Galway Harbour Company

Galway Harbour Extension

Environmental Impact Statement

Chapter 8

Water

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GLOSSARY, ABBREVIATIONS & DEFINITIONS

ARPD	Apparent Redox Potential Discontinuity
Bathymetric	Of or relating to measurements of the depths of oceans or lakes
Benthic	That portion of the marine environment inhabited by organisms living at or near the bottom of the ocean
Benthos	Plants and animals that live in the sea bed below low water.
CEFAS	The Centre for Environment Fisheries and Aquaculture Science
Chart Datum	A chart datum is the level of water that charted depths displayed on a nautical chart are measured from
Coriolis Effect	An effect whereby a mass moving in a rotating system experiences a force perpendicular to the direction of motion and to the axis of rotation. With regard to the island of Ireland, this force causes water from rivers to move in a clockwise direction around the island
cSAC	candidate Special Area of Conservation
Epifauna	Animals living on the surface of the seabed or a riverbed, or attached to submerged objects or aquatic animals or plants
Eulittoral	A subdivision of the benthic division of the littoral zone of the marine environment, extending from high-tide level to about 200 feet (60 meters), the lower limit for abundant growth of attached plants
GPS	Global Positioning Satellite
Granulometry	The measurement of the size distribution in a collection of grains
Gyre	Any system of rotating ocean current.
IBA	Important Bird Area
Infauna	The animals living in the sediments of the ocean floor or river or lake beds
Intertidal	Of or being the region between the high tide mark and the low tide mark
MDS	Multi-Dimensional Scaling
Neritic	Of, relating to, or denoting the shallow part of the sea near a coast and overlying the continental shelf
North Atlantic Drift	A continuation of the Gulf Stream across the Atlantic Ocean and along the coast of northwestern Europe
OSI	Organism Sediment Index
PCB	Polychlorinated biphenyl
ppm	parts per million
Refusal Depth	Depth to bedrock
RIB	Rigid Inflatable Boat
Ruggedised	Produce in a version designed to withstand rough usage
SBR	Surface Boundary Roughness
SCUBA	Self Contained Underwater Breathing Apparatus
Sedimentation	The phenomenon of sediment accumulation
Smolt	A young salmon after the parr stage, when it becomes silvery and migrates to the sea for the first time
SPA	Special Protection Area
SPI	Sediment Profile Imagery
Subtidal	The sea bed environment below low tide that is always covered by water.
Velocity	The speed of something in a given direction

8 WATER

8.1 INTRODUCTION

This section of the EIS was commissioned to carry out aspects of marine work in relation to the development of the Galway Harbour Extension. The work was to include a bathymetric survey of the site for later use in modelling studies, a review of water quality in inner Galway Bay, a mathematical model to predict any changes in current velocities or directions, salinity and sedimentation patterns where the development is to take place and wave modelling studies to predict wave heights at the proposed new development and to assess the potential impacts and identify any mitigation measures. Figure 8.1.1 shows the sensitive receptors in relation to Water.

Figure 8.1.1 – Sensitive receptors in relation to Water

8.2 BATHYMETRY

8.2.1 Introduction

As part of the study on the proposed Galway Harbour Extension a survey of the bathymetry at the proposed development area, approach channel to Galway Docks, inlet channel to Lough Atalia and Lough Atalia and Renmore Lough were carried out to augment the existing available bathymetric surveys listed at 8.4.2.3.

8.2.2 Methodology

The surveys were completed using a SonarLite Portable Echo Sounder with Trimble NT300D DGPS:

- SonarLite Echo Sounder
- Tranducer frequency – 200 KHz Active Tranducer
- Depth Range 0.30 to 50 m

- Accuracy +/- 0.025 m
- Sound Velocity Range – 1400 to 1600 m/s
- Pulse Frequency – 1 Hz
- Data Output – ASCII, NMEA, Navitronic, Odom, Atlas, Elac, Geotronics
- Trimble NT300D
- 12 channel, parallel tracking, L1 C/A code with carrier phase filtered measurements
- 5 Hz position updates, latency <200 m/s
- Accuracy, less than 1 m when operating within the broadcast area of a reference station conforming to the International Association of Lighthouse Authorities Standards.

Prior to the survey being carried out, the sounder was calibrated by the bar test method and also against a 1.5 m depth probe marked in centimetres and any difference was adjusted as necessary. This exercise was carried out at regular intervals during the survey to confirm that soundings were correct. Once the equipment was set up, the survey vessel followed predetermined transect lines at a speed of approximately 1.5 m/s. The sounder automatically recorded depths every second and DGPS positions every two seconds.

In order to rule out depth variations over the survey period due to the tidal differences and to standardise the depths relative to Chart Datum, tidal information was recorded.

8.3 WATER QUALITY

8.3.1 Introduction

This section provides a description of the water quality status in inner Galway Bay and is based on a number of both published and unpublished texts on the subject. Before the sewage treatment plant located in Mutton Island became operational in late 2003, water quality in inner Galway Bay was impacted by two main water inputs: the municipal sewage system (with sewage outfalls located at various sources, including the Docks, South Park, Knocknacarra and various combined storm overflows) and freshwater from the Corrib River. In September 2003, foul sewage from the municipal system was diverted to the treatment plant where it was treated and then pumped into Galway Bay to the south of Mutton Island.

8.3.2 Water movement within inner Galway Bay and long shore drift

Current flow on the west coast of Ireland has been studied by Tulloch & Tait (1959) and, in Galway Bay, by Booth (1974). Both studies show that water enters the bay from the south, primarily through the South Sound with the Foul and Gregory Sounds being less important in this respect. Circulation is anticlockwise with water leaving the bay chiefly through the North Sound (Booth 1974). This overall south to north net flow agrees with Monahan's (1977) findings and directs fresh (largely Corrib) water outflow along the North Shore with suspended materials being deposited over this area. The deflection of the Corrib water westwards along the north shore of Galway Bay is driven largely by the Coriolis effect. While predominantly neritic and "estuarine" (Booth 1974) in nature, the bay is subject to periodic intrusions of oceanic water masses (O'Brien 1975, 1977; Fives & O'Brien 1976). Both Lusitanian and North Atlantic Drift indicator species have been taken in the plankton of the Inner Bay (Fives & O'Brien 1976).

Transport of nutrients and bacteria in inner Galway Bay depends on a number of variables including current speed and direction, wind speed and direction, river flow and tidal conditions.

Water movement in Galway Bay is complex and variable and is strongly influenced by wind. Current directions are mainly between NE and E during flood tide and between SW and NW during the ebb tide. For Spring tides, the ebb pattern is dominated by tidal currents while the flood pattern is diverse with direction influenced to a large extent by wind directions (An Foras Forbartha, 1988).There is also a strong tidal influence under some ebb tides with calm to moderate breezes during Neap tides. The presence of a gyre in the inner portion of the Bay has been suggested by Booth (1974) and Harte *et al.* (1982). O'Connor *et al.* (1986) comment on this feature in the light of the distribution of *Amphiura filiformis*.

There is some stratification on different stages of the tidal cycle and other wind conditions. The vertical mixing of the water column in terms of salinity and temperature is weaker during Neap tides than during Spring tides.

The River Corrib is by far the largest input of freshwater to Galway Bay. The river has a very strong effect in structuring the water column in the northeastern section of the inner bay, especially during spate periods. This freshwater does not follow the anticlockwise flow of the Atlantic seawater within the bay but rather is influenced by the wind velocity and direction (Smith *et al*., 1998). In calm conditions, the river water flows in a westerly direction along the north shore but, when westerly gales are blowing, this water can be backed up into Oranmore Bay and New Harbour (An Foras Forbartha, 1986).

The flow of the River Corrib affects surface salinities in the area, *i.e.* northeast of Mutton and Hare Islands. From there, the freshwater tends to flow seawards in a west/southwest direction. Low salinity at the surface also extended to Mweeloon Bay, New Harbour and Oranmore Bay (An Foras Forbartha, 1988).

On the turn of the tide after low water, the water fills from the southwest as it makes its way eastwards towards Oranmore Bay. Due to the presence of the Mutton Island causeway, the flooding tide is directed around the island and enters the mouth of the River Corrib. The movement of the ebbing tide water is essentially the reverse of this.

Winds coming from the west to the south west sector are the strongest winds in inner Galway Bay. These winds can modify surface water current speeds causing water to be forced either to the north during southerly wind flows or easterly if the wind comes from the west. These prevailing wind conditions generate an easterly moving long shore drift. The Mutton Island causeway intercepts any sediment mobilised by the long shore drift and thereby reduces the extent of material being carried onto Ballyloughan Beach and into the area of the proposed new Harbour Extension.

In terms of river flow, there is a strong seasonality regarding to volume between Winter and Spring months with Winter having the largest flows.

8.3.3 Water Quality

Water samples from different locations around Galway Bay were taken from 1988 to 2003, before and after the Mutton Island waste water treatment plant was operational. Water samples were analysed for both faecal and total coliforms and the results compared against limits established in the EU Directive on Bathing Waters. The majority of these results were below the maximum admissible limits established in the EU regulations for bathing waters giving a clear indication of improvement in the water quality in inner Galway Bay.

An examination of bacteriological results (see Table 8.3.1) for faecal coliforms and streptococci provided on the Galway City Council website for Grattan Road beach for 2003 – 2013 (ex. 2010) showed that out of a total of 154 samples, 9 (5.8%) exceeded Mandatory or National guideline levels for faecal coliforms. Guideline levels for faecal coliforms were exceeded on 39 occasions and 16 times for faecal streptococci.

	2003 (20)*			2004 (18)			2005 (13)			2006 (19)			2007 (16)			2008 (22)			2009 (17)			2011 (8)			2012 (13)			2013 (8)		
	G[1]	M[2]	N[3]	G	M	N	G	M	N	G	M	N	G	M	N	G	M	N	G	M	N	G	M	N	G	M	N	G	M	N
Faecal coliforms	5	0	0	6	0	0	3	1	1	5	1	1	2	0	0	8	0	1	3	0	0	3	0	0	6	2	0	0	0	0
Faecal streptococci	3	NA	0	5	NA	3	2	NA	0	2	NA	1	0	NA	0	1	NA	0	2	NA	1	0	NA	0	1	NA	0	0	NA	0

Table 8.3.1 - Number of exceedences per year 2003-2013 (ex. 2010) for faecal coliforms & faecal streptococci at Gratten Beach, Galway (data from Galway City Council).

* Number of samples analysed in that year. NA: Not applicable.
[1]G = Guideline: <100 colony forming units (cfu) per 100 ml
[2]M = Mandatory: <2,000 colony forming units (cfu) per 100 ml
[3]N = National: <1,000 colony forming units (cfu) per 100 ml

With regard to nutrients concentrations, data come from a survey by AQUAFACT in 1999 to establish the quality of the water in Galway Bay prior to the commissioning of the Mutton Island sewage treatment plant. Water samples from six stations around Galway Bay were analysed for nitrate, nitrite, ammonia and phosphate levels. While satisfactory results were obtained for nitrates, nitrites and phosphates, ammonia levels were slightly over the EU Regulations for salmonid waters (0.025 mg/l NH_3) in some occasions. These data were obtained before diversion of sewage water to the Mutton Island treatment plant and are higher than present day data.

Heavy metal levels measured over the past (An Foras Forbartha, CAAS and AQUAFACT reports) in water samples taken in inner Galway Bay were always very low.

8.3.4 Conclusions

Bathing water quality analyses in terms of bacteriological content at Grattan Beach which is closest to the treatment works at Mutton Island indicate that although there are breaches of Guideline, Mandatory and National levels, the number of exeedences is low.

During the construction period of the proposed Galway Harbour Extension, turbidity levels of the water where the construction activities will take place will increase. Typically, turbidity levels rise to *ca.* 150 mg/l (AQUAFACT obs.) in water samples collected directly beside the bucket of a machine. During the operational period of the harbour, there will be the potential for impacts on water quality from spillages from vessels or from land. These issues are further addressed in the “Impacts and Mitigation” section.

8.4 HYDRODYNAMIC, SEDIMENT, WAVE CLIMATE, FLOOD RISK AND SALINITY STUDIES

8.4.1 Introduction

Hydrodynamic and sediment modelling of the proposed development area was carried out to assess and quantify the potential impacts of the development on tidal circulation, water quality, sedimentology, wave dynamics, flood risk and changes in salinity.

8.4.2 Hydrodynamic and sediment modelling

8.4.2.1 Hydrodynamic Software description

The TELEMAC package was the software of choice for modelling the complicated hydrodynamics of the Galway bay area and particularly the varying refinement of the computation required (*i.e.* inner harbour and proposed extension area requiring high resolution and the open sea requiring less resolution). TELEMAC is a software system designed to study environmental processes in free surface transient flows. It is therefore applicable to seas and coastal domains, estuaries, rivers and lakes. Its main fields of application are in hydrodynamics, water quality, sedimentology and water waves.

TELEMAC is an integrated, user friendly software system for free surface waters. TELEMAC was developed by Laboratoire National d’Hydraulique of the French Electricity Board (EDF-LNHE), Paris. It is now under the directorship of a consortium of organisations including EDF-LNHE, HR Wallingford, SOGREAH, BAW and CETMEF. It is regarded as one of the leading software packages for free surface water hydraulic application and with more than 300 Telemac Installations Worldwide.

8.4.2.2 Scientific background

The TELEMAC System is a set of finite element programs designed to solve free water surface problems. A series of modules are available for solution of hydrodynamics, transport and dispersion of pollutants, sediment transport and wave dynamics. These are:

- TELEMAC-2D: 2-dimensional depth averaged hydrodynamics and transport and dispersion of tracers

- TELEMAC-3D: 3-dimensional hydrodynamics, transport and dispersion and sediment movement

- SISYPHE: Sediment transport module solving bed and suspended load of cohesive and non-cohesive sediments

- TOMAWAC: A third generation spectral wave model representing the generation of waves due to winds or offshore climates and propagation into shallow waters.

- ARTEMIS: A harbor wave model that solves the mild slope equation in elliptical form and includes the processes of refraction by bed shoaling, wave breaking, diffraction and reflection of waves due to structures.

Each TELEMAC Module uses a completely flexible unstructured mesh of triangular elements allowing it to efficiently model complex geometry problems such as harbours.

8.4.2.3 Model development

A finite element unstructured finite element mesh was fitted to Galway Bay from Laghtnagliboge Point near Spiddal and Black Head on the north Clare coastline eastward including both north and south Galway Bays which includes Oranmore Bay, New Harbour, The Doorus Strait, Kinvara Bay and Ballyvaughan Bay (see Figure 8.4.1). In the vicinity of the subject extension site at Galway harbour the finite element mesh was refined to include better detail of the bathymetry and shoreline geometry (see Figure 8.4.2). This refinement area included Mutton Island, the Claddagh Basin and Lough Atalia. Figure 8.4.3 shows the model mesh with the proposed port structure.

The bathymetry specified in the model came from the following Sources:-

- AQUAFACT Surveys of the Approach Channel, Claddagh area, Lough Atalia, the Proposed Port area east of the approach channel;
- The Infomar (GSI) Lidar Data Set of Galway Bay – survey 22nd May to 14th June 2008;
- Galway Bay and Approaches Admiralty Charts.

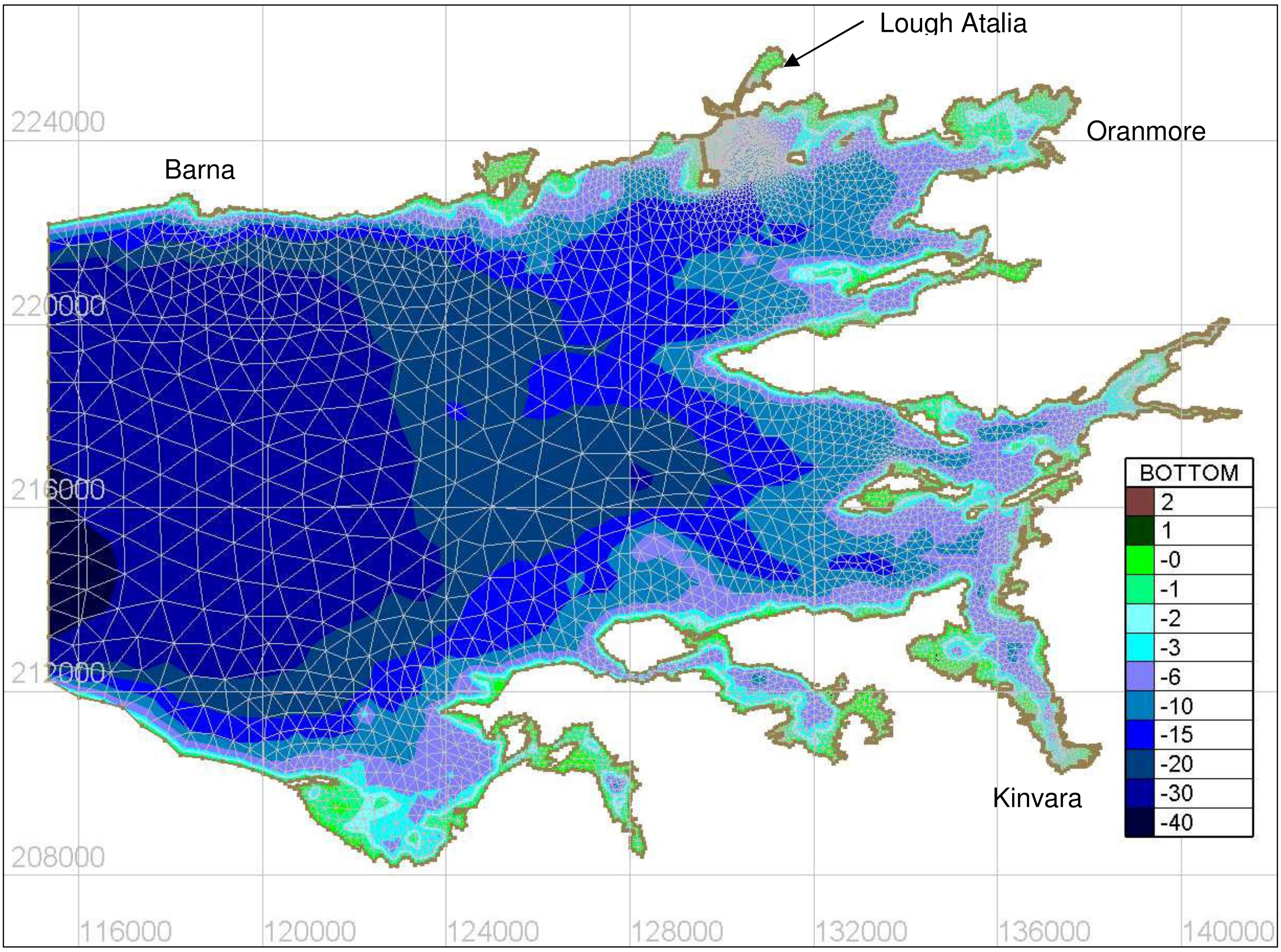


Figure 8.4.1 - Hydrodynamic model finite element mesh and bathymetry for existing case. (Bottom refers to bed elevations mOD Malin)

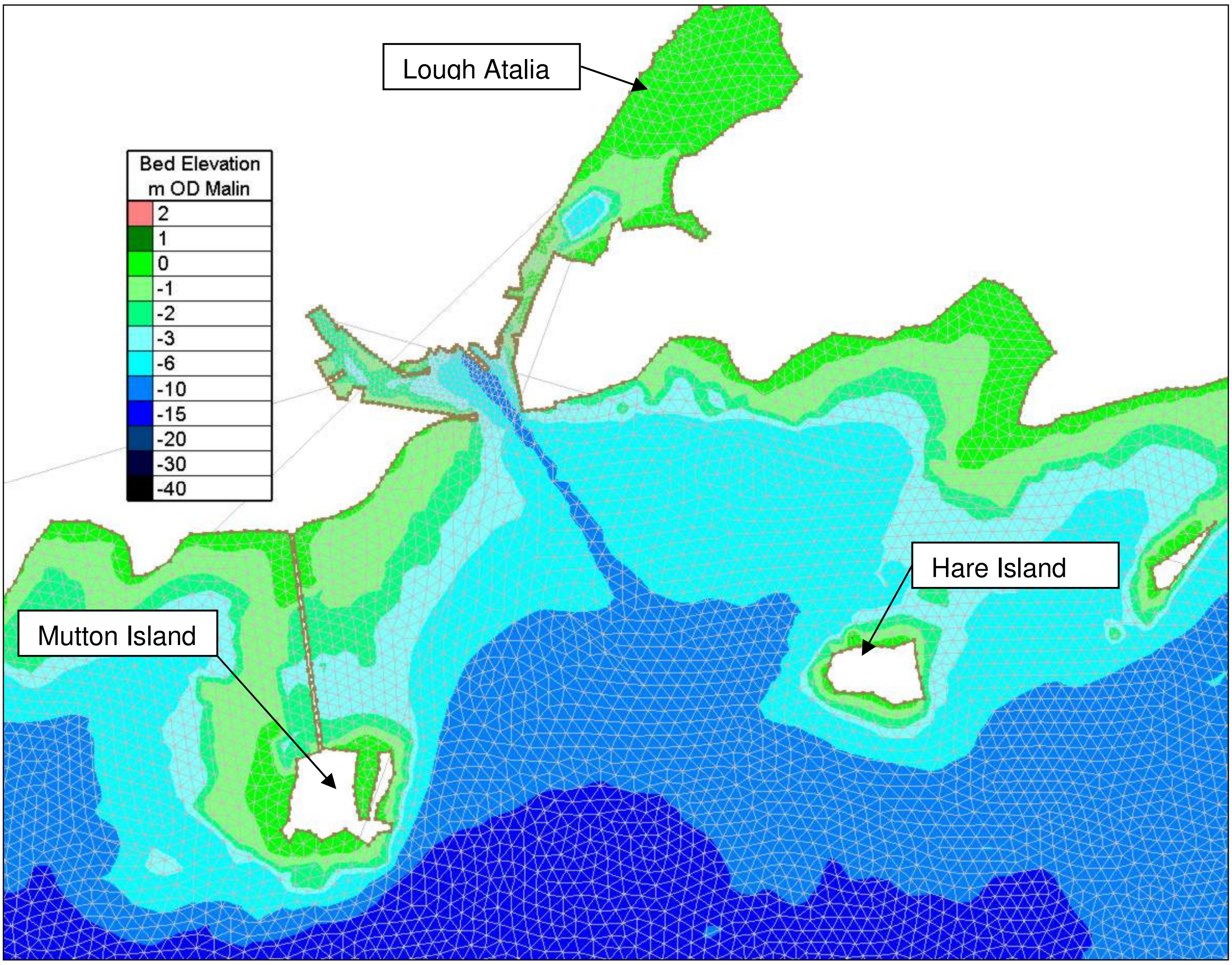


Figure 8.4.2 - Close-up view of Hydrodynamic model refinement in the vicinity of the proposed harbour extension development

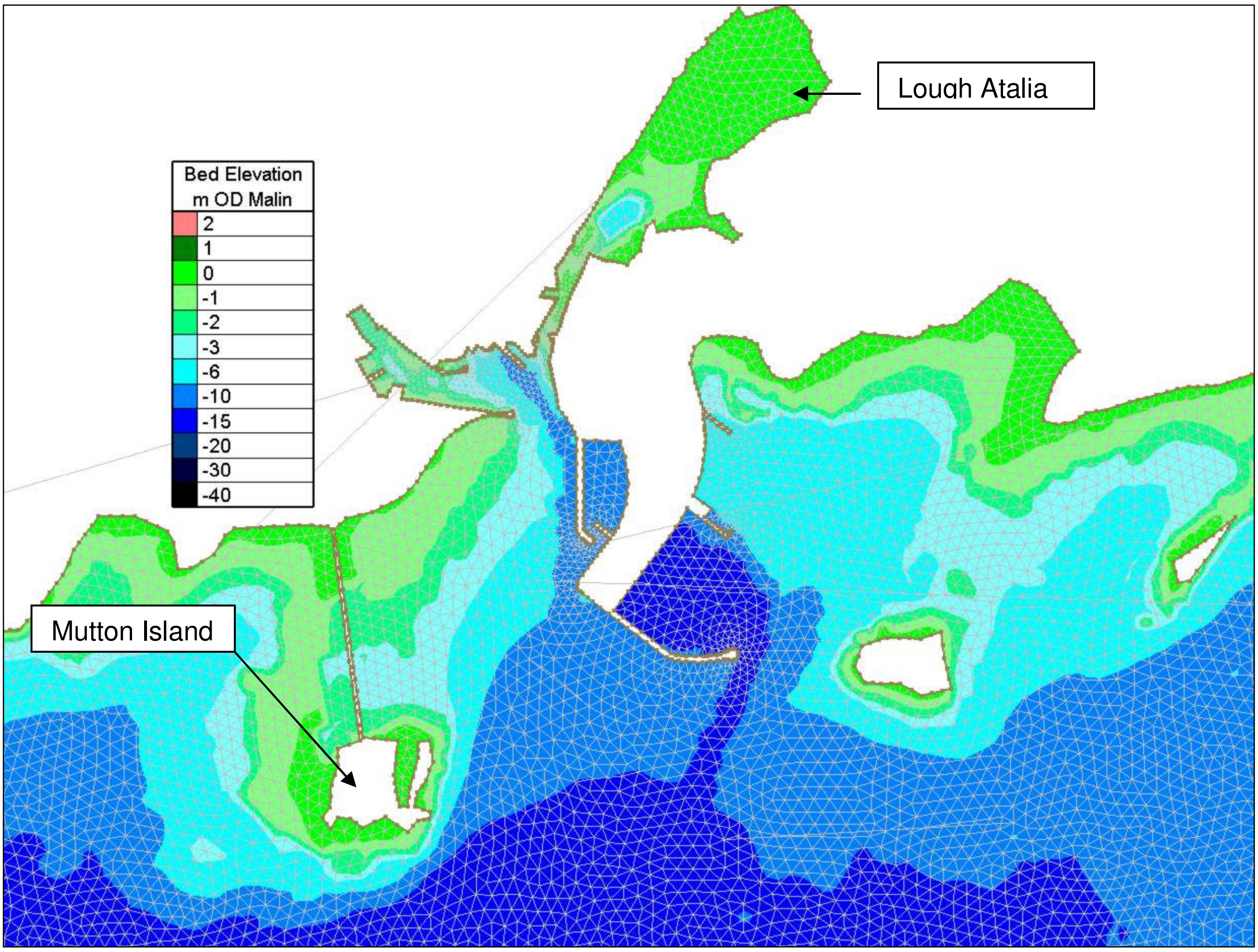


Figure 8.4.3 - View of Hydrodynamic model mesh for proposed case in the vicinity of the proposed harbour extension

8.4.2.4 Hydrodynamic simulations

In order to assess the implications of the proposed harbour extension development on flooding, water quality, sediment transport and salinity, the key input to the analysis is the hydrodynamics, which need to be resolved spatially and temporally in terms of current speeds, direction and depths and tidal elevations throughout the study area. The hydrodynamics dictate the rate of mixing, the horizontal and vertical dispersion, settlement patterns and bed erosion rates. The depth averaged hydrodynamics were resolved using the Telemac2d and Telemac3d hydraulic modules for the domain presented in Figure 8.4.1. Zoomed in views of the model finite element structure surrounding the extension area and Mutton Island are presented in Figures 8.4.2 and 8.4.3. These figures show the high degree of refinement used for the immediate areas of interest. The hydrodynamics vary depending on astronomical, fluvial and meteorological conditions. The following six hydrodynamic conditions were simulated by the model so as to provide sufficient representation of the hydrodynamics, transport and flooding properties under the natural range of ambient conditions. The forcing tide was specified at the westerly open sea boundary of the model, the fluvial flow boundary condition specified at the boundary node points located at Wolfe Tone Bridge and the wind shear specified at each surface nodal point. All other sources of fluvial inflow were considered insignificant in respect to their effect on the hydrodynamics within the area of interest.

Hydrodynamic simulations were performed for a combination of tide, wind and river inflow conditions, with and without the proposed harbour extension. The seven hydrodynamic conditions considered were:

1. 95% percentile Corrib River flow (26.4 m^3/s) with mean Neap tide
2. 95% percentile Corrib River flow (26.4 m^3/s) and mean Spring tide
3. Median river flow (82 m^3/s) with mean Spring tide.
4. Median Corrib River flow (82 m^3/s) with mean Neap tide
5. 100 year return period Corrib River flow (458 m^3/s) with mean Spring tide
6. Historical storm surge tide (of 3.49 m O.D.) combined with the 100 year return period Corrib River flow that includes 20% climate change allowance (549 m^3/s).
7. Historical storm surge tide (of 3.49m O.D.) combined with the 100 year plus design flood flow that includes 20% climate change allowance (549 m^3/s) plus Sea climate change allowance of 500mm

Astronomical tidal information was obtained from the Galway Docks sea climate change tide Gauge, published Spring and Neap tide ranges from the Nautical Almanac and Corrib River flows from the published information for the Wolfe Tone Bridge Gauge (refer to OPW hydrometric Section

8.4.2.5 Discussion on hydrodynamics

The flood simulations list 5 to 7 above were performed to quantify the potential impact that the harbour extension development would have on upstream flooding in the Claddagh basin and docks area, north and northwest of Nimmo's pier. These simulations show clearly no discernible impact on the peak flood levels.

It should noted that simulation 6 which is the occurrence of the 100-year Corrib flood with the historical maximum recorded storm surge of 3.49 m O.D. Malin (which is a 25-year tide event) will arise less frequently than a combined 200-year design flood event.

Larger storm surge events of 4 to 4.5 m OD will result in even greater water depth which reduces the frictional losses and so the Corrib flood flow will have an even lower impact on the flood then arising as a consequence of the sea level.

The simulation runs for these cases showed no discernible increase in peak flood level within the Claddagh basin and within the existing docks area. The flood risk assessment study demonstrates that the proposed development will not negatively impact on the flood risk in the adjacent areas.

The Spring tide hydrodynamics under median Corrib flow (82 m^3/s) are presented for mid-ebb and mid-flood tidal stages (*i.e.* most active) in Figures 8.4.4 to 8.4.7 with and without the proposed harbour extension. These figures illustrate the potential impact that the structure will have on tidal and river flow circulation. The principal impact is the deflection of the Corrib outflow more westwards giving it a more southerly heading towards Mutton Island resulting in a concentration of flow along the proposed dredged channel past the marina breakwater and southwards. The simulation indicates slightly higher velocities and more persistent flow in the new channel over the tidal cycle than that predicted for the existing dredge channel (without the development). One other principal impact on the flow regime is an increase in tidal velocity past the head of the southern breakwater protecting the commercial Harbour area with velocities increasing from 0.1 – 0.15 m/s to 0.2 - 0.25 m/s.

On Neap tides (see Figures 8.4.8 to 8.4.11) there is little discernible impact on velocities except for the dredged approach channel to Galway docks. Neap tides result in extremely slack tidal velocities of 0.05 to 0.15 m/s in the area of interest north of Mutton to Hare Island. The dominant influence on velocities is the river flow.

Simulation output for the 100-year Corrib Flood combined with mean Spring tide conditions are presented in Figures 8.4.12 to 8.4.15 for the with and without port development scenarios. Under flood conditions, the proposed development funnels the Corrib flood flow southwards along the dredge channel with little variation in the direction (rectilinear flow). This results in consistently higher velocities in the proposed westerly approach channel. Under the existing case, the Corrib plume velocities are slightly lower and vary in direction from southwards to southeastward depending on the tidal stage.

Note in the following figures (8.4.4 to 8.4.15), the various colour bands represent depth averaged current speed (m/s) and the arrows indicate flow direction.

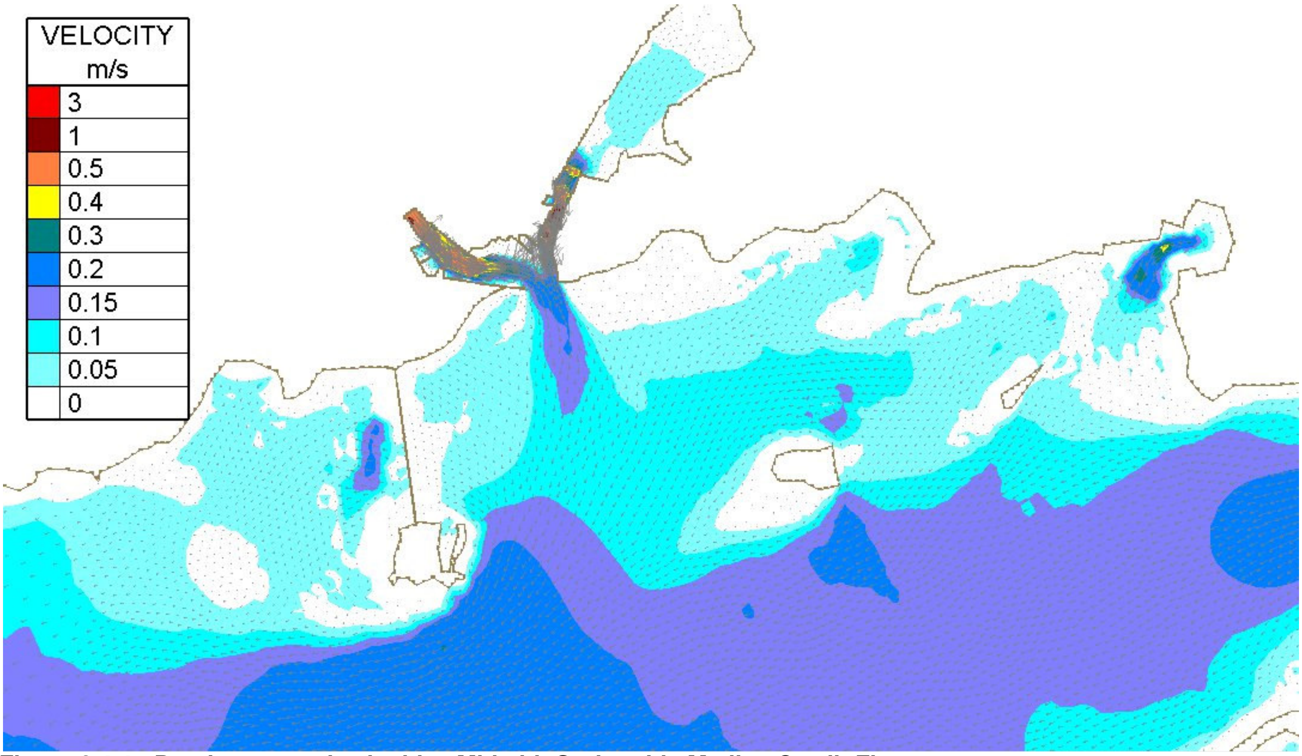


Figure 8.4.4 - Depth averaged velocities Mid-ebb Spring tide Median Corrib Flow

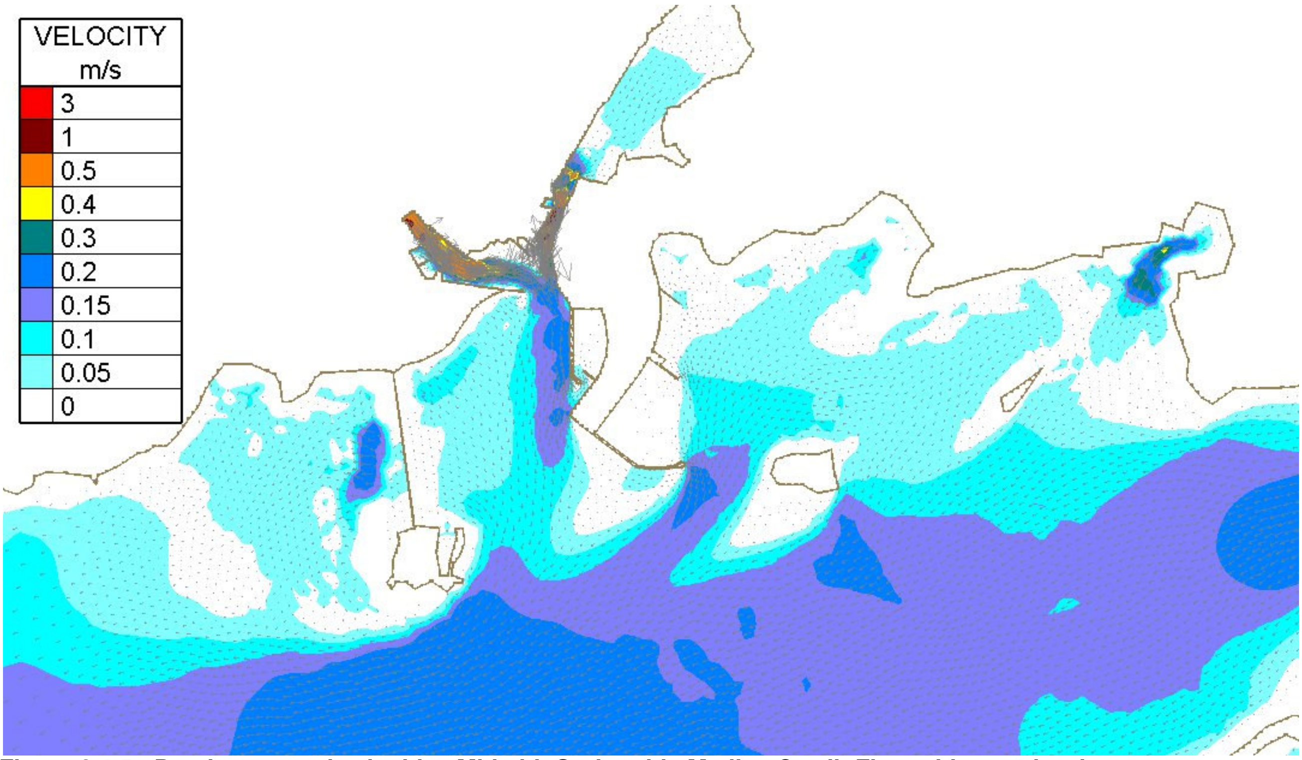


Figure 8.4.5 - Depth averaged velocities Mid-ebb Spring tide Median Corrib Flow with new development

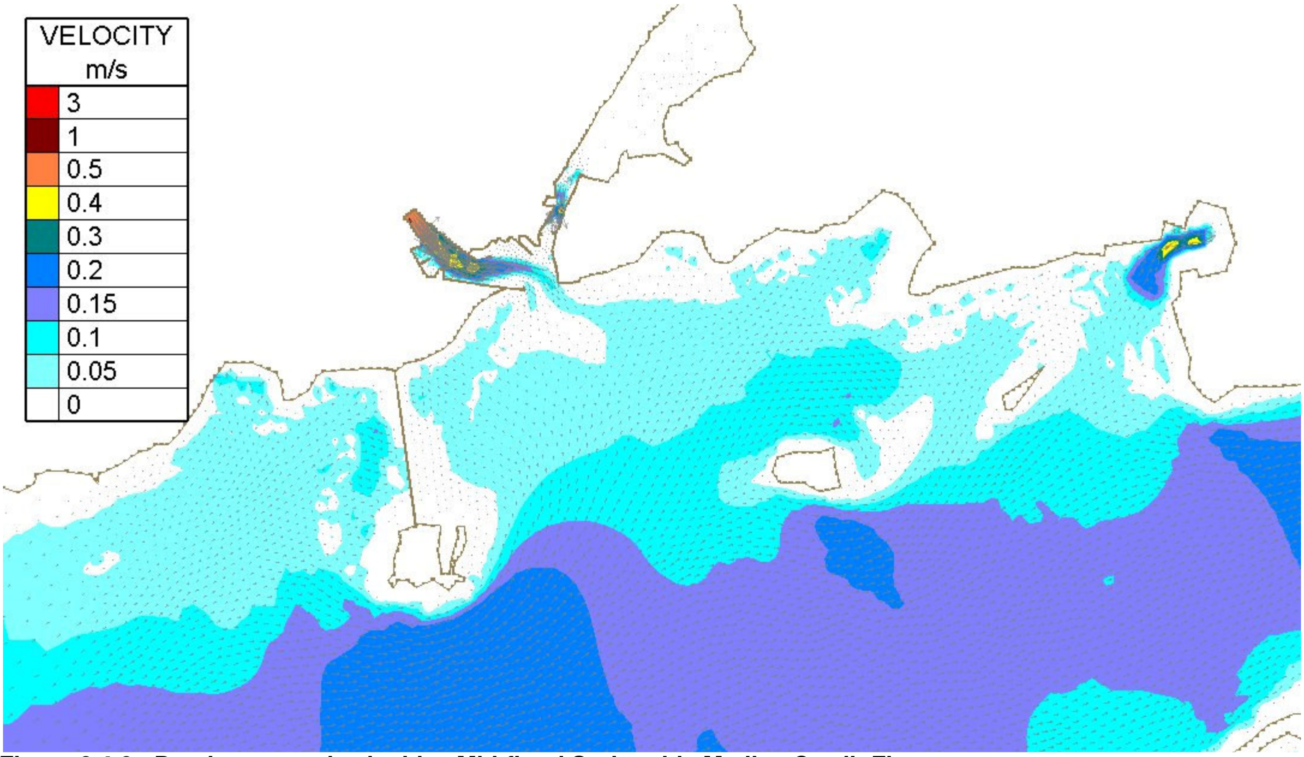


Figure 8.4.6 - Depth averaged velocities Mid-flood Spring tide Median Corrib Flow

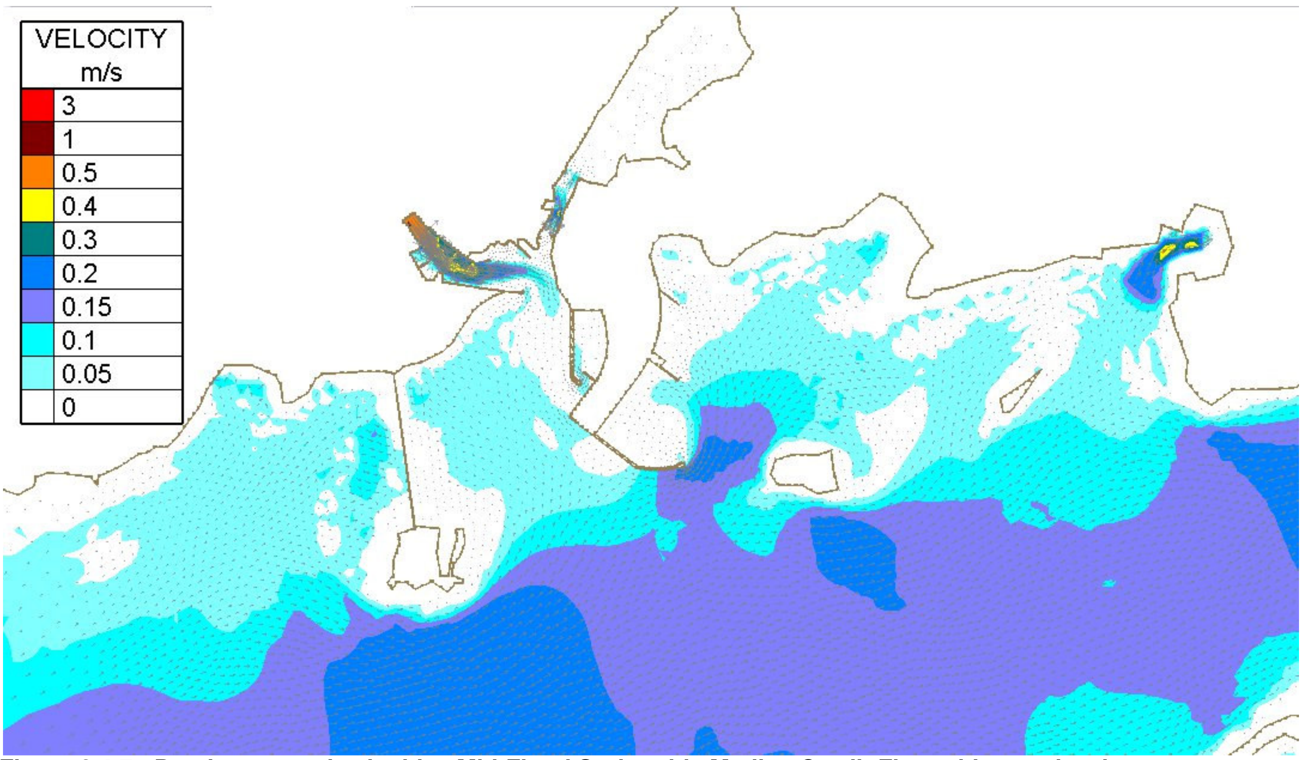


Figure 8.4.7 - Depth averaged velocities Mid-Flood Spring tide Median Corrib Flow with new development

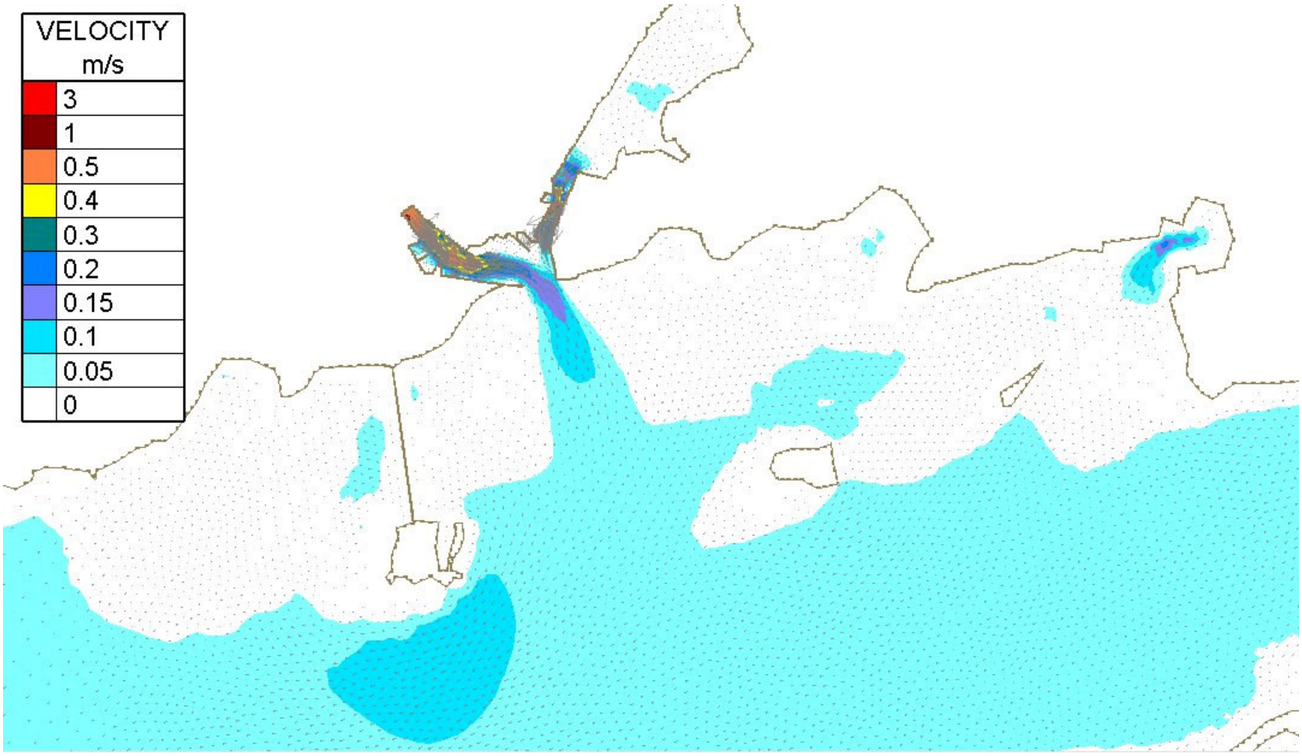


Figure 8.4.8 - Depth averaged velocities Mid-ebb Neap tide Median Corrib Flow

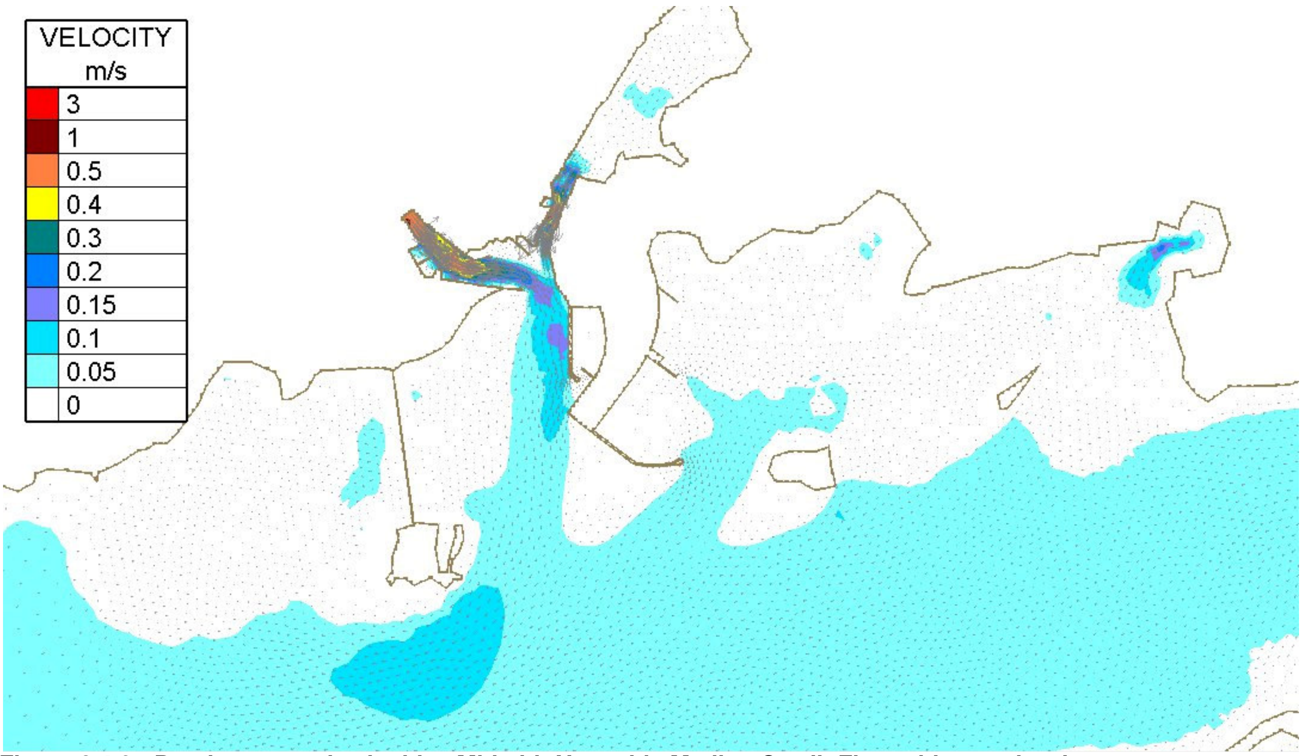


Figure 8.4.9 - Depth averaged velocities Mid-ebb Neap tide Median Corrib Flow with new development

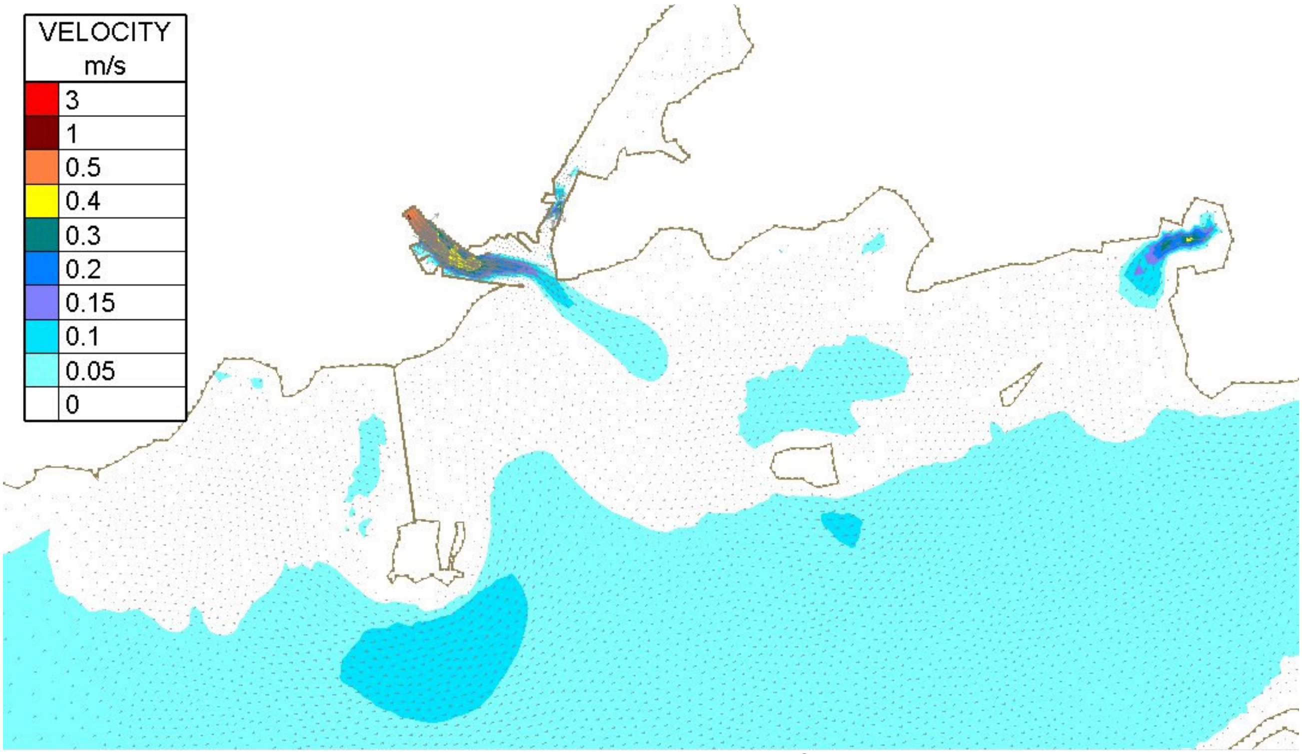


Figure 8.4.10 - Depth averaged velocities Mid-Flood Neap tide Median Corrib Flow

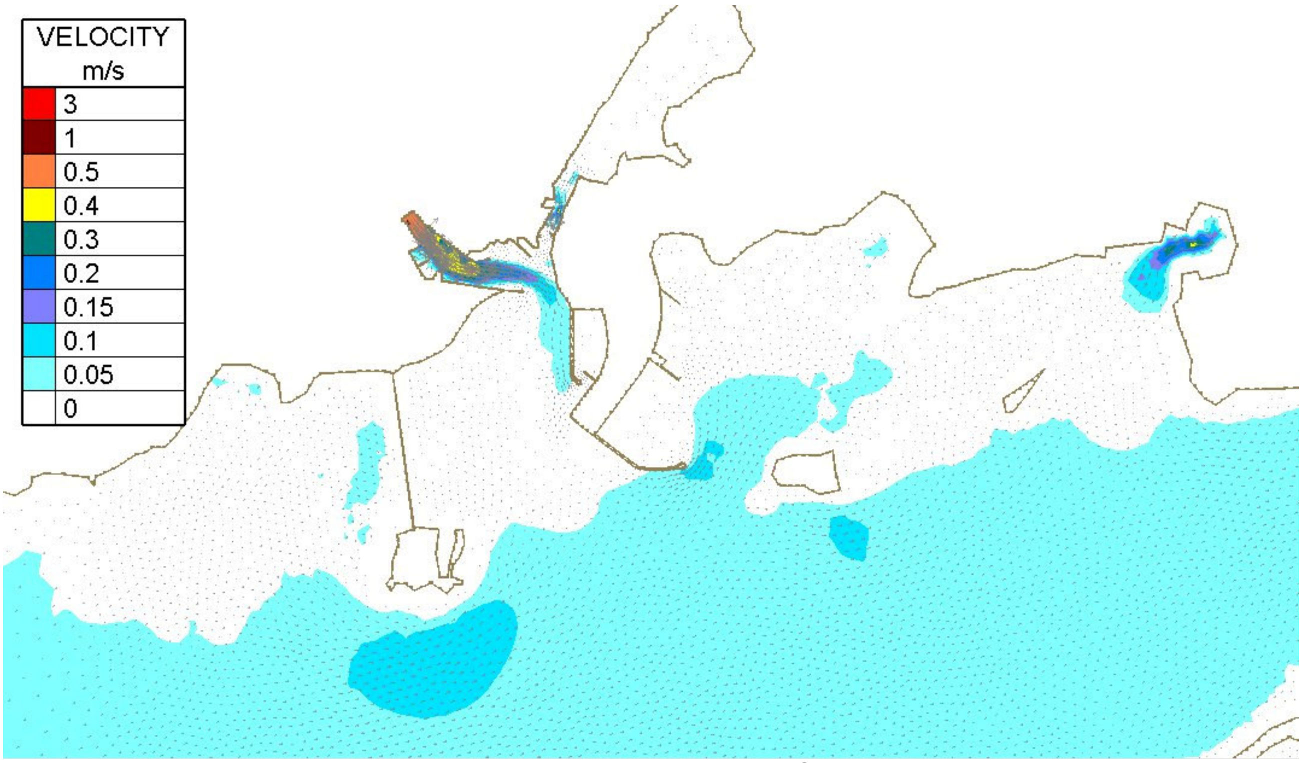


Figure 8.4.11 - Depth averaged velocities Mid-Flood Neap tide Median Corrib Flow with new development

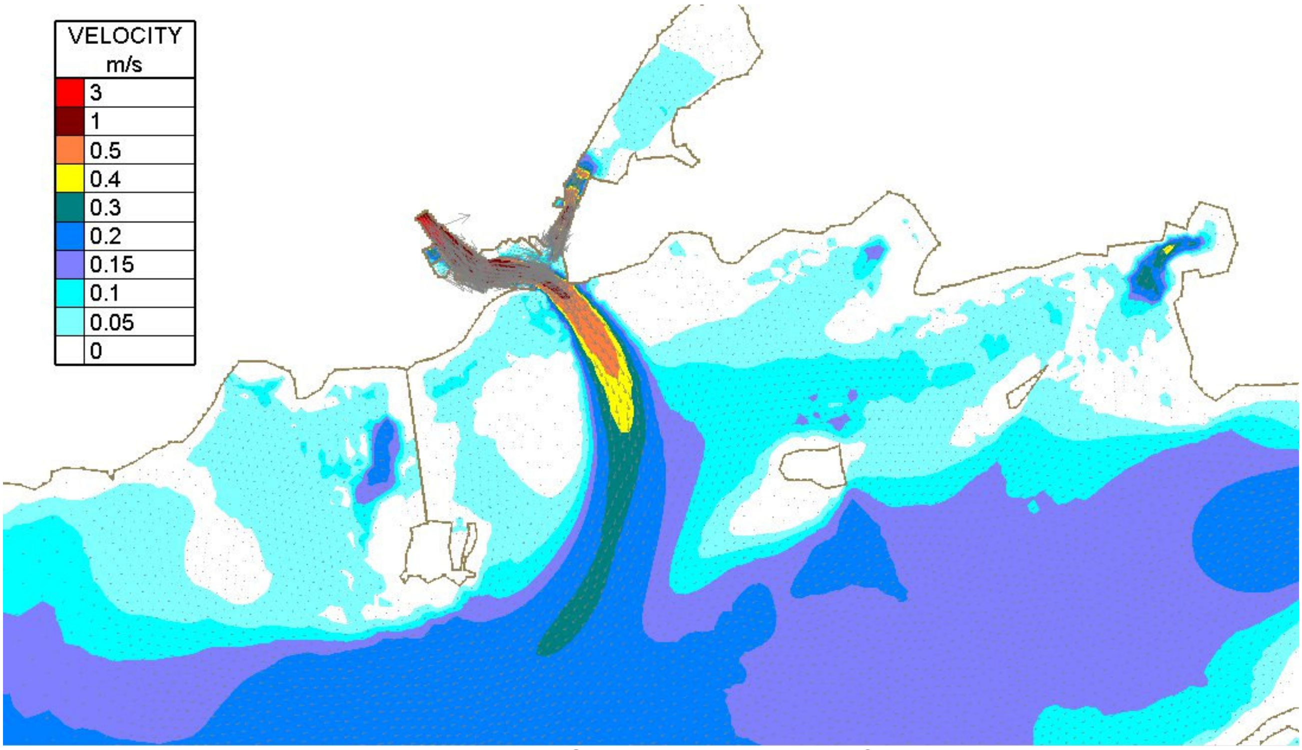


Figure 8.4.12 - Depth averaged velocities Mid-ebb Spring Tide and 100 year Corrib Flow

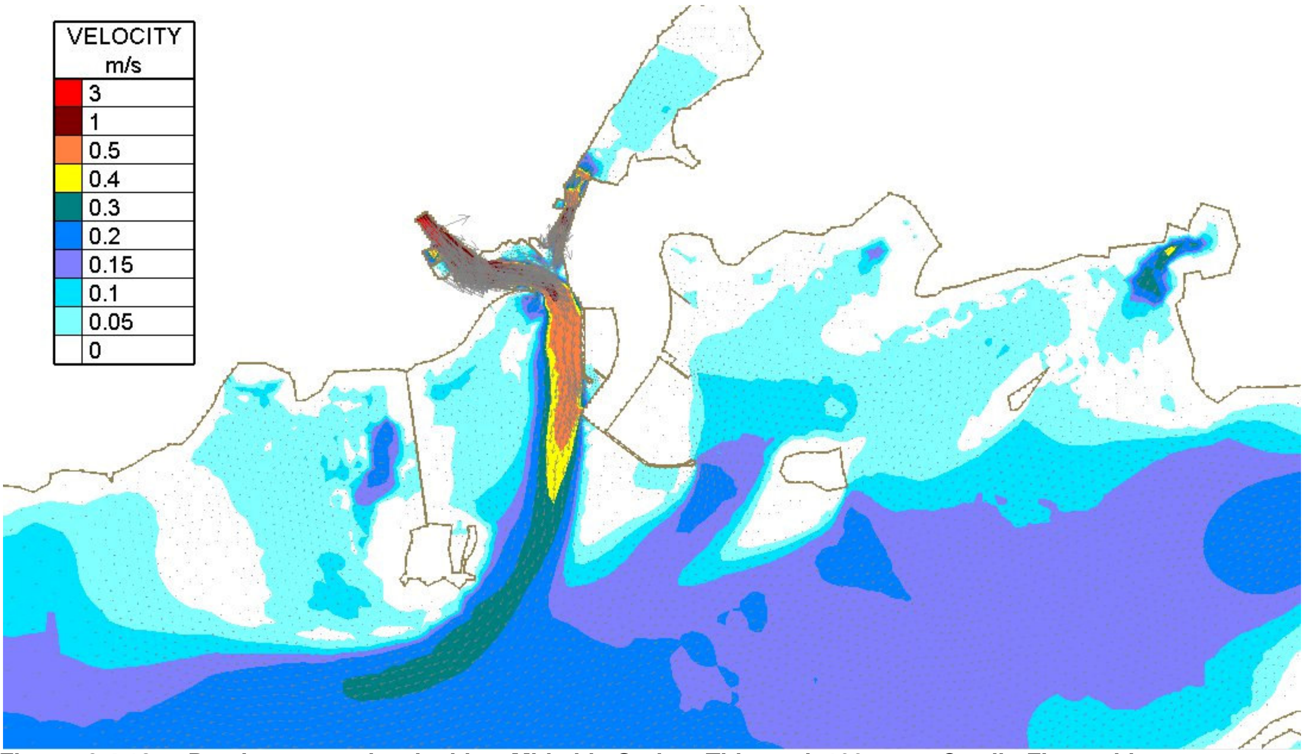


Figure 8.4.13 - Depth averaged velocities Mid-ebb Spring Tide and 100 year Corrib Flow with new development

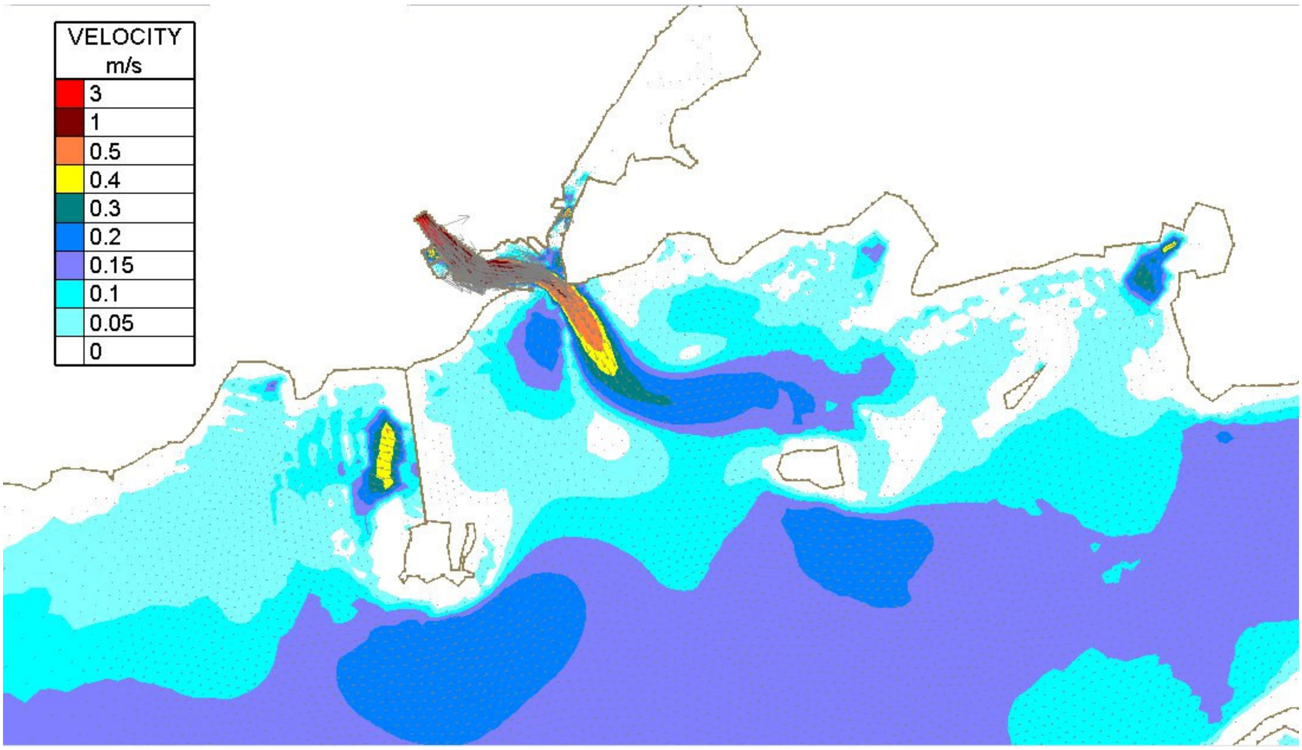


Figure 8.4.14 - Depth averaged velocities Mid-Flood Spring tide and 100 year Corrib Flow

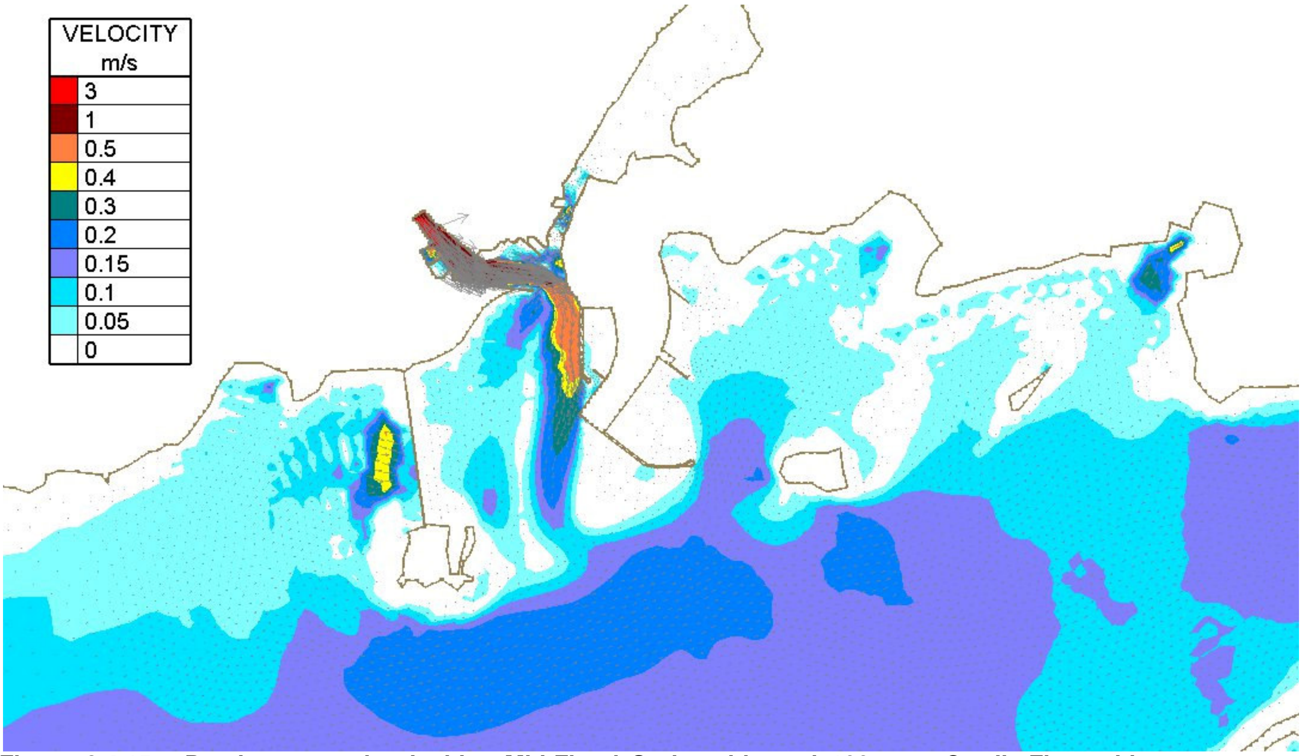


Figure 8.4.15 - Depth averaged velocities Mid-Flood Spring tide and 100 year Corrib Flow with new development

8.4.2.6 Discussion on sedimentation

One of the features of the proposed harbour extension development is the creation of a dredged access channel and the deflection to the west of the Corrib inflow and outflow stream to and from the existing harbour area, Lough Atalia and the Corrib estuary as a result of the reclamation area and associated breakwater area. The implications of this on sedimentation is shown to be minor in respect to erosion and deposition impacts with the comparisons between tidal stream velocities and bed shear stresses (the bed shear stress dictates the rate of erosion and susceptibility of a location for deposition) showing only small to minor changes. The main changes in shear stresses were found to occur along the new dredged channel to Galway Docks and past the head of the southern breakwater which are beneficial in respect to maintaining the dredged channel and reducing the deposition of silt within the dredged channel.

The sedimentological survey of the area carried out found that bed sediment was generally classified as a fine to medium sand with silt content varying from 5 to 45%. The majority of the sampling stations were dominated by fine sands. The movement of the sediment on the seabed is dependent on the tidal currents and the sediment type (grain size). The tidal flow gives rise to generating shear stress along the seabed. When the shear stress increases to a critical value, the sediment will move (refer to Table 8.4.1 below for critical shear stress values for different sediment sizes). Shear stresses above 0.1 N/m^2 will erode the silt fraction with the fine to medium sand requiring shear stresses of 0.18 to 0.23 N/m^2.

Sediment size classification and critical shear stress for erosion		
Material Type	**Sediment Size (mm)**	**Critical shear stress (N/m^2)**
Fine gravel	6	5.24
Very fine gravel	3	2.16
Very coarse sand	1.5	0.83
Coarse sand	0.75	0.37
Medium sand	0.38	0.23
Fine sand	0.19	0.18
Very fine sand	0.09	0.14
Coarse silt	0.047	0.11

Table 8.4.1 - Sediment size classification and critical shear stress for erosion

Figures 8.4.16 – 8.4.39 present the computed bed shear in N/m^2 for the Spring, Neap and river flood simulations (hydrodynamic simulations 3, 4, and 5) with and without the proposed development. The Spring tide simulation presented in Figures 8.4.16 to 8.4.23 show high erosive velocities in the Corrib estuary upstream of Nimmo's pier and the Lough Atalia entrance channel for both existing and proposed cases. At the mouth to Lough Atalia, the mid ebb and flood velocities are high due to the channel entrance giving rise to correspondingly higher shear stresses. It should be noted similar to the Corrib estuary bed adjacent to the Spanish Arch and Claddagh Basin, the channel bed is rocky with small boulders, cobbles, gravel and sand indicating the fines and silts have been removed as a result of the higher shear stresses. The simulations also show erosive velocities in shallows adjacent to the causeway.

The results show that the proposed development produces shear stresses during spring tides sufficient to erode silt and fine sand along the proposed dredge channel to Galway Docks and also in the approach channel past the head of the southern breakwater. This is considered desirable in respect to maintaining the dredge channels. The simulation shows no erosive impact elsewhere.

The Neap tides are sufficiently slack not to result in erosive shear stresses outside of the Corrib estuary for both proposed and existing cases and therefore no erosive impact is predicted under Neap tide conditions.

Under River Corrib flood conditions, the proposed development restricts the erosive flow to the proposed dredge channel immediately to the west; this is considered beneficial in respect to reducing the dredging maintenance requirement which is currently not very excessive (500 mm removed at approximately 10-year interval). Similar to the Spring tide simulation, shear stresses sufficient to erode fine sand are generated in the vicinity of the southern breakwater head. This is also considered beneficial as this is located in the dredge channel. No significant impacts are predicted elsewhere.

The overall conclusion is that the proposed harbour extension configuration confines the high flows and critical bed shear to the approach channel and will not result in any erosive impact elsewhere over the existing situation. This will reduce deposition in the new approach channel to Galway Docks while avoiding scour elsewhere.

8.4.2.7 Sediment from the River Corrib

The upstream characteristics of the River Corrib, with its very large lake (Lough Corrib) for settlement, results in the sediment content comprising primarily of the finer silt and sand fractions (even under flood conditions). Simulation of the fine sediment from the River Corrib showed the proposed development pushing the river plume and thus suspended sediment southwards out to sea past Mutton Island on the ebbing tide and away from the Renmore area only returning in a much more dilute plume on the flooding tide. The simulation results indicate a reduction generally of between 40 and 60% in fine sediment load east of the proposed development.

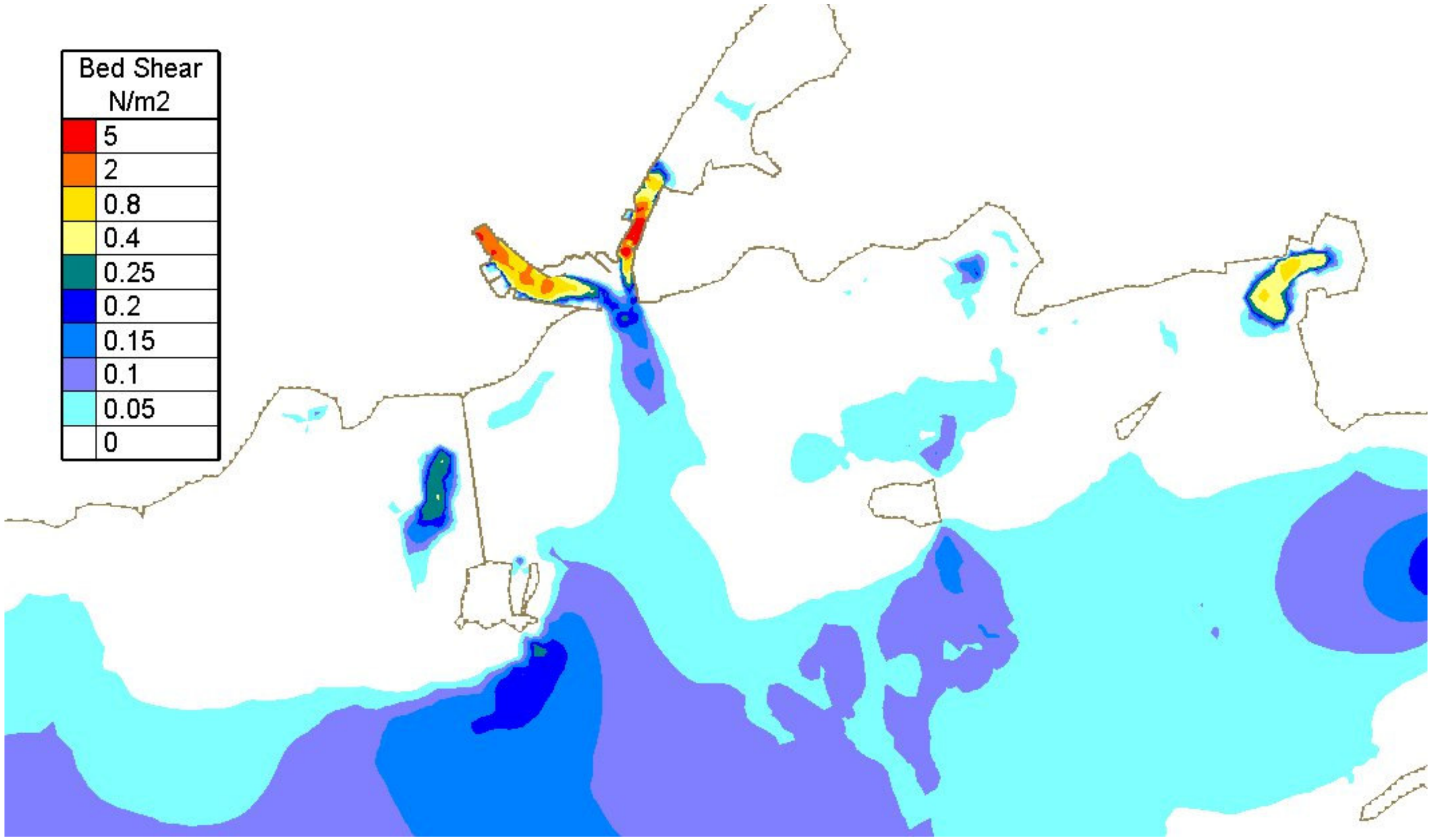


Figure 8.4.16 - Computed bed shear at mid-ebb for existing case – Spring Tide and Median River Corrib Flow (82 m3/s)

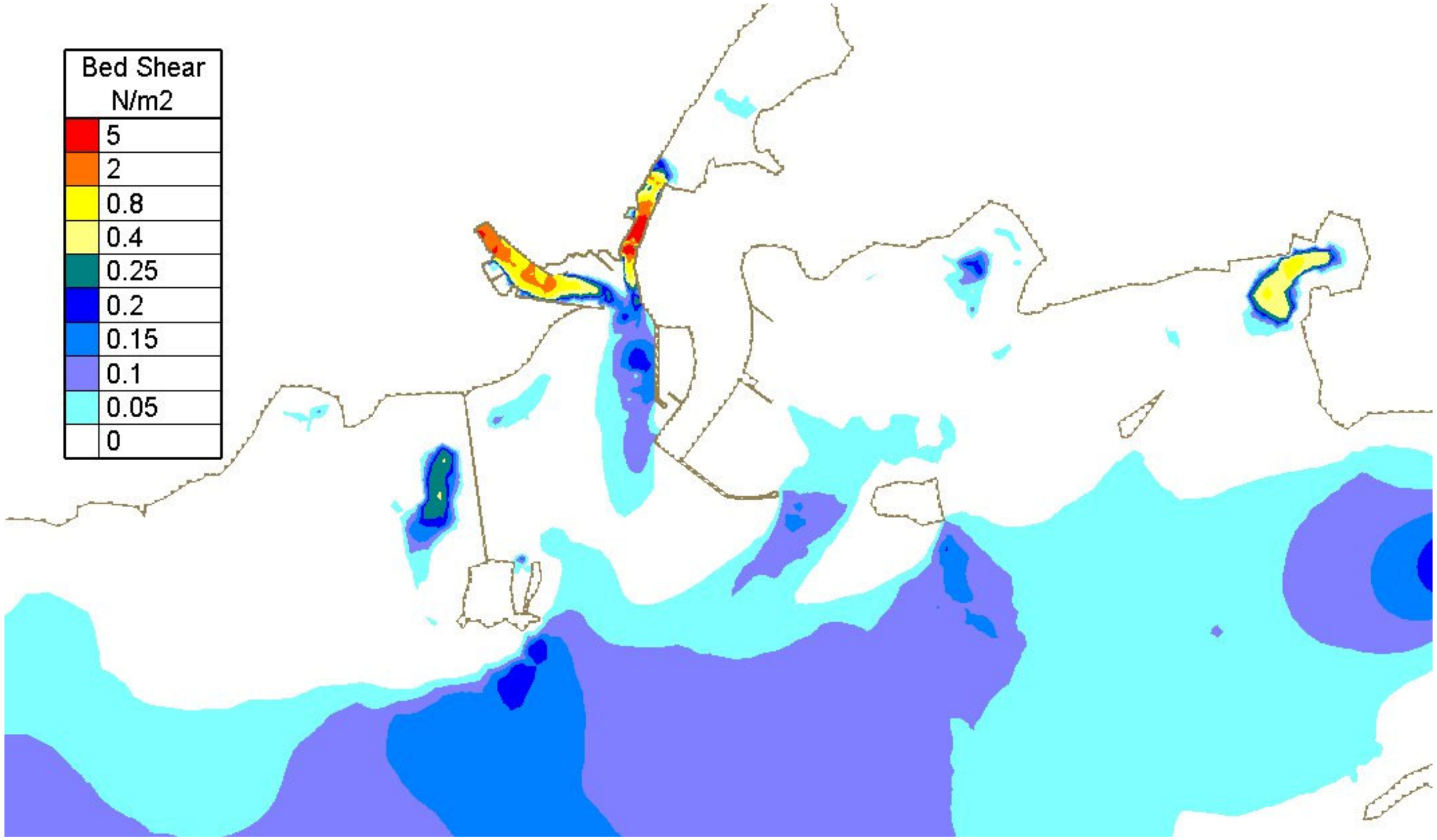


Figure 8.4.17 - Computed bed shear at mid-ebb with new harbour extension development case – Spring Tide and Median River Corrib Flow (82 m³/s)

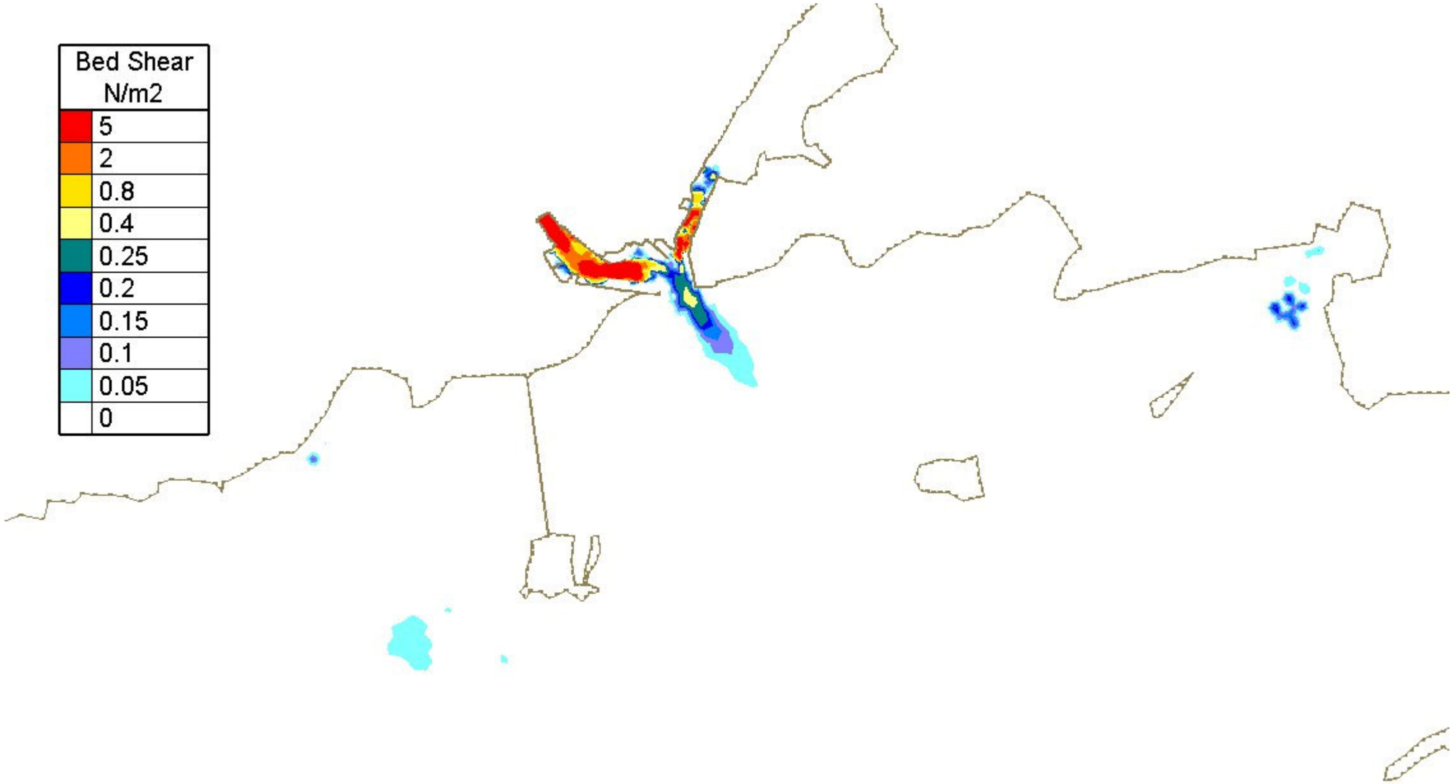


Figure 8.4.18 - Computed bed shear at low water for existing case – Spring Tide and Median River Corrib Flow (82 m3/s)

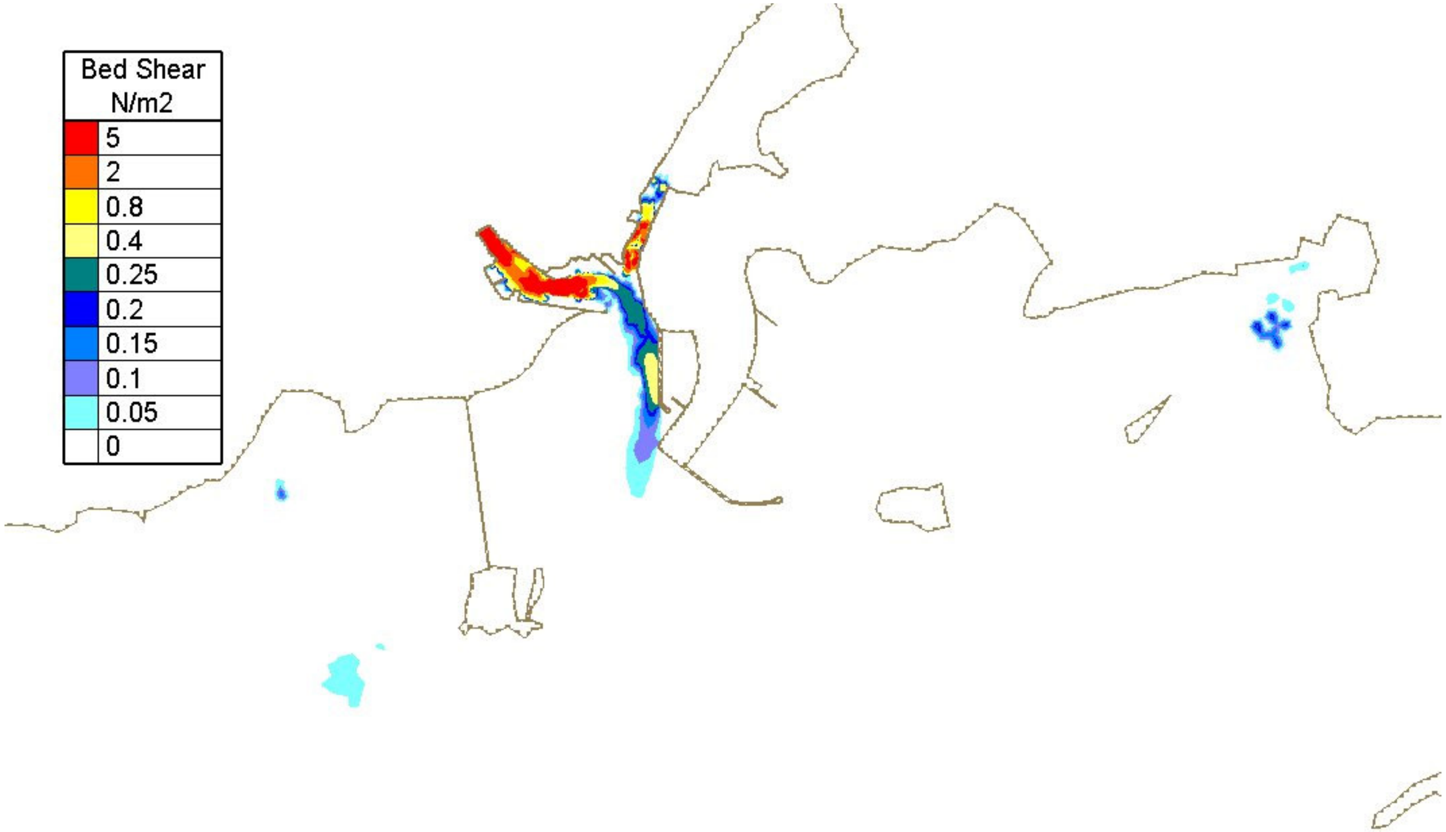


Figure 8.4.19 - Computed bed shear at low water with new harbour extension development case – Spring Tide and Median River Corrib Flow (82 m3/s)

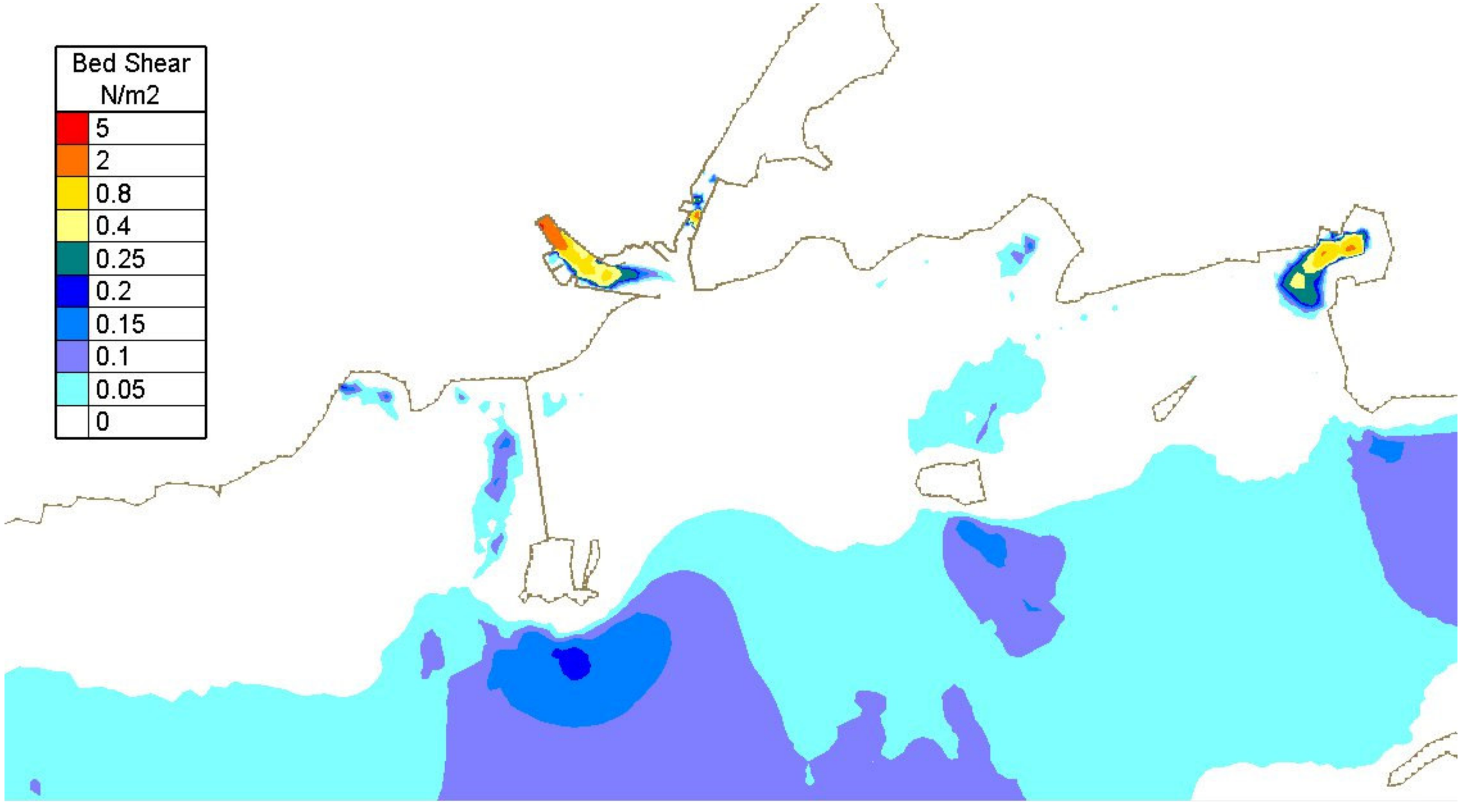


Figure 8.4.20 - Computed bed shear at mid-flood for existing case without development – Spring Tide and Median River Corrib Flow (82 m3/s)

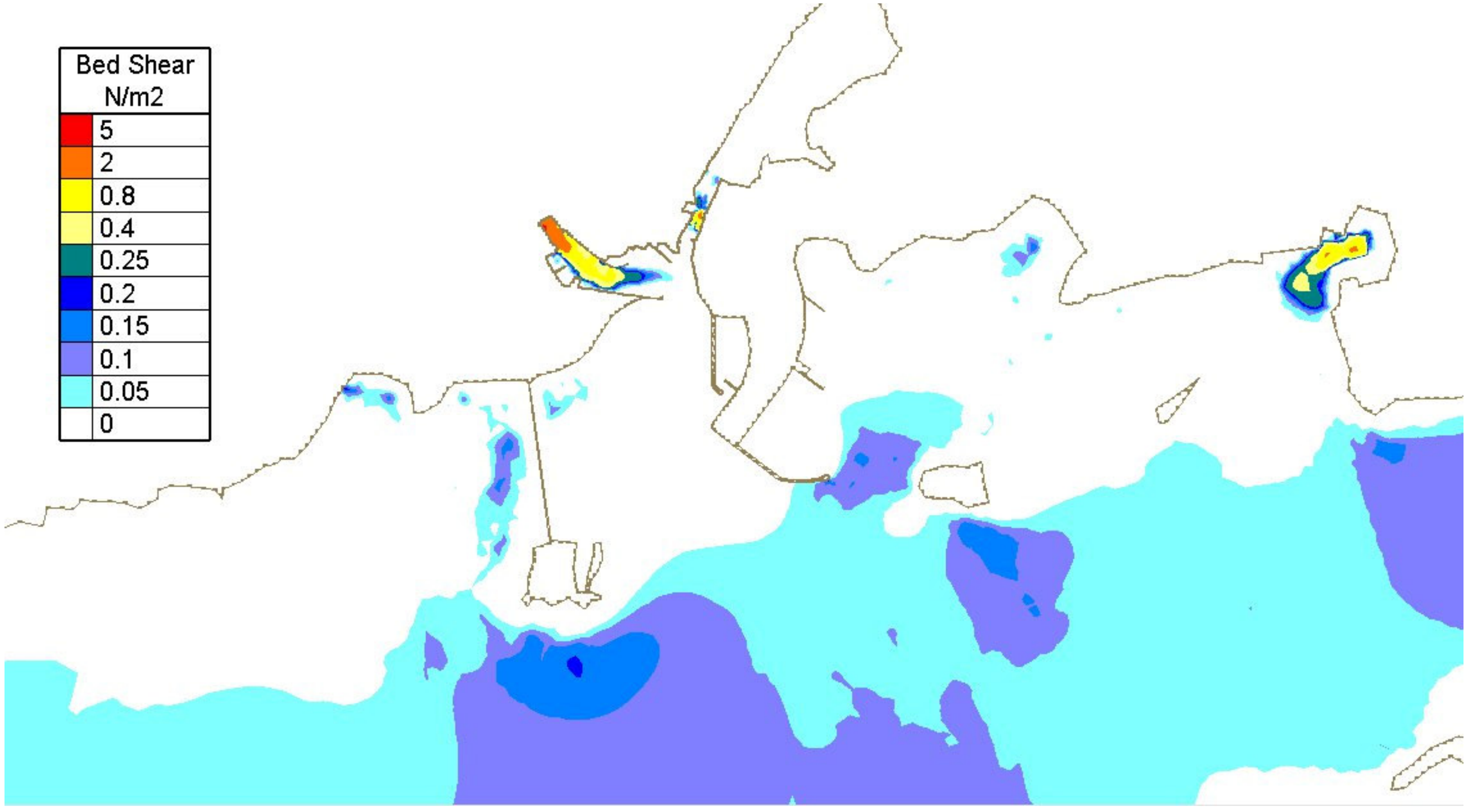


Figure 8.4.21 - Computed bed shear at mid-flood with new harbour extension development – Spring Tide and Median River Corrib Flow (82 m3/s)

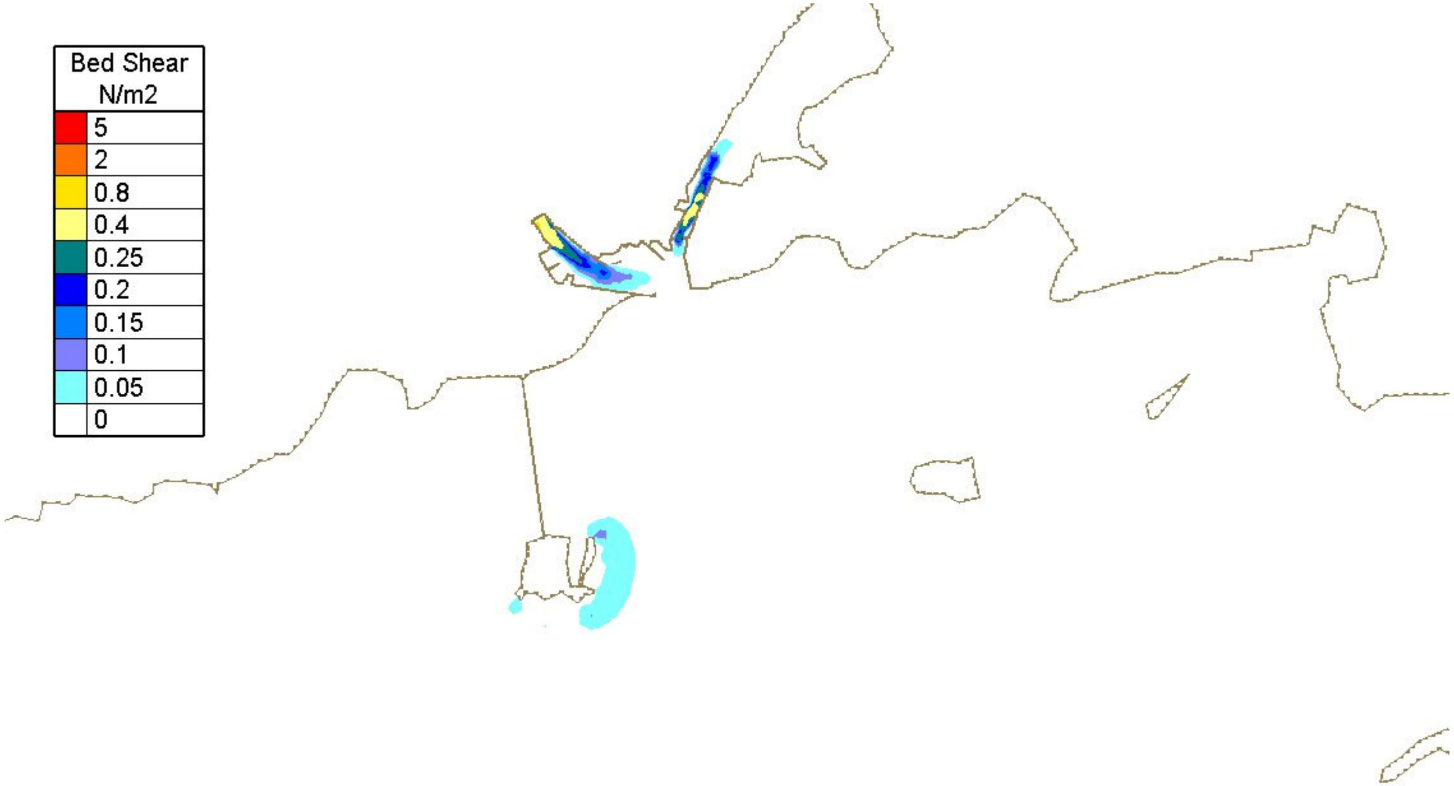


Figure 8.4.22 - Computed bed shear at Highwater for existing case without development – Spring Tide and Median River Corrib Flow (82 m3/s)

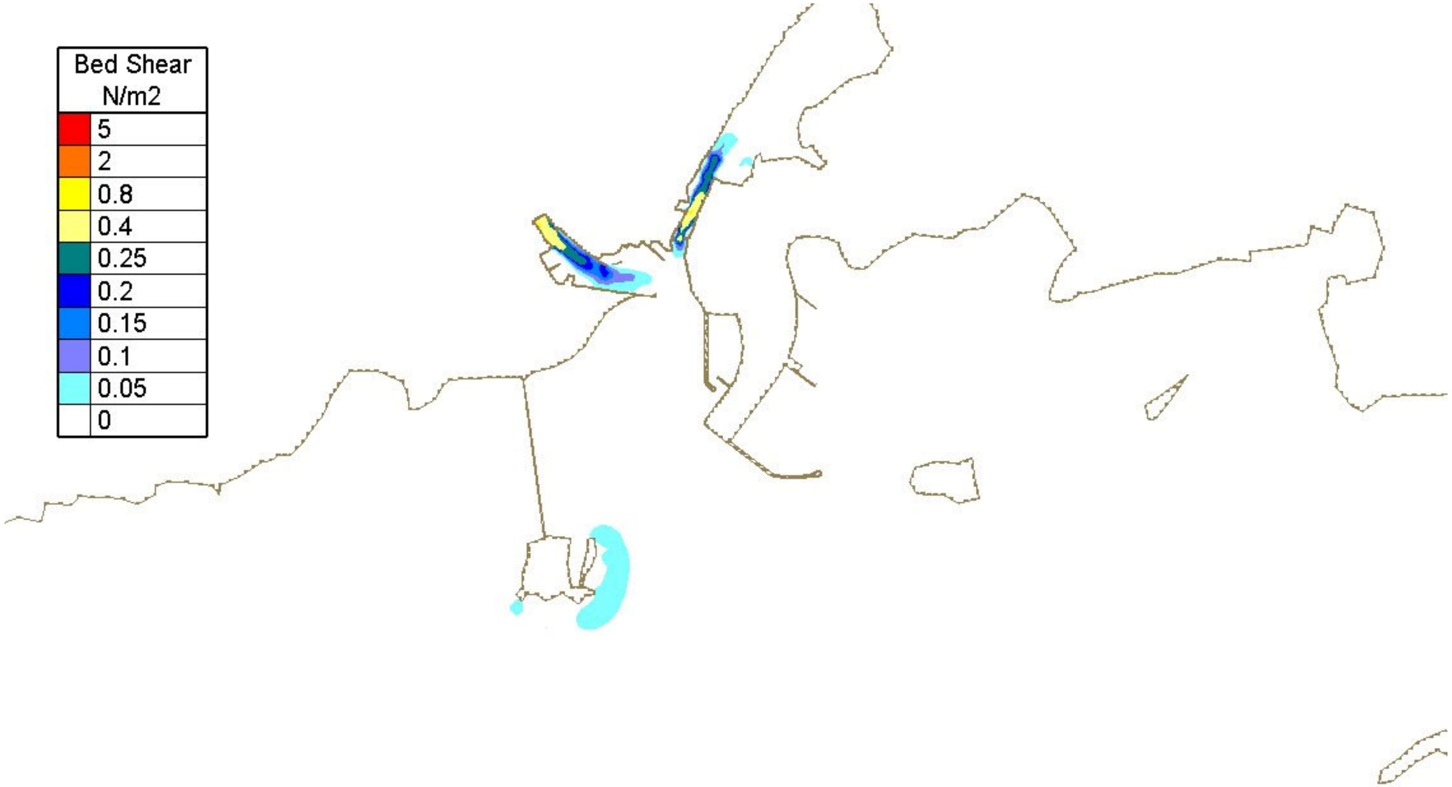


Figure 8.4.23 - Computed bed shear at Highwater with new harbour extension – Spring Tide and Median River Corrib Flow (82 m3/s)

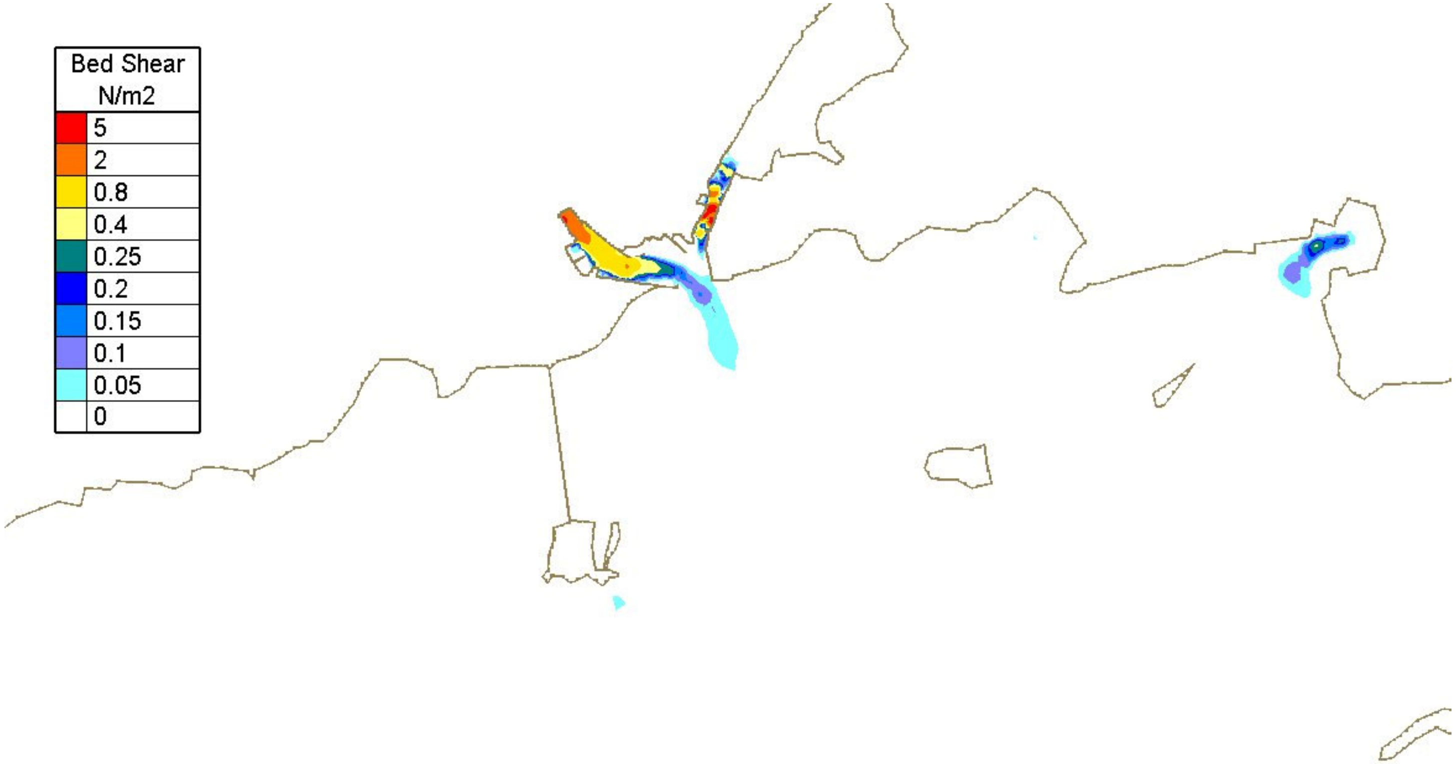


Figure 8.4.24 - Computed bed shear at mid-ebb existing case without new harbour extension development – Neap Tide and Median River Corrib Flow (82 m3/s)

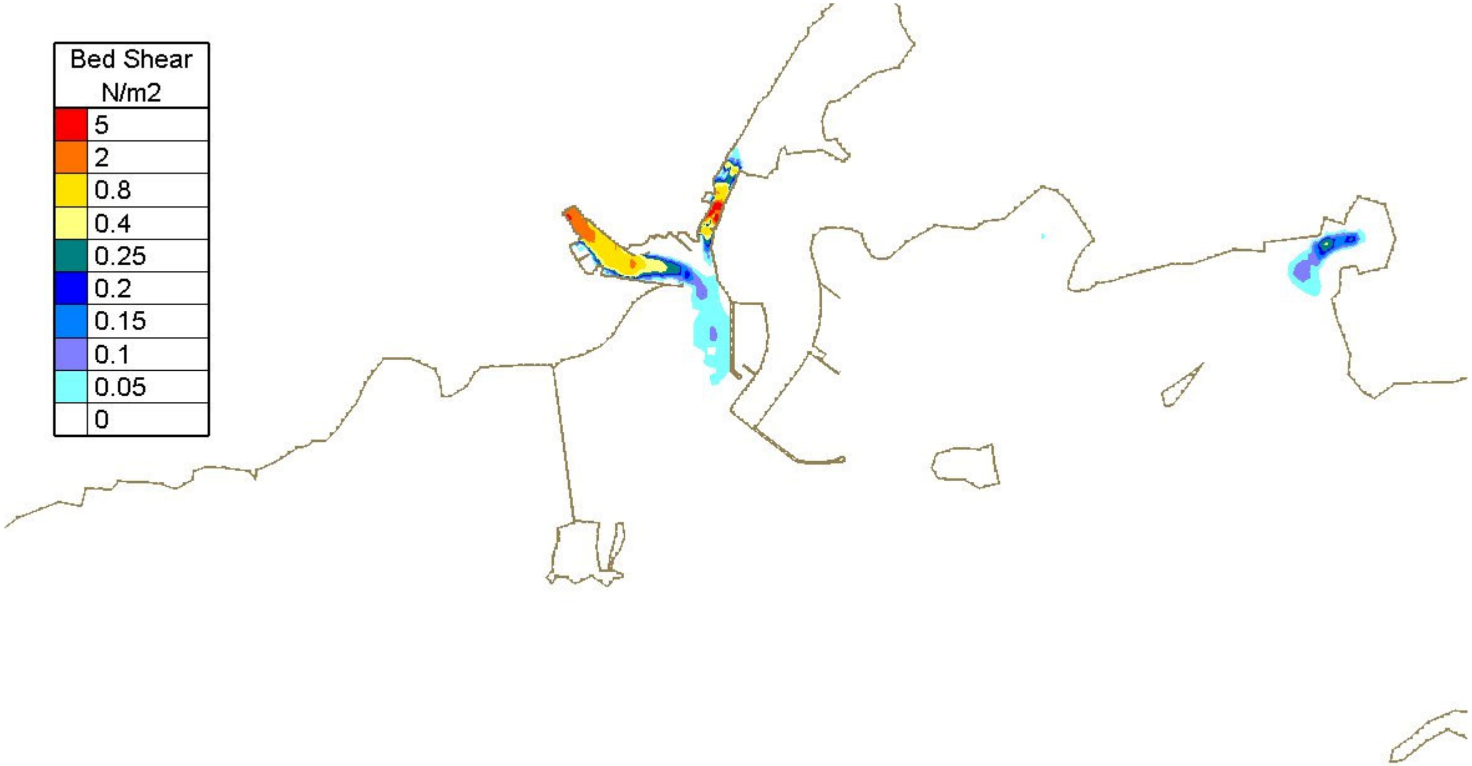


Figure 8.4.25 - Computed bed shear at mid-ebb with new harbour extension development – Neap Tide and Median River Corrib Flow (82 m3/s)

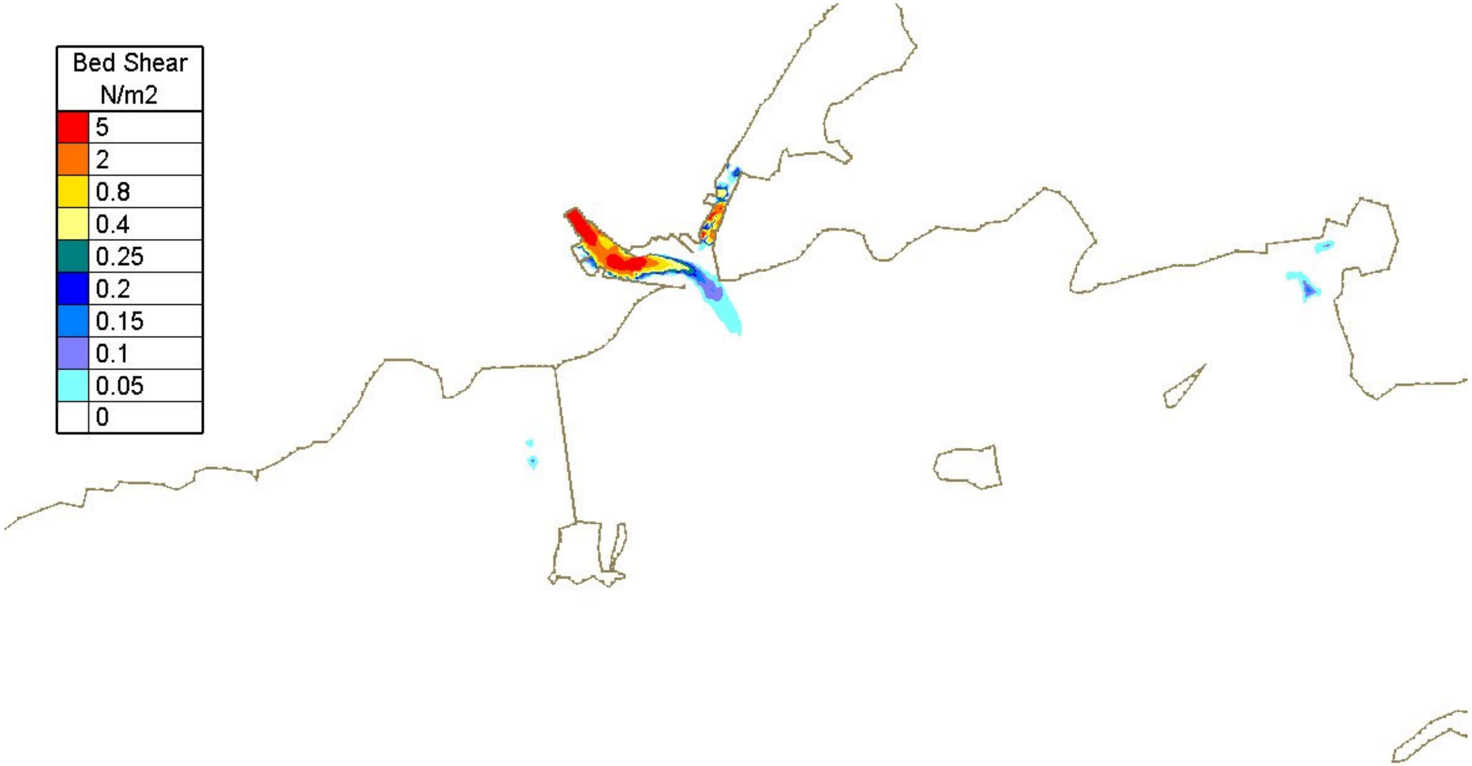


Figure 8.4.26 - Computed bed shear at low water existing case without new harbour extension development – Neap Tide and Median River Corrib Flow (82 m3/s)

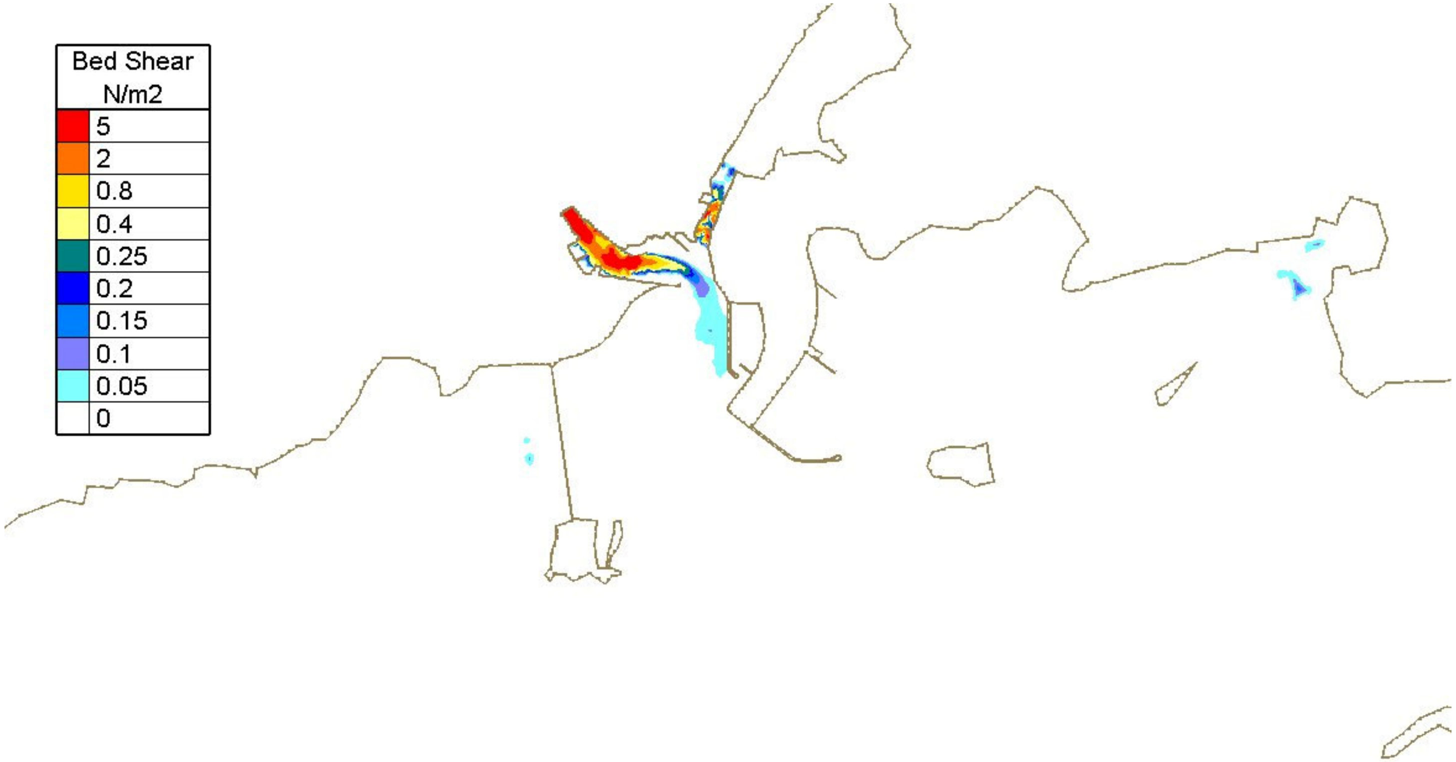


Figure 8.4.27 - Computed bed shear at low water with new harbour extension development – Neap Tide and Median River Corrib Flow (82 m3/s)

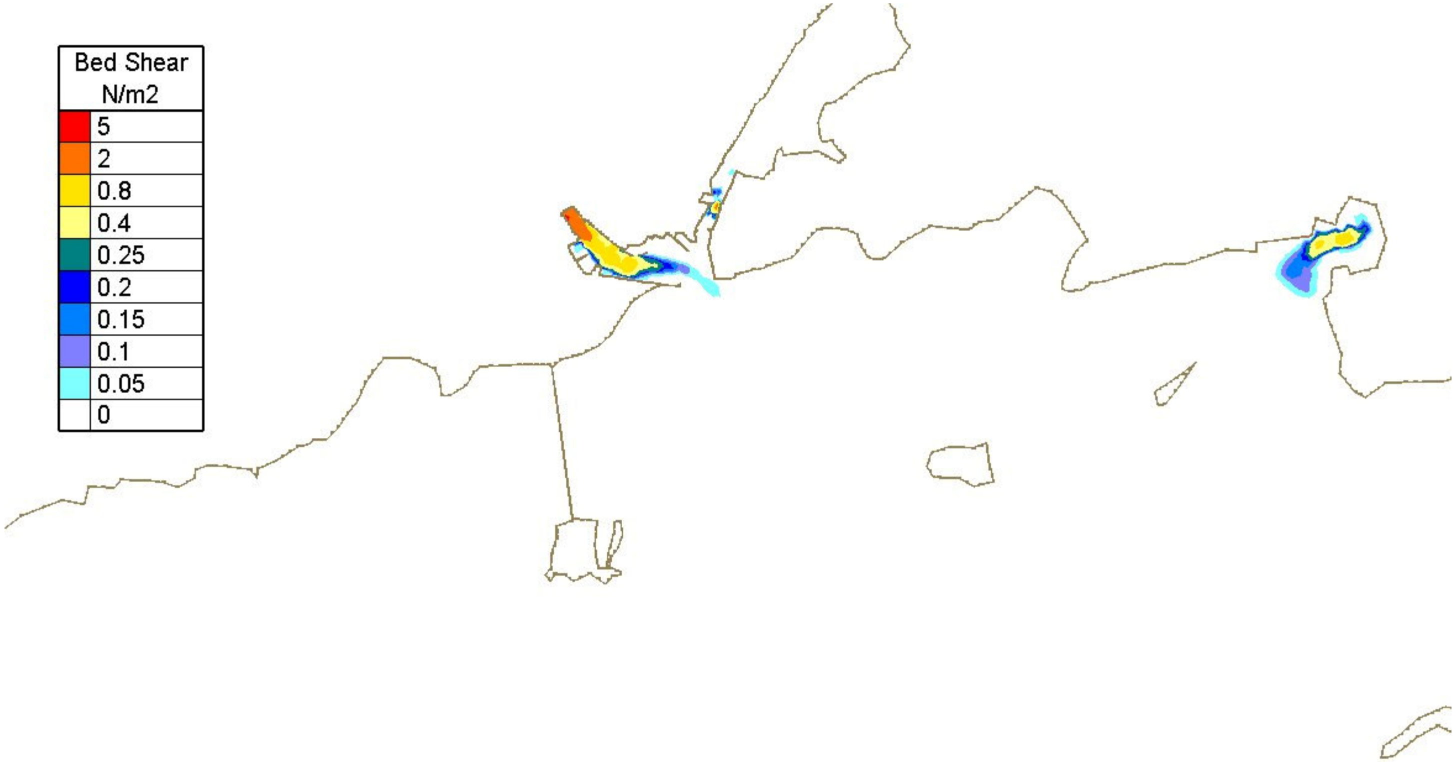


Figure 8.4.28 - Computed bed shear at mid-flood for existing case without development – Neap Tide and Median River Corrib Flow (82 m3/s)

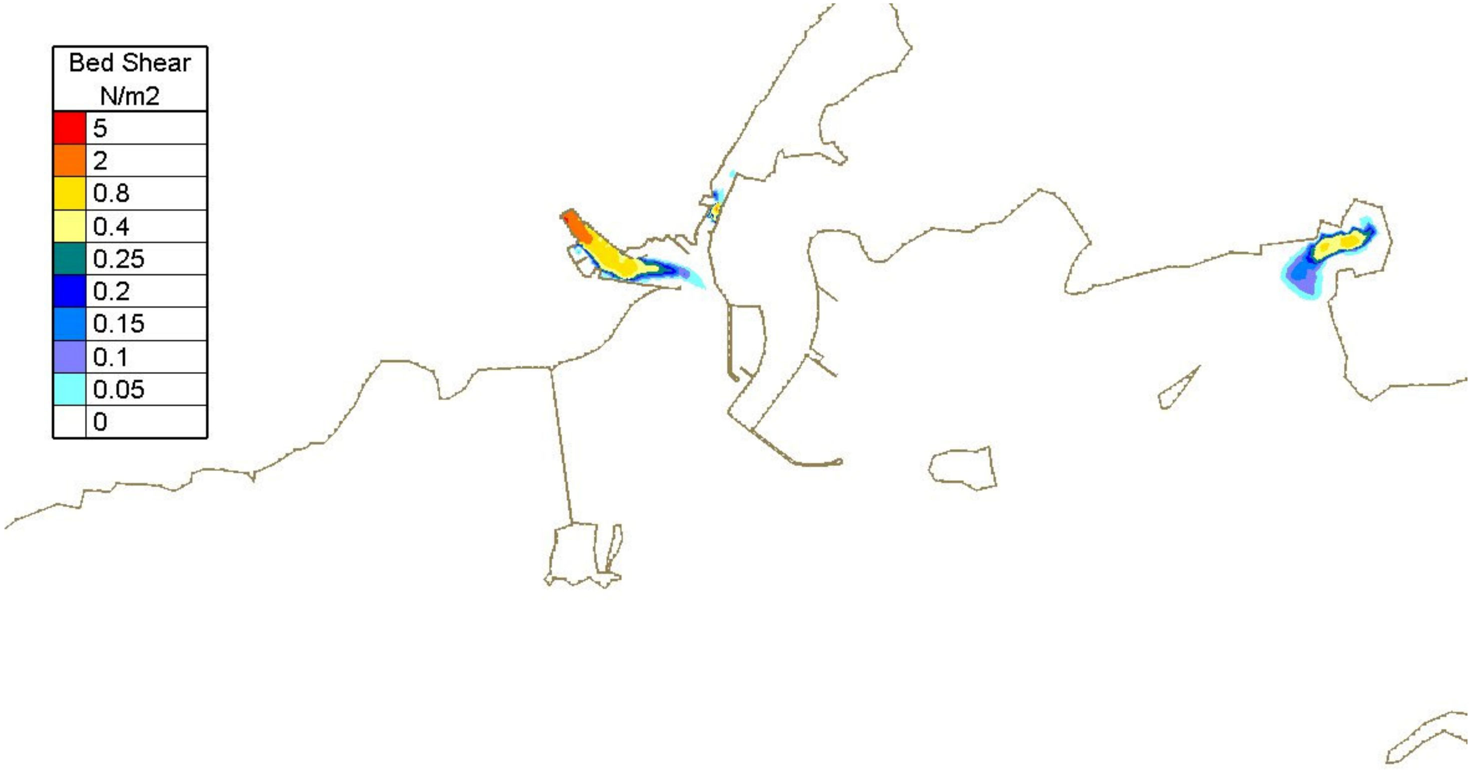


Figure 8.4.29 - Computed bed shear at mid-flood with new harbour extension development – Neap Tide and Median River Corrib Flow (82 m3/s)

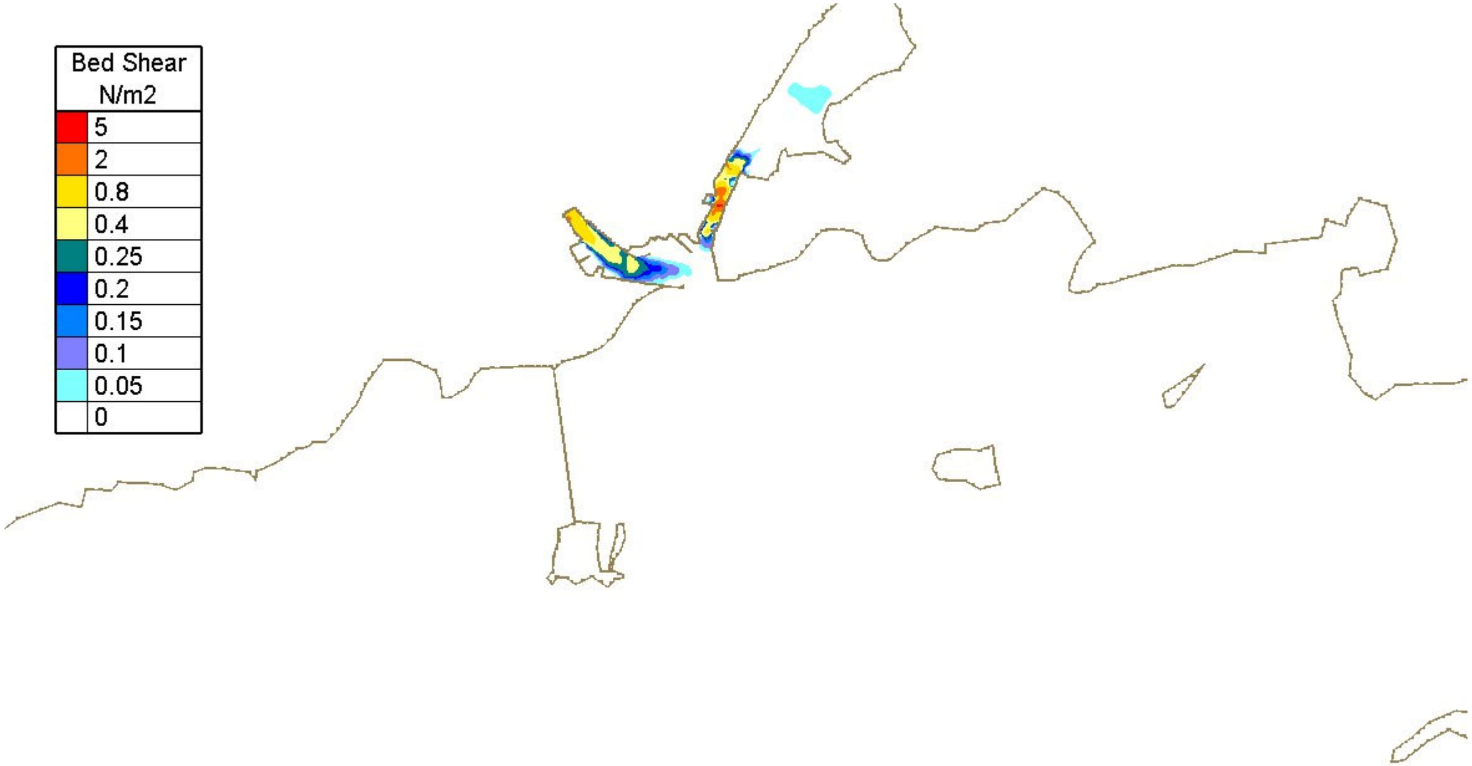


Figure 8.4.30 - Computed bed shear at mid flood high tide for existing case without development – Neap Tide and Median River Corrib Flow (82 m3/s)

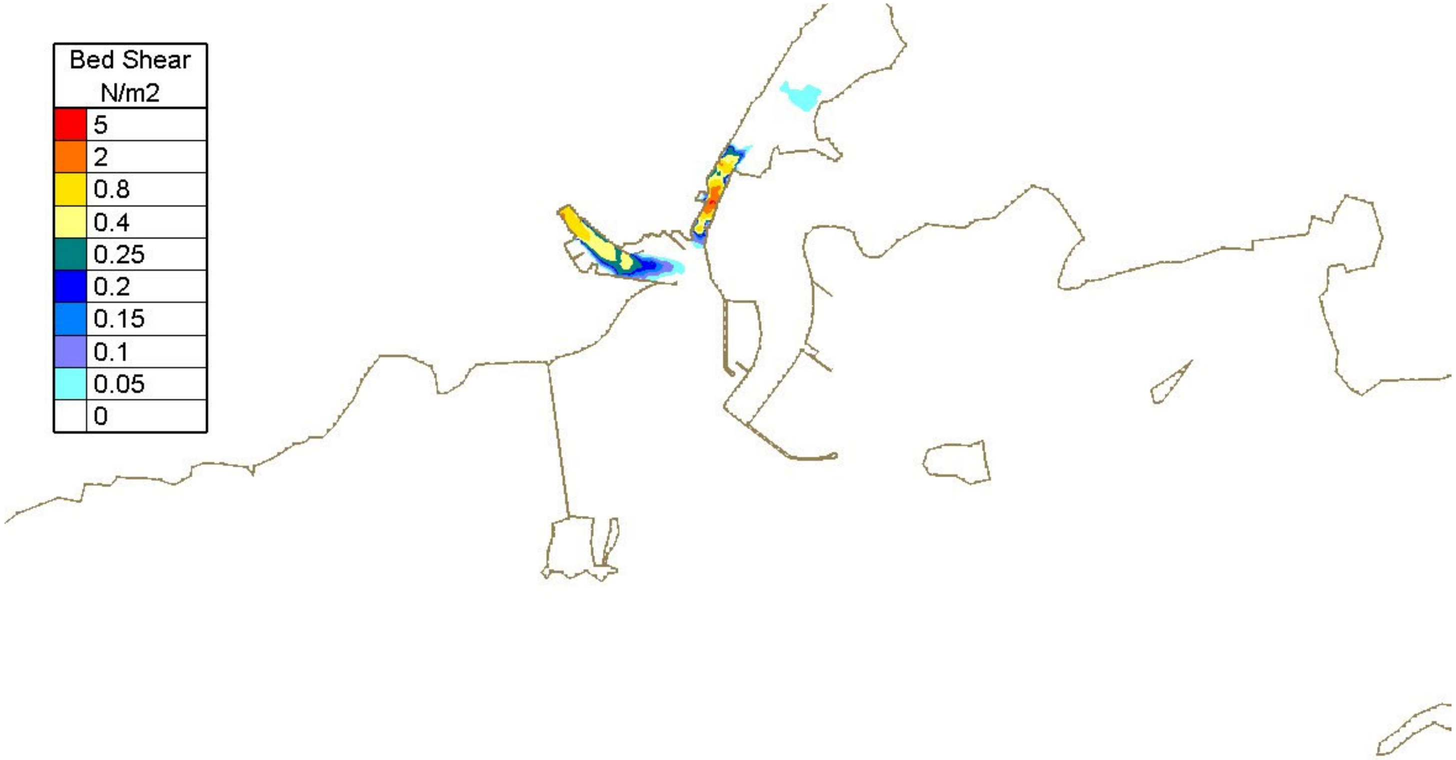


Figure 8.4.31 - Computed bed shear at mid flood high tide with new harbour extension development – Neap Tide and Median River Corrib Flow (82 m3/s)

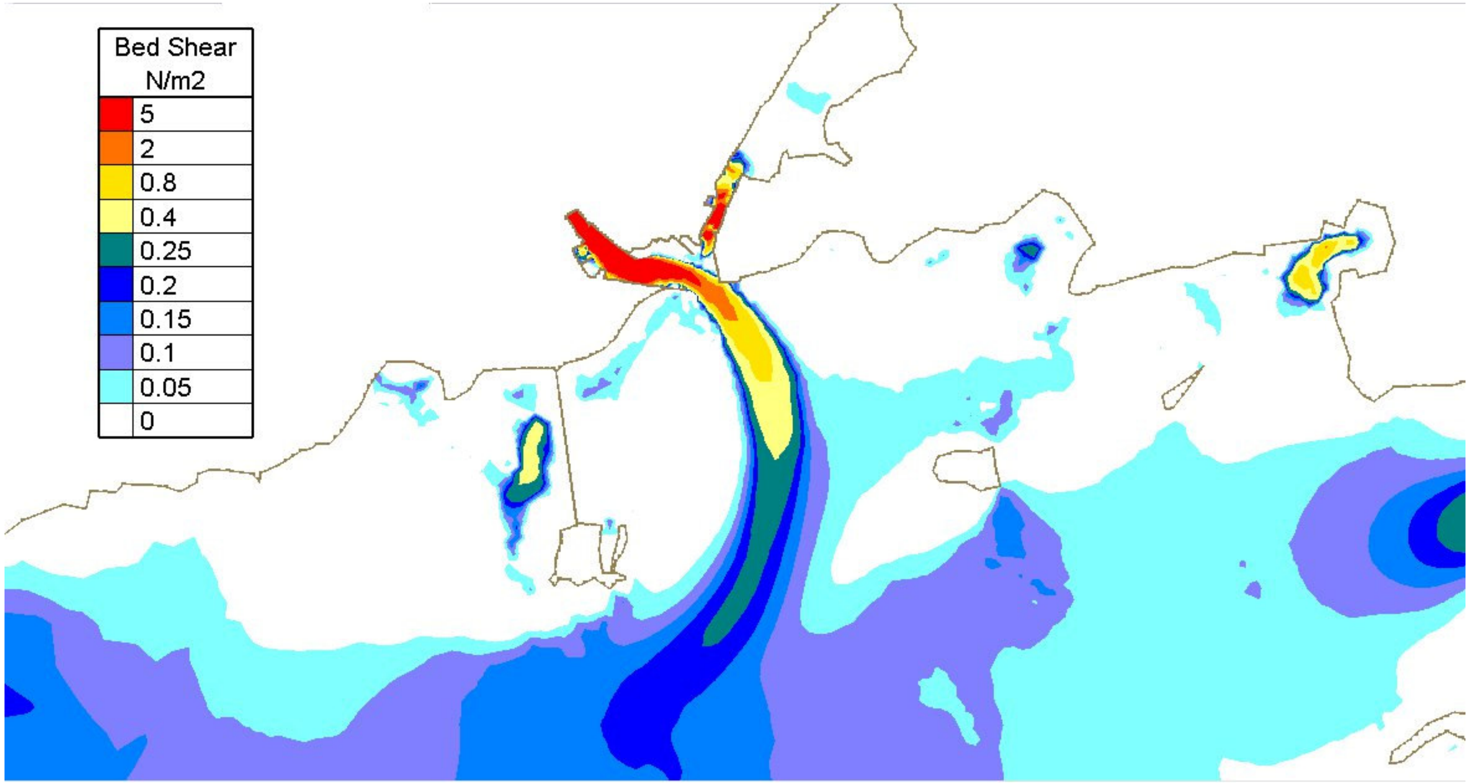


Figure 8.4.32 - Computed bed shear at mid-ebb Spring Tide with 100 year Corrib Flood Flow for existing case

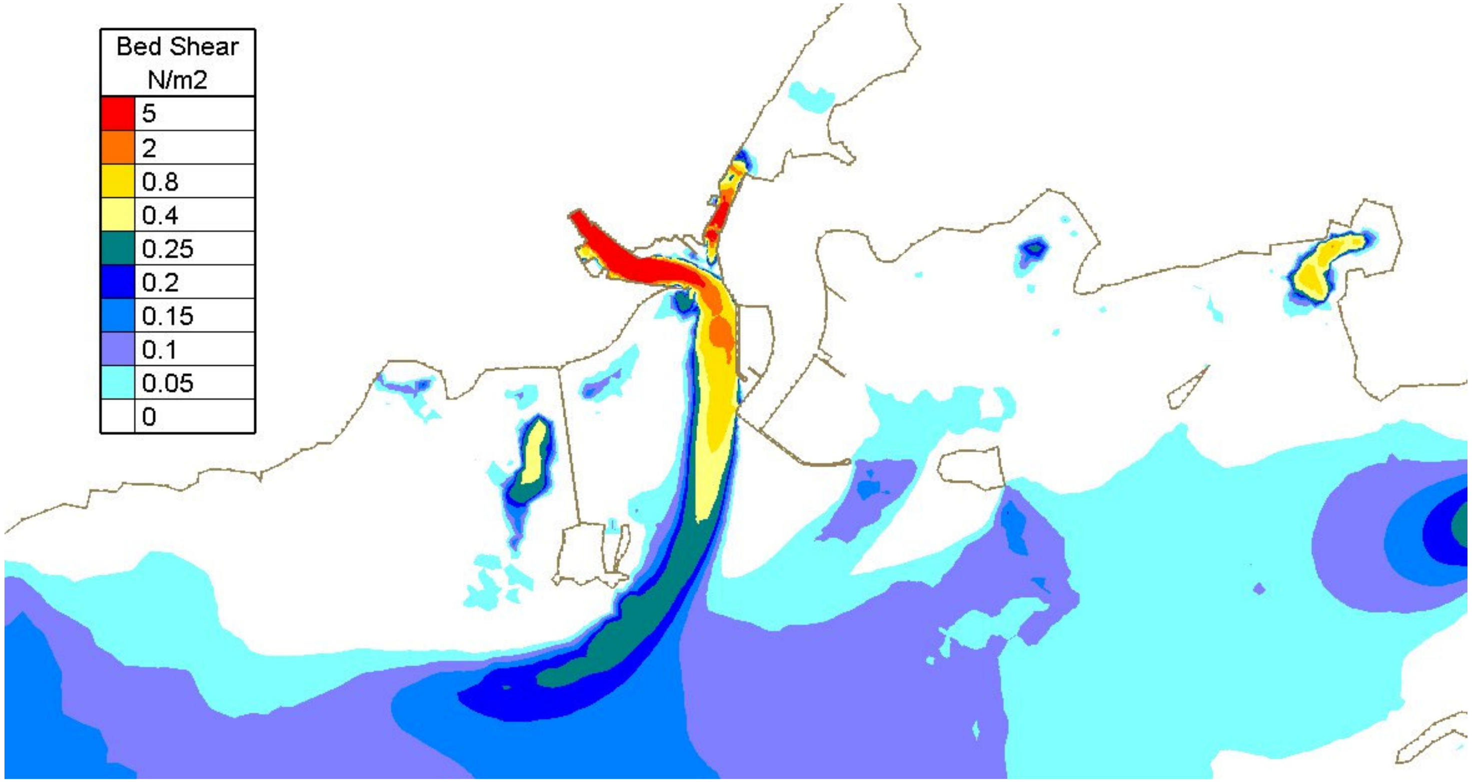


Figure 8.4.33 - Computed bed shear at mid-ebb Spring Tide with 100 year Corrib Flood Flow with new harbour extension development

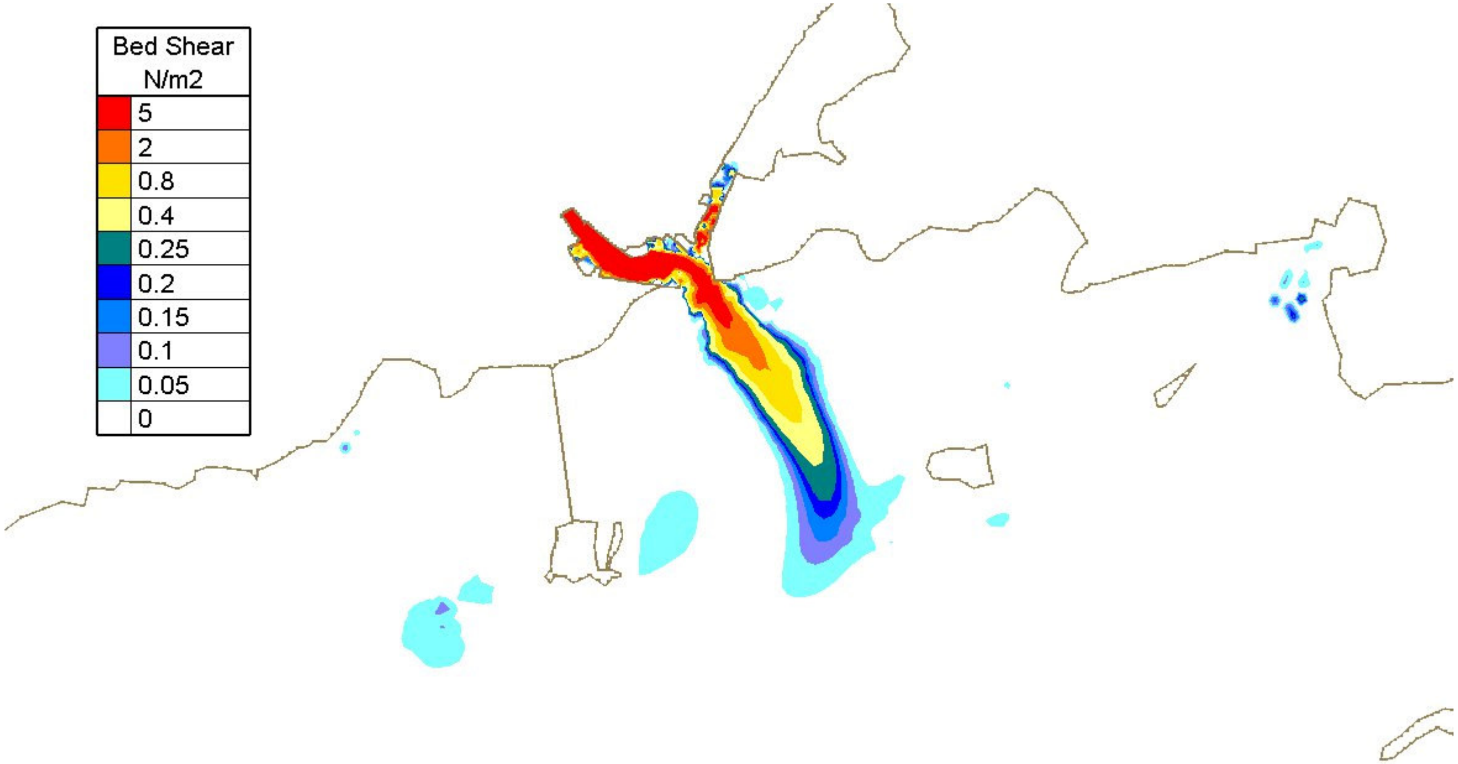


Figure 8.4.34 - Computed bed shear at Low Water Spring Tide with 100 year Corrib Flood Flow for existing case

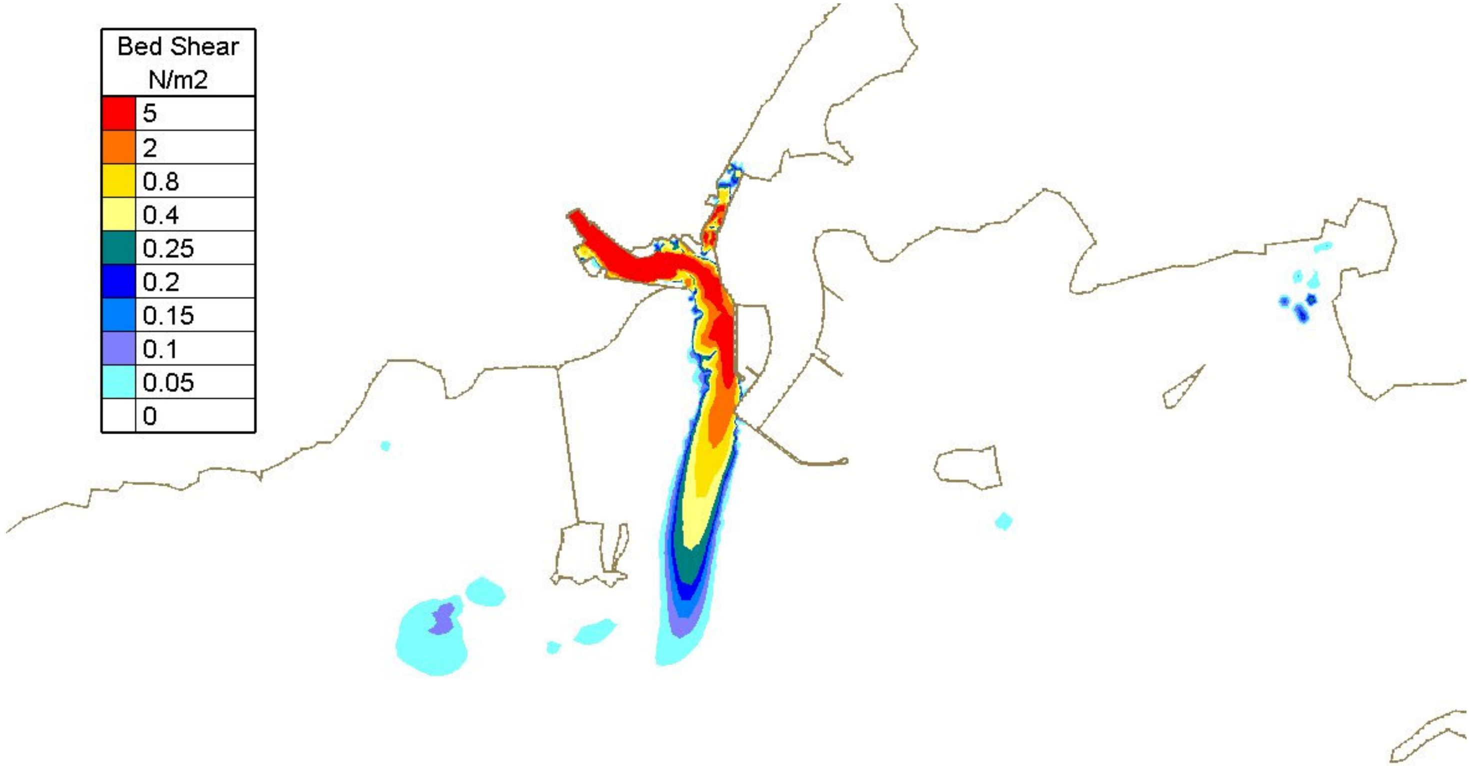


Figure 8.4.35 - Computed bed shear at Low Water Spring Tide with 100 year Corrib Flood Flow with new harbour extension development

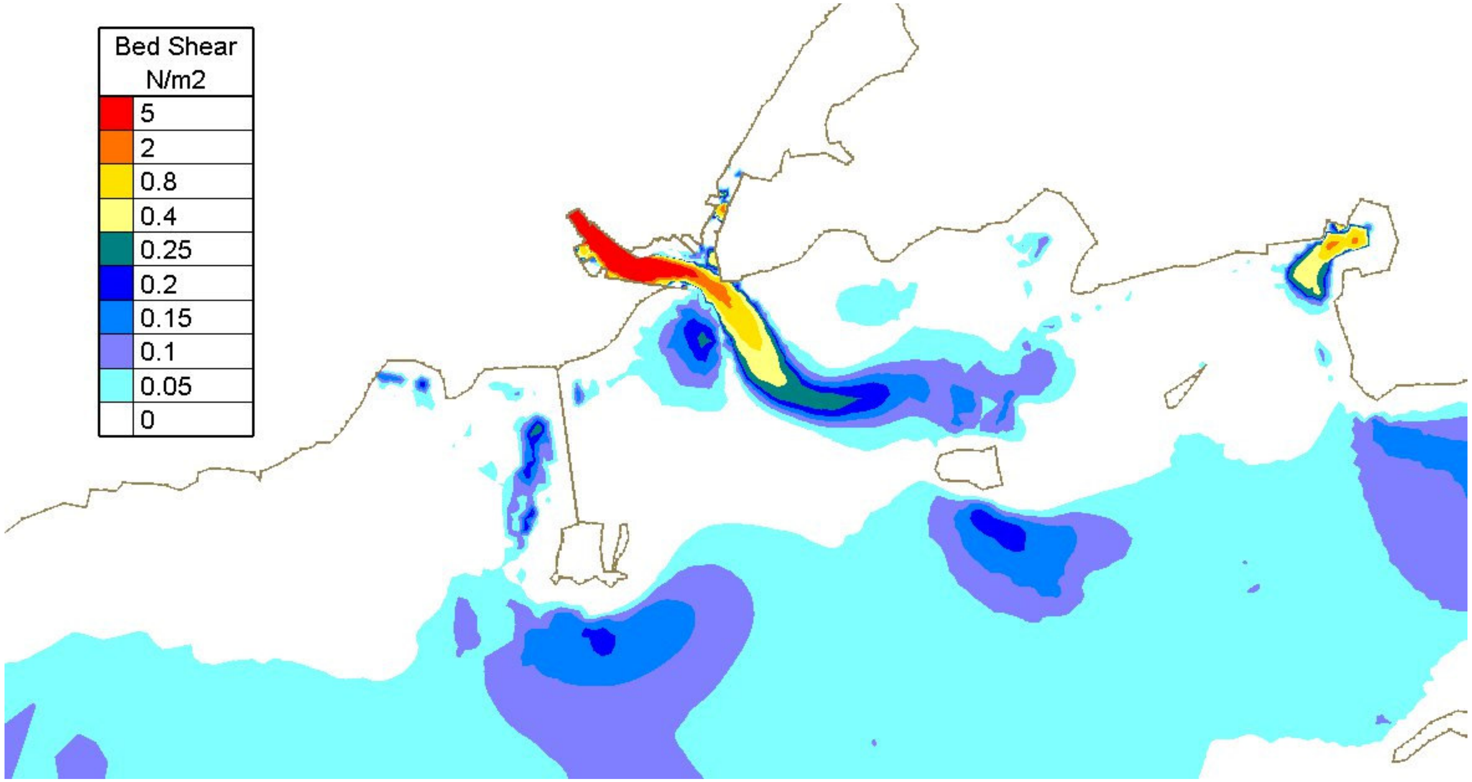


Figure 8.4.36 - Computed bed shear at mid-flood Spring Tide with 100 year Corrib Flood Flow for existing case

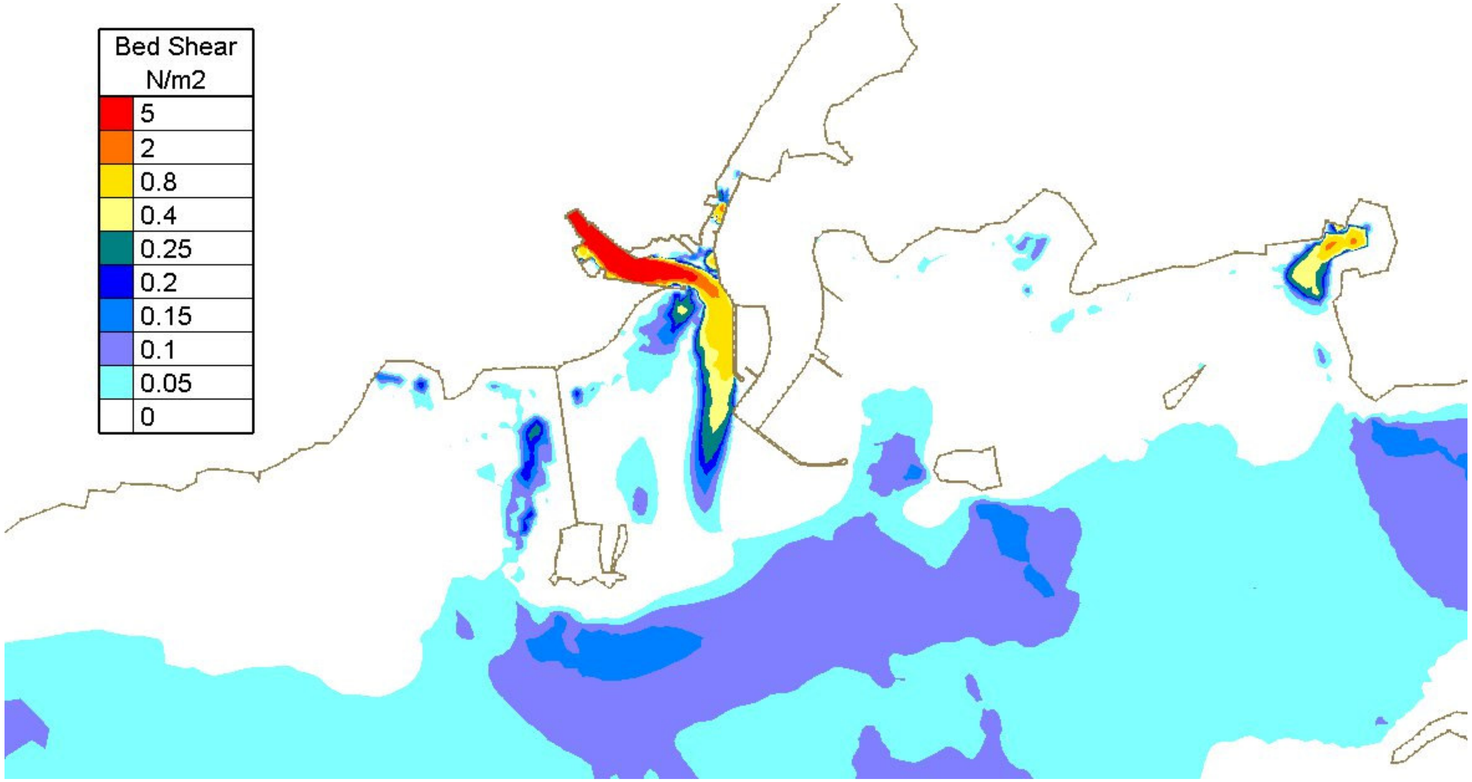


Figure 8.4.37 - Computed bed shear at mid-flood Spring Tide with 100 year Corrib Flood Flow with new harbour extension development

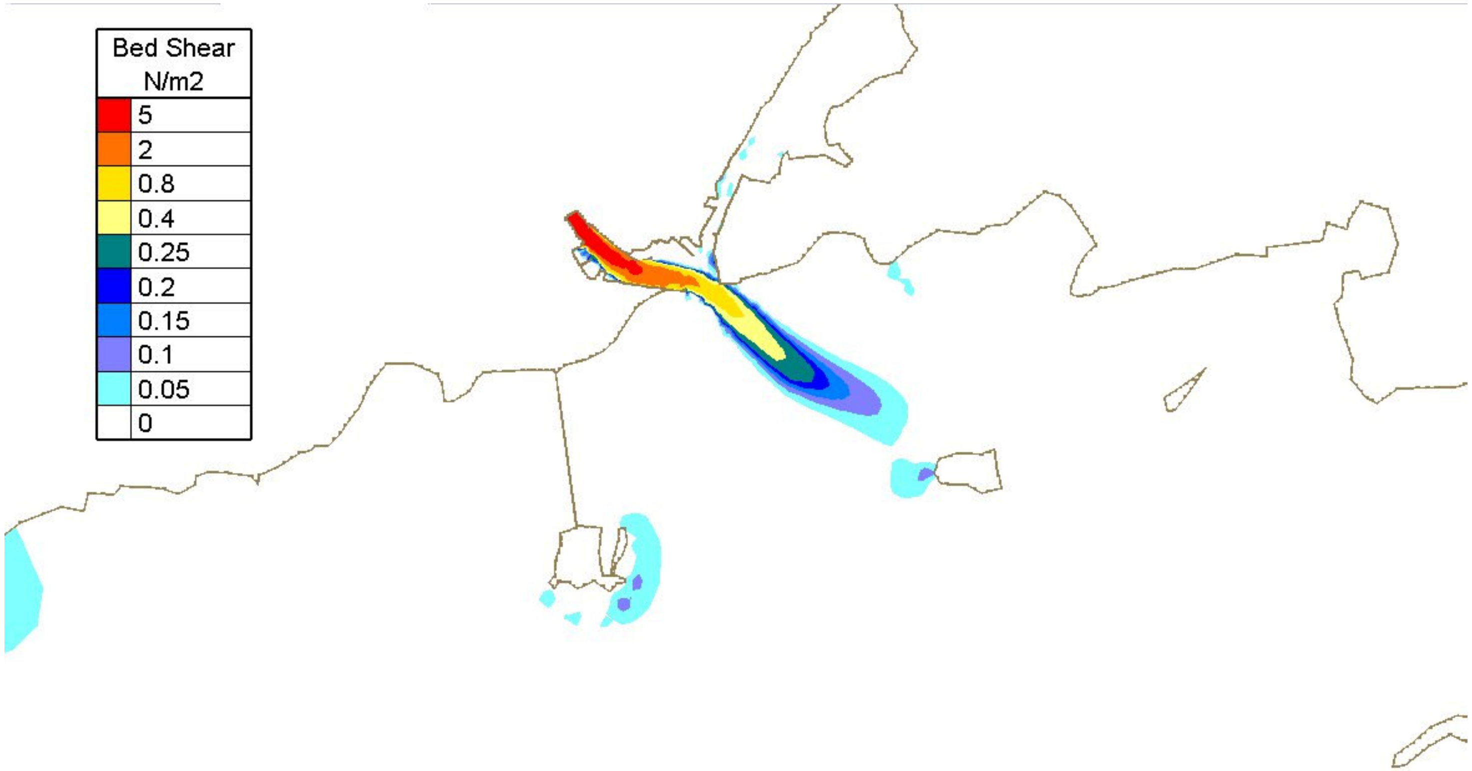


Figure 8.4.38 - Computed bed shear at High Water Spring Tide with 100 year Corrib Flood Flow for existing case

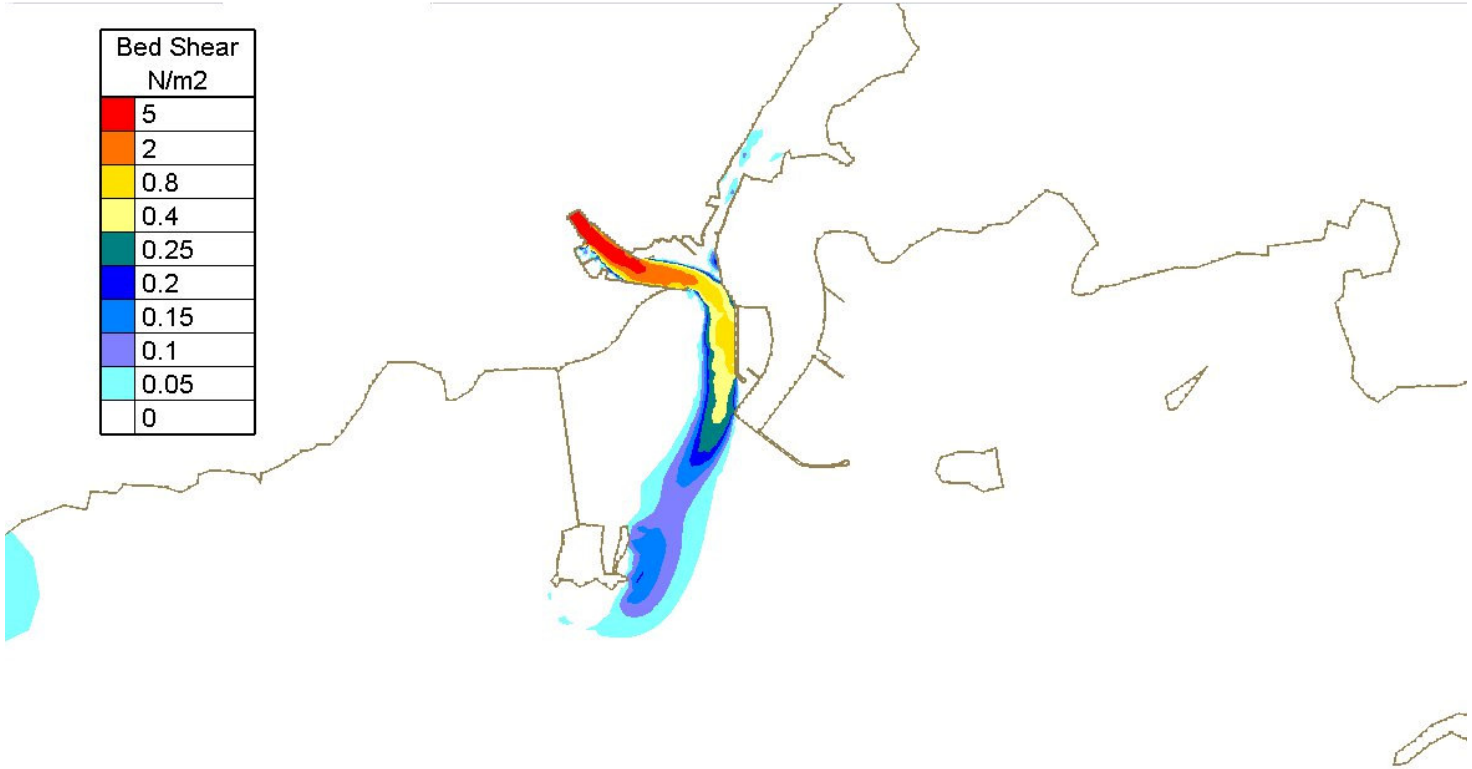


Figure 8.4.39 - Computed bed shear at High Water Spring Tide with 100 year Corrib Flood Flow with new harbour extension development

8.4.2.8 Capital dredge suspended sediment analysis

8.4.2.8.1 Introduction

In order to assess the impact of sediment that will be released to the water column during the proposed capital dredging stage, a sediment plume model study using TELEMAC3D was carried out. Telemac3D models the water body in vertical layers and has an integrated sediment transport model that is coupled with the hydrodynamics.

The model was set-up with an immobile bed and an initial condition of a water column free of suspended solids. For this application, it is assumed that the sediment is non-cohesive, even the finer silt and the sediment settling velocity is based on the Van Rijn equation (1984) developed for non-cohesive sediments which ensures conservatism in respect to the prediction of suspended solids concentrations. In reality some degree of flocculation would happen with the finer sediments and the flocculated sediments would acquire a higher settling velocity and therefore a smaller sediment plume.

8.4.2.8.2 Model Runs

In order to evaluate the likely impact on the water column, four dredging locations were selected (see Figure 8.4.41 for location of these representative dredging points). The dredge plume from each of these locations was modelled separately under critical conditions of Summer low Corrib flow (24.6 m^3/s) and mean Spring tides. Two sediment fractions were investigated namely a fine silt and a fine sand. These simulations were carried out for three days continuous dredging per location so as to evaluate the plume pattern, its dispersion and return over successive tides. These fine sediment fractions were selected so as to ensure conservatism in respect to predicting plume extent and suspended solids concentrations. The AQUAFACT bed sediment sampling results (sample reference numbers 1 to 6 in Figure 8..4.40) show the bed sediment to be generally classified as a fine sand (refer to Table 8.4.2 below). The Soils Report confirms a considerable sand silt content (refer to chapter 6).

The ambient velocities and associated bed shear stresses predicted by the hydrodynamic modelling indicate that the majority of the sediment will settle out close to the dredging location. Typical settling velocities for sands and silt are presented below in Table 8.4.3.

The simulation modelled a silt having a settling velocity of 0.00175 m/s and a critical bed shear for deposition of 0.12 N/m^2. A fine silt fraction was also modelled having a settling velocity of 0.0001 m/s and a critical shear for deposition of 0.08 N/m^2.

For the purpose of modelling the dredging work the dredging rate is specified at 40.5 l/s based on Tobin Consulting Engineers calculations. An S-factor for the released concentration as a result of the dredging work of 6000 mg/l (based on the CIRIA Report C547 guidance document based on field measurements of losses from a trailing suction Hopper Dredger) was specified. This represents a sediment release rate of 875 kg of sediment per hour into the water column at the dredge site.

In modelling both fine and coarse fractions it was assumed that for both fractions the release rate will be the 875 kg/hour (*i.e.* the dredged sediment is assumed to comprise 100% fine and 100% coarse silt). In reality the average is an 80% fine fraction.

Sediment size distribution							
Stations	Gravel (>1.5mm)	Very coarse sand (1.5mm)	Coarse sand (0.75mm)	Medium sand (0.38mm)	Fine sand (0.19mm)	Very fine sand (0.09mm)	Silt (<0.063mm)
1	0	0	0	17.65	75.29	2.3	4.77
2	0	20.19	0.36	5	21.01	22.09	31.35
3	0	0	0	28.98	65.87	0.6	4.54
4	0	2.27	0.99	4.19	23.19	24.73	44.62
5	0	18.38	0.07	17.92	53.05	4.34	6.24
6	0	0	0.7	32.69	63.44	0.33	3.47
Median	0	1.14	0.22	17.79	58.25	3.32	5.51
Maximum	0	20.19	0.99	32.69	65.87	24.73	44.62

Table 8.4.2 Sediment size distribution (percentage) at Proposed Harbour Site from AQUAFACT surface sampling (refer to Fig 8.41 below for sampling locations see also Soils Chapter 6)

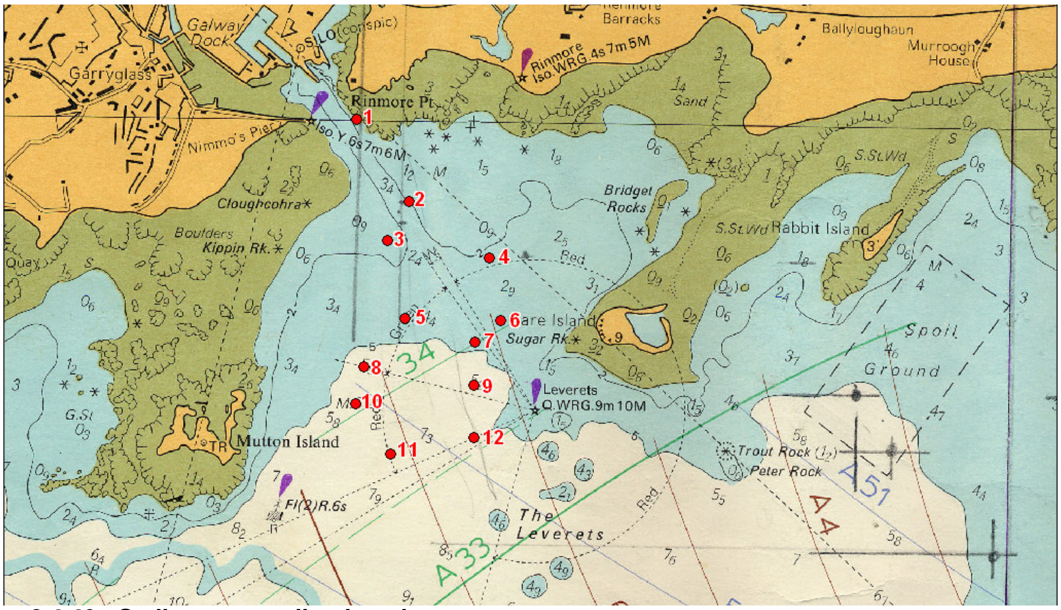


Figure 8.4.40 - Sediment sampling locations

Note: Samples No.7 – 12 are outside the now proposed works area

Settling velocities for non-cohesive sands and silts		
Material Type	Sediment Size (mm)	Settling velocity (m/s)
Coarse sand	0.75	0.093
Medium sand	0.38	0.046
Fine sand	0.19	0.020
Very fine sand	0.09	0.0056
Coarse silt	0.047	0.0015
Very fine silt	0.01	0.00006

Table 8.4.3 Typical settling velocities for non-cohesive sand and silts. Note: settling velocities computed using the Van Rijn (1984) formula

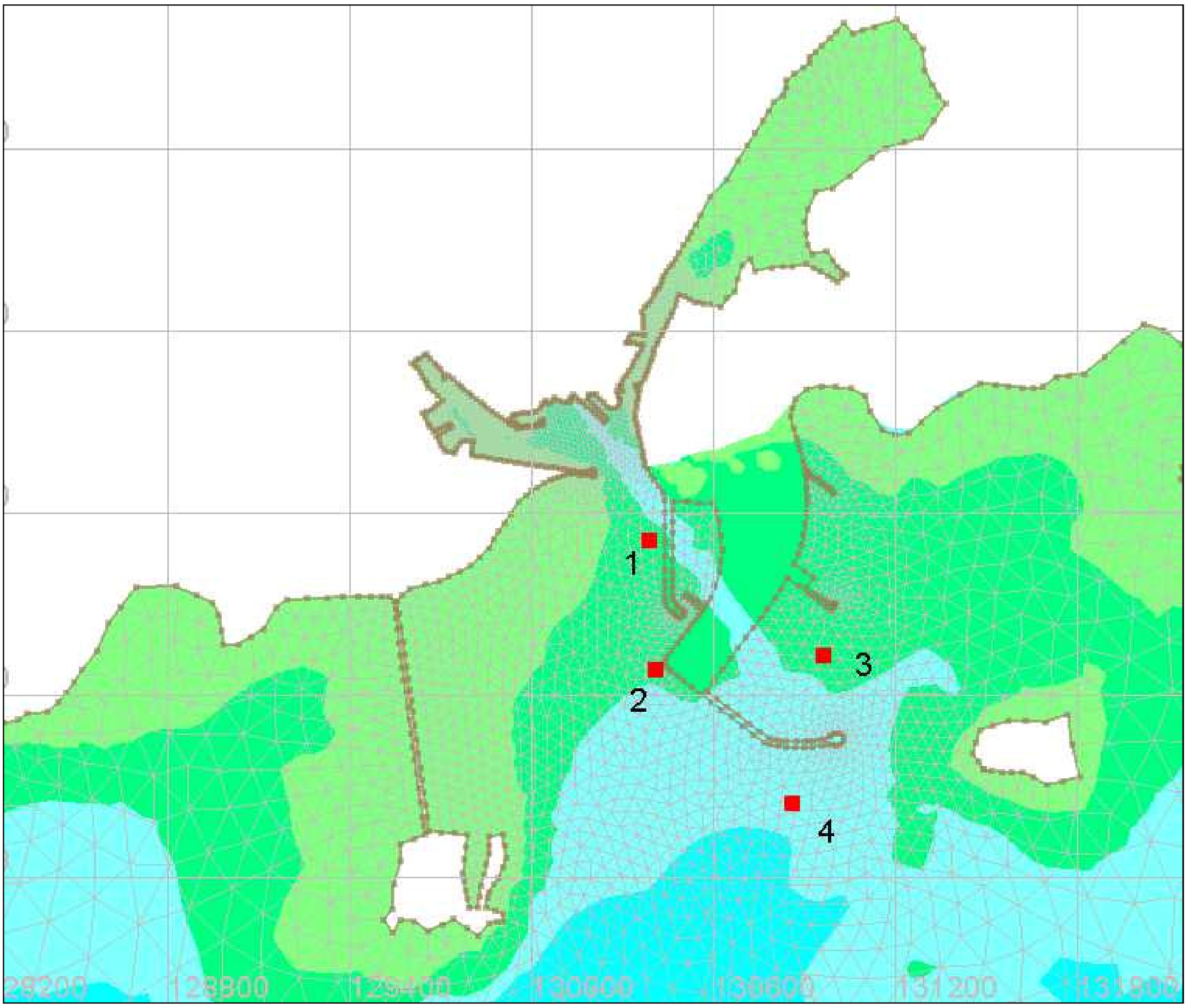


Figure 8.4.41 - Dredging locations for sediment plume simulations

8.4.2.8.3 <u>Discussion</u>

<u>Dredge Point 1</u>

Dredge location No. 1 was chosen to represent the potential effect of dredging the new approach channel to Galway Docks adjacent to the marina breakwater (See Figure 8.4.41 for location).

The coarse silt simulations show a highly localised impact on the receiving water with the sediment falling out of suspension within a reasonable short distance from the dredger (see Figure 8.4.42). Immediately adjacent to the dredger activities the median and maximum concentrations are 8.4 mg/l and 20.4 mg/l. The predicted suspended solids concentrations fall below 2 mg/l within 180 m (max. distance for all tidal stages) and below 1 mg/l within a maximum distance of 320 m of the dredger. Neap tide simulations would result in a somewhat less dispersed plume with a higher settlement rate due to the lower ambient velocities (approximately 40 to 50% lower on Neap tides)

The fine silt simulations show a considerably larger area of impact due to the poor ability for fines to settle out. The simulations show that concentrations are generally less than 10 mg/l away from the source. The plume excursion shows migration with the ebbing tide out around Mutton Island and on the flooding tide into Lough Atalia as illustrated in Figure 8.4.43 where increases of 1 mg/l are predicted. Given the extremely high naturally occurring background levels of suspended solids under storm conditions where values of 65,000 mg/l were recorded, this level of increase representing 4 orders of magnitude lower than naturally occurring is considered as being capable of having no impact on the functioning of this habitat. Peak concentrations occur immediately adjacent to the dredging activities with predicted median and maximum concentrations of 20.74 mg/l and 52.9 mg/l.

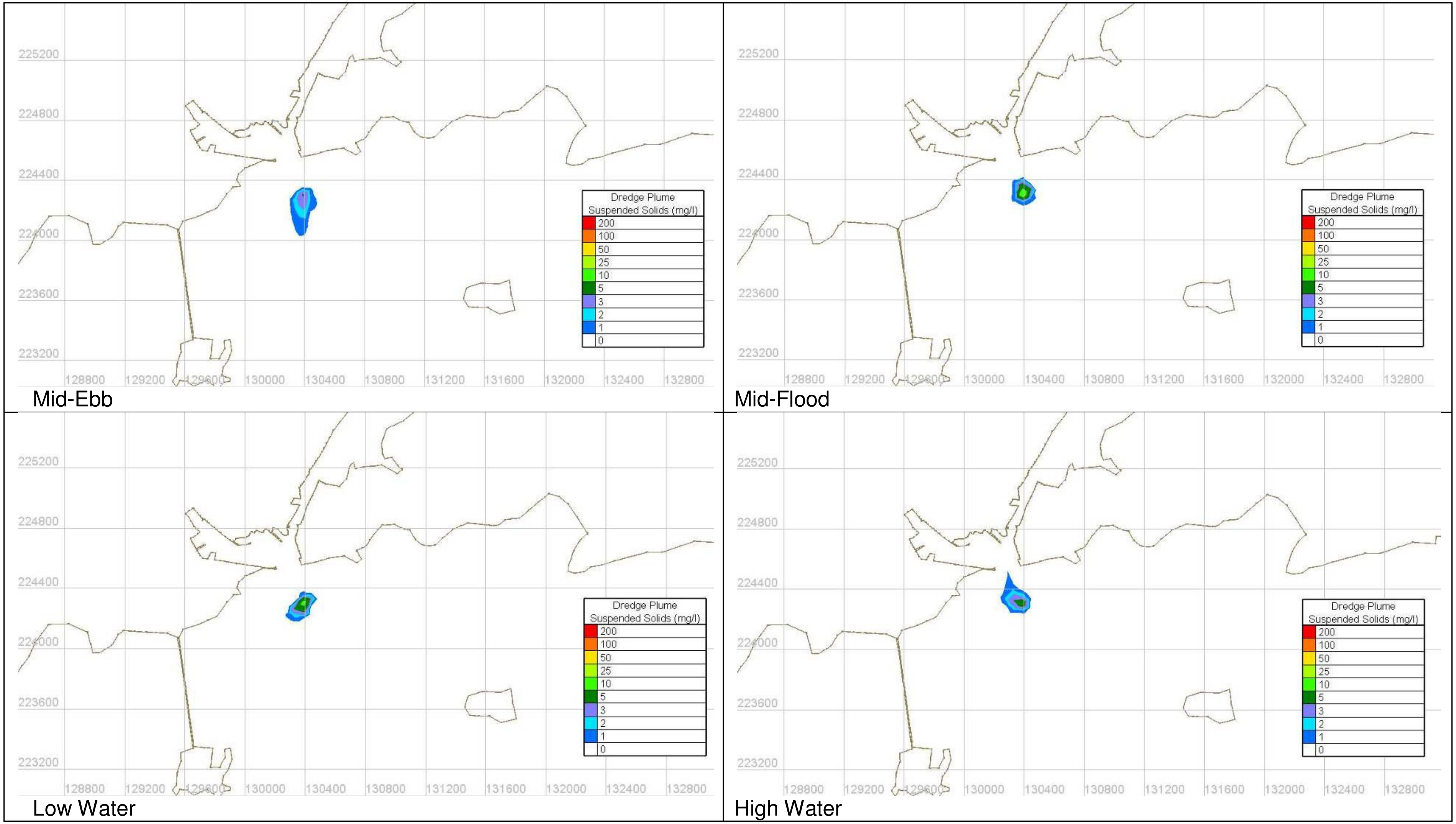


Figure 8.4.42- Coarse silt suspended sediment plume simulation at dredge location 1 – Spring tide and Corrib Summer low flow

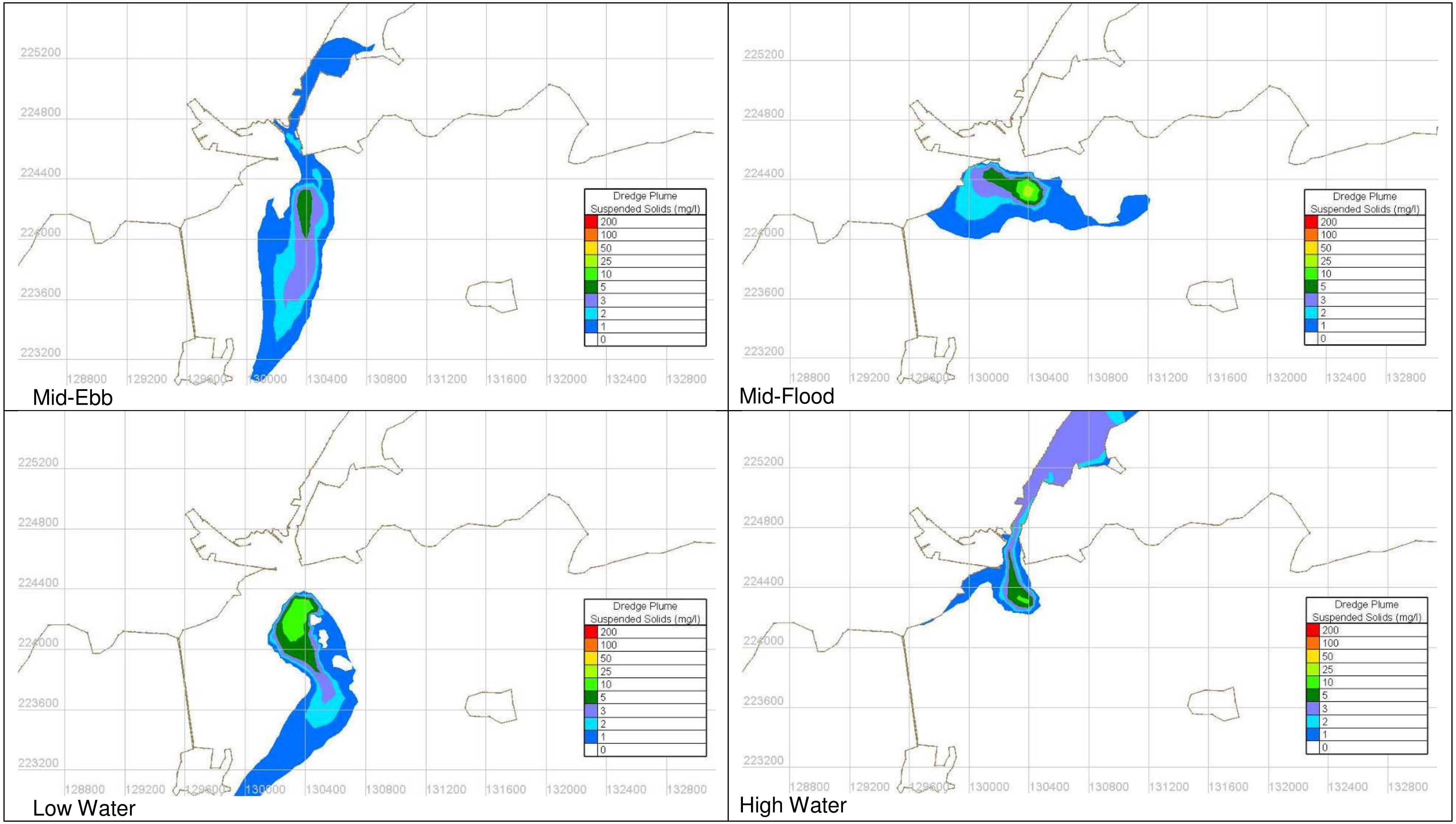


Figure 8.4.43 Fine silt suspended sediment plume simulation at dredge location 1 – Spring tide and Corrib Summer low flow

Dredge Point 2

Dredge location No. 2 was chosen to represent the potential effect of dredging towards the outer (southern) end of the new approach channel to Galway Docks adjacent to the port breakwater (refer to Figure 8.4.41 for location).

The coarse silt simulations show a very localised impact on the receiving water with the sediment falling out of suspension within a relatively short distance from the dredging area (Figure 8.4.44). Immediately adjacent to the dredger activities the median and maximum concentrations are 17.6 mg/l and 62.5 mg/l. The predicted suspended solids concentrations fall below 2 mg/l within 220 m on the ebb excursion and 180 m on the flooding tide excursion.

The fine silt simulations show a considerable larger area of impact due to the poor ability for fines to settle out. The simulations show that concentrations are generally less than 10 mg/l away from the source. The plume excursion shows migration with the ebbing tide out around Mutton Island and on the flooding tide and north and northwest towards the South Park shoreline, as illustrated in Figure 8.4.45. Peak concentrations occur immediately adjacent to the dredging activities with predicted median and maximum concentrations of 21.7 mg/l and 210 mg/l respectively. The predicted suspended solids concentrations fall below 1 mg/l within 500 m on the flooding tide excursion and within 210 m on the ebbing tide excursion.

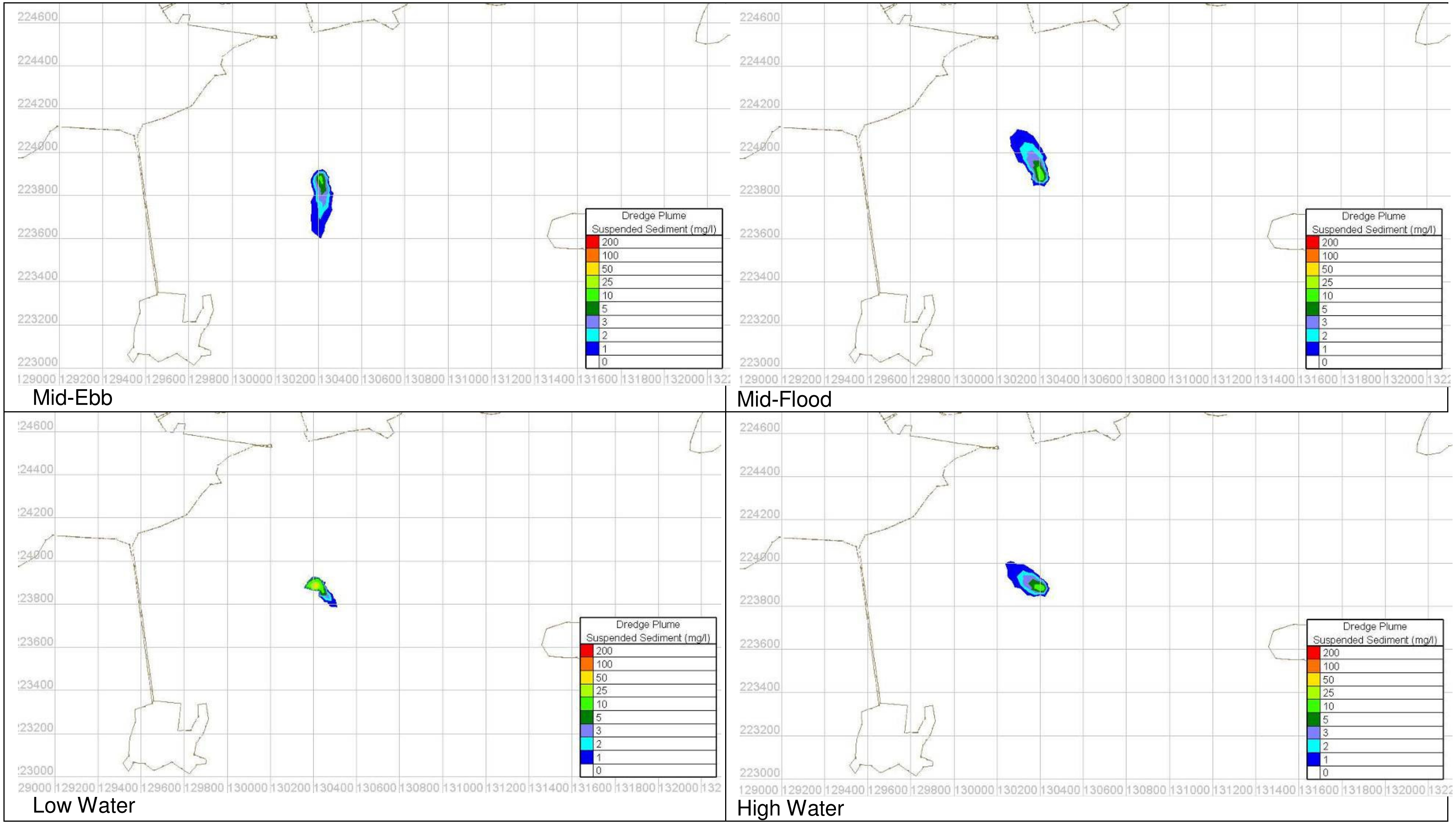


Figure 8.4.44 Coarse silt suspended sediment plume simulation at dredge location 2 – Spring tide and Corrib Summer low flow

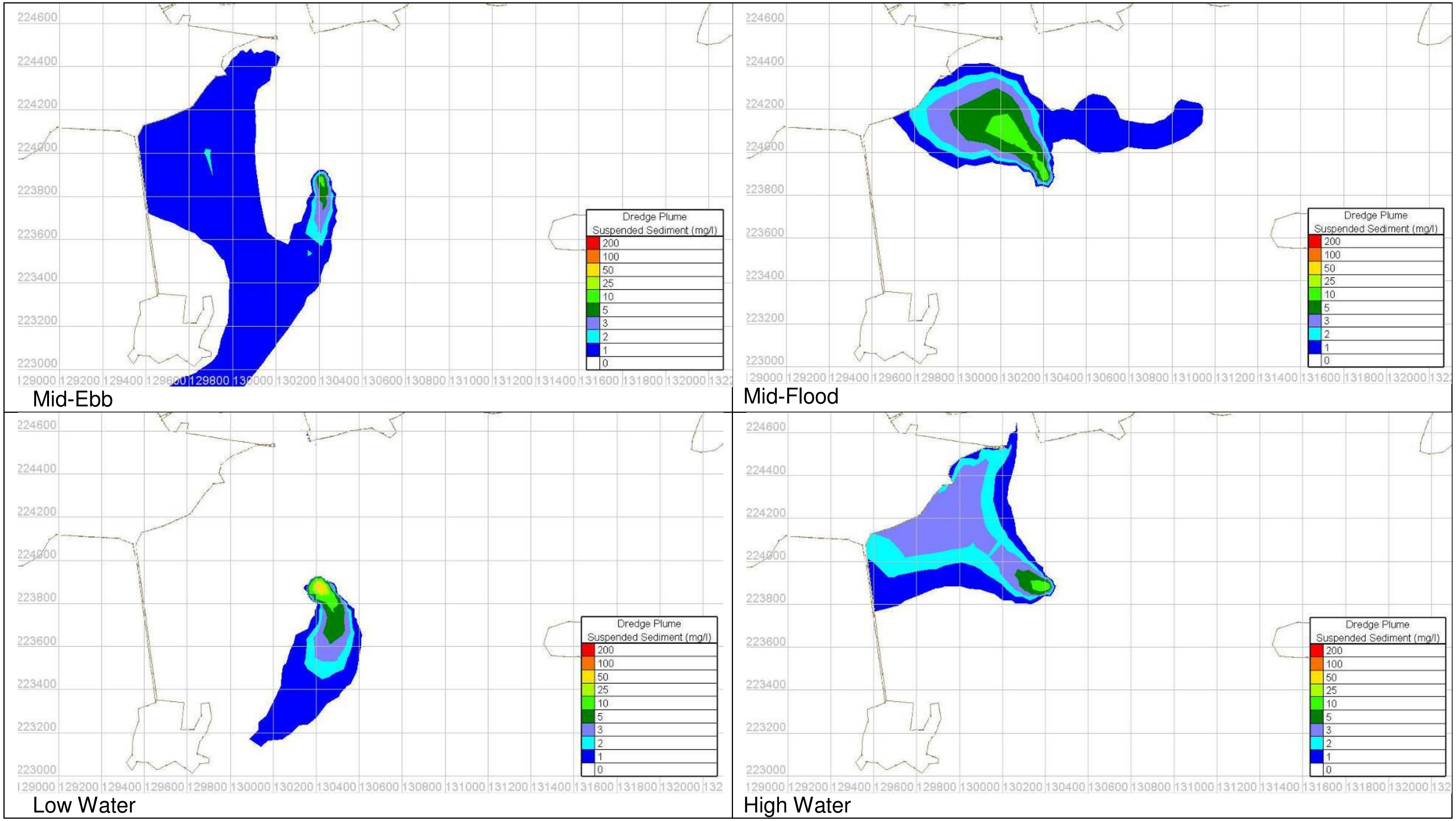


Figure 8.4.45 Fine silt suspended sediment plume simulation at dredge location 2 – Spring tide and Corrib Summer low flow

Dredge Point 3

Dredge location No. 3 was chosen to represent the potential effect of dredging within the port area (turning circle), refer to Figure 8.4.41 for location.

The coarse silt simulations show a very localised impact on the receiving water with the sediment falling out of suspension within a short distance from the dredging area (Figure 8.4.46). Immediately adjacent to the dredger activities the median and maximum concentrations are 4.17 mg/l and 12.65 mg/l. The predicted suspended solids concentrations fall below 2 mg/l within a radius distance of 130 m on the Spring tide. Neap tide excursions will be even smaller and thus the sediment concentration and deposition will be localised to the dredging works area.

The fine silt simulations show a considerably larger area of impact due to the poor ability for fines to settle out (Figure 8.4.47). The simulations show that concentrations are generally less than 10 mg l away from the source. The plume excursion shows migration with the ebbing tide initially south and then southwest towards Mutton Island. On the flooding tide the plume migrates northwards in a widely dispersed mushroom-shaped plume towards the Renmore shoreline area (concentrations are shown to be well less than 1 mg/l along the shoreline). At an average sediment concentration of 1 mg/l, the daily deposition rate of fine silt would only be 8.64 g/m^2 which represents a very small rate of deposition, in the context of baseline sediment concentrations (natural deposition rates). Peak concentrations occur immediately adjacent to the dredging activities with predicted median and maximum concentrations of 6.06 mg/l and 33.8 mg/l respectively. The predicted suspended solids concentrations fall below 5 mg/l within 800 m on the flooding tide excursion and within 210 m on the ebbing tide excursion.

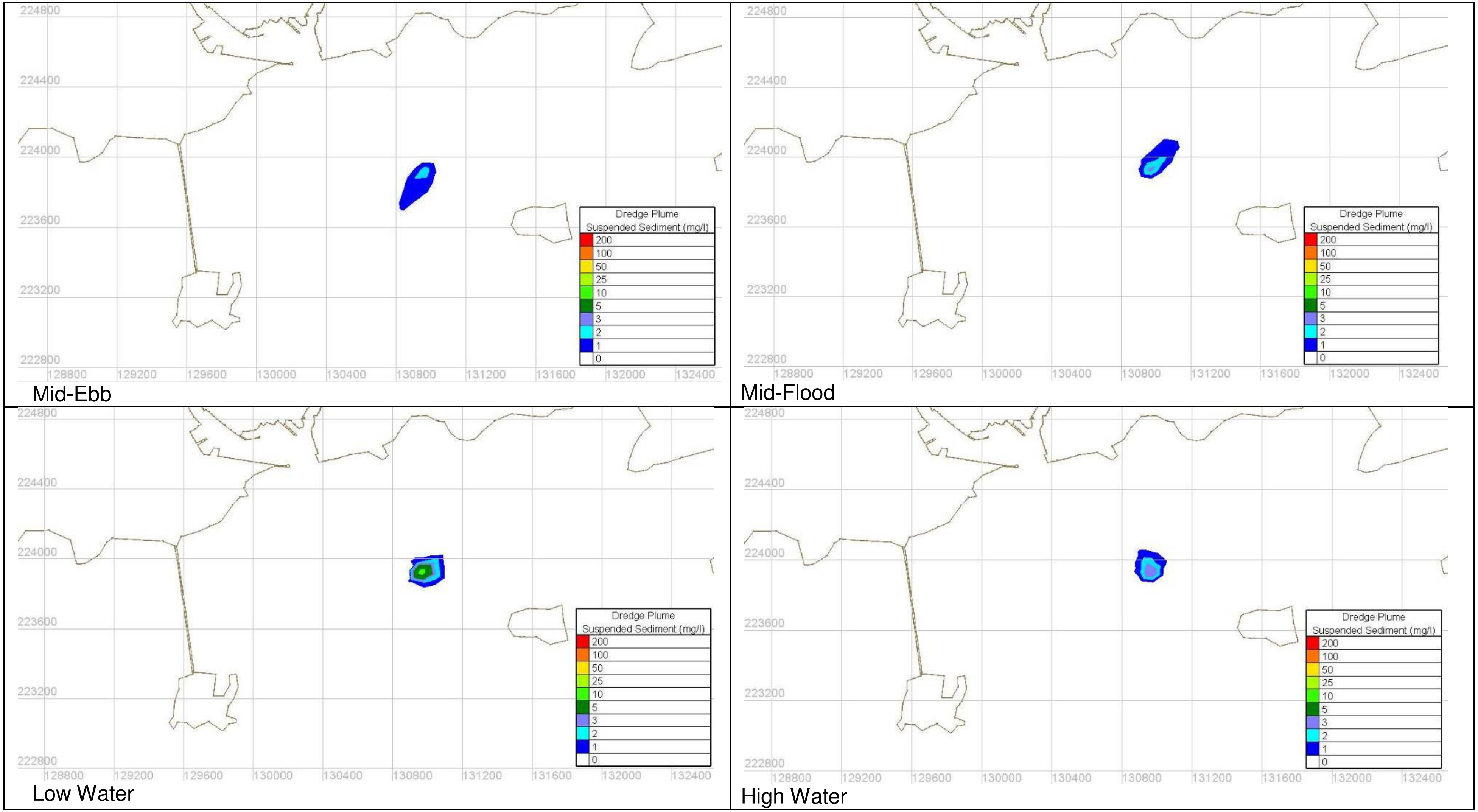


Figure 8.4.46 Coarse silt suspended sediment plume simulation at dredge location 3 – Spring tide and Corrib Summer low flow

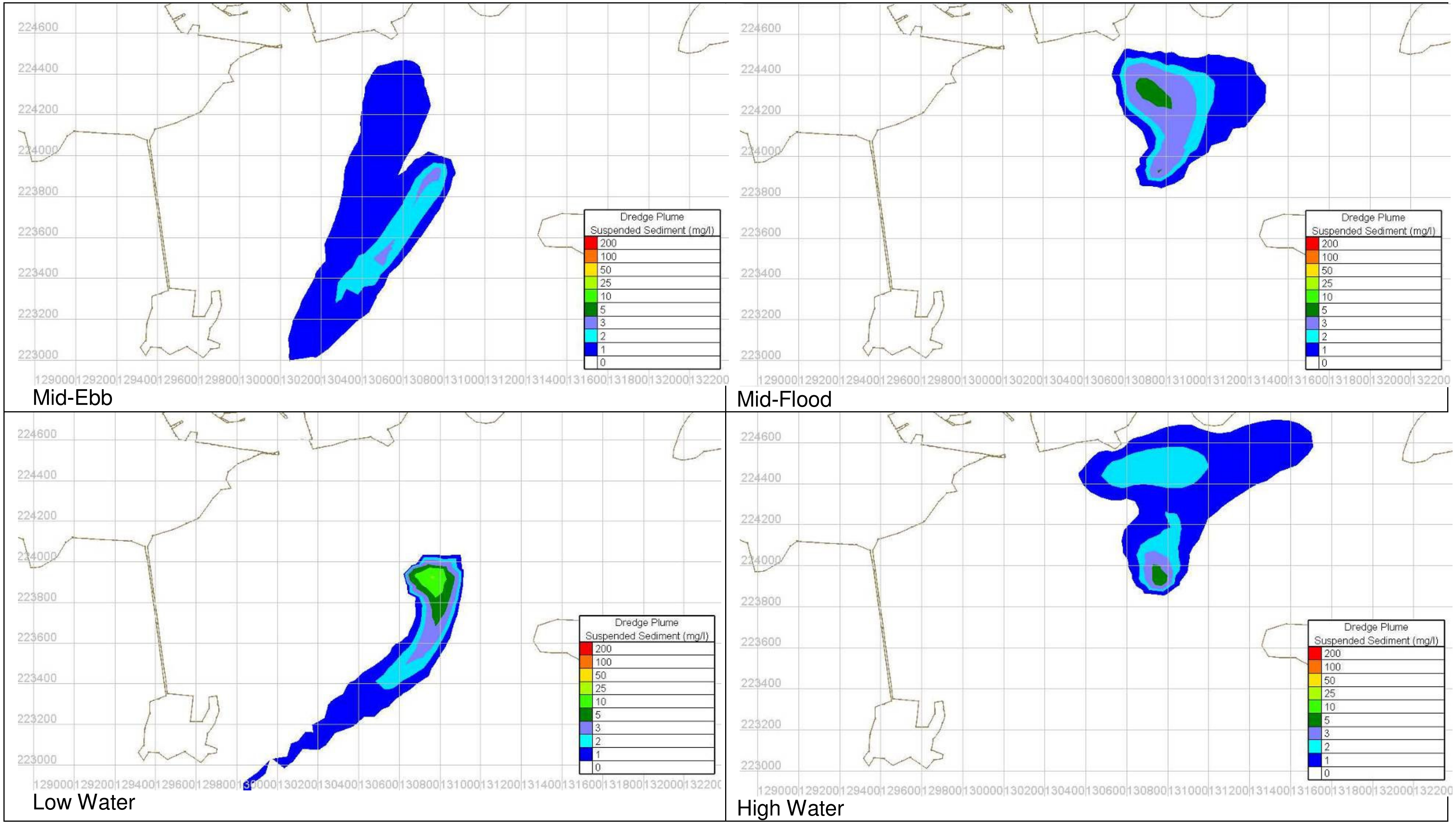


Figure 8.4.47 Fine silt suspended sediment plume simulation at dredge location 3 – Spring tide and Corrib Summer low flow

Dredge Point 4

Dredge location No. 4 was chosen to represent the dredging activities in the proposed approach channel to the port, refer to Figure 8.4.41 for location map.

The coarse silt simulations show a very localised impact on the receiving water with the sediment falling out of suspension within a short distance from the dredging area (Figure 8.4.48). Immediately adjacent to the dredger activities the median and maximum concentrations are 3.57 mg/l and 11.39 mg/l. The predicted suspended solids concentrations fall below 2 mg/l within 180 m on the flooding westward tide and 120 m on the ebbing tide travelling south southwest. Neap tide excursions will be even smaller and thus the sediment concentration and deposition will be very localised to the dredging works area.

The fine silt simulations show a considerable larger area of impact due to the poor ability for fines to settle out (Figure 8.4.49). The simulations show that concentrations are generally less than 5 mg/l away from the immediate dredging location. The plume excursion shows migration with the ebbing tide initially southwest past Mutton Island. On the flooding tide the plume migrates northwest past Hare Island. The plume remains well off shore and elongated in a southwest to north east axis influenced by slightly stronger tidal currents. Peak concentrations occur immediately adjacent to the dredging activities with predicted median and maximum concentrations of 4.38 mg/l and 16.42 mg/l respectively. The predicted suspended solids concentrations fall below 5 mg/l within 290 m on the flooding east northeast tidal excursion and within 110 m on the ebbing southwest tide excursion.

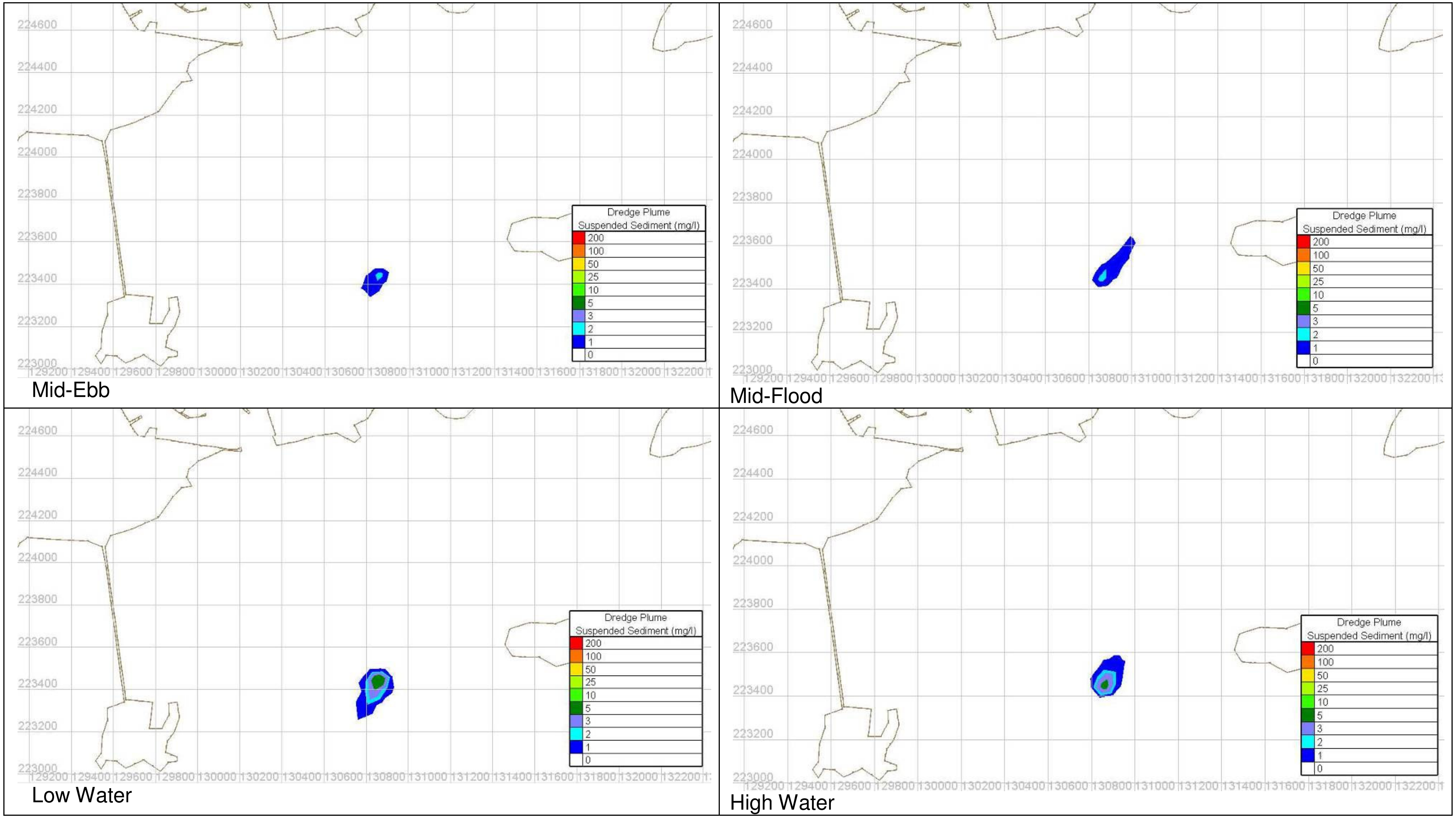


Figure 8.4.48 Coarse silt suspended sediment plume simulation at dredge location 4 – Spring tide and Corrib Summer low flow

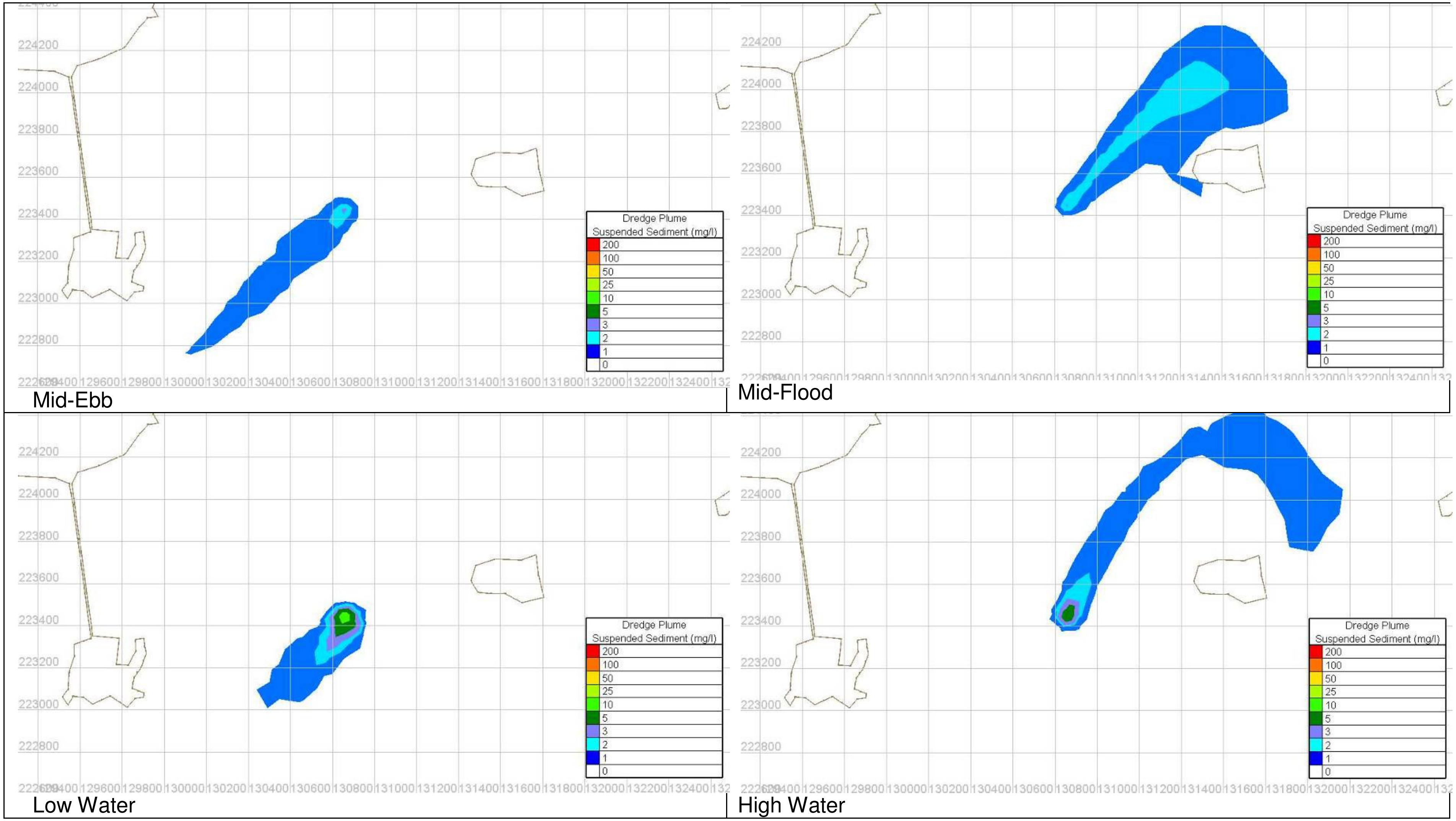


Figure 8.4.49 Fine silt suspended sediment plume simulation at dredge location 4 – Spring tide and Corrib Summer low flow

8.4.2.8.4 Mitigation:

As Lough Atalia is classified as lagoonal and is designated a priority habitat, dredging will cease before the tide could move the silt plume into the Lough Atalia Channel. In light of the model output shown in Figure 8.4.43, dredging will be controlled to ensure that no waters carrying a silt load above ambient will enter Lough Atalia. Hydrodynamic analysis has shown that the available inflow period to Lough Atalia is limited to approximately 2 to 2.5 hours per tidal cycle with the principal tidal inflow occurring on spring tides and very limited inflow occurring on neap tides due to the presence of a raised shelf on the inflow channel.

8.4.2.8.5 Conclusion

The sediment plume modelling for the four test sites chosen to represent the capital dredge area showed sediment deposition to be generally localised close to the dredging point. The simulations demonstrated that even when modelling a 100% fine silt (conservative approach), the suspended sediment concentrations are only significantly elevated above background in the vicinity of the dredging point with the plume enjoying reasonable dispersal thereafter. The actual monitored sediment characteristics classify the sediment as varying between a fine sand and a fine silt. The coarse to fine sand fraction will deposit close to the dredge point whereas the silt will disperse with the inflowing and out flowing tides. Generally, concentrations remote from the dredging point are 1 mg/l or less. At a concentration of 1 mg/l of silt, the depositional rate based on a settling velocity of .0001 m/s is 8.64 g/m^2 per day which is considered insignificant and particularly so, given the temporary nature of the capital dredge activity. The suspended solids concentrations of less than 1 mg/l above ambient that may enter Lough Atalia are extremely low compared to naturally occurring background levels and it is considered that they will not to have any effect of the functioning of this ecosystem.

Under larger river flows, the sediment plume would have greater dispersal out to sea resulting in lower sediment plume concentrations. The critical hydrodynamic conditions for Lough Atalia are Spring tides and low Corrib Flow conditions.

There will be two periods during which the sedimentary conditions in the mouth of the River Corrib will change and these relate to the dredging/construction period and for a period of a number of weeks post-completion.

Firstly the dredging/construction sediment will be brought into suspension by the dredging activities and the model has been used to predict concentrations at the selected sites within the works area. As sediments can only travel northwards on a flooding tide and as maximum flow occurs on Spring tides, only these conditions were modelled. The predicted deposition levels at all four observation sites indicate that the majority of suspended sediments will fall out within short distances of the dredging activity. In order to establish naturally occurring levels of suspended solids at Ballyloughan Beach, a water sample was taken on May 24th, 2011 during a period of extreme wind conditions. The sea water at the beach was very turbid and suspended solid values were recorded at 65,000 mg/l, which are 4 orders of magnitude above the values predicted by the model *i.e.* less that 1 mg/l.

Secondly, the model predicts increased velocities in the deeper water between the new structure and the causeway, there will be a period of erosion post-completion of Phase 1 in which fine surface sediments will be transported southwards. The model predicts that the material will deposit eastwards of Mutton Island. Sedimentary conditions in this area are characterized by muddy sands and the eroded material that will deposit in this location are fine muds of less than 63µm. Given the low predicted volumes and the fact the sediments in the area of fall out already comprise of such sediments, it is considered that the addition of these fines will not have a significant impact on the biological communities in the area.

8.4.2.9 Maintenance dredge suspended sediment analysis

Following on from the capital dredge impact assessment presented above a series of dredge plume simulations for similar hydrodynamic conditions were carried out with the proposed harbour extension and dredged channels in place. These simulations were performed to further assess the potential impact from the future maintenance dredge operations on water quality at sensitive receptors such as Lough Atalia and Renmore Beach. Similar to the previous simulations a fine sediment having a dredge resuspension S-Factor of 6 kg/m^{3}and a dredge rate of 40.5 l/s were specified in the model as per expected losses from a trailing suction hopper dredger reported by CIRIA C547 Guidelines. The equilibrium concentration after 3 days continuous release of sediment was output for the 4 principal stages of the tidal cycle and for seven locations four associated with the commercial harbour navigation channel and harbour turning circle (A1 to A4) and three locations along the marina and docks relocated navigation channel (B1 to B3), refer to Figure 8.4.50 below.

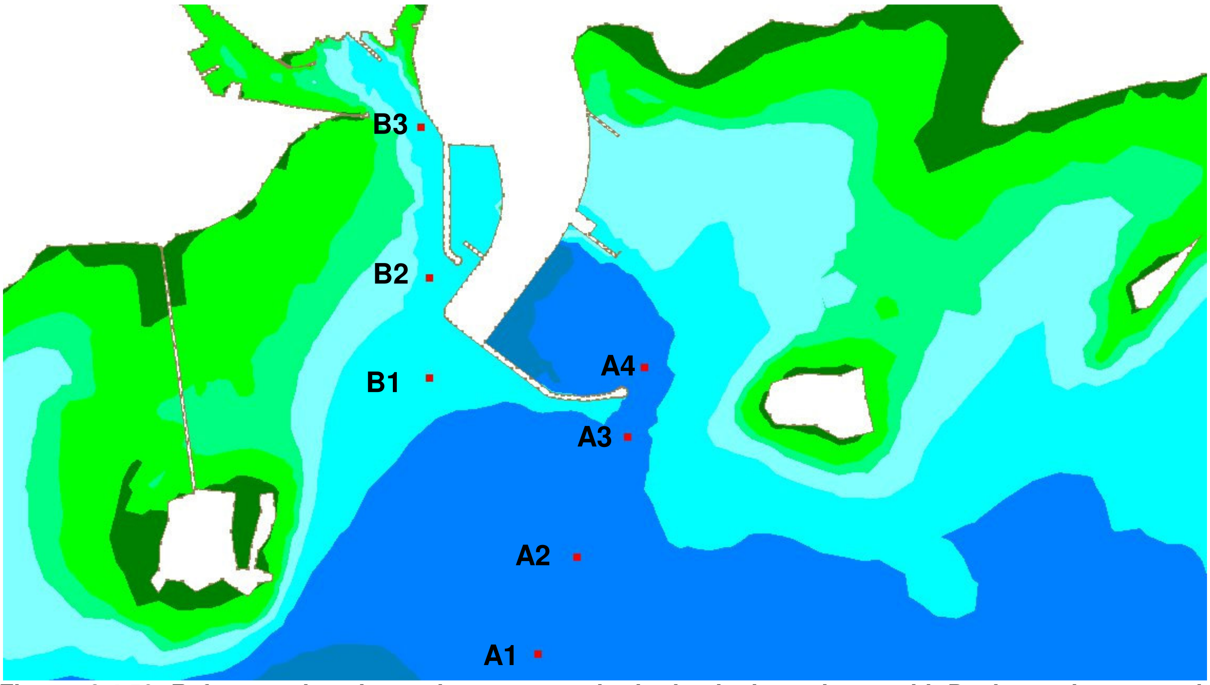
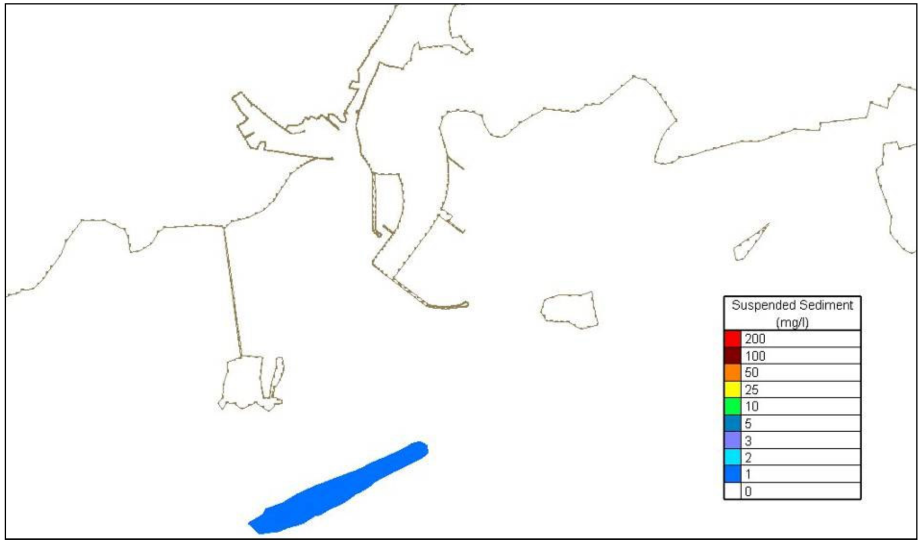


Figure 8.4.50 Reference locations along approach dredged channels to old Docks and proposed commercial port to assess suspended solids plume impact maintenance dredge operations

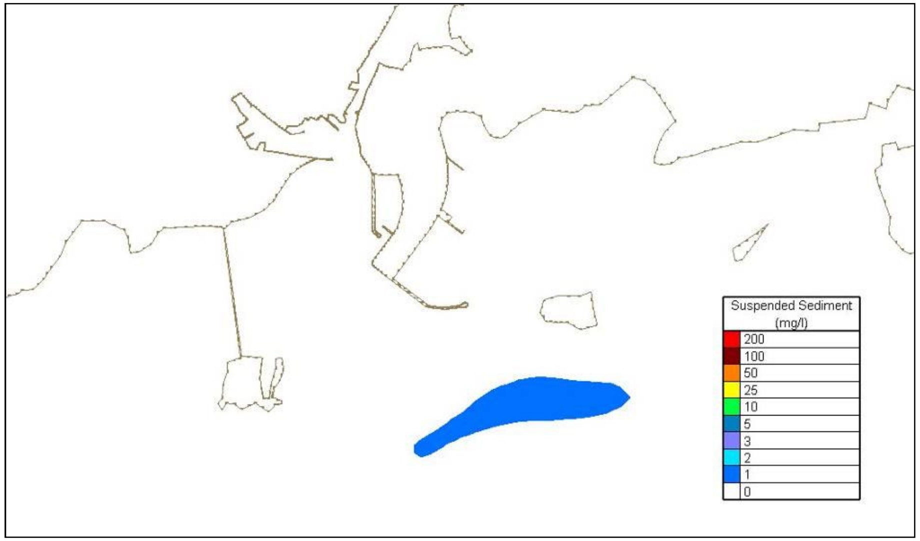
The suspended solids plume plots for the maintenance dredging activities by a trailing suction hopper dredger at each of the dredging sites (A1-A4 and B1-B3) are presented in Figures 8.4.51 to 8.4.57 representing snapshots of sediment plume after three days of continuous dredging at the four principal stages of the tidal cycle. Concentrations down to 1 mg/l are shown in these plots which is well below natural ambient suspended solids levels for these coastal waters. The findings from these simulations clearly show that dredging activities in the new approach channel to the old docks and Marina (as represented by B1 to B3) without mitigation would result in an increase of suspended solids concentration in excess of 1 mg/l entering Lough Atalia on the incoming tide. The simulation results for sites A1 to A4 in the port and approach channel show no impact to Lough Atalia or to Renmore beach with the plume undergoing high dispersal and dilution as a result of the deep water at the dredge sites.

Mitigation to protect Lough Atalia will involve confining dredging activities to the outgoing ebbing flow for the channel to the Docks and Marina represented by B1 to B3. No mitigation measures will be required for the main commercial harbour approach channel, turning circle and berths as the suspended sediment disperses quickly due to the large depths and the dredging methods proposed during maintenance dredging work would be undertaken on the ‘A’ areas during rising tide and the ‘B’ areas during ebbing tides only. The worst case of ‘B’ maintenance dredging is plotted *i.e.* low river flow. Strong river flow would significantly reduce such silt access to Lough Atalia; however it would not entirely eliminate same. Monitoring at the entrance to the Lough Atalia Channel will be undertaken during maintenance dredging to ensure that dredging during ebbing flow is controlled and ceases sufficiently in time before rising flow into Lough Atalia.

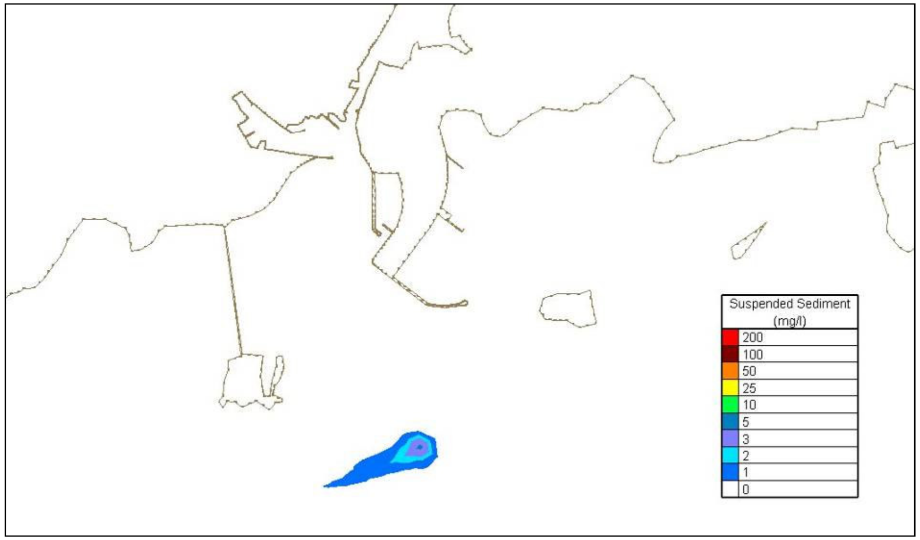
The predicted hydrodynamics for the post construction model indicate that that for sites B1 to B3 there will be little opportunity for accretion and hence little need for maintenance dredging in this area.



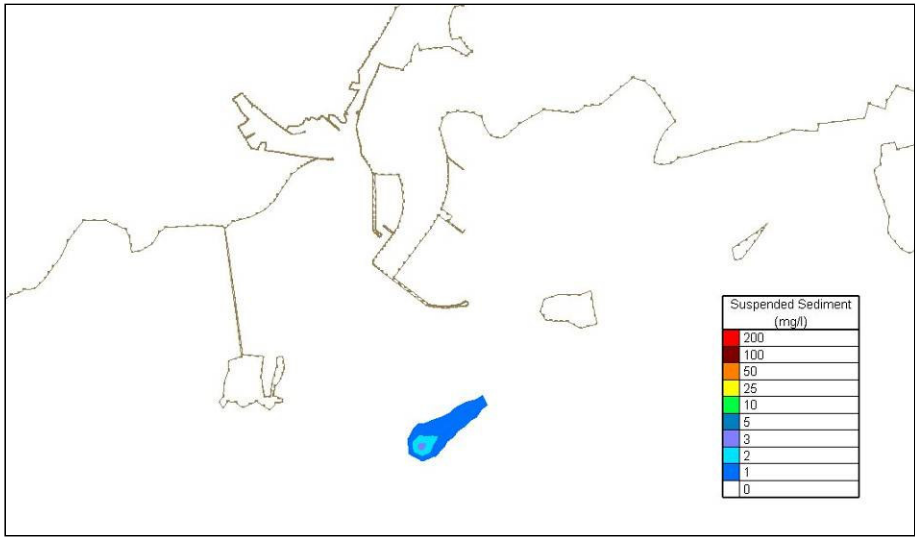
(i) Mid-ebb



(iii) Mid-Flood

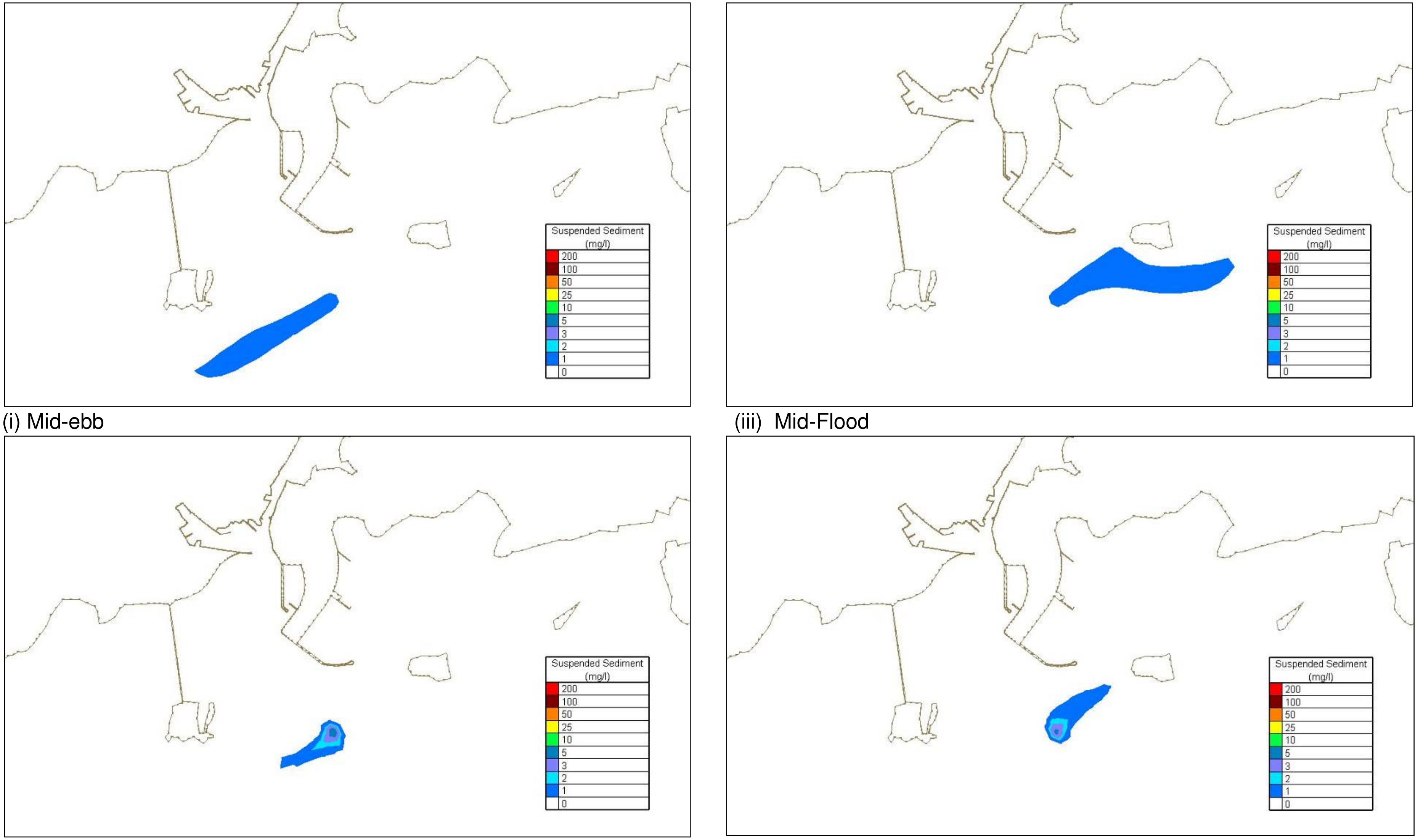


(ii) Low Water



(iv) High Water-

Figure 8.4.51 Dredge Plume at Location A1 for a Mean Spring tide and Summer Corrib low flow



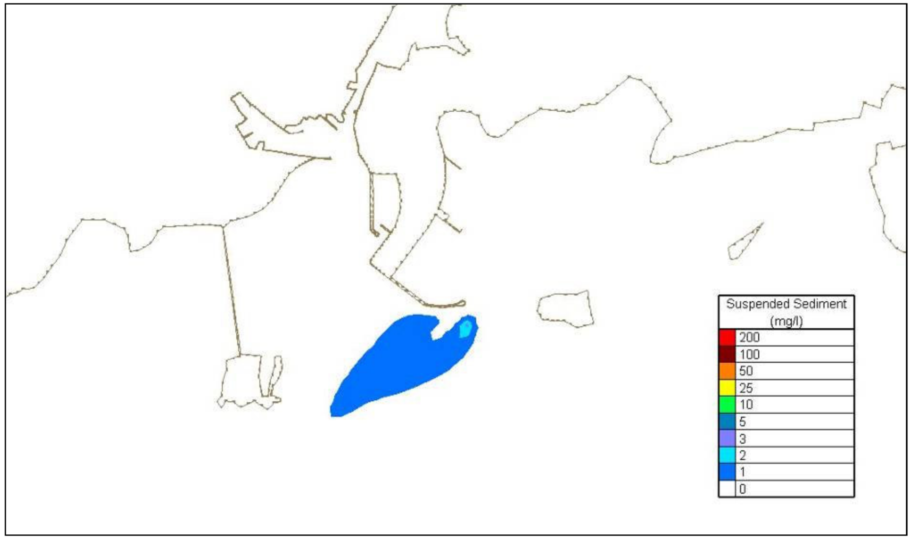
(i) Mid-ebb

(iii) Mid-Flood

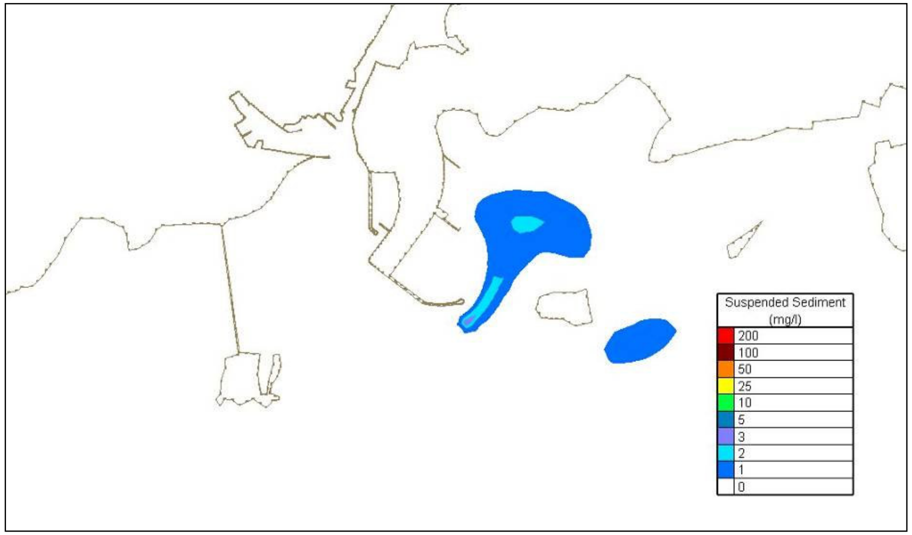
(iii) Low Water

(iv) High Water-

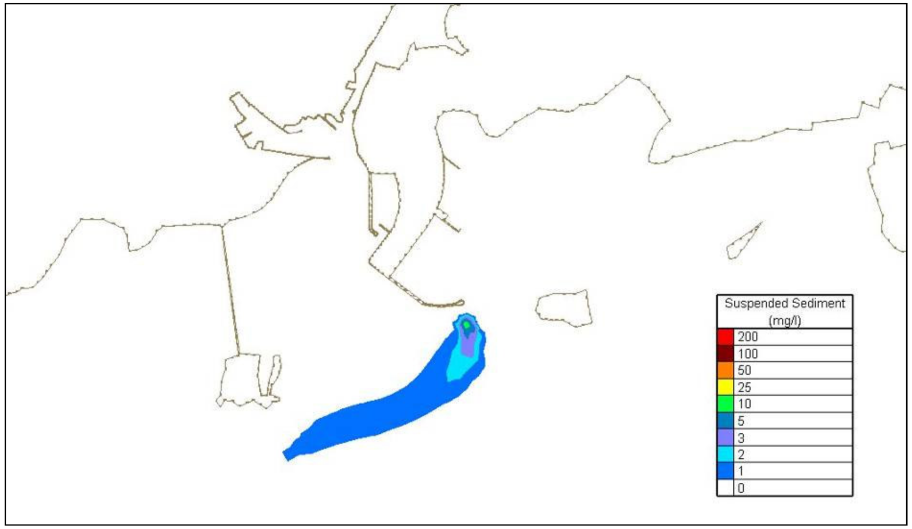
Figure 8.4.52 Dredge Plume at Location A2 for a Mean Spring tide and Summer Corrib low flow



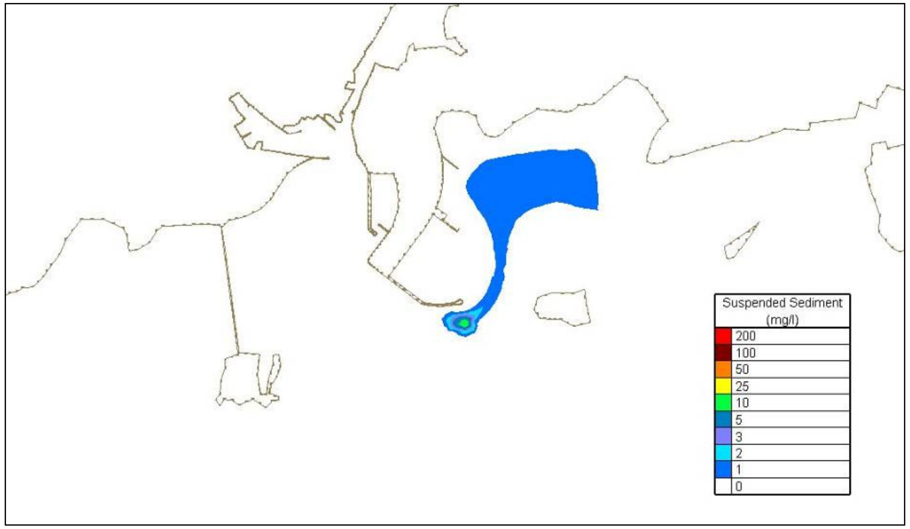
(i) Mid-ebb



(iii) Mid-Flood

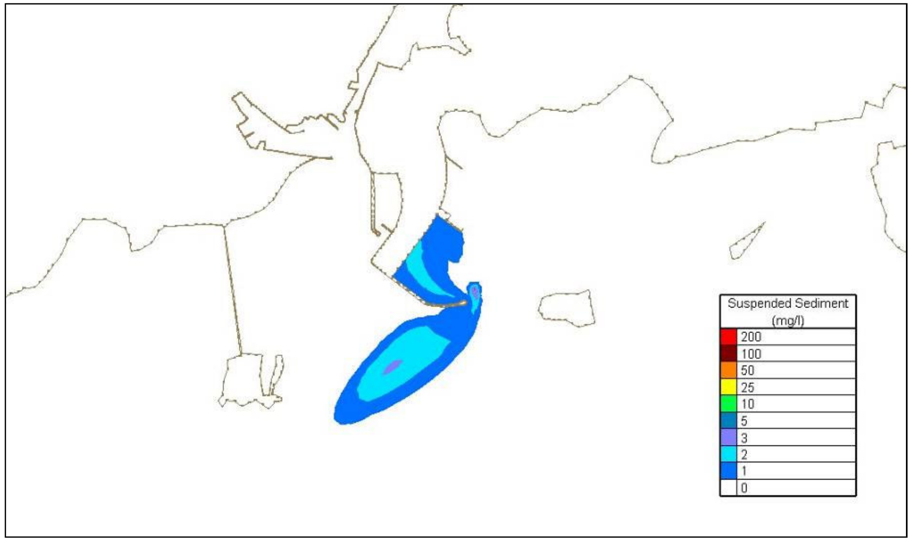


(iii) Low Water



(iv) High Water-

Figure 8.4.53 Dredge Plume at Location A3 for a Mean Spring tide and Summer Corrib low flow



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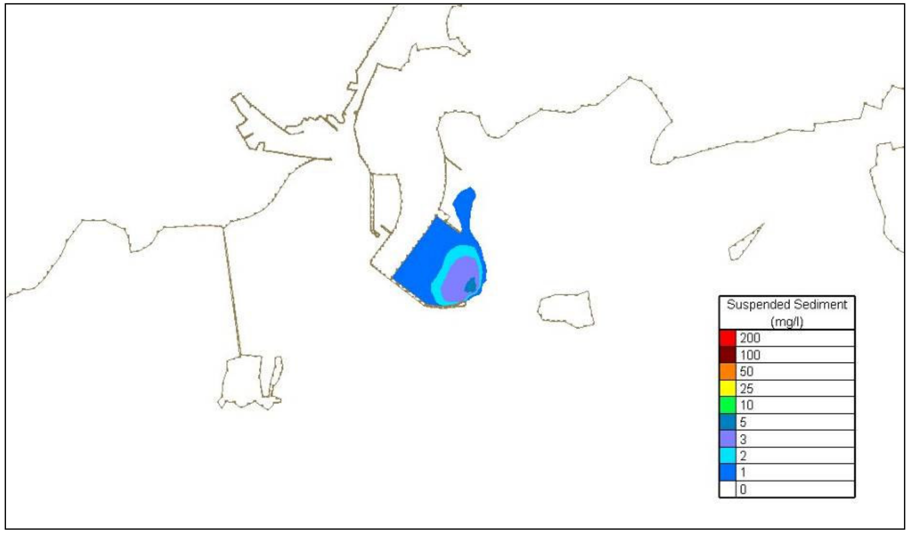
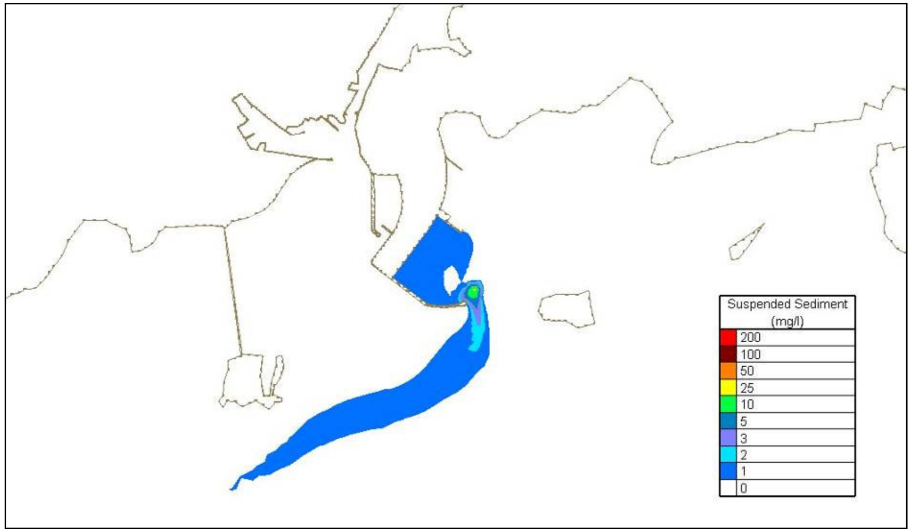
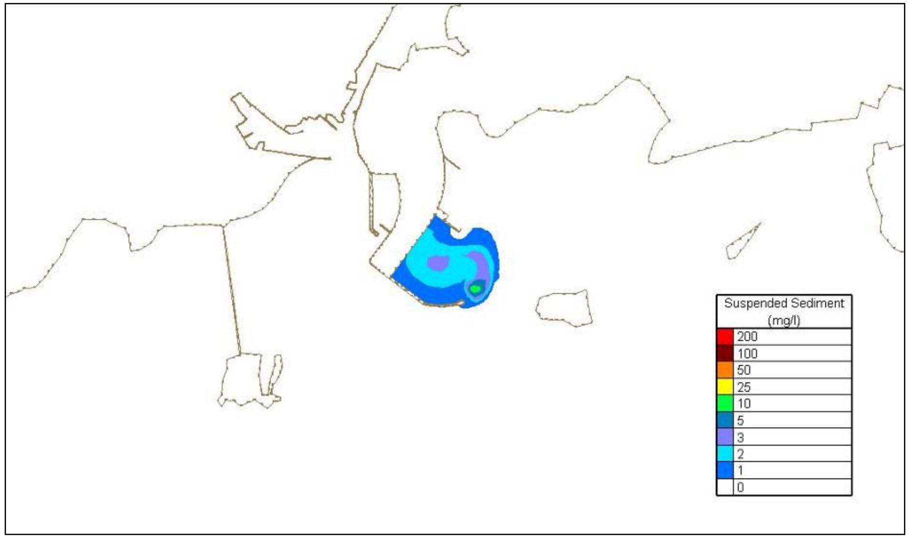


(iii) Mid-Flood

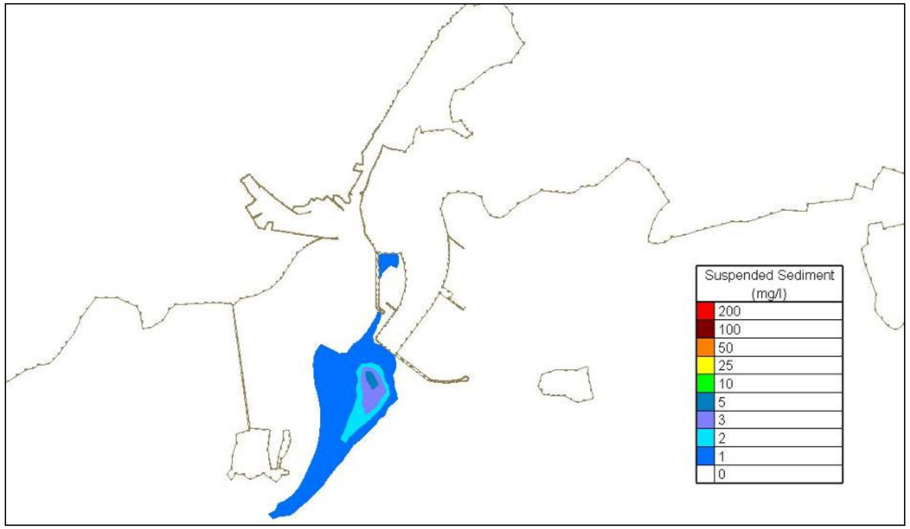


(iii) Low Water



(iv) High Water-

Figure 8.4.54 Dredge Plume at Location A4 for a Mean Spring tide and Summer Corrib low flow



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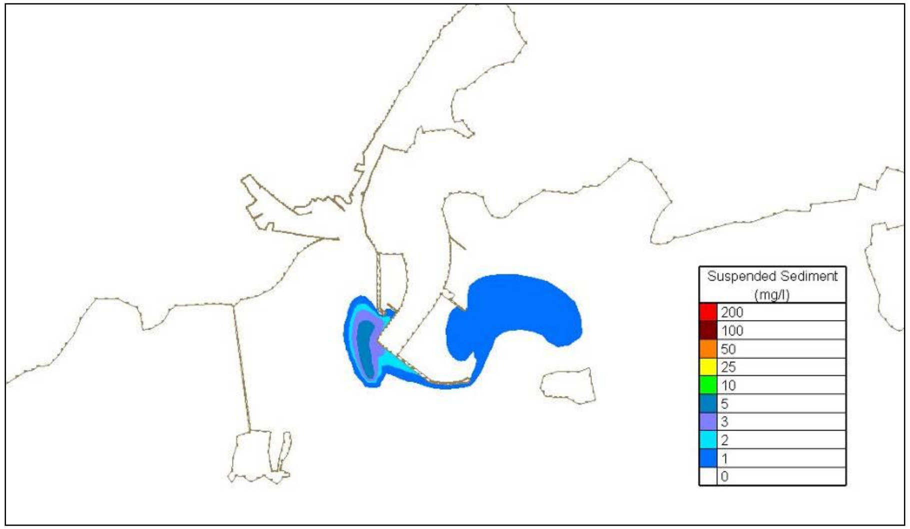
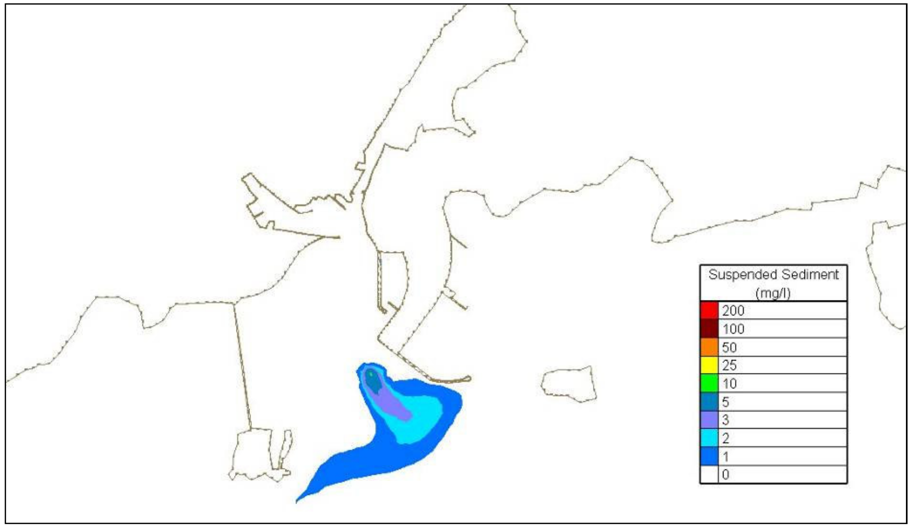
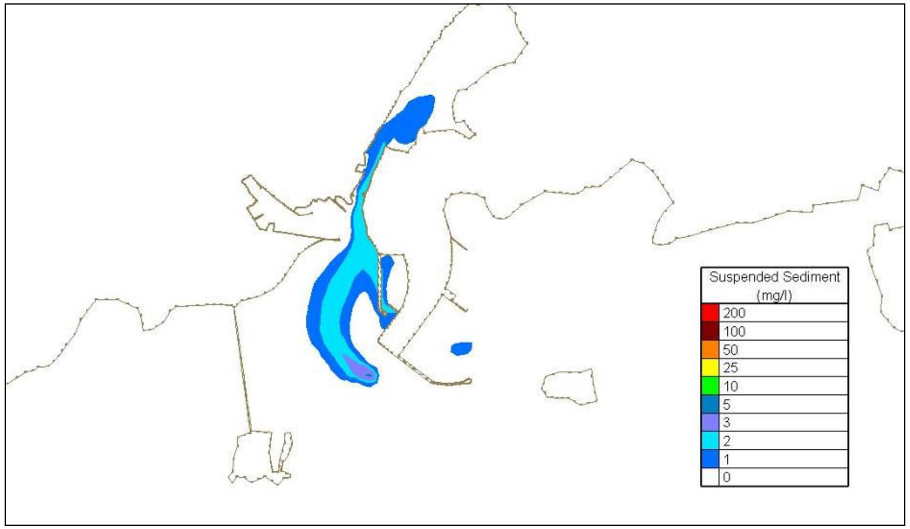


(iii) Mid-Flood

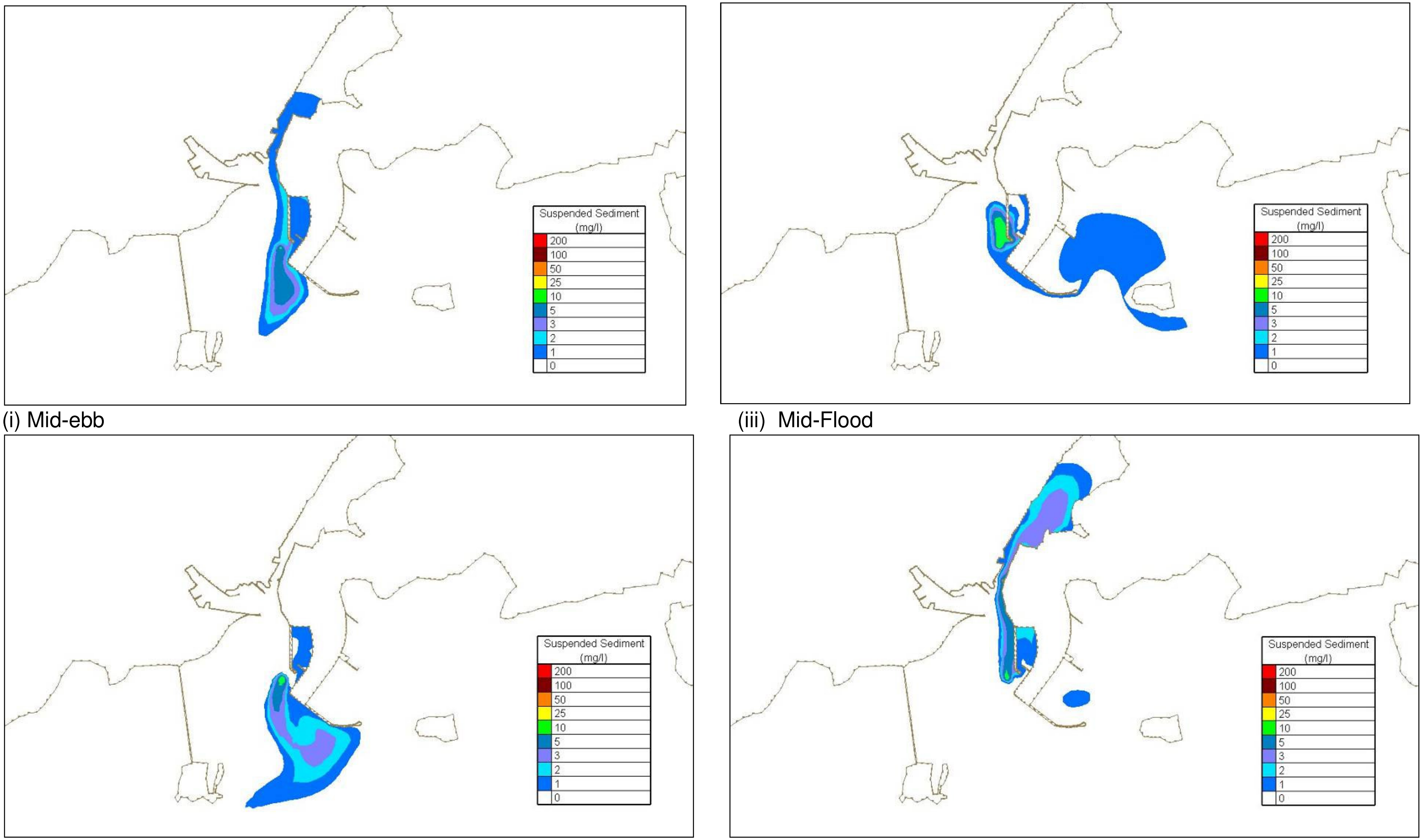


(iii) Low Water



(iv) High Water-

Figure 8.4.55 Dredge Plume at Location B1 for a Mean Spring tide and Summer Corrib low flow



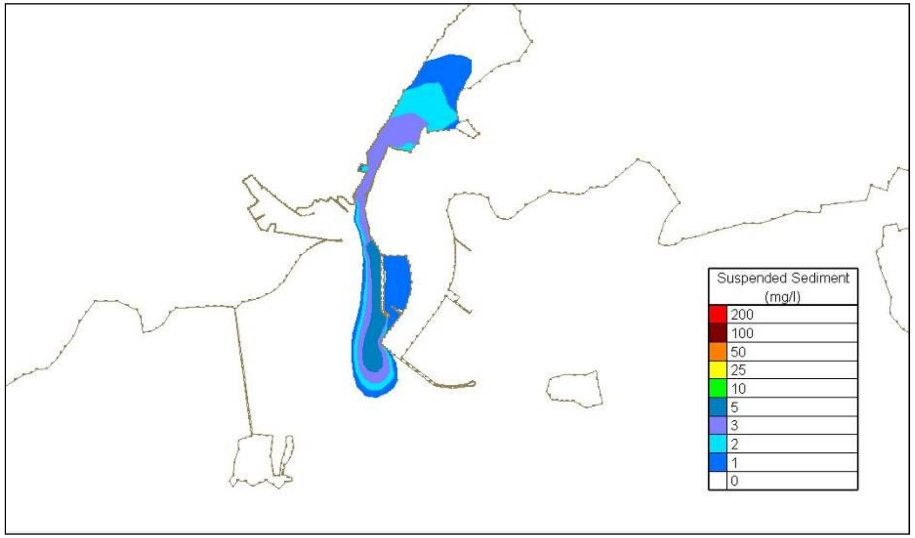
(i) Mid-ebb

(iii) Mid-Flood

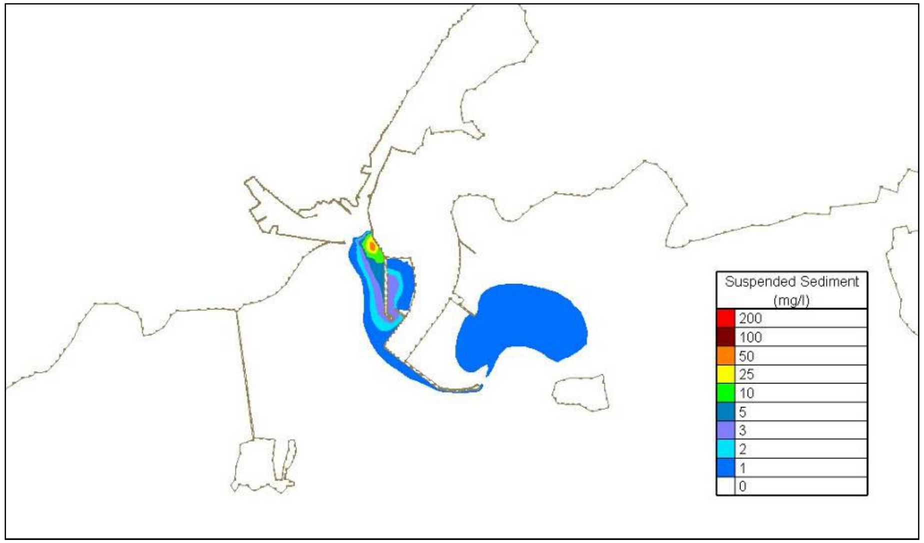
(iii) Low Water

(iv) High Water-

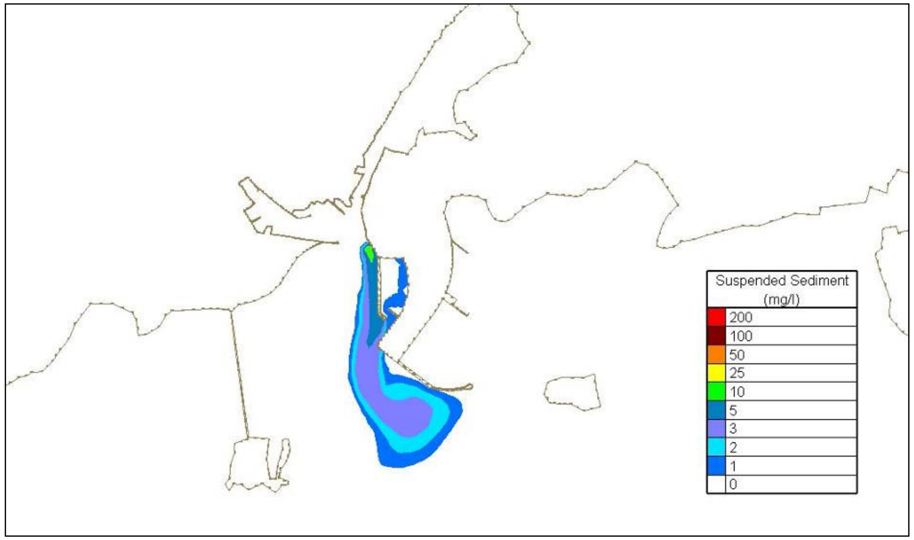
Figure 8.4.56 Dredge Plume at Location B2 for a Mean Spring tide and Summer Corrib low flow



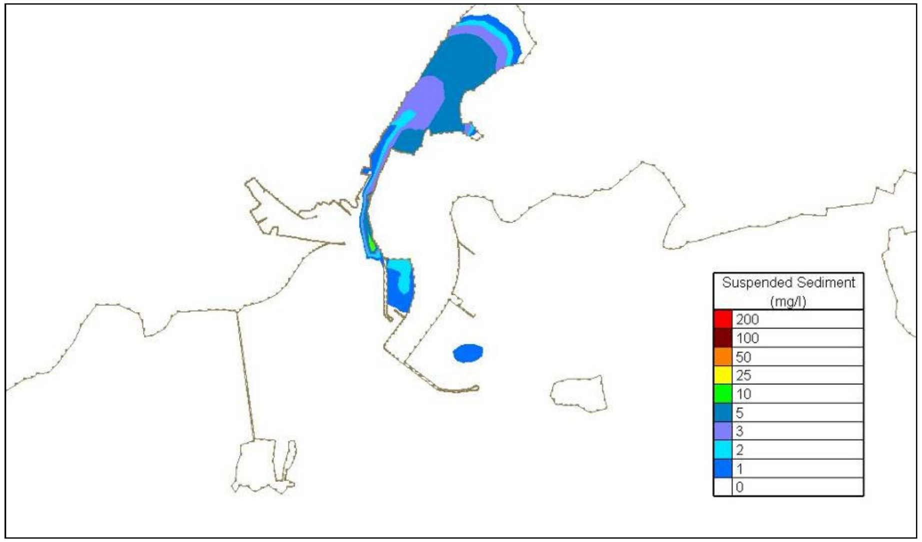
(i) Mid-ebb



(iii) Mid-Flood



(iii) Low Water



(iv) High Water-

Figure 8.4.57 Dredge Plume at Location B3 for a Mean Spring tide and Summer Corrib low flow

8.4.3 Salinity in Inner Galway Bay

8.4.3.1 Introduction

The River Corrib is the largest point source input of freshwater to Galway Bay. The river has a very strong effect in structuring the water column in the north-eastern section of the inner bay, especially during spate periods. This freshwater does not follow the anticlockwise flow of the Atlantic seawater within the bay but rather is influenced by the wind velocity and direction. In calm conditions, the river water flows in a west south-westerly direction along the north shore but when westerly gales are blowing, this water can be backed up into Oranmore Bay and New Harbour (Nolan, 1977).

The flow of the River Corrib affects surface salinities in the northeastern part of the Corrib estuary, *i.e.* northeast of Mutton, Lough Atalia and in the vicinity of Hare Island. The direction of the river flow is governed to a great extent by the man-built retaining walls of the river between the Salmon Weir Bridge and Wolfe Tone Bridge and further down to Long Walk to the east and the Claddagh Basin and Nimmo's Pier to the west and south at the river's mouth. On the ebbing tide, these structures funnel the flow into a strong linear easterly direction. Once the flow meets the lay-by wall and the westerly edge of the Enterprise Park, the flow is deflected southwards to the open sea. (This has always been the case historically as this southerly deflection occurred due to the presence of Renmore Point). The southerly flow is enhanced due to the outflow from Lough Atalia and the presence of the navigation channel. From there, the freshwater tends to flow seawards in a west/southwest direction and is regulated by the Coriolis effect (with rivers on the west coast flowing north) and prevailing wind direction. The Connemara coastline along the north shore of Galway Bay forces the freshwater to flow in a westerly direction. Low salinity at the surface also extended to Mweeloon Bay, New Harbour and Oranmore Bay (Nolan, 1977).

On the turn of the tide after low water, seawater fills from the southwest as it makes its way eastwards towards Oranmore Bay. After low tide, the flooding water flows south of Mutton Island, turning to the north and making its way towards the docks. Prior to the construction of the Mutton Island causeway, the tide would flood over an area to the south of South Park, but now with the causeway in place, it almost entirely flows around Mutton Island. The movement of the ebbing tide water is essentially the reverse of this.

Winds coming from the west to the south west sector are the strongest winds in inner Galway Bay. These winds can modify surface water current speeds causing water to be forced either to the north during southerly wind flows or easterly if the wind comes from the west.

In terms of river flow, there is a strong seasonality regarding the volume, with Winter and Spring months being the ones with the largest flows. Approximate maximum flows recorded by an OPW gauge in the River Corrib give a figure of *ca* 320 m^3/s.

Estuaries are well studied in terms of their oceanographic characteristics; freshwater, being less dense than sea water, floats on the surface of the water column and through entrainment, becomes mixed with salt water until it becomes of the same salinity as the surrounding sea water. The greater the river flow, the greater the area that will have lower salinity values. Tidal conditions play a very large role in determining the extent of the area: when the tide is low, the fresh water extends over a large area and the underlying "salt wedge" displays a shallow angle. This situation is completely reversed under high tide conditions with the freshwater being restricted to a much smaller area and the angle of the salt wedge being much more pronounced. The greatest variability of these features will be under Spring tides and spate flow conditions.
Salt wedges occur when the mouth of a river flows directly into seawater. The circulation is controlled by the sea as it pushes back in the seawater on the flood tide or is forced back out by the river under ebb tides. This creates a sharp salinity boundary that separates an upper less salty layer from an intruding wedge-shaped salty bottom layer.

Due to the high variations in salinity at the mouth of the Corrib, several different surveys were carried out to document this variability. These included surveys of Inner Galway Bay, Lough Atalia and Renmore Lagoon. Results for each of these areas are given below.

Lough Atalia and the small off shoot called Renmore Lagoon comprise an area of *ca* 40 ha of mostly shallow (less that 1 m) water. Oliver (2007) prepared a report on it for the National Parks and Wildlife Service and this is included as Appendix 7. In this, Oliver comments on fluctuations in salinity and as part of a wider lagoonal study. In 2013 additional sampling was carried out as part of a detailed dispersion modelling exercise to assess potential changes to water salinity in both Lough Atalia and Renmore Lough. The outcome of this study is discussed at Section 8.4.5.6.

8.4.3.2 Materials and methods

Survey work was carried out on the 1st of April 2009 from a 6.8 m RIB and salinity measurements were made using a salinity probe (see Figure 8.4.58 for station locations). The vessel's position was recorded continuously using a GPS. For the most part only surface salinity measurements were taken but some profiles were also collected. Historical salinity data (2/84 – 10/88) collected at a number of sites in inner Galway Bay using a temperature/salinity probe (Figure 8.4.59) were also examined and are presented as part of this report. Archived information (O'Connor, pers. obs) on the position of the Corrib River front is also included as are findings from a M.Sc. thesis (Nolan 1997) on the River Corrib plume.

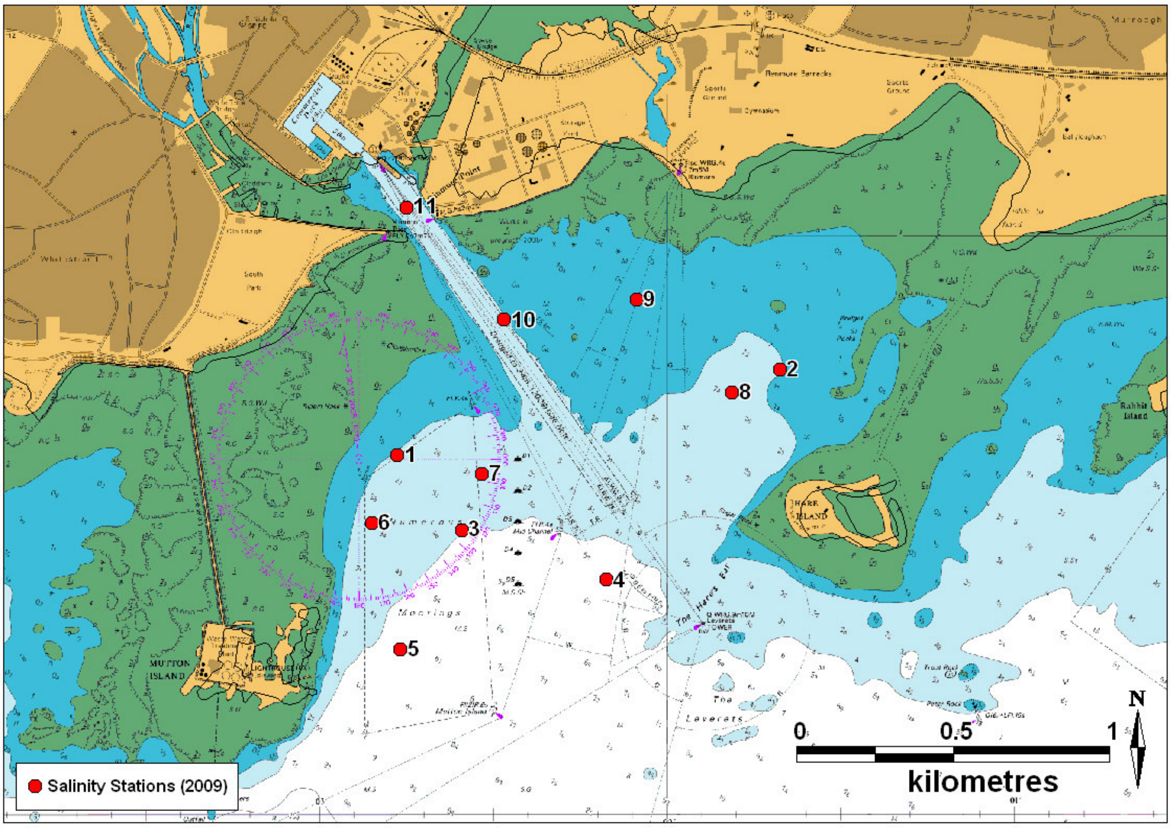


Figure 8.4.58 The stations where salinity readings were taken on the 1st of April 2009

8.4.3.2.1 <u>Lough Atalia and Renmore Lagoon</u>

On the 11th and 12th of August 2011, four sampling stations were selected along the western shore of Lough Atalia (see Figure 8.4.60). Surface salinities at each station were recorded. A single site in Renmore Lagoon was sampled from the eastern shore. On August 16th 2011, a transect from south to north was undertaken on an inflatable and vertical salinity profiles were taken at 7 stations (see Figure 8.4.60).

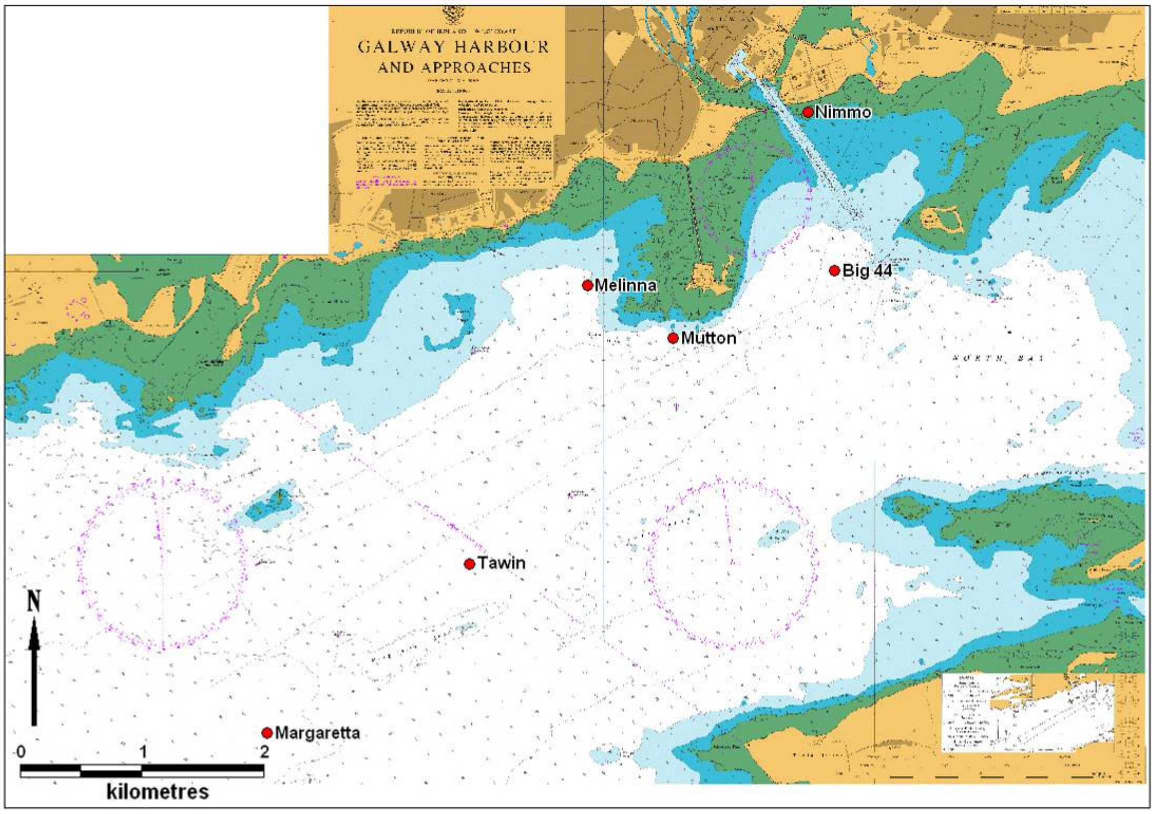


Figure 8.4.59 Locations of the stations where salinity data were collected from 1984 to 1988 by B.O'Connor

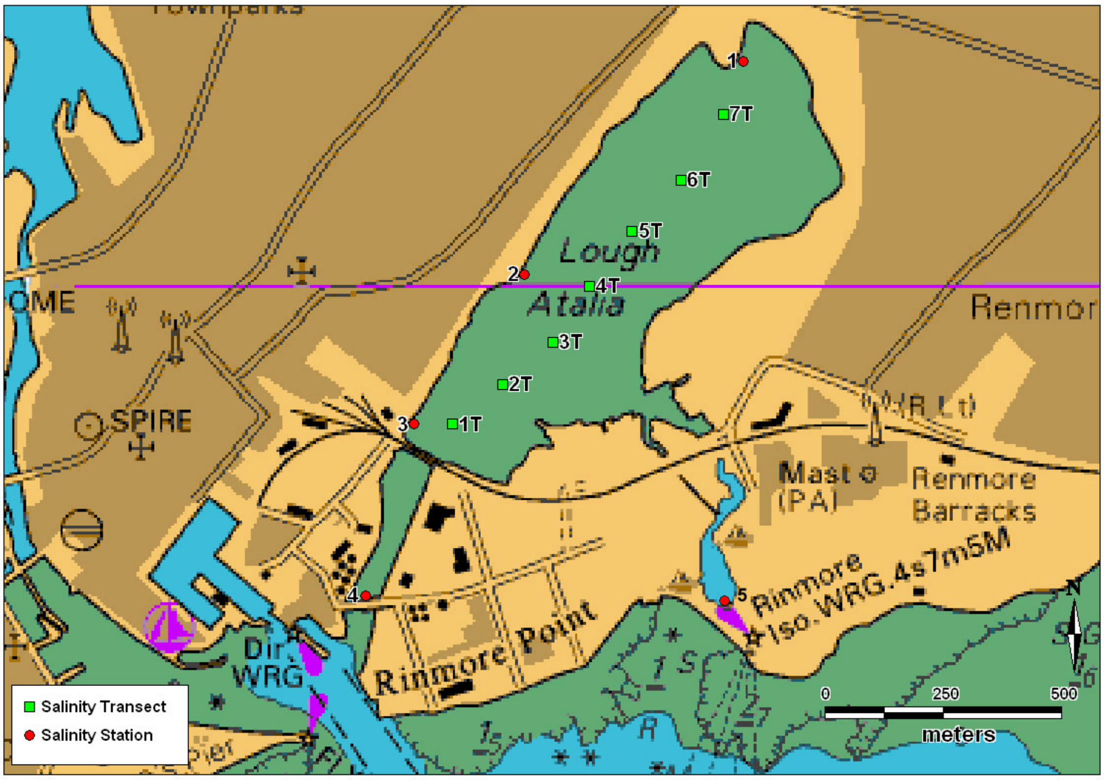


Figure 8.4.60 Salinity sampling sites, August 2011

Another shore survey took place between 12th September 2011 and 2nd May 2012 during which surface salinities were taken in five locations in Lough Atalia and Renmore Lagoon (see Figure 8.4.61). Stations 1 to 3 are located in Renmore lagoon, while stations 4 and 5 are located in Lough Atalia.

Figure 8.4.61 The five locations at which salinities were measured from shore (survey 12th September 2011 and 2nd May 2012).

Salinity profiles of Lough Atalia were again measured between 4th April 2012 and 4th May 2012. The locations at which these salinity profiles were taken are shown in Figure 8.4.62. The locations consist seven transects, each of three stations, giving a total of 21 stations.

Figure 8.4.62 Survey stations in Lough Atalia April/May 2012

8.4.3.3 Results

8.4.3.3.1 Galway Bay

Figure 8.4.63 shows the results of the surface salinity measurements and Figures 8.4.69-8.4.73 show the graphs of salinity profiles collected on the same date and previous dates. Depressed surface salinities (<10 S_P) are found in a band that runs from the northwest to the southeast while higher surface values (>10 S_P) are found to the south of this. The profile data collected in 2009 and in the period 1984 – 1988 show that surface salinities can show depressed values in the top 2 m while deeper water maintains typical inshore values of *ca* 33 S_P. Figures 8.4.64-8.4.66 how the position of fronts observed on previous dates and indicates that lower salinities can dominate extensive areas of the area between Mutton Island and Hare Island.

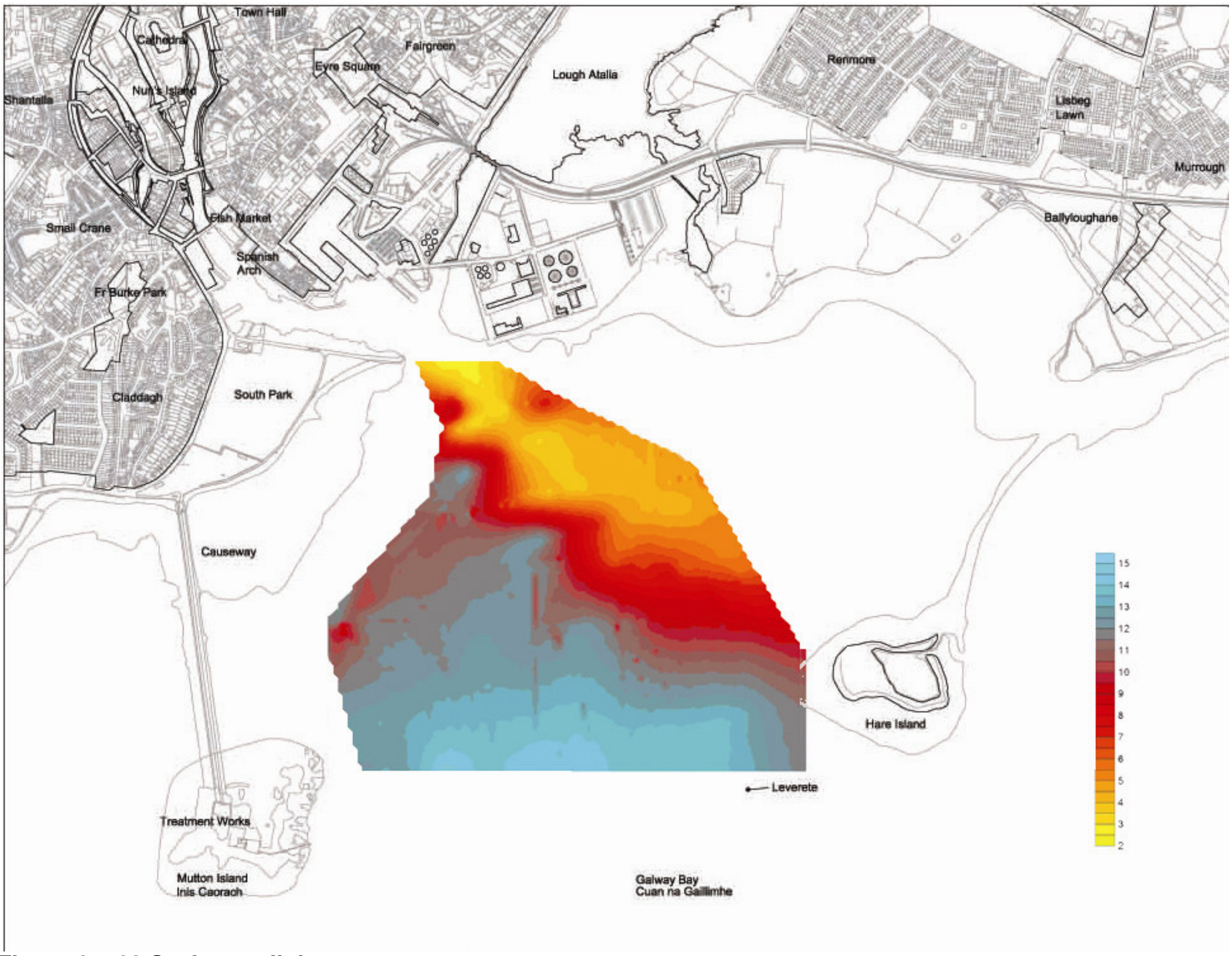


Figure 8.4.63 Surface salinity contours

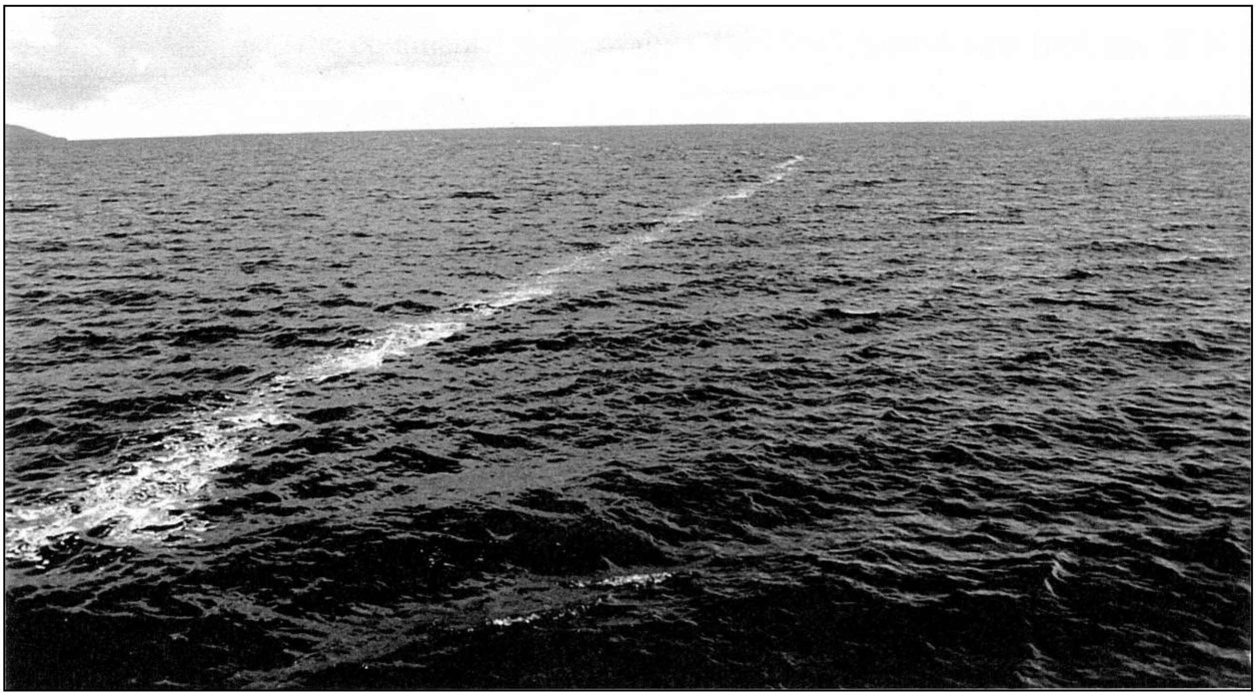


Figure 8.4.64 Saltwater / Freshwater Front (halocline) observed in Galway Bay

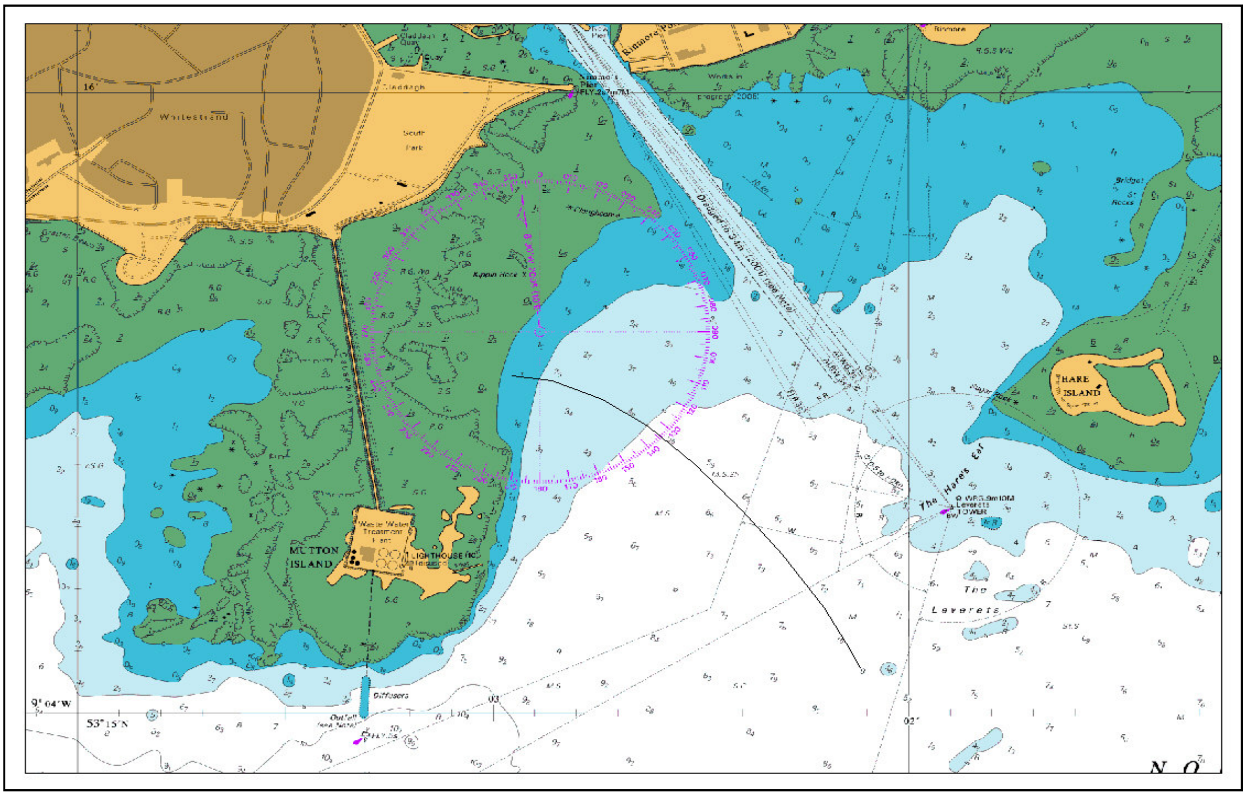


Figure 8.4.65 The black line indicates a portion of a saltwater/freshwater front recorded at the Corrib estuary in 1984

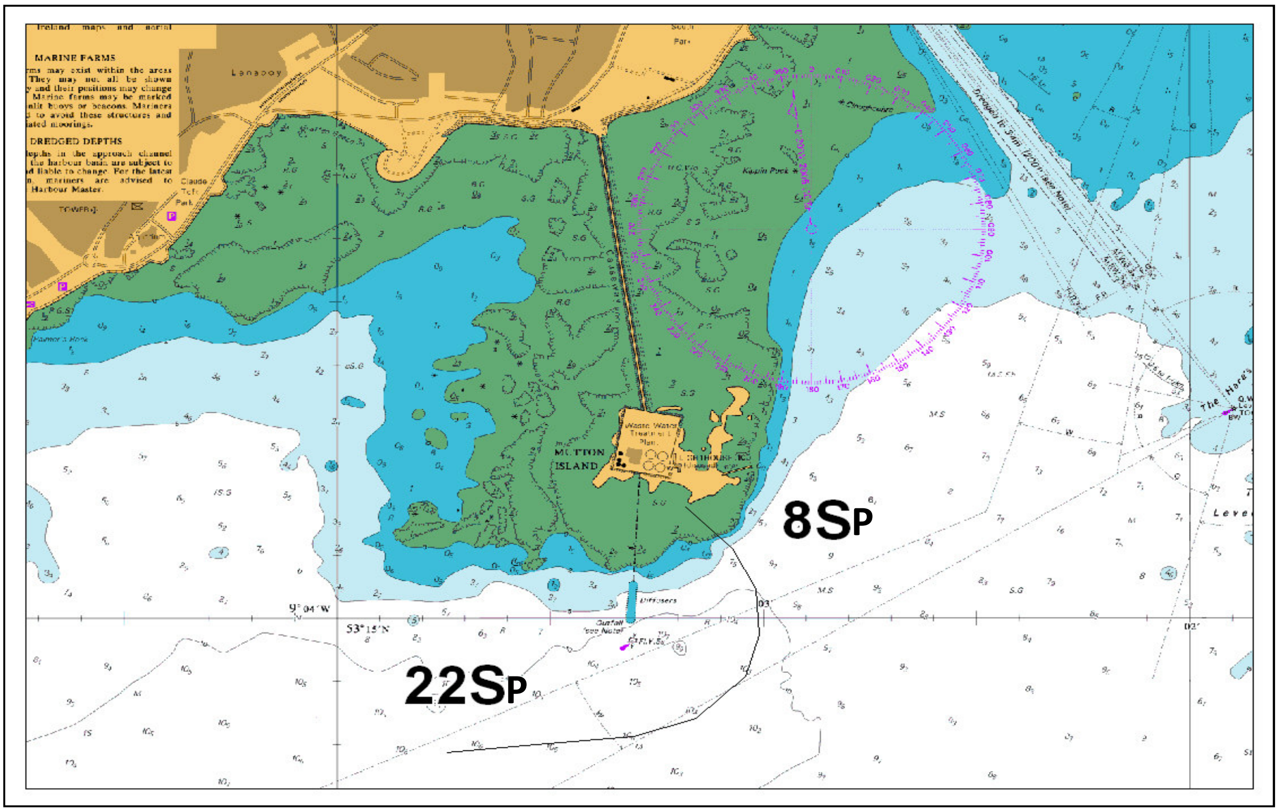


Figure 8.4.66 The black line indicates a portion of a saltwater/freshwater front recorded at Mutton Island during 1988

8.4.3.3.2 Lough Atalia and Renmore lagoon

Historic records of salinity data for Lough Atalia can be found in Moloney *et al.* (1990) and Sotillo *et al.* (2011). Values in Maloney *et al.* (1990) range from 4 – 13 S_P while Sotillo *et al.* (2011) record surface values from 18 – 20 S_P and bottom values from *ca* 19 – 24 S_P. The lough is dominated by low salinities during neaps and higher salinities during springs.

In the shore study of August 2011, values ranged from 2.5 – 17.6 S_P. Rainfall was heavy during the week, especially on the 10th of August and is likely to have had a bearing on readings taken that week. There was a high influx of fresh water from the nearby River Corrib entering Lough Atalia at the time and this was further influenced by the rising tide on both occasions. Table 8.4.4 shows the salinity reading from the four stations in Lough Atalia.

Lough Atalia Surface Salinities (S_P) August 2011.		
Station	Date: 11-08-11 Time: 14:32 – 15:07 Tide: HW @ 16:37 (4.6m CD)	Date: 12-08-11 Time: 14:28 – 14:55 Tide: HW @ 17:19 (4.8m CD)
1	13.1	5.0
2	16.0	17.0
3	17.6	17.5
4	3.0	2.5

Table 8.4.4 L. Atalia surface salinity values at 4 locations on two dates in August 2011 (mean tides)

On August 16th 2011, a transect from south to north was undertaken on an inflatable and vertical salinity profiles were taken at 7 stations. The values recorded from the 7 stations along the transect are shown in graphical form in Figure 8.4.67. Surface values ranged from to 20 – 23 S_P while salinities from 1 m depth ranged from 23.9 – 25.3 S_P. Deeper waters occur towards the mouth of Lough Atalia as shown by lines 1, 2 and 3. The deeper water can be seen in Figure 8.4.62, illustrated by the darker areas. Conversely, many areas outside this are shallow *ca* 1 m at high tide. Due to the shallow waters at the northern end of the lagoon, only surface salinities could be measured in this area so results of 4 to 7 appear only on the vertical axis 8.4.67.

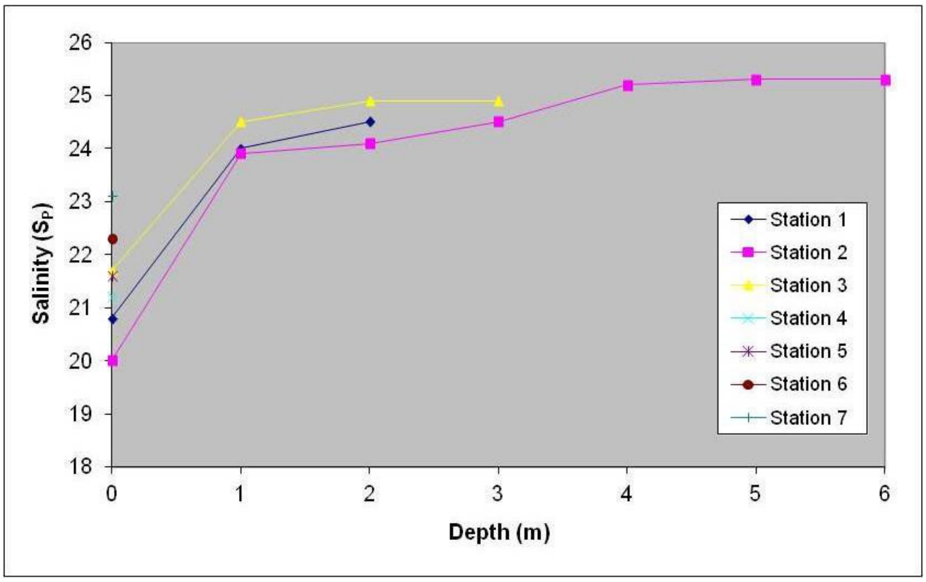


Figure 8.4.67 L. Atalia salinity profiles at 7 stations in August 2011

Salinity values for Renmore Lagoon varied from 12.7 on the surface to 13.1 S_P at 50 cm.

Results of the shore survey of Lough Atalia and Renmore Lagoon (refer to 8.4.61 for station location) between 12th September 2011 and 2nd May 2012 can be seen in Figure 8.4.68.

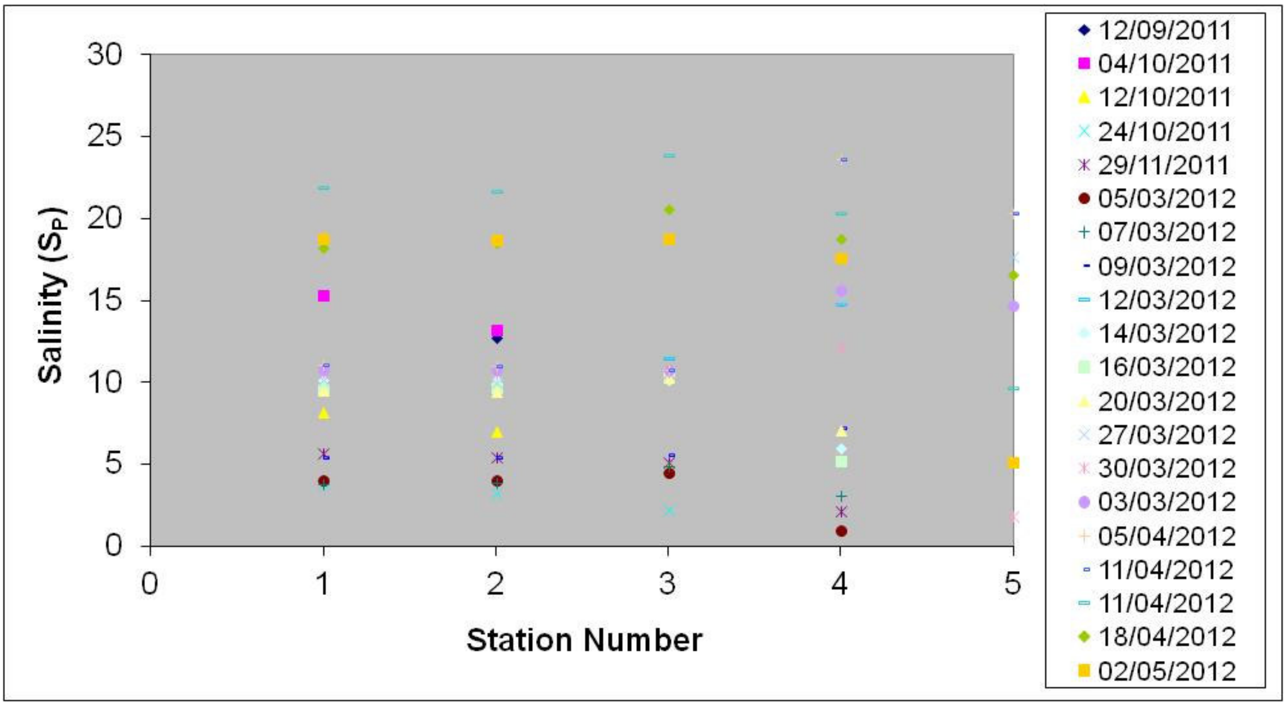


Figure 8.4.68 Salinities measured over a series of dates at three stations in Renmore lagoon and two stations on Lough Atalia

As can be seen in Figure 8.4.68, the range of salinities in Renmore lagoon (Stations 1-3) ranged from 2.2 to 23.9 S_P, both at Station 3 at the northern end of the lagoon. When taking the five stations into account, the highest salinity remains the same, while the lowest is 1 S_P found at Station 4. The salinities within the lagoon remain relatively constant between stations for the same dates, with Station 4 having slightly lower salinities and Station 5 the lowest salinities. The lower salinities at stations 4 and 5 relative to Renmore lagoon can be accounted for by the freshwater influence which enters Lough Atalia from the river Corrib. Station 5 is seen to be more affected by freshwater than station 4. The salinities within Renmore lagoon remain more or less constant between the southerly end and the northerly end, which is further from the sea, suggesting that there are no pathways directly between the sea and Renmore lagoon through the narrow land bank. This was tested statistically by comparing salinity values from Stations 1 and 3 using ANOVA. A p value of 0.89 means that there is no statistical difference between these two data sets.

Results of the second series of salinity profiles taken in Lough Atalia can be seen in Figures 8.4.69-8.4.73. The surveys were carried out at various stages of ebbing/flooding and Spring/Neap tides and thus show great variability between surveys. The salinity ranged from 7.5-29.4 S_P both of which occurred at the southern end of the lough. Surface salinities were generally lower near the southern end of the lough where the mouth is located. However, low surface salinities were also recorded towards the northern end. Salinities increase with depth, leading to the highest salinities being recorded at the deepest areas of the lough, namely Station 5. There is some evidence to suggest the formation of a temporary halocline in Lough Atalia under conditions of low mixing which disappears in high mixing conditions such as during a flooding tide.

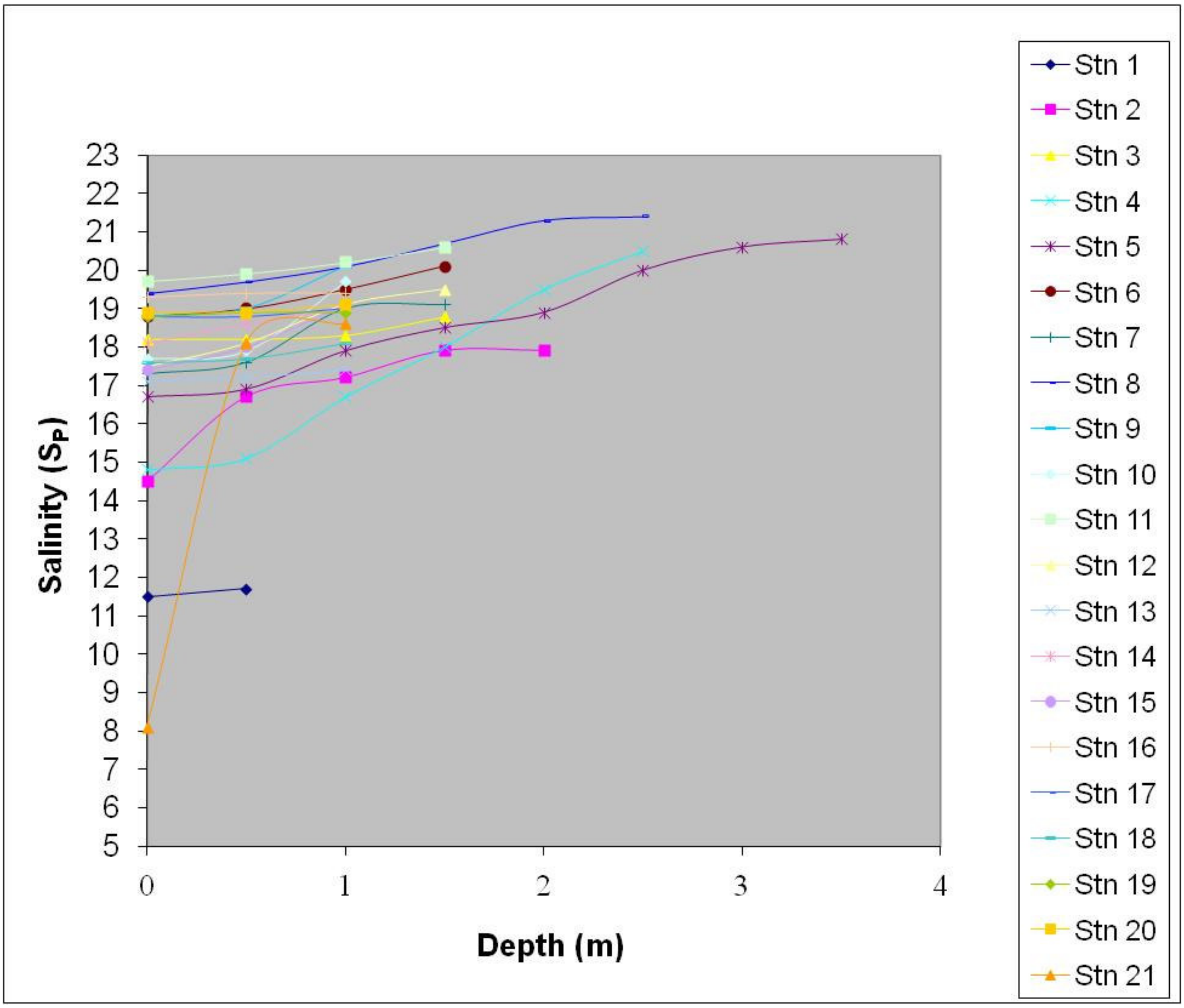


Figure 8.4.69 Salinity profiles of Lough Atalia 04/04/2012 (carried out in the two hours preceding high tide, three days prior to Spring tide)

Locations shown on Figure 8.4.62

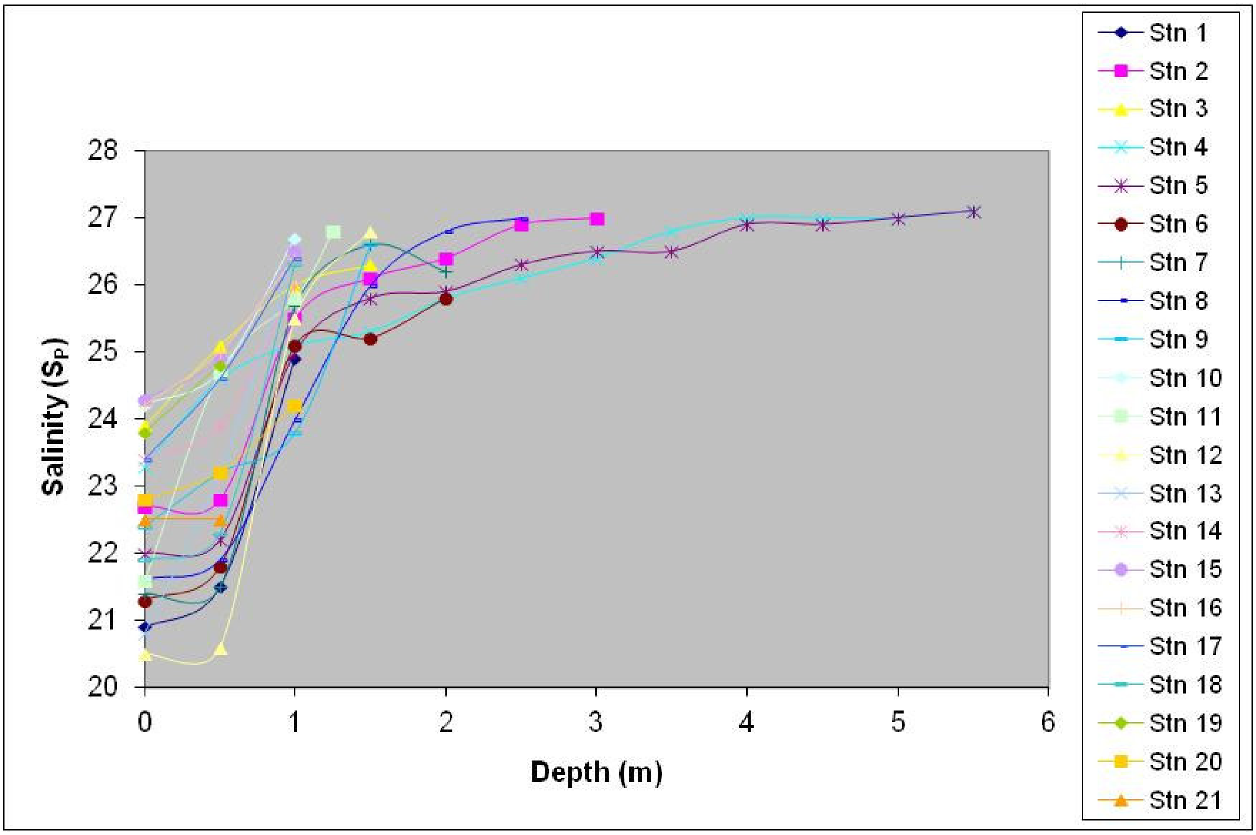


Figure 8.4.70 Salinity profiles of Lough Atalia 10/04/2012 (carried out on an ebbing tide, from approximately 2 to three hours after high water, on a diminishing Spring tide *i.e.* four days after a full moon)

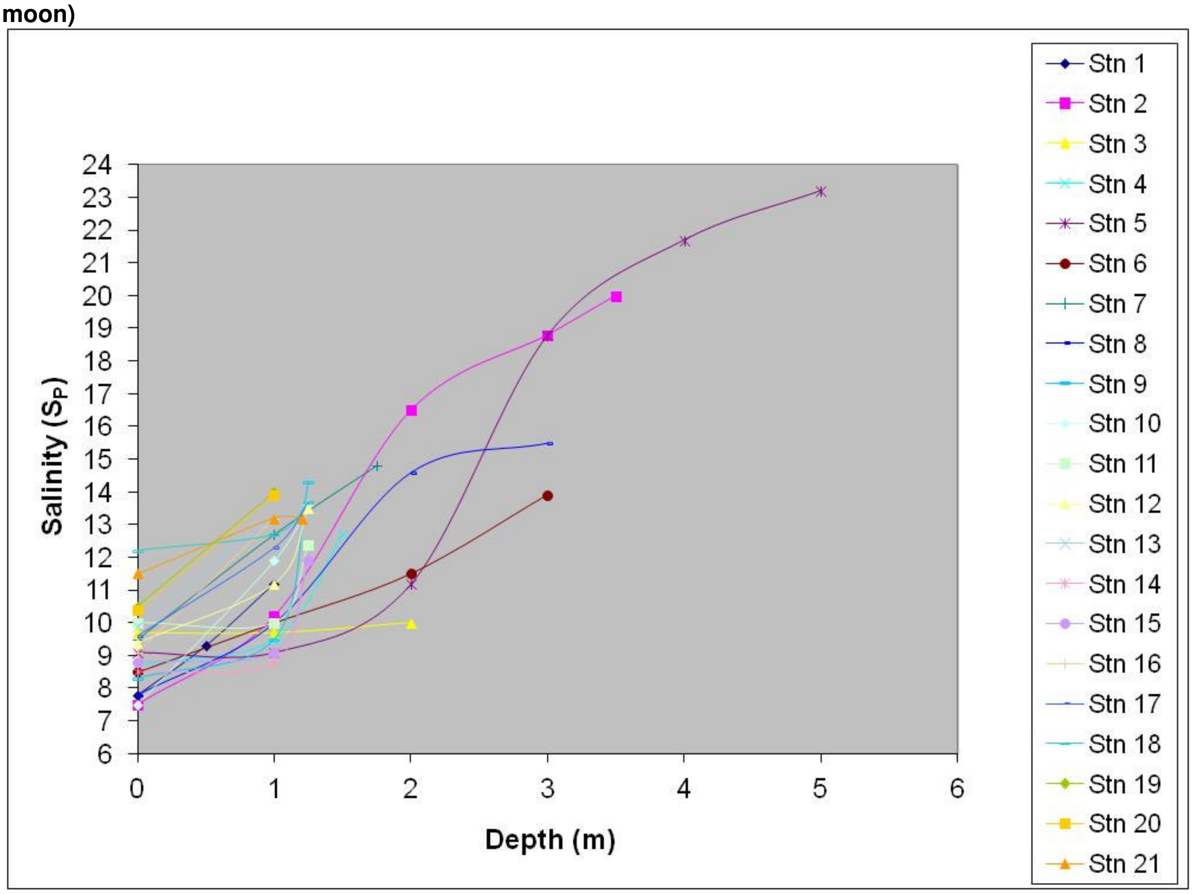


Figure 8.4.71 Salinity profiles of Lough Atalia 16/04/2012 (carried out approximately three-quarters of an hour either side of high tide, occurring two days after the peak of a Neap tide)

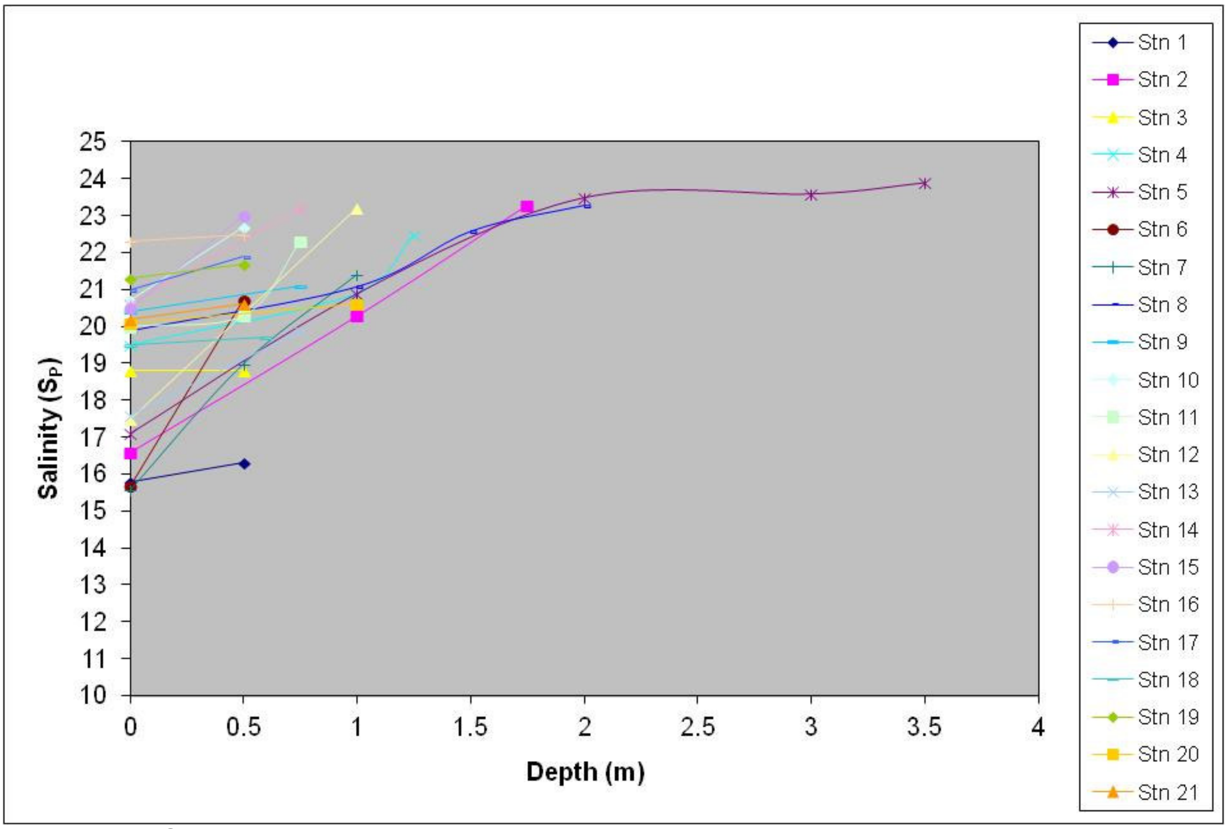


Figure 8.4.72 Salinity profiles of Lough Atalia 19/04/2012 (was carried out approximately 15 minutes before to 40 minutes after low tide, two days before the peak of a Spring tide)

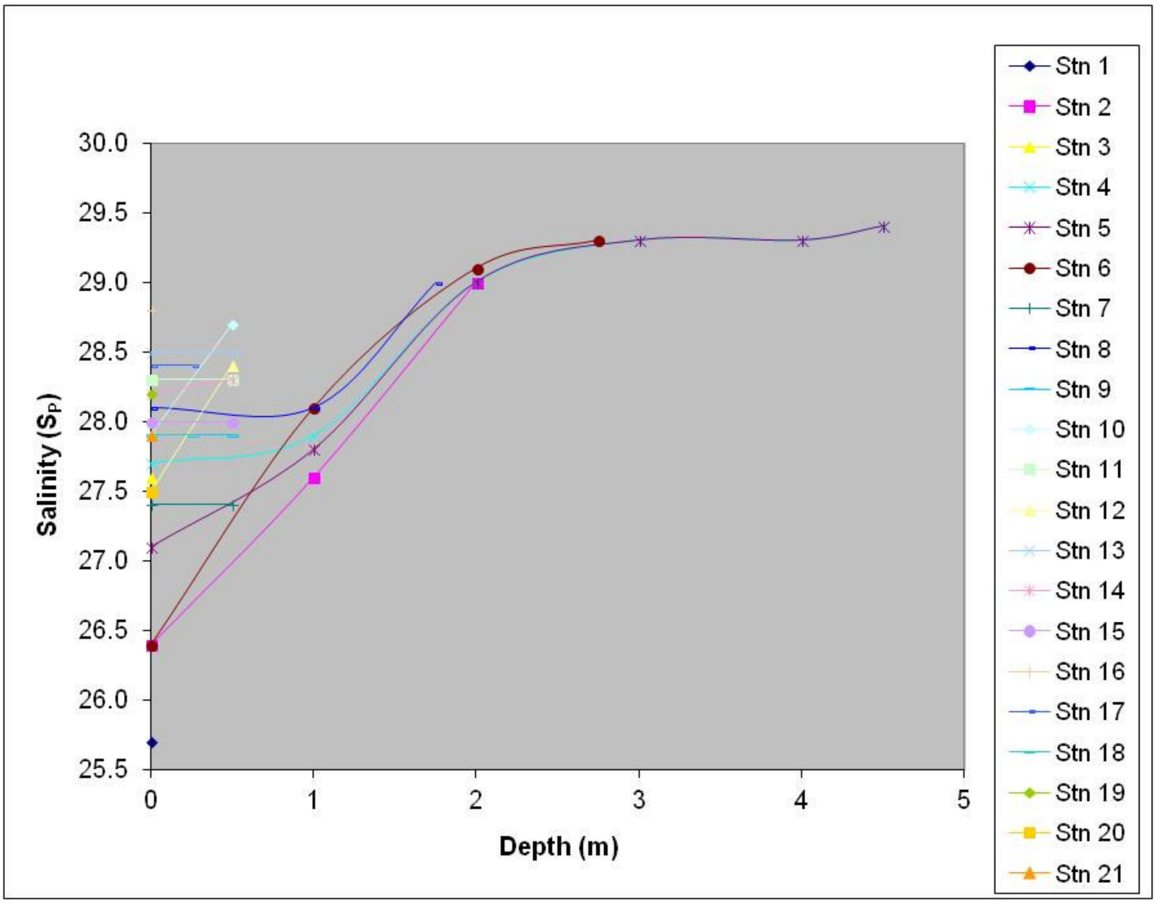


Figure 8.4.73 Salinity profiles of Lough Atalia 04/05/2012 (carried out during 1.5 hours directly after low water, two days before the peak of a Spring tide)

During the course of this series of salinity surveys, values in Lough Atalia ranged from 1.0 – 29.4 S_P with lowest values being recorded from the shallow, northern end of the lough and highest values at the deeper parts at the opening to the sea to the south. The low values at the northern end reflect the effects of surface run off. Low surface salinities recorded at the mouth are attributed to Corrib River water being brought back in to the lough by flooding tides.

Salinities within the River Corrib Estuary by nature are highly variable. During times of spate, such as in Winter, volumes of freshwater enter the system and surface salinities are depressed. The opposite effect occurs in periods of low river flow, resulting in higher surface salinities. Salinities in the area are also affected by the tidal state and the direction and strength of the prevailing wind, as discussed in the introduction to this section.

8.4.3.4 Model study

8.4.3.4.1 <u>Introduction</u>

A range of model simulation runs were performed with and without the proposed Galway Harbour Extension to assess and quantify the overall impact of the development on salinity levels in Lough Atalia, the Galway docks and approaches area, Renmore Shoreline area and Galway Bay in the vicinity of the proposed Harbour Extension. Model simulations were performed for a range of open sea tide and freshwater inflows from the Corrib. All other sources of freshwater inflow (*i.e.* small streams, storm outfalls, groundwater base flow and springs) were ignored as their flow contribution was considered minor in comparison to the River Corrib source.

A full baroclinic (density varying) three-dimensional hydrodynamic model TELEMAC-3D had to be employed to tackle this complex problem of a buoyant freshwater flow interacting with the more dense saline tidal waters at the Mouth to Galway Docks and Lough Atalia.

Salinity Units ppt and psu
Please note that the salinity measurement data referred to in this report are in the units of psu, whereas the hydrodynamic salinity model TELEMAC-3D refers to salinities in grams of salt per kilogram of solution (g/l or parts per thousand (ppt)). The modern oceanographic definition of salinity is the Practical Salinity Scale of 1978 (PSS-78). The numeric unit from PSS-78 is psu (practical salinity unit) and is distinct from the previous physical quantity ppt (kg salt per kg water in parts per thousand). Salinity values in ppt and psu are nearly equivalent by design, and for the purposes of this assessment can be treated as equivalent.

The model simulations were performed for a 16.5 day period (32 tidal cycles) and a time step of 2 seconds. This was sufficient to attain equilibrium salinity concentrations within Lough Atalia and in the vicinity of the Harbour Extension area, as it provided a 2.5day warm-up period and 14day spring-neap-spring tidal cycle. The time varying tidal curve specified at the open sea western boundary to drive the model simulations is presented below in Figure 8.4.74.

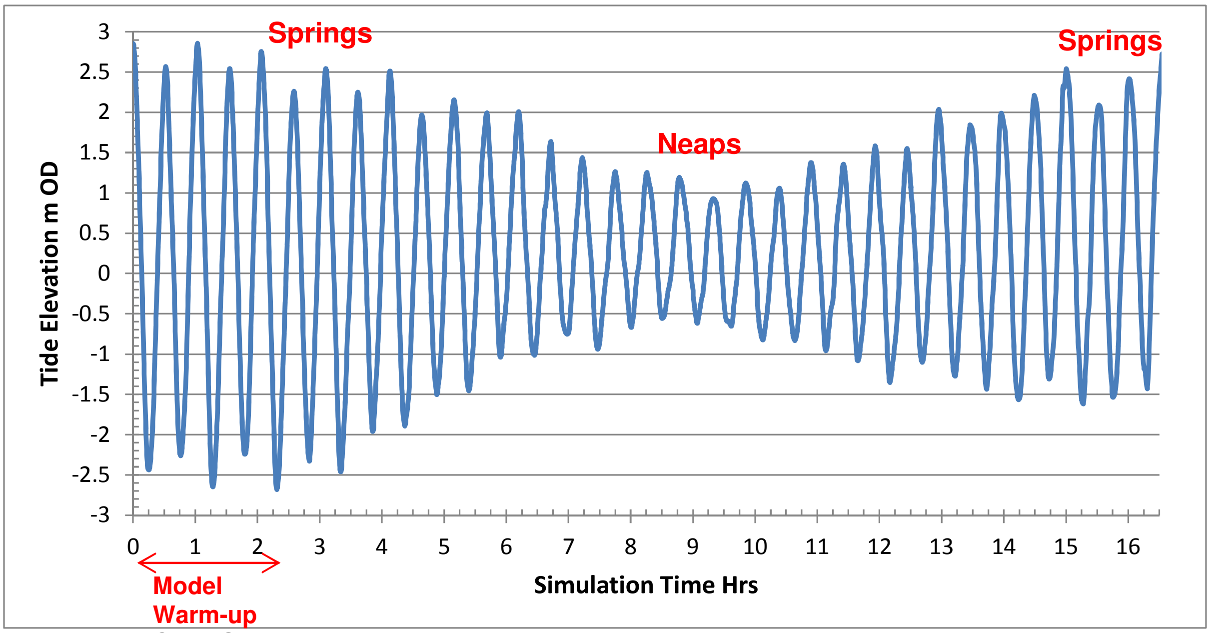


Figure 8.4.74 Open Sea Tidal conditions used in salinity simulations

Using the River Corrib flow duration curve information at Wolfe Tone Bridge gauge (30061) the following range of flow conditions were examined in order to quantify the overall impact on salinity by the proposed development:

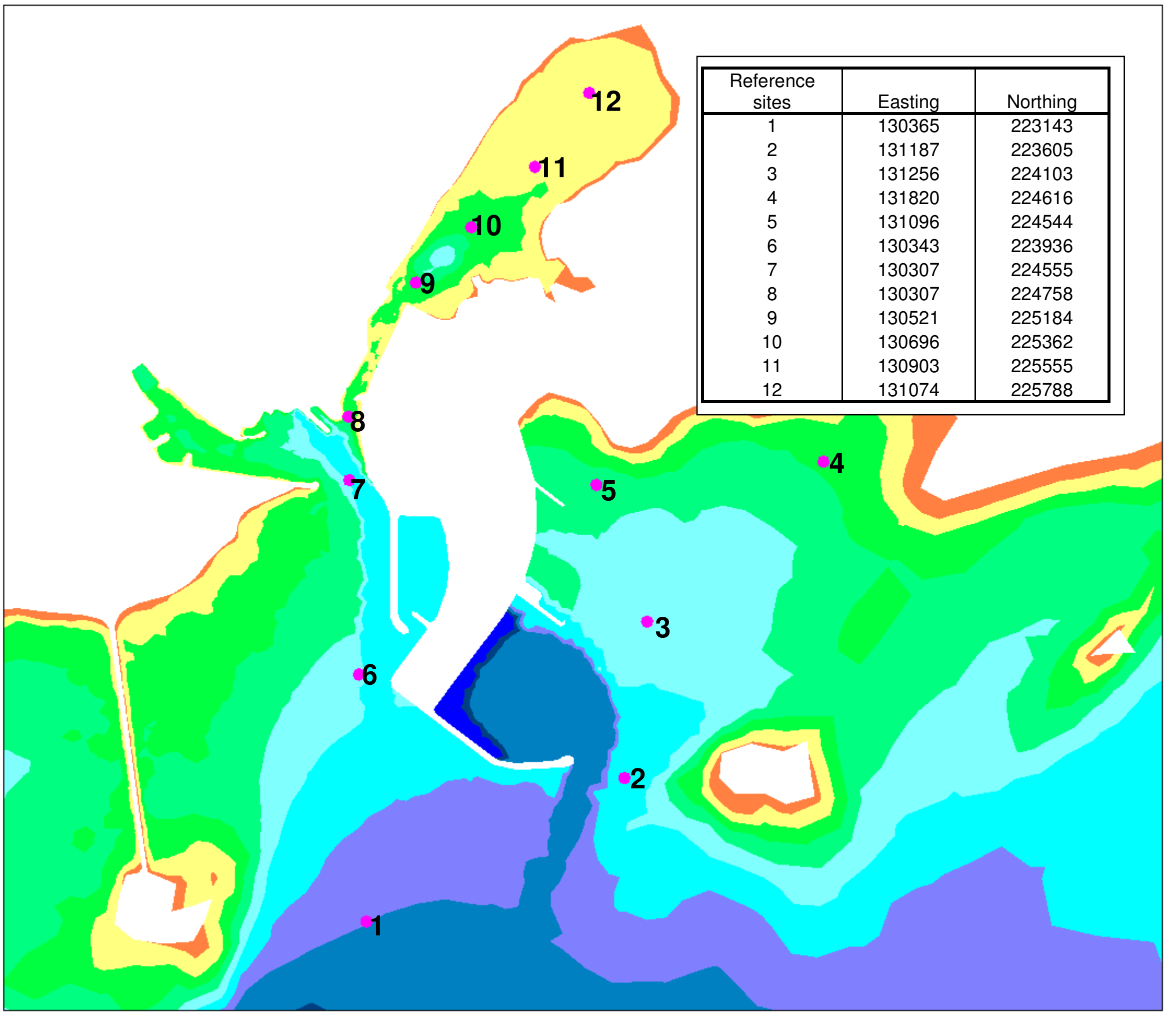
1. 99-percentile River Corrib low flow of 9.1 cumec
2. 90-percentile River Corrib flow of 28.5cumec
3. 50-percentile River Corrib flow of 82 cumec
4. 10-percentile River Corrib flow of 200 cumec
5. 1-percentile River Corrib flood flow of 272cumec

These flows were specified as constant inflows as opposed to a time varying flow hydrograph. This approach is considered reasonable and appropriate for the River Corrib given the highly damped nature due to the large Lough Corrib and its flow regulation by OPW at the Salmon Weir Sluice Barrage in Galway City.

8.4.3.4.2 <u>Simulation results</u>

In order to compare the predicted salinities with and without the proposed Harbour Extension, a number of reference sites within Lough Atalia the Approach Channel and the proposed harbour extension area were selected. At these reference sites time series of salinity concentrations were generated and analysed for each simulation run so as to directly compare the change in salinity value. These reference sites are presented below in Figure 8.4.75.

A salinity of 33 ppt was specified at the western open sea boundary and 0 ppt in the Corrib and an initial starting condition of 32 ppt throughout the bay. Simulations were then run for a 16.5day (32 tidal cycle period (spring-neap-spring) so as to obtain equilibrium conditions within the area of interest and particularly for the final 14 tidal cycles representing neap to spring tides.



Reference sites	Easting	Northing
1	130365	223143
2	131187	223605
3	131256	224103
4	131820	224616
5	131096	224544
6	130343	223936
7	130307	224555
8	130307	224758
9	130521	225184
10	130696	225362
11	130903	225555
12	131074	225788

Figure 8.4.75 Reference site for time series output of computed salinities

For each simulation run the temporal mean for the final 14 tidal cycles (neaps to springs) was performed and salinity contour plots of these mean salinities with and without the proposed development in the bottom, mid-depth and surface layers are presented in Figures 8.4.76 to 8.4.90. These demonstrate the stratification between the freshwater surface layer and the underlying saline layers, with the bottom layer being the most saline. The plots also demonstrate the sheltering effect that the harbour extension will have on the buoyant freshwater outflow resulting in more saline conditions to the East of the harbour extension (Renmore Bay area) and less saline conditions to the west and south of the development.

For each of the five hydrodynamic simulations summary tables of the salinity predictions at the 12 reference sites for proposed and existing cases, along with a summary of salinity differences are presented in Tables 8.4.5 to 8.4.14.

8.4.3.5 Discussion of Results

The tide simulations for various freshwater inflows from the Corrib show the deflection of the Corrib freshwater plume westward due to the harbour extension site with freshwater only arriving into Renmore Bay and Ballyloughan area on the subsequent flooding tide. In the existing case there is a wider area for the plume to disperse with no physical structure to prevent the plume migrating east and southeast on the ebbing tide and thus availing of a greater area for dispersion. With the proposed development, the Corrib plume is directed more southwards with reduced opportunity for the freshwater plume to directly disperse into the Renmore Bay area on the returning tide.

The modelling demonstrates significant increases in salinity to the east of development with greatest changes occurring to the northeast of proposed harbour extension, with the model reference sites 3, 4, and 5 showing an average rise in salinity of 2.4, 4.2 and 5.4ppt respectively. These changes are to be expected as the plume of the River Corrib which under present conditions is able to flow on a ebbing tide eastwards past the existing Enterprise Park and Ballyloughan, will be unable to access this area with the new structure in place. As this area will receive less freshwater, it will also receive less suspended sediments and debris that are carried by the River Corrib. These changes will bring about improved bathing water conditions at Renmore Beach and at Ballyloughan. These increases in salinity may bring about a change in benthic fauna whereby lower salinity-intolerant species such as echinoderms may colonise the muddy sands/sands in this area.

Less significant changes in salinities levels (reduction in salinity) are predicted to take place to the west of the structure and very minor changes predicted for Lough Atalia or the waters beyond Mutton Island. In the approaches to Galway Docks, south of Nimmo's Pier (reference sites 6 and 7) reduction in average salinity concentrations of 1.5 to 2ppt are predicted.

The impact of the proposed harbour extension on salinity in Lough Atalia (using reference Sites 9, 10, 11 and 12 of Figure 8.4.75) and integrating the results over the five hydrodynamic simulation runs considered, gives an overall predicted reduction in the mean salinity within Lough Atalia of 1.29ppt. A more detailed discussion of the salinity impact to Lough Atalia and Renmore Lough follows in Section 8.4.5.6.

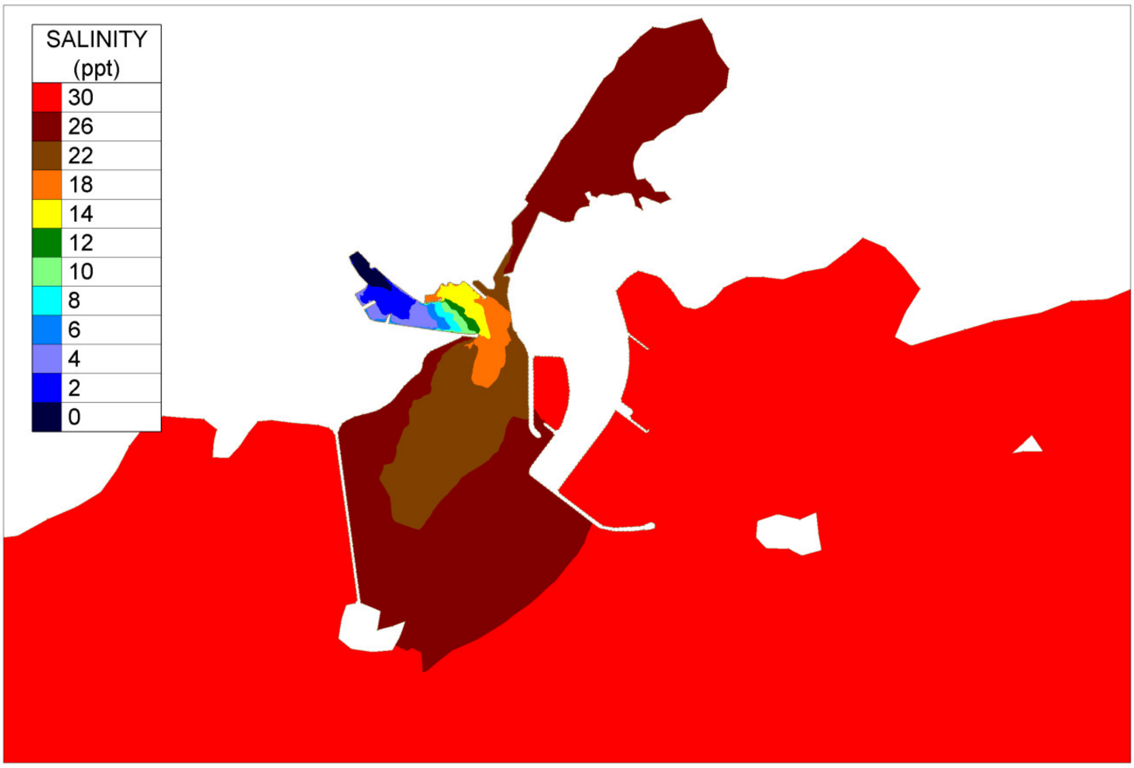


Figure 8.4.76 Mean Salinity concentration in bottom layer for existing and proposed cases under 99-percentile Corrib Low Flow (9.1 cumec)

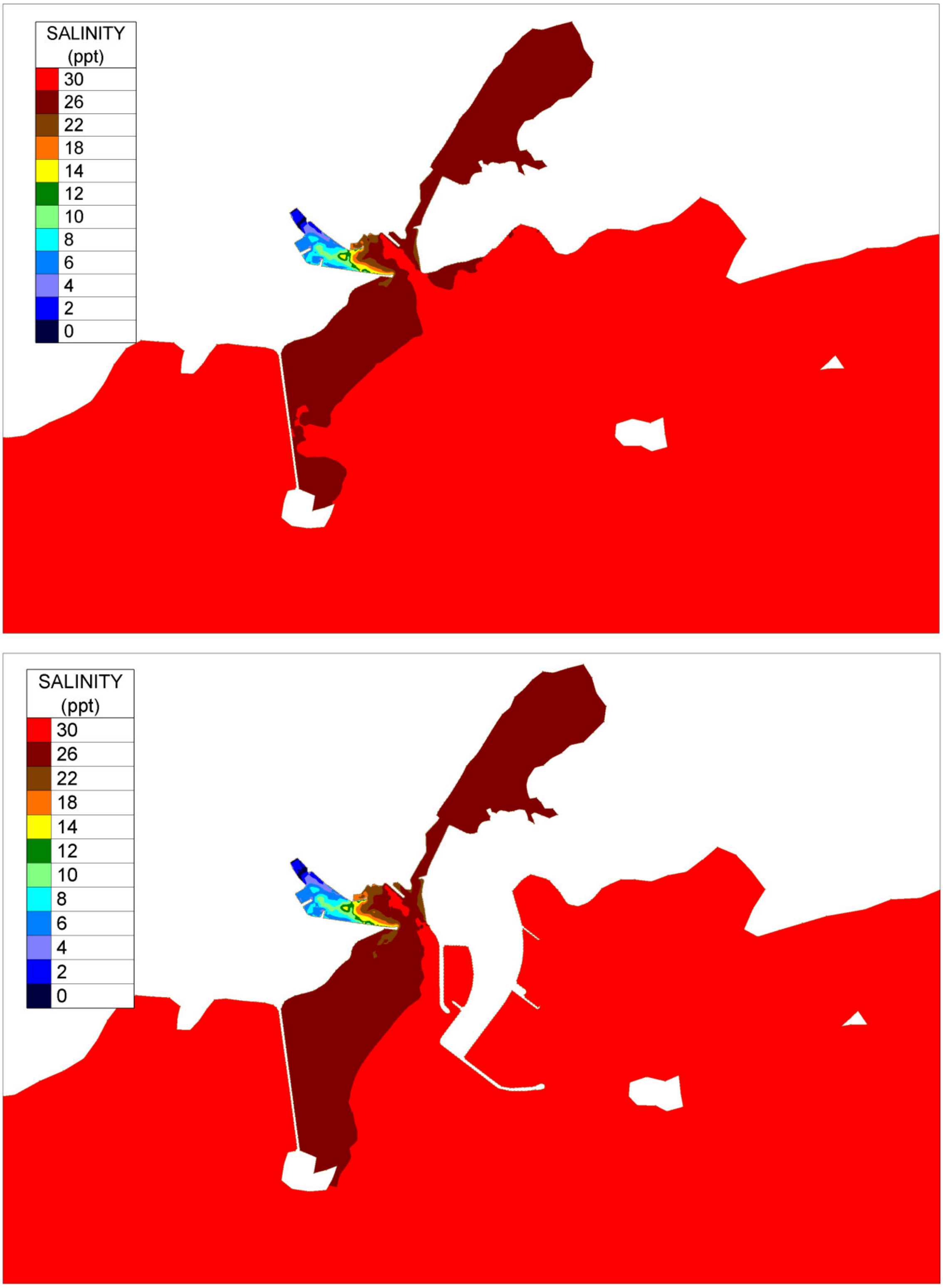


Figure 8.4.77 Mean Salinity concentration in mid-depth layer for existing and proposed cases under 99-percentile Corrib Low Flow (9.1 cumec)

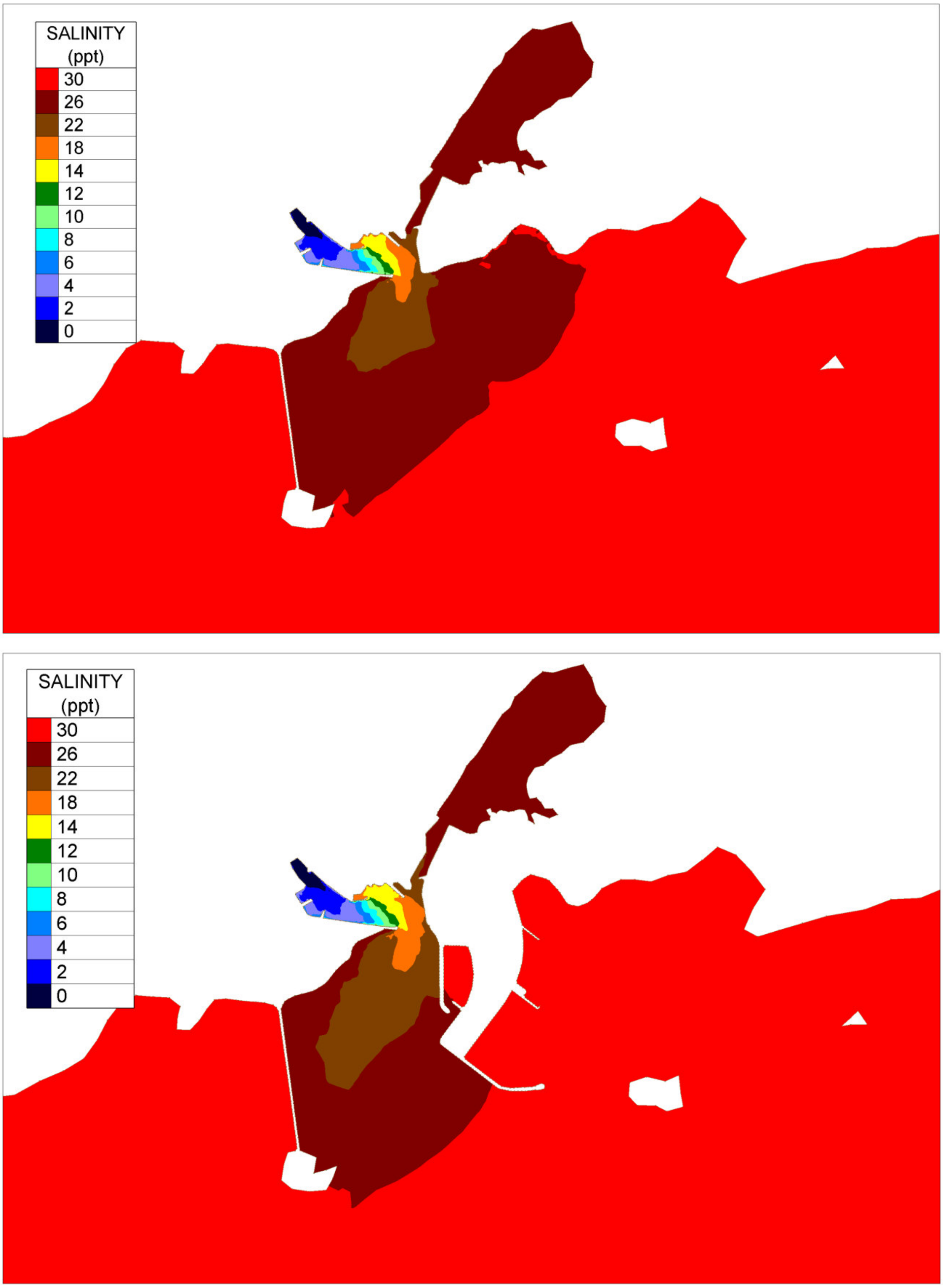


Figure 8.4.78 Mean Salinity concentration in surface layer for existing and proposed cases under 99-percentile Corrib Low Flow (9.1 cumec)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	32.31	29.74	30.81	32.71	32.16	32.48	32.87	32.58	32.75
2	32.02	30.27	30.91	32.64	31.86	32.34	32.8	32.19	32.63
3	31.31	29.2	30.28	32.39	31.44	32.01	32.76	32.08	32.52
4	32.03	29.28	30.54	32.03	30.75	31.42	32.25	31.4	31.9
5	30.99	26.39	29.41	31.91	29.05	31.1	32.43	29.5	31.77
6	30.69	25.22	27.89	32.43	30.18	31.88	32.79	32.22	32.62
7	25.76	15.58	21.31	31.54	26.64	30.2	32.51	29.96	31.94
8	29.17	21.98	26.03	30.62	25.56	27.76	31.83	25.57	28.44
9	29.39	25.5	27.2	29.41	25.62	27.31	29.44	25.89	27.45
10	29.1	26.13	27.12	29.11	26.2	27.25	29.15	26.36	27.48
11	29.09	26.29	27.22	29.09	26.28	27.29	29.09	26.28	27.32
12	28.26	26.64	27.25	28.27	26.65	27.29	28.29	26.66	27.31

Table 8.4.5 Salinity Concentrations for neap to spring tides under 99-percentile low flow in Corrib – Existing Case (without development)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	32.02	29.58	30.4	32.64	32.16	32.47	32.87	32.49	32.76
2	32.38	30.51	31.65	32.58	31.87	32.35	32.75	32.15	32.53
3	32.12	31.53	31.88	32.56	32.07	32.31	32.64	32.12	32.43
4	32.19	31.7	31.94	32.21	32.03	32.11	32.27	32.06	32.17
5	32.14	31.87	32.05	32.22	32.08	32.14	32.31	32.09	32.2
6	29.92	23.45	26.91	32.25	28.82	31.45	32.8	31.82	32.64
7	25.71	14.43	20.45	31.08	25.09	29.61	32.37	28.88	31.78
8	28.76	21.19	25.43	30.27	24.97	27.2	31.65	24.9	27.96
9	28.98	24.8	26.62	29.01	24.91	26.75	29.05	25.22	26.89
10	28.66	25.47	26.52	28.7	25.55	26.67	28.75	25.72	26.91
11	28.65	25.64	26.62	28.66	25.64	26.69	28.65	25.64	26.72
12	27.73	26.01	26.64	27.74	26.02	26.68	27.77	26.04	26.7

Table 8.4.6 Salinity Concentrations (ppt) for neap to spring tides under 99-percentile low flow in Corrib – Proposed Case (with Harbour Extension)

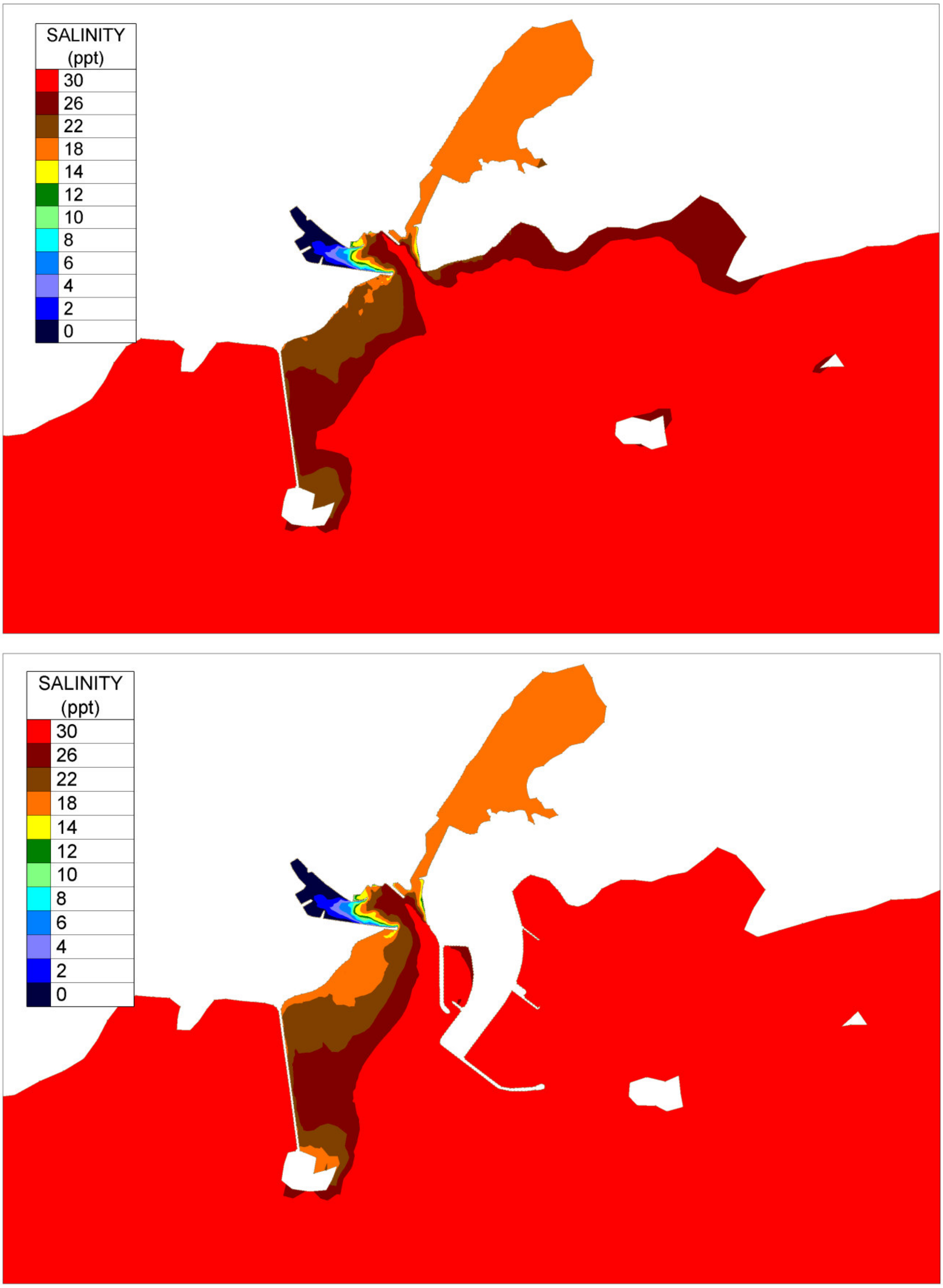


Figure 8.4.79 Mean Salinity concentration in bottom layer for existing and proposed cases under 90-percentile Corrib Flow (28.5cumec)

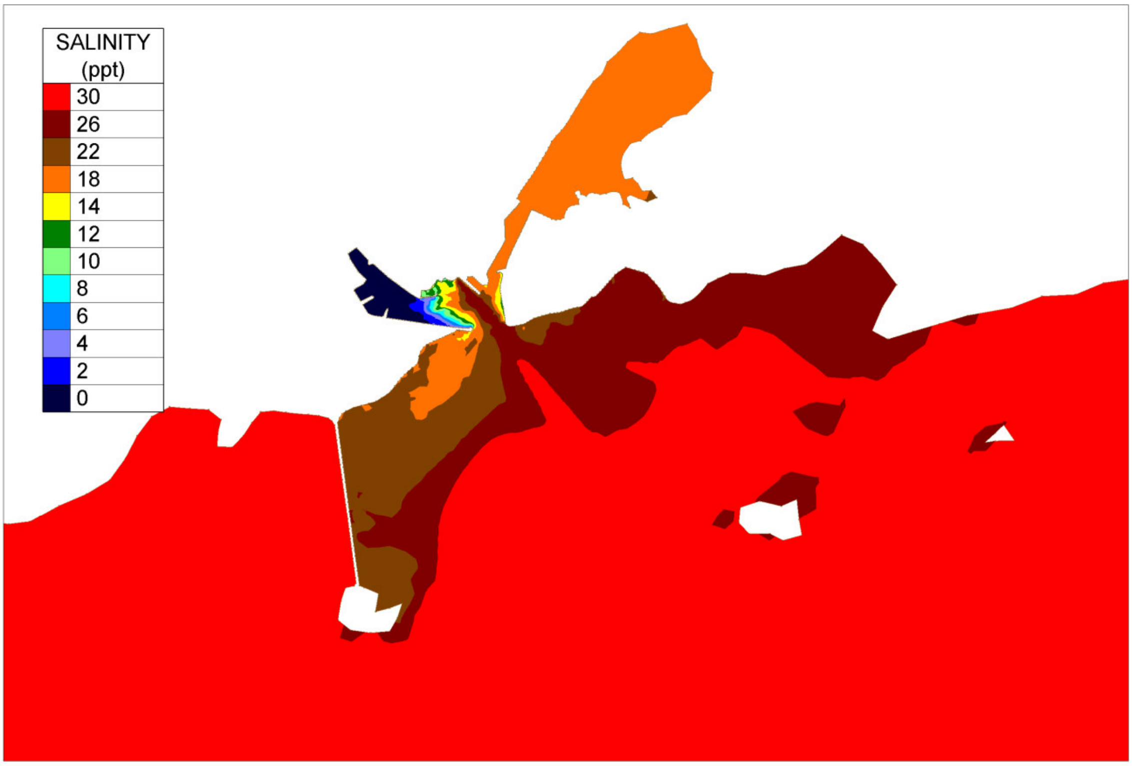


Figure 8.4.80 Mean Salinity concentration in mid-depth layer for existing and proposed case under 90-percentile Corrib Flow (28.5cumec)

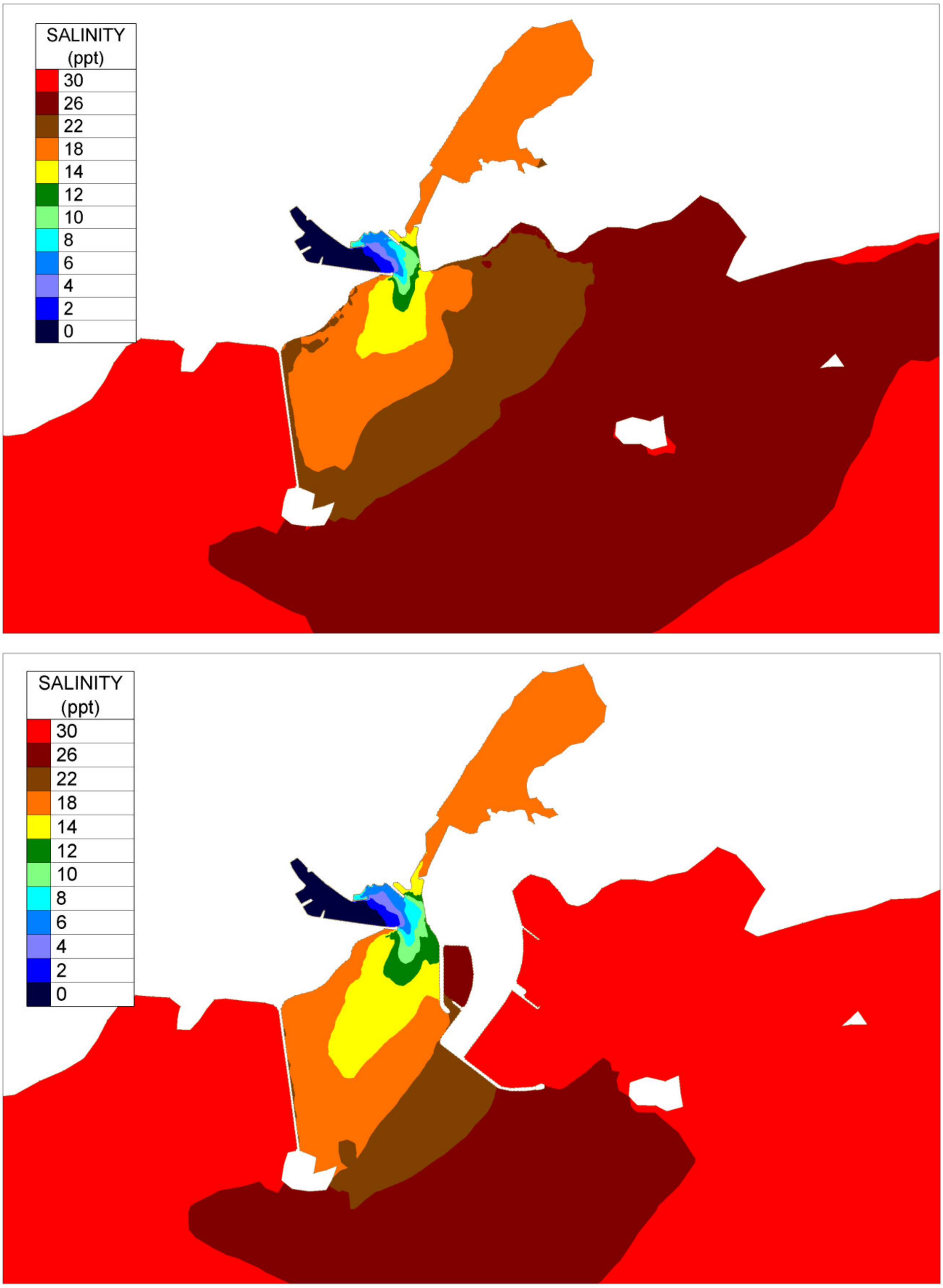


Figure 8.4.81 Mean Salinity concentration in surface layer for existing and proposed cases under 90-percentile Corrib Flow (28.5cumec)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	30.76	25.73	27.45	32.37	31.50	32.10	32.85	32.31	32.70
2	29.41	25.83	27.31	32.11	30.24	31.58	32.72	31.25	32.42
3	28.88	23.62	26.36	31.85	29.56	30.91	32.62	31.22	32.18
4	30.37	24.03	26.84	30.60	27.02	29.01	31.74	28.69	30.49
5	28.09	19.37	24.67	30.75	23.45	28.49	32.06	24.22	30.21
6	26.48	15.63	21.22	31.98	26.64	30.69	32.72	32.12	32.49
7	18.13	5.13	11.17	29.69	18.22	26.21	32.03	26.37	30.72
8	24.33	12.71	18.17	27.70	16.24	21.10	30.41	17.62	22.76
9	25.03	16.86	20.26	25.14	17.11	20.55	25.27	17.87	20.93
10	23.96	18.03	20.02	24.41	18.24	20.40	24.67	18.61	20.88
11	23.82	18.40	20.15	23.97	18.42	20.34	24.16	18.43	20.51
12	22.24	19.02	20.23	22.27	19.04	20.33	22.75	19.06	20.43

Table 8.4.7 Salinity Concentrations for neap to spring tides under 90-percentile flow in Corrib – Existing Case (without development)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	29.19	24.68	26.54	32.39	31.30	32.02	32.86	32.14	32.68
2	31.38	27.75	29.70	32.05	30.58	31.58	32.54	31.32	32.09
3	30.91	29.79	30.47	31.90	31.05	31.44	32.27	31.26	31.85
4	31.39	29.97	30.54	31.40	30.84	30.99	31.51	31.03	31.20
5	30.99	30.49	30.79	31.19	30.97	31.07	31.51	31.10	31.27
6	25.34	14.74	20.16	31.33	23.54	29.26	32.67	31.27	32.38
7	17.46	3.71	9.84	28.66	14.32	24.58	31.67	22.53	29.90
8	23.39	11.32	16.93	26.64	14.66	19.91	29.66	16.29	21.64
9	23.92	15.56	19.07	24.04	15.82	19.37	24.17	16.61	19.79
10	22.78	16.79	18.82	23.29	16.99	19.21	23.57	17.37	19.71
11	22.64	17.15	18.93	22.79	17.17	19.13	23.03	17.18	19.31
12	21.10	17.79	19.02	21.11	17.81	19.11	21.60	17.83	19.22

Table 8.4.8 Salinity Concentrations (ppt) for neap to spring tides under 90-percentile flow in Corrib – Proposed Case (with Harbour Extension)

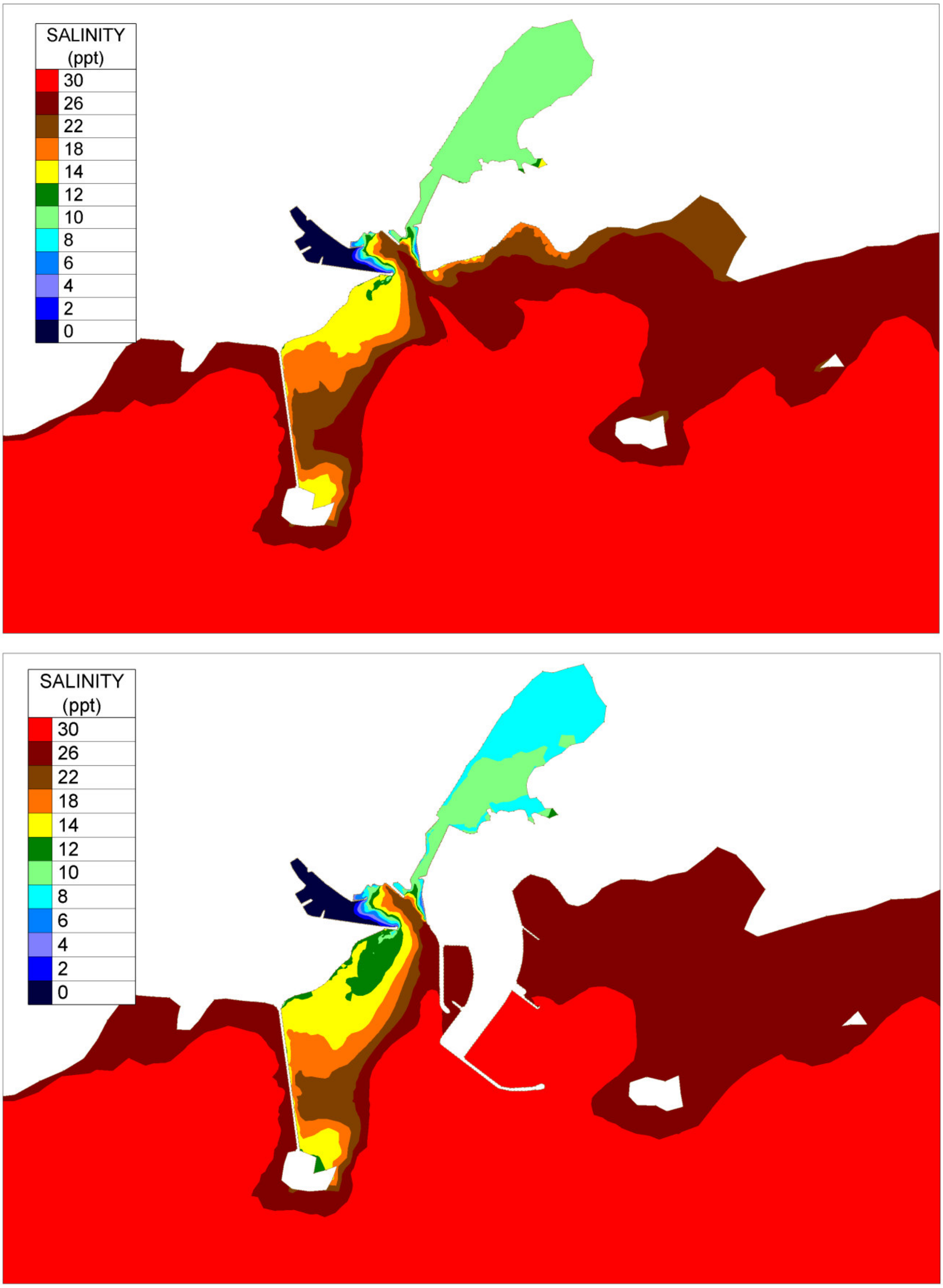


Figure 8.4.82 Mean Salinity concentration in bottom layer for existing and proposed cases under 50-percentile Corrib Flow (82cumec)

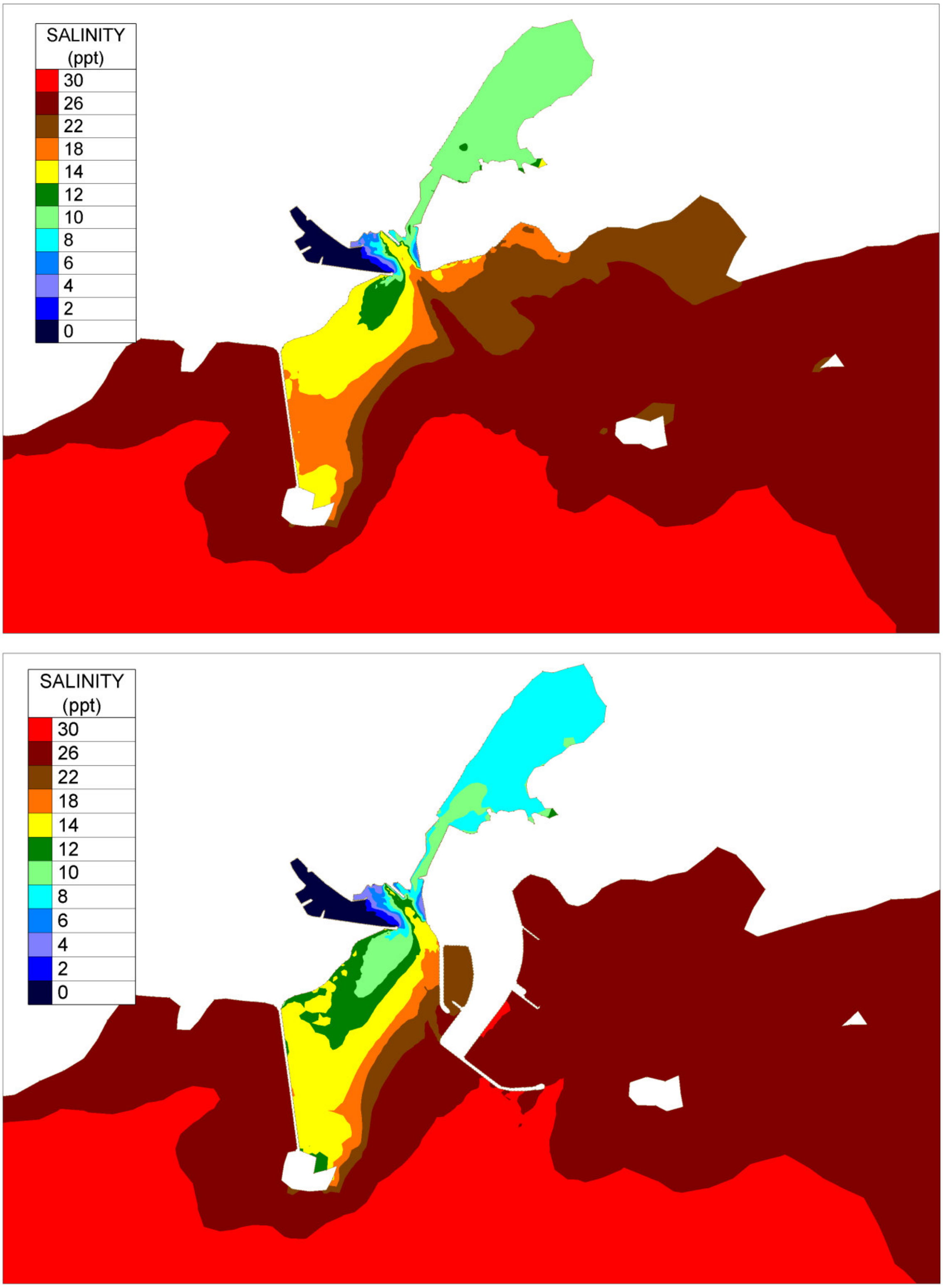
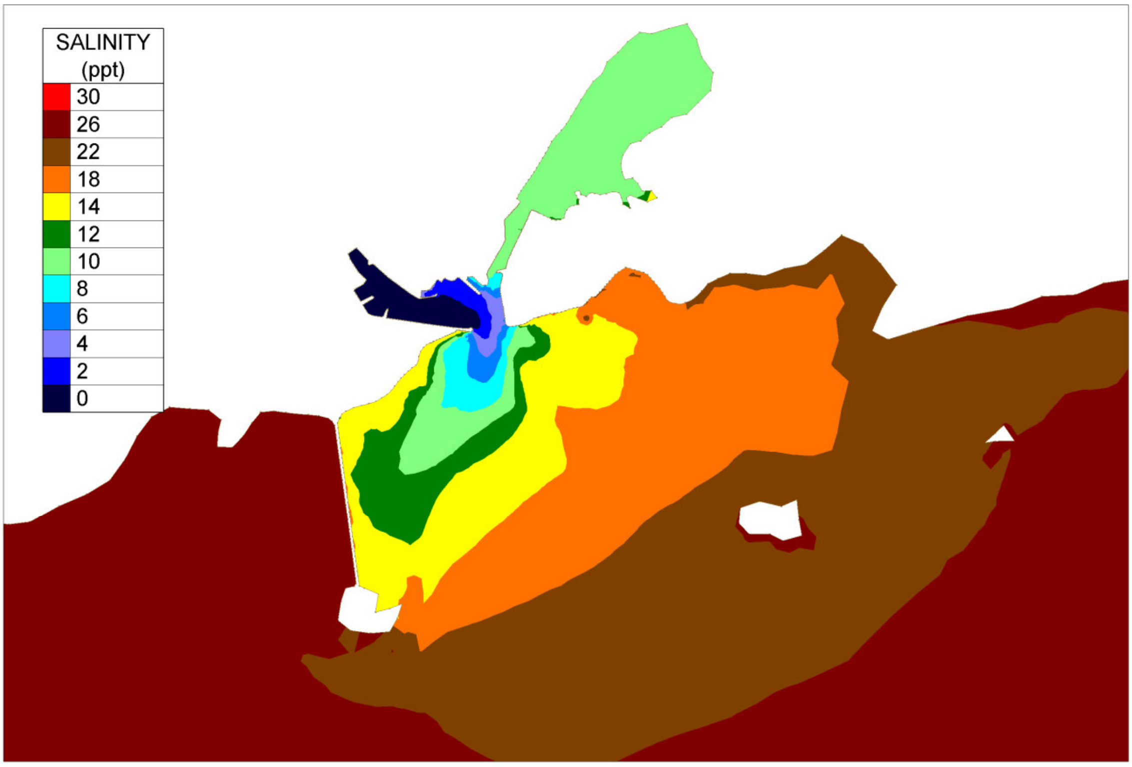


Figure 8.4.83 Mean Salinity concentration in mid-depth layer for existing and proposed cases under 50-percentile Corrib Flow (82 cumec)



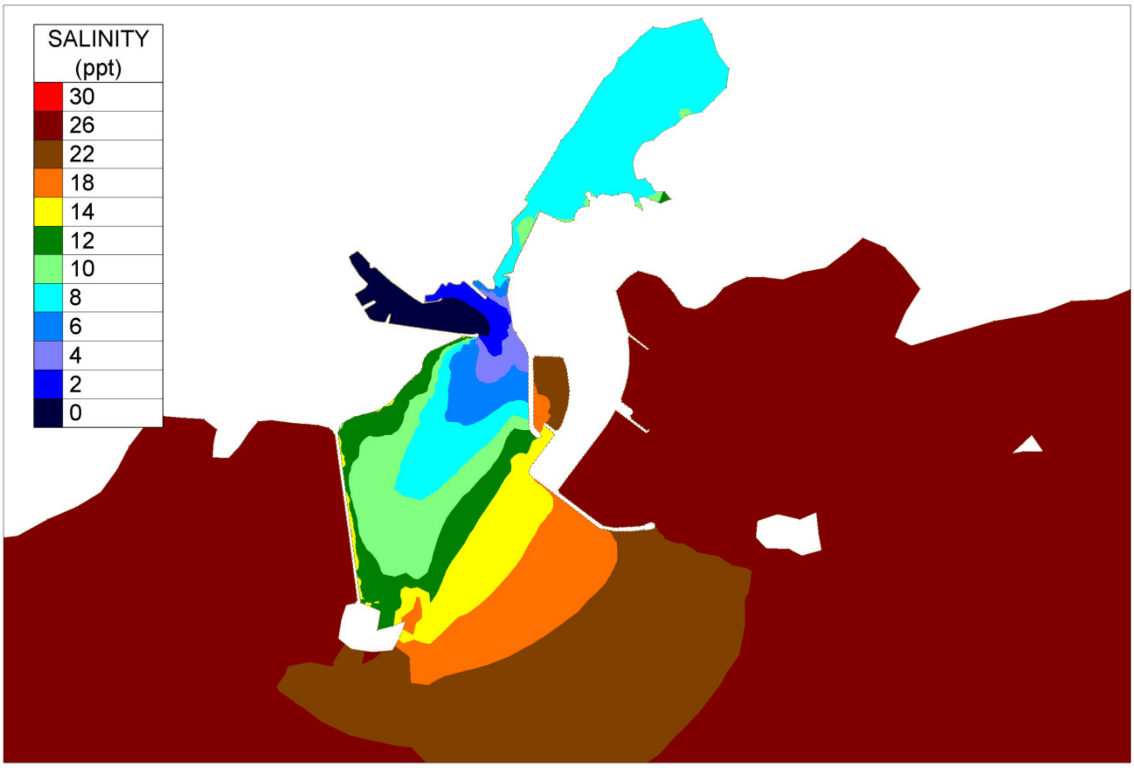


Figure 8.4.84 Mean Salinity concentration in surface layer for existing and proposed cases under 50-percentile Corrib Flow (82cumec)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	27.46	19.80	22.33	31.13	29.70	30.62	31.89	31.14	31.67
2	25.07	19.68	22.25	30.62	27.29	29.54	31.62	29.35	31.17
3	24.40	16.00	20.82	30.20	25.23	28.28	31.39	28.77	30.70
4	24.80	16.92	21.42	27.99	20.65	24.72	30.29	22.32	27.47
5	23.29	10.97	18.43	28.34	15.31	23.99	30.63	16.39	27.02
6	21.07	8.05	14.40	30.28	20.29	27.39	31.70	30.72	31.35
7	10.51	0.99	4.16	25.15	4.14	17.79	30.41	15.58	27.27
8	16.95	4.29	9.42	21.28	6.55	11.95	26.78	7.95	14.34
9	18.02	7.28	11.19	18.19	7.49	11.58	18.40	8.23	12.17
10	16.39	8.41	10.83	17.21	8.64	11.37	17.66	9.03	12.04
11	15.80	8.79	10.92	16.11	8.83	11.19	16.66	8.84	11.51
12	13.90	9.48	11.04	14.02	9.51	11.18	14.88	9.54	11.35

Table 8.4.9 Salinity Concentrations for neap to spring tides under 50-percentile flow in Corrib – Existing Case (without development)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	25.10	18.13	21.06	31.14	29.17	30.43	31.90	30.85	31.62
2	29.05	24.00	26.22	30.57	28.06	29.54	31.35	29.34	30.55
3	28.54	26.45	27.65	30.00	28.77	29.29	30.78	29.26	30.11
4	29.36	26.83	27.84	29.38	28.29	28.58	29.81	28.70	29.06
5	28.65	27.62	28.16	29.05	28.48	28.70	29.77	28.79	29.15
6	19.32	7.53	13.32	29.27	16.45	24.65	31.63	28.75	30.95
7	9.56	0.17	3.13	23.82	0.83	14.39	29.86	1.15	24.30
8	15.68	3.16	8.07	19.60	4.95	10.42	25.81	6.37	12.74
9	16.39	5.79	9.68	16.56	6.01	10.08	16.78	6.74	10.69
10	14.75	6.92	9.32	15.59	7.15	9.85	16.04	7.55	10.53
11	14.18	7.31	9.40	14.48	7.35	9.68	15.02	7.35	9.99
12	12.31	8.02	9.52	12.41	8.05	9.66	13.25	8.08	9.82

Table 8.4.10 Salinity Concentrations (ppt) for neap to spring tides under 50-percentile flow in Corrib – Proposed Case (with Harbour Extension)

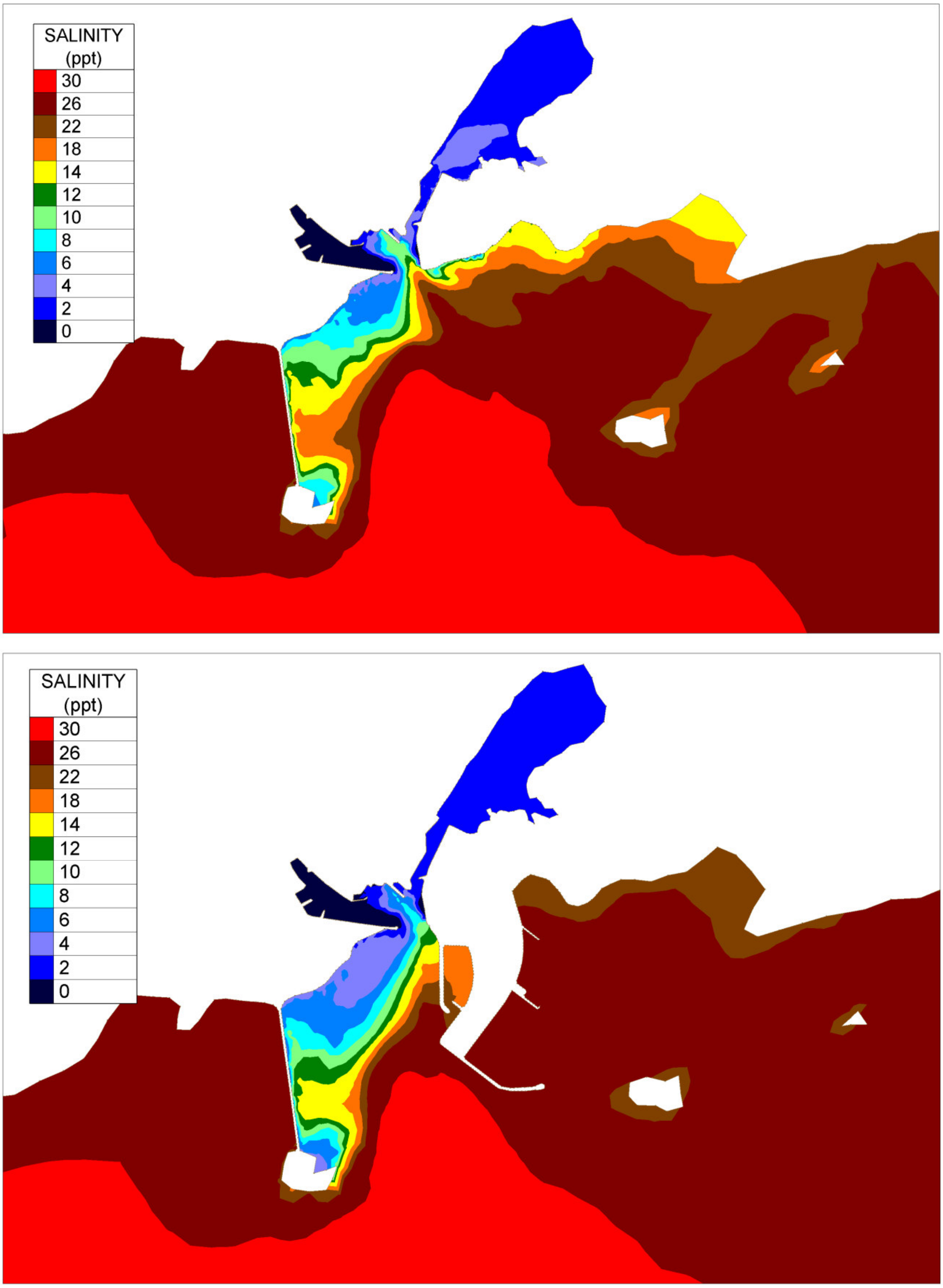


Figure 8.4.85 Mean Salinity concentration in bottom layer for existing and proposed cases under 10-percentile Corrib Flow (200cumec)

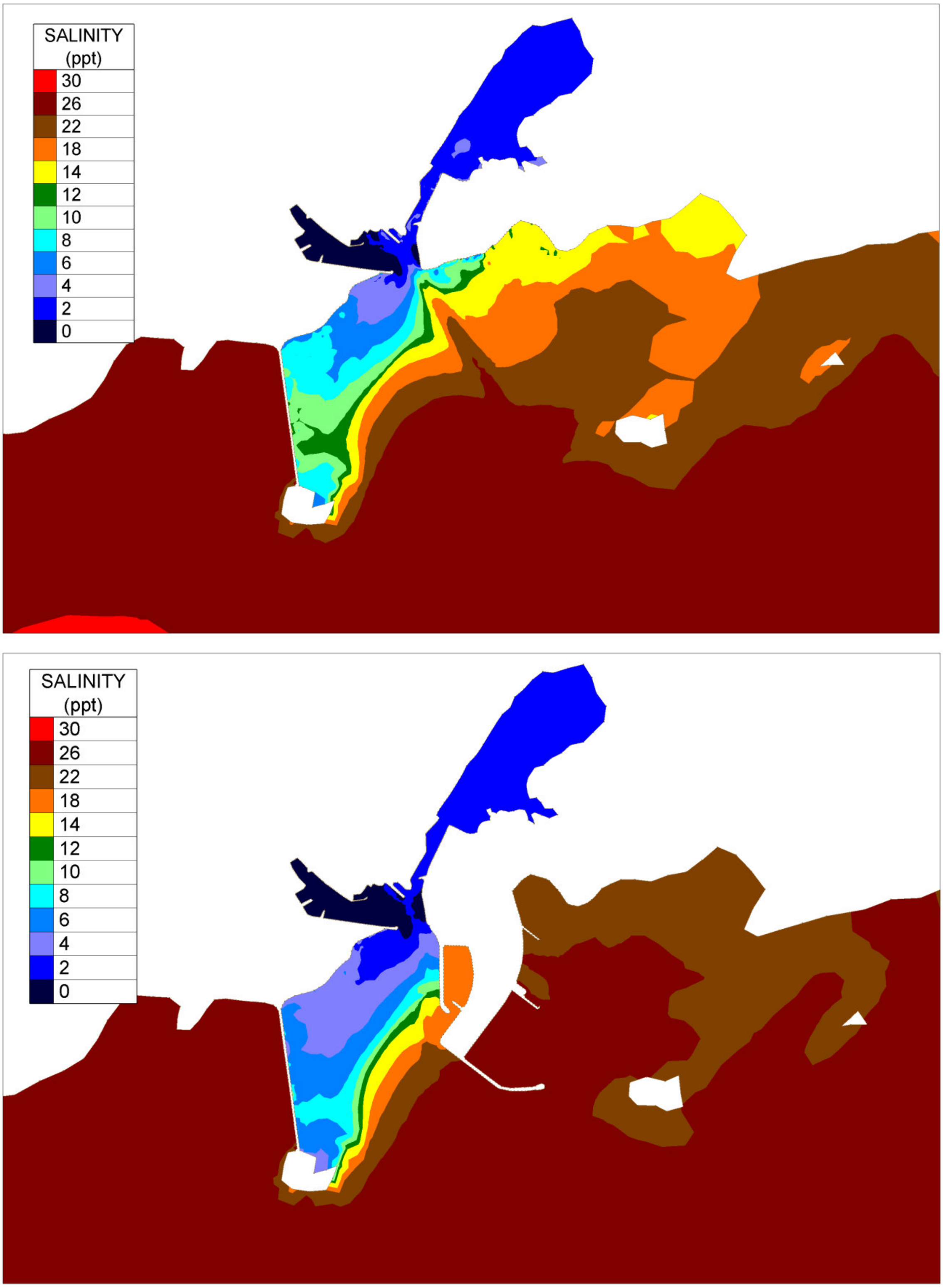


Figure 8.4.86 Mean Salinity concentration in mid-depth layer for existing and proposed cases under 10-percentile Corrib Flow (200cumec)

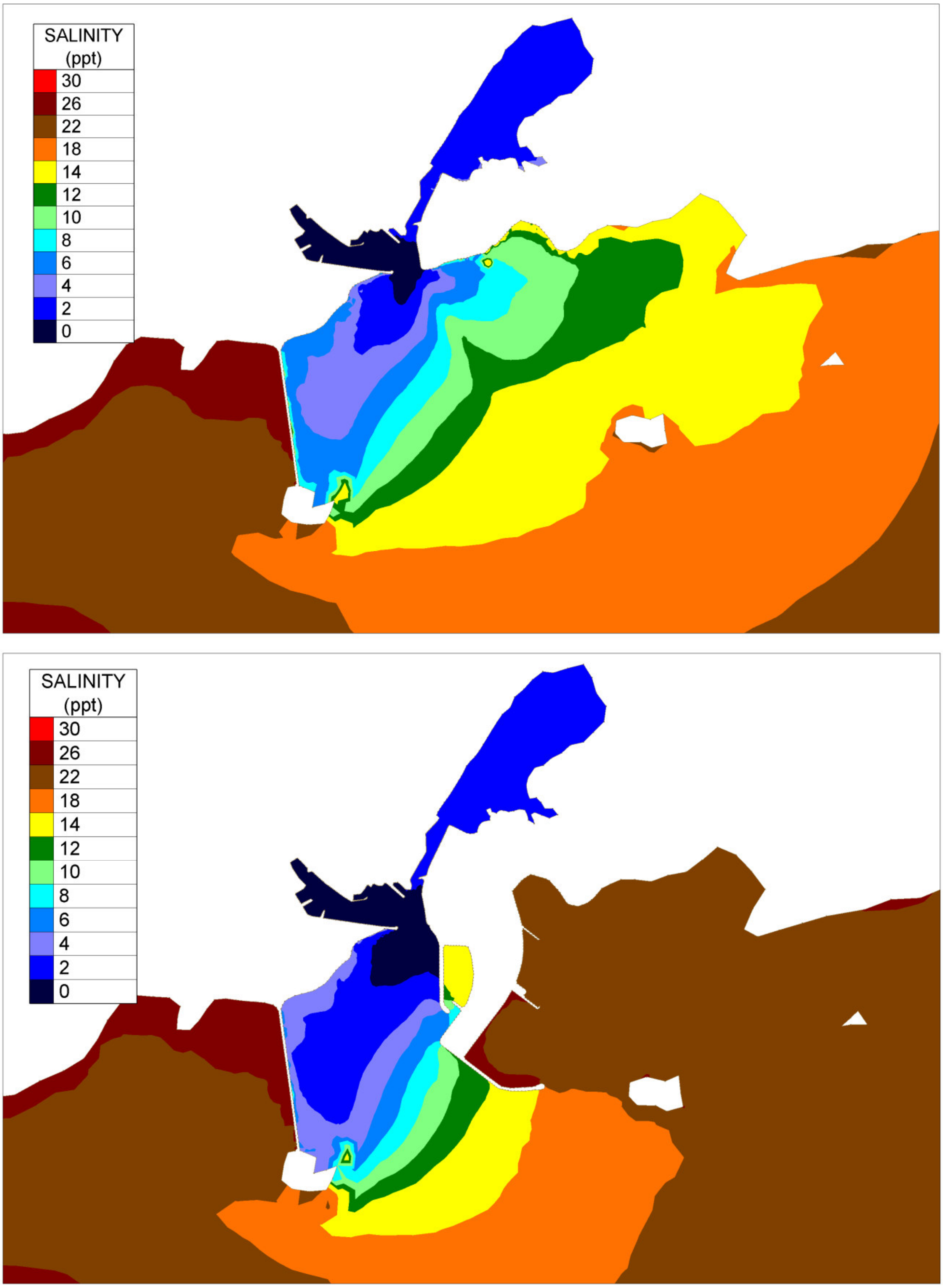


Figure 8.4.87 Mean Salinity concentration in surface layer for existing and proposed cases under 10-percentile Corrib Flow (200cumec)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	21.66	12.44	15.62	29.64	27.37	28.91	30.91	29.76	30.56
2	20.06	10.82	16.26	28.59	22.61	26.71	30.61	27.52	29.82
3	17.98	6.36	13.28	27.88	16.61	23.92	30.12	24.37	28.96
4	18.12	6.52	13.77	23.99	10.43	18.28	28.23	13.84	22.72
5	16.99	1.90	10.49	24.32	3.67	16.90	28.69	4.50	21.61
6	14.50	2.82	8.04	27.41	11.86	21.06	30.65	28.21	30.12
7	3.51	0.00	0.81	15.31	0.02	4.44	27.66	0.05	14.34
8	7.06	0.66	3.09	10.43	1.13	3.89	18.85	1.98	5.34
9	8.00	1.62	3.68	8.13	1.70	3.89	8.31	1.99	4.18
10	6.95	2.27	3.50	7.56	2.38	3.76	7.89	2.62	4.13
11	6.35	2.50	3.56	6.63	2.50	3.68	7.07	2.50	3.78
12	4.97	2.88	3.61	4.99	2.89	3.66	5.46	2.90	3.72

Table 8.4.11 Salinity Concentrations for neap to spring tides under 10-percentile flow in Corrib – Existing Case (without development)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	19.46	10.38	14.31	29.68	26.14	28.39	30.89	29.61	30.50
2	25.62	18.40	21.57	28.61	24.78	26.83	30.09	26.72	28.71
3	25.45	22.11	23.88	27.47	25.73	26.44	29.12	26.64	27.96
4	26.51	22.89	24.30	26.63	24.92	25.37	27.52	25.54	26.24
5	25.65	23.75	24.71	26.35	25.21	25.58	27.63	25.67	26.38
6	12.08	0.97	6.68	25.54	5.14	16.59	30.55	20.17	28.73
7	2.74	0.00	0.44	12.30	0.00	2.35	25.90	0.01	9.12
8	5.82	0.09	2.09	8.34	0.25	2.63	16.41	0.74	3.71
9	6.36	0.61	2.42	6.46	0.67	2.63	6.56	0.83	2.92
10	5.42	1.08	2.24	5.97	1.17	2.48	6.30	1.41	2.84
11	4.89	1.28	2.30	5.10	1.29	2.40	5.46	1.29	2.48
12	3.55	1.69	2.35	3.55	1.71	2.39	3.90	1.71	2.44

Table 8.4.12 Salinity Concentrations (ppt) for neap to spring tides under 10-percentile flow in Corrib – Proposed Case (with Harbour Extension)

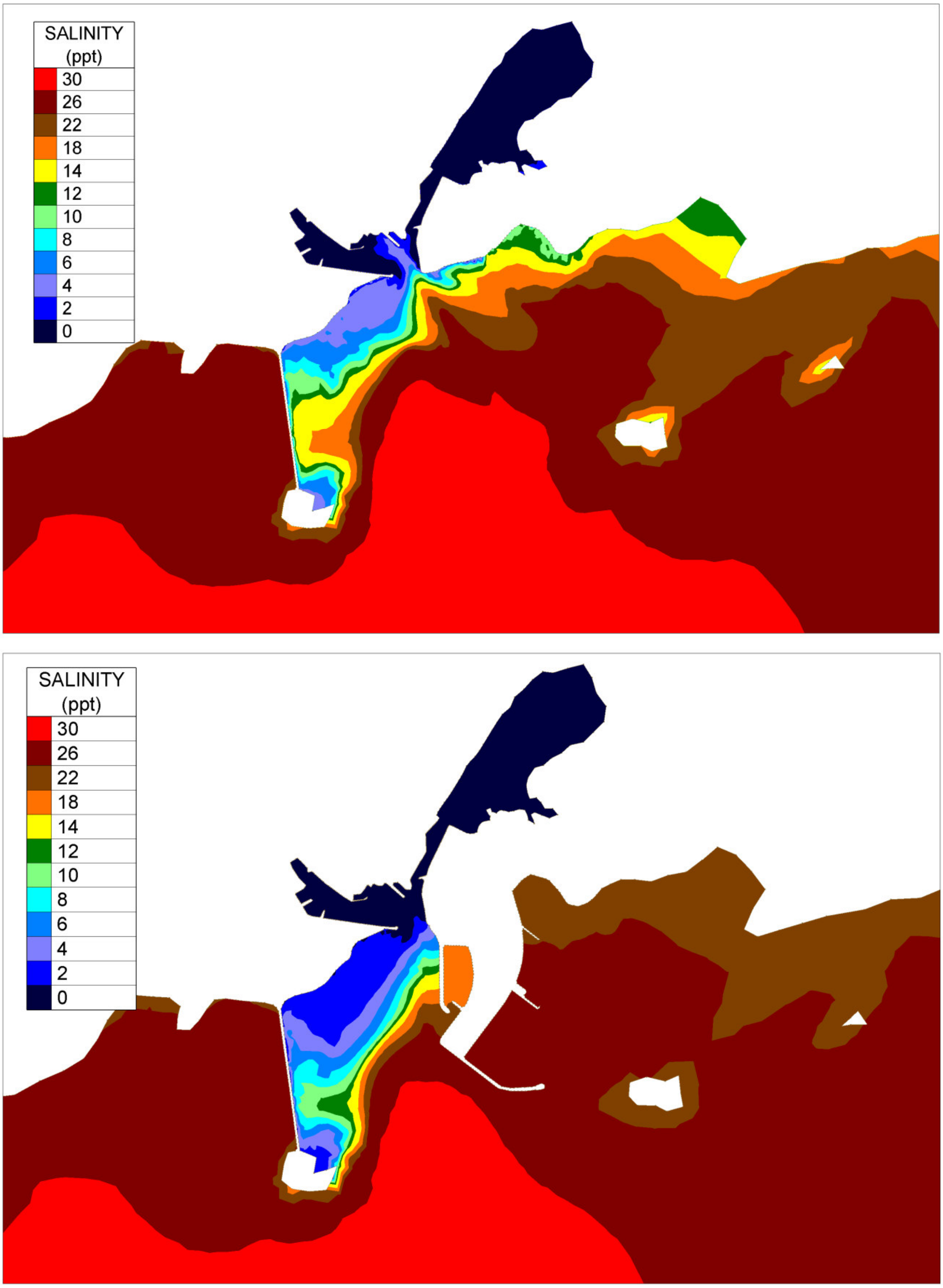


Figure 8.4.88 Mean Salinity concentration in bottom layer for existing and proposed cases under 1-percentile Corrib Flood Flow (272cumec)

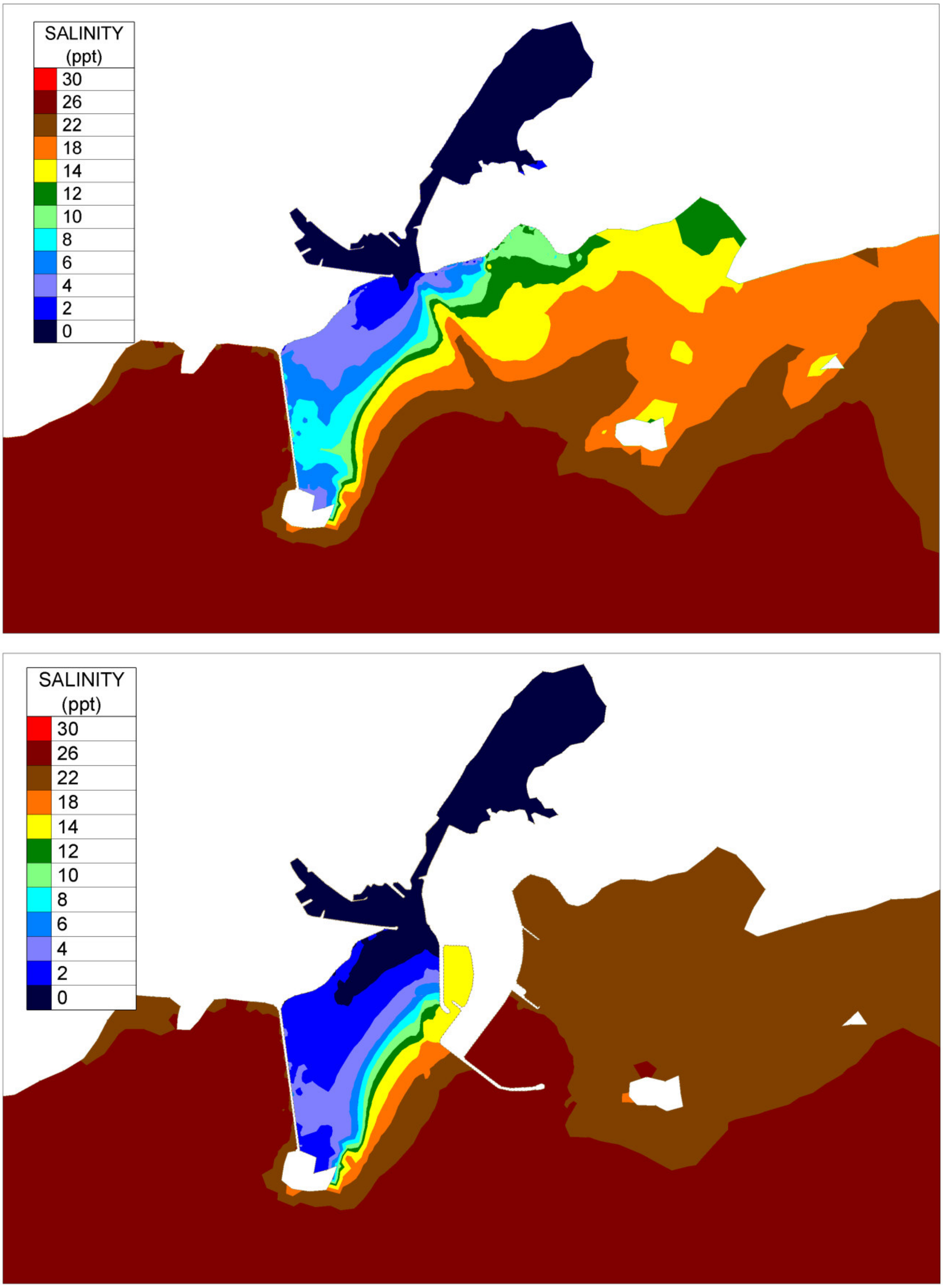
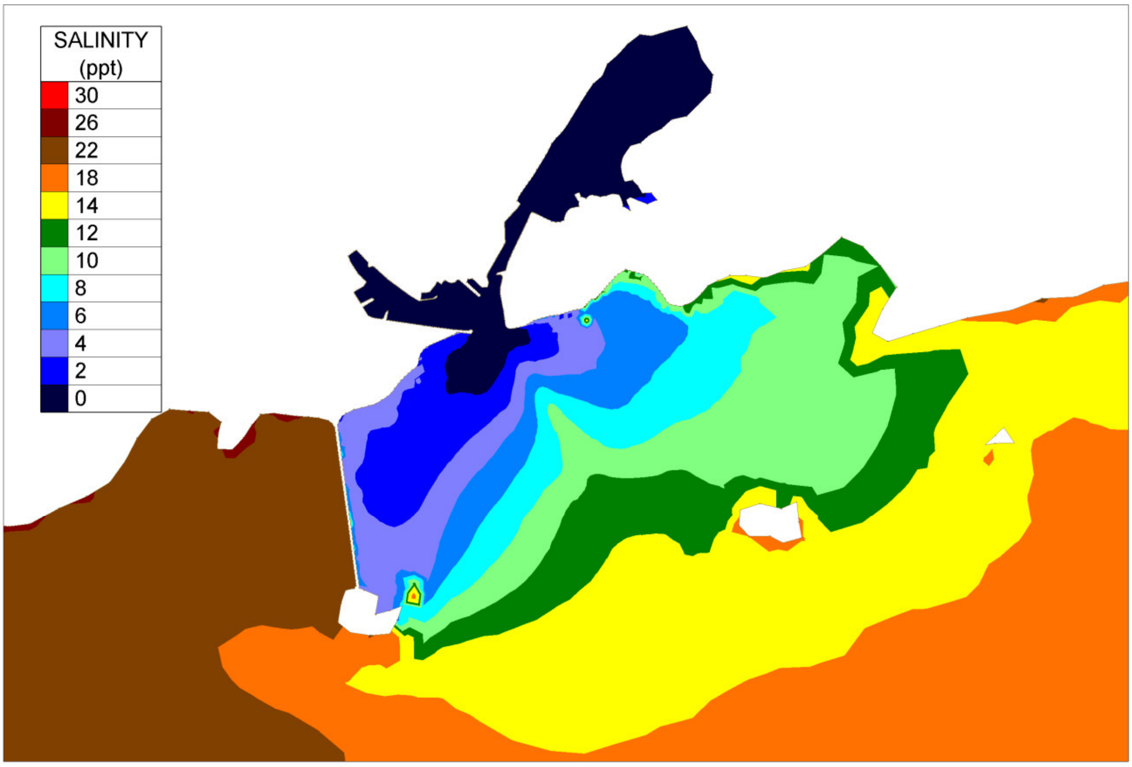


Figure 8.4.89 Mean Salinity concentration in mid-depth layer for existing and proposed cases under 1-percentile Corrib Flood Flow (272cumec)



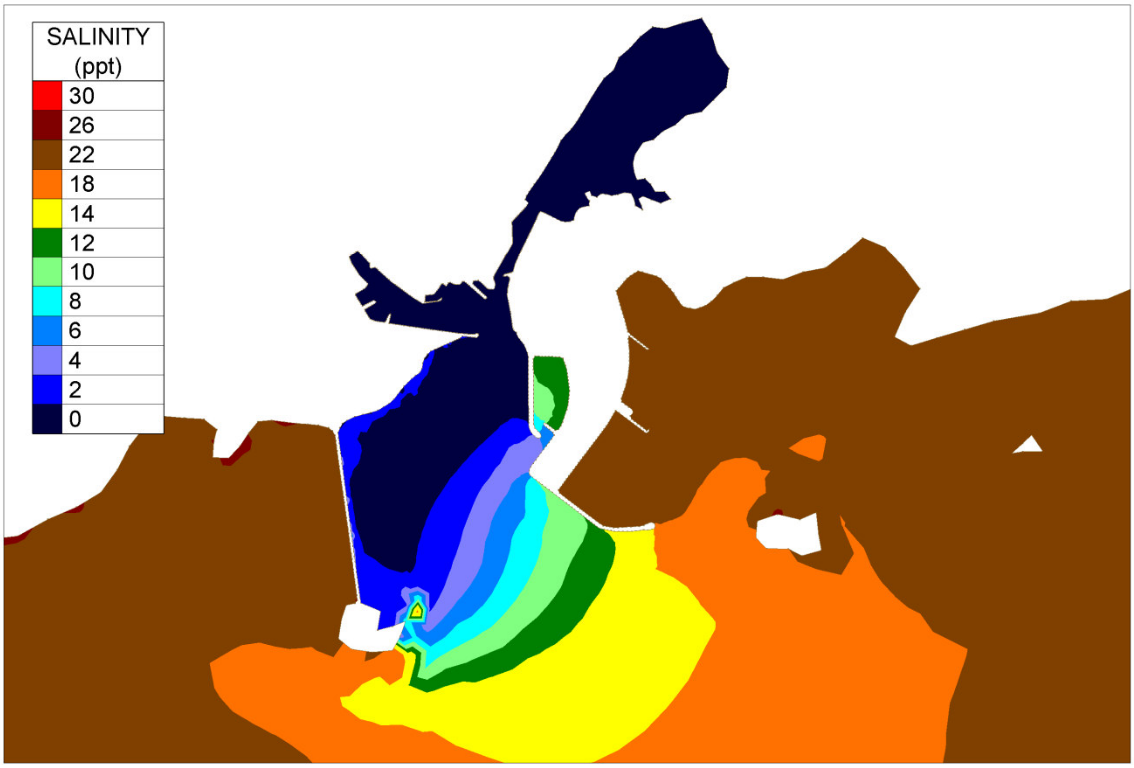


Figure 8.4.90 Mean Salinity concentration in surface layer for existing and proposed cases under 1-percentile Corrib Flood Flow (272cumec)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	17.80	10.43	13.47	29.34	26.69	28.56	30.91	29.65	30.50
2	17.78	7.35	13.51	28.01	20.42	25.83	30.47	27.67	29.74
3	15.60	3.38	10.09	26.88	12.51	21.48	30.12	22.59	28.78
4	15.74	4.41	10.25	22.31	6.72	15.08	27.32	9.93	20.37
5	14.61	1.75	7.17	22.27	3.23	13.24	27.88	3.92	18.74
6	12.25	1.57	6.51	25.87	8.28	18.61	30.63	26.01	29.87
7	1.97	0.00	0.26	9.52	0.00	1.53	25.82	0.00	7.36
8	4.48	0.04	1.49	6.53	0.08	1.78	13.60	0.27	2.45
9	5.09	0.31	1.63	5.20	0.33	1.80	5.34	0.38	2.05
10	4.27	0.61	1.48	4.74	0.65	1.68	5.06	0.81	1.98
11	3.82	0.72	1.52	4.00	0.73	1.60	4.41	0.73	1.66
12	2.56	1.04	1.56	2.56	1.04	1.59	2.86	1.04	1.63

Table 8.4.13 Salinity Concentrations for neap to spring tides under 1-percentile flood flow in Corrib – Existing Case (without development)

Reference	Surface Layer (5)			Mid-depth Layer (3)			Bottom Layer (1)		
Sites	max	min	mean	max	min	mean	max	min	mean
1	16.63	8.25	11.97	29.34	24.72	27.71	30.89	29.31	30.43
2	24.42	16.68	19.87	27.72	23.54	25.94	29.91	25.78	28.25
3	24.32	20.72	22.56	26.53	24.70	25.49	28.86	25.73	27.35
4	25.60	21.62	23.10	25.63	23.78	24.25	26.69	24.44	25.27
5	24.53	22.44	23.52	25.43	24.13	24.52	26.85	24.60	25.48
6	9.59	0.11	4.73	23.12	0.46	12.99	30.38	3.12	26.94
7	1.25	0.00	0.09	6.32	0.00	0.51	22.52	0.00	3.03
8	3.24	0.00	0.64	4.44	0.00	0.78	9.91	0.00	1.09
9	3.54	0.02	0.66	3.63	0.02	0.78	3.72	0.02	0.94
10	2.62	0.08	0.57	3.11	0.08	0.68	3.43	0.11	0.87
11	2.29	0.14	0.60	2.41	0.14	0.63	2.74	0.14	0.65
12	1.22	0.29	0.62	1.21	0.29	0.63	1.31	0.29	0.65

Table 8.4.14 Salinity Concentrations (ppt) for neap to spring tides under 1-percentile flood flow in Corrib – Proposed Case (with Harbour Extension)

8.4.3.6 Salinity in Lough Atalia and Renmore Lough

The following discussion is a summary of the approach and findings on the separate studies on Lough Atalia and Renmore Lough. The full text of the reports are included in Appendices 8.1 and 8.2.

8.4.3.6.1 Hydrology of Lough Atalia

General Description

Lough Atalia is a tidal Lough of some 39ha in area, located to the northeast of Galway Docks in Galway City. The Lough is connected to the sea via a 430m long inlet channel which has a railway bridge crossing at its north end, the Galway Harbour Enterprise Park road bridge crossing towards its southern seaward end and a low stone boulder weir located across a wide section of the channel towards the north end. The surrounding catchment area to the Lough is of the order of 2.2km^2 and is an urbanised catchment with approximately 30 to 40% paved area.

The bedrock geology of the catchment and the majority of the Lough is a Visean pure bedded limestone, which is classified as regionally important karstic (conduit flow) bedrock aquifer. The southern end of the Lough near the Railway Bridge is classified as a Metagabbro and Orthogneiss bedrock which is a metamorphic rock derived from igneous rock. This represents a hard and impervious rock formation whereas the Visean Limestone is softer and prone to weathering and solution. The bedrock underlying the Docks and the proposed Harbour Extension area is also shown to be Metagabbro and Orthogneiss bedrock.

The Bathymetry of Lough Atalia reveals generally a shallow bay except for a deep pocket towards the southern end of the Lough inside the inlet channel. This deep pocket coincides with the interface between the igneous and limestone bedrock formations, with the softer limestone bedrock being eroded over time by the locally high velocities inflowing to the Lough and the igneous rock being much more resistant to erosion. There is only one identified spring feature referred to on the older OSI maps and which is marked on-site by a white Cross as St. Augustine's Well with no other springs being identified either on the OSI or GSI karst database in the vicinity of Lough Atalia.

The salinity in Lough Atalia has been shown earlier to vary significantly with the tidal range and the River Corrib flow rate. Recorded salinities within the lough varied from 1 up to 29 psu over a range of sampling dates in 2012 and 2013. The lough is relatively shallow and is practically completely flushed in a single spring tide. The incoming spring tide initially pushes freshwater into the lough and then as the tide rises sufficiently a more saline wedge is introduced. On neap tides the tidal range (< 0.4m) is insufficient to draw the deeper saline wedge into the lough and consequently the water entering primarily comprises Corrib freshwater from the buoyant surface layer. This affect significantly lowers the salinity within the Lough during the neap tide period. As the tidal cycle proceeds from neap to spring tides more saline conditions are returned to the Lough by the deeper saline flows. The magnitude of the Corrib freshwater flow has a significant effect on salinity levels within Lough Atalia being the principal source of freshwater inflow to this tidal Lough.

Because of the large attenuating effect of Lough Corrib and the control of flows and water levels in the Corrib by the OPW at the Salmon Weir sluice facilities, the magnitude of the flow rate discharging to the estuary is a gradually varying discharge with the majority of storm fluctuations dampened out by the lake control (being retained as lake storage for slower release).

Hydrodynamics of Lough Atalia

Hydrometric measurements carried out in Lough Atalia in January and March 2013 indicate low water levels in the Lough of 0.5 to 0.6m O.D. and highwater levels of 2.3 to 2.4m OD Malin on spring tides. This tidal range is significantly lower than the tidal range measured outside the Lough at the Galway Docks tidal gauge which registers a spring tide low water level of -2.5m O.D. and highwater level of 2.5m O.D.

On neap tides the tidal range in Lough Atalia was found to be very weak and practically non-existent at 0.2 to 0.3m range (low water between 0.3 and 0.4m and high water 0.5 to 0.6m OD). The neap tidal range registered for this period at Galway Docks had a low water level of -0.6m and high water level of 1.2m O.D.

The tidal inflow period on spring and neap tides was found to be approximately 2 to 2.5hours and the outflow period from the Lough being a slow release for nearly 10hours. At a monitoring location just inside the Lough the measured ADCP (Acoustic Doppler Current Profiler) current profiles showed a strong pulse of inflow to Lough Atalia on spring tides and reducing to little or no appreciable pulse on the neap tides.

<u>Tidal Exchange</u>
Lough Atalia from inside the Railway Bridge is approximately 39ha in Area. The entrance channel to Lough Atalia dictates the tidal range and tidal flows entering the Lough. This channel acts as a low profile weir maintaining a typically a low water level within the Lough of 0.3 to 0.6m O.D. Malin for neap and spring tides respectively. The channel width varies typically from 45 to 70m with the narrowest point at the Road culvert having an opening width of c. 35m. A stone boulder weir located approximately 100m downstream of the Railway Bridge crosses approximately 75% of the channel width with a top elevation of 0.8 to 1m O.D.

On a spring tide the surface area of Lough Atalia is typically 39ha at high water and 25ha at Low water. The volume of the Lough at highwater is estimated to be 771,200 m^3 (water level of 2.4m O.D.) and 197,500 m^3 at Low water (water level of 0.6m OD). This represents an exchange volume of 573,700m^3 over a tidal cycle (approx 75% of the Lough volume at high water). This exchange volume practically flushes out the entire Lough on a single tide. This tidal exchange represents an average inflow rate over the 2.5hr inflow period of 64m^3/sec (13m^3/sec averaged over the full 12.4hr tidal cycle). The hydraulic residence time within the Lough for a spring tide is 4.2hours which is very short representing excellent flushing characteristics.

On neap tides the surface area of Lough Atalia is typically 25ha at high water and 21ha at Low water. The volume of the Lough at highwater is estimated to be 197,500 m^3 at high water (water level of 0.6m O.D.) and 139,900 m^3 at Low water (water level of 0.35m OD) This represents an exchange volume of 57,600m^3 (approx 30% of the Lough Volume at high water). This tidal exchange represents an average inflow rate over the 2.5hr inflow period of 6.4m^3/sec (a factor of 10 lower than the spring tide rate) or 1.3m^3/sec averaged over the full 12.4hr tidal cycle. The hydraulic residence time within the Lough for a neap tide is about 30hours.

<u>Sources of freshwater inflow</u>
The principal source of freshwater flow to Lough Atalia is from the Corrib which enters Lough Atalia during the relatively short inflow period of 2 to 2.5 hours around high water at Galway Docks. Other potential sources of freshwater inflow are from groundwater and direct storm runoff to the Lough from the surrounding urban catchment via a number of storm outfalls.

Groundwater inflow contribution to Lough Atalia is estimated to be less than 0.1cumec based on an empirical base flow equation from the FSR method (NERC 1975) for a catchment area of 2.2km^2 and annual rainfall amount of 1200mm. This rate is not significant in comparison to the tidal exchange volumes entering lough Atalia. However given the karstic nature of the limestone bedrock larger groundwater inflows cannot be ruled out. A further source of freshwater inflow is from direct storm water runoff from surrounding roads and paved areas. On balance such a contribution will generally be minor given the relatively small contributing catchment area.

<u>Salinity Measurements</u>
Salinity measurements from discrete sampling surveys within Lough Atalia were carried out on 5 dates in 2012 at 21 sampling sites and for a range of depths through the water column. The dates were 4th, 10th, 16th, 24th April and 4th May 2012.

A second series of discrete sampling surveys at 10 sites within Lough Atalia and for a range of depths through the water column was conducted in January 2013. The dates were 11th, 14th, 15th, 18th, 21st, 23rd and 24th of January 2013. Discrete sampling of salinities on the inlet channel to Lough Atalia at the Docks Enterprise Park Road Bridge was carried out for a spring and neap cycle on the 11 and 18th February 2013.

The discrete salinity surveys confirmed that spatially there is not generally significant variation in salinity concentrations with the southern end of the Lough being slightly less saline due to its proximity to the inlet channel. The different sampling dates did reveal significant variation between dates in the salinity concentration with neaps being considerably less saline than spring tide periods. The measurements showed increasing salinity with depth particularly for the deeper southern section of the Lough towards the inlet/outlet channel. In the shallower areas of the Lough the variation in salinity with depth was only slight.

8.4.3.6.2 Hydrology of Renmore Lough

Renmore Lough is a small tidal Lough located to the south of the Railway line and measures 0.88 ha in area with retaining banks at 2.3 to 2.6m. The inflow to Renmore Lough is via a narrow long channel running northeast under the Railway track from Lough Atalia. The ground level to the south separating Galway Bay from Renmore Lough is typically at 3m OD. The Lough is founded on metamorphic rock of igneous origin and drains down to the invert crest level of its inflow/outflow channel with Lough Atalia (1.77m O.D). The Lough is permanently wet but is shallow with water depths varying from 0.8 to 0.3m at low water (low water is typically 1.77m O.D.). A small local freshwater contribution enters the Lough from runoff from the Railway embankment, from Mellows Drive and the Playing Pitches and Par 3 golf course to the east and also direct rainfall from immediate surroundings lands to the west and the lough surface itself.

The saline inflow only occurs for a relatively short period of time when tide levels in Lough Atalia exceed c. 1.75m O.D. and consequently only occurs on spring tides with the inflow period generally less than 2hours. Salinity measurements within the Lough generally reflect a wide range of salinities depending on spring tide salinities in Lough Atalia which in turn are influenced by the magnitude of the Freshwater flow in the River Corrib. The average salinity in Renmore Lough will be slightly higher than Lough Atalia as the salt contribution only occurs during spring tides which generally have a higher salt content than neap tides in Lough Atalia.

The salinity range in Lough Atalia however is greater and is from c. 30 to nil ppt, whereas in Renmore Lough due to the tidal restriction this salinity range is less from c 23 to 2ppt.

8.4.3.6.3 Conclusions on Salinity Studies on Lough Atalia and Renmore Lough

The tide simulations for various freshwater inflows from the Corrib show the deflection of the Corrib freshwater plume westward due to the harbour extension site resulting in reduced dispersion and lower salinities (*i.e.* more fresh) in the upper water column layers off Nimmo's pier (mouth to Lough Atalia and Galway Docks) and west of the Harbour extension site. The impact of this reduced dispersion of the Corrib freshwater plume is to introduce a slightly fresher water into Lough Atalia resulting in a slight lowering of the salinity concentration there. Conversely considerably more saline conditions are predicted east of the Harbour Extension in the Renmore Bay area and north of Hare Island.

Within Lough Atalia the measurement and model study combined show for both existing and proposed cases that the lowest salinities and tidal variation of salinities is when the River Corrib is in flood (maximum flows) and the tide range is at its minimum (*i.e.* neap tides). The measured and modelled data indicate that the salinity within Lough Atalia will tend towards complete freshwater (nil salinity) during the larger flood flows. On neaps tides the tidal range is extremely weak and the water introduced on the inflowing period is from the surface layer and is basically freshwater. The Lough is relatively small and shallow with a high exchange/flushing rate which eliminates any significant build-up / storage of salinity in the Lough that could be used to maintain salinities during neap and Corrib flood flow periods. The high flushing rate of the Lough ensures a dynamic Lough having large temporal variation in salinities over a single tidal cycle, over lunar cycles and seasonally.

The Impact of the Harbour Extension Development on salinity concentrations within Lough Atalia is to reduce salinities by on average by 1.29ppt over the complete range of flow and tide conditions. Given the relative range of salinities within the Lough from c. 30ppt to nil ppt, this reduction of 1.29ppt in salinity, which is only 10% of the mean salinity, is not considered significant. The model analysis also demonstrates that the range of salinities (maximum to minimum) within Lough Atalia will not alter as a result of the harbour extension, only the frequency of occurrence will change.

Periodic large and extreme flood flows in the Corrib will reduce salinities to practically nil in Lough Atalia for both the existing and proposed cases, principally during neap tides but also on spring tides for a less frequent more extreme flood flow. Over the full tidal range the probability of nil Salinity in a given year occurring within Lough Atalia will increase from 0.08% to 0.21% (7 to 18hours in an average year).

The overall impact on salinity within Renmore Lough by the proposed Harbour extension will be to decrease the median salinity within the Lough by 1.22ppt. The overall water balance and inflows to and from Renmore Lough will not be affected by the proposed development as the tidal elevations in Lough Atalia will not be altered by the development and thus the inflow rates to Renmore Lough will remain unchanged.

8.4.4 Outfall Dispersion Simulations

8.4.4.1 Introduction

The potential impact on transport and dispersion of the Existing Mutton Island outfall and the proposed Galway East outfalls was examined using the TELEMAC2D Hydrodynamic model for the existing and proposed development cases.

A continuous discharge of conservative tracer was modelled simultaneously at the two outfall locations under mean Spring and Neap tidal cycles with a median (50-percentile) River Corrib Flow of 82 m^3/s specified. The Effluent Flows simulated were the projected future mean flows of 0.667 m^3/s at Mutton Island outfall and 0.488 m^3/s at the proposed Galway East outfall. A concentration of 1000 mg/l was specified for the tracer inflow. For comparison between the with and without harbour extension snapshots of the Tracer plume at the four principal stages of the tide are presented in Figures 8.4.91 to 8.4.98 and 8.4.99 to 8.4.106 for the Spring and Neap tides conditions respectively.

The Mutton Island outfall was specified at grid point 129628, 222729 and the proposed Galway East outfall at 131892, 222010. Note the finite element scheme translates the outfall location simulated to the nearest computational node to the specified coordinates.

8.4.4.2 Discussion

It can be concluded from the results presented in Figures 8.4.91 to 8.4.106 show that the Galway East proposed outfall location at E131892, N222010 will not be impacted by the proposed port development with the predicted tracer plume shape, extent and concentration almost identical between the with and without development cases. Any slight plume variation between the scenarios is due to the differences in model mesh structure (boundary fitted mesh) between the existing case model mesh and the harbour development case model mesh. The different meshes generated can result in nodal points not exactly coinciding and thus slight differences in the outfall location used by the models, as the simulated outfall point takes the nearest nodal point to the specified location.

The outfall dispersion results show for the existing Mutton Island outfall some variation in the plume characteristics to the east of Mutton Island. These changes in plume shape and extent are brought about by local changes in the velocity pattern as a result of the physical harbour extension structure. The overall impact is considered to be local and minor and importantly the simulations show no impact along the Salthill/Silverstrand, South Park and Renmore shoreline areas or upstream at the existing Galway Harbour where amenity and bathing standards are important. The simulations show practically imperceptible impact to the westerly excursion of the tracer plume from the outfall site and no perceptible impact to water quality at the more remote bathing waters of Silver Strand, Barna and Furbo. The simulations also show no impact to the designated shellfishery waters located in the south inner Galway Bay area.

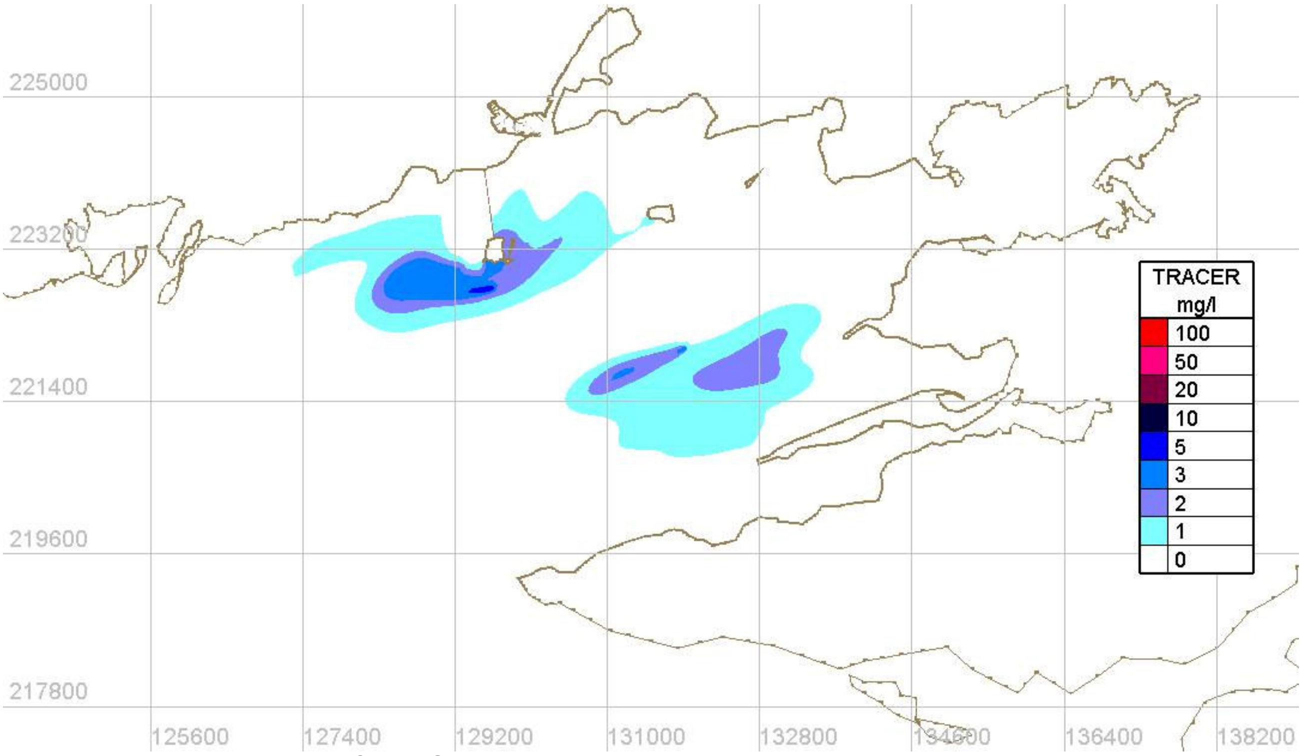


Figure 8.4.91 Mid-ebb Existing Case - Spring tides

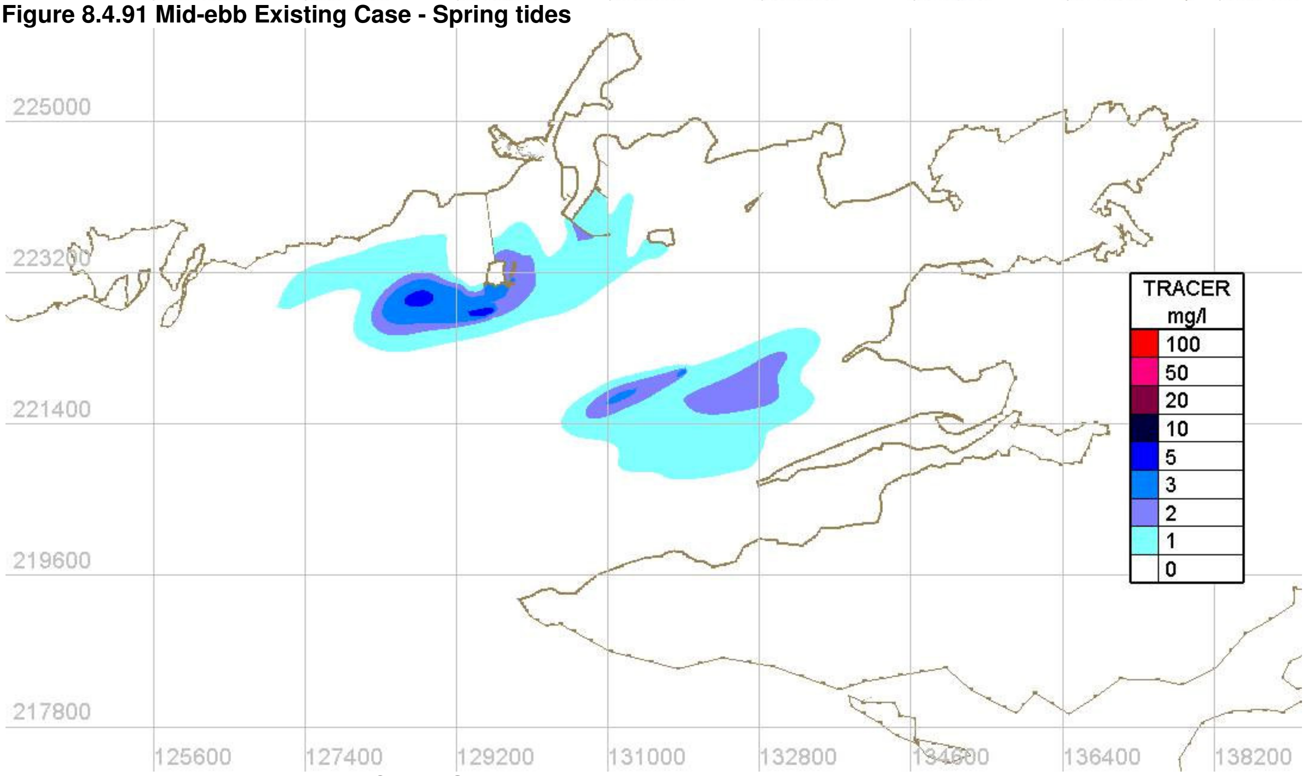


Figure 8.4.92 Mid-ebb Proposed Case - Spring tides

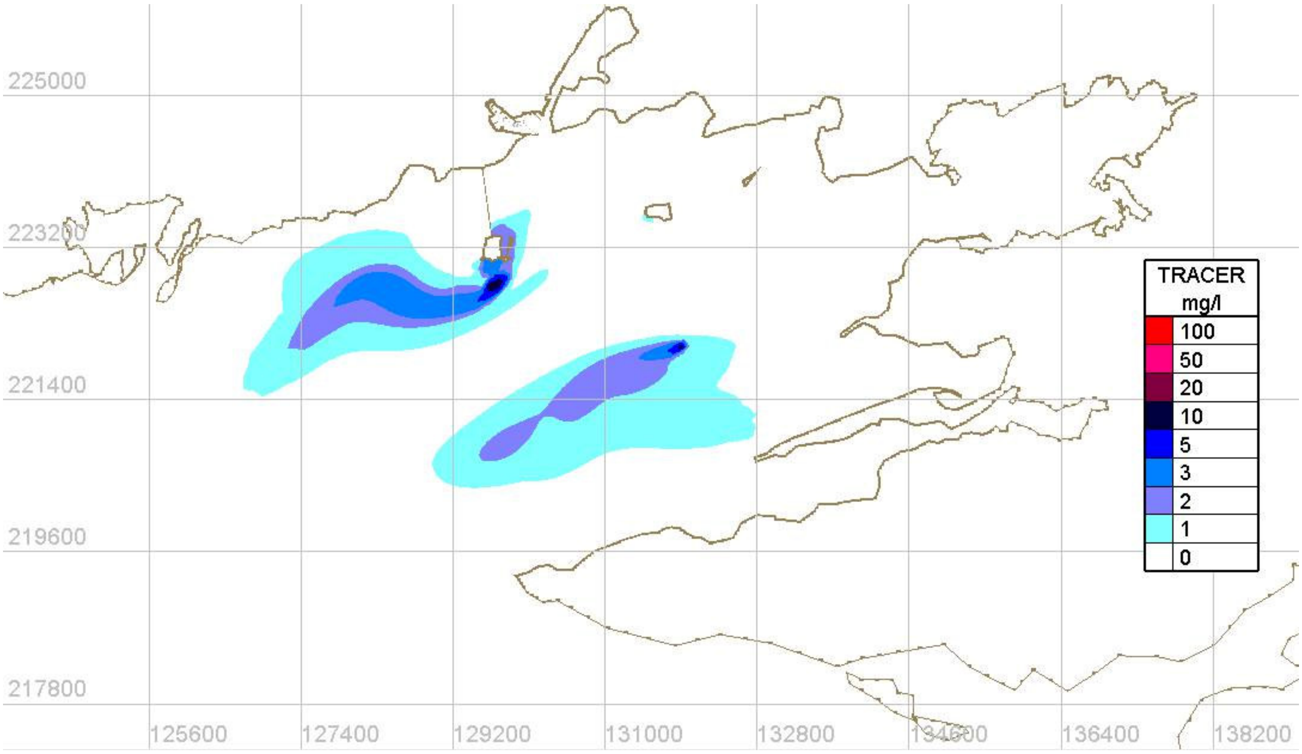


Figure 8.4.93 Low water - Existing Case - Spring tides

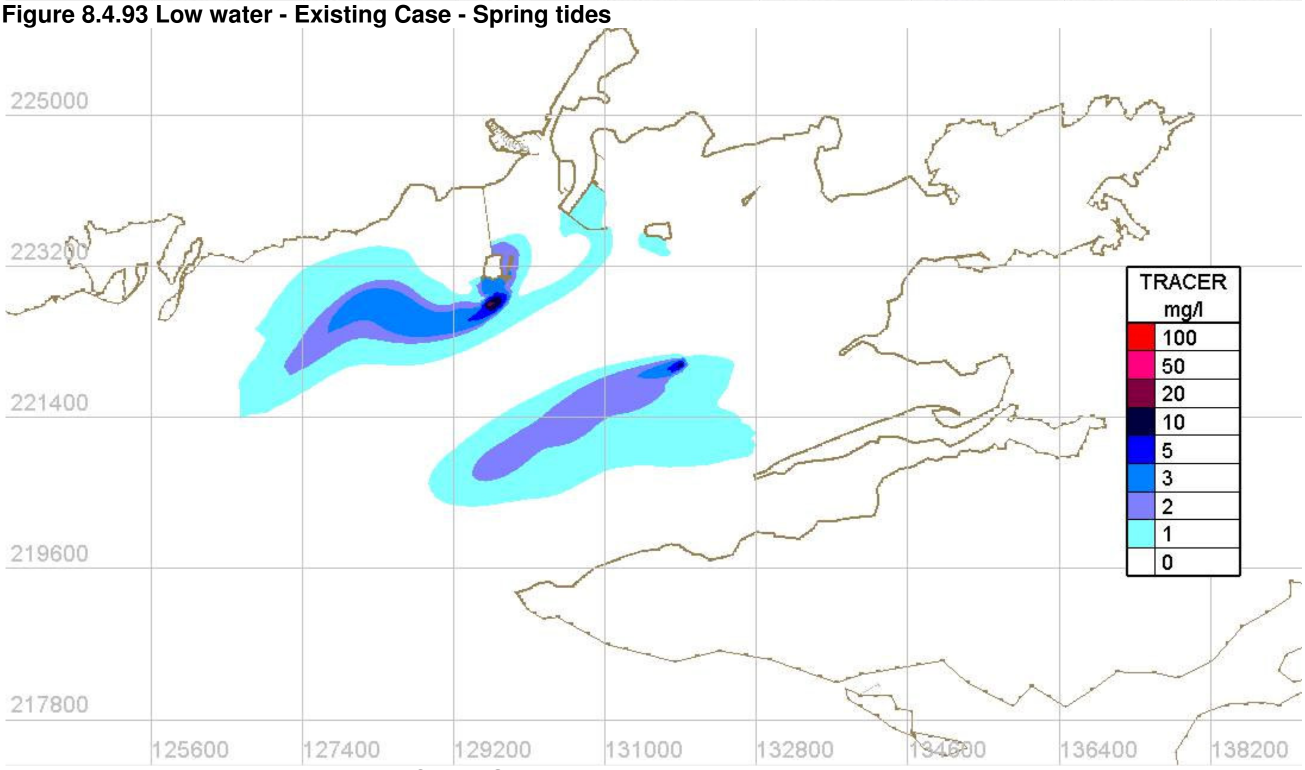


Figure 8.4.94 Low water - Proposed Case - Spring tides

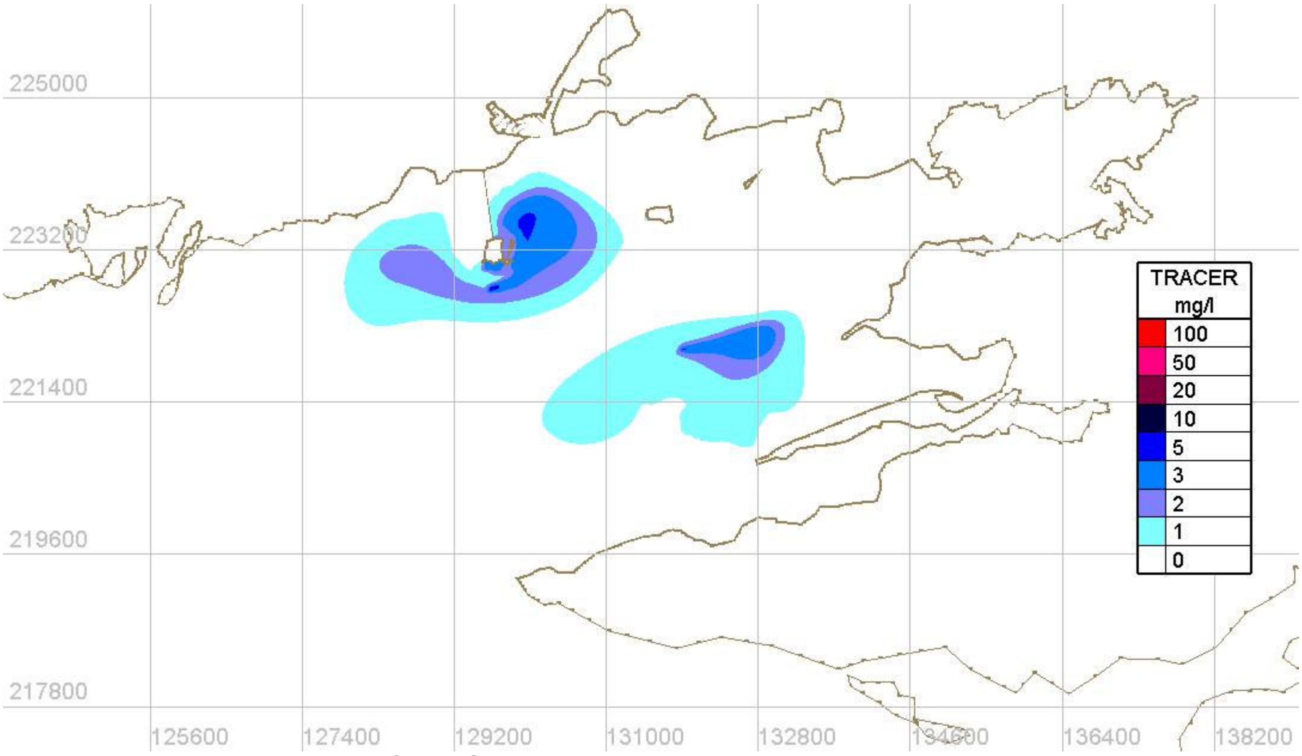


Figure 8.4.95 Mid Flood - Existing Case - Spring tides

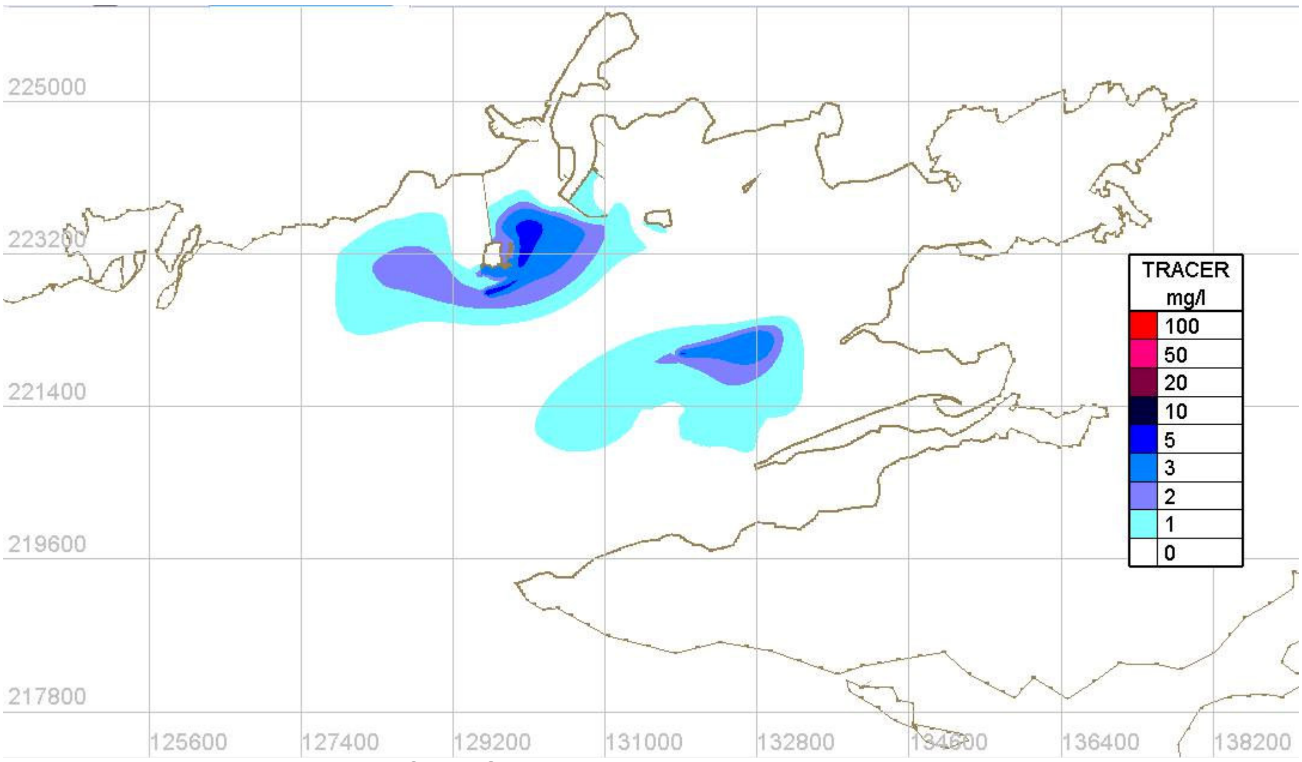


Figure 8.4.96 Mid Flood - Proposed Case - Spring tides

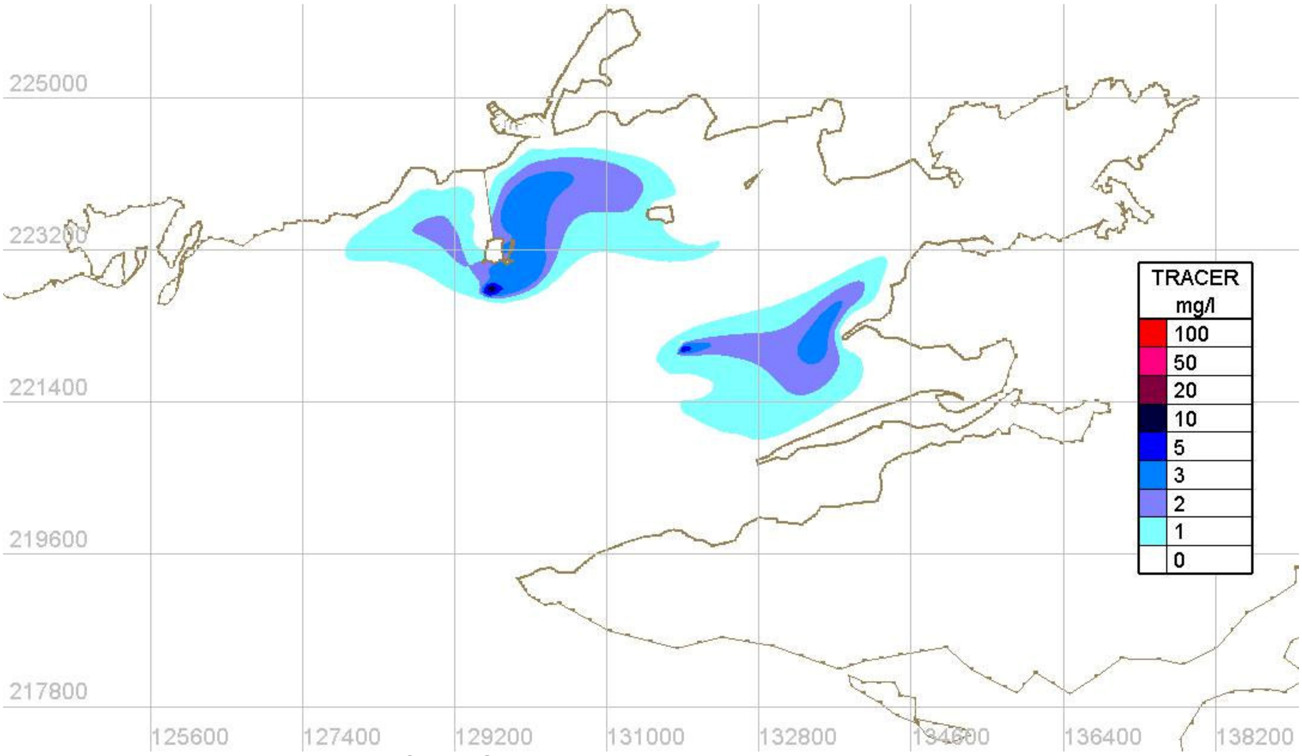


Figure 8.4.97 High Water - Existing Case - Spring tides

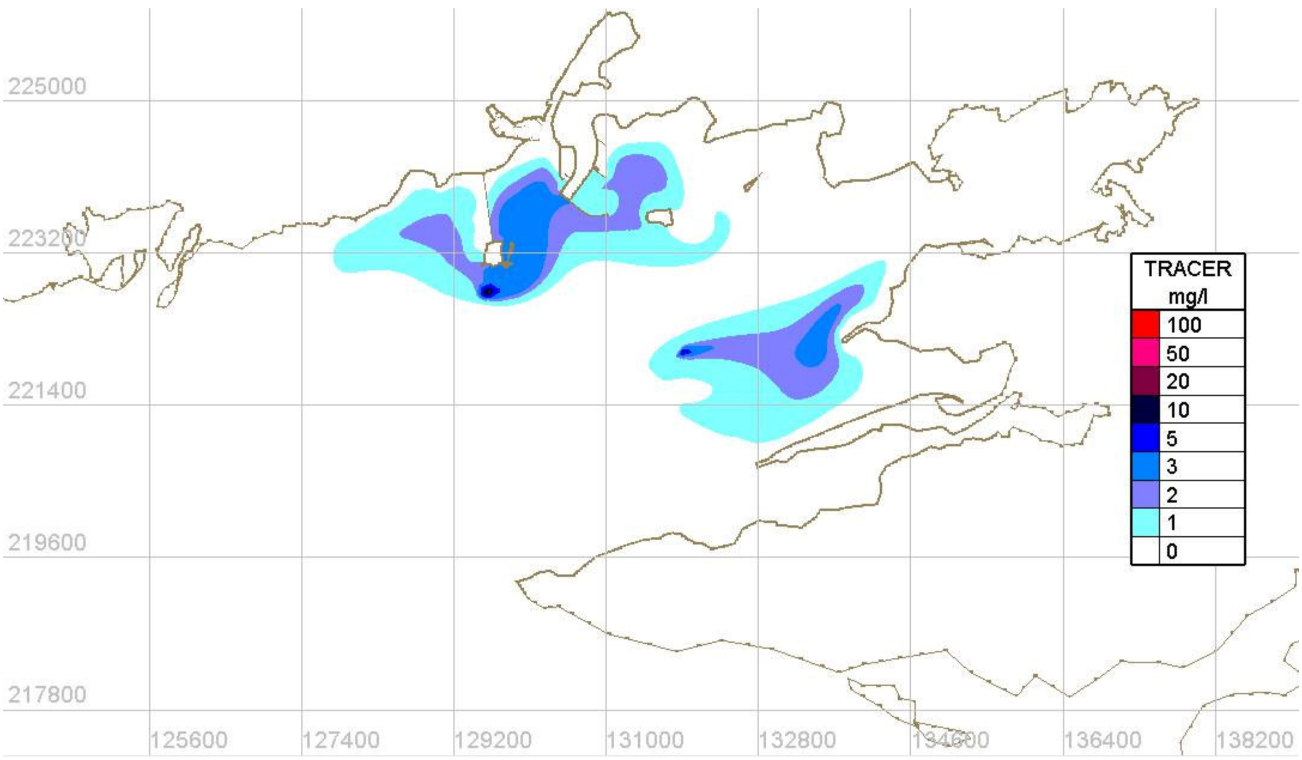


Figure 8.4.98 High Water - Proposed Case - Spring tides

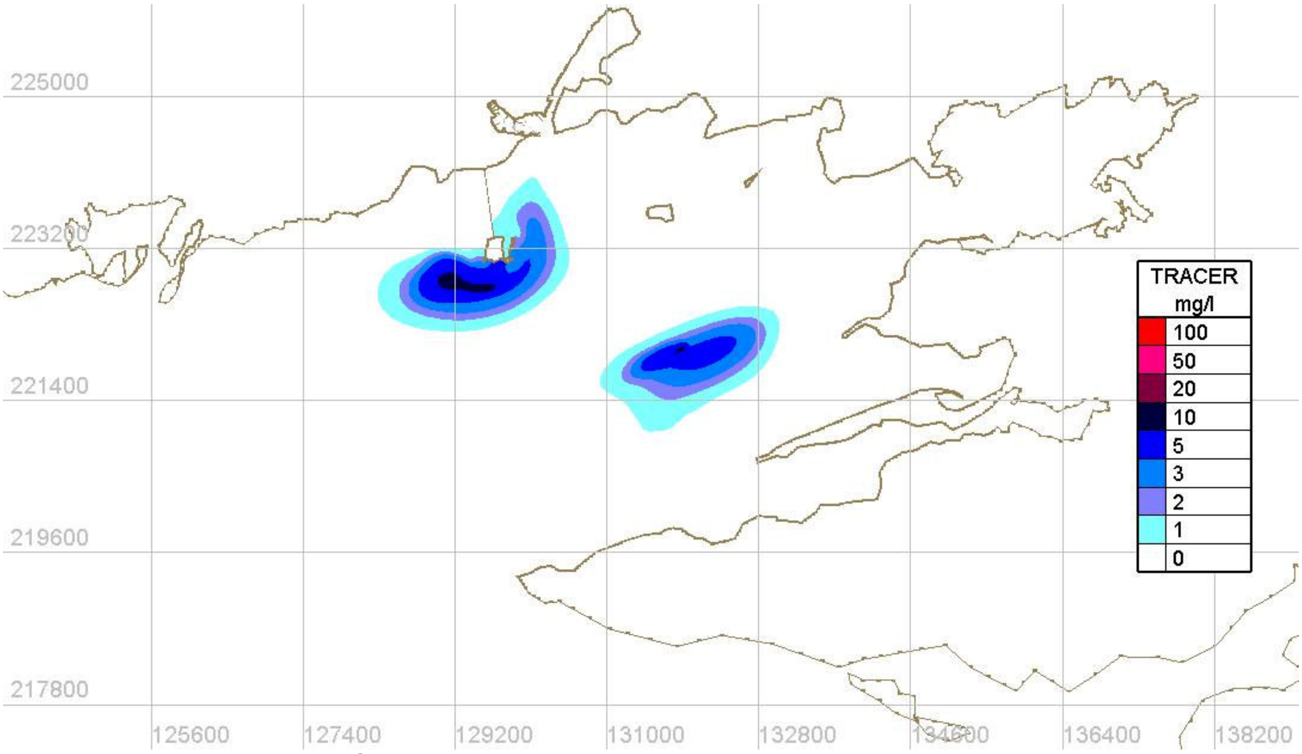


Figure 8.4.99 Mid-ebb Existing Case - Neap tides

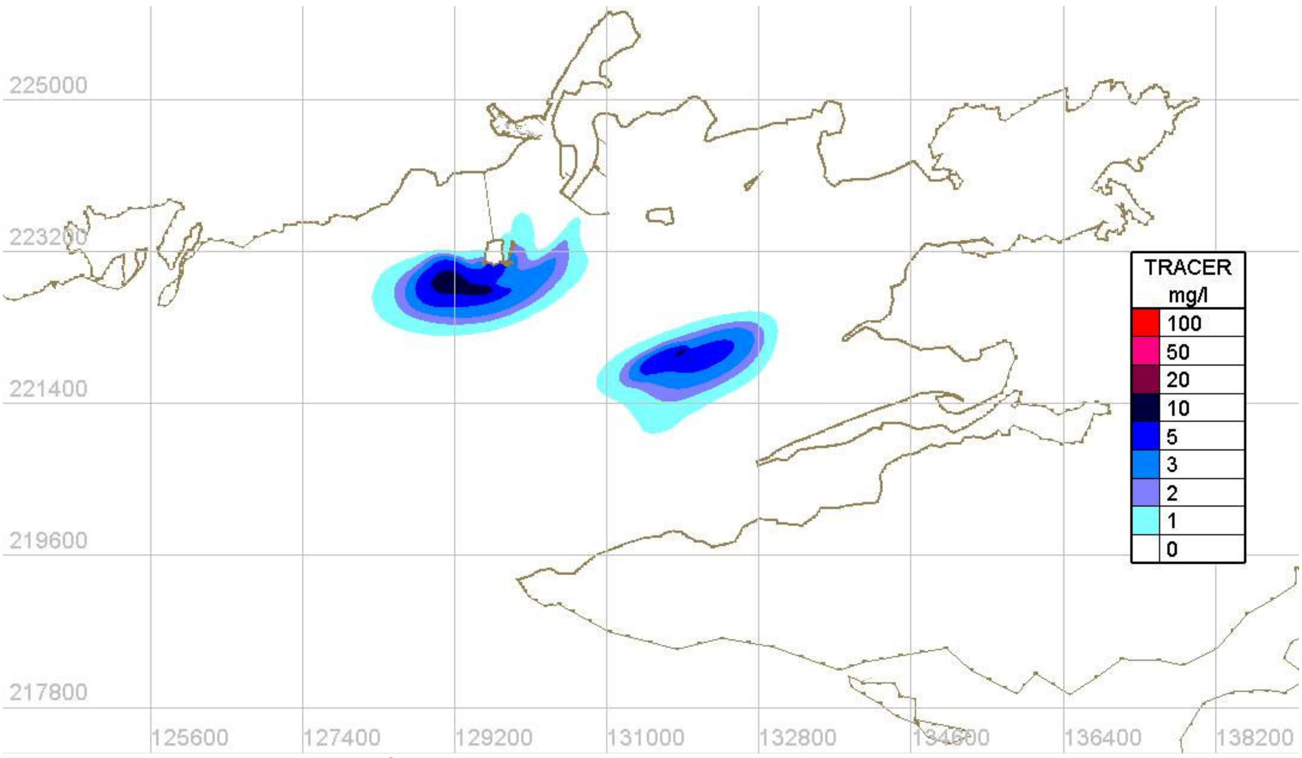


Figure 8.4.100 Mid-ebb Proposed Case - Neap tides

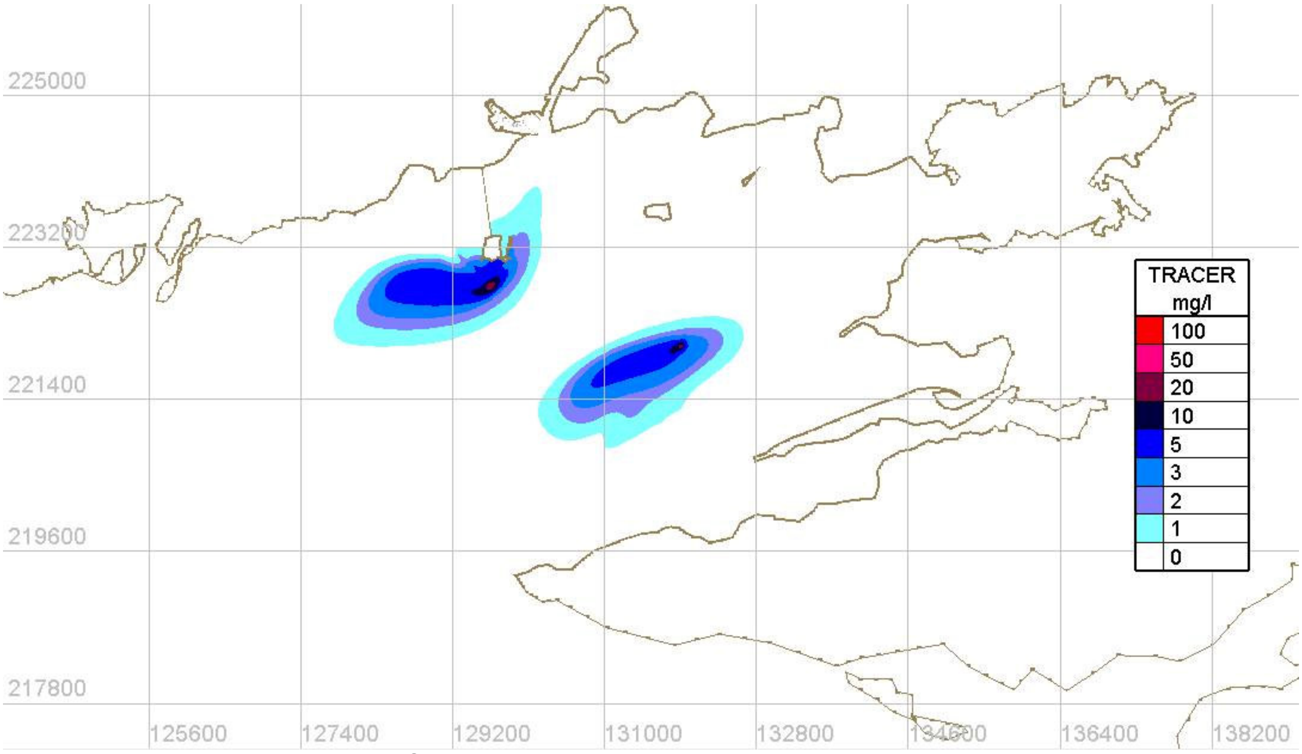


Figure 8.4.101 Low water - Existing Case - Neap tides

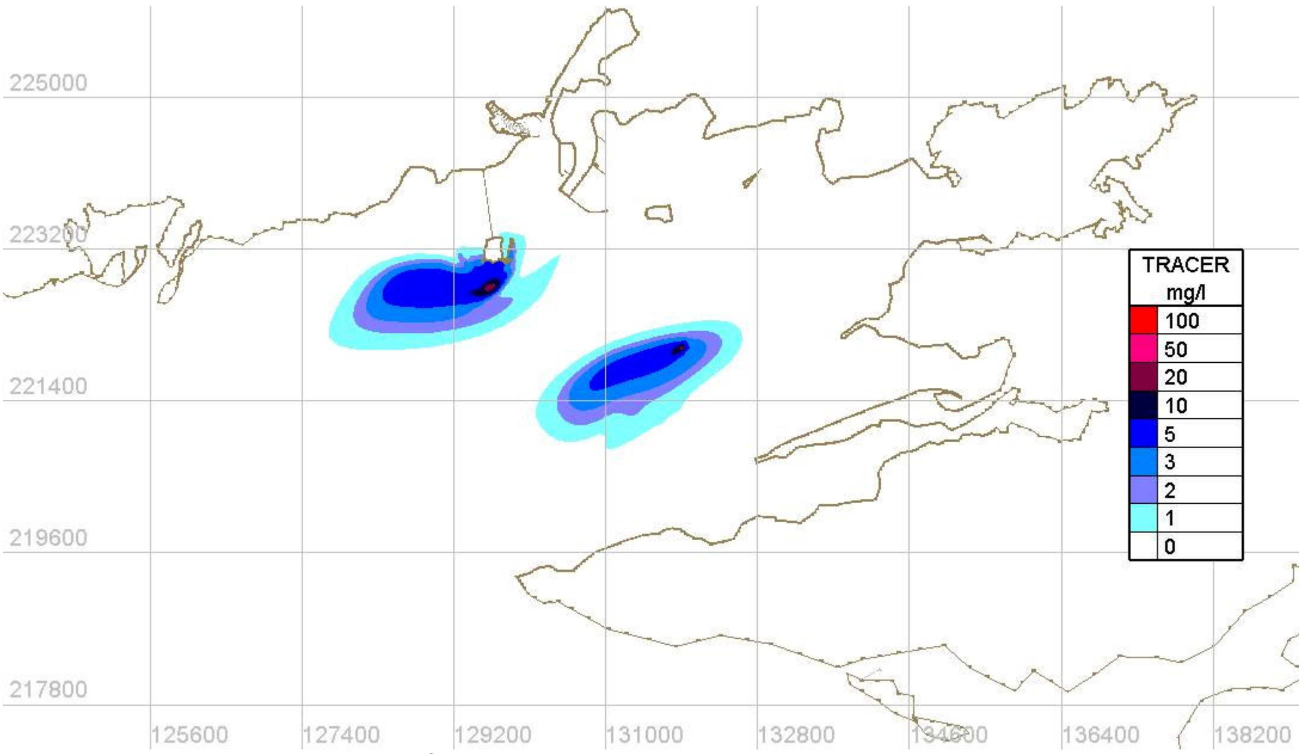


Figure 8.4.102 Low water - Proposed Case - Neap tides

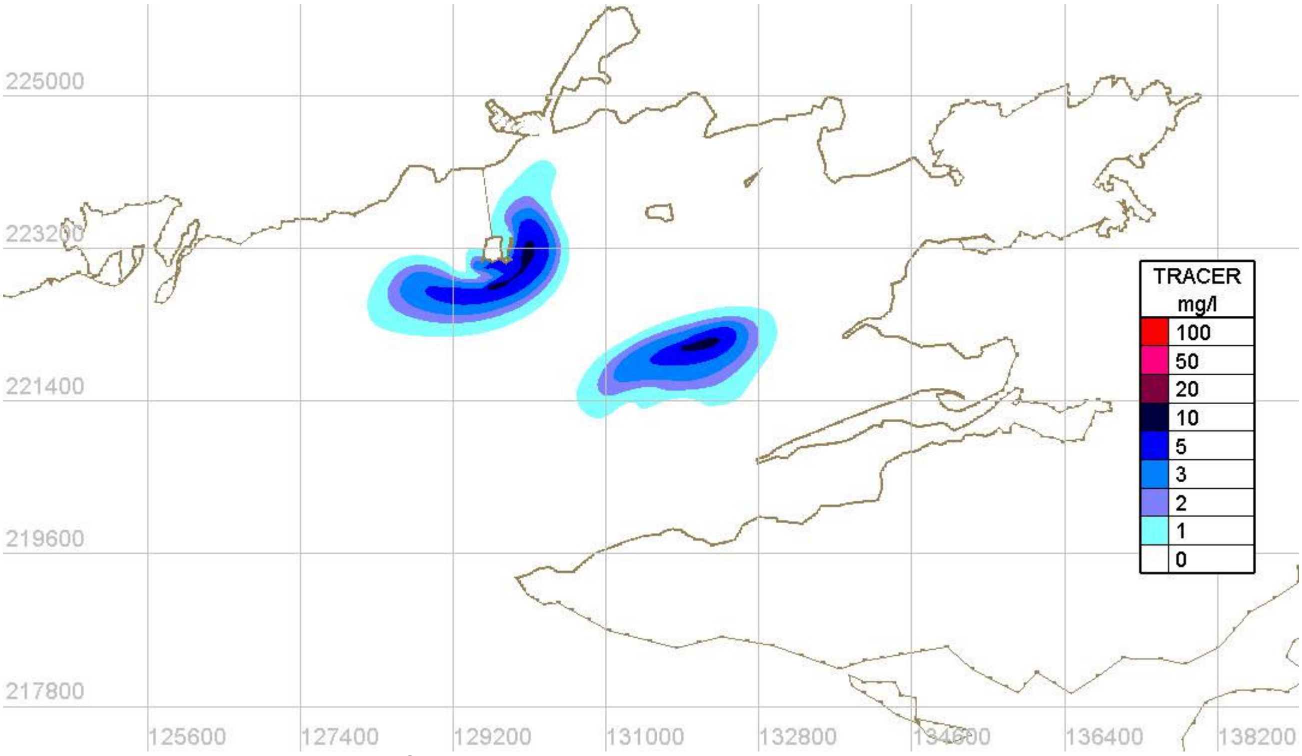


Figure 8.4.103 Mid Flood - Existing Case - Neap tides

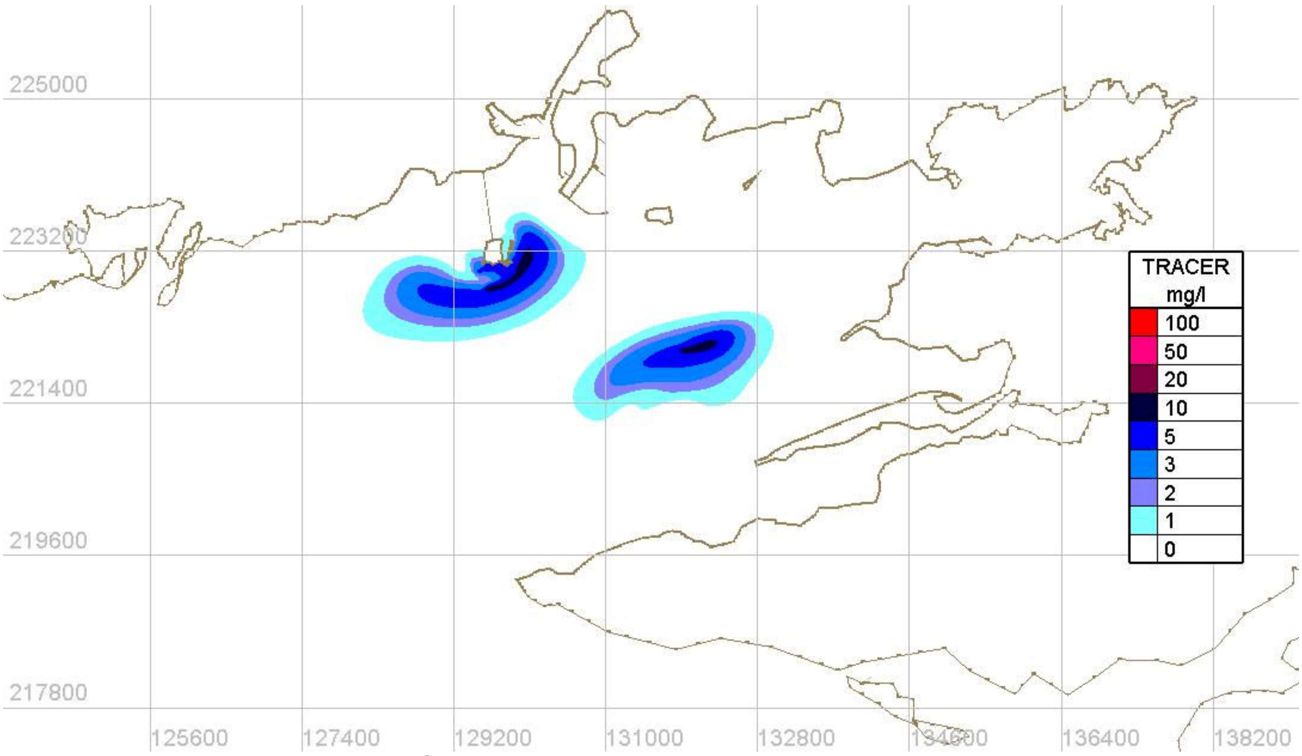


Figure 8.4.104 Mid Flood - Proposed Case - Neap tides

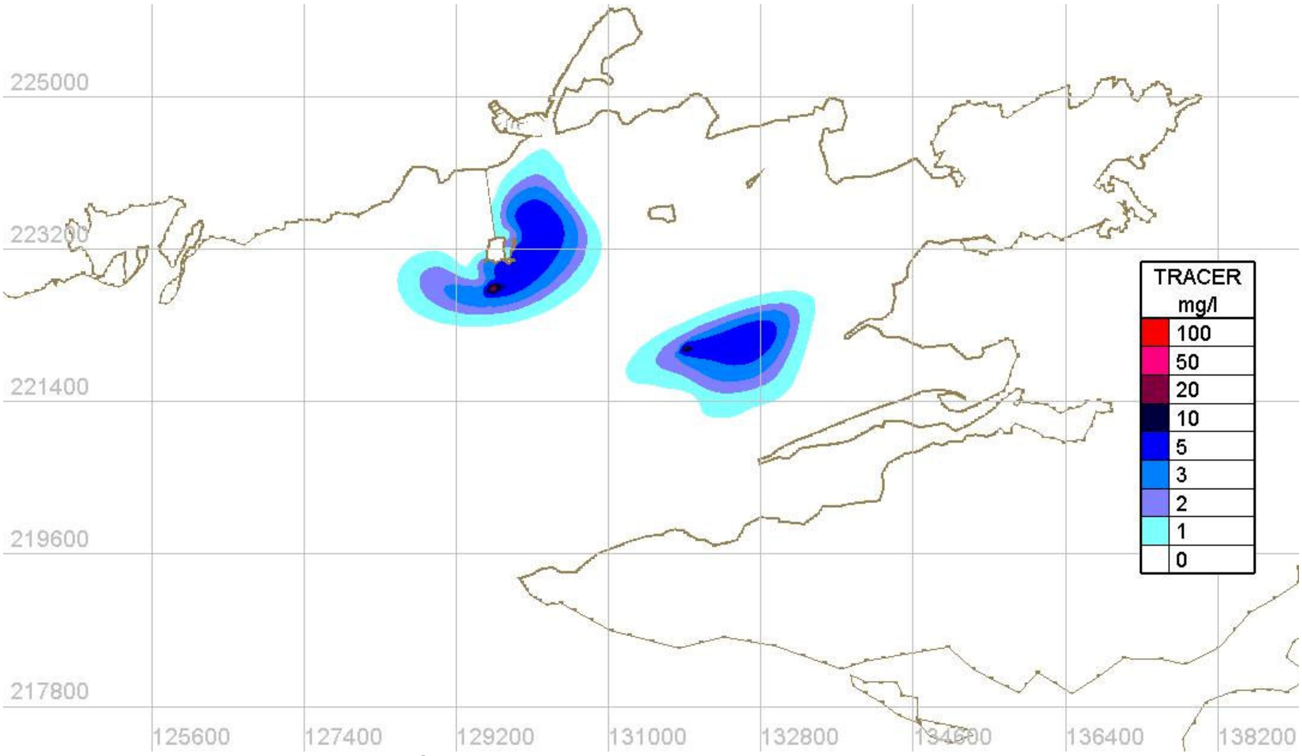


Figure 8.4.105 High Water - Existing Case - Neap tides

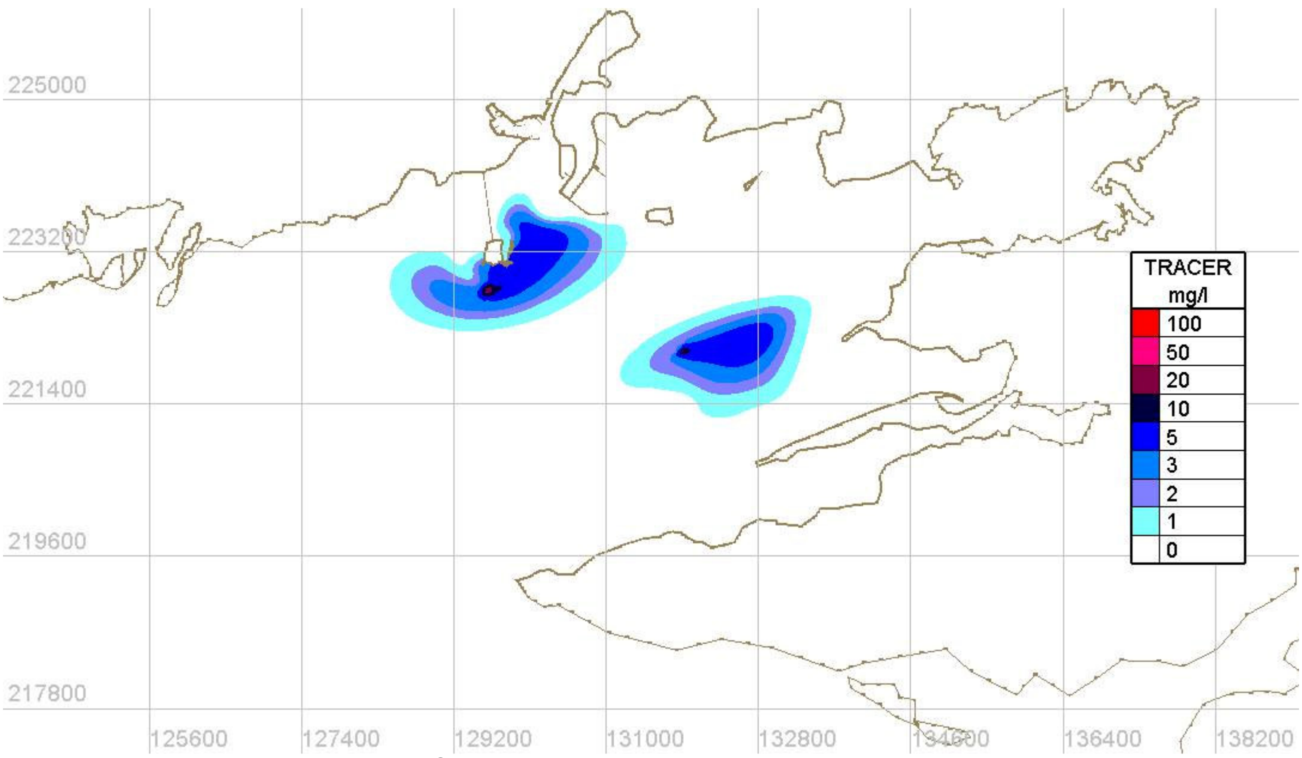


Figure 8.4.106 High Water - Proposed Case - Neap tides

8.4.5 *In Combination effect of the Mutton Island Causeway on Hydrodynamics and Salinities*

As part of the environmental assessment the cumulative effect of the proposed Galway harbour extension in combination with the more recent changes to the Galway City shoreline geometry is investigated in this section.

In order to assess the cumulative impact of the proposed harbour extension development on the hydrodynamics of Inner Galway Bay, an understanding of the hydrodynamics of Galway Bay prior to recent major developments is required. The most significant recent change to the coastline of the Galway City is the Mutton Island causeway which was completed in 2002.

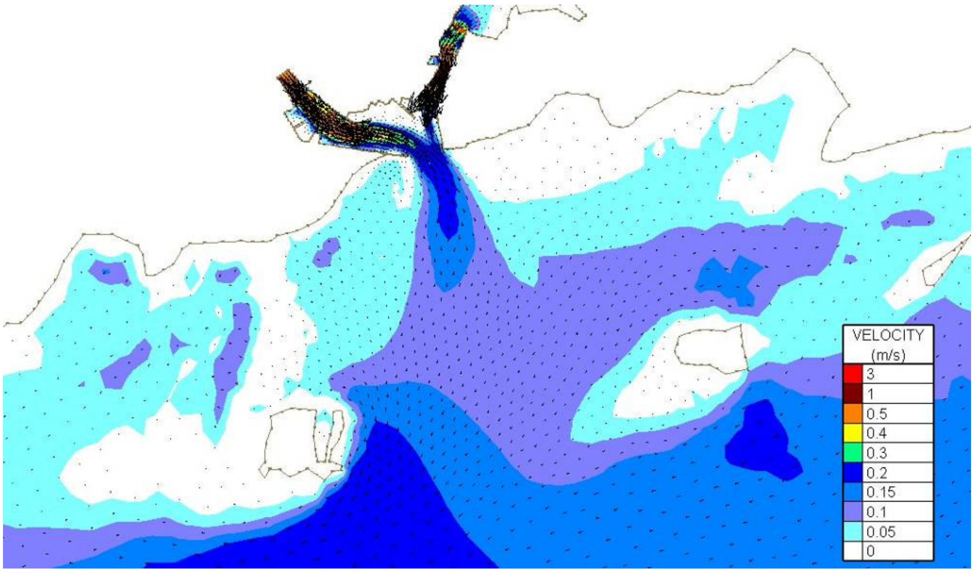
The hydraulic impact of the causeway structure is to force all flow (both ebbing and flooding flows) from and to the docks, Lough Atalia and Renmore area southwards past Mutton Island. The overall effect of the causeway on flows is found to be small with peak velocity magnitudes in the shallower water depths close to Mutton Island increasing by less than 0.05 m/s and with no apparent effect on the deeper waters further offshore of the island. The shallow relatively slack waters north, northwest and northeast of Mutton Island are shown to have become even slacker as a result of the causeway.

The causeway is shown to essentially partition the shallow shoreline area to the west of the Causeway (Grattan road and Whitestrand beach area) from the estuarine waters of the Corrib estuary to the east. The effect of this is to increase salinity along the shoreline to the west of the causeway. The impact of the causeway on velocities, tide levels at the entrance to the docks and Lough Atalia and more remote at Renmore is shown to be negligible.

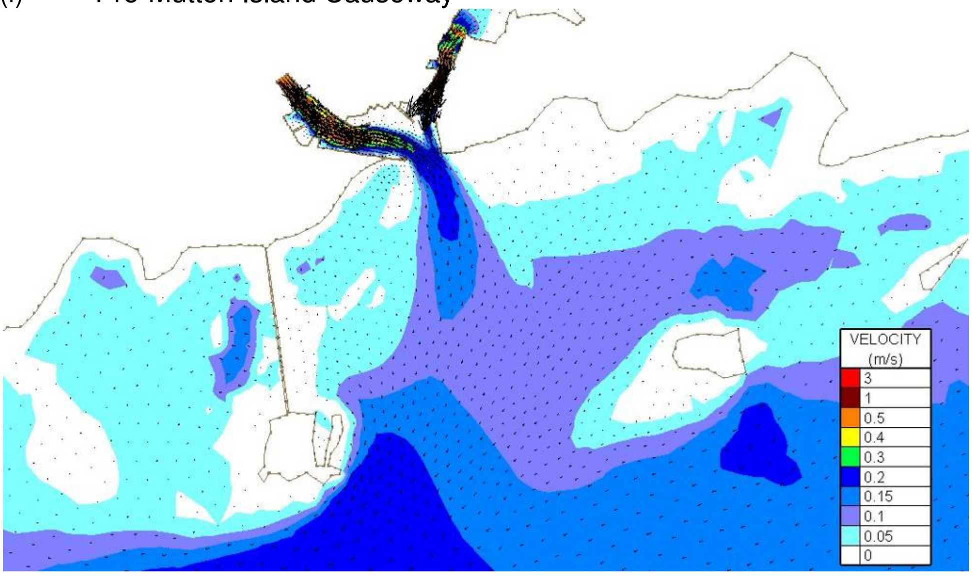
Spring and Neap hydrodynamic simulations for low and median Corrib flows are presented in the following figures 8.4.107 to 8.4.118 for the following three cases:

- Pre Mutton Island Case (without Causeway);
- Current existing Case with Causeway; and
- Proposed Development Case with proposed Galway Harbour Extension.

Contour plots of tidally averaged salinities for Spring and Neap tides are presented in Figures 8.4.119 to 8.4.122 for the above three cases. These salinity plots demonstrate that the major impact within the subject area is produced by the causeway preventing freshwater mixing and transport north past Mutton Island and thus significantly increasing salinity along the shoreline area to the west of the causeway. The combined effect of the causeway and the proposed harbour extension will be to concentrate the plume of Corrib freshwater flow southwards between the proposed harbour and the causeway and thereby reduce salinities within the new approach channel to the docks area and increase salinities along the shoreline to the east of the new harbour towards Renmore Beach.



(i) Pre-Mutton Island Causeway



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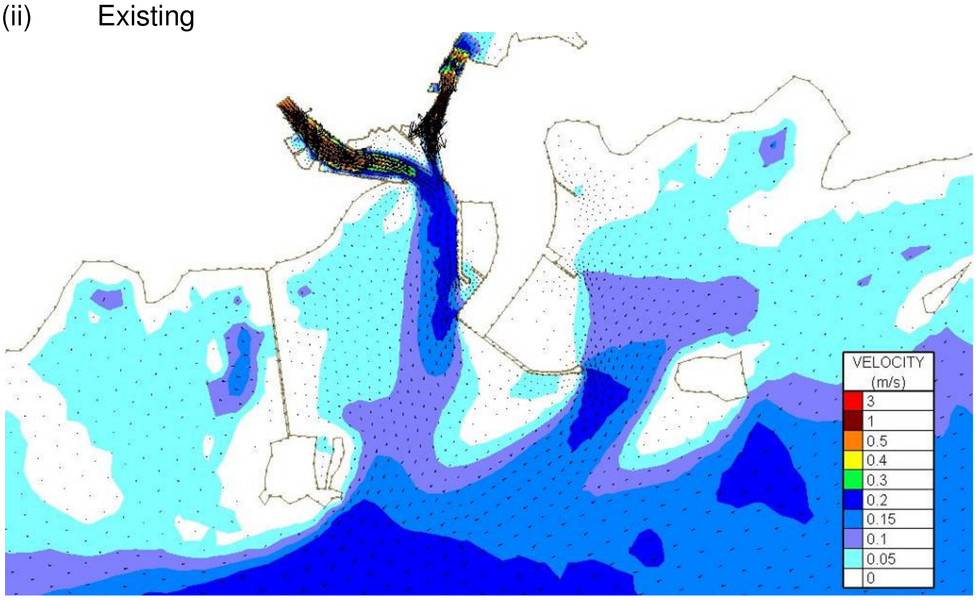
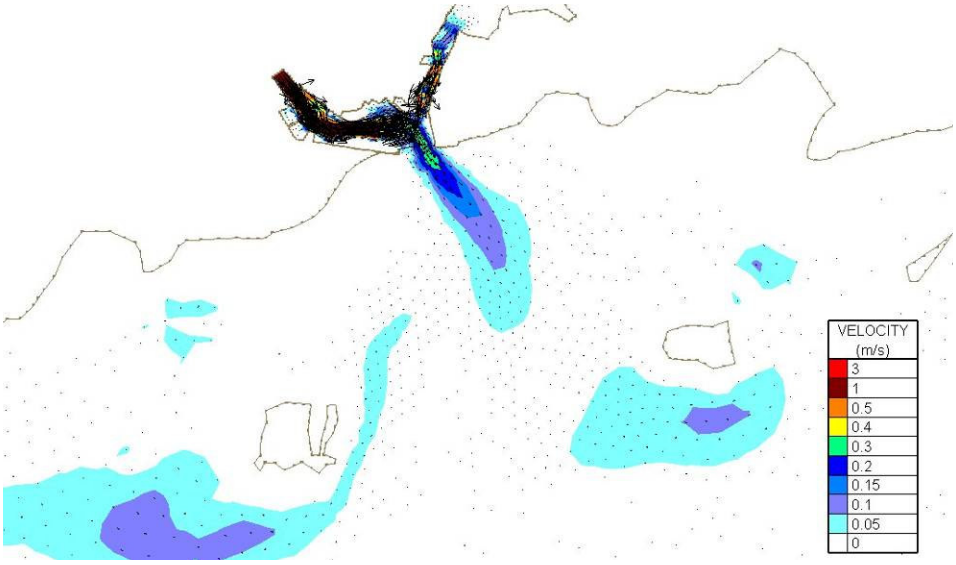
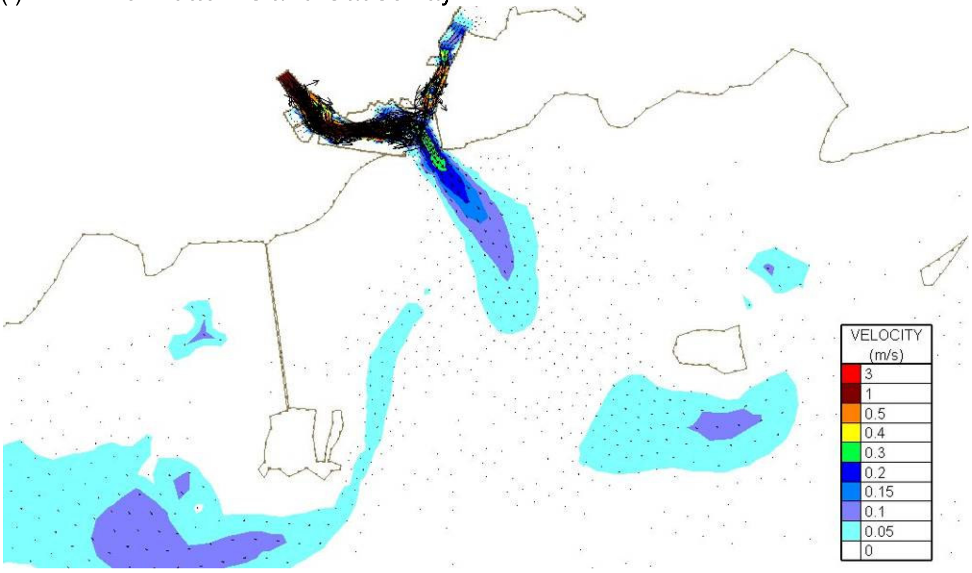


(iii) Proposed

Figure 8.4.107 Mid-ebb Velocities – Spring Tide Median Flow



(i) Pre-Mutton Island Causeway



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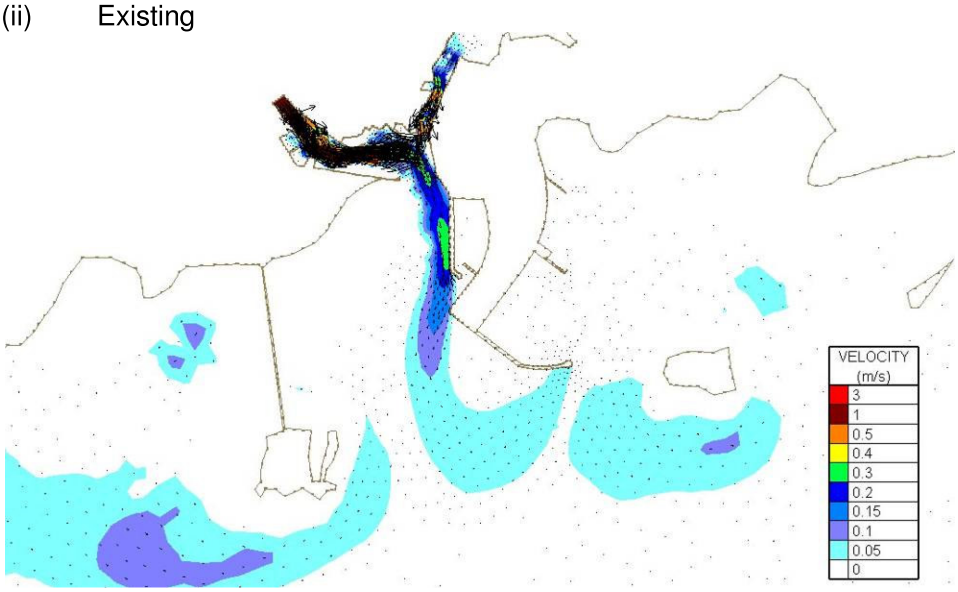
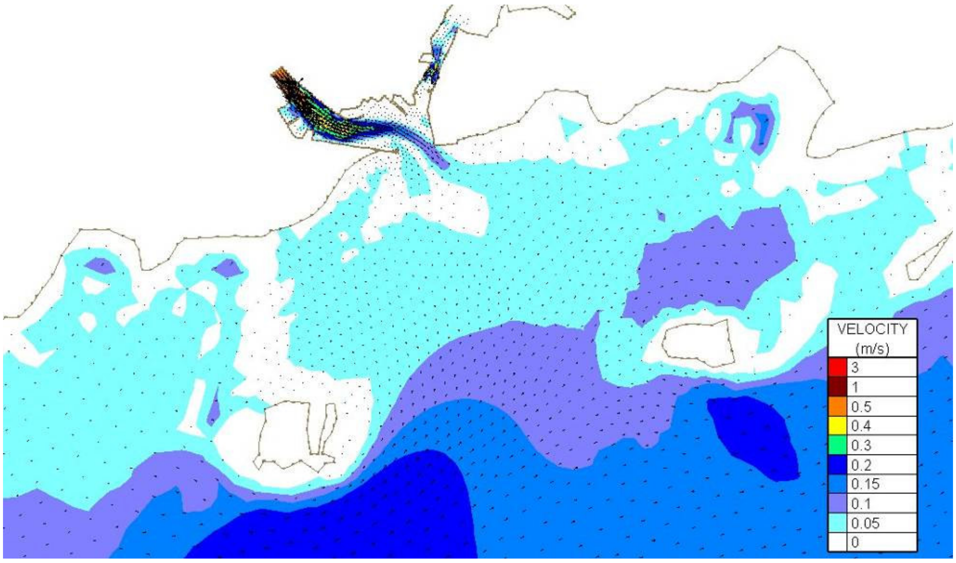


(iii) Proposed

Figure 8.4.108 Low-Water Velocities – Spring Tide Median Flow



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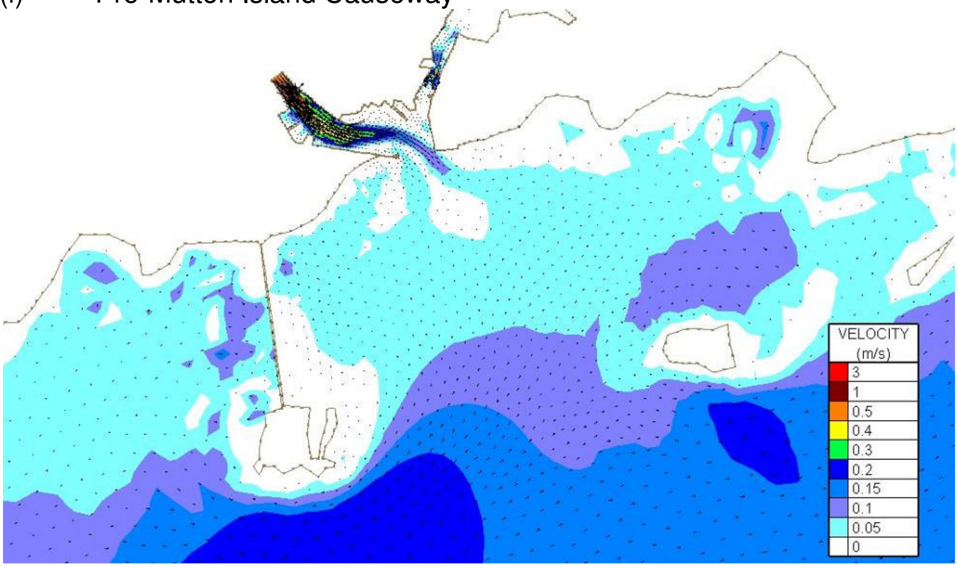

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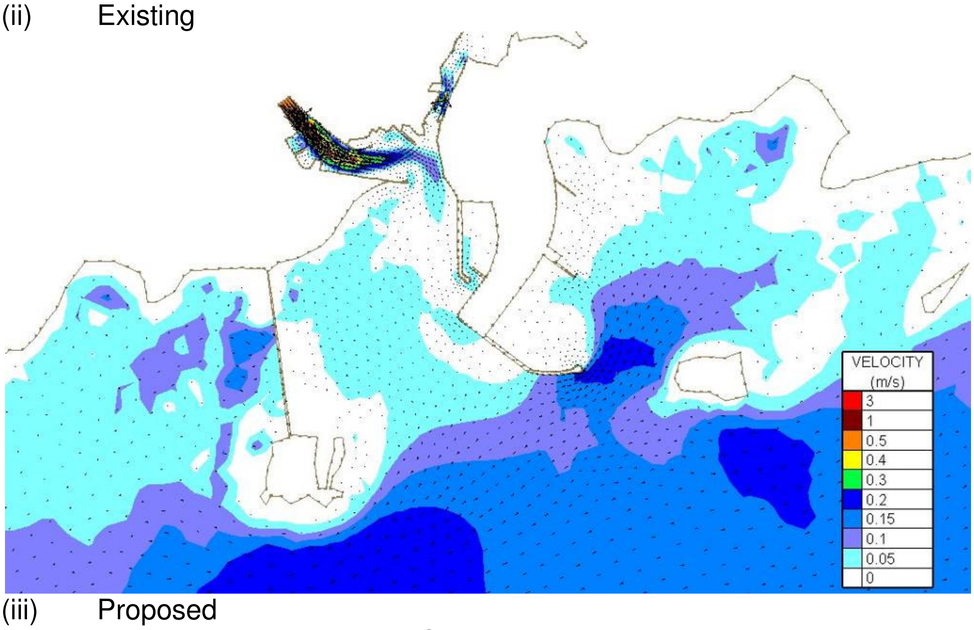
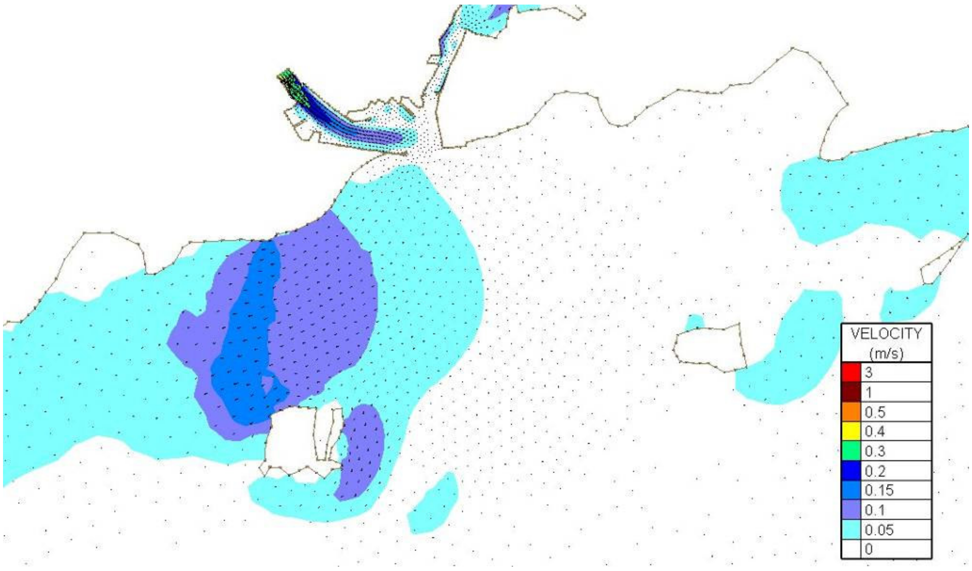


(iii) Proposed

Figure 8.4.109 Mid-flood Velocities – Spring Tide Median Flow



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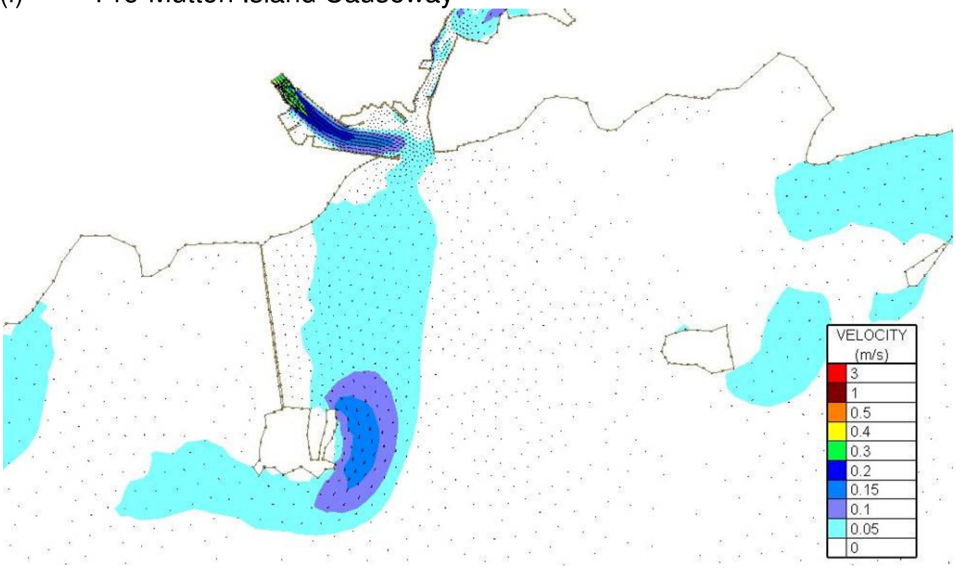

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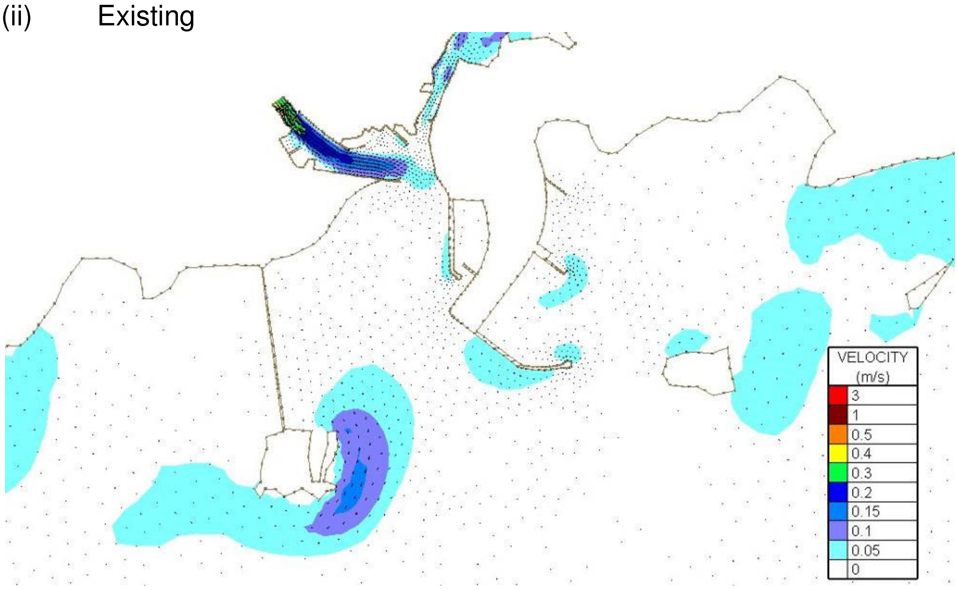
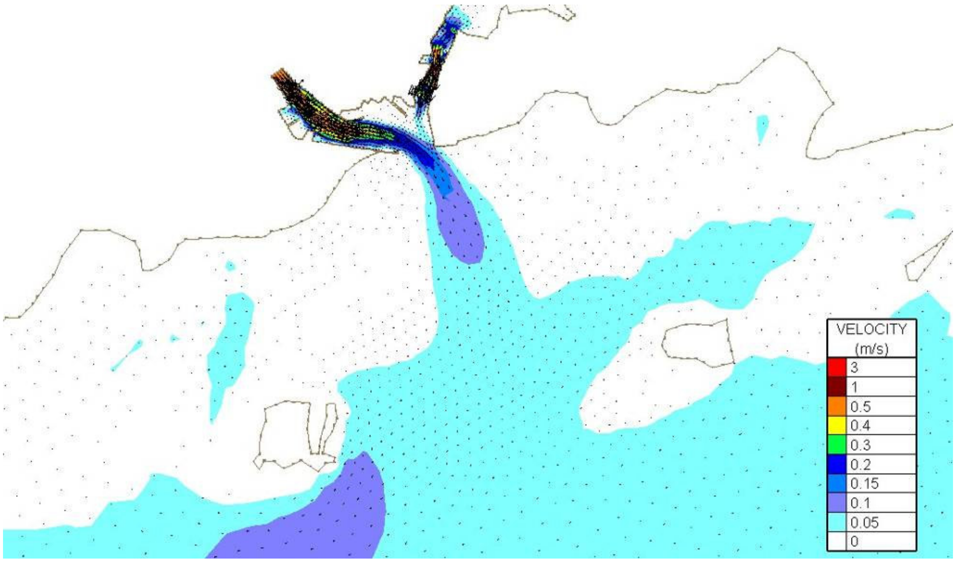


(iii) Proposed

Figure 8.4.110 High water Velocities – Spring Tide Median Flow



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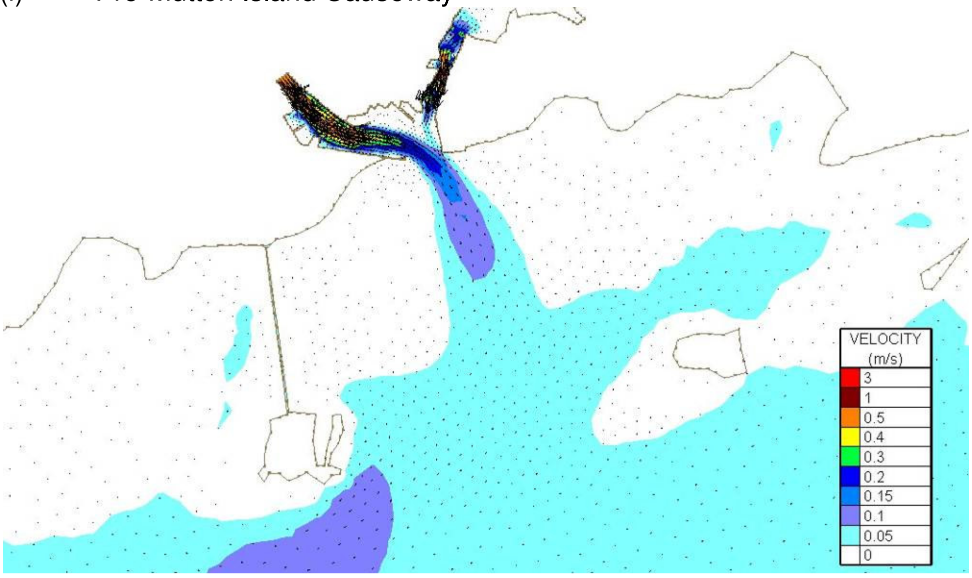

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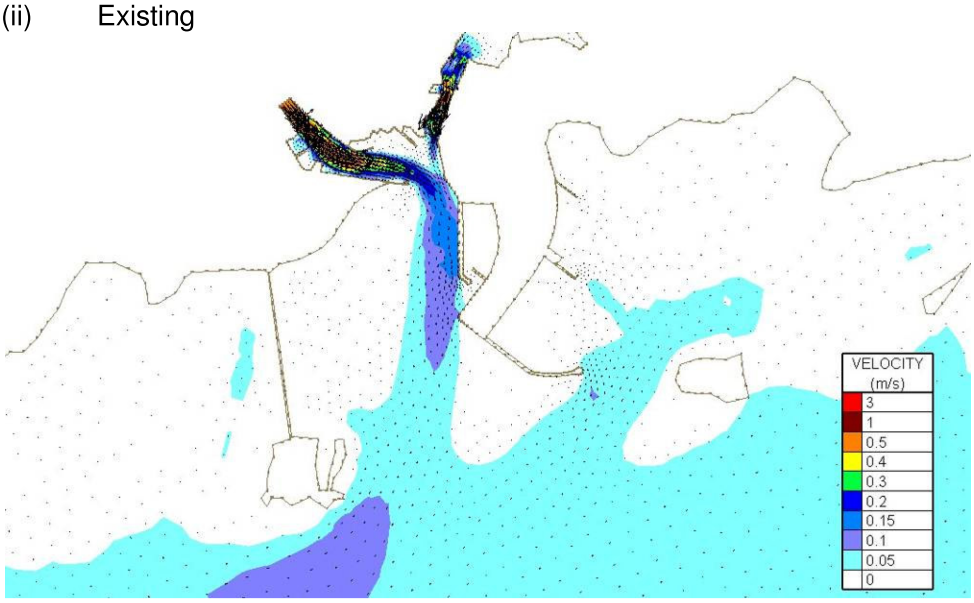
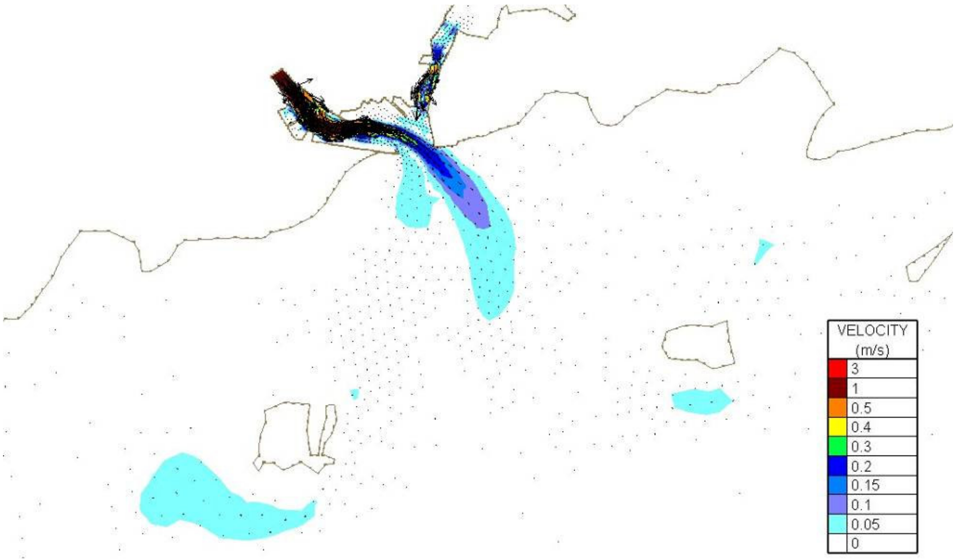


(iii) Proposed

Figure 8.4.111 Mid-ebb Velocities – Neap Tide Median Flow



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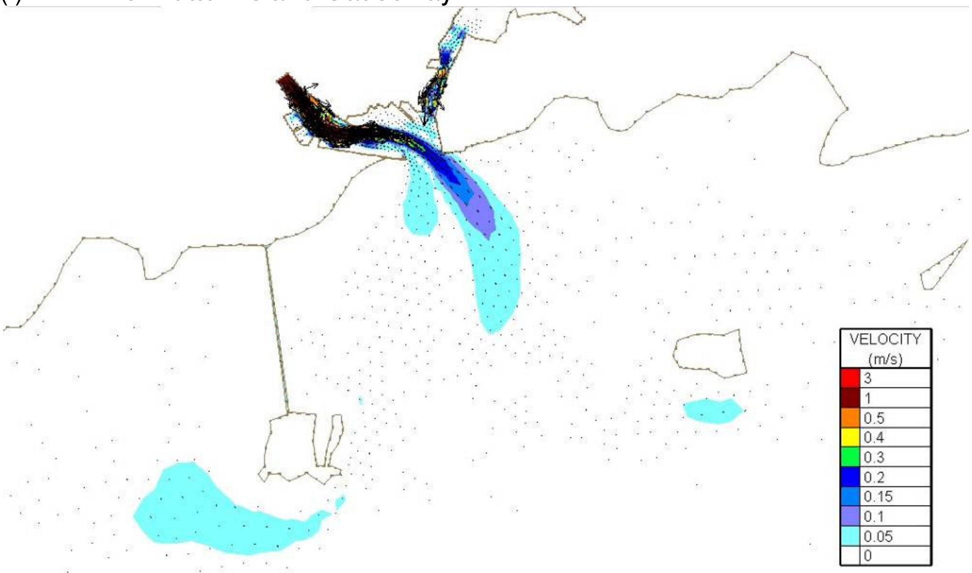

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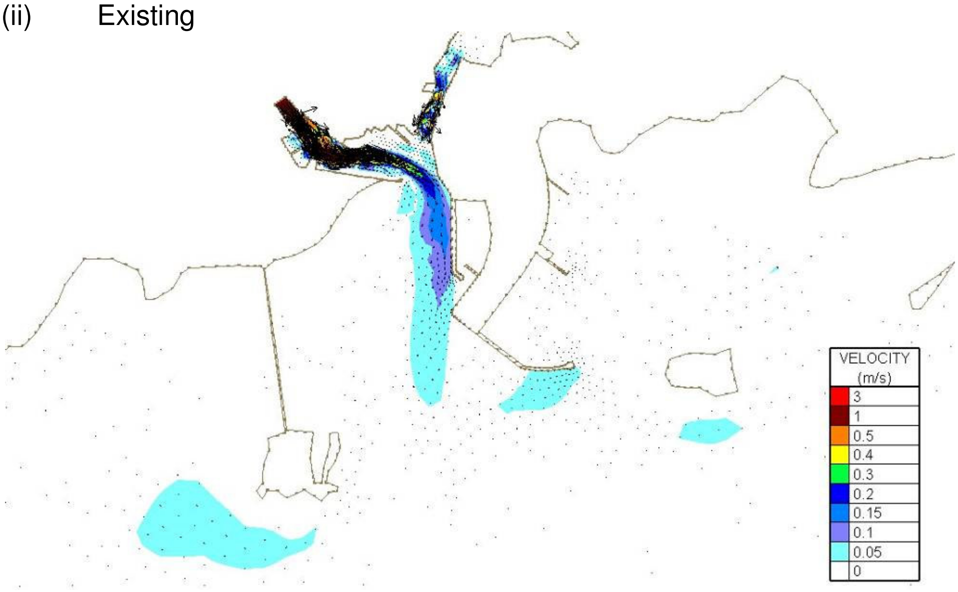
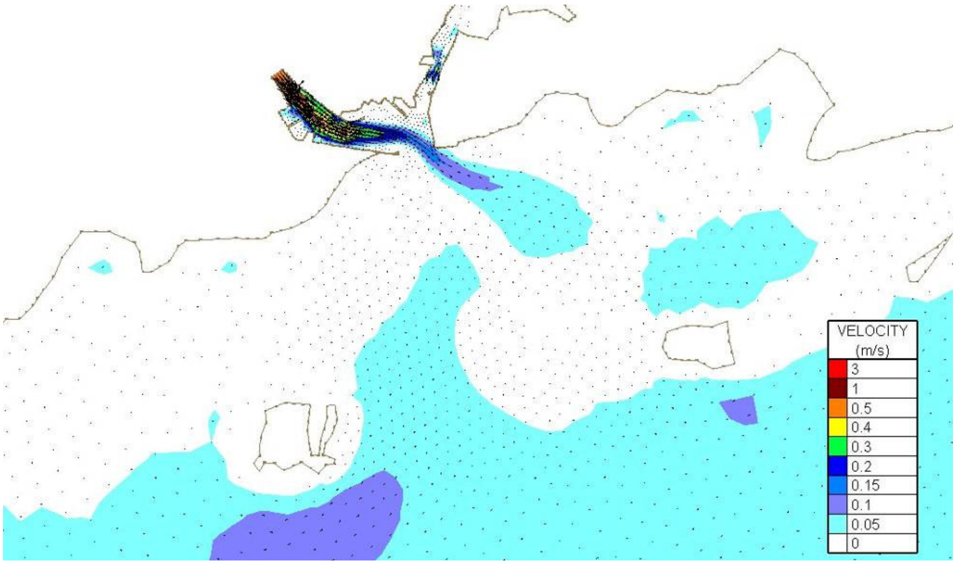


(iii) Proposed

Figure 8.4.112 Low water Velocities – Neap Tide Median Corrib Flow



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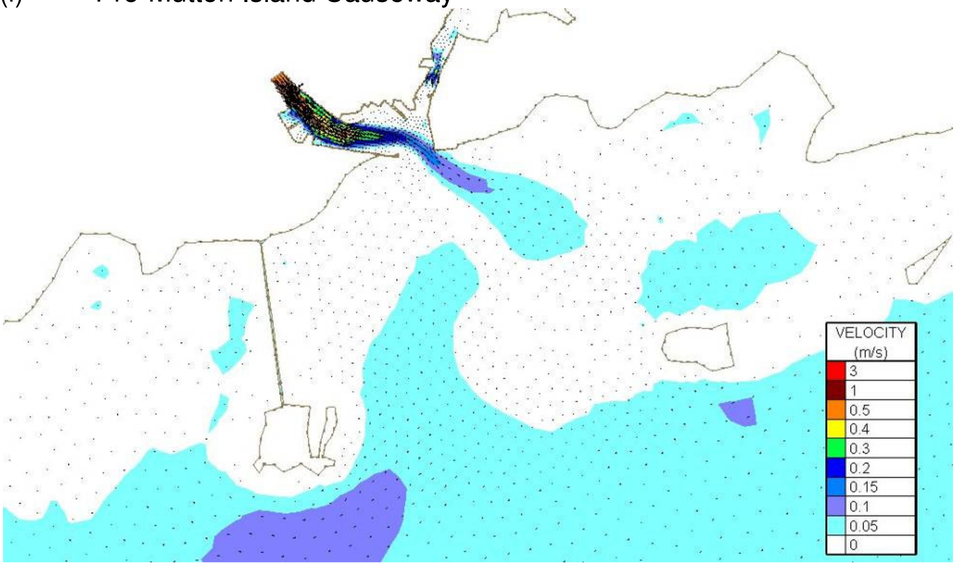

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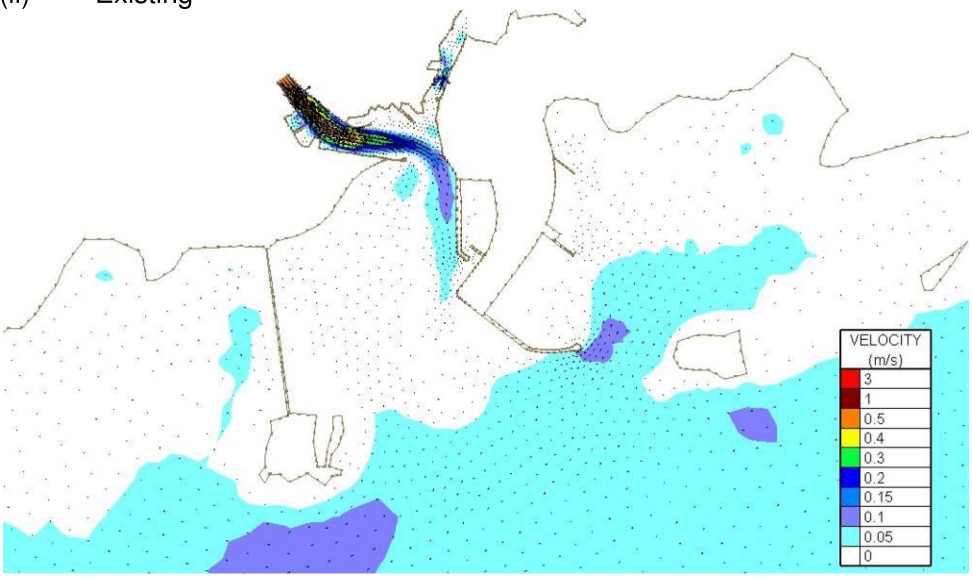

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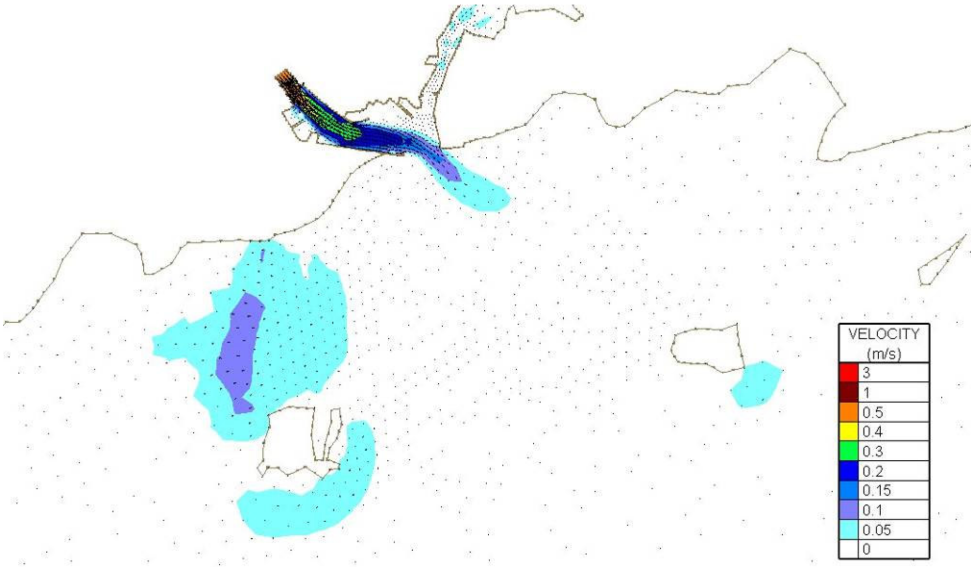
Figure 8.4.113 Mid-flood Velocities – Neap Tide Median Corrib Flow



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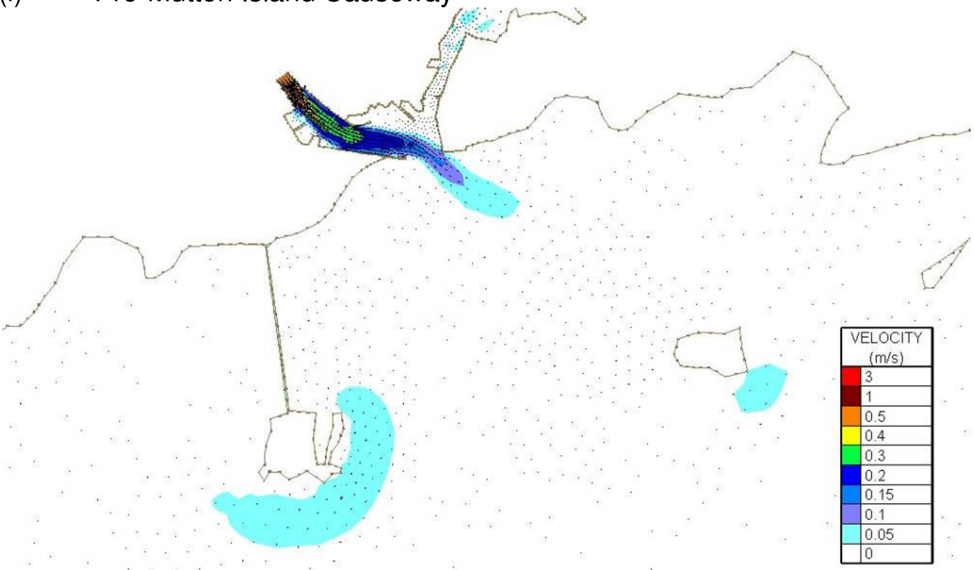

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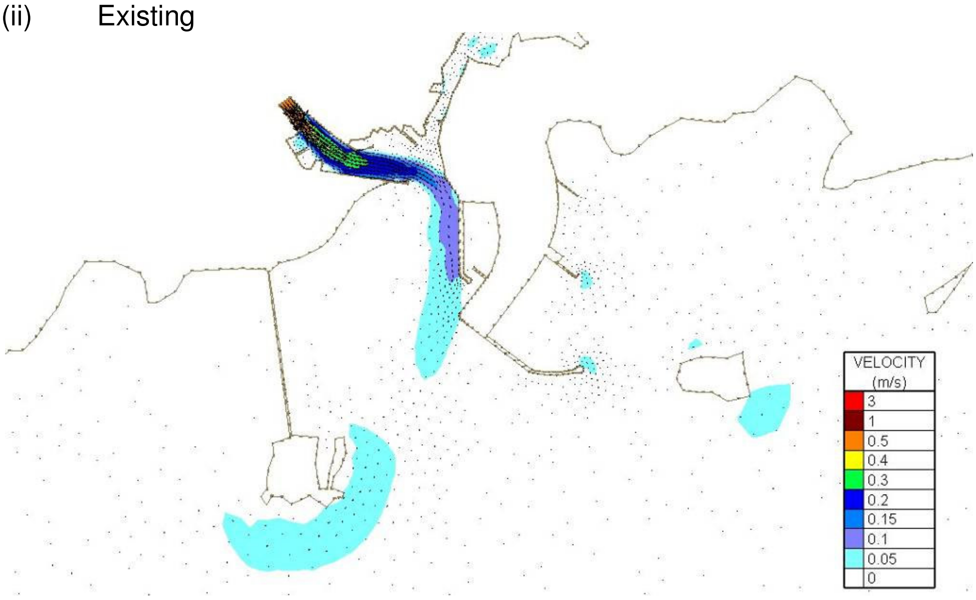
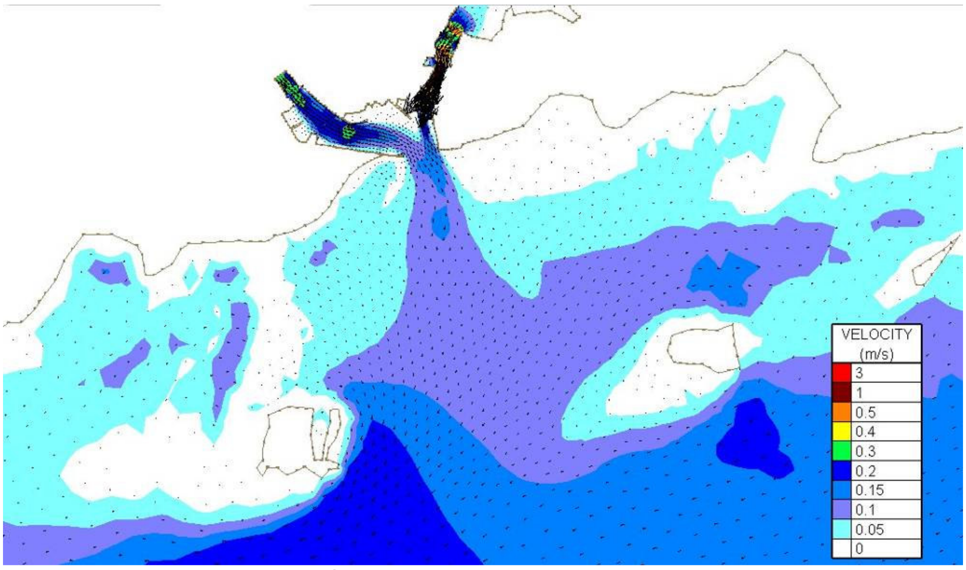


(iii) Proposed

Figure 8.4.114 High Water Velocities – Neap Tide Median Corrib Flow



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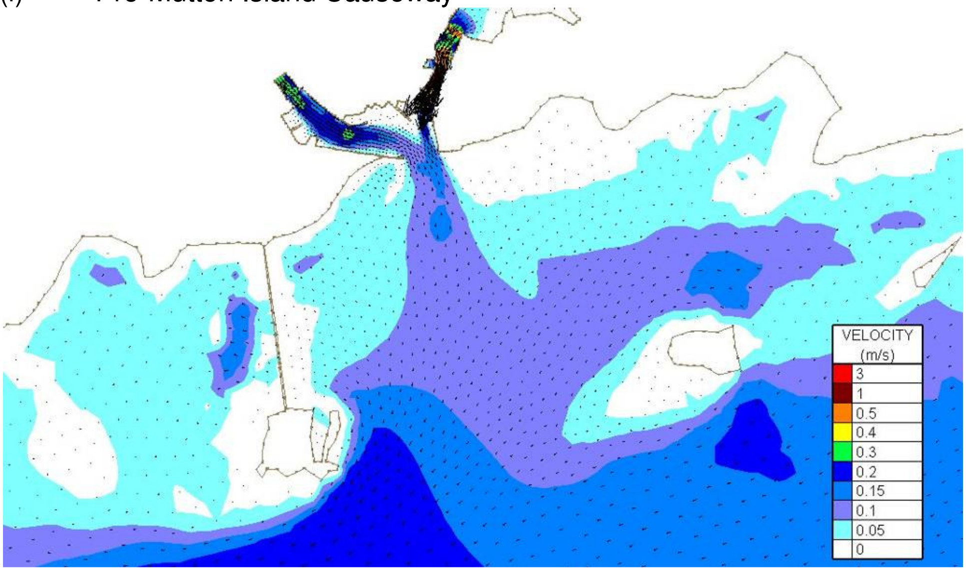

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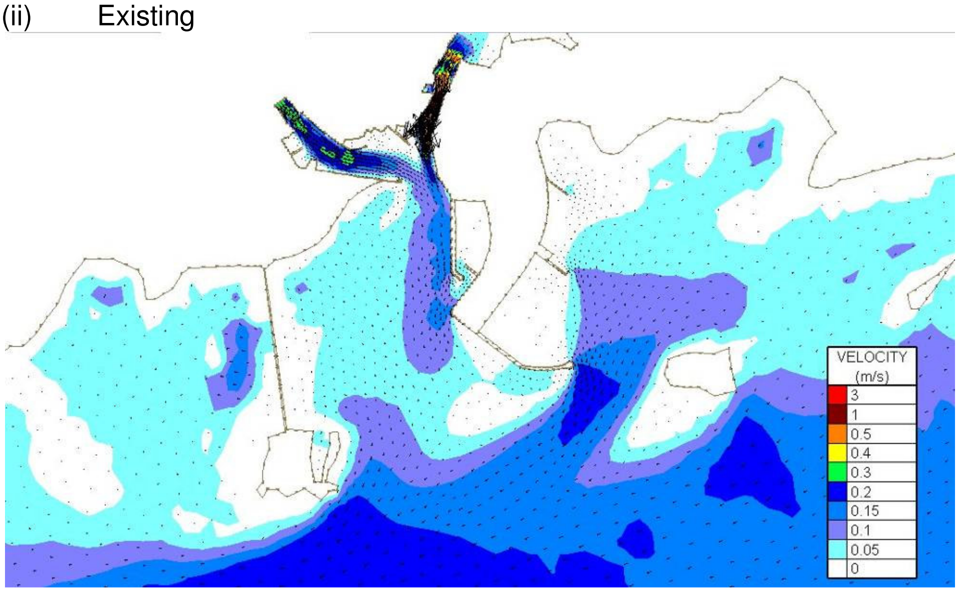
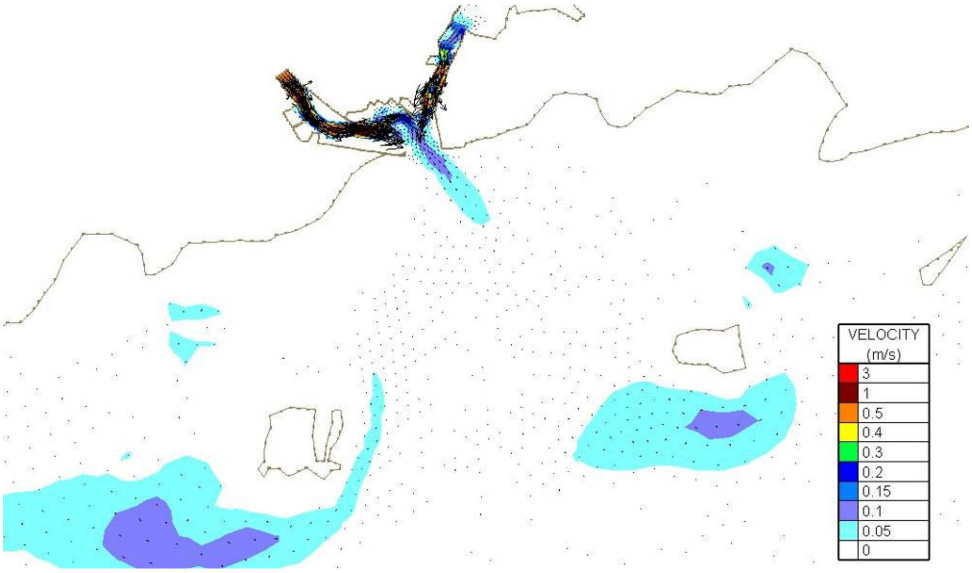


(iii) Proposed

Figure 8.4.115 Mid-ebb Velocities – Spring Tide Corrib Low Flow



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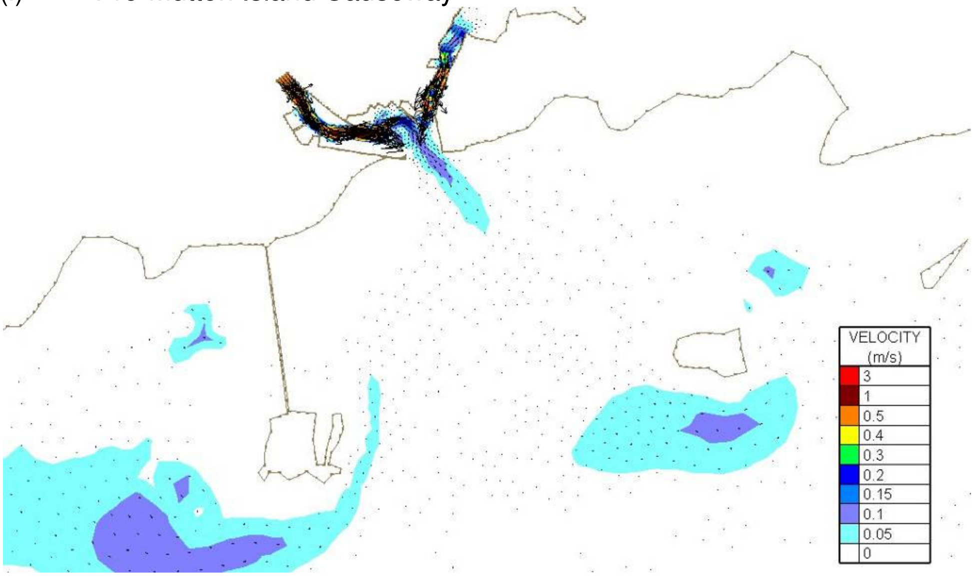

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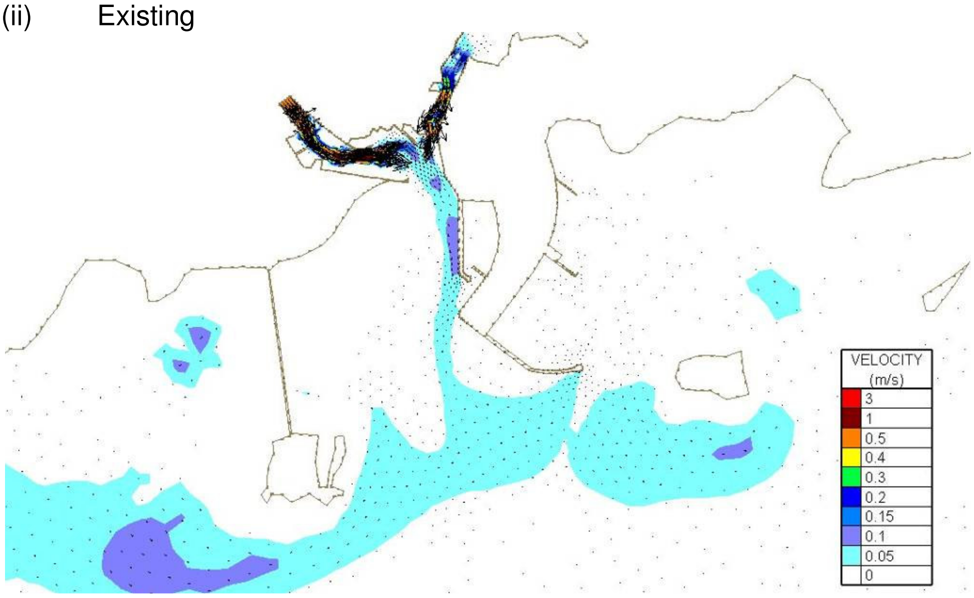
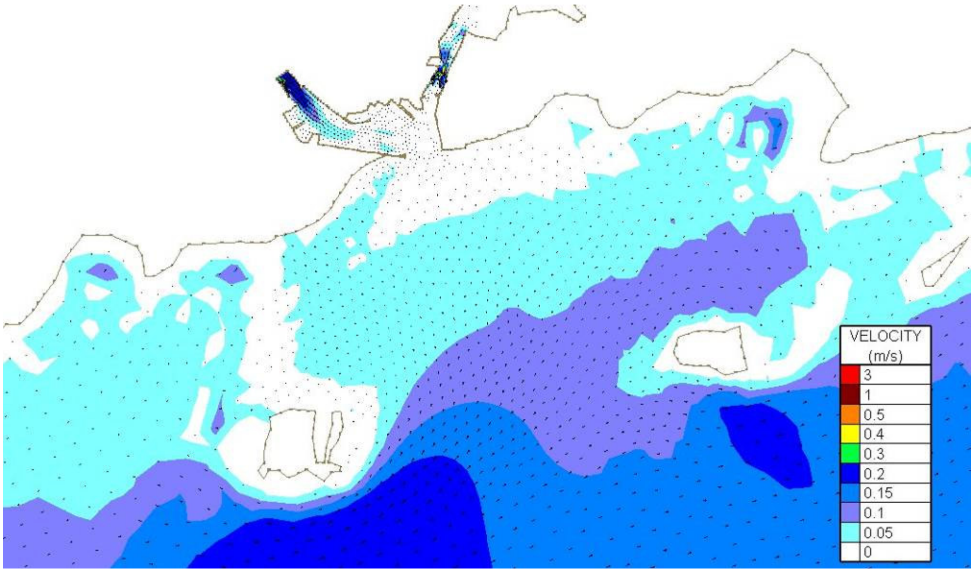


(iii) Proposed

Figure 8.4.116 Low water Velocities – Spring Tide Corrib Low Flow



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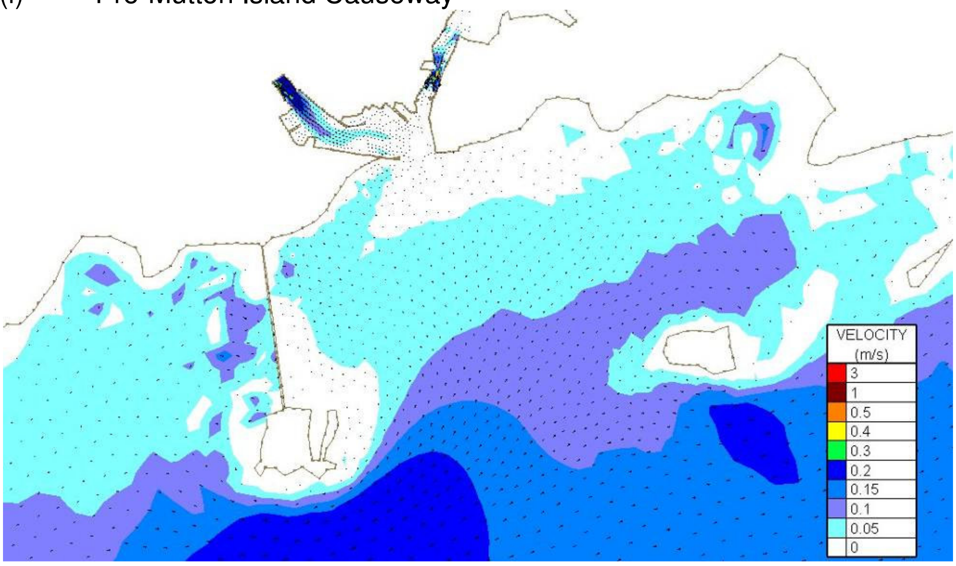

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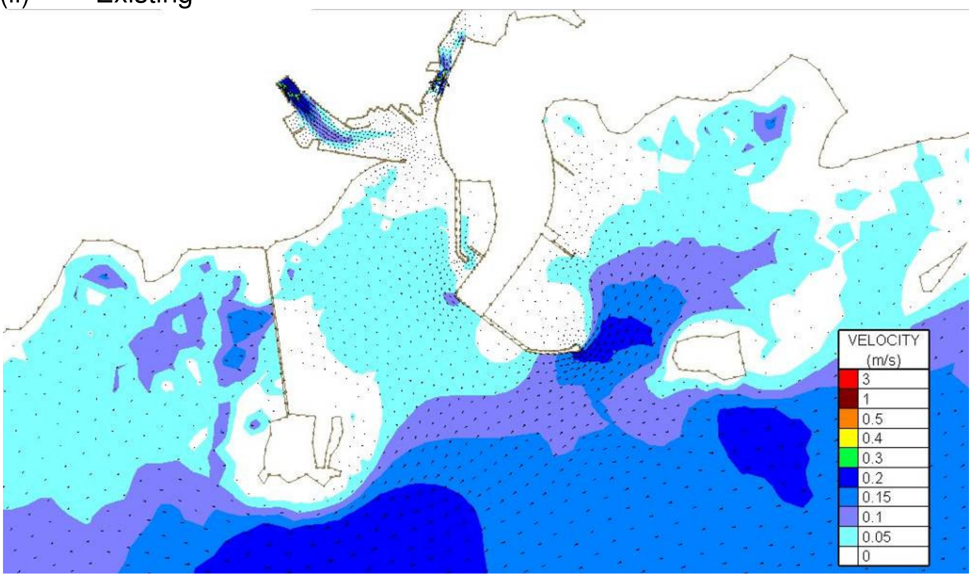
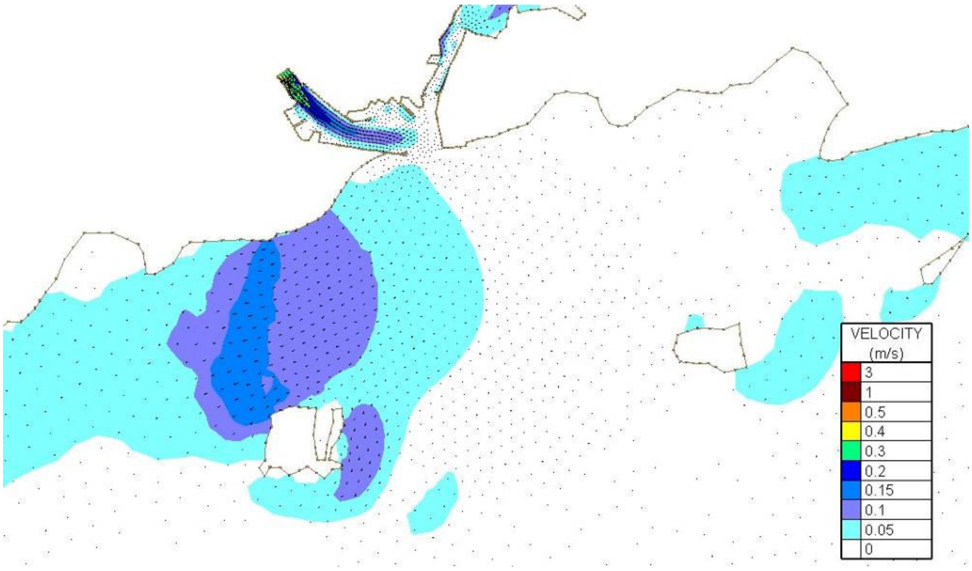


(iii) Proposed

Figure 8.4.117 Mid-flood Velocities – Spring Tide Corrib Low Flow



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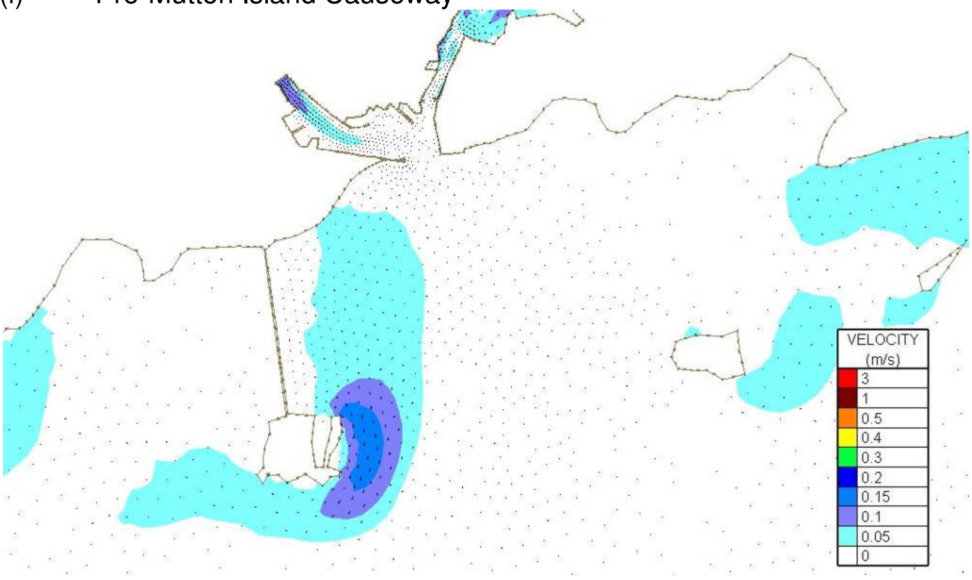

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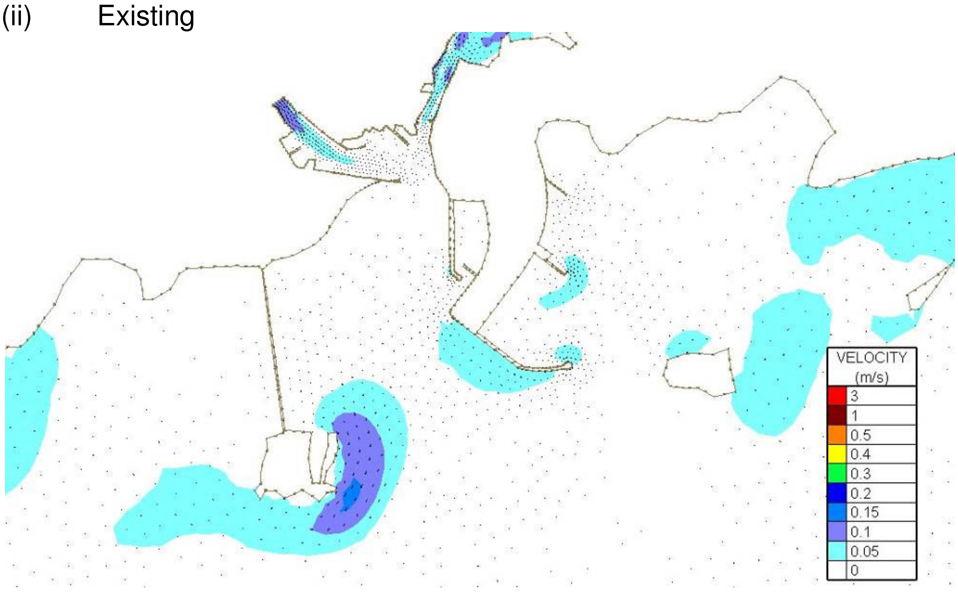


(iii) Proposed

Figure 8.4.118 High water Velocities – Spring Tide Corrib Low Flow

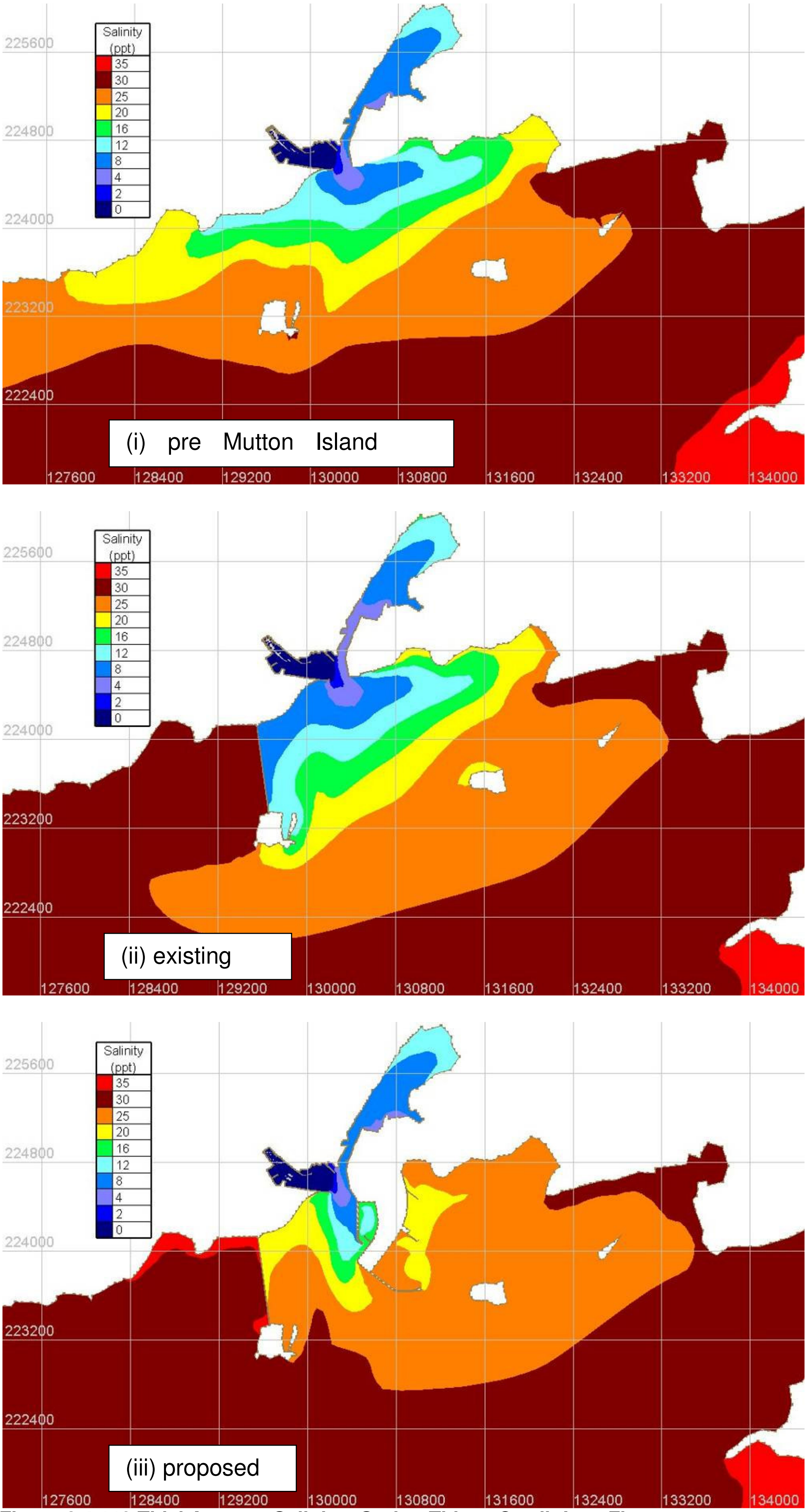


Figure 8.4.119 Tidal Average Salinity –Spring Tides –Corrib Low Flow

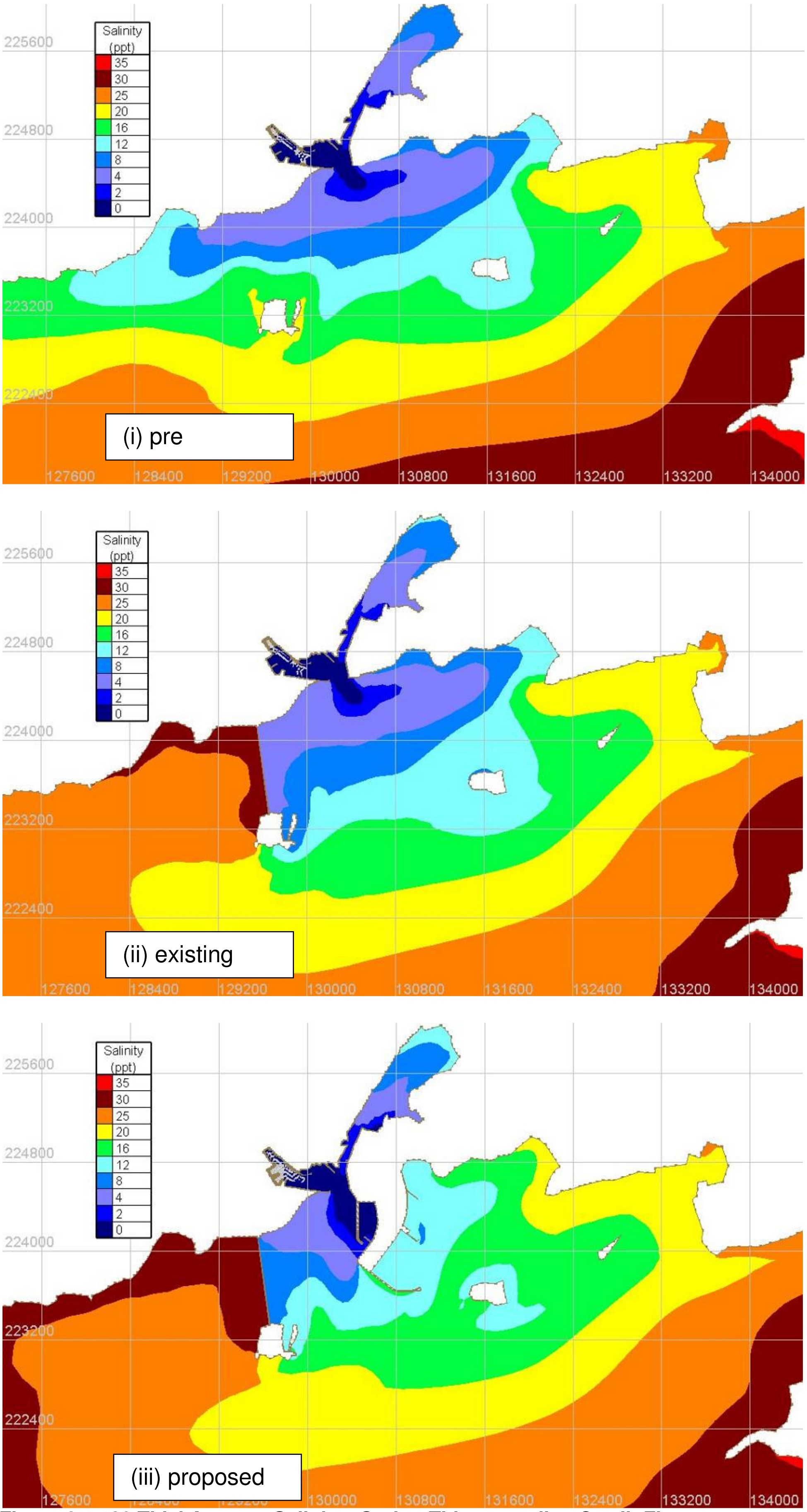


Figure 8.4.120 Tidal Average Salinity –Spring Tides – median Corrib Flow

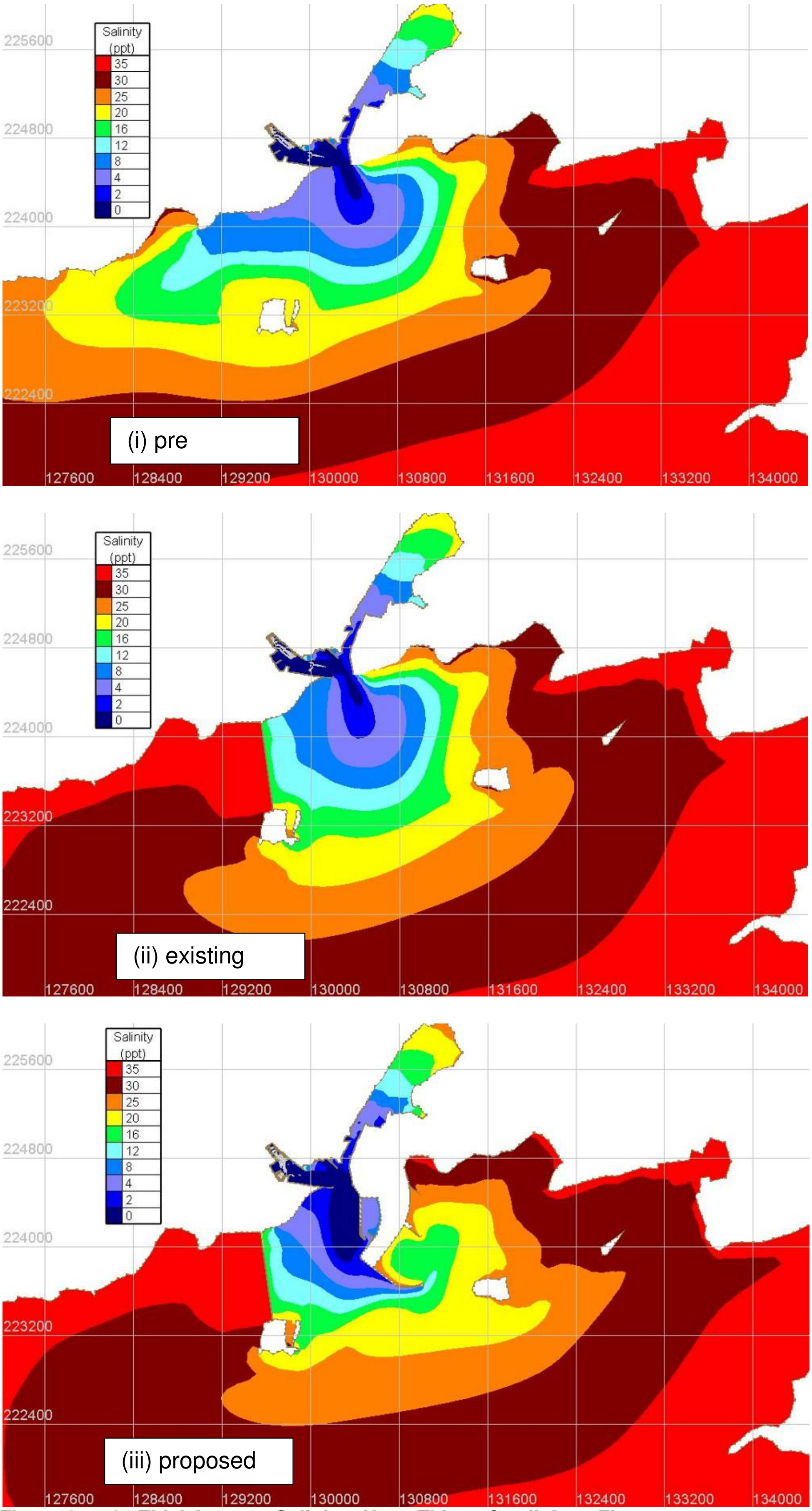


Figure 8.4.121 Tidal Average Salinity –Neap Tides –Corrib Low Flow

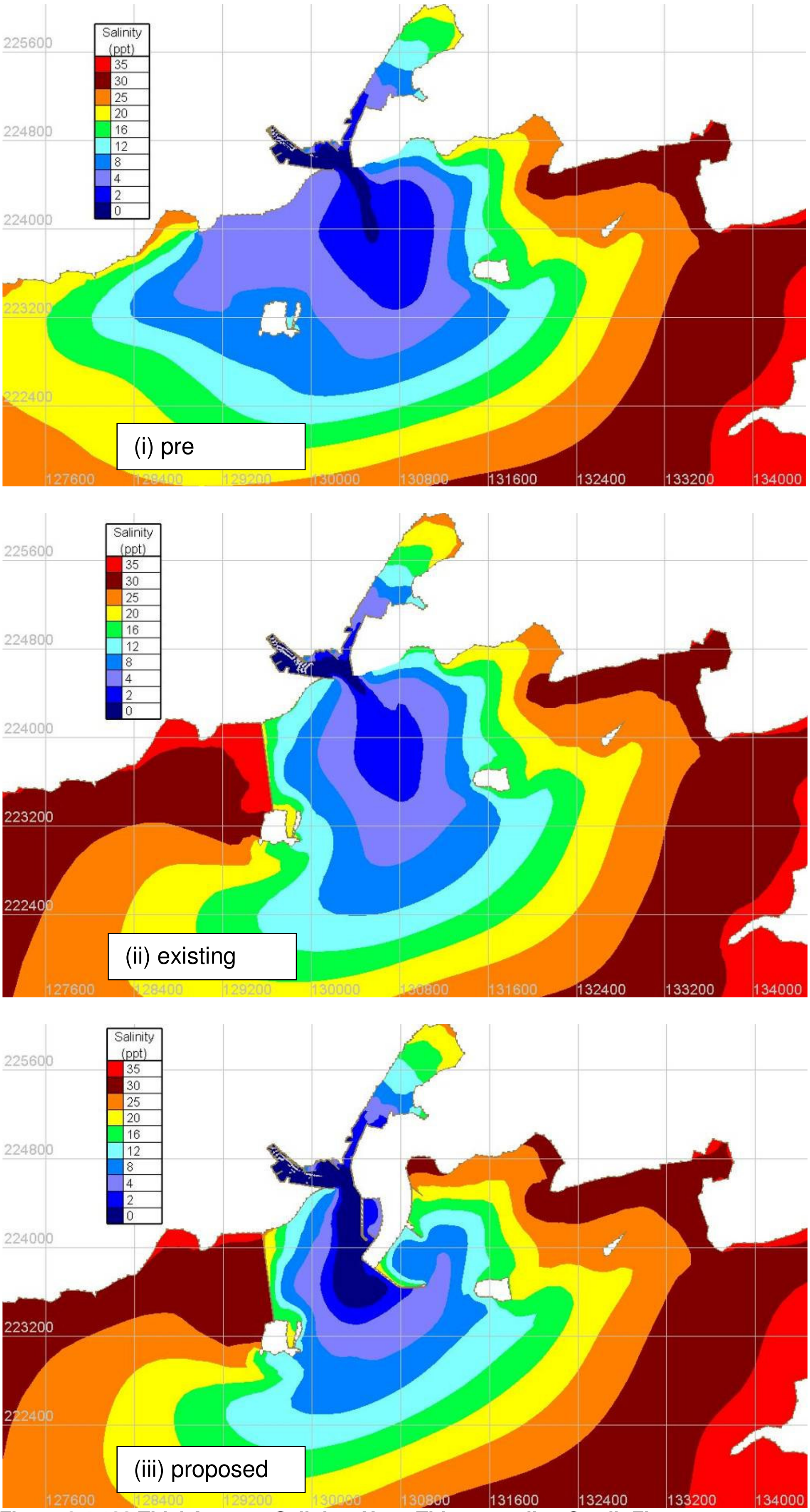


Figure 8.4.122 Tidal Average Salinity –Neap Tides – median Corrib Flow

8.4.6 Wave Climate Prediction

8.4.6.1 Introduction

A significant source of flood risk to the port and surrounding lands and urban shoreline areas is flooding by wave overtopping combined with highwater tides. In particular the protecting breakwaters proposed for the harbour extension are dependent on accurate prediction of design waves so as to set the height and location of the breakwater/quay wall. To this aim wave climate modelling of Galway Bay westward beyond the Aran Islands was carried out to predict the design waves at the Harbour and surrounding shoreline. To derive wave heights in the Proposed harbour area and at all other relevant locations of interest within the outer harbour area (*i.e.* between Mutton Island, Hare Island and the shore), two wave models have been used; a spectral wave model TOMAWAC used to transform deep water waves to nearshore waves and a harbour agitation model ARTEMIS suitable to studying wave disturbance within enclosed bays and harbour areas.

8.4.6.2 Methodology

Two wave models from the TELEMAC hydraulic computational suite of hydrodynamic software were used to assess and predict the wave climate at the new Harbour and along the adjacent shoreline. The first model used was TOMAWAC to model the propagation of deepwater waves into inner Galway Bay. A second more refined model ARTEMIS was used to model the proposed port area, its sea defences and to assess the effect of the new port extension on the local wave climate.

TOMAWAC is a third generation spectral wave model representing the generation of waves due to winds and offshore climates and propagation of these waves into shallow waters.

The following energy dissipation, transfer and propagation processes are modeled by TOMAWAC using an unstructured finite element mesh.

<u>Dissipation processes</u>

- white capping dissipation or wave breaking, due to an excessive wave steepness during wave generation and propagation;
- bottom friction-induced dissipation, mainly occurring in shallow water (bottom grain size distribution, ripples, percolation);
- dissipation through bathymetric breaking. As the waves come near the coast, they swell due to shoaling until they break when they become too steep; and
- dissipation through wave blocking due to strong opposing currents.

<u>Energy transfer processes:</u>

- non-linear resonant quadruplet interactions, which is the exchange process prevailing at great depths; and
- non-linear triad interactions, which become the prevailing process at small depths.

<u>Wave propagation-related processes:</u>

- wave propagation due to the wave group velocity and, in this case, to the velocity of the medium in which it propagates (sea currents);
- depth-induced refraction which, at small depths, modifies the directions of the wave-ray and then implies an energy transfer over the propagation directions;
- shoaling: wave height variation process as the water depth decreases, due to the reduced wavelength and variation of energy propagation velocity;
- current-induced refraction which also causes a deviation of the wave-ray and an energy transfer over the propagation directions; and

- interactions with unsteady currents, inducing frequency transfers (e.g. as regards tidal seas).

Model limitations
Due to model solution structure the following important physical processes are not addressed by the TOMAWAC wave Spectral model:

- diffraction by a coastal structure (breakwater, pier, etc.) or a shoal, resulting in an energy transfer towards the shadow areas beyond the obstacles blocking the wave propagation;
- reflection (partial or total) from a structure or a pronounced depth irregularity; and
- Unable to include Drying/mudflat areas

The limitations of the TOMAWAC Model in respect to diffraction and reflection were overcome by using the harbor agitation model ARTEMIS in the vicinity of the subject development area.

ARTEMIS solves the modified Elliptic Mild Slope Equation (EMSE) for wave propagation. It can be applied for the computation of agitation, resonance and seiching in harbours. It may also be used to calculate the wave field under combined refraction-diffraction and reflection effects in small bays.

ARTEMIS is used in various situations for harbour design and coastal hydraulics studies considering small domains for typical wave characteristics (a few kilometres) or larger ones for resonance computations (large periods). Wave deformation including refraction, diffraction, reflection and energy dissipation (wave breaking and bottom friction) processes is modelled. It is therefore applicable to estuarine and coastal engineering in the frame of the following typical studies:

- Wave agitation in harbours,
- Seiching in coastal channels,
- Shoaling in a small size coastal domain with or without important diffraction effects,
- Wave diffraction behind a dike,
- Wave reflection on sea bed features or obstacles (islands, harbour structures).

Random waves are considered in ARTEMIS as being a superimposition of several monochromatic waves of different periods, which are randomly out of phase with one another. The real wave energy is the sum of the energies of the constituent monochromatic waves.

8.4.6.3 Wind data

There is no absolute maximum wind speed at a given location, as it is always possible that a stronger wind may occur in the future. The most commonly used wind for wave climate studies is a 50-year return period wind. This represents the steady wind speed that is likely to be exceeded once in 50 years and so it has been used for this study.

Wind data were obtained from the Meteorological Office from the Belmullet monitoring station in Co. Mayo. This is the closest monitoring station to Galway Bay. The data consist of a series of maximum daily wind speeds and directions recorded over the stated period. The wind data for each year were segregated into 30° sectors and a 50-year wind speed was calculated for each direction category using the well-documented Gumbel (EV1) Distribution Method (Linacre, 1992). Table 8.4.15 lists the 50-year wind speeds calculated for each direction category.

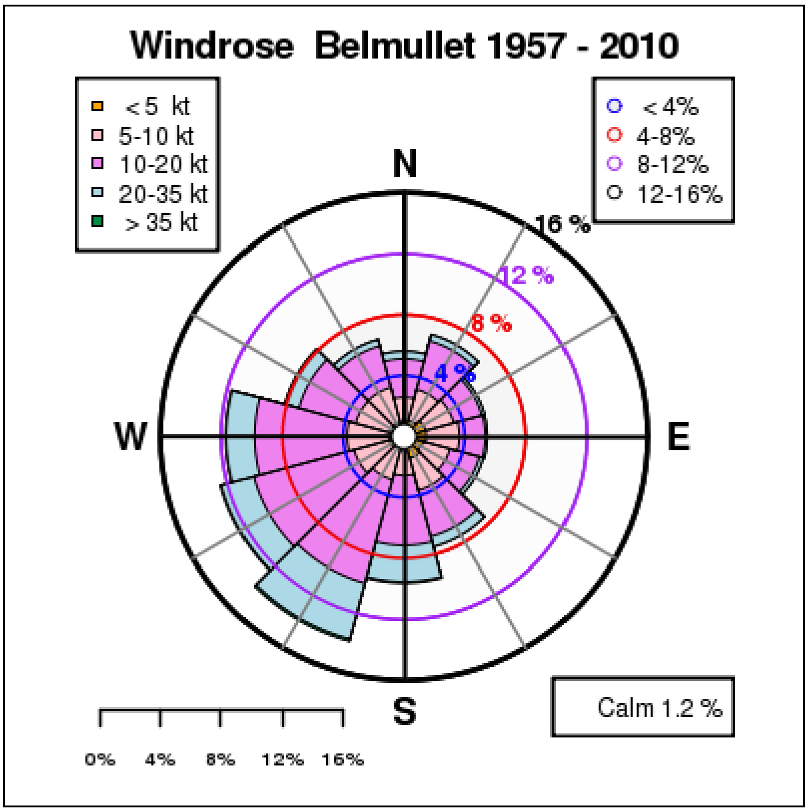
50-year wind speeds		
Wind Direction	Category	50-year Wind Speed [m/s]
350 – 10	N	17.42
20 – 40	N – NE	19.81
50 – 70	E – NE	16.46
80 – 100	E	18.21
110 – 130	E – SE	20.92
140 – 160	S – SE	21.54
170 – 190	S	19.67
200 – 220	S – SW	24.15
230 – 250	W – SW	29.01
260 – 280	W	30.59
290 – 310	W – NW	28.30
320 – 330	N – NW	20.92

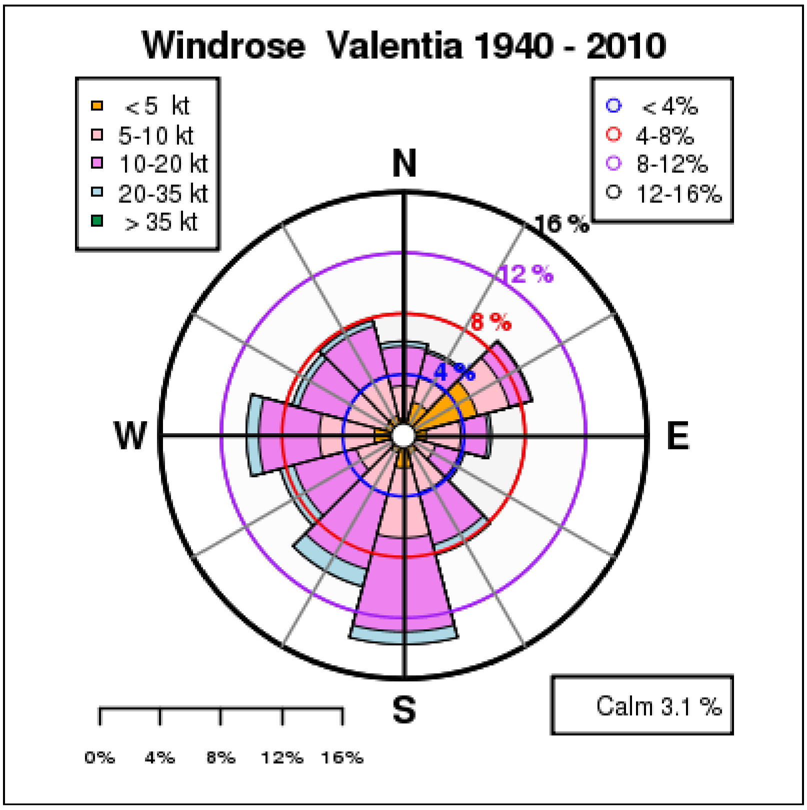
Table 8.4.15 50-year wind speeds calculated for selected wind direction categories

Met Eireann wind roses based on long-term observations for the coastal and estuarine stations of Belmullet, Valentia and Shannon Airport are presented below in Figure 8.4.123. It is clear from these wind roses that the principal directions are from the South to West sector with winds from the easterly sector considerably less frequent. In terms of distance the Shannon wind Met Station is closest to Galway but is considered to be more inland as it is located well up the Shannon Estuary and thus more sheltered than the Galway Bay area.

The Irish Met service provide a contour map of Ireland with 50year 1hour and 10min duration wind speeds, refer to figures 8.4.124 to 8.4.125. For the Inner Galway Bay area the 10 minute mean wind speed with return period of 50years is 30.5 to 31.0m/s (Met Eireann, 2005). The 1 hour mean wind speed with return period of 50years is 28m/s. In this study a wind speed of 30m/s is used in the local wave analysis.

The wind data in combination with the Shore Protection Manual (SPM) (1984) method were used to define the deepwater non-fetch limited significant wave heights and periods at the open sea boundaries west of the Aran Islands propagating onshore from the southwest and westerly sectors. These wind field data were also used to determine the magnitude of the local shallow water (fetch limited) wave climate using the Shore Protection Manual (SPM) method. This wind field information was also specified as the local wind shear generating force in the TOMAWAC Spectral model which allows the additional propagation and growth of the wave as it travels inshore.





Note 1kt (knot) = 0.514m/s

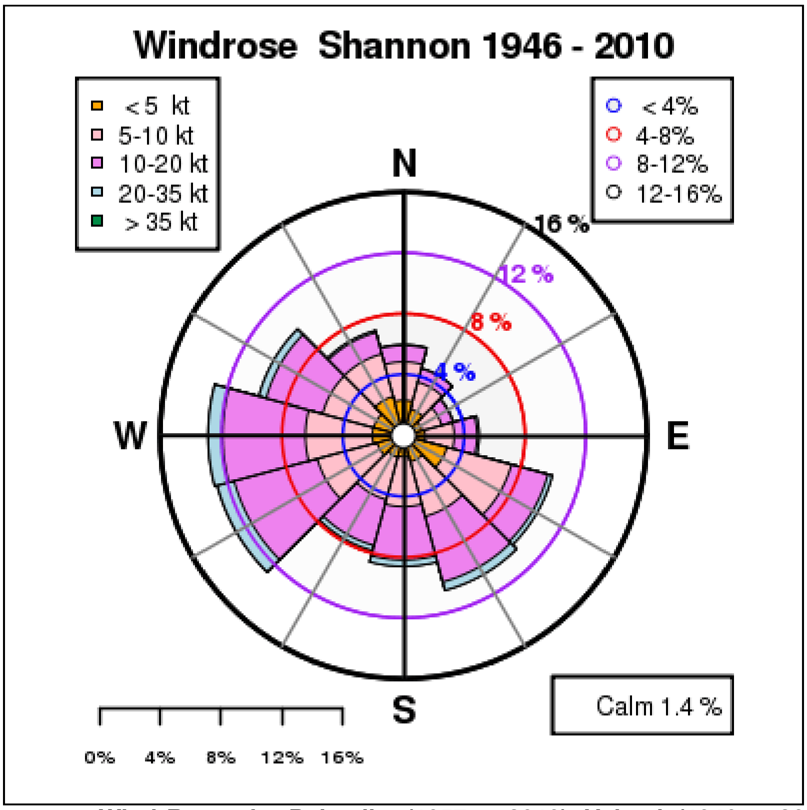


Figure 8.4.123 Long-term Wind Roses for Belmullet (1957 to 2010), Valentia(1940 to 2010) and Shannon Airport (1946 to 2010)

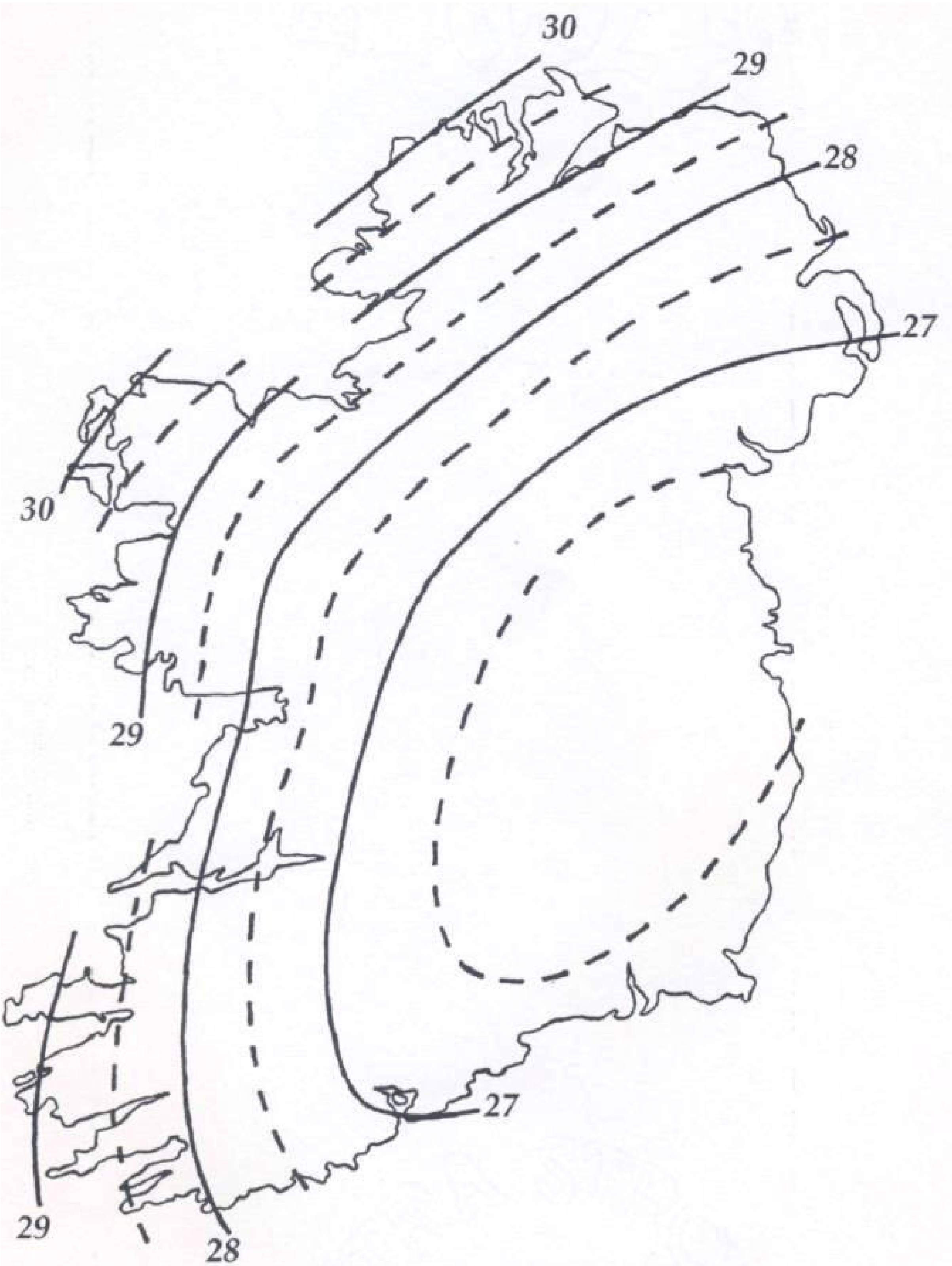


Figure 8.4.124 Met Eireann 1 hour mean wind speed of 50year return period

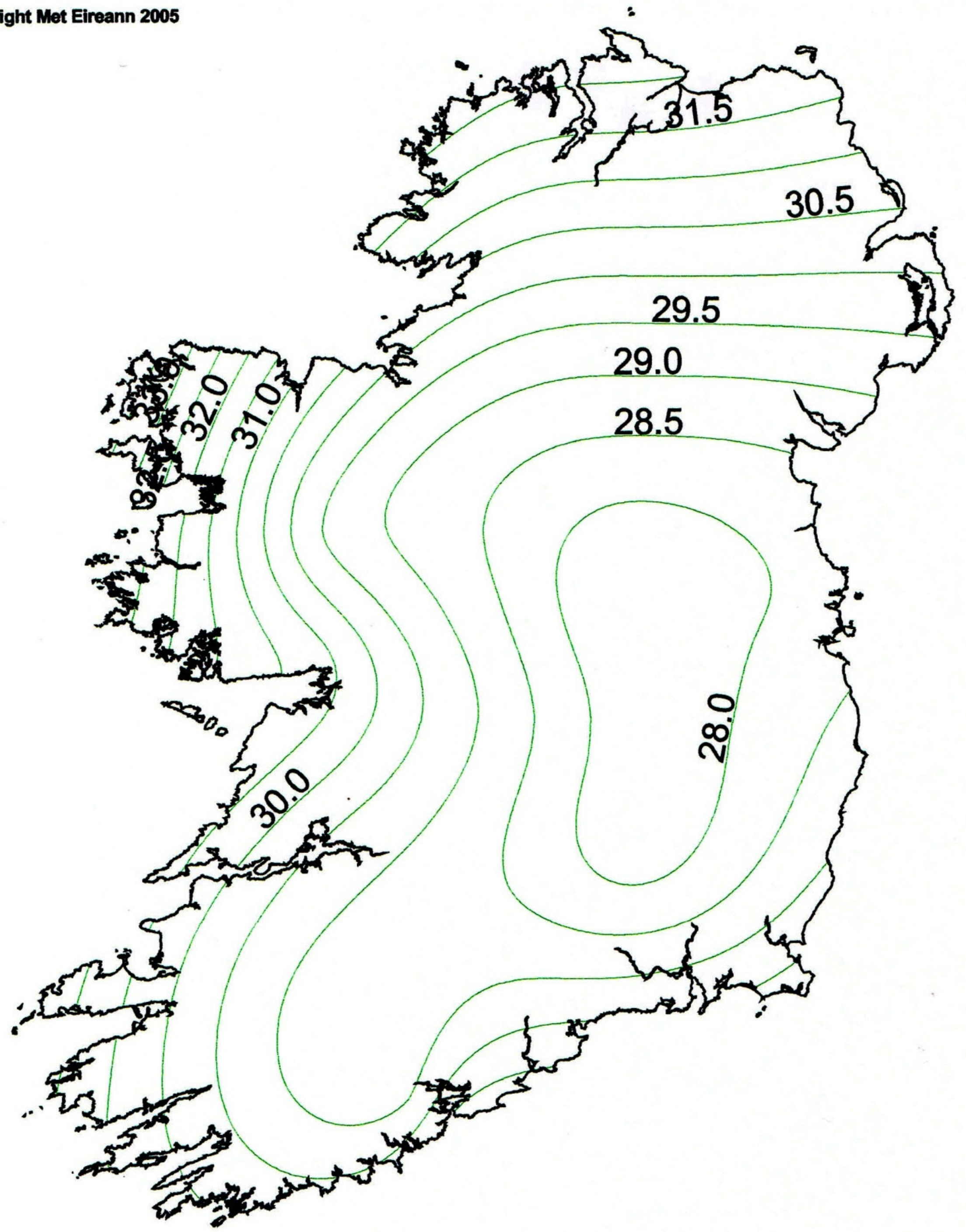


Figure 8.4.125 10minute mean wind speed of 50year return period (Met Eireann 2005)

8.4.6.4 TOMAWAC Model Simulations

8.4.6.4.1 Introduction

The boundary conditions along the seaward extent of the TOMAWAC Spectral model (west and south sea boundaries, refer to Figure 8.4.126) model were specified in terms of the significant height and period of the appropriate deepwater wave. These conditions were then used as the forcing function for the model and significant wave heights were predicted for each finite element nodal point within the domain. Deepwater wave propagations from the Southwest, West-southwest and West were examined (Table 8.4.16).

Deepwater wave conditions at model Open sea boundary			
Direction	**50-yr Wind Speed [m/s]**	**Deepwater Wave Height [m]**	**Period [sec]**
Southwest	26	15.	15.4
West-southwest	29	17.	16.4
West	29	17	16.4

Table 8.4.16 Deepwater wave conditions at model deepwater boundary

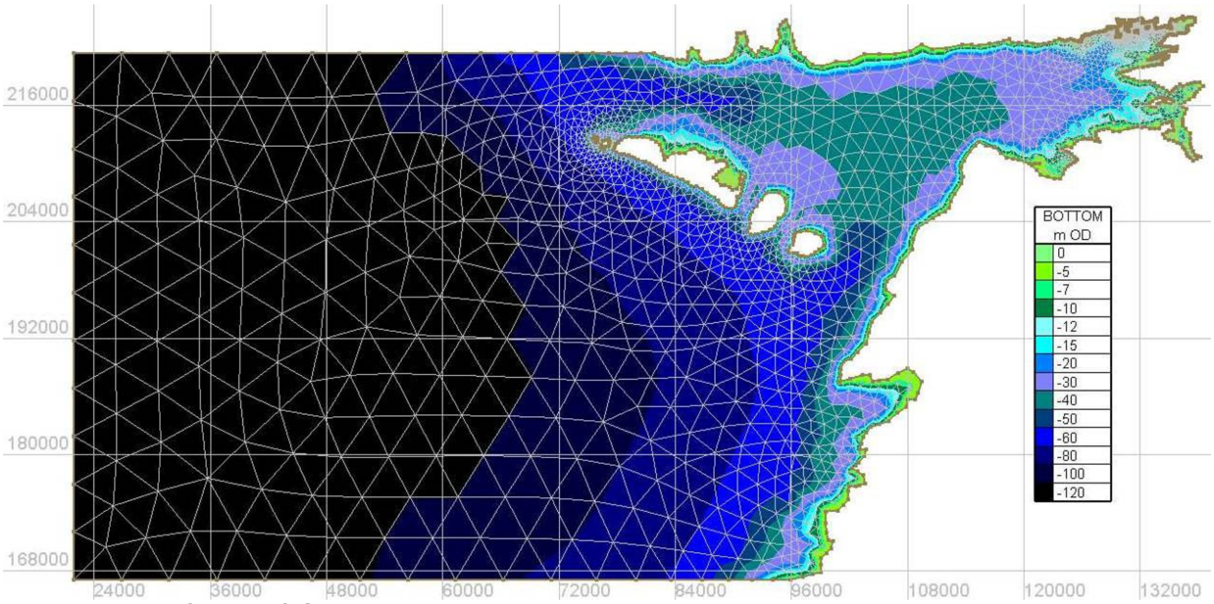


Figure 8.4.126 TOMAWAC Spectral Wave Model Domain

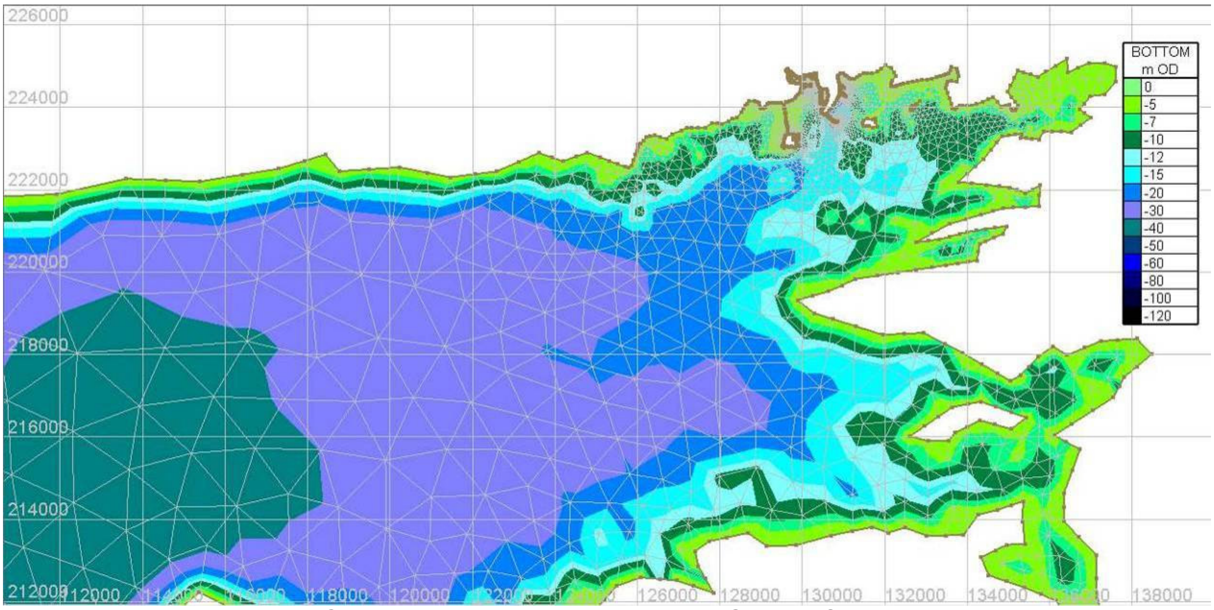


Figure 8.4.127 View of Inner Galway Bay as represented in the TOMAWAC Model

8.4.6.4.2 <u>Wave Climate Results</u>

A 50-year steady wind blowing from a south, southwest, west-southwest and west directions produces steady wind speeds of c. 26 to 31 m/s. The fetch length in these directions is considered to be unlimited with sea depths in excess of 100 m. These conditions result in significant wave heights varying from 15 to 18 m and wave periods from 15 to 17 seconds. In the analysis for the harbour extension, a significant wave height of 20 m with significant period of 17seconds and local wind speeds of 30 m/s was specified in the model for all offshore directions so as to ensure a degree of conservatism in predicting the 50-year wave climate.

Figures 8.4.128 to 8.4.131 show contoured plots of the significant wave heights predicted by the TOMAWAC spectral wave model as a result of the deepwater waves propagating inshore. The maximum value of the significant wave height that reaches inner Galway Bay just to the southwest of Mutton island (wave Input point for ARTEMIS wave agitation model) was found to be slightly less than 4 m (3.77 m on Southwest and 3.3 m for a west southwest wind and offshore condition). For westerly winds the significant wave height at this location is 2.9 m. The mean wave direction is typically 58 to 63 degrees for all of the critical off shore wave directions, (southwest, west-southwest and west). The mean and peak periods in the inner bay area are 8 to 8.5 and 10.2 to 10.3 seconds. Southerly and north-westerly offshore waves have very limited effect on the inner Galway Bay area. It is clear that the Aran Islands and the reducing sea depth east of the islands provide crucial protection to the inner Galway Bay area. This is primarily due to the position of the Aran Islands at the entrance to Galway Bay which act as a very effective breakwater for deepwater waves entering outer Galway Bay at particular angles.

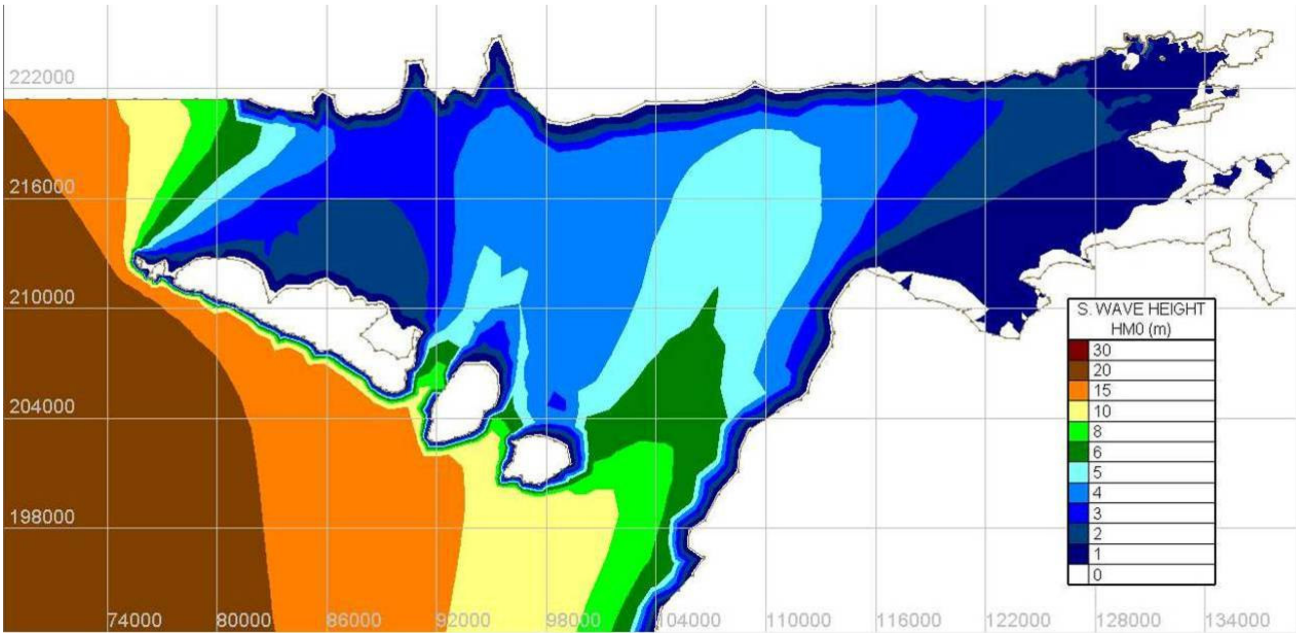


Figure 8.4.128 Wave climate under 50-year southerly wind conditions

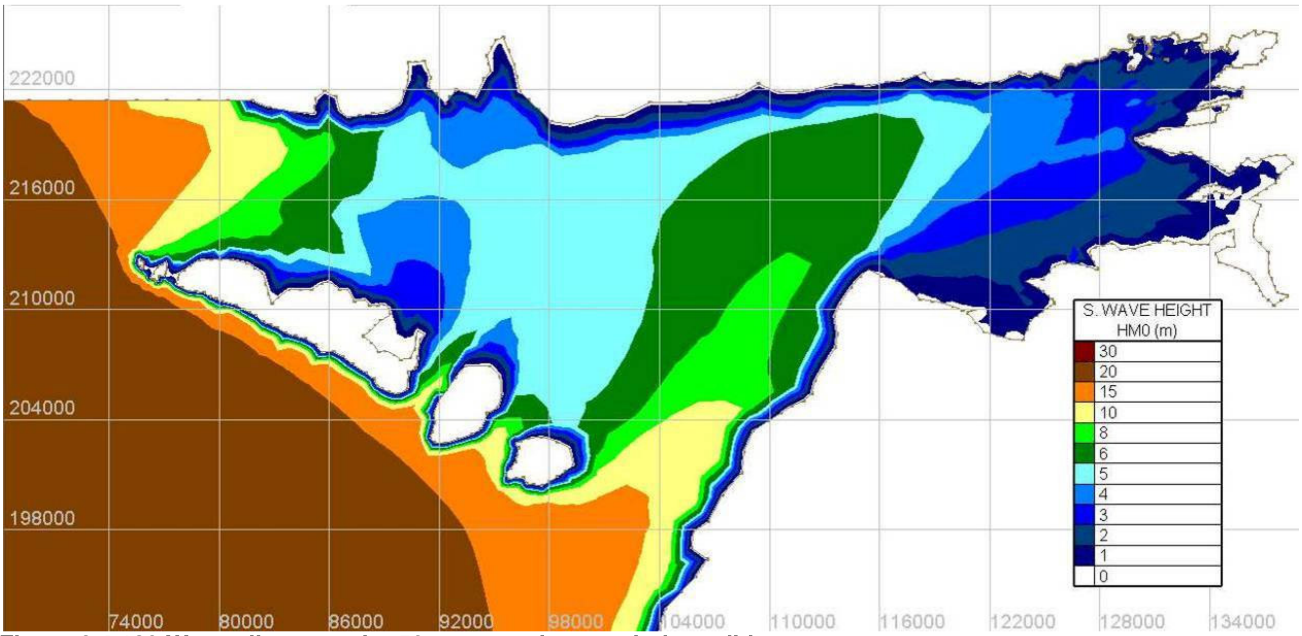


Figure 8.4.129 Wave climate under 50-year southwest wind conditions

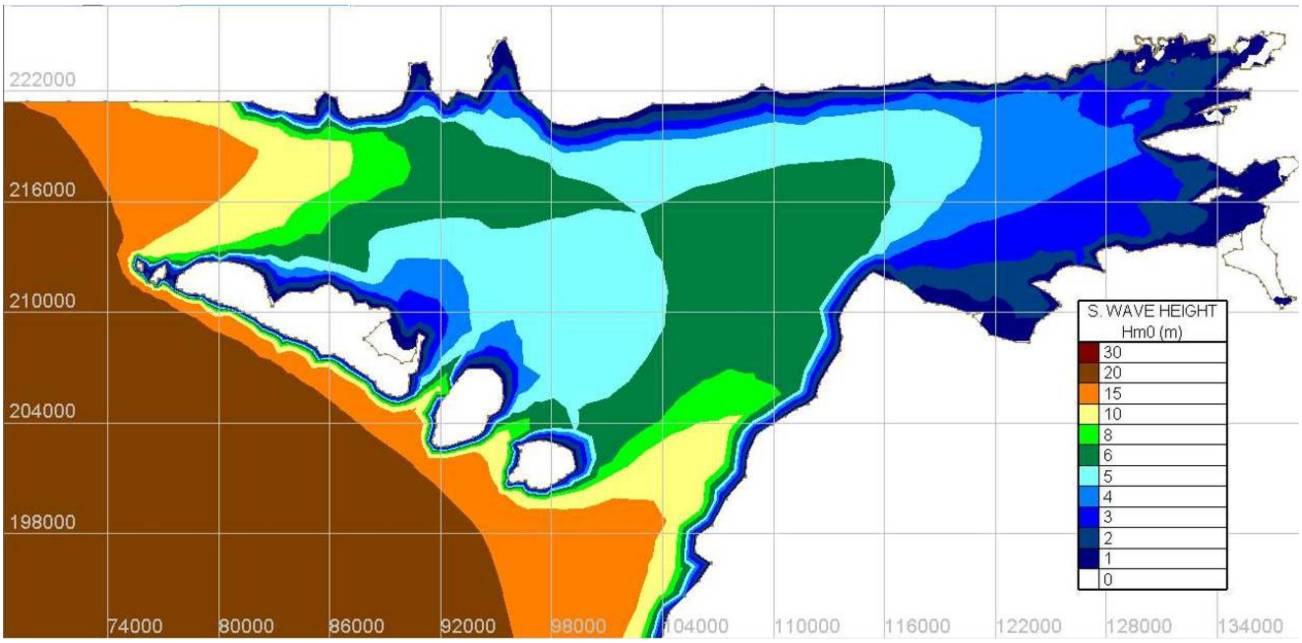


Figure 8.4.130 Wave climate under 50year west southwest wind conditions

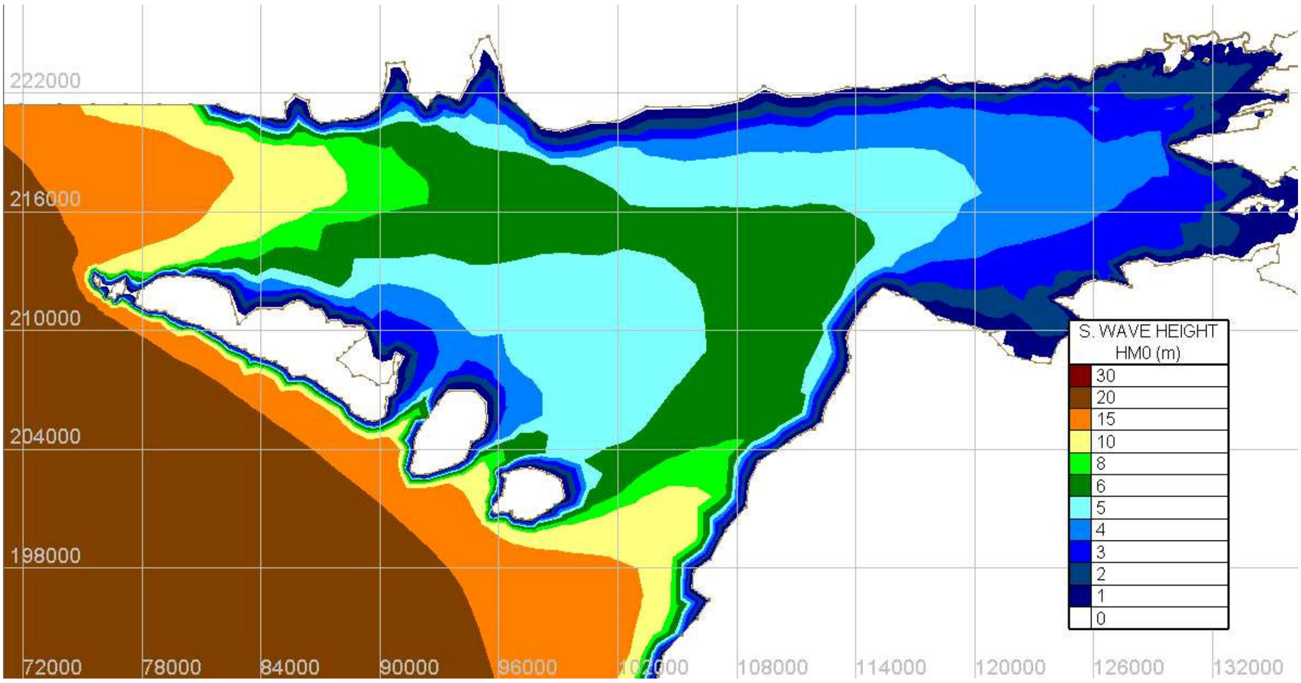


Figure 8.4.131 Wave climate under 50year Westerly wind conditions

8.4.6.5 ARTEMIS Model Simulations

8.4.6.5.1 Introduction

Because The ARTEMIS model software solves directly the modified Elliptic Mild Slope Equation (EMSE) for wave propagation, a very refined meshing of the order of metres is required particularly when modelling the short duration/high frequency waves (generated by winds over local shallow fetch lengths). A single model was developed with an element spacing of 3m to model the wave field for long period Atlantic storms and the shorter period local fetch storms both for existing and proposed harbour cases. The ARTEMIS Model was run in random wave mode both in respect to the period about the significant period and direction about the principal direction.

Simulations were carried out for the various sectors from West to East to assess the potential impact of the proposed harbour development on wave climate and quantify the design conditions for the proposed harbour protective breakwaters and quay walls.

The reflection coefficient used in the ARTEMIS model for the harbour extension quay wall and vertical sheet piled breakwaters was set at 0.8 to 0.9 with the incident /reflective wave direction determined through trial and error by running simulations and outputting incident wave direction. The reflection coefficient for the shoreline area along South Park was specified at 0.25 to 0.5, 0.15 for the Mutton Island Causeway (designed to absorb wave energy) and 0.25 along the Renmore shoreline and Hare and Mutton Islands respectively.

8.4.6.5.2 Design Tide Inputs

The design wave inputs to the Artemis model are presented in Table 8.4.17 as follows:

Design Wave Inputs to ARTEMIS Models		
Direction	**Significant Wave Height Hs [m]**	**Ts [sec]**
East (Local Fetch, 3.3km)	1.21	3.8
East-South-East (Local Fetch, 3.9km)	1.25.	3.9
South-East (Local Fetch, 4.1km)	1.34.	4.0
South-South-East (Local Fetch, 4.9km)	1.41	4.2
South (Local Fetch, 7.4km)	1.52	4.4
South South West (Local Fetch, 8.1km)	1.68	4.75
South-West (Atlantic Storms Deepwater Wave)	3.77	10.3
West South West (Atlantic Storms Deepwater Wave)	3.3	10.2

Table 8.4.17 Design wave inputs to ARTEMIS Models

8.4.6.6 Discussion of Results

The ARTEMIS Model was run for storm waves generated by local fetch from the East, East South East, South East, South South East, South and South South West sectors respectively. Longer period Atlantic waves propagating from the southwest to the West were also examined. All of the above runs were specifically aimed at assessing the protection afforded by the proposed breakwaters in respect to conditions within the mooring areas of the Commercial Harbour and Fisherman’s pier and within the proposed marina area and any other operational areas. The southerly sector was also considered the critical direction for storm waves acting on the proposed Harbour and the vulnerable South Park shoreline area (inside the Mutton Island Causeway) and the mouth of the Corrib Estuary and the existing docks entrance adjacent to Nimmo's Pier.

The simulations show the breakwaters protecting well the harbour and marina areas against the dominant wave directions from the south to the west. The southwesterly deepwater wave simulation represents the design condition for the Commercial Harbour Breakwater (southern Breakwater) with wave heights along the breakwater increasing south-eastward along the breakwater from 1.5 m towards the northwest corner to just less than 4 m at the outer most exposed tip adjacent to Hare Island (refer to Figure 8.4.132 and 8.1.33).

The breakwater protection is not designed to protect the commercial harbour against storm waves propagating locally from the east and southeast with model results predicting 0.7 to 0.8 m waves within part of the commercial harbour for the southeast design storm conditions with the local waves propagating northwestward through the opening between the breakwater and Hare Island. The model predicts waves slightly in excess of 1 m at the south face of the Fisherman's pier for the east-south-east direction. Refer to Figure 8.4.137 to 8.4.139 for southeast to East design wave simulation plots. Hare Island is shown to provide some protection against south-easterly to easterly storms.

A simulation was also carried out assuming the causeway to be completely submerged by 200-year Tide with Sea level Rise (4.635 m O.D. Malin) >1 m water depth and a southwesterly (SW and WSW) deepwater design wave of 3.77 m significant wave height applied. The simulation shows that the Mutton Island Causeway would under these submerged conditions break the storm waves and dissipate energy and thus provide protection to the westerly face of the development even under submerged conditions (refer to Figure 8.4.142).

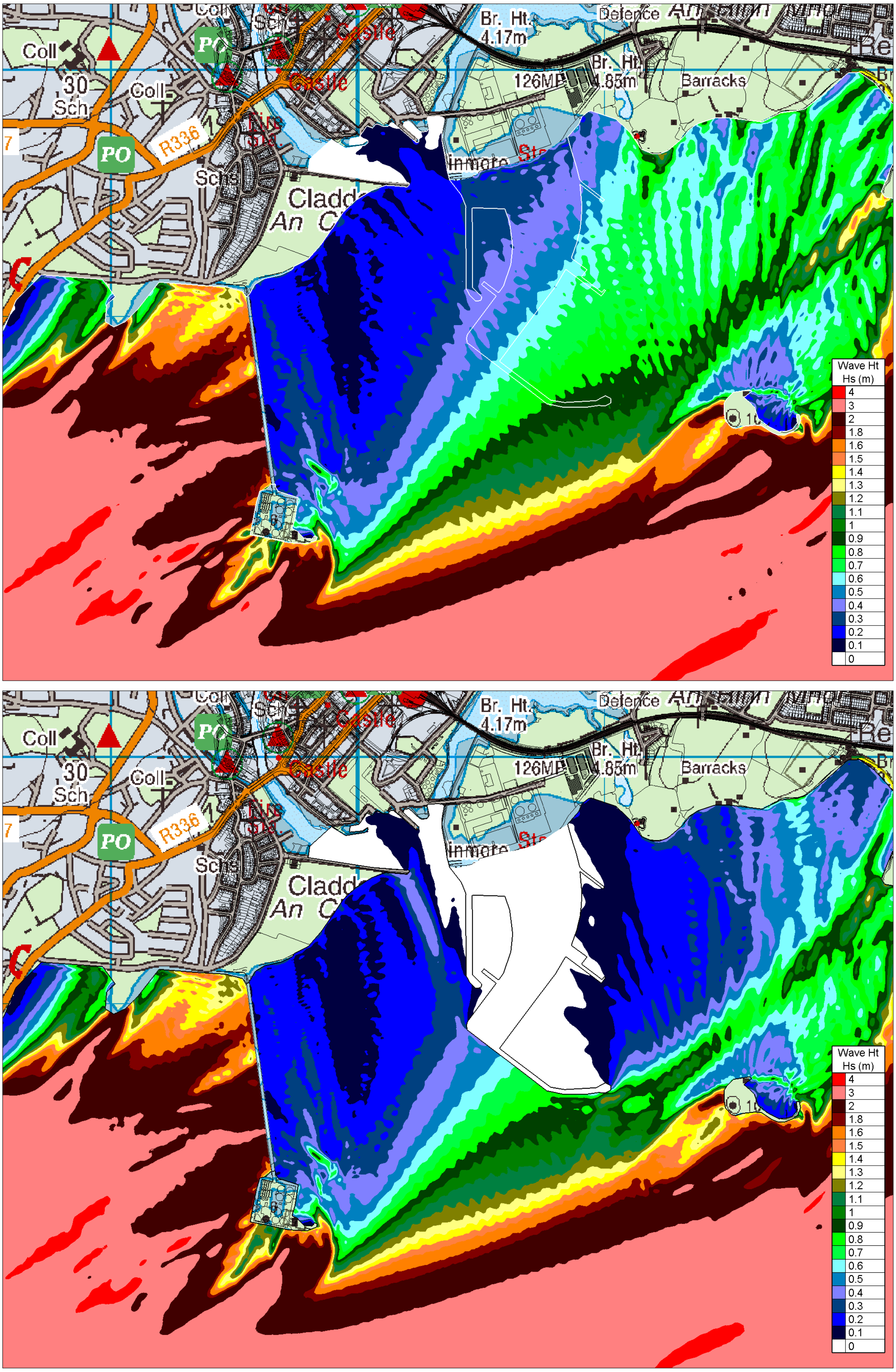


Figure 8.4.132 ARTEMIS Significant Wave Heights for Atlantic Storm from the West-South-West for Existing and Proposed Case

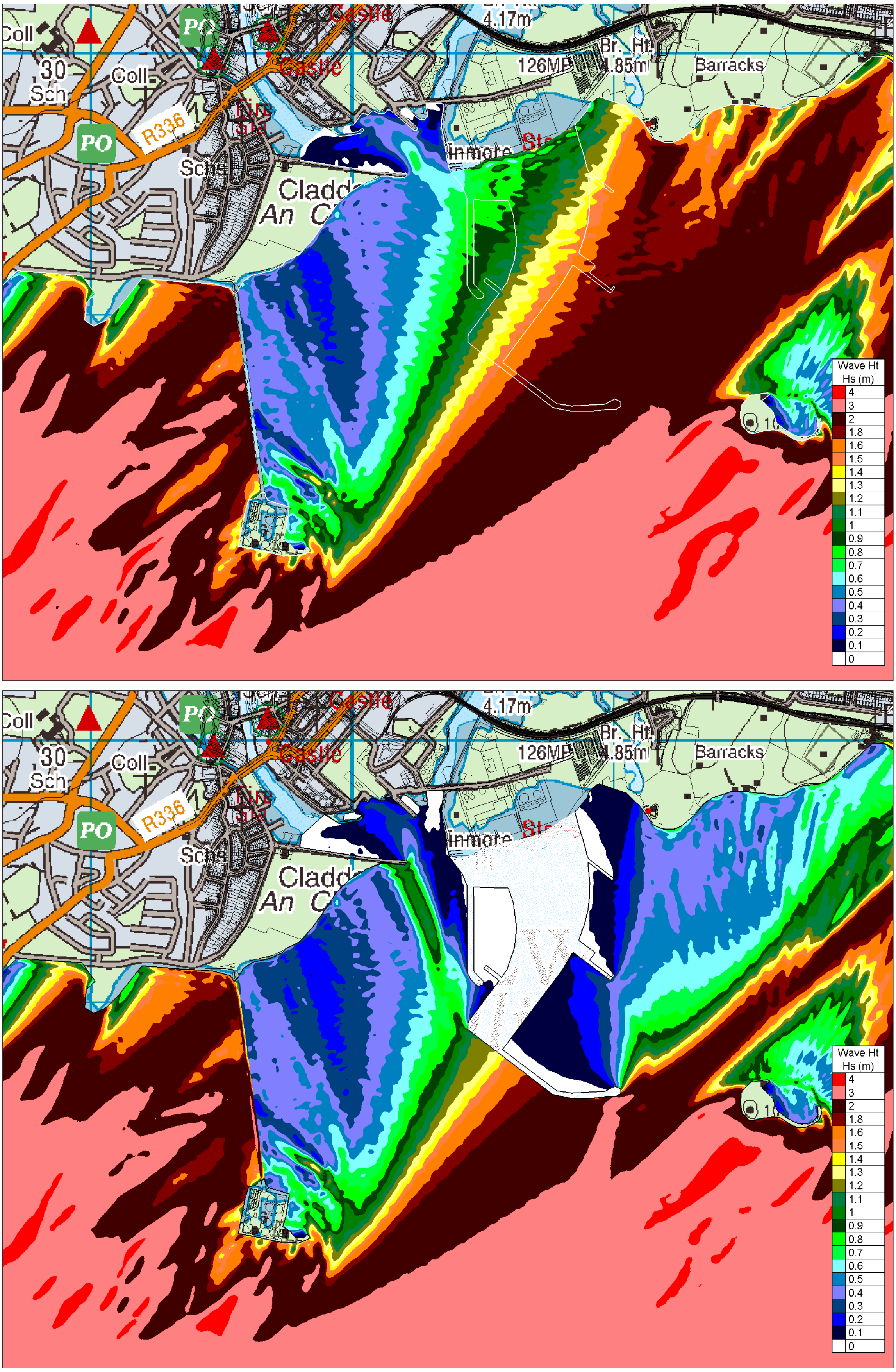


Figure 8.4.133 ARTEMIS Significant Wave Heights for Atlantic Storm from the South-West for Existing and Proposed Case

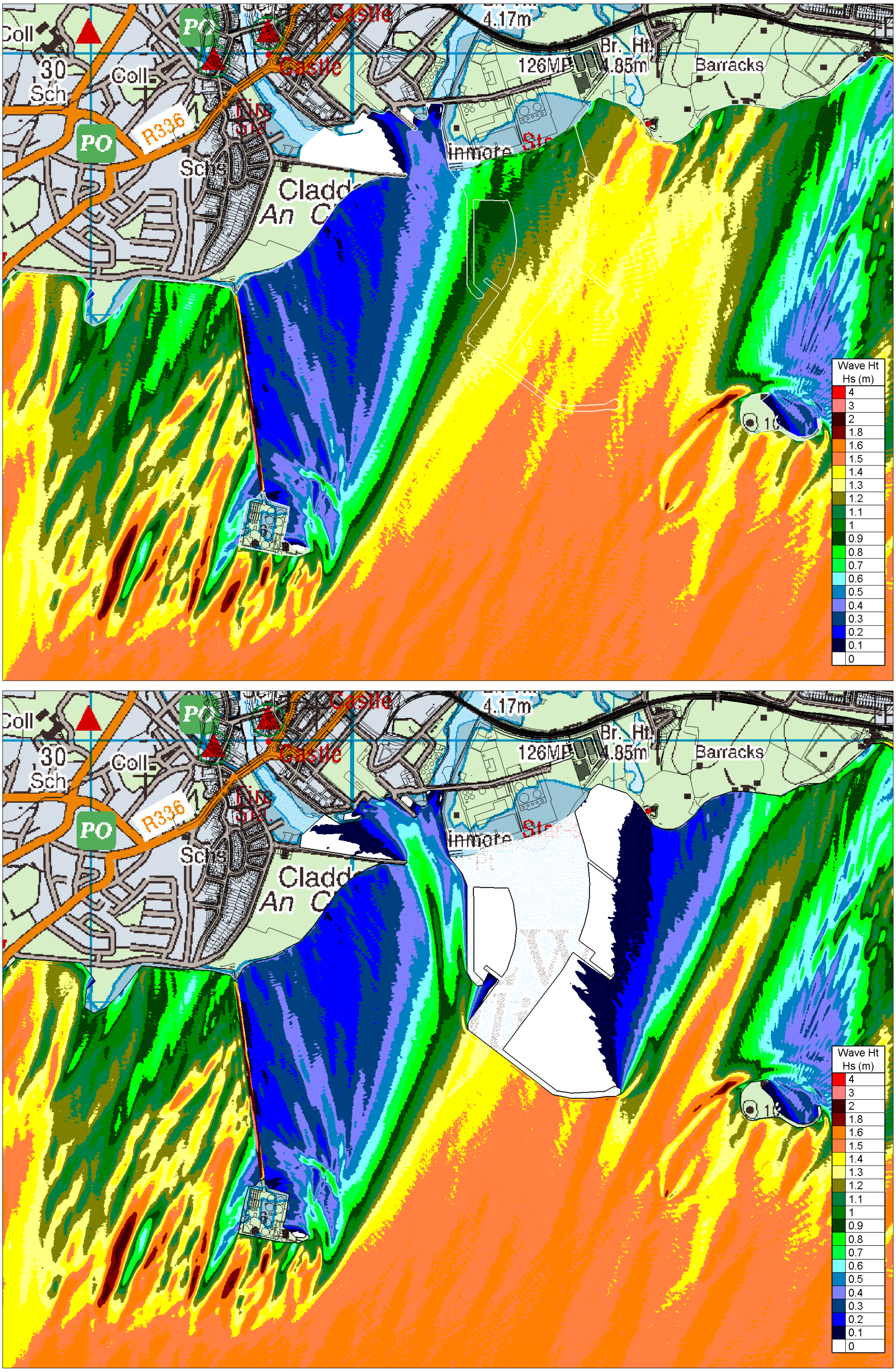


Figure 8.4.134 ARTEMIS Significant Wave Heights for local Design Storm Waves from the South-South-West for Existing and Proposed Case

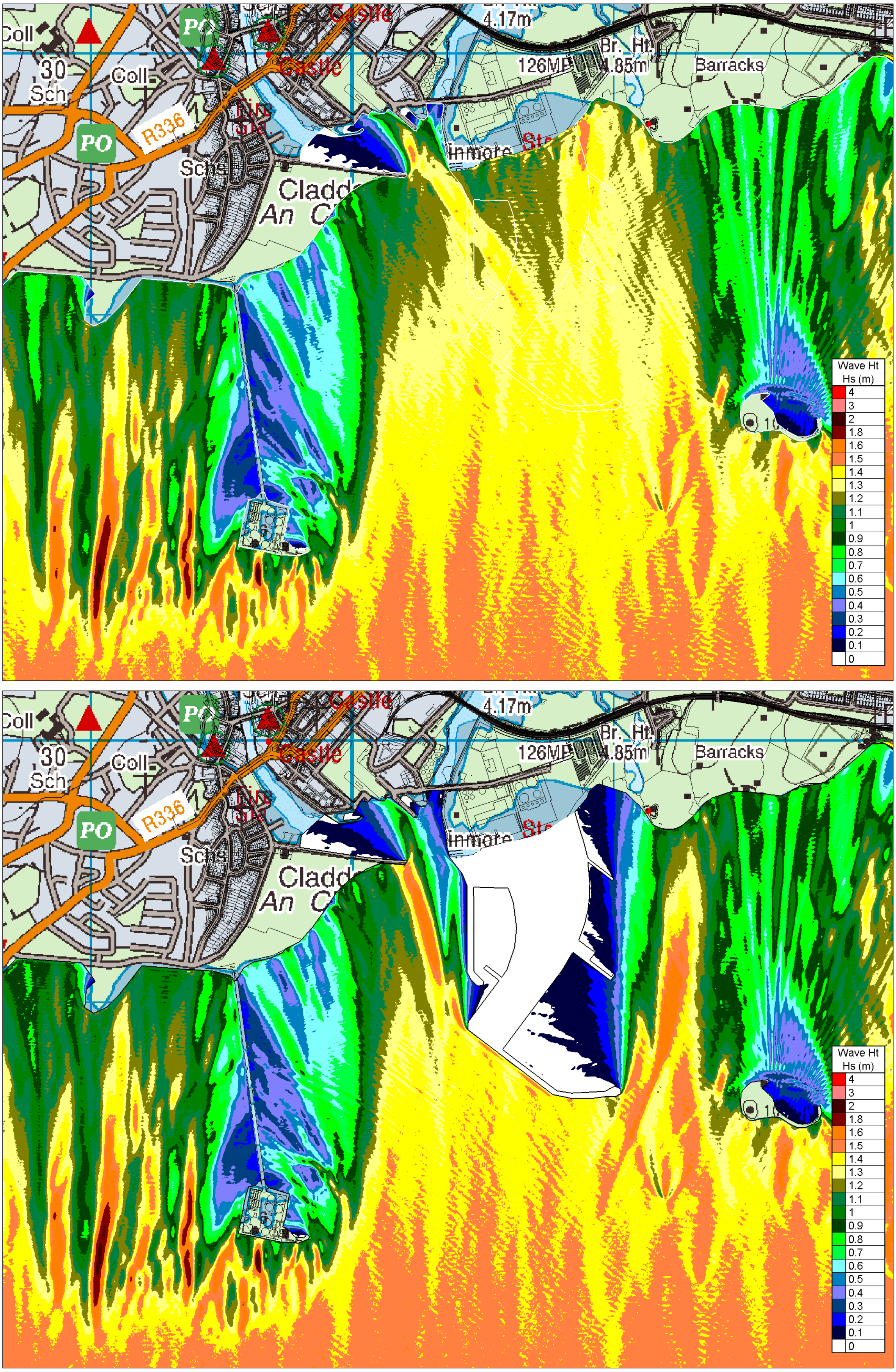
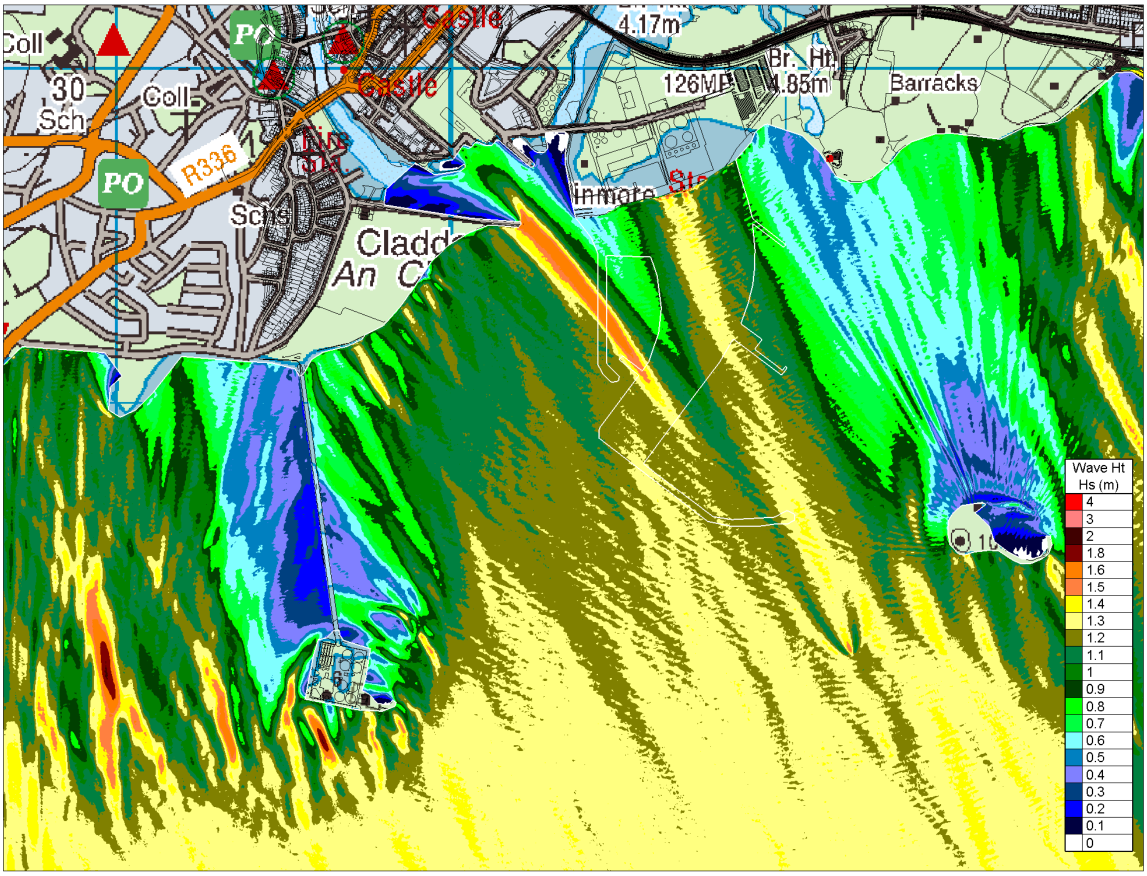


Figure 8.4.135 ARTEMIS Significant Wave Heights for local Design Storm Waves from the South for Existing and Proposed Case



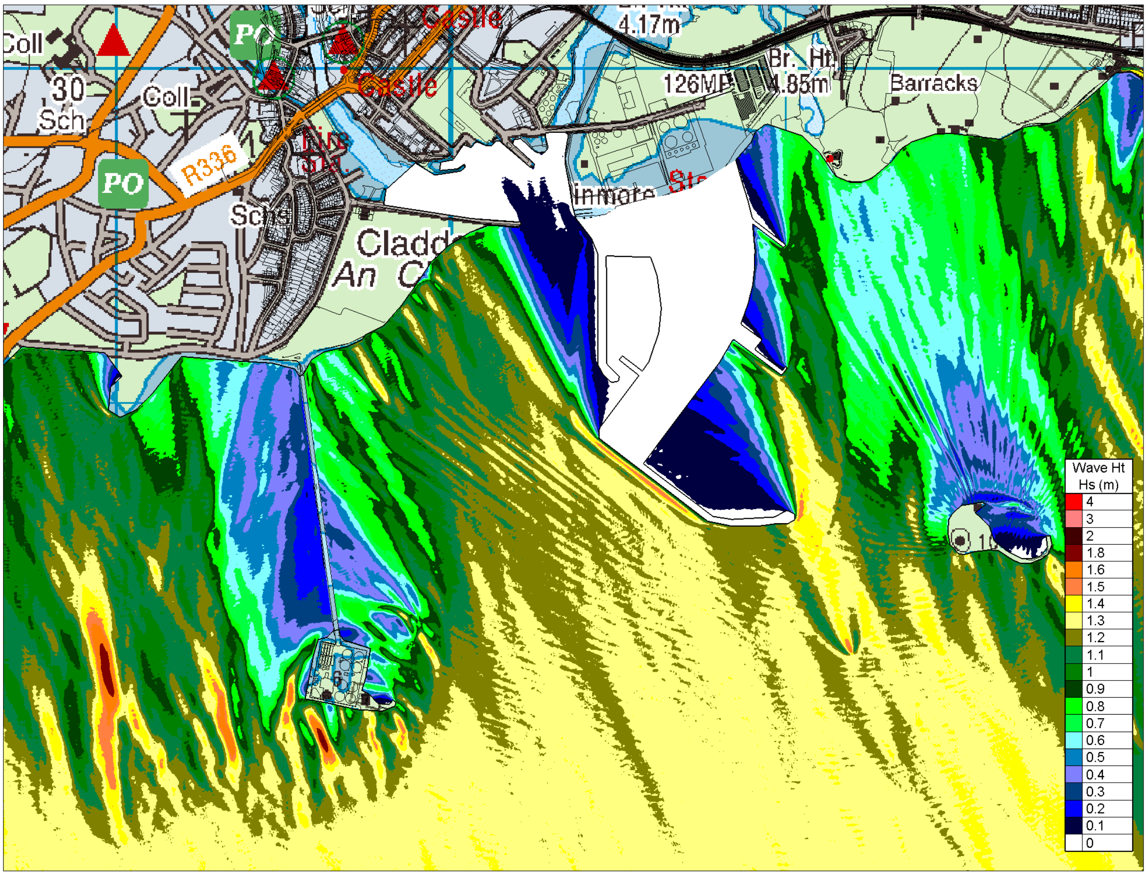


Figure 8.4.136 ARTEMIS Significant Wave Heights for local Design Storm Waves from the South-South-East for Existing and Proposed Case

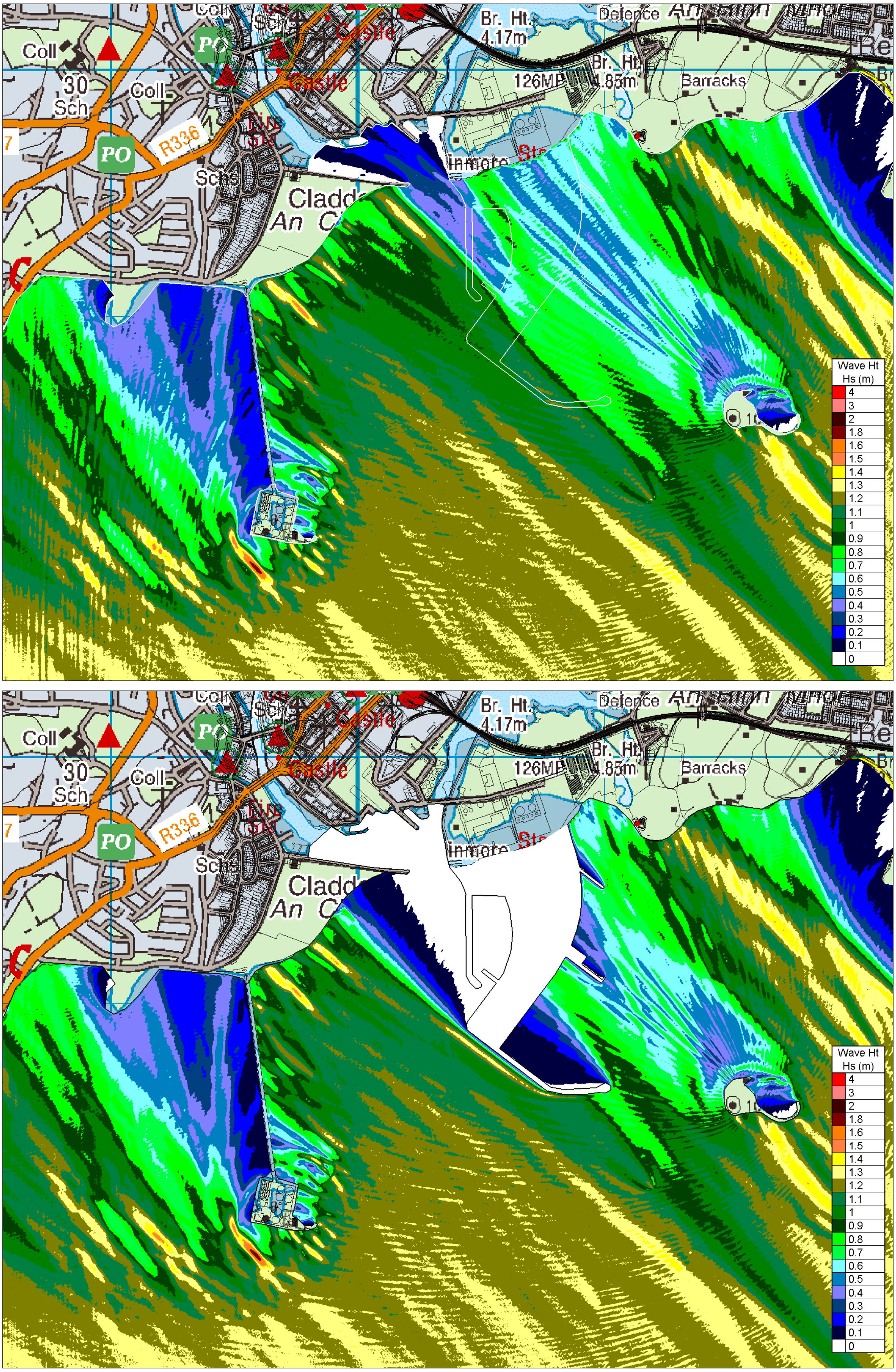


Figure 8.4.137 ARTEMIS Significant Wave Heights for local Design Storm Waves from the South-East for Existing and Proposed Case

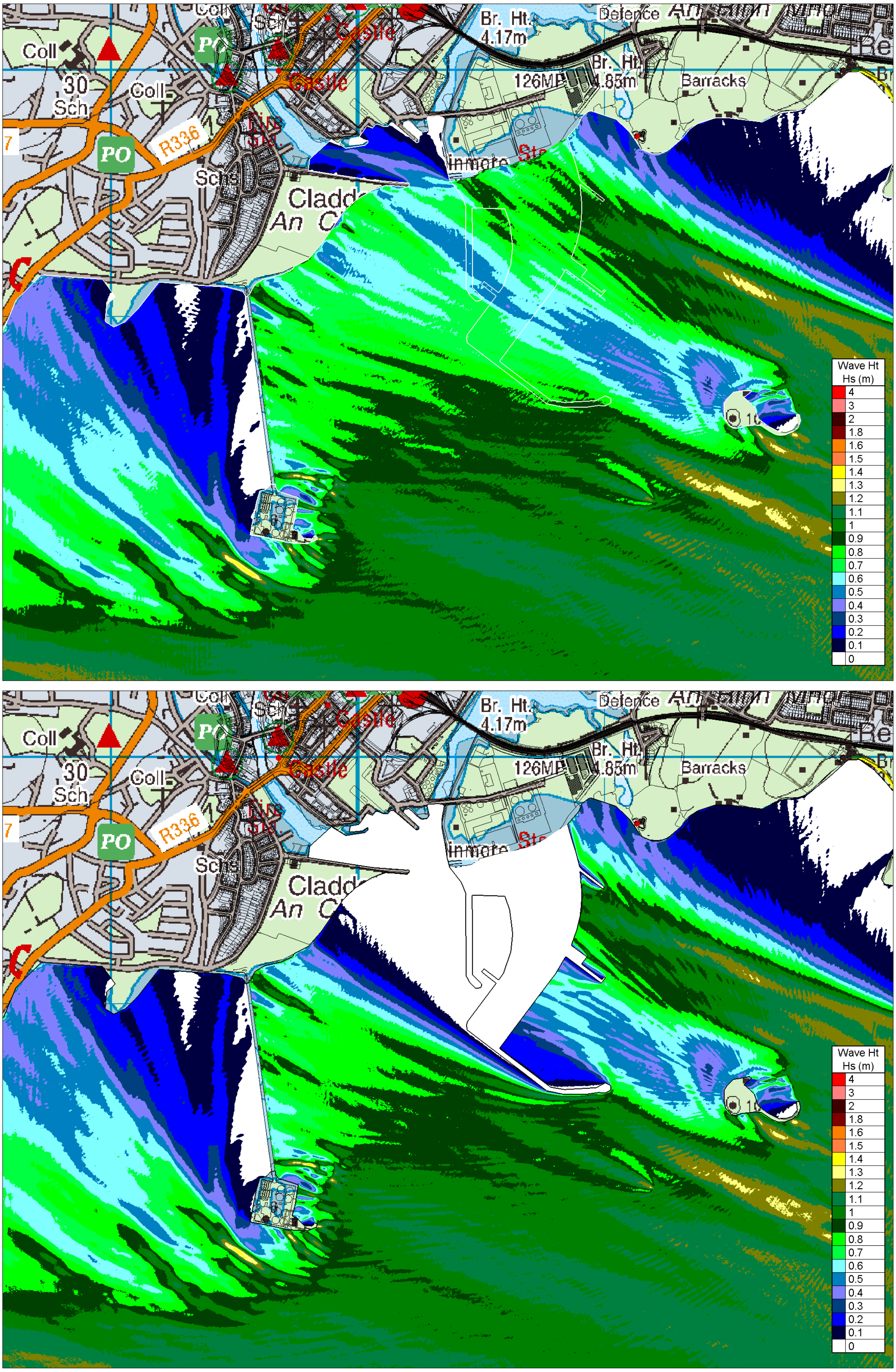
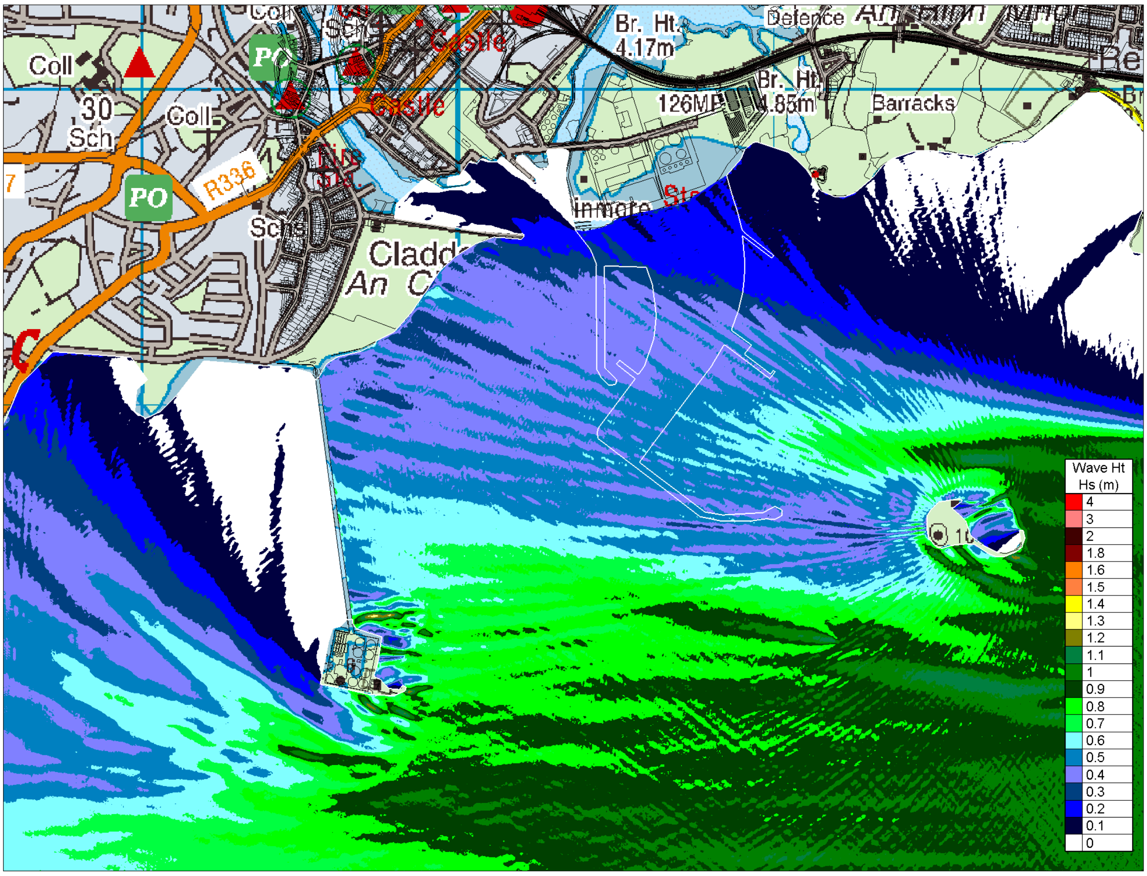


Figure 8.4.138 ARTEMIS Significant Wave Heights for local Design Storm Waves from the East-South-East for Existing and Proposed Case



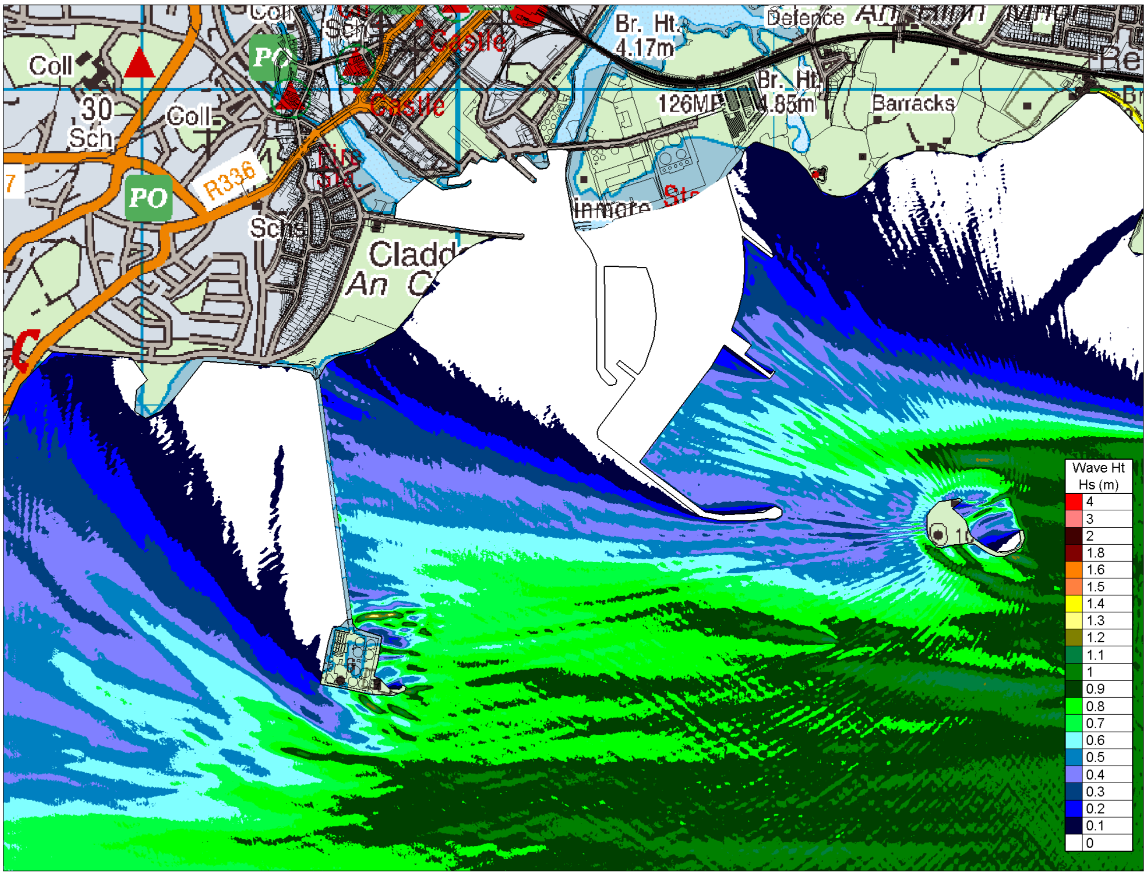


Figure 8.4.139 **ARTEMIS Significant Wave Heights for local Design Storm Waves from the East for Existing and Proposed Case**

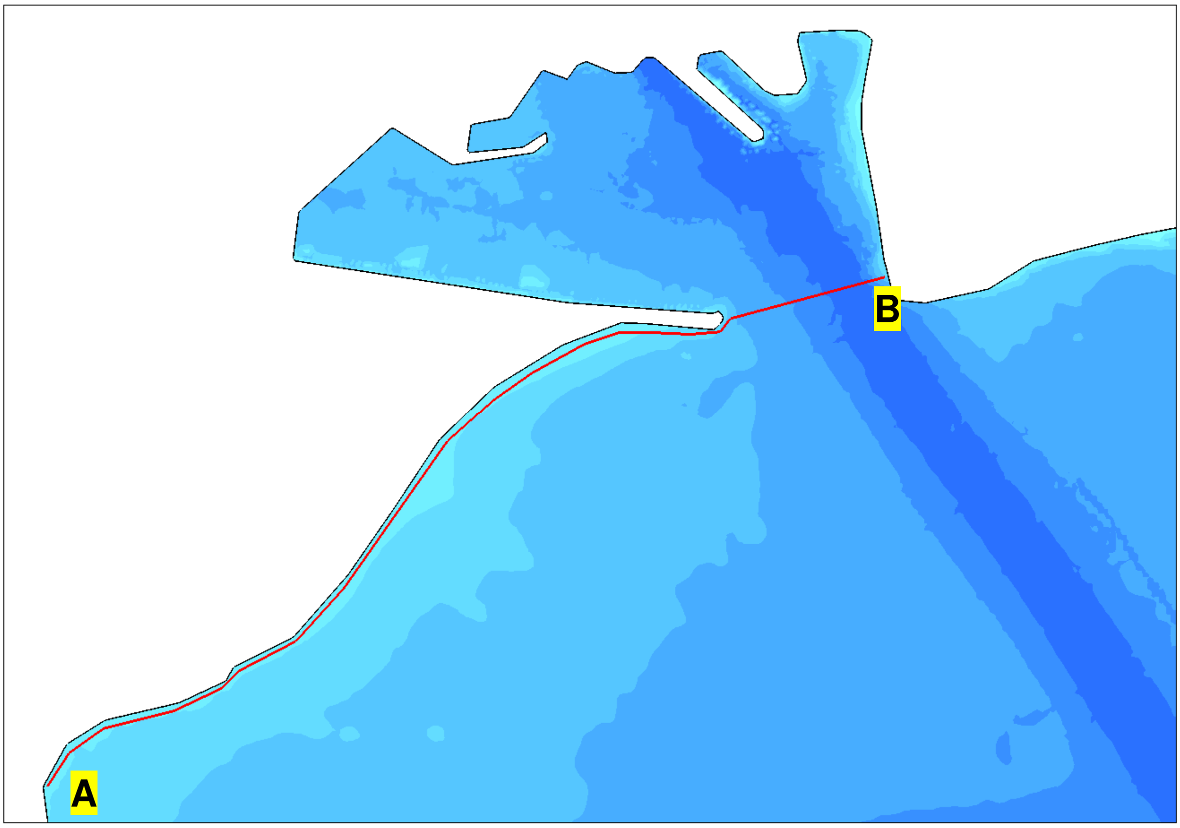


Figure 8.4.140 Shoreline Section A-B along Southpark, Nimmo's Pier and entrance to GalwayDocks / Claddagh Basin.

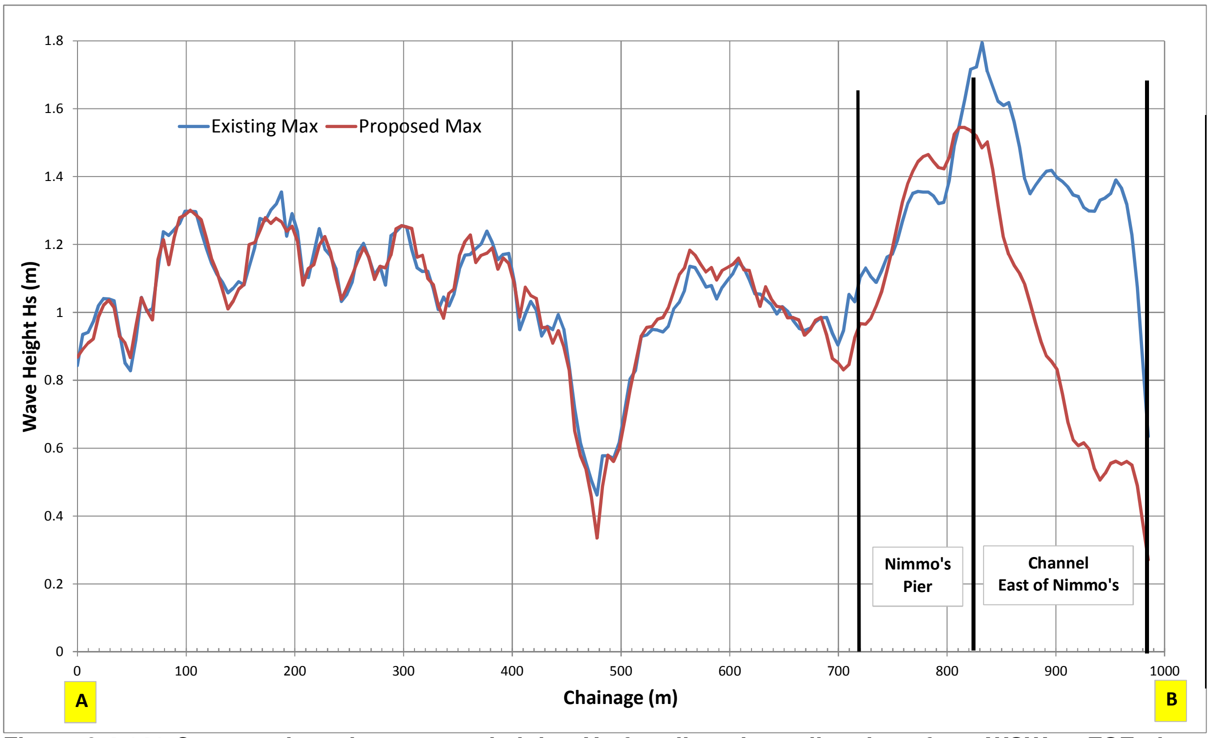
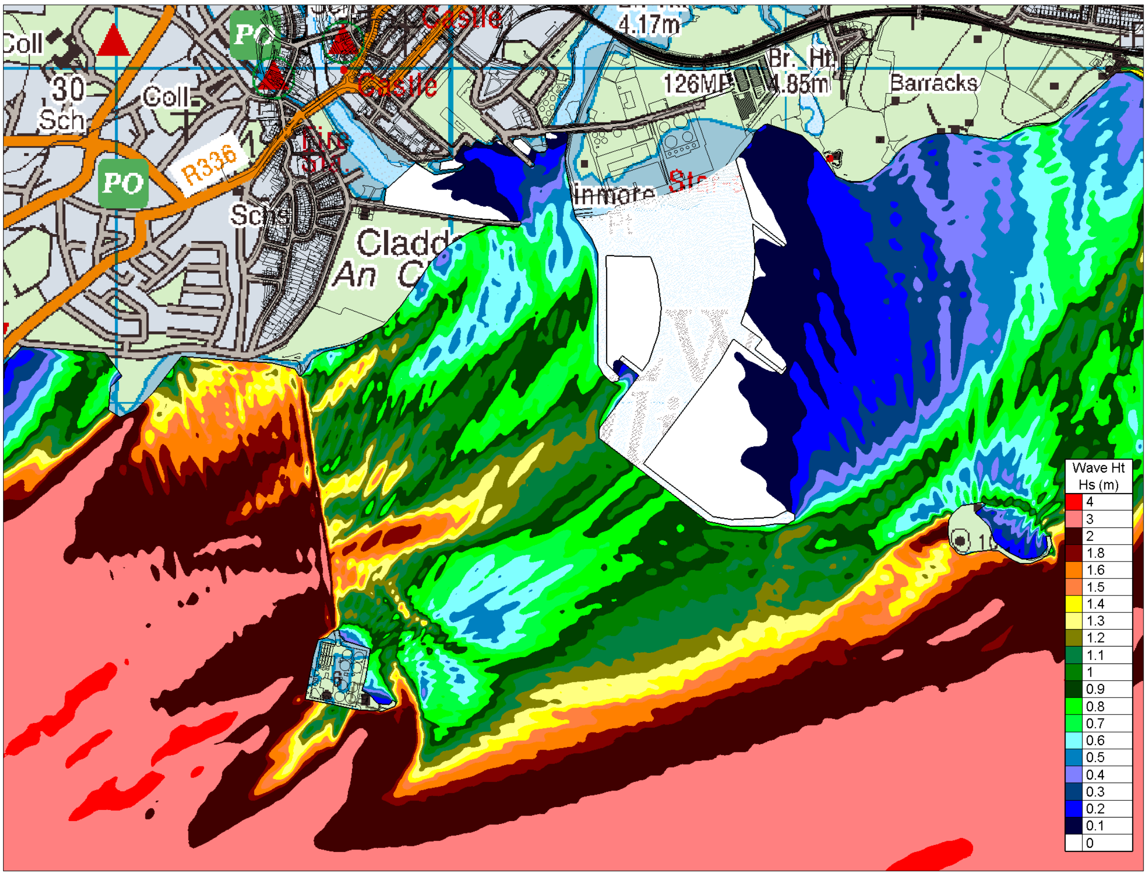


Figure 8.4.141 Computed maximum wave heights Hs for all onshore directions from WSW to ESE along Section A-B for Existing and Proposed Cases.



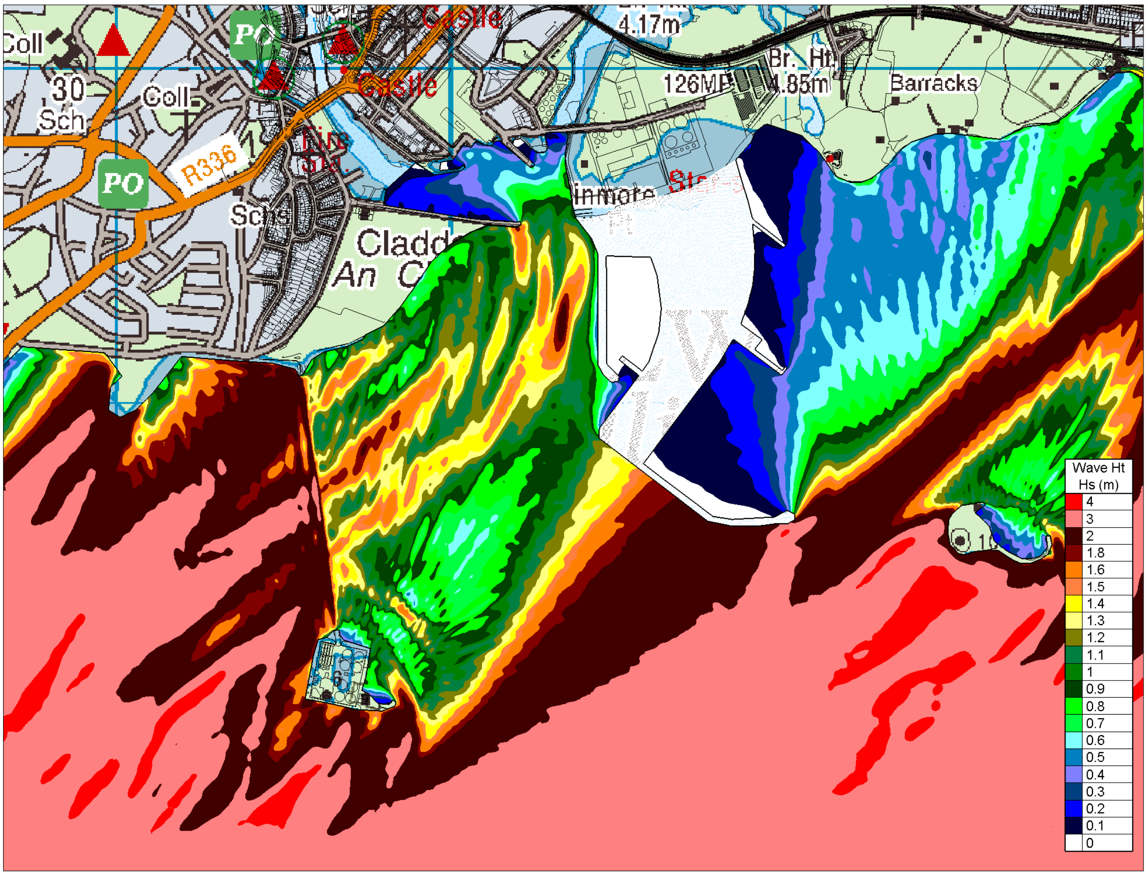


Figure 8.4.142 West-South-West Design Wave at extreme highwater of 4.635 m O.D. Malin to examine the ability of Mutton Island Causeway to protect the Harbour and Marina area from West-South-West and south-west deepwater design waves

8.4.6.7 Impact of development on surrounding wave climate

Wave climate simulations were carried out with and without the proposed harbour development to evaluate the potential impact that the development will have on the local wave climate of the shoreline areas to the west and east of the development.

The model simulations show a significant sheltering effect from the head of Nimmo's Pier to the Renmore Shoreline area including Claddagh Basin, Spanish Arch/Long Walk under southerly to easterly storms, refer to Figure 8.4.135 to 8.4.139.

For the southwesterly (SSW to WSW) sector the harbour development will result in increased wave heights in the vicinity of Nimmo's pier with the wave field being diffracted westward by the proposed Harbour and breakwater structures, refer to figure 8.4.132 to 8.4.134. Further west along the Southpark shoreline there is little or no predicted change in wave climate.

In terms of maximum wave heights (refer to Figures 8.4.140 to 141) along Southpark shoreline and Docks Area (i.e. Shoreline from the Causeway to the docks/Claddagh Basin entrance channel) the critical wave directions are southerly SSW to SSE. Under such conditions the proposed development reduces the maximum predicted wave height at the entrance channel to the Docks/Claddagh Basin area by between 0.3 and 0.5m, from 1.4 to 1.8 under the existing case to 0.8 to 1.5m under the proposed case. At the Nimmo's pier section here is a slight increase of less than 0.15m in maximum wave height as a result of the proposed harbour development.

Further westward along the Southpark Shoreline section the impact on the wave heights is minor, refer to figures 8.4.132 to 8.4.139. The analysis shows only slight increases and decreases of less than 0.05m in the maximum predicted wave heights along the Southpark shoreline, refer to Figure 8.4.141.

The simulations show no impact to the Wave climate to the west of the Causeway (i.e. Grattan Road shoreline area) which is more exposed and vulnerable area in respect to wave overtopping during southwesterly storms.

The wave modelling shows the Claddagh Basin to north of Nimmo's pier to be generally sheltered from wave climate except under East-South-East wave storm which is shown to propagate into the basin producing wave heights of 0.2m under the existing case. The proposed Harbour development is shown to completely shelter the Claddagh Basin against this direction.

8.4.6.8 Conclusions

The breakwater protection varies in height depending on the location and exposure to wave climate with southerly breakwater having a crest elevation of 9.1 to 10.1 m O.D. which provides 4.465 to 5.465 m above the design tide level (4.635 m O.D.) for wave climate and wave run-up effects. This level of protection will minimise the risk of overtopping of the breakwater structure by extreme waves. The westerly breakwater located in the more sheltered waters has a top elevation 6.65 to 6.95 m O.D. which based on wave climate analysis will protect this area from overtopping by the waves predicted for these locations.

A simulation was also carried out assuming the Mutton Island causeway to be completely submerged by 200-year Tide with Sea level Rise (4.635 m O.D. Malin), covered by over 1m of water depth and a westerly deepwater design wave of 3.77 m significant wave height applied. The simulation shows that the Mutton Island Causeway would under these submerged conditions break the storm waves and dissipate much of its energy and thus provide protection to the westerly face of the proposed development even under submerged conditions.

The wave climate simulations show that the proposed harbour development impacts the local wave climate environment through a combination sheltering via dissipation and reflection off its breakwaters and diffraction and refraction of the wave field around the development and over the dredged channels. The development generally shelters the eastern section of the adjacent Renmore shoreline against storms from the south to southwesterly sector. It protects the Galway Docks entrance and much of the Southpark shoreline against storms from the south to the east.

The simulations show under south and south westerly storms increased wave activity along the south face of Nimmo's Pier and the entrance to Galway Docks and the Corrib channel. These are not the most significant waves which presently occur at this location and theses waves are directed across the Corrib channel as opposed running up along it.

The wave simulations show that this increased wave activity at Nimmo's pier entrance does not appreciably impact wave heights within the inner Claddagh Basin area and such impacts are less than those which presently arise from the southeast direction which will now be blocked by the proposed development.

8.4.7 Flood Risk Assessment Study

8.4.7.1 Background

It is clear that the proposed development site is substantially located within the High Flood Risk Zone (*i.e.* Zone A of the Flood risk Management Planning Guidelines) given its nature as a proposed Harbour and the fact that the majority of the Harbour Extension land area will be reclaimed from the sea. Therefore under the Planning Guidelines the Proposed development will require a detailed Flood Risk Assessment.

This Flood Risk Assessment has been prepared in accordance with the Flood Risk Management Planning Guidelines Published by *Department of the Environment, Heritage and Local Government's* (Nov. 2009).

The Flood Risk Assessment (FRA) was therefore carried out to

- Assess and quantify flood risk at the Proposed Development and in the adjoining areas
- Identify sources of Flood Risk within the study area
- Identify Flood Zones within the development site boundary
- Assess potential hydrological impacts by the development on flooding and flood risk locally and in the wider area.
- Develop appropriate flood risk mitigation and management measures
- Identify whether the proposed Development meets the requirements of the Flood Risk Management Planning Guidelines

In particular the objectives of this study were to establish:-

- The Impact of the Proposed Development on flooding in the adjoining flood susceptible areas of the Spanish Arch, Claddagh Quay and Frenchville areas.
- The impact on the proposed port development from potential flooding by tidal and fluvial Flood events both combined and independent.
- The breakwater and Wave Wall protection required for the Harbour Extension development.

8.4.7.2 Methodology

In order to assess the impact that the proposed development will have on flooding locally and on the wider area, two-dimensional hydrodynamic modelling using TELEMAC2D was carried out. This modelling allows a combination of fluvial flood events from the River Corrib and tidal events in Galway Bay both astronomical and storm surge along with adverse wind conditions to be examined and evaluated. Return period Tidal storm surge and river flood flows inputs to the TELEMAC2D model were determined through statistical analysis of gauged tide levels in Galway Bay and River Corrib flood flows at Galway.

In order to evaluate the wave height magnitude at the proposed harbour development and surrounding flood risk lands wave climate modelling using two wave models from the TELEMAC system was carried out. These wave models allow the simulation of local and deepwater waves propagating inshore and the impact that harbour structures (*i.e.* breakwaters and quay walls) have on the resultant wave climate including for partial and full reflection of waves off structures and diffraction around structures.

The inputs to the Flood Risk Assessment Analysis are:-

- Tides and tidal predictions using gauged tide level for Galway Bay (Corrib Estuary Gauge at Wolfe Tone Bridge and Oranmore Bay at old Dublin Road Bridge)
- River Corrib gauged flows including flood duration curve and flood flow statistical analysis
- Wind speeds and directions for nearest coastal meteorological station at Belmullet Co. Mayo

- Offshore Deepwater wave characteristics using the Shore Protection Manual US Army Corp of Engineers (1984)
- Recommended climate change allowances for flood flows and sea level rise

8.4.7.3 Principal Flood Risk Areas

The main flood risk areas within the study area are low-lying lands below 4.2 m O.D. Malin, namely Claddagh Quay, Long Walk, Nimmo's Pier, Flood Street, Lower Quay Street and Quay Lane, Spanish Parade, Dock Street and Dock Road, Merchant's Road and Merchants Road Lower, Fr. Griffin Road, Fairhill, Dominick Street, Munster Avenue, William Street West, Grattan Road, Frenchville, Claddagh Avenue and South Park Place. These areas are at risk from flooding by tidal inundation during storm surge events. To date the highest sea storm event recorded was a flood highwater level of 3.49 m O.D. Malin. that occurred on the 17^{th} January 1995.

Areas that have previously flooded as a result of high tides are Grattan Road and adjoining dwellings primarily at Frenchville and properties fronting the road at South Park Place and Claddagh Avenue, Claddagh Quay, Docks Road, Merchant's Road Lower, Spanish Parade, Quay Lane and Flood Street.

Recurring flooding *(ca* 1 in 5 years) at Merchant's Road Lower, Spanish Parade, Quay Lane and Flood Street occurs due to urban storm drainage capacity which can be compounded by high tide levels.

Areas subject to potential Flood Risk from Wave Climate is Grattan Road, Mutton Island Causeway, South Park Shoreline Walkway to Nimmo's Pier and the Salthill promenade area.

There is no history of fluvial flooding by the River Corrib of these areas.

8.4.7.4 Sources and Mechanisms for Flooding

The main sources of flooding are:

- High Storm Surge Tides
- Wave overtopping and wave run-up
- Local surface/storm water runoff
- Flooding by the River Corrib

The primary source of Flood Risk to Galway City comes from tidal inundation of the unprotected low-lying lands below 200-year tide levels. Wave run-up and overtopping of coastal defences along Salthill and Grattan Road coinciding generally with high Spring tides represents a significant flood risk to these more exposed locations.

Flooding by the River Corrib was not found by itself to represent a significant source of flood risk but combined with tidal surge events it represents a significant flood risk to Claddagh Basin and Spanish Arch areas which adjoin the estuary.

Urban drainage issues also represent a significant source of flooding and some low-lying areas in the vicinity of Flood Street see regular inundation due to short duration intense rainstorm events (cloud bursts) and also backing up of storm drainage during very high tides.

The River Corrib under fluvial flood conditions (by itself) will not result in the flooding of the Claddagh area downstream of Wolfe Tone Bridge due to the relatively wide river width and deep flow depth available below quay level in the Claddagh basin which is sufficient to easily accommodate extreme fluvial flood flows.

The critical condition is a combination of tidal flooding and fluvial flooding which will be examined using the hydrodynamic model. Even under these conditions the additional contribution to the highwater level by the Corrib will be small as ample cross-section flow area is available for flow conveyance given the relatively wide river channel width and sizable flow depth at the critical highwater stage of the tide.

Wave overtopping of flood defences was a major contribution to flooding during the January 1995 flood event whereby a combined effect of wave overtopping near the Grattan Road/Fairhill junction to the west of the causeway entrance and high tide levels (3.49 m O.D.) produced significant flooding of South Park, Grattan Road and dwellings at Frenchville. A similar high tide event in 1997 with no adverse wave climate saw no flood damage to these areas.

The main vulnerability to wave overtopping is the shore defences west of and to a lesser extent east of the Mutton Island causeway. Currently the sea defences along the exposed section west of Mutton Island causeway have rock armouring to a crest level of c. 5.3 m O.D. At the historical highwater tide level of 3.49 m O.D. Malin this provides a protection height of c. 1.8 m against storm waves and only 1.15 m at the 200 year high tide level. Development to the east of the Mutton Island causeway as proposed by the new harbour will not impact on the shoreline to the west of the causeway.

The meteorological conditions that produce storm surge tides and large storm waves are similar and consequently have a reasonable high probability of coinciding. Under such conditions, storm waves of up to 3 m could result to the west of the Mutton island Causeway and up to 1.6m to the east of the causeway.

Those to the west would be capable of overtopping the existing flood defences. The proposed development will not worsen this situation.

East of the causeway along the South Park cycle path an extent of wave defences have been formed with rock armouring to levels of 5.0 m O.D. This protection is considered to be sufficient for the predicted wave climate.

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8.4.7.5 Description of the New Harbour Extension

The proposed harbour development will involve 28.07 ha of developed lands with 23.89 ha to be reclaimed from the sea. It will provide 660m of Sheltered Quays, a Commercial Port, a western Marina with 216 berths, a Fisherman's Pier and a Nautical Centre Slipway. This development will involve substantial dredging in order to form a new approach channel (more westerly than the existing channel) to the existing Galway Harbour having an invert of -3.5 m C.D. and 80 m wide, a deep water approach channel to the Commercial Harbour having an invert of -8 m CD and 100 m wide, a ship turning circle of 400 m radius with -8 m CD invert, Commercial berths at -12 m CD and 660m quay length, Fisherman's Quay at -3.5 m CD and -8.0m CD and 216 berth Marina at -3.5 m CD, refer to Table 8.4.18 (Note OS Malin Head datum is 2.9 m above Chart Datum (CD)). Based on the borehole investigations carried out, it is estimated that the total dredge volume is 1.839 million m^3 of which 1.815 million m^3 is sediment and soils.

Dredging Depth and Dredge Areas of New Port			
Description	**Dredge Depth (CD)**	**Dredge Area (ha)**	**Notes**
Entry Channel to Commercial Harbour	-8.0 m	11.42	From -8m sea bed contour
Turning Circle for Commercial Harbour	-8.0 m	12.56	400m Turning Circle
Commercial Quay Berths and approach from turning circle	-12.0 m	2.89	660m of Quay berth in total
Access Channel to Western marina and Existing Harbour	-3.5 m	8.27	To match existing channel dredge depth of -3.5m
Western Marina Dredge	-3.5 m	4.72	216 Marin Berths
Access Channel and Fishermen Pier Berths	-3.5 m -8.0m	2.41	180m of Fisherman's

Table 8.4.18 Dredging depth and dredge areas of New Port

The total dredged area is 46.48ha.

From a flood risk perspective all quays, yards and internal roadways *etc*. within the New Harbour Extension will be set at a minimum level of 7.6 m CD (4.7 m above Malin Head Datum).

Breakwater protection against local and offshore wave climate will be provided along the south and west facing aspects of the New Harbour so as to protect the Marina area, access road to the commercial Harbour and the Commercial Port and Fishing Pier. The crest (top) elevation of the proposed southerly breakwater protecting the more exposed commercial port area will be set at 12 to 13 m CD (9.1 m to 10.1 m OD) with breakwater height increasing south-eastward. The westerly breakwater protecting the Marina Area will be set at 9.55 to 9.85 m CD (6.65 to 6.95 m O.D.). These breakwaters will be founded on sheet piles drilled into bedrock with sheet pile heads extended between 2.2 and 5.15 m OD and 2 layer rock armour placed above this.

The final height, configuration and orientation of breakwaters and quay walls were informed by several iterations of the wave climate modelling analysis.

<u>Buildings</u>
The following buildings will be applied for in this Planning Application

- Harbour Office/Port Centre;
- Marina Office;
- Passenger Terminal;
- Harbour Company Warehouse; and
- Ancillary Buildings (Pump house for fire fighting purposes, Security Building at Port main gate and ESB Substation).

All proposed buildings within the Port area will have a minimum Finish Floor Level of 8.4 m above CD (5.5 m OD Malin).

<u>Storm Drainage</u>
A conventional storm water drainage network is proposed for the Port collecting all roof and hard paved runoff (roads, yards and quays). It is proposed to discharge the storm runoff directly to the sea at four locations, (refer to Table 8.4.19). Each outfall will be fitted with a valve and a Tideflex (or similar) non-return valve and upstream of the outfall a Class 1 petrol interceptor. As the discharges are directly to the Sea no Storm water attenuation is proposed as the impact of these storm outfalls on flooding will be imperceptible.

Location of Proposed Storm Outfalls and Peak Flow Rates		
Outfall	**OS grid location**	**Peak Storm Flow**
A - A1	130860, 224705	3003 l/s
A2	130860, 224703	735 l/s
B	130843, 224186	754 l/s
C	130442, 223905	706 l/s
D	130581, 223815	202 l/s

Table 8.4.19 Location of proposed storm outfalls and peak flow rates (refer to Drawing 2139-2214)

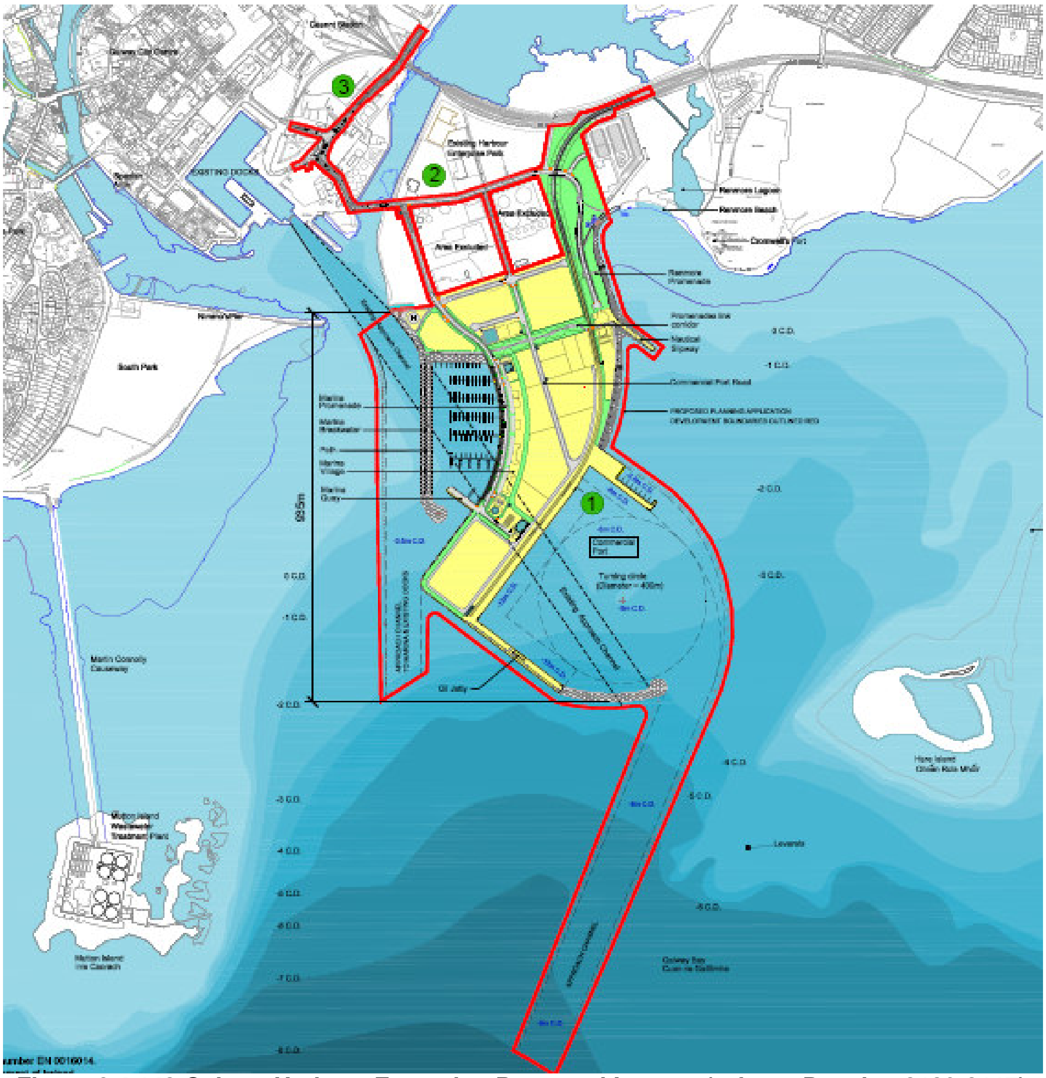

Figure 8.4.143 Galway Harbour Extension Proposed Layout (refer to Drawing 2139-2117)

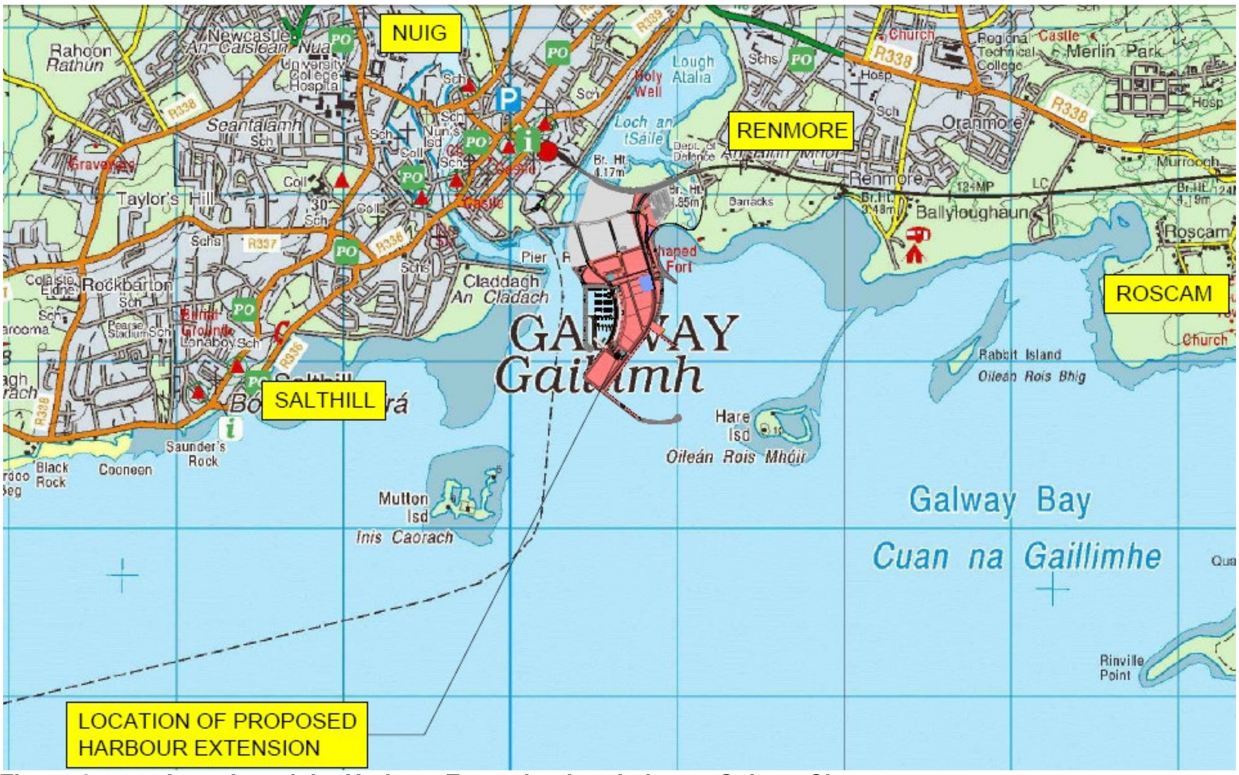

Figure 8.4.144 Location of the Harbour Extension in relation to Galway City

8.4.7.6 Planning Guidelines Concerning Flood Risk Management

8.4.7.6.1 Background

In September 2008, the OPW and DoEHLG jointly published for public consultation new draft Planning Guidelines on Planning System and Flood Risk Management which are aimed at ensuring a more consistent, rigorous and systematic approach to fully incorporate flood risk assessment and management into the planning system. These guidelines after consultation were finalised and published in November 2009.

The document gives guidance on how to assess and manage flood risk potential and also includes guidance on the preparation of flood risk assessments by developers.

The recommended stages of assessment are:
Screening Assessment : to identify whether there may be flooding or surface water management issues related to a plan area or proposed development site that may warrant further investigation;

Scoping assessment: to confirm sources of flooding that may affect a plan area or proposed development site, to appraise the adequacy of existing information and to scope the extent of the risk of flooding and potential impact of a development on flooding elsewhere and of the scope of possible mitigation measures.

Appropriate risk assessment: to assess flood risk issues in sufficient detail and to provide a quantitative appraisal of potential flood risk to a proposed or existing development, of its potential impact on flood risk elsewhere and of the effectiveness of any proposed mitigation measures.

8.4.7.6.2 Site Specific Flood Risk Assessment

The Flood Risk Management Guidance suggests the following information that might accompany a site specific Flood Risk Assessment report:

Mapping:
- A location map;
- A plan that shows existing site and proposed development(s);
- Identification of any structures which may influence the hydraulics; and
- Flood Inundation map showing flood zone areas on the subject site/area.

Surveys:
- Site levels related to Ordnance Datum; and
- Appropriate cross-section(s) showing finished *etc.* or other relevant levels in respect to flooding.

Assessments:
- Consideration of flood zone in which the site falls and demonstration that development meets the vulnerability criteria set out in the Guidance;
- Flood alleviation measures already in place;
- Information about potential sources of flooding that may affect the site; and
- The impact of flooding on a site.

Design Standards
- The FRA should generally be undertaken on the basis of a design event of the appropriate design standard:-
  - 100 year Fluvial Flood or 1% Annual Exceedence Probability (AEP) for River Flow
  - 200 year combined Return Period event or 0.5% AEP for tide affected sites

Decision Making Process
Management of flood hazard and potential risks in the planning system is based on
1. Sequential Approach
2. Justification Test

1. Sequential Approach

The aim of the sequential approach is to guide development away from areas at risk from flooding. The approach makes use of flood risk zones, ignoring presence of flood protection structures, and classifications of vulnerability of property to flooding. Definitions of flood risk zones are as follows:

- Zone A High Probability – Highest risk of flooding: More than 1% probability of river flooding and more than 0.5% probability of tidal flooding. Development should be avoided and/or only considered through application of Justification test. Only water compatible development , such as docks and marinas, dockside activities that require a waterside location, amenity open space, outdoor sports and recreation and essential transport infrastructure that cannot be located elsewhere would be considered appropriate for this zone (*i.e.* not requiring application of Justification test)

- Zone B Moderate Probability: Between 1 and 0.1% probability of river flooding or between 0.5 and 0.1% probability of coast flooding. Development should only be considered in this zone if adequate land or sites are not available in Zone C or if development in this zone would pass the Justification Test.

- Zone C Low Probability : Less than 0.1% probability of river or coastal flooding. Development in this zone is appropriate from a flooding perspective.

2. Justification Test

Further sequentially based decision making should be applied when undertaking the Justification Test for development that needs to be in flood risk areas for reasons of proper planning and sustainable development:

1. within Zone or site, development should be directed to areas of lower flood probability;
2. where impact of the development on adjacent lands is considered unacceptable the justification of the proposal or Zone should be reviewed
3. where the impacts are acceptable or manageable, appropriate mitigation measures within the site and if necessary elsewhere should be considered.

8.4.7.6.3 Application of the Justification Test in Development management

Where a planning Authority is considering proposals for new development in areas at a high or moderate risk of flooding that include types of development that are vulnerable to flooding and that would generally be inappropriate, the planning authority must be satisfied that the development satisfies all of the criteria of the Justification Test as it applies to development management outlined in Figure 8.4.145 below.

Box 5.1 Justification Test for development management (to be submitted by the applicant)

When considering proposals for development, which may be vulnerable to flooding, and that would generally be inappropriate as set out in Table 3.2, the following criteria must be satisfied:

1. The subject lands have been zoned or otherwise designated for the particular use or form of development in an operative development plan, which has been adopted or varied taking account of these Guidelines.
2. The proposal has been subject to an appropriate flood risk assessment that demonstrates:
   (i) The development proposed will not increase flood risk elsewhere and, if practicable, will reduce overall flood risk;
   (ii) The development proposal includes measures to minimise flood risk to people, property, the economy and the environment as far as reasonably possible;
   (iii) The development proposed includes measures to ensure that residual risks to the area and/or development can be managed to an acceptable level as regards the adequacy of existing flood protection measures or the design, implementation and funding of any future flood risk management measures and provisions for emergency services access; and
   (iv) The development proposed addresses the above in a manner that is also compatible with the achievement of wider planning objectives in relation to development of good urban design and vibrant and active streetscapes.

The acceptability or otherwise of levels of residual risk should be made with consideration of the type and foreseen use of the development and the local development context.

Note: See section 5.27 in relation to major development on zoned lands where sequential approach has not been applied in the operative development plan.

Refer to section 5.28 in relation to minor and infill developments.

Figure 8.4.145 Justification Test for development management

The planning implications for each of the flood zones are summarised as follows:

Zone A – High probability of Flooding. Most types of development would be considered inappropriate in this zone. Development in this zone should be avoided and/or only considered in exceptional circumstances, such as in city and town centres, or in the case of essential infrastructure that cannot be located elsewhere, and where the justification test has been applied. Only water-compatible development, such as docks and marinas, dockside activities that require a waterside location, amenity open space, outdoor sports and recreation, would be considered appropriate in this zone.

Zone B - Moderate probability of flooding. Highly vulnerable development, such as hospitals, residential care homes, Garda, fire and ambulance stations, dwelling houses and primary strategic transport and utilities infrastructure, would generally be considered inappropriate in this zone, unless the requirements of the Justification Test can be met. Less vulnerable development, such as retail, commercial and industrial uses, sites used for short-let for caravans and camping and secondary strategic transport and utilities infrastructure, and water-compatible development might be considered appropriate in this zone. In general however, less vulnerable development should only be considered in this zone if adequate lands or sites are not available in Zone C and subject to a flood risk assessment to the appropriate level of detail to demonstrate that flood risk to and from the development can or will adequately be managed.

Zone C - Low probability of flooding. Development in this zone is appropriate from a flood risk perspective (subject to assessment of flood hazard from sources other than rivers and the coast) but would need to meet the normal range of other proper planning and sustainable development considerations.

8.4.7.7 Flood Hydrology and Tides

8.4.7.7.1 Hydrometric Data

OPW hydrometric gauges measuring water level are available at Wolfe Tone Bridge (30061) and Oranmore Bridge (29015). These gauges are tidal and their annual maximum flood level series provides information on high tide levels particularly the Oranmore gauge which has a relatively small fluvial flood flow contribution. The Wolfe Tone gauge on the Corrib provides a combined fluvial and tidal annual maximum series with peak levels particularly during Winter periods influenced by a combination of high River Corrib flood flows coinciding with Spring tides. The Wolfe Tone gauge for the annual maximum flood level series is located on the upstream face (has recently been relocated downstream of the bridge) and recorded high tide levels can be affected by bridge afflux and standing waves during flood conditions. For the purposes of predicting tidal surge flooding the Oranmore gauge is considered to be more appropriate and reliable and applicable to the Galway Port area. Galway Port also monitors and more recently records tidal height (Marine Institute Operational Tide Gauge for the West Coast). Unfortunately there is not a sufficient continuous record length available for this station to perform statistical analysis of return period high tide levels.

8.4.7.7.2 Frequency Analysis of Annual Maximum Tide Levels

Oranmore Estuary Gauge 29015

A gauging station with annual maximum series of tide/flood elevations recorded continuously since 1982 is available at Oranmore Bridge (gauge reference 29015) on the Old Dublin Road (see Table 8.4.20). This provides a 28-year annual maximum high water series on which to carry out an EV1 statistical analysis. The EV1 distribution was fitted by the method of Gringorten plotting positions and least squares fit. The standard error was also computed for this analysis and plotted to provide the 95-percentile upper and lower confidence intervals for the series, see Figure 8.4.146. Based on the EV1 statistical analysis, the return period tidal highwater estimates for the Galway Bay Area are presented in Table 8.4.21.

The predicted 200-year flood elevation is H_{200} = 3.800 m O.D. Malin (which is 0.31 m higher than the recorded maximum tide level at gauge 29015) having a statistical standard error (s.e.) of 0.169 m giving an upper 95% confidence interval (*i.e.* H_{200} plus twice the standard error) for the 200-year tidal prediction of 4.146 m O.D. Malin. The 1000-year tide level is estimated to be 4.057 m O.D. and the upper 95% CI of the fit is estimated to be 4.497 m O.D. Because of the relatively short duration of record length (28 years) in comparison to the return period magnitude of 200 years and 1000 years it is prudent to use the upper 95% confidence Interval estimate.

OPW Annual Maximum Flow Levels (Oranmore Gauge (29015)				
Year	m OD Malin	Stage (m)	Date	Comment
1982	2.81	1.79	08/09/1983	Levels are tidal peaks
1983	2.88	1.86	19/02/1984	Levels are tidal peaks
1984	3.04	2.02	23/11/1984	Levels are tidal peaks
1985	2.82	1.8	28/03/1986	Levels are tidal peaks
1986	2.92	1.9	03/12/1986	Levels are tidal peaks
1987	3.02	2.00	27/09/1988	Levels are tidal peaks
1988	3.09	2.07	09/03/1989	Levels are tidal peaks
1989	3.05	2.03	26/02/1990	Levels are tidal peaks
1990	3.40	2.38	05/01/1991	Levels are tidal peaks
1991	2.93	1.91	29/08/1992	Levels are tidal peaks
1992	3.07	2.05	12/01/1993	Levels are tidal peaks
1993	3.16	2.14	12/01/1994	Levels are tidal peaks
1994	3.49	2.47	17/01/1995	Levels are tidal peaks
1995	2.94	1.92	18/09/1996	Levels are tidal peaks
1996	3.48	2.46	10/02/1997	Levels are tidal peaks
1997	2.99	1.97	30/03/1998	Levels are tidal peaks
1998	3.10	2.08	02/01/1999	Levels are tidal peaks
1999	2.89	1.87	26/12/1999	Levels are tidal peaks
2000	2.99	1.97	09/03/2001	Levels are tidal peaks
2001	3.26	2.24	02/02/2002	Levels are tidal peaks
2002	2.78	1.76	08/10/2002	Levels are tidal peaks
2003	2.88	1.86	19/03/2004	Levels are tidal peaks
2004	3.28	2.26	08/01/2005	Levels are tidal peaks
2005	3.02	2.00	29/03/2006	
2006	3.00	1.98	19/01/2007	Maximum levels are tidal peaks
2007	2.82	1.8	27/10/2007	Maximum levels are tidal peaks
2008	2.83	1.81	22/08/2009	Maximum levels are tidal peaks
2009	3.26	2.24	03/03/2010	Maximum levels are tidal peaks

Table 8.4.20 The OPW Annual Maximum Flood level series for Oranmore gauge (29015)

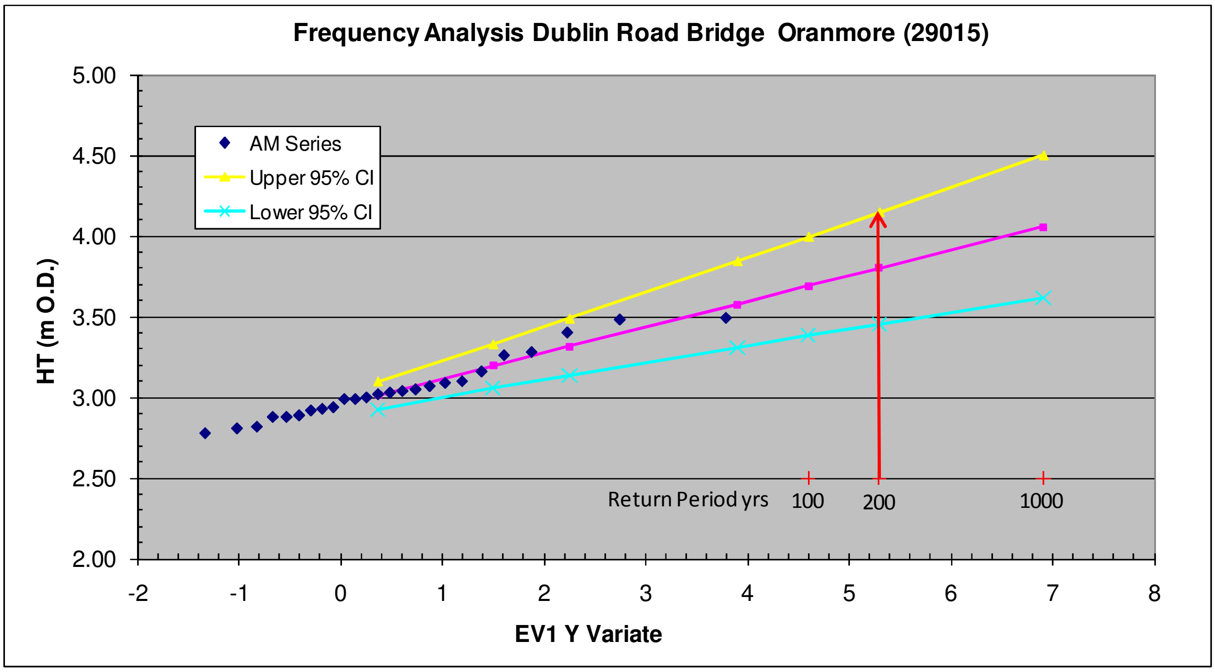

Figure 8.4.146 EV1 Fit of the Oranmore AM highwater tidal series showing the statistical upper and lower 95-percentile confidence interval of the fit

Tide level predictions at Oranmore Bridge Gauge			
T (years)	Y_{EV1}	Tide Highwater H_T m OD Malin	Statistical standard error(m)
2	0.37	3.012	0.035
5	1.50	3.193	0.058
10	2.25	3.313	0.079
50	3.90	3.577	0.127
100	4.60	3.689	0.148
200	5.30	3.800	0.169
1000	6.91	4.057	0.219

Table 8.4.21 Tide level predictions at Oranmore Bridge Gauge

Wolfe Tone Bridge Gauge – Corrib Estuary 30061

This gauge has a tidal record length of 25 years (1982 to 2006) (see Table 8.4.22). The gauge was relocated in 2006 and new annual maximum (AM) records are not currently available. The 200-year flood level prediction for the Corrib Estuary at Wolfe Tone Bridge using the available AM flood level series is 3.96 m O.D. Malin and including twice the statistical standard error for the upper 95% confidence interval gives a flood level of 4.368 m O.D. Malin (Figure 8.4.147). Because of the relatively short duration of record length (25 years) in comparison to the return period magnitude of 200 years and 1000 years it is prudent to use the upper 95% confidence Interval estimate. Table 8.4.23 shows the tide level predictions.

The inclusion of the upper 95% confidence interval through the addition of the twice the standard error ensures conservatism in estimates of 100, 200 and 1000year flood events notwithstanding the relatively short annual maximum period in relation to the return period.

Annual Maximum Series of Recorded Water Levels (Wolfetone Bridge 30061)			
Hydrometric Year	Water Level (mAOD – Malin)	S.G. Reading (m)	Date
1982	2.88	2.66	08/09/1983
1983	3.01	2.79	19/02/1984
1984	2.96	2.74	23/11/1984
1985		Break	28/03/1986
1986	**2.71**	**2.49**	**03/12/1986**
1987	3.06	2.84	27/09/1988
1988	3.08	2.86	09/03/1989
1989	2.87	2.65	26/02/1990
1990	3.02	2.80	05/01/1991
1991	3.07	2.85	29/08/1992
1992	3.27	3.04	12/01/1993
1993	3.23	3.02	12/01/1994
1994	3.48	4.26	07/01/1995
1995	3.03	3.81	28/09/1996
1996	3.63	4.41	10/02/1997
1997	3.08	3.85	30/03/1998
1998	3.139	2.9	02/01/1999
1999	3.309	2.86	26/12/0999
2000	3.069	2.84	12/12/2000
2001	3.569	4.34	01/02/2002
2002	2.989	3.76	08/10/2002
2003	2.872	3.643	02/08/2004
2004	3.175	2.946	08/01/2005
2005	3.189	3.96	30/03/2006
2006	3.229	4.0	20/02/2007

Table 8.4.22 Wolfe tone gauge Annual maximum Water Level series (combined)

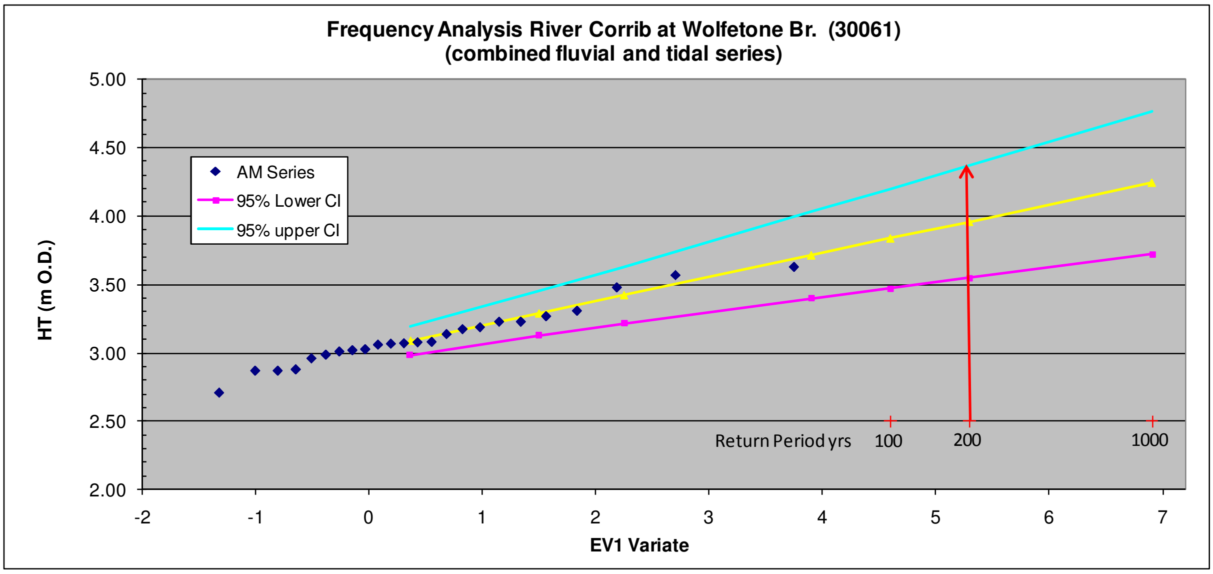

Figure 8.4.147 EV1 Fit of the Corrib Estuary Gauge at Wolfe Tone Bridge Gauge AM highwater series showing upper and lower 95-percentile confidence interval

Tide level predictions at Wolfe Tone Bridge Gauge			
T (years)	Y_{EV1}	Tide Highwater H_T m OD Malin	Statistical standard error(m)
2	0.37	3.088	0.053
5	1.50	3.288	0.081
10	2.25	3.420	0.104
50	3.90	3.712	0.158
100	4.60	3.835	0.182
200	5.30	3.958	0.205
1000	6.91	4.242	0.261

Table 8.4.23 Tide level predictions for Corrib Estuary at Wolfe Tone Bridge Gauge

8.4.7.7.3 Recommended Design Tide Level

The recommended 200 year and 1000 year design surge tide levels for the New development at Galway Harbour are the upper 95-percentile estimates obtained from statistical analysis of the Oranmore tidal gauge A.M. Series (4.146 and 4.45 m O.D. respectively). The Oranmore gauge provides a slightly longer Annual Maximum Series than Wolfe Tone Gauge and importantly its AM flood level series is not influenced by fluvial flood events. The Wolfe Tone Bridge AM series is not completely independent of the influence of the larger fluvial flood events which compromise the resultant annual maximum series in respect to tidal flood statistics.

8.4.7.7.4 Frequency Analysis of Flood flows in the River Corrib

The annual maximum flood flow series for the Wolfe Tone Bridge Gauge (30061) was obtained from the OPW Hydrometric Section (refer to Table 8.4.24, flow estimates since 2002 are not available from OPW). This gauging station is tidal and the fluvial flows used in the AM series were extracted during the non-tidal periods. A statistical analysis of the annual maximum flow series was carried out using an EV1 probability distribution fitted by the method of least squares (see Figure 8.84). The return period flood flow estimates are presented in Table 8.4.28 below.

Flows [m^3/sec] exceeded							
1%	5%	10%	50%	80%	90%	95%	99%
272	230	200	82.1	35	28.5	24.6	9.12

Table 8.4.24 Annual Maximum Flood Flow Series

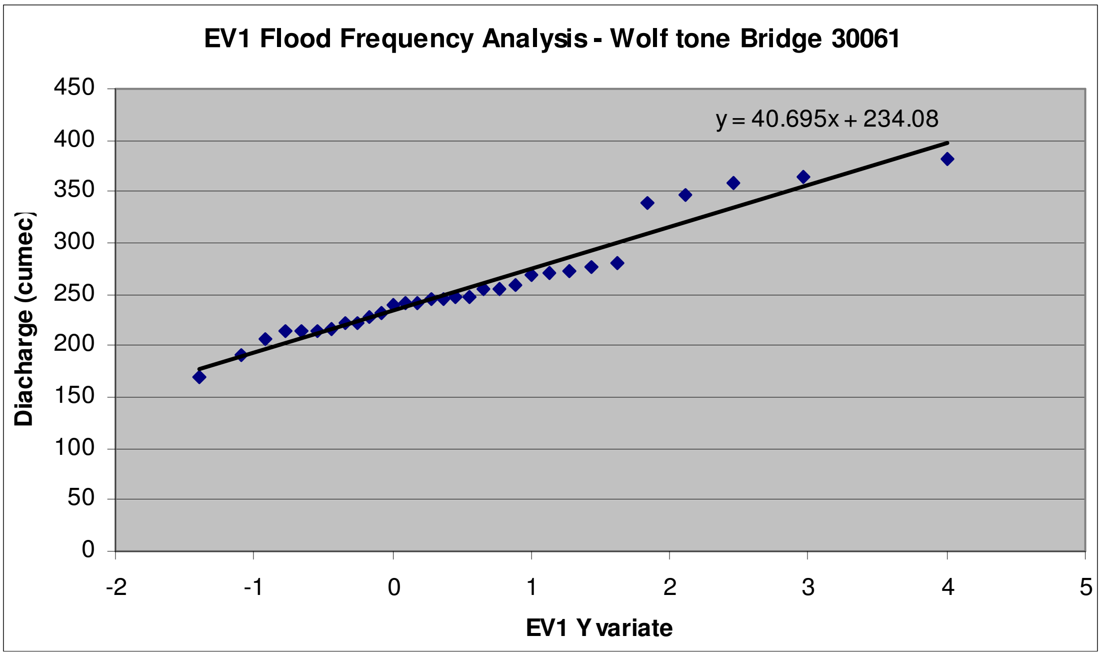

Figure 8.4.148 Flood Flow Frequency Analysis – River Corrib at Wolf Tone Bridge Gauge

Annual Maximum Fluvial Flood Flow series for River Corrib at Galway						
Hydrometric Year	**Water Level (mAOD-Malin)**	**S.G. Reading (m)**	**Estimated Flows (m^3/s)**	**Reliable Limit (m^3/s)**	**Date**	**COMMENTS / NOTES**
1972	1.12	0.90	169	335	**16/12/1972**	Fluvial Max (tidal level likely exceeded)
1973	1.17	0.95	215	335	**25/09/1974**	Fluvial Max (tidal level likely exceeded)
1974	1.48	1.26	231	335	**25/01/1975**	Fluvial Max (tidal level likely exceeded)
1975	1.04	0.82	347	335	**02/12/1975**	Fluvial Max (tidal level likely exceeded)
1976	1.16	0.94	190	335	**22/02/1977**	Fluvial Max (tidal level likely exceeded)
1977	1.52	1.30	228	335	**12/11/1977**	Fluvial Max (tidal level likely exceeded)

Table 8.4.25 Annual Maximum Fluvial Flood Flow series for River Corrib at Galway (more recent years not available from OPW for fluvial range)

Annual Maximum Fluvial Flood Flow series for River Corrib at Galway						
Hydrometric Year	**Water Level (mAOD-Malin)**	**S.G. Reading (m)**	**Estimated Flows (m^3/s)**	**Reliable Limit (m^3/s)**	**Date**	**COMMENTS / NOTES**
1978	1.2	0.98	364	335	**15/12/1978**	Fluvial Max (tidal level likely exceeded)
1979	1.46	1.24	241	335	**17/12/1979**	Fluvial Max (tidal level likely exceeded)
1980	1.22	1.00	339	335	**20/12/1980**	Fluvial Max (tidal level likely exceeded)
1981	1.21	0.99	248	335	**16/03/1982**	Fluvial Max (tidal level likely exceeded)
1982	1.28	1.06	245	335	**30/01/1983**	Fluvial Max (tidal level likely exceeded)
1983	1.29	1.07	269	335	**17/01/1984**	Fluvial Max (tidal level likely exceeded)
1984	1.2	0.98	273	335	**27/12/1984**	Fluvial Max (tidal level likely exceeded)
1985	1.22	1.00	241	335	**07/08/1986**	Fluvial Max (tidal level likely exceeded)
1986	1.24	1.02	248	335	**18/12/1986**	Fluvial Max (tidal level likely exceeded)
1987	1.3	1.08	255	335	**09/02/1988**	Fluvial Max (tidal level likely exceeded)
1988	1.12	0.90	276	335	**22/03/1989**	Fluvial Max (tidal level likely exceeded)

Table 8.4.26 Annual Maximum Fluvial Flood Flow series for River Corrib at Galway (more recent years not available from OPW for fluvial range)

Annual Maximum Fluvial Flood Flow series for River Corrib at Galway						
Hydrometric Year	**Water Level (mAOD-Malin)**	**S.G. Reading (m)**	**Estimated Flows (m^3/s)**	**Reliable Limit (m^3/s)**	**Date**	**COMMENTS / NOTES**
1989	1.56	1.34	215	335	**21/02/1990**	Fluvial Max (tidal level likely exceeded)
1990	1.46	1.24	381	335	**05/01/1991**	Fluvial Max (tidal level likely exceeded)
1991	1.4	1.18	223	260	**19/03/1992**	Fluvial Max (tidal level likely exceeded)
1992	1.63	1.41	281	260	**07/12/1992**	Fluvial Max (tidal level likely exceeded)
1993	1.66	1.44	271	260	**28/12/1993**	Fluvial Max (tidal level likely exceeded)
1994	1.98	2.75	358	260	**27/01/1995**	Fluvial Max (tidal level likely exceeded)
1995	1.41	2.18	207	260	**30/11/1995**	Fluvial Max (tidal level likely exceeded)
1996	1.61	2.38	255	260	**26/02/1997**	Fluvial Max (tidal level likely exceeded)
1997	1.57	2.34	245	260	**14/01/1998**	Fluvial Max (tidal level likely exceeded)
1998	1.67	2.44	215	260	**26/01/1999**	Fluvial Max (tidal level likely exceeded)
1999	1.8	2.57	239	260	**27/12/1999**	Fluvial Max (tidal level likely exceeded)
2000	1.71	2.48	223	260	**14/12/2000**	Fluvial Max (tidal level likely exceeded)

Table 8.4.27 Annual Maximum Fluvial Flood Flow series for River Corrib at Galway (more recent years not available from OPW for fluvial range)

Annual Maximum Fluvial Flood Flow series for River Corrib at Galway						
Hydrometric Year	**Water Level (mAOD-Malin)**	**S.G. Reading (m)**	**Estimated Flows (m^3/s)**	**Reliable Limit (m^3/s)**	**Date**	**COMMENTS / NOTES**
2001	1.78	2.55	259	334	**13/02/2002**	Fluvial Max (tidal level likely exceeded)
2002	1.56	2.33	216	248.78	**15/11/2002**	Fluvial Max (tidal level likely exceeded)

Table 8.4.27 contd/. Annual Maximum Fluvial Flood Flow series for River Corrib at Galway (more recent years not available from OPW for fluvial range)

Return Period Flow Estimates (Wolfetone Bridge)					
Return Period T years	**F(T)**	**EV1 variate**	**Q_T (m^3/s)**	**Standard Error (m^3/s)**	**Total (m^3/s)**
2	0.5	0.37	249	8.5	258
5	0.8	1.50	295	14.4	310
10	0.9	2.25	326	19.4	346
25	0.96	3.20	364	26.1	390
50	0.98	3.90	393	31.3	424
100	0.99	4.60	421	36.4	458
200	0.995	5.30	450	41.6	492

Table 8.4.28 Return period Flow Estimates – Wolf Tone Bridge

The median (50percentile or 2-year return period) flood flow value (Qmed) value for this gauge is 245 m^3/s (standard statistical error (s.e. = 8.4 m^3/s)), the mean annual maximum flood flow QBAR = 257 m^3/s (s.e. = 9.3 m^3/s) and the maximum recorded flow (1972 – 2002) is 381 m^3/s (recorded on the 5th Jan 1991).

The estimated 100-year flood flow peak for the River Corrib is 421 m^3/s plus a standard statistical error of 36.4 m^3/s (8.6%) giving a 66.7% upper confidence estimate of 457.4 m^3/s.

8.4.7.8 Climate Change Allowance

8.4.7.8.1 Sea Level Rise

The OPW in its guidance documentation issued for its flood relief schemes suggests the following climate change allowances:

A Climate Change Allowance of 300 mm to be added to design levels in all tidal situations, except for locations on the south coast, where an allowance of 350 mm is to be added. The allowance is also applicable to all sea levels that act as downstream boundary conditions for fluvial flood risk issues, where such conditions arise. This allowance is to be treated as a component part of the design water level and is not to be included as part of the freeboard which is supplementary. The OPW guidance is currently under review as part of the flood studies upgrade and is likely to be increased to at least 500 mm.

In the UK DEFRA (2006) in their most recent guidance (DEFRA, 2006) have presented the following net sea level rise allowances for the UK (Table 8.4.29), which represent a dramatic increase over previous guidance.

Regional net sea level rise allowances						
Region	Assumed vertical land movement (mm/yr)	Nett Sea-Level Rise (mm/yr)				Previous Allowances
		1990-2025	2025-2055	2055-2085	2085-2115	
East of England	-0.8	4.0	8.5	12.0	15.0	6mm/yr constant
South West and Wales	-0.5	3.5	8.0	11.5	14.5	5mm/yr constant
NW & NE England, Scotland	+0.8	2.5	7.0	10.0	13.0	4 mm/yr constant

Table 8.4.29 The UK Flood and Coastal Defence Appraisal Guidance (DEFRA, 2006) Regional net sea level rise allowances

Updated figures now reflect an exponential curve and replace the previous straight line graph representations.

Applying the regional values for Wales for the period 2007 to 2107, a total increase in sea level of 967 mm is predicted by the year 2107 (average rate of 9.7 mm/year). This is significantly higher than any previous sea level rise predictions by the ICCP.

The 2007 IPCC report does indicate an improvement in the accuracy of modelling processes from that in the previous IPCC TAR report; however, considerable further refinements are required on the modelling of ice flow processes. The report suggests that if contributions of ice melt from Greenland and Antarctica were to grow linearly with temperature rates, then the upper range for the A2 emissions scenario will increase from 0.52 m to 0.62-0.72 m.
Based on current research and current uncertainties on the behaviour of the Greenland and Arctic/Antarctic ice shelf, it is considered that a 500mm tidal increase estimate remains a suitable average sea level rise estimate for infrastructure planning purposes over the next 50 to 100-years in Irish coastal waters (*i.e.* 5 mm per annum sea level rise).

Therefore the estimated 200-year tide plus 500 mm (Climate Change) sea level rise produces a predicted high tide level of 4.646 m O.D. Malin for the Galway City area.

8.4.7.8.2 Climate Change Allowance for Fluvial Flood Flows

Climate change scenarios produced by the UK Hadley centre suggest fluvial floods in the 2080's increasing by up to 10% (low and medium low scenarios) or by up to 20% (medium high and high scenarios). Present recommendations are to include in the design flow a 20% increase in flood peaks over 50 years return period as a result of climate change. This scenario based on the Irish growth curve will result in a present day 100 year flood becoming a 25-year flood in approximately 50 years time.

Other predicted climate change effects for the UK are:
- A 4 to 5 mm per annum rise in mean sea level
- Additional intensity of rainfall of 20%
- An additional 30% Winter rainfall by the 2080's
- A reduction of 35/45% rainfall in Summer
- The 1 in 100 year rainfall storm to increase by 25%

DEFRA Guidance

In the UK research is ongoing to assess regional variations in flood allowances and the rate of future change. Current research thus far does not provide any evidence for the rate of future change let alone consider regional variations in such a rate. The UK Flood and Coastal Defence Appraisal Guidance (DEFRA, 2006) gives the following sensitivity climate change ranges, per Table 8.4.30. As a pragmatic approach it is suggested that 10% should be applied up to 2025, rising to 20% beyond 2025.

UK Flood and coastal appraisal guidance (DEFRA, 2006)				
Parameter	**1990 - 2025**	**2025 - 2055**	**2055 - 2085**	**2085 - 2115**
Peak rainfall intensity (preferably for small catchments)	+5%	+10%	+20%	+30%
Peak river flow (preferably for larger catchments)	+10%	+20%		

Table 8.4.30 UK flood and coastal defence appraisal guidance (DEFRA, 2006)

The proposed climate change allowance is a 20% increase in peak flow rates. This rate has been adopted by the OPW for its Catchment Flood Risk Assessment and Management Studies (Lee, Dodder, Tolka CFRAMs).

8.4.7.9 Hydrodynamic Assessment

8.4.7.9.1 Introduction

This hydrodynamic assessment examines the effect of the Harbour Extension development on the combined events of high tides and fluvial flood flows in the River Corrib so as to assess and quantify the potential rise in flood levels in the Claddagh Basin and approaches to Galway Docks as a result of the proposed harbour development. In order to predict the potential flood impact arising from the harbour development on the hydrodynamics of Galway Harbour, TELEMAC2D Model was used.

8.4.7.9.2 Model development

An element unstructured finite element mesh was fitted to Galway Bay from Laghtnagliboge Point near Spiddal and Black Head on the north Clare Coastline Eastward including both north and South Galway Bays which includes Oranmore Bay, New Harbour, Doorus Strait, Kinvarra Bay and Ballyvaughan Bay. In the vicinity of the port site at Galway Harbour, the finite element mesh was refined to include better detail of the bathymetry and shoreline geometry. This refinement area included Mutton Island, the Claddagh Basin and Lough Atalia.

The TELEMAC package was the software of choice for modelling the complicated hydrodynamics of the Galway bay area and particularly the varying refinement of the computation required (*i.e.* harbour and port area requiring high resolution and the open sea requiring less resolution).

8.4.7.9.3 Hydrodynamic Simulations

Hydrodynamic Simulations were performed for various tide and river Inflow conditions with and without the proposed harbour development. The hydrodynamic conditions considered in respect to flooding and flood impact were:

1 100-year Corrib Flood Flow (458m^3/sec) combined with a mean Spring tide

2 100-year Corrib Flood Flow (458m^3/sec) combined with the Historical Maximum Storm Surge Tide of 3.49 m O.D.

3 100-year Corrib Flood Flow that includes 20% climate change allowance (549m^3/sec) combined with the Historical Maximum Storm Surge Tide levels of 3.49 m O.D.

4 100-year River Corrib Flood Flows that includes 20% climate change allowance (549m^3/sec) combined with the 200-year Design Tide level of 4.146 m O.D. Malin

Consideration of higher surge tide levels such as the predicted 200-year tide level with and without sea level rise will result in higher water depths for flow conveyance and thus lower flow velocities resulting in a potentially smaller impact from the proposed port development on upstream flooding during the critical highwater stage of the tide.

The simulations were carried out for 12.4 hour Spring tidal curve having a tidal range set to 4.5 m (Spring tide range) with peak level set to the required highwater level (*i.e.* mean Spring high tide level, historical maximum tide level of 3.49 m O.D. or the 200-year design tide level). Given the relatively long duration of flooding in the River Corrib a constant peak flow rate was specified.

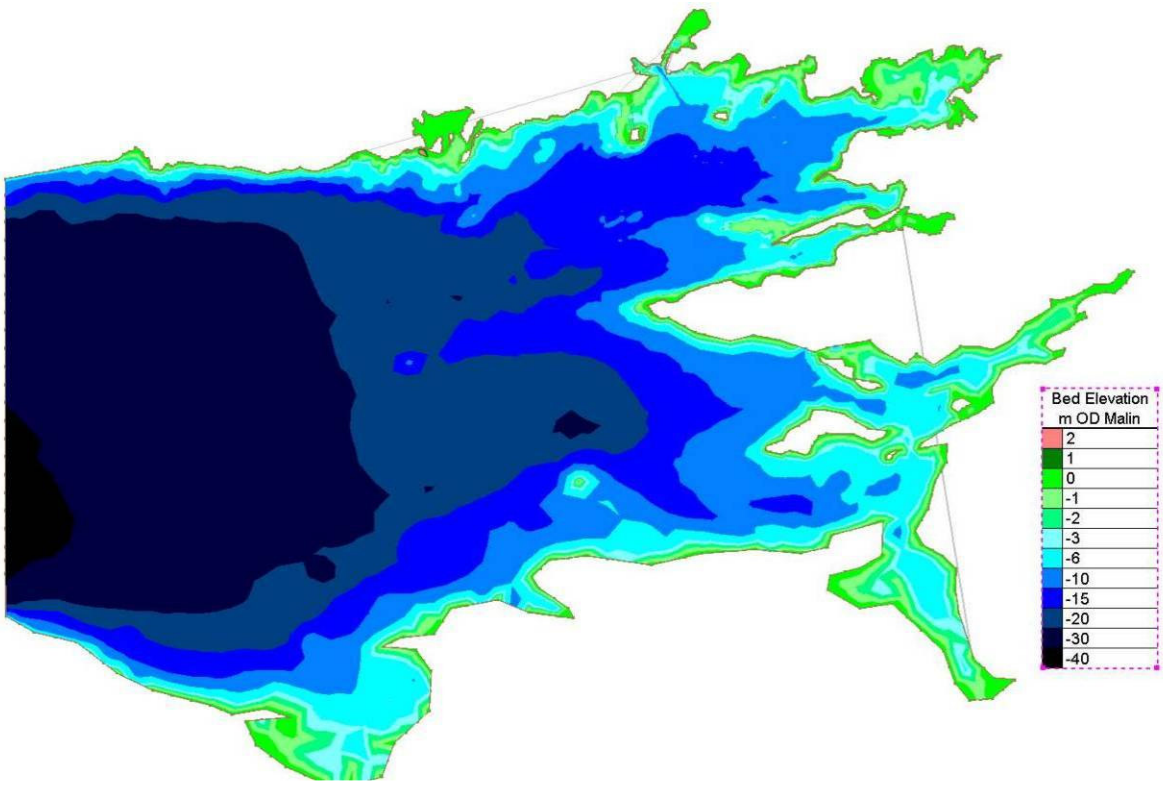

Figure 8.4.149 Hydrodynamic Model Finite element domain area and bathymetry for existing case

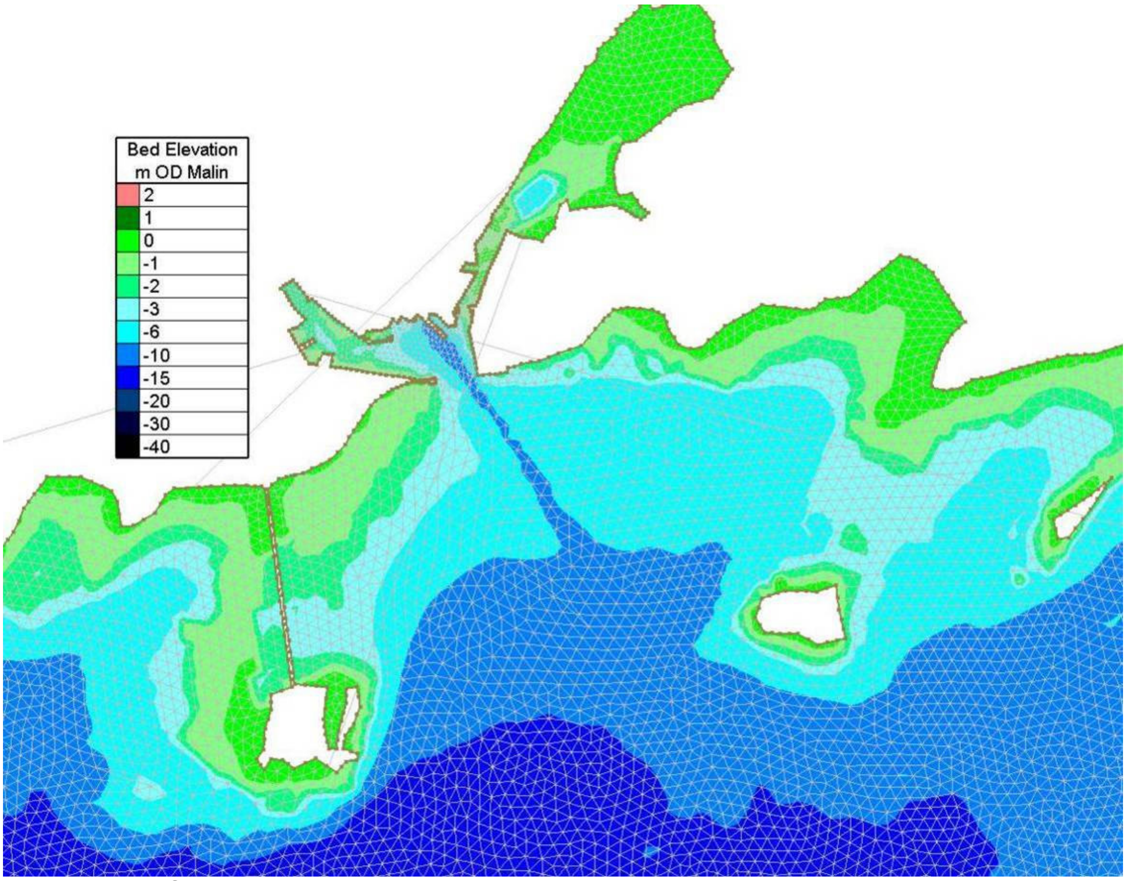

Figure 8.4.150 Close-up view of model refinement in the vicinity of the proposed harbour development

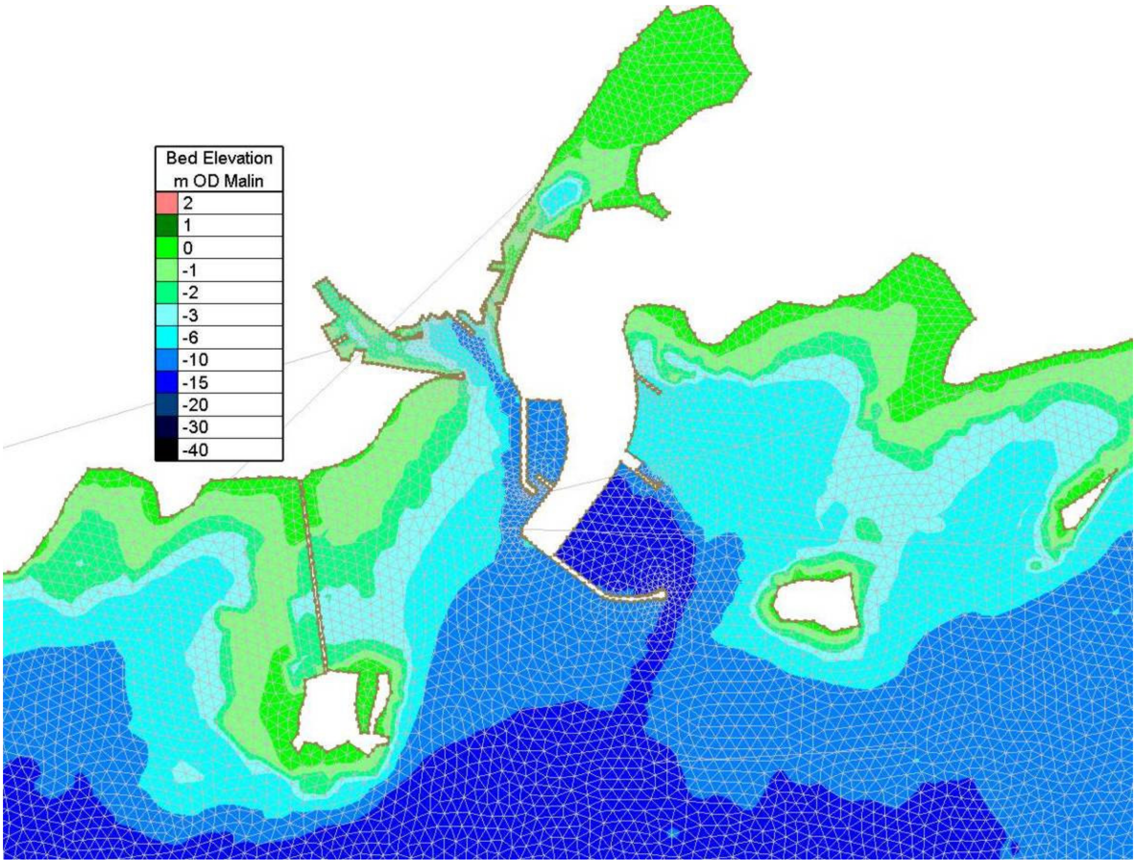

Figure 8.4.151 View of Hydrodynamic model mesh for proposed case in the vicinity of the Harbour Extension

8.4.7.9.4 Discussion on Flood Impact

The Hydrodynamic Simulations 1 to 4 listed above were performed to quantify the potential impact that the Harbour Extension structure and land reclamation would have on flooding upstream in Claddagh Basin and in the Docks Area north and northwest of Nimmo's Pier.

The velocity magnitudes at the four principal stages of the tidal cycle (*i.e.* Mid-ebb, Low water, Mid-flood and Highwater) are presented in Figures 8.4.152 to 8.4.183 for existing and proposed cases and for the four flood simulations listed in Section 8.4.3.8.9. These plots demonstrate the high velocities in the Corrib Channel at the Claddagh Basin throughout the tidal cycle associated with the Corrib inflow. Some reduction in flow velocities occur with the rising tide with the lowest velocities coinciding with the peak of the tide at highwater as a result of the increased flow depth by the tide. The highest velocities in the Corrib Estuary are associated with the low water period where the minimum flow depth occurs. High velocities are also evident in the channel to Lough Atalia where the width and depth are constrained resulting in locally high velocities during the short tidal filling and ebbing periods. It is clear from the plots that the proposed development diverts the plume westward along the new channel with a consistently southerly flow path towards Mutton Island. It is also apparent from the plots that no discernible change in the velocity magnitudes occur upstream of Nimmo's Pier as a result of the proposed development.

This is further verified from the time series plots of water surface elevation for each of the simulations 1 to 4 for 6 selected location points (refer to Figure 8.4.184 for location of these output points) presented in Figures 8.4.185 to 8.4.208. These time series of water elevation under varying tide and constant peak inflow from the River Corrib demonstrate that the proposed port development will not affect the maximum high water tide and combined fluvial flood levels within the study area and specifically upstream of Nimmo's Pier. Some slight increase in water levels is noted at a number of the output locations (sites 3, 4 and 5) during the low water period. The Low water period is not critical in respect to flooding and flood risk.

The analysis also demonstrates that a tidal surge specified at the open sea boundary of the model near Spiddal will arrive within the Claddagh basin producing virtually similar levels indicating the limited influence that flooding from the River Corrib has on peak tidal surge levels and that the source of flood risk to the Claddagh, Spanish Arch and docks area is a tidal storm surge event.

These fluvial or combined simulation runs show no discernible impact on the peak flood levels within the Claddagh Basin Area. The combination of the 100-year tide with the historical maximum recorded storm surge of 3.49 m O.D. Malin (25 year tide event) will easily exceed a 200 year flood event.

A 200-year combined event is typically a 200-year tide occurring with a median (2year) fluvial flood or a 100 year fluvial flood and a 5-year tide level. Larger tidal storm surge events (4 to 4.5 m OD) will result in a greater water depth for flow to take place and thus will result in no discernible impact on the upstream highwater flood levels within the Claddagh Basin as a result of the proposed development.

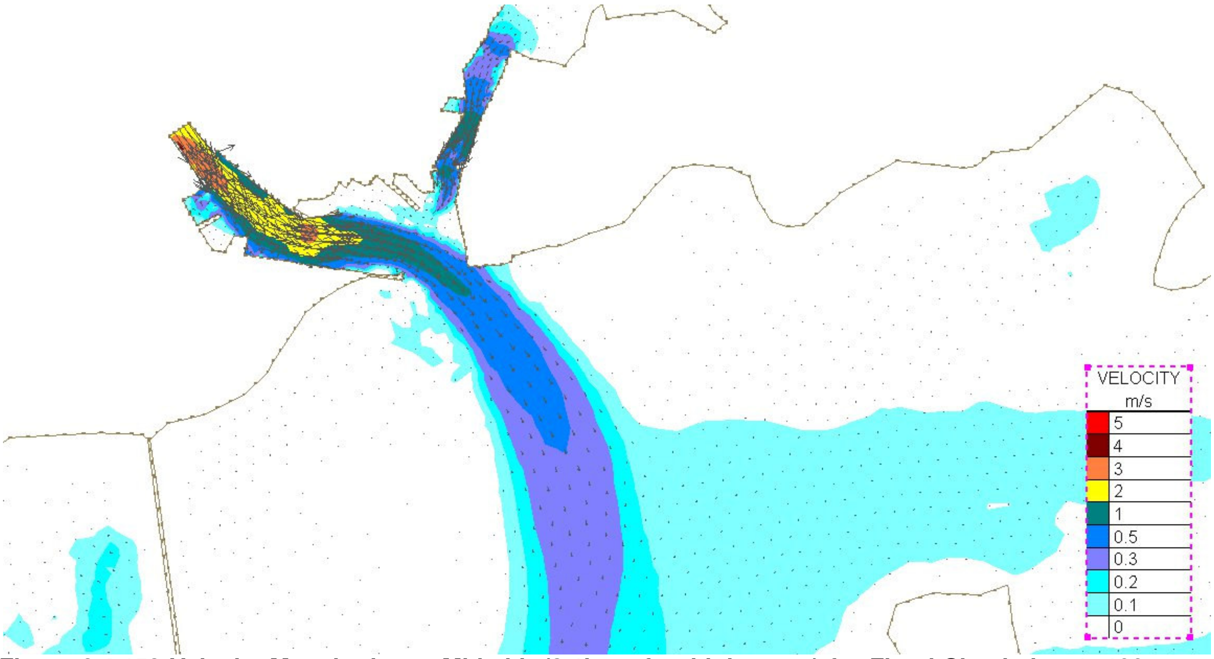

Figure 8.4.152 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 1- 100 year river flow and mean spring tide– Existing Case

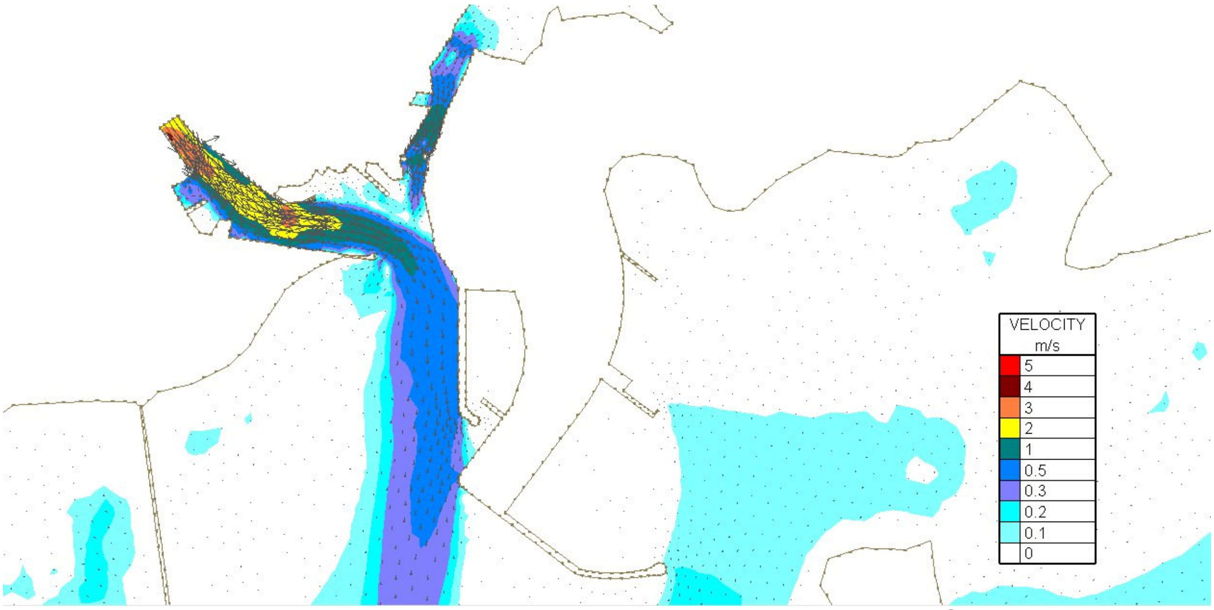

Figure 8.4.153 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 1- 100 year river flow and mean spring tide – Proposed Harbour Extension Case

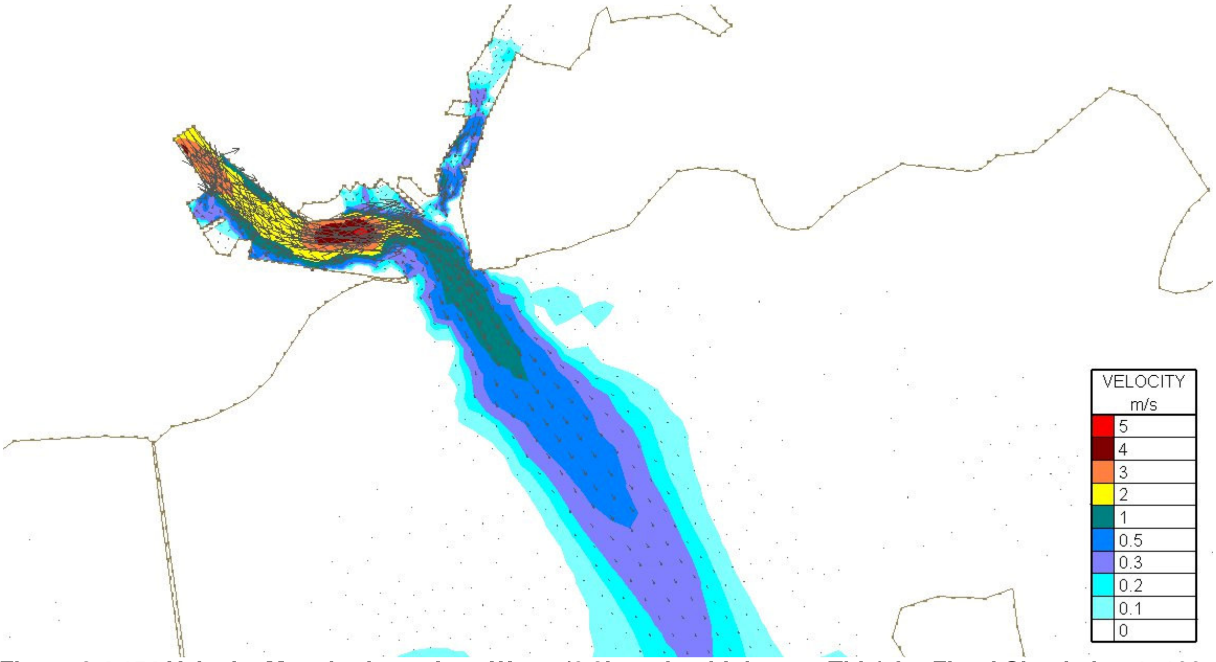

Figure 8.4.154 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 1- 100 year river flow and mean spring tide – Existing Case

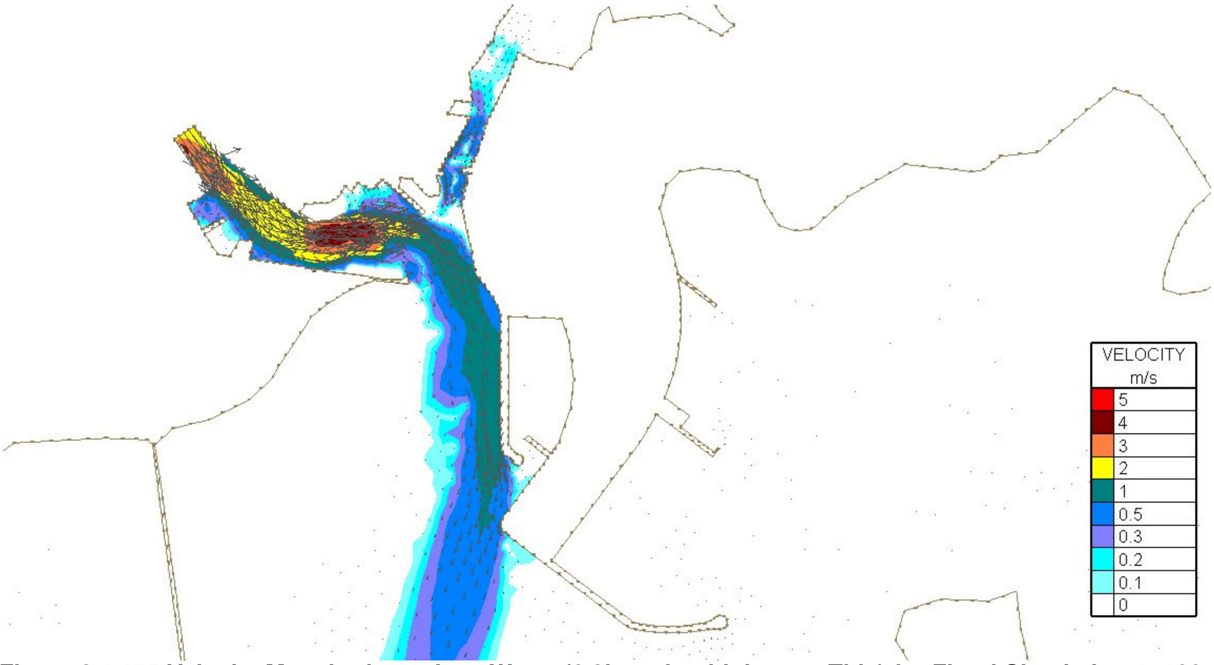

Figure 8.4.155 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 1- 100 year river flow and mean spring tide – Proposed Harbour Extension Case

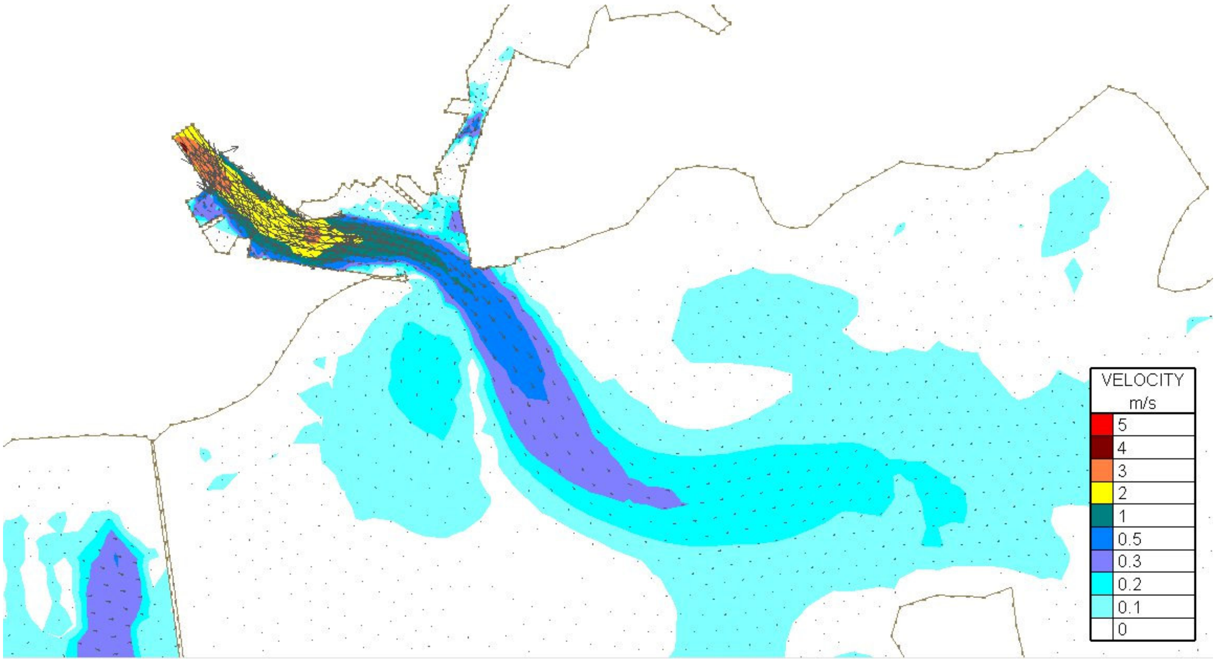

Figure 8.4.156 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 1 - 100 year river flow and mean spring tide – Existing Case

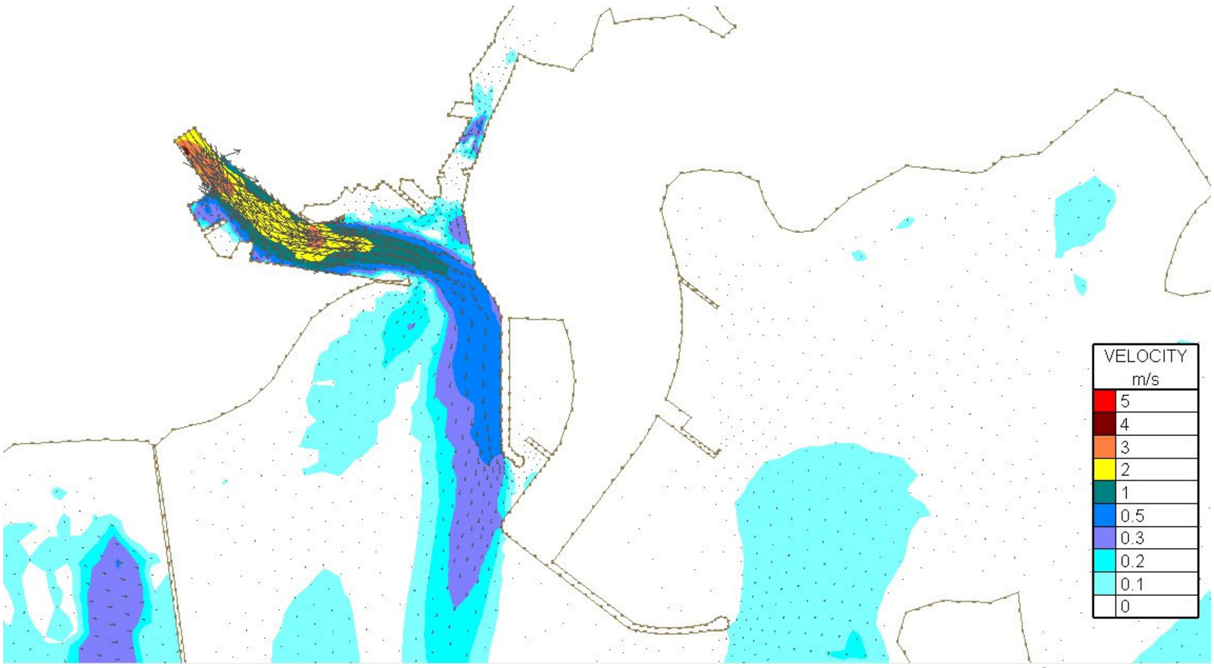

Figure 8.4.157 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 1 – 100 year river flow and mean spring tide – Proposed Harbour Extension Case

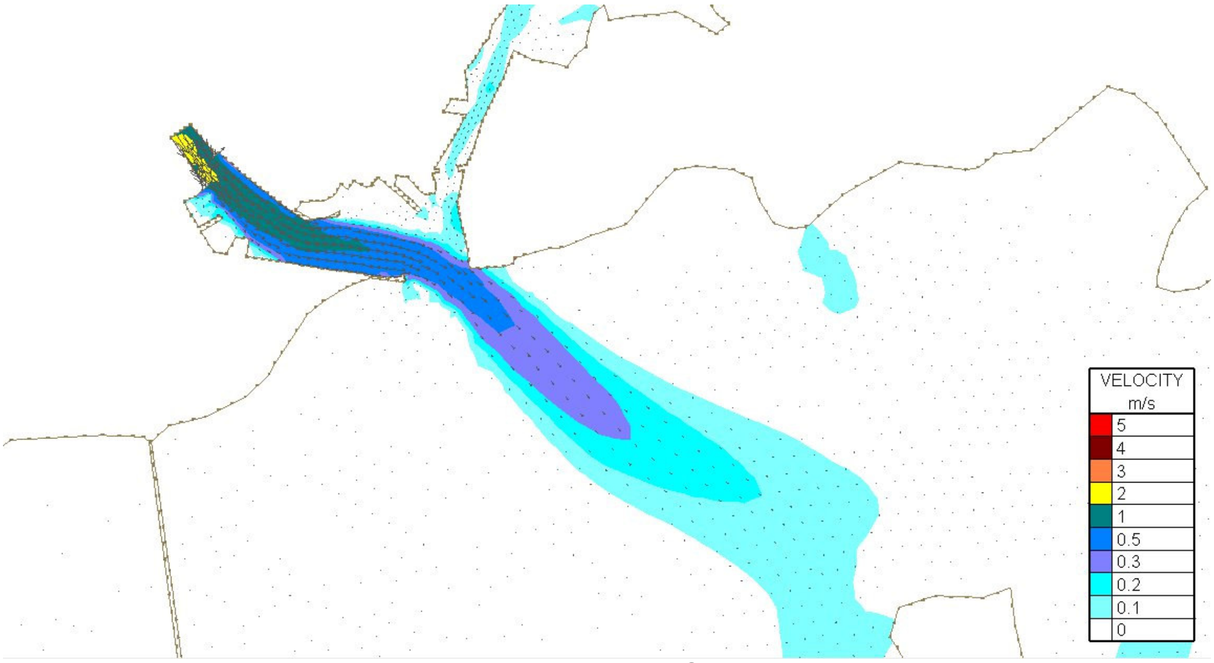

Figure 8.4.158 Velocity Magnitudes at highwater for Flood Simulation 1 – 100 year river flow and mean spring tide – Existing Case

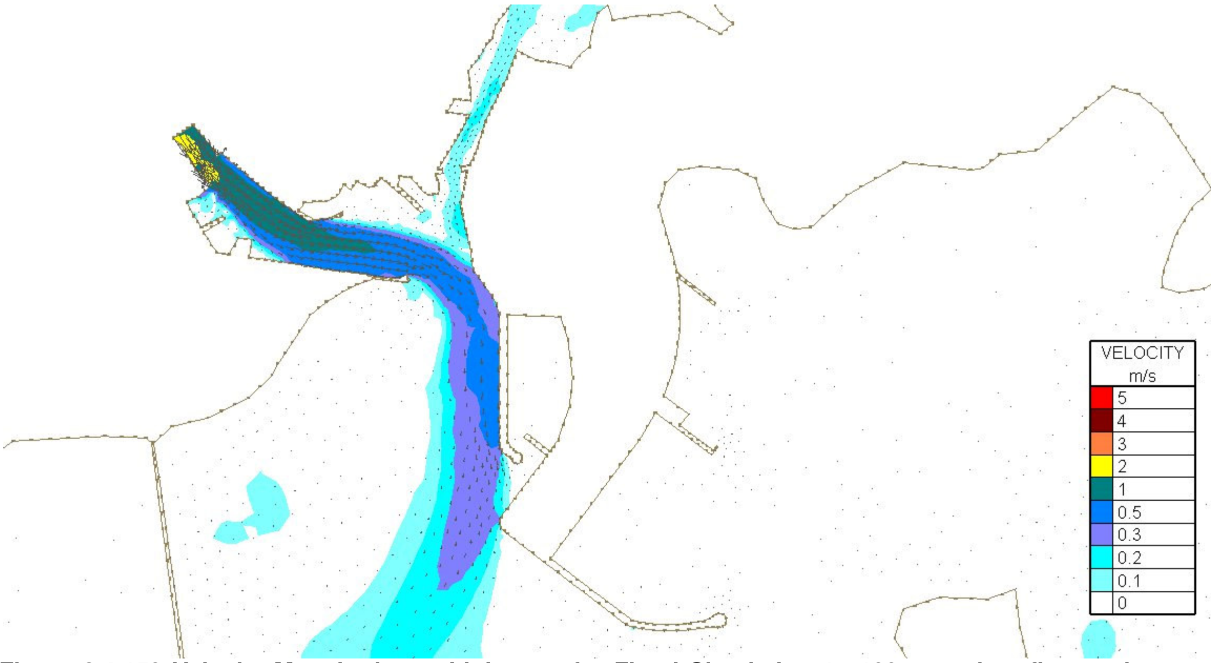

Figure 8.4.159 Velocity Magnitudes at highwater for Flood Simulation 1– 100 year river flow and mean spring tide – Proposed Harbour Extension Case

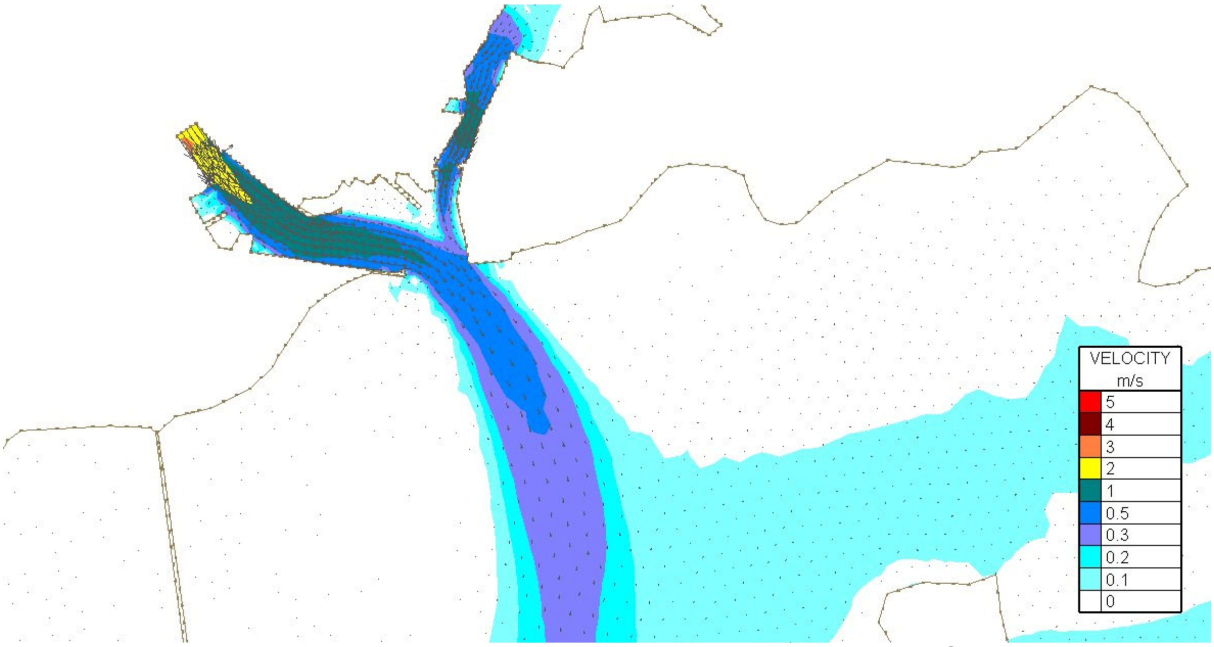

Figure 8.4.160 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 2- 100 year river flow and a 3.44m tide – Existing Case

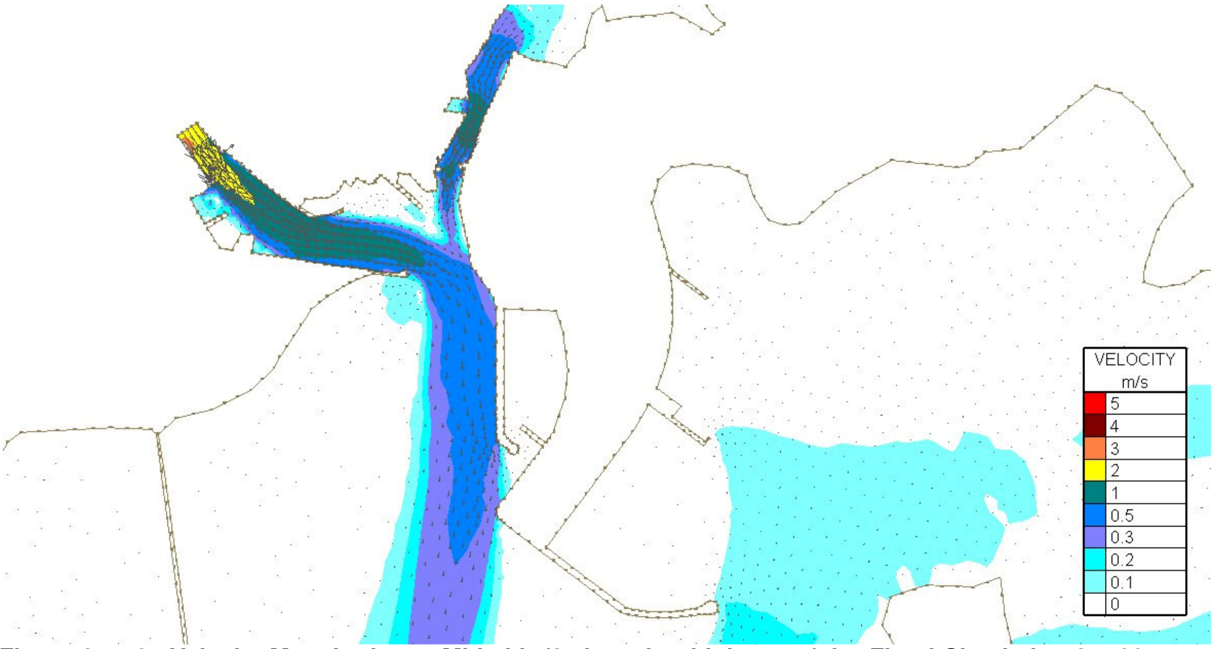

Figure 8.4.161 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 2- 100 year river flow and a 3.44m tide – Proposed Harbour Extension Case

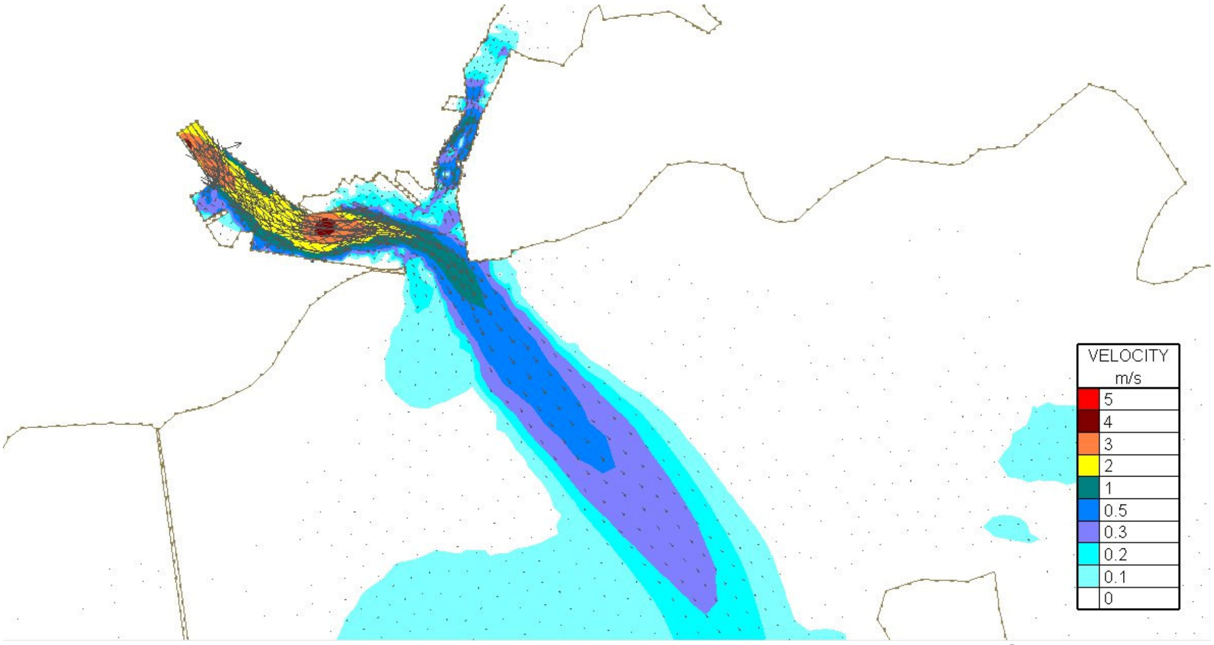

Figure 8.4.162 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 2- 100 year river flow and a 3.49m tide – Existing Case

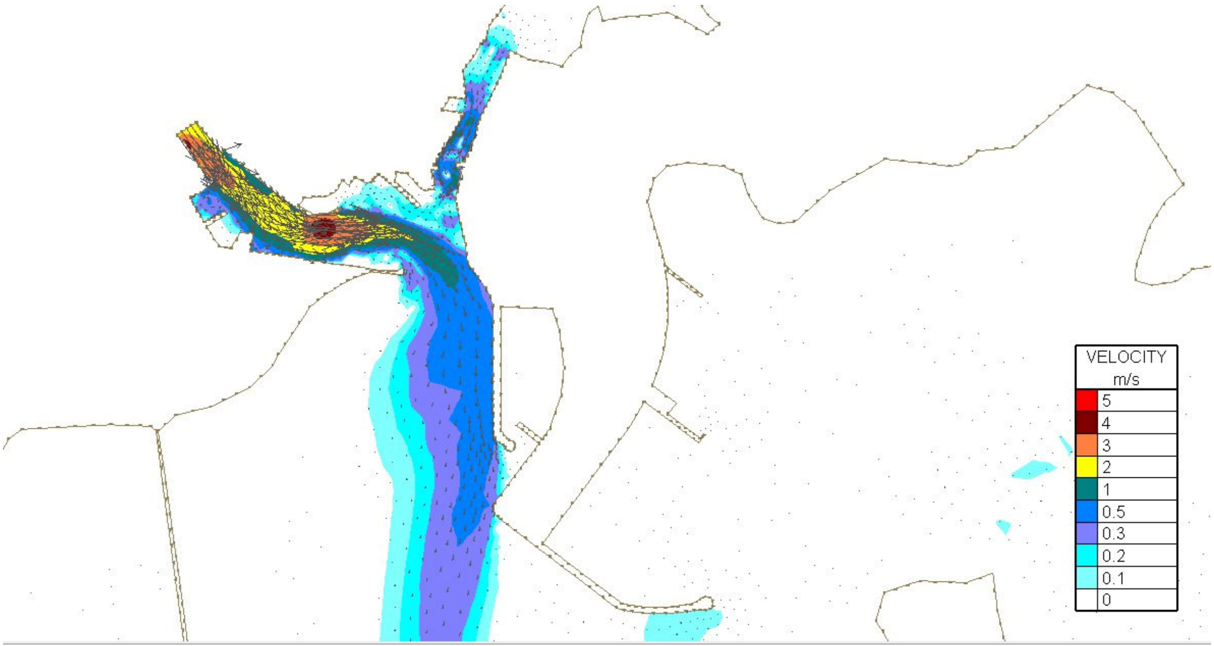

Figure 8.4.163 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 2 - 100 year river flow and a 3.49m tide – Proposed Harbour Extension Case

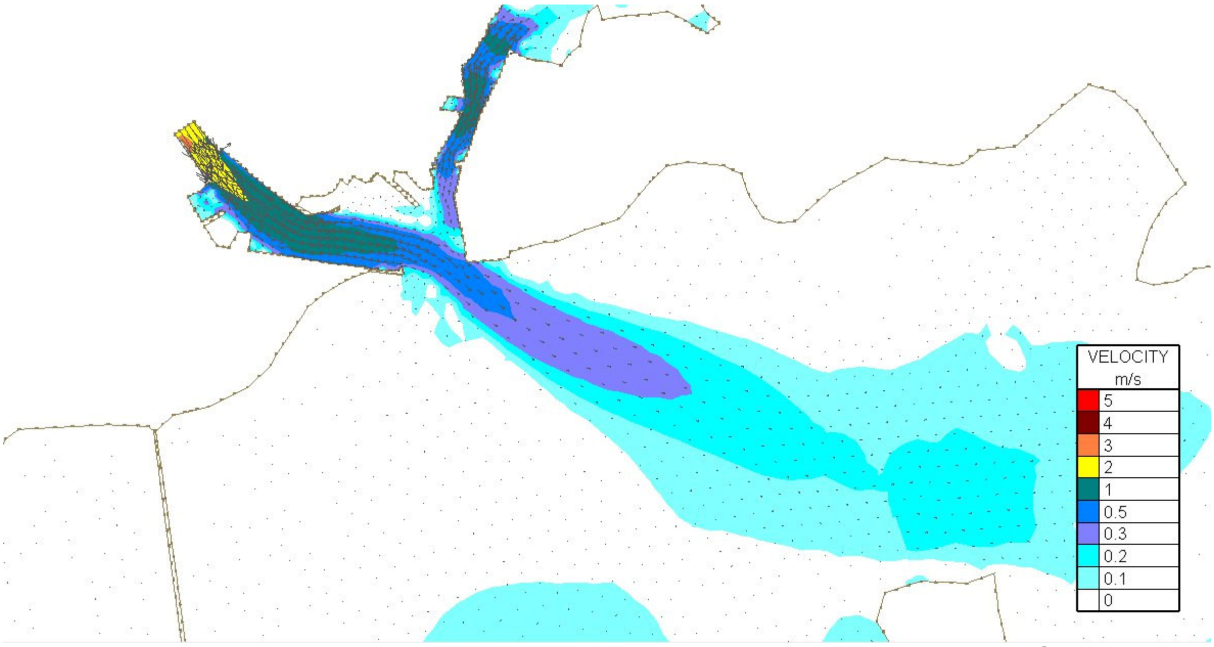

Figure 8.4.164 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 2 – 100 year river flow and a 3.49m tide – Existing Case

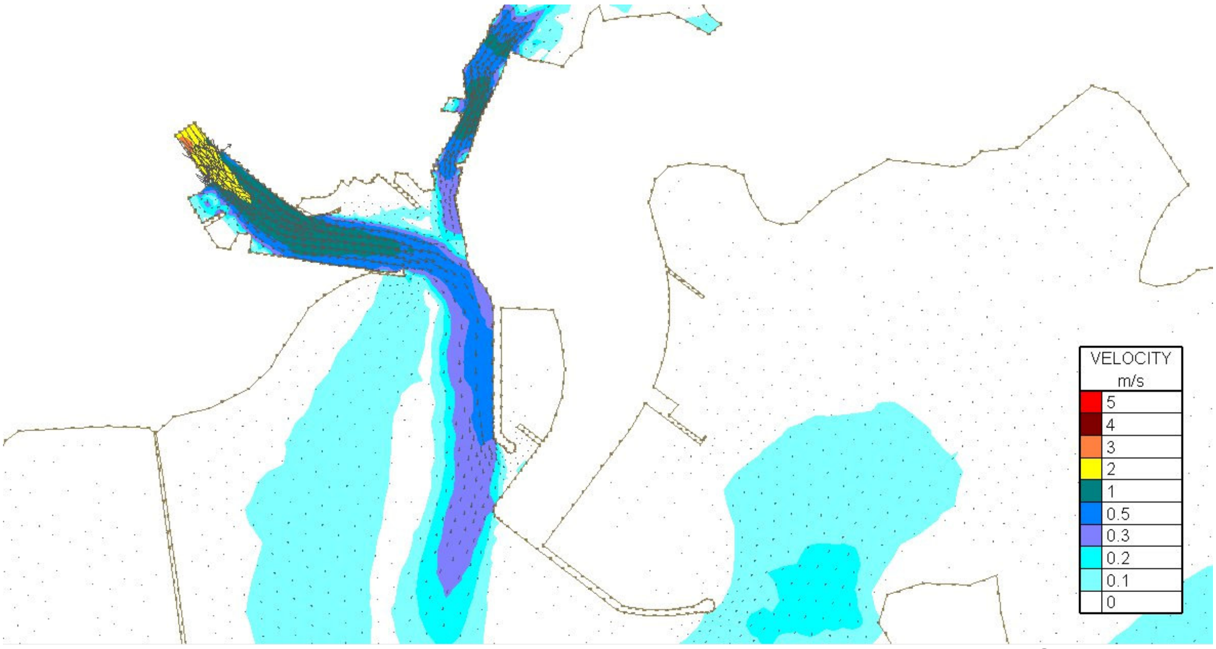

Figure 8.4.165 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 2 – 100 year river flow and a 3.49m tide – Proposed Harbour Extension Case

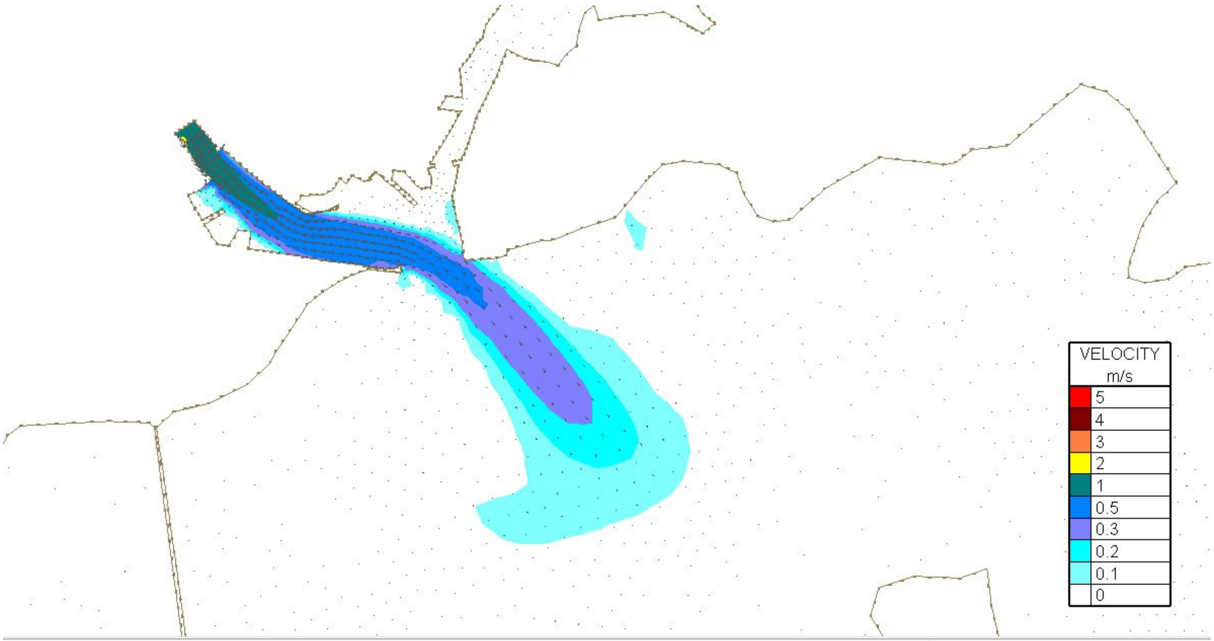

Figure 8.4.166 Velocity Magnitudes at highwater for Flood Simulation 2 – 100 year river flow and a 3.49m tide – Existing Case

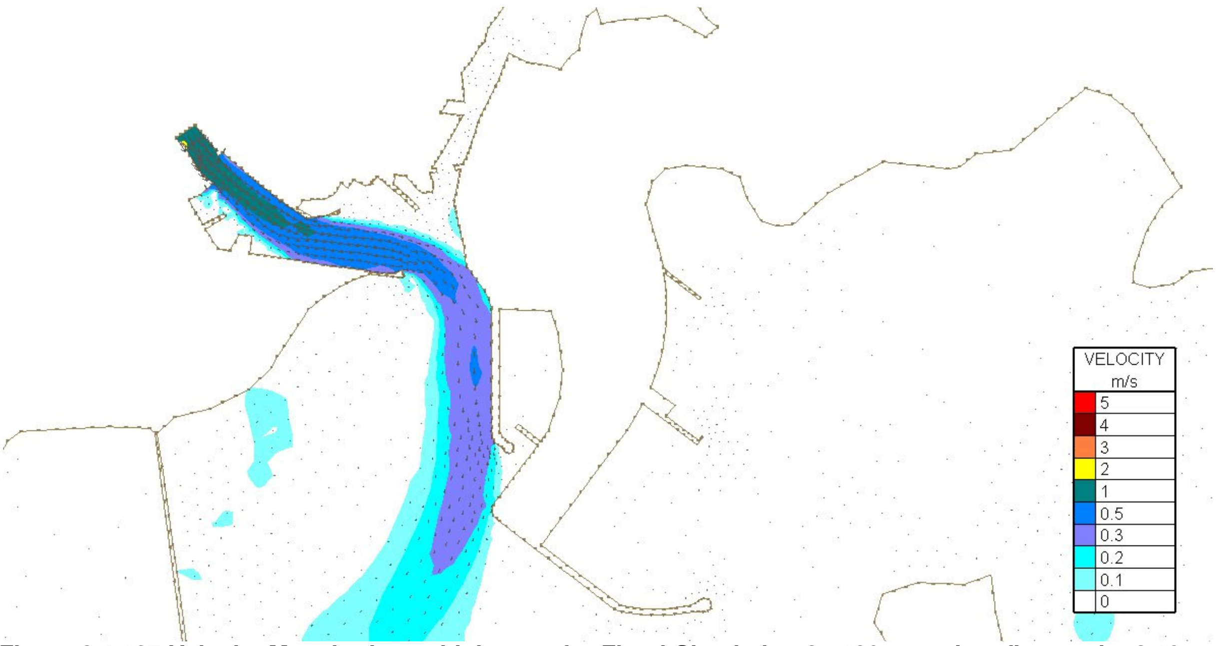

Figure 8.4.167 Velocity Magnitudes at highwater for Flood Simulation 2– 100 year river flow and a 3.49m tide – Proposed Harbour Extension Case

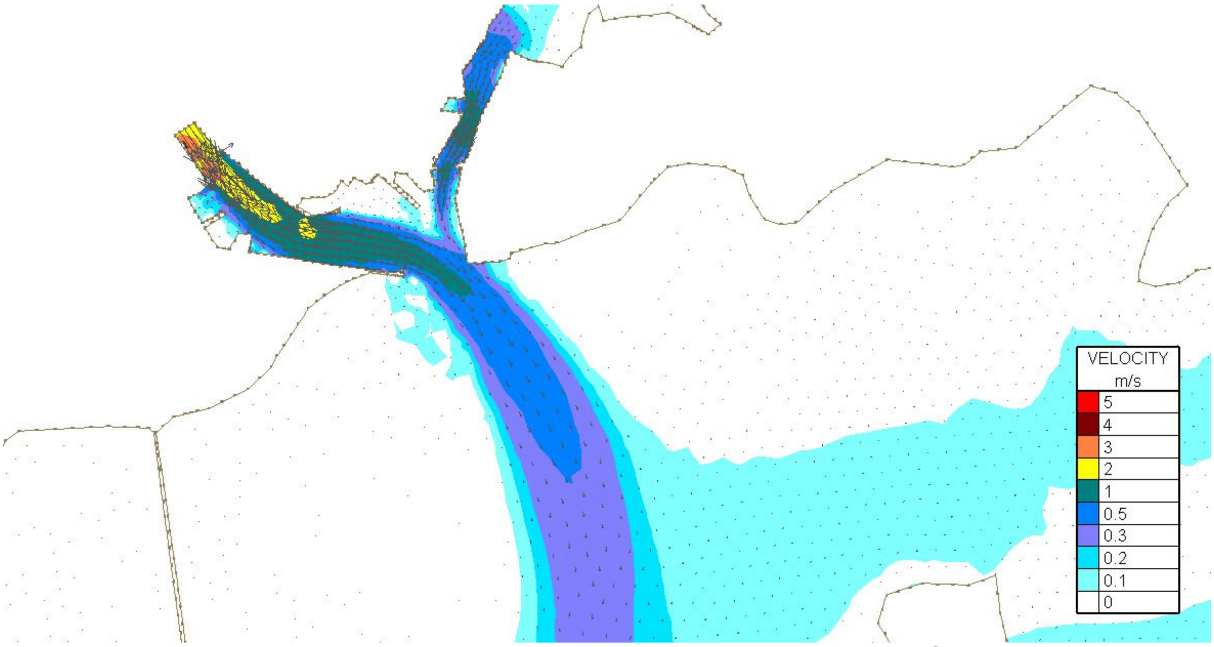

Figure 8.4.168 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 3 - river plus 20% climate change and a 3.49m tide – Existing Case

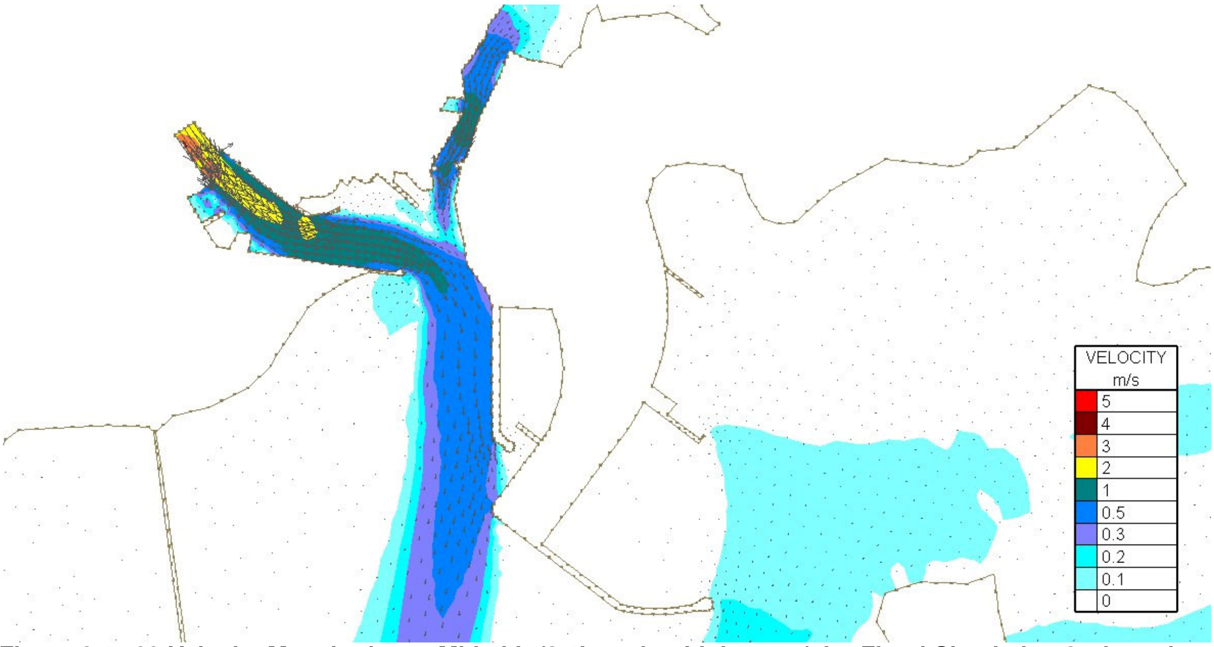

Figure 8.4.169 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 3- river plus 20% climate change and a 3.49m tide – Proposed Harbour Extension Case

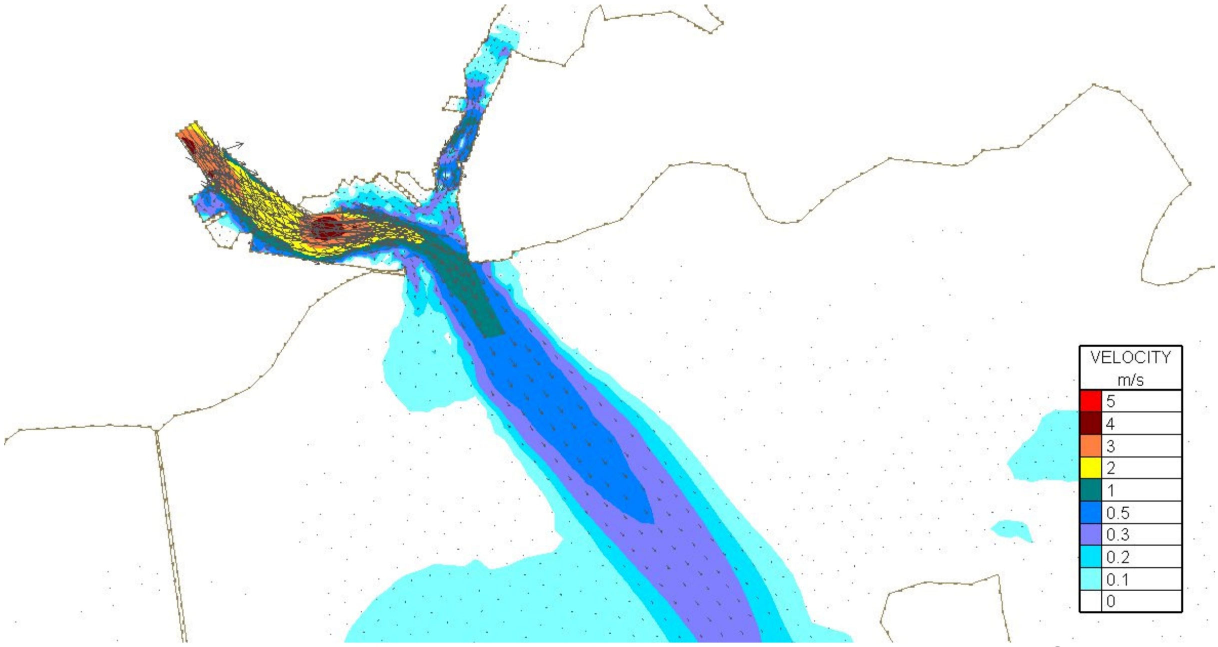

Figure 8.4.170 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 3- river plus 20% climate change and a 3.49m tide – Existing Case

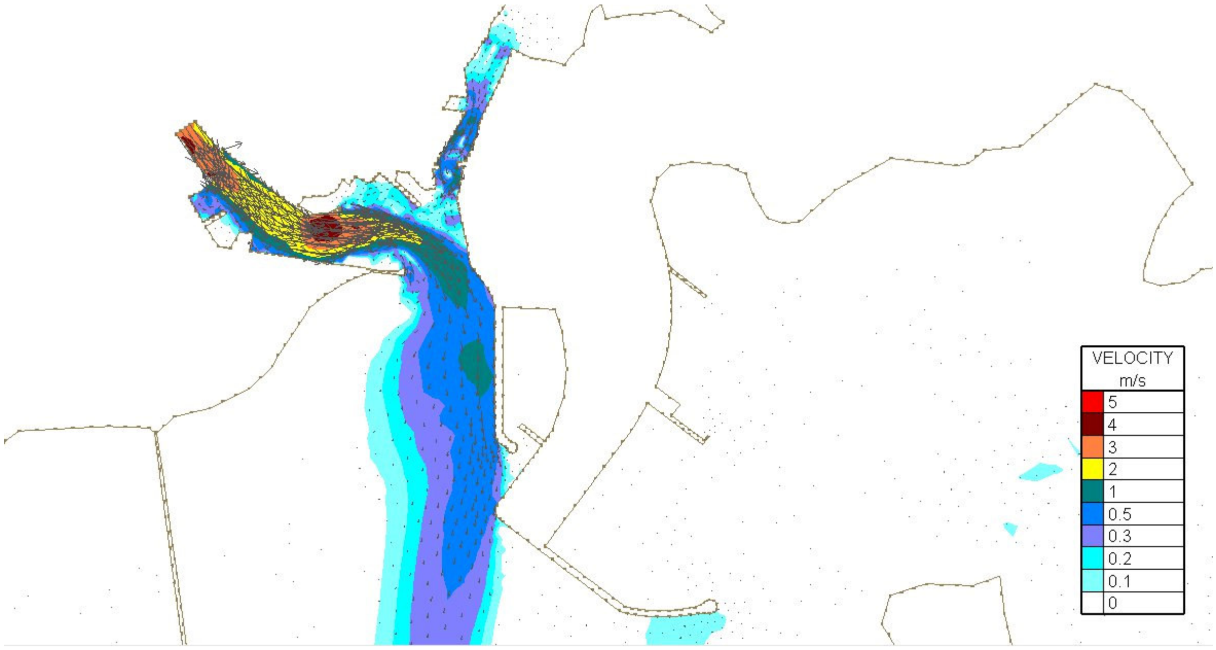

Figure 8.4.171 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 3- river plus 20% climate change and a 3.49m tide – Proposed Harbour Extension Case

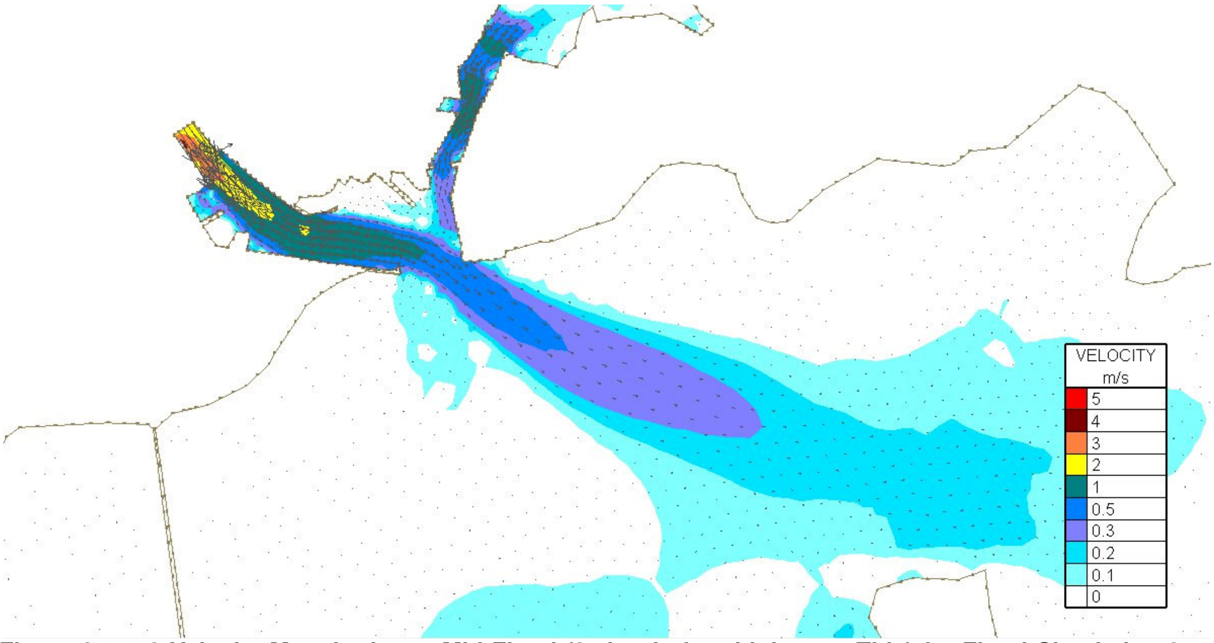

Figure 8.4.172 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 3 – river plus 20% climate change and a 3.49m tide – Existing Case

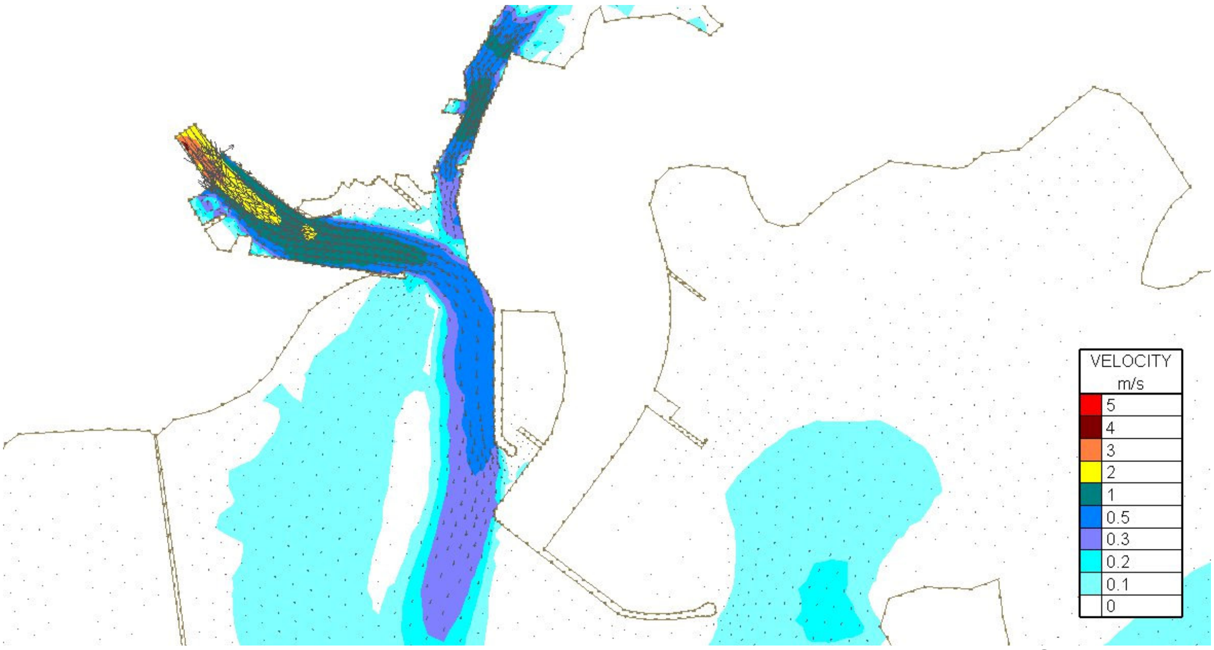

Figure 8.4.173 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 3 – river plus 20% climate change and a 3.49m tide – Proposed Harbour Extension Case

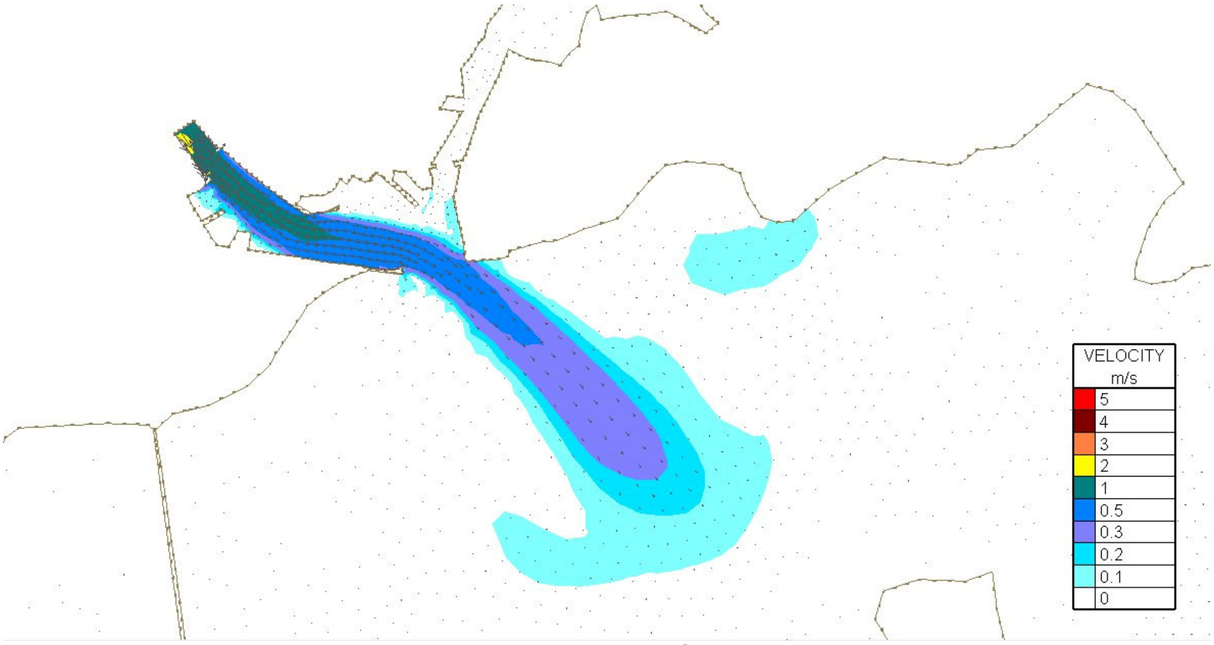

Figure 8.4.174 Velocity Magnitudes at highwater for Flood Simulation 3 – river plus 20% climate change and a 3.49m tide – Existing Case

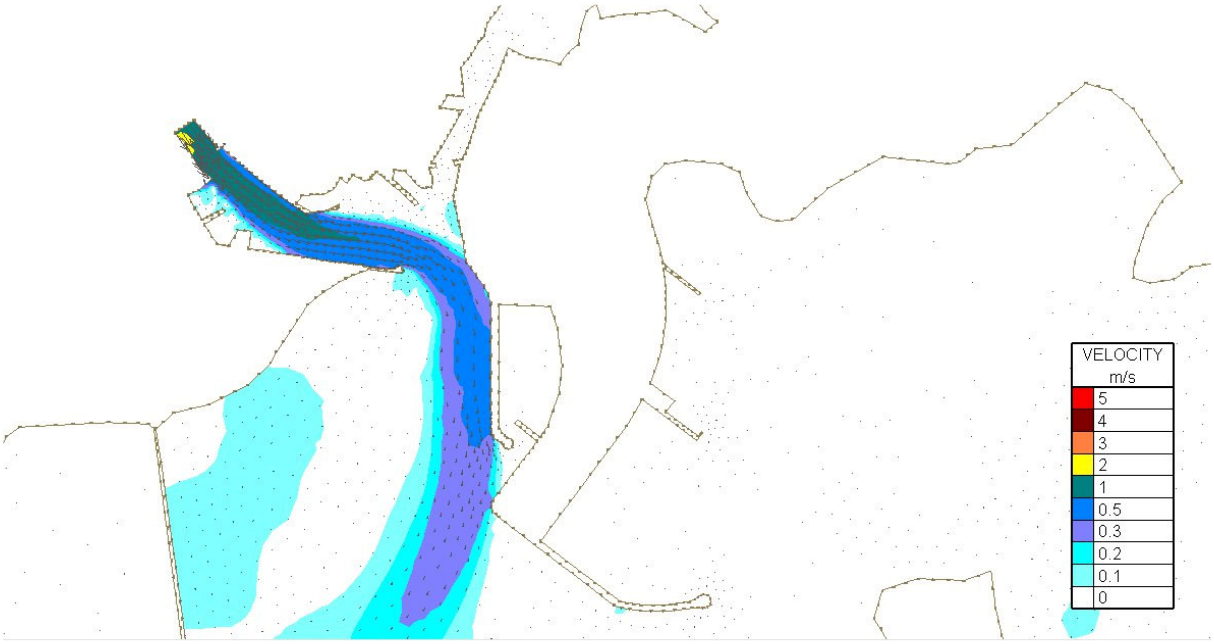

Figure 8.4.175 Velocity Magnitudes at highwater for Flood Simulation 3 – river plus 20% climate change and a 3.49m tide – Proposed Harbour Extension Case

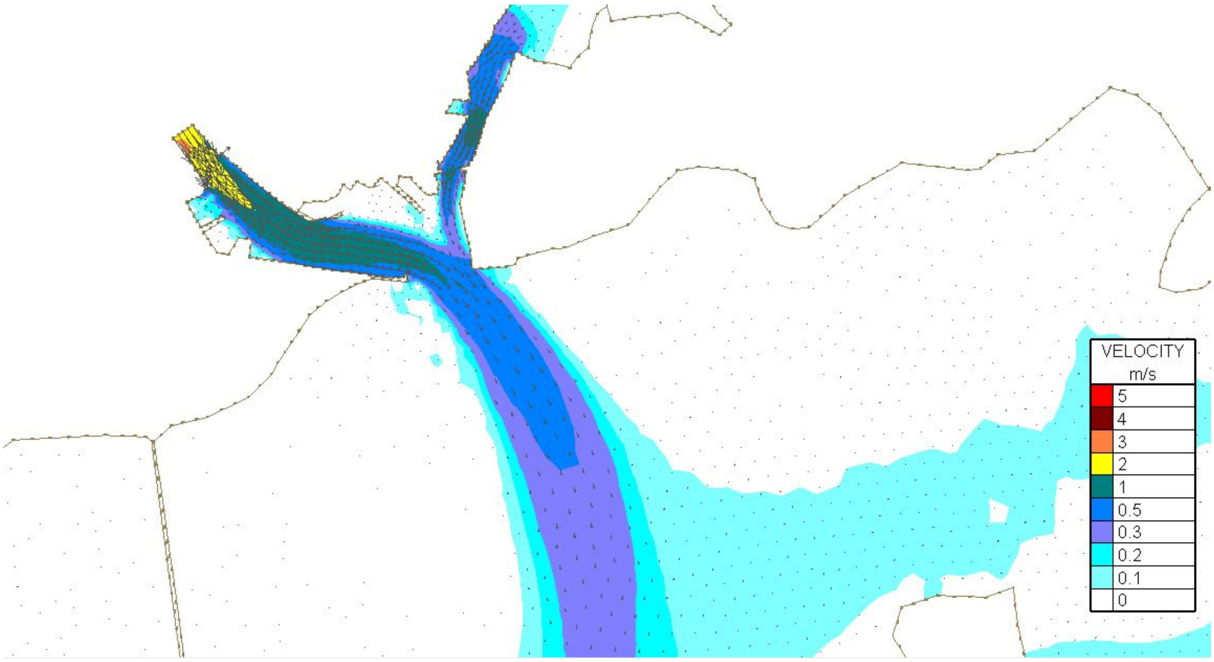

Figure 8.4.176 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 4 – 100 year river flow and a tide of 4.146m (200 year tide) – Existing Case

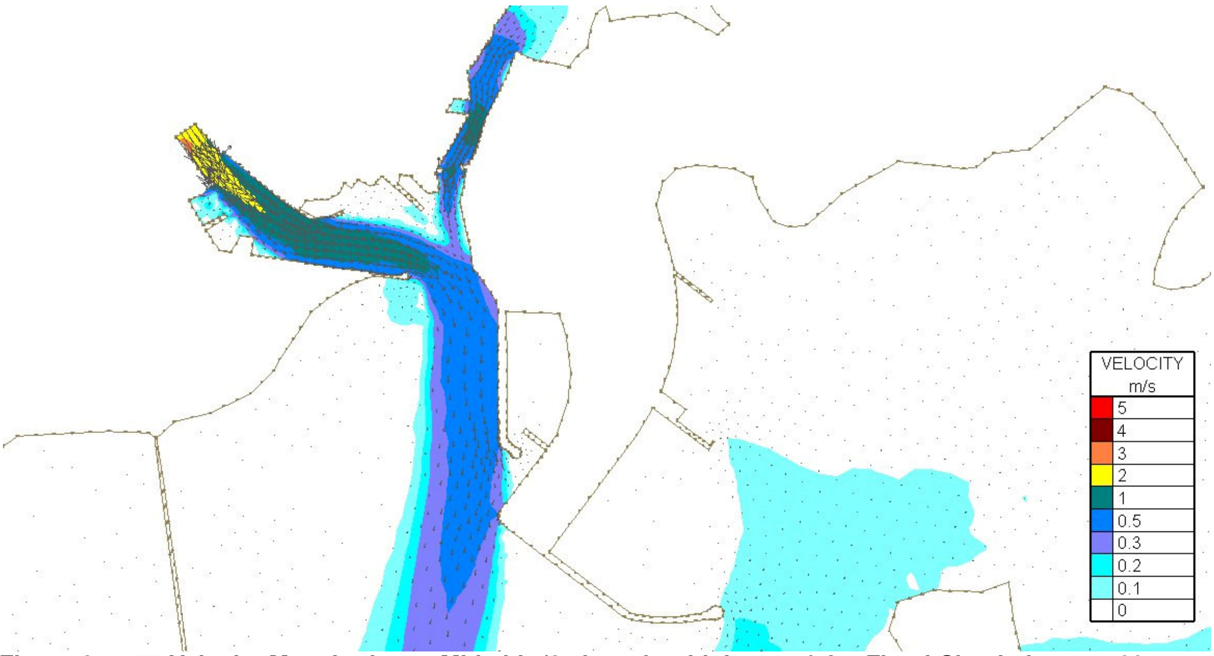

Figure 8.4.177 Velocity Magnitudes at Mid-ebb (3.1hrs after highwater) for Flood Simulation 4– 100 year river flow and a tide of 4.146m (200 year tide) – Proposed Harbour Extension Case

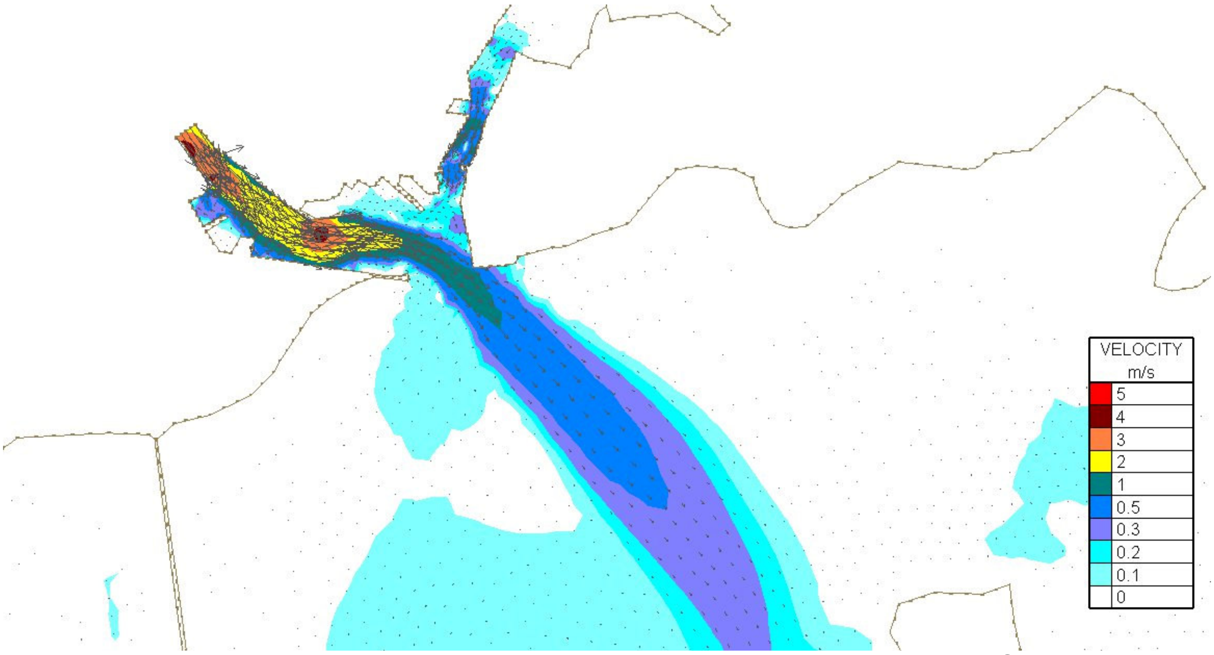

Figure 8.4.178 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 4– 100 year river flow and a tide of 4.146m (200 year tide) – Existing Case

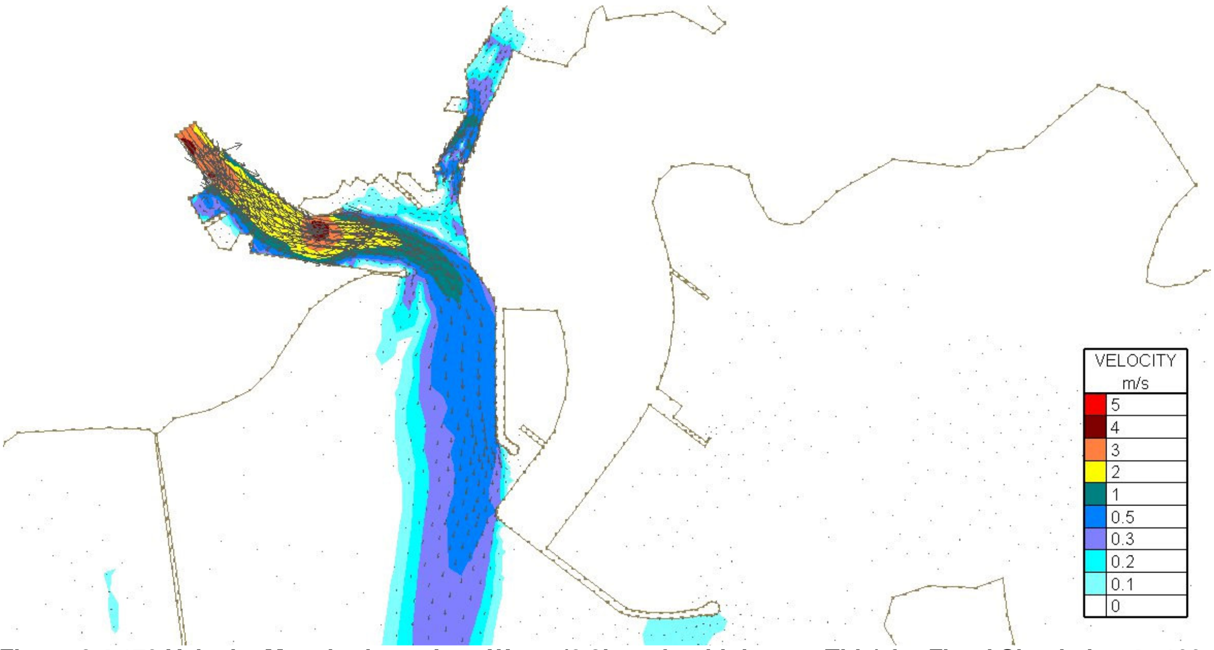

Figure 8.4.179 Velocity Magnitudes at Low Water (6.2hrs after highwater Tide) for Flood Simulation 4– 100 year river flow and a tide of 4.146m (200 year tide) – Proposed Harbour Extension Case

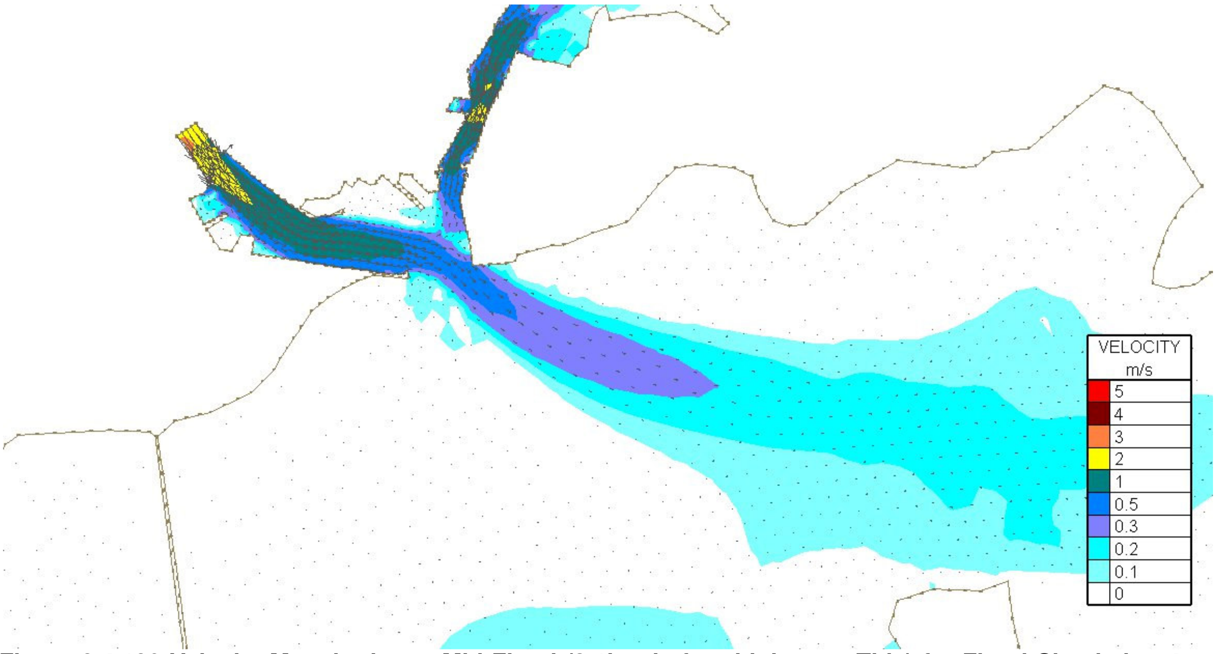

Figure 8.4.180 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 4 – 100 year river flow and a tide of 4.146m (200 year tide) – Existing Case

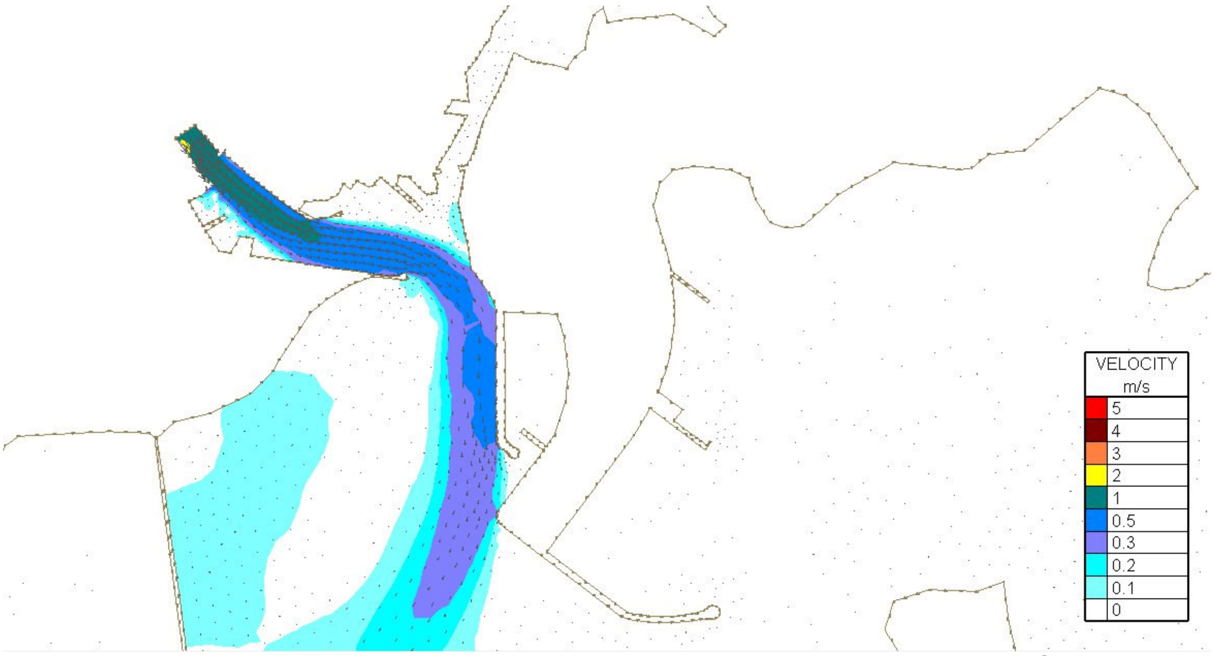

Figure 8.4.181 Velocity Magnitudes at Mid Flood (3.1hrs before highwater Tide) for Flood Simulation 4 – 100 year river flow and a tide of 4.146m (200 year tide) – Proposed Harbour Extension Case

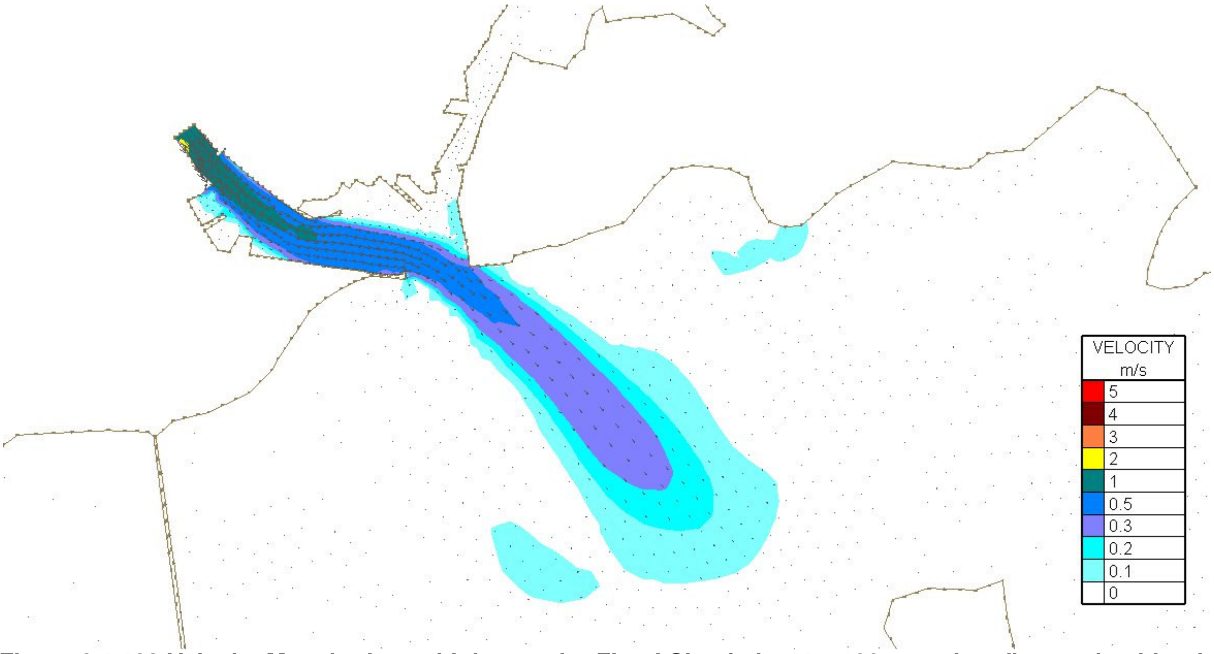

Figure 8.4.182 Velocity Magnitudes at highwater for Flood Simulation 4 – 100 year river flow and a tide of 4.146m (200 year tide) – Existing Case

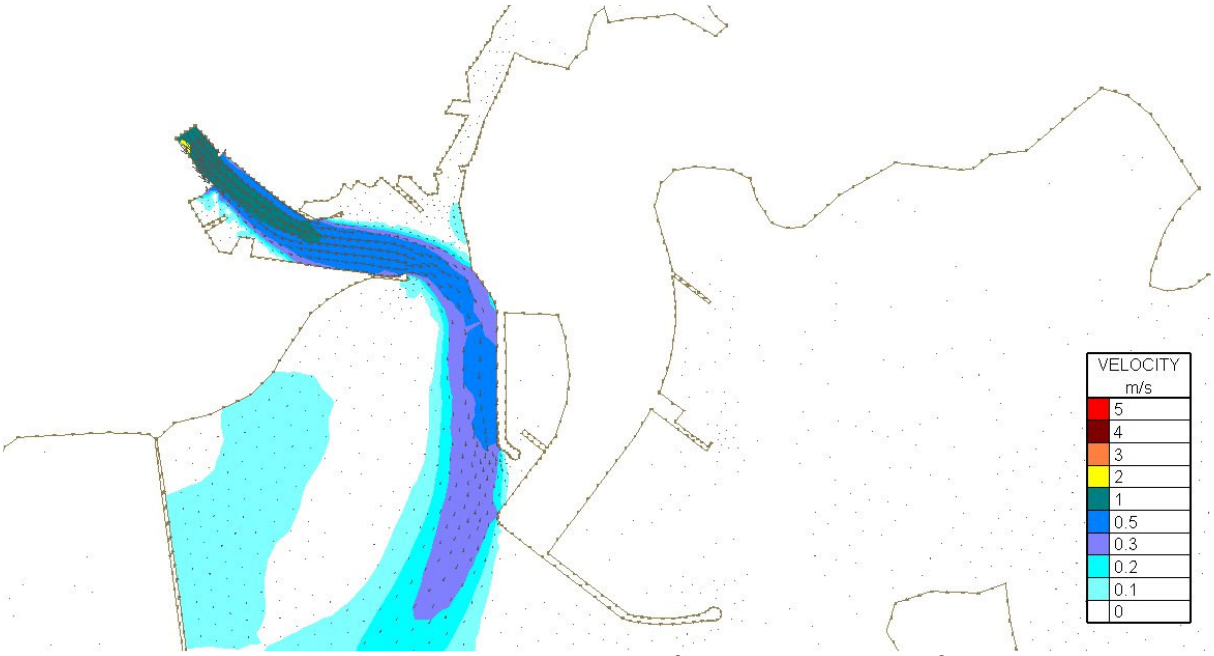

Figure 8.4.183 Velocity Magnitudes at highwater for Flood Simulation 4 for Flood Simulation 1 – 100 year river flow and a tide of 4.146m (200 year tide) – Proposed Harbour Extension Case

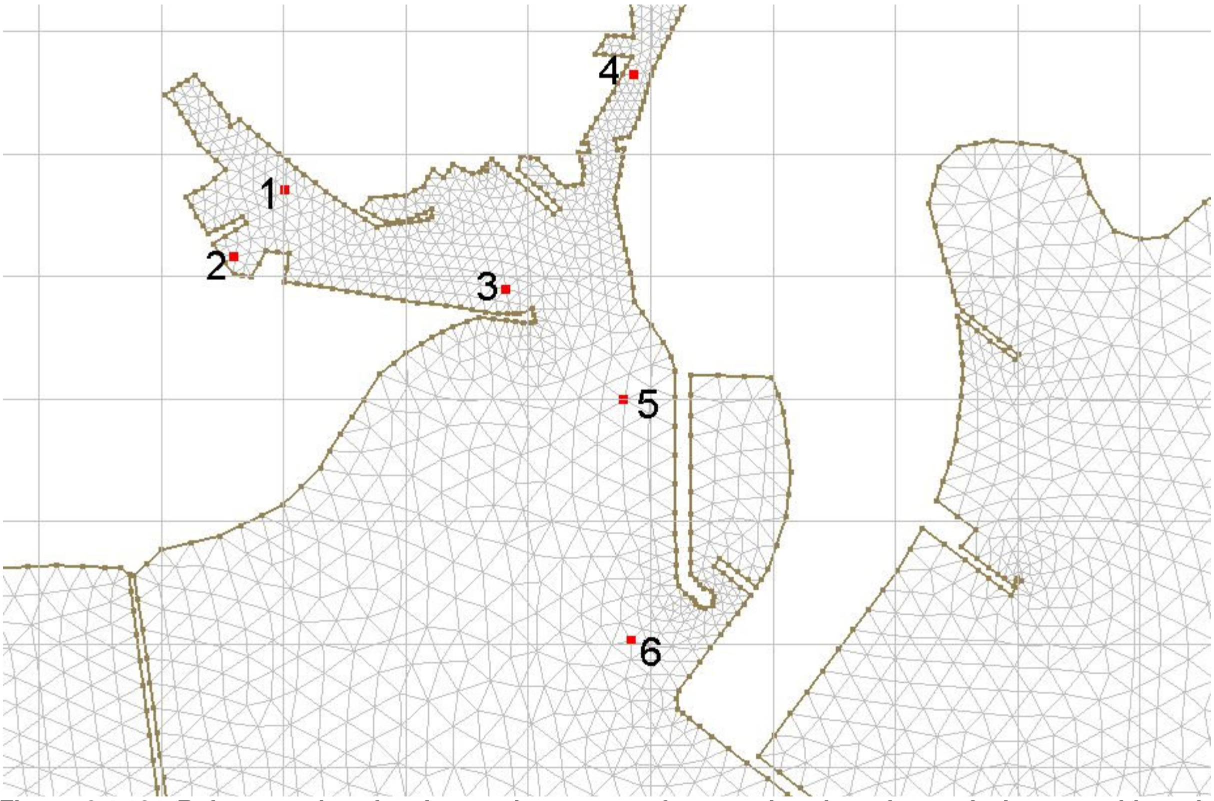

Figure 8.4.184 Reference sites for time series output of water elevations for analysis runs with and without proposed development

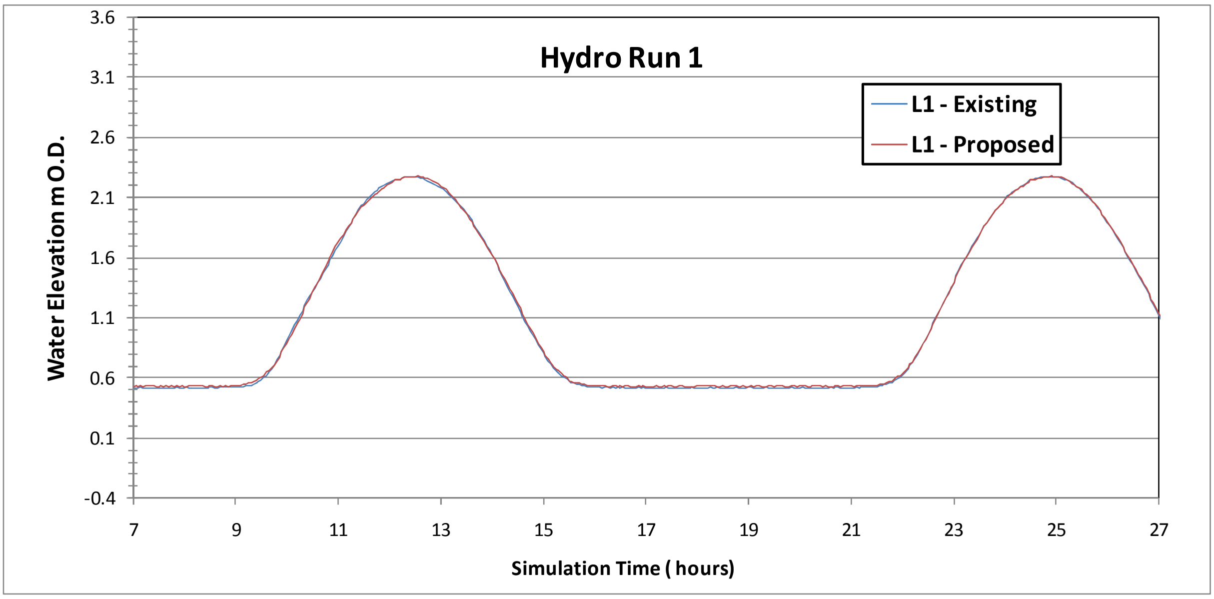

Figure 8.4.185 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 1 at reference site 1 in Claddagh Basin adjacent to Spanish Arch

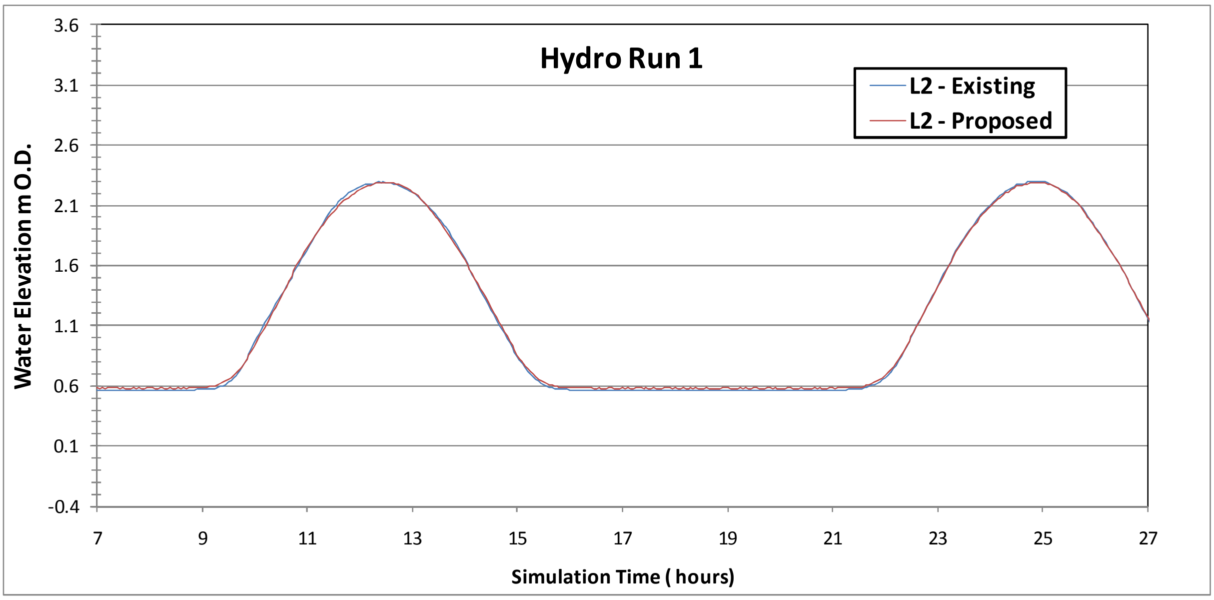

Figure 8.4.186 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 1 at reference site 2 in Claddagh Basin

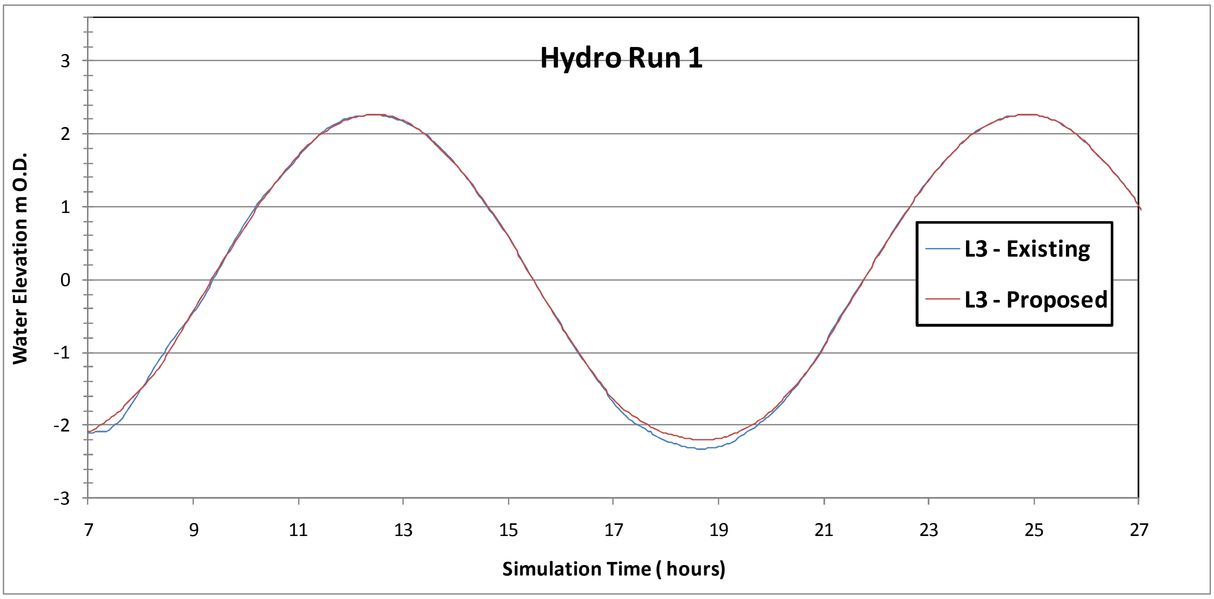

Figure 8.4.187 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 1 at reference site 3 in estuary inside Nimmo's Pier

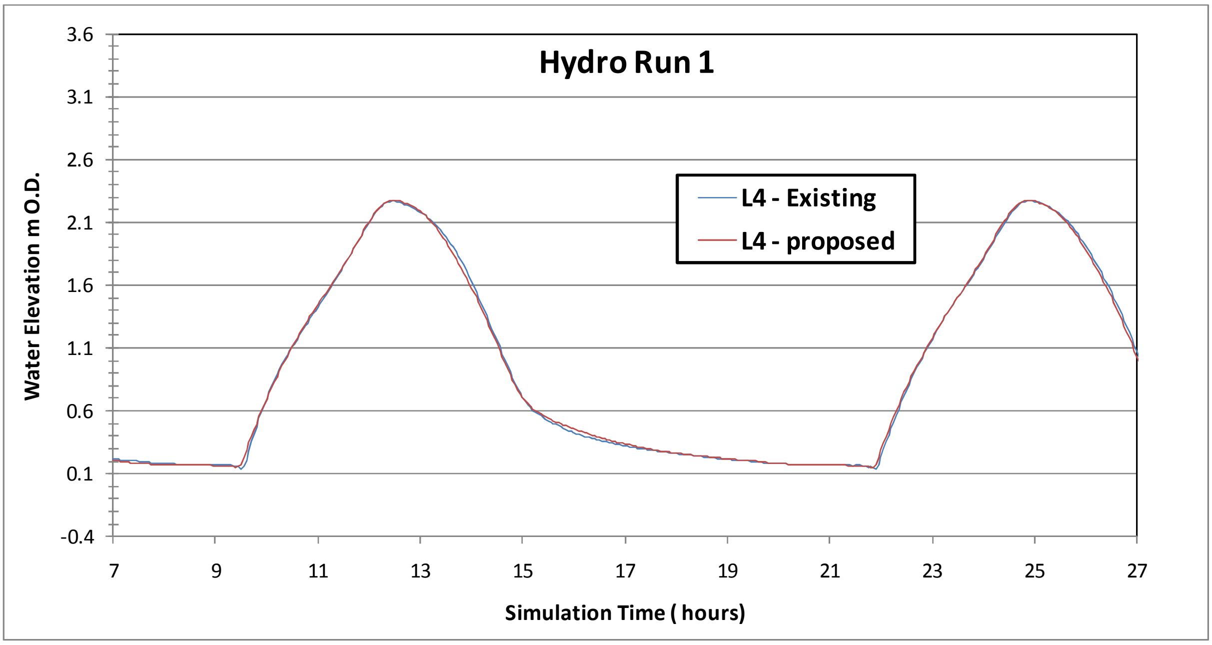

Figure 8.4.188 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 1 at reference site 4 – entrance channel to Lough Atalia

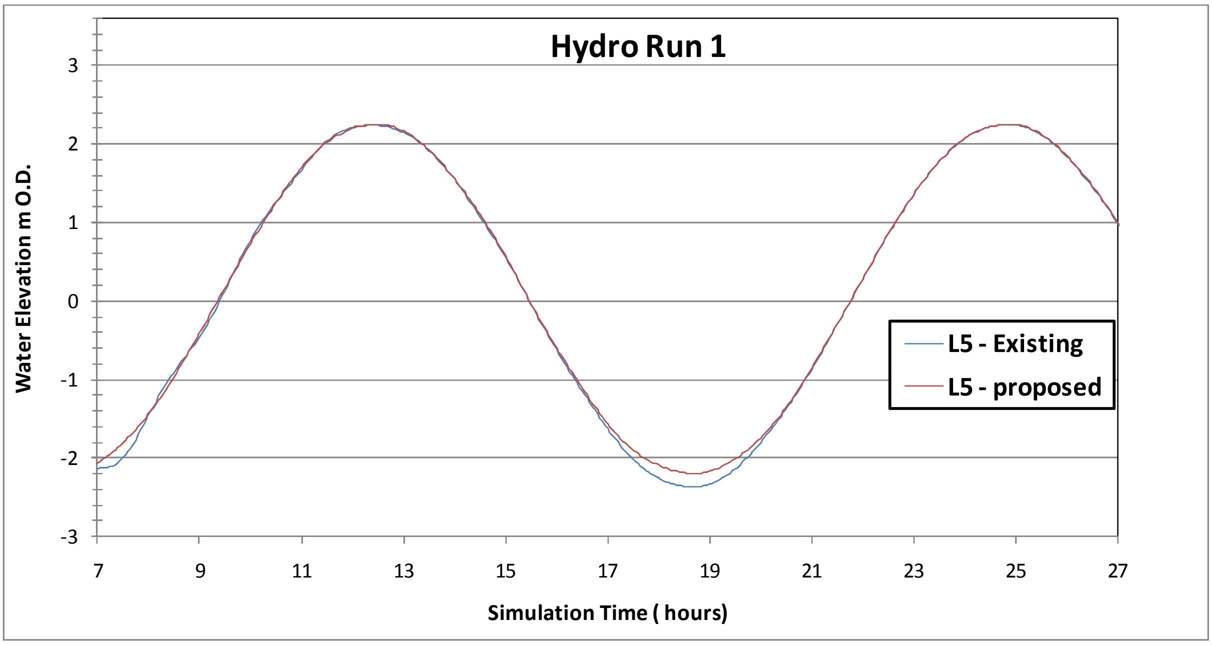

Figure 8.4.189 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 1 at reference site 5 – new dredge channel at Marina Breakwater

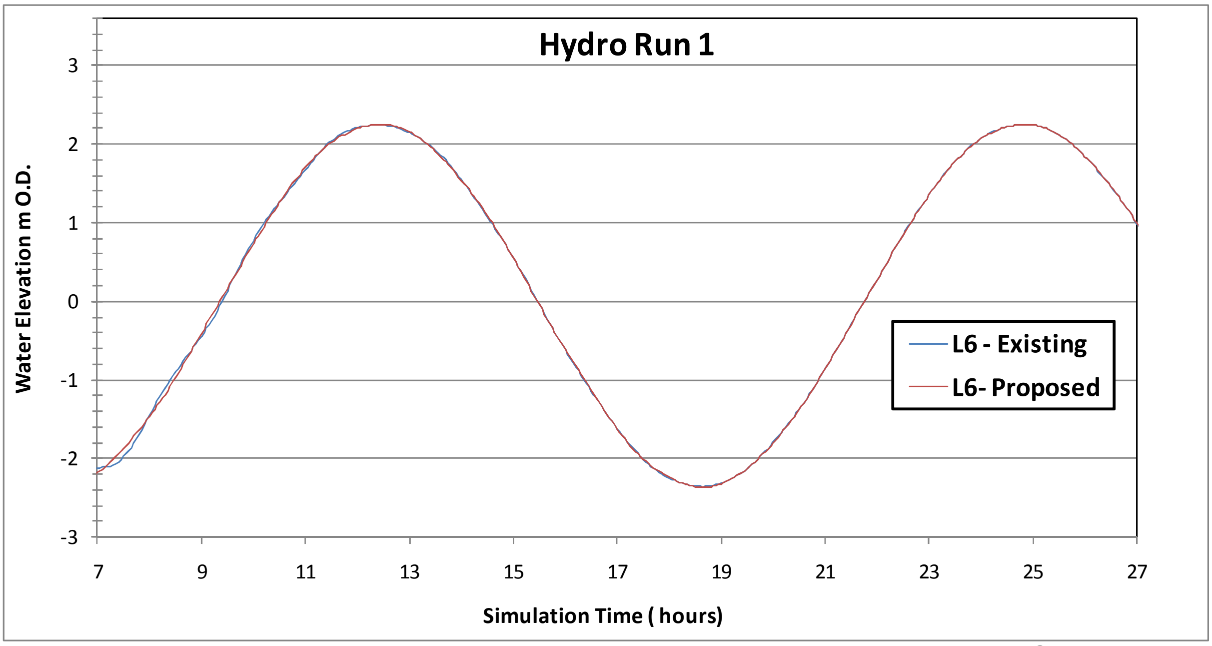

Figure 8.4.190 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 1 at reference site 6 – new dredge channel south of Marina entrance

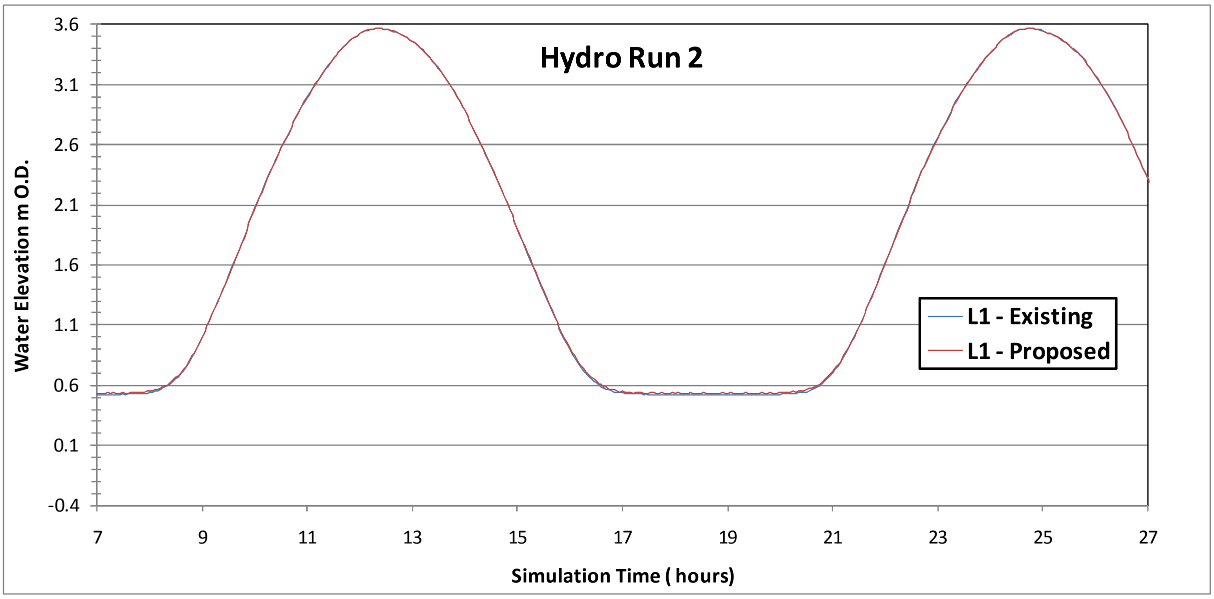

Figure 8.4.191 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 2 at reference site 1 in Claddagh Basin adjacent to Spanish Arch

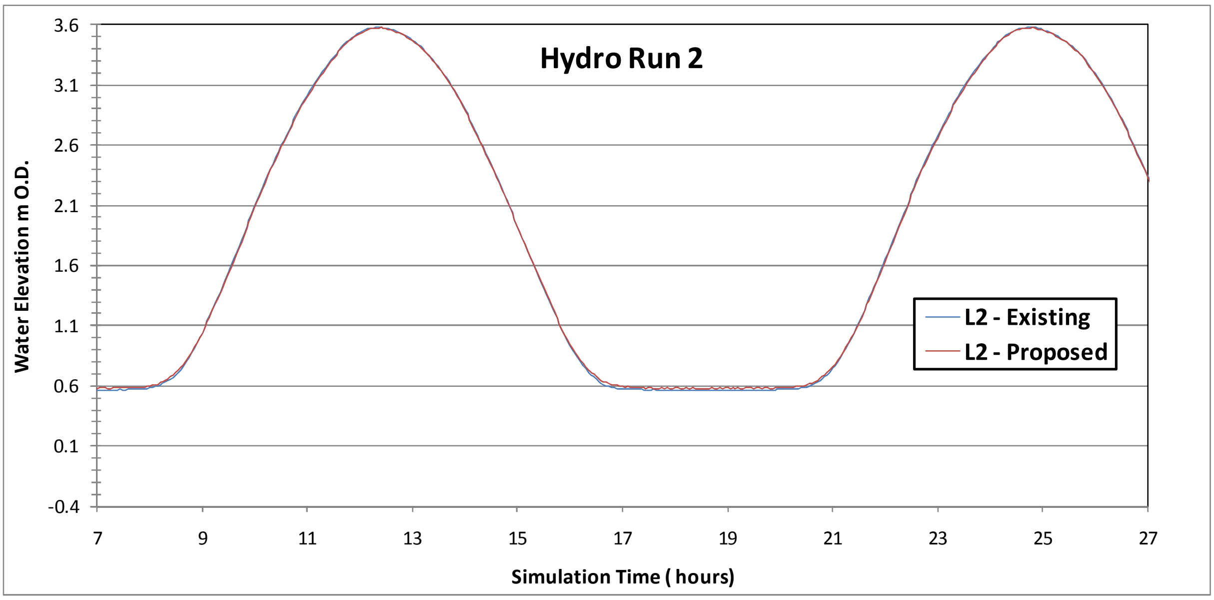

Figure 8.4.192 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 2 at reference site 2 in Claddagh Basin

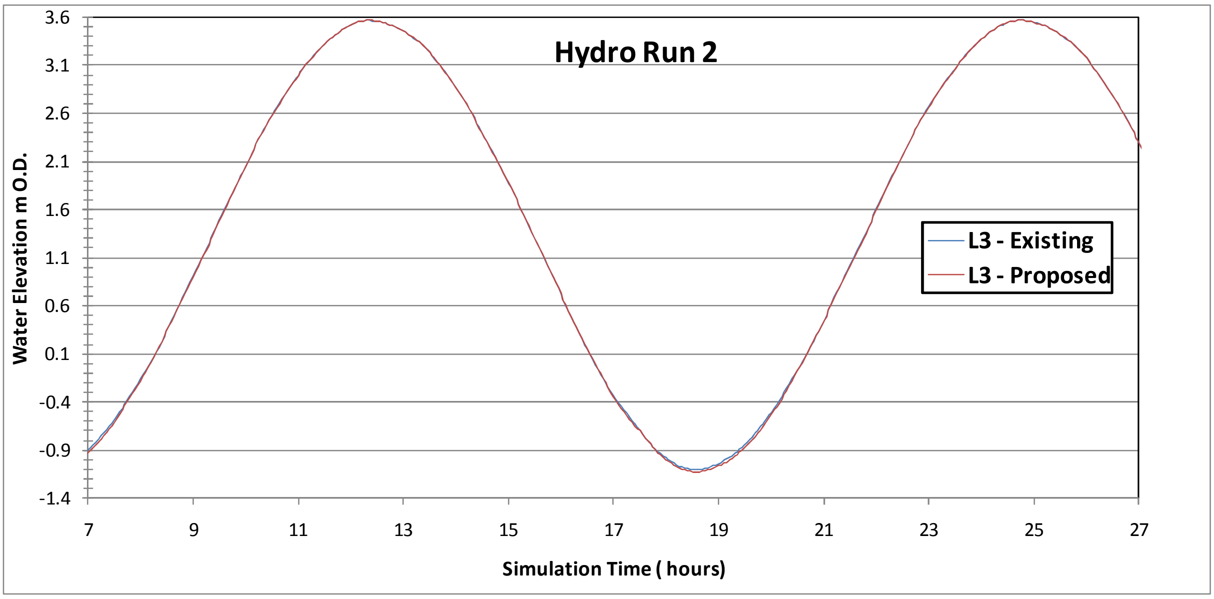

Figure 8.4.193 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 2 at reference site 3 in estuary inside Nimmo's Pier

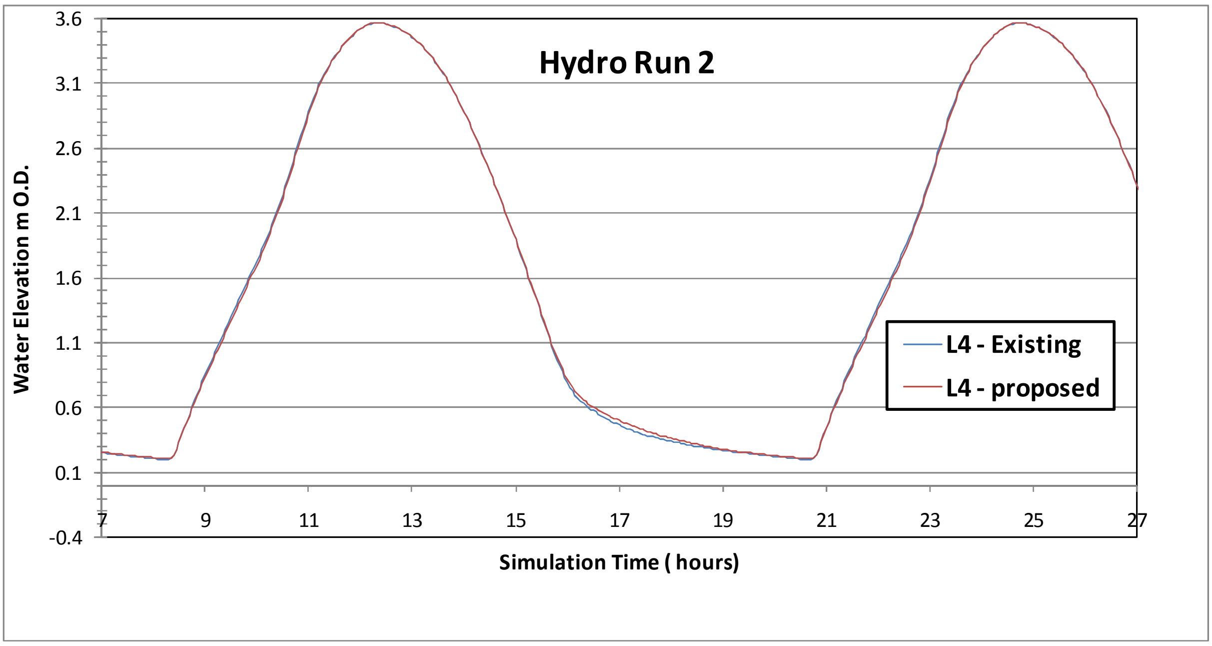

Figure 8.4.194 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 2 at reference site 4 – entrance channel to Lough Atalia

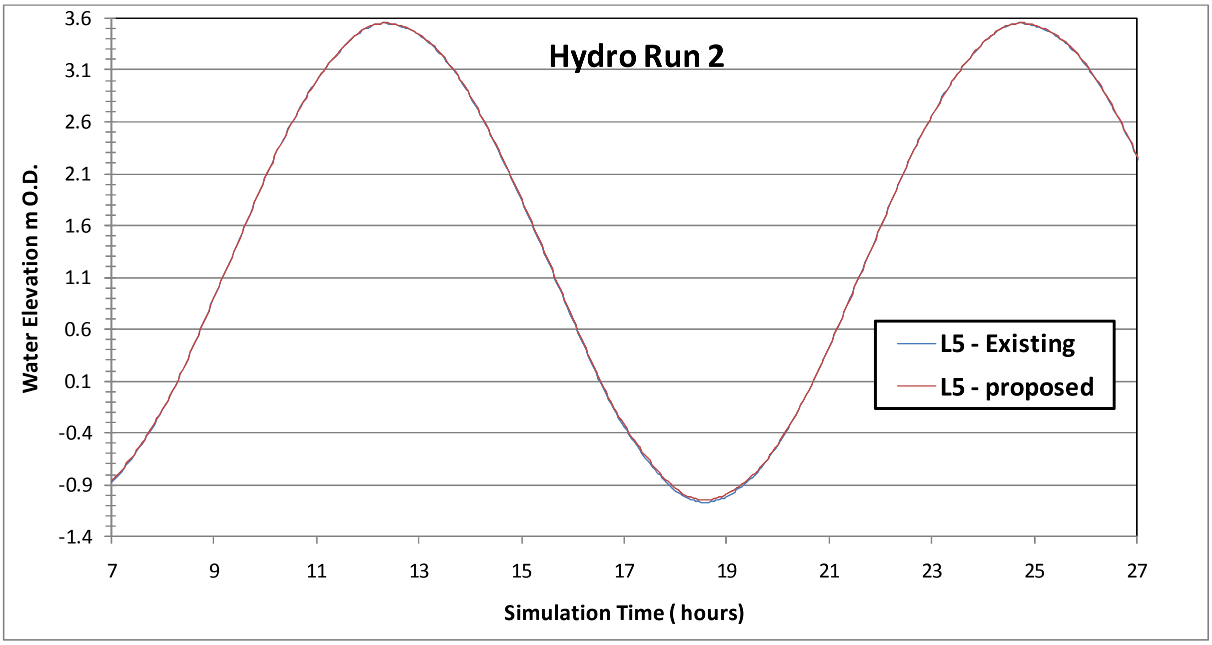

Figure 8.4.195 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 2 at reference site 5 – new dredge channel at Marina Breakwater

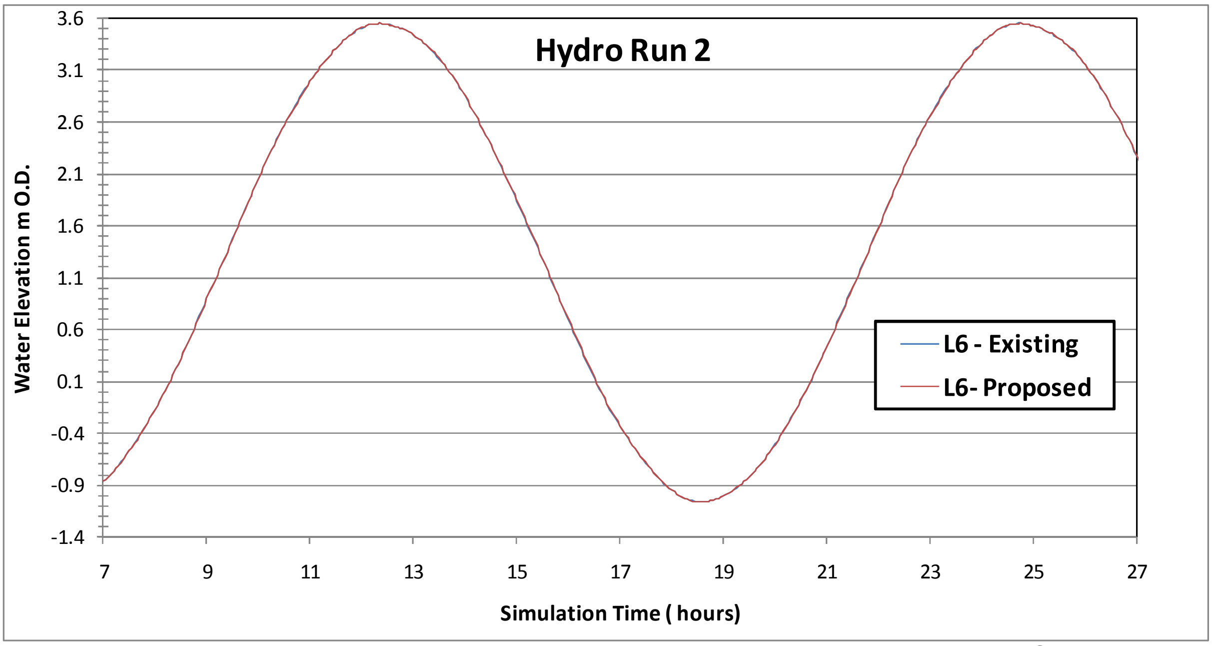

Figure 8.4.196 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 2 at reference site 6 – new dredge channel south of Marina entrance

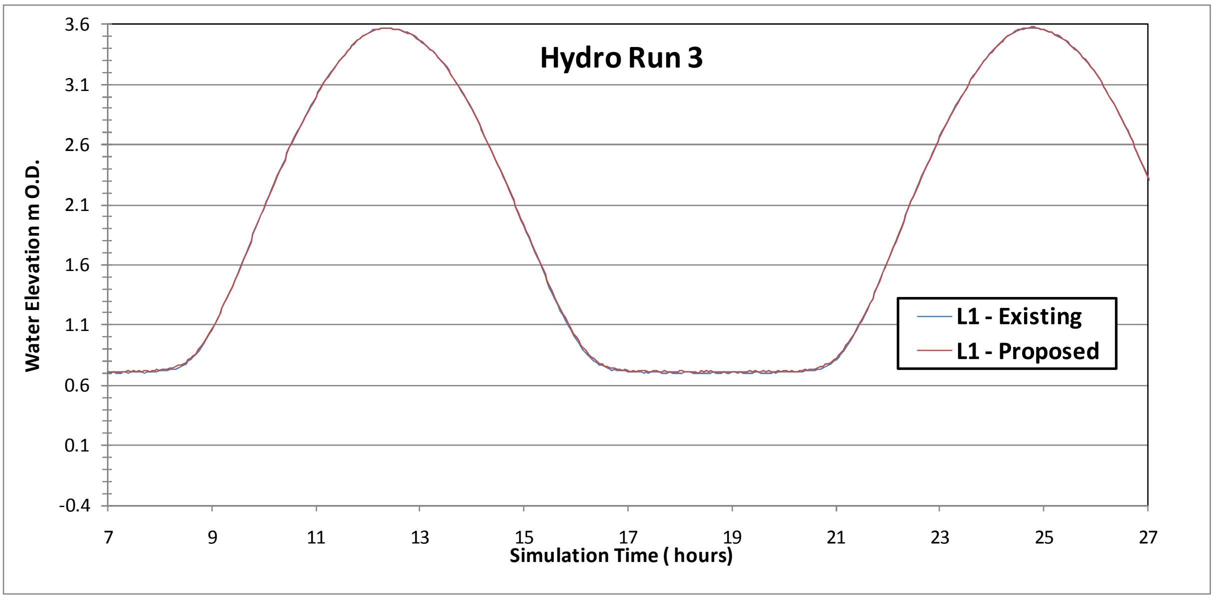

Figure 8.4.197 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 3 at reference site 1 in Claddagh Basin adjacent to Spanish Arch

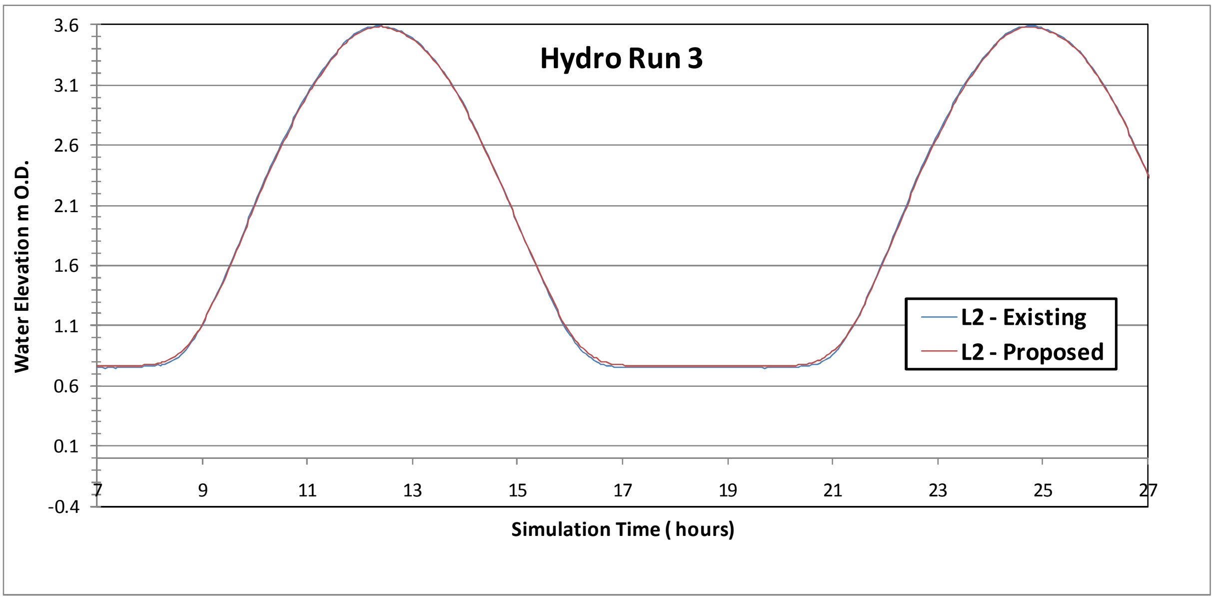

Figure 8.4.198 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 3 at reference site 2 in Claddagh Basin

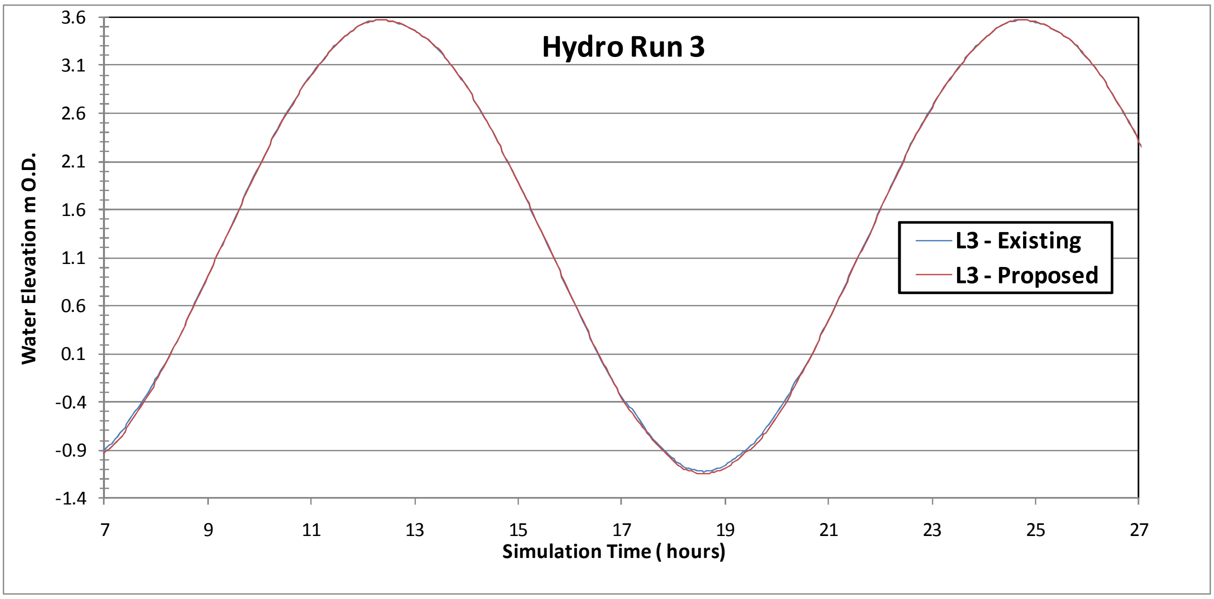

Figure 8.4.199 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 3 at reference site 3 in estuary inside Nimmo's Pier

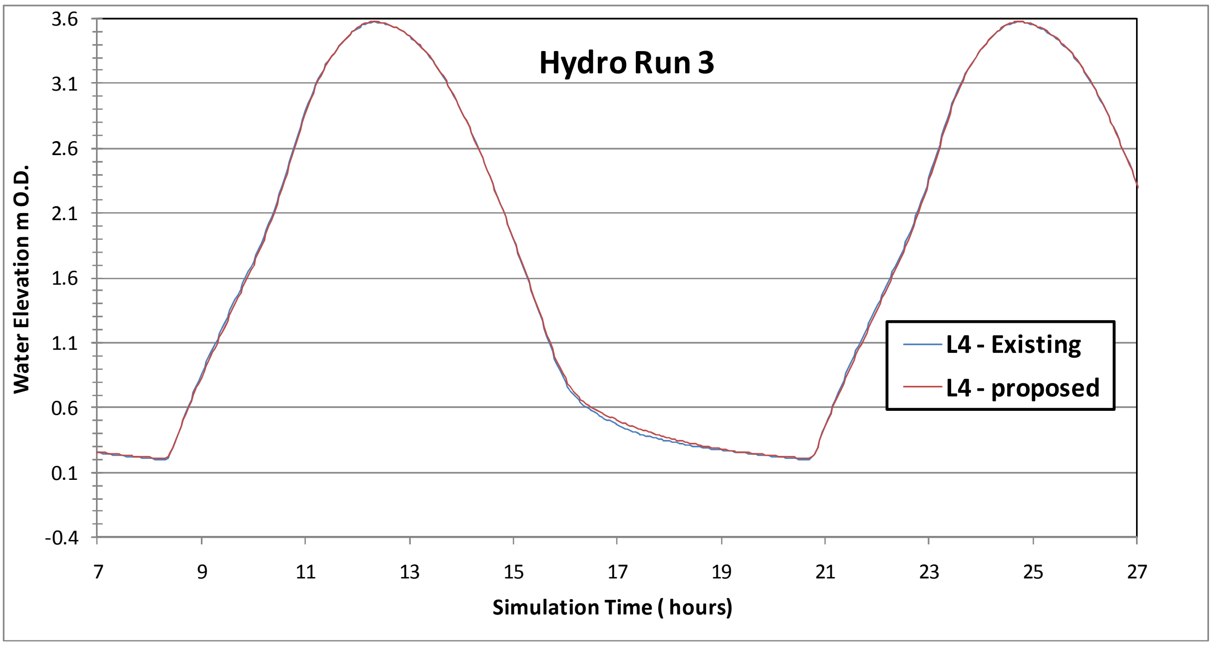

Figure 8.4.200 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 3 at reference site 4 – entrance channel to Lough Atalia

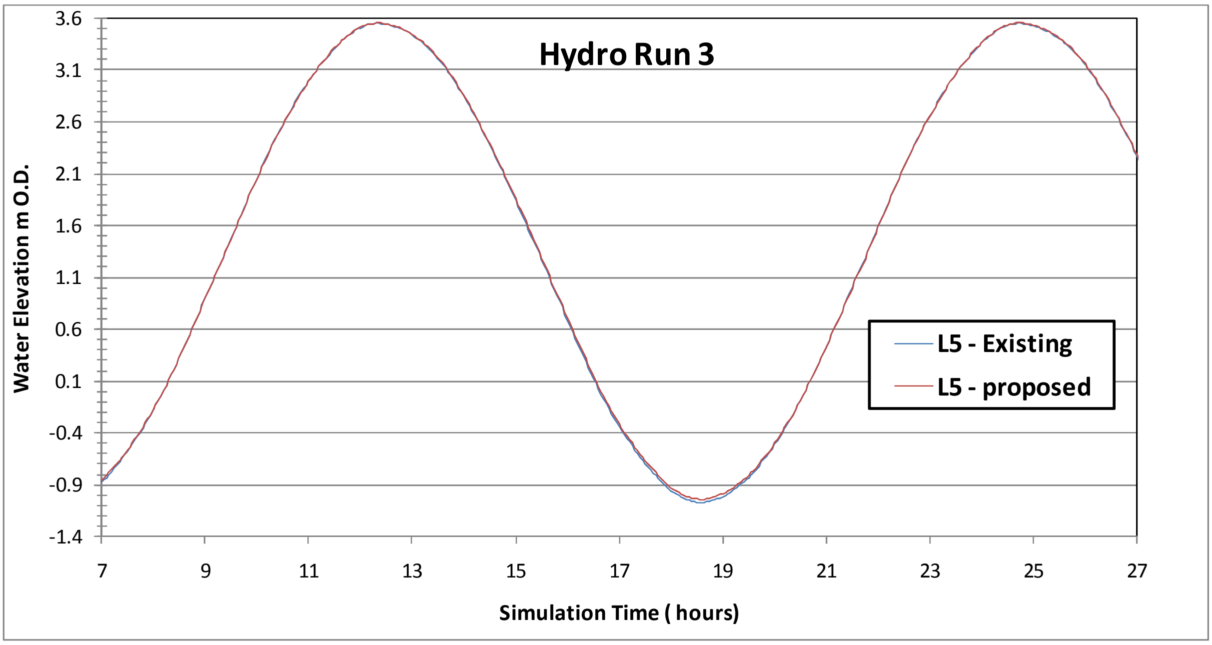

Figure 8.4.201 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 3 at reference site 5 – new dredge channel at Marina Breakwater

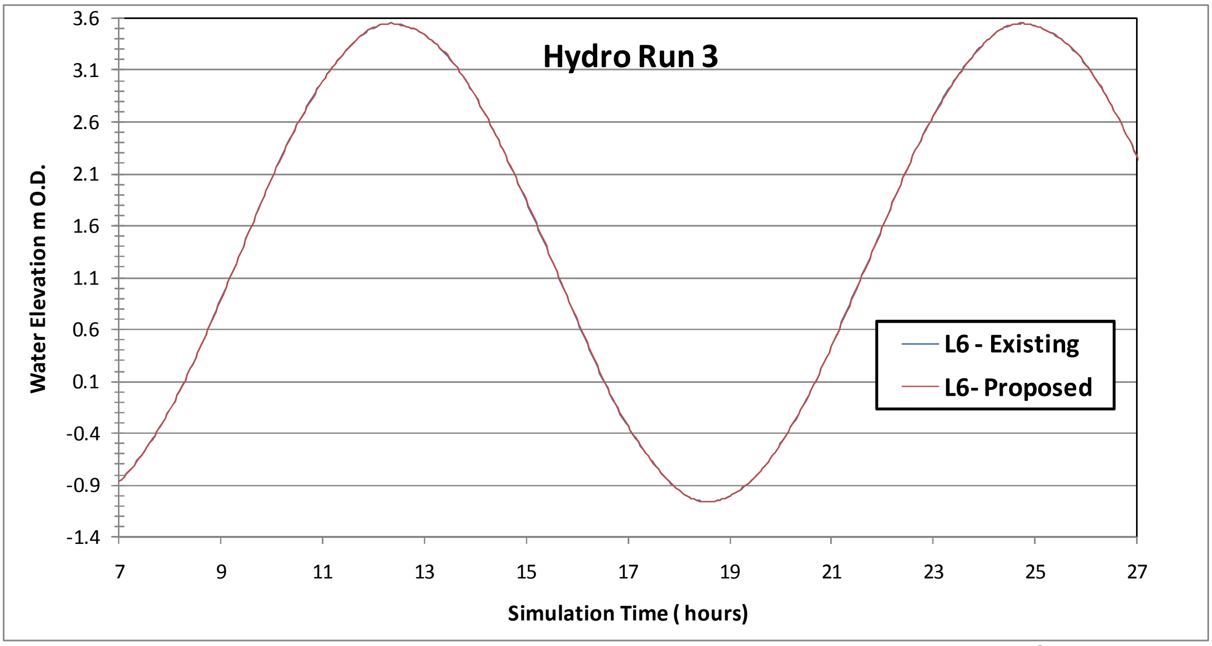

Figure 8.4.202 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 3 at reference site 6 – new dredge channel south of Marina entrance

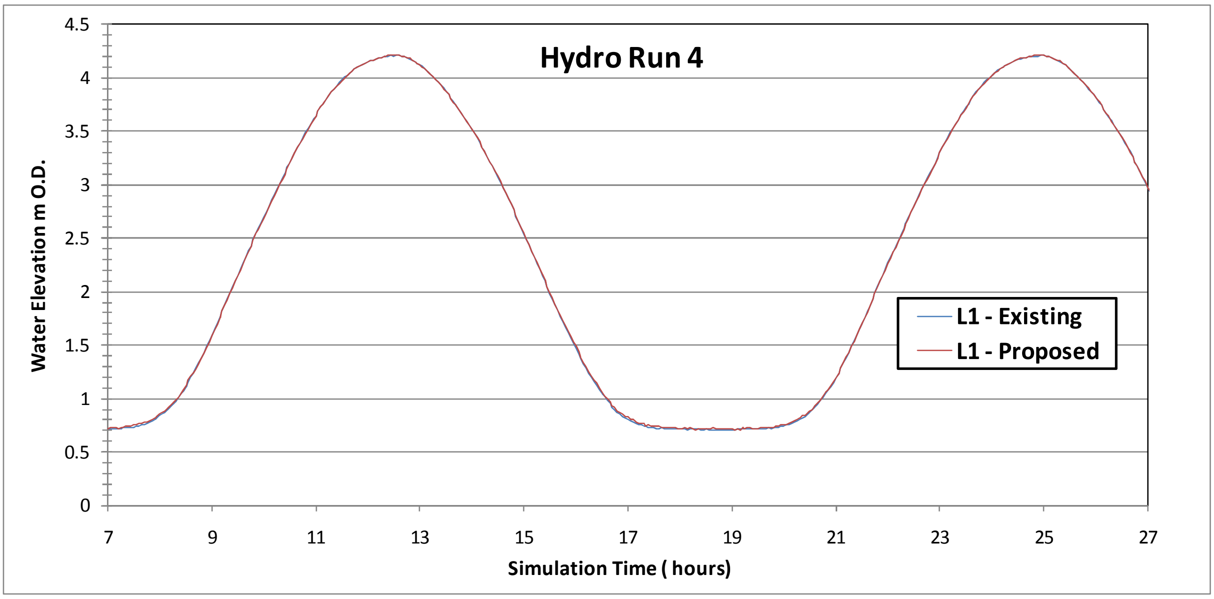

Figure 8.4.203 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 4 at reference site 1 in Claddagh Basin adjacent to Spanish Arch

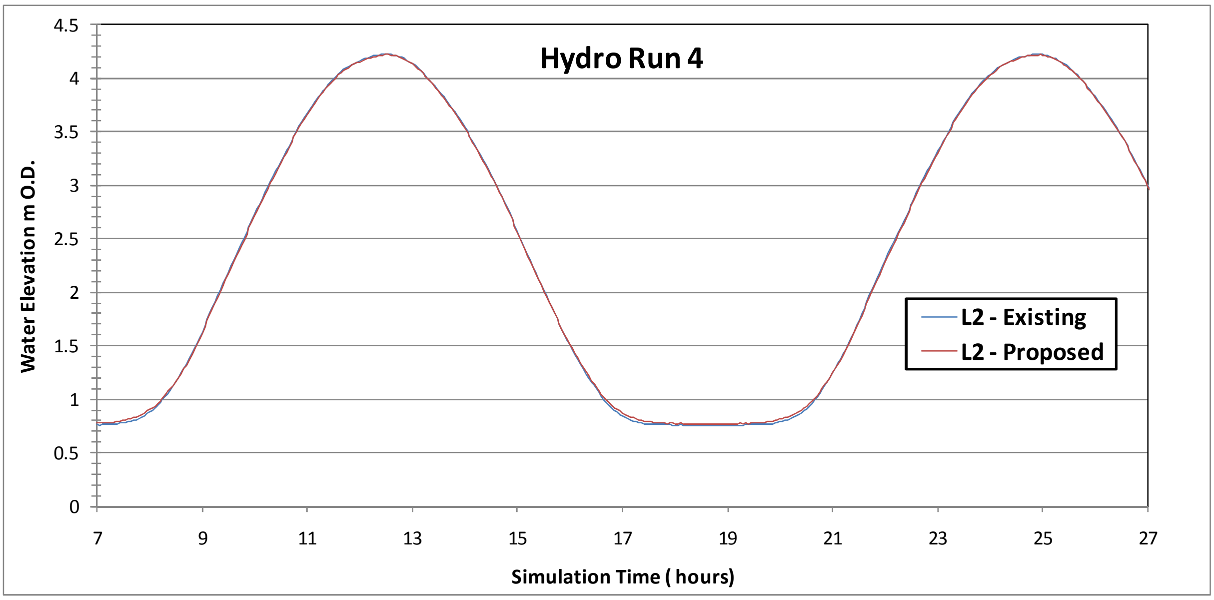

Figure 8.4.204 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 4 at reference site 2 in Claddagh Basin

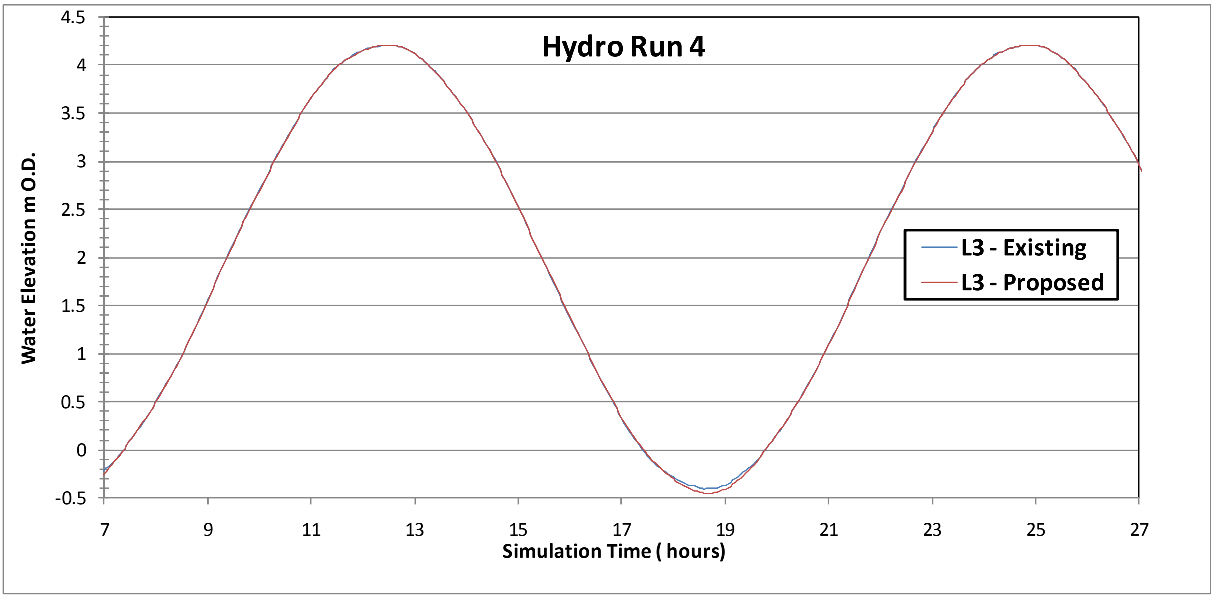

Figure 8.4.205 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 4 at reference site 3 in estuary inside Nimmo's Pier

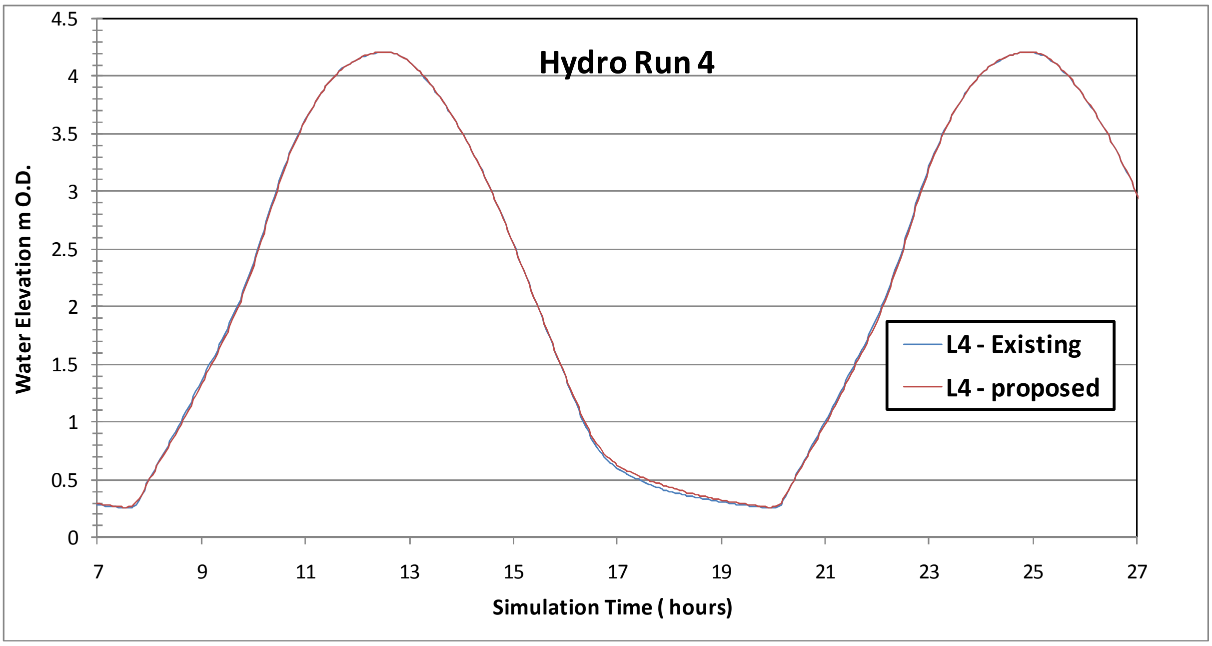

Figure 8.4.206 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 4 at reference site 4 – entrance channel to Lough Atalia

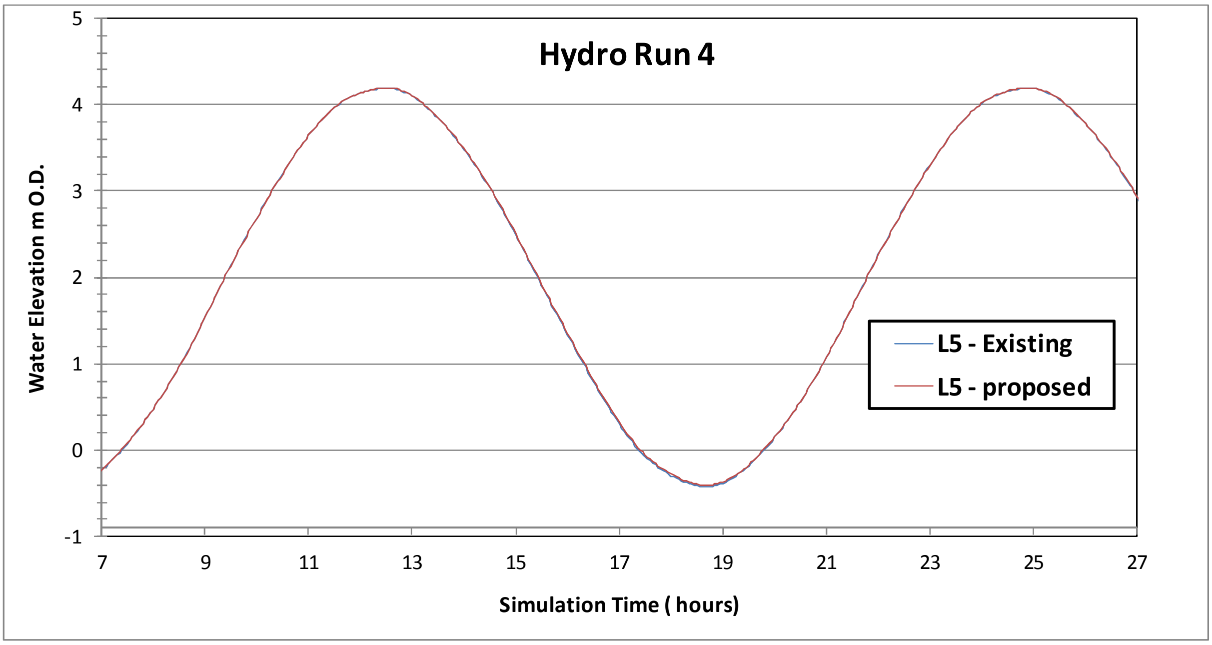

Figure 8.4.207 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 4at reference site 5 – new dredge channel at Marina Breakwater

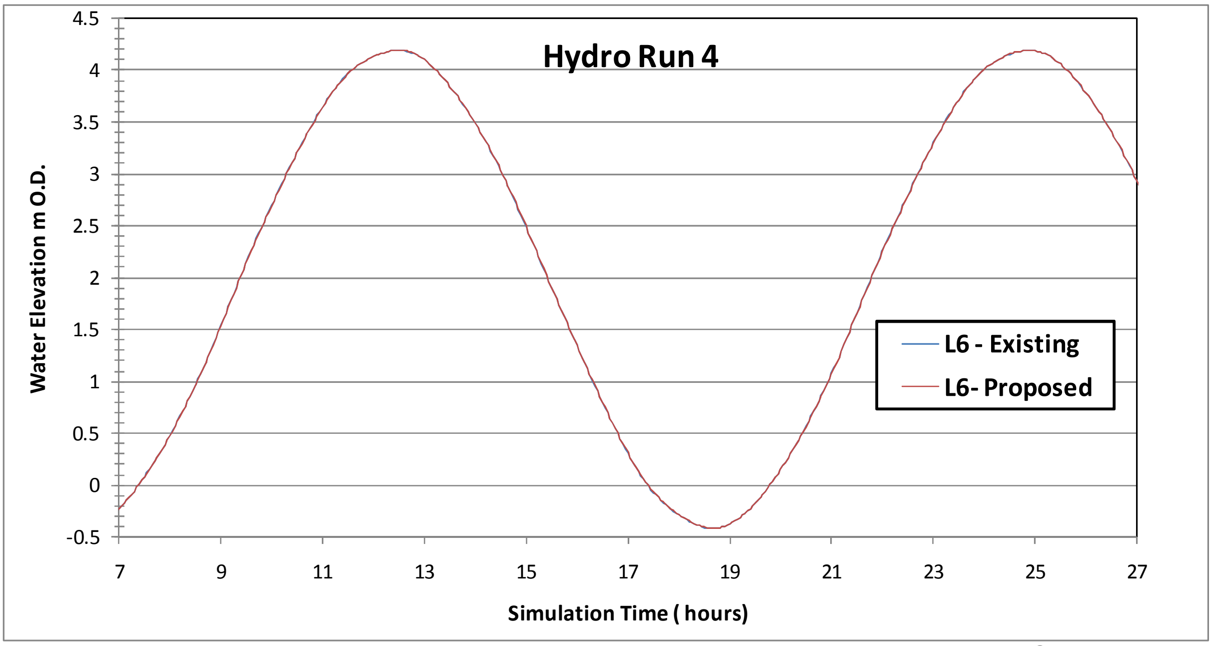

Figure 8.4.208 Flood level comparison between existing and proposed for Hydrodynamic Simulation No. 4 at reference site 6 – new dredge channel south of Marina entrance

8.4.7.10 Summary and Conclusions

8.4.7.10.1 Design Flood Level Prediction

The critical flood level for the harbour and surrounding areas is produced by a tidal storm surge event of 4.146 m O.D. Malin (200year tide) plus a climate change allowance (sea level rise) of 0.5m over the next 100 years giving a flood design level of 4.646 m. Such an event would under existing protection inundate a large portion of the city centre.

A detailed wave climate analysis was carried out to examine the exposure of the site and proposed development and assist in designing the required breakwater protection for the Commercial Port and proposed Marina. The principal area of exposure is from offshore waves propagating inshore from west to southwest directions diffracting around Mutton Island and impacting on the southern breakwater. These wave heights have been used to design the new port wave walls.

8.4.7.10.2 Flood Risk Zoning for Site

The proposed development site is located within the High Flood Risk Zone (*i.e.* Zone A of the Planning Guidelines). Flood Zone A is the high flood risk zone and represents lands that are below the 100year fluvial Flood level or the 200-year tidal or combined (tidal and fluvial) flood level. From the Flood risk assessment the critical condition for the harbour is the 200-year tidal storm surge event. The proposed development being a Commercial Harbour and Marina with associated dockside activities is classified as a water compatible development and recognised as appropriate development for Flood Zone A in the Flood Risk Management Planning Guidelines (Nov 2009) and therefore under these guidelines is justifiable from a flood risk management perspective provided suitable flood risk mitigation is provided.

8.4.7.10.3 Flood Risk to Proposed Development

The quay height and operational ground level are set at 4.7 m O.D. Malin which is above the design flood level of 4.646 m O.D. (assume 4.65m OD) and therefore considered safe from inundation from storm surge tides. The minimum finish floor level for all buildings on the port site is to be 5.5 m O.D. which is well above the design flood level providing a freeboard of 850 mm and thus not considered at risk of flooding from tidal/combined fluvial flood inundation.

The breakwater protection varies in height depending on the location and exposure to wave climate with southerly breakwater having a crest elevation of 9.1 to 10.1 m O.D. which provides 4.45 to 5.45 m above the design tide level (4.65 m O.D.) for wave climate and wave run-up effects. This level of protection will minimise the risk of overtopping of the breakwater structure by extreme waves. The westerly breakwater located in the more sheltered waters has a top elevation 6.35 to 6.65 m O.D. which based on wave climate analysis will protect this area from overtopping by the extreme waves predicted for these locations.

8.4.7.10.4 Flood Impact

The proposed port development has been shown not to impact on flood risk for the adjoining areas. It has no perceptible impact on peak combined tide levels within the Claddagh Basin, Spanish Arch and Galway Docks area upstream of Nimmo's Pier. The development does not adversely impact on wave climate in respect to flooding and flood risk with the harbour development generally sheltering the shoreline areas at South Park, the existing docks and the Renmore shoreline area against local and offshore generated waves.

8.4.7.10.5 Conclusion

In conclusion the Flood Risk Assessment shows that the proposed development is appropriate development for Flood Zone A being a harbour extension and that it will not increase flood risk to adjacent lands and developments from sea levels, wave climate and river flows.

8.5 IMPACTS AND MITIGATION

8.5.1 Impacts & Mitigation

The construction of the proposed new development will have obvious impacts on the physical and chemical characteristics of the aquatic environment in the vicinity of the mouth of the Corrib and these have been demonstrated by the output of the mathematical modelling studies. These effects relate to both the construction phase and the operational phase. During construction, the most significant impact will arise due to sediment being brought into the water column by dredging activities but the modelling results show that the area of significant impact is close to where the dredger is operating and that the levels of suspended sediments outside this zone are highly significantly lower than the maximum measured disturbed background levels. The impact of the deposition of these sediments on benthic fauna has been discussed in Chapter 7. Once construction is completed, impacts include permanent changes to current direction and velocity, temporally short but permanent changes in sediment distribution in the area to the east of Mutton Island and permanent changes in salinity values and wave climate characteristics. Due to the physical and chemical nature of some of these impacts, it is not possible to mitigate for them. However, the scales of change indicated by the model output are not considered to be great enough to significantly effect the functioning of the ecosystem. No changes are predicted for flooding episodes in Galway due to the construction of the proposed development. The impacts to the marine environment are listed in Table 8.5.1 below as are mitigation measures (where possible) and the residual impacts.

Summary Table of Impacts and Mitigation Measures					
No.	**Type of Impact**	**Resulting From**	**Potential Level of Impact**	**Proposed Mitigation**	**Residual Impact**
1	'Do Nothing' Impact	Decline of existing port usage.	Long Term Slight/ Imperceptible Positive Impact	None	Long Term Slight/ Imperceptible Positive Impact
2	Alteration of salinity levels in the vicinity of the Corrib River outflow following construction. Resulting in impacts to salinity sensitive species.	Increased current velocities or changes in current direction due to the construction of the proposed development	Potential impact is considered low, as the freshwater current regime from the Corrib River will not be significantly altered between the new structure and the Mutton Island causeway. Salinities will increase to the east of new structure and low salinity intolerant species may colonise the area.	None	Permanent Slight Positive Impact

Table 8.5.1 Summary table of impacts and mitigation measures

Summary Table of Impacts and Mitigation Measures					
No.	**Type of Impact**	**Resulting From**	**Potential Level of Impact**	**Proposed Mitigation**	**Residual Impact**
3	Alteration to current velocities at the proposed development site will impact the sedimentary environment resulting in a shift of existing scouring and deposition sites and a subsequent alteration of benthic habitat types.	Construction of proposed development in the intertidal and subtidal zone in proximity to the Corrib outflow.	Moderate potential impact. Existing current velocities are predicted to increase along west side of the solid structure of the development. Sedimentary conditions will stabilise after a significant spate condition.	None	Permanent Slight Negative Impact
4	Alteration to current directions at the proposed development site will impact the sedimentary environment resulting in a shift of existing scouring and deposition sites and a subsequent alteration of benthic habitat types.	Construction of proposed development in the intertidal and subtidal zone in proximity to the Corrib outflow.	Low potential impact. Variations in current directions will have little environmental impact.	None	Permanent Imperceptible Impact
5	Release of grey water from construction site	Construction activities	High Potential Impact.	Implementation of best practice and EMP. Use of well managed bunds will prevent grey water entering the sea	Temporary Slight Negative Impact
6	Release of bilge water from construction vessels	Leakage from construction vessels	High Potential Impact.	Bilge water is collected from vessels and disposed of by licenced operators.	None

Table 8.5.1 contd/. Summary table of impacts and mitigation measures

Summary Table of Impacts and Mitigation Measures					
No.	**Type of Impact**	**Resulting From**	**Potential Level of Impact**	**Proposed Mitigation**	**Residual Impact**
7	Release of sewage from construction site	Leakage from construction site and vessels	High Potential Impact.	Implementation of best practice and EMP. Sewage will be controlled at the construction site.	Temporary Slight Negative Impact
8	Release of diesel from construction site	Leakage from construction site and vessels	High Potential Impact.	Implementation of best practice and EMP. The use of a bund will prevent leakages entering the sea.	Temporary Slight Negative Impact
9	Oil spills and other accidental release of fluids/solids during loading/off loading of vessels	Accidental spillage during loading/off loading vessels	High Potential Impact.	Oil spill contingency plan.	Temporary High Negative Impact
10	Disposal of ballast waters	From construction vessels	High Potential Impact. Ballast waters can introduce non-native species that have the potential to out-compete native species.	Disposal of ballast waters is regulated by the International Maritime Organisation and vessels that make transnational passages may not dispose of ballast waters inside the EEZ (Exclusive Economic Zone)	None.
11	Impacts from maintenance dredging	Sedimentation and smothering arising from dredging and disposal	Short Term Serial Localised Negative Impacts	None	Short Term Serial Localised Negative Impacts
12	Changes in wave climate	Increases and decreases due to new structure	Low potential impact. Variations wave climate conditions will have little environmental impact.	None	Permanent Low Impact

Table 8.5.1 contd/. Summary table of impacts and mitigation measures

8.5.2 Contingency plans

The main source of significant damage could occur if the construction area was flooded during construction, when wet cement was still soluble or in the event of a diesel spillage. If this occurred during concrete pouring, the wet material could be washed out into the marine environment. However, as wet concrete will only be used underwater and otherwise significantly above high tide level for grouting, it is considered that the impact will be immeasurable.