the wave climate at Kyleakin Pier is dominated by wind waves generated over short fetches and thus long period swell does not reach the Pier site at Kyleakin

The SW wave model was used to simulate the transformation of waves over each relevant 15^o sector at high tide levels. As storms from the south west to the west north west directions are frequently accompanied by storm surges, all the wave simulations were undertaken including the appropriate level of storm surge. Thus the storm simulations were run with water levels for each return period as shown in Table 6.2 below.

Table 6.2: Tidal levels used in storm wave simulations.

Storm Direction [Deg N]	1 in 1 year Water Level [m CD]	1 in 50 year Water Level [m CD]	1 in 100 year Water Level [m CD]	1 in 500 year Water Level [m CD]
15 to 120	5.30	5.30	5.30	5.30
225	5.80	6.10	6.30	6.50
240	5.80	6.10	6.30	6.50
255	5.80	6.10	6.30	6.50
270	5.80	6.10	6.30	6.50
285	5.65	5.90	6.00	6.20
300	5.50	5.70	5.80	5.90
315	5.35	5.40	5.50	5.60
330	5.30	5.30	5.30	5.30
345	5.30	5.30	5.30	5.30
360	5.30	5.30	5.30	5.30

The largest waves will approach the proposed Pier from the north west direction during a storm from 300°N. A plot showing how these extreme waves propagate towards the Pier is shown in Figure 6.3 below. It will be seen that waves with significant heights in excess of approximately 3.0m can approach the site during extreme a 1 in 100 year return period event.

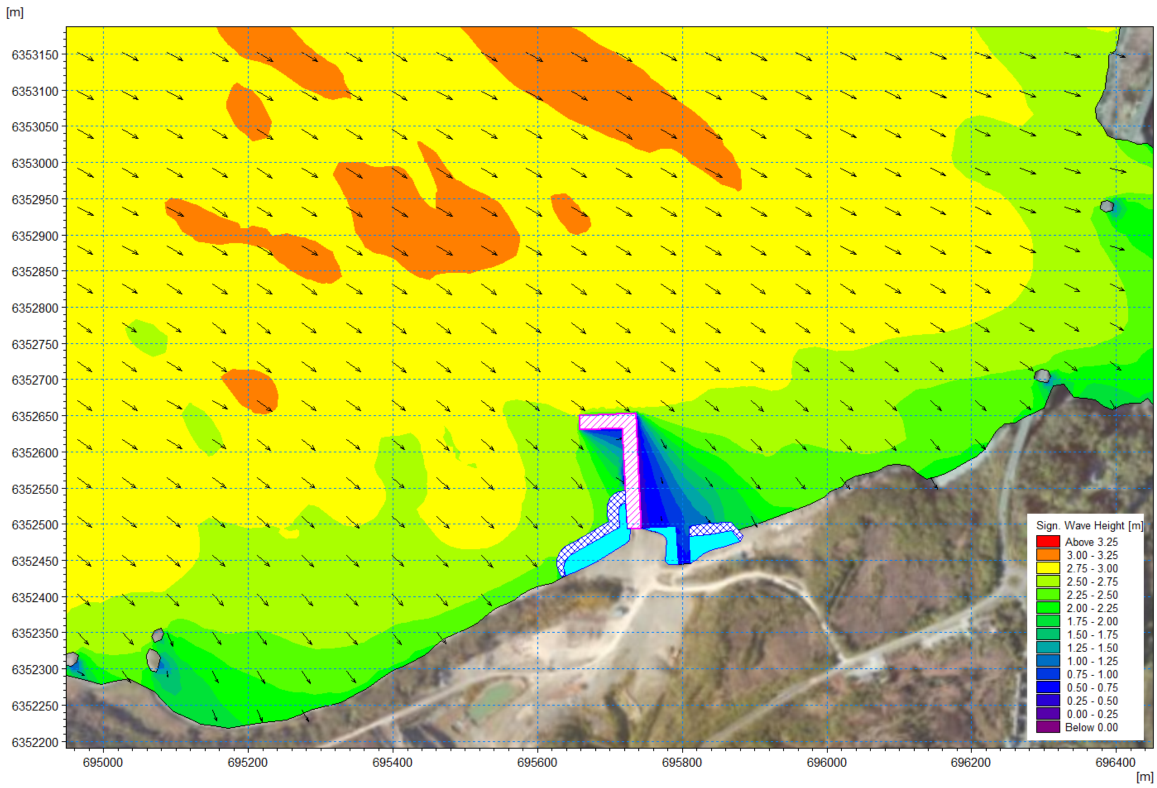


Figure 6.3: Significant wave height and mean wave direction1 in 100 year return period storm from 300°.

It should be noted that the SW wave model does not include wave to wave interactions which results from wave reflections from the Pier structure which will affect the wave climate along the berthing face of the Pier itself. Thus simulations were undertaken using the Boussinesq Wave (BW) model to evaluate the conditions at the berths.

The output from of a BW simulation representative of a 1 in 100 year north westerly storm event is shown in Figure 6.4. The input to the BW model simulation was taken from the results of the SW model simulations.

It will be seen from Figure 6.4 that the wave reflections from the structure will increase the storm wave heights locally along the berthing faces of the Pier.

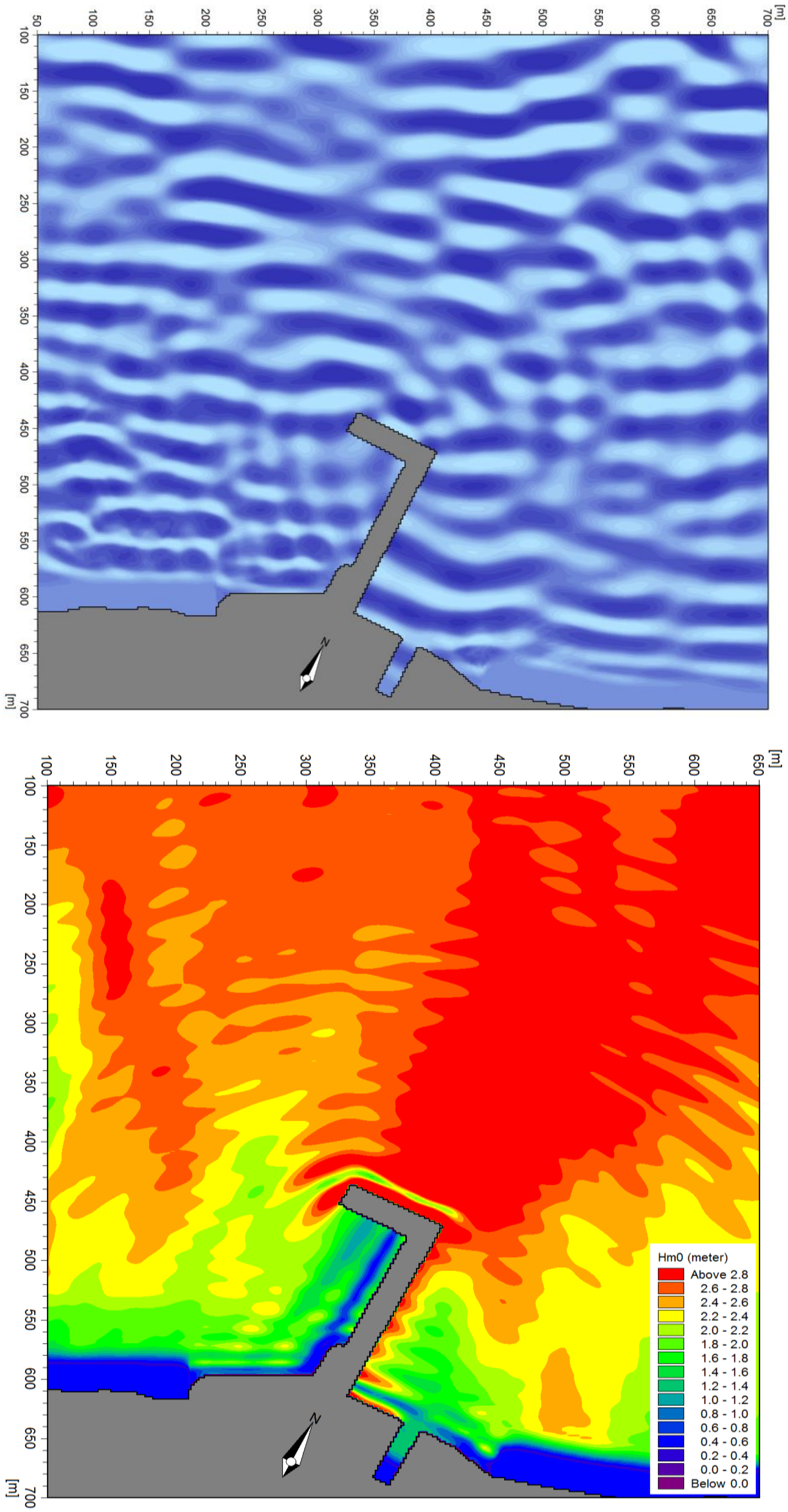


Figure 6.4: Storm wave patterns (upper) and significant wave heights around the proposed Pier during a 1 in 100 year storm event from 300° N at MSL (lower).

6.5 IMPACT OF THE PROPOSED SCHEME ON THE WAVE CLIMATE

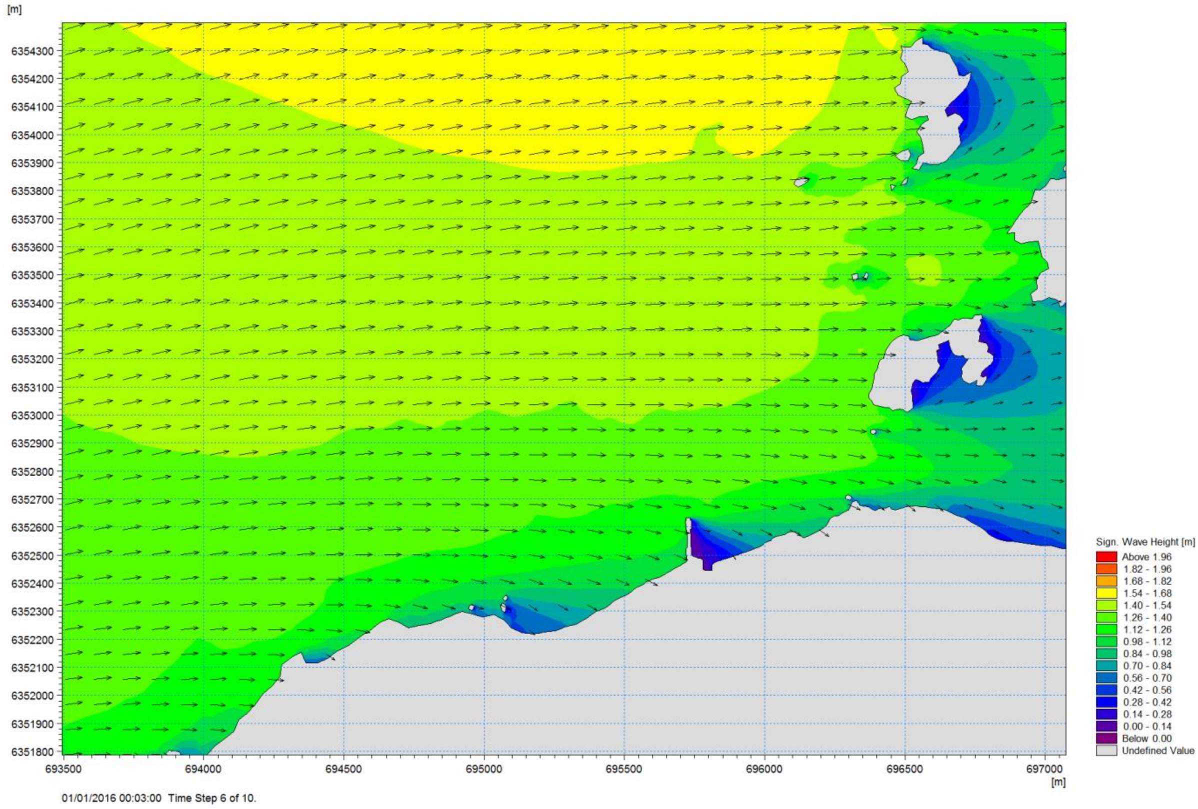
The impact of the proposed Pier on the existing wave climate in the adjoining area was assessed and quantified by running 1 in 1 year return period storms for both the existing and the proposed Pier using the Spectral Wave module which has been described in further detail in Appendix 1.

The simulations were undertaken for wind directions of 240^o, 300^o, 000^o and 45^o and were set to run during a spring high tide. However as noted previously, due to the water depths around the Pier area the spectral wave results will also be applicable to tidal levels other than spring high water.

The significant wave heights and mean wave directions for 1 in 1 year return period storms from 240^o, 300^o, 000^o and 45^o sectors are illustrated in Figure 6.6 to Figure 6.8. The difference in 1 in 1 year wave climates during storms from 240^o through to 45^o in the study area as a result of the proposed Pier and associated dredging works has been shown in Figure 6.9 to Figure 6.12. It should be noted that the extent of these Figures have been adjusted to clearly illustrate the differences around the Pier at Kyleakin.

It will be seen from these wave height difference plots that the changes in wave climate as a result of the proposed new Pier development will be restricted to the area adjoining the Pier itself. The greatest changes to the wave climate are observed during storm events from 240^o when significant wave heights are decreased by up to 0.65m on the lee side of the existing Pier structure. However, changes to the wave climate do not generally exceed ±0.25m during storm events from the other modelled directions.

For most storm direction these changes are most frequently observed within the nearshore area proximal to the development, except during storms from 240^o when minimal changes of ±0.15m can be observed at the base of the Isle of Skye Bridge.



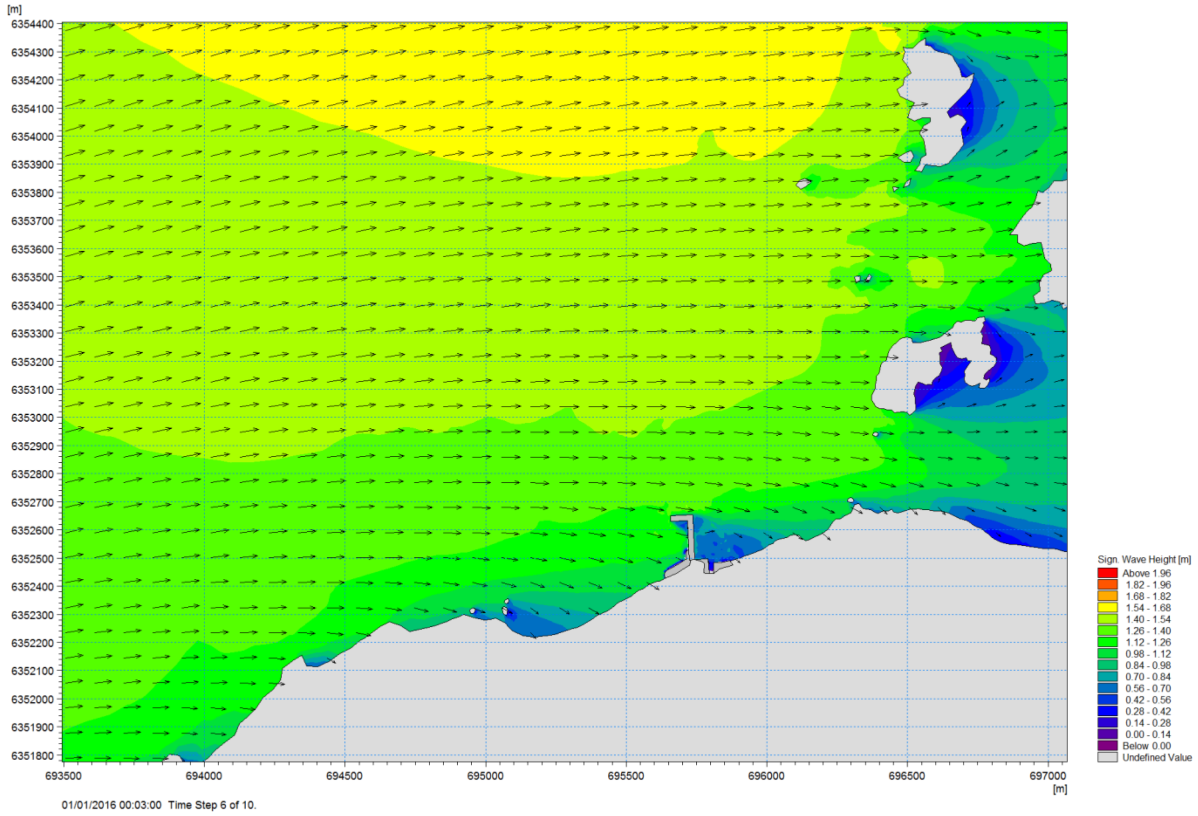
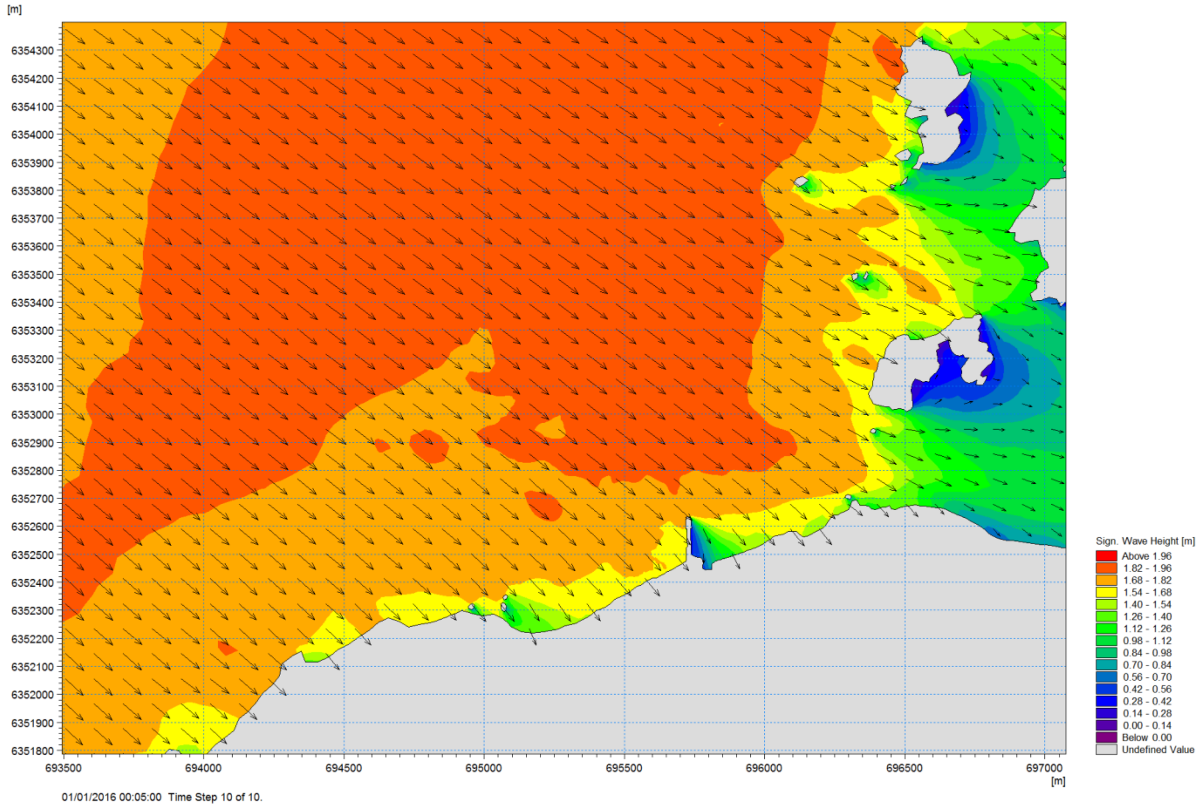


Figure 6.5: Significant wave height and mean wave direction at the existing (upper) and proposed Pier during a 1 in 1 year return period storm from 240^{0}N.



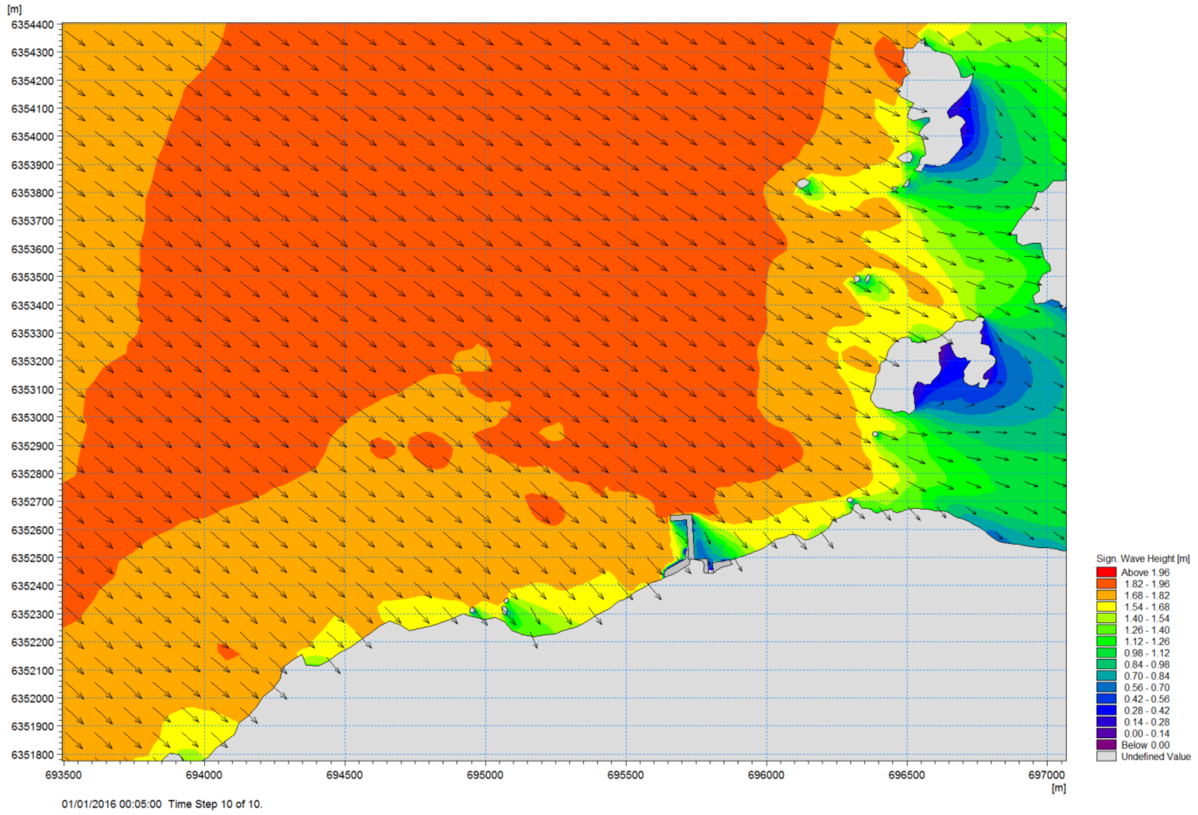
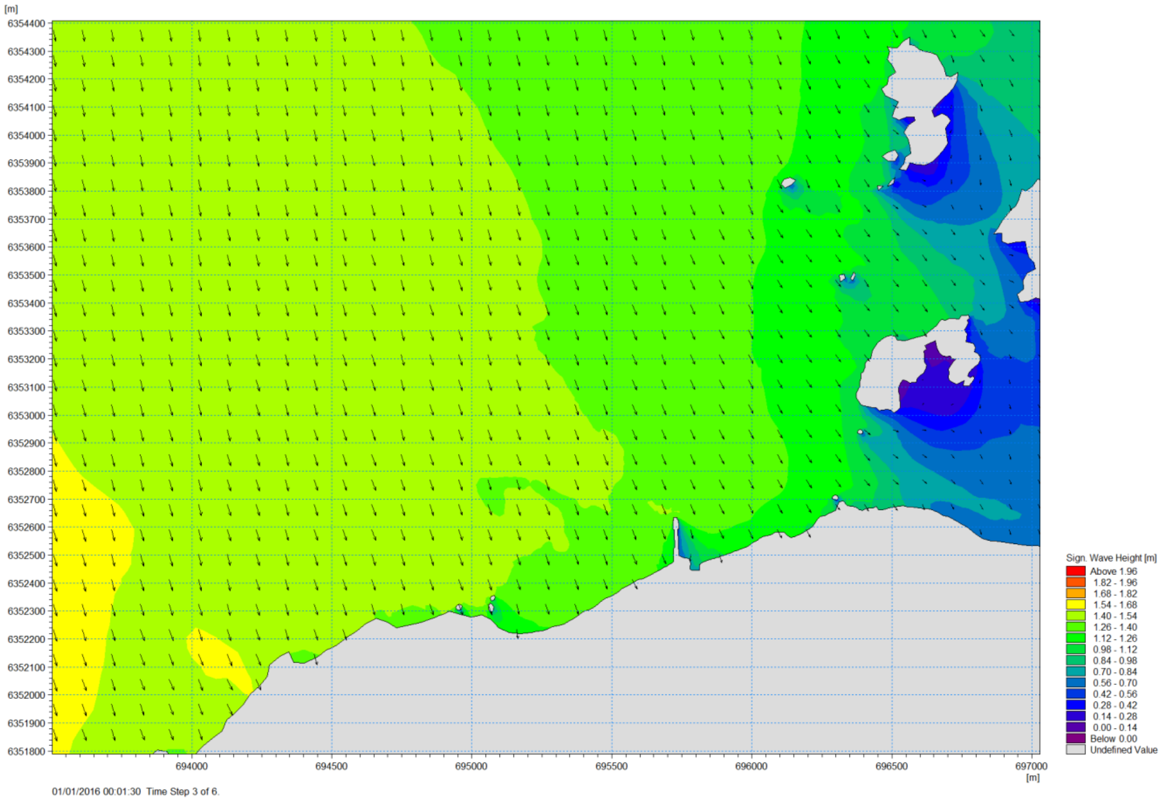


Figure 6.6: Significant wave height and mean wave direction at the existing (upper) and proposed Pier during a 1 in 1 year return period storm from 270^{0}N.



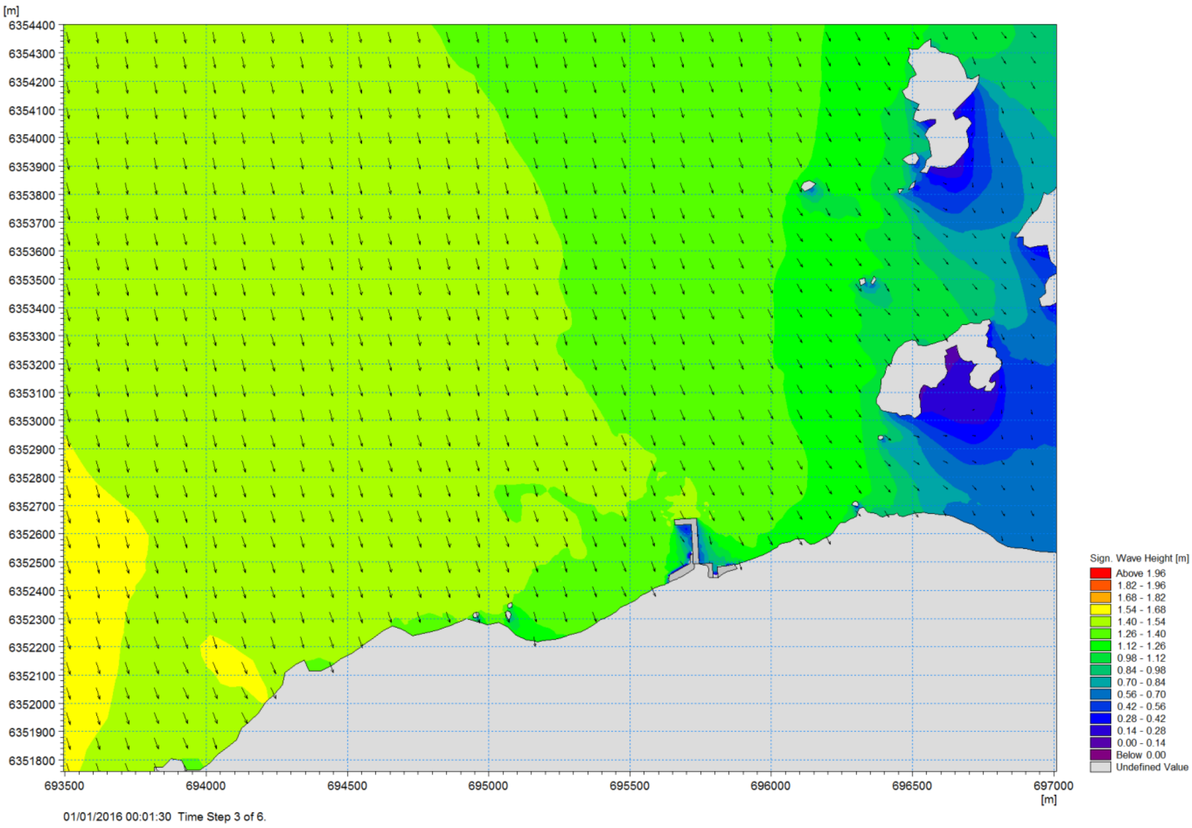
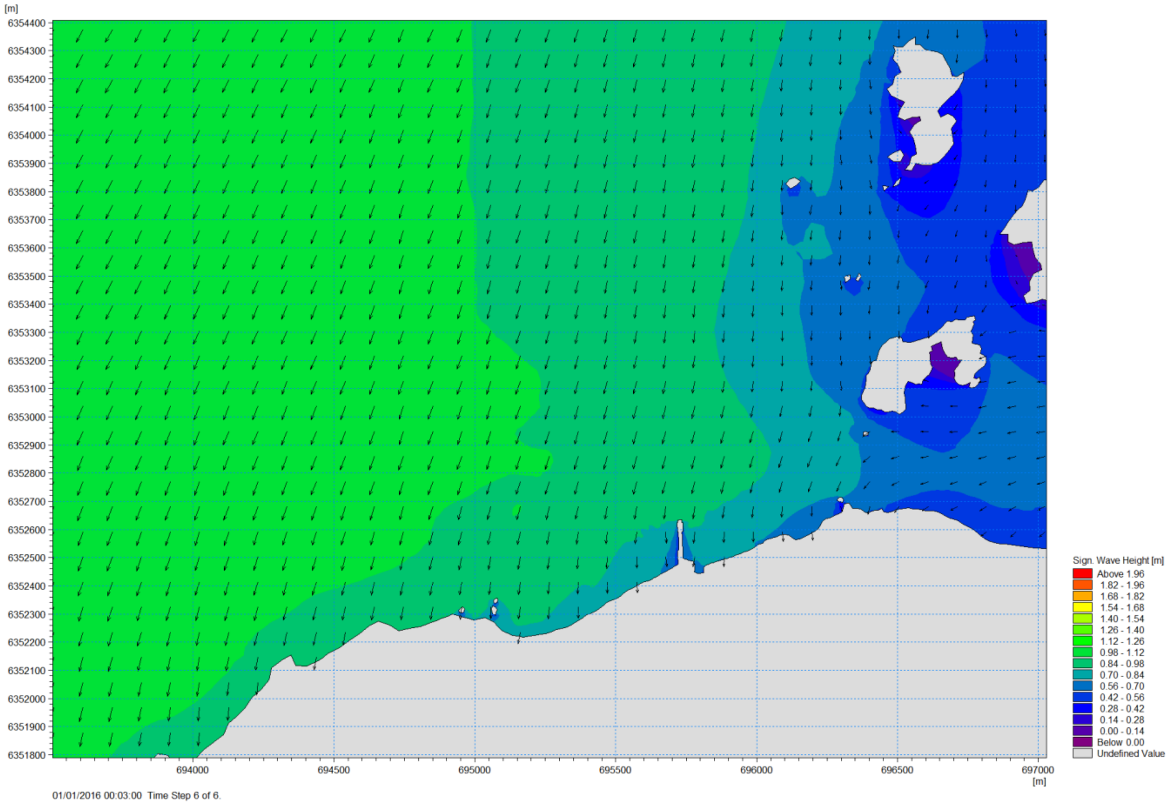


Figure 6.7: Significant wave height and mean wave direction at the existing (upper) and proposed Pier during a 1 in 1 year return period storm from 300^{0}N.



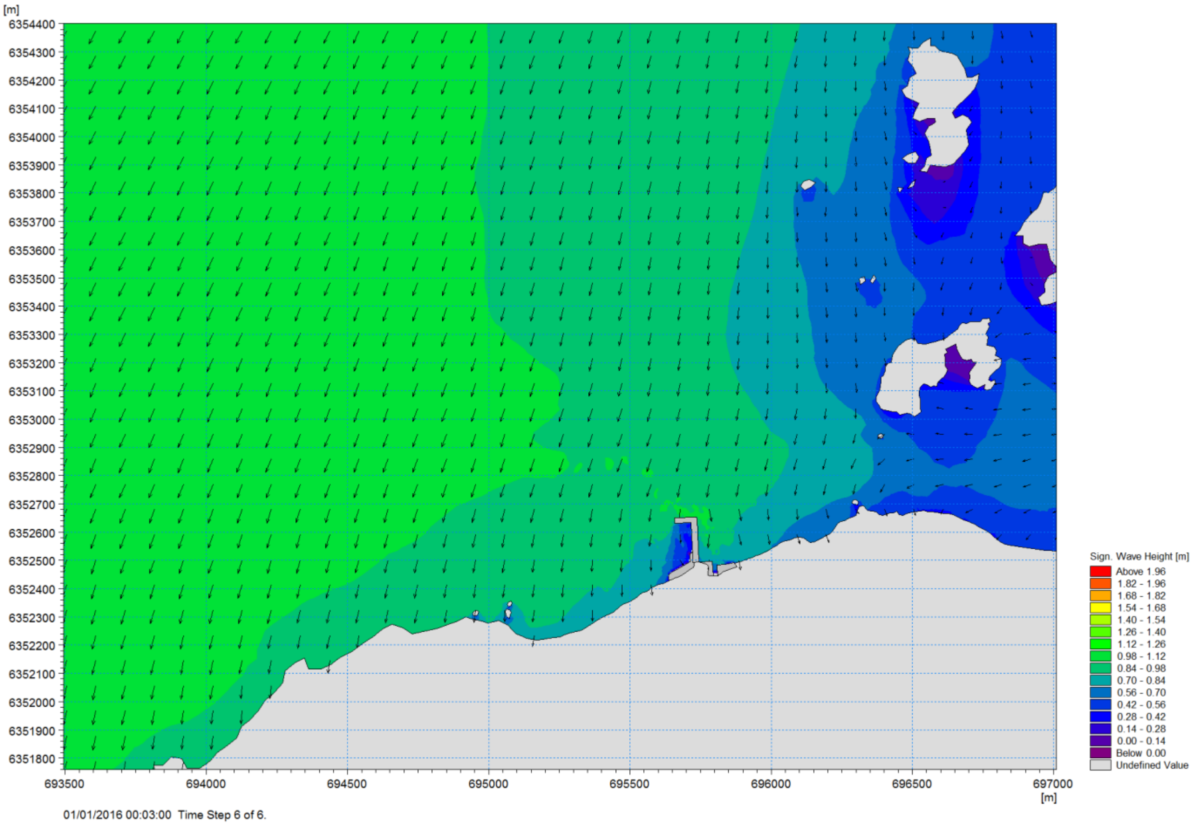


Figure 6.8: Significant wave height and mean wave direction at the existing (upper) and proposed Pier during a 1 in 1 year return period storm from 330^{0}N.

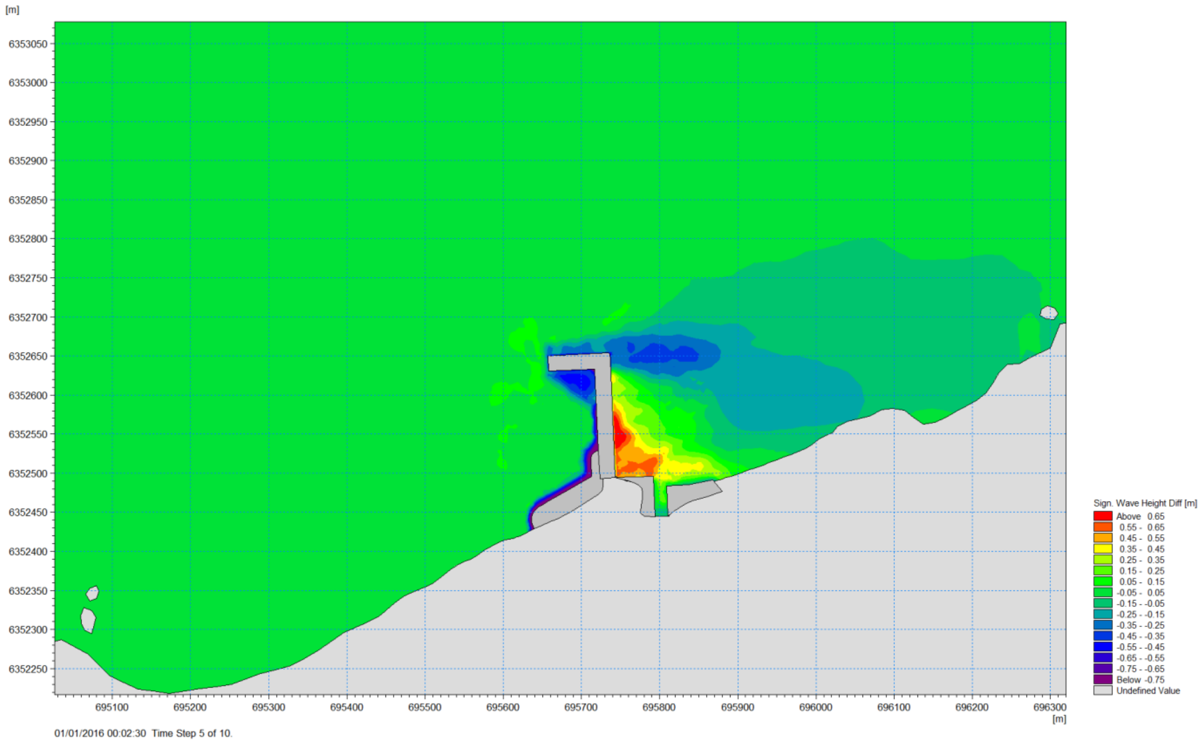


Figure 6.9: Difference in significant wave heights during a 1 in 1 year return period storm from 240°N – Proposed minus existing.

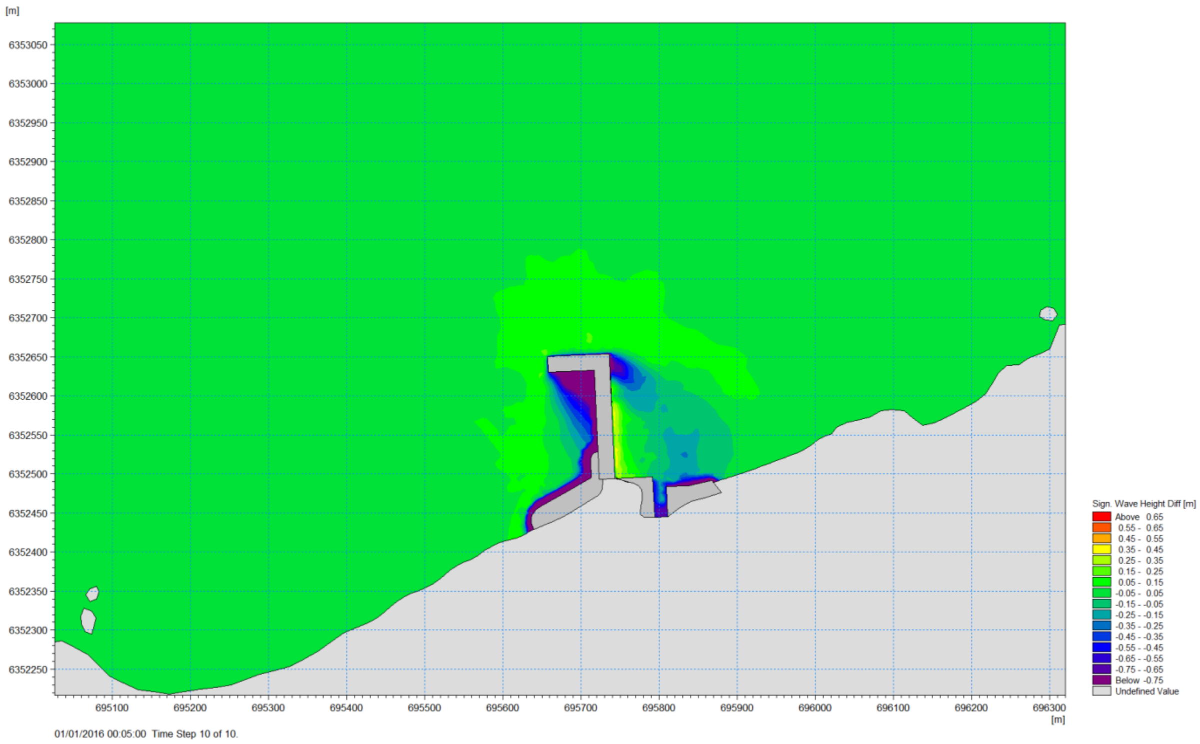


Figure 6.10: Difference in significant wave heights during a 1 in 1 year return period storm from 270°N – Proposed minus existing.

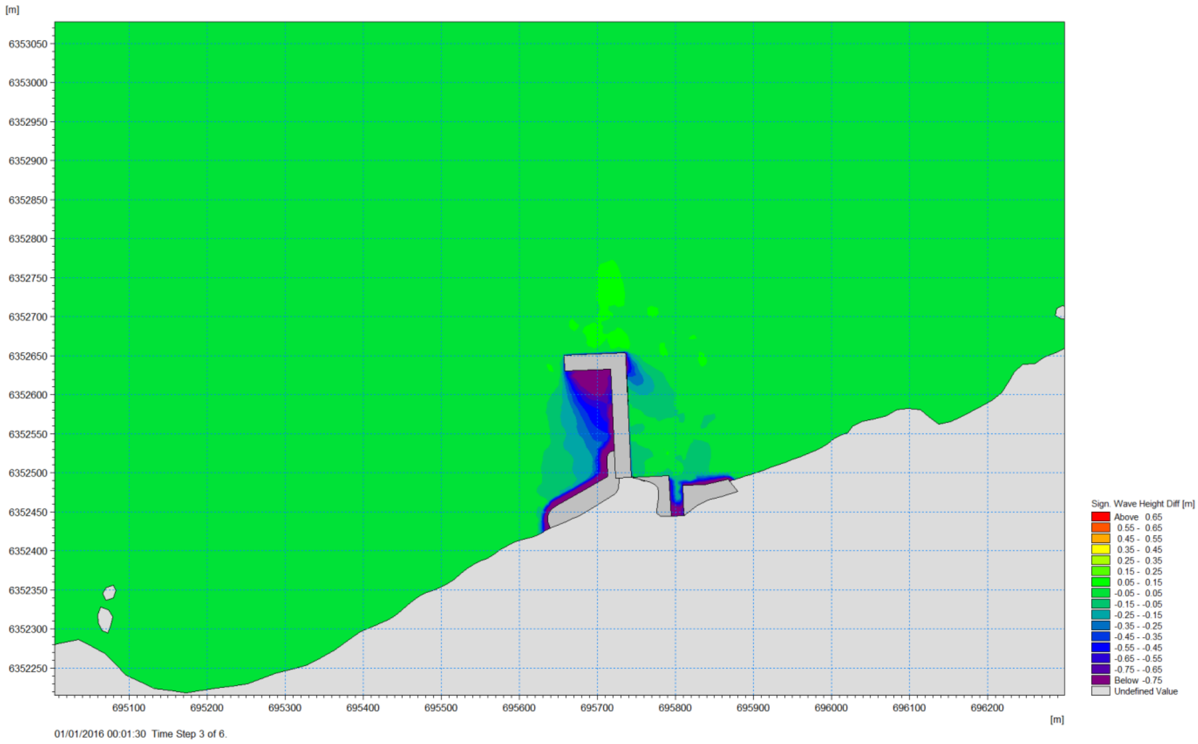


Figure 6.11: Difference in significant wave heights during a 1 in 1 year return period storm from 300°N – Proposed minus existing.

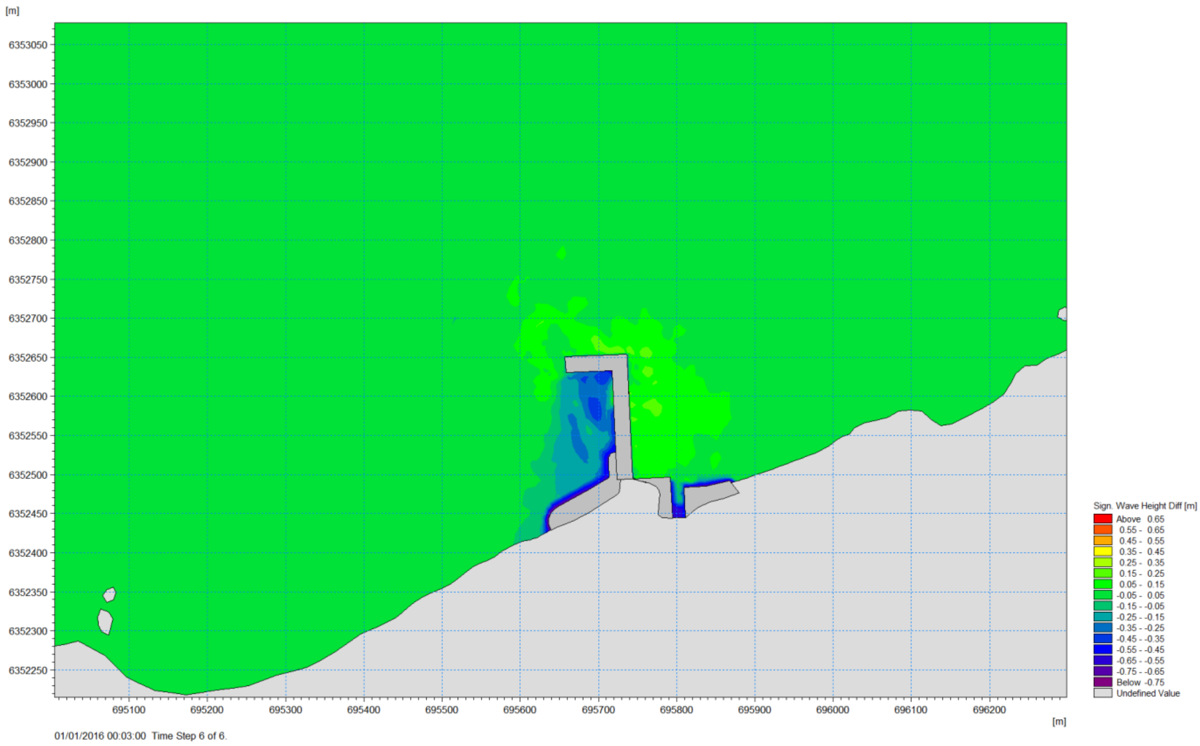


Figure 6.12: Difference in significant wave heights during a 1 in 1 year return period storm from 330°N – Proposed minus existing.

7 SEDIMENT TRANSPORT AT KYLEAKIN PIER

7.1 BACKGROUND

The sea bed to the north of Kyleakin Pier is generally composed of coarse material and the natural levels of suspended sediment in the water column are very low. Thus the main movement of sediments in the area result from waves breaking along the coast particularly on the sections of shoreline that have fine beach material.

Only the larger waves in the overall wave climate generate littoral currents of sufficient strength to produce significant longshore sediment drift. As can be seen from Figure 7.1 which illustrates the wave rose for these larger elements of the overall wave climate, the waves capable of driving longshore sediment drift approach the Pier from the north and west.

As there is a very limited supply of beach material on the coast to the east of the Pier (see Figure 7.1), the only sediment drift at Kyleakin Pier will be along the western shoreline from a south westerly direction.

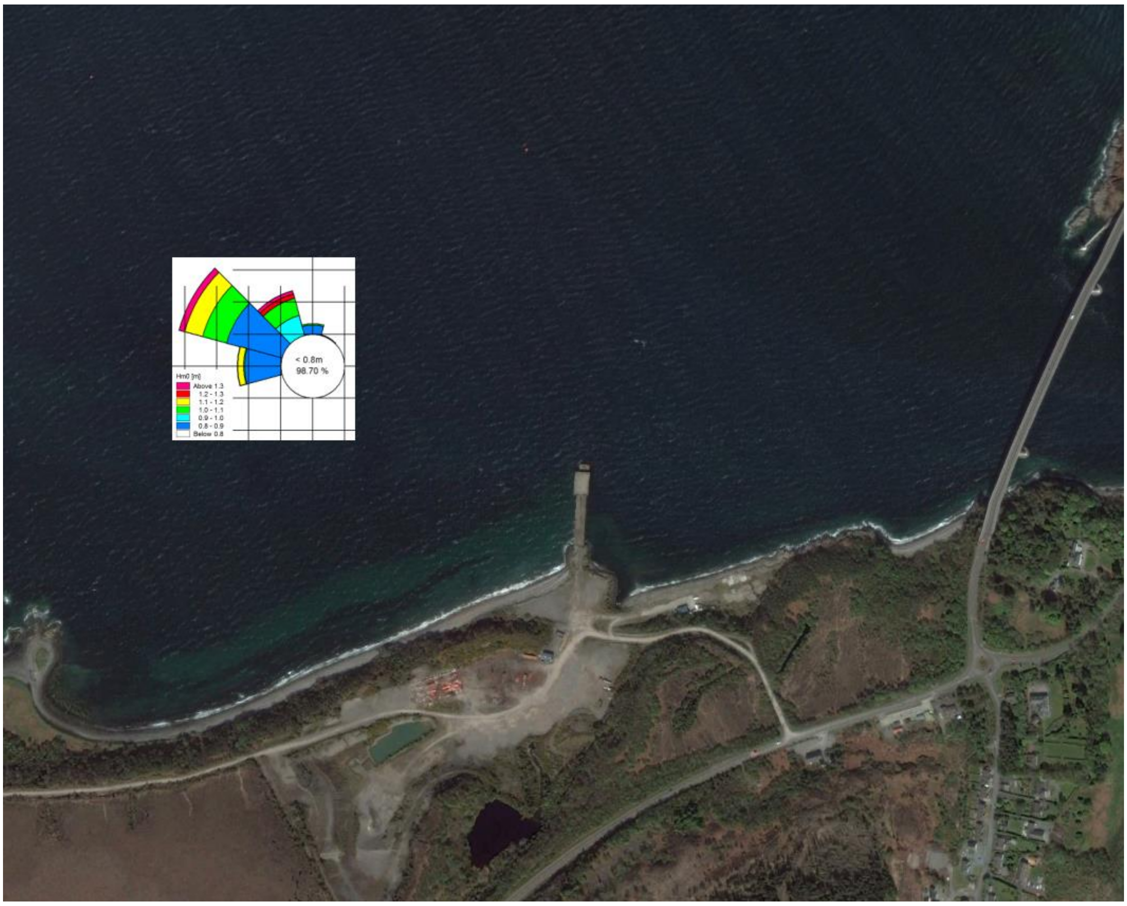


Figure 7.1: Wave rose for more energetic waves in the overall wave climate at Kyleakin Pier site.

7.2 SEDIMENT TRANSPORT MODELLING

As described in Section 4.5 sediment transport simulations were undertaken using the MIKE 21 coupled Hydrodynamic, Spectral and Sediment Transport model to assess the impact of the proposed Pier and capital dredging requirements on the sediment transport regime.

It is well established that in any coastal site sediment transport rates vary both temporally and spatially. However storm current-wave interactions are recognised as being one of the most effective drivers of sediment transport and re-suspension, particularly within the surf zone. This study has therefore taken a conservative approach to investigating the impact of the proposed Pier on the stability of the seabed in within the vicinity of the proposed works by simulating the transport of sediment under typical 1 in 1 year storm conditions.

The coupled sediment transport model was initially run using the existing model domain for a four day period under 1 in 1 year storm conditions from the north-west sector (details of 1 in 1 year wind and wave conditions can be found in Section 6). This coupled model numerical resolves the transport of non-cohesive sediment material on the seabed (i.e. the bed level change) based tidal information from the HD model, the radiation stresses generated by the wind and wave action in the surf zone and the physical properties of the sediment comprising the seabed such as grain size etc.

The composition of seabed for this numerical model was specified using information gathered during the geotechnical survey undertaken in August 2016 (detailed in Section 3.2). In brief, the model seabed was represented by a gravel material in the outer area of the proposed works whilst the material in the inner area ranged between very fine sand and coarse sand material.

7.3 IMPACT OF THE PROPOSED PIER ON BED STABILITY

By comparing Figure 7.2and Figure 7.3 which presents the change in the sea bed level after the north westerly storm event for the existing and the proposed Pier respectively, it will be seen that the morphological response of the sea bed to the storm has a similar pattern. In both scenarios, there is a notable build-up of sediment to the west of the Pier. On the eastern side, it can be seen that material in the area just beyond the proposed side slope is transported towards the shore by the littoral currents to result in a notable build-up of material in the nearshore area.

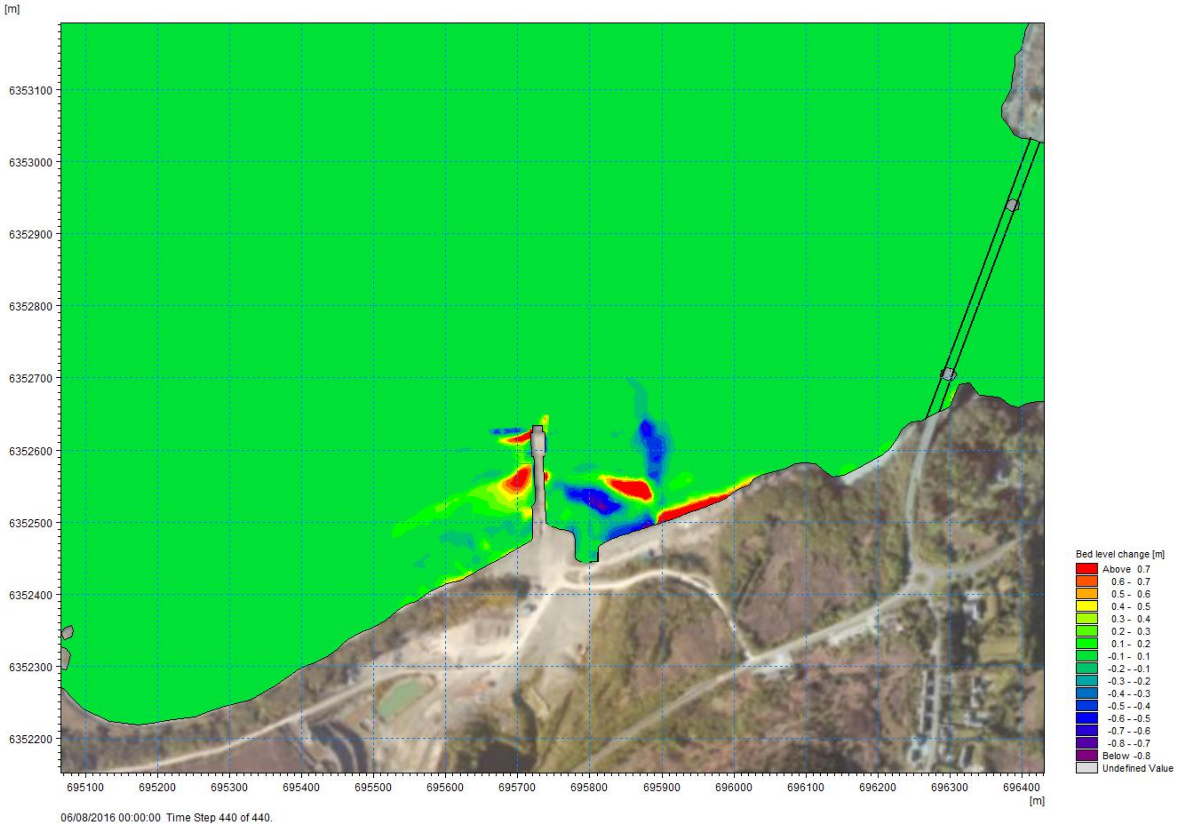


Figure 7.2: Change in bed level after a 4 day 1 in 1 year storm from the NW – Existing Layout.

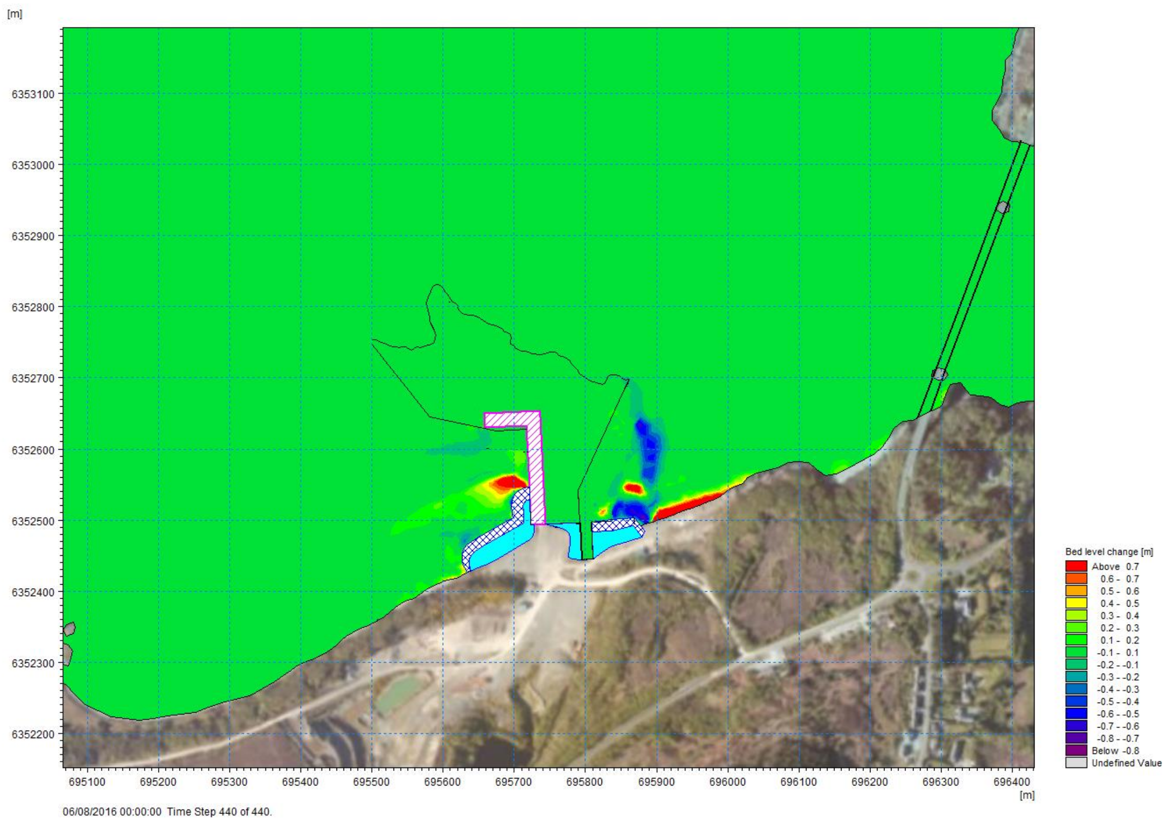


Figure 7.3: Change in bed level after a 4 day 1 in 1 year storm from the NW –Proposed Layout.

Importantly, it will be seen from Figure 7.4 which illustrates the difference in bed level changes between the proposed and the existing that the majority of the differences are within the nearshore area of the proposed structure.

Minor differences can be seen to the left of the proposed revetment; these differences of ±0.2m result from the new 79m quay deflecting the direction of the littoral currents and simply represent a minor displacement of sediment. To the east of the proposed Pier structure, the re-graded side slopes can be seen to cause a minor difference in bed level change. However these minor changes are generally within the nearshore area proximal to the development

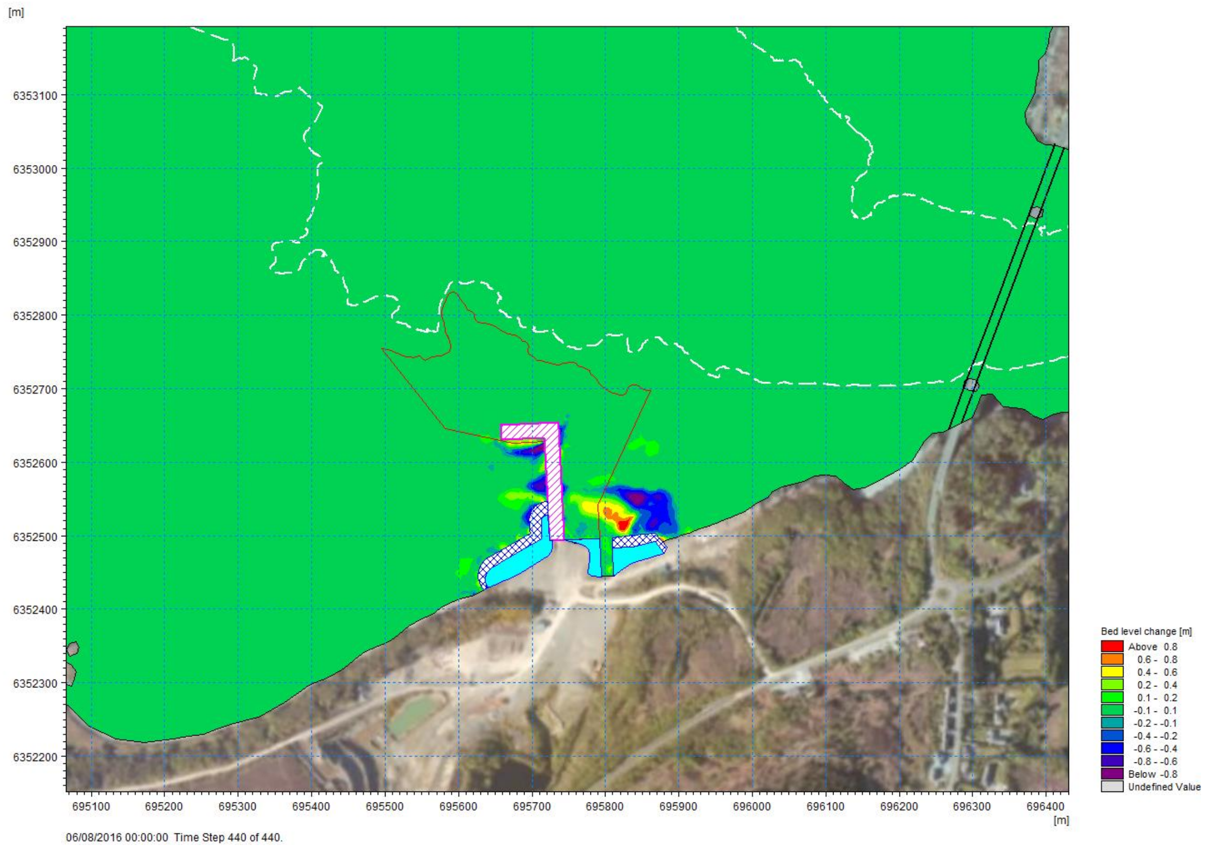


Figure 7.4: Difference in bed level change a 4 day 1 in 1 year storm from the NW – Proposed minus existing.

7.3.1 Summary of the effect of the Proposed Pier development on sediment transport

Based on the results of the sediment transport simulations presented in this section of the report, it be can be concluded that even under 1 in 1 year storm conditions virtually all of the bed changes are contained within the upper surf zone of the site where waves would be breaking. The results robustly demonstrate that the proposed works will only result in minimal changes to the sediment transport regime beyond the immediate vicinity of the works.

7.4 PROPELLER INDUCED SCOUR PROTECTION

7.4.1 Operational Phase

A navigational risk assessment study undertaken by ABPmer (2016) has shown that vessels will approach the proposed Pier at Kyleakin from the north, see Figure 7.5 below.

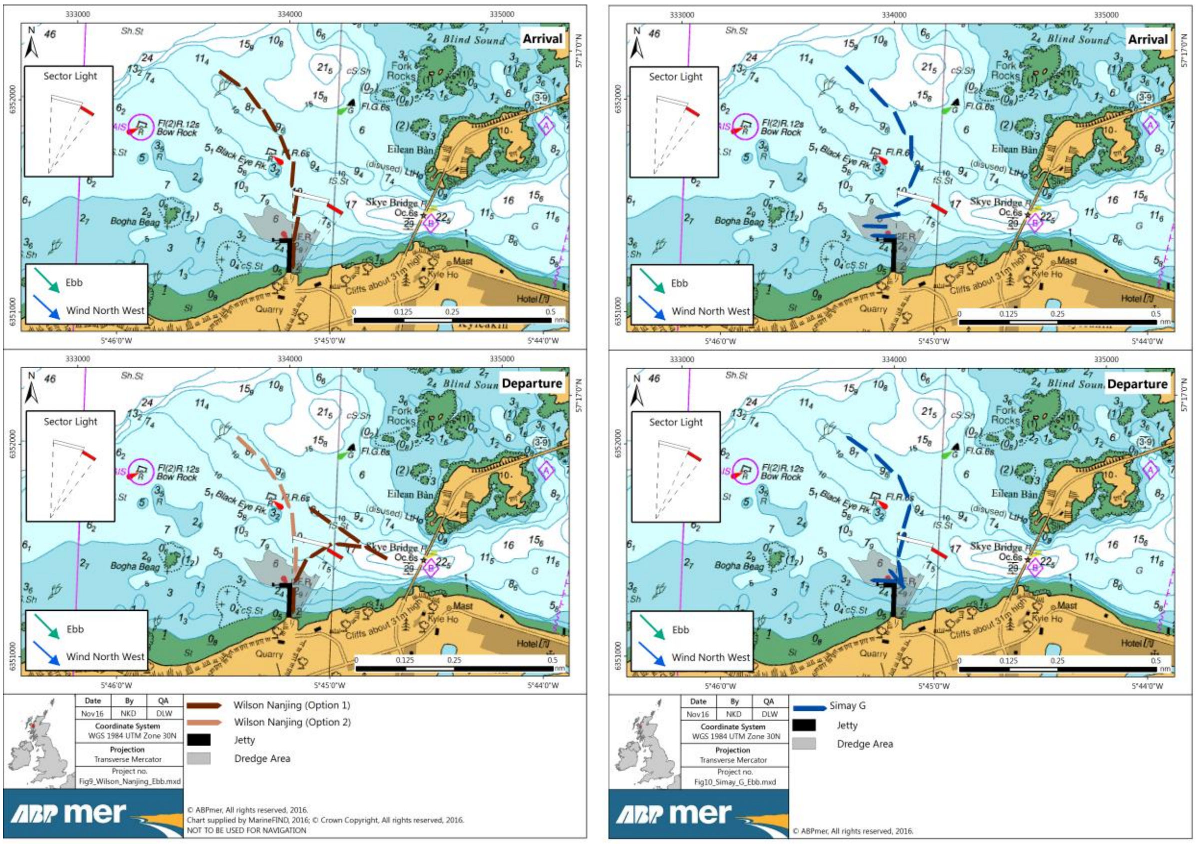


Figure 7.5: Navigation simulation plots prepared by ABPmer.

It has been established that the sea bed to the north of the Pier is composed of exposed bed rock, cobbles together with occasional boulders and during storm conditions can be exposed to relatively strong tidal conditions with storm waves in excess of 2.5 metres.

Based on the information presented in ABPmer's navigational risk assessment study the worst case for propeller wash disturbance on the sea bed to the north of the -9.5m CD contour will be when a fully laden vessel approaches the Kyleakin Pier area from the north at a time of LAT. At this time the under keel clearance for the largest fully laden ship, the Wilson Nanjing, will be 2.2m. The vessel will be approaching the area at about 4 knots using less than 10% of the installed power.

Based on these conditions propeller wash calculations have been undertaken, in accordance with the BAW code of practice 2010. The calculations demonstrate that the maximum near bed velocity from the propeller wash will occur at about 20 metres aft of the propeller and that the peak near bed velocity will be 0.85 m/s. This velocity is comparable with existing tidal velocities in this area which for a typical spring tide has a near bed velocity of about 1 m/s. Thus it will be seen that the impact of the propeller wash on the bed will be less than that already experienced during spring tidal conditions. As such, no additional disturbance is expected to sea bed to seaward of the -9.5m CD contour.

However, as the ships enter the dredged area and approach the berths they will require to use both the main engine and thrusters to safely come alongside the proposed quays. It is anticipated that the ships will come alongside the proposed 160m berth bow first, i.e. bow in towards the shoreline. Information gathered during geotechnical surveys demonstrated that bed material along this quay is comprised primarily of sand. Thus scour protection should be installed along at least the outer half of the berth to prevent the propeller wash eroding a hole in the bed around the draft end of the ship and depositing the material in the inner part of the berth.

Scour protection may also be required along parts of the inner section of the 160m berth if ships masters find they frequently need to use thrusters to safely approach or leave this berth. Scour protection may be added to this area at a later date if it is found that high thruster use is required for navigational purposes. The berth along the north face of the proposed 79m quay will also require scour protection from ship propeller action as ships arrive and depart from this berth. In addition during severe storms wave reflections from this structure will result in locally high scour currents at the base of the wall particularly adjacent to the eastern end of the berth.

7.4.2 Construction Phase

The vessels using the Kyleakin Pier during construction will generally be work boats, barges, tugs and dredging equipment. The largest vessel is likely to be a trailer hopper dredger which will be working almost exclusively within the dredged area. The loaded depth of a dredger suitable for this site is about 2.8m. Other vessels used during construction would be limited to a draft of less than 3 metres as this is the depth alongside the existing quay.

The most powerful vessels operating outside the dredged area is likely to a tug. Typically this vessel would be 31 metres long with a draft of about 2.2 metres. Installed power is likely to be of the order of 290 kW per propeller and the worst case for propeller wash on the sea bed will occur when there is a rudder directly behind the propeller.

Propeller wash calculations have been undertaken, in accordance with the BAW code of practice 2010, for this vessel operating at LAT at the -9.5m CD contour. For these calculations it has been assumed that the vessel would be manoeuvring slowly (which gives high wash conditions), such as when manoeuvring caissons towards the site. These calculations have demonstrated that the maximum near bed velocity due to propeller wash will be *c.* 0.984 m/s. As noted above this velocity is comparable with existing tidal velocities in this area which for a typical spring tide has a near bed velocity of about 1 m/s. Thus it is expected that construction vessels will not disturb the seabed to seaward of the -9.5m CD contour.

7.5 SLOPE STABILITY ANAYLSIS

7.5.1 Slopes on eastern side of Pier

The stability of the proposed dredged 1 in 7 slope on the eastern side of the dredged berthing area has been investigated using a variety of Mike21 wave and sediment transport models. These include the coupled flexible mesh tide, spectral wave and sand transport model and the Boussinesq wave model in combination with the STP_Q3 model which can calculate the sediment transport for combined waves and currents including sloping beds.

Vibrocore samples vc1, vc2, vc3, vc5 and vc6 collected during the geotechnical site investigation (see Section 3.2) indicate that in the area of the proposed dredged 1 in 7 slope the bed sediments below the existing surface are fine or very fine sands. Thus for the purpose of this stability modelling a Dn_{50} grain size for the slope material has been taken as 0.1mm.

For the purposes of this stability modelling a 1 in 100 year return period storm from 300^o to 330^o N has been used in combination with spring tide flows. The 300^o to 330^o N storm direction was chosen as these conditions tend to produce the most arduous conditions at the Pier (see Section 6). Furthermore, the dredged 1 in 7 slope is more exposed to waves from this direction as opposed to storms from a more westerly direction.

The waves approaching the Pier during a 1 in 100 year storm will have significant wave heights of about 2.75 metres with spectral peak periods of 5.6 seconds.

7.5.2 Results of eastern slope stability analysis

In order to take account of the effects of wave reflections from the Pier and the quay walls a Boussinesq wave model was set up to simulate how storm waves would be modified due to wave refraction, diffraction and wave reflections around the proposed Pier area. The model was run for 1 in 100 year return period storm waves with the water levels set at mean sea level. Figure 6.4 in Section 6.4 shows the typical storm wave patterns around the Pier with 2.75m significant height waves approaching the Pier from 330^o. The wave heights around the Pier area are also illustrated in Figure 6.4.

It will be seen that there is considerable wave reflection from the outer caisson structure and the quay wall as well as diffraction of the waves around the north eastern corner of the Pier all of which will have an effect on the wave climate at the proposed 1 in 7 dredged slope to the east of the Pier. The Boussinesq wave model can also calculate the likely wave driven currents around the Pier area and these are shown in Figure 7.6 below.

The currents field shown in Figure 7.6 illustrate wave driven currents only and do not include any tidal currents in the area. Nevertheless Figure 7.6 demonstrates that there is a circulation behind the projecting caisson structure to the west of the Pier and a north west going current of up to about 0.35 m/s in the area of the bank that lies immediately to the east of the slipway projection. This wave driven current combined with the wave height of about 2 metres significant height may destabilise the 1 in 7 slope in this area if the surface of the bed is exposed sand.

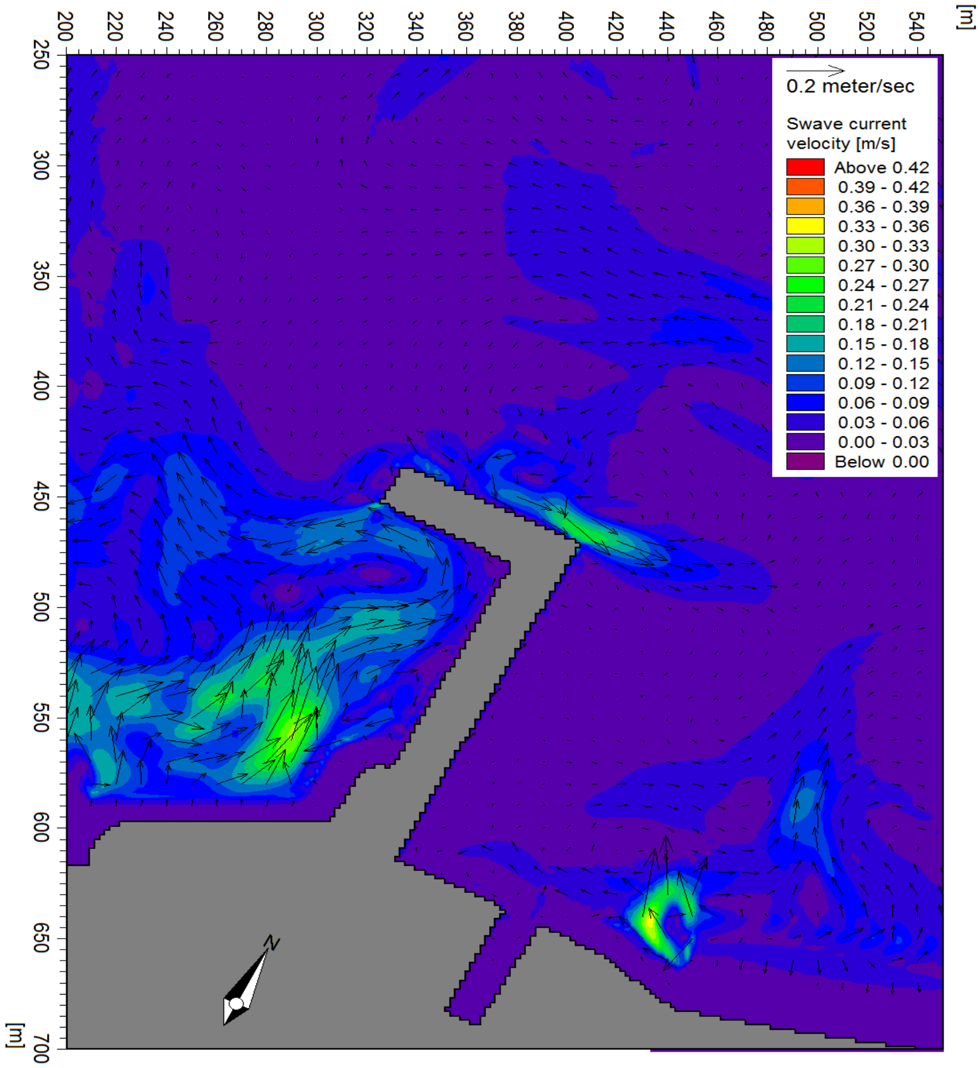


Figure 7.6: Wave driven currents derived from Boussinesq wave model simulation.

The tidal currents across the other parts of the 1 in 7 slope to the east of the Pier were taken from the flexible mesh model spring tide simulations. The typical values at peak flow were found to be in the order of 0.32 m/s flowing in 120^o – 300^o directions across the bank. These tidal currents were combined with the wave heights derived from the Boussinesq wave model simulations and the rate of sediment transport calculated using the STP_Q3 program which forms the basis for all the Mike21 non cohesive sediment transport models.

The calculations showed that the rate of sediment transport on the 1 in 7 dredged slope would peak at about $0.95173e^{-03}$ m^3/m/s across the bank at the time of maximum north-west going tide with the 1 in 100 year return period storm. The sediment transport reduced to about $0.1428e^{-03}$ m^3/m/s across the bank when the tidal stream reduced to a velocity of 0.1 m/s. Thus the computational modelling and analysis indicates that for a fine sand bed material the 1 in 7 dredged slope to the east of the Pier will not be stable under storm conditions.

However, the seabed around the existing Kyleakin Pier is generally composed of a surface layer of cobble and gravel and in many places the natural slope of this bed material is considerably steeper than 1 in 7. In addition the proposed capital dredging regime will produce a very considerable amount of gravel and cobble material which can be used to provide a protective cover layer across the 1 in 7 dredged side slopes in the same manner as the natural seabed in large parts of the Kyleakin Pier area. It is expected that this material will be placed onto the side slopes using a hydraulically operated backhoe type dredging equipment which is eminently suitable for this type of operation. This equipment can also be used to compact the placed material on these side slopes.

Modelling and analysis was therefore undertaken for the stability of the bank covered with cobble sized material using the Boussinesq wave, tidal and STP_Q3 model programs. These simulations showed that there would be no movement of the cobble sized material on the 1 in 7 slopes even under 1 in 100 year storm conditions and spring tide flows. Thus it was concluded that the 1 in 7 side slopes with a cover layer of cobble sized material would provide a stable bed form for the eastern boundary of the dredged area at Kyleakin Pier as the cobble sized stone is able to resist current induced scour even under 1 in 100 year return period storm conditions.

7.5.3 Slopes on western side of Pier

The stability of the proposed dredged 1 in 7 slope on the western side of the dredged berthing area has been investigated using a variety of Mike21 wave and sediment transport models. The results of the site investigation indicate that the material comprising the side slopes to western side of the Pier is comprised primarily of a coarse sand with mean grain diameters of 0.5mm to 0.6mm. However the material also contains sediments sizes up to 5 to 10mm diameter.

For the purposes of this stability modelling a 1 in 100 year return period storm from 300^{o} N has been used in combination with spring tide flows. The 300^{o} N storm direction was chosen as these conditions tend to produce the most arduous conditions at the Pier (see Section 6). Furthermore, the dredged 1 in 7 slope is more exposed to waves from this direction as opposed to storms from a more westerly direction.

It was found that the most arduous conditions for slope stability on the western side of the Pier resulted from a combination of the littoral current regime and the wave climate during the 1 in 100 year return period storm near to low tide. At this time the littoral current field during a 1 in 100 year storm will be as shown in Figure 7.7 with a current of velocity of 0.33 m/s flowing an a direction of about 60^{o} N across the outer part of the dredged slope. The wave climate at the slope at this time will have significant wave heights of about 1.87 metres with spectral peak periods of 5.38 seconds.

Around the time of mid tide the littoral currents and the east going tide results in a north west going flow around the south western corner of the western caisson which could affect the stability of the dredged slope in this area. The current velocity is *c.*0.51 m/s flowing in a 300^{o}N direction. The wave climate at the part of the dredged slope at this time has a significant wave height of 1.98 metres and a spectral peak wave period of 5.32 seconds approaching from 321^{o}N.

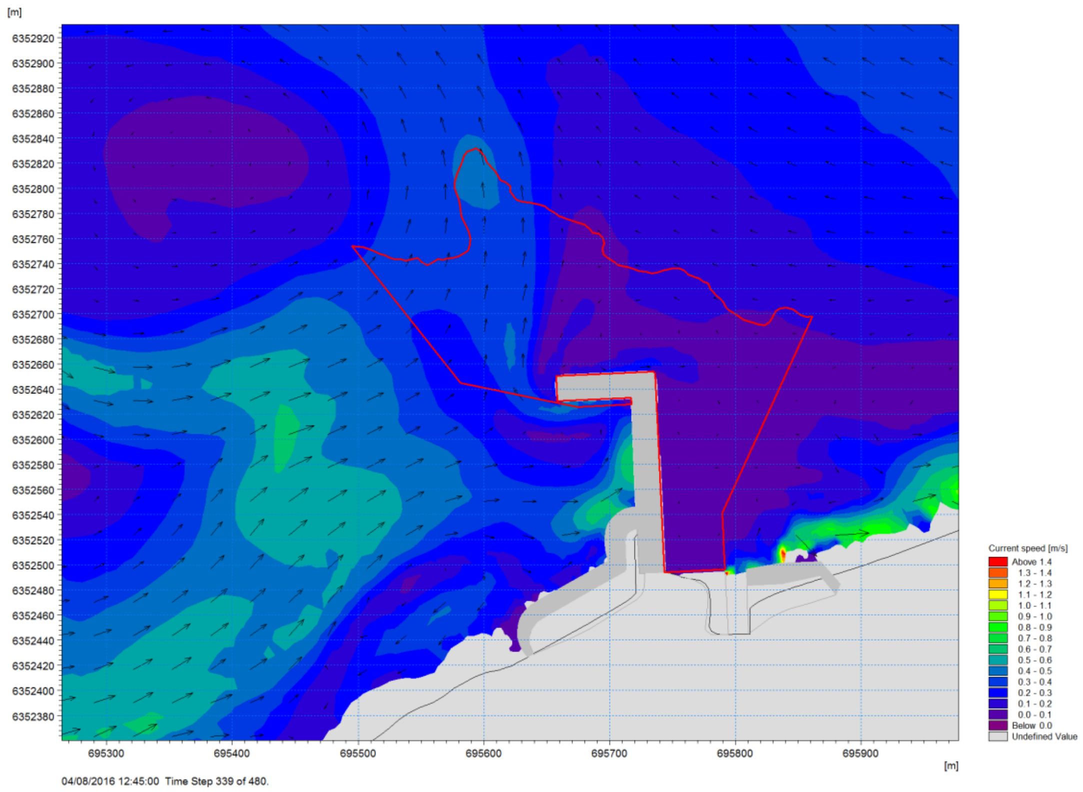


Figure 7.7: Littoral current during 1 in 100 storm from 300o with spring tides and a water level of -1.874m MSL.

7.5.4 Results of stability analysis

The stability analysis was undertaken suing the Q3_STP module which can calculate the sediment transport for combined waves and currents including sloping beds. The analysis showed that for the Dn_{50} sized material of 0.5mm on the outer slope the sediment transport across the slope during a 1 in 100 year storm would peak at about 0.005mm/s/m length of slope. However the grading of the natural material on the slope contains gravel type material with grain diameters in excess of 4mm. The sediment transport analysis showed that this material would be stable even during a 1 in 100 year return period storm. This shows that the coarser parts of the sediment grading of the material on the slope will naturally armour the surface of the slope so that the slope will have long term stability.

A similar analysis was undertaken for the inner section of the 1 in 7 slope close to the end of the caissons. In this case the rate of sediment transport across the slope for the Dn_{50} sized material of 0.64mm was found to be 0.002 mm/s/m length of slope. However the fraction of the material in excess of 2mm will be stable and as with the outer slope the coarser fraction of the material will effectively armour the slope so that slope will have long term stability..

8 IMPACT OF CAPITAL DREDGING ON WATER QUALITY

8.1 BACKGROUND

8.1.1 Description of Capital Dredging Requirements

In order to facilitate the development of the proposed Pier at Kyleakin, dredging and reclamation activities have been planned to increase the depth of the seabed within the immediate vicinity of the proposed Pier to -8.50m CD with slopes of 1/7 to meet existing seabed levels. The proposed dredge area including the 1/7 side slopes in relation to the existing Pier is illustrated below in Figure 8.1**.**

In total, Wallace Stone LLP have anticipated that approximately 190,000m^3 of material will be dredged from the study site to achieve the desired -8.5m CD depth and associated side slopes.

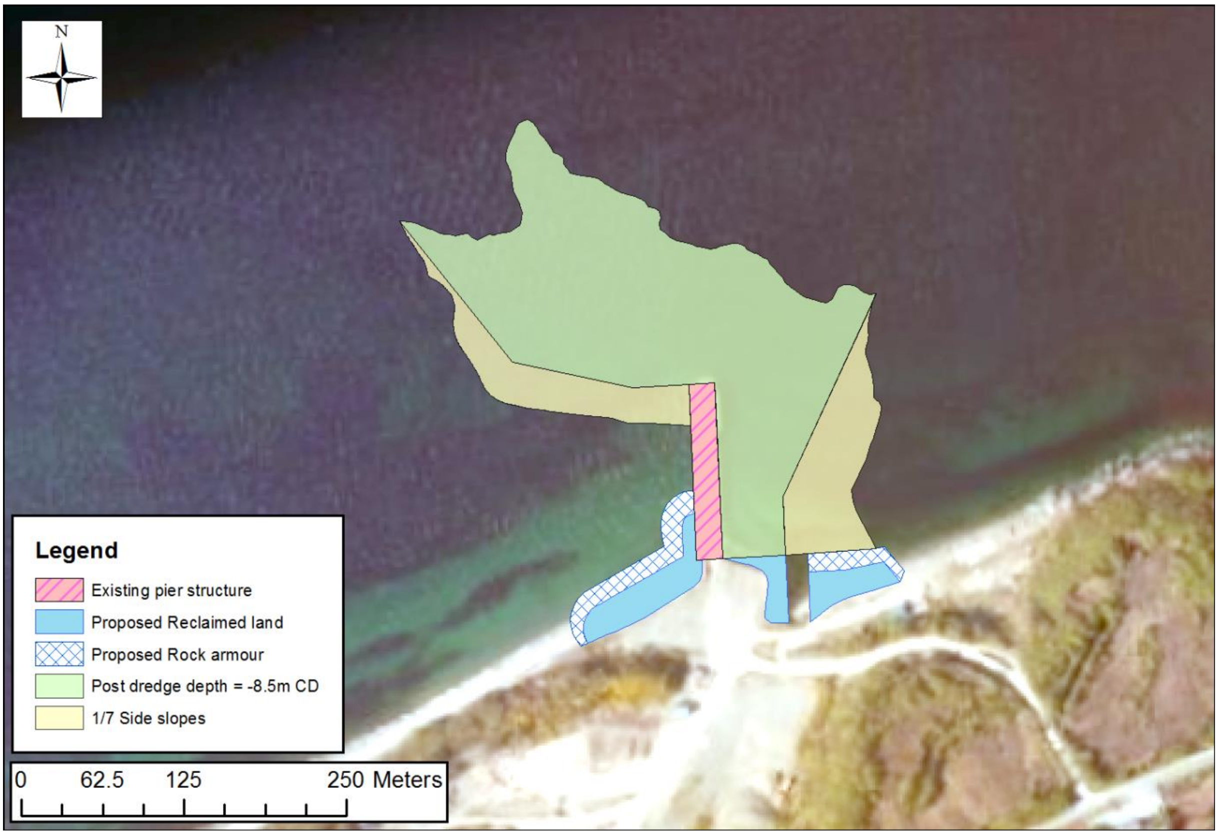


Figure 8.1: The extent of the capital dredging works required as part of the proposed development.

8.1.2 Characterisation of seabed

Particle Size Analyses (PSA) of the sediment samples collected during the geotechnical survey detailed in Section 3.2 indicated that much of the outer dredge area was dominated by coarse gravel material whilst the inner dredge area was characterised by a wider distribution of fine silt to coarse sand material.

To accurately reflect the heterogeneous nature of the proposed dredge material RPS has applied different sediment characteristics to the outer and inner dredge areas before undertaking any numerical modelling.

Based on the hydrographic and geotechnical surveys of the area, it was found that *c.* 85,500m^3 and 104,500m^3 of material had to be dredged from the outer and inner areas respectively. The location and extent of these dredge areas are illustrated in Figure 8.2 below.

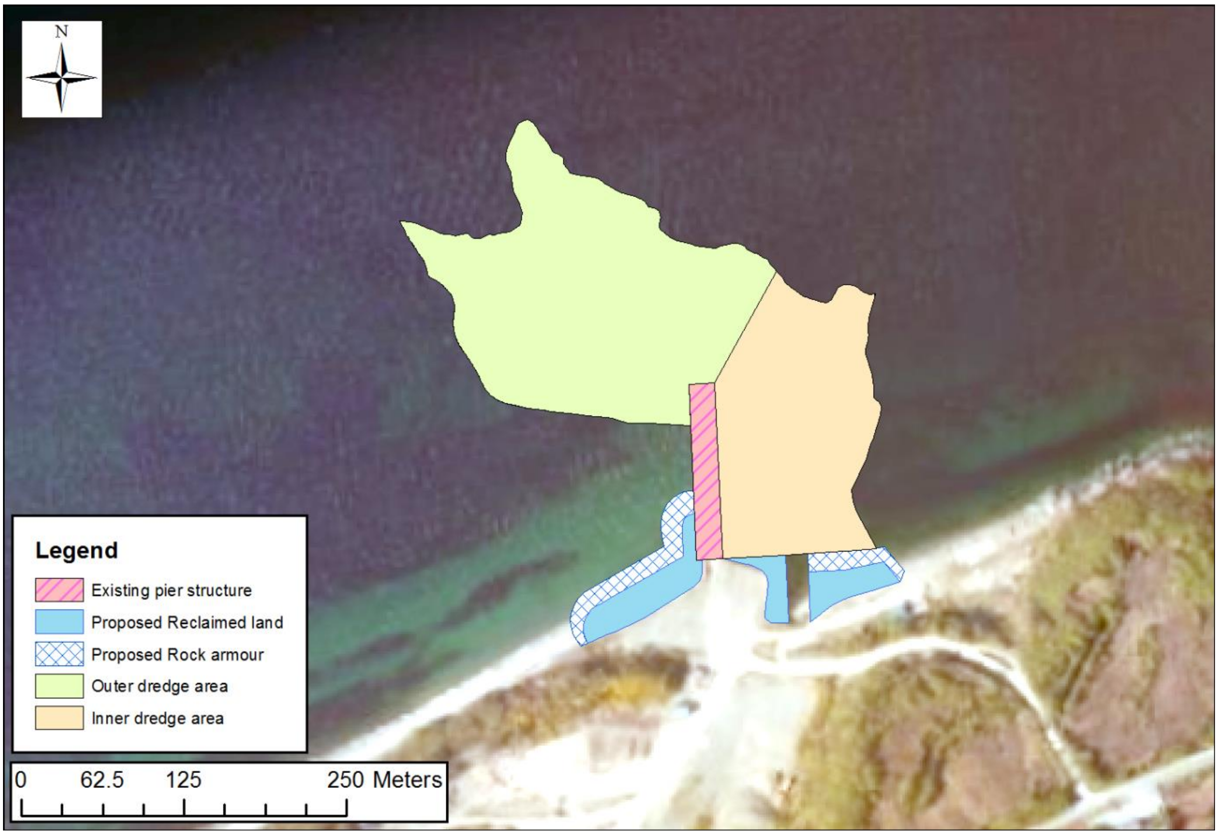


Figure 8.2: Location and extent of the inner and outer dredge areas at Kyleakin Pier.

A common method of assessing the potential environmental impact of dredging plumes involves the use of coupled hydrodynamic and sediment transport modelling techniques. An important step in this approach is stipulating appropriate source terms to define the input of dredge material into the model. One of the primary factors in determining the dredging source term is based on the total amount of available fines within the dredge material.

Other studies (Becker *et al*. 2015; Koningsveld 2015) that investigated the large scale spatial and temporal fate of dredge plumes have only accounted for the fraction of fine material (<63µm) as coarser particles are known to settle to the seabed within the near zone (Land *et al*. 2004).

However as this study is interested in assessing the fate of dredge plumes in relation to the adjacent environment and, specifically, the flame shell beds which are found just beyond the -9.5m CD contour (see Figure 8.3), RPS has also accounted for material as large as coarse sand (1000µm) in the sediment transport simulations detailed in the following Sections. Preliminary investigations found that material coarser than 1000µm quickly fell back the seabed and did not re-enter suspension.

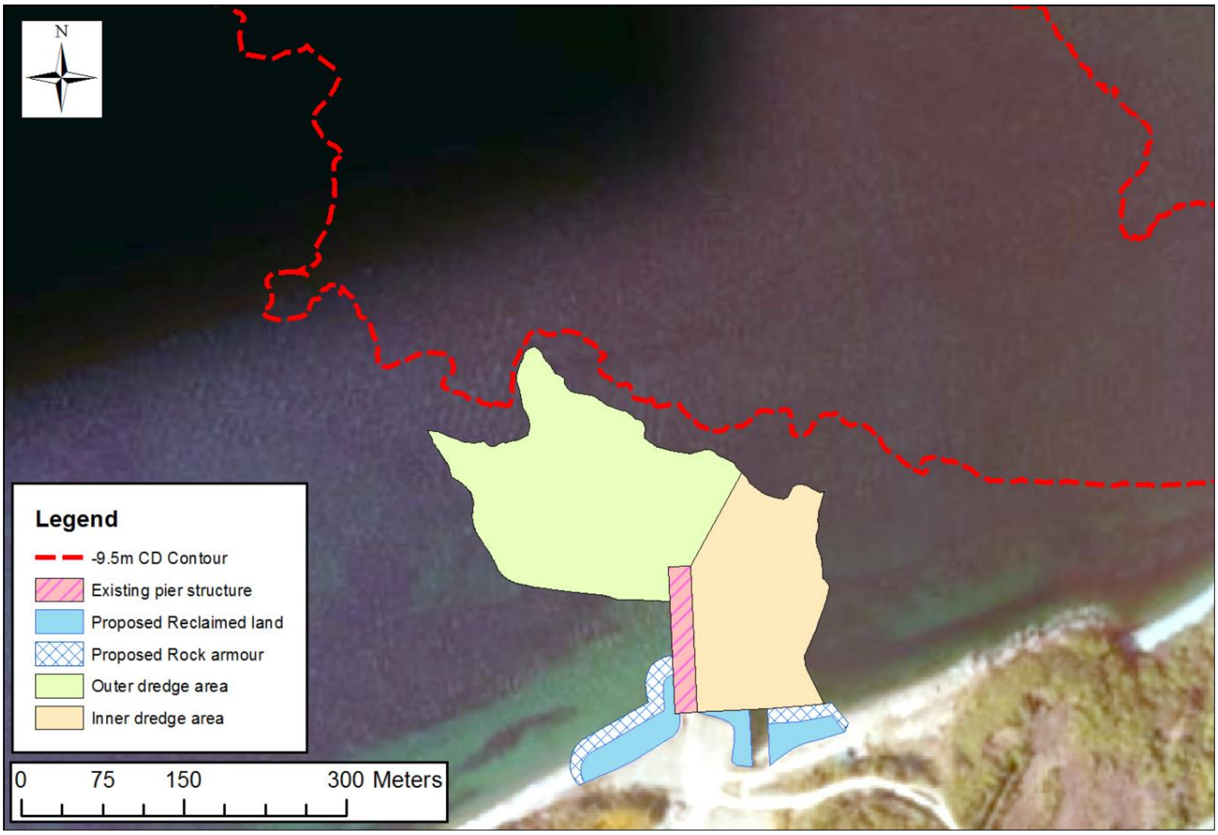


Figure 8.3: Location and extent of the proposed dredge area in relation to -9.5m CD contour.

Based on the PSA of the sediment samples retrieved during the geotechnical survey detailed in Section 3.2 it was found that approximately 42.5% and 60% of the bed material in the outer and inner dredge areas respectively was comprised of sandy material (1000 - 63µm). Furthermore, analyses indicated that *c.* 3.5% and 20% of the total dredge volume in the outer and inner dredge areas respectively was comprised of silty material (<63µm). The composition of the bed material in both dredge areas is summarised in Table 8.1.

The dispersion of the silt and sand fractions were modelled in the numerical simulations based on the physical characteristics of the six sediment classes detailed in Table 8.2 below.

Table 8.1: Composition of bed material in the inner and outer dredge areas at Kyleakin.

Dredge area	Capital Dredge Requirements [m^3]	% material <1000um	% Sand material (1000 – 63 µm)	% Silt material (<63 µm)
Outer	85,500	46	42.5	3.5
Inner	104,500	80	60	20
Total	**190,000**			

Table 8.2: Modelled sediment characteristics.

General Classification	Class	Mean Particle Diameter [um]
Sands	1	1000
	2	500
	3	125
Silts	4	63
	5	44
	6	22

8.1.3 Modelled dredging techniques

The type of dredging equipment that can be used for any project is defined by the type of soil to be dredged and environmental constraints amongst other factors. After consultation with dredging experts it was determined that given the considerable volume of coarse sand and gravel material to be dredged together with the close proximity of the flame shell beds, it was highly likely the capital dredging requirements would be undertaken using the following or a combination of the following dredging techniques:

- Trailer Suction Hopper Dredge (TSHD);
- Backhoe Dredger (BHD);
- Cutter Suction Dredger (CSD).

To allow for greater flexibility for prospective dredging contractors, this study has investigated the dispersion and fate of the sediment plumes created through the use of all three types of equipment listed above.

Based on the capital dredge requirements of both the outer and inner dredge areas, together with typical production values associated with each dredging technique it is anticipated that the duration of the total capital dredging programme could range between 30 and 78 days. This has been summarised in Table 8.3 below.

Table 8.3: Possible combinations of dredging methods and corresponding durations to undertake the capital dredging in the Inner and Outer areas.

Approach	Dredging Method		Dredging Duration [Days]		
	Outer area	Inner area	Outer area	Inner area	Total Duration
1	TSHD (without overspill)		25	50	75
2	TSHD	BHD	18	60	78
3	CSD		10	20	30

The corresponding source terms for each of the dredging methods summarised in Table 8.3 have been described in full detail in Sections 8.2 to 8.4. The source terms described in Sections 8.2 to 8.4 vary based primarily on the dredge equipment, however each dredger will follow the same path in the inner and outer dredge areas; these paths are illustrated overleaf in Figure 8.4

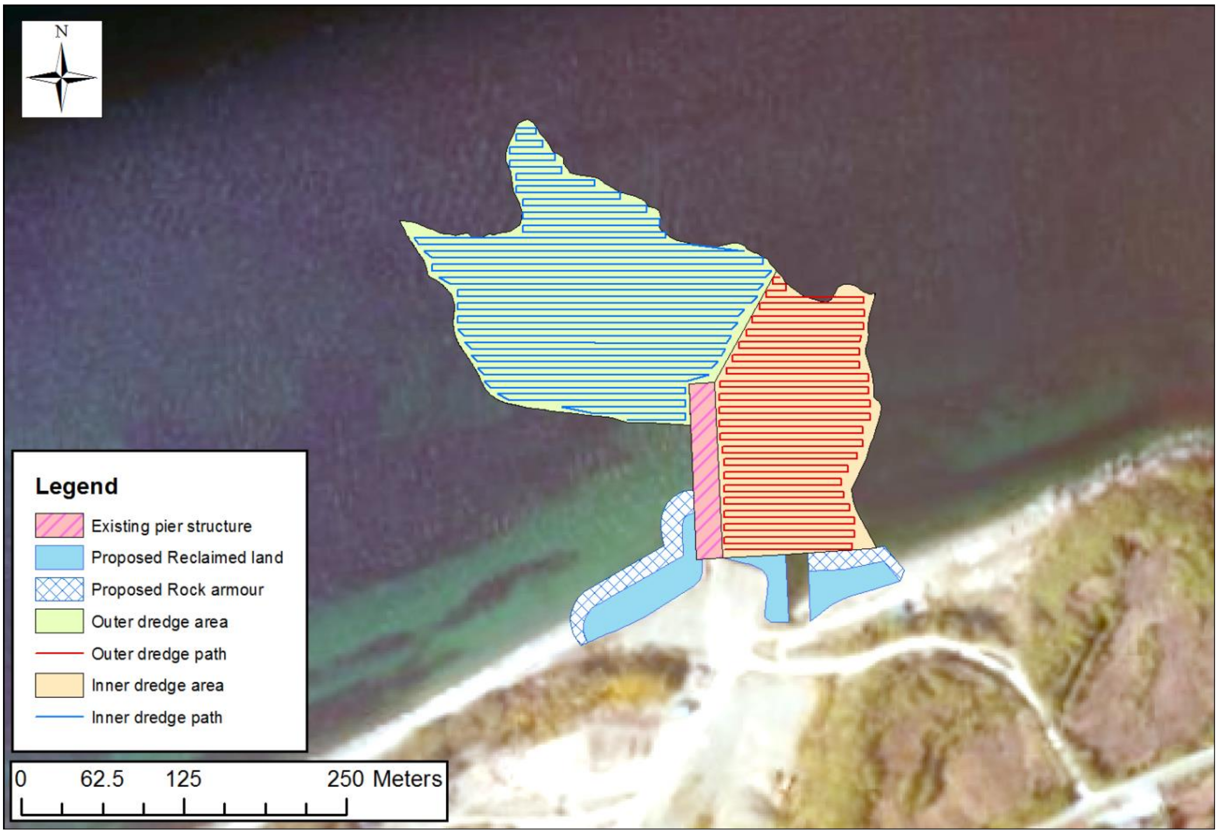


Figure 8.4: Anticipated dredge paths of the inner and outer dredge areas.

8.2 DREDGING APPROACH 1: TSHD (WITHOUT OVERSPILL) EQUIPMENT

8.2.1 Assumptions

In this scenario it has been assumed that a Trailer Suction Hopper Dredger will pump the dredged material into the hopper without overflow until filled. Once fully laden, the TSHD will sail to the existing quay wall and pump the material to the quarry area. The overspill from the stilling ponds in the quarry area will be discharged through the reclamation works to an area just west of the existing structure.

The TSDH is expected to dredge at full capacity for 30minutes, whilst unloading and sailing times are expected to account for another 60 minutes, resulting in a total cycle time of 90 minutes which will be repeated 24/7. Based on this information it expected to take 25 and 50 days to dredge the outer and inner areas respectively.

8.2.2 Source term analysis

The dredging undertaken by the TSHD without overspill can be represented by two source terms: the loss of material from the drag head and the overspill of material from the stilling pond into the placement area.

The loss of material at the drag head was taken as 3% of the total sand and silt material in the inner and outer areas whilst the overspill material at the placement area was taken as 10% of the total silt material in both areas. These source term fractions are in agreement with those presented by Becker *et al*. 2015 and are summarised in Table 8.4 below.

Table 8.4: Source terms and fractions for the TSHD with no overspill in the inner and outer dredge areas.

Area	Dredge Equipment	Source	Fraction
Inner	TSHD (no overspill)	Head loss	3% of Sand and Silt
		Placement overspill	10% of Silts
Outer	TSHD (no overspill)	Head loss	3% of Sand and Silt
		Placement overspill	10% of Silts

8.2.3 Numerical representation

Using the source terms summarised in Table 8.4 to represent the input of sand and silt material into the marine environment, the sediment plume simulations were run over the course of a 15 day period which included a full range of spring and neap tidal conditions, this 15 day period relative to the tidal cycle has been illustrated in Figure 8.5. The results of the model simulations were then scaled up to represent the full 75 day dredging campaign across the inner and outer dredge areas.

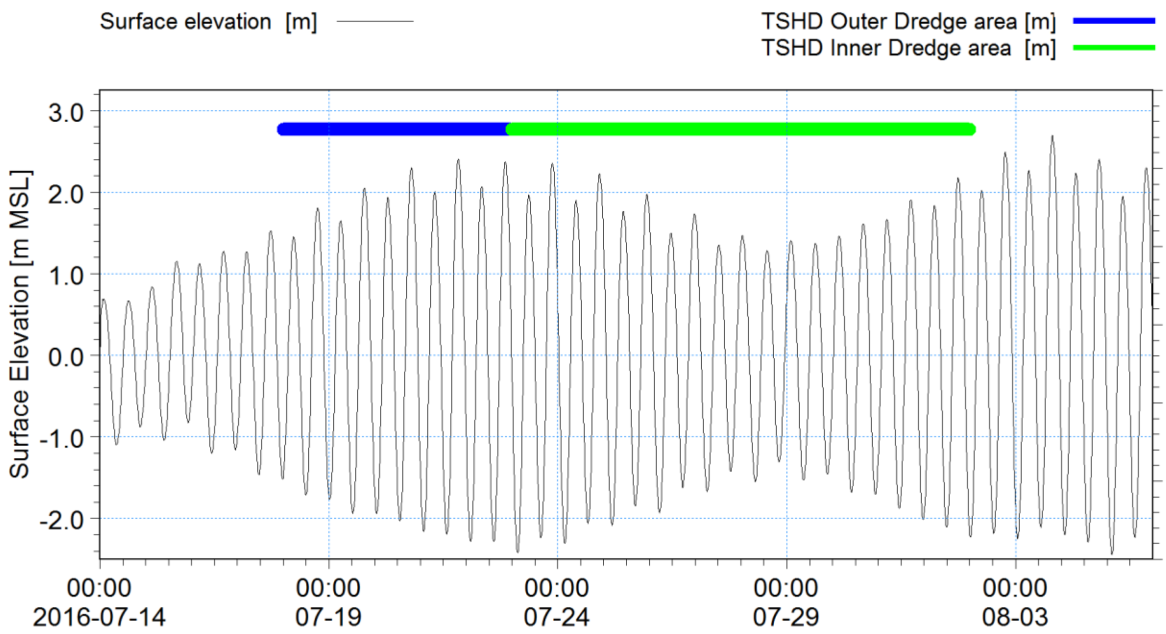


Figure 8.5: The simulated 15 day TSHD dredger programme in relation to the tidal cycle at Kyleakin Pier.

To account for the possible effect of wind driven currents on the dispersion of the dredge material, a force 3 southerly wind was applied throughout the entire domain for the duration of the dredging programme.

To accurately reflect the 90 minute dredging cycle, the head and placement source terms were alternatively switched on and off throughout the entire duration of the dredge campaign.

Given that the capital dredging programme will be undertaken prior to the construction of the new 79m concrete caisson, the coupled hydrodynamic and sediment transport simulations were run using the existing Kyleakin Pier model domain. This model has been described in more detail in Section 4.2.3 and is illustrated in Figure 4.4.

8.2.4 Dredging Approach 1: Simulation results

As the sediment plume created during the course of the dredging programme would be greatest when dredging the inner area due to the higher fraction of fine material, the total increase in suspended sediment concentrations (SSCs) as a result of using the TSHD in this area has been illustrated in Figure 8.6. This Figure illustrates the increase in SSCs at spring peak flood, high water, peak ebb and low water tidal conditions when the TSHD is nearest to the -9.5m CD contour.

It will be seen from this figure that under typical conditions the increase in SSCs due to the losses at the TSHD head do not generally exceed 0.03kg/m^3 (or 30mg/L). Furthermore, the plume envelope does not extend beyond the immediate vicinity of the dredge area (illustrated by the black polygon in Figure 8.6) except for a short period either side of low water.

Figure 8.6 also demonstrates how the overspill of material from the stilling pond to an area just west of the existing Pier quickly disperses to result in a very small increase in SSCs generally <0.03kg/m^3. This minimal change is generally contained within *c*. 10m radius close to the nearshore area.

The average increase in SSCs as a result of a TSHD undertaking the capital dredging work across the inner and outer dredge areas in 75 days is illustrated in Figure 8.6. It will be seen from this Figure that aside from the minimal increase of SSCs in the placement area there are virtually no changes in SSCs beyond the overall dredge area.

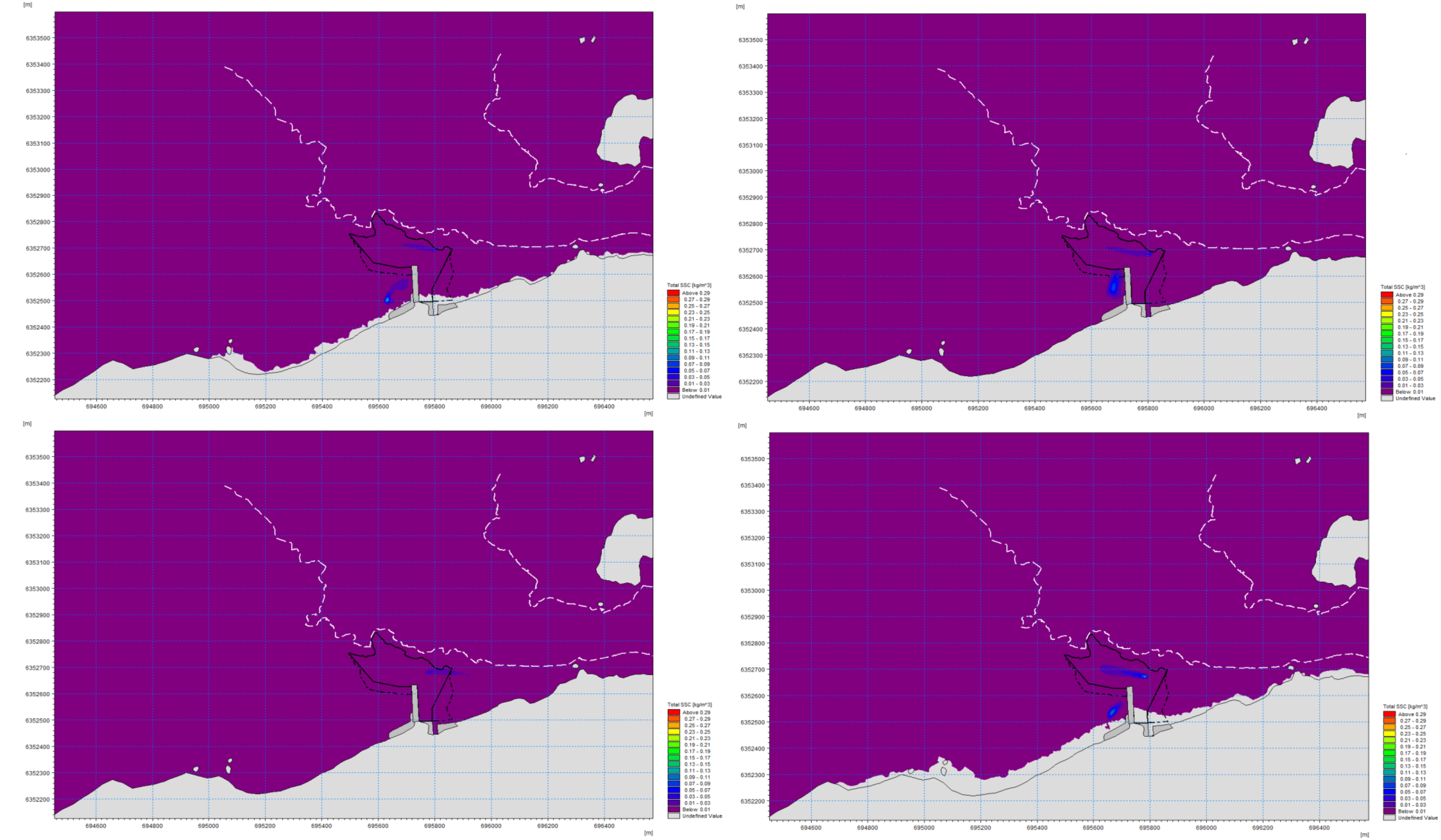


Figure 8.6: The increase in total SSCs created by a TSHD in the inner dredge area at Peak flood (top left), High Water (top right), Peak Ebb (bottom left) and Low Water (bottom right) spring tidal regimes.

The deposition of material <1000µm at the end of the 75 day TSHD dredging campaign is illustrated in Figure 8.8. It will be seen that the deposition of material is strongly influenced by the residual tidal current regime (see Figure 5.4) which acts to transport material in a westerly direction. Beyond the nearshore area proximal to the development, depositions levels can reach *c.*0.10m within 800m to the west of the Pier and fall to < 0.01m within another 800m.

It is important to note that it is common practice for dredging contractors to account for the effect of sediment deposition during the dredging programme by making very minor adjustments to the final target dredge depth. As such, only material beyond the dredge extent should be considered when assessing sediment plume deposition levels.

8.2.5 Summary of the TSHD w/o overspill dredging campaign

Based on the modelling results presented in Section 8.2 it was found that the increase in SSCs as a result of losses from the TSHD under typical tidal conditions would not generally exceed 0.03kg/m^3 outside of the dredge area. The overspill of material from the stilling pond into the placement area to the west of the Pier was found to result in a minor increase in SSCs however this change was generally contained within *c*. 10m radius close to the nearshore area.

The TSHD modelling results demonstrated how the residual tidal currents transported the suspended sediments in a westerly direction to result in depositions levels of *c.*0.10m within 800m to the west of the Pier which then decreased to < 0.01m within another 800m.

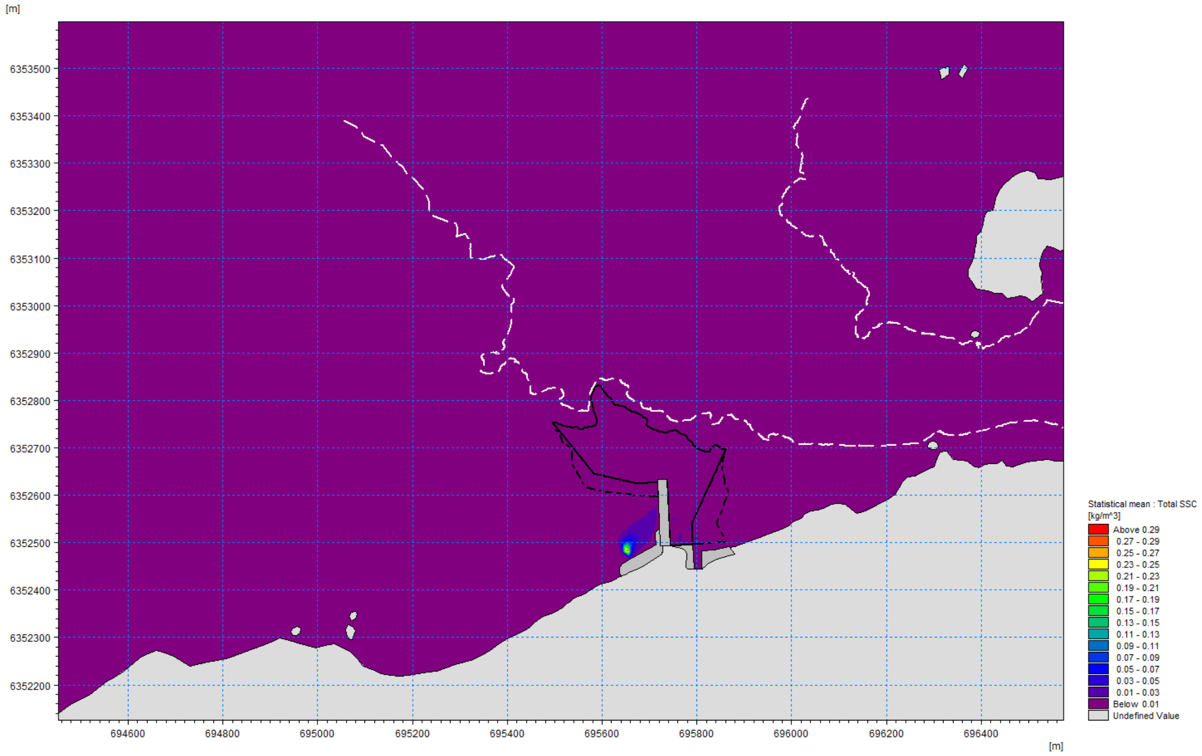


Figure 8.7: The mean total SSCs created by the TSHD during the entire capital dredging programme.

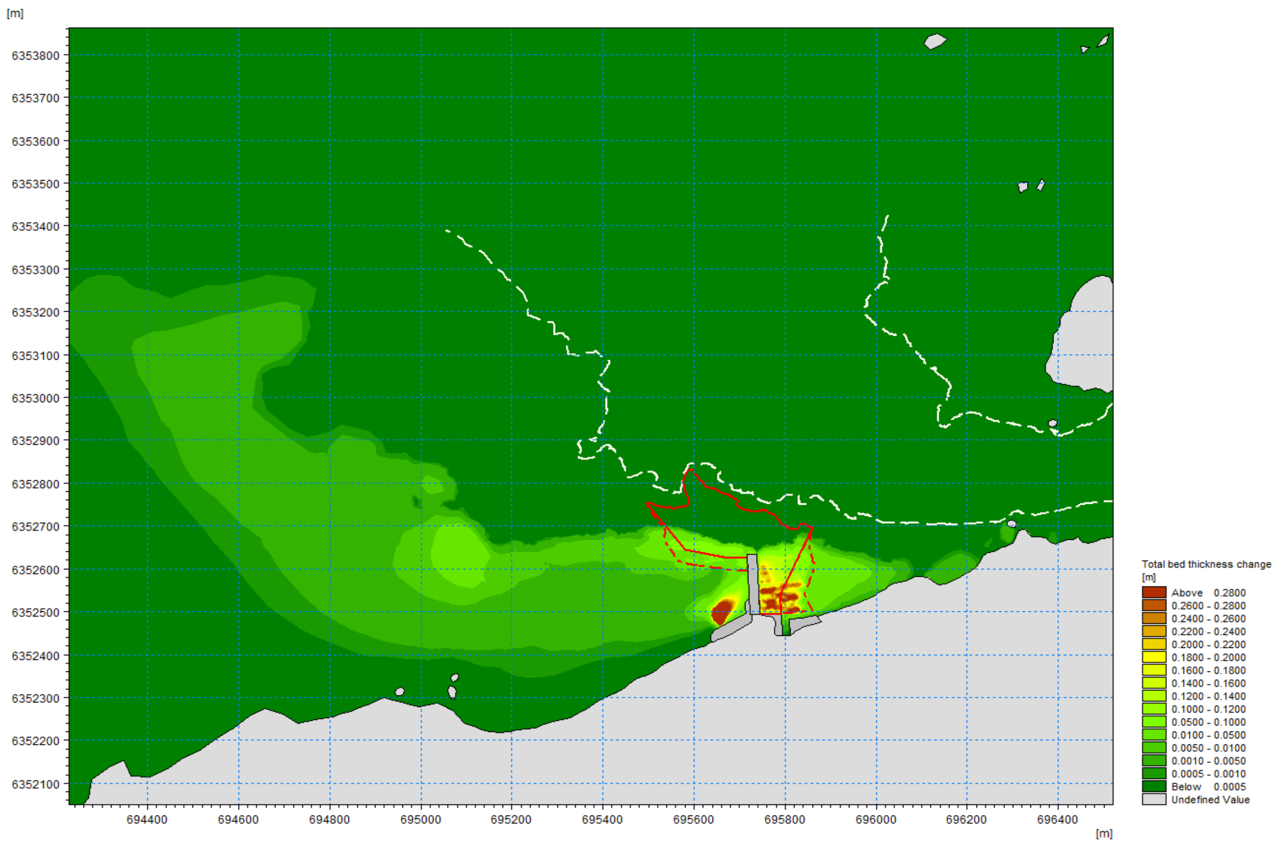


Figure 8.8: Deposition levels at the end of the 75day TSHD dredging campaign.

8.3 DREDGING APPRAOCH 2: TSHD AND BHD EQUIPMENT

8.3.1 Assumptions

In this scenario it has been assumed that in the outer dredge area a Trailer Suction Hopper Dredger will pump dredged material into the ship's hopper with overflow until filled. The fully laden TSHD will then sail to the existing quay wall where the material will be off-loaded before being transported to the quarry area.

In the inner dredge area, a Backhoe Dredger (BHD) will excavate the dredge material using a hydraulically operated bucket that will be used to transfer the dredge material into a barge. The barge will take the material to the quay wall where it will be off-loaded and transported to quarry area.

Both pieces of dredge equipment are expected to operate on a 24/7 until the dredging has been completed. Based on this assumption the TSHB is expected to take 18 days to complete the dredging of the outer area whilst the BHD is expected to take 60 days to complete the dredging of the inner area.

8.3.2 Source term analysis

The material introduced into the marine environment as a result of TSHD dredging can be represented by two source terms: the loss of material from the drag head and the overspill of material from the hopper of the dredger. The losses at the TSHD drag head was taken as 3% of the sand and silt material in the outer dredge area whilst the overspill from the hopper was taken as 20% of the silt material.

The source term for the BHD has been taken as 3% of the total sand and silt material. This source was represented by introducing half of this quantity in the bottom layer of the numerical model and the other half in the top layer of the numerical model.

The source terms for the TSHD (with overspill) in the outer area and the BHD in the inner dredge area a have been summarised in Table 8.5 below.

Table 8.5: Source terms and fractions for the BHD the TSHD with overspill in the inner and outer dredge areas respectively.

Area	Dredge Equipment	Source	Fraction
Inner	BHD	Losses - bottom	1.5% of Sand and Silt
		Losses - top	1.5% of Sand and Silt
Outer	TSHD (with overspill)	Head loss	3% of Sand and Silt
		Placement overspill	20% of Silts

8.3.3 Numerical representation

Using the source terms summarised in Table 8.5 to represent the input of sand and silt material into the marine environment, the sediment plume simulations were run over the course a 13 day period which included a full range of spring and neap tidal conditions, this 13 day period relative to the tidal cycle has been illustrated in Figure 8.6. The results of the model simulations were then scaled up to represent the full 78 day dredging campaign across the inner and outer dredge areas.

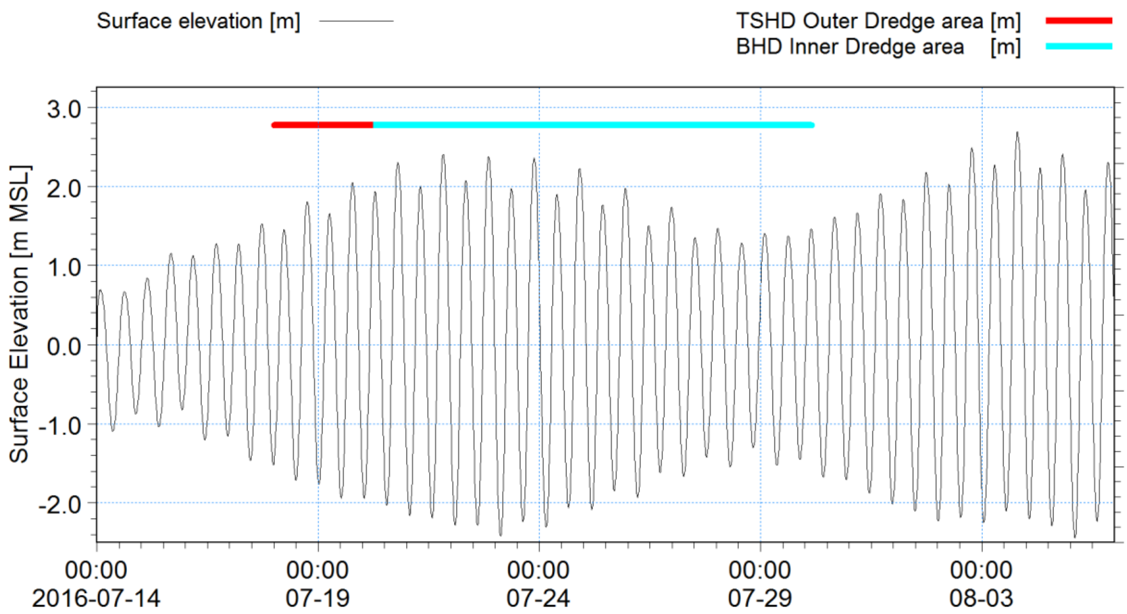


Figure 8.9: The simulated 13 day TSHD and BHD dredger programme in relation to the tidal cycle at Kyleakin Pier.

To account for the possible effect of wind driven currents on the dispersion of the dredge material, a force 3 southerly wind was applied throughout the entire domain for the duration of the dredging programme.

Given that the capital dredging programme will be undertaken prior to the construction of the new 79m concrete caissons, the coupled hydrodynamic and sediment transport simulations were run using the existing Kyleakin Pier model domain. This model has been described in more detail in Section 4.2.3 and is illustrated in Figure 4.4.

8.3.4 Dredging Approach 2: Simulation results

As the sediment plume created during the course of the dredging programme would be greatest when dredging the inner area due to the higher fraction of fine material, the total increase in suspended sediment concentrations (SSCs) as a result of using the BHD in this area has been illustrated in Figure 8.10. This Figure illustrates the increase in SSCs at spring peak flood, high water, peak ebb and low water tidal conditions when the BHD is nearest to the -9.5m CD contour.

It will be seen from this figure that under typical conditions a small increase in SSCs of <0.07kg/m^3 (or <70mg/L) is contained within the extent of the proposed dredge area.

The average increase in SSCs over the course of the entire 78 day dredging campaign is illustrated in Figure 8.11. It will be seen from this figure that due to the lower production and spill rates associated the BHD, the mean increase in SSCs does not exceed 0.01kg/m^3 except in a small area near the proposed slipway where the increase in SSCs do not exceed 0.03kg/m^3. The mean increase in SSCs beyond the immediate dredge area does not exceed 0.01kg/m^3 (or 10mg/L).

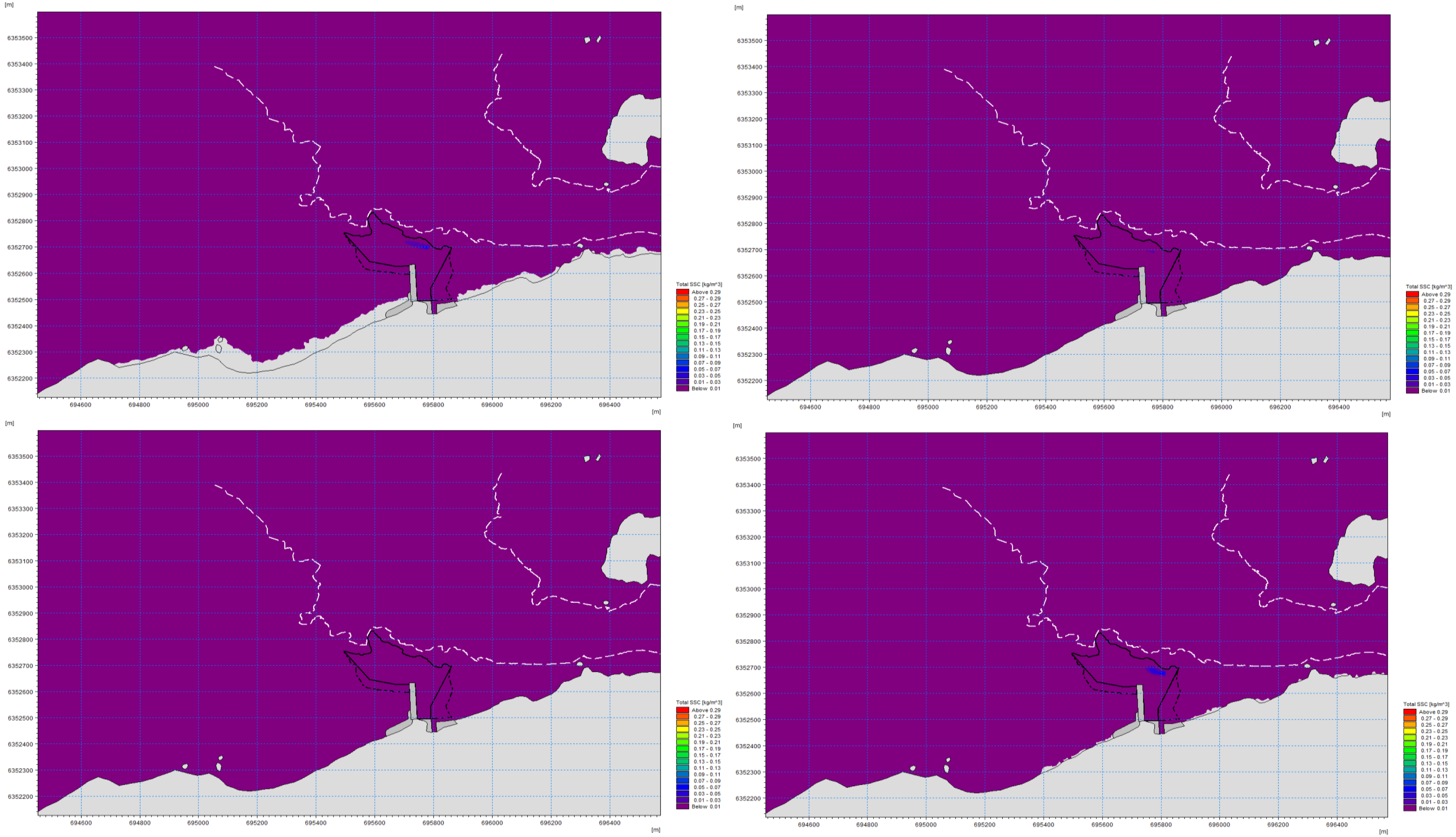


Figure 8.10: The increase in total SSCs created by a BHD in the inner dredge area at Peak flood (top left), High Water (top right), Peak Ebb (bottom left) and Low Water (bottom right) spring tidal regimes.

The deposition of material <1000µm at the end of the 78 day combined TSHD and BHD dredging campaign is illustrated in Figure 8.16. It will be seen that the deposition of material is strongly influenced by the residual tidal current regime (see Figure 5.4) which acts to transport material in a westerly direction. Beyond the nearshore area proximal to the development, depositions levels can reach *c.*0.10m within 800m to the west of the Pier and fall to < 0.01m within another 800m.

It is important to note that it is common practice for dredging contractors to account for the effect of sediment deposition during the dredging programme by making very minor adjustments to the final target dredge depth. As such, only material beyond the dredge extent should be considered when assessing sediment plume deposition levels.

8.3.5 Summary of the TSHD and BHD dredging campaign

Based on the modelling results presented in Section 8.3 it was found that the increase in SSCs as a result of losses from the BHD under typical tidal under typical conditions a minimal increase in SSCs of <0.07kg/m^3 (or <70mg/L) is contained within the extent of the proposed dredge area. Furthermore, results demonstrated that mean increase in SSCs beyond the immediate dredge area did not exceed 0.01kg/m^3 (or 10mg/L).

The TSHD and BHD modelling results demonstrated how the residual tidal currents transported the suspended sediments in a westerly direction to result in depositions levels of *c.*0.10m within 800m to the west of the Pier which then decreased to < 0.01m within another 800m.

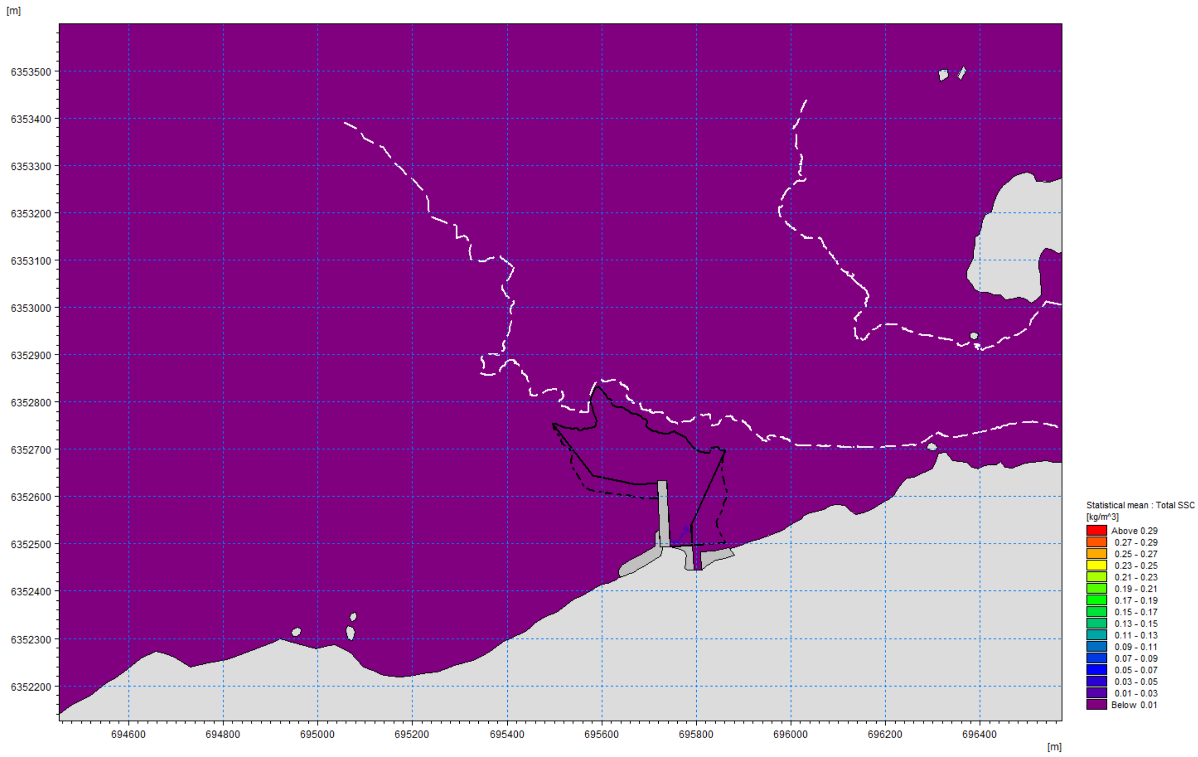


Figure 8.11: The mean total SSCs created by the TSHD & BHD during the entire capital dredging programme.

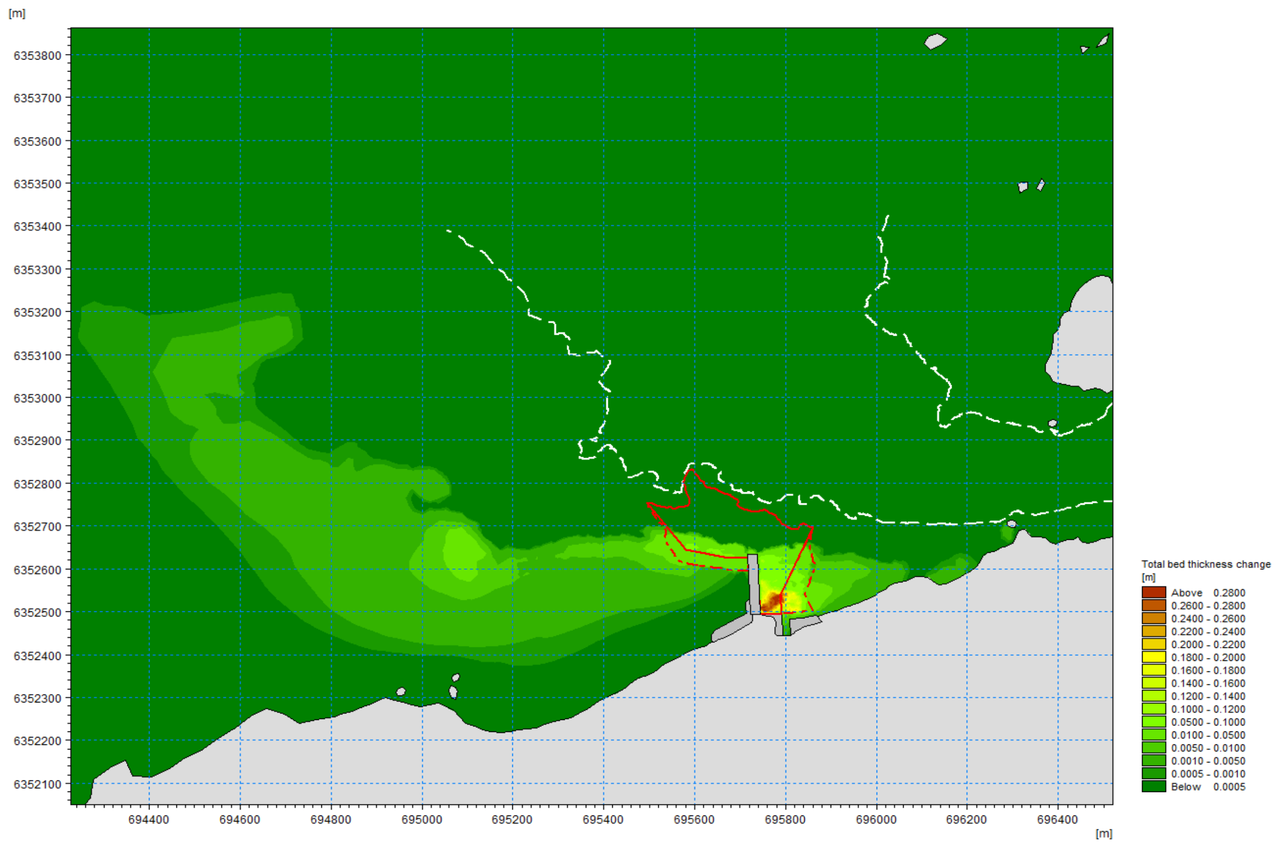


Figure 8.12: Deposition levels at the end of the 78 day TSHD and BHD dredging campaign.

8.4 DREDGING APPROACH 3: CSD EQUIPMENT

8.4.1 Assumptions

For this final scenario it has been assumed that Cutter Suction Dredger (CSD) will be used to remove material from both the inner and outer dredge areas. The CSD will dislodge material using a rotating cutter equipped and then pump the mixture of dredge material and water to the quarry area by means of a discharge pipeline. The overflow from the stilling ponds will be discharged into the nearshore area just west of the existing Pier.

The CSD is expected to operate at full capacity on a 24/7 basis in both of the dredge areas. Based on this information it expected to take 10 days and 20 days to dredge the inner and outer areas respectively.

8.4.2 Source term analysis

The material introduced into the marine environment as a result of CSD dredging operations can be represented by two source terms: the loss of material from the drag head and the overspill of material from the stilling pond into the placement area.

The losses at the CSD drag head were taken as 5% of the sand and silt material in the inner and outer areas whilst the overspill at the placement area was taken as 10% of the silt material in both areas.

Table 8.6: Source terms and fractions for the CSD in the inner and outer dredge areas.

Area	Dredge Equipment	Source	Fraction
Inner	CSD	Head loss	5% of Sand and Silt
		Placement overspill	10% of Silts
Outer	CSD	Head loss	5% of Sand and Silt
		Placement overspill	10% of Silts

8.4.3 Numerical representation

Using the source terms summarised in Table 8.6 to represent the input of sand and silt material into the marine environment, the sediment plume simulations were run over the course of a 15 day period which included a full range of spring and neap tidal conditions, this 15 day period relative to the tidal cycle has been illustrated in Figure 8.5. The results of the model simulations were then scaled up to represent the full 30 day dredging campaign across the inner and outer dredge areas.

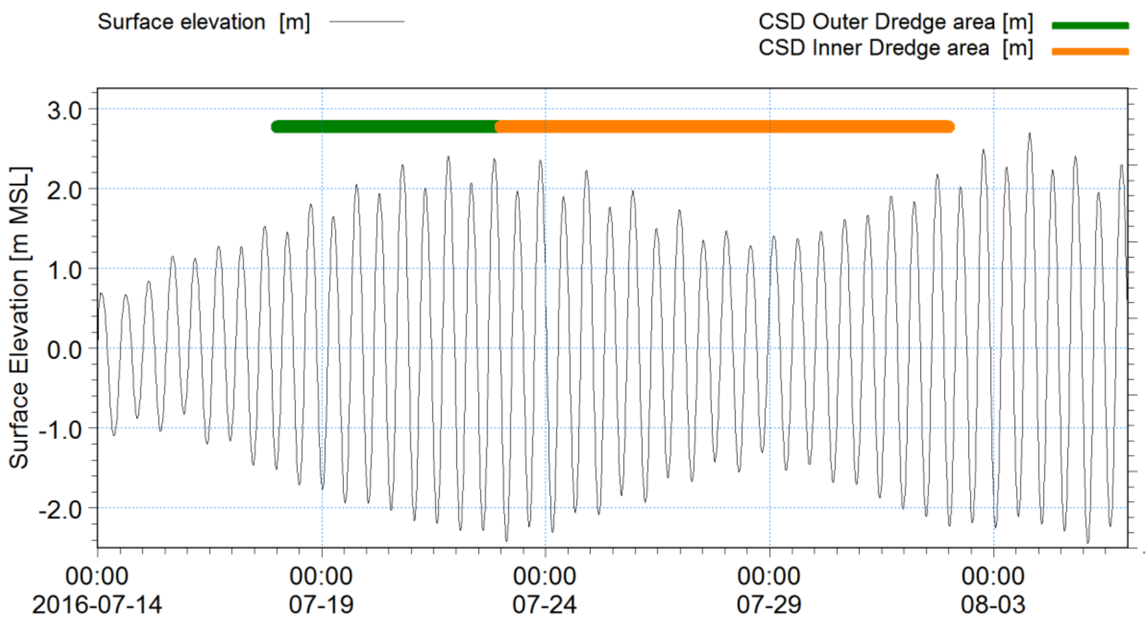


Figure 8.13: The simulated 15 day CSD dredger programme in relation to the tidal cycle at Kyleakin Pier.

To account for the possible effect of wind driven currents on the dispersion of the dredge material, a force 3 southerly wind was applied throughout the entire domain for the duration of the dredging programme.

Given that the capital dredging programme will be undertaken prior to the construction of the new 79m concrete caisson, the coupled hydrodynamic and sediment transport simulations were run using the existing Kyleakin Pier model domain. This model has been described in more detail in Section 4.2.3 and is illustrated in Figure 4.4.

8.4.4 Dredging Approach 3: Simulation results

As the sediment plume created during the course of the dredging programme would be greatest when dredging the inner area due to the higher fraction of fine material, the total increase in suspended sediment concentrations (SSCs) as a result of using the CSD in this area has been illustrated in Figure 8.14. This Figure illustrates the increase in SSCs at spring peak flood, high water, peak ebb and low water tidal conditions when the CSD is nearest to the -9.5m CD contour.

It will be seen from this figure that the increase in SSCs due to the losses at the CSD head can reach 0.07kg/m^3 600m to the west of the dredge area. Under normal tidal conditions, this change decreases to <0.05kg/m^3 within another 200m to the west of the immediate dredge extent. Under normal tidal conditions it will be seen that there is generally no increase in SSCs beyond the -9.5m contour.

Figure 8.14 demonstrates how the overspill of material from the stilling pond to an area just west of the existing Pier is generally dispersed in an easterly direction due to the nearshore currents. The change in SSCs as a result of this source term is confined within the nearshore area proximal to the development.

The average increase in SSCs as a result of a CSD undertaking the capital dredging work across the inner and outer dredge areas in 30 days is illustrated in Figure 8.15. It will be seen from this Figure that aside from the minimal increase of SSCs in the placement area there are virtually no changes in SSCs beyond the overall dredge area.

The deposition of material <1000µm at the end of the 30 day CSD dredging campaign is illustrated in Figure 8.16. Like the other dredging scenarios it will be seen that the deposition of material is strongly influenced by the residual tidal current regime which acts to transport material in a westerly direction. Beyond the nearshore area proximal to the development, depositions levels can reach *c.*0.10m within 800m to the west of the Pier and fall to < 0.01m within another 800m.

It is important to note that it is common practice for dredging contractors to account for the effect of sediment deposition during the dredging programme by making very minor adjustments to the final target dredge depth. As such, only material beyond the dredge extent should be considered when assessing sediment plume deposition levels.

8.4.5 Summary of the CSD dredging campaign

Based on the modelling results presented in Section 8.4 it was found that the increase in SSCs as a result of losses from the CSD under typical tidal conditions would not generally exceed 0.07kg/m^3. This change can be observed up to 600m to the west of the dredge area and is seen to decrease to <0.05kg/m^3 within another 200m to the west

It was found that the overspill material from the spilling pond into the placement area dispersed in an easterly direction to ultimately end up within the dredge area.

The CSD modelling results demonstrated how the residual tidal currents transported the suspended sediments in a westerly direction to result in depositions levels of *c.*0.10m within 800m to the west of the Pier which then decreased to < 0.01m within another 800m.

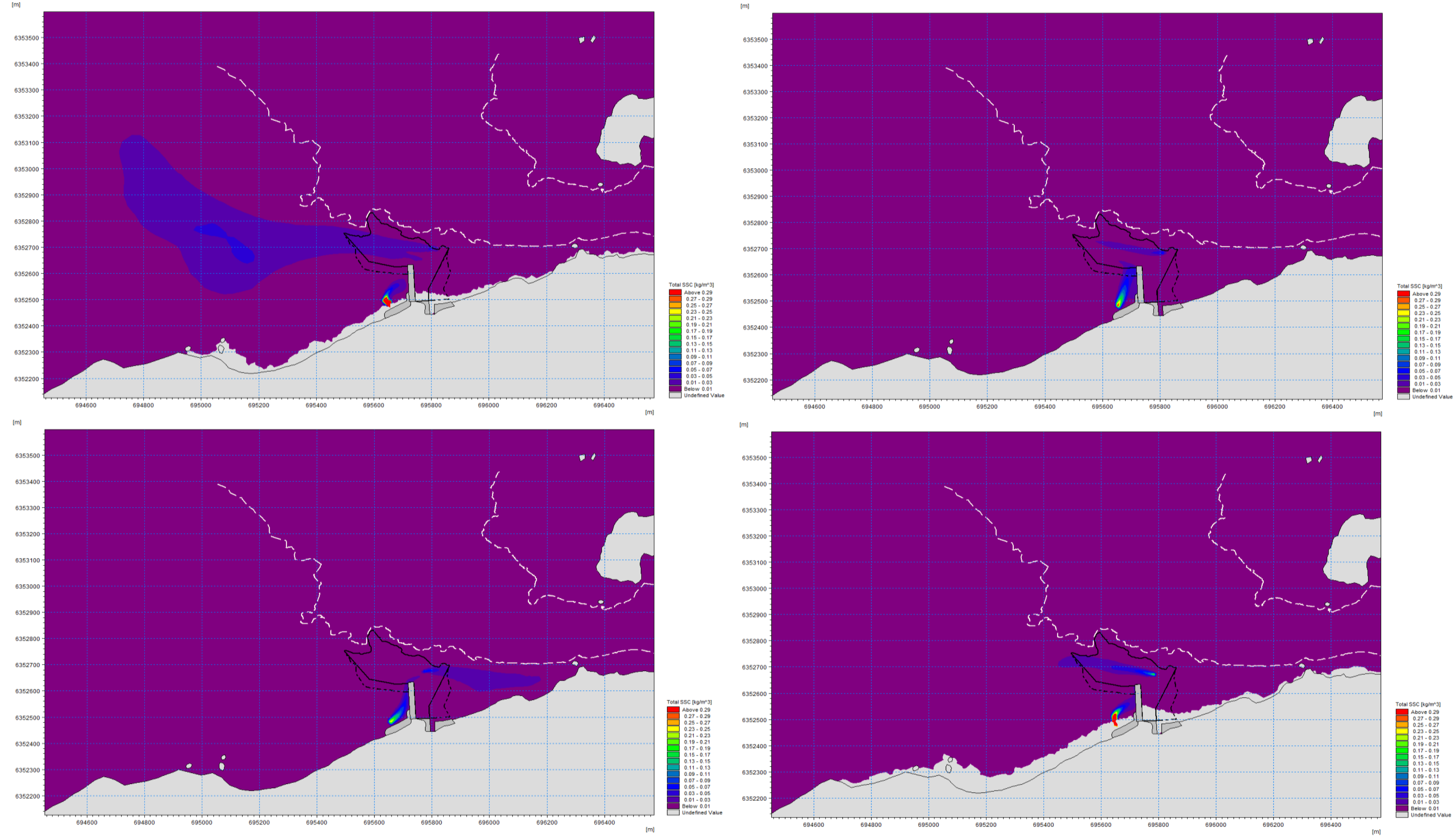


Figure 8.14: Total SSCs created by a CSD in the inner dredge area at Peak flood (top left), High Water (top right), Peak Ebb (bottom left) and Low Water (bottom right) spring tidal regimes.

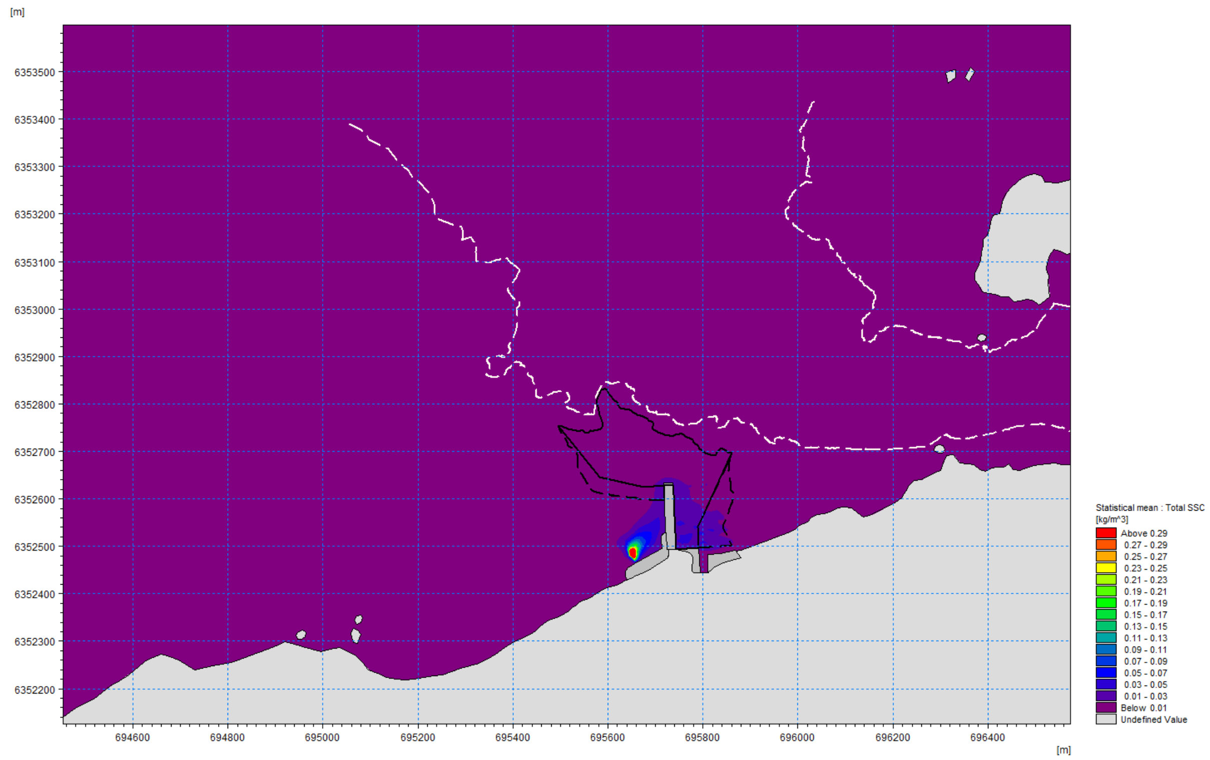


Figure 8.15: The mean total SSCs created by the CSD during the entire capital dredging programme.

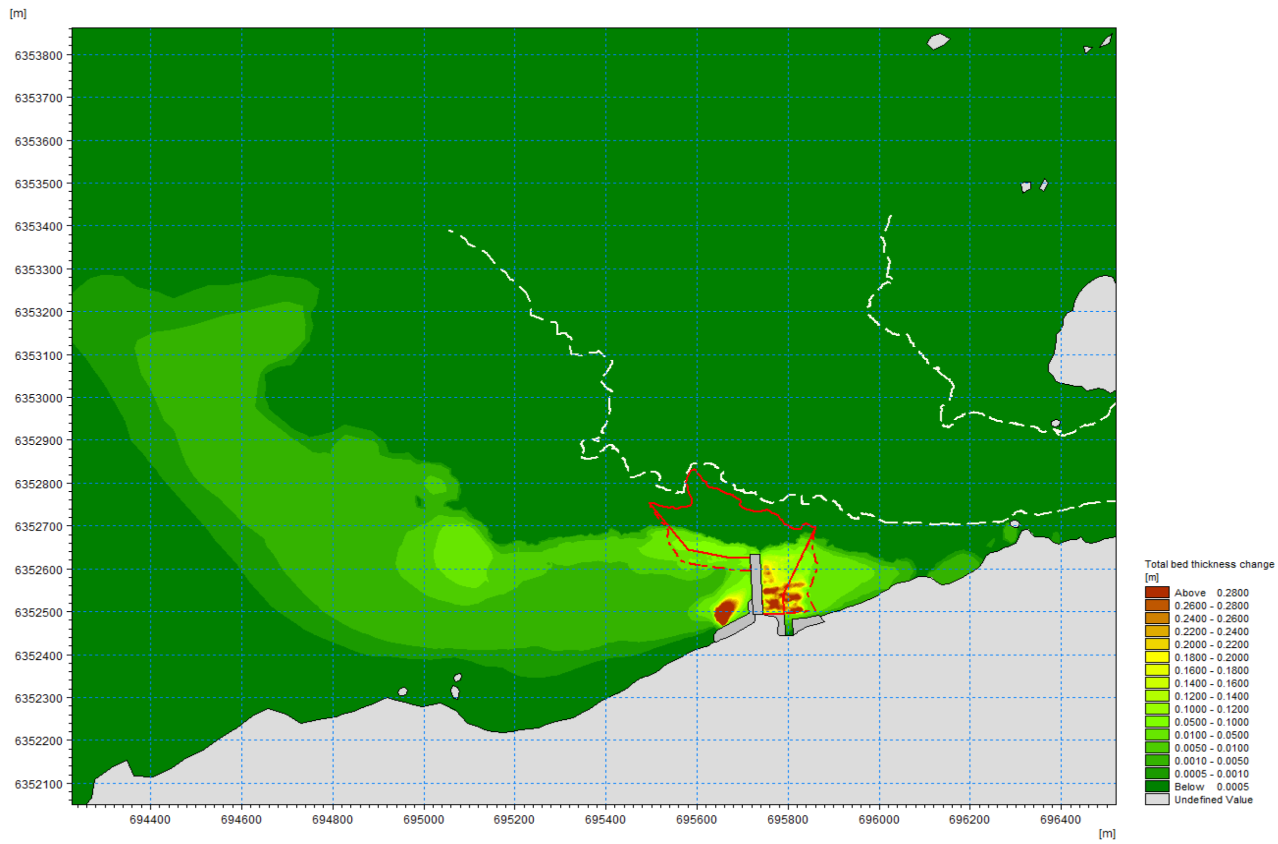


Figure 8.16: Deposition levels at the end of the 30 day CSD dredging campaign.

8.5 FUTURE MAINTENANCE DREDGING REQUIREMENTS

The normal suspended sediment loading in the waters around the Kyleakin Pier is very low and the as outlined in Section 7 the prominent feature of the sediment transport regime is a movement along the shoreline from south west to east. The existing Pier at Kyleakin currently acts like a groyne and intercepts the majority of the longshore transport which approaches the Pier from the south west.

The proposed 79m caisson structure at the end of the existing Pier will enhance the performance of the existing Pier in retaining sediment on the western side of the Pier. The storm wave induced circulation to the west of the Pier shown in Figure 7.6 in Section 7 serves to retain sediment in the lee of the caisson structure. The west going tide also tends to produce an eddy on the western side of the Pier which will serve to retain sediment in this area.

Although there is not a strong storm wave induced eddy on the eastern side of the Pier the east going tide does induce a local west going flow near the shoreline on the eastern side of the Pier. Fortunately the amount of available sediment on the eastern side of the Pier is relatively small as the site investigations indicate that the bed sediments in this area are gravels which are stable under the existing hydraulic conditions.

Thus it is expected that maintenance dredging requirements will be relatively small and that rapid infilling of the dredged area under storm conditions is unlikely to occur. Nevertheless it is expected that some sedimentation will occur particularly in the corners of the dredged area as a results of ship movements acting on fine material which is deposited in the dredged area. It is anticipated that maintenance dredging can be undertaken using an excavator mounted on the quay in combination with some local ploughing within the dredged basin area.

9 SUMMARY AND CONCLUSIONS

Marine Harvest Scotland Ltd plan to develop an existing Pier site located immediately to the west of Kyleakin. The proposed scheme includes developing the existing Pier to create a 160m long berth and using a series of concrete caissons to create a smaller 79m long berth at the end of the existing Pier. The proposed development also includes the capital dredging of *c*. 190,000m^3 of material to achieve a minimum operating depth of -8.5m CD.

RPS was commissioned to undertake an extensive modelling programme to evaluate the effect of the proposed development on the existing coastal processes at Kyleakin. This included analysing and assessing any changes to the existing flow regime, wave climate and sediment transport regime. The computational modelling was undertaken using RPS' in house suite of MIKE coastal process modelling software by the Danish Hydraulic Institute (DHI). A summary of the study findings have been presented in Sections 9.1 to 9.5.

9.1 EFFECT OF THE PROPOSED DEVELOPMENT ON THE TIDAL REGIME

It was found that under typical tidal conditions the proposed Pier configuration will only result in minimal changes to the tidal regime. The greatest changes in velocities of ±0.10m/s were observed during peak-flood and peak-ebb flows and were generally limited to within the nearshore area proximal to the development. Similar changes to the residual tidal and residual littoral current regimes were also observed.

Therefore based on the results of the numerical simulations it can be concluded that the proposed Pier structure will not result in any major changes to the existing tidal or littoral current regime beyond the nearshore area proximal to the development.

9.2 EFFECT OF THE PROPOSED DEVELOPMENT ON THE WAVE CLIMATE

An assessment of the wave climate at Kyleakin Pier using the Spectral Wave model demonstrated that under 1 in 1 year storm conditions, the greatest change to the wave climate as a result of the proposed development is observed during events with wind blowing from 240^o. During these particularly events significant wave heights are decreased by up to 0.65m on the lee side of the existing Pier structure. For 1 in 1 year events originating in other relevant sectors (300^o, 000^o and 45^o) changes to the wave climate as a result of the proposed development did not generally exceed ±0.25m.

9.3 EFFECT OF THE PROPOSED DEVELOPMENT ON SEDIMENT TRANSPORT

Results from the hydrographic and geotechnical field investigations indicated that the sea bed to the north of Kyleakin Pier is characterised by cobbles overlying coarse gravel material. Given that there is a very limited supply of beach material on the coast to the east of the Pier, only the larger waves in the overall wave climate generate littoral currents of sufficient strength to produce significant longshore sediment drift. As such the only sediment drift at Kyleakin Pier will be along the western shoreline from a south westerly direction.

Based on the results of sediment transport simulations undertaken to assess the stability of the seabed under 1 in 1 year conditions it was found that the bed level changes caused by the proposed development would be contained within the upper surf zone of the site where waves would be breaking. The proposed development was found to result in only minimal bed level changes beyond the immediate vicinity of the works

As the outer area of the seabed at Kyleakin is characterised by cobbles overlying coarse gravel material, propeller wash from ships using the proposed Pier at Kyleakin will not result in any measureable bed erosion seaward of the -8.5m CD contour. However, along the outer section of the 79m quay, propeller induced scour is expected to mobilise sediment and will therefore require scour protection to be installed. Scour protection may also be required along the inner length of the 160m quay if it is found that high thruster use is required for navigational purposes.

Calculations undertaken to determine the stability of the proposed side slopes indicated that for a fine sand bed material the 1 in 7 dredged slopes to the east of the Pier would not be stable under storm conditions. However given that the proposed capital dredging regime would produce a very considerable amount of gravel and cobble material which was found to be stable under the same conditions, it is proposed to use this material to provide a protective cover layer across the 1 in 7 dredged side slopes.

A similar analysis demonstrated that on the western side of the Pier, where the bed material is composed of coarse sands and gravels, that the coarser parts of the sediment grading of the material on the slope will naturally armour the surface of the slope so that the slope will have long term stability.

9.4 EFFECT OF THE PROPOSED DEVELOPMENT ON WATER QUALITY

A series of coupled hydrodynamic and sediment transport simulations were undertaken to assess effect of dredging the 190,000m^3 of material from the proposed dredge area on the existing water quality at Kyleakin. The study investigated three scenarios in which the capital dredging requirements would be undertaken using three most likely pieces of dredging equipment, i.e. a Trailer Hopper Suction Dredgers (TSHD), Backhoe Dredgers (BHD) or Cutter Suction Dredgers (CSD).

As the production rate of each piece of dredging equipment is different, the duration of the dredging programmes ranged between 30 days for the CSD and 78 days for the TSHD/ BHD.

Results indicated that using a CSD resulted in the greatest increase in ambient Suspended Sediment Concentrations (SSCs). Even under this scenario the greatest increase in SSCs during a typical spring tidal cycle did not exceed 0.07kg/m^3 (70mg/L). An increase in SSCs of 0.07kg/m3 was observed up to 600m to the west of the dredge area and were found to decrease to <0.05kg/m^3 within another 200m. In all dredging scenarios, no increase in SSCs was found to occur beyond the -9.5m CD contour to the north at the flame shell beds.

All dredging techniques were found to result in similar minimal increases in SSCs in the nearshore area proximal to the development. It was found that regardless of the dredging equipment, the residual tidal currents transported suspended sediments in a westerly direction to result in depositions levels of *c.*0.10m within 800m to the west of the Pier which then decreased to < 0.01m within another 800m.

9.5 FUTURE MAINTENANCE DREDGING REQUIREMENTS

This study found that the prominent feature of the sediment transport regime at Kyleakin is a movement of material along the shoreline from south west to east. The proposed 79m caisson structure at the end of the existing Pier will enhance the performance of the existing Pier which currently acts as a groyne in retaining sediment on the western side of the Pier.

Therefore it is expected that maintenance dredging requirements will be relatively small and that rapid infilling of the dredged area under storm conditions is unlikely to occur. Nevertheless it is expected that some sedimentation will occur partially due to ship movements acting on fine material in the berthing area. It is anticipated that maintenance dredging can be undertaken using an excavator mounted on the quay in combination with some local ploughing within the dredged basin area.

9.6 CONCLUSION

Based on the results of an extensive modelling programme it was found that the proposed Pier development at Kyleakin will only result in minimal changes to the existing tidal regime, wave climate and sediment transport regime. Furthermore it was found that the majority of these changes were within the nearshore area proximal to the development.

It is anticipated that the outer half of the proposed berth may require protection against propeller induced scour. Dependent of the use of ship thrusters, scour protection may also be required along the inner berth at a later date. Furthermore, it has been proposed to stabilise the 1 in 7 dredge slopes with dredge gravel and cobble material.

Undertaking the capital dredging requirements using a Trailer Suction Hopper Dredger, Backhoe Dredger or a Cutter Suction Dredger was found to result in minor increases to ambient suspended sediment concentrations over the duration of the works. However, undertaking the dredging using a Trailer Suction Hopper Dredger and a Backhoe Dredger was found to result in the least change to the marine environment.

Regardless of the dredging equipment used, the residual tidal currents were found to transport suspended sediments in a westerly direction to result in depositions levels of < 0.01m within 2km to the west of the Pier. Furthermore, under normal tidal conditions none of the dredging techniques were found to increase SSCs beyond the -9.5m CD contour at the flame shell beds to the north.

Overall the hydraulic modelling shows that the proposed Pier development at Kyleakin only results in minimal changes to the existing coastal processes at Kyleakin. These findings are in line with what would be expected at a site where there is already a structure of similar projection to that of the Pier that has been proposed as part of this development.

10 REFERENCES

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APPENDIX 1

MIKE Modelling Modules

1 MIKE MODELLING MODULES

1.1 MIKE 21/3 COUPLED FM

The MIKE 21 and MIKE 3 Coupled Modelling modules which are 2D and 3D numerical modelling systems respectively were used to simulate the coastal processes within Dublin Port and the greater Dublin bay area. The MIKE 21/ are truly dynamic modelling systems for application within coastal and estuarine environments and can be used for investigating the morphological evolution of the nearshore bathymetry due to the impact of engineering works (coastal structures, dredging works etc.). The engineering works may include breakwaters (surface-Piercing and submerged), groynes, shoreface nourishment, harbours etc. MIKE 21/3 Coupled Model FM can also be used to study the morphological evolution of tidal inlets.

MIKE 21/3 Coupled Model FM is composed of the following modules:

- Hydrodynamic Module
- Transport Module
- ECO Lab Module
- Mud Transport Module
- Sand Transport Module
- Particle Tracking Module
- Spectral Wave Module

The Hydrodynamic Module and the Spectral Wave Module are the basic computational components of the modelling system. Using MIKE 21/3 Coupled Model FM it is possible to simulate the mutual interaction between waves and currents using a dynamic coupling between the Hydrodynamic Module and the Spectral Wave Module. The MIKE 21/3 Coupled Model FM also includes a dynamic coupling between the Mud Transport, Particle Tracking and the Sand Transport models and the Hydrodynamic Module and the Spectral Wave Module. Hence, a full feedback of the bed level changes on the waves and flow calculations can be included.

The main features of the MIKE 21 Coupled Model FM are as follows:

- Dynamic coupling of flow and wave calculations
- Full feedback of bed level changes on flow and wave calculations
- Easy switch between 2D and 3D calculations (hydrodynamic module and process modules)

- Optimal degree of flexibility in describing bathymetry and ambient flow and wave conditions using depth-adaptive and boundary-fitted unstructured mesh

1.2 HYDRODYNAMIC MODULE

The Hydrodynamic Module simulates water level variations and flows in response to a variety of forcing functions in lakes, estuaries and coastal regions. The effects and facilities include:

- Flooding and drying
- Momentum dispersion
- Bottom shear stress
- Coriolis force
- Wind shear stress
- Barometric pressure gradients
- Ice coverage
- Tidal potential
- Precipitation/evaporation
- Wave radiation stresses
- Sources and sinks

The Hydrodynamic Module can be used to solve both three-dimensional (3D) and two-dimensional (2D) problems. In 2D the model is based on the shallow water equations - the depth-integrated incompressible Reynolds averaged Navier-Stokes equations.

1.3 MUD TRANSPORT (MT) MODULE MODELLING SYSTEM

The Mud Transport (MT) module of the MIKE 21/3 Flow Model FM describes erosion, transport and deposition of mud or sand/mud mixtures under the action of currents and (if appropriate) waves. The hydrodynamic basis for the MT Module is calculated using the Hydrodynamic Module of the MIKE 21/3 Flow Model FM modelling system and the MT is implemented as a couple model with the two running concurrently. The MT module is applicable for mud fractions and also sand/mud mixtures.

The following processes may be included in the simulation.

- Forcing by waves
- Salt-flocculation
- Detailed description of the settling process
- Layered description of the bed, and
- Morphological update of the bed

In the MT-module, the settling velocity varies, according to the salinity, if included, and the concentration taking into account flocculation in the water column. Bed erosion can be either non-uniform, i.e. the erosion of soft and partly consolidated bed, or uniform, i.e. the erosion of a dense and consolidated bed. The bed is described as layered and is characterised by the density and shear strength.

1.4 SAND TRANSPORT (ST) MODULE MODELLING SYSTEM

The hydrodynamic basis for the Transport Module is calculated using the Hydrodynamic Module of the MIKE 21/3 Flow Model FM modelling system. The transport module calculates the resulting transport of material based on these flow conditions coupled with the other appropriate aforementioned modules. A number of components may be specified with each component defining a separate transport equation. The time integration of the transport (advection-dispersion) equations is then performed using an explicit scheme to calculate the resulting sediment transport.

1.5 MIKE21 FM FLEXIBLE MESH SPECTRAL WAVE MODELLING SYSTEM

Modelling the wave transformation from the offshore boundary of the Dublin bay model to the sites of interest was undertaken using the MIKE 21 Spectral Wave (SW) model which is a new generation spectral wind-wave model based on unstructured meshes. The model simulates the growth, decay and transformation of wind-generated waves and swell in offshore and coastal areas.

MIKE 21 SW accounts for the following physical phenomena:

- Wave growth by wind action
- Non-linear wave-wave interaction
- Dissipation due to white-capping
- Dissipation due to bottom friction
- Dissipation due to depth-induced wave breaking
- Refraction and shoaling due to depth variations
- Diffraction
- Wave-current interaction
- Effect of time-varying depth and flooding and drying

The discretisation of the governing equation in geographical and spectral is performed using a cell-centred finite volume method. In the geographical domain, an unstructured mesh technique is used. The time integration is performed using a fractional step approach where a multi-sequence explicit method is applied for the propagation of wave action.

The MIKE 21 SW includes two different formulations:

- Directional decoupled parametric formulation
- Fully spectral formulation

The directional decoupled parametric formulation is based on a parameterization of the wave action conservation equation. The parameterization is made in the frequency domain by introducing the zeroth and first moment of the wave action spectrum as dependent variables following Holthuijsen (1989).

1.6 BED ROUGHNESS

When using the two-dimensional hydrodynamic models, the bed resistance was specified using the Manning number. According to the MIKE 21 manual, the relationship between the Manning number,M, and the Nikuradse roughness length, k_s can be estimated using

$$M = \frac{25.4}{k_s^{1/6}}$$

Using one of the several relationships recommended by Soulsby (1997), over flat beds of sediment, k_s is related to the median grain diameter (D_{50}) as approximately

$$k_s = 2.5\ D_{50}$$

For the three-dimensional models, the bed resistance was specified using the bed roughness height of the sea bed which is dependent on the von Karman constant.

It was therefore possible to impose a uniform bed resistance coefficient at the seabed for both the two and three dimensional models - the value of which was determined using the simple relationships presented above and by calibrating of the Dublin Port model.

1.7 TURBULENCE MODULE

The turbulence model used by MIKE21/3 modelling system is based on a standard k-epsilon model ($k - \varepsilon$) with a buoyancy extension. The model uses transport equations for the turbulent kinetic energy (TKE), k, and the dissipation of TKE, ε, to describe the turbulence.

APPENDIX 2

Model Calibration

2 MODEL CALIBRATION

Surveys were undertaken during across July and August of 2016 to provide tidal height, speed and direction data in order to calibrate the hydrodynamic models.

The surveys were undertaken by Aspect Land & Hydrographic Surveys Ltd and involved the deployment of two acoustic Doppler current profilers (ADCP) in the vicinity of the Kyleakin Pier for a period of *c.*6 weeks. One device was deployed to the north west of the existing Pier along the -11mCD contour whilst the second device was deployed to the north east of the existing Pier along the -9.0mCD contour. Both ADCP devices were set up to record information at 0.5m intervals. The deployment location of the two devices in relation to Kyleakin Pier is presented in Figure 3.2 below.

The data recorded by these devices were used to compare, verify and validate the simulated data with recorded data.

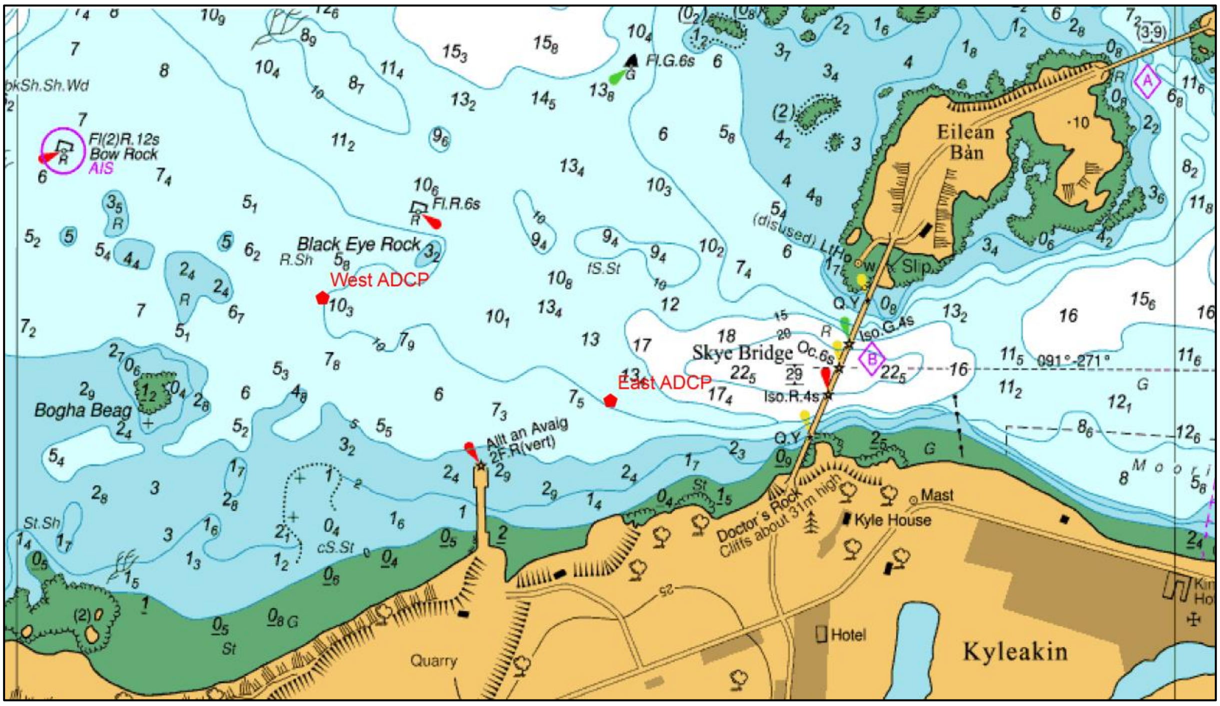


Figure 2.1: Location of the ADCP surveys in relation to Kyleakin Pier.

The data recorded by both ADCP devices across the entire month confirmed the complexity of the tidal current regime at Kyleakin. Figure 2.2 illustrates the variability of the tidal current speeds and directions across three consecutive layers of the water column (i.e. 1.5m). It will be seen that the current speeds can be as low as 0.02m/s in one layer but as high as 0.60m/s in another. Similar variability can be observed in the current direction recordings throughout the majority of the 6 week deployment period.

Weather records that covered the deployment date confirmed that the variability amongst the water column could not be attributed to particularly bad weather. Instead, the extreme variability has been attributed to both the complex nature of the tides in this region and the relatively small bin sizes to which both devices were set up to record data to (i.e. 0.50m) which could have affected the accuracy of the recorded data.

Given the limitations of this dataset, the calibration plots presented in this section should be assessed with caution.

Figure 2.2: Extreme variability of current speeds and directions across three consecutive 0.5m bins – West ADCP.

2.1 MODEL VERFICATION BASED ON TIDAL STREAM INFORMATION

As well as verifying the outputs with data recorded by the two ADCPs detailed in the previous Section of this chapter, the hydrodynamic model was also verified against Tidal Stream information published by the United Kingdom Hydrographic Office (UKHO). There are two tidal streams in close proximity to Kyleakin Pier, however as ADCP data is considered to be more representative, the data provided for these streams were dismissed in favour of the ADCP data.

There was one tidal stream point located at the bottom of the Sound of Raasay as illustrated in Figure 5.2. Model verification against this information would ensure that the model is accurately simulating the fundamental hydrodynamic process in the far-field of the model domain. Tidal stream data detailed by the UKHO provides a reasonably estimation of the current direction and velocities six hours before and after High Water (HW) and can therefore be used as indicator of model accuracy.

Figure 2.4 and Figure 2.5 illustrates a comparison between the simulated current velocities and the reported current velocities at tidal stream 2209 A for both spring and neap tides respectively. In general, the current velocity data taken from the simulation compared well with the values reported by the UKHO for this particularly tidal stream.

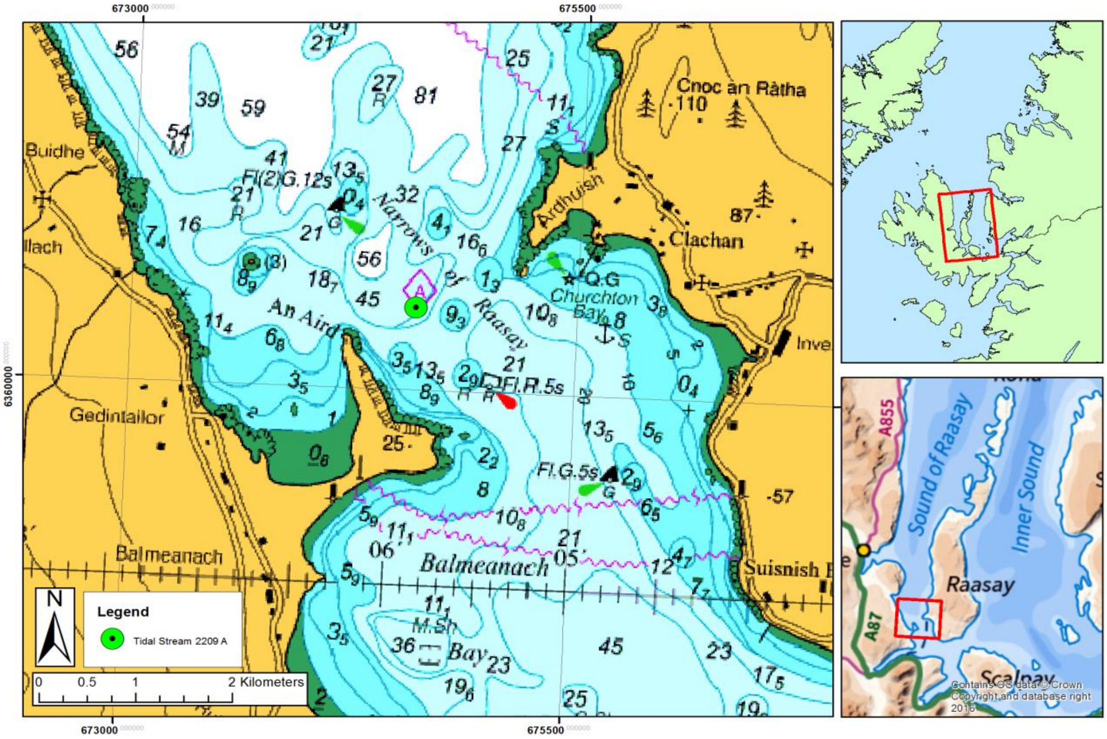
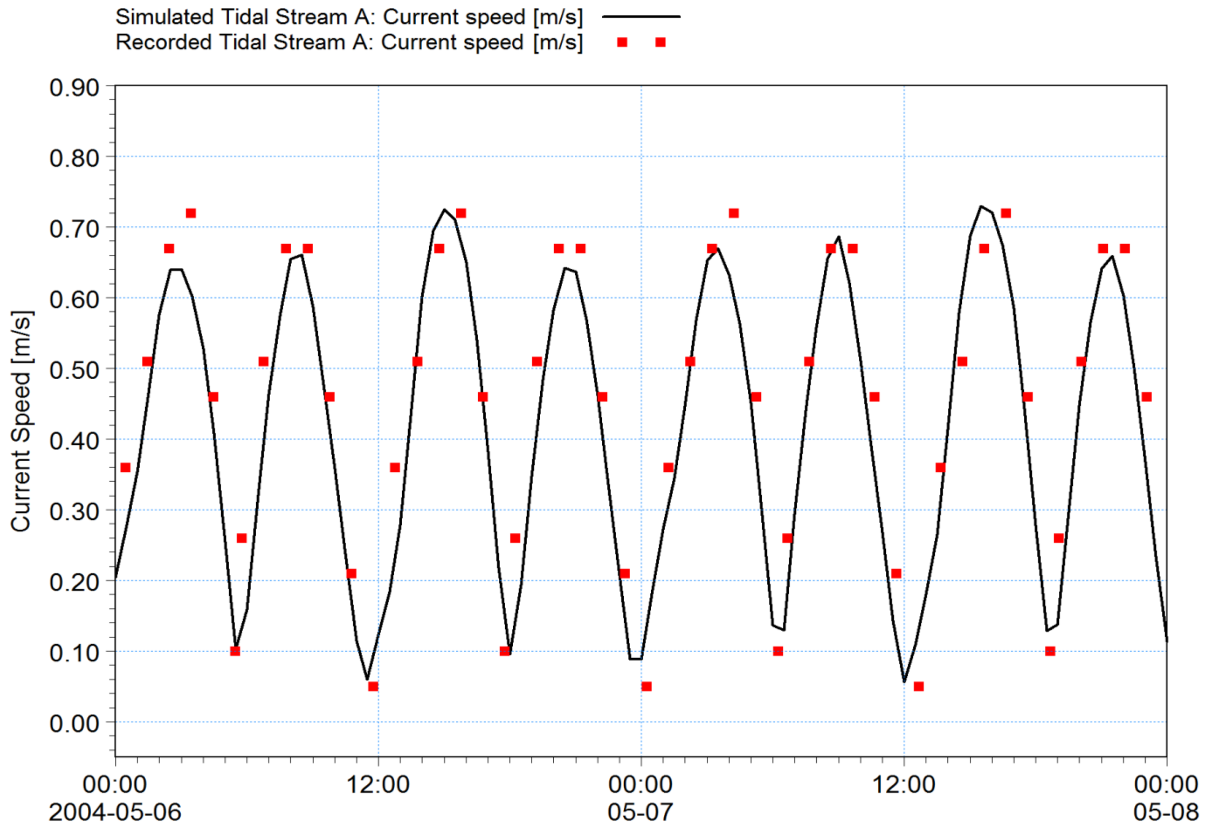


Figure 2.3: Location of Tidal Stream 2209 A in the Sound of Raasay.



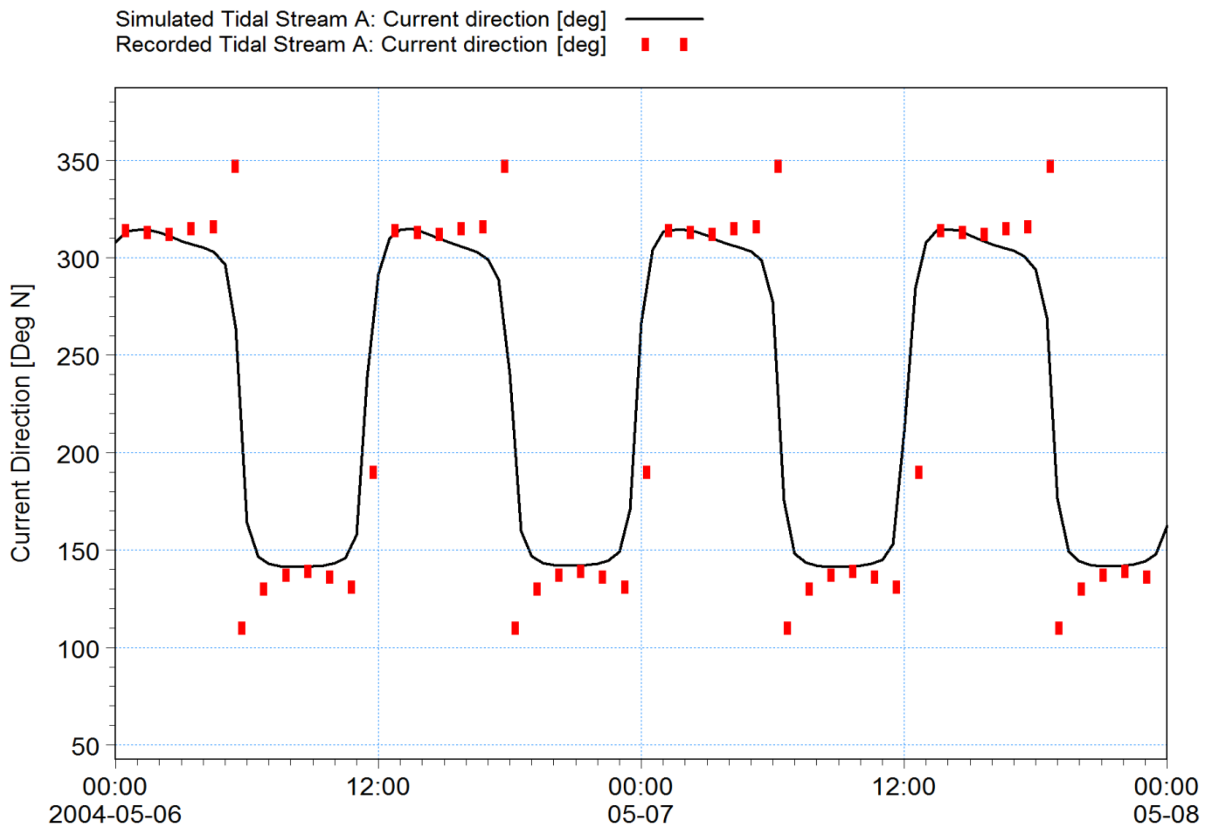
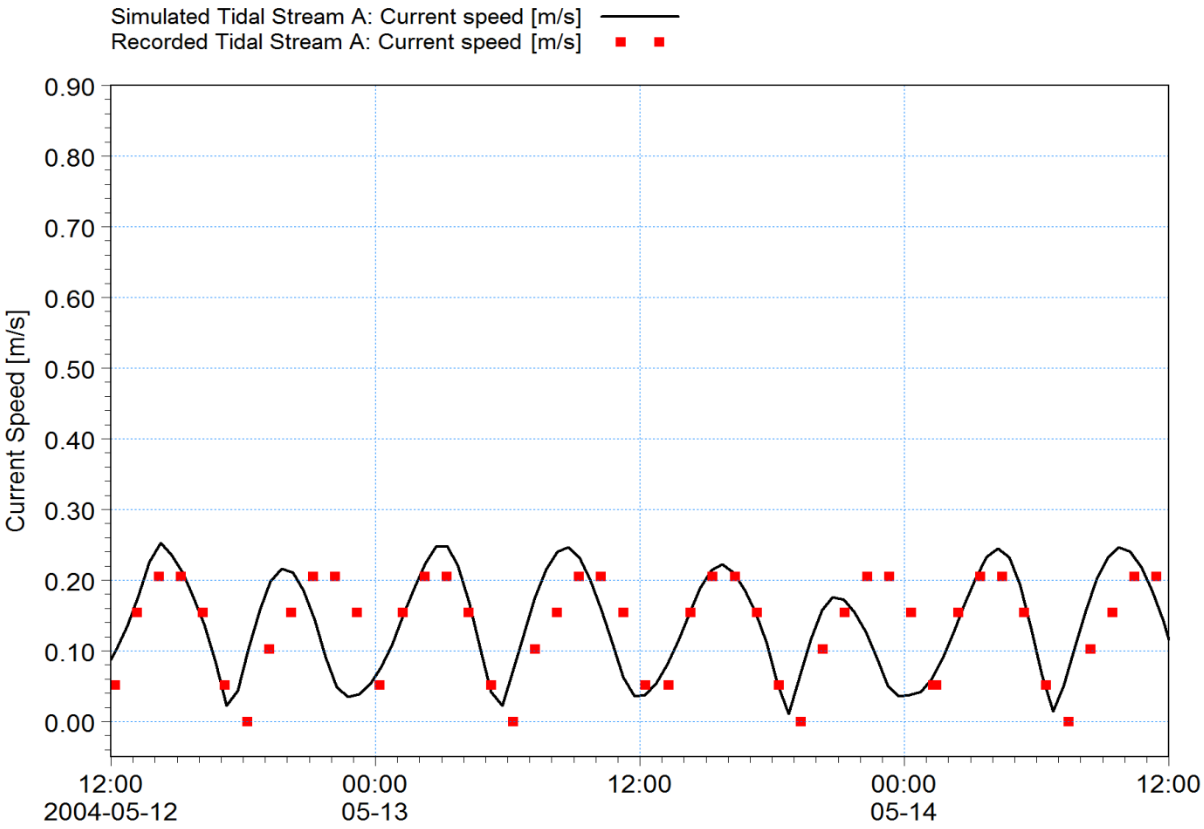


Figure 2.4: Comparison of modelled and observed Current Speed and Directions during a typical Spring tidal cycle – Tidal Stream 2209 A.



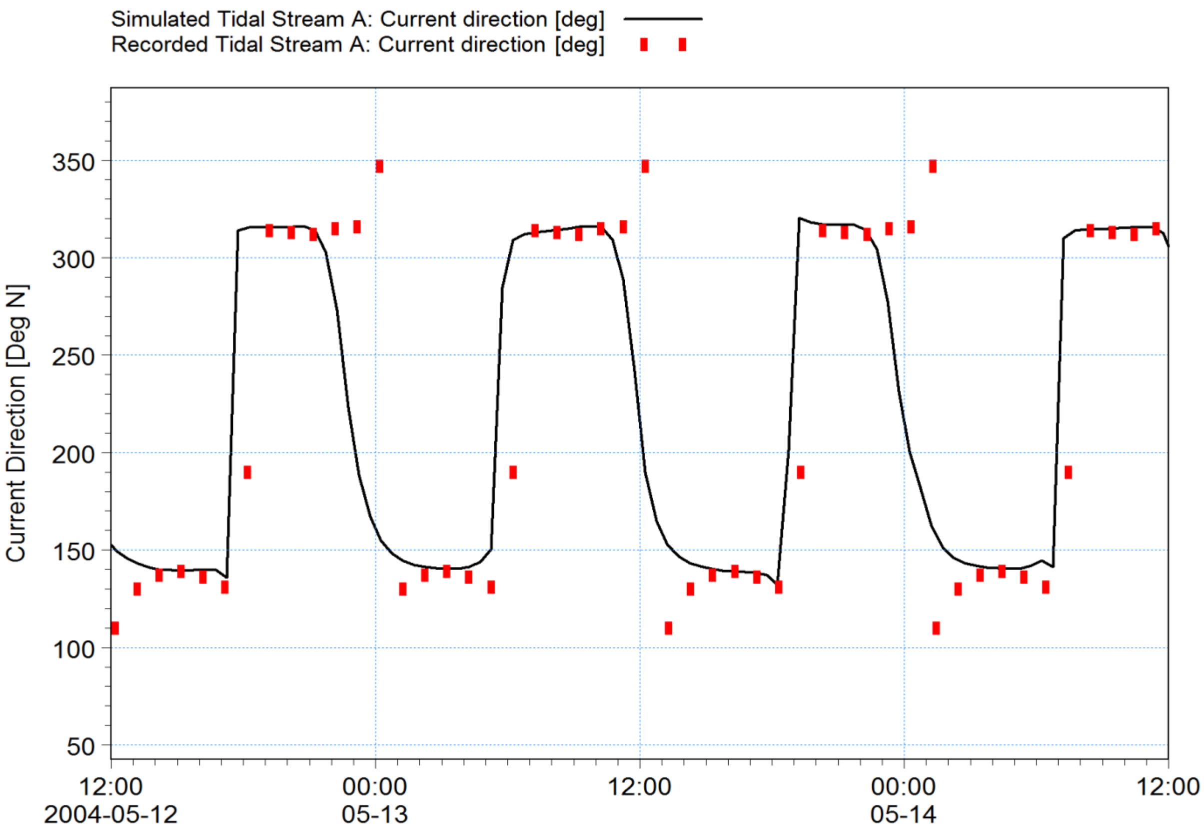


Figure 2.5: Comparison of modelled and observed Current Speed and Directions during a typical Neap tidal cycle – Tidal Stream 2209 A.

2.2 MODEL VERFICATION BASED ON WEST ADCP DATA

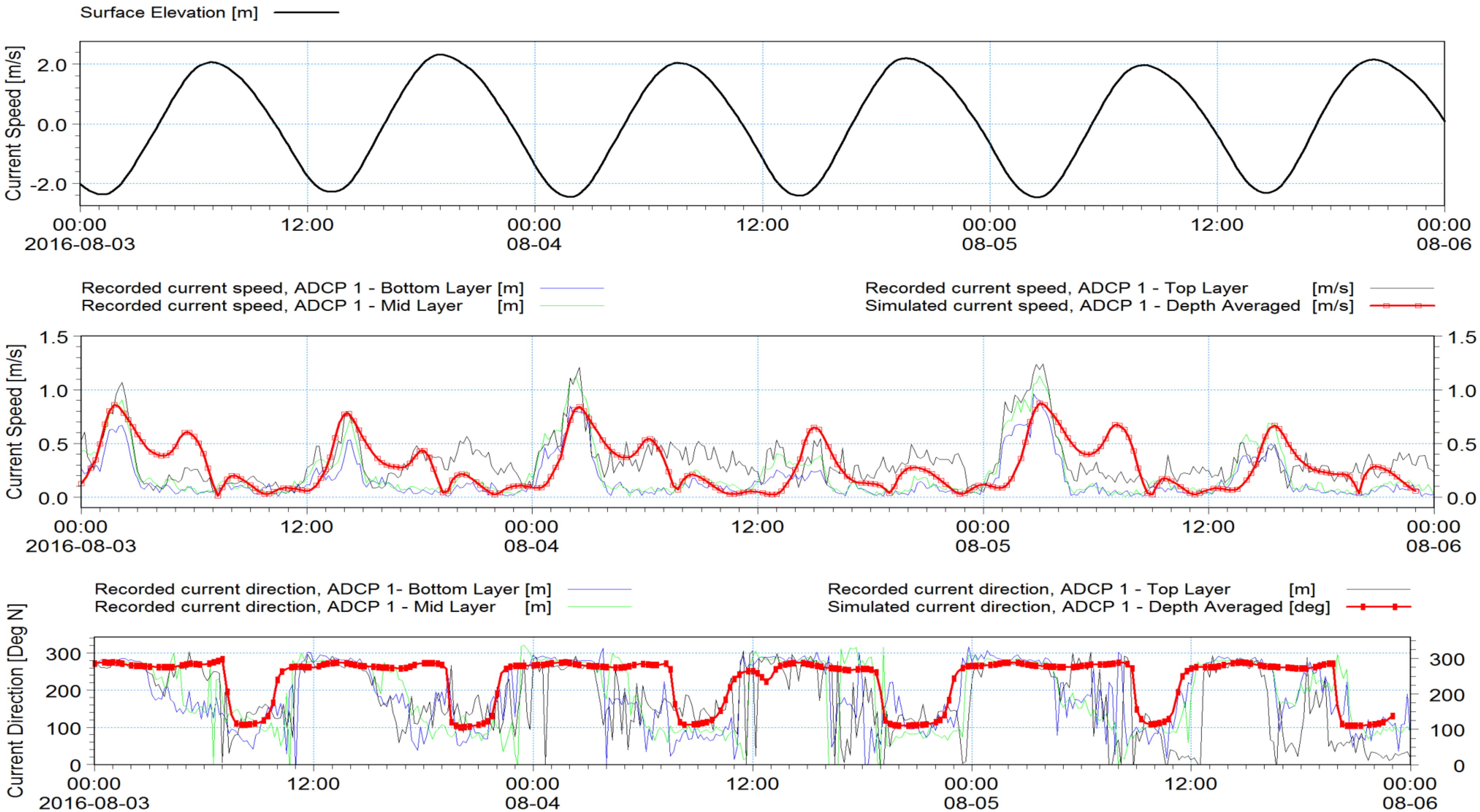


Figure 2.6: Comparison of modelled and observed Current Speed and Directions during a typical Spring tidal cycle - West ADCP.

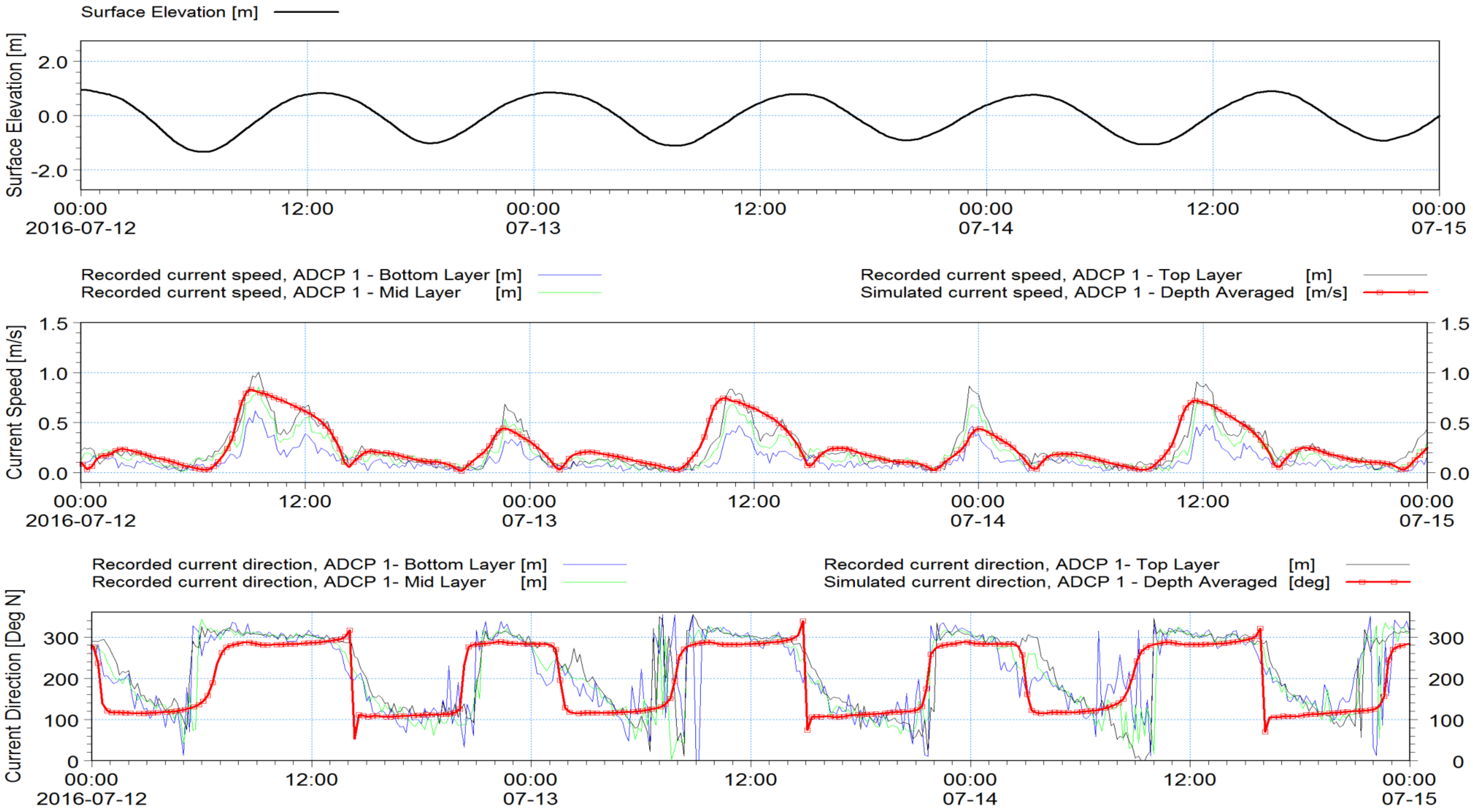


Figure 2.7: Comparison of modelled and observed Current Speed and Directions during a typical Neap tidal cycle - West ADCP.

2.3 MODEL VERFICATION BASED ON EAST ADCP DATA

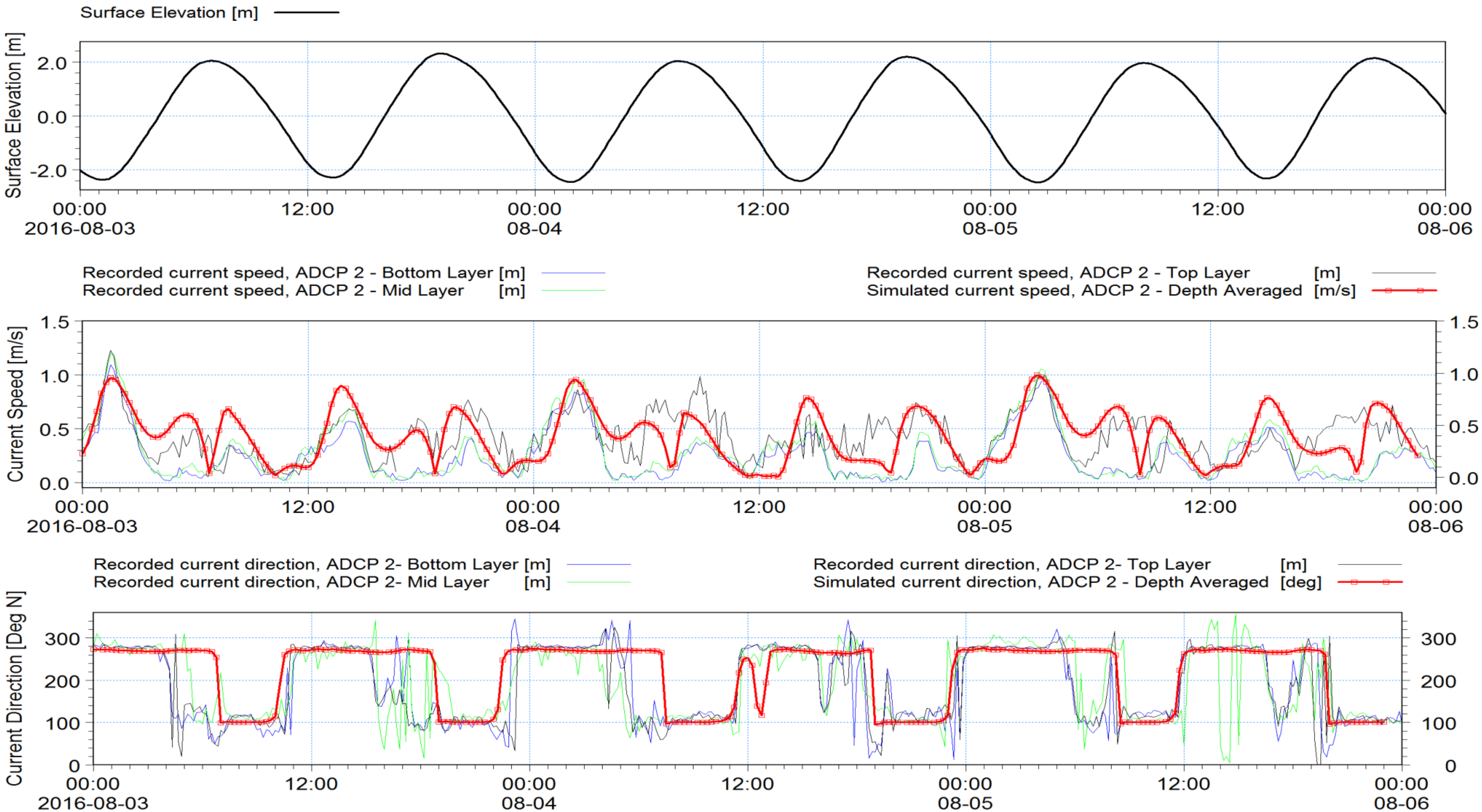


Figure 2.8: Comparison of modelled and observed Current Speed and Directions during a typical Spring tidal cycle – East ADCP.

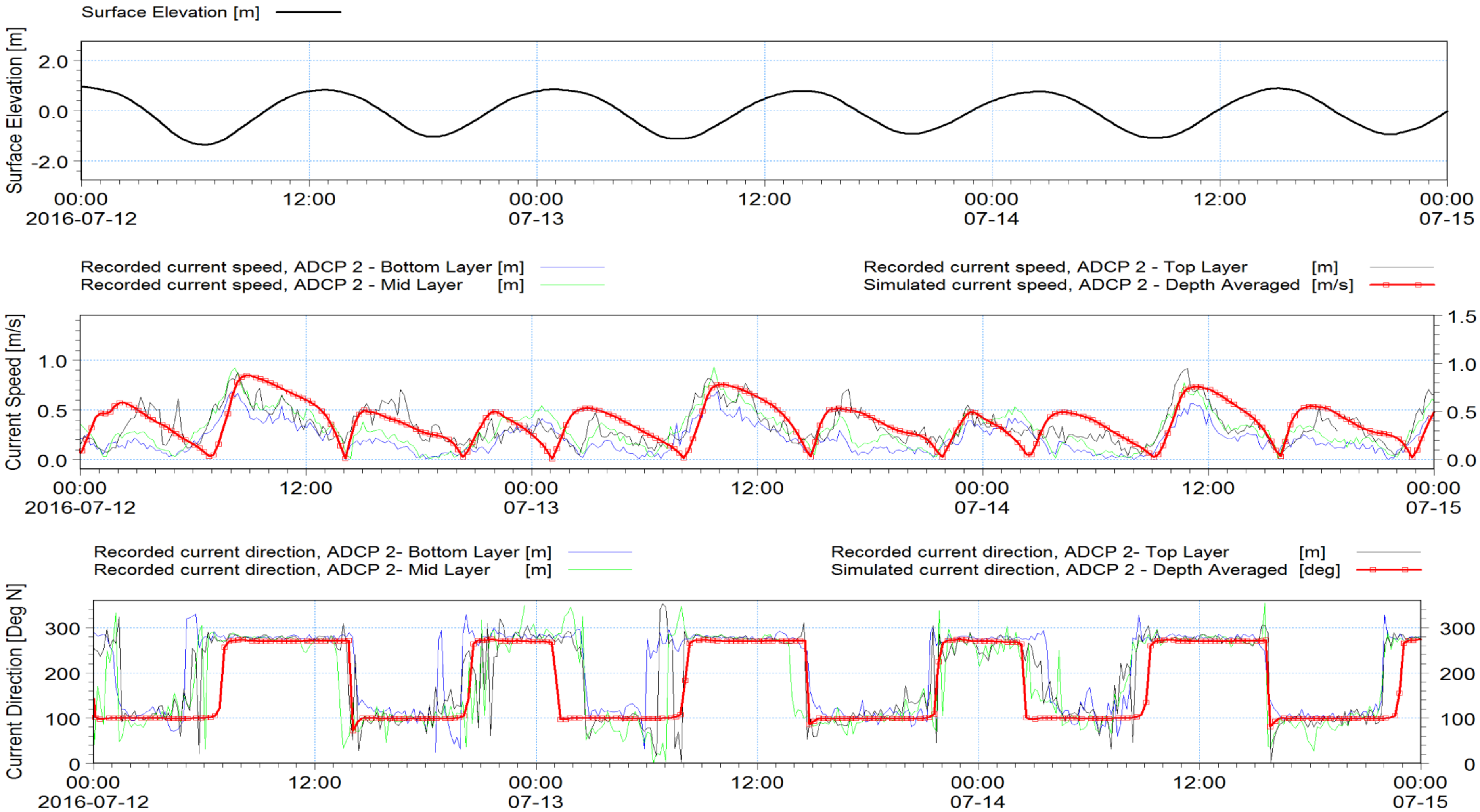


Figure 2.9: Comparison of modelled and observed Current Speed and Directions during a typical Neap tidal cycle – East ADCP.

APPENDIX 4

ALHS Geotechnical Survey Report

3 ALHS GEOTECHNICAL SURVEY REPORT

See attached.

APPENDIX 3

Tidal regime at Kyleakin Pier – Neap tidal conditions

4 TIDAL REGIME AT KYLEAKIN PIER – NEAP TIDAL CONDITIONS

See overleaf

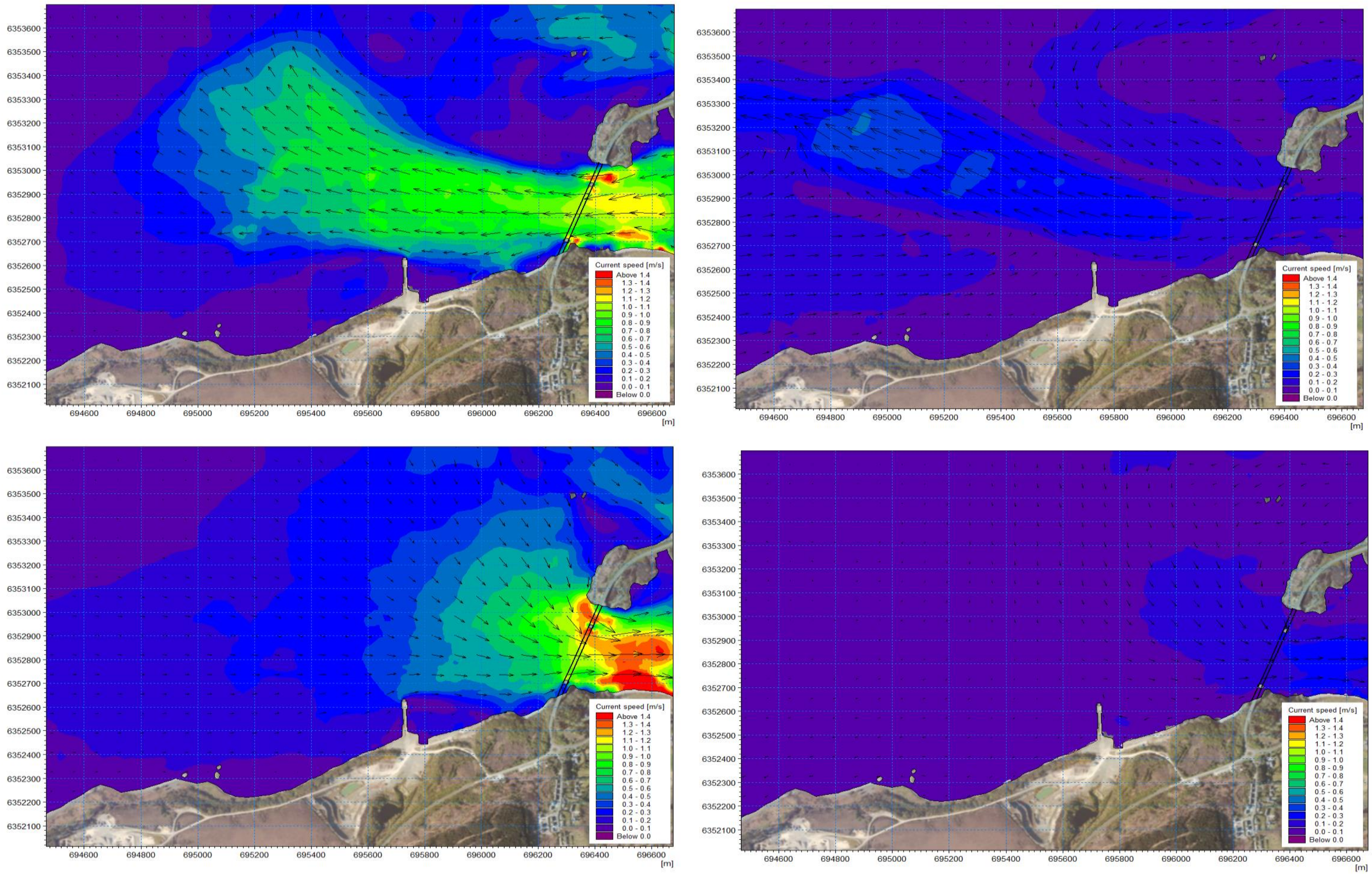


Figure 4.1: Typical peak flood (top left), high water (top right), peak ebb (bottom left) and low water (bottom right) neap tidal regimes – Existing Layout.

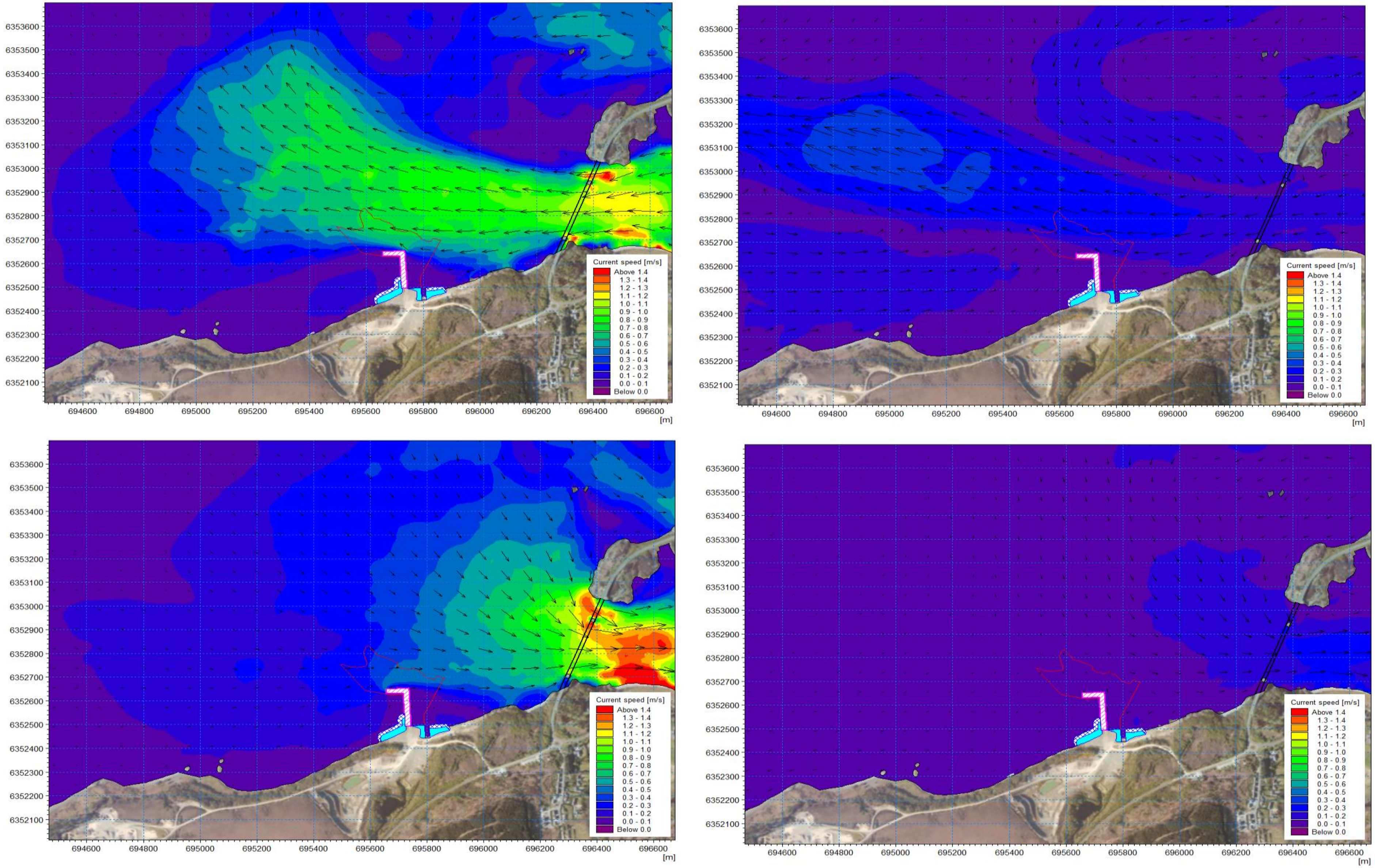


Figure 4.2: Typical peak flood (top left), high water (top right), peak ebb (bottom left) and low water (bottom right) spring tidal regimes – Proposed Layout