# Great Yarmouth Third River Crossing

## Application for Development Consent Order

# Document 6.2: Environmental Statement Volume II: Technical Appendix 11C: Sediment Transport Assessment

**Planning Act 2008**

**The Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009 (as amended) ("APFP")**

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# Glossary of Abbreviations and Defined Terms

| | |
|---|---|
| AAP | Area Action Plan |
| AEP | Annual Exceedance Probability |
| AOD | Above Ordnance Datum |
| FRA | Flood Risk Assessment |
| HAT | Highest Astronomical Tide |
| LAT | Lowest Astronomical Tide |
| MHWS | Mean High Water Spring Tide |
| MLWS | Mean Low Water Spring Tide |
| MHWN | Mean High Water Neap Tide |
| MLWN | Mean Low Water Neap Tide |
| GYBC | Great Yarmouth Borough Council |

# 1 Introduction

## 1.1 Overview

1.1.1 A Sediment Transport Assessment for the proposed Great Yarmouth Third River Crossing (hereinafter referred to as “the Scheme”) within the town of Great Yarmouth on the East Anglian coast of England has been prepared as part of the DCO Application. A Rochdale Envelope approach has been adopted and the reasonable worst-case scenario for sediment transport has been assessed. A hydraulic model has been built to assess the impact of the Scheme on the sediment regime in Great Yarmouth and this report details the model build and outputs.

1.1.2 This assessment investigates the impact of the Scheme on the sediment regime within the River Yare, looking specifically at the magnitude and range of the impact. The assessment has been carried out for a Spring and Neap Tide and likely extreme events. For this assessment, out of channel flooding events have not been considered, therefore no floodplains have been included in the model. This is because once the water level is sufficient to overtop the flood defences, the velocity magnitude in the channel is unlikely to increase as water flowing out onto the floodplain increases the flow area and limits the velocity magnitude in channel. In addition, the focus of this assessment is on regular, everyday events and as floodplain flows occur infrequently, it has not been necessary to include them in this assessment.

## 1.2 Sediment Assessment Study Area

1.2.1 Great Yarmouth is a seaside town in Norfolk on the east coast of England. The River Yare flows through the centre of the town and is a commercial port with a number of large ship berths along both quays. Tidal defences line the river edge, providing protection from coastal flooding to the town and containing the water flow during the normal tidal cycle. The river flows in a southerly direction, under two existing bridges before turning at almost a right angle to discharge in an easterly direction into the sea.

1.2.2 The River Yare is one of the sea boundaries of the Broadlands Rivers Catchment and is tidally driven. The tidal boundary drives the levels in the River Yare and across the Norfolk Broads. Great Yarmouth currently has two road bridge crossings over the River Yare; Breydon Bridge and Haven Bridge as shown in Plate 1-1. These are currently the only two ways for traffic to cross the River Yare in Great Yarmouth. Both bridges are constructed using traditional methods each supporting the bridge deck on vertical support columns built into the river bed.


**Plate 1.1 – Study Location**



*Plate 1-1: Study Location*

1.2.3 The River Bure is a tributary which flows into the River Yare approximately 240m downstream of the A47 Bridge. Upstream of Breydon Bridge, the River Yare forms a lake known as Breydon Water. Breydon Water is an area of intertidal mud flats and salt marshes and contributes a significant volume of storage to the estuary.

## 1.3 The Scheme

1.3.1 The Principal Application Site is located approximately 2.3km upstream of the harbour mouth and 2.3km downstream of Breydon Water. The Satellite Application Sites have not been included in this assessment as they are remote from the river channel and do not have an impact on the sediment regime within the river. The Scheme consists of twin bascule bridge decks supported on vertical columns, which extend from the east and west quay walls. The columns are surrounded by small knuckles with ship fenders attached, which provides a 50m navigable channel for vessels. The total width of the opening under the bridge deck is approximately 55m. Each side of the bridge has an approach road sloping from the deck height to the existing ground level on either side of the bridge. Both approach roads are on an embankment to provide vehicular access to the bridge deck. On either side of the bridge, each embankment has an opening allowing access underneath the approach roads for local traffic. For the full Scheme description refer to Chapter 2 of the Environmental Statement. Document reference 2.1 and 2.2 shows the design of the Scheme.

# 2 Data Collection and Review

## 2.1 Overview

2.1.1 The data listed in Table 2-1 has been collected as part of this study. All the data has been reviewed and its suitability for use in this assessment determined.

*Table 2-1: Collected Data Summary*

| Data | Source |
|---|---|
| 1D/2D ISIS TUFLOW Classic River Yare, 2011 Model | Environment Agency/Halcrow |
| 1D/2D Flood Modeller/TUFLOW Classic flooding model, 2018 | WSP |
| General Arrangement Plans (document reference 2.2)<br>Engineering Plans, Drawings and Sections (document reference 2.11) | BAM Farrans |
| OS mastermap<br>As-built construction drawing of Haven Bridge | Norfolk County Council (NCC)/Environment Agency |
| Bathymetry Survey (2017)<br>P16-General-Port-Pilotage-information.pdf | Peel Ports Great Yarmouth |
| Sediment Particle Size Survey (July 2018) | Norfolk County Council |
| Velocity Survey (April 2018) | Norfolk County Council |
| 2015, 0.5m LiDAR<br>2009, 1m LiDAR<br>Extreme Sea Levels<br>15-minute gauge data for Haven Bridge, Great Yarmouth (Gorleston on Sea), Three Mile House and Burgh Castle | Environment Agency |

2.1.2 As part of the Sediment Transport Assessment, the Environment Agency has provided a 1D/2D ISIS-TUFLOW flooding model which has been used in previous projects in Great Yarmouth and as part of this application, WSP have developed a new 1D/2D Flood Modeller-TUFLOW model to assess flooding. A review of both models has been carried out to understand if any elements can be used in the sediment assessment. However, the flood models were specifically developed to assess flooding within Great Yarmouth and it was decided that only level information (including the channel bed and flood defence levels) from the flood models would be useful within the sediment transport model developed for this assessment.

2.1.3 In addition to the 1D/2D hydraulic models received as part of the Scheme, various reports and datasets have also been collected. The design information has been used to schematise the bridge within the model. As-built drawings for Haven Bridge have been received which have been used to schematise the existing bridge in the model. The drawings provide sufficient information to specify the bridge dimensions in the model.

2.1.4 Several surveys have been carried out to provide information for use in the sediment model. Bathymetric survey of the river channel is carried out regularly by Peel Ports Great Yarmouth and the latest survey dataset (2017) was made available for this assessment. The data has been used to set the bathymetry in the water channel within the model. A sediment survey has been carried out in the channel near the Principal Application Site. The sediment survey provides particle size distribution information at ten sample locations, for further information on the sediment survey see Section 4.2.

2.1.5 Peel Ports Great Yarmouth produced a document (Ref 11C.1) providing general information to mariners who use the port. This document provides anecdotal evidence suggesting the current speed peaks around 3 knots (1.5m/s) on the incoming tide and up to 3 to 4 knots (1.5m/s to 2m/s) on the outgoing tide. There is no mention of where these velocity magnitudes have been observed and as such they have only been used for information purposes.

2.1.6 As part of the calibration process, a velocity survey was carried out. The survey was undertaken over a two-day period at the weekend during a relatively quiet period for port operations to minimise disturbance due to vessel movement. The survey has been used to validate the velocity outputs of the model, see Section 5.3.

2.1.7 The Environment Agency own several datasets that can assist with model development. LiDAR has been obtained from the Environment Agency's data website, the 50cm resolution, 2015 flight dataset has been used predominately and the 1 m resolution, 2009 flight dataset has been used to fill in any gaps in data. The Environment Agency has provided 15-minute level gauge data for Haven Bridge, Gorleston-on-Sea, Three Mile House and Burgh Castle, all of which are within the Broadlands catchment area. This data has been used to generate the tidal boundaries in conjunction with the extreme sea levels and calibrate the model.

# 3 Tidal Boundaries

## 3.1 Overview

3.1.1 Tidal levels have been derived to define the eastern boundary of the hydraulic model that represents the sea level along the Great Yarmouth coastline. The tidal boundaries have been generated in two ways; firstly, an extract from the gauge at Gorleston-on-Sea for a Spring and Neap cycle has been extracted to simulate the typical tidal cycle and used to represent an everyday event.

3.1.2 Secondly, Environment Agency guidance on estimating design sea levels (Ref 11C.2) has been used to derive the extreme tidal boundary inflows used in the model. An extreme tide curve has also been derived for several scenarios scaled to the 5% Annual Exceedance Probability (AEP) tidal event; 2.84mAOD, taken from the guidance. These scenarios represent an extreme event which the Scheme is likely to experience during its lifetime (assumed design life is 120 years).

3.1.3 The events that have been simulated in the model are as follows:

- Everyday Events:
  - Spring; and
  - Neap.
- Extreme Events:
  - Mean High Water Spring (MHWS) to Mean Low Water Spring (MLWS) + 5% AEP Sea Surge Event; and
  - Mean High Water Neap (MHWN) to Mean Low Water Neap (MWWN) + 5% AEP Sea Surge Event.

3.1.4 This section provides an overview of the tidal curve derivation process, for full details see Annex A.

## 3.2 Everyday Scenario

3.2.1 In order to generate the “everyday” tidal boundary, the recorded tidal data at the Gorleston-on-Sea gauge was downloaded from the British Oceanography Data Centre (BODC) website for 2018. Plate 3-1 shows the water elevation recorded for the full year for 2018 at the gauge.

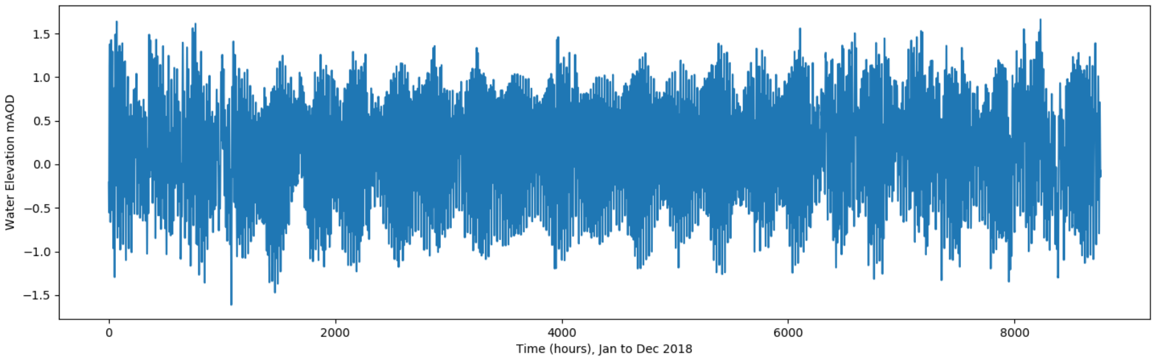


*Plate 3-1: 2018 – January to December Tidal Levels Recorded at Gorleston on Sea Gauge*

3.2.2 Plate 3-1 shows the full year of recorded data at Gorleston-on-Sea for 2018. The time series plot shows the typical spring/neap cycle repeating approximately every week throughout the year and several surge tides particularly around the early part of the year from January to February. For the purpose of this assessment a typical spring/neap tide cycle is required; therefore, the curve shown in Plate 3-2 has been extracted making sure no surge events are captured.

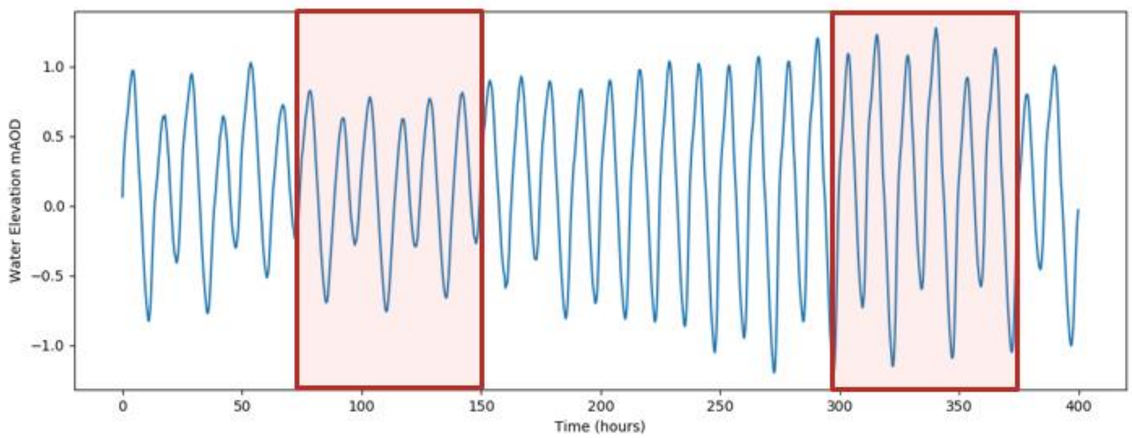


*Plate 3-2: Extracted Tidal Curve*

3.2.3 Plate 3-2 shows a typical water level time series ranging from a Neap to Spring tide. The data has been selected from the yearly recorded data shown in Plate 3-1 to represent a typical tide with minimal surge events. At this point, the date of the profile is no longer relevant, therefore the plots show the tidal cycle time in hours starting at zero hours. In an effort to reduce model simulation time, the curve shown in Plate 3-2 has been split into two separate simulations (shown in the red boxes) of approximately 75 hours; one simulating a spring tide and one simulating a neap tide. These simulations will be used to approximate the amount of sediment movement on a typical spring and neap tide.

## 3.3 Extreme Tide Event

3.3.1 In order to understand the impact of likely extreme tidal events, Environment Agency guidance on estimating design sea levels (Ref 11C.2) has been used to derive the extreme tidal boundary inflows used in the model. The Environment Agency guidance has a ten-step procedure to create a tidal boundary for the model:

1. Check study location is outside of the estuary boundaries;
2. Select an appropriate chainage point for extreme sea levels;
3. Select an AEP peak sea level;
4. Consider allowance for uncertainty;
5. Identify base astronomical tide;
6. Convert levels to Ordnance Datum (OD);
7. Identify surge shape to apply;
8. Produce the resultant design tide curve;
9. Sensitivity testing;
10. Apply allowance for climate change (if required).

3.3.2 The guidance is the best method currently available for tidal curve derivation in UK waters. An overview of the derivation is provided here, for a full description, see Annex A.

3.3.3 Steps one and two require the estuary boundaries and extreme sea level datasets provided with the guidance. Using the datasets, checks have been carried out to ensure the location of the tidal boundary is outside of the River Yare estuary and the nearest chainage node is 4,150.

3.3.4 Steps three and four select the appropriate AEP event and the measure of uncertainty. For this assessment, it has been decided that 5% AEP event represents the likely extreme event. This is because the event remains in channel and it is probability says this is likely to happen in the Scheme's design life. To that end, Table 3-1 shows the extreme sea level for the 5% AEP taken from the guidance.

*Table 3-1: Extreme Sea Level*

| Annual Exceedance Probability (AEP) | Extreme Sea Level (mAOD) |
|---|---|
| 5% | 2.84 |

3.3.5 The uncertainty value is +/- 0.2m, this is a measure of the uncertainty in the modelling used to generate the extreme sea levels. This is considered an acceptable uncertainty for this assessment because the water level is not the focus of this assessment.

3.3.6 In order to generate the astronomical tide, the gauge data at Gorleston-on-Sea has been used. In addition to the Gorleston-on-Sea gauge, in line with the Environment Agency guidance, the MHWS, MLWS, MHWN and MLWN levels have been obtained from the nearest primary gauge at Lowestoft. The Environment Agency guidance states that when generating the base tidal curve, the tidal parameters from the nearest primary gauge should be used. Lowestoft harbour is 12km south of Great Yarmouth and therefore it is considered appropriate to use these gauge parameters for this assessment. Table 3-2 lists the Lowestoft tidal gauge parameters.

*Table 3-2: Lowestoft Primary Gauge Properties*

| Property | Value (mAOD) |
|---|---|
| MHWS | 1.08 |
| MLWS | -0.86 |
| MHWN | 0.74 |
| MLWN | -0.34 |

3.3.7 To generate the tidal curve, gauge data from the Gorleston-on-Sea gauge at Great Yarmouth has been analysed and a typical tidal cycle has been extracted. The extracted tidal profile has been repeated to create a minimum of 75 hours and scaled to the appropriate levels in Table 3-2 for a given event. In following this method, the shape of the tidal profile is replicated in the model. This is particularly important because the shape and rate of change in water level drives the velocity in the harbour. Plate 3-3 shows the tidal cycle extracted from the gauge data which represents the tidal levels in Great Yarmouth.

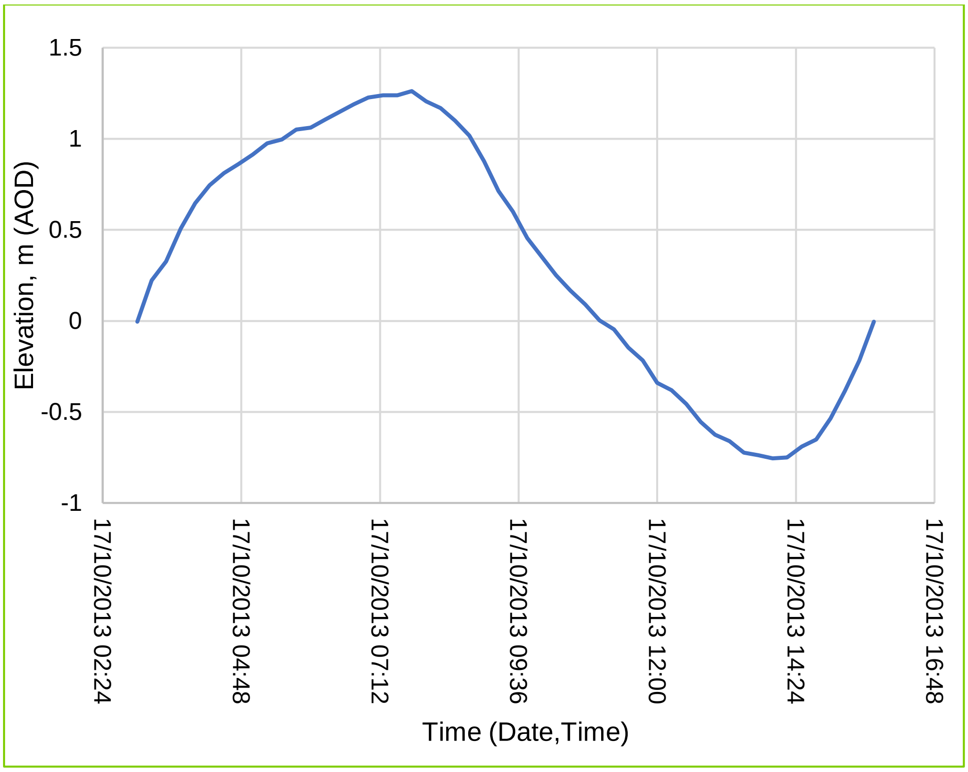


*Plate 3-3: Typical Tidal Curve (Extracted from Gauge Data)*

3.3.8 Following the extraction of the typical tidal curve shown in Plate 3-3, the peaks and troughs are scaled to the appropriate levels in order to create the base tidal curve events. Plate 3-4 shows the final base curves for the MHWS to MLWS and the MHWN to MLWN events.

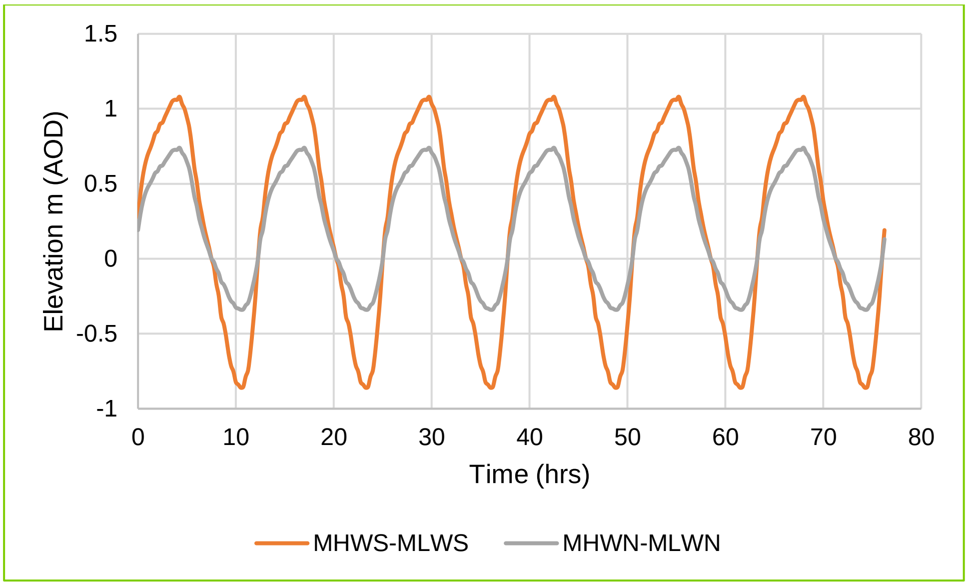


*Plate 3-4: Base Tidal Profiles MHWS – MLWS and MHWN - MLWN*

3.3.9 Once the extreme sea level and the base tidal profiles have been identified, sea surge is applied. This has been carried out by obtaining the normalised surge shape from the Environment Agency guidance. For Great Yarmouth, the normalised surge shape is number 9 in the dataset provided with the guidance documentation.

3.3.10 The guidance states that the resultant design tide curve is derived by combining the extreme sea level, base tide and surge shape. The first process is to align the base tides and surge shape peaks, in this case this is at 42.5 hours.

3.3.11 Once the base tide and surge shape are aligned, it is necessary to scale the base tide to the required extreme sea level. To explain this procedure, the MHWS-MLWS + 5% AEP event has been used as an example. Firstly, the difference between the required extreme sea level (2.84 m AOD) and the base tide peak (1.48 m AOD) is calculated, which in this example is 1.36 m. As the surge shape is aligned with the peak water level time in the base tide, the maximum surge value of 1.0 occurs at the same time as the peak water level. The surge shape can now be scaled by the coefficient 1.36/1.0 = 1.36m AOD, thus creating a surge height which can be added to the base tide curve resulting in the required tidal profile for the event.

3.3.12 The procedure has been carried out for the events shown in Plate 3-4 to produce the two extreme tidal boundaries required for this assessment as shown in Plate 3-5.

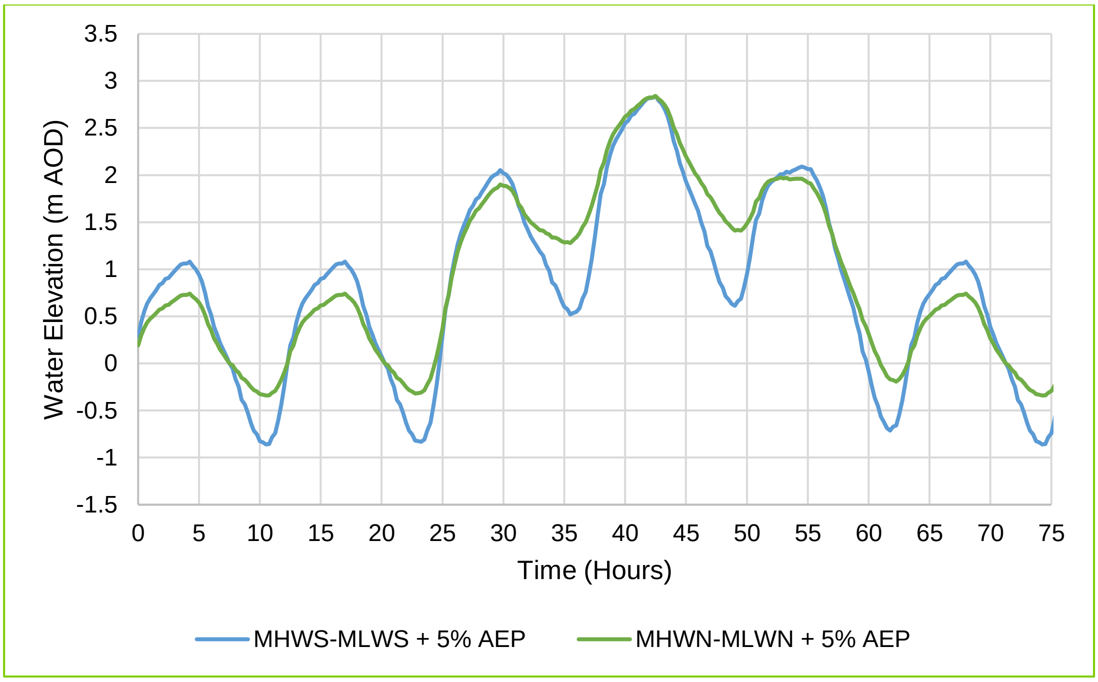


*Plate 3-5: Extreme Tidal Curves*

3.3.13 The final step in the Environment Agency guidance is to consider climate change. For this assessment climate change (sea level rise) is not considered. This is because the velocity in the channel is predominately driven by the rate of change of water level and simply increasing the base profile elevation will not dramatically increase the velocity in the River Yare. In addition, during high water level events, the flood defences will be overtopped allowing water to flow onto the floodplain outside of the channel. Once the water level is sufficient to overtop the flood defences, the velocity magnitude in the channel is unlikely to increase as water flowing out onto the floodplain increases the flow area and limits the velocity magnitude in channel.

# 4 Existing Regime

4.1.1 The existing tidal regime has been investigated to understand the baseline environment in which the Scheme will be constructed. This section provides information on the existing sediment regime including particle size analysis, tidal prism, typical cross-sections in the River Yare channel and Breydon Water, tidal symmetry and tidal dominance.

## 4.2 Particle Size Analysis

4.2.1 A sediment survey was carried out in 2018 to ascertain the particle sizes of sediment in the River Yare channel at the Principal Application Site, the survey was carried out at ten locations as shown on Plate 4-1. Samples were taken from the channel and tested in a laboratory to determine the Particle Size Distribution (PSD).

4.2.2 The sediment survey suggests that the D50 particle size ranges from 0.03mm to 0.55mm diameter in the river at the Principal Application Site. Table 4-1 lists all the particle size data received from the sediment sampling. In cross referencing the D50 particle size with the locations in Plate 4-1, it is possible to see that smaller particle sizes are typically found closer to the western quay wall with larger particle sizes nearer to the eastern quay.

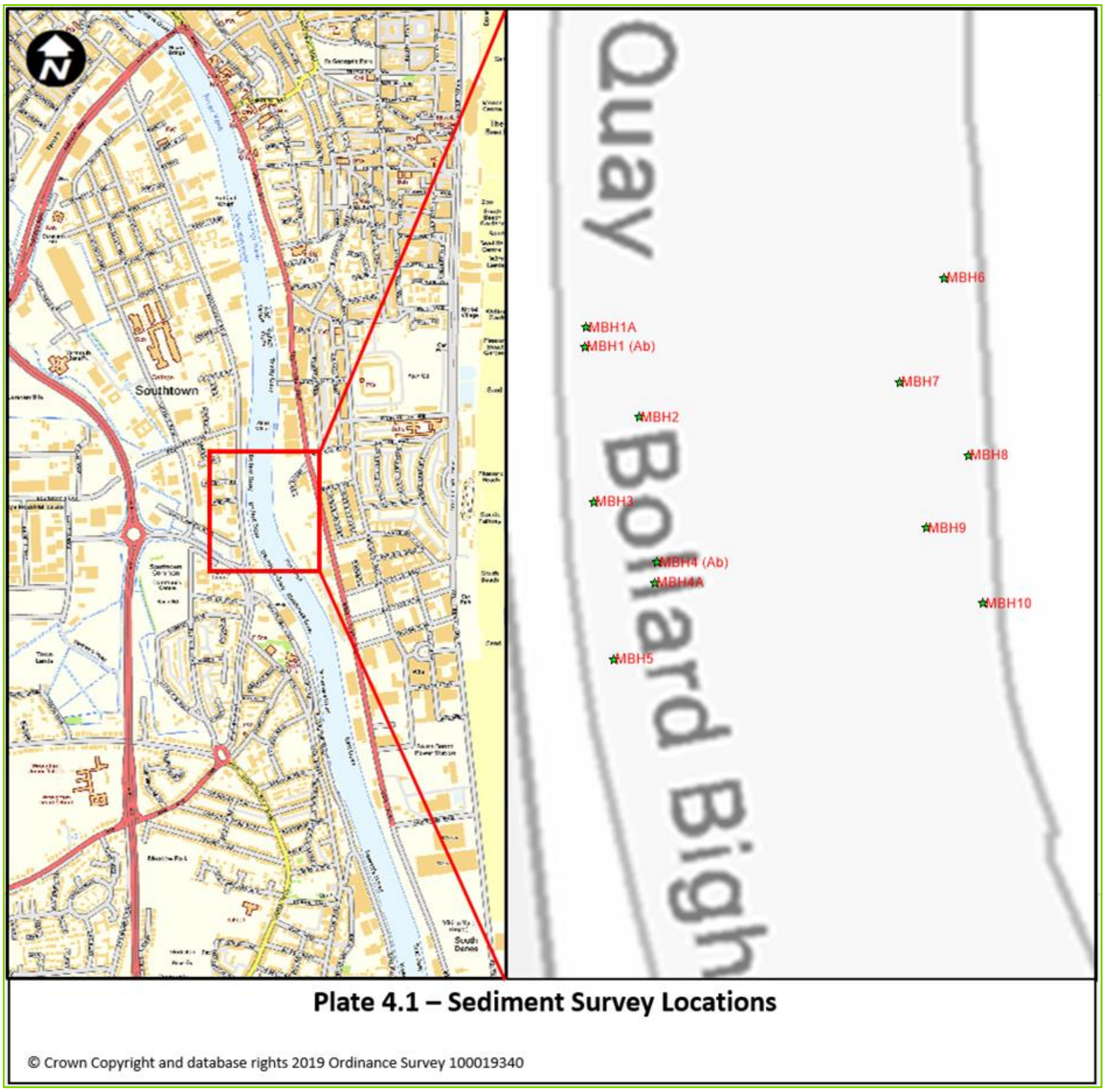


*Plate 4-1: Sediment Survey Locations*

*Table 4-1: Sediment Survey Results*

| | | | mm Passing | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Sample** | **Deck Level (mOD)** | **Date** | **D10** | **D50** | **D60** | **D100** | **Moisture Content (%)** | **Comments** |
| MBH1 (Abandoned) | 2.71 | 11/06/2018 | NA | NA | NA | NA | NA | NA |
| MBH1A | 3.32 | 19/06/2018 | 0.00 | 0.07 | 0.15 | 14.00 | 85.00 | Soft grey clayey gravelly very silty fine to coarse SAND. Gravel is fine to medium subrounded flint and quartz. |
| MBH2 | 3.13 | 13/06/2018 | 0.00 | 0.03 | 0.05 | 10.00 | 37.00 | Soft brownish-grey very sandy, very silty CLAY. |
| MBH3 | 3.53 | 18/06/2018 | 0.11 | 0.40 | 0.54 | 20.00 | 35.00 | Dark grey very gravelly silty fine to coarse SAND. Gravel is fine to coarse subrounded flint. |

| mm Passing | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Sample** | **Deck Level (mOD)** | **Date** | **D10** | **D50** | **D60** | **D100** | **Moisture Content (%)** | **Comments** |
| MBH4 (noted as abandoned however there is a PSD card.) | 2.95 | 15/06/2018 | 0.00 | 0.20 | 0.35 | 38.00 | 28.00 | Soft grey very silty very gravelly clayey fine and medium SAND. Gravel is medium to coarse subrounded to subangular flint, concrete and sandstone. Some shell fragments. |
| MBH4A | 3.01 | 17/06/2018 | 0.22 | 0.49 | 0.62 | 38.00 | 17.00 | Olive rapidly Weathering to brown very gravelly medium SAND. Gravel is fine to coarse subrounded to angular flint |
| MBH5 | 3.23 | 20/06/2018 | 0.00 | 0.08 | 0.14 | 5.00 | 86.00 | Very soft dark greyish orange very sandy clayey SILT. |
| MBH6 | 2.94 | 24/06/2018 | | | | | | NO REPORT CARD |

| | | | mm Passing | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Sample** | **Deck Level (mOD)** | **Date** | **D10** | **D50** | **D60** | **D100** | **Moisture Content (%)** | **Comments** |
| MBH7 | 3.15 | 25/06/2018 | 0.25 | 0.55 | 0.92 | 14.00 | 16.00 | Greyish brown very gravelly medium to coarse SAND. Gravel is fine and medium angular to subrounded flint, concrete and occasional shell fragments. |
| MBH8 | 3.30 | 21/06/2018 | 0.09 | 0.24 | 0.26 | 14.00 | 47.00 | Grey and brown slightly gravelly silty fine and medium SAND. Gravel is medium angular to subangular flint. |
| MBH9 | 3.07 | 03/07/2018 | 0.24 | 0.38 | 0.41 | 10.00 | 17.00 | Brown medium SAND. |
| MBH10 | 3.33 | 22/06/2018 | 0.14 | 0.29 | 0.32 | 2.00 | 27.00 | Dark grey and black medium SAND. |

## 4.3 Tidal Prism

4.3.1 The tidal prism of an estuary is defined as the volume of water between the mean high-water level and mean low-water level or in other words the volume of water that exits the estuary on the ebb tide. The prism is used to gain an understanding of the potential sediment movement through the estuary because it is this water that contains the sediment and directly links to sedimentation/erosion.

4.3.2 The River Yare has an unusual estuary mouth because the first section of the estuary is a narrow, defended channel through the town centre which then opens into the large mudflats and saltmarsh of Breydon Water. In order to calculate the tidal prism, the estuary boundary has been defined as the section of the Yare through Great Yarmouth town centre and Breydon Water.

4.3.3 Plate 4-2 shows the parts of the channel considered the estuary for the purposes of calculating the tidal prism. The river area is shown by the blue polygon and Breydon Water area has been shown by the red polygon. To calculate the tidal prism, the baseline model has been used to calculate the surface area of the water at the MHWS and the MLWS. The volume between the two surfaces is then calculated. To further understand how the estuary works, the tidal prism for only the River Yare channel has also been calculated. This helps to understand the impact of Breydon Water on the tidal dynamics in the area. Table 4-2 lists the tidal prism calculated in the estuary rounded to the nearest 1,000 $m^3$.


**Plate 4.2 – Tidal Prism Boundary**



*Plate 4-2: Tidal Prism Boundary*

*Table 4-2: Calculated Tidal Prism*

| | |
|---|---|
| **MHWS Level** | 1mAOD |
| **MLWS level** | -0.6mAOD |
| **Baseline Tidal Prism River Yare** | 617,000m$^3$ |
| **Baseline Tidal Prism Breydon Water** | 4,504,000m$^3$ |
| **Total Baseline Tidal Prism** | 5,121,000m$^3$ |

## 4.4 Bathymetry

4.4.1 Peel Ports Great Yarmouth have provided bathymetry data of the River Yare collected in 2017. The bathymetry collected is within the port's jurisdiction between the river mouth and Haven Bridge. Plate 4-3 shows a typical cross section in the River Yare channel at the Principal Application Site.

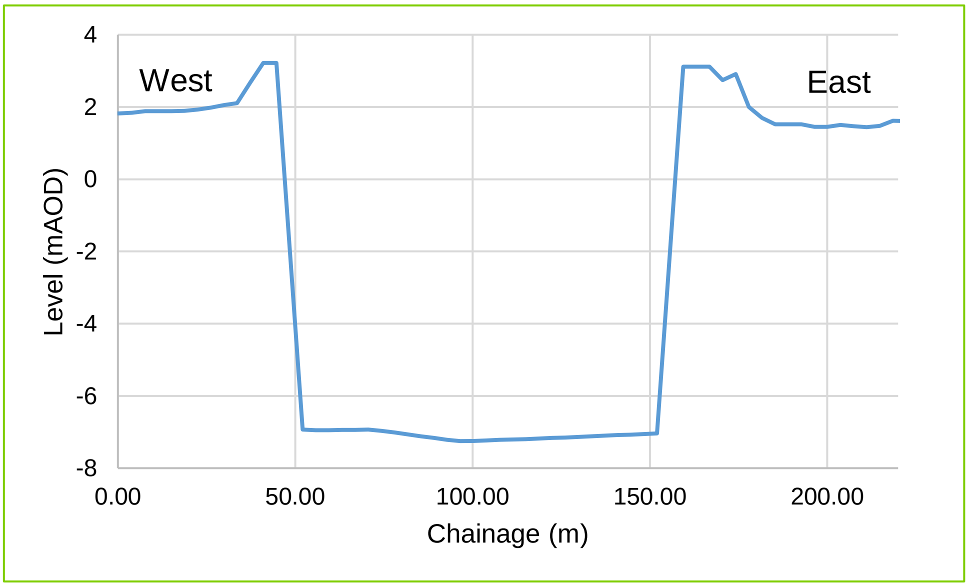


*Plate 4-3: Typical River Cross Section: River Yare*

4.4.2 Plate 4-3 shows that the channel bed is around -7mAOD. This is consistent along the full length of the channel through Great Yarmouth and is maintained by regular dredging undertaken by Peel Ports Great Yarmouth. No bathymetry data has been obtained for Breydon Water, however the 2011 Halcrow/EA flood model uses 1D cross sections to represent the lake. Plate *4-4* shows a typical cross section through Breydon Water, there is a deep central channel with slope sections either side representing the mudflats and saltmarsh of Breydon Water.

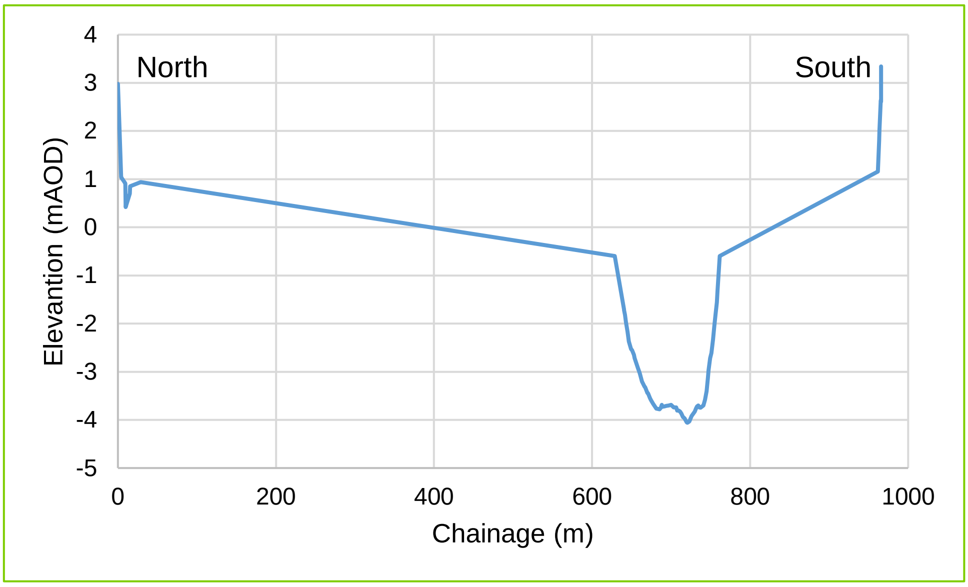


*Plate 4-4: Typical Lake Cross Section - Breydon Water*

## 4.5 Tidal Symmetry

4.5.1 Tidal symmetry compares speed against elevation to show whether a tidal system is ebb or flood dominant. For this assessment, the model results from the 13th-16th April 2018 tidal cycle simulation have been plotted on Plate 4-5 and Plate 4-6. This cycle has been obtained at the gauge at Gorleston-on-Sea and represents the period of time when the velocity survey was conducted.

4.5.2 Plate *4-5* shows the water level and speed plotted against time and Plate 4-6 shows the water level plotted against speed for the 13th-16th April 2018 tidal cycle in the channel near the Principal Application Site. The plots suggest that the estuary is almost tidally symmetrical (a perfectly symmetrical tide would be shown as a circle or an oval on the graph) in the engineered River Yare channel with a slight skew at high water. As there is a need for periodic dredging, it is assumed that sediment is deposited in the channel during slack water and is carried on both the ebb and flood tide.

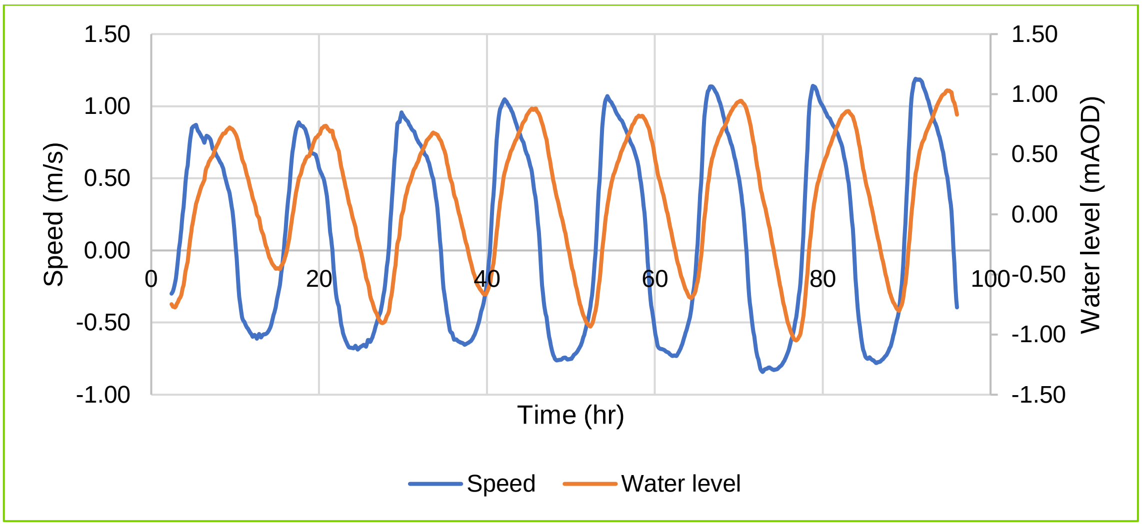


*Plate 4-5: Tidal Boundary*

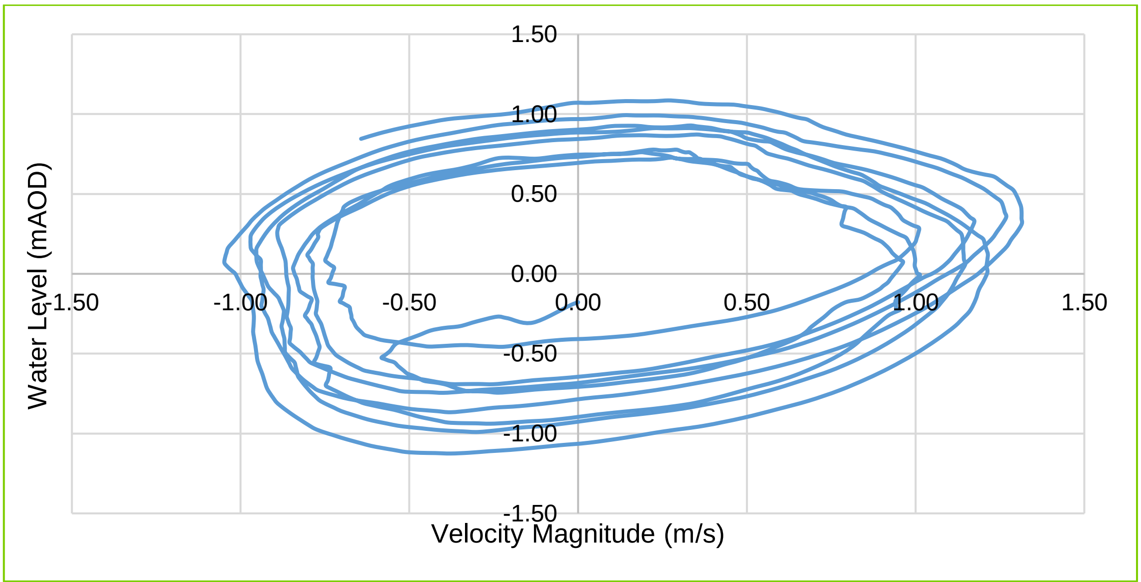


*Plate 4-6: Velocity Magnitude against Water Level at the Scheme Site – Baseline Model*

## 4.6 Dronker's Ratio and Estuary Type

4.6.1 The Dronker's Ratio is a measure of tidal dominance and is used to assign a type to an estuary. This is used here to assess the tidal dominance of the estuary as a whole. There are two types of estuary; Type I and Type II. A type I estuary is a deep, wide channel that is typically filling up with sediments. As the intertidal flats of the estuary develop, the sediment supply on the flood is reduced and new morphology is attained. Type II estuaries typically excrete sediment on the flood tide, which has the effect of eroding the intertidal plain and reverting the estuary to Type I. A typical estuary oscillates between Type I and Type II in a dynamic equilibrium.

4.6.2 The Dronker's Ratio provides a numerical measure of tidal dominance and is calculated using the surface area and volume of the high and low tidal levels in the estuary following EA guidance (Ref 11C.3). The estuary is defined as shown in Plate 4-7 and Plate 4-8 wetted areas. The high and low tide levels are 1mAOD and -0.6mAOD respectively.

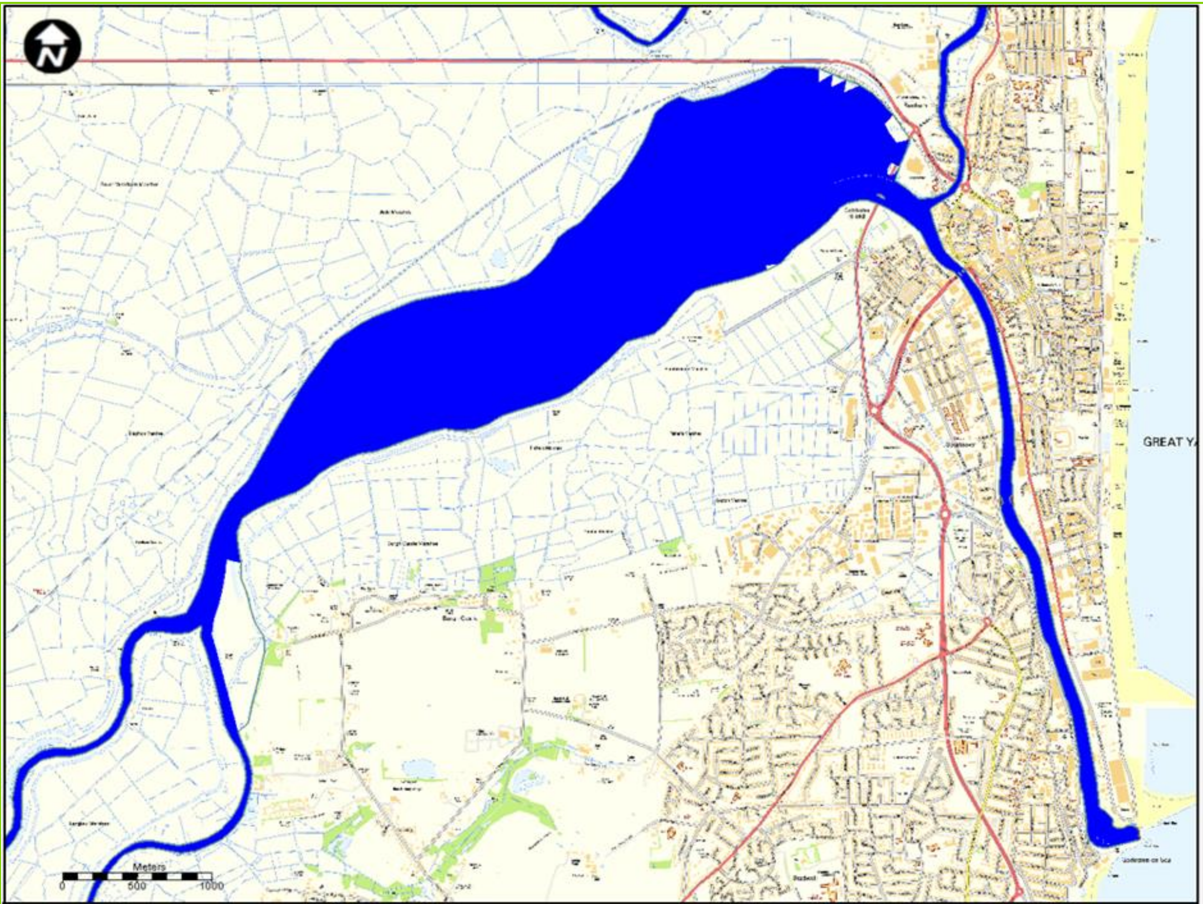


Plate 4.7 – Wetted Area, High Tide



*Plate 4-7: Wetted Area, High Tide*


**Plate 4.8– Wetted Area, Low Tide**



*Plate 4-8: Wetted Area, Low Tide*

*Table 4-3: Baseline Dronker's Ratio Calculation*

| Measure | Baseline |
|---|---|
| Hydraulic depth, dh | 3.88 |
| Tidal Amplitude, a | 0.58 |
| Surface area at low water, Slw | 1318636m$^2$ |
| Surface area at high water, Shw | 4916929m$^2$ |
| Volume at high water, Vhw | 9475544m$^3$ |
| Volume at low water, Vlw | 4357856m$^3$ |
| Dronker's (dh) | 0.49 |

4.6.3 The Dronker's Ratio shown in Table 4-3 shows that the estuary is an ebb dominated environment. The Dronker's Ratio of 1 shows no tidal dominance.

A value lower that one highlights an Ebb dominant environment and great than one shows a Flood dominant environment. However, Great Yarmouth is not a typical estuary because of the narrow channel through Great Yarmouth town centre. The engineered channel hydraulically controls the flow of the water and by extension the sediment transport in and out of Breydon Water. To that end, the impact of the engineered channel means that Breydon Water is excreting sediments at a slower rate than would otherwise be expected in such an estuary.

4.6.4 The combination of the cross section shown in Plate 4-4 which shows the shape of the lake and the Dronker's Ratio suggests the estuary is Type II and considered Ebb dominant.

# 5 Model Build

## 5.1 Overview

5.1.1 A 3D tidal model has been built in TUFLOW-FV to represent the River Yare including Breydon Water at Great Yarmouth. Baseline and Scheme versions of the model have been created. The model built for this study is detailed in Section 5.2. Section 5.3 describes the model calibration process that has been undertaken. TUFLOW-FV uses an unstructured grid to resolve the 3D flow characteristics of the watercourse. A 3D model can significantly increase the amount of information and detail compared to a 2D model.

5.1.2 In addition to the hydraulic calculations, the TUFLOW-FV model built for this assessment includes an explicit sediment transport module. This module explicitly calculates the bed load, erosion and deposition rates of sediment particles in the watercourse by using the velocity magnitude to calculate the bed shear stresses. The model provides detailed velocity magnitude results to be used in the sediment transport module. This is beneficial when considering sediment transport as it is the velocity magnitude in the lower section of the water column that drives sediment transport.

5.1.3 The unstructured grid (flexible mesh) method allows the user to efficiently use the computational power available by specifying a high resolution in areas of interest and lower resolution elsewhere. This is particularly useful when the results needed are focused in a small spatial area, as for the Scheme, for example, around bridge supports.

## 5.2 Model Build

### Model Domain

5.2.1 The model domain extends from the harbour entrance at Gorleston-on-Sea to Breydon Water and includes representation of the River Yare and the River Bure upstream of Breydon Water. It is assumed that the worst-case scenario for the velocity magnitude will be before the water level exceeds the harbour walls therefore it is not considered necessary to include any floodplain representation within the model. The harbour entrance is approximately 2.5km from the Principal Application Site, which is sufficient distance to ensure that any boundary effects do not influence the area of interest. Plate 5-1 shows the model domain used in this assessment.

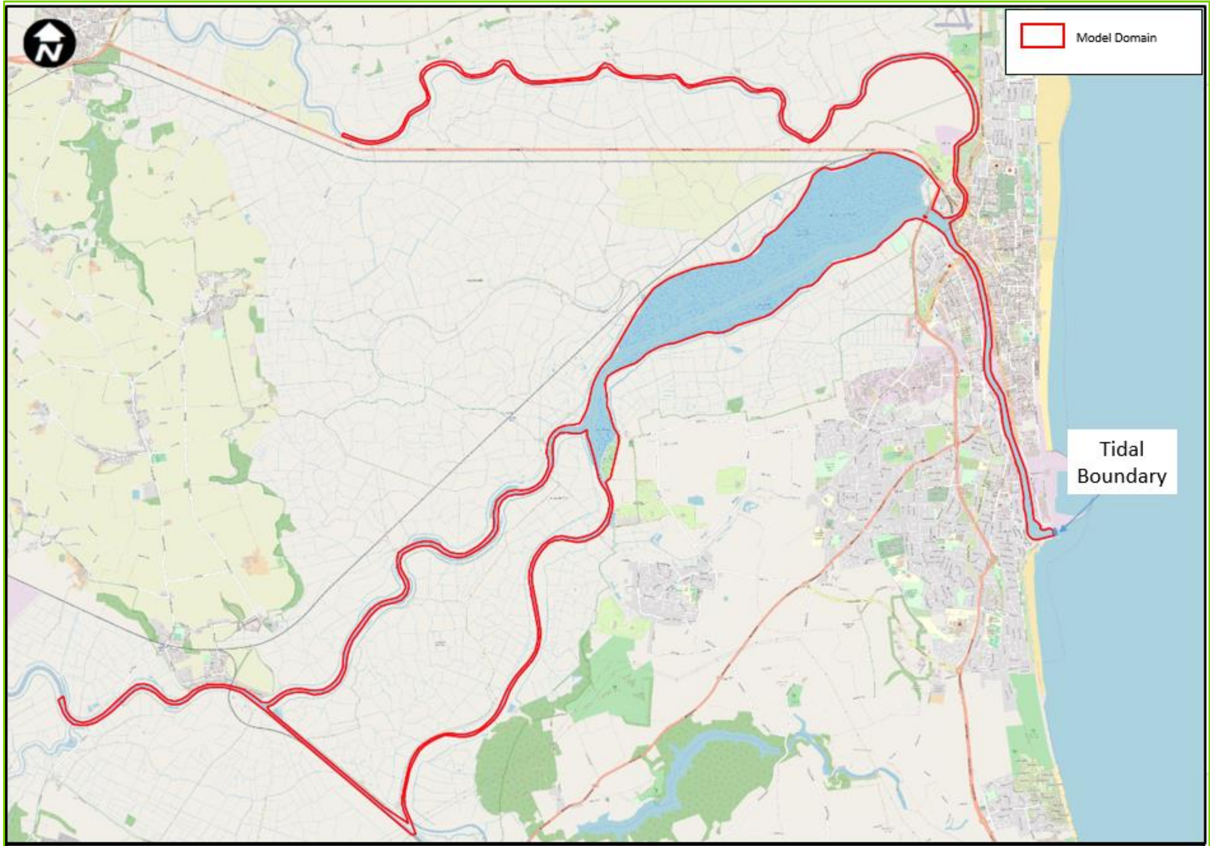


Plate 5.1 – Model Domain Boundary



*Plate 5-1: Model Domain Boundary*

5.2.2 The major benefit of the flexible mesh is the ability to vary the resolution across the model domain. The cell size through the domain is dependent on the level of accuracy required in specific locations and computational time. In this model build, it was considered necessary to simulate the channel at the Principal Application Site at an ultra-high resolution (approximately 3m by 3m) to obtain the highest level of detail in the area where the largest impacts will occur. The cell size increases further away from the Scheme to approximately 5m by 5m in channel. Breydon Water and the reaches of the River Yare and River Bure upstream of this have been simulated at a lower resolution. The lower resolution is considered appropriate to simulate the areas that are a significant distance from the Principal Application Site. Plate 5-2 shows the resolution of various areas within the model domain. Plate 5-3 shows the Scheme representation in the model grid where the bridge knuckles extend into the channel from both quays leaving an approximately 50m wide channel between them.

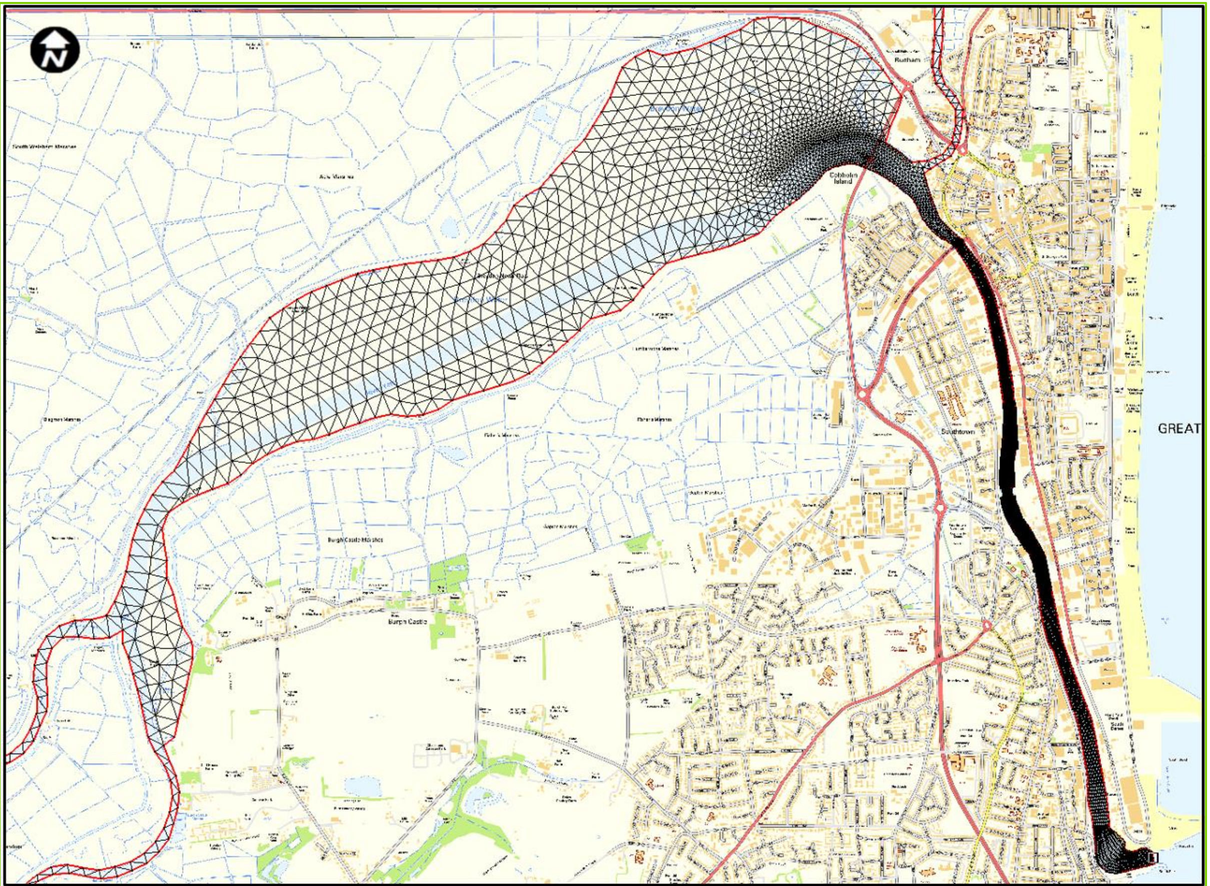


**Plate 5.2 – Model Mesh**



*Plate 5-2: Model Mesh*

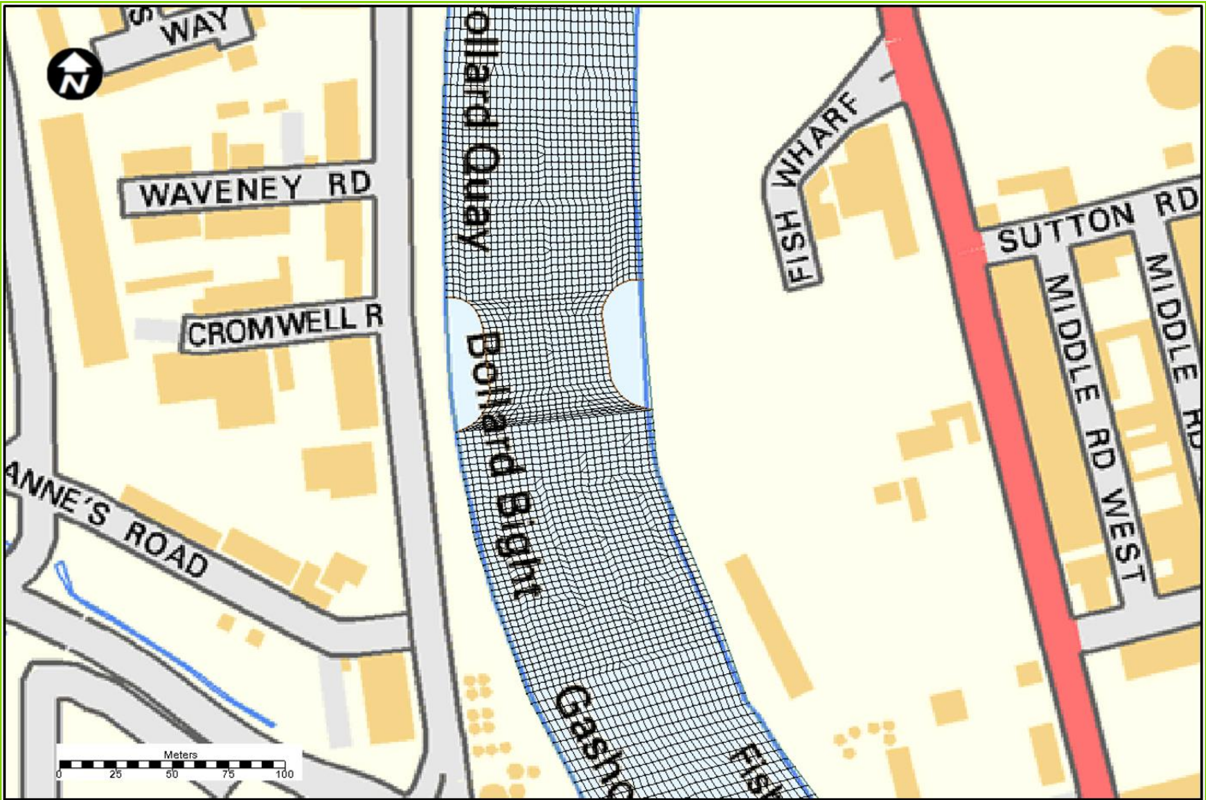


Plate 5.3 – Model Mesh at the Principal Application Site



*Plate 5-3: Model Mesh at the Principal Application Site*

5.2.3 The benefit of the flexible mesh, finite volume method in TUFLOW-FV is that different sized polygons can be used with no connection/flux errors, which are possible in a finite difference model. This means triangles and quadrilaterals can be used alongside each other in the model mesh, however it is considered best practice to use quadrilaterals where possible because it improves run times. In addition, different sized polygons can be used next to each other providing they share two node connections without any impact on the calculations, a visual check of all the outputs was carried out to ensure connectivity. In this assessment, the best representation was to use predominantly triangular cells. Higher model run times have been accepted in order to improve the model calculations in this case.

5.2.4 There are currently two bridge crossings in Great Yarmouth; Haven Bridge and Breydon Bridge represented in the model. For the purpose of this assessment, the bridges have been represented by simulating the bridge knuckles in the mesh and no representation of the bridge decks. This is because both existing bridge decks are higher than the events simulated, therefore they will not interact with the water. It is the bridge support structures that have an impact on the sediment transport. Haven Bridge has two main support structures which have been explicitly modelled. Breydon Bridge has one large support and several smaller supports. The large support, which supports the bascule bridge section and lifting mechanism has been explicitly represented in the model. Due to the resolution of the model at Breydon Bridge, the smaller piers are not represented. This is considered suitable because the supports, when compared to the main structure are much smaller and the impacts of the supports will not affect the watercourse at the modelled resolution.

### Roughness Values

5.2.5 As part of the model setup, initial roughness values have been applied to the model. Following review of the study area, it was considered appropriate to split the model up into three different environments, which each have a different roughness value. Table 5-1 shows the roughness values used in the model. The values have been selected using typical values and following engineering guidance.

*Table 5-1: Roughness Values*

| Area | Roughness (Manning's *n*) |
|---|---|
| Smooth dredged Channel | 0.03 |
| Natural (un-dredged) river channel | 0.04 |
| Lake/mudflats | 0.05 |

5.2.6 The domain was split into three roughness regions; smooth dredged channel, natural (un-dredged) river channel and lake/mudflats. The smooth dredged channel roughness has been applied to the channel through Great Yarmouth from the North Sea boundary at Gorleston-on-Sea to Haven Bridge. A Manning's *n* value of 0.03 has been used for this section because of the periodic dredging activity which will remove any vegetation growth on the river bed that causes additional drag. The channels of the River Bure and River Yare upstream of Haven Bridge have been defined as a natural (un-dredged) river channel. This is defined as an un-dredged channel where vegetation may grow and therefore cause increased energy losses, a Manning's *n* roughness value of 0.04 has been applied to these areas. Breydon Water has been defined as an area where vegetation can grow in large quantities, a Manning's *n* value of 0.05 has been applied in this area to simulate the energy losses associated with this.

### Model Topography

5.2.7 The bathymetry data provided by Peel Ports Great Yarmouth has been used to define the bed levels in the River Yare. Peel Ports Great Yarmouth conducted the survey between the harbour entrance and Haven Bridge, as shown on Plate 5-4. The dataset, recorded in 2017, consists of data points taken from a boat traversing the harbour.

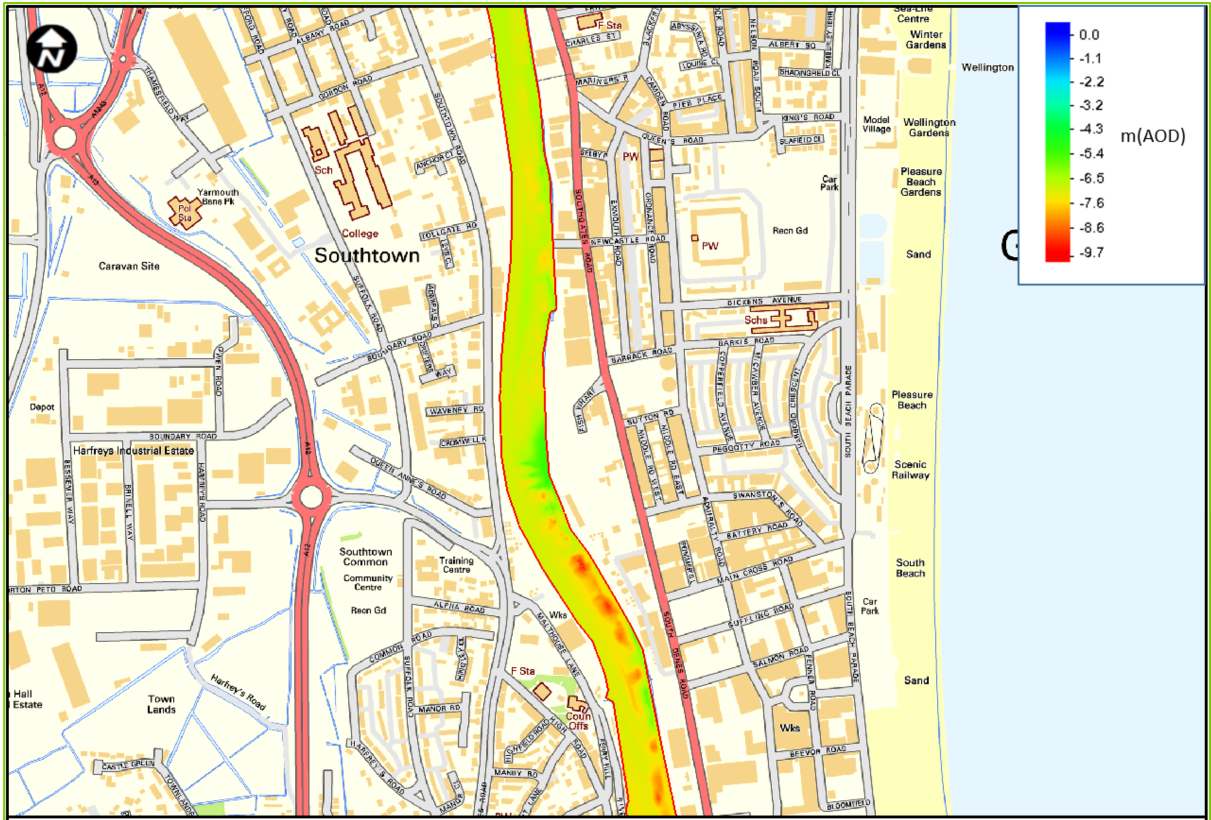


**Plate 5.4 – Bathymetry at the Principal Application Site**



*Plate 5-4: Bathymetry at the Principal Application Site*

5.2.8 There is limited information available for the river bed upstream of Haven Bridge. The flood models received for use in this project, contain 1D cross-sections defining the river channels upstream of Haven Bridge, the bed levels in these sections range from -7mAOD to -4mAOD. It was not clear from the flood model supporting information where the data used to define the levels was from. In order to be conservative, the main channels of the River Yare and River Bure have been set at a constant depth of -7m AOD upstream of Haven Bridge. This approach has been adopted because the upper reach of the model has been included to provide sufficient storage within the system and there is not a need to represent the river sections in detail.

5.2.9 In order to represent Breydon Water, LiDAR levels have been used. The flights are often flown at or near low tide therefore the dataset can be used to set the bathymetry in the lake assuming the water levels will always be greater than this. Breydon Water has been represented using a coarse resolution approach, therefore LiDAR provides sufficient information for the bathymetry for this model.

### Boundary Conditions

5.2.10 The North Sea tidal boundary is located to the south east of the Principal Application Site. The tidal curves derived for this assessment as summarised in Section 3 have been applied to this boundary in the model. The tidal boundary is applied at the river mouth and forces the water levels and flows in the model. No fluvial boundaries have been applied to the model because the catchment has a strong tidal dominance which can be seen on gauges much further upstream. To that end, it is unlikely that a small fluvial inflow will have a measurable impact on the hydraulics within the River Yare through Great Yarmouth.

### Structures

5.2.11 There are two existing structures on the River Yare in Great Yarmouth, these are Haven Bridge and Breydon Bridge. Both the Haven Bridge support structures have been represented in the model, this creates a constriction in the channel simulating the impact of the bridge on the water flow. Breydon Bridge has been represented by explicitly simulating the main support for the bascule bridge span, the smaller support piles are not modelled because they are significantly smaller than the grid resolution. This means that any impact of the piles would not be seen in the calculation. This approach is considered appropriate because the impact of the piles on the hydraulics of the channel will be very small and highly unlikely to affect the Principal Application Site location, which is 2.5km away.

### Salinity and Temperature

5.2.12 As the River Yare is tidally dominated, the water in the estuary is mostly saline, warm coastal ocean water. Salinity (35g/kg) and temperature (20°C) has been applied to water coming in through the tidal boundary in the model. TUFLOW recommends the use of these values as they represent typical value in the coastal oceans around the UK. The use of salinity and temperature values impact the density calculations undertaken by the model, therefore these parameters are considered important in the sediment transport modelling.

### Sediment Parameters

5.2.13 A number of sediment samples have been collected from sample locations close to the Principal Application Site as reported in Section 4.2. The PSD assessment has been carried out detailing the size and type of the particles found. Using the D50 (the 50$^{th}$ percentile particle size passing through the sieve) value, the sediment found ranges from 0.03mm to 0.55mm in size with the larger particles typically found close to the eastern quay wall. The model has been set up to simulate silt and sand sediment types that are typically found in the River Yare channel.

5.2.14 TUFLOW-FV has the capability of simulating sediment deposition using a range of methods from applying a simple settling velocity to each particle type to a full salinity induced flocculation and hindering assessment. TUFLOW recommends the use of the simplest method (the settling velocity method) first. It is only when the expected results cannot be achieved that more complicated methods should be considered. As such, this assessment calculates sediment deposition by assigning each sediment type with a settling velocity. In this assessment, the sediment settling velocity has been obtained using the Ferguson and Church method (Ref 11C.4).

5.2.15 Erosion is dealt with by calculating the critical shear stress using the bed velocity magnitude. Each sediment type has an assigned critical erosion shear stress, which is used to determine when the sediment becomes mobile.

5.2.16 Following the sediment sample survey, the PSD survey concluded that there are two main sediment types in the channel; sand and silt. The model has been set up to simulate these sediment types using the parameters specified in Table 5-2.

*Table 5-2: Sediment Type Model Parameters*

| Parameter | Sand | Silt |
|---|---|---|
| Settling Velocity (m/s) | $2x10^{-2}$ | $1x10^{-5}$ |
| Critical Shear for deposition ($N/m^2$) | Nan – special treatment for sand in Tuflow FV. | 0.1 |
| Material Density ($kg/m^3$) | 2650 | 2650 |
| Critical Shear for erosion | Top Layer: 0.12<br>Bottom Layers: 0.2 | Top Layer: 0.12<br>Bottom Layers: 0.2 |

5.2.17 TUFLOW-FV uses a layered approach to simulate a river bed. For example, if a silt layer is found on top of a sand layer then it follows that the silt will be eroded first before the sand layer can be mobilised. All deposited material will always be on the top layer. For this model, it was appropriate to represent the bed initially using a two-layer approach; the first layer is silt dominant and the second layer sand dominant. Plate 5-5 shows a graphical representation of the bed as simulated in the model.

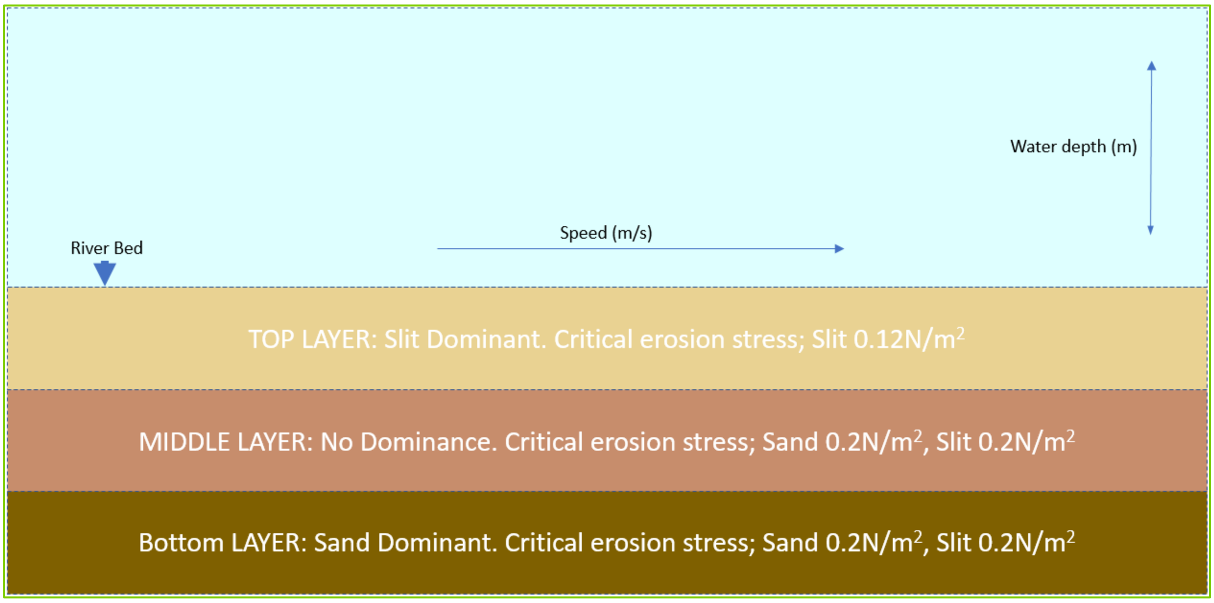


*Plate 5-5: Sediment Model Layers Schematisation*

5.2.18 In order to set up the model, the sediment has to be initially distributed around the model manually. The model is set up to have 1325kg/$m^2$ of silt in the top layer, 1325 kg/$m^2$ of sand and silt in the middle layer and 5300kg/$m^2$ Sand in the bottom layer. The purpose of this approach is to introduce sediment into the model with can be transported around the domain using the hydrodynamic calculations.

### Baseline Model

5.2.19 Once the initial baseline model had been developed as described above, a series of calibration tests have been carried out to ensure the model is an accurate representation of the River Yare through Great Yarmouth. The calibration process has been carried out by comparing the model predicted velocities to the velocity survey outputs from 2018. The calibration process is discussed in Section 5.3.

### Scheme Model

5.2.20 The Scheme has been represented by modelling the bridge knuckles as blocked out areas of the river channel as shown in Plate 5-3. As the water levels in this assessment will not exceed the defences, there is no requirement to represent any of the Scheme that is outside of the water channel including the embankments for the approach roads or any of the Satellite Application Sites.

### Construction Phase Model

5.2.21 The construction method for the Scheme is expected to take up the same footprint as the finished Scheme knuckles. This means the results of the model created to assess the final Scheme arrangement is the worst possible case. As such, no additional modelling is required and the Scheme model results have been used to also assess the impact on the sediment regime during construction.

### 3D Representation

5.2.22 The model will be simulated using the hybrid 3D discretisation. The initial layer density has been set as 1m resolution to balance computational time and the accuracy of the calculations. The model has been simulated using the 1m vertical resolution at the bed. This resolution is considered appropriate for the assessment of the sediment in Breydon Water and the River Yare. The sediment transport model uses the velocity at the river bed to calculate the shear stress, which drives the sediment transport and therefore uses high resolution results

## 5.3 Model Calibration

5.3.1 The model described in Section 5.2 has been calibrated to a number of parameters. As part of the Scheme, a velocity survey has been carried out at nine locations in the River Yare through Great Yarmouth as shown in Plate 5-6.

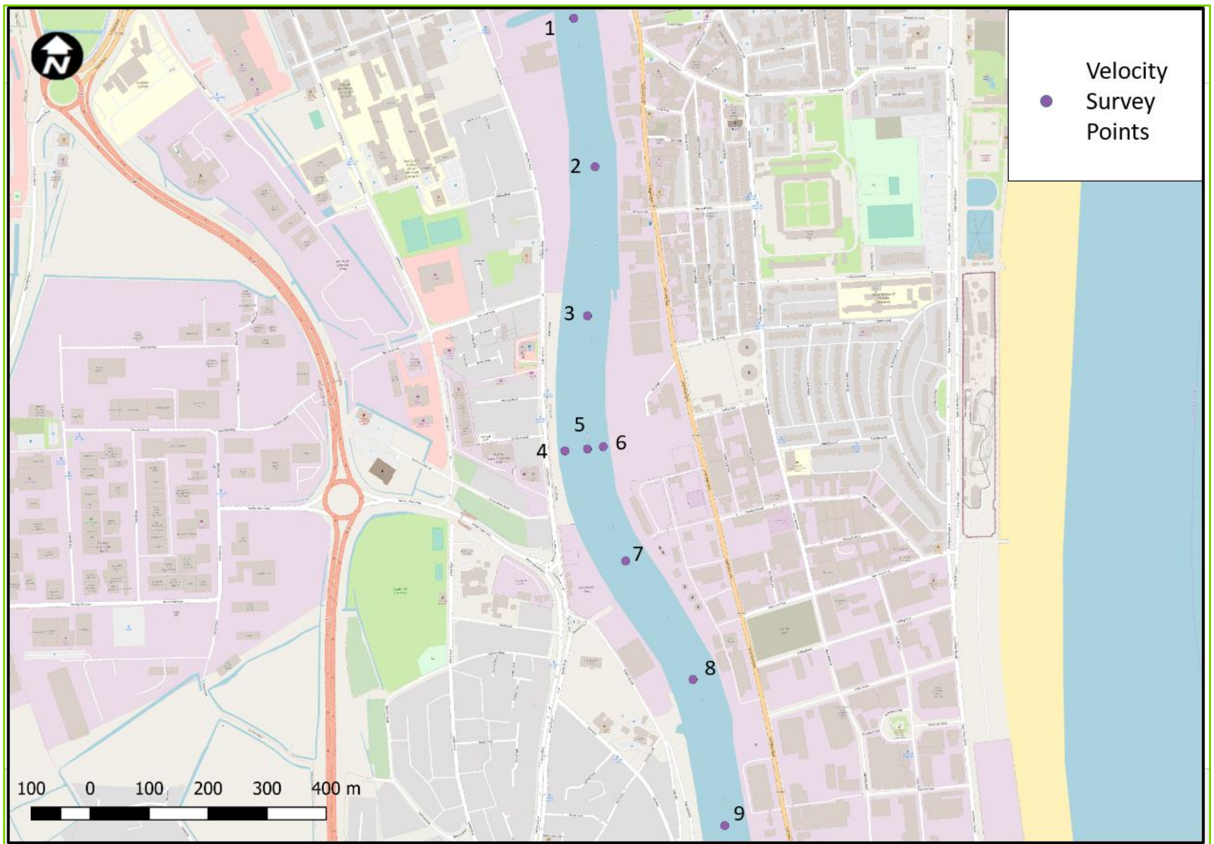


Plate 5.6– Velocity Survey Locations



*Plate 5-6: Velocity Survey Locations*

5.3.2 At each location shown in Plate 5-6, an Acoustic Doppler Current Profiler (ADCP) has been used to obtain the velocity. The data has been recorded for a period of 5 minutes every hour for a day at each location and the velocity magnitude through the water column has been recorded. The model has been set up to simulate the same period of time ($13^{th}$-$15^{th}$ April 2018) by obtaining the tidal levels from the Gorleston-on-Sea gauge for this period. To calibrate the model, predicted 2D velocity magnitude data has been exported from the model at each of the locations shown in Plate 5-6 and compared to the survey data. The model is calibrated to the 2D depth averaged velocity, this helps to negate the effect of specific differences in flows due to potential small sources of water such as drainage pipes or moving boats on the surface which the model cannot predict. This method is considered suitable for this model.

5.3.3 The calibration model run presented uses the tidal cycle for a weekend in April 2018 and simulates a four-day period (13th-16th). Plate 5-7 shows the water level plotted against hours used as the model boundary in the calibration event.

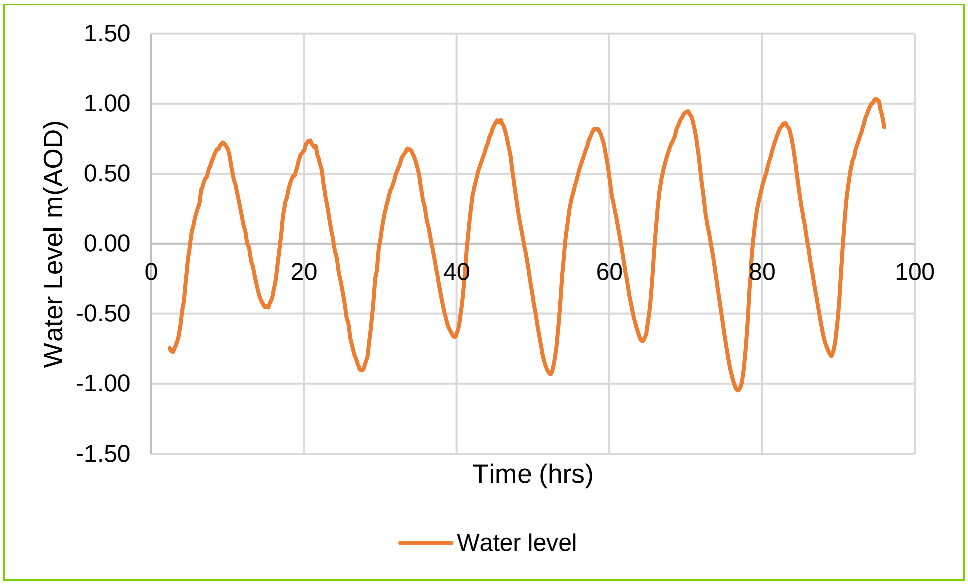


*Plate 5-7: Gorleston-on-Sea Gauge Recorded Water Level - 13th-16th April 2018*

5.3.4 The model has been simulated for the four day tidal period in April 2018 and Plate 5-8, Plate 5-9 and Plate 5-10 show a comparison of 2D depth averaged velocity magnitude between the model and the recorded data at velocity survey locations 4, 5 and 6 respectively. Plate 5-8 shows that the model predicts the peak velocity well at survey point 4. There are some differences between the model results and the survey data, which are likely due to local impacts such as vessel movements that can impact the survey results. It is not possible to replicate these impacts in the model. Plate 5-9 shows the model predicts velocity magnitudes well, although there are some discrepancies. Plate 5-10 shows that the model matches the survey data very well in this location. In the central section of the graph, the survey and speed match very closely.

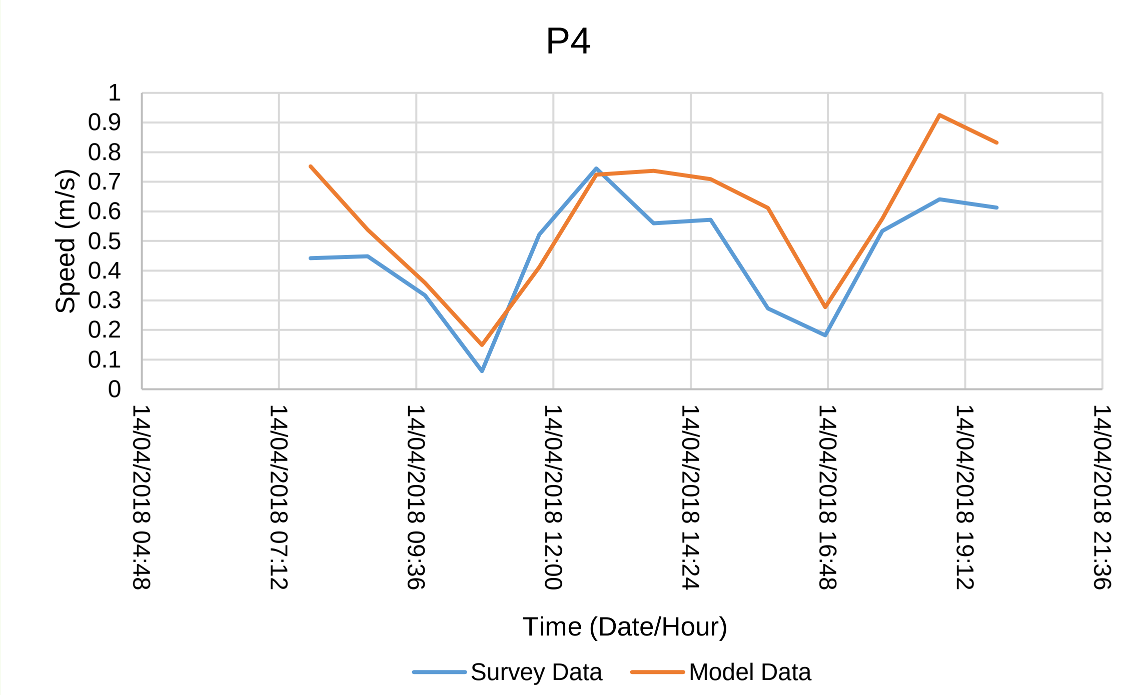


*Plate 5-8: Comparison of Modelled and Recorded Water Speed at Survey Point 4*

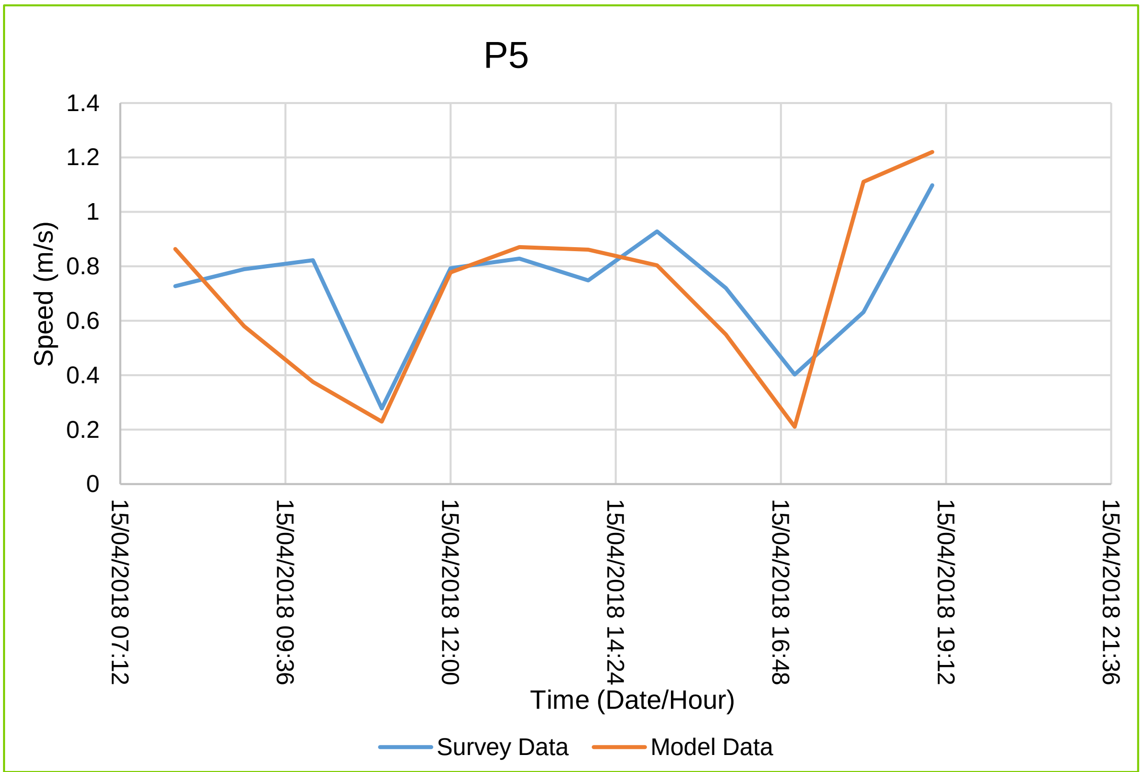


*Plate 5-9: Comparison of Modelled and Recorded Water Speed at Survey Point 5*

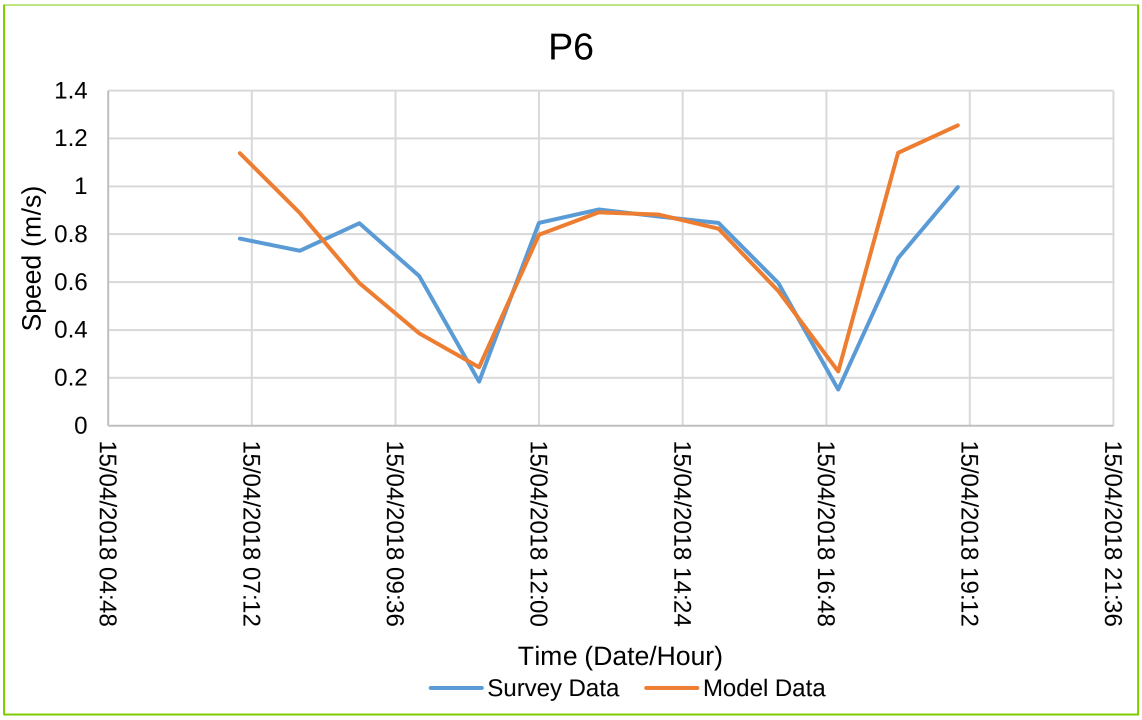


*Plate 5-10: Comparison of Modelled and Recorded Water Speed at Survey Point 6*

5.3.5 In addition to the plates shown, the model represents the depth average velocities well. The calibration process has shown that the model is capable of predicting the velocity magnitude in the River Yare near the Scheme well by matching the velocity magnitude of the recorded data well. Following the calibration process, the model is considered suitable for use in the sediment assessment.

# 6 Impacts of the Scheme

## 6.1 Model Runs

6.1.1 The model has been used to assess the sediment transport by simulating the four different tidal events described in Section 3 for the Baseline and the Scheme scenarios. For each tidal event, the impact of the Scheme has been determined by comparing the model results between the Baseline and Scheme scenarios. The events that have been simulated in the model are listed in Table 6-1.

*Table 6-1: Model Simulations*

| Baseline | Scheme |
|---|---|
| **Everyday Events** | |
| Spring | Spring |
| Neap | Neap |
| **Extreme Events** | |
| MHWS to MLWS + 5% AEP sea surge | MHWS to MLWS + 5% AEP sea surge |
| MHWN to MLWN + 5% AEP sea surge | MHWN to MLWN + 5% AEP sea surge |

6.1.2 The model has been simulated for a 75 hour tidal period for each event, the first 25 hours of each run is used to stabilise the model. In order to simulate the required resolution at the Principal Application Site in 3D, each model run takes approximately 30 hours to run 75 hours simulation time.

6.1.3 The model has been simulated using the setup described in Section 5. The results have been processed to produce plots and plates to show the difference in sediment transport due to the Scheme. The main driver for sediment transport is velocity magnitude which is used to calculate the bed stress. Bed stress is the parameter used to predict the sediment deposition and erosion therefore assessing the bed stress provides a good estimate of sediment transport.

6.1.4 In addition to the bed stress, the instantaneous average erosion/deposition rate has been calculated. This rate has been calculated to give a measure of sediment erosion and deposition and to show the areas that will be affected. The model does not include morphological updates because the changes in bathymetry are small and will not significantly change the hydrodynamics and is likely to increase the total time and instability of the model.

6.1.5 Whilst absolute values are used where appropriate, averages are used to provide a measure of erosion/deposition accounting for the influence of the ebb and flood tide and to understand the longer term impacts of the Scheme.

## 6.2 Results – Everyday Tide

6.2.1 The results presented in this section show the impact of the Scheme on the tidal environment and sediment transport processes using a simulation of 75 hours for the Spring and Neap tidal boundary. By using a Spring and a Neap tide, the upper and lower limits of impact can be assessed for a typical year without explicit simulation of a full tidal cycle as this would mean excessive run times. For the purposes of this assessment, the Baseline and Scheme model have been simulated using the same boundary and the results of each compared. Time series outputs of velocity magnitude, water level and bed stress from the model at four locations in the domain; Harbour Entrance, Scheme, Haven Bridge and Breydon Water are shown on Plate 6-1.

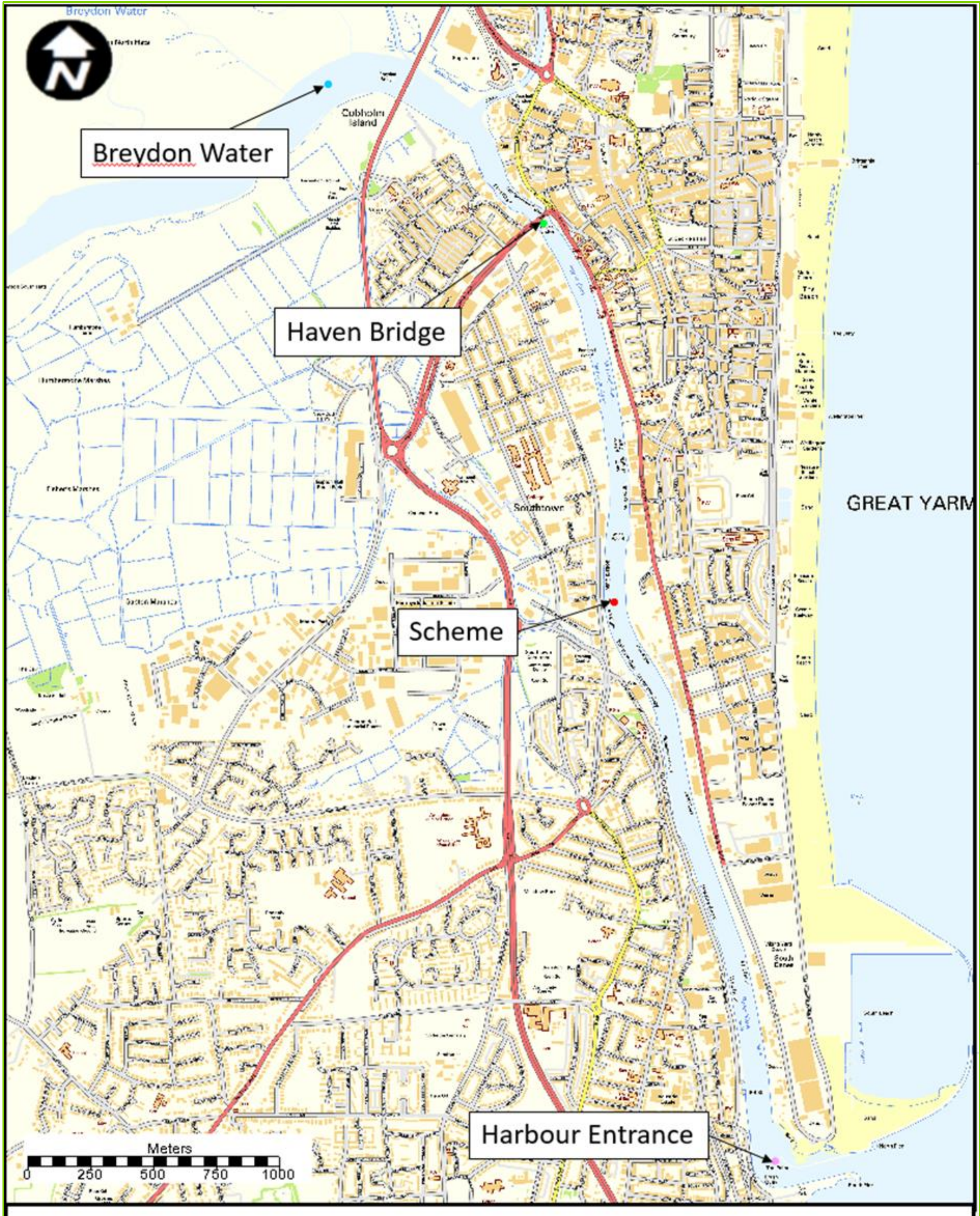


**Plate 6.1 –Time Series Locations**



*Plate 6-1: Time Series Locations*

### Velocity Magnitude and Elevation

6.2.2 The velocity magnitude and water level are fundamental to sediment movement. In narrowing the channel caused by the Scheme, the velocity magnitude will increase in order to retain the same capacity. In this section, the velocity magnitude and elevation impacts of the Scheme are discussed for the Spring and Neap tidal events.

Spring Tide Event

6.2.3 Plate 6-2 shows the depth-averaged velocity magnitude between the bridge knuckles for the Spring tidal simulation in the Baseline and Scheme scenarios. The plot shows the Baseline velocity magnitude at the Principal Application Site location peaks at approximately 1m/s as shown by the orange line on the plot. The plot shows that due to the presence of the Scheme (blue line), the water velocity magnitude increases by around 100% to up to 2m/s for the duration of the simulation. This is because the bridge knuckles constrict the change and in order for a similar volume of water to transit the channel, the velocity increases.

6.2.4 Plate 6-3,

6.2.5 Plate *6-4* and Plate 6-5 show the velocity magnitude at Haven Bridge, the harbour entrance and Breydon Water respectively. The plots show there is a negligible change in velocity magnitude due to the Scheme remote from the Principal Application Site.

6.2.6 Plate 6-6 shows the difference (Scheme – Baseline) in velocity magnitude for the four locations in the channel. What is clear from the plot is that the main difference in velocity magnitude is at the Principal Application Site. The plot shows that the constriction that the new bridge causes increases the Baseline velocity magnitude by up to 1m/s in between the bridge knuckles. There are a few times in the tidal event near the harbour entrance where the velocity is affected slightly however, the differences in velocity magnitude are typically less than 0.1m/s.

6.2.7 Plate 6-7 shows a 2D plot of the velocity magnitude for the Baseline and Scheme Spring simulation at 37hr which corresponds with the largest difference in Plate 6-6. The plate highlights the differences in velocity magnitude caused by the Scheme. There is a small change (approximately 1m/s increase) in velocity magnitude at Haven Bridge due to the presence of the Scheme. There is a negligible impact on velocity magnitude elsewhere in the domain.

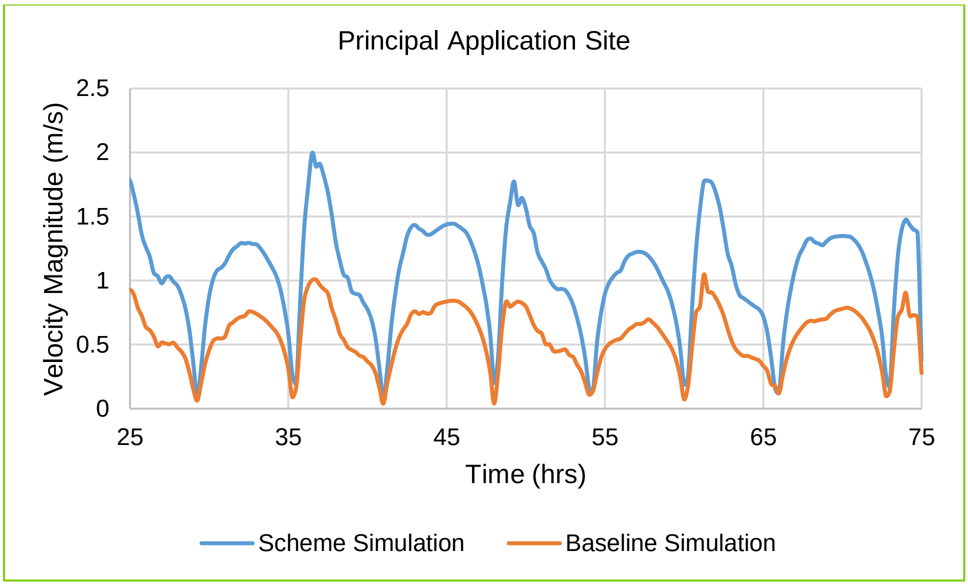


*Plate 6-2: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at the Principal Application Site (between the bridge knuckles) for the Spring Tide*

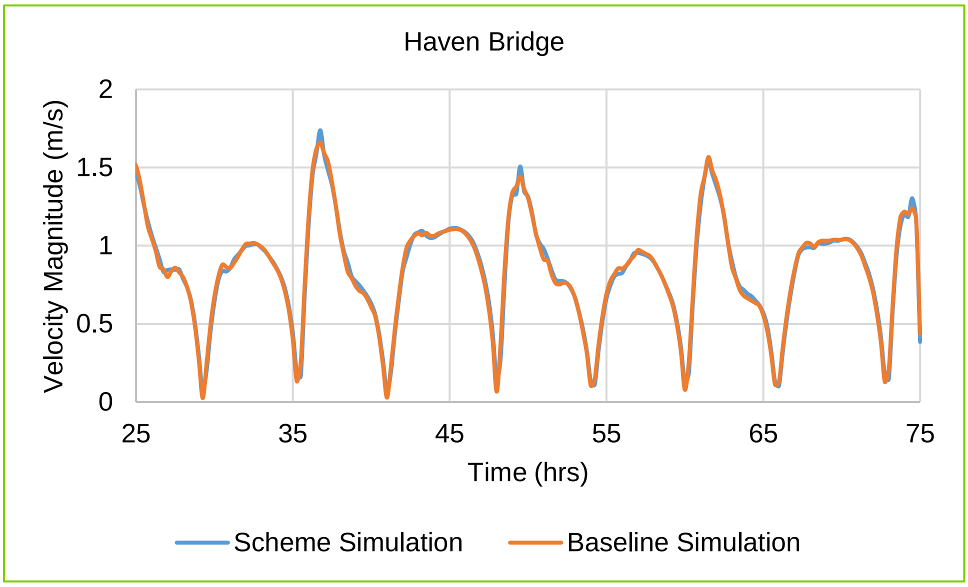


*Plate 6-3: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at Haven Bridge for the Spring Tide*

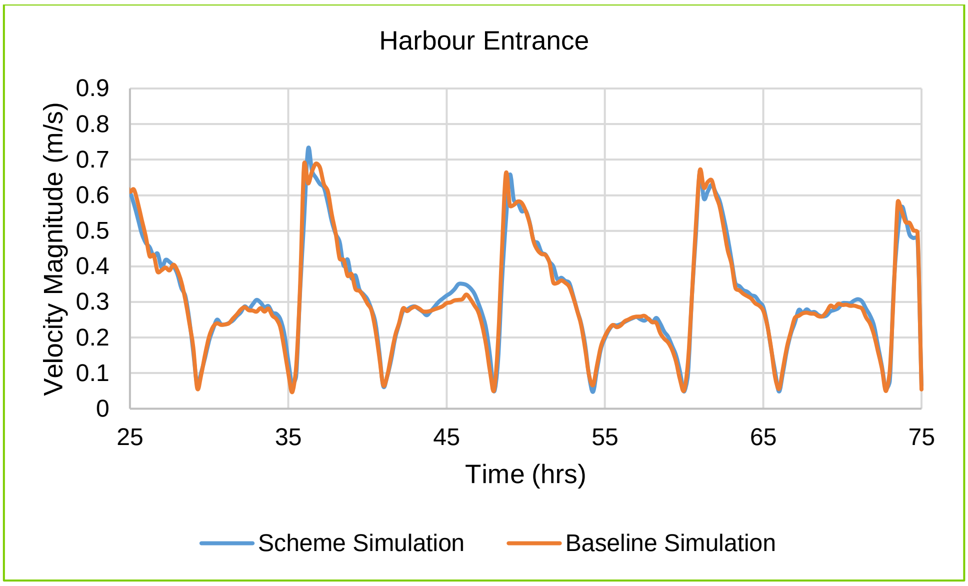


*Plate 6-4: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at the Harbour Entrance for the Spring Tide*

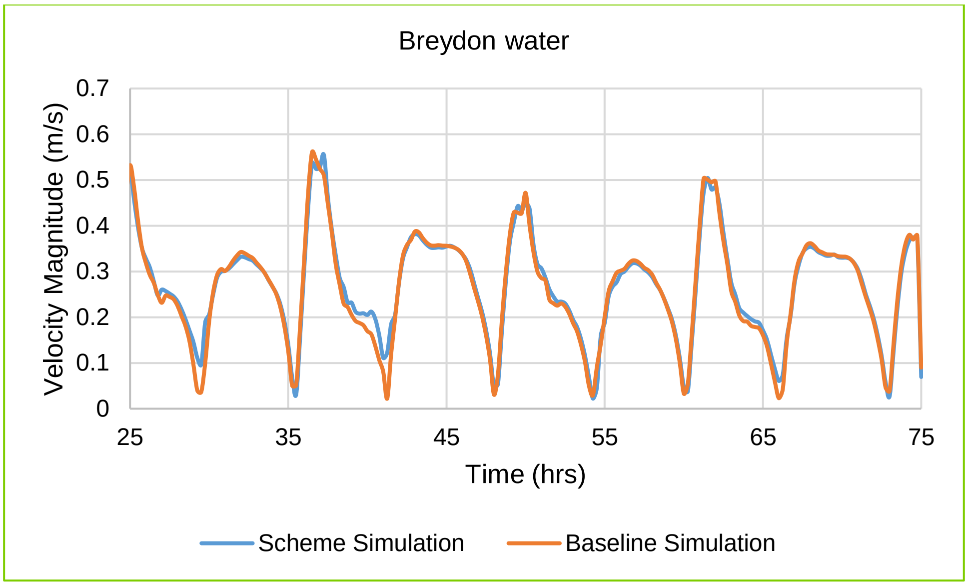


*Plate 6-5: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at Breydon Water for the Spring Tide*

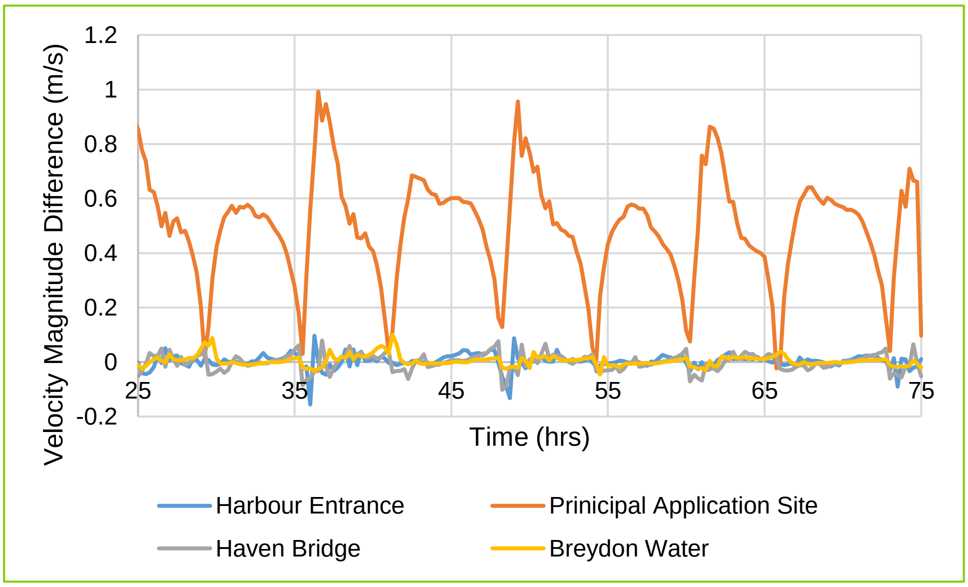


*Plate 6-6: Velocity Magnitude Difference between the Baseline and Scheme Scenarios (Scheme-Baseline) for the Spring Tide*

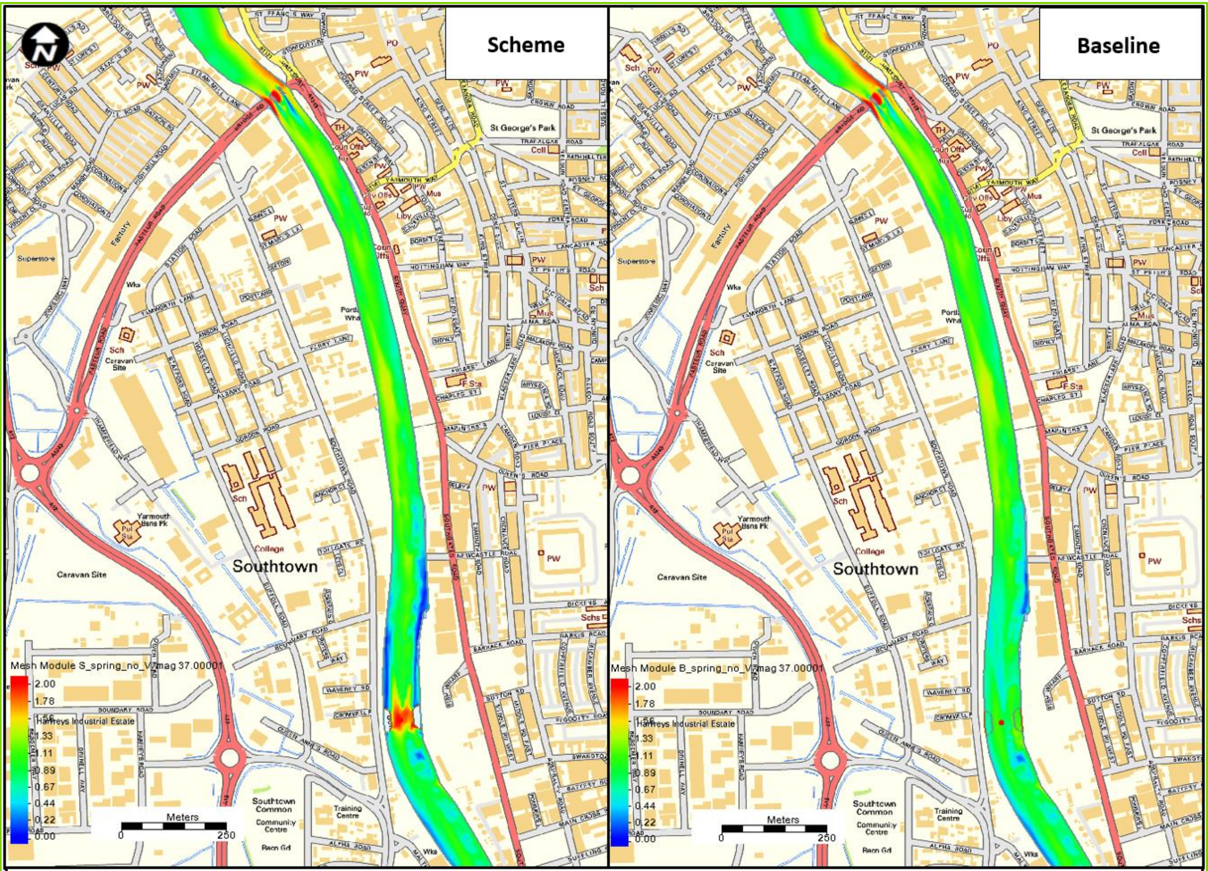


**Plate 6.7– Spring Velocity Magnitude, Scheme and Baseline at 37hr**



*Plate 6-7: Spring Velocity Magnitude*

6.2.8 Plate 6-8, Plate 6-9, Plate 6-10 and Plate 6-11 show the water level at the Principal Application Site, Breydon Water, Harbour Entrance and Haven Bridge respectively. The plates show the Scheme has a negligible impact on the water level in the Spring tide event. Plate 6-12 shows the water level difference between the Scheme and Baseline at the four locations in the domain. There is a small difference in water levels at the Principal Application Site. This is a result of the increase in water velocity magnitude caused by the Scheme, which in turn slightly reduces the local water level. This can be seen on the flood tide where the blue line representing the Scheme is visible on Plate 6-8. The water level difference is less than 0.15m, considering the bed elevation at the Scheme is approximately -7mAOD giving a water depth of between 6m and 8.5m in the tidal cycle, this difference is negligible.

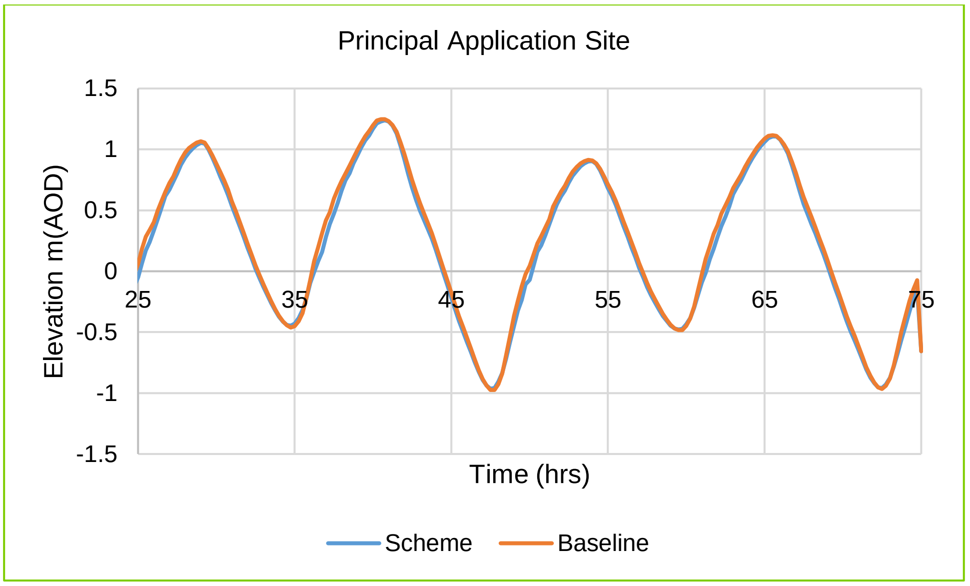


*Plate 6-8: Comparison of Water Level between the Baseline and Scheme Scenarios at the Principal Application site (between the bridge knuckles) for the Spring Tide*

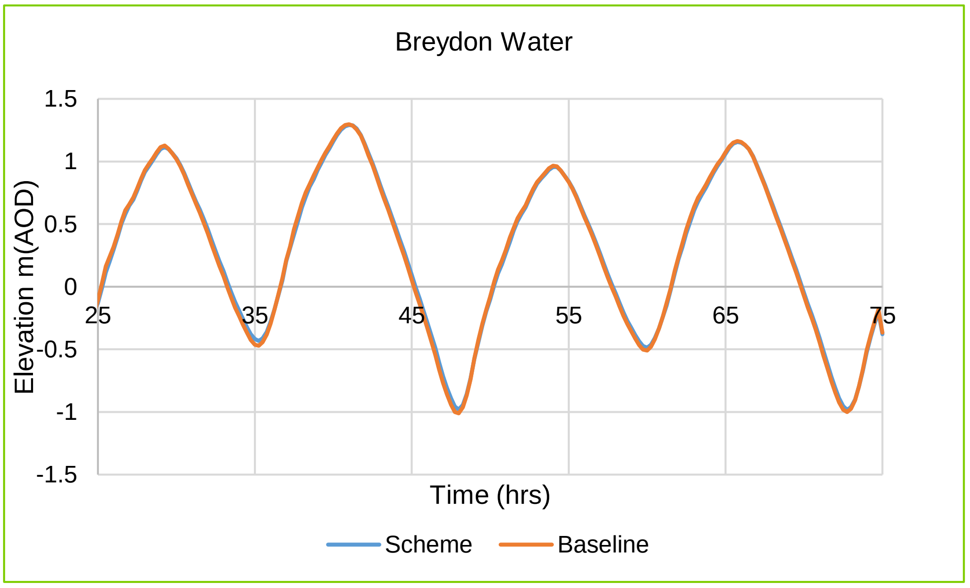


*Plate 6-9: Comparison of Water Level between the Baseline and Scheme Scenarios at Breydon Water for the Spring Tide*

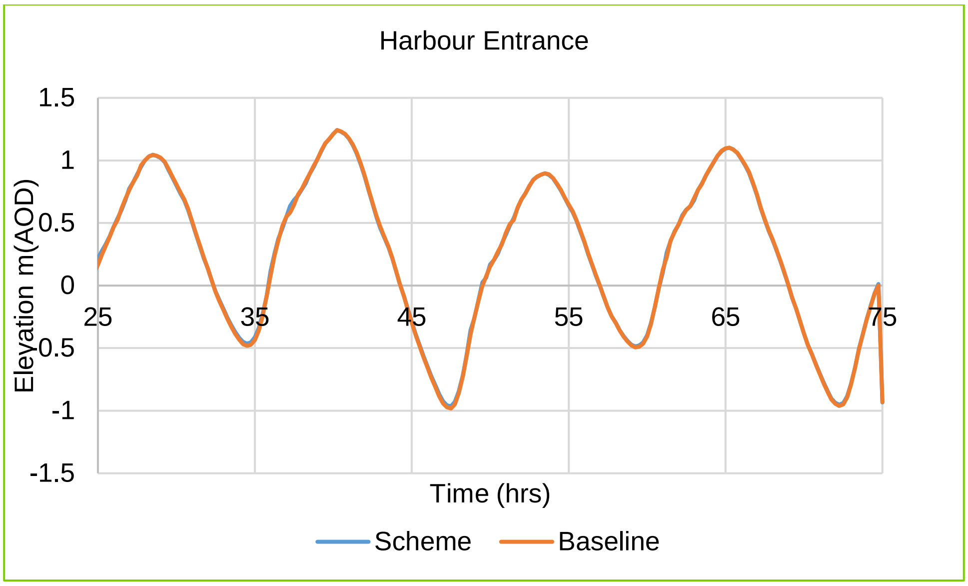


*Plate 6-10: Comparison of Water Level between the Baseline and Scheme Scenarios at the Harbour Entrance for the Spring Tide*

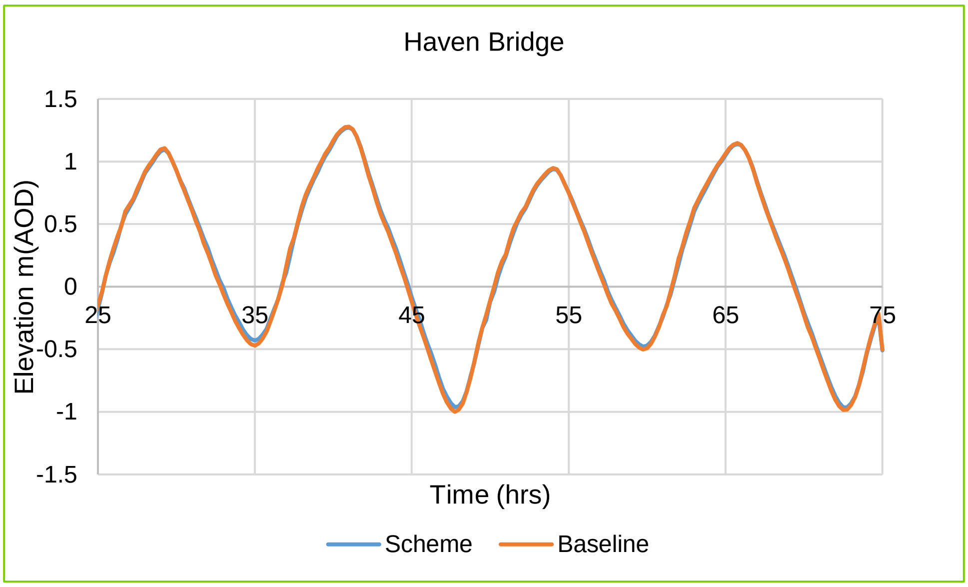


*Plate 6-11: Comparison of Water Level between the Baseline and Scheme Scenarios at Haven Bridge for the Spring Tide*

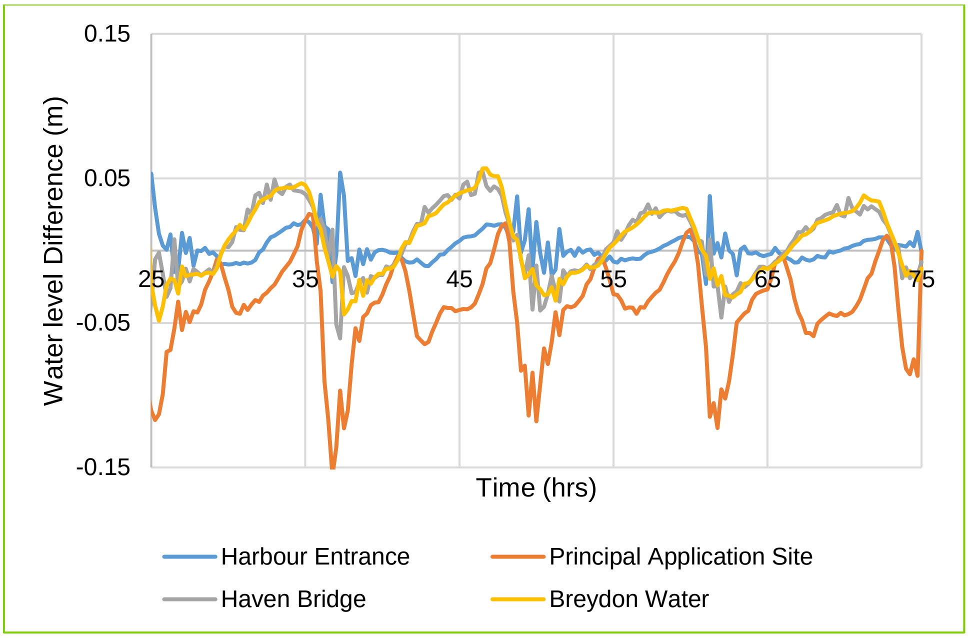


*Plate 6-12: Water Level Difference between Baseline and Scheme Scenarios (Scheme – Baseline) for Spring Tide*

6.2.9 The results show during the Spring tidal event, the Scheme has a negligible impact on the water level in the model domain. The main effect of the Scheme is to increase the local velocity magnitude by up to 1m/s at the Principal Application Site because of the constriction caused in the channel by the bridge knuckles. The differences in velocity magnitude in Breydon Water and at the Harbour Entrance are negligible.

<u>Neap Tidal Event</u>

6.2.10 Plate 6-13 shows the velocity magnitude between the bridge knuckles at the Principal Application Site for the Neap tidal profile. The Baseline velocity magnitude at the Principal Application Site location in the neap tide reaches a peak of approximately 0.7m/s during the simulation. The plot shows that due to the presence of the Scheme, the water velocity magnitude approximately doubles for the duration of the simulation. This is because the bridge knuckles cause a constriction the channel and in order for a similar volume of water to transit the channel, the velocity must increase. Plate 6-14, Plate 6-15 and Plate 6-16 show the velocity magnitude change is small elsewhere in the domain during the neap tide. Plate 6-15 shows a large difference in velocity magnitude at the harbour entrance during the model warm up time. This is considered a localised model error and likely due to the inflow boundary and initial conditions and therefore is not attributed to the Scheme. The difference is not seen in any of the other model runs and is not consistent with later tidal cycles in the simulation.

6.2.11 Plate 6-17 shows the difference (Scheme – Baseline) in velocity magnitude for the four locations in the channel. The plot shows the largest difference in velocity magnitude is at the Principal Application Site. With the exception of a peak near the harbour mouth at around 40 hours into the simulation, the differences in velocity magnitude elsewhere in the domain are less than 0.1m/s and considered negligible.

6.2.12 Plate 6-18 shows a 2D plot of the velocity magnitude for the Baseline and Scheme Neap simulation at 50 hours, which corresponds with the largest difference on Plate 6-17. The plate shows the localised increase in velocity magnitude due to the Scheme. The range of the impact on velocity is approximately 500m upstream and 500m downstream of the Principal Applications Site during the Neap tide simulation.

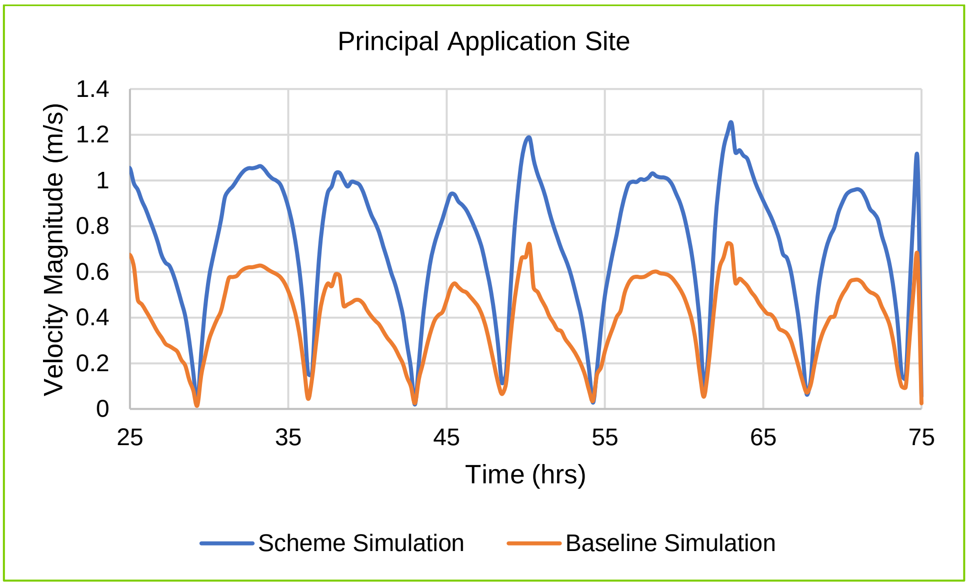


*Plate 6-13: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at the Principal Application Site for the Neap Tide*

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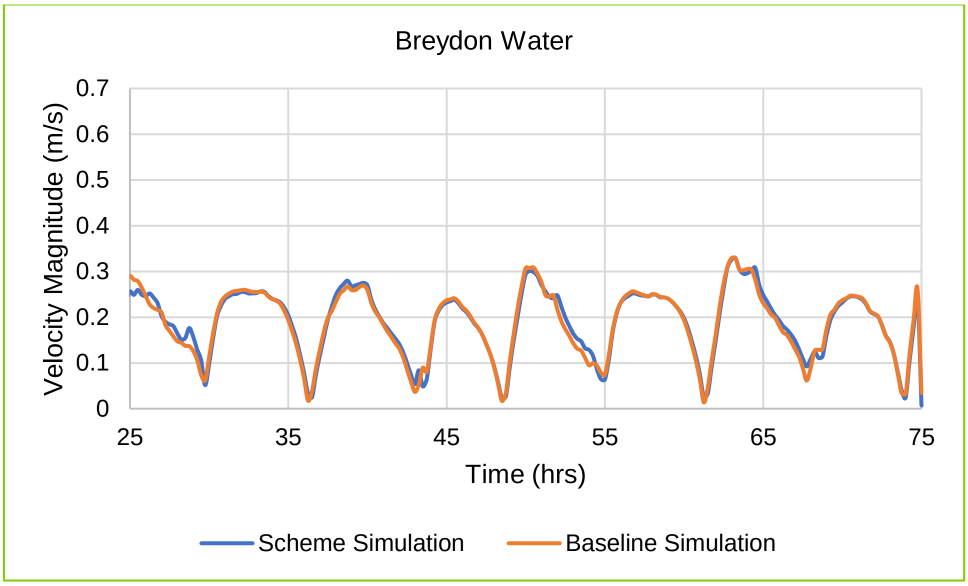


*Plate 6-14: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at Breydon Water for the Neap Tide*

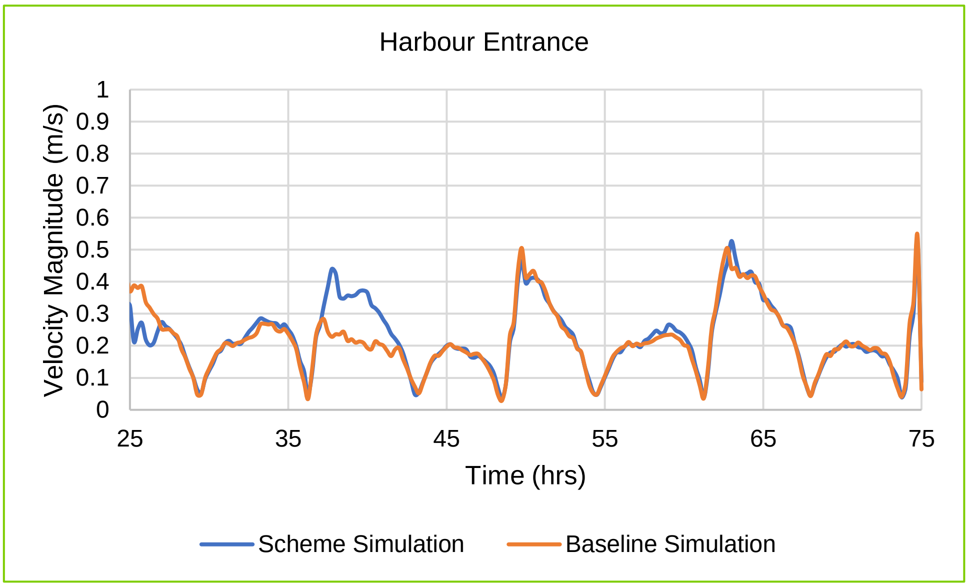


*Plate 6-15: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at the Harbour Entrance for the Neap Tide*

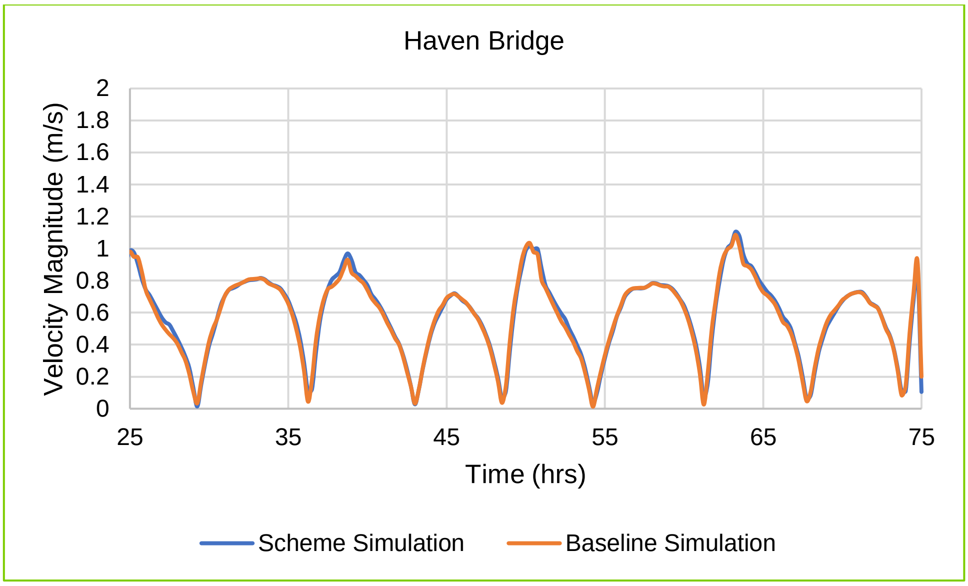


*Plate 6-16: Comparison of Velocity Magnitude between the Baseline and Scheme Scenarios at Haven Bridge for the Neap Tide*

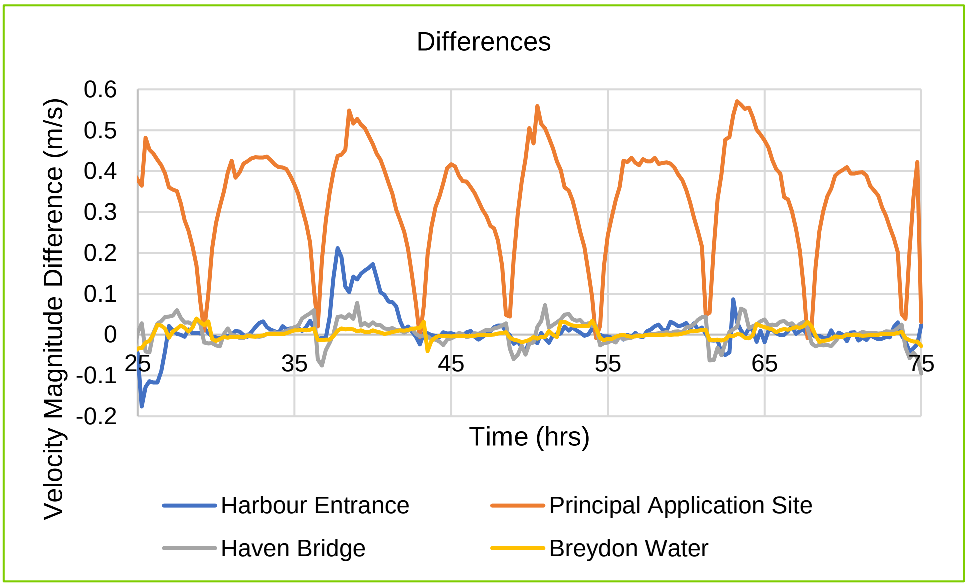


*Plate 6-17: Difference in Velocity Magnitude between the Baseline and Scheme Scenarios for the Neap Tide*

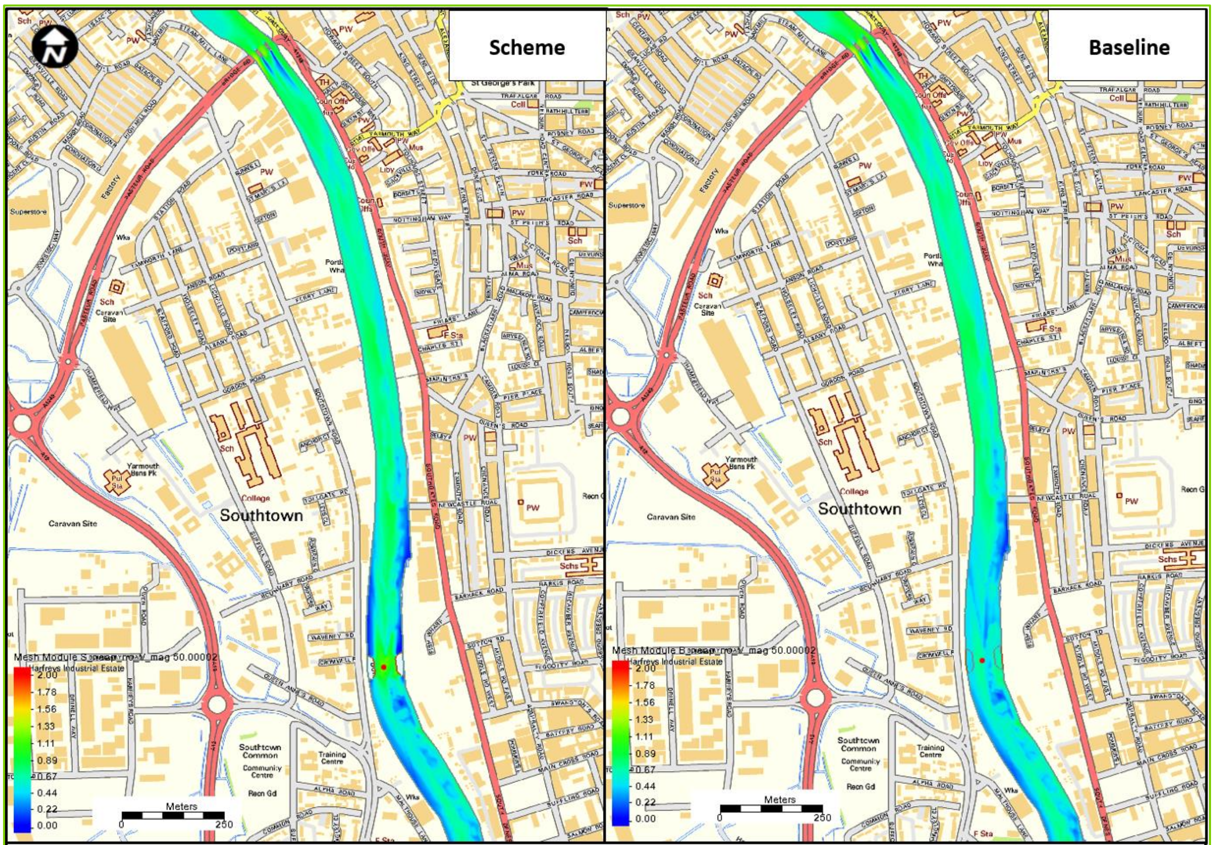


**Plate 6.18– Neap Velocity Magnitude, Scheme and Baseline at 50hr**



*Plate 6-18: Neap Velocity Magnitude*

6.2.13 Plate 6-19, Plate 6-20, Plate 6-21 and Plate 6-22 show the water level at the Principal Application Site, Breydon Water, Harbour Entrance and Haven Bridge respectively. The plates show the Scheme has a negligible impact on the water level at the Scheme in the Neap event. There is a negligible impact on water level elsewhere in the domain. Plate 6-23 shows the water level difference between the Scheme and Baseline at the four points in the domain. The water level difference is less than 0.1m at the Principal Application Site, considering the bed elevation at the Principal Application Site is approximately -7mAOD giving a water depth of between 6m and 8m in the Neap cycle, this difference is considered negligible.

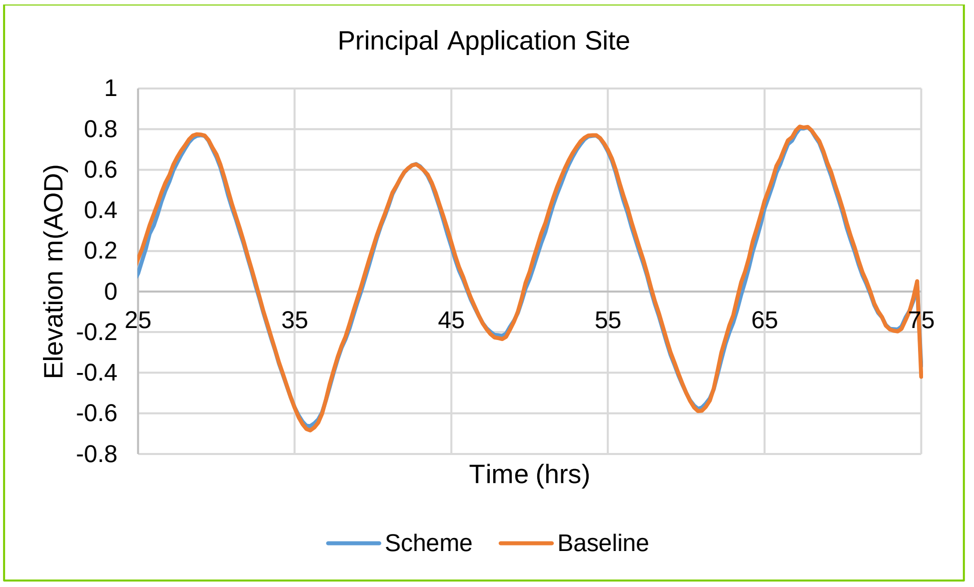


*Plate 6-19: Comparison of Water Level between the Baseline and Scheme Scenarios at the Principal Application Site for the Neap Tide*

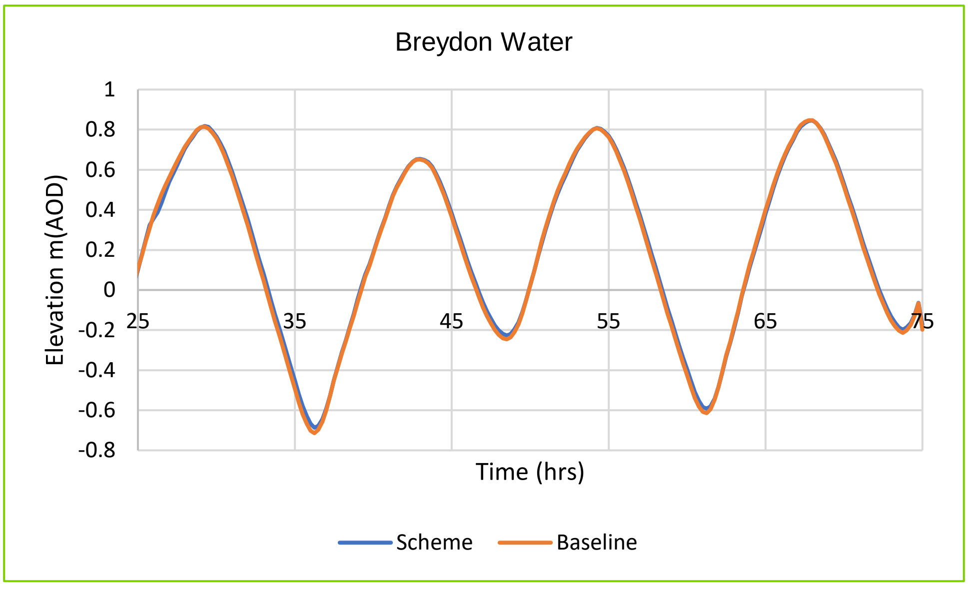


*Plate 6-20: Comparison of Water Level between the Baseline and Scheme Scenarios at Breydon Water for the Neap Tide*

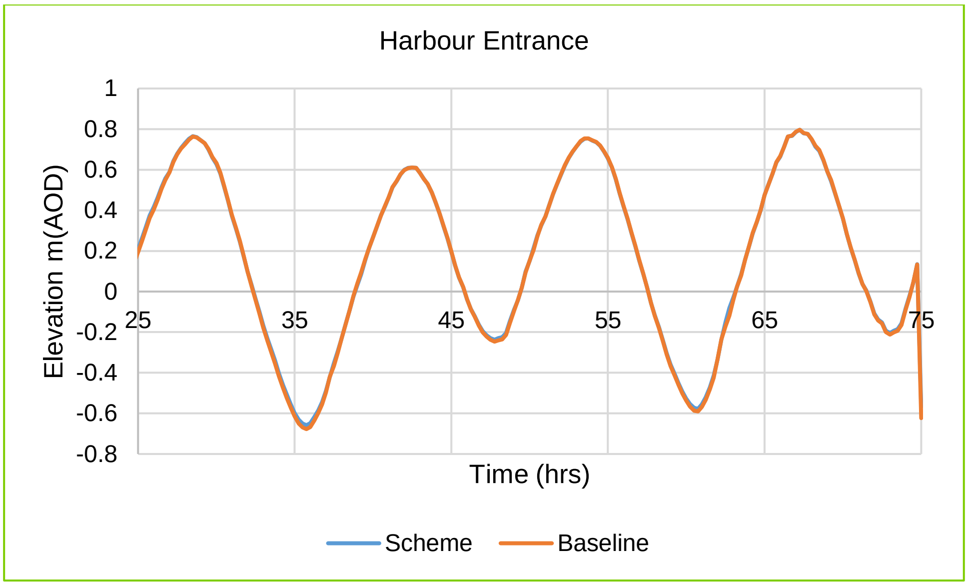


*Plate 6-21: Comparison of Water Level between the Baseline and Scheme Scenarios at the Harbour Entrance for the Neap Tide*

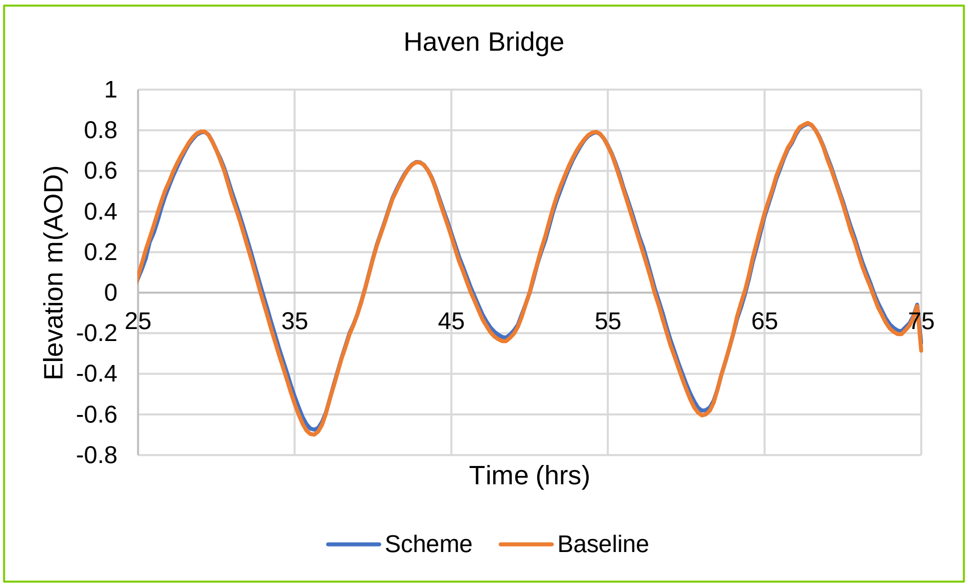


*Plate 6-22: Comparison of Water Level between the Baseline and Scheme Scenarios at Haven Bridge for the Neap Tide*

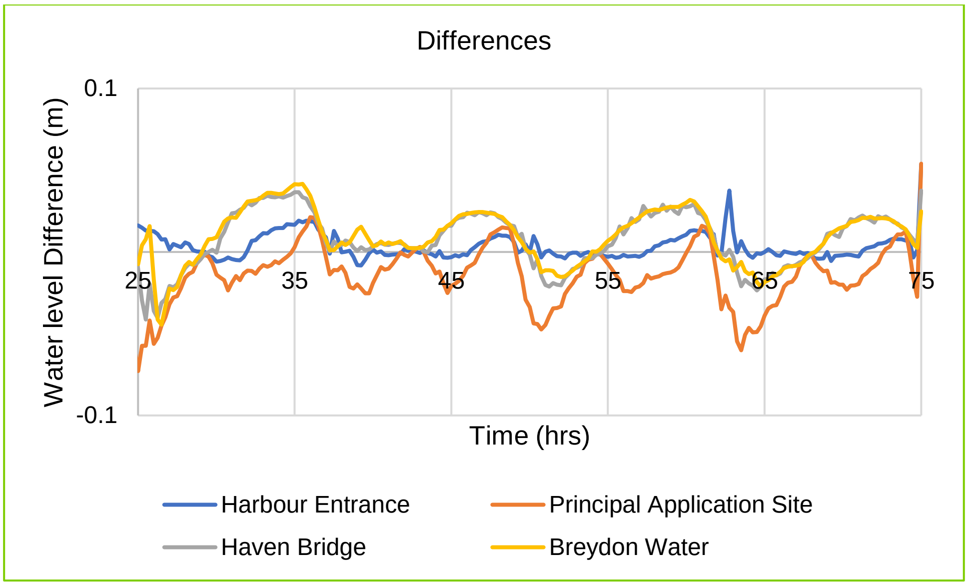


*Plate 6-23: Difference in Water Level between the Baseline and Scheme Scenarios for the Neap Tide*

6.2.14 The results show during the Neap tidal event, the Scheme has a negligible impact of the water level across the domain. The main effect of the Scheme is to increase the local velocity magnitude at the Principal Application Site because of the constriction caused in the channel by the bridge knuckles.

6.2.15 When comparing the Neap and Spring tide events, the Baseline velocity magnitude in the Neap event peaks at approximately 0.7m/s and the Baseline velocity magnitude over the Spring tide peaks at approximately 1m/s. The difference is driven by the increased water level and tidal amplitude in the Spring tide when compared to the Neap.

6.2.16 The velocity magnitude increase due to the Scheme is greater in the Spring tide event than the Neap event. This is because the velocity magnitude is dependent on the rate of change in water level, which is greater during the Spring tide than the Neap tide.

### Bed Stress

6.2.17 Bed stress is the parameter that drives erosion and deposition. Therefore, assessing the bed stress predicted by the model highlights areas where erosion and deposition occurs. The bed stress is calculated using the bottom velocity magnitude in the model. The bed stress results for the Spring and Neap tide are presented below. To put the bed stress into context, the critical erosion rate for the top layer of material (silt) in the channel is 0.12Pa, therefore where the stress exceeds this value sediment erosion will occur.

### Spring Tide Event

6.2.18 Table 6-2 shows the bed stress average and extremes for the Harbour Entrance, Principal Application Site, Haven Bridge and Breydon Water. The table shows that at all locations, the bed stress rates in the Baseline scenario are sufficient on average to erode material over the duration of the Spring tide event.

6.2.19 The results show that in the Spring tide event, on average across the simulation, the Scheme increases the localised bed stress, this is in line with the increased velocity magnitude. The results show that on average the bed stress is increased by 1.55Pa in the Scheme scenario compared to the Baseline at the Principal Application Site. When comparing the Scheme model to the Baseline model results at the Harbour Entrance, Haven Bridge and in Breydon Water, the Scheme has a negligible impact on average bed stress.

*Table 6-2: Bed Stress – Spring Tide*

| Tide | Harbour Mouth | Scheme | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Average Baseline (Pa) | 0.26 | 0.63 | 1.55 | 0.14 |
| Average Scheme (Pa) | 0.26 | 2.14 | 1.54 | 0.14 |
| Average Difference (Pa) | 0.00 | 1.55 | -0.01 | 0.00 |
| **Baseline** | | | | |
| Maximum Baseline (Pa) | 1.01 | 1.86 | 5.44 | 0.53 |
| Minimum Baseline (Pa) | 0.00 | 0.00 | 0.00 | 0.00 |
| **Scheme** | | | | |
| Maximum Scheme (Pa) | 1.01 | 6.73 | 5.86 | 0.52 |
| Minimum Scheme (Pa) | 0.01 | 0.01 | 0.01 | 0.00 |
| **Difference** | | | | |
| Maximum Difference (Pa) | 0.20 | 5.02 | 0.43 | 0.08 |
| Minimum Difference (Pa) | -0.32 | -0.01 | -0.33 | -0.05 |

6.2.20 Plate 6-24 and Plate 6-25 show the time series results for the bed stress at the four locations in the domain. This shows that the highest bed stress is seen on the flood tide as water is entering the estuary. A lower bed stress can be seen on the ebb tide. The impact of the Scheme approximately mirrors the impact of Breydon Bridge in the Spring tide. This result shows the Scheme will have a similar impact on the estuary as Breydon Bridge currently has during a Spring tide. Plate 6-26 shows the difference in bed stress through the timeseries. The plot shows the largest difference is at the Principal Application Site on the flood tide, as water is entering the estuary. The differences in bed stress elsewhere are negligible.

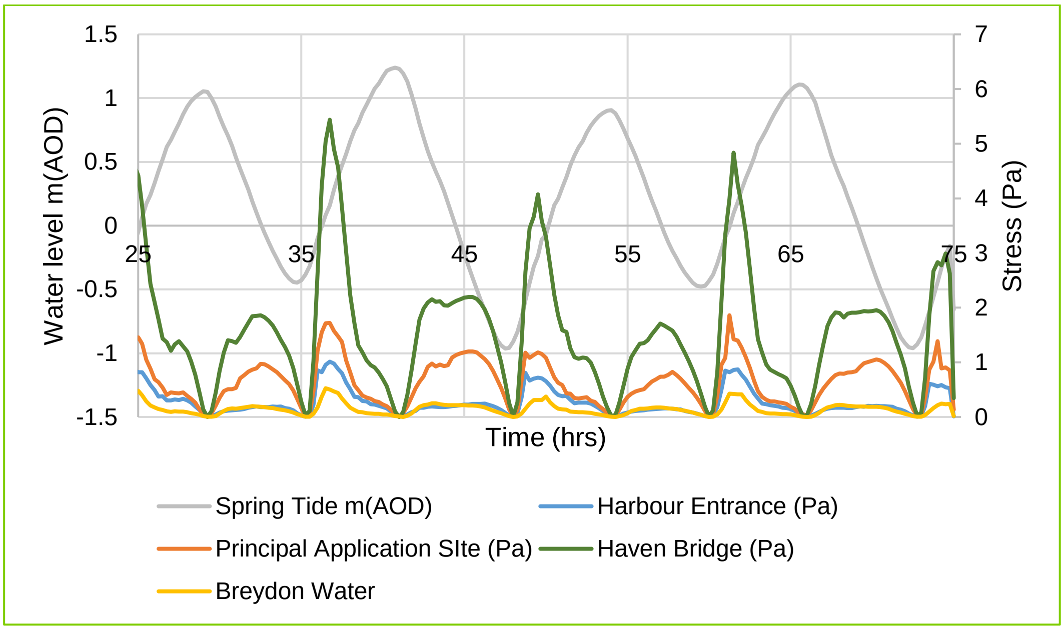


*Plate 6-24: Model Predicted Baseline Bed Stress – Spring Tide*

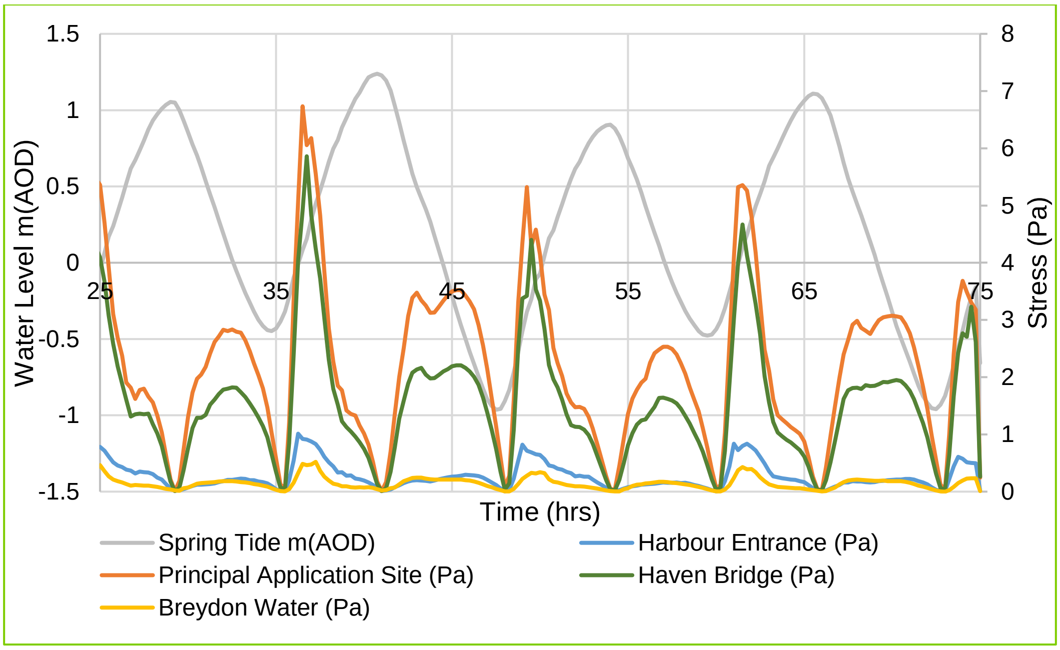


*Plate 6-25: Model Predicted Scheme Bed Stress – Spring Tide*

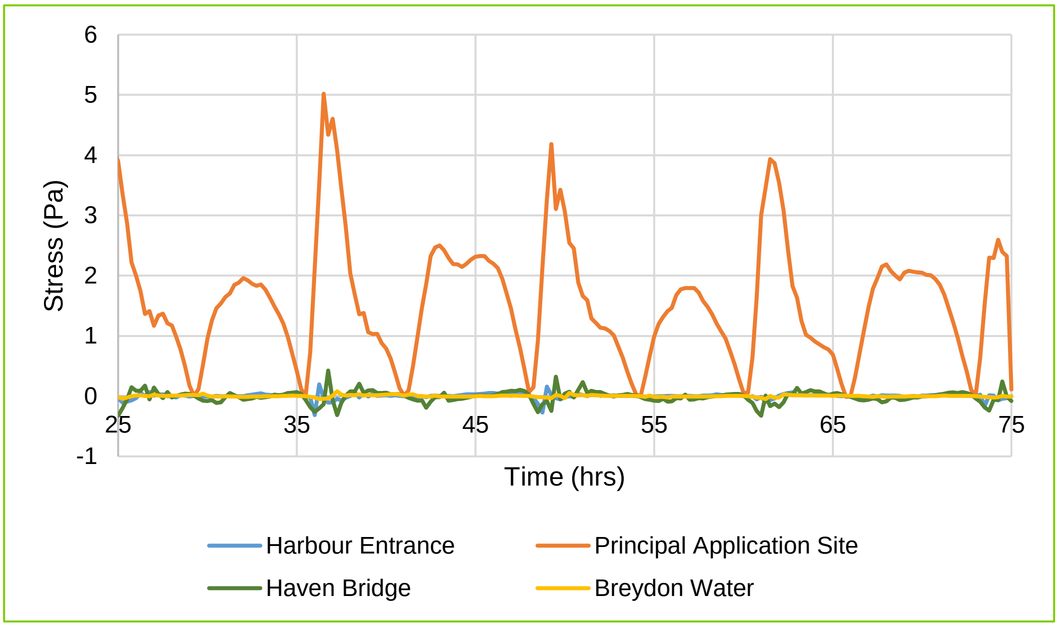


*Plate 6-26: Bed Stress Difference (Scheme – Baseline) – Spring Tide*

### Neap Tide Event

6.2.21 Table 6-3 shows the bed stress average and extremes for the Harbour Entrance, the Principal Application Site, Haven Bridge and Breydon Water during the Neap tidal event. The table shows at all locations during the Baseline Scenario, the bed stress rates in the model domain are sufficient on average to erode material over the duration of the Neap event.

6.2.22 Table 6-3 shows that in the Neap tide event, on average across the model run, the Scheme increases the bed stresses due to the increase velocity magnitude. The main difference between the Baseline and Scheme scenarios is at the Principal Application Site where the bed shear is significantly increased due to the increase in velocity magnitude. The results show that on average the bed stress is increased by 0.74Pa at the Principal Application Site. When comparing the Scheme model to the Baseline model results at the Harbour Entrance, Haven Bridge and in Breydon Water there is a negligible impact on bed stress.

*Table 6-3: Bed Stress - Neap Tide*

| | Harbour Mouth | Schem e | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Average Baseline (Pa) | 0.12 | 0.32 | 0.73 | 0.07 |
| Average Scheme (Pa) | 0.13 | 1.06 | 0.74 | 0.07 |
| Average Difference (Pa) | 0.01 | 0.74 | 0.01 | 0.00 |
| **Baseline** | | | | |
| Maximum Baseline (Pa) | 0.48 | 0.87 | 2.01 | 0.16 |
| Minimum Baseline (Pa) | 0.00 | 0.00 | 0.00 | 0.00 |
| **Scheme** | | | | |
| Maximum Scheme (Pa) | 0.51 | 2.65 | 2.29 | 0.18 |
| Minimum Scheme (Pa) | 0.00 | 0.00 | 0.00 | 0.00 |
| **Difference** | | | | |
| Maximum Difference (Pa) | 0.22 | 1.79 | 0.26 | 0.03 |
| Minimum Difference (Pa) | -0.17 | 0.00 | -0.18 | -0.03 |

6.2.23 Plate 6-27 and Plate 6-28 show the time series results for bed stress at the Harbour Entrance, the Principal Application Site, Haven Bridge and Breydon Water. The plots show that the highest bed stress is seen on the flood tide as the water is entering the estuary. A lower increase in bed stress can be seen on the ebb tide. This result shows the Scheme will have a similar impact on the estuary as Breydon Bridge currently has during a Neap tide. Plate 6-29 shows the difference in bed stress between the Baseline and Scheme scenarios through the Neap tide simulation. This plot shows the largest difference is at the Principal Application Site on the flood tide, as water is entering the estuary.

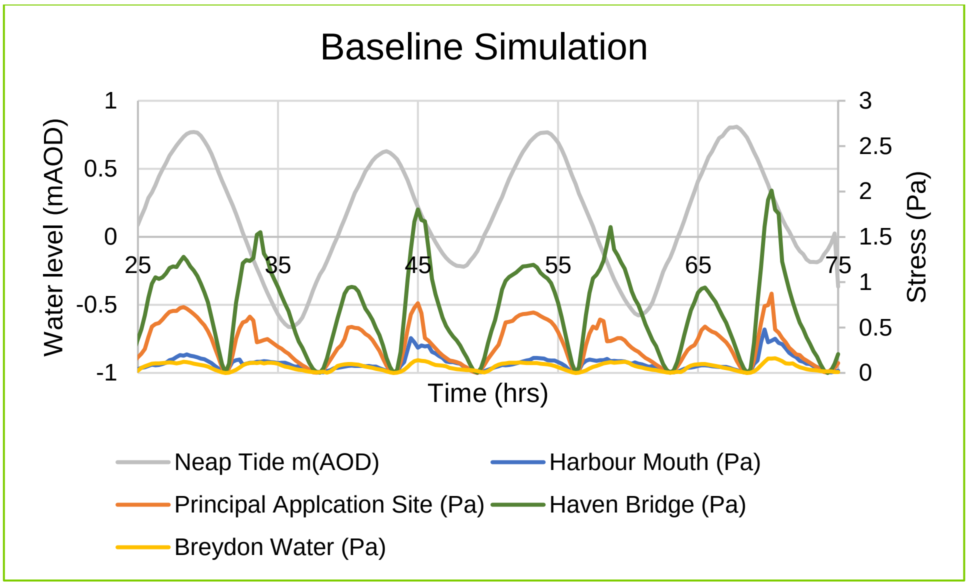


*Plate 6-27: Model Predicted Baseline Bed Stress – Neap Tide*

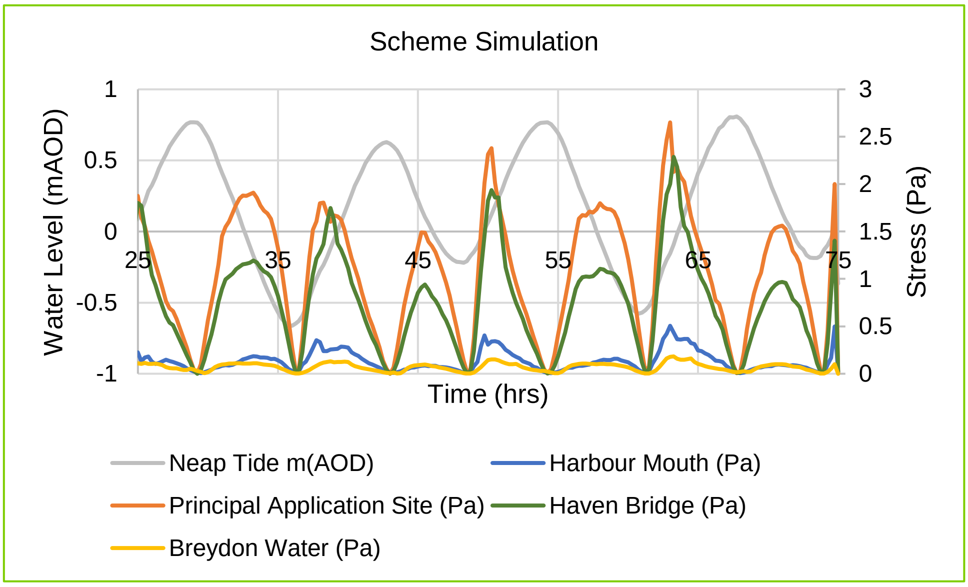


*Plate 6-28: Model Predicted Scheme Bed Stress – Neap Tide*

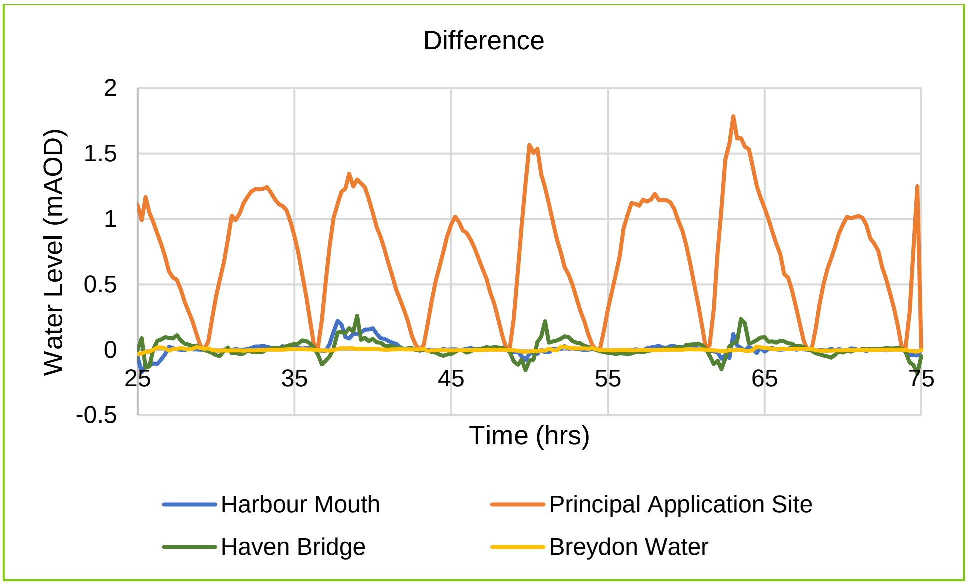


*Plate 6-29: Bed Stress Difference (Scheme – Baseline) – Neap Tide*

6.2.24 When comparing the impacts on bed stress in the Spring and Neap simulations, the Scheme causes a larger increase in bed stress during the Spring tide. This is because the velocity magnitude is greater during this simulation. In both simulations, the impact of the Scheme is mainly seen locally at the Principal Application Site with a negligible difference predicted elsewhere.

6.2.25 On average, throughout the year, the change in bed stress is likely to fall between the Spring and Neap values because of the approximately two weekly spring/neap cycle experience in the estuary.

### Erosion Rate

6.2.26 In order to make an assessment of sediment transport, a calculation of the average erosion/deposition rate has been carried out. The purpose of this value is to give an understanding of the worst case impacts on the sediment regime, given that no bed morphology can be included in the calculation. The erosion/deposition rate takes into account both the scour and deposition occurring through the model simulations.

6.2.27 The average erosion rate has been calculated by taking the difference in bed material (kg/m$^2$) at the start and finish of the model simulation and dividing by the total simulation time. The calculation provides a number which can be extrapolated to give an estimation over a required period of time. It should be noted that long term changes in bed level will affect the velocity magnitude due to the continuity equation (Q=VA, where Q is flow rate, V is velocity magnitude and A is cross sectional area). For example, if sediment built up in a location, assuming flow remains the same, local velocity would increase because the cross-sectional area would decrease until it was sufficient to trigger erosion. Explicit modelling of these bed elevation changes cannot be undertaken given resolution of the model in this assessment as the model run times would be too long.

### Spring Tide Event

6.2.28 Table 6-4 shows the calculated erosion rates for the Spring tide event. A positive rate shows scour and a negative rate shows deposition. In both the Scheme and the Baseline models, the Harbour entrance, the Principal Application Site, Haven Bridge and Breydon Water are found to be experiencing scour on average throughout the model run.

6.2.29 Table 6-4 shows that the Scheme reduces the scour rates in Breydon Water and at the Harbour Entrance compared to the Baseline scenario. As a result, the existing sediment does not erode as fast as predicted for the Baseline scenario and less material is moved around the model domain.

6.2.30 There is additional scour at the Principal Application Site in the Scheme scenario compared to the Baseline, this is due to the increase in velocity magnitude which drives higher bed stresses causing localised scour pits. The results also show that the rate of scour is slightly increased near Haven Bridge in the Scheme scenario compared to the Baseline, this is due to the small increase in velocity due to the Scheme at Haven Bridge.

*Table 6-4: Spring Erosion Rate*

| **Baseline** | **Harbour Entrance** | **Principal Application Site** | **Haven Bridge** | **Breydon Water** |
|---|---|---|---|---|
| Average Erosion rate (kg/m$^2$/hr) | 0.98 | 1.87 | 3.75 | 0.77 |
| Average Erosion depth rate (m/hr) | 0.00037 | 0.00071 | 0.00141 | 0.00029 |
| **Scheme** | | | | |
| Average Erosion rate (kg/m$^2$/hr) | 0.74 | 7.35 | 5.75 | 0.28 |
| Average Erosion depth rate (m/hr) | 0.000277 | 0.002775 | 0.002170 | 0.000105 |
| **Differences** | | | | |
| Average Erosion rate (kg/m$^2$/hr) | -0.24 | 5.48 | 2.00 | -0.49 |
| Average Erosion depth rate (m/hr) | -0.00009 | 0.00207 | 0.00076 | -0.00018 |

6.2.31 Plate 6-30 shows the 2D plot of the average erosion rate comparison (kg/$m^2$/hr) in the model domain. There are two areas which show higher erosion rates with the Scheme in place compared to the Baseline; the Principal Application Site and Haven Bridge. These are areas where the increase in velocity locally impacts the sediment regime. However, the plate shows there is little change elsewhere in the domain. The Scheme locally scours the material in the channel between the bridges and most of the material is deposited close to the Principal Application Site near to the quay walls, with a small amount deposited elsewhere upstream and downstream of the Principal Application Site in the engineered channel. The modelling shows there is a negligible decrease in the erosion rate when comparing the Scheme scenario to the Baseline scenario in Breydon Water, this has the effect of slowing down the ambient erosion occurring naturally in the lake.

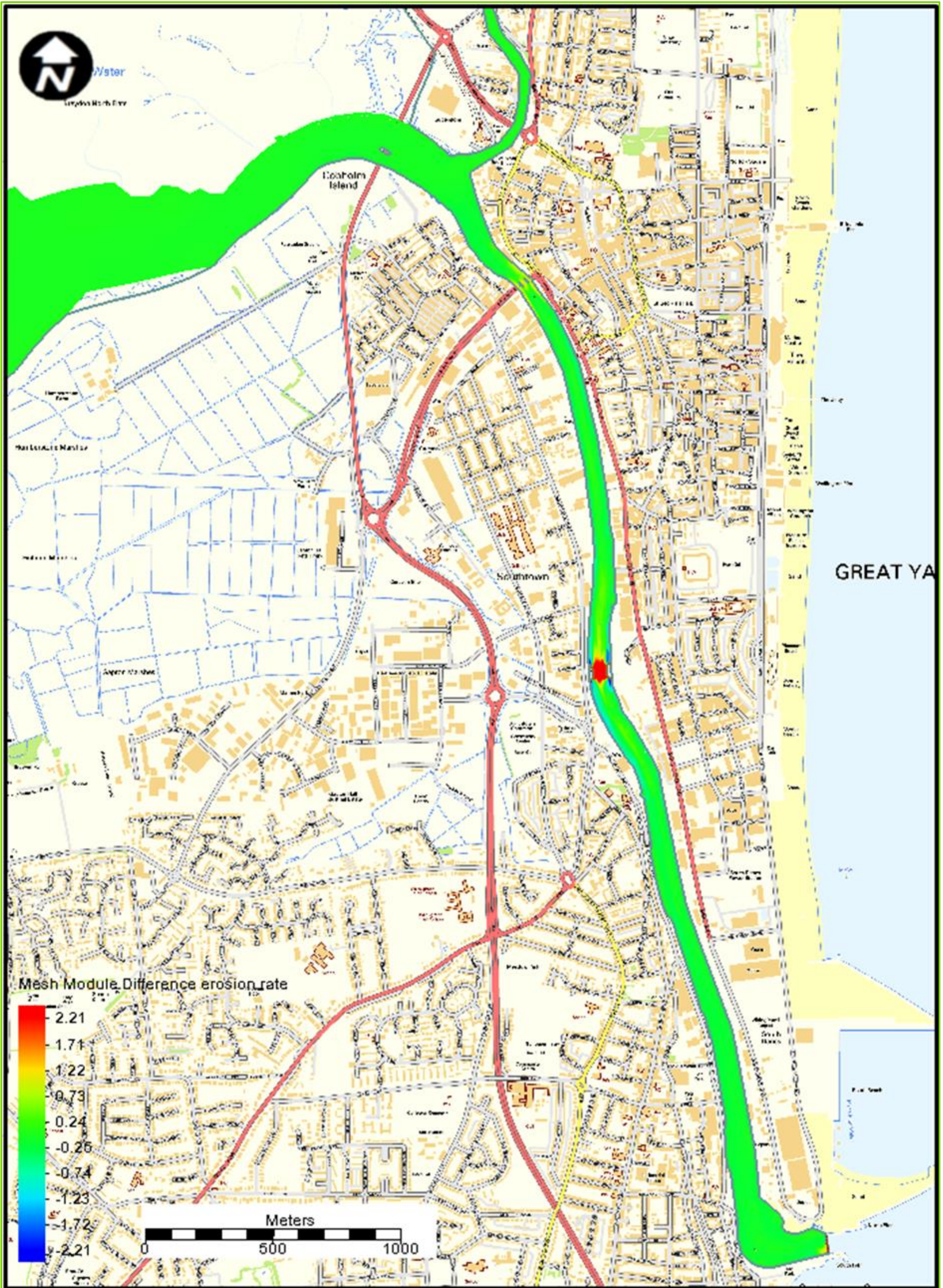


**Plate 6.30 – Spring Average Erosion Rate Comparison**



*Plate 6-30: Spring Average Erosion Rate Comparison*

6.2.32 It should be noted that the rates presented here are rates calculated over a relatively short period of time and do not consider morphological changes therefore they should be considered worst case. In reality, bed levels would likely find an equilibrium before the scour is increased significantly as a result of the Scheme. Assuming the flow in the estuary remains the same, velocity magnitude will increase as a result of the continuity equation. The velocity magnitude on the Spring tide increases by up to 10%, therefore when sufficient scour has occurred to increase the cross-sectional area by 10%, the scour rates will likely return to pre-Scheme conditions. In this assessment, the channel is approximately 100m wide, Breydon Bridge is assumed to be 50% blockage and a water depth of 8m is assumed then the expected maximum scour depth would be 0.8m.

### Neap Tide Event

6.2.33 Table 6-5 shows the calculated erosion rates for the Neap tide event. A positive rate shows scour and a negative rate shows deposition. In both the Scheme and the Baseline scenarios, scour is shown at the four locations in Table 6-5 as the velocity magnitudes are sufficient to erode the bed material.

6.2.34 When comparing the Baseline scenario to the Scheme scenario, the Scheme has a negligible impact on the scour rates in Breydon Water and at the Harbour entrance during the Neap event.

6.2.35 There is additional scour between the bridge knuckles at the Principal Application Site due to the increase in velocity magnitude, which drives higher bed stresses causing localised scour. The results also show that the rate of scour is slightly increased near Haven Bridge due to the change in velocity magnitude as a result of the Scheme.

*Table 6-5: Neap Erosion Rate*

| **Baseline** | **Harbour Mouth** | **Scheme** | **Haven Bridge** | **Breydon Water** |
|---|---|---|---|---|
| Average Instantaneous Erosion rate (kg/m$^2$/hr) | 0.10 | 1.13 | 2.75 | 0.01 |
| Average Instantaneous Erosion depth rate (m/hr) | 0.00004 | 0.00043 | 0.00104 | 0.00000 |
| **Scheme** | | | | |
| Average Instantaneous Erosion rate (kg/m$^2$/hr) | 0.15 | 4.55 | 3.20 | 0.01 |
| Average Instantaneous Erosion depth rate (m/hr) | 0.000055 | 0.001716 | 0.001209 | 0.000003 |
| **Differences** | | | | |

| Baseline | Harbour Mouth | Scheme | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Average Instantaneous Erosion rate (kg/m$^2$/hr) | 0.04 | 3.42 | 0.45 | 0.00 |
| Average Instantaneous Erosion depth rate (m/hr) | 0.00002 | 0.00129 | 0.00017 | 0.00000 |

6.2.36 Plate 6-31 shows the 2D plot of the average erosion rate comparison (kg/m$^2$/hr) in the domain. There is one area which show higher erosion rates with the Scheme in place; the Principal Application Site. This is where an increase in the velocity magnitude locally as a result of the Scheme impacts the sediment regime. However, the plate shows there is a negligible change elsewhere in the domain. The Scheme locally scours the material in the channel between the bridge knuckles and most of the material is deposited close to the Principal Application Site close to the quay walls, with a small amount deposited in the elsewhere upstream and downstream of the Principal Application Site in the engineered channel. The modelling shows there is a negligible decrease in the erosion rate when comparing the Scheme scenario to the Baseline scenario in Breydon Water, this has the effect of slowing down the ambient erosion occurring naturally in the Lake.

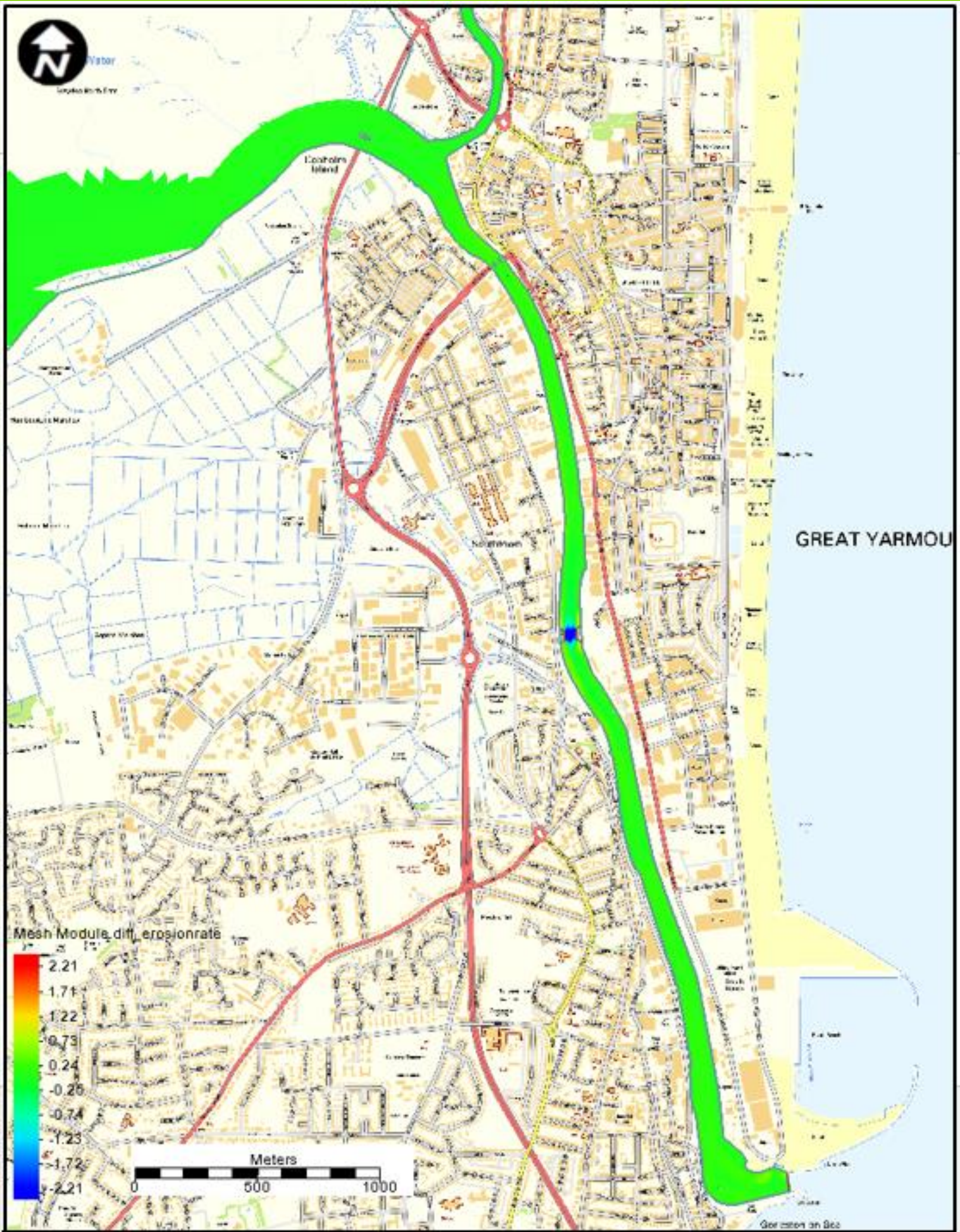


Plate 6.31– Neap Average Erosion Rate Comparison



*Plate 6-31: Neap Average Erosion Rate Comparison*

6.2.37 In general, the modelling shows that there is change to erosion rate in both the Neap and Spring tides at the Principal Application Site as a result of the

Scheme and the increased velocity magnitude between the bridge knuckles, which causes scour. The model shows that the scoured material is deposited predominately locally near to the quay walls where the velocity magnitude is lower either side of each bridge knuckle. In the Spring tide simulation, there is a small increase in erosion rates at Haven Bridge. In both the Neap and Spring tide simulations the erosion rates in Breydon Water and at the harbour entrance are negligible.

### Depth, Shape and Volume of Scour at the Scheme in a Typical Event

6.2.38 The model cannot be run for a long enough time to gain a full equilibrium in 3D to ascertain the full depth of the scour at the Principal Application Site. However, it is possible to estimate the depth of the scour using the continuity equation.

6.2.39 The model has shown that there is likely to be increased scour in the middle of the River Yare channel between the knuckles as a result of the Scheme. At the Principal Application Site, the width of the channel is 100m and the Scheme constricts the channel by approximately 50%, therefore the depth of the scour depth is likely to be limited to approximately double the average water depth. The average water level from the Gorleston-on-Sea gauge 2018 dataset shown in Plate 3-2 is 0.17mAOD. The bed level at the Principal Application Site is approximately -7mAOD, giving an average existing water depth of approximately 7m. This would mean in order to return to pre-Scheme conditions, the worst case depth of the scour pit would be approximately 7m below existing bed level between the Scheme knuckles. This depth should be considered a worst case scenario and a detailed assessment of scour should be carried out on the final design. Consideration is required to ensure the foundations are not compromised and scour protection will be required as part of the final design to reduce the depth of the scour pit at the Scheme.

6.2.40 Plate 6-7 shows the velocity magnitude between the Scheme and the Baseline scenarios in the Spring event at simulation time 37 hours. The figure shows the Scheme has localised impacts at the Principal Applications Site and negligible impacts at Haven Bridge. The impacts of the Scheme are reduced further away from the Principal Application Site. Plate 6-32 shows the average velocity magnitude difference, the figure shows on average, the velocity changes are localised to close to the Scheme. This plate shows the extent of the likely erosion in due to the Scheme.

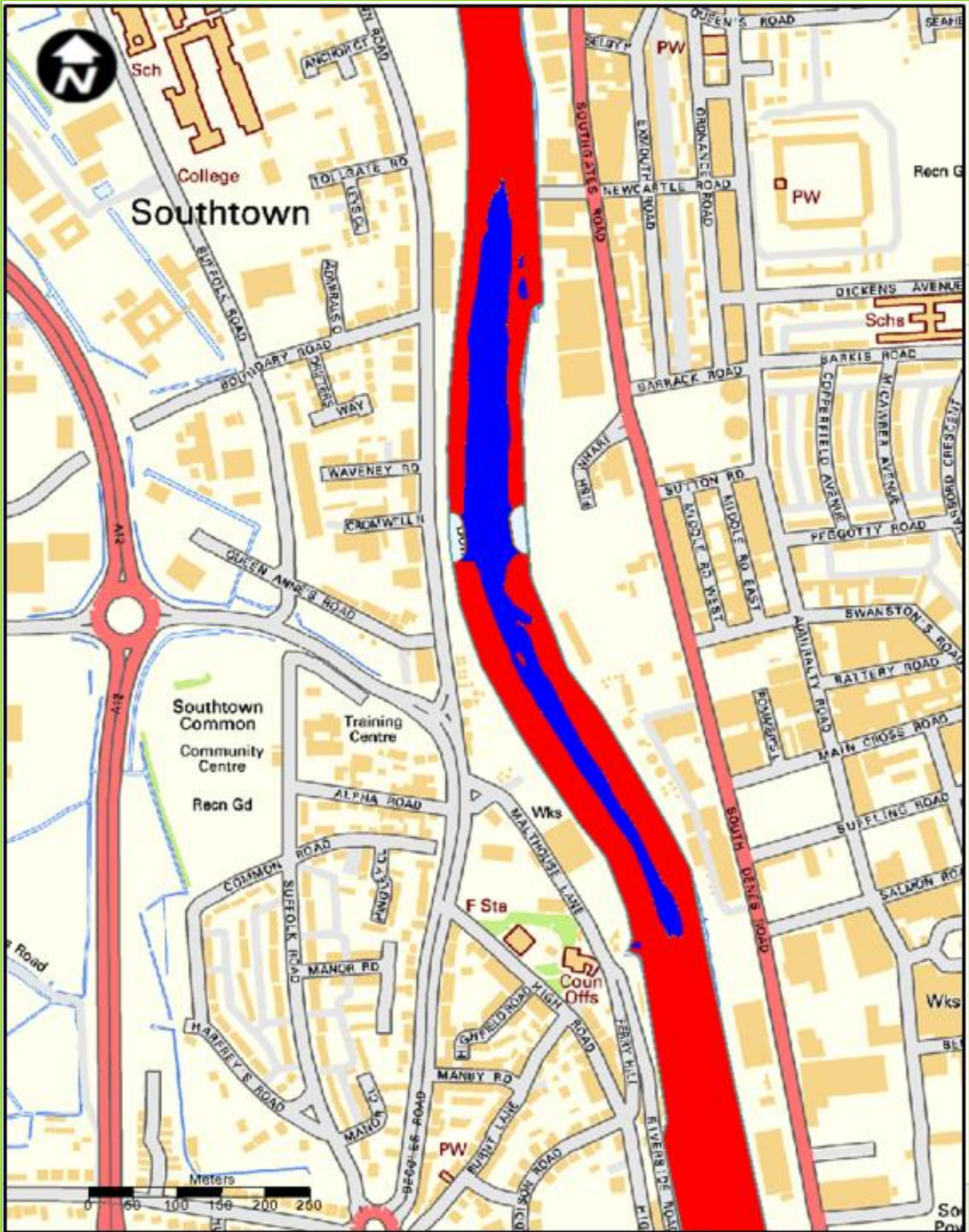


**Plate 6.32–Extent of Average Velocity Magnitude Change**



*Plate 6-32: Extent of Average Velocity Magnitude Change*

6.2.41 Plate 6-33 shows the localised scour pattern with the Scheme in place. Assessing the volume of scoured material is difficult as it depends on many variables, however it is possible to provide a rough estimate assuming the worst case scour depth of 7mAOD. The area between the bridge knuckles is approximately 50m x 50m, which assuming a maximum scour depth of 7m would mean an estimated scour volume of 17,500m3. This value should be considered worst case as it does not take into account any engineered scour protection at the bridge and is a conservative estimation.

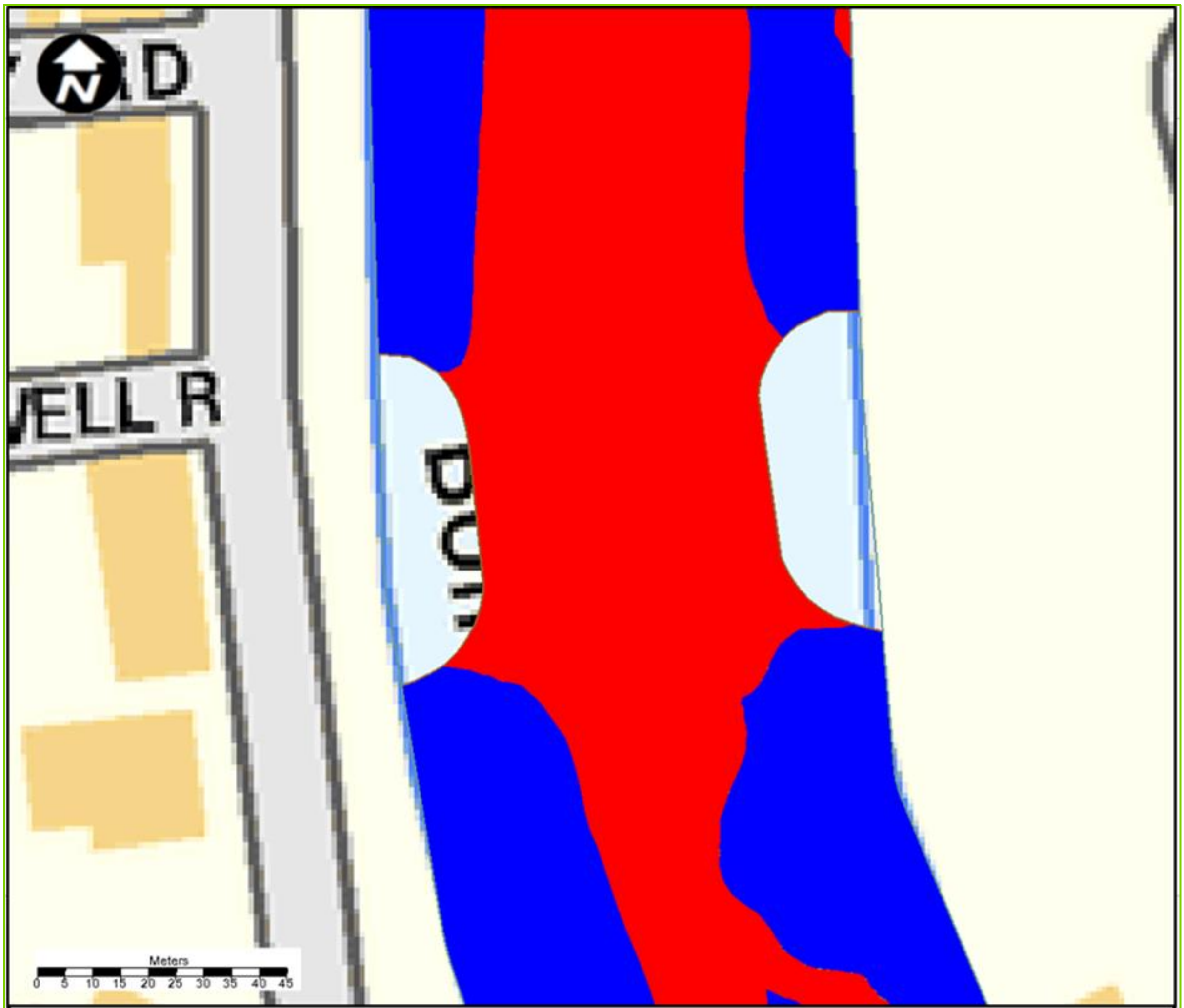


**Plate 6.33 –Typical Scour Pattern**



*Plate 6-33: Typical Scour Pattern*

6.2.42 Plate 6-34 shows the likely areas of deposition and erosion, where red is erosion, blue is deposition, green shows a negligible change. The figure clearly shows the main impacts are localised near the Scheme where the eroded material typically moves towards the Quay walls. There is also increased deposition near the quay walls at Haven Bridge. This is likely to be from the small amount of additional scour at the Haven Bridge. There is a negligible elsewhere in the domain.


**Plate 6.34– Erosion/Deposition Areas**



*Plate 6-34: Erosion/Deposition Areas*

6.2.43 The modelling of the everyday Spring and Neap events has shown that the impacts of the Scheme on sediment transport are local, creating some areas of additional sediment deposition and erosion near the Principal Application Site. There is no net change in sediment volume in the engineered section of the River Yare channel, therefore the Scheme has no impact on the volume of dredged material but will change the areas that will need to be dredged slightly. The modelling has shown that there is a negligible impact on the sediment regime at Breydon Water.

## 6.3 Results - Extreme Events

6.3.1 Two likely extreme events have been considered in this assessment, these are;

- MHWS to MLWS + 5% AEP Sea Surge Event; and
- MHWN to MLWN + 5% AEP Sea Surge Event.

6.3.2 The bed stress, peak velocity magnitude and bed erosion rates have been calculated for each event. This section provides an understanding of the likely impacts from a single surge event in the estuary to provide a likely worst case scour rate for an extreme event.

### Peak Velocity

6.3.3 In order to assess the impact of the Scheme during an extreme tide, the peak velocity magnitudes at the Harbour Entrance, Principal Application Site, Haven Bridge and Breydon Water are presented in Table 6-6. The table shows that during the extreme Baseline simulation the peak velocities are greater than the velocities predicted by the model for the Spring and Neap events, which is to be expected. The MHWS-MLWS +5% event has a greater velocity magnitude than the MHWN-MLWN +5% event. This is because there is a larger difference between high and low tide during the Spring surge event, which causes higher velocity magnitudes.

6.3.4 The results show that the Scheme increases the velocity magnitude at the Principal Application Site in both the MHWS-MLWS +5% and MHWN-MLWN +5% events due to the constriction caused by the bridge knuckles. There is small decrease in velocity magnitude at Haven Bridge as a result of the Scheme, this is due to the slight delay in water arriving at the bridge on the flood tide. There is a negligible impact on velocity at the harbour entrance and Breydon Water in both events.

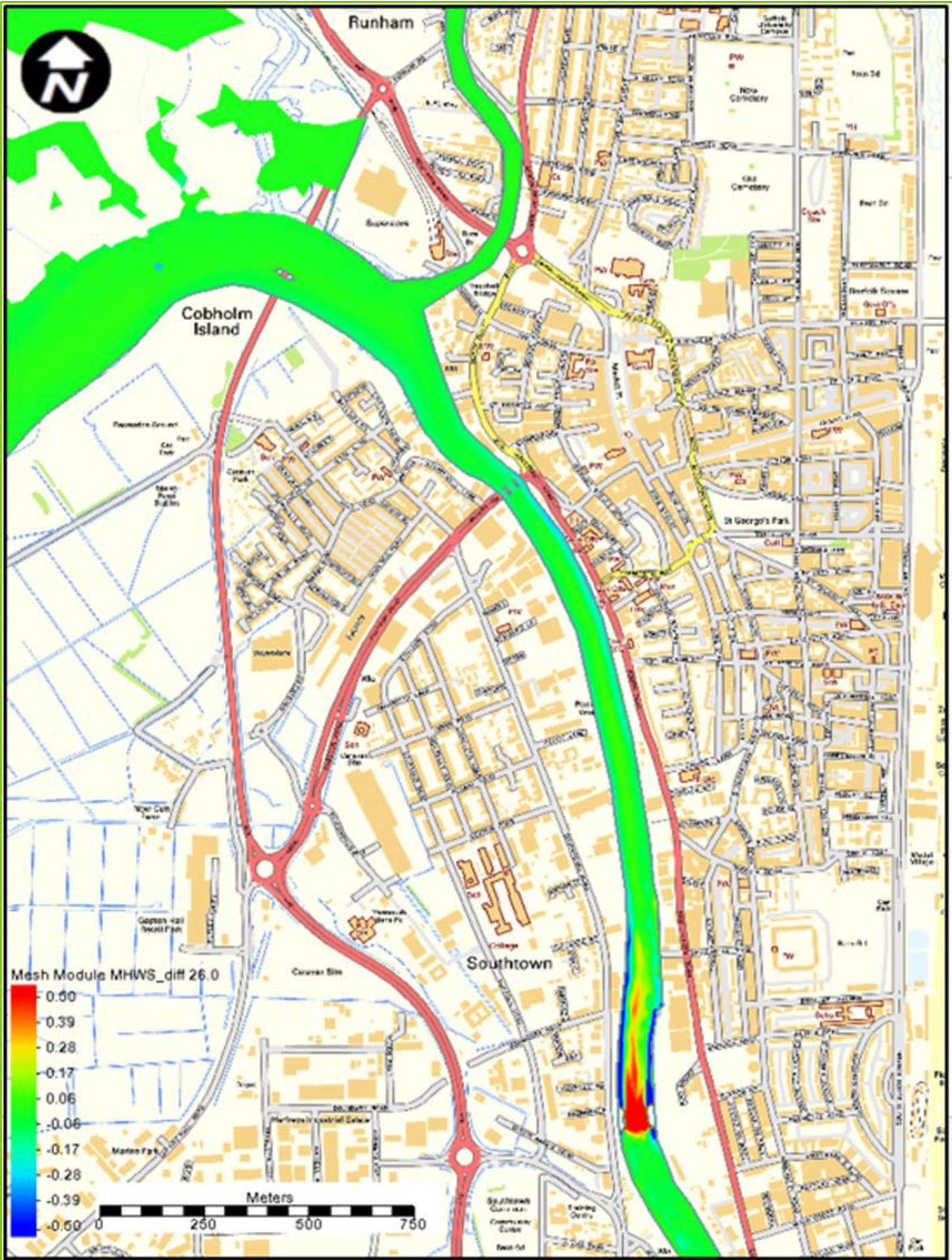


**Plate 6.35– Velocity Magnitude at 26hr, MHWS to MLWS +5% AEP**



6.3.5 Plate 6-35 and Plate 6-36 show the velocity magnitude difference between the Baseline and Scheme scenarios in the MHWS-MLWS +5% AEP and MHWN-MLWN +5% AEP respectively. These figures show that velocity magnitude increases at the Principal Application Site between the bridge knuckles and decreases immediately upstream and downstream of the knuckles. There in a negligible impact within the River Yare channel between the Principal Application Site and Haven Bridge showing a slight reduction along the quay walls in both events. There is a negligible change at Breydon Water and the harbour entrance due to the Scheme.

*Table 6-6: Extreme Tide, Peak Velocity*

| MHWS-MLWS + 5% | Harbour Entrance | Principal Application Site | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Baseline | 0.87 | 1.35 | 2.19 | 0.76 |
| Scheme | 0.85 | 2.52 | 2.09 | 0.75 |
| Difference | -0.02 | 1.17 | -0.10 | -0.01 |
| **MHWN-MLWN + 5%** | | | | |
| Baseline | 0.73 | 1.15 | 1.74 | 0.60 |
| Scheme | 0.71 | 2.08 | 1.72 | 0.59 |
| Difference | -0.03 | 0.93 | -0.02 | -0.01 |

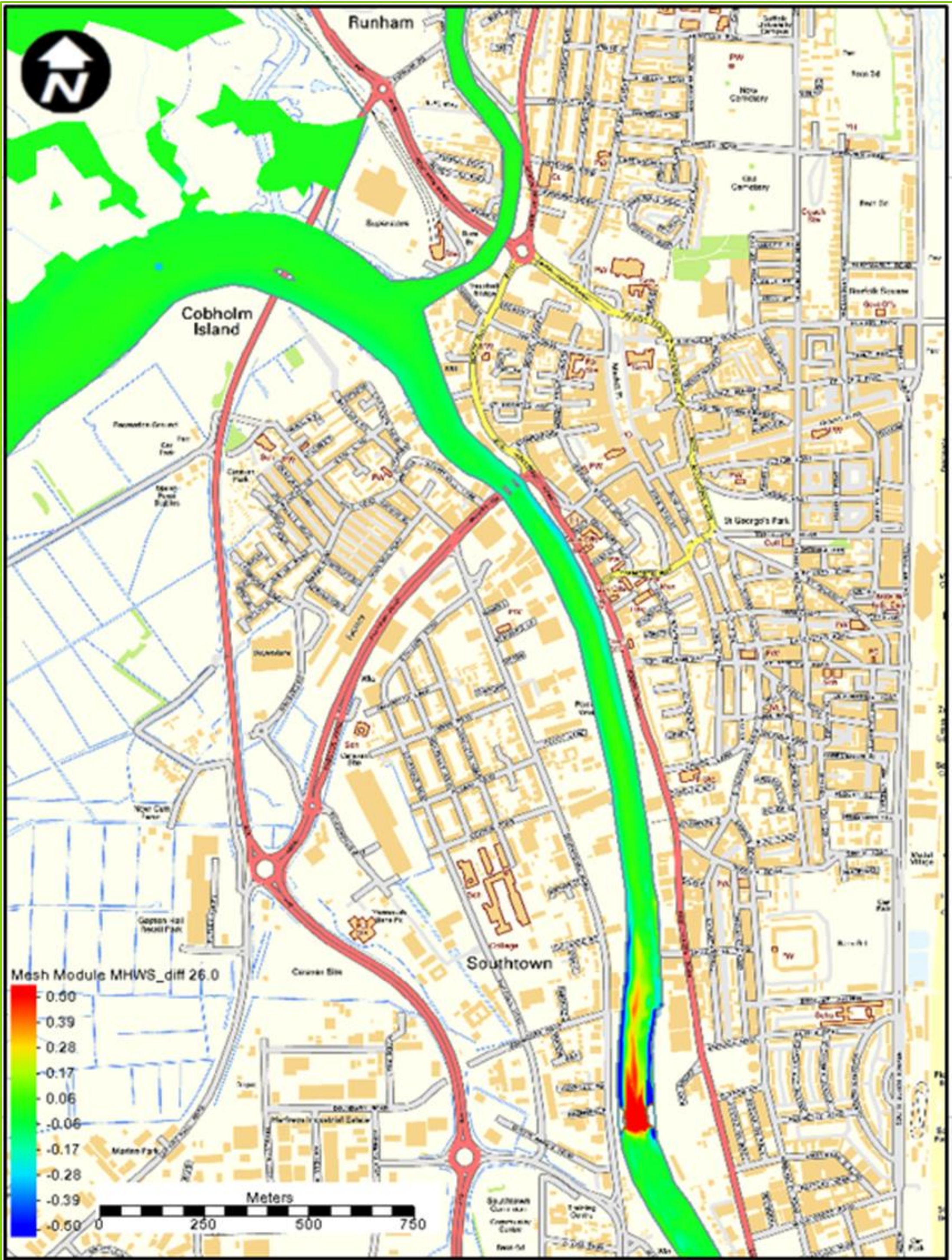


**Plate 6.35– Velocity Magnitude at 26hr, MHWS to MLWS +5% AEP**



*Plate 6-35: Velocity Magnitude, MHWS to MLWS*

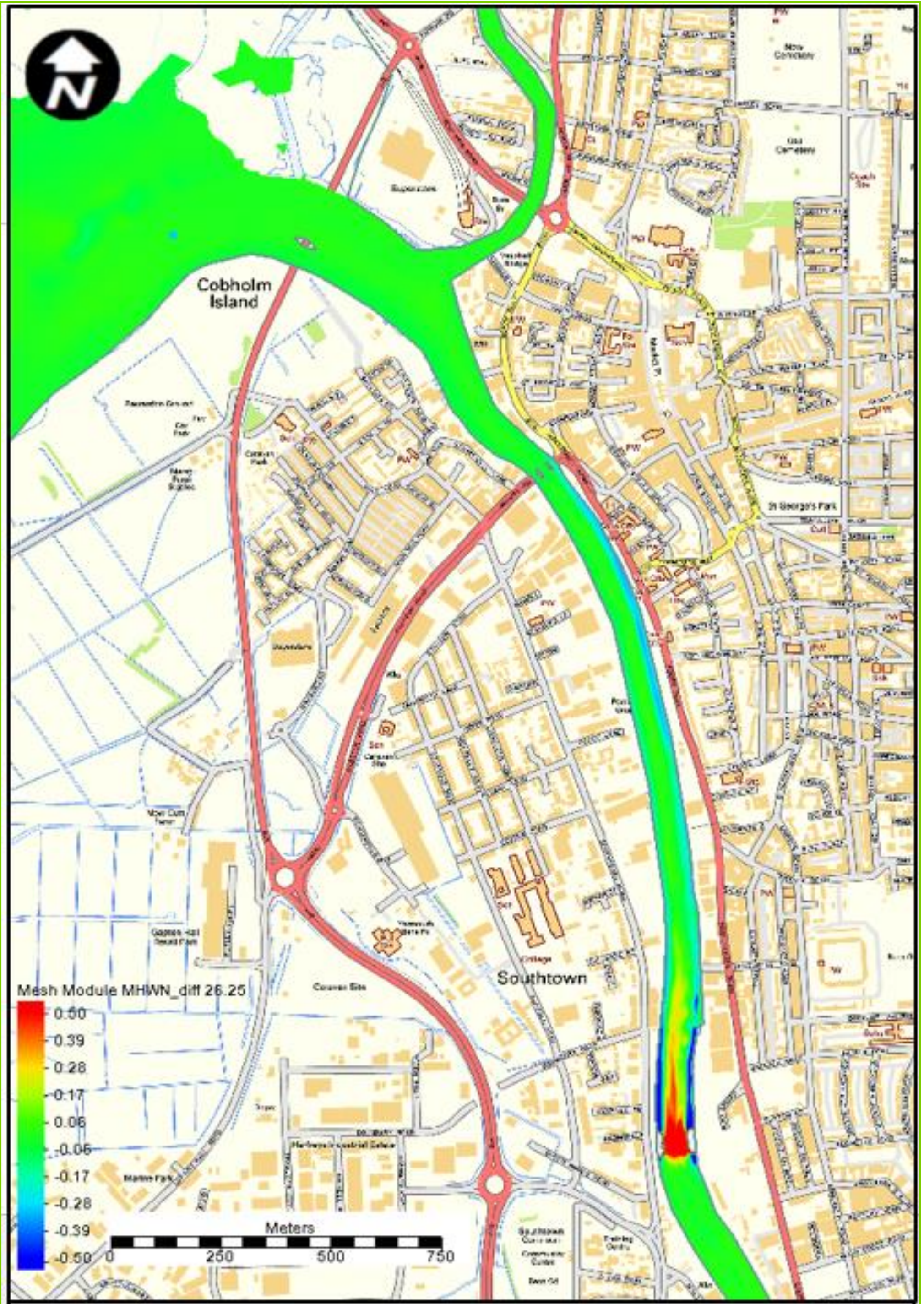


**Plate 6.36– Velocity Magnitude at 26.25hr, MHWN to MLWN +5% AEP**



*Plate 6-36: Velocity Magnitude, MHWN to MLWN*

### Bed Stress

6.3.6 Table 6-7 shows the bed stress for the Baseline and Scheme scenarios at the harbour entrance, Principal Application Site, Haven Bridge and Breydon Water for the MHWS-MLWS+ 5% AEP event. The average bed stress at the Principal Application Site is increased due to the Scheme. There is a negligible impact on bed stress elsewhere in the domain during the MHWS-MLWS+ 5% AEP event.

*Table 6-7: Extreme Tide MHWS-MLWS+ 5% AEP, Bed Stress*

| MHWS-MLWS + 5% | Harbour Mouth | Scheme | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Baseline Average | 0.39 | 0.81 | 2.19 | 0.17 |
| Scheme Average | 0.38 | 2.90 | 2.13 | 0.17 |
| Average Difference | -0.01 | 2.10 | -0.06 | 0.00 |
| **Baseline** | | | | |
| Maximum | 1.63 | 3.11 | 9.28 | 0.97 |
| Minimum | 0.00 | 0.00 | 0.00 | 0.00 |
| **Scheme** | | | | |
| Maximum | 1.57 | 10.74 | 8.54 | 0.95 |
| Minimum | 0.01 | 0.00 | 0.01 | 0.00 |

6.3.7 Table 6-8 shows the bed stress at the harbour entrance, Principal Application Site, Haven Bridge and Breydon Water for the MHWN-MLWN+ 5% AEP event. The average bed stress at the Principal Application Site is increased due to the Scheme. There is a negligible impact on bed stress elsewhere in the domain. The results for the MHWN-MLWN+ 5% AEP event are lower than the MHWS-MLWS+ 5% AEP as the overall velocity magnitudes are lower.

*Table 6-8: Extreme Tide MHWN-MLWN+ 5% AEP, Bed Stress*

| MHWN-MLWN + 5% | Harbour Mouth | Scheme | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Baseline Average | 0.27 | 0.50 | 1.29 | 0.11 |
| Scheme Average | 0.26 | 1.78 | 1.29 | 0.11 |
| Average Difference | -0.01 | 1.28 | 0.00 | 0.00 |
| **Baseline** | | | | |
| Maximum | 1.18 | 2.26 | 5.88 | 0.60 |
| Minimum | 0.00 | 0.00 | 0.00 | 0.00 |
| **Difference** | | | | |
| Maximum | 1.07 | 7.33 | 5.66 | 0.59 |
| Minimum | 0.00 | 0.00 | 0.00 | 0.00 |

### Bed Erosion Rate

6.3.8 To understand the instantaneous impact of the surge event, the bed erosion rates have been compared in

6.3.9 Table 6-9 and Table 6-10. Comparing the Scheme and Baseline values provides an understanding of the likely erosion and deposition due to a single surge event.

6.3.10 Table 6-9 shows the average erosion rate for the duration of the MHWS-MLWS+ 5% AEP surge event. The results show that the erosion rate at the Principal Application Site location increases because of the Scheme due to the increased velocity magnitude. This means additional scour at the Principal Application Site is likely. The average erosion rate at Haven Bridge decreases from the Baseline scenario to the Scheme scenario, this is because the velocity slightly lower in the Scheme scenario. This means the Scheme reduces the rate the material is being scoured at Haven Bridge during the surge events. The results show there is a negligible impact on sediment erosion elsewhere in the domain due to the Scheme.

*Table 6-9: Extreme Tide MHWS-MLWS + 5% AEP, Bed Erosion*

| Baseline | Harbour Entrance | Principal Application Site | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Average Erosion rate (kg/m2/hr) | 1.48 | 3.86 | 10.73 | 0.51 |
| **Scheme** | | | | |
| Average Erosion rate (kg/m2/hr) | 1.44 | 14.10 | 10.46 | 0.51 |
| **Differences** | | | | |
| Average Erosion rate (kg/m2/hr) | -0.04 | 10.24 | -0.27 | 0.00 |

6.3.11 Table 6-10 shows the average erosion rate for the duration of the MHWN-MLWN+ 5% AEP surge event. The results show that the erosion rate at the Principal Application Site is increased because of the Scheme due to the increased velocity magnitude. This means that during the surge event, additional scour at the Principal Application Site is likely. The results show that there is a negligible impact on sediment erosion/deposition elsewhere during the MHWN-MLWN + 5% AEP Surge event.

*Table 6-10: Extreme Tide MHWN-MLWN+ 5% AEP, Bed Erosion*

| Baseline | Harbour Entrance | Principal Application Site | Haven Bridge | Breydon Water |
|---|---|---|---|---|
| Average Erosion rate (kg/m2/hr) | 0.89 | 2.21 | 6.29 | 0.27 |
| **Scheme** | | | | |
| Average Erosion rate (kg/m2/hr) | 0.85 | 8.71 | 6.28 | 0.27 |
| **Differences** | | | | |
| Average Erosion rate (kg/m2/hr) | -0.04 | 6.50 | -0.01 | 0.00 |

6.3.12 In conclusion, the impact of a likely extreme event is that water flushes through the River Yare channel through Great Yarmouth at a higher ambient velocity magnitude than during the everyday events and the velocity magnitude increases locally at the Principal Application Site due to the presence of the Scheme. This in turn increases the instantaneous scour near the Principal Application Site for the short period over which the extreme tide occurs. The results show the impact on erosion/deposition elsewhere is negligible.

## 6.4 Construction Phase

6.4.1 To construct the Scheme, cofferdams will be installed that will be filled in to create the bridge Knuckles. There will be no additional increase in the footprint of the Scheme in the water during construction compared to the operational phase. This means there is no need to simulate a separate model for the construction phase as the results presented above for the operational phase will apply.

## 6.5 Impact of the Scheme on Tidal Parameters

6.5.1 In order to assess the wider impacts of the Scheme on the watercourse, the tidal parameters calculated in Section 4 has been assessed using the model for the Scheme scenario.

### Tidal Asymmetry

6.5.2 Plate 6-37 shows the velocity magnitude against the water elevation at the Principal Application Site. The plot shows that when compared to the Baseline plot in Plate 4-6, the scheme does not have an impact on the tidal asymmetry in the model. The area taken up by the bridge knuckles is relatively small when compared to the estuary as a whole and the localised increase in velocity magnitude ensures that the same volume of water reaches the upper estuary and Breydon Water. The Scheme model shows that the tide is still almost symmetrical with a slight skewness at high water.

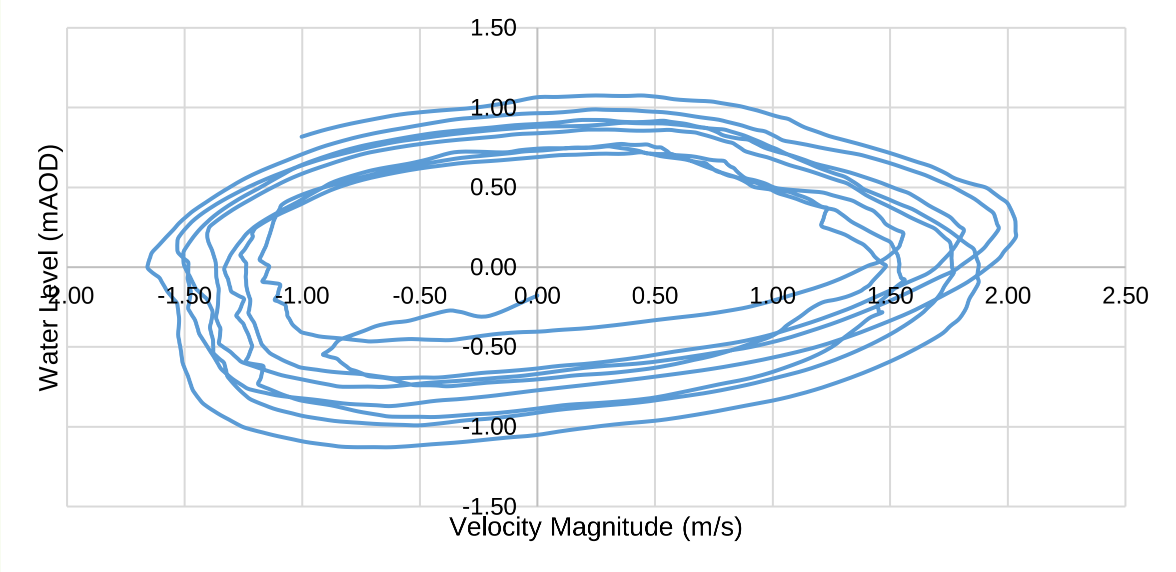


*Plate 6-37: Velocity Magnitude against Water Level at the Principal Application Site in the Scheme Scenario*

### Tidal Dominance

6.5.3 Table 6-11 shows the Dronker's Ratio calculated for the Scheme scenario, the surface area and volume are slightly decreased due to the presence of the Scheme in the watercourse. However, this is no difference in the Dronker's Ratio when rounded to two decimal places. This shows that the Scheme does not change the estuary type which has been shown to be Type II and considered ebb dominant.

*Table 6-11: Scheme Dronker's Ratio*

| | Baseline | Scheme |
|---|---|---|
| Hydraulic depth, dh | 3.88 | 3.89 |
| Tidal Amplitude, a | 0.58 | 0.58 |
| Surface area at low water, Slw | 1318636 | 1318636 |
| Surface area at high water, Shw | 4916929 | 4916929 |
| Volume at high water, Vhw | 9475544 | 9489944 |
| Volume at low water, Vlw | 4357856 | 4369376 |
| Dronker's | 0.49 | 0.49 |

### Tidal Dominance and Climate Change

6.5.4 In order to gain an understanding of the effects of climate change on the tidal dominance, The Dronker's Ratio has been calculated when the water level increases by 1.88m. This level has been obtained using the UK Climate Projections 18 (UKCP18) estimated sea level rise dataset and extrapolated for a 120 year design life. The increase of 1.88m creates an average high water level of 2.88mAOD. When considering the river cross-section in Plate 4-4, the 2.88mAOD water level is retained within both banks of the Breydon Water therefore the assumption is that the water will not overtop the defences and flow onto the floodplain.

6.5.5 Table 6-12 shows that with the increase in sea levels due to climate change, the Dronker's Ratio suggests that the estuary will change to a Type I, flood dominant Estuary. The Scheme is not shown to impact on the estuary type and tidal dominance.

*Table 6-12: Climate Change, Dronker's Ratio*

| Measure | Climate Change - Baseline | Climate Change - Scheme |
|---|---|---|
| Hydraulic depth, dh | 3.35 | 3.35 |
| Tidal Amplitude, a | 0.58 | 0.58 |
| Surface area at low water, Slw | 4916929 | 4915129 |
| Surface area at high water, Shw | 4916929 | 4915129 |
| Volume at high water, Vhw | 9253660 | 9253660 |
| Volume at low water, Vlw | 18743605 | 18729205 |
| Dronker's | 2.01 | 2.01 |

6.5.6 The results show that the Scheme has no impact on the tidal parameters when considering the estuary as a whole. This is because the relative size of the Scheme in the watercourse compared to the whole estuary is very small and the Scheme is not large enough to have a significant impact on the overall tidal regime of the estuary. The overall volume of sediment movement through the estuary will not be impacted significantly by the Scheme to cause a visible change in the estuary wide sediment regime.

# 7 Summary

7.1.1 A 3D flexible mesh hydraulic model of Great Yarmouth has been developed to assess the impact of the Scheme on sediment transport in the River Yare and Breydon Water. The tidal curve for the Spring and Neap has been extracted from Gorleston on Sea level gauge and used to force the model for the ‘everyday’ scenario. For ‘extreme’ events, the hydrology of Great Yarmouth has been analysed and the MHWS to MLWS +5% AEP Surge and the MHWN to MLWN +5% AEP Surge have been derived. The tidal boundaries have been applied at the boundary to the south of the Scheme at the North Sea.

7.1.2 Calibration testing has been carried out by comparing the model output to a velocity survey carried out using an ADCP device in April 2018. The model has been simulated using the levels extracted from Gorleston on Sea gauge and the velocity points compared to the model results. The 3D depth-averaged results show that the model can predict the velocity magnitude in the channel well. There are a few differences which are likely to be local impacts such as disturbances from vessel moves for example. The model is considered fit for use in the sediment assessment.

7.1.3 The D50 Sediment particle size ranges from 0.03mm to 0.55mm and defined as predominately silt and sand. The sediment model has been set up to simulate silt and sand and chart the evolution through the system. The model has been used to simulate the Spring and Neap tidal events to represent the everyday events and likely extreme events.

7.1.4 The Everyday tide results show that the Scheme locally increases the velocity magnitude because of the constriction of the Scheme knuckles in both the Spring and Neap simulations. This locally increases the scour in the centre of the channel and the material is typically moved the Quay walls where the velocity magnitude is decreased. During the Neap tide there is a negligible impact on velocity magnitude elsewhere in the domain. The Spring tide shows there is a small impact on scour rates at Haven Bridge which causes a small amount of erosion and deposition locally. There is a negligible impact in Breydon Water and at the Harbour entrance. There is a localised impact on bed stress and erosion rates due to the presence of the Scheme in the Spring and Neap tide.

7.1.5 The extreme tide events show that the velocity magnitude experiences an increase due to the presence of the Scheme in the water course. The localised impacts are greater at the Scheme when compared to the Everyday scenarios. There is a small reduction in velocity magnitude at Haven Bridge which means the bed erodes slower due to the presence of the Scheme. There is a negligible impact elsewhere in the domain during the extreme events. It should be noted that due to the low frequency of such events in the channel, the change in scour patterns are negligible.

7.1.6 The tidal parameters analysis has shown the Scheme has no change on the tidal prism, water level, asymmetry and Dronker's Ratio. This is because when considering the estuary, the area taken up by the knuckles is negligible therefore the increase in velocity magnitude ensures the same volume of the water transits the estuary. This means that the overall volume of sediment transport in the estuary is not affected by the Scheme simply because the volume taken up by the knuckles is negligible when compared to the estuary as a whole.

7.1.7 In conclusion, the modelling and tidal analysis has shown that the presence of the Scheme does increases the scour and deposition within the Principal Application Site. The modelling has shown there is small impacts in the engineered channel up to Haven Bridge, however the additional scoured material remains in the engineered channel. There is a negligible change in the sediment regime of Breydon Water due to the presence of the Scheme. The Scheme has no impact on the tidal parameters of the estuary.

7.1.8 There is no additional material transported into the engineered channel due to the presence of the Scheme. Therefore, there is no change to the overall dredging regime in the harbour needed. However, some dredging areas may change due to the physical presence of the Scheme in the channel. Engineering scour protection should be considered at the Scheme in order to reduce the impact of the increased velocity magnitude and reduce the volume of sediment scoured.

# 8 References

Ref 11C.1: General Port and Pilotage Information P16, Peel Ports Great Yarmouth, October 2014.

Ref 11C.2: SC060064/TR4: Practical Guidance Design Sea Levels and Open Coast (CFBD) Flood Risk Study (2014) JBA for the Environment Agency.

Ref 11C.3: Analysis and Modelling Guide, EA/ABPmer, 2008

Ref 11C.4: Ferguson, R & A. Church, M. (2004). A Simple Universal Equation for Grain Settling Velocity. Journal of Sedimentary Research - J SEDIMENT RES. 74. 933-937. 10.1306/051204740933.

# Great Yarmouth Third River Crossing

## Application for Development Consent Order

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## Document 6.2: Environmental Statement Volume II: Technical Appendix 11C, Annex A: Tidal Boundary Derivation – Sediment Assessment

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**Planning Act 2008**

**The Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009 (as amended) ("APFP")**

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PAGE No.

# Tables

# Plates

# 1 Introduction

1.1.1 This note records the process and decision making that has been followed to generate the tidal boundaries for the Sediment Transport Assessment carried out as part of the Great Yarmouth Third Crossing (hereafter known as 'the Scheme').

1.1.2 The purpose of the assessment is to simulate an 'everyday' scenario and likely extreme scenarios which do not cause out of bank flooding to get an understanding of the impact of the Scheme on the existing sediment regime. Out of bank flooding is not considered in the Sediment Transport Assessment because the focus of this assessment is on in-channel everyday events where the water is predominately moving up and down the channel. The likely extreme scenarios consider the impact during of small tidal surges. The tidal boundary has been created using two different processes; firstly selecting a typical Spring/Neap tidal cycle from existing data to simulate the everyday event and secondly, deriving a tidal boundary for likely extreme tides.

1.1.3 The everyday Spring/Neap boundary has been extracted from the recorded gauge data at Gorleston-on-Sea level gauge located at the harbour mouth. The extreme tidal boundary derivation detailed here follows the recommendations set out in SC060064/TR4 (Ref 11C.2).

# 2 Everyday Scenario

2.1.1 In order to generate the "everyday" tidal boundary, the recorded tidal data was downloaded from the British Oceanography Data Centre (BODC) website for 2018. Plate 2.1 shows the water elevation recorded for the full year for 2018.

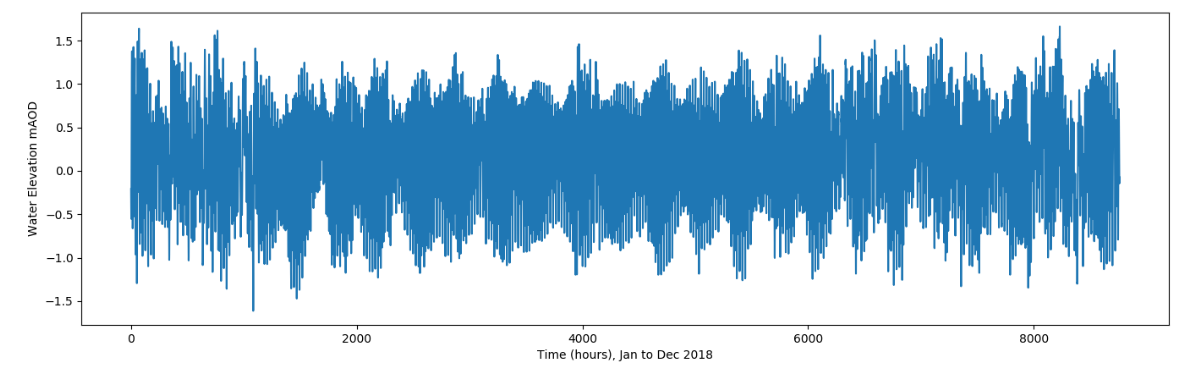


*Plate 2.1: 2018 - January to December*

2.1.2 Plate 2.1 shows the full year of recorded data at Gorleston-on-Sea for 2018. The time series plot shows the typical Spring/Neap cycle repeating approximately every 2 weeks throughout the year and several surge tides particularly around the early part of the year around January to February. For the purpose of this assessment a typical Spring/Neap tide cycle is required; therefore, the curve shown in Plate 2.2 has been extracted making sure no surge events are captured.

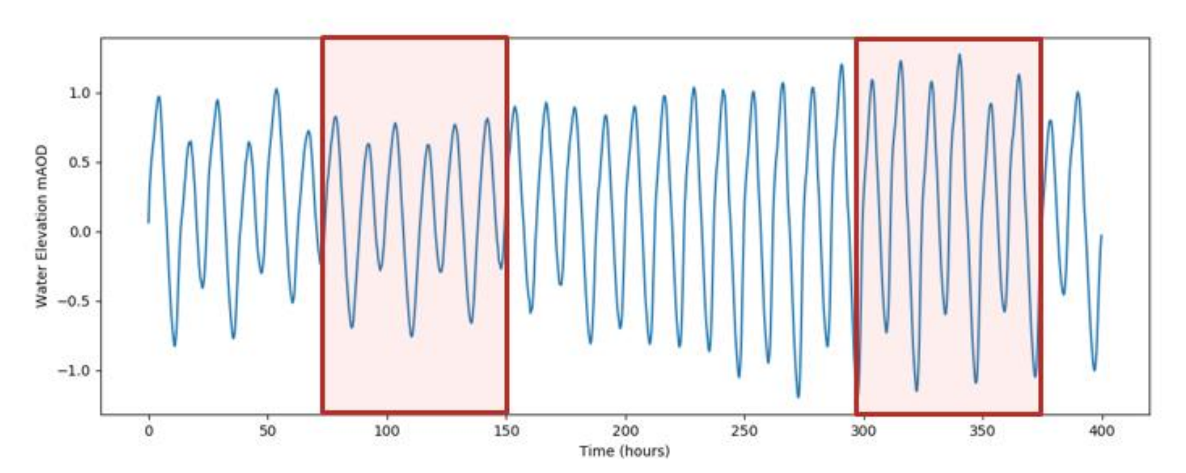


*Plate 2.2: Extracted Tidal Curve*

2.1.3 Plate 2.2 shows a typical water level time series ranging from a Neap to Spring tide which includes the shape of the tide which can be replicated in the model. The data has been selected from the yearly recorded data shown in Plate 2.1 to represent at typical tide with minimal surge events. At this point, the date of the profile is no longer relevant therefore the plate plots the tidal cycle against time in hours starting at zero hour. In an effort to reduce simulation time, the curve shown in Plate 2.2 has been split into two separate simulations (shown in the red boxes) of approximately 75 hours; one simulating a Spring tide and one simulating a Neap tide. These simulations will be used to approximate the amount of sediment movement on a typical Spring and Neap tide.

2.1.4 The aim of this event is to simulate a typical tidal profile and assess the impact of the Scheme on the sediment regime due to everyday flow. The tidal boundaries shown in Plate 2.2 will be simulated in the 3D model and the sediment transport will be assessed.

# 3 Extreme Tidal Curve Derivation

3.1.1 The purpose of this curve is to assess the impact of a sudden, likely extreme event on the sediment and what the impact of the Scheme is on sediment transport.

3.1.2 The section records the steps carried out to generate a number of sea surge events showing the peak of the 5% AEP event from the JBA 2014 has been applied to the base profiles. The extreme events are;

- MHWS to MLWS + 5% AEP sea surge;
- MHWN to MLWN + 5% AEP sea surge.

## 3.2 Extreme Tide Calculations

3.2.1 To investigate the impact of a likely extreme tide level, tidal curves have been derived using the SC060064/TR4 guidance (Ref 11C.2) to create curves with the peak water level of the 5% AEP level provided by JBA in Open Coast (CFBD) Flood Risk Study (Ref 11C.3). Table 3.1 lists all the steps set out in the Environment Agency guidance.

*Table 3.1: Guidance Steps*

| Ten Step procedure |
|---|
| 1. Check study location is outside of estuary boundaries |
| 2. Select an appropriate chainage point for extreme sea levels |
| 3. Select an annual exceedance probability peak sea level |
| 4. Consider allowance for uncertainty |
| 5. Identify base astronomical tide |
| 6. Convert levels to Ordnance Datum |
| 7. Identify surge shape to apply |
| 8. Produce the resultant design tide curve |
| 9. Sensitivity testing |
| 10. Apply allowance for climate change |

3.2.2 The guidance is part *of* the larger project, 'Coastal flood boundary conditions for UK mainland and islands' (Ref 11C.5) and is the best method currently available for tidal curve derivation in UK waters. As part of this project several additional datasets are also provided, as shown in Table 3.2.

*Table 3.2: Additional Data Sets*

| Additional Data |
|---|
| Estuary Boundaries |
| Extreme Sea Levels |
| Gauge Sites |
| Confidence Interval |
| Surge Shapes |

3.2.3 In following the guidance steps set out in Table 3.1 and using the datasets in Table 3.2 the extreme event tidal curves are generated.

### Check Study Location in Outside of Estuary Boundaries

3.2.4 The guidance states that it is only valid for areas outside of estuaries, and as such the first check is to make sure the boundary is not in a major estuary. As part of the SC060064/TR4 guidance (Ref 11C.2), a shape file is provided with all major estuary locations highlighted.

3.2.5 On reviewing the Estuary Boundary dataset, the proposed location of the tidal boundary is outside any estuary.

### Select the Appropriate Chainage Point for Extreme Sea Levels

3.2.6 The guidance recommends that the extreme sea level node nearest to a horizontal line drawn from the tidal boundary should be used to define the extreme sea levels for the site of interest. A horizontal line drawn from the Great Yarmouth tidal boundary passes closest to 4,150 chainage node.

### Select an Annual Exceedance Probability Peak Sea Level

3.2.7 For each chainage node, an extreme sea level for the full range of return periods is provided in the additional data supplied alongside the guidance. The extreme sea levels modelled by JBA on behalf of the Environment Agency (Ref 11C.2) at node 4,150 are provided in Table 3.3 for the event considered in this study.

*Table 3.3: Extreme Sea Level*

| AEP | Extreme Sea Levels (m AOD) |
|---|---|
| 5% | 2.84 |

### Consider Allowance for Uncertainty

3.2.8 As part of the SC060064/TR4 project (Ref 11C.2), confidence in the extreme sea levels are provided as shown in Table 3.4 for the event considered in this study. The confidence levels are a measure of the potential error in the Environment Agency extreme sea level modelled results.

*Table 3.4: Uncertainty Levels (node 4,150)*

| AEP | Uncertainty (+/-m) |
|---|---|
| 5% | 0.2 |

### Identify a Base Astronomical Tide

3.2.9 Gauge data at the Great Yarmouth gauge has been made available however the MHWN and MLWN levels have not been obtained because Great Yarmouth is not a Primary Gauge on the network. In the interest of consistent, the tidal parameters should all be obtained from the same source. In this situation, EA guidance recommends using the properties of the nearest Primary Gauge to the site of interest. The nearest Primary Gauge is in Lowestoft harbour approximately 12km to the south. Table 3.5 shows the tidal properties from the Lowestoft harbour gauge that will be used to create the base tide profiles.

*Table 3.5: Lowestoft Primary Gauge Properties*

| Property | Value (mAOD) |
|---|---|
| *HAT* | 1.48 |
| *LAT* | -1.38 |
| *MHWS* | 1.08 |
| *MLWS* | -0.86 |
| *MHWN* | 0.74 |
| *MLWN* | -0.34 |

3.2.10 As part of this assessment, Gauge data from the Great Yarmouth gauge at Gorleston-on-Sea has been obtained. The data has been recorded from December 1992 and continues to be in operation recording the sea level at the mouth of the River Yare. Plate 3.1 shows an extract from the gauge data.

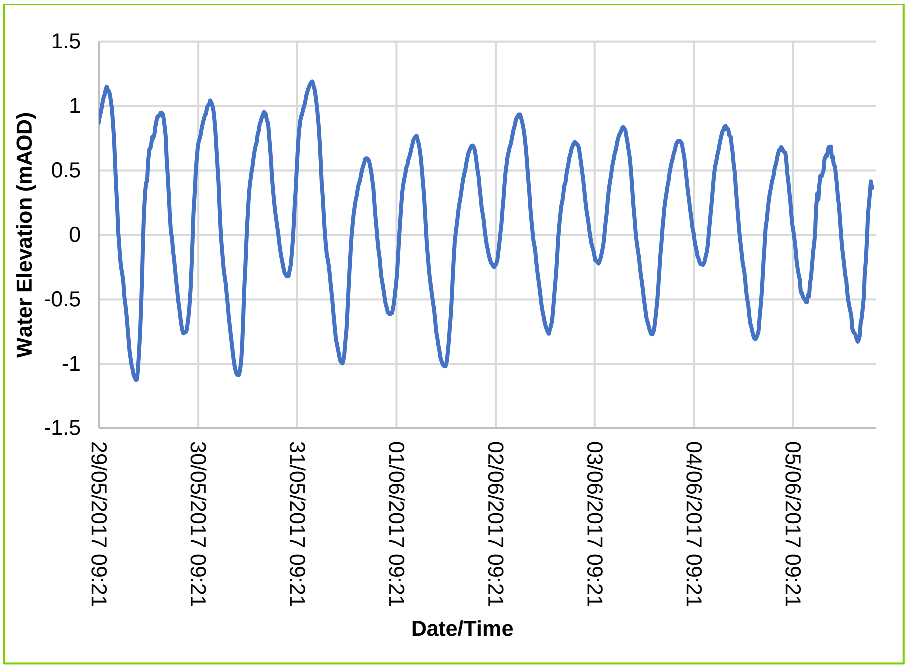


*Plate 3.1: Extract from the Great Yarmouth Gauge*

3.2.11 In order to properly represent the tidal curve shape, the gauge data has been reviewed and a typical tidal cycle has been extracted. This tidal cycle has then been scaled so the peak and trough matches the required water level. Plate 3.2 shows the typical tidal cycle extracted from the gauge.

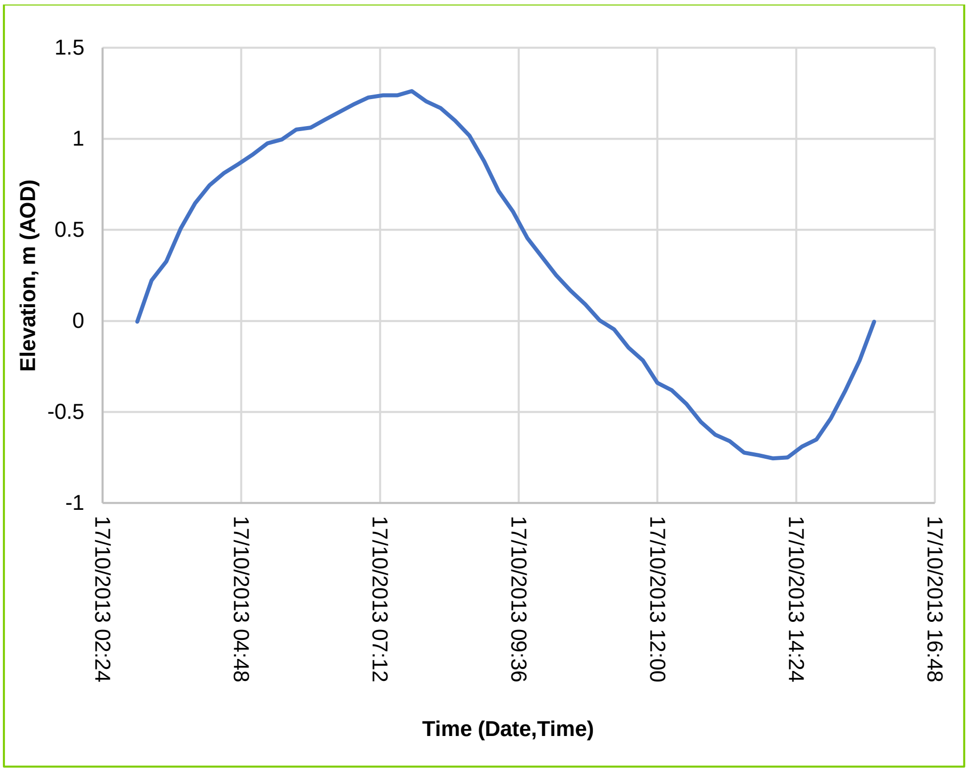


*Plate 3.2: Typical Tidal Curve*

3.2.12 Plate 3.2 shows a typical tidal curve extracted from the Gorleston-on-Sea Gauge. Extracting a typical tidal profile from the gauge accurately predicts the shape of the tide taking into account the skewness, symmetry and the period.

3.2.13 Following the extraction of the typical curve from the gauge data, the curve shown in Plate 3.2 has been extended by repeating the tidal cycle to create the base curve to run the model for 75 hours. At this point, the peak and trough for the curves have been scaled to the required levels in order to create the base tidal profiles for the assessment, as shown in Plate 3.3.

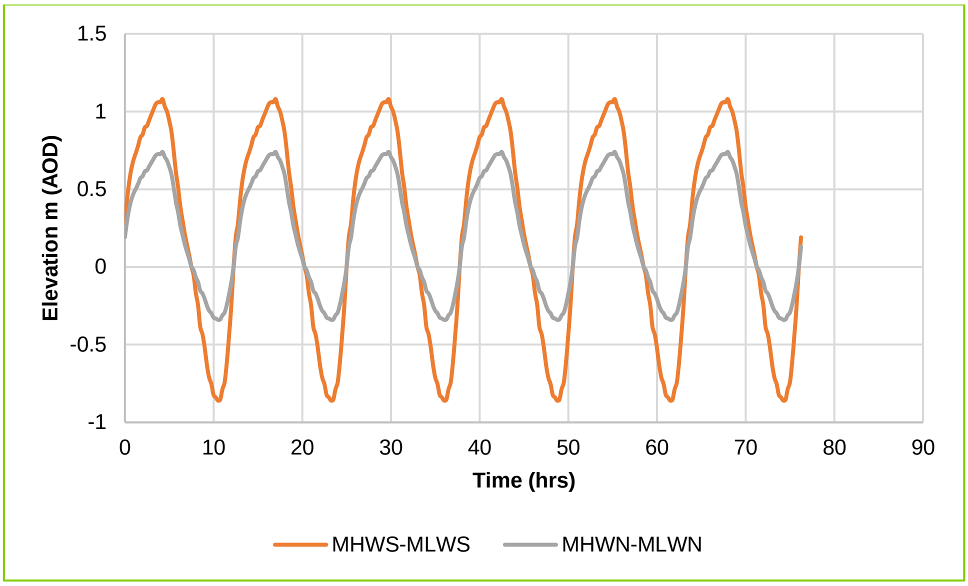


*Plate 3.3: Base Tide Profiles*

3.2.14 Plate 3.3 shows the base tidal profiles used to generate the extreme events simulated in the sediment model.

### Convert Levels to Ordinance Datum

3.2.15 All levels are assessed with respect to Ordinance Datum. Any local levels may be recorded in Chart Datum and, for Great Yarmouth, the chart datum conversion is -1.56m.

### Identify Surge Shape

3.2.16 As part of the SC060064/TR4 (Ref 11C.2) project surge shapes where derived for key locations around the UK. For this assessment the nearest surge shape is number 9 in the Design_Surge_Shapes.xls provided with the guidance documentation.

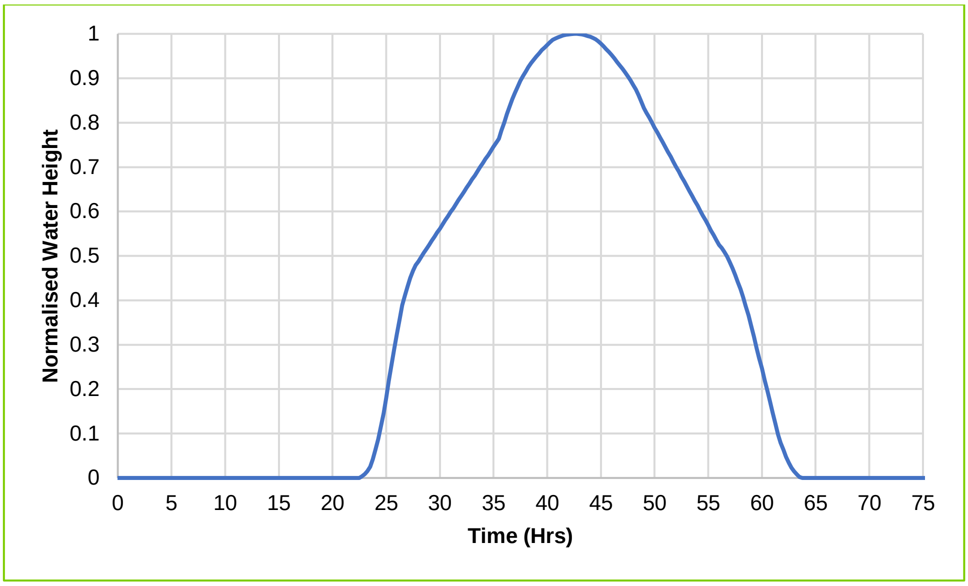


*Plate 3.4: Shape 9 – Lowestoft Surge*

3.2.17 Plate 3.4 shows the normalised surge shape which when combined with the base tidal profiles, the design tidal curves are derived.

### Produce the Resultant Design Tide Curve

3.2.18 The guidance states that the resultant design tide curve is derived by combining the extreme sea level, base tide and surge shape. The first process is to align the base tide and surge shape peaks, in this case this is at 42.5 hours in line with the base tidal curve.

3.2.19 Once the base tide and surge shape are aligned, it is necessary to scale the base tide to the required extreme sea level. To explain this procedure, the HAT-LAT - 5% AEP event will be used as an example. Firstly, the difference between the required extreme sea level (2.84mAOD) and the base tide peak (1.48mAOD) is calculated, which in this example is 1.36m. As the surge shape is aligned with the peak water level time in the base tide, the maximum surge value of 1.0 occurs at the same time as the peak water level. The surge shape can now be scaled by the coefficient 1.36/1.0 = 1.36m AOD, thus creating a surge height which can be added to the base tide curve resulting in the required tidal profile for the event.

3.2.20 This procedure is carried out of each tidal profile to produce the three tidal boundaries required for this extreme scenario assessment as shown on Plate 3.5.

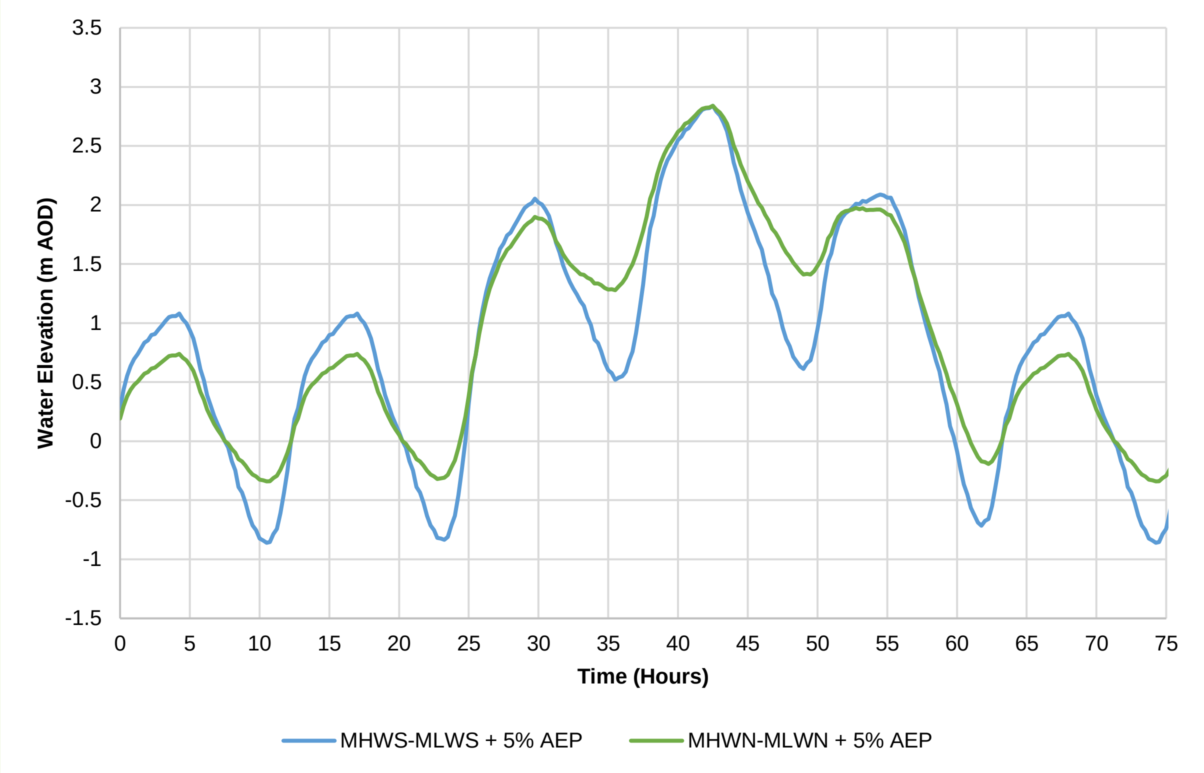


*Plate 3.5: Tidal Curves for all Events*

### Sensitivity Testing

3.2.21 For this assessment, no additional curves are required for the sensitivity testing.

### Climate Change Calculations

3.2.22 For this assessment, climate change scenario is not considered therefore no climate change curves have been created.

### Conclusions

3.2.23 For the purpose of the sediment transport assessment, the tidal curves for each of the events have been created (Plate 3.5).The final curves generated will be used as the inflow boundary for the 3D hydraulic sediment model developed for the Scheme.

### Limitations

3.2.24 There are a number of limitations highlighted in the guidance documents. These are presented in Plate 3.6.

*Table 3.6: Limitations of the Tidal Curve Derivation Method*

| Limitation | Description |
|---|---|
| Extreme sea levels are considered accurate to one decimal place. | The extreme sea levels are considered accurate to one decimal place, two decimal places are provided only to differentiate between nodes on the chainage. |
| Extreme sea levels do not consider wave impacts. | The sea level values presented include effects from the storm surge but do not include any impact on local sea level due to onshore wave action. |

3.2.25 The guidance document recognises flaws in the data used to produce the extreme sea levels, this is due to difficulty recording long-term sea level data. However, it is stated that this is the best possible method currently available and uses the most accurate initial conditions available. The limitations are considered acceptable for the accuracy required in a flood risk assessment therefore the extreme sea level curves will be used to assess flooding in Great Yarmouth due to the Scheme.