The Lake Lothing (Lowestoft) Third Crossing Order 201[*]

Document SCC/LLTC/EX/37: Environmental Statement Volume 3 – Appendix 17C Sediment Transport Assessment – Tracked changes Revision 1

Planning Act 2008

The Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009

Regulation Number: 5(2)([e])

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Foreword

This Sediment Transport Assessment relates to an application ('the Application') submitted by Suffolk County Council ('the Council' / 'the Applicant') to the Secretary of State (through the Planning Inspectorate) for a Development Consent Order ('DCO') under the Planning Act 2008.

If made by the Secretary of State, the DCO would grant development consent for the Applicant to construct, operate and maintain a new bascule bridge highway crossing, which would link the areas north and south of Lake Lothing in Lowestoft, and which is referred to in the Application as the Lake Lothing Third Crossing (or 'the Scheme').

This Sediment Transport Assessment has been prepared in accordance with the requirements of section 37(3)(d) of the Planning Act 2008 and regulation 5(2)(e) of the Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009 ('the APFP Regulations'), and in compliance with relevant guidance. It has been updated to reflect comments from the Environment Agency as part of their Relevant Representations.

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Abbreviations

ABP	Associated British Ports
AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
CD	Chart Datum
DCO	Development Consent Order
EA	Environment Agency
ESL	Extreme Sea Levels
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
NPPF	National Planning Policy Framework
NTSLF	National Tidal and Sea Level Facility
ReFH	Revitalised Flood Hydrograph
SCC	Suffolk County Council
UKCP09	UK Climate Projections
WDC	Waveney District Council
HAT	Highest Astronomical Tide
LAT	Lowest Astronomical Tide

Figures

Tables

1 Introduction

1.1 Overview

1.1.1 As part of the Lake Lothing Third Crossing (hereafter known as the 'Scheme'), a 3D hydraulic model has been built to assess the impact of the Scheme on water speed in the channel. This report describes the development of the hydraulic model built to assess the impact of the Scheme on the sediment regime within Lake Lothing and the results of it. The pile arrangement for the main support structures have been explicitly modelled. An assessment of the construction cofferdam has been carried out, this is considered the worst-case arrangement for the Scheme and represents the greatest impact on the sediment regime.

1.1.2 As part of the Development Consent Order (DCO) application, the Environment Agency (EA) submitted a Relevant Representation relating to certain aspects of the Scheme, which identified some concerns with the Sediment Transport Assessment that was included as Appendix 17C to the Environmental Statement (ES) (Document Reference 6.1). Following further discussion with the EA, that report has therefore been updated to take into account comments made by the EA.

1.2 Study area

1.2.1 Lake Lothing is used as a commercial transport hub with a number of large ship berths on the north and south side of the lake. The lock at Mutford Bridge at the upstream end of the lake controls the water flow between Oulton Broad and Lake Lothing and allows the passage of small leisure vessels. Lowestoft currently has two road bridge crossings; the A47 Bascule Bridge and Mutford Bridge as shown in Figure 1-1. These are the only two methods for traffic to cross Lake Lothing. In addition to the road crossings there is a railway crossing near Mutford Bridge as shown in Figure 1-1.

1.2.2 Three small fluvial catchments discharge into Lake Lothing; the watercourses associated with these catchments are Kirkley Stream and two small unnamed drainage channels. Kirkley stream is approximately 4.4km long and flows in a northerly direction into the southern side of Lake Lothing. One of the unnamed drainage channels is also on the south side of Lake Lothing and the other is on the northern side however the precise location of the outfalls is unknown.

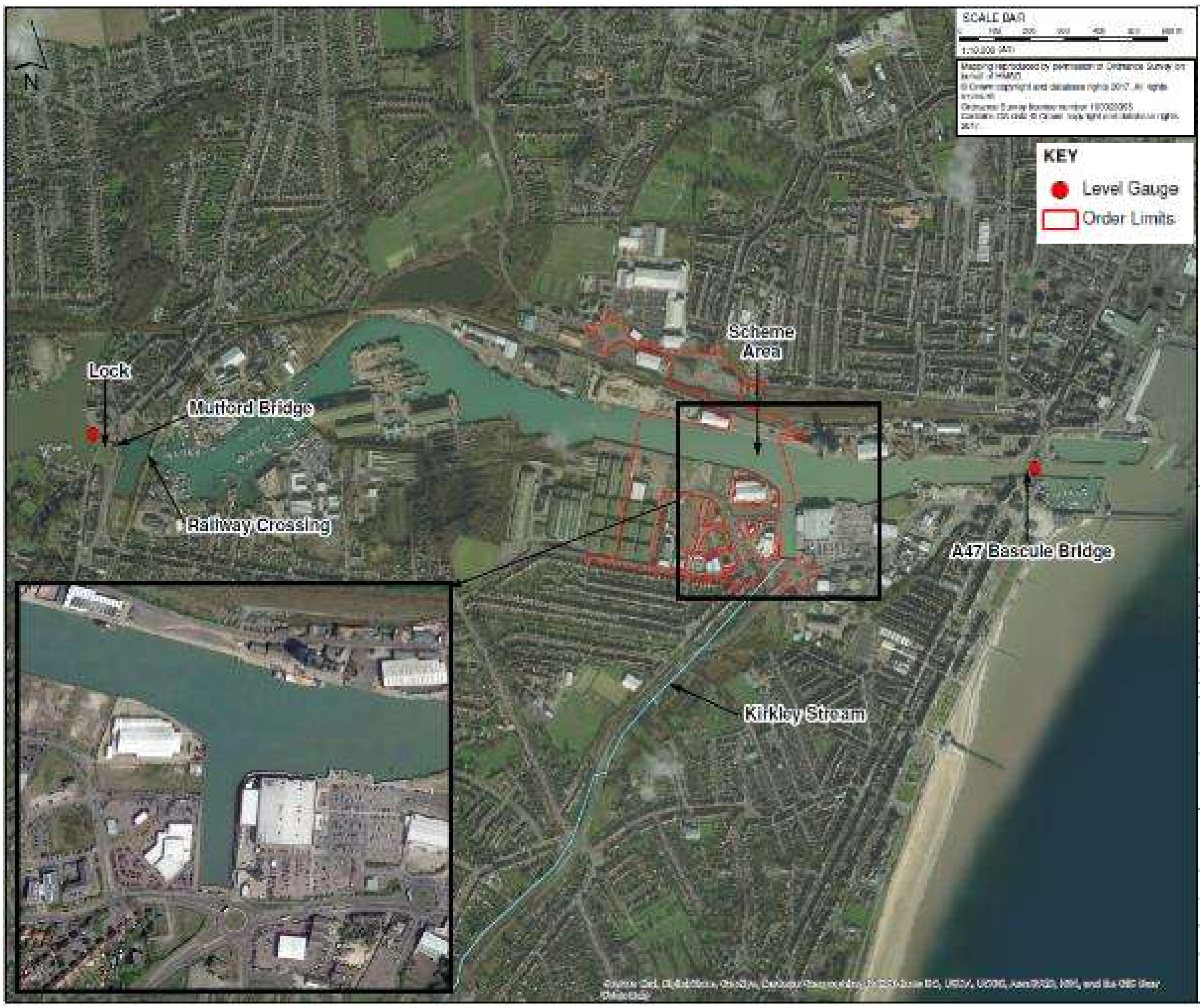

Figure 1-1 – Location Plan

1.3 The Scheme

1.3.1 The bridge and the new road layouts are shown indicatively on Figure 1-2. The Scheme consists of a central bascule bridge supported by a total of six concrete piers and is located approximately 0.8km upstream of the existing A47 Bascule Bridge.

1.3.2 Two large central piers support the centre bascule span in the lake. The access road layout includes two new roundabouts, two embankments and a small network of paved roads. Consequential amendments are also made to the existing road network on the north and south side of the Scheme, including a new access road on the south side linking Riverside Road and Waveney Drive.

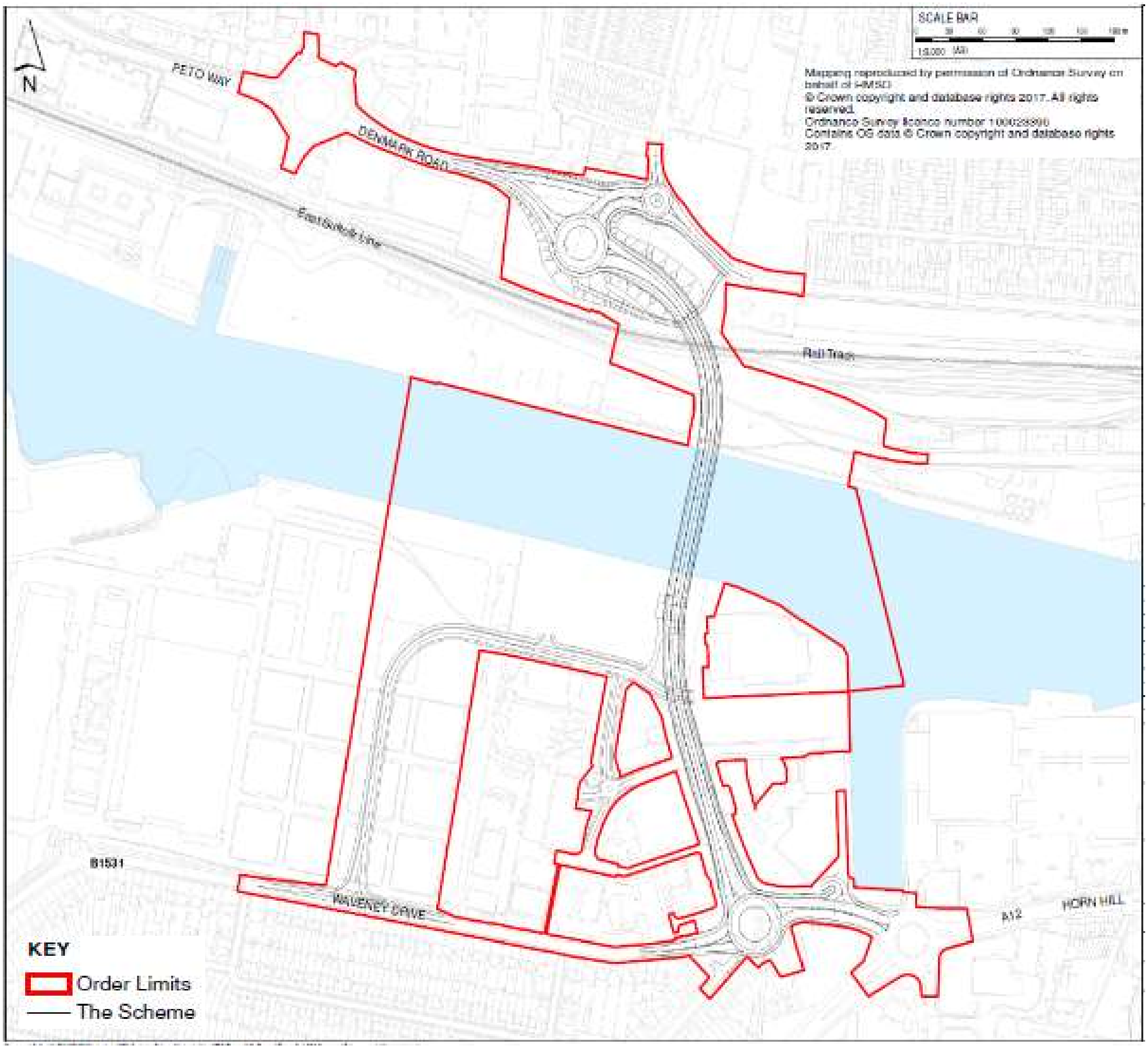

Figure 1-2 - Road Alignment of the Scheme

2 Data Collection and Review

2.1 Overview

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

Table 2-1 - Summary of data collected

Data	Source
1D-2D ISIS TUFLOW Lake Lothing model	Waveney District Council / CH2M Hill
Version C19 design for the Scheme	WSP
OS Mastermap As built construction drawings for existing crossings. (A47 Bascule Bridge, Lock at Mutford Bridge and Mutford Bridge) Previous study reports[1]	Suffolk County Council (SCC)
Bathymetric survey	Associated British Ports (ABP)
2015, 0.5m LiDAR Extreme sea levels Daily average gauge data for Lake Lothing and Oulton Broad	Environment Agency (EA)
Tidal levels in Lake Lothing charts	National Tidal and Sea Level facility (NTSLF)
Topographic data on the North and South Quay	WSP

2.2 Data Review

2.2.1 CH2M Hill developed a 1D-2D ISIS TUFLOW model as part of the Lowestoft Flood Risk Management Strategy. The model was reviewed by WSP to determine whether it could be used in this assessment. The review has shown that whilst the model is suitable for its intended purpose it does not provide sufficient detail needed for the sediment assessment.

2.2.2 The 1D-2D model provides a depth-averaged value at each cell location for each parameter modelled. Whilst the level of detail is sufficient for flood mapping activities, it has been decided by WSP more detail is required in terms of water speed on the Lake Lothing bed to determine sediment transport. To that end, a TUFLOW FV 3D model is considered more appropriate because it models and reports on parameters at different depths throughout the water column. A detailed description of the model build has been provided in section 4.1.

2.2.3 Levels data from the reference design drawings have been used to represent the Scheme in the post-development model built for this study. Topographic survey data collected on the

[1] *Lowestoft tidal barrier - outer harbour water level modelling investigation – 2016*
Lowestoft Tidal Defences Additional Modelling Studies – 2014
Lowestoft Flood Risk Management Strategy - 2016

North Quay and South Quay by the WSP Highways team was also provided for use in this study.

2.2.4 SCC provided a number of datasets and documents for use in this assessment. OS mastermap data covering Lowestoft was used, which includes land use classification, as were as-built drawings for the existing road crossings over Lake Lothing.

2.2.5 ABP has provided a detailed bathymetric survey of Lake Lothing and the outer harbour. The dataset contains points measured from Chart Datum (CD) within the harbour taken on a boat that traversed the inner and outer harbour. The levels on the bathymetric survey have been sensibility checked against topographic survey levels on the north and south quays of Lake Lothing and the bed levels appear reasonable. In order to use the data collected during the bathymetric survey, it was necessary to convert the levels provided from CD to mAOD as all other level data used in this assessment is in mAOD. Lowestoft CD is -1.5mAOD and is defined as the approximate level of the lowest astronomical tide at Lowestoft. The bathymetric survey data points are converted to mAOD by adding -1.5m from each level recorded in the survey.

2.2.6 The Environment Agency (EA) has provided several datasets; the 2015, 0.5m resolution LiDAR dataset, Extreme Sea Levels (ESL)[2] and daily water level data recorded in Lake Lothing (at the A47 Bascule Bridge) and in Oulton Broad (at Mutford Bridge) as shown on Figure 1-1. LiDAR levels were checked against topographic survey where possible and a good correlation was found, therefore the LiDAR was deemed suitable for use in this assessment. There have been no significant changes in the Lowestoft area since 2015 that would impact on the tidal dynamics, therefore the LiDAR flown in 2015 is deemed to be valid to represent the present day (2017) floodplain levels. The daily level data provided by the EA was analysed to determine the relationship between levels in Lake Lothing and Oulton Broad. Levels in Lake Lothing are higher than those on Oulton Broad as shown in Figure 2-1. This shows that the water level on Oulton Broad is mainly controlled by the water level of the River Waveney, which flows into Oulton Broad and not directly influenced by the water level in Lake Lothing. However, during high tidal events the lock at Mutford Bridge can be overtopped allowing water from Lake Lothing into Oulton Broad.

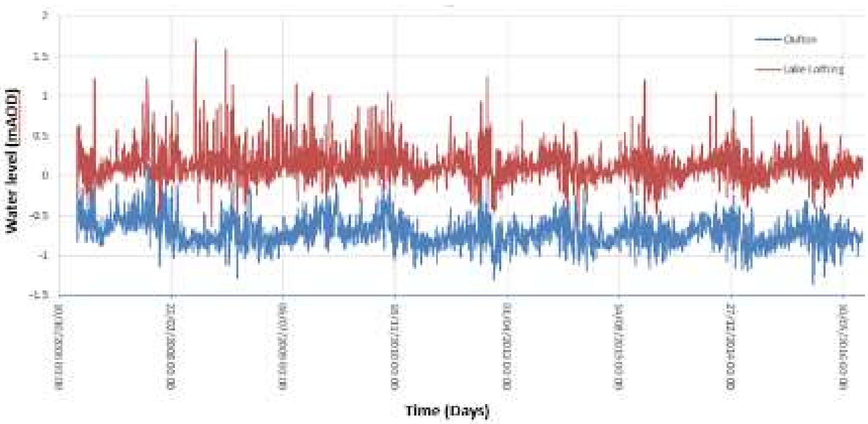

Figure 2-1 - EA level gauge comparison

[2] Open Coast (CFBD) Flood Risk Study, JBA, 2014

3 Hydrology

3.1 Overview

3.1.1 The hydrology of Lake Lothing has been analysed. Tidal levels have been derived to define the eastern boundary of the hydraulic model that represents sea levels along the Lowestoft coast. EA guidance on estimating design sea levels[3] has been used to derive the tidal boundary used in the model. Fluvial flows have been calculated on the three watercourses that discharge into Lake Lothing to allow the fluvial inputs to be included in the hydraulic model. Fluvial inflows to the model have been estimated following the EA Flood Estimation Guidelines[4]. This method has been used to specify an astronomical tidal profile on to which different scenarios are applied using the surge shape.

3.1.2 For this assessment scenarios based on the Lowest Astronomical Tide (LAT), Highest Astronomical Tide (HAT), an astronomical (typical tide) and a 5% AEP return period event to assess a likely extreme event have been investigated. The four design tidal profiles are used to simulate different likely ebb/flood tidal profiles in Lake Lothing. The four design events have been assessed for the present day (2017). This approach provides a recognised structured framework to produce tidal boundaries for this assessment. A 0.1% AEP event has been created for a high tide sensitivity test as reported in Section 4.3. No climate change event has been simulated because the water speed is based on the amplitude of the tidal curve which is not affected by climate change sea level rise. Rising sea levels cause significant out of bank flooding which is not the focus of this assessment.

3.1.3 A summary of the calculations undertaken to define the hydrological boundaries of the model is provided below with more detail provided in Section 2 Appendix A.

3.2 Tidal Curve Derivation

3.2.1 The EA guidance sets out a 10-step procedure to generate a tidal curve:

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; and

[3] SC060064/TR4: Practical Guidance Design Sea Levels and Open Coast (CFBD) Flood Risk Study (2014) JBA for the Environment Agency.

[4] Flood Estimation Guidelines Technical Report 197_08, Environment Agency, 2015

10. Apply allowance for climate change (This step has not been undertaken in this assessment as climate change has not been considered as explained in paragraph 3.1.3).

3.2.2 The procedure above makes use of several datasets which are provided as part of the guidance:

- Estuary Boundaries;
- ESLs from Open Coast (CFBD) Flood Risk Study, JBA 2014;
- Gauge Sites;
- Confidence Intervals; and
- Surge Shapes.

3.2.3 The tidal curve has been derived using the process set out in Section 3.2.1. As discussed in detail in Section 2 – Appendix A, the first four steps in the process make use of the datasets provided to obtain the required data for the site. The remaining steps require the manipulation of the data to obtain the tidal curve.

3.2.4 The procedure uses the available data to create an astronomical tidal profile. In the assessment it was deemed appropriate to use the tidal curve from the CH2M Hill existing model as only low-resolution data was available from the gauge; the tidal curve was scaled to the required peaks for the extreme sea level events and HAT. The LAT event has been created by scaling the trough to the LAT level and reducing the amplitude of the tide to create a profile akin to the small ebb/flood profile. The existing model tidal curve was scaled to the ESLs using the surge shape for Lowestoft provided with the guidance. This procedure is explained in detail in Section 2 – Appendix A.

3.2.5 Figure 3-1 shows the tidal curves that have been derived for use in this assessment.

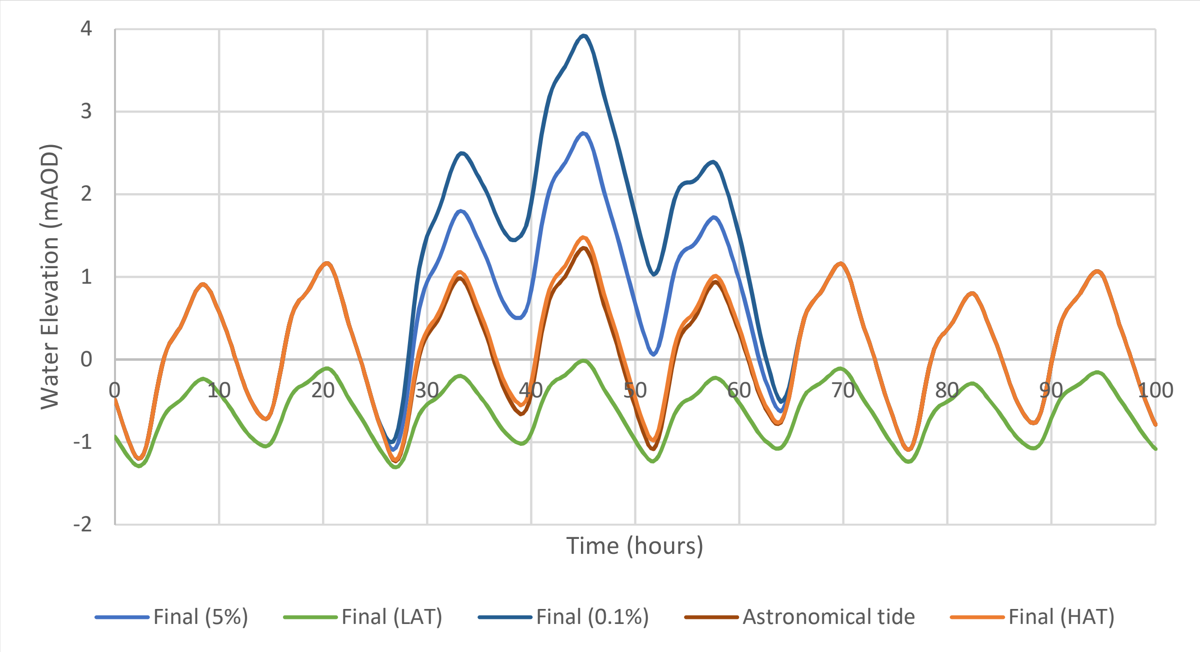

Figure 3-1 - Tidal curve for 5% AEP, LAT, 0.1% AEP, Astronomical tide and HAT present day events

4 Modelling Methodology

4.1 Overview

4.1.1 A 3D TUFLOW-FV model of Lake Lothing and the outer harbour has been developed for this assessment. Baseline and Scheme versions of the model have been created and other scenarios have been used to test the sensitivity of the model to a range of parameters. The model build for this study is detailed in Section 4.2. Section 4.3 describes the sensitivity testing undertaken on the model developed for this study and the outcomes of this. Section 4.4 describes the model verification process that has been undertaken.

4.1.2 TUFLOW-FV uses an unstructured grid to resolve the 3D flow characteristics of the water body. A 3D model can significantly increase the amount of information and the detail available to the project team. The model provides detailed water speed results at the seabed which can be used to assess particle transport in the model domain. This is beneficial when considering sediment transport as it is the water speed in the lower portion of the water column that drives which sediment particles are moved and how far they are moved before deposition.

4.1.3 The unstructured grid 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 small spatial areas, as for the Scheme, for example, around a support pier.

4.2 Model Build

Model Domain

4.2.1 The model domain extends from west of the A47 Bascule Bridge to the Lock at Mutford Bridge. Figure 4-1 shows the modelled extent. It was assumed that the worst case for the water speed impact will be before the water level exceeds the harbour walls therefore it was not considered necessary to include a large flood plain. The eastern boundary is 90m west of the A47 Bascule Bridge where the channel narrows and this was considered sufficient because the impact of the Scheme is not expected to extend past the narrow section of channel into the outer harbour, this has been discussed further in section 5. It is acknowledged that the outer harbour will create variations in flow due to eddies generated by the quays in the outer harbour, however due to the constriction of the A47 Bascule bridge and the resolution of the model it is unlikely any impact will be transmitted to the location of the Scheme. The extent of the domain has been set to accommodate the local out of bank flow from the 5% AEP event. The 5% AEP event is the largest event that has been simulated as part of the sediment assessment. A sensitivity test has been carried out assessing the impact of the glass walling effect[5] in larger events by simulating a 0.1% AEP event and the results are discussed in section 4.3.

[5] ‘Glass Walling’ is a common modelling term used to describe the situation when a simulated water extent reaches the edge of a model domain where no boundary has been set. When this occurs, the water will not pass through the boundary and will be impacted in the same way as it would if a wall were present in the domain.

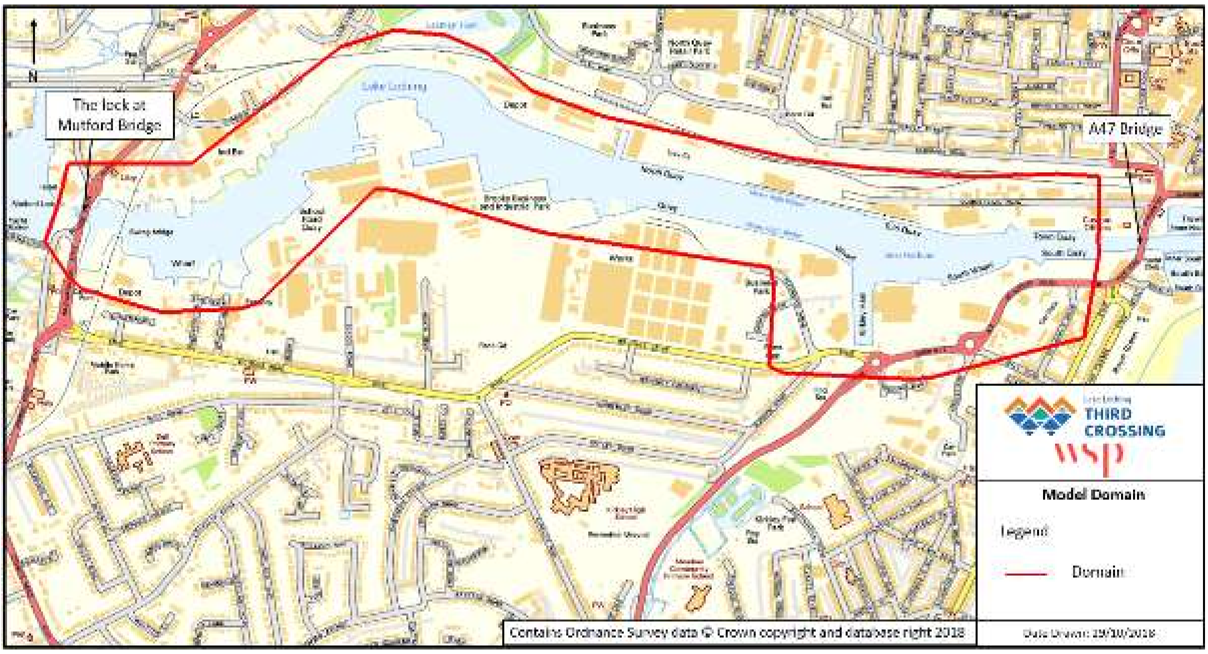

Figure 4-1 – Model Domain

4.2.2 The cell size through the domain is flexible and 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 around the Scheme at ultra-high resolution (approximately 1mx1m) to obtain the highest level of accuracy possible in the area where the impacts will be seen. The cell size in the upper lake near the lock and on the floodplain have been simulated at a lower resolution because detailed flow modelling is not required here. It has been ensured that the model grid is the same pre- and post Scheme to ensure that any changes between the models are as a result of the Scheme rather than changes in the model grid. The Scheme has been represented by increasing the local height of a number of cells. The individual piles are represented by increasing a cell where the pile is situated as shown in figure 4-7. An area of ultra-high-resolution cells has been used at the Scheme site in both baseline and Scheme simulations as shown in figure 4-2, the entire mesh across the domain has been shown in figure 4-3.

4.2.3 The benefit of the flexible mesh is that different sized polygons can be used. This means triangles and quadrilaterals can be used alongside each other, 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 however a visual check of all the outputs was carried out to ensure connectivity. Figure 4-2 shows that quadrilaterals fit well in the estuary channel therefore they have been specified in line with best practice.

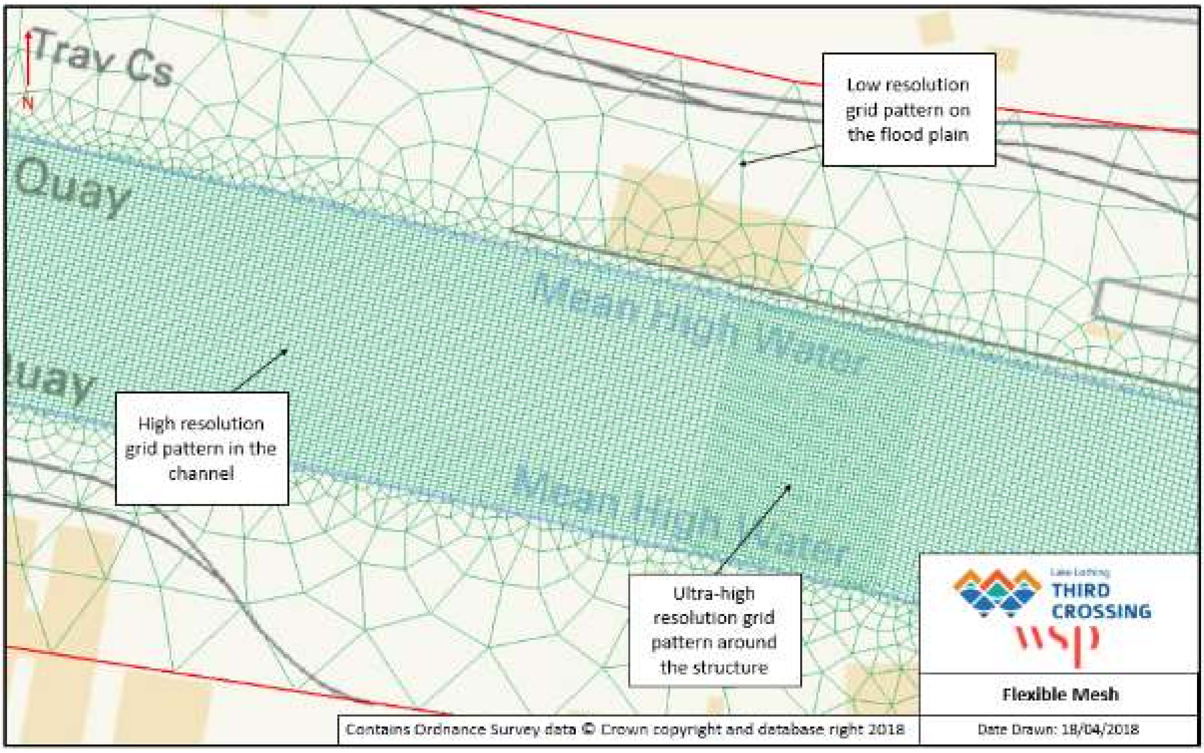

Figure 4-2 – Flexible mesh

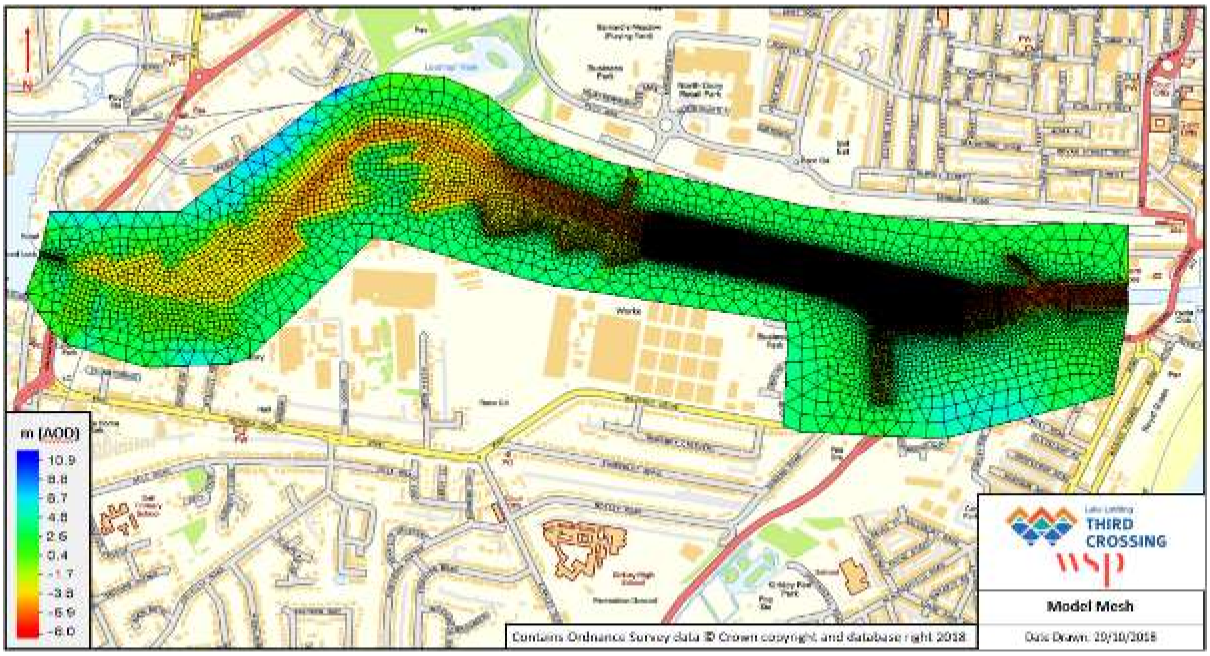

Figure 4-3 - Model Domain Mesh

4.2.4 The lock at Mutford Bridge has been simulated as closed for all scenarios modelled. It has been assumed that during high water events, the lock gates will remain closed. It has also been assumed that the impact on the Lake when the lock discharges water will remain the same after the Scheme is built as the Scheme does not impact the Lock. In addition, the impacts of the bridge piers on water speed are not expected to extend to the lock, this has been discussed in section 5.

Roughness Values

4.2.5 The roughness value in the water channel has been set to an initial value of 0.03 Manning's n based on engineering judgement representing a clean straight channel. The buildings on the floodplain have not been explicitly simulated in the model. The focus of this assessment is water speed within the Lake Lothing channel, therefore simulating the flow paths around individual buildings is not required. The floodplain roughness has been set to 0.05 to represent a general land value. Roughness sensitivity tests including +/- 20% an increase of roughness on the flood plain have been carried out to assess the sensitivity of the model to the Manning's values and the roughness values will be changed if required. This is reported in section 4.3.

Model Topography

4.2.6 The bathymetric data provided by ABP, once converted from CD to mAOD (see Appendix A) has been used to define the bed levels of Lake Lothing and the outer harbour within the model. The dataset recorded in spring 2016 consists of some 180,000 data points taken from a boat traversing the harbour. Towards Mutford Bridge, only the centre of Lake Lothing was included in the bathymetric survey as shown on Figure 4.4. In this area a lower resolution grid has been used, therefore the bed levels provided in the survey are sufficient for representing the channel in the model.

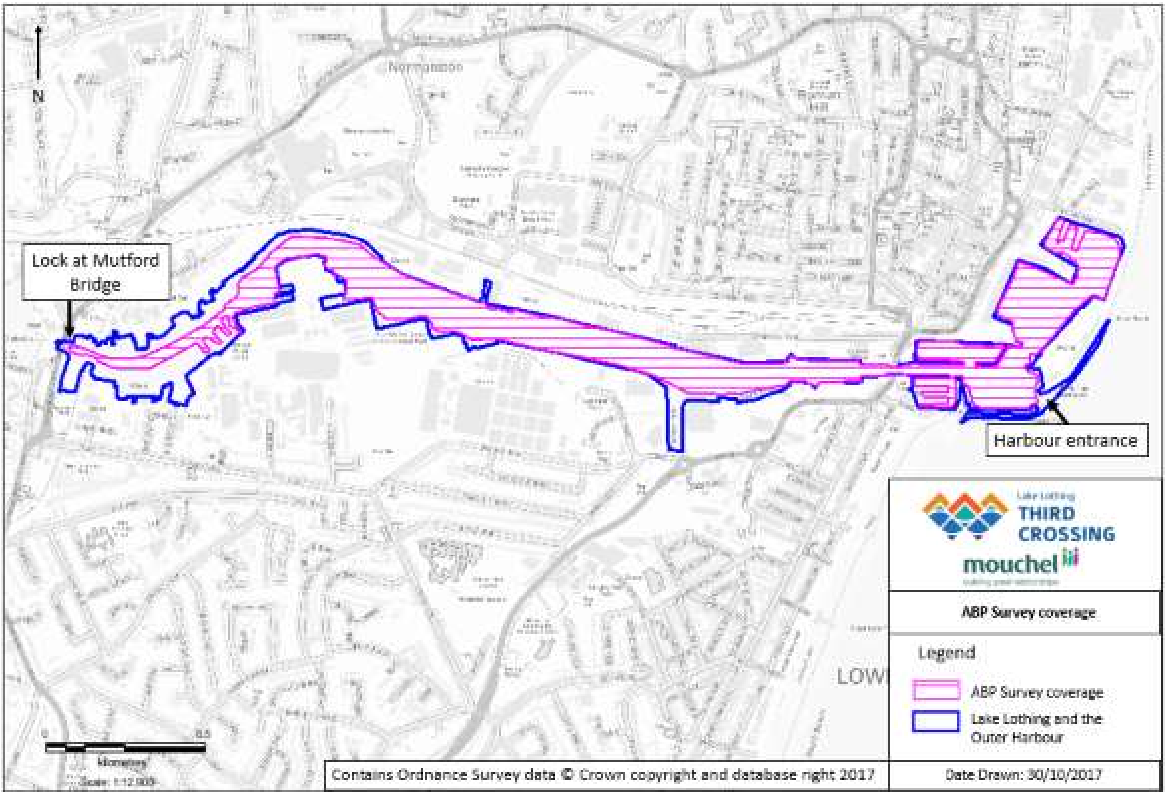

Figure 4-4 - ABP bathymetric survey coverage in Lake Lothing

4.2.7 LiDAR from the 2015 flight at 0.5m horizontal resolution has been used for the floodplain elevations. There is complete coverage of the 2D domain (Figure 4-1) using this dataset. The levels in the LiDAR dataset were checked against topographic survey collected by WSP on the south quay of Lake Lothing, the LiDAR shows the ground levels adjacent to Lake Lothing at approximately 3mAOD and this correlates with the survey data and the quay wall heights provided by the EA. In addition to the LiDAR levels, quay wall levels have been stamped to the domain to create the vertical walls of the harbour. Figure 4-5 shows the complete level data across the model domain.

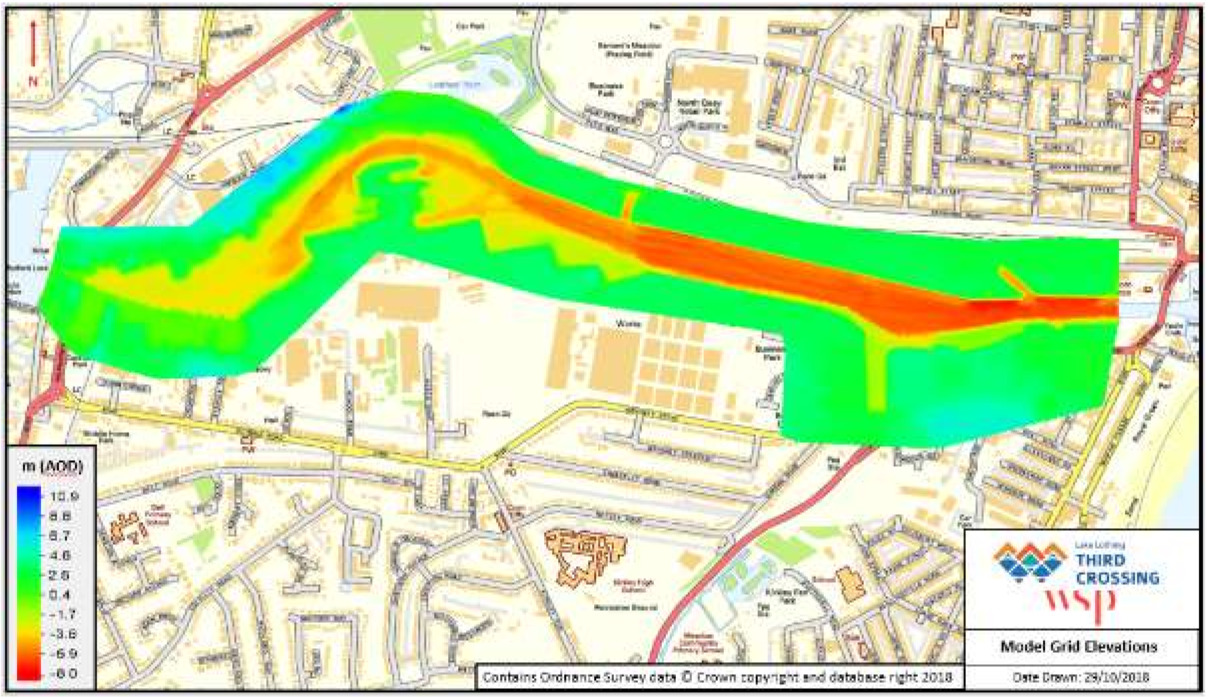

Figure 4-5 – Model Bathymetry

Boundary Conditions

4.2.8 The North Sea tidal boundary is located to the east of Lowestoft as shown on Figure 4-6. The tidal curves derived for this assessment as summarised in Section 3.2 have been applied to this boundary in the model. The tidal boundary is applied to the west of the A47 Bridge (Figure 4-1), whilst there is likely to be some losses due to the bridge and outer harbour, applying the boundary in this location represents a conservative scenario as the velocities simulated in Lake Lothing are likely to be higher than in reality. It is acknowledged that the outer harbour structures will create eddies which will result in energy loss. However due to the width of the constriction (approximately 22m), the length scale of the eddies created in the outer harbour will be smaller than the model can explicitly simulate through the A47 constriction.

4.2.9 The Lake is contained by the harbour walls and the lock at Mutford Bridge upstream therefore no other boundaries are required in this model. A sensitivity test has been carried out to assess the impact of the fluvial inflows within the model (section 4.3), the fluvial inflows have been applied as point inflows in the locations shown in Figure 4-6.

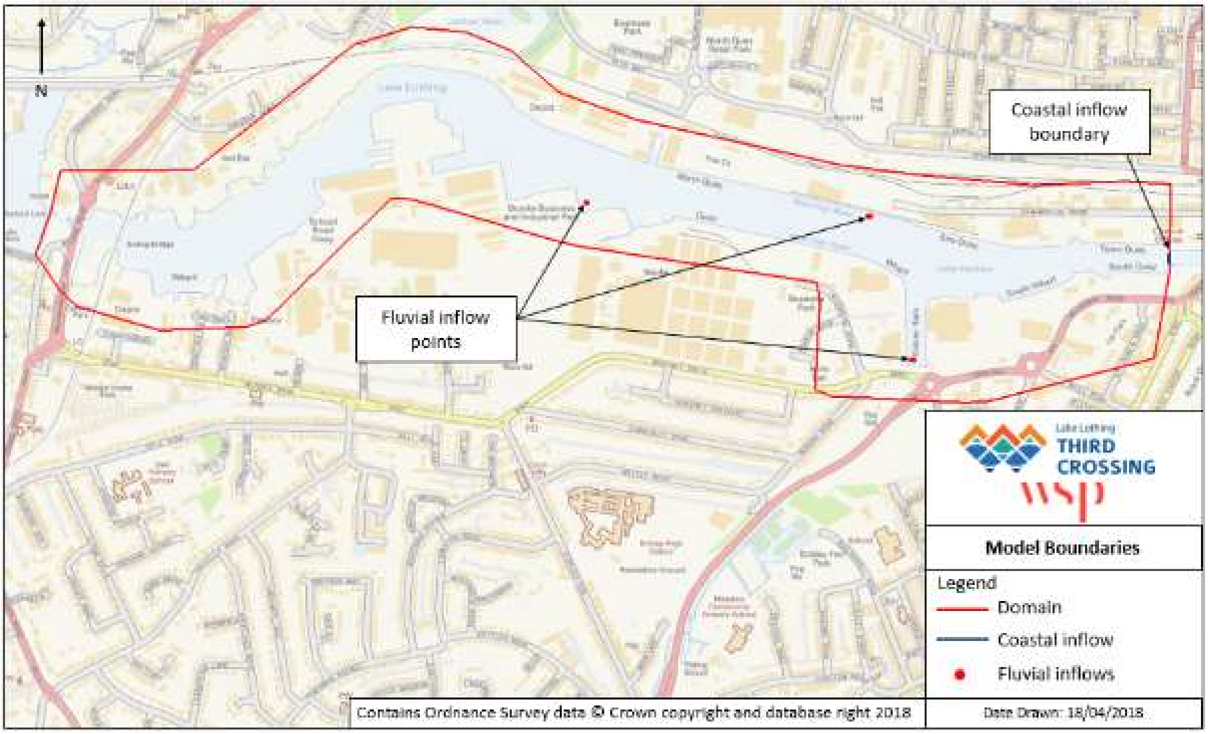

Figure 4-6 - Model boundary conditions

Initial Water Level

4.2.10 The initial water level in the model is set to -0.5mAOD across the entire 2D domain as this is the initial water level in the design tidal curves calculated as described in Section 3. Setting the initial water level to the same elevation as the start of the tidal boundary reduces the potential for model instabilities likely with sudden movements of large volumes of water in the model domain. This also reduces the need for an extended period of time to 'warm up' the model, reducing the overall simulation length.

Structures

4.2.11 There are no bridges simulated in the baseline model domain. The model domain starts to the west of the A47 Bridge therefore there is no need to simulate the bridge. The Mutford Bridge is not explicitly simulated because the resolution of the model in this area is not sufficient to accurately represent the structure and the speed is very low. It is considered unnecessary to directly simulate the impacts of Mutford Bridge because any impact on sediment from the bridge will not affect the Scheme site due to the distance between the two locations and the local velocity levels. The extent of the impact of the proposed crossing has been discussed in Section 5.

Baseline Model

4.2.12 Once the baseline model had been developed as described above, sensitivity tests were undertaken to determine the sensitivity of the model to various parameters as described in Section 4.3 below. Following the sensitivity testing, a Scheme version of the model was constructed as described below.

Scheme Model

4.2.13 To represent the Scheme in the model, the resolution of the grid around the Scheme is such that individual cells can be raised to represent each of the piles on which the bridge will be supported. The localised raising of the cell to the capping platform creates an obstruction in the lower levels of the water column. Due to the limitations of the model, the structural shape of the column cannot be modelled explicitly, i.e. piles, pile cap and bridge supports. To assess the sediment movements, the water speed on and near the bed are important. This is because the sediment is mobilised by shear forces generated between the water and river bed. These forces are present in the boundary layer between the bed material and the water velocity which is found in the lower portion of the water column. To that end, it was appropriate to explicitly simulate the piles at the river bed only within this assessment. The piles are represented up to 3mAOD meaning they are approximately 11m in height. In reality, the piles are circular and approximately 1m diameter, in the model they have been represented as approximately 1m x 1m squares. The pile coordinates have been taken from the reference design for the Scheme.

4.2.14 There is no representation of the Scheme on land because the largest return period simulated does not flood the approach roads. It was considered unnecessary in this model because the water does not breach the harbour walls in this location in the 5% AEP event. Figure 4-7 shows the representation of the piles in the model.

Figure 4-7 - Scheme Representation

3D representation

4.2.15 TUFLOW FV's 3D solver has been used to provide a detailed assessment of water flow in the channel. The model has been simulated using the hybrid z-sigma vertical discretisation option into 10 layers leading to an approximate layer thickness of 1m. The bed elevation at the location of the piles is approximately -8mAOD, therefore the water depth ranges from approximately 7m to 11m. An approximate 1m vertical resolution has been chosen as a balance between accuracy of the prediction, computational stability and model runtimes. This utilises sigma of layers in the shallow zones and equal spaced layers in deeper areas making the simulation more computationally efficient. In addition, the model automatically reverts to 2D when the water depth is less than 1m, i.e. on a large proportion of the flood plain. This is considered favourable for this application as details of the flood outlines are not required from the model. A comparison has been carried out to understand the difference in results between the 1m and 0.5m vertical resolutions. Typically, the 0.5m vertical resolution model results show a lower current speed when compared to the 1m resolution model. This however is only a maximum 0.01m/s reduction at peak flow. It is considered unnecessary to simulate the model at greater resolution and increase model run times in order to model such a small difference in current speed.

4.2.16 The 3D solver requires the model to use the baroclinic-barotropic mode splitting method and automatically sets the Courant-Friedrich-Levy (CFL) value to produce a stable simulation. The model uses a parametric method to solve the vertical mixing and the Smagorinsky method for horizontal mixing. This method provides a better approximation of the gradients through the water column than the TUFLOW default option (constant eddy viscosity/diffusivity method) and is the recommended approach when using the software for tidal applications.

4.3 Model Sensitivity Testing

4.3.1 The sensitivity of the model developed for this study to various parameters has been tested, the baseline model (described in Section 4.2) was used in the sensitivity testing and tests were carried out for the 5% AEP event. Testing is carried out on the 2D model as recommended by TUFLOW which states that the model should be built, tested and the global parameters set in 2D prior to moving the 3D. Sensitivity of the model to the following parameters has been tested:

- Bed Roughness;
- Building Roughness;
- Fluvial inflows; and
- Tidal levels.

4.3.2 Table 4-1 explains the versions of the model that have been developed for the sensitivity testing and how each of the parameters listed above has been changed to test their impact.

4.3.3 To assess the impact of the parameters listed in Table 4-1 on the model results, a number of points across the model domain have been selected as comparison points. At these points the depth averaged water speed results of the sensitivity tests have been compared to the baseline model results.

Table 4-1 - Sensitivity tests

Sensitivity Test	Purpose	Change
Baseline	Baseline model	N/A
Overall Roughness	To test the effect of the roughness of the domain	Change roughness across model by +/-20%
Floodplain Roughness	To test the impact of increasing the roughness on the floodplain outside of the channel to simulate a more densely packed urban environment.	Increase the roughness value on the floodplain outside of the channel only from 0.05 to 0.7.
River inflows	To test the impact of the fluvial river inflows on flood levels and extent	Fluvial inflows are applied in the 2D domain. 5% AEP fluvial and tidal events.
Extreme tide	To test the effect of the tidal boundary and the effect of glass walling at the model domain boundary.	The 0.1% AEP event was simulated through the model.
3D	To test the effect of simulating the model in 3D on the depth average velocity	Simulating the model using the 3D module.

4.3.4 Figure 4-8 shows the locations of the comparison points and Table 4-2 lists the locations and provides the coordinates for each point.

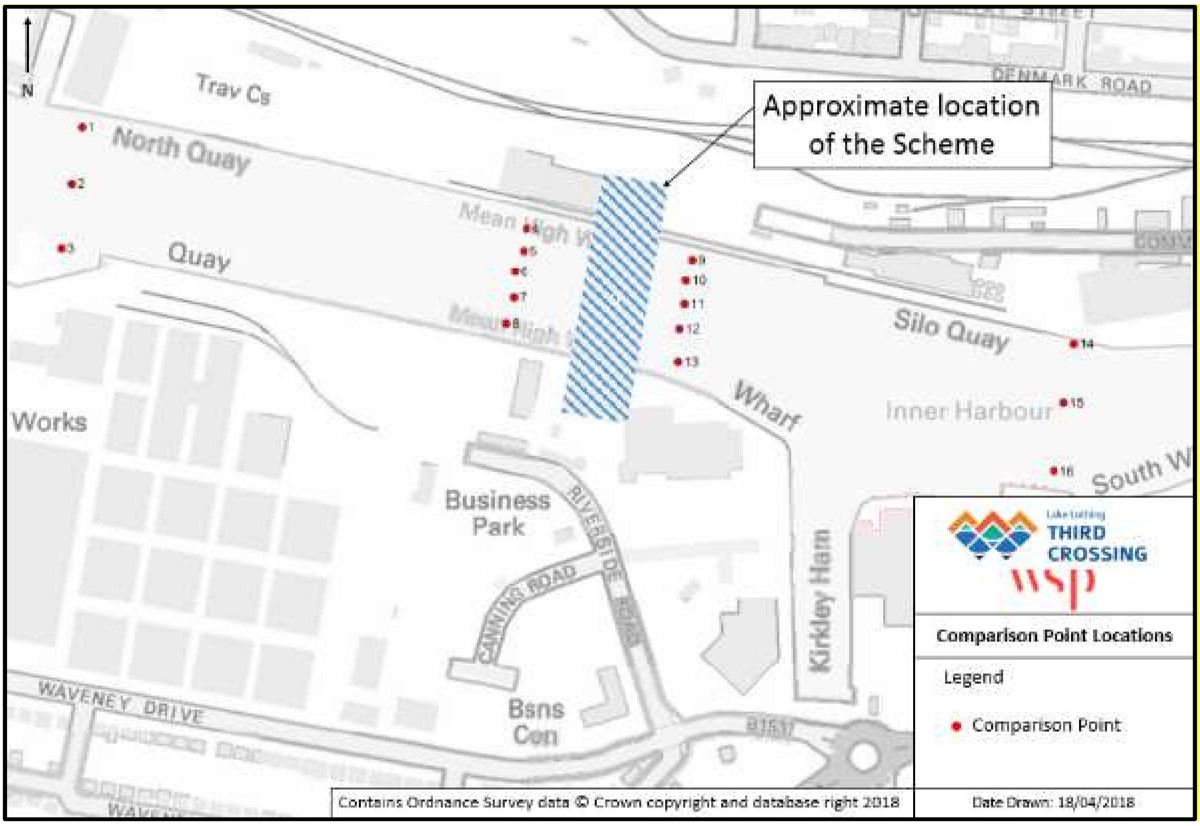

Figure 4-8 – Comparison points

Table 4-2 - Comparison point data

Point	Location	Easting	Northing
P1	Western far, North Quay	653490	292902
P2	Western far, Central	653482	292858
P3	Western far, South Quay	653474	292808
P4	Western close, North Quay	653834	292823
P5	Western close, North central	653832	292806
P6	Western close, central	655825	292790
P7	Western close, South central	653824	292770
P8	Western close, South Quay	653818	292750
P9	Eastern close, North Quay	653962	292799
P10	Eastern close, North central	653957	292783
P11	Eastern close, central	653956	292765
P12	Eastern close, South central	653952	292746
P13	Eastern close, South Quay	653951	292720
P14	Eastern far, North Quay	654258	292734
P15	Eastern far, Central	654250	292689
P16	Eastern far, South Quay	654242	292635

4.3.5 Table 4-3 shows the water speed for each sensitivity test rounded to the nearest 0.01m/s at each of the comparison points listed in Table 4-2 with the points adjacent to the Scheme in red text. It has been decided to show the results of the sensitivity tests at 41.25hr model simulation time because this is when the largest water speeds occur at most of the observation points. It should be noted that the velocities predicted in the model are inherently low due to the nature of the harbour.

Table 4-3 - Sensitivity Testing results, Depth Average Speed.

Comparison point	Baseline (m/s)	Roughness + (m/s)	Roughness - (m/s)	High Property Roughness (m/s)	Extreme Tide (m/s)	River inputs (m/s)	3D (m/s)
P1	0.05	0.05	0.05	0.05	0.07	0.05	0.05
P2	0.07	0.07	0.07	0.07	0.10	0.07	0.08
P3	0.03	0.03	0.03	0.02	0.04	0.03	0.02
P4	0.09	0.09	0.09	0.09	0.10	0.09	0.09
P5	0.06	0.07	0.06	0.06	0.08	0.06	0.06
P6	0.11	0.11	0.11	0.11	0.13	0.11	0.11
P7	0.07	0.07	0.06	0.07	0.09	0.07	0.07
P8	0.06	0.06	0.06	0.06	0.08	0.06	0.05
P9	0.12	0.12	0.13	0.12	0.14	0.12	0.13
P10	0.05	0.06	0.05	0.06	0.07	0.06	0.05
P11	0.10	0.10	0.10	0.11	0.13	0.11	0.10
P12	0.08	0.09	0.08	0.09	0.11	0.08	0.09
P13	0.06	0.06	0.06	0.06	0.08	0.06	0.05
P14	0.13	0.12	0.13	0.13	0.17	0.12	0.13
P15	0.13	0.13	0.14	0.13	0.18	0.13	0.13
P16	0.07	0.07	0.07	0.07	0.12	0.07	0.06

4.3.6 Figure 4-9 shows the variation in depth average speed throughout the model runs at comparison point 16 to show the impact of the sensitivity tests over time.

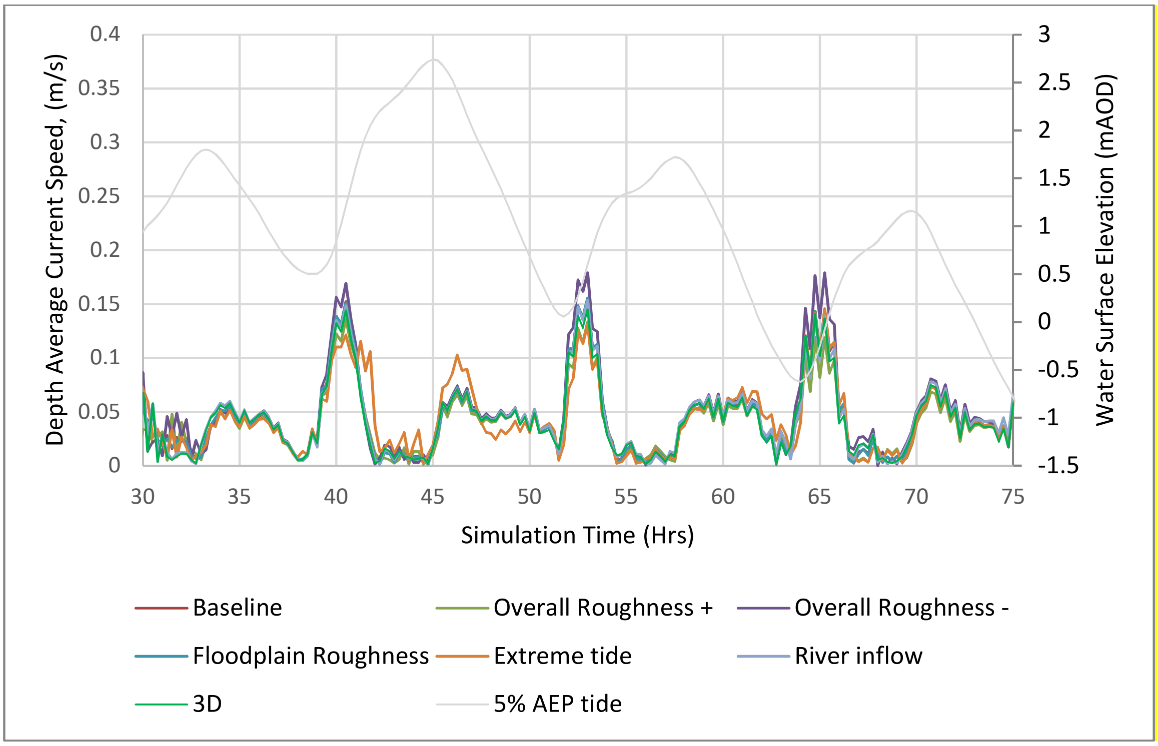

Figure 4-9 Depth Averaged Water Speed at comparison point 16

4.3.7 Following the sensitivity testing, it became clear that the model requires a period of time to stabilise at the start of the run. This is a common issue in tidal models and is caused by the oscillating tidal boundary. To that end, the first 5 hours of the model have been used as a spin up period and have not been considered when comparing the velocities.

4.3.8 The results show that the model water speed is sensitive to the amplitude of the tidal inflow curve as shown by the increase in water speed in the Extreme Tide event. This is because the large volume of water is being forced through a narrow constriction and not yet overtopping the harbour walls in the location of the Scheme.

4.3.9 The sensitivity testing shows that the fluvial inflows have no impact on the water speed predicted by the model, this is as expected as the harbour is tidally driven. As the river inflows have a negligible impact on the water speed, it has not been deemed necessary to include their representation in the final model developed for this assessment.

4.3.10 The sensitivity of the model to the overall roughness values has been tested. Neither an increase nor a decrease of 20% in the roughness values causes a significant change to the water speed predicted in Lake Lothing by the model. As such, the roughness values used as described in Section 4.2 are deemed appropriate.

4.3.11 The high floodplain roughness test shows that the model is not sensitive to the roughness value representing the floodplain. As such, it is deemed appropriate not to explicitly represent the buildings on the floodplain.

4.3.12 Table 4-3 shows that using the 3D option to simulate the water speed has very little impact on the depth average water results when compared to the Baseline simulation. This means the parameters chosen for the 2D simulation are suitable for use when modelling the watercourse in 3D.

4.3.13 The sensitivity testing has shown that the model is most sensitivity to the boundary conditions at the tidal boundary just after low tide when the speed is highest as shown in Figure 4-9, before the water level is sufficiently high enough to leave the channel and cause flooding. The model water speed is not sensitive to fluvial inflows because the harbour is predominately tidally driven. The sensitivity testing has also shown that the standard roughness values used are appropriate for use in assessing the bed water speed in the harbour.

4.4 Model Stability

4.4.1 The finite volume discretization technique is inherently stable and as such takes serious modelling errors to cause the model to crash. On one hand this is a useful quality for solving tidal flows due to the complex flow patterns, however this can hide bad model setup. As such, checking the model is important. The best measure of model mesh performance is to assess the minimum CFL number which sets the timestep of the simulation and the associated grid cell which is considered best practice. Tuflow-FV writes out a log file, which can be used to analyse the grid performance.

4.4.2 On reviewing the CFL numbers in the log file, none of the cells have a significantly lower value that any others with the smallest CFL number calculated to be 0.075. This means none of the cells are particularly smaller or deeper than any other which would increase computational time. This shows that the model grid is generally optimised across the whole domain and is computationally stable ready for use in this assessment.

4.5 Model verification

4.5.1 As there is limited information available for a calibration, it was decided to verify the model by simulating the 2013 flooding event and investigate the water levels in the domain. Additionally, anecdotal information provided by the Harbour Master suggests that the water speed is very low and controlled by the narrow constriction of the A47 Bascule Bridge.

4.5.2 The event chosen for model verification was the 2013 tidal surge event in Lowestoft between the 5th and 6th December. The event caused widespread flooding due to a tidal surge in the North Sea. The surge, combined with the high tide tracked down the east coast of England causing damage to properties near the coastline. Due to the size of the 2013 event, and as it occurred relatively recently, there is a good amount of data and anecdotal evidence from the EA for the flood event.

4.5.3 The level data provided by the EA for this assessment at the Lowestoft gauge is daily averaged levels, therefore the NTSLF website was checked and showed that the peak high tidal level was approximately 4.75m CD (3.25mAOD) during the 2013 event. Figure 4-10 shows the NTSLF gauge data at the time of the 2013 tidal surge event. The graph shows the water elevation in chart datum. The conversion to mAOD from chart datum in Lowestoft is -1.5m. On the chart the red line represents the predicted tidal curve and the blue dots represent the recorded data at the gauge site.

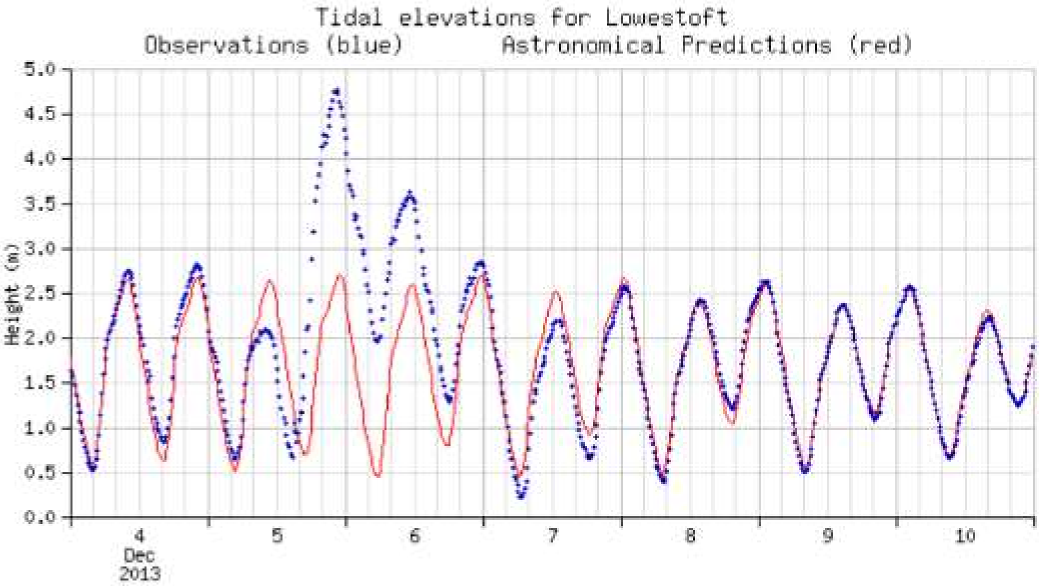

Figure 4-9 - Levels (mCD) at Lowestoft Gauge. (National Tidal and Sea Level Facility website, extracted 2016)

4.5.4 Figure 4-11 shows the tidal curve derived in mAOD for the 2013 event used in the model to simulate the 2013 event. The curve was initially created by scaling each peak of the astronomical tide to the corresponding peak in the NTSLF data set. Further scaling was required to simulate the 2013 event in the model to ensure the recorded water level of 3.25mAOD at the gauge site was achieved. The tidal curve simulated at the eastern tidal boundary of the model was scaled to a peak of 3.35mAOD for the verification run as this ensured that the level of 3.25mAOD was achieved at the gauge site within the model. With the tidal inflow scaled to a peak of 3.25mAOD, the water level at the gauge site (on the eastern side of the A47 Bascule Bridge) was predicted to be too low compared to the level recorded during the 2013 event.

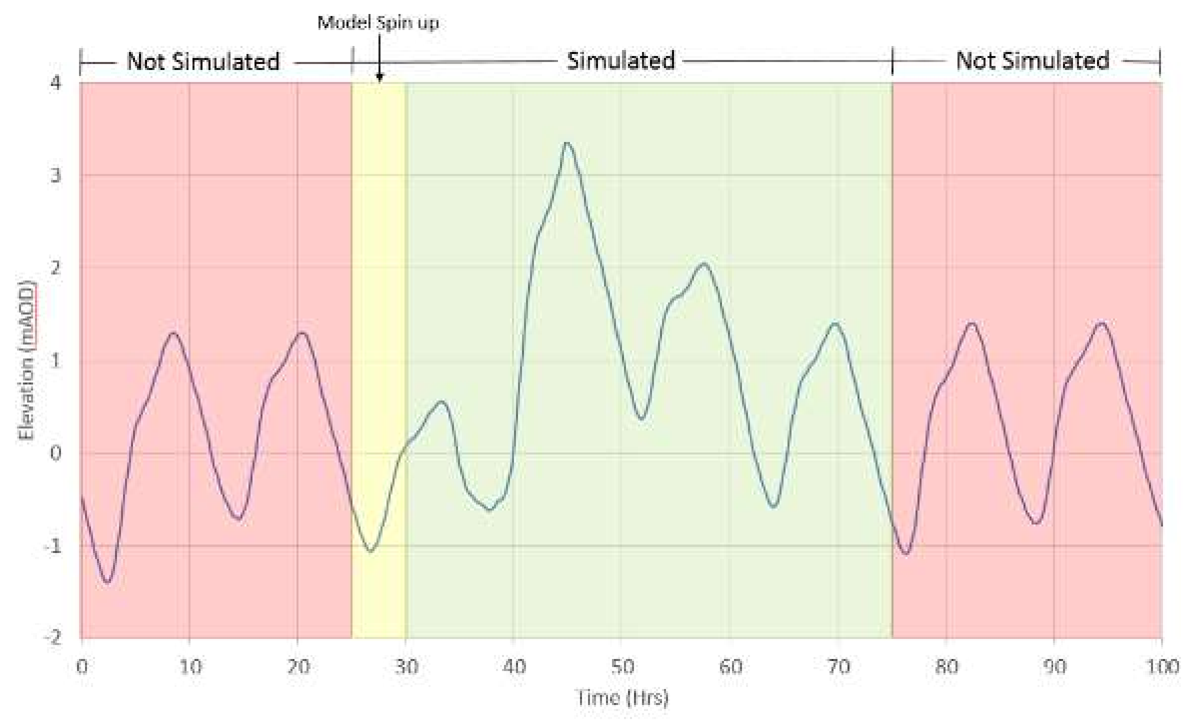

Figure 4-10 - 2013 Event Tidal Curve

4.5.5 The model has been simulated for 50 hours from 25h-75h to simulate the peak tidal event as shown on Figure 4-11. Figure 4-12 shows the water level at the 16 comparison points compared to the inflow event tidal curve. The figure shows the model can retain the expected water level through Lake Lothing. The water speed is driven by the amplitude of the tidal curve, Figure 4-13 shows the water speed at observation point 6. The graph shows that the maximum water speed is the 0.25m/s (2.d.p). This value is in line with the anecdotal evidence provided by the local harbour staff which states that the water speed is very low in the harbour its self.

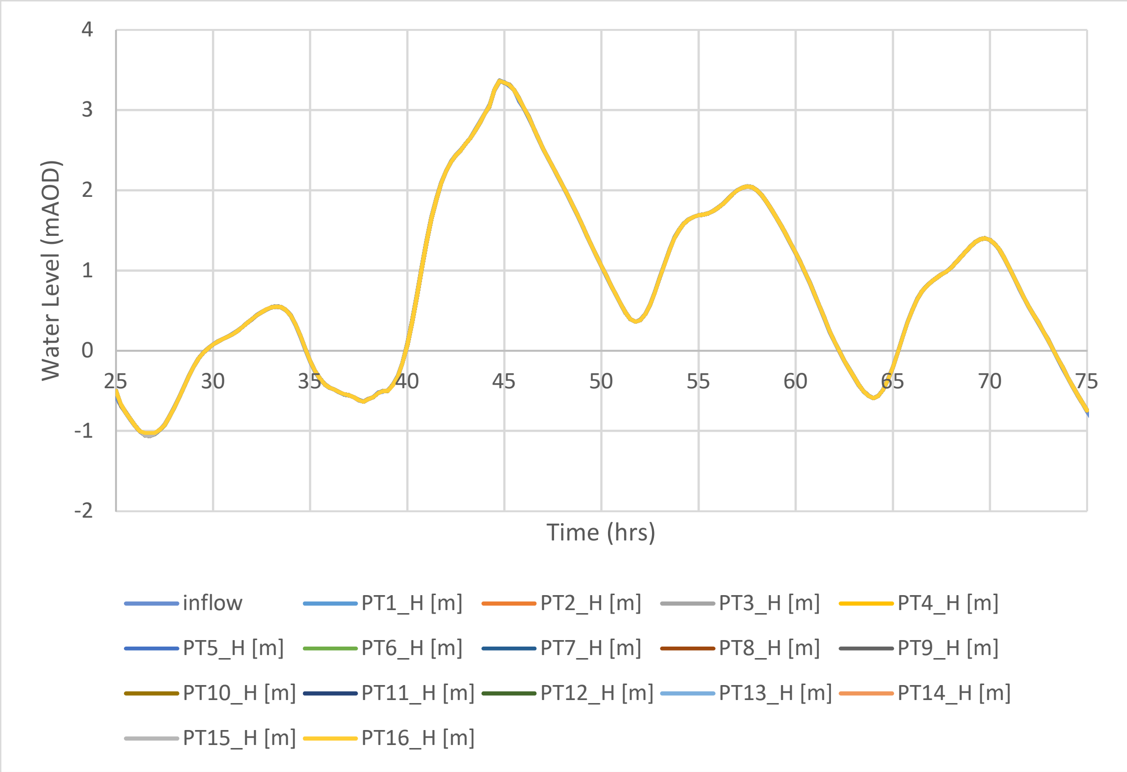

Figure 4-11 - Water level timeseries comparison

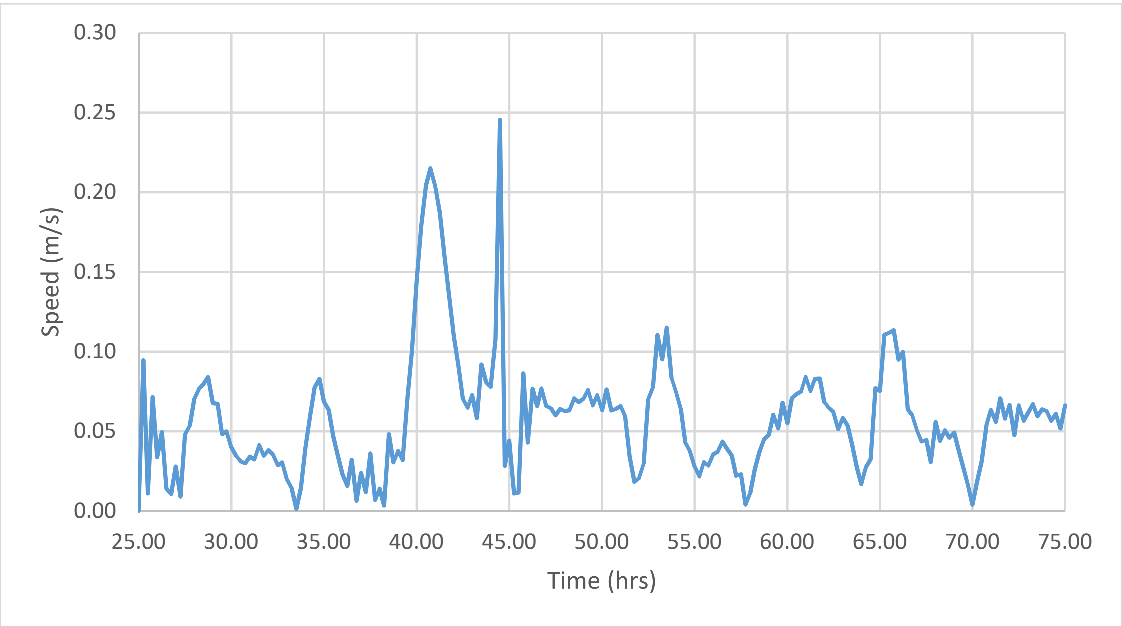

Figure 4-12 – Water Speed timeseries at comparison point 6

4.5.6 In conclusion, the sensitivity testing and stability checks have been carried out and shown that the initial parameter values chosen in Section 4 are suitable for use in this assessment and the model is not particularly sensitive to initial conditions because of the spin-up time. It is considered best practice to calibrate/verify the model in 2D initially to confirm the model parameters are suitable for use before converting to 3D for the scenario simulations.

5 Hydraulic Modelling Results

5.1 Model Runs

5.1.1 A total of eight 3D model runs have been undertaken as part of this assessment, four return periods have been simulated. Table 5-1 shows all the model runs that have been undertaken as part of this assessment. As discussed in section 4, the model has been simulated for a total of 50 hours and the first 5 hours have been used to spin up the model to produce stable results. The central part of the tidal curve has been used, therefore the results show 30h-75h and include 2 full tidal cycles.

5.1.2 No climate change events have been simulated for this assessment because the water speed is a function of the tidal curve amplitude and the rate of change of water elevation and not a global sea level rise. In addition, there is not a major fluvial water source that could contribute to the current speed. Therefore, a climate change simulation would result in similar water speeds and would not provide any further useful information.

Table 5-1 - Model Simulations

Scenario	Return Period	Present Day 2017
Baseline	LAT	Baseline_003_3D_LAT
	HAT	Baseline_003_3D_HAT
	Astro	Baseline_003_3D_Astro
	5% AEP	Baseline_003_3D
Scheme	LAT	Scheme_003_3D_LAT
	HAT	Scheme_003_3D_HAT
	Astro	Scheme_003_3D_Astro
	5% AEP	Scheme_003_3D

5.1.3 A sediment survey (May 2018, see Appendix 12B/APP-192) has been carried out in Lake Lothing to determine the sediment particle size at the bed. This showed the spatial particle distribution ranges from 0.002mm (Clay) to 0.02mm (Medium Slit), however the majority of the domain is in the 0.002mm to 0.003mm range. Figure 5-1 shows a range of particle sizes found in estuaries from silt to sand (0mm to 2mm), this shows the particle size found in Lake Lothing are at the lower end of the range.

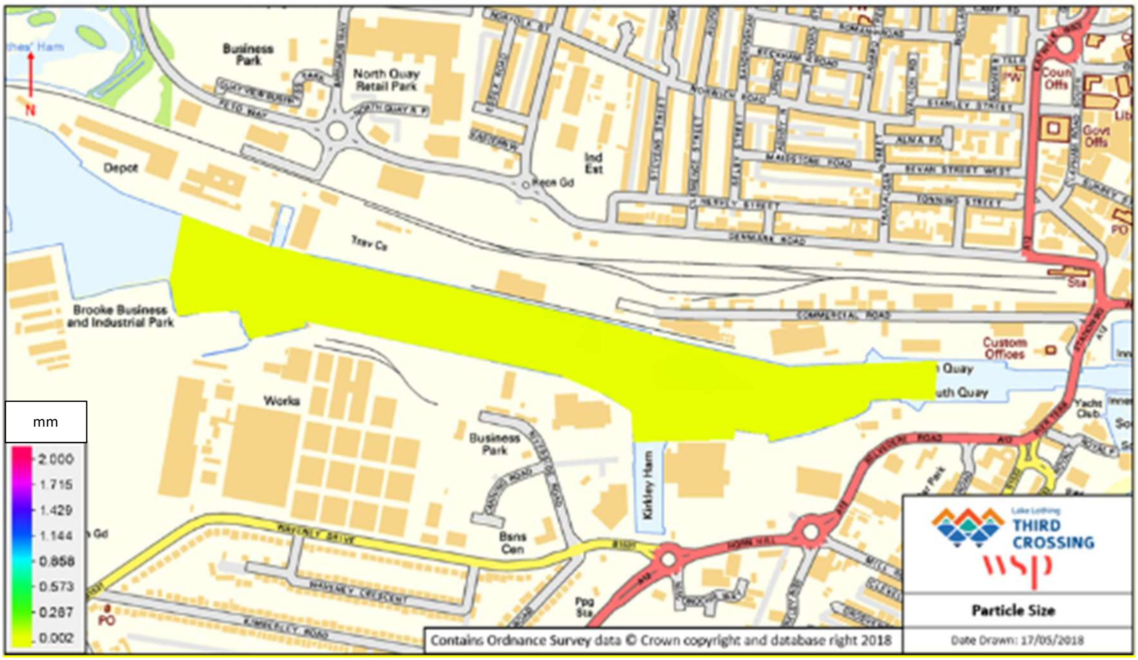

Figure 5-1 – Particle size distribution

5.1.4 To assess the impact of the Scheme on sediment transport, an analysis of the sediment particle size and change in water speed has been carried out using The Hjulström Curve diagram (Figure 5-2). The curve plots the particle size against the water speed of the watercourse and plots erosion, transport and deposition zones according to the cohesive forces. It is important to note that the diagram plots water speed in cm/s where as in this assessment water speed has been considered in m/s which is in line with other assessments for the Scheme. In section 5.2, the timeseries plots show the erosion and deposition zones for typical sediment sizes found at the comparison points.

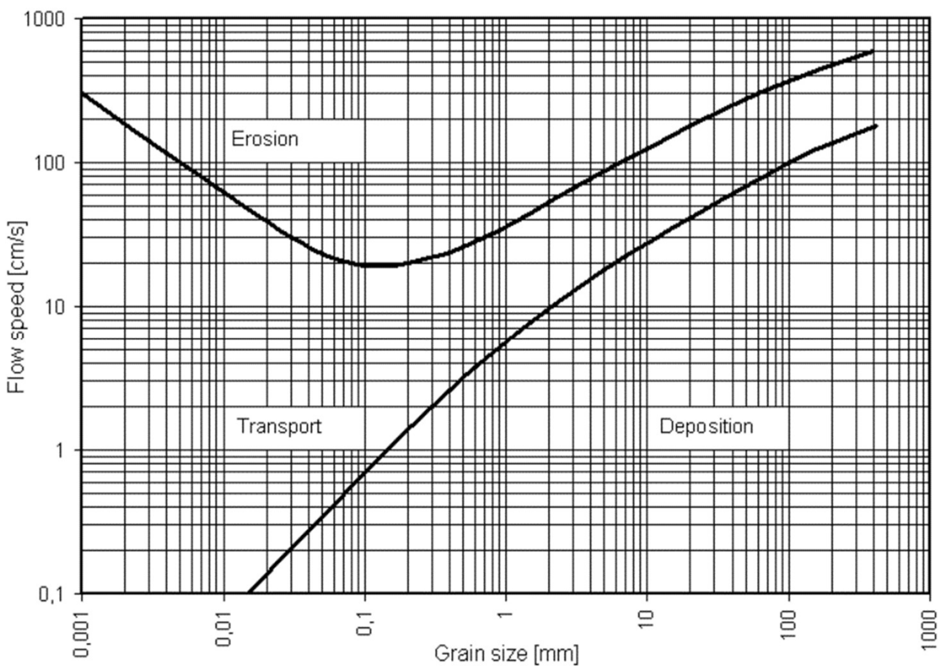

Figure 5-2 - Hjulström Curve

5.2 Results

5.2.1 The 3D model presented in section 4 has been used to simulate the water speed in Lake Lothing and the velocity at the bottom 1m of the watercourse has been investigated. This method has been used to obtain a detailed approximation of the bed water speed within Lake Lothing. 1m vertical resolution provides sufficient accuracy for this assessment and was chosen as a balance between numerical accuracy, model run time and results resolution.

Existing Sediment Regime

5.2.2 The harbour requires periodic dredging to prevent siltation over time and a dredging programme is in place. The speed of the water around the Scheme is typically within the 0.06-0.08 m/s range. When looking at the Hjulström Curve (Figure 5-2), and comparing the typical sediment particle size of 0.003mm a current speed of approximately 1m/s is needed to resuspend sediment. The current speed is not sufficient to resuspend sediments already within the harbour however the speed and particle size is within the 'transport' area of the Hjulström Curve (Figure 5-2). Taking into account the anecdotal evidence provided by the Harbour Master of speeds and the need for dredging each year, this increase in sedimentation must come from external sources. As the flow upstream of Lake Lothing is controlled by a lock gate which is highly unlikely to be the main source of the sediment, the incoming tide must contain suspended sediments picked up outside of the study area and carried in on the incoming tide.

Impact of the Scheme

5.2.3 In order to determine the impact of the Scheme, a comparison has been made of the current speed between the baseline and Scheme models for each tidal scenario modelled.

LAT (low ebb/flood) Scenario

5.2.4 The purpose of this scenario is to show the impact of the Scheme in a low ebb/flood tidal event. The tidal amplitude is significantly smaller than the other scenarios causing the current speed to be lower than other scenarios. Figure 5-3 shows the current speed in m/s for the LAT scenario at the time of the highest recorded speed at point 15 (53.25hrs) for the baseline and

Scheme simulations. Comparison point 15 was chosen because is it the closest point to the boundary in the centre of the channel.

Figure 5-3 - LAT Scenario, Baseline and Scheme Current Speed.

5.2.5 Figure 5-3 shows the baseline speed in the domain is low, showing around 0.1 to 0.13 m/s near the inflow boundary to the east and reduces significantly in a short distance to less than 0.1 m/s. When comparing the speed to the Scheme simulation, the piles have a small local impact strengthening two flow channels to the centre and north of the channel. This is because the piles have created a small localised constriction between the piles and harbour walls.

5.2.6 As part of the simulation, v and u velocity timeseries for 1m above the seabed have been extracted at 16 locations in the domain shown in figure 4-8 and processed to show the current speed. These points have been chosen to provide a numerical comparison between the models at locations immediately upstream and downstream of the scheme and further away from the scheme along the centreline of the channel.

5.2.7 Figures 5-4, 5-5, 5-6 and 5-7 show the timeseries plots for points 2, 6, 11 and 15 respectively. Each figure shows the Baseline and Scheme current profiles, the LAT scenario tidal boundary and the erosion/deposition threshold speed for the sediment particle size at the comparison point.

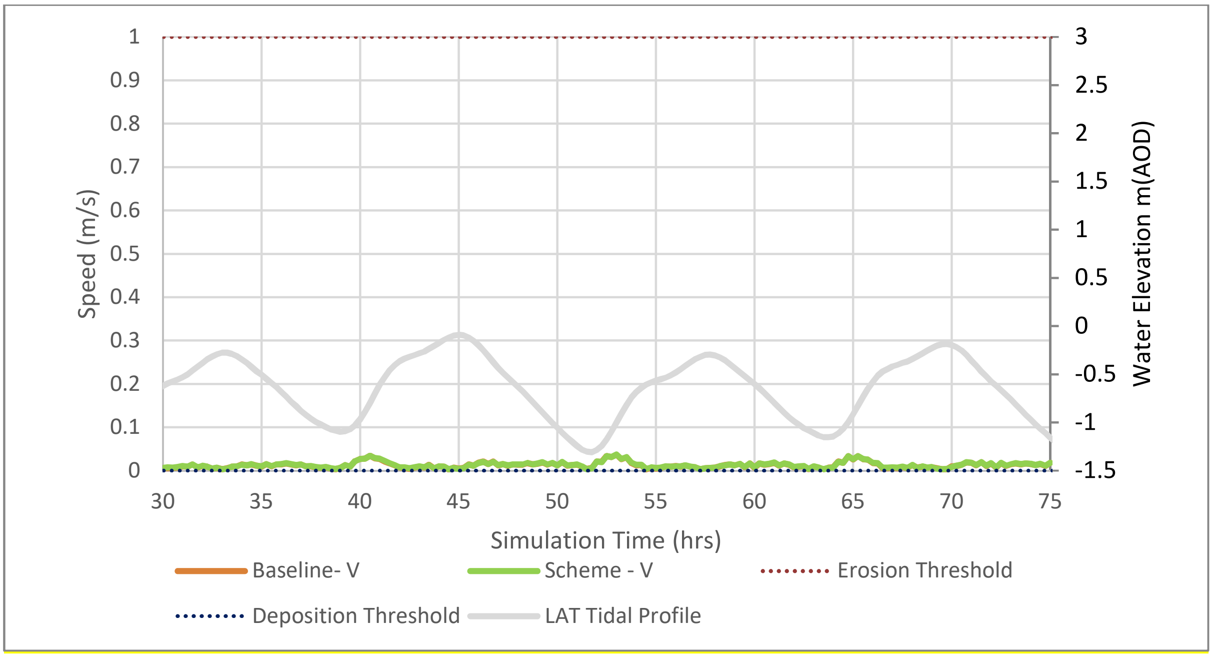

Figure 5-4 - LAT Scenario, Point 2

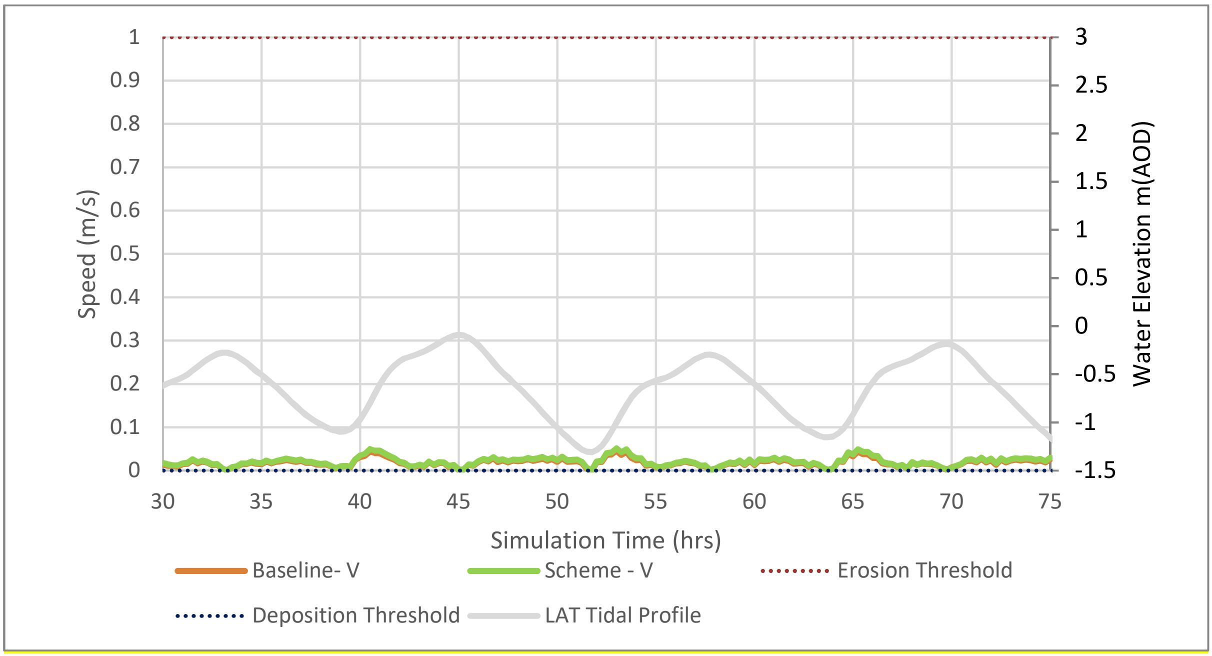

Figure 5-5 - LAT Scenario, Point 6

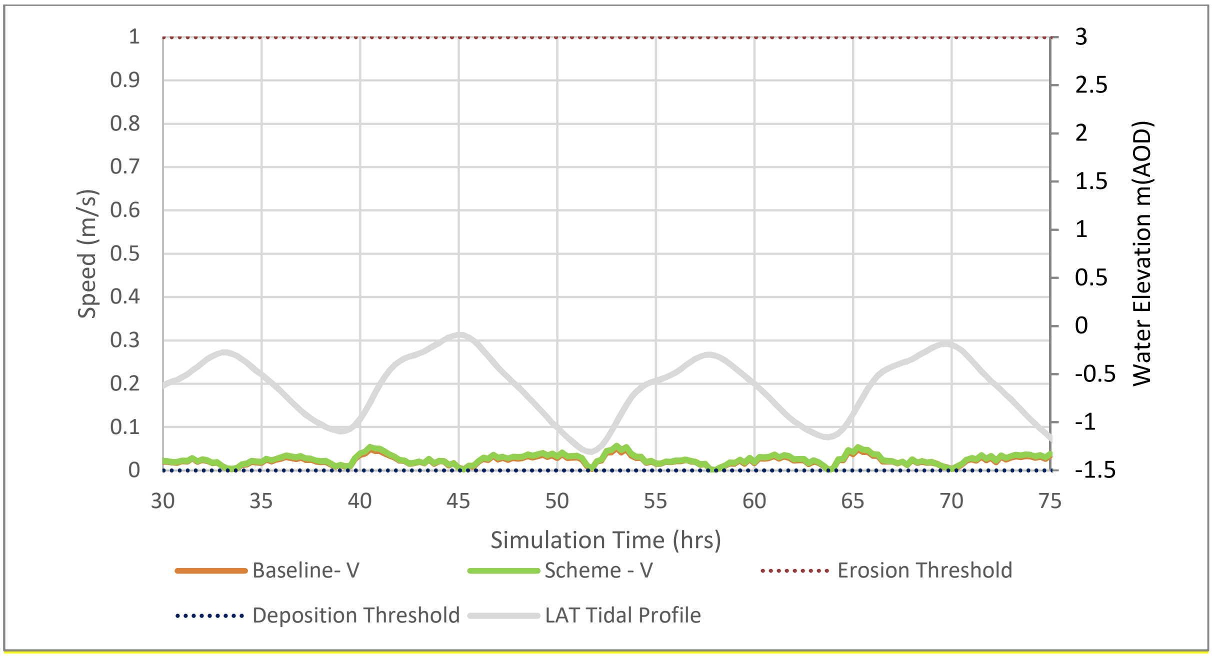

Figure 5-6 - LAT Scenario, Point 11

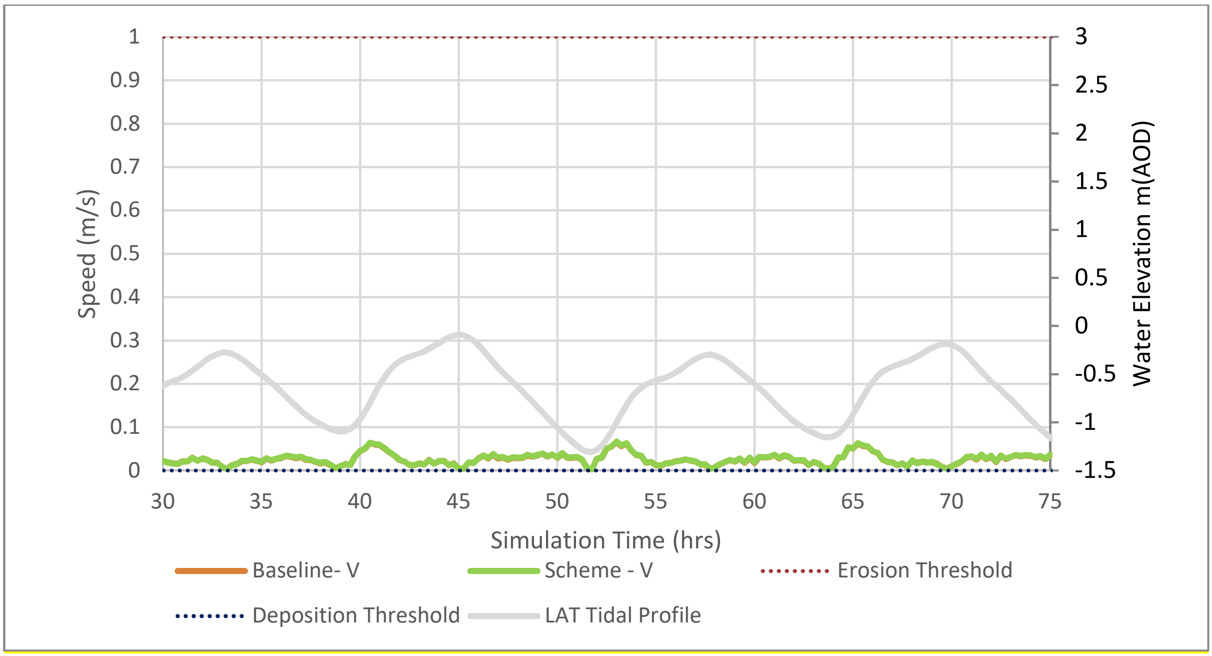

Figure 5-7 - LAT Scenario, Point 15

5.2.8 Figures 5-4, 5-5, 5-6 and 5-7 all show the current speeds are not sufficient to cause erosion at any point in the simulation in the Baseline and Scheme simulations. The figures show that the Scheme has very little impact on the speed at the centre of the channel, a slight increase in velocity is seen when the baseline speed is largest however this does not increase the current sufficiently to cause additional erosion. The figures show that the highest speeds are seen on the flood tide which is where the steepest part of the tidal profile is seen. This shows that in the absence of a significant fluvial inflow the current speed is driven by the rate of change of water elevation.

5.2.9 In addition to the figures showing the current speed along the centreline, Figure 5-8 shows the timeseries for comparison point 5. Point 5 is positioned directly upstream of one of the groups of piles (Figure 4-8) and shows the greatest change in current speed at any of the comparison points.

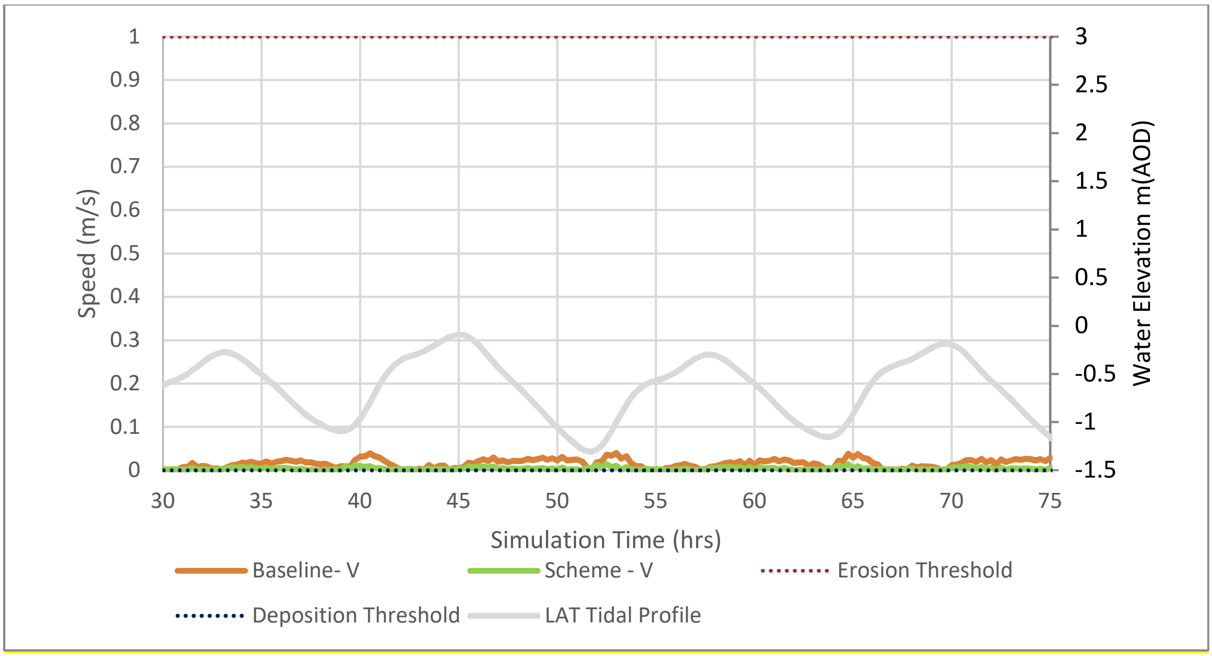

Figure 5-8 - LAT Scenario, Point 5

5.2.10 Figure 5-8 shows the current speed reduces significantly immediately upstream of the piles. This is because of the wake effect from the piles. The deposition threshold for the particle size found at point 5 is 0m/s, i.e. when the tide changes. To that end, whilst the piles do cause a measurable reduction in current speed there is not a change in regime.

5.2.11 In conclusion, the low ebb/flood tide scenario (LAT Scenario) shows that at low current speeds, the Scheme does not have a sufficient impact on current speeds to change the current sediment regime across the domain.

Astronomical Tide Scenario

5.2.12 The purpose of this scenario is to show the impact of the Scheme in an astronomical ebb/flood tidal event based on the tidal curve shape to a peak between the MHWS and HAT at the Lowestoft gauge. Figure 5-9 shows the current speed in m/s for the Astronomical tide scenario at the time of the highest recorded speed at point 15 (52.75hrs) for the baseline and scheme simulations. Comparison point 15 was chosen because is it the closest point to the boundary in the centre of the channel.

Figure 5-9 - Astronomical Tide Scenario, Baseline and Scheme Current Speed

5.2.13 Figure 5-9 shows the baseline speed in the domain is low, showing around 0.1 to 0.3 m/s near the inflow boundary to the east and reduces significantly in a short distance to less than 0.13 m/s. When comparing the speed to the Scheme simulation, the piles have a small local impact strengthening two flow channels to the centre and north of the channel. This is because the piles have created a small localised constriction between the piles and harbour walls. The wake can be seen upstream and downstream of the piles and extends approximately 400m upstream and 400m downstream.

5.2.14 Figures 5-10, 5-11, 5-12 and 5-13 show the timeseries plots for points 2, 6, 11 and 15 respectively. Each figure shows the Baseline and Scheme current profiles, the Astronomical scenario tidal boundary and the erosion/deposition threshold speed for the sediment particle size at the comparison point.

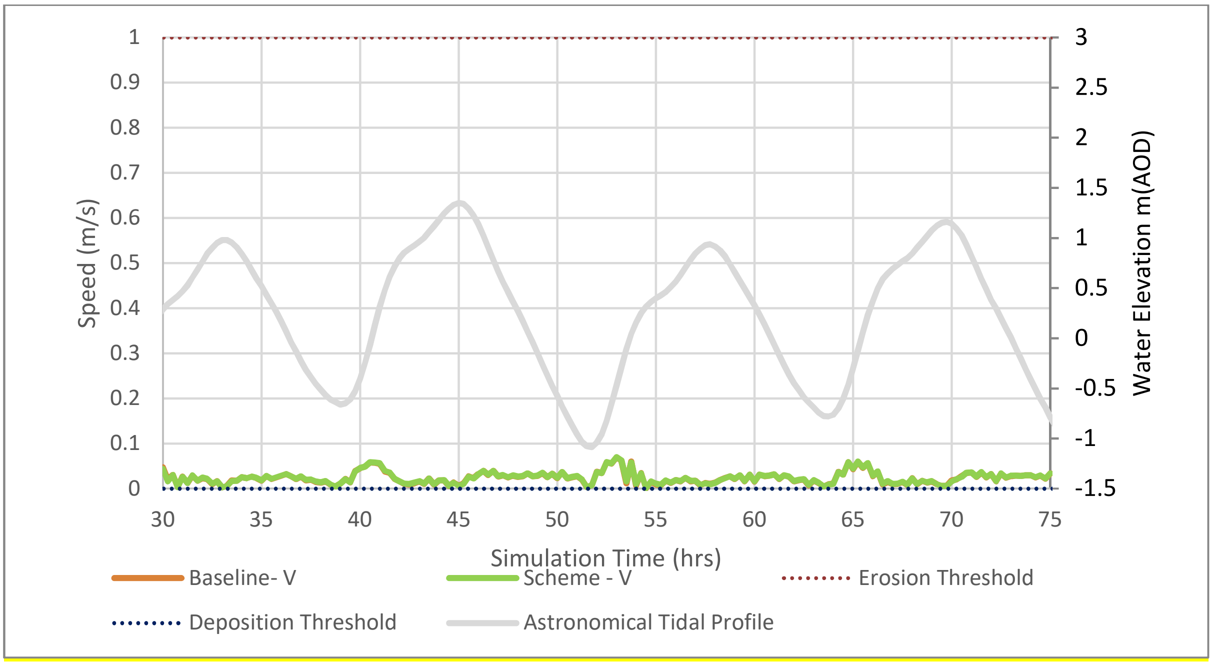

Figure 5-10 - Astronomical Tide Scenario, Point 2

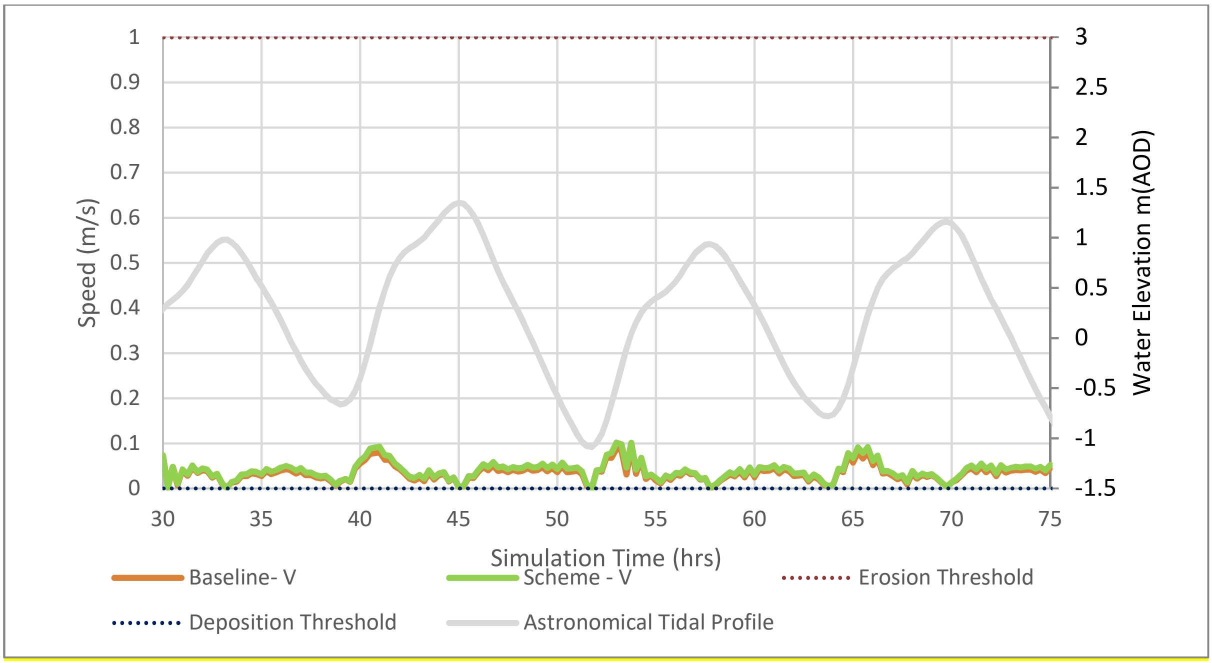

Figure 5-11 - Astronomical Tide Scenario, Point 6

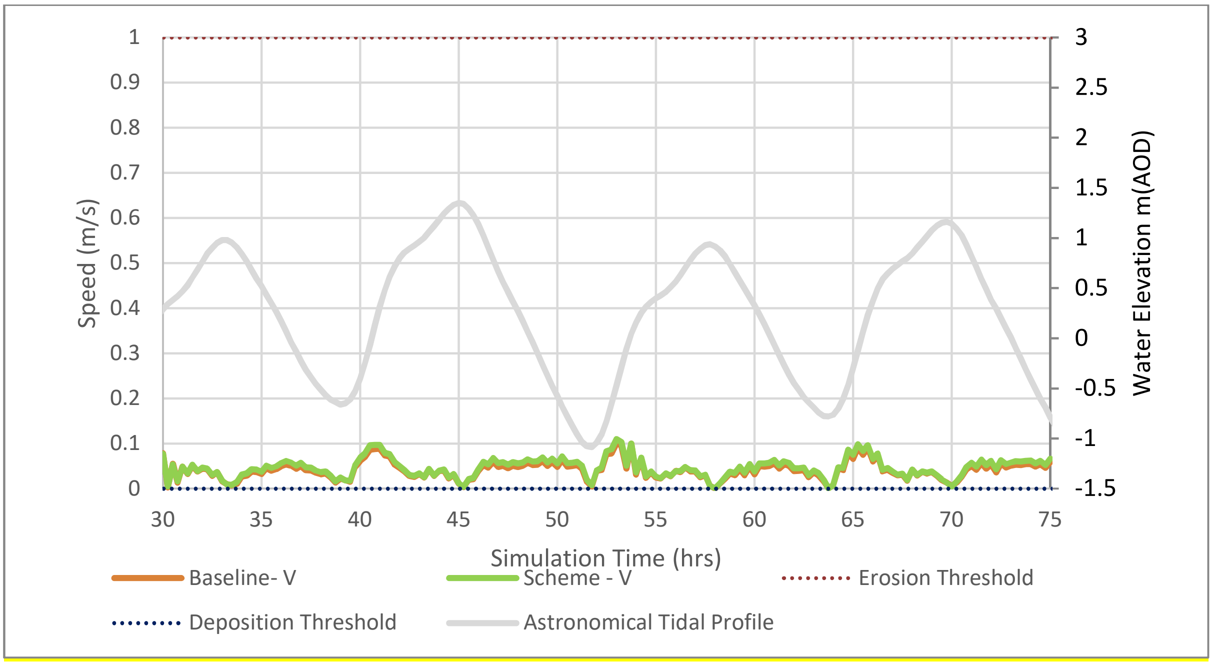

Figure 5-12 - Astronomical Tide Scenario, Point 11

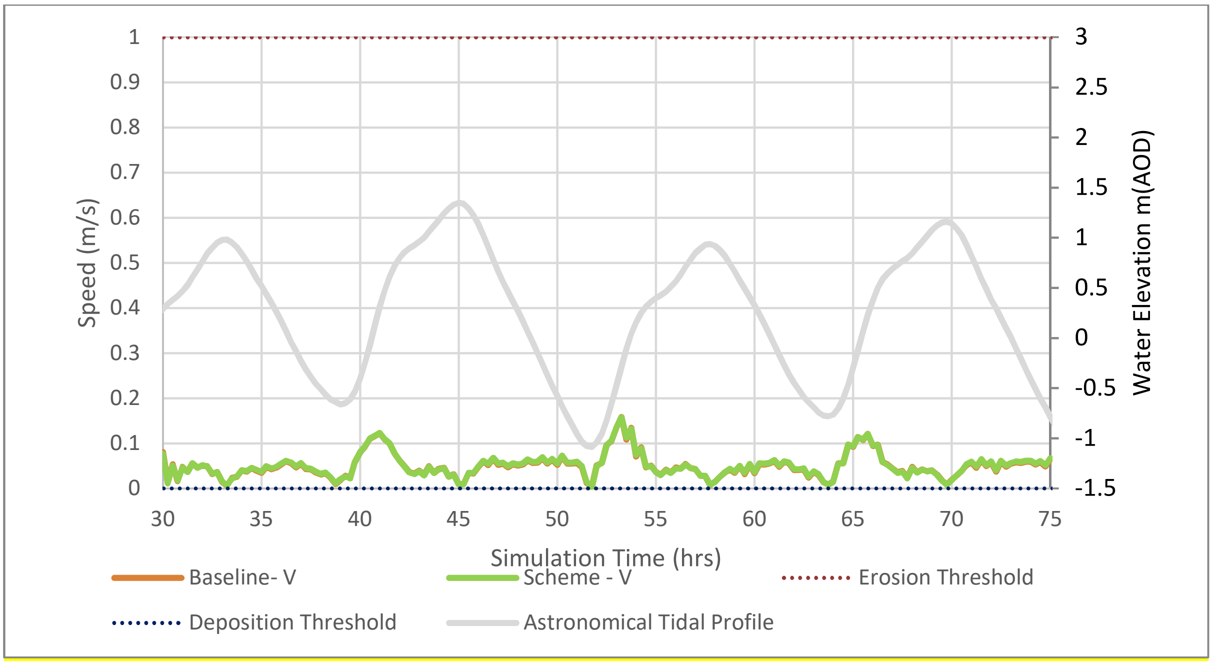

Figure 5-13 - Astronomical Tide Scenario, Point 15

5.2.15 Figures 5-10, 5-11, 5-12 and 5-13 all show the current speeds are not sufficient to cause erosion at any point in the simulation in the Baseline and Scheme simulations. The figures show that the Scheme has a small localised impact close the piles and this impact is reduced further away. The increase in velocity is not significant enough to change the sediment regime in the area. The figures show that the highest speeds are seen on the flood tide which is where the steepest part of the tidal profile is seen. This shows that in the absence of a significant fluvial inflow the current speed is driven by the rate of change of water elevation.

5.2.16 In addition to the figures showing the current speed along the centreline, Figure 5-14 shows the timeseries for comparison point 5. Point 5 is positioned directly upstream of one of the groups of piles (Figure 4-8) and shows the greatest change in current speed at any of the comparison points.

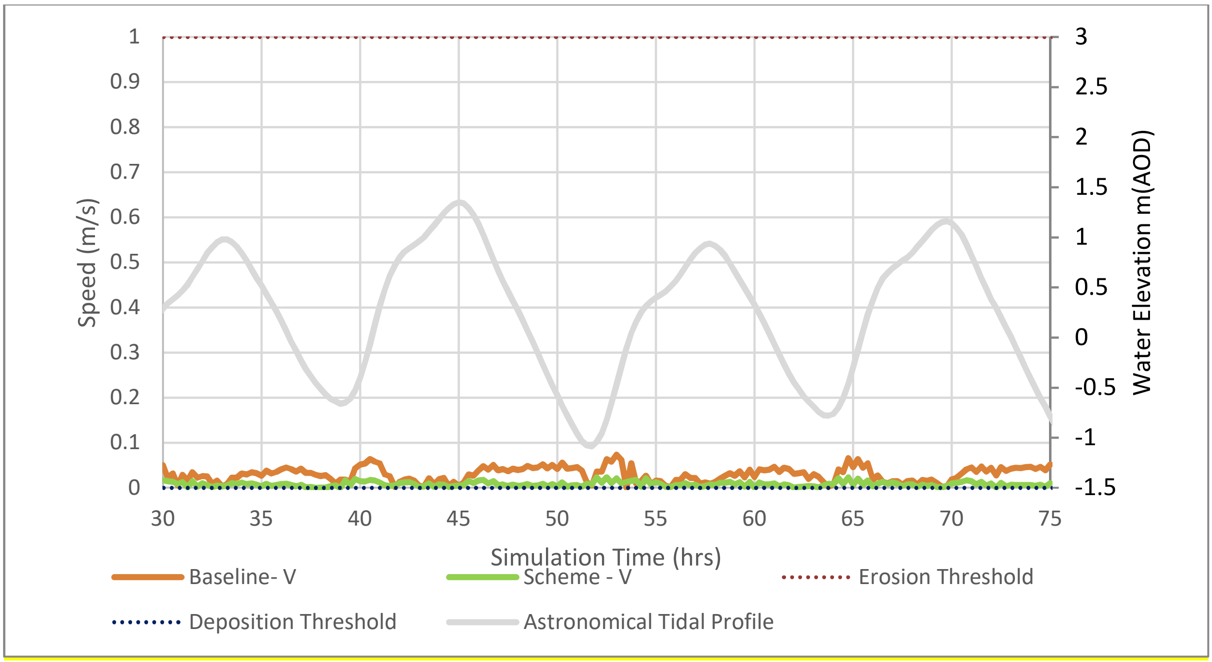

Figure 5-14 - Astronomical Tide Scenario, Point 5

5.2.17 Figure 5-14 shows the current speed reduces significantly immediately upstream of the piles. This is because of the wake effect from the piles. The deposition threshold for the particle size found at point 5 is 0m/s, i.e. when the tide changes. To that end, whilst the piles do cause a measurable reduction in current speed there is not a change in regime.

5.2.18 In conclusion, the astronomical ebb/flood tide scenario shows that in an expected current speed scenario, the Scheme does not have a sufficient impact on current speeds to change the current sediment regime across the domain.

HAT (High ebb/flood) Scenario

5.2.19 The purpose of this scenario is to show the impact of the Scheme in a high ebb/flood tidal event. Figure 5-15 shows the current speed in m/s for the HAT scenario at the time of the highest recorded speed at point 15 (53.5hrs) for the baseline and scheme simulations. Comparison Point 15 was chosen because is it the closest point to the boundary in the centre of the channel.

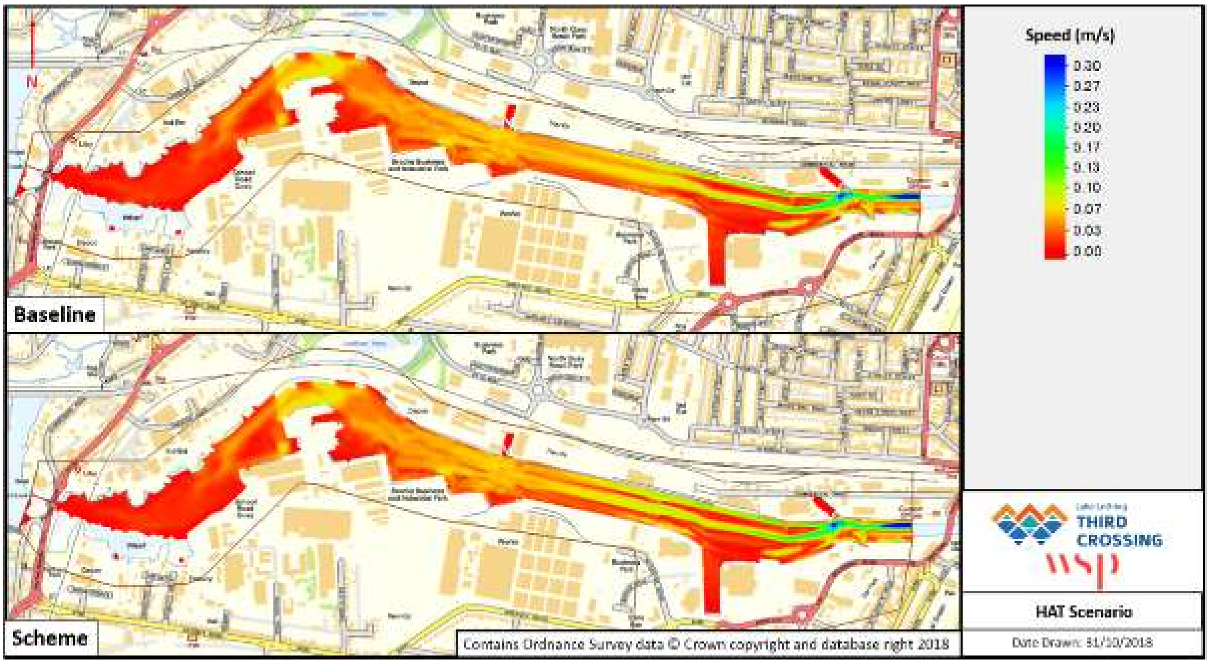

Figure 5-15 - HAT Tide Scenario, Baseline and Scheme Current Speed

5.2.20 Figure 5-15 shows the baseline speed in the domain is low, showing around 0.1 to 0.3 m/s near the inflow boundary to the right and reduces significantly in a short distance to less than 0.13 m/s. When comparing the speed to the Scheme simulation, the piles have a small local impact strengthening two flow channels to the centre and north of the channel. This is because the piles have created a small localised blockage caused a constriction between the piles and harbour walls. The wake can be seen upstream and downstream of the piles and extends approximately 400m upstream and 400m downstream.

5.2.21 Figure 5-16, 5-17, 5-18 and 5-19 show the timeseries plots for points 2, 6, 11 and 15 respectively. Each figure shows the Baseline and Scheme current profiles, the HAT scenario tidal boundary and the erosion/deposition threshold speed for the sediment particle size at the comparison point.

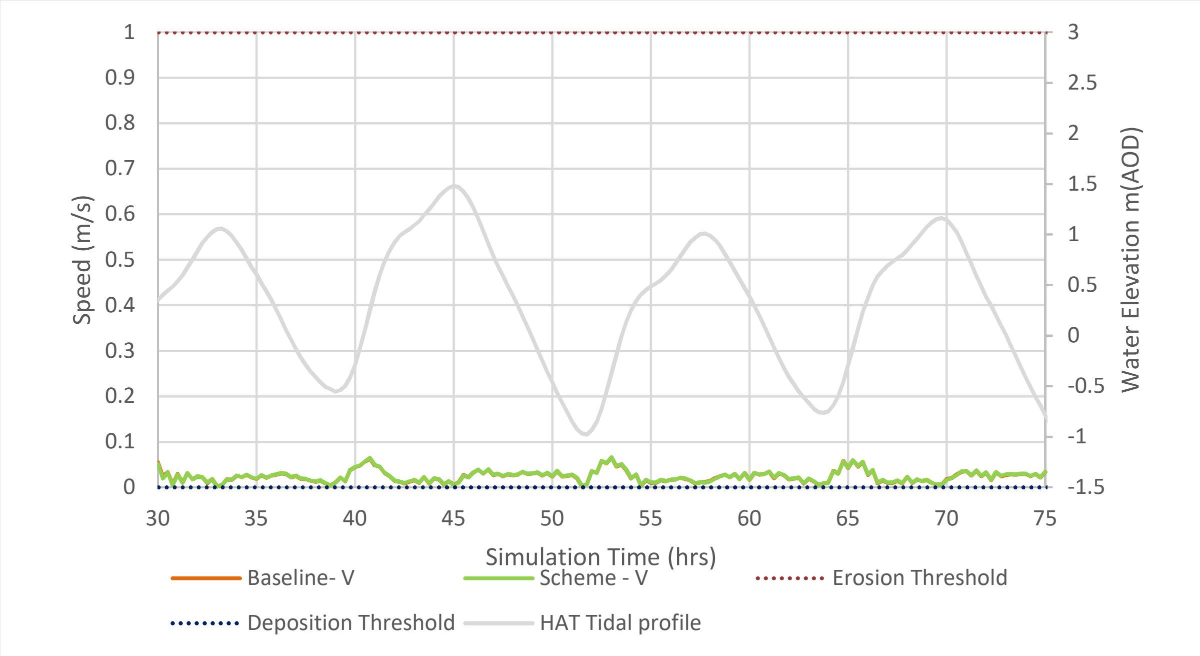

Figure 5-16 - HAT Tide Scenario, Point 2

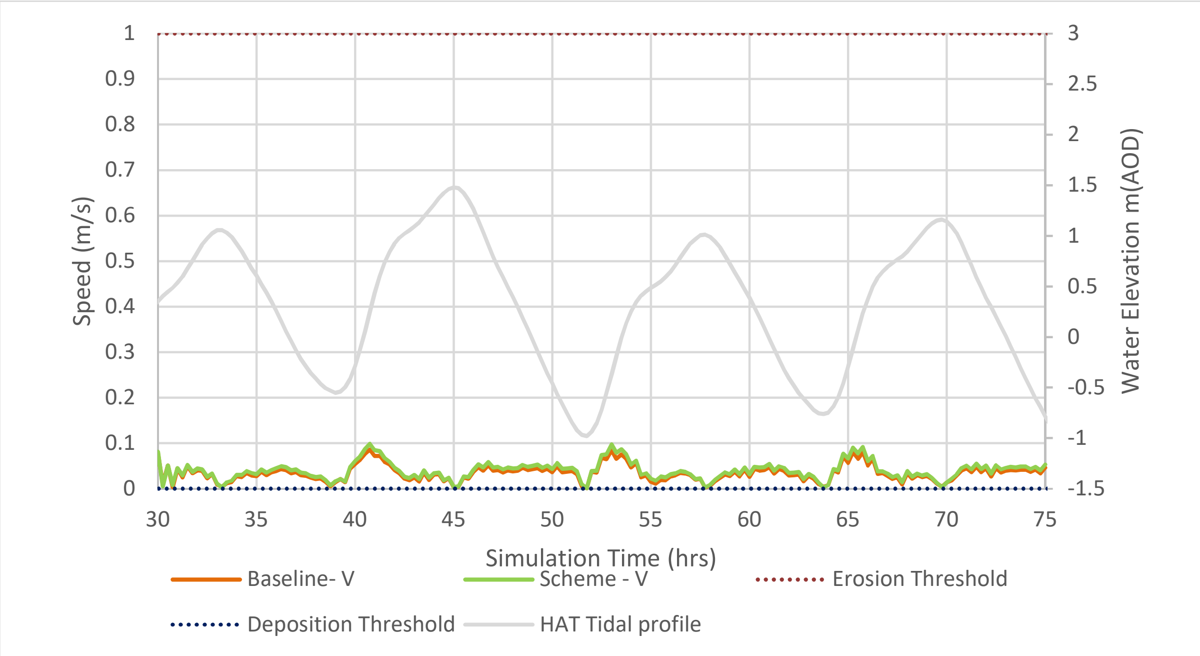

Figure 5-17 - HAT Tide Scenario, Point 6

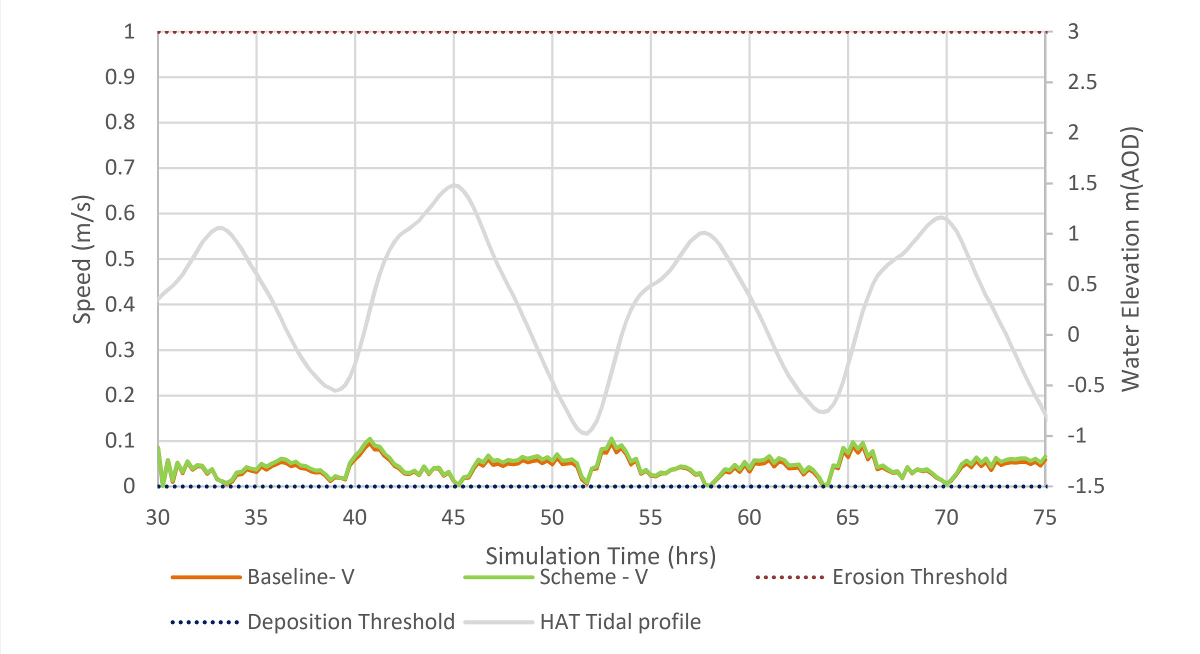

Figure 5-18 - HAT Tide Scenario, Point 11

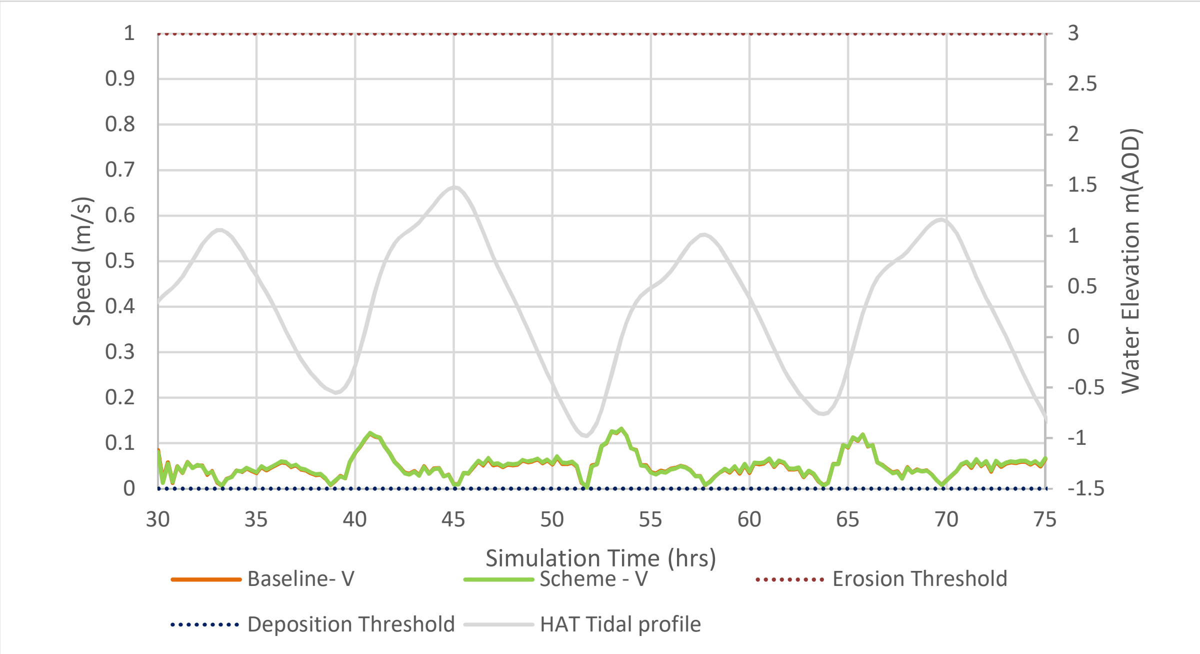

Figure 5-19 - HAT Tide Scenario, Point 15

5.2.22 Figures 5-16, 5-17, 5-18 and 5-19 all show the current speeds are not sufficient to cause erosion at any point in the simulation in the Baseline and Scheme simulations. The figures show that the Scheme has a small localised impact close the piles and this impact is reduced further away. The increase in velocity is not significant enough to change the sediment regime in the area. The figures show that the highest speeds are seen on the flood tide which is where the steepest part of the tidal profile is seen. This shows that in the absence of a significant fluvial inflow the current speed is driven by the rate of change of water elevation in the HAT scenario.

5.2.23 In addition to the figures showing the current speed along the centreline, Figure 5-20 shows the timeseries for comparison point 5. Point 5 is positioned directly upstream of one of the groups of piles (Figure 4-8) and shows the greatest change in current speed at any of the comparison points.

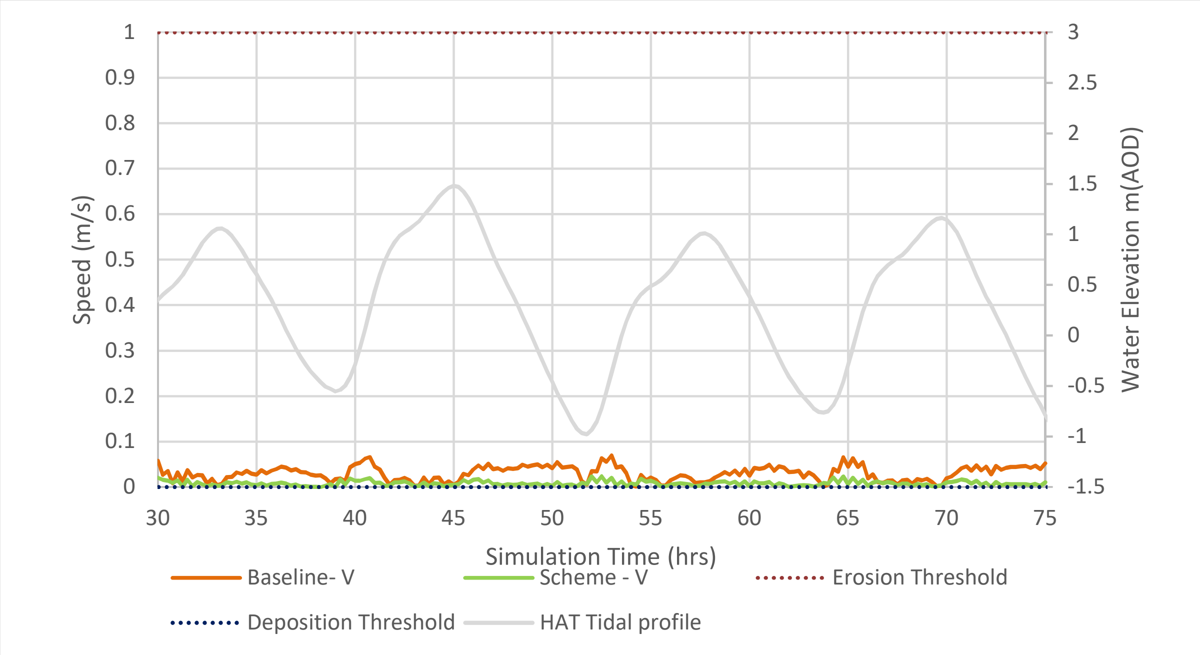

Figure 5-20 - HAT Scenario, Point 5

5.2.24 Figure 5-20 shows the current speed reduces significantly immediately upstream of the piles. This is because of the wake effect from the piles. The deposition threshold for the particle size found at point 5 is 0m/s, i.e. when the tide changes. To that end, whilst the piles do cause a measurable reduction in current speed there is not a change in regime.

5.2.25 In conclusion, the HAT, high ebb/flood tide scenario shows that in a high expected current speed, the Scheme does not have a sufficient impact on current speeds to change the current sediment regime across the domain.

Extreme (5% AEP) Tidal Scenario

5.2.26 The purpose of this scenario is to show the impact of the scheme in a likely extreme ebb/flood tidal event. Figure 5-21 shows the current speed in m/s for the 5% AEP scenario at the time of the highest recorded speed at point 15 (40.75hrs) for the baseline and scheme simulations. Comparison Point 15 was chosen because is it the closest point to the boundary in the centre of the channel.

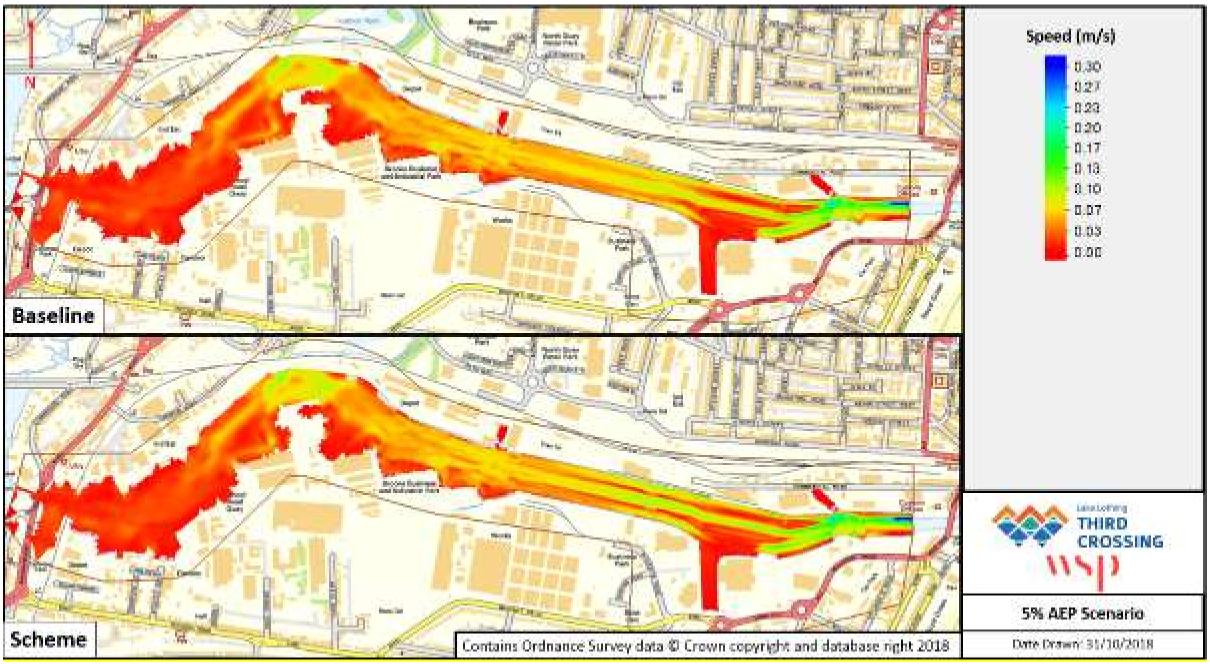

Figure 5-21 - 5% AEP Tide Scenario, Baseline and Scheme Current Speed

5.2.27 Figure 5-21 shows the baseline speed in the domain is low, showing around 0.1 to 0.3 m/s near the inflow boundary to the east and reduces significantly in a short distance to less than 0.13 m/s. When comparing the speed to the Scheme simulation, the piles have a small local impact strengthening two flow channels to the centre and north of the channel. This is because the piles have created a small localised blockage caused a constriction between the piles and harbour walls. The wake can be seen upstream and downstream of the piles and extends approximately 400m upstream and 400m downstream.

5.2.28 Figures 5-22, 5-23, 5-24 and 5-25 show the timeseries plots for points 2, 6, 11 and 15 respectively. Each figure shows the Baseline and Scheme current profiles, the 5% AEP scenario tidal boundary and the erosion/deposition threshold speed for the sediment particle size at the comparison point.

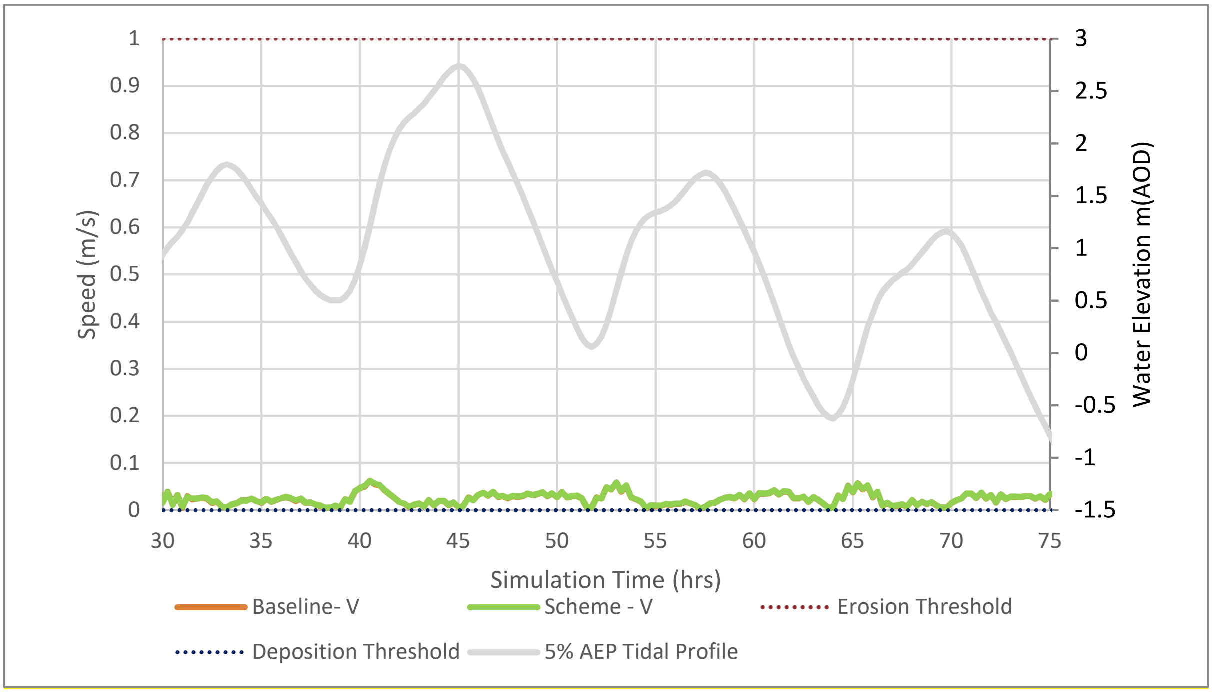

Figure 5-22 - 5% AEP Scenario, Point 2

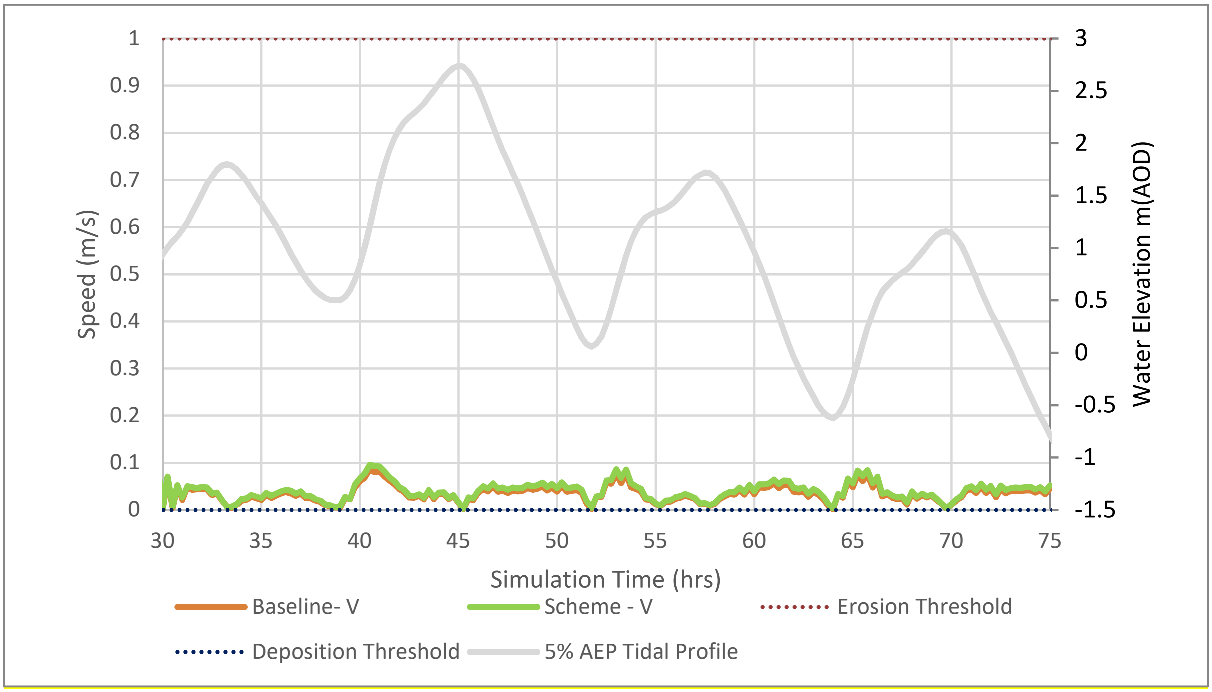

Figure 5-23 - 5% AEP Scenario, Point 6

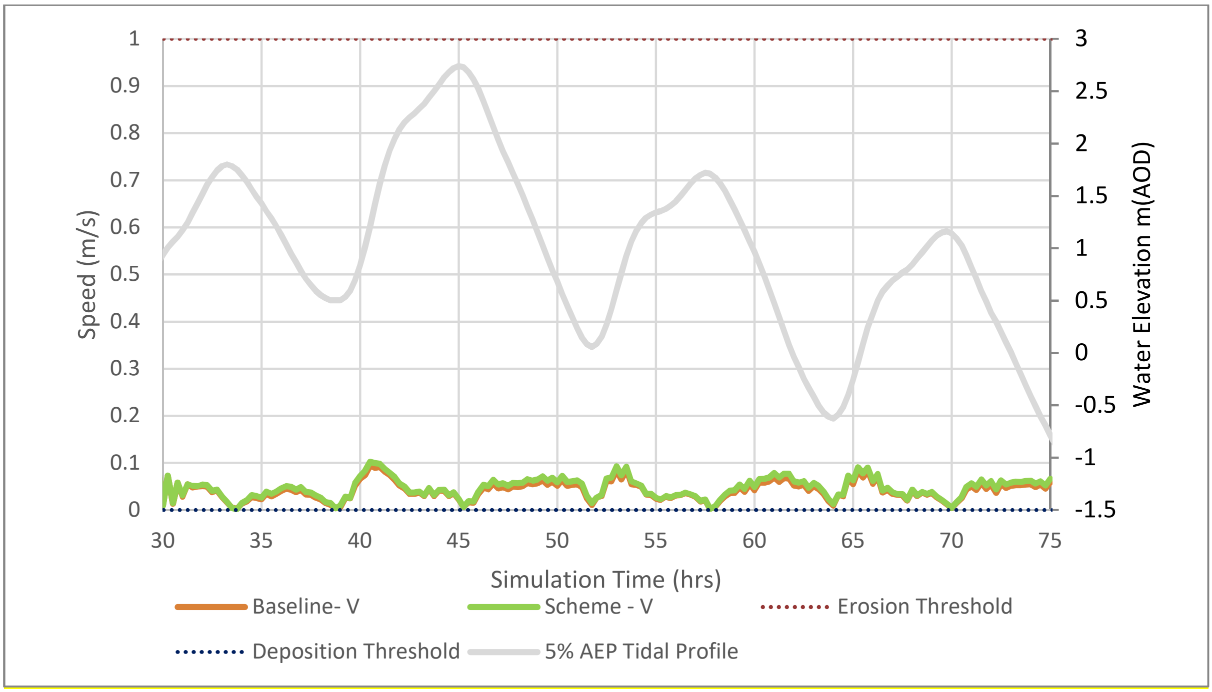

Figure 5-24 - 5% AEP Scenario, Point 11

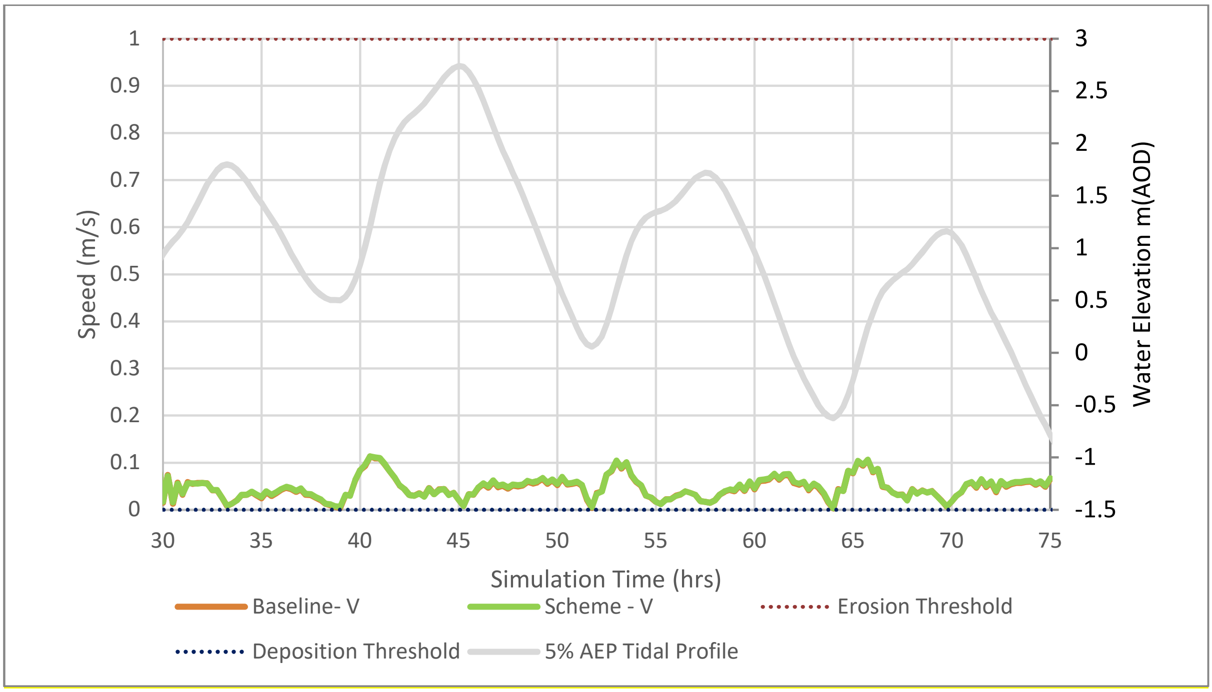

Figure 5-25 - 5% AEP Scenario, Point 15

5.2.29 Figures 5-22, 5-23, 5-24 and 5-25 all show the current speeds are not sufficient to cause erosion at any point in the simulation in the Baseline and Scheme simulations. The figures show that the Scheme has a small localised impact close the piles and this impact is reduced further away. The increase in velocity is not significant enough to change the sediment regime in the area. The figures show that the highest speeds are seen on the flood tide which is where the steepest part of the tidal profile is seen. This shows that in the absence of a significant fluvial inflow the current speed is driven by the rate of change of water elevation in the 5% AEP scenario.

5.2.30 In addition to the figures showing the current speed along the centreline, Figure 5-26 shows the timeseries for comparison point 5. Point 5 is positioned directly upstream of one of the groups of piles (Figure 4-8) and shows the greatest change in current speed at any of the comparison points.

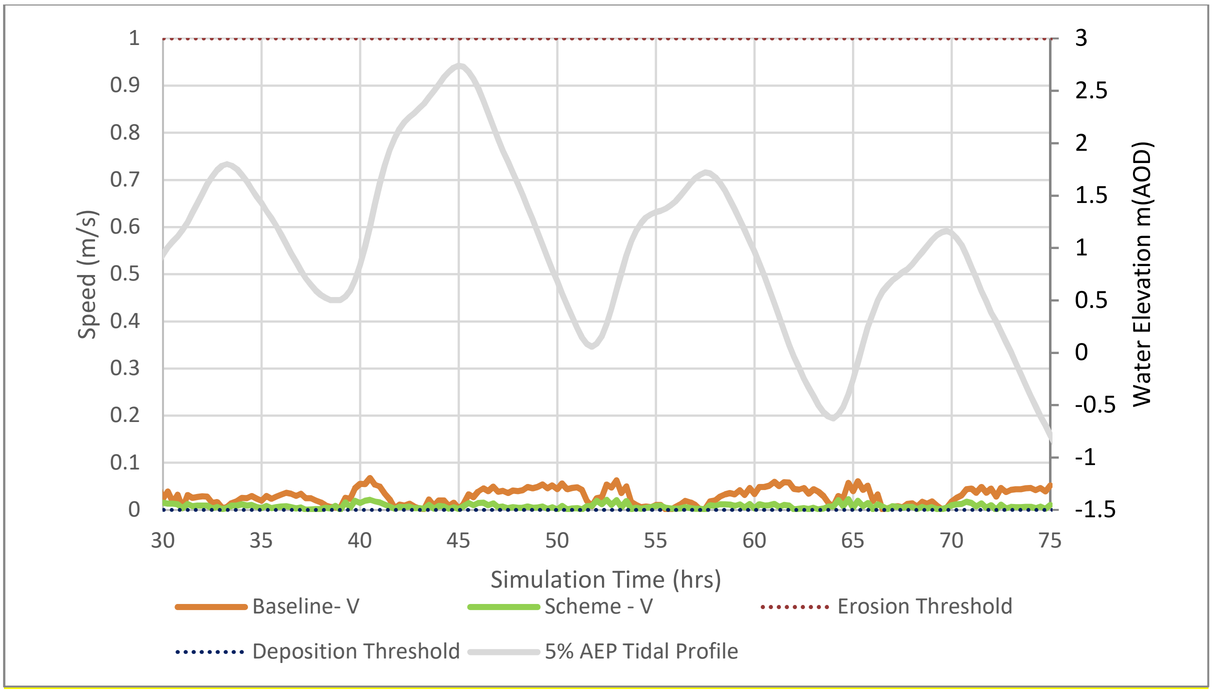

Figure 5-26 - 5% AEP Scenario, Point 5

5.2.31 Figure 5-26 shows the current speed reduces significantly immediately upstream of the piles. This is because of the wake effect from the piles. The deposition threshold for the particle size found at point 5 is 0m/s, i.e. when the tide changes. To that end, whilst the piles do cause a measurable reduction in current speed there is not a change in regime.

5.2.32 In conclusion, the 5% AEP, extreme ebb/flood tide scenario shows that in a likely extreme current speed event, the Scheme does not have a sufficient impact on current speeds to change the current sediment regime across the domain.

5.2.33 When comparing the scenarios, there is no significant difference in the impacts during the typical events (HAT and Astronomical tidal profiles) and the likely extreme event (5% AEP). The lowest current speed is in the low ebb/flood scenario. This has shown that the water speed is dependent on the change in water level between a peak and trough and not the overall depth of water.

5.2.34 To conclude, the modelling has shown that the impact of the Scheme on the harbour bed current speed is not sufficient to change the existing sediment regime and cause erosion elsewhere in the Lake Lothing.

The extent of the Impact

5.2.35 To assess the extent of the impact of the Scheme, difference plots (figures 5-27, 5-28, 5-29 and 5-30) for comparison points P2, P6, P11, P15 (shown figure 4-8) have been created. The difference plots show the difference in water speed between the Scheme and baseline for 30hr-75hr simulation time for the Astronomical, LAT, HAT and 5% AEP events.

5.2.36 Figure 5-4 shows the largest difference is in the astronomical tide approximately 53 hours into the tidal curve. The plot shows the difference is less than 0.01m/s and this is considered negligible. This shows that the impact of the Scheme is measurable up to approximately 400m west of the Scheme, beyond which no impact is seen in the model.

5.2.37 Figure 5-5 and 5-6 shows the largest difference in water speed is approximately 53 hours into the tidal curve. The model shows the impact of the Scheme is close to the Scheme site. Figure 5-7 shows the largest difference is in the astronomical tide approximately 53 hours into the tidal curve. The plot shows the difference is less than 0.01m/s and this is considered negligible. This shows that the impact of the Scheme is measurable up to approximately 400m east of the Scheme and does not extend as far as the A47 Bascule Bridge. The tidal levels are plotted for reference.

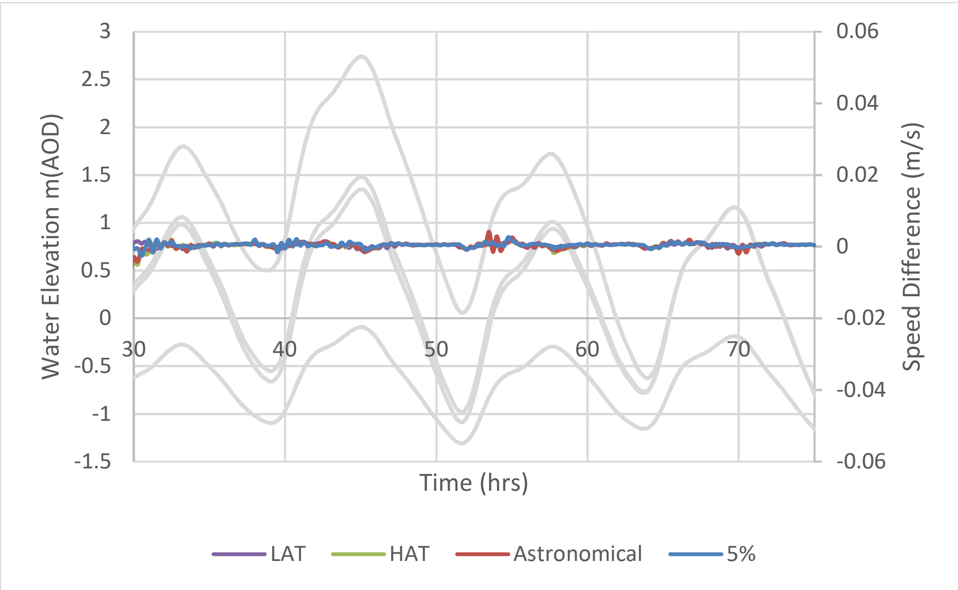

Figure 5-27 – Point 2 timeseries: Far west – Comparison between Baseline and Scheme scenarios

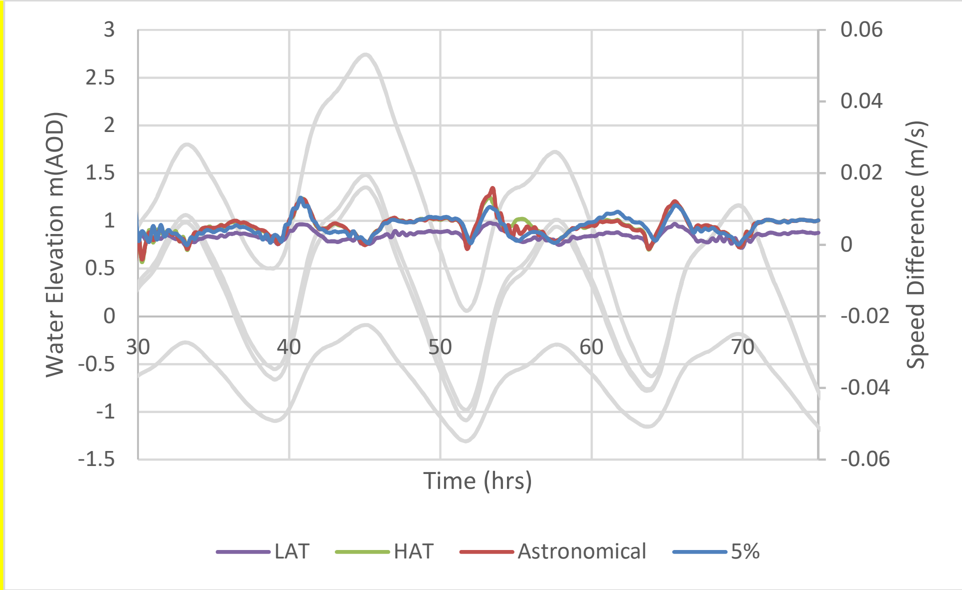

Figure 5-28 – Point 6 timeseries: Near West – Comparison between Baseline and Scheme scenarios

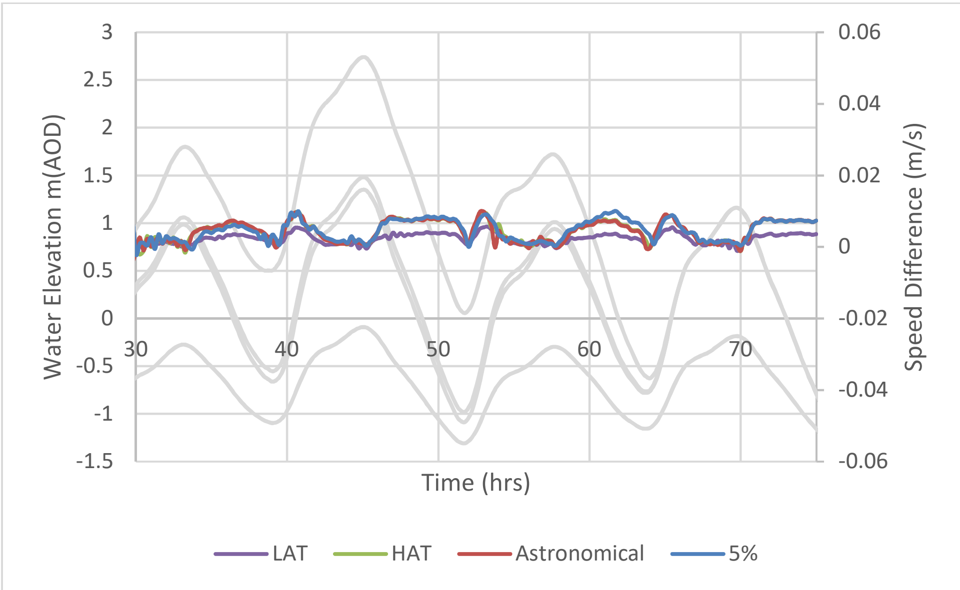

Figure 5-29 - P11 timeseries: Near East – Comparison between Baseline and Scheme scenarios

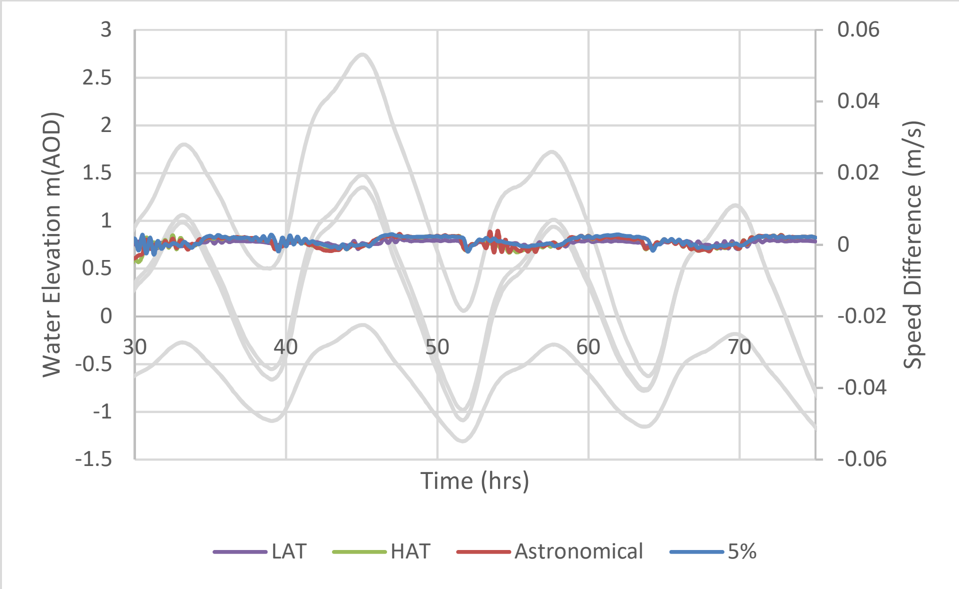

Figure 5-30 - P15 timeseries: Far East - – Comparison between Baseline and Scheme scenarios

5.2.38 In conclusion, Figures 5-27, 5-28, 5-29 and 5-30 show the impact of the Scheme in the model is limited to approximately 400m to the east and 400m to the west before the impact becomes negligible. There is no impact in the upper reaches of the Lake towards Mutford Lock or to the east towards the A47 Bascule Bridge. The model has shown the impacts of the Scheme are small and found in the channel close to the Scheme location.

Construction Impacts

5.2.39 The assessment has shown that the impact on the sediment regime is small during the operational phase of the Scheme. However, there will also be an impact on the sediment regime during the construction phase. Construction of the Scheme will be undertaken over an approximate period of two years as shown in Chapter 5 of the ES. In order to assess the impact on the sediment regime, a hydraulic assessment has been carried out to assess the worst-case impact of the construction phase on the expected water current speeds.

5.2.40 As identified in Chapter 5 of the ES, cofferdams will be used for approximately ten months to construct the bridge piers. The cofferdams will constrict the flow channel by up to 60% and will be a significantly larger blockage than the fully constructed piers. The cofferdams will be constructed using a two-zone approach; an outer removable cofferdam extending from the harbour wall to the location of the piers and an inner cofferdam built around the piers themselves. For this assessment, the model represents the outer cofferdam as this has the larger footprint and assesses the change in current speed to infer the impact on the sediment regime.

Construction Phase Modelling Approach

5.2.41 The assumed construction sequence for the piers in the reference design is to create two sheet pile outer cofferdams from the north and south quays. The cofferdams would be de-watered allowing the central pier foundations to be constructed. It has been assumed that the height of the outer cofferdams will be the same as the quay wall levels, the inner cofferdam will be of a higher level than the quay walls to maintain essential equipment during the construction phase should a flood event occur. The reference design used for this assessment has piles used to support fenders for which the final arrangement is yet to be determined. Therefore, a worst-case arrangement has been assessed. As the location of the piles is within the construction cofferdam modelled as part of this assessment, it is appropriate to consider the cofferdams as the worst-case arrangement in the water channel. This assesses the worst case environmentally and means that when the cofferdams are removed, the impact of small piles will be significantly less than has been assessed in the construction stage.

5.2.42 Within the model, the cofferdams are represented by increasing the elevation of the mesh. This represents the worst-case scenario where both cofferdams are fully installed at the same time. This simulates the maximum constriction in the Lake and allows water to flow between the cofferdams during the scenario. Figure 5-31 shows the model domain used in the assessment. This assessment does not simulate any events where the water level is greater than the top of the cofferdam, therefore there was no need to simulate the cofferdam as a hollow structure.

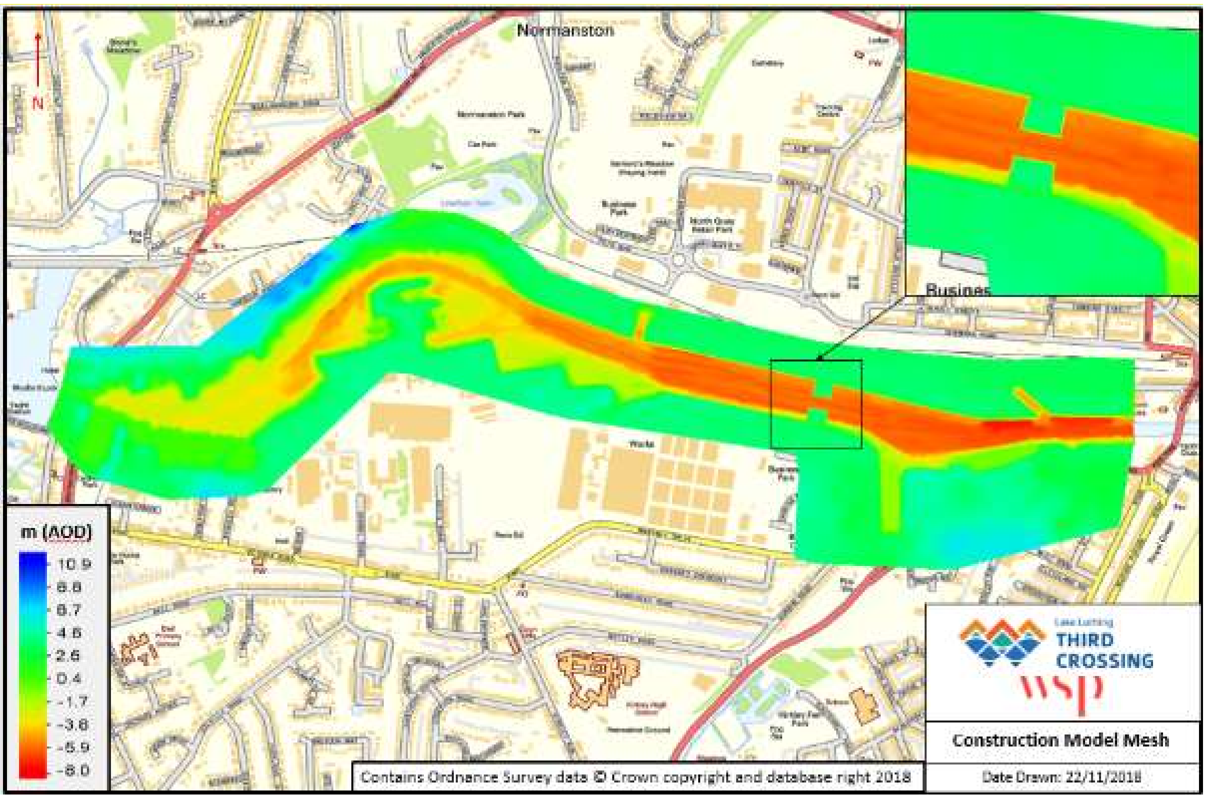

Figure 5-31 - Construction Model Mesh

Results

5.2.43 The model has been simulated for the astronomical tide scenario for the during construction assessment. This represents a tidal scenario which the cofferdam will be exposed to repeatedly during the construction period. Figure 5-32 shows the bed current speed at 40.5hr for the construction phase and baseline scenarios. Figure 5-33 shows a timeseries of bed current speed between the locations of the cofferdams in the baseline and construction phase scenarios.

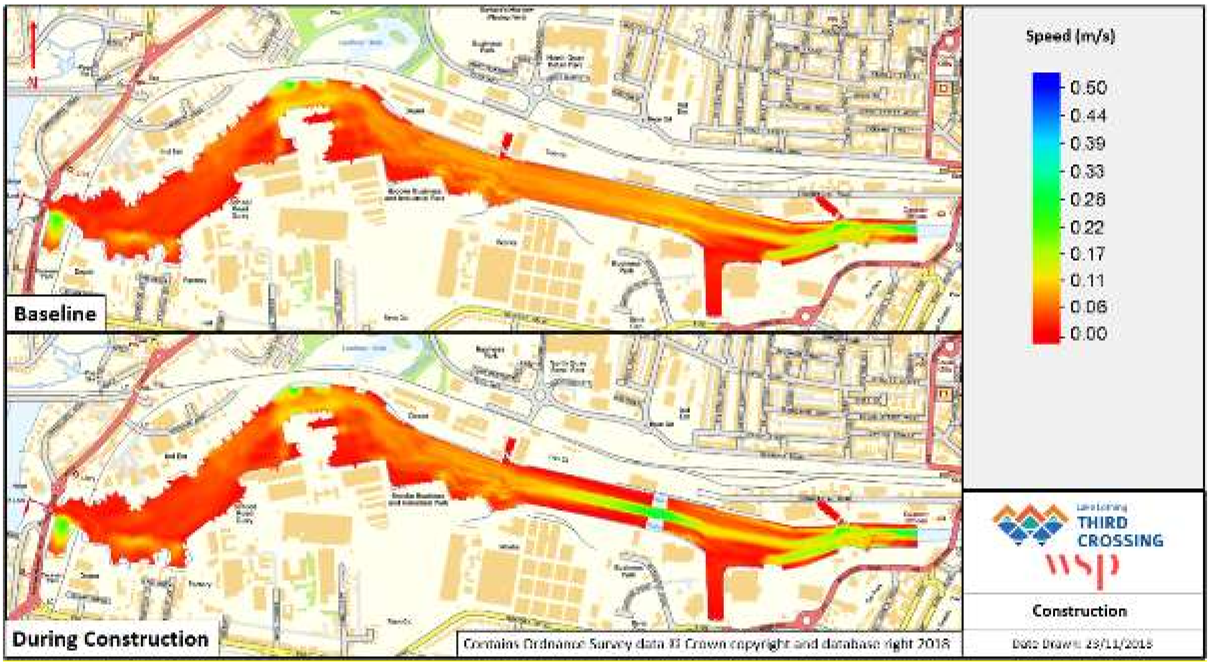

Figure 5-32 – Bed current speed plot at 40.5hr

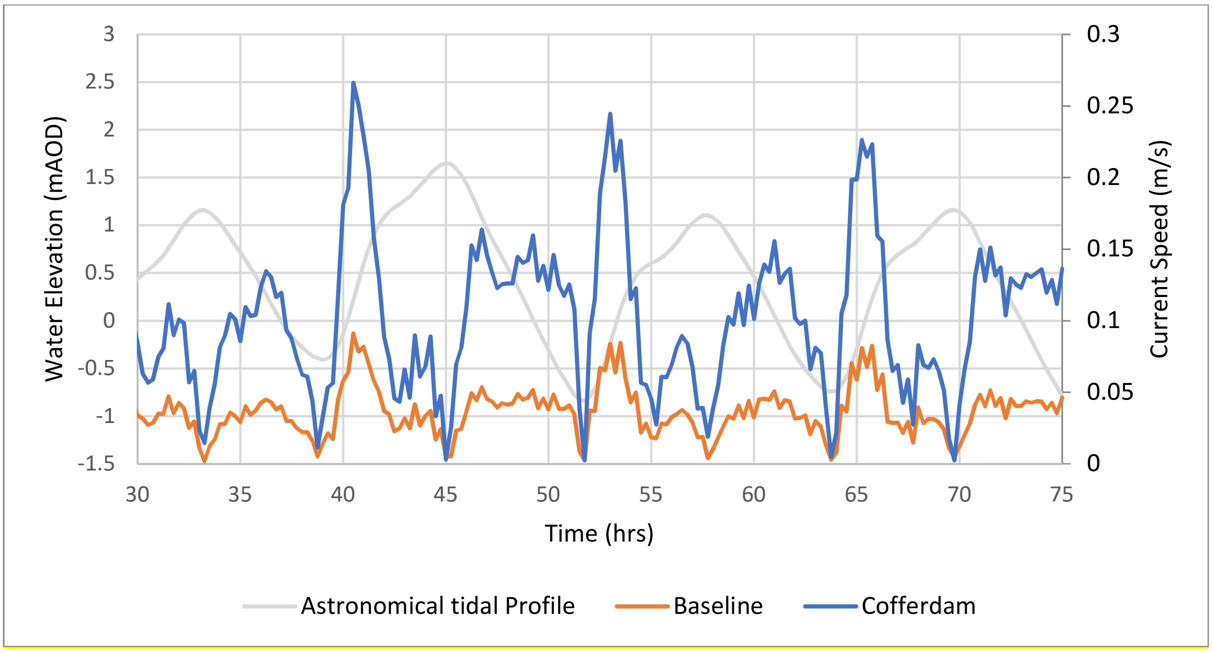

Figure 5-33 - Bed current speed comparison between cofferdams

5.2.44 The results show that when the cofferdams are in place, the peak bed current speed is increased to 0.27m/s between the cofferdams compared to 0.09m/s in the baseline scenario. A localised change to the flow pattern is also seen near the location of the Scheme. This is expected and is because of the narrowing of the channel by the cofferdams causing a localised funnelling effect. The model shows that the predicted increase in bed current speed is not sufficient to remobilise sediment as the peak flow in the construction scenario is significantly lower than the 1m/s required to mobilise sediment in Lake Lothing. To that end there will not be an increase in sediment erosion whilst the cofferdams are in place.

5.2.45 It should be noted that whilst the cofferdams are in place, there will be localised areas of increased sedimentation immediately upstream and downstream of the cofferdam walls. Sheltered areas of low current speed will be created upstream and downstream of the cofferdams, where water will slow and small eddies will be created, which will enable suspended sediment to settle at slack water. However, when dredging activity resumes post construction, any localised increase in sedimentation will be removed meaning the bathymetry will be returned to the target depth for the navigation channel. There will be not be a net increase in the sediment volume deposited in Lake Lothing due to the presence of the cofferdams. In addition to the magnitude of the impact, it was important to understand on the extent of the impact. Figure 5-32 shows that the impact reaches approximately 500m away from the Scheme and does not extend as far as the A47 Bascule Bridge.

5.2.46 In conclusion, the results show the increase in water speed where over half the channel is blocked by cofferdams is not sufficient to change the sediment regime in the channel. Therefore, it is possible to conclude that any arrangement of piles within the footprint of the cofferdams will not have an impact large enough to significantly change the sediment regime in Lake Lothing. The modelling assessment has shown that the impact of the cofferdam does not extend as far as the A47 Bascule Bridge.

5.2.47 In addition to the impact of the cofferdams on the sediment regime in lake Lothing while they are in place, the impacts of the installation/removal of the cofferdams should also be acknowledged. Modelling the impact of the construction of the cofferdams on sediment dispersion has not been undertaken as part of this assessment. It is difficult to quantify the volume of material disturbed by the construction method at this time due to the number of variables that will affect the result, these include but are not limited to; the amount of sediment at the location prior to construction, the timing since the last dredging activity at the time that this activity is carried out and the river current speed at the time installation. The sediment assessment modelling has shown the baseline current speed in Lake Lothing is low (>0.2m/s) therefore any resuspended sediment (as a result of cofferdam installation) is highly unlikely to be carried far from the construction site before settling back on the bed. It should also be noted that dredging activity is carried out approximately twice a year in Lake Lothing by the port authority within the navigation channel, which has a much greater impact on the sediment in Lake Lothing than the short-term construction activity required to build a cofferdam.

6 Summary

6.1.1 A 3D flexible mesh hydraulic model of Lake Lothing has been developed to assess the impact of the Scheme on sediment transport in the harbour. It was necessary to develop a 3D model to increase the accuracy of the water speed simulation. The flexible mesh has allowed for a high-resolution grid close to the Scheme and lower resolution elsewhere to provide an efficient balance between model run time and accuracy.

6.1.2 The hydrology of Lake Lothing has been analysed and 5% AEP, HAT, LAT and an astronomical tide event have been simulated. Tidal levels have been derived to define the eastern boundary of the hydraulic model that represents sea levels at the A47 Bascule Bridge. EA guidance on estimating design sea levels has been used to derive the tidal boundary used in the model. Fluvial flows have been calculated on the three watercourses that discharge into Lake Lothing to allow the fluvial inputs to be included in the hydraulic model. Fluvial inflows to the model have been estimated following the EA Flood Estimation Guidelines. However, sensitivity testing later showed that it was not necessary to include the fluvial flows in the hydraulic model developed for this assessment as the main flood risk to Lowestoft is tidal.

6.1.3 Sensitivity testing has been carried out to assess the model sensitivity to the roughness values, extreme tide and fluvial inflows. The testing has shown that the model is not sensitive to roughness changes therefore the Manning's values used were considered fit for purpose. The model is sensitive to the tidal boundary as this is the main inflow into the model. The fluvial inflows do not have an effect on the water speed in the channel therefore it was considered appropriate to not include the fluvial sources in the model.

6.1.4 A verification process has been carried out for the baseline model by simulating the 2013 tidal event which caused significant flooding in Lowestoft. It was not possible to calibrate the model as there is only one level gauge within Lake Lothing and this has been used to define the tidal boundary for the verification model run, it was found that the inflow had to be scaled up in order for the recorded levels at the A47 gauge to be replicated in the model. The water levels in model have been checked and they simulate the expected tidal curve at the comparison points. Anecdotal data from the Harbour Master suggests Lake Lothing experiences low water speed because of the narrow constriction at the A47 Bascule Bridge and the control imposed upstream flow by the Lock at Mutford Bridge, this flow pattern has been replicated in the model. As a result, the model was considered suitable for use in the sediment assessment.

6.1.5 The sediment survey of the harbour has shown that the sediment particle size typically ranges from 0.002mm – 0.003mm near the Scheme. The results for the baseline scenario has shown that the water speed is sufficient to maintain transport during the flood tide. As the tide changes from flood to ebb, the sediment will be deposited in the harbour. This is consistent with the existing sediment regime in Lake Lothing.

6.1.6 A comparison of the baseline and Scheme scenarios has shown that for the Astronomical, LAT, HAT and 5% AEP tidal events, there are very small localised changes in water speed due to the piles. The Scheme does not impact the water speed sufficiently in any area to cause a change in sediment regime in any event simulated.

6.1.7 The model has shown that there is no significant difference between the impacts during the typical events and the extreme events. The water speed is dependent on the difference in water level between the peak and trough of the tidal curve as there are no significant sources of inflow affecting the water speed.

6.1.8 At this stage, detailed plans for the construction stage are not available therefore specific modelling of tasks such as installation and removal of cofferdams cannot be undertaken. However, due to the low current speeds showing the modelling and anecdotal evidence from the harbour master any localised impact from construction activity is likely to have a short-term impact, i.e. until reinstatement of target depths during the first dredge post-construction. It should also be noted that dredging activity is carried out approximately twice a year in Lake Lothing within the navigation channel, which has a much greater impact on sediment mobilisation in Lake Lothing than the short-term construction activity required to build a cofferdam.

6.1.9 The modelling assessment has shown that the Scheme has a small localised increase in peak current velocity, however it will have no significant impact on the overall sediment transport regime and as such the overall volumes of maintenance dredging required in the Lake will not be affected. The presence of the bridge structure will require a change in the method of dredging used in its vicinity as certain methods would not be considered suitable for use in proximity to the piers.

6.1.10 The modelling of the cofferdam represents the worst-case scenario environmentally, therefore any arrangement of piles/pile cap within the footprint of the cofferdam remaining after construction will not significantly change the sediment regime in Lake Lothing.

Appendix A

CALCULATION CONTROL SHEET
PROJECT: Lake Lothing, Third Crossing
PART OF PROJECT: Design Sea Level Calculations
CALCULATION TITLE: Design Sea Level Calculations record
FILE LOCATION: G:\1403\7.0 Projects\7.05 Live Projects\1073877 Lake Lothing\09 Documents\Reporting\Sediment modelling\Sediment modelling Report.docx

CALCULATION SUMMARY
This report provides a record of the calculations and decisions made during the derivation of the tidal boundary inflows using the recommendations in SC060064/TR4: Practical Guidance design sea levels. Following the review of the model by the Environment Agency (EA), the extreme sea levels from Open Coast (CFBD) Flood Risk Study (JBA, 2014) have been used.
Purpose of Calculations To derive design tidal inflow for the sea boundary in the Lowestoft sediment model.

CHECKING AND REVIEW STATUS

Rev	Purpose	Author	Reviewed	Authorised	Date
1	Draft for model build	DE	JH		

REVISION HISTORY

Revision Ref./ Date Issued	Date	Purpose and description of Amendments	Issued to
1		Draft for model build	

1 Introduction

This document provides a record of the calculations and decisions made during design sea level estimation. It will often be complemented by more general hydrological information given in a project report. This version of the report is for when a single tidal boundary is required.

2 Method Statement

Item	Comments
Purpose of study Give an overview which includes: Purpose of study Approx. no. of tidal boundaries required	The Lowestoft Lake Lothing and Outer Harbour Area Action Plan was adopted in 2012 and identifies Waveney District Council's long-term ambition for a third vehicular crossing of Lake Lothing. The Lake Lothing, Third Crossing has been designated a Nationally Significant Infrastructure Project (NSIP) and is a key objective in regeneration of the harbour area of Lowestoft. This document presents the tidal curve calculation for the sea boundary in Lake Lothing. This is achieved by combining extreme water level, astronomical tide profile and a surge shape. Each component is derived following the SC060064/TR4: Practical Guidance Design Sea Levels (Environmental Agency (EA), 2011) using the extreme sea levels from Open Coast (CFBD) Flood Risk Study (JBA, 2014).
Description of catchment Brief description of catchment, or reference to section in accompanying report	Lowestoft is a seaside town in Suffolk on the east coast of England. The harbour, known as Lake Lothing is one of the sea boundaries for the Broadlands rivers catchment. Lake Lothing is a tidal driven lake which has a boundary with the North Sea downstream and Mutford Lock upstream. The lake is split into two areas, the inner harbour and the outer harbour. The downstream end of Lake Lothing is subject to approximately a 12-hour tidal cycle from the North Sea which causes changes in water levels in the lake basin.
Flood estimates required	Flow hydrographs / peak flow estimates are required for the assessment are: • 5% Annual Exceedance Probability (AEP) • Highest Astronomical Tide (HAT) • Lowest Astronomical Tide (LAT) • Astronomical Tide

Table A2.1: Overview of Study

What is the source of the sea level data? • Admiralty Tidal Time Charts • Gauge Data	Gauge data situated at the A47 Bascule Bridge in Lowestoft harbour (NGR: 652127 292785)

Table A2.2: Source of Sea Level Data

Watercourse	Station Name	Gauging authority number	Grid reference	Period of available data	Type of Data
Lake Lothing Harbour	LAKE LOTHING	T341907	TM5212792785	9 years	Tidal (Level)
Comments	Data for the gauge is provided in two formats, checked daily average sea levels from the EA and 15 minute 'live data' from the National Tidal and Sea Level Facility[6], (NTSLF) which has not been quality checked and is extracted at 4hr intervals from an online graph meaning it is labour intensive and prone to human error.				

Table A2.3: Site information

Item	Comments
Other Flow / levels gauging sites	Level gauge at the lock at Mutford Bridge records levels within Oulton Broads
Historic flood data	Tidal flooding of properties on and near the coastline in Lowestoft and Lake Lothing. (2013 and 1953). A shape file showing the flood extent in an event is provided in the Environmental Agency's Historic flood maps. As the time or date information for the event is unavailable, it was assumed that the extent is the 2013 event because shapefiles were not available in 1953.
Flow data for events	No flow data available, level only gauges at Lake Lothing and Oulton Broad.
Results from previous studies / models	Lowestoft Tidal defences: Additional Modelling Studies, 2014 Lowestoft Estuary Inception Study, 2013 Lowestoft Tidal Flood Study, 2013 1D BESL model (simulates Oulton Broads fluvial system)
Other data (e.g. Groundwater, tidal	Photographs taken during 2013 flood event published in a local newspaper.

Table A2.4: Other Data Available

[6] http://www.ntslf.org/data/realtime?port=Lowestoft

Item	Comments
Outline the method	The conceptual method chosen here follows the guidance; *SC060064/TR4: Practical Guidance design sea levels.* In April 2008, the EA undertook a strategic overview of the coasts in England. The guidance was created for the EA project, *Coastal flood boundary conditions for UK mainland and Islands (*SC060064/TR2: Design sea levels[7]), with the aim to update and consolidate the outdated methods for producing tidal curves suitable for Flood Risk Assessments. The aims of the project were to: • Provide a consistent set of extreme sea levels around the coasts of England, Wales and Scotland. • Provide a means of generating total storm tide curves for use with the extreme sea levels. • Offer practical guidance on how to use these new datasets. This method is acknowledged as the best method for calculating the tidal curves in the UK using the most up-to-date method and the best data available. EA recommends its use for tidal curve derivation when undertaking Flood Risk Assessments.

Table A2.5: Sea Level Derivation Method

[7] Coastal flood boundary conditions for UK mainland and islands SC060064/TR2: Design sea levels, Environmental Agency, 2011

3 Tidal Curve Calculations

The extreme tidal curves are derived using the guidance from *SC060064/TR4: Practical Guidance Design Sea Levels.* All decisions and reasons are presented.

Ten Step procedure
• Check study location is outside of estuary boundaries
• Select an appropriate chainage point for extreme sea levels
• Select an annual exceedance probability peak sea level
• Consider allowance for uncertainty
• Identify base astronomical tide
• Convert levels to Ordnance Datum
• Identify surge shape to apply
• Produce the resultant design tide curve
• Sensitivity testing
• Apply allowance for climate change

Table A3.1: Guidance

The guidance is part of the larger project, *Coastal flood boundary conditions for UK mainland and islands, (Environmental Agency, 2011)* and is the best method currently available for tidal curve derivation in UK waters. As part of this project a number of additional datasets are provided:

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

Table A3.2: Additional Data sets

Since the guidance was published, there has been an update to the extreme sea levels carried out by JBA for the EA. Following the guidance and the updated extreme sea levels, the event tidal curves are generated.

3.3 Check Study Location is Outside of Estuary Boundaries

The guidance is valid only 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,* a shape file is provided with all major estuary locations highlighted, Figure A3.1 shows a comparison between the Lowestoft estuary boundary and the Lowestoft model tidal boundary.

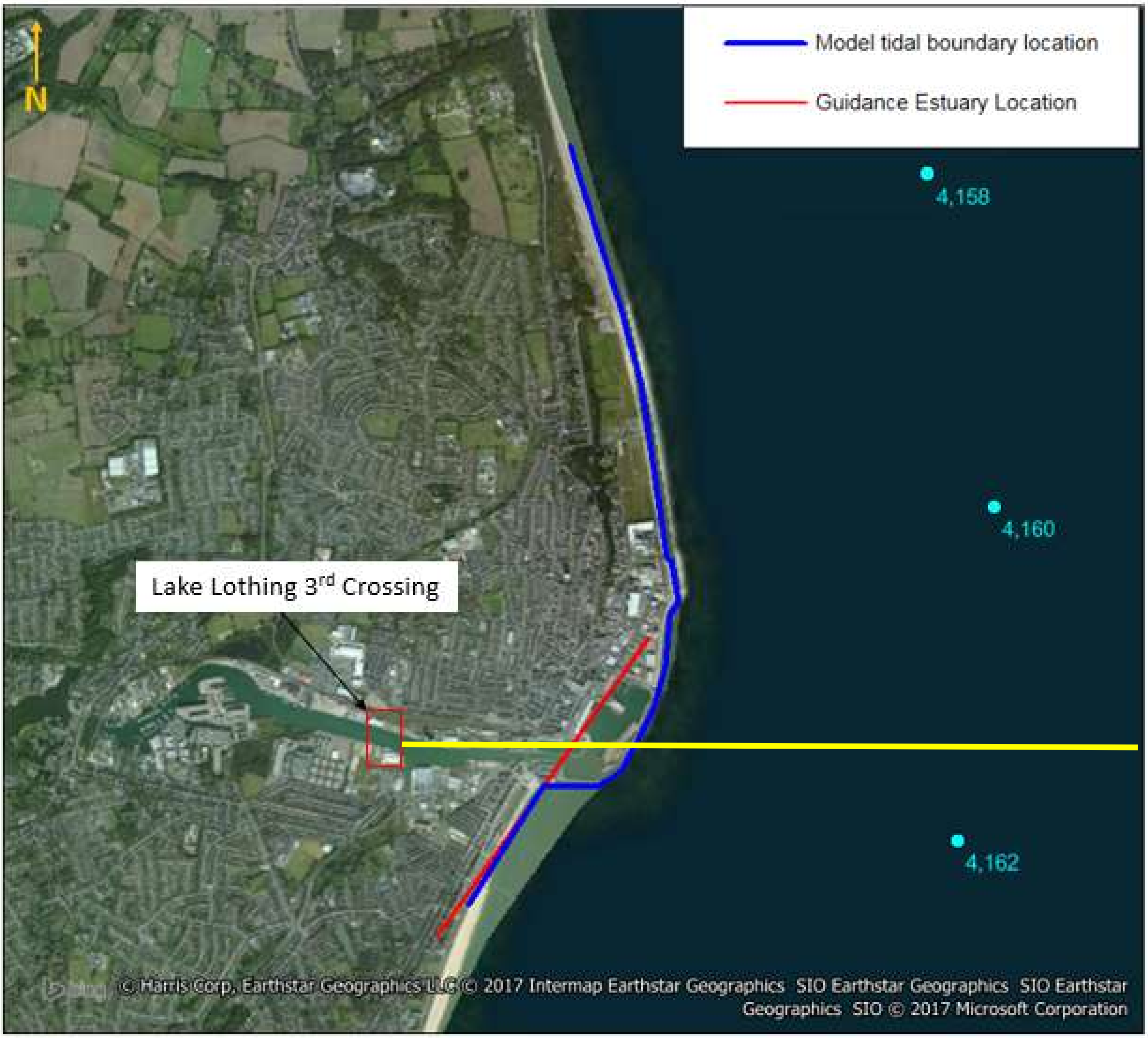

Figure A3.1 Estuary Boundary Check

Figure A3.1 shows the estuary boundary of Lake Lothing in red and the proposed tidal boundary of the Lowestoft tidal model in blue. The location of the model boundary is close to the estuary location in the guidance. At most locations, the tidal boundary is outside of the estuary however there is a small section which is on the estuary line because it follows the coastline. In this model, this is deemed acceptable because Lowestoft has an engineered harbour and in reality, the estuary discharges north of the area where the tidal boundary touches the estuary line. Based on this assessment, this guidance is deemed appropriate for use to generate the tidal curve.

3.4 Select the Appropriate Chainage Point for Extreme Sea Levels

The guidance recommends that the extreme sea level node nearest to a line drawn from the tidal boundary should be used to define the extreme sea levels for the site of interest. A yellow line drawn from the Lowestoft tidal boundary passes closest to 4162 chainage node as shown on Figure A3.2.

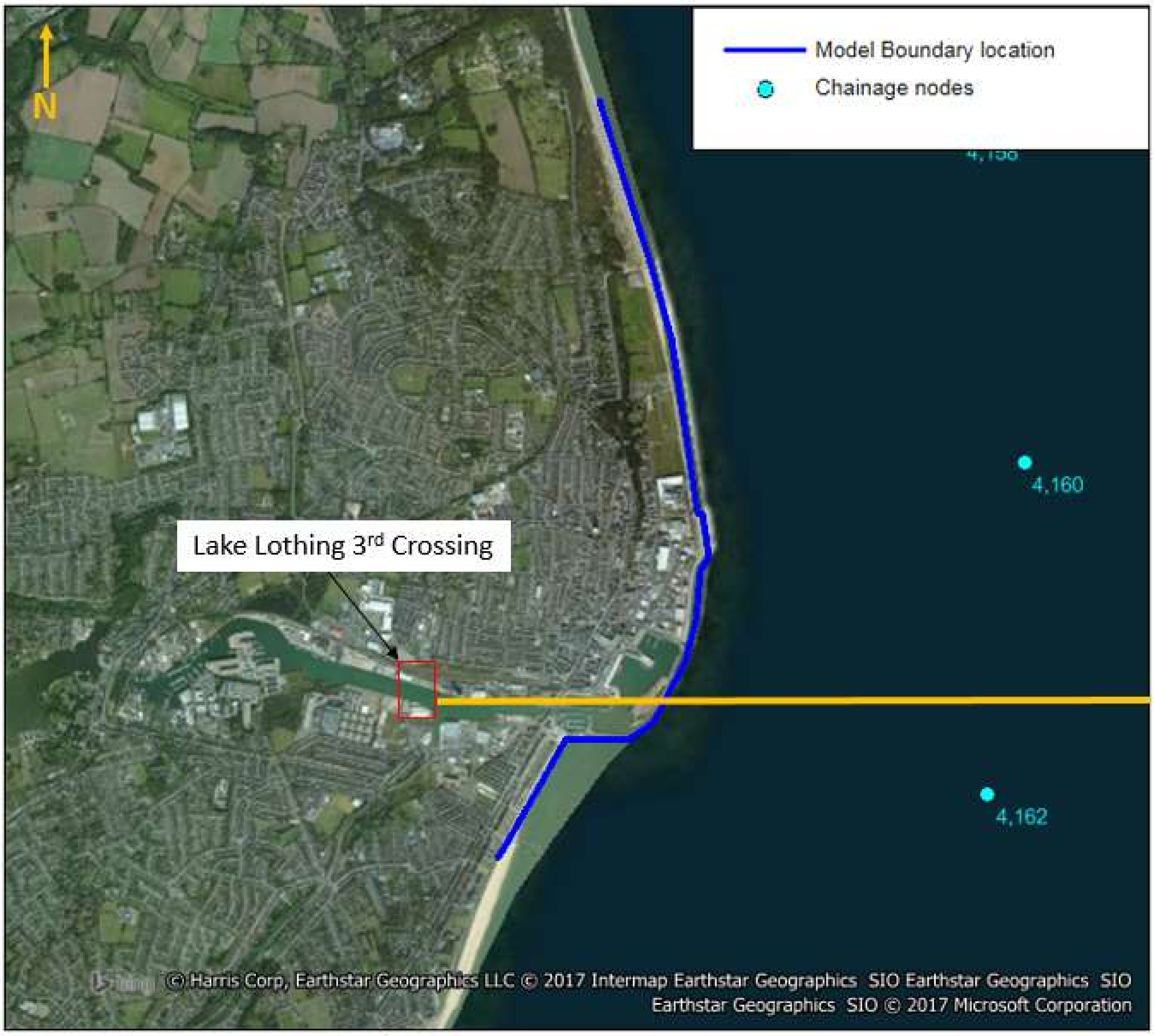

Figure A3.2: Chainage

3.5 Select an Annual Exceedance Probability Peak Sea Level

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 provided in the Open Coast (CFBD) Flood Risk Study by JBA for the EA at node 4162 are provided in Table A3.3 for the events considered in this study.

AEP	Extreme sea levels (m AOD)
5%	2.74
HAT	1.48
LAT	-1.38
Astronomical	1.35
0.1%	3.92

Table A3.3: Extreme Sea Levels

3.6 Consider Allowance for Uncertainty

As part of the *SC060064/TR4* project, confidence in the extreme sea levels are provided as shown in Table A3.4 for the events considered in this study. The confidence levels are a measure of the potential error in the EA extreme sea level modelled results. The uncertainty is considered acceptable for this project. The EA require the Scheme to be assessed against the high impact, low probability (H++) event. Modelling of the H++ event will demonstrate the sensitivity of the model to the levels forced at the tidal boundary.

AEP	Uncertainty (+/- m)
5%	0.2
0.1%	0.4

Table A3.4: Uncertainty levels (node 4162)

3.7 Identify Base Astronomical Tide

The next stage of the tidal curve derivation is to identity the base astronomical tide. *SC060064/TR4* guidance states that the astronomical tide used for the tidal curve should have a peak between the Highest Astronomical Tide (HAT) and the Mean High-Water Springs (MHWS). Table A3.5 shows the HAT and MHWS values for Lowestoft from the National Tidal and Sea Level Facility[8] (NTSLF). The tidal levels are provided in chart datum in Lowestoft harbour. Conversion to ordnance datum is to add -1.5m, this is carried out in part 3.8.

HAT (mCD)	MHWS (mCD)
2.98	2.58

Table A3.5: HAT and MHWS for Lowestoft

The SC060064/TR4 guidance states that the Admiralty tidal tables should be used to estimate the astronomical tide. This step is unnecessary because Lowestoft has a tidal gauge in the harbour meaning that an astronomical tide can be obtained from recorded data.

Browsing the gauge data, a tidal profile with a peak tide of 2.85mCD was found. The NTSLF website publishes tidal levels on an interactive graph. The numerical dataset is not available; therefore, a sample was taken from the graph at approximately a four-hour resolution.

As part of the Lake Lothing Third Crossing study, WSP received the 1D-2D hydraulic model developed for the Lowestoft Tidal Defences study[9] carried out by CH2M Hill on behalf of Waveney District Council. The tidal curve for the original 1% AEP event was provided with the CH2M Hill model.

The data from the NTSLF is too coarse to be used for the tidal curve, therefore the CH2M Hill tidal curve data was considered for use in this study. In order to test the suitability of the CH2M Hill tidal curve, it was scaled to a peak of 2.85mCD and compared to the NTSLF data as shown in Figure A3-3.

[8] http://www.ntslf.org/tgi/portinfo?port=Lowestoft

[9] Lowestoft Tidal defences: Additional Modelling Studies, 2014

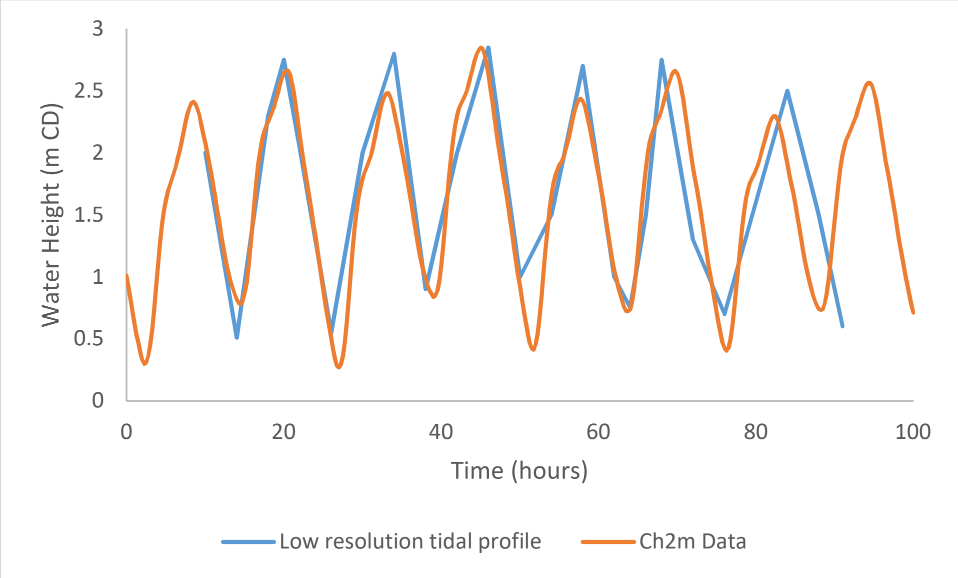

Figure A3-3: Astronomical tidal profile comparison

Figure A3-3 shows the CH2M Hill scaled tidal curve and the low resolution tidal profile taken from NTSLF graph. The peaks and troughs of both profiles align and are comparable. The largest peak, 2.85mCD is identical because of the scaling procedure. Some of the other peaks are different, this is a consequence of the scaling. However, for the Lake Lothing Third Crossing study the maximum water level is most important and the other peaks are less relevant. Therefore, due to the good comparison between the two data sets, it was deemed appropriate to use the CH2M Hill tidal curve to define the astronomical tide for the Lowestoft model tidal boundary.

3.8 Convert Levels to Ordnance Datum

The tidal levels are quoted in chart datum and need to be converted to ordnance datum. A chart datum conversion is provided at key ports around the UK. Lowestoft chart datum conversion is -1.5m. The data from the gauge site in Lowestoft is quoted in chart datum therefore this needs to be converted to ordnance datum to be comparable with the extreme sea levels and suitable for use in the hydraulic model.

3.9 Identify Surge Shape

As part of the *SC060064/TR4* project surge shapes were derived for key locations around the UK, the Lowestoft surge shape is number 9 in the Design_Surge_Shapes.xls provided with the guidance documentation.

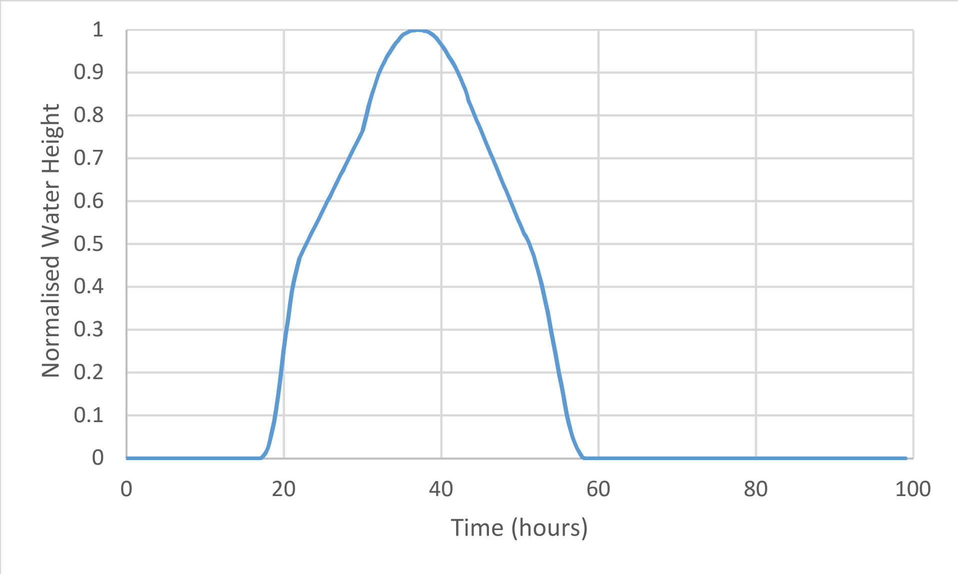

Figure A3-4: Shape 9 – Lowestoft Surge

Figure A3-4 shows the normalised surge shape at Lowestoft which is combined with the CH2M Hill model curve to derive the design tide curve.

3.10 Produce the Resultant Design Tide Curve

The guidance states that the resultant design tide curve is derived by combining the extreme sea level, base astronomical tide and surge shape. The first process is to align the astronomical tide and surge shape peaks, in this case this is at 45 hours in line with the CH2M Hill tidal curve.

Once the CH2M Hill tidal curve and surge shape are aligned, it is necessary to scale the astronomical tide to the required extreme sea level. To explain this procedure, a 0.5% AEP event will be used as an example. Firstly, the difference between the required extreme sea level (3.4m AOD) and the peak CH2M Hill (3.11m AOD) is calculated which in this example is 0.29m. As the surge shape is aligned with the peak water level time in the CH2M Hill tidal curve, 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 0.19/1.0 = 0.19m AOD, thus creating a surge height which can be added to the CH2M Hill tidal curve resulting in the required peak water level for the event.

This procedure is carried out of each return period, scaling to the extreme sea level for a given design event (Table A3.3)

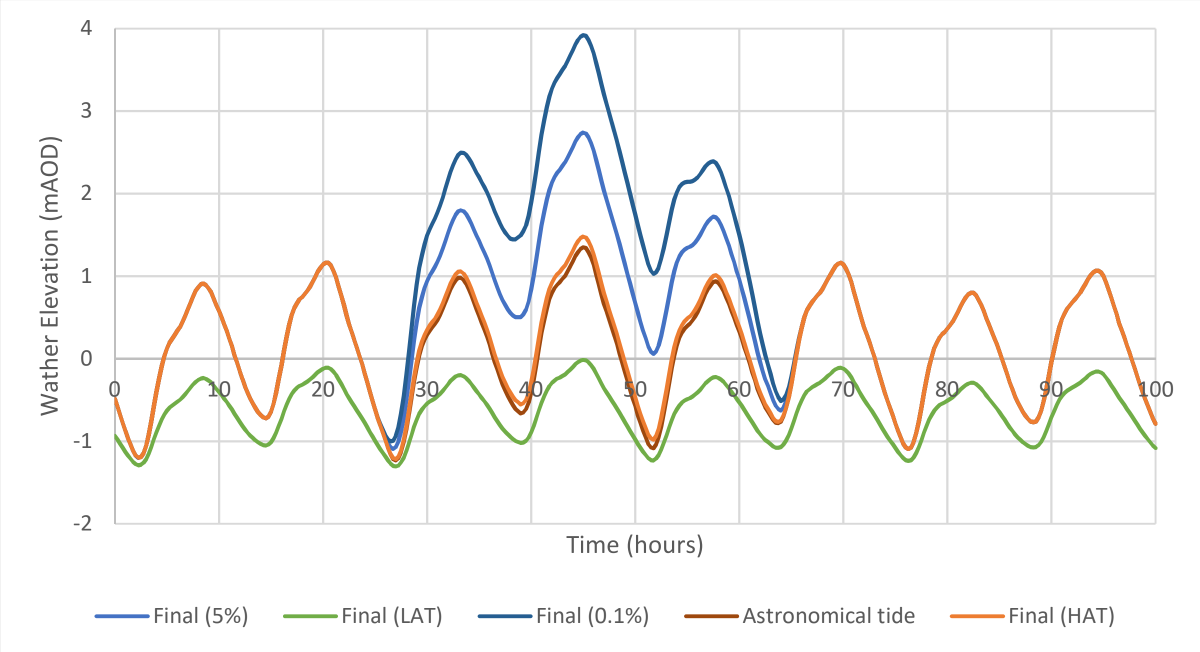

Figure A3-1: Final design event tidal curves

Figure A3-5 shows the final tidal curves for the 5% AEP, 0.1% AEP, HAT, LAT and astronomical tidal events used in the model simulations.

3.11 Sensitivity Test

The guidance, *SC060064/TR4* requires the surge shape to be offset. This is to see the impacts of the surge arriving at a different time on the tidal curve. This is unnecessary for this study because the extreme tidal level remains at the same level which is the driving factor in tidal flooding. Other tests will be undertaken to determine the sensitivity of the model to certain parameters.

3.12 Climate Change Calculations

Climate change impacts have not been considered as part of this assessment.

4 Conclusions

The extreme tidal levels in Table A4.1 have been derived following the guidance, SC060064/*TR4* and discussed in the previous section.

Event	5% AEP (m AOD)	0.5% AEP (m AOD)	0.1% AEP (m AOD)
Present day extreme sea level (2017)	2.74	3.40	3.92

Event	Peak (m AOD)
HAT	1.48
LAT	-1.38
Astronomical	1.35

Table A4.1 - Final calculated tidal peaks

The final tidal curves generated will be used as the inflow boundary to the hydraulic model developed for the Lake Lothing Third Crossing FRA.

4.3 Limitations

There are a number of limitations highlighted in the guidance documents. These are presented in table A4.2.

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.

Table A4.6: Limitations of the tidal curve derivation method

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 sediment assessment therefore the extreme sea level curves will be used to assess flooding in Lowestoft due to the Third Crossing Development.

Appendix B

CALCULATION CONTROL SHEET
PROJECT: Lake Lothing, Third Crossing
PART OF PROJECT: Hydrology calculations
CALCULATION TITLE: FEH Calculation Record
FILE LOCATION: G:\1403\7.0 Projects\7.05 Live Projects\1073877 Lake Lothing\09 Documents\Hydrology\Report

CALCULATION SUMMARY
This report provides a record of the calculations and decisions made during design flood estimation using the techniques of the Flood Estimation Handbook (Institute of Hydrology, 1999).
Purpose of Calculations To derive design hydrographs and peak flows for three catchments that flow into Lake Lothing in Lowestoft.

CHECKING AND REVIEW STATUS					
Rev	**Purpose**	**Author**	**Reviewed**	**Authorised**	**Date**
1	Draft for model build	DE	JH	TJ	29/03/17

REVISION HISTORY			
Revision Ref./ Date Issued	**Date**	**Purpose and description of Amendments**	**Issued to**
1	03/2017	Draft version for hydraulic model update	

Abbreviations

AM ....Annual Maximum
AREA...............................Catchment area (km^2)
BFI..................................Base Flow Index
BFIHOST.........................Base Flow Index derived using the HOST soil classification
CD ....Catchment Descriptors
CFMP ....Catchment Flood Management Plan
CPRE ....Council for the Protection of Rural England
FARL ....FEH index of flood attenuation due to reservoirs and lakes
FEH ....Flood Estimation Handbook
FSR ....Flood Studies Report
HOST ....Hydrology Of Soil Types
NRFA..............................National River Flow Archive
POT ....Peaks Over a Threshold
QMED.............................Median Annual Flood (with return period 2 years)
RaDAR ....Radio Detection and Ranging
ReFH ....Revitalised Flood Hydrograph method
SAAR..............................Standard Average Annual Rainfall (mm)
SPR ....Standard Percentage Runoff
SPRHOST ....Standard percentage runoff derived using the HOST soil classification
Tp(0)...............................Time to peak of the instantaneous unit hydrograph
UAF ....Urban Adjustment Factor
URBAN...........................Flood Studies Report index of fractional urban extent
URBEXT1990..................FEH index of fractional urban extent
URBEXT2000..................Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH..................Windows Frequency Analysis Package – used for FEH statistical method

1. Introduction

This calculation record document provides a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This version of the record is for studies where flood estimates are needed at multiple locations.

2. Method Statement

Table B2.1: Overview of study

Purpose of study Give an overview which includes: Purpose of study Approx. no. of flood estimates required Peak flows or hydrographs?	This document will present the flood estimation calculations for three small tributaries that discharge directly into the Lake Lothing. Peak flows and hydrographs are required.
Description of catchment Brief description of catchment, or reference to section in accompanying report	Lake Lothing is a tidally influenced, salt water lake in Lowestoft through which part of the Norfolk Broads discharges into the North Sea. The largest fluvial inflow source is from Mutford Lock which controls flow into Lake Lothing from Oulton Broad. This lock is in daily use as it provides access to the Norfolk Broads for sailing vessels. There are 3 smaller catchments flowing directing into Lake Lothing Harbour. Kirkley stream (National Grid Reference: 653900, 292650) and two smaller, unnamed catchments (National Grid Reference: 653400 292750 and 654050 292850).
Flood estimates required	Flow hydrographs / peak flow estimates are required for: 20 (5% AEP), 200 (0.5% AEP), 1000 (0.1% AEP), 20+cc, 200+cc, 1000+cc years flood design events.

Table B2.2: Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – As part of the pooling group analysis (HiFlows, v4.1).

Table B2.3: Gauging Stations (flow or level data available at sites or nearby donor catchments)

Watercourse	Station Name	Gauging authority number	NWA number (used in FEH)	Grid reference	Catchment Area (km^2)	Rating?	Period of available data
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

No suitable Gauge Station was found in the vicinity of the study catchments.

Table B2.4: Data available at each Gauging Station

Station Name	Start & end of data in Hi Flows UK	Update from EA for this study? Dates?	Suitable for QMED?	Suitable for pooling?	Data Quality Check needed?
N/A	N/A	N/A	N/A	N/A	N/A
Comments on data quality (inc. rating) and any checks made	N/A				

Table B2.5: Rating Equations

Station Name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Reasons e.g. availability of recent flow gaugings, amount of scatter in the rating.
N/A	N/A	N/A	N/A

Table B2.6: Other data available

Item	Comments
Flow / level gauges	Level gauge at the Lock records levels within Oulton Broads Level gauge at A47 Bascule Bridge (eastern end of Lake Lothing) Data for both gauges within the study area has been supplied by the Environment Agency for the period January 2007 – August 2016.
Historic flood data	Tidal flooding of properties on and near the coastline.
Extra data for other sites in pooling groups	none
Flow data for events	none
Rainfall data for events	none
Potential evaporation data / MORECS data	none
Results from previous studies / models	Lowestoft Tidal defences: Additional Modelling Studies, 2014 Lowestoft Estuary Inception Study, 2013 Lowestoft Tidal Flood Study, 2013 1D BESL model (simulates Oulton Broad's fluvial system)

Table B2.6: Other data available

Item	Comments
Other data (e.g. Groundwater, tidal	Lake Lothing is predominately tidally driven.

Table B2.7: Initial choice of approach

Item	Comments
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	The study area for the hydraulic assessment covers three small tributaries which flow into Lake Lothing in Lowestoft harbour. These will be used as point inflows into an existing 2D Lake Lothing model to simulate the fluvial inflow. While the main source of flow in Lake Lothing is tidal, including the riverine inflows will allow the ‘worst case’ flood scenario to be modelled. Standard FEH approach – ReFH and FEH Statistical method will be used to derive the flow estimated for the small catchments. A preferred method will be chosen based on the analysis of the methods used.

Table B2.7: Initial choice of approach

Item	Comments
Any unusual catchment features to take into account? e.g. 5 highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% 6 highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments 7 pumped watercourse – consider lowland catchment version of rainfall-runoff method 8 major reservoir influence (FARL<0.90) – consider flood routing, extensive floodplain storage – consider choice of method carefully	For Kirkley stream (11.07km² catchment area) 1 BFIHOST is 0.638 and SPRHOST is 30.59%. The catchment is therefore permeable. 2 URBEXT1990 and 2000 are 0.1547 and 0.1549 respectively. This relates to the 'Heavily Urbanised' category. 3 FARL is 1 for Kirkley Stream so no extensive floodplain storage. For Catchment 1, 653400 292750 (0.56km² catchment area) 4 BFIHOST is 0.721 and SPRHOST is 27.02%. The catchment is therefore highly permeable and the ReFH method is likely to be unsuitable. ♦ URBEXT1990 and 2000 are 0.4219 and 0.4799 respectively. This relates to the 'Very heavily urbanised' category. ♦ FARL is 1 for Catchment 1 so no extensive floodplain storage. For Catchment 2, 654050 292850 (0.71 km² catchment area) 5 BFIHOST is 0.755 and SPRHOST is 22.32%. The catchment is therefore highly permeable. ♦ URBEXT1990 and 2000 are 0.5158 and 0.5193 respectively. This relates to the 'Very heavily urbanised' category. ♦ FARL is 1 for Catchment 2 so no extensive floodplain storage.
Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) **Outline the choices available and whether appropriate for the sites of interest:** 9 FEH Statistical (single site or pooled?) 10 FEH rainfall-runoff 11 Revitalised rainfall runoff 12 IoH124 13 Rational Method 14 Hybrid approach?	Small catchments (0.71km² and 0.56km²) for the two unnamed catchments therefore FEH may not be appropriate however calculations are carried out to confirm. High BFIHOST for two unnamed catchments therefore ReFH may not be appropriate however calculations are carried out to confirm. Urbanisation has been taken into account in ReFH and FEH Statistical methods. Choices to be used are: 1 FEH Statistical Pooled analysis 2 ReFH

Table B2.7: Initial choice of approach

Item	Comments
Initial choice of method(s) and reasons 15 Will the catchment be split into sub-catchments? If so, how?	ReFH and FEH Statistical Pooled analysis to be undertaken for 3 catchments flowing into Lake Lothing. These are: • Kirkley Stream • Catchment 1, 653400 292750 Catchment 2, 654050 292850
Software to be used (with version numbers)	FEH CD_ROM v3.0[10] WINFAP-FEH v3[11] (with HiFlows v4.1)

3. Locations where flood estimates are required

The table below lists the locations of hydrological points of interest (subject sites). The site codes listed below are used in all subsequent tables to save space. A map showing the hydrological boundaries and downstream points of interest is shown in Figure B3.1.

Table B3.1: Summary of hydrological points of interest (all subject sites)

Site code	Watercourse	Site	Grid Reference		Catchment area from FEH WEB (km^2)	Revised area if required (km^2)
1	Kirkley Stream	Kirkley Stream	653900	292650	11.07	-
2	Unnamed Catchment	Catchment 1	653400	292750	0.56	-
3	Unnamed Catchment	Catchment 2	654050	292850	0.71	-
Reasons for choosing hydrological points of interest (subject sites)	Kirkley Stream is the point of confluence with Lake Lothing and the area represents the full catchment. Catchment 1 is the point of confluence with Lake Lothing and the area represents the full catchment. Catchment 2 is the point of confluence with Lake Lothing and the area represents the full catchment.					
How catchment descriptors were checked	Catchment area was checked by inspection of Ordnance Survey maps and LiDAR data. Checks of soil types and drainage show that the soil type are sandy (Kirkley and Catchment 1) and loamy (catchment 2) with natural drainage. This correlates with the BFIHOST values for the catchments.					

[10] FEH CD-ROM v3.0 © NERC (CEH). © Crown copyright. © AA. 2009. All rights reserved.

[11] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2016.

Figure B3.1: Map of catchments

Catchment Descriptors from FEH CD ROM Version 3 at the five hydrological points of interest have been extracted for use on this study.

For the design runs, the URBEXT 2000 values were updated to 2016 for each sub-catchment. The method used to adjust QMED for urbanisation, for both subject sites and donor sites, is that published in Kjeldsen (2010)[12] in which PRUAF (percentage runoff urban adjustment factor) is calculated from BFIHOST. The result will differ from that of WINFAP-FEH v3.0.003 which does not correctly implement the urban adjustment of Kjeldsen (2010). Significant differences will occur only on urban catchments that are highly permeable.

[12] Kjeldsen, T. R. (2010). Modelling the impact of urbanization on flood frequency relationships in the UK. Hydrol. Res. 41. 391-405.

PRUAF= 1 +0.47*URBEXT$_{2000}$(BFIHOST/(1-BFIHOST))

N.B. The FEH CD-ROM Version 3 provides URBEXT values for the year 2000 (URBEXT$_{2000}$). URBEXT$_{2000}$ is not simply an update of URBEXT$_{1990}$ but it is based on new data produced using different mapping techniques.

Table B3.2 shows the re-statement of the categories of urbanisation distinguished in the FEH according to their URBEXT 1990 values, together with 'equivalent' URBEXT 2000 values.

Table B3.2: Categories of catchment urbanisation related to FEH CDROM (2007)

Category	URBEXT1990	URBEXT2000
Essentially rural	0.000 <_URBEXT$_{1990}$ < 0.025	0.000 <_URBEXT$_{2000}$ < 0.030
Slightly urbanised	0.025 _ URBEXT$_{1990}$ < 0.050	0.030 _ URBEXT$_{2000}$ < 0.060
Moderately urbanised	0.050 _ URBEXT$_{1990}$ < 0.125	0.060 _ URBEXT$_{2000}$ < 0.150
Heavily urbanised	0.125 _ URBEXT$_{1990}$ < 0.250	0.150 _ URBEXT$_{2000}$ < 0.300
Very heavily urbanised	0.250 _ URBEXT$_{1990}$ < 0.500	0.300 _ URBEXT$_{2000}$ < 0.600
Extremely heavily urbanised	0.500 _ URBEXT$_{1990}$ _ 1.000	0.600 _ URBEXT$_{2000}$ _ 1.000

In addition to updating URBEXT to 2015 values, URBEXT values were checked and no adjustments were made based on visual inspection of Ordnance Survey maps and neighbouring catchment descriptors. The final values of URBEXT 2015 (and all other catchment descriptors) used in the analysis and reason for any adjustments are provided in Table B3.2.

Table B3.3 shows the catchment descriptors to be used for each point of interest.

Table B3.3: Catchment descriptors used in analysis

CD			
Grid Ref	Kirkley Stream	Catchment 1	Catchment 2
AREA	11.07	0.56	0.71
ALTBAR	11	7	18
ASPBAR	2	22	185
ASPVAR	0.2	0.58	0.71
BFIHOST	0.638	0.721	0.755
DPLBAR	5.33	1.02	1.14
DPSBAR	8.4	5.3	27
FARL	1	1	1
FPEXT	0.391	0.3482	0.0912
FPDBAR	3.081	1.138	0.323

FPLOC	0.858	0.84	0.478
LDP	8.51	2.44	1.88
PROPWET	0.27	0.27	0.27
RMED-1H	11.1	10.9	11
RMED-1D	30.2	28.7	28.8
RMED-2D	38	36.3	35.6
SAAR	602	599	600
SAAR4170	610	605	600
SPRHOST	30.59	27.02	22.32
$URBCONC_{1990}$	0.844	0.881	0.924
$URBEXT_{1990}$	0.1547	0.4219	0.5158
$URBLOC_{1990}$	0.549	1	0.912
$URBCONC_{2000}$	0.84	0.94	0.964
$URBEXT_{2000}$	0.1549	0.4799	0.5193
$URBLOC_{2000}$	0.586	0.947	0.961
C	-0.02436	-0.02489	-0.02484
D1	0.32889	0.31713	0.31822
D2	0.36447	0.3667	0.35466
D3	0.22802	0.2359	0.23607
E	0.31795	0.31713	0.31925
F	2.46533	2.45479	2.45752
C(1km)	-0.024	-0.025	-0.024
D1(1km)	0.31	0.322	0.31
D2(1km)	0.372	0.346	0.372
D3(1km)	0.237	0.231	0.237
E(1km)	0.317	0.318	0.317
F(1km)	2.46	2.461	2.46
Adjusted URBEXT[13] $_{2016}$	0.1602	0.4964	0.5372
Adjusted URBEXT[14] $_{2016}$	0.1667	0.4546	0.5558
Notes on changes made:	$URBEXT_{2000}$ factor used = 1.034 $URBEXT_{1990}$ factor used = 1.077	$URBEXT_{2000}$ factor used = 1.034 $URBEXT_{1990}$ factor used = 1.077	$URBEXT_{2000}$ factor used = 1.034 $URBEXT_{1990}$ factor used = 1.077

4. Statistical method

The FEH statistical method constructs a flood frequency curve based on the estimation of QMED, which is then used to calculate peak flow estimates for each return period. FEH methods should not normally be applied on heavily urbanised catchments (with an URBEXT value greater than 0.5) or catchments smaller than 0.5km². Catchment 2 has a URBEXT was of greater than 0.5, in this case the ReFH urban method is favoured.

The statistical method was carried out for the three locations at the points of hydrological interest mentioned in Section 2.

For this study, the sub-catchments are un-gauged, therefore the QMED value has been estimated based on catchment descriptors extracted from FEH CD-ROM 3 and the most recent equation published by CEH for QMED estimations. No potential donor catchments were found close to the subject sites. Statistical pooling analysis was undertaken using FEH WINFAP software to produce a growth curve and calculate flood flows for range of return periods.

[13] $URBEXT_{2000}$ Adjustment

[14] $URBEXT_{1990}$ Adjustment

The final FEH statistical method flow estimates for the sub-catchments are presented at the end of the section in Table B4.7 for the following range of return periods: 20, 20+cc, 200, 200+cc, 1000 and 1000+cc

The initial estimates of QMED are displayed in Table B4.3. They are based on catchment descriptors alone and use the following equation:

$$QMED = 8.3062 AREA^{0.8510} 0.1536^{\left(\frac{1000}{SAAR}\right)} FARL^{3.4451} 0.0460^{BFIHOST^2}.$$

Table B4.1: Search for Donor Sites (if applicable)

Comment on potential donor sites: 16 Number of potential donor sites available 17 Distances from subject site 18 Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors 19 Quality of flood peak data 20 Include a map if necessary. Note that donor catchments should usually be rural.	No Donor sites are applicable for the catchments in this report.

Methods:
AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.
When QMED is estimated from POT data, it should also be adjusted for climatic variation.

Table B4.2: Donor sites chosen and QMED adjustment factors

NWA number	Watercourse	Station	Reason	AM or POT	QMED from flow data (A)	QMED from CDs (B)	Adj ratio (A/B)
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
If a spreadsheet has been used to calculate QMED insert link here: G:\1403\7.0 Projects\7.05 Live Projects\1073877 Lake Lothing\09 Documents\Hydrology\Calculations\Qmed Calculation.xls							

The data transfer procedure is the revised one from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table B4.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

$$a = 0.4598*\exp(-0.020*d_{ij})+(1-0.4598)*\exp(-0.4785*d_{ij})$$

If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

Table B4.3: Estimation of QMED at subject sites

Site Code	Method	Initial QMED from CD's (m^3/s)	Initial QMED from CD's (m^3/s)				If more than one donor		Final estimate of QMED (m^3/s)
			NRFA no. used	Distance between centroids d_{ij}	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	Weight	Weighted ave. adjustment	
Kirkley Stream	CD	0.99	-	-	-	-	-	-	0.99
Catchment 1	CD	0.08	-	-	-	-	-	-	0.08
Catchment 2	CD	0.10	-	-	-	-	-	-	0.10

Are the values of QMED consistent, for example at successive points along the watercourse and at confluence?	Successive QMED is not appropriate for this study as all catchments flow into Lake Lothing independent of each other

Pooling groups were derived using the revised procedures from Science Report SC050050 (2008). Several subject sites may use the same pooling group. The composition of the edited pooling groups is given in the Appendix.

Table B4.4: Derivation of pooling groups

Name of Group	**Site code for which group initially derived**	**Subject site treated as gauged?** (enhanced single site analysis)	**Changes made to default pooling group with reasons.** Note also any sites that were investigated but retained in the group.	**Weighted average L-moments,** L-CV and L-skew (before urban adjustment)
Kirkley	**Kirkley Stream**	No	Four stations removed due to low SPRHOST values. One station removed due to low FARL value. Four sites added to edited pooling group to total 526 years.	L-CV $_{initial}$ = 0.231 L-Skew $_{initial}$ = 0.061 L-CV $_{final}$ = 0.244 L-Skew $_{final}$ = 0.106
C1	**Catchment 1**	No	One station removed for short record. Three stations removed due to low SPRHOST values. One station removed due to low FARL value. One station removed for high discordancy. Four sites added to edited pooling group to total 520 years.	L-CV $_{initial}$ = 0.218 L-Skew $_{initial}$ = 0.199 L-CV $_{final}$ = 0.237 L-Skew $_{final}$ = 0.243
C2	**Catchment 2**	No	One station removed for short record. Four stations removed due to low SPRHOST values. One station removed due to low FARL value. One station removed for high discordancy. Six sites added to edited pooling group to total 533 years.	L-CV $_{initial}$ = 0.231 L-Skew $_{initial}$ = 0.234 L-CV $_{final}$ = 0.219 L-Skew $_{final}$ = 0.228

If a spreadsheet has been used for pooling group growth curves insert link here:
G:\1403\7.0 Projects\7.05 Live Projects\1073877 Lake Lothing\09 Documents\Hydrology\Calculations:

- kirkley stream_Results_FEH statistical.xls
- catchment 1_Results_FEH statistical.xls
- catchment 2_Results_FEH statistical.xls
-

Notes:
The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.

Table B4.5: Derivation of flood growth curves at each subject site

Site code	**Method:**	**Distribution(s) chosen and reason, include goodness of fit parameters**	**Any urban adjustment or permeable adjustment?**	**Parameters of chosen distribution(s)**
Kirkley	P	GL and GEV recommended by FEH, with GL producing a steeper growth curve that is more conservative at higher return periods	V3 (Kjeldsen, 2010) applied to growth curve	Location: 1.000 Scale: 0.227 Shape: -0.131 Bound: -0.741
C1	P	GL and GEV recommended by FEH, with GL producing a steeper growth curve that is more conservative at higher return periods	V3 (Kjeldsen, 2010) applied to growth curve	Location: 1.000 Scale: 0.163 Shape: -0.331 Bound: -0.005

Site code	Method:	Distribution(s) chosen and reason, include goodness of fit parameters	Any urban adjustment or permeable adjustment?	Parameters of chosen distribution(s)
C2	P	GL and GEV recommended by FEH, with GL producing a steeper growth curve that is more conservative at higher return periods	V3 (Kjeldsen, 2010) applied to growth curve	Location: 1.000 Scale: 0.147 Shape: -0.324 Bound: 0.547

Notes:
Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis
A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.
Urban adjustments are all carried out using the v3 method: Kjeldsen (2010).
Growth curves were derived using the revised procedures from Science Report SC050050 (2008).

Table B4.6: Growth Curves

Site Code	Growth Curve Factor for the following return periods	
	20	200
Kirkley	1.82	2.73
C1	1.82	3.36
C2	1.72	3.06

Table B4.7 provides the final peak flow estimates calculated using the statistical method with an urban adjustment factor (UAF) of 1.036 applied to the QMED$_{rural}$ and urban adjustment applied to the growth curves utilising Kjeldsen, Version 3 (2010).

Table B4.7: Statistical Method Estimate of Peak Flows

Name	Flood peak (m^3/s) for the following return periods in years								
	20	20cc (25%)[15]	20cc (65%)[16]	200	200cc (25%)[6]	200cc (65%)[7]	1000	1000cc (25%)[6]	1000cc (65%)[7]
Kirkley	1.79	2.24	2.95	2.7	3.38	4.46	5.08	6.35	8.38
C1	0.15	0.19	0.25	0.27	0.34	0.45	0.54	0.68	0.89
C2	0.17	0.21	0.28	0.3	0.38	0.5	0.65	0.81	1.07

Table B4.7 provides the peak flow estimates calculated using the statistical method with an urban adjustment factor (UAF) applied to the QMED$_{rural}$; Kirkley = 1.21, Catchment 1 = 1.79, Catchment 2 = 2.04.

[15] Based on Anglian river basin district for a design life of 100 years – central value. (*Adapting to climate change – Guidance 2016)*

[16] Based on Anglian river basin district for a design life of 100 years – upper value. (*Adapting to climate change – Guidance 2016)*

5. Revitalised FSR/FEH rainfall runoff method

The Revitalised Flood Hydrograph (ReFH) method was developed by CEH to provide a more realistic representation of flood hydrology. This method is generally believed to perform reasonably well on most catchments. However, this method is not currently appropriate for either 'heavily urbanised' or 'very heavily urbanised' based on the values of URBEXT2000 extracted from FEH CD-ROM 3 because its summer design event was only calibrated on seven urban catchments, and further research to improve the ReFH method has been recommended.

The ReFH Urban is an enhancement of the existing ReFH rainfall-runoff technique in order to better estimate design flows in heavily or very heavily urbanised catchments. This alternative method which is based on the study published by Kjeldsen (2009) can be applied when there is a difference between the boundaries of the topographic and sewer catchments.

Peak flow estimates from the Revitalised FEH rainfall-runoff model calculated without using urban subdivisions for this heavily urbanised watercourse, however these flows will be calculated separately to determine the impact of using these extra calculations on flow estimates compared to those calculated without urban subdivisions.

Parameters used to derive the ReFH method hydrographs are provided in Table B5.1 and B5.2. For this study, the critical storm duration of each sub-catchment has been calculated in order to generate the maximum peak flow estimate each catchment individually.

The ReFH method flow estimates for each sub-catchment are presented in Table B5.3.

Table B5.1: Parameters for ReFH model

Site Code	**Method** OPT: Optimisation BR: Baseflow Recession Fitting CD: Catchment Descriptors DT: Data Transfer	T_p (hours)	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow Recharge
Kirkley	CD	6.053	533.1	38.011	1.441
C1	CD	1.275	598.78	15.213	1.644
C2	CD	0.698	625.573	13.13	1.728
Brief description of any flood event analysis carried out (further details should be or in a project report)					

Table B5.2 Design events for Standard ReFH method

Site Code	Urban or Rural	Season of design (summer or winter)	Storm Duration (hours)	Storm area for ARF (if not catchment area)
Kirkley	Urban	Summer	29.5	-
C1	Urban	Summer	3.5	-
C2	Urban	Summer	1.7	-
Are the storm durations likely to be changed in the next stage of the study? (e.g. by optimisation within a hydraulic model?)			Storm duration have been optimised for 100-year event.	

Table B5.3: Peak flow estimates from the Revitalised FEH rainfall-runoff model

Site code	Flood peak (m^3/s) for the following return periods in years								
	20	20cc (25%)	20cc (65%)	200	200cc (25%)	200cc (65%)	1000	1000cc (25%)	1000cc (65%)
Kirkley	3.03	3.79	5	5.51	6.89	9.09	9.52	11.9	15.71
C1	0.29	0.36	0.48	0.57	1.72	0.94	1.1	1.38	1.82
C2	0.47	0.59	0.78	0.99	1.23	1.62	2	2.51	3.3

Table B5.4 shows the parameters for the ReFH Urban method for each of the catchments.

Table B5.4: Parameters for ReFH urban model

Site Code	Undeveloped area (km^{2}))	Paved, draining away from the watercourse (km^{2}))	Paved, draining towards the watercourse (km^{2}))	Sewer Capacity (yr RP)	Runoff (%)
Kirkley	8.68	0	2.39	70	70
C1	0.02	0	0.54	30	70
C2	0.67	0	0.04	30	70
Brief description of any flood event analysis carried out (further details should be or in a project report)					

Table B5.5: Design events for ReFH urban model

Site Code	Urban or Rural	Season of design (summer or winter)	Storm Duration (hours)	Storm area for ARF (if not catchment area)
Kirkley	Urban	Winter	11.01	-
C1	Urban	Summer	5.01	-
C2	Urban	Summer	1.03	-
Are the storm durations likely to be changed in the next stage of the study? (e.g. by optimisation within a hydraulic model?)			Storm duration have been optimised for 100-year event.	

The ReFH Urban method combines a flow prediction for paved and unpaved areas within the catchment to create a more realistic flow hydrograph for an urban catchment. This typically results in the combination of a short flashy peak (urban flow) and longer slower peak (rural flow). Appendix A shows the hydrographs for 100-year return period in three catchments.

Table B5.6 shows the peak flows for each of the catchments.

Table B5.6: Peak flow estimates from the revitalised FEH rainfall runoff urban model

Site code	Flood peak (m^3/s) for the following return periods in years								
	20	20cc (25%)	20cc (65%)	200	200cc (25%)	200cc (65%)	1000	1000cc (25%)	1000cc (65%)
Kirkley	3.09	3.86	5.09	5.50	6.88	9.08	8.45	10.57	13.95
C1	0.49	0.61	0.81	0.93	1.16	1.53	1.5	1.87	2.47
C2	1.70	2.12	2.81	3.56	4.44	5.87	5.95	7.43	9.81

6. Summary of results

Peak discharges were calculated for each sub-catchment for the following range of return periods: 20, 20+cc, 200, 200+cc, 1000 and 1000+cc. The following methods were investigated: FEH Statistical and ReFH. Table B6.1 summarises the 200-year peak flows for both methods for all sub-catchments.

Table B6.1: Summary of 100-year return period peak flow estimated for the different methods

Site code	Site name	Peak flow for 200 year return period (m^3/s)		
		FEH Statistical	ReFH	ReFH-Urban
Kirkley	Kirkley Stream	2.7	5.51	15.42
C1	Catchment 1	0.27	0.57	0.33
C2	Catchment 2	0.3	0.99	0.62

Table B6.2: Choice of method

Item	Comments
Final choice of method and reasons Include: 7 Reference to type of study 8 Nature of catchment 9 Type of data available	For all catchments a hybrid method for the hydrograph will be adopted. This will use the ReFH Urban hydrograph shape and fit it to the statistical peak. This is because the statistical method is the most suitable method for a catchment of this size and by using the ReFH Urban method for the shape then the short storm periods typical of an urban area is somewhat accounted for.

Table B6.3: Assumptions, limitations and uncertainty

Item	Comments
List the main assumptions made (specific to this study)	
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	There is a number of limitations with the methods used of the hydrograph derivation. Neither the statistical or standard ReFH methods are suitable for small, heavily urbanised catchments, therefore the use of ReFH Urban for the hydrograph shape negated some of the issues. Using the hybrid method for the catchments (ReFH Urban scaled of statistical peak) negates some of the issues with using the ReFH urban method in a catchment with less urban area whilst still showing a realistic hydrograph shape and storm period.

Item	Comments
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).	The ReFH urban method is designed to improve the performance of the standard ReFH method in urban catchments therefore there is confidence with this method in catchment 1 and 2.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	The results have made use of the most up-to-date data and methods and could be applied to future studies within Lake Lothing
Give any other comments on the study, for example suggestions for additional work.	N/A

Table B6.4: Checks

Item	Comments
Are the results consistent, for example at confluences?	N/A
What do the results imply regarding the return periods of floods during the period of record?	There are no gauges within the catchments, (only within Lake Lothing itself) therefore it is not possible at this stage to determine the return period of previous flood events within the catchment.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	The 100-year growth factor • Kirkley stream = 2.43 • Catchment 1 = 2.77 • Catchment 2 = 2.55
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	The 100/1000-year ratio is: • Kirkley stream = 2.12 • Catchment 1 = 2.42 • Catchment 2 = 2.61

Item	Comments
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	The specific runoff rates: • Kirkley stream = 2.44 • Catchment 1 = 4.82 • Catchment 2 = 4.22
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	There is a Kirkley stream study which shows localised flooding upstream in the catchment. This is consistent with the water levels derived here. There are no previous studies for catchment 1 and 2 that can be used as a comparison.
Are the results compatible with the longer-term flood history? **Are there any amendments to parameters after verification / Calibration?**	There is some flooding due to fluvial sources upstream of Kirkley stream however the main source of flooding in the area is from the tidal levels. There are no flooding events attributed to catchment 1 and 2.
Describe any other checks on the results	

Supporting Information

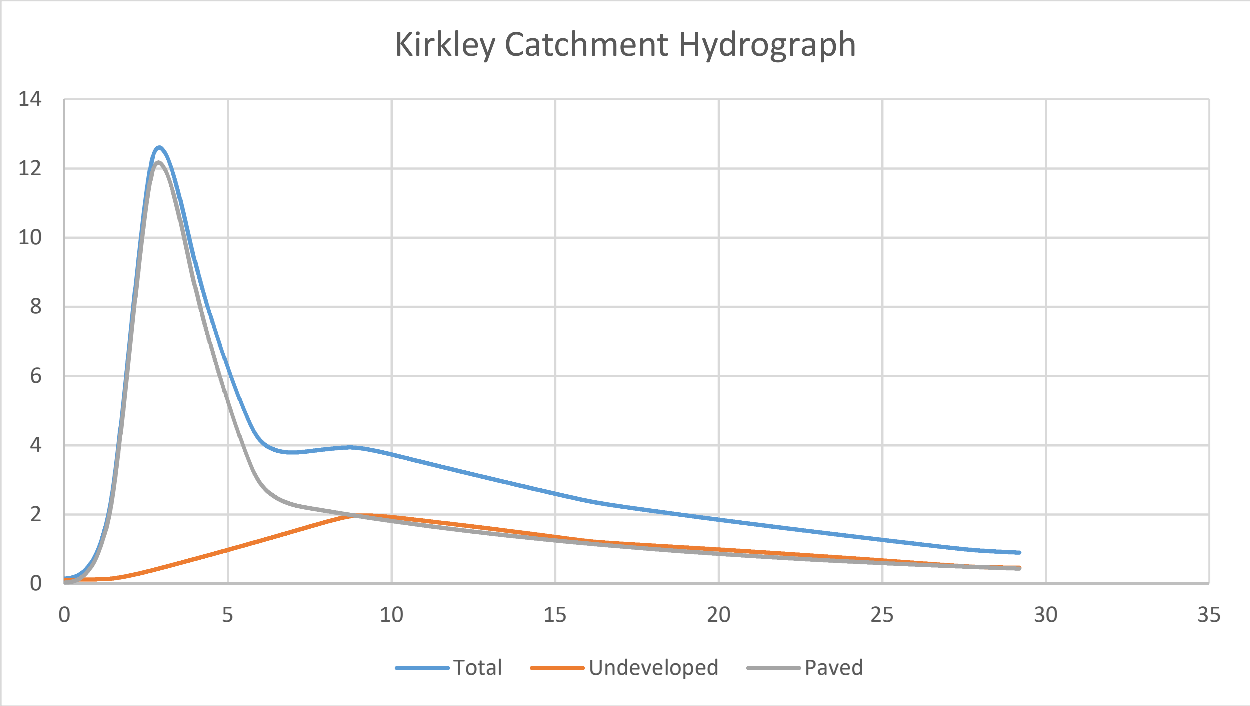

Figure BS.1, Kirkley stream hydrograph

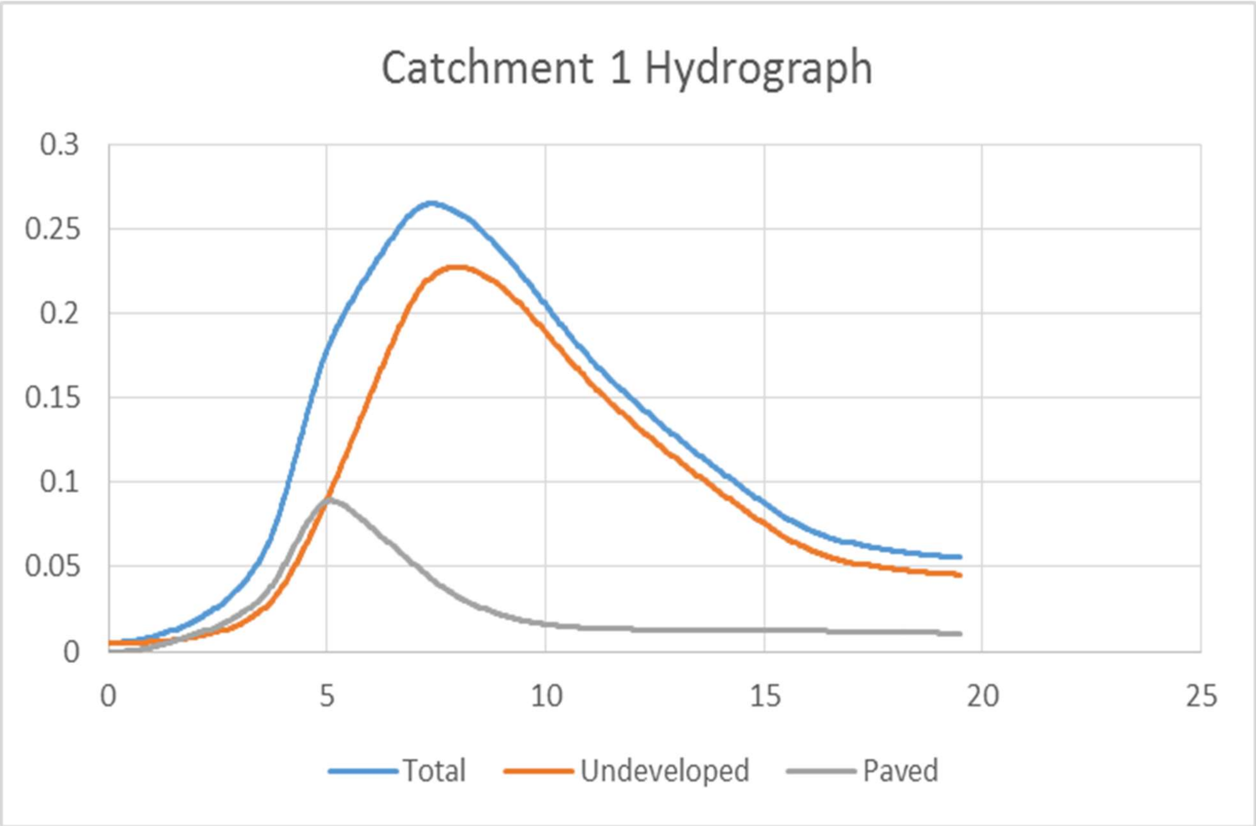

Figure BS.2, Catchment 1 Hydrograph

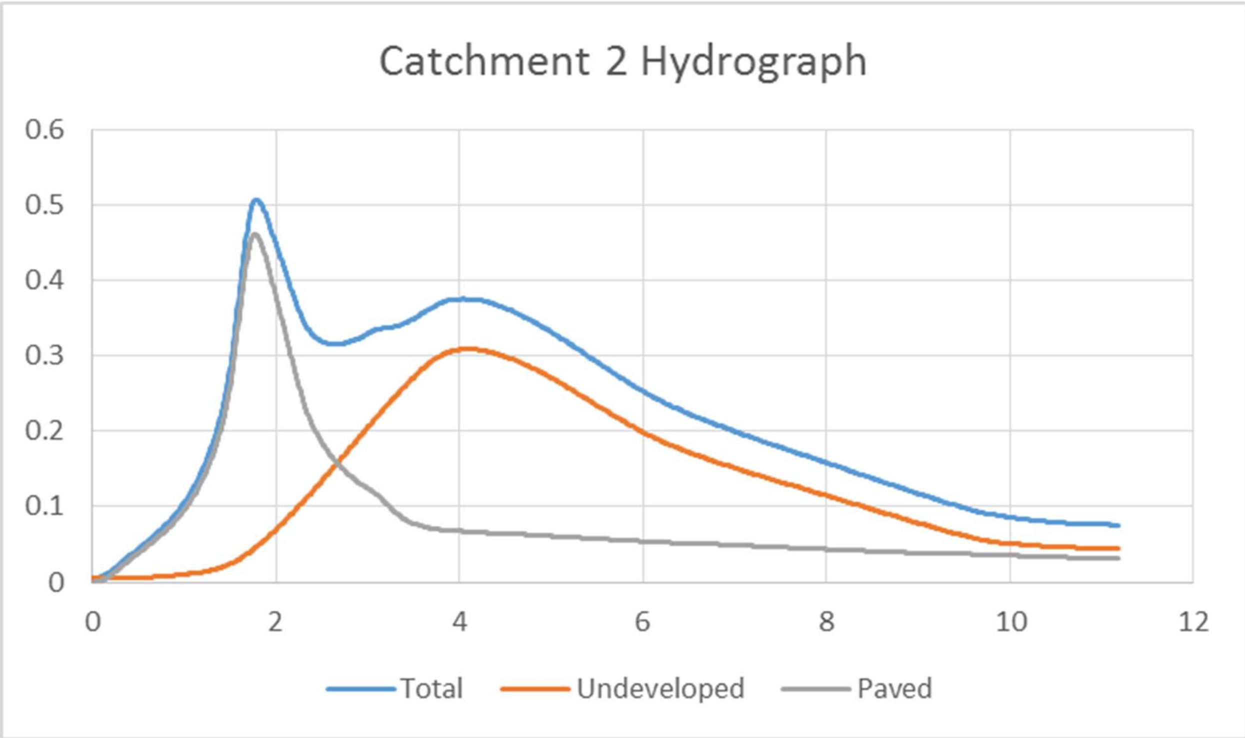

Figure BS.3, Catchment 2 Hydrograph