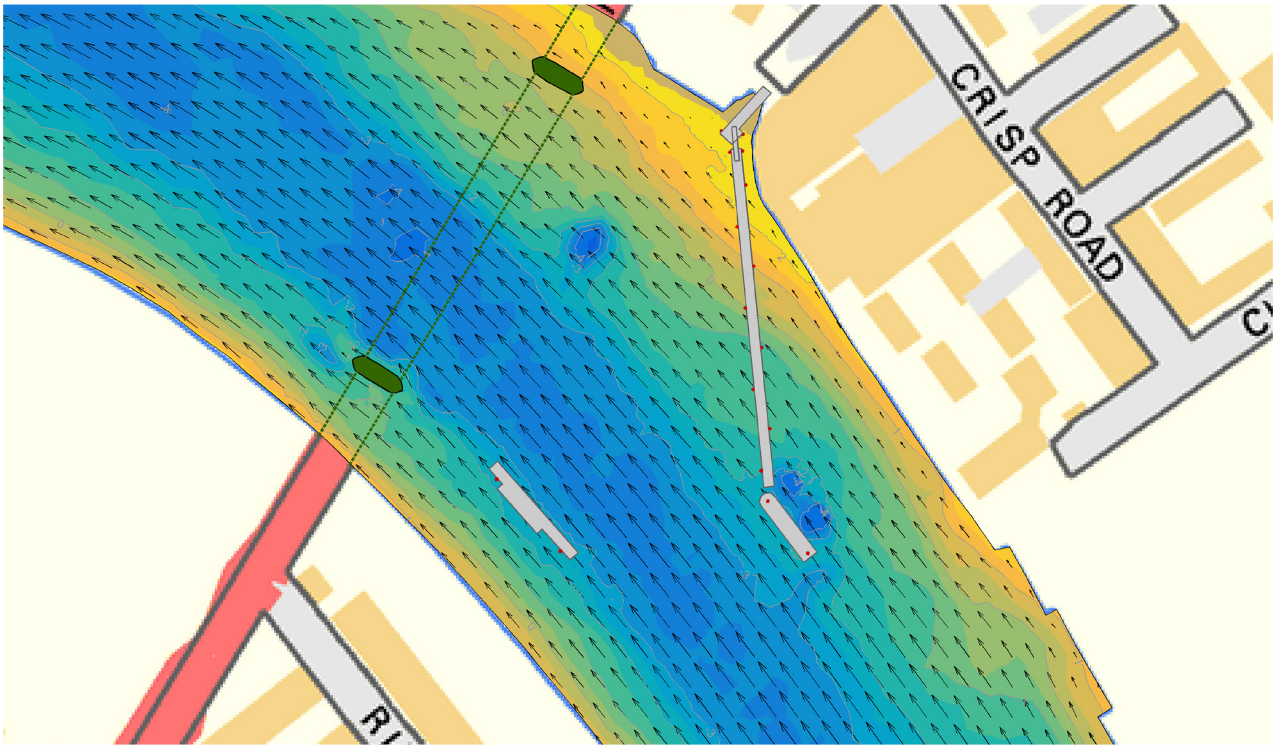
HR Wallingford
Working with water

Hammersmith Temporary Ferry

Hydrodynamic and scour assessment



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Summary

An option for a temporary ferry crossing is being investigated by Transport for London (TfL) to run nearby to the existing Hammersmith Bridge during the bridge's refurbishment. Thames Clippers supported by Beckett Rankine recently won the tender to design and develop the ferry crossing and associated marine elements. HR Wallingford have been commissioned to support the consents process, including hydrodynamic, scour, noise and ecological assessments.

This report details the methods and results for the hydrodynamic and scour assessments. The findings of the study are summarised below.

Hydrodynamics

Only very limited effects of the temporary piers and walkway on hydrodynamics are predicted. The piers and piles do provide some speed reductions due to their blockage to and drag on the passing flow. Changes greater than 0.05 m/s only occur within 30 m up and downstream of the piers. There is no discernible effect of the walkway on flows. At the time of late ebb a small area of speed increase on the Barnes foreshore is predicted. All changes are less than 0.1 m/s suggesting the effects of the temporary piers are likely to be within natural variability in flows at the site.

For a representative flow event for the outfalls that are close to the Hammersmith Temporary Pier there is the possibility for speed differences of +/- 0.1 m/s to occur at and around the pier and restraining piles during low water discharge events. Otherwise the potential effect of the outfall on the proposed structures is concluded to be extremely small.

Erosion/accretion and morphology

An analysis of changes to the peak bed shear stress calculated from the model results indicates small patches of increased maximum bed shear stress underneath the temporary piers, indicating that in these very localised areas some bed material coarsening, possibly leading to a small amount of erosion, may occur. A small area of increase in maximum bed shear stress on the Barnes foreshore is predicted, suggesting some coarsening the sediment in this area - removing some of the finer fraction material, if present.

Scour

Scour predictions are very sensitive to local geotechnical data. There is limited data at the site so assumptions based on nearby data have been used.

Local scour may occur around the proposed piles at the Hammersmith and Barnes Temporary Piers, and the piles restraining the floating walkway, to depths no deeper than 1 m, but more than likely restricted to less than 0.5 m.

This prediction is unlimited by the presence of a stronger underlying layer of clay, which is known to be present in the tidal Thames with varying thicknesses of overlying mobile material. It is the thickness of this mobile material (sandy gravel at the Hammersmith site) that will ultimately control the scour depths that develop around the piles. The limited available geotechnical data defining this layer indicates that it is less than 1 m thick in the vicinity of the works, which would limit scour depths to a similar level. It is recommended that scour predictions are updated if and when site specific geotechnical data becomes available.

The scour observed at the existing southern Hammersmith Bridge pier is observed to occur to depths of 0.6 m on the downstream side, which provides an analogy for the maximum scour depths that can be expected for the conditions at the site. The observed scour depths at the bridge help support the predictions made above.

Consideration has been given to the potential flow speed increases at the Hammersmith Temporary Pier piles during an outfall discharge event. The results show that there is limited increased risk of scour due to the proximity to the outfall.

The risk of local scour occurring of the grounded floating walkway is considered to be low. Any scour that does occur during flooding and draining is expected to be within the bounds of natural variability.

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1. Introduction

An option for a temporary ferry crossing is being investigated by Transport for London (TfL) to run nearby to the existing Hammersmith Bridge during the bridge's refurbishment. Thames Clippers supported by Beckett Rankine recently won the tender to design and develop the ferry crossing and associated marine elements. HR Wallingford have been commissioned to support the consents process, including hydrodynamic, scour and ecological assessments.

This report details the methods and results for the hydrodynamic and scour assessment, including:

1. Establishment of a hydrodynamic model of the area around the proposed ferry operation, demonstrating its effects on tidal flow for typical conditions;
2. A scour assessment of the proposed in-river structure. Similarly demonstrate any increase in scour risk for nearby third party assets due to the proposed bridge.

Numerical modelling studies for the tidal Thames carried out by HR Wallingford since 2001 have made use of the Thames Base model. This model was set up by HR Wallingford in partnership with the Environment Agency (EA) and the Port of London Authority (PLA) to aid them with their regulatory responsibilities and therefore provides a model of known provenance for the EA and PLA. The model has been extensively calibrated against many tidal and freshwater conditions and its bathymetry updated with the model's continuing accuracy confirmed several times (HR Wallingford, 2004, 2006 and 2009).

The Thames Base model has been applied to the baseline and developed 'with works' scenarios at the temporary bridge location, using updated bathymetry and a refined computational mesh.

The potential effects of the works on the physical processes of the tidal River Thames are:

- changes to tidal propagation;
- changes to the pattern and magnitude of flows around the site;
- changes to the pattern of accretion and erosion, including scour at the structure, of the sub tidal area and intertidal foreshore.

The hydrodynamic model used to simulate the effect of the proposed works is described in Section 2, while the model results are described in Section 3. The scour assessment is described in Section 4. The summarised results of both parts of the assessment are undertaken given in Section 5.

1.1. Project appreciation

Temporary piers to provide passenger access to the ferry will be located on either side of the river (Figure 1.1), immediately downstream of Hammersmith Bridge. Hammersmith Temporary Pier on the north bank will land at the end of Queen Caroline Street, while Barnes Temporary Pier will land on the Thames towpath on the south bank.

Both the Hammersmith Temporary Pier and Barnes Temporary Pier which make up the Hammersmith Ferry service are to be temporary installations for an intended period of 3 years with a maximum of 5 years. The design of each structure has therefore been completed with ease of removal as a key criterion.

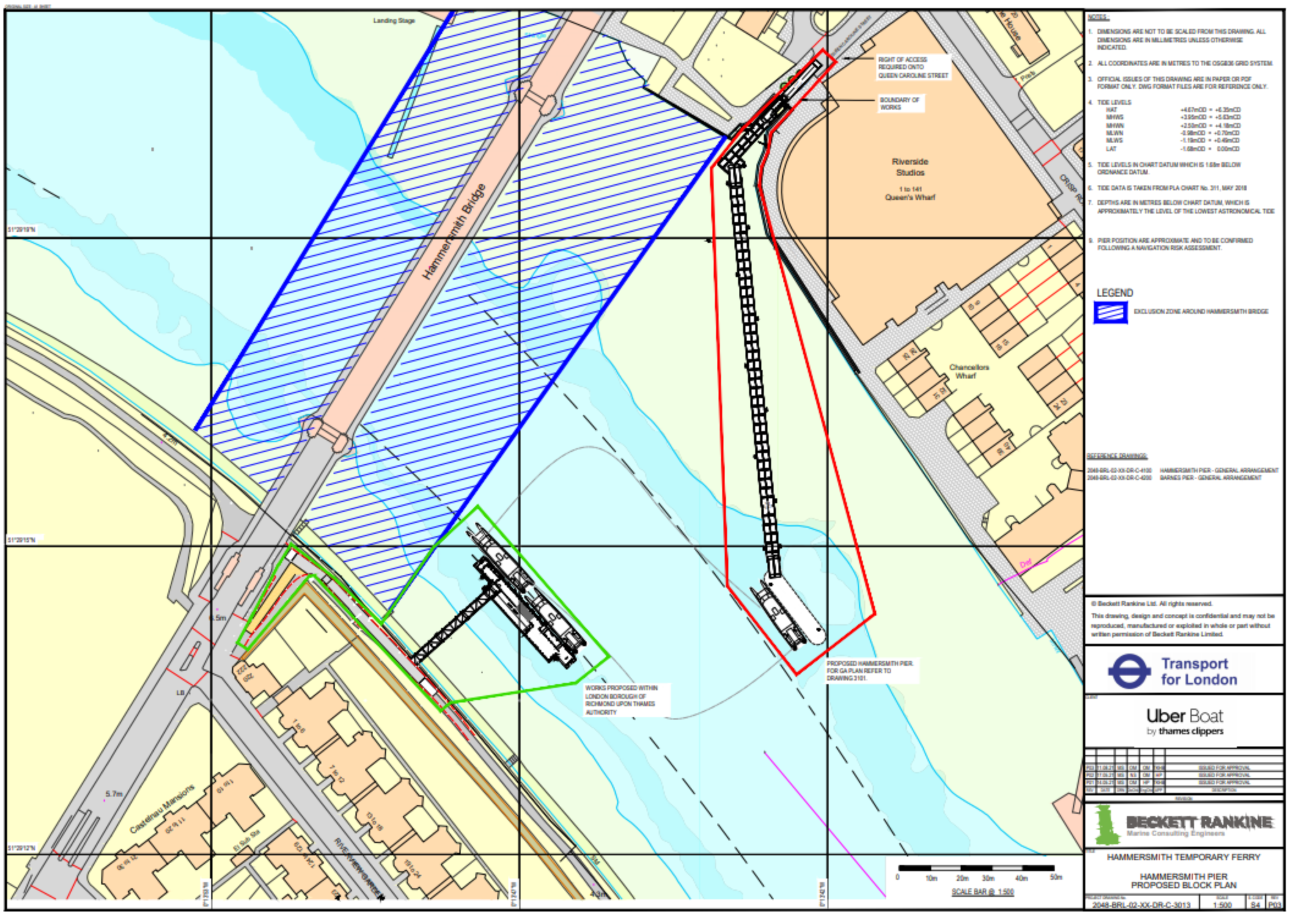

Figure 1.1: General arrangement of the temporary ferry project.

Source: Beckett Rankine, Drawing 2048-BRL-02-XX-DR-C-3013 P03

1.1.1. Hammersmith Temporary Pier

The proposed Hammersmith Temporary Pier (Figure 1.2 and Figure 1.3) is to land on the public slipway located at the end of Queen Caroline Street. The slipway is seldom used and is closed off with timber flood boards. Access to the pier is to be via a lightweight steel ramp that will span over the flood boards.

A modular floating walkway (using units by EZ Dock) will span between the flood defence wall and a second-hand barge, modified for use as a pier. The walkway will be restrained by tubular piles of up to 0.5m in diameter. The required piling is to be minimised to avoid major impacts and disturbance to the river environment. As identified in Figure 1.3, parts of the floating walkway will ground at low water.

The barge will be restrained by a pair of spud legs – these have been selected given their temporary nature and lesser impact when compared to piles. The pier is skewed downstream to facilitate passage of large vessels beneath Hammersmith bridge (the bridge is open for occasional navigation when no works are in progress on the bridge). The position also makes use of the deeper water related to the outfalls as shown in Figure 1.2.

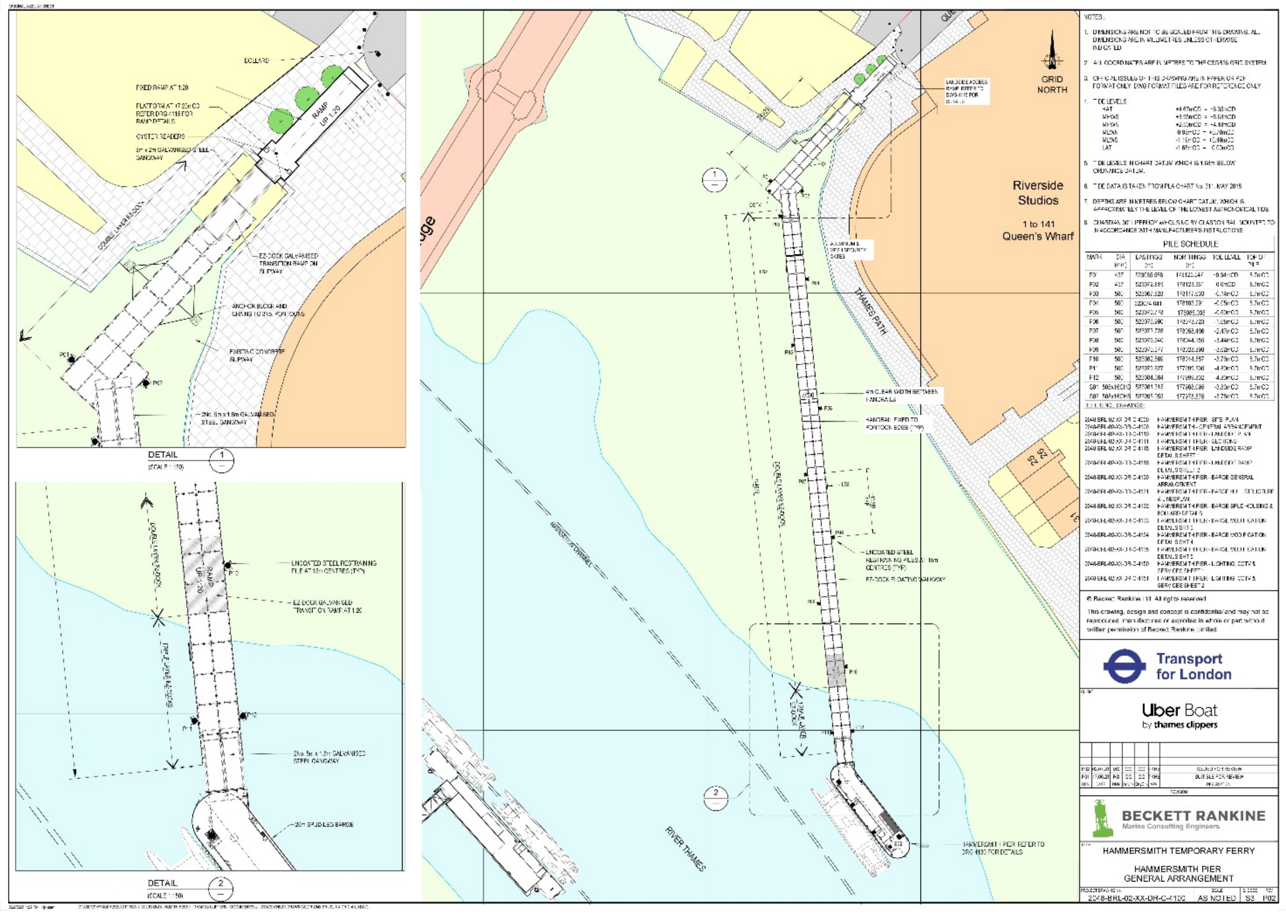

Figure 1.2: Hammersmith Temporary Pier general arrangement

Source: Beckett Rankine, Drawing 2048-BRL-02-XX-DR-C-4100_P02 HSMTH BRG-GA

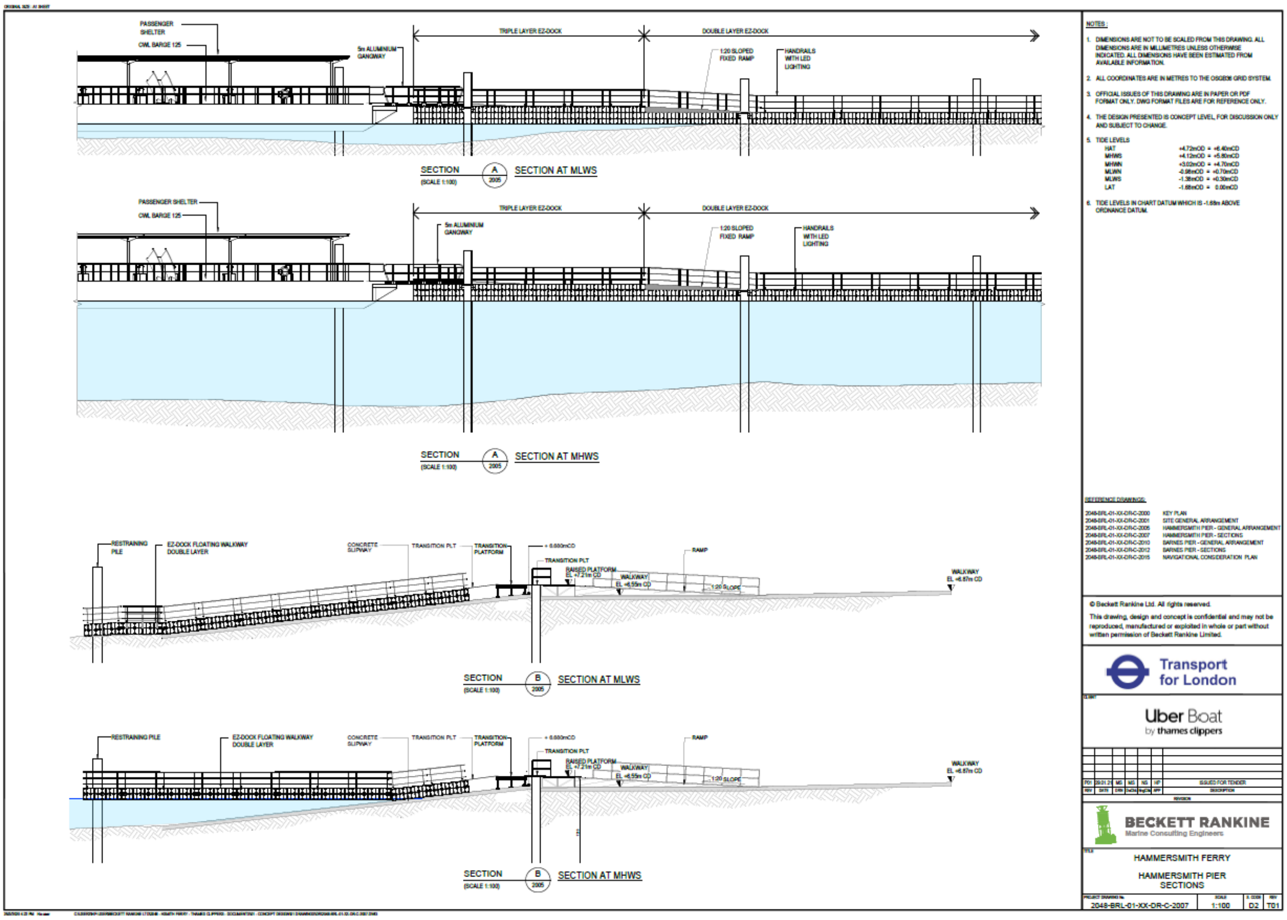

Figure 1.3: Cross-sections of the floating walkway proposed to access Hammersmith Temporary Pier

Source: Beckett Rankine, Drawing 2048-BRL-01-XX-DR-C-2007_T01

1.1.2. Barnes Temporary Pier

The proposed Barnes Temporary Pier (Figure 1.4) is formed from the old Savoy Pier, itself a temporary structure, which will be repurposed for this development. The pier will be modified such that is restrained by a pair of spud legs rather than its current radial arms to minimise the impact on the foreshore.

Access to the pier is by an aluminium linkspan, connecting to the landside towpath. The towpath is located beneath Flood Defence Level and floods on some spring tides. As part of the works, a lightweight steel frame walkway will be installed to allow dry access to the pier.

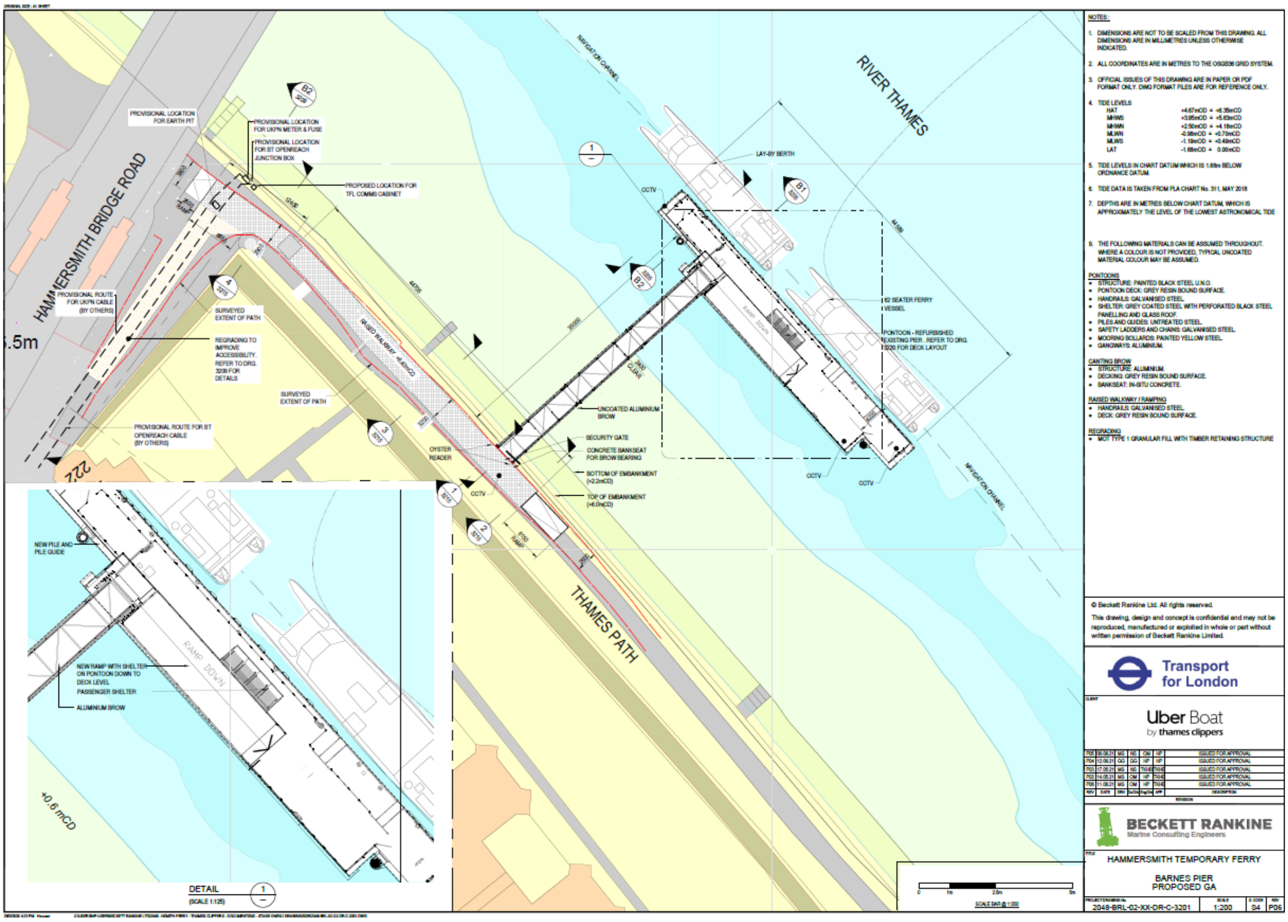

Figure 1.4: Barnes Temporary Pier general arrangement

Source: Beckett Rankine, Drawing 2048-BRL-02-XX-DR-C-3201 P06 BARNES PIER PROPOSED GA (002)

1.1.3. Program

Offsite construction activities are underway. Works on site are due to start in early September and are to be completed by end of October. These dates continue to be subject to attaining the relevant licensing and consents for the works.

1.1.4. Plough dredging

Approximately 120 m^3 of sediment is to be levelled by plough dredging in and around the area of the Hammersmith Temporary Pier (Figure 1.5), with an additional c.34 m^3 to be plough dredged at Barnes Temporary Pier (Figure 1.6), to allow vessels to come alongside at low tide.

The maximum height to be levelled at any location is circa 450 mm. The total c.154 m^3 of sediment will be plough dredged downstream. The effect on hydrodynamics of such small volumes of removed material are considered to be within the limits of model accuracy, and are therefore not worthwhile including in the hydrodynamic modelling assessment.

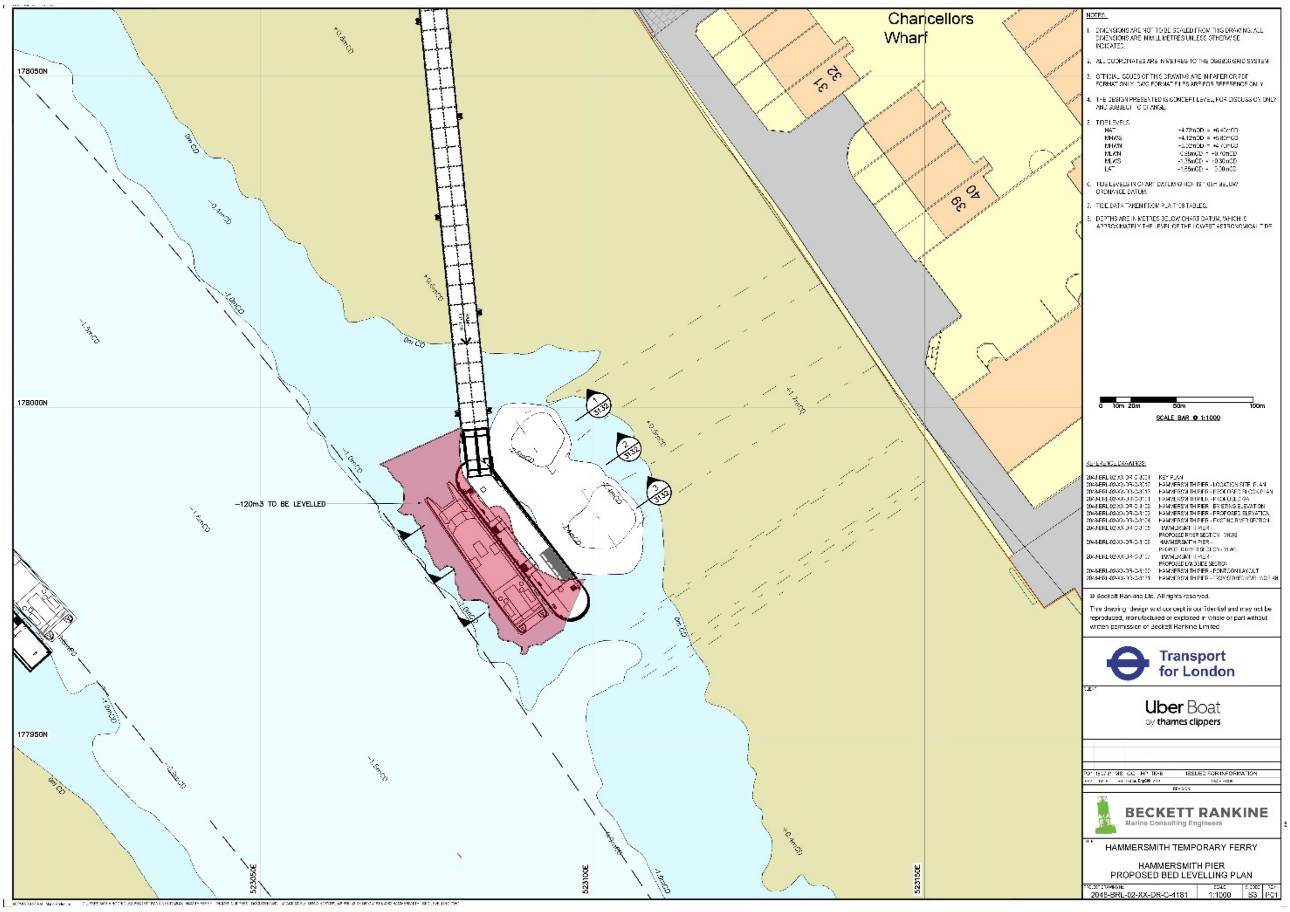

Figure 1.5: Location of sediment to be levelled via plough dredger at Hammersmith Temporary Pier

Source: Beckett Rankine, Drawing 2048-BRL-02-XX-DR-C-4181 HAMMERSMITH - BED LEVELLING PLAN

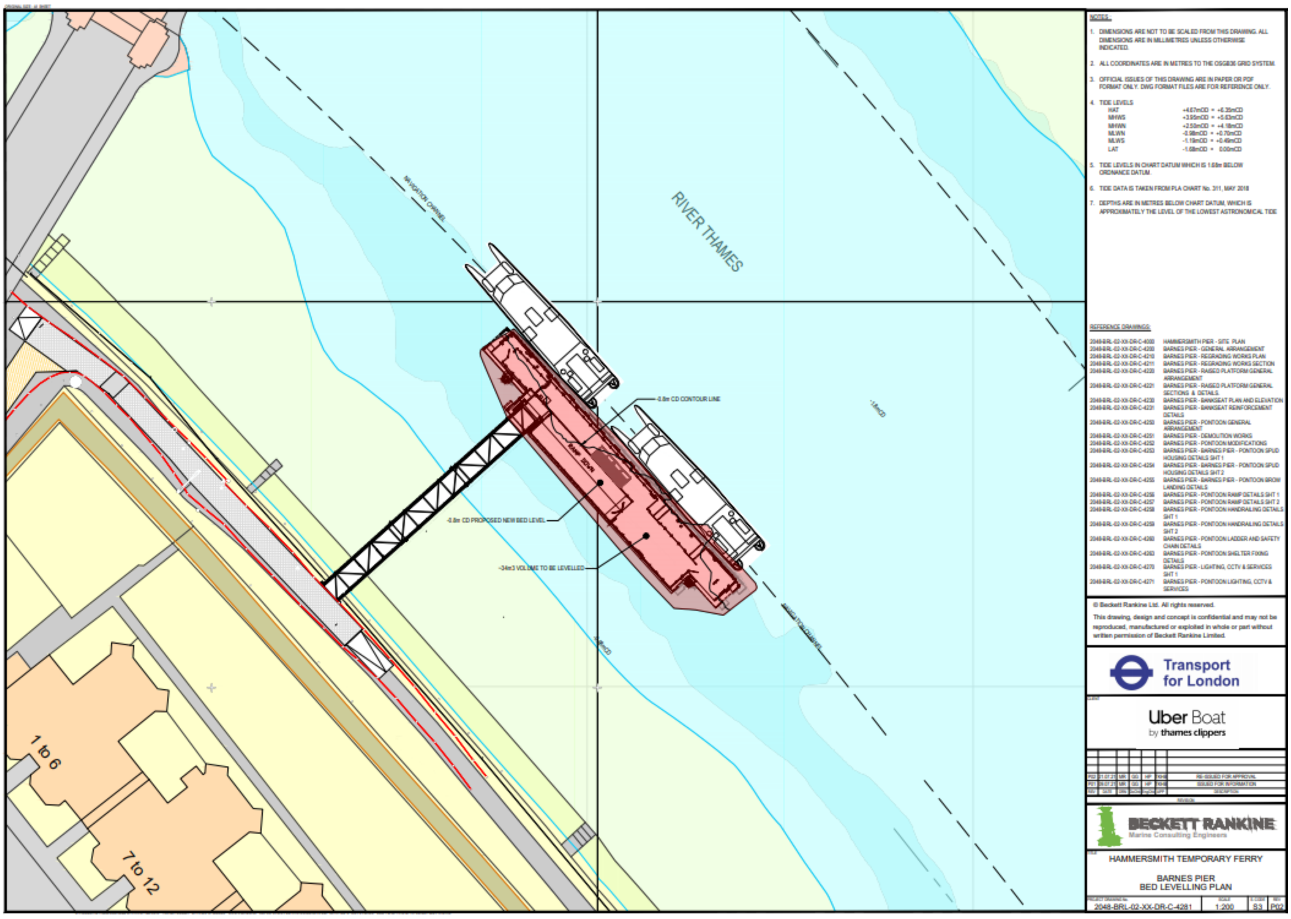

Figure 1.6: Location of sediment to be levelled via plough dredger at Barnes Temporary Pier

Source: Beckett Rankine, 2048-BRL-02-XX-DR-C-4281_P02 BARNES - BED LEVELLING PLAN

1.1.5. Construction

The first activity on site will be the bathymetric and UXO surveys. A proof dig at the pile line will also be carried out. Following this, the temporary piers will be installed following Red7 Marine's method statement. All piles will be driven by the crawler crane mounted on a jack-up barge. In the case of the 4 most northern piles, a landside excavator will act as the piling gate. For the remainder of the piles the excavator will be mounted on the jack-up barge where it will also act as a piling gate. A supply barge will operate adjacent to the jack-up barge to store the piles. Where necessary for the spud leg piles at Barnes Pier, the excavator will be mounted on the supply barge.

Non-percussive piling methods will be used to install the tubular piles. Soft-start vibratory piling methods (high-frequency, variable moment resonant free vibratory hammer) will be used instead to embed the piles ~4 m into the riverbed, therefore, the noise and vibratory effects will be significantly reduced and less harmful to the surroundings. Piles will be driven dry where possible, and in the minimum water level possible where not possible. The plant requires a minimum water depth of 2 m to safely carry out the works. The methodology utilises low water piling techniques to reduce noise and vibration effects throughout the works.

2. Hydrodynamic model

2.1. Model set up

2.1.1. Model description

The Thames Base model has previously been used to investigate the hydrodynamic regime around developments in many areas along the Thames as well as investigating the estuarine hydrodynamic and sediment transport processes themselves. The modelling tool used was TELEMAC2D. Developed by EDF-LNHE, TELEMAC2D solves the 2D shallow water equations which assume the vertical structure of the flow can be represented as a logarithmic profile, an appropriate assumption in a well-mixed, macro tidal estuary such as the Thames. The model uses a triangular grid which allows the model mesh resolution to continually vary in space resulting in good representation of features such as the various bridge piers, vessels, structures and the riverbank.

The Thames Base model covers the whole tidal Thames from Southend to Teddington to enable straightforward setting of boundary conditions. Noting the location of the bridge in an area of generally shallow water with large areas of drying, the model was run in 2-dimensional depth averaged mode.

2.1.2. Model set-up

For the present study the shape of the piers and piles were included in the model mesh as part of the refinement of the model in the study area. The resultant model mesh included a smallest mesh size of 0.2 m, and is shown for the wider study area in Figure 2.1 and in more detail at the proposed works in Figure 2.2.

The aim of the modelling is to investigate the effect of the works on the overall tidal and sediment transport regimes of the tidal River Thames. To implement the effect of piled structures in the model, an approach based on the drag on the passing flow was used. The amount of drag was calculated from the size and shape of the piles for the structures considered within a 0.5 m square polygon at the location of each pile:

- Hammersmith Temporary Pier – 2No, upstream and downstream;
- Barnes Temporary Pier – 2No, upstream and downstream;
- Floating walkway – 9No, alternating at 15 m intervals from the riverward end to the slipway.

For the temporary piers, the water surface was suppressed to a level equivalent to their draught; communicated by Beckett Rankine to be 0.5 m and 0.61 m for the Hammersmith and Barnes Temporary Piers, respectively. The draught of the floating walkway is stated as 0.06 m unloaded. To account for potential loading from passengers etc, the floating walkway draught implemented in the model is 0.1 m.

To account for the potential effect of the episodic discharge associated with the two outfalls approx. 15 m shoreward of the Hammersmith Temporary Pier, an additional scenario was run including a source at the position shown in Figure 2.1. In lieu of timely available data, the values associated with the discharge were estimated based on HR Wallingford’s experience of similar, relatively small, outfalls as:

- Discharging for 1 hour before and after local HW, ramping up to and down from total 3 m^3/s at HW;
- Discharging for 1 hour before and after local LW, ramping up to and down from total 3 m^3/s at LW.

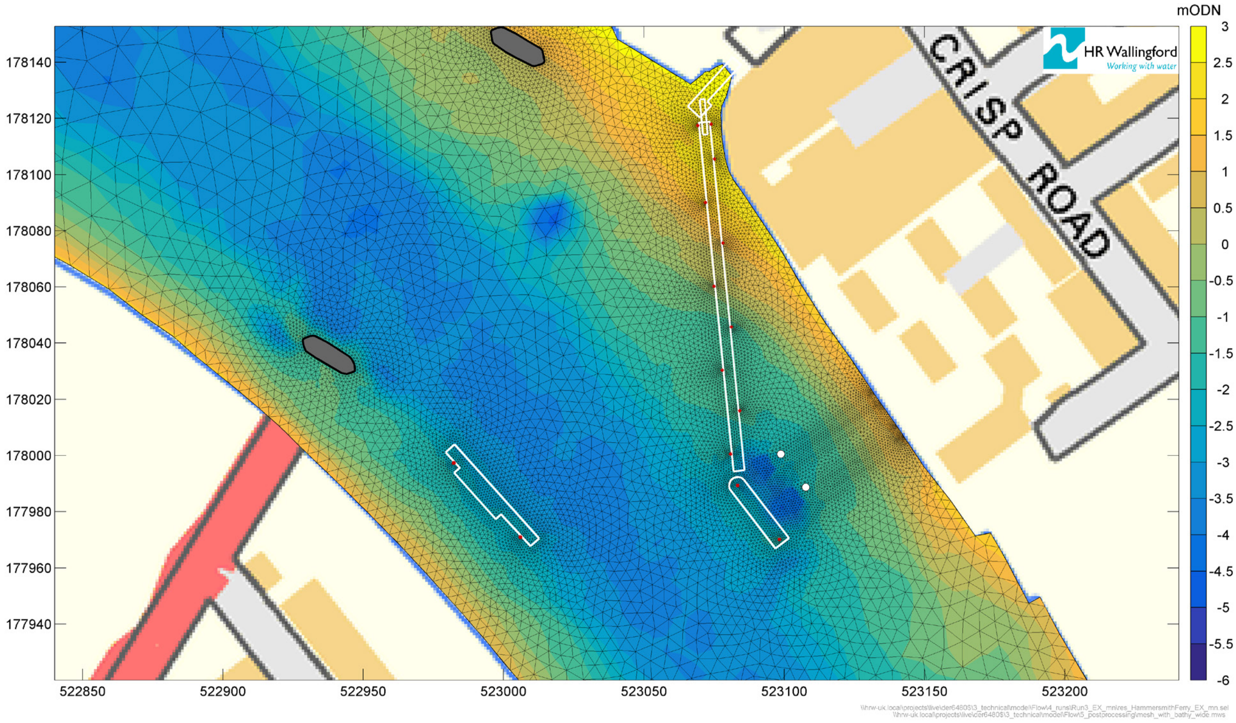

Figure 2.1: Model mesh and bathymetry in the wider study site, including the temporary piers in white, the piles in red and the outfall locations as circles

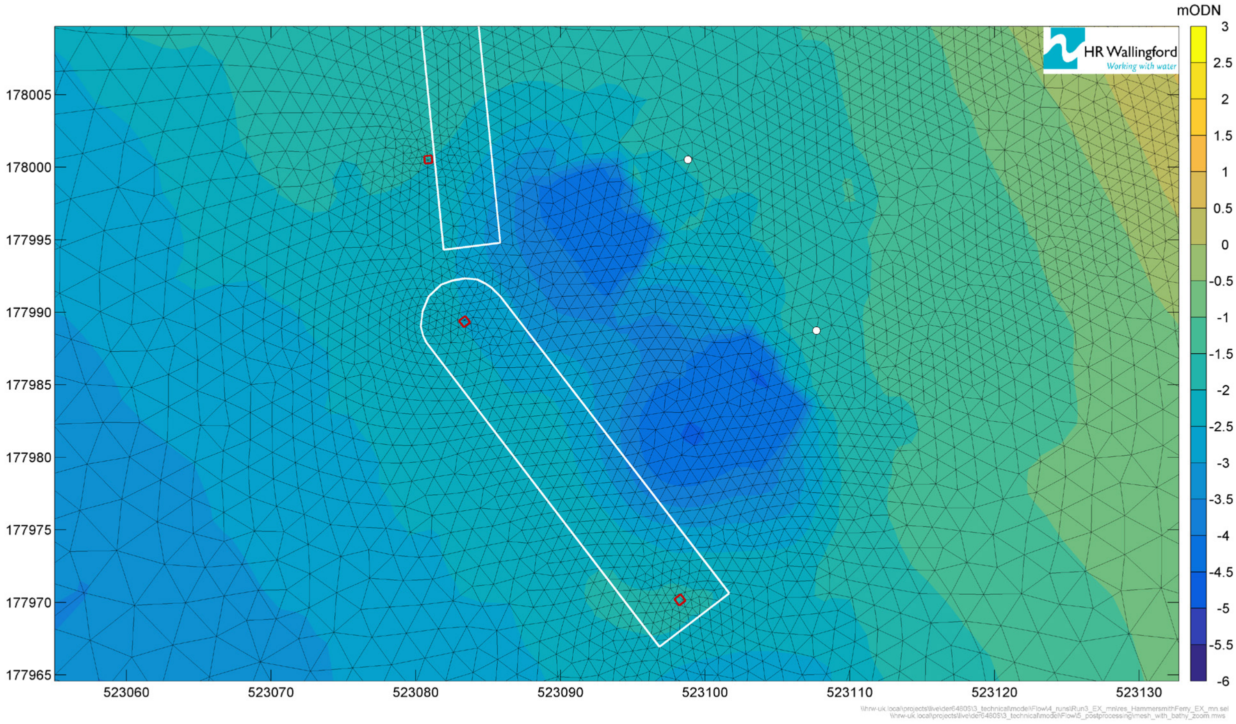

Figure 2.2: More detailed model mesh at the Hammersmith Temporary Pier

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2.1.3. Bathymetry data

The bathymetry database of the Thames Base numerical model was developed from the bathymetric data published by the Port of London Authority (PLA). All depths are reduced to a common flat datum of Ordnance Datum Newlyn (ODN) from the local Chart Datum which changes up the tidal River Thames in line with the change to low water level. In the area of Hammersmith Bridge, Chart Datum is 1.68 m below ODN.

Additional bathymetry data at the site was provided by TfL. These data were reduced to ODN and incorporated into the Base model bathymetry. The final model bathymetry is shown in the vicinity of the project area in Figure 2.3, noting the deep areas related to the presence of the outfalls at Hammersmith Temporary Pier, and another immediately to the south of Hammersmith Bridge.

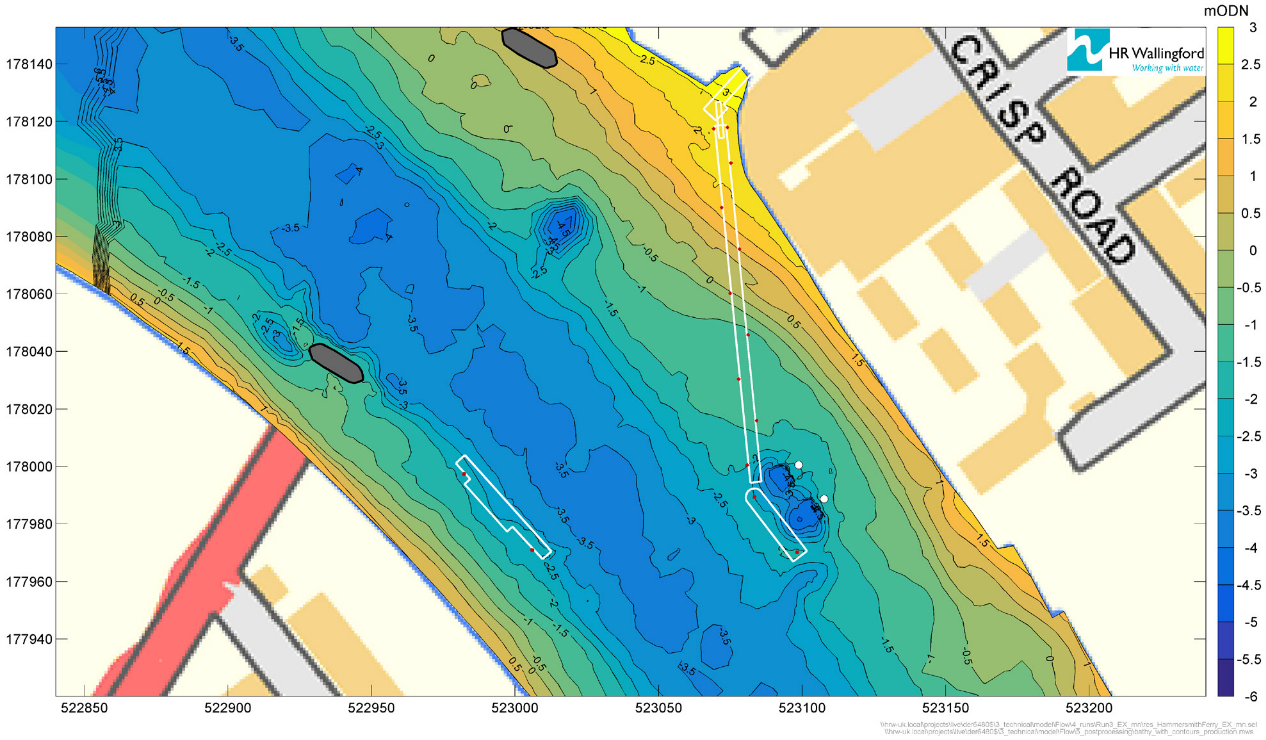

Figure 2.3: Model bathymetry in the vicinity of the project area

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2.1.4. Boundary conditions

The simulations require the imposition of landward and seaward boundary conditions. The model domain covers the whole length of the tidal Thames Estuary so the tidal elevation at Southend-on-Sea and water discharge at Teddington Weir define the seaward and landward boundary conditions respectively.

Data for the tidal elevation boundary comes from those observed at the Port of London Authority's tide gauge on Southend Pier. The freshwater flow data is calculated from the gauged flow at Kingston (http://nrfa.ceh.ac.uk/data/station/meanflow/39001).

2.2. Choice of hydrodynamic conditions

2.2.1. Tidal conditions

The typical tidal conditions used comprised a series of spring tides of range 5.06 to 5.86 m at Southend-on-Sea; which include high water levels of 2.80 to 3.18 m above ODN as shown in Table 2.1.

The typical tidal conditions chosen were chosen to be close to a mean spring tide at Southend-on-Sea (tide range 5.3 m, HW of 2.9 mODN).

As well as the tidal effect the tide gauge observations include non-tidal effects such as those from meteorological factors (wind, pressure). The difference of the observed tidal signal from that predicted for purely tidal factors is also shown on Table 2.1. The difference is of the order of 0.2 m, ranging from a small negative surge at the start of the period to a small positive surge. This amount of difference from the predicted tide is small confirming the chosen period as reasonably typical of the tidal conditions that occur.

Table 2.1: High and low waters of imposed boundary tide for typical spring tidal conditions

Date	Time (GMT)	Observed (mODN)	Difference from predicted tide (m)
28-Sep-04	00:10	2.93	-0.12
	06:20	-2.26	-0.13
	12:10	2.8	-0.11
	19:00	-2.68	0.03
29-Sep-04	00:50	3.18	0.06
	06:50	-2.08	0.20
	12:40	3.06	0.09
	19:30	-2.64	0.05

2.2.2. Freshwater flow

The freshwater river flow at the tidal limit of the Thames Estuary is gauged and recorded at Teddington/Kingston. The data is available via the UK National River Flow Archive (http://www.ceh.ac.uk/data/nrfa/uk_gauging_station_network.html).

The monthly average flows are shown in Table 2.2. For simulations of typical conditions the annual mean flow of 65 m^3/s was used.

Table 2.2: Monthly mean daily freshwater flow at Teddington

	Flow (m^3/s)		
Month	Mean	Maximum	Minimum
January	126	581	1
February	122	527	4
March	101	709	6
April	75	348	2

Month	Flow (m^3/s)		
	Mean	**Maximum**	**Minimum**
May	52	330	2
June	35	377	2
July	22	204	1
August	21	188	0
September	22	581	0
October	38	371	0
November	71	800	2
December	101	547	2
All	65	800	0

Source: UK National River Flow Archive

3. Model results

3.1. Description of model results presentation

The study programme comprised three simulation scenarios:

- typical hydrodynamic conditions for baseline layout;
- typical hydrodynamic conditions with proposed works in place;
- typical hydrodynamic conditions plus outfall discharge for two hours across low water and high water, respectively, with proposed works in place.

The simulations are presented in four ways to help assessment of the near and mid field hydrodynamic impacts of the works.

Vector plots of flow alignment

These figures present snapshots of the model results focussing on the effect of the proposed works on flow direction and thus alignment of flow at the works. The proposed flow vectors are shown in red and are overlaid by the existing conditions flow vectors in black. Any effect is indicated by a change in the vector direction.

The times of peak ebb and peak flood tides were chosen to be representative of the largest ebb or flood tide currents at the works site to provide a precautionary view of the effects of the works on current speeds. In addition the results are shown during the late ebb when of the greatest difference between baseline and proposed has been predicted, to fully represent the variation in effect across the tidal cycle.

Spatial plots of current speed and speed difference

These figures present snapshots of the simulated current speed at the times of peak ebb and flood tide. The upper two frames show contoured current speed magnitude for the hydrodynamic conditions tested. The top left frame shows baseline conditions, and the top right frame shows the proposed conditions. The bottom frame shows the difference in speed magnitude when comparing the with works scenario to the baseline conditions. Yellow through to red colours indicate increases in flow speed, while green through to blue colours indicates decreases in flow speed.

Generally in presenting model predictions in the tidal River Thames, any changes less than 0.1 m/s are not plotted as they are considered insignificant compared to the peak currents or the natural variability that occurs in the area. These limits have been reduced here so that the differences between the model scenarios can be more easily discerned. However it should be noted that changes less than 0.1 m/s are unlikely to be significant for the purposes of considering the navigational or morphological implications of the works.

Time series plots of current speed and direction

These plots are included to provide additional information throughout the tidal period. Eight points covering the area around the proposed temporary piers have been chosen to characterise the effects. The locations of the chosen points are shown in Figure 3.1, and have been selected to demonstrate the through-tide impacts upstream and downstream of the proposed design, on the foreshore under the floating walkway, and at the existing bridge pier to understand any potential effects on this third-party asset.

At each point, the baseline current speed and direction (speed plotted as a blue line, direction as a green square) are overlaid with the results for the 'with works' scenario (speed plotted as a black line, direction as a black diamond).

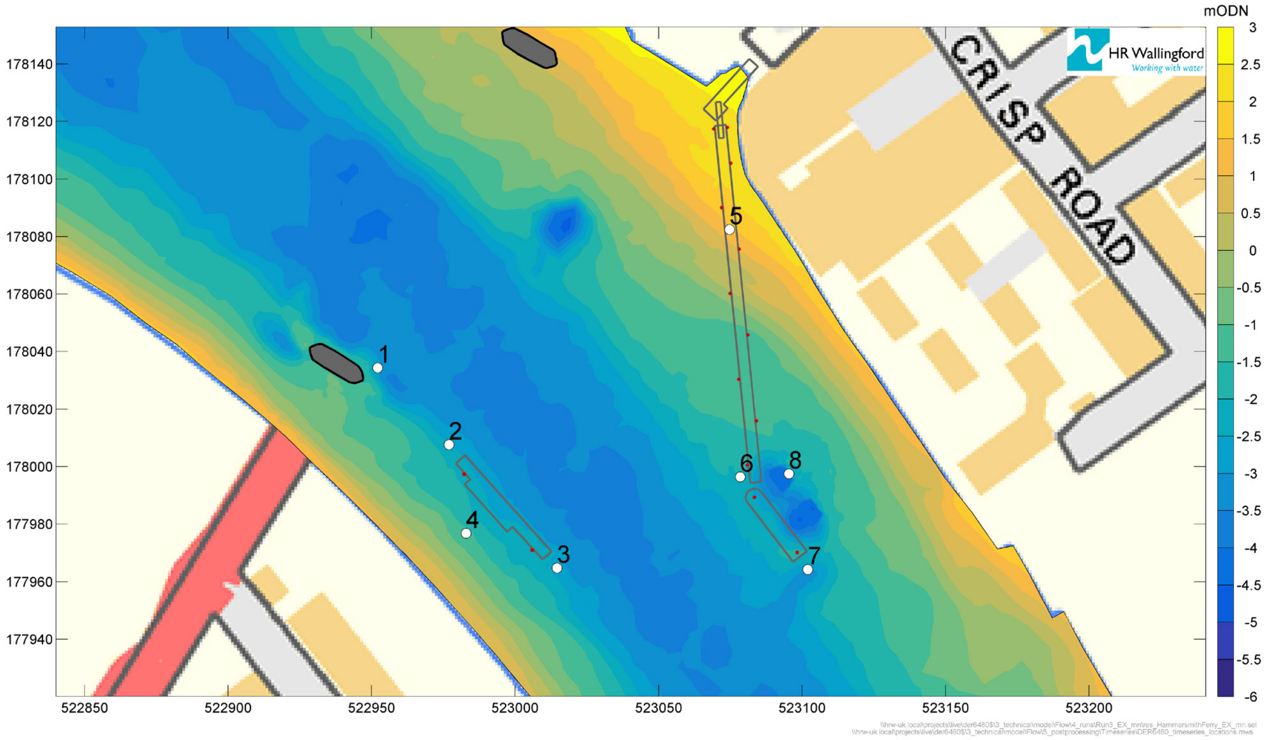

Figure 3.1: Time series locations selected for assessment

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Spatial plots of maximum bed stress

The first-order implications of the predicted currents for the bed sediments and morphology of the area were assessed by analysis of the type of bed material that would be expected for the bed shear stress generated by the currents alone. The bed shear stress is the force of the passing flow on the riverbed and is used as a measure of the potential for the bed to be eroded or to allow mobile sediment to settle permanently onto the bed. To allow comparison of the results the predicted bed shear stresses were coloured according to the type of material that would be expected to be present for the given peak bed shear stress.

For lower values of peak bed shear stress two colour bandings were used; one for bed shear stress low enough to allow long term accretion (build up) of fine, muddy sediment on the bed; and a second colour band for bed shear stresses which would allow temporary fine sediment deposition. For higher values of bed shear stress three bandings were used based on the critical stress required to mobilise sediments of diameter up to 5 mm, 10 mm and 20 mm.

In reality the riverbed is likely to be stronger than that predicted by consideration of sediment size alone due to the additional forces of local wind or vessel generated waves and turbulence near structures. The presence of a mix of sediment types would also be expected to reduce sediment movement. Allowing for the above caveats, this simple methodology is considered to be a useful tool to investigate where the bed may experience a change in currents sufficient to allow redistribution of the bed material.

The figures are coded to the following colours:

- Orange - Bed Stress values allowing fine sediment accumulation;
- Yellow - Bed stress values allowing occasional fine sediment accretion;
- Blue - Bed stress values appropriate for sand and gravel up to 5mm;
- Pink - Bed stress values appropriate for gravels 5mm – 10mm;
- Purple - Bed stress values appropriate for gravels 10mm – 20mm;
- Red - Bed stress values appropriate for gravels 20mm+.

3.2. Flow alignment

Figure 3.2 and Figure 3.3 illustrate the baseline (existing) flow direction (black vectors) and the effect of the proposed temporary piers (overlaid red vectors) at times of peak ebb and flood tide, respectively. Any effect is indicated by a change in the vector direction. Both of the temporary piers appear well aligned to the flow along the longitudinal axis (i.e. the baseline current vectors line up with the along-river axis of each structure). There is no discernible change in the current direction as indicated by the vectors for the proposed case.

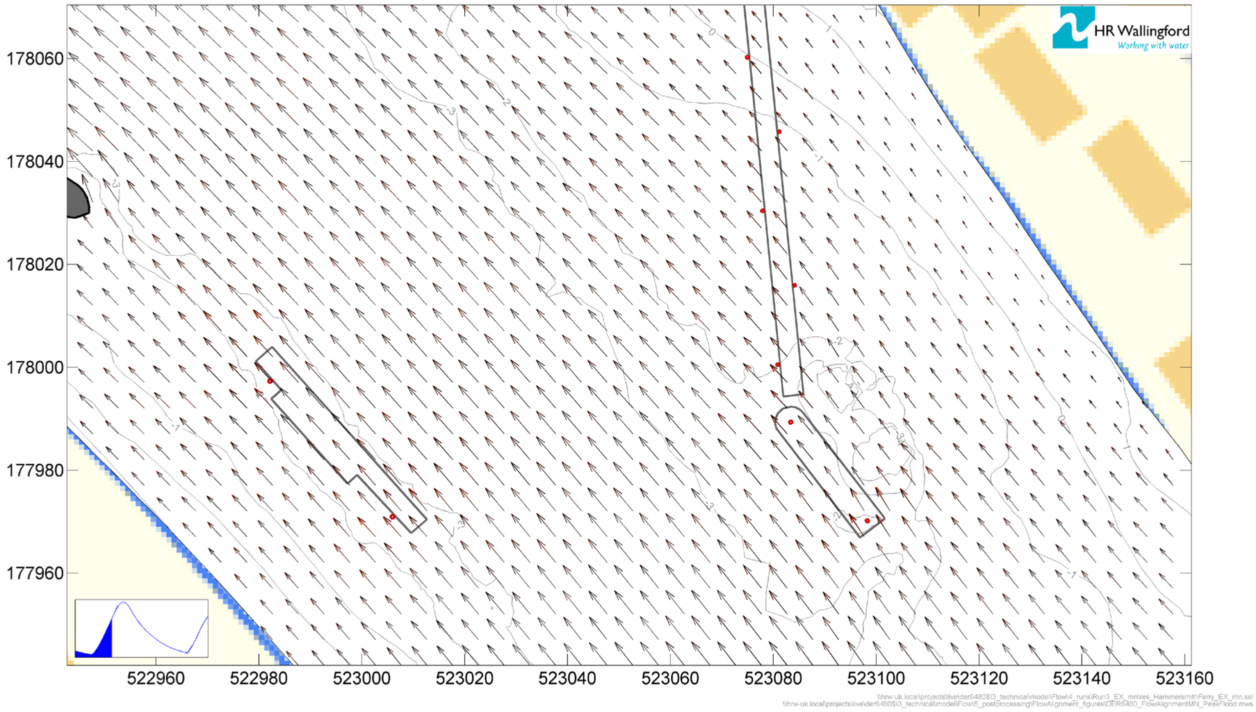

Figure 3.2: Flow alignment, peak flood depth averaged current. Black and overlying red arrows indicate flow direction for baseline and proposed cases, respectively, with depth contours in light grey

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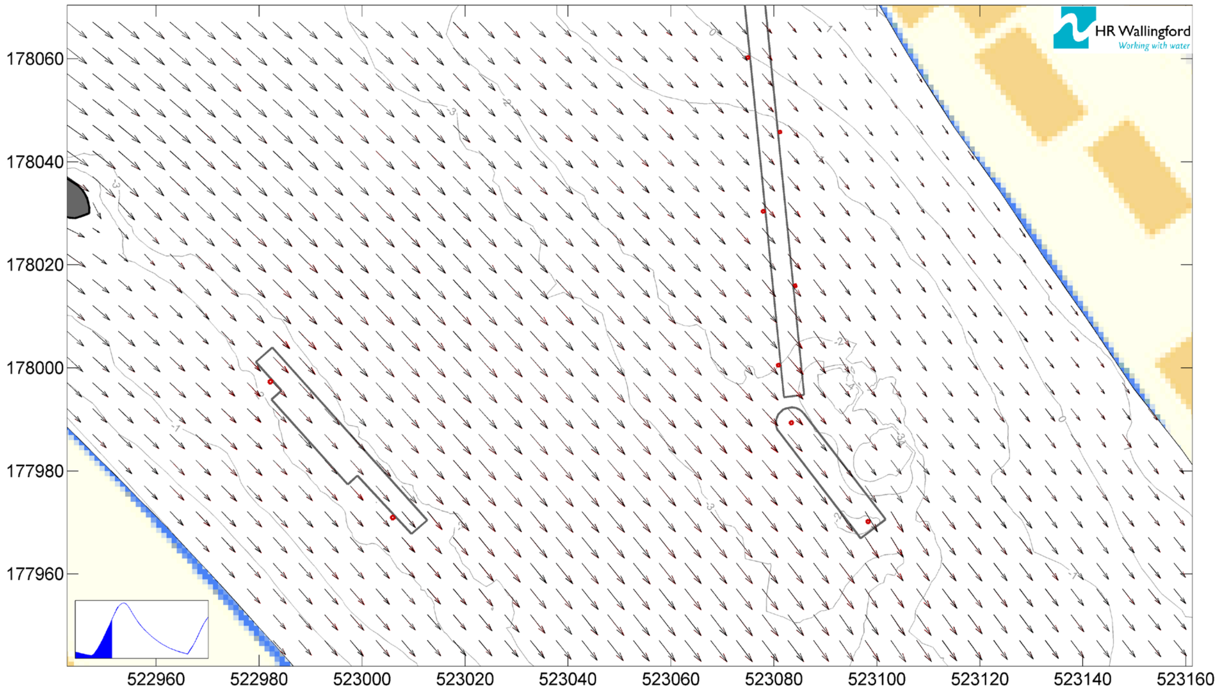

Figure 3.3: Flow alignment, peak ebb depth averaged currents. Black and overlying red arrows indicate flow direction for baseline and proposed cases, respectively, with depth contours in light grey

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3.3. Impact on hydrodynamics – spatial plots

The impacts of the works associated with the proposed temporary piers are shown as spatial plots in this section, and secondly as temporal plots in the following Section 3.4.

In the spatial plots, the impacts of the proposed works are shown at times of peak currents for mean spring tide and river flow conditions, to indicate the likely maximum extent of any effect. Showing first currents at *peak flood* in Figure 3.4 to Figure 3.6, followed by currents at *peak ebb* in Figure 3.7 to Figure 3.9. Peak flood flow speeds reach a maximum of 1.5 m/s in the main channel between the bridge piers, reducing to 1.3 m/s for peak ebb conditions.

The results are also shown for the largest differences evident across the tidal cycle, occurring in the late ebb as water depths decrease but there is still up to 1.3 m/s of flow speed in the main channel. The figures for the late ebb are provided in Figure 3.10 to Figure 3.12.

Flow speed differences between baseline and proposed conditions are for the most part manifest as speed reduction due to the drag associated with the piles and blockage due to the piers, which the model predicts will cause flow speed decreases locally up to 0.2 m/s but generally less than 0.1 m/s. For the later ebb case, there are small areas of flow speed increase, very locally up to 0.2 m/s at the up and downstream ends of the piers, associated with the reduced water depth caused by the draughts of the floating piers. There is also a small footprint of flow speed increase less than 0.1 m/s on the foreshore at Barnes Temporary Pier apparent for the later ebb case but not for either of the peak flow cases.

The upstream and downstream footprints of flow speed differences in Figure 3.6, Figure 3.9 and Figure 3.12 are very localised, with flow speed differences on the whole less than 0.1 m/s. These changes are within the natural variability in currents that occurs in the area due to changes in tide and river flow and will not significantly impact on the main navigation channel. Additionally, there is no discernible impact of the Hammersmith Temporary Pier on the nearby outfalls.

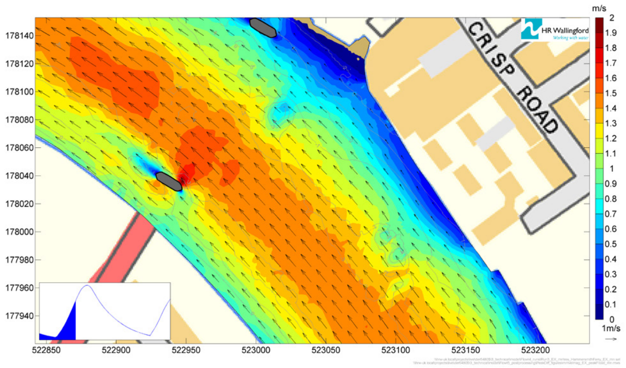

Figure 3.4: Baseline conditions, peak flood depth averaged currents

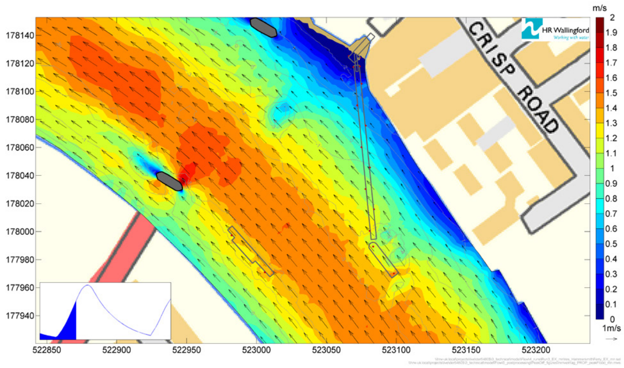

Figure 3.5: With proposed changes, peak flood depth averaged currents

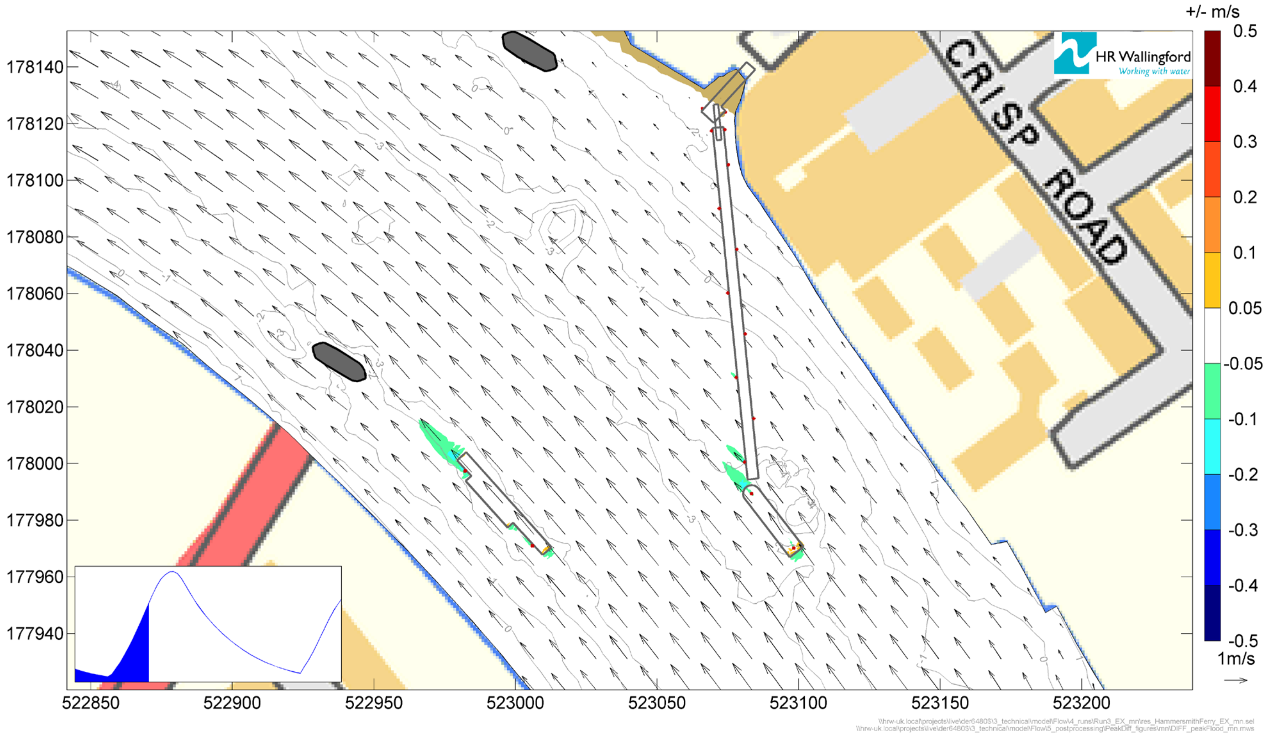

Figure 3.6: Difference in peak flood depth averaged currents associated with the proposed changes

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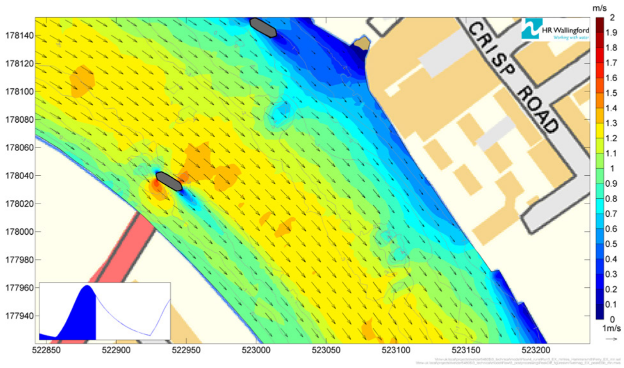

Figure 3.7: Baseline conditions, peak ebb depth averaged currents

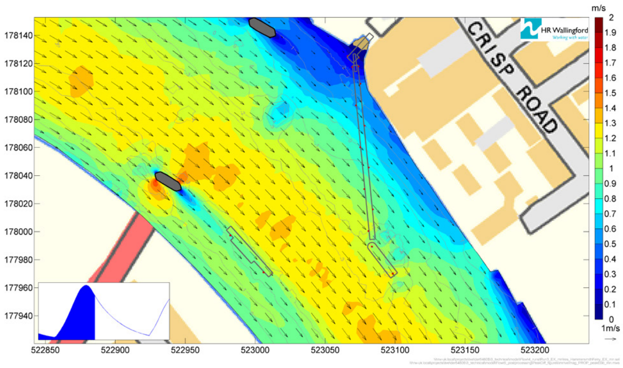

Figure 3.8: With proposed changes, peak ebb depth averaged currents

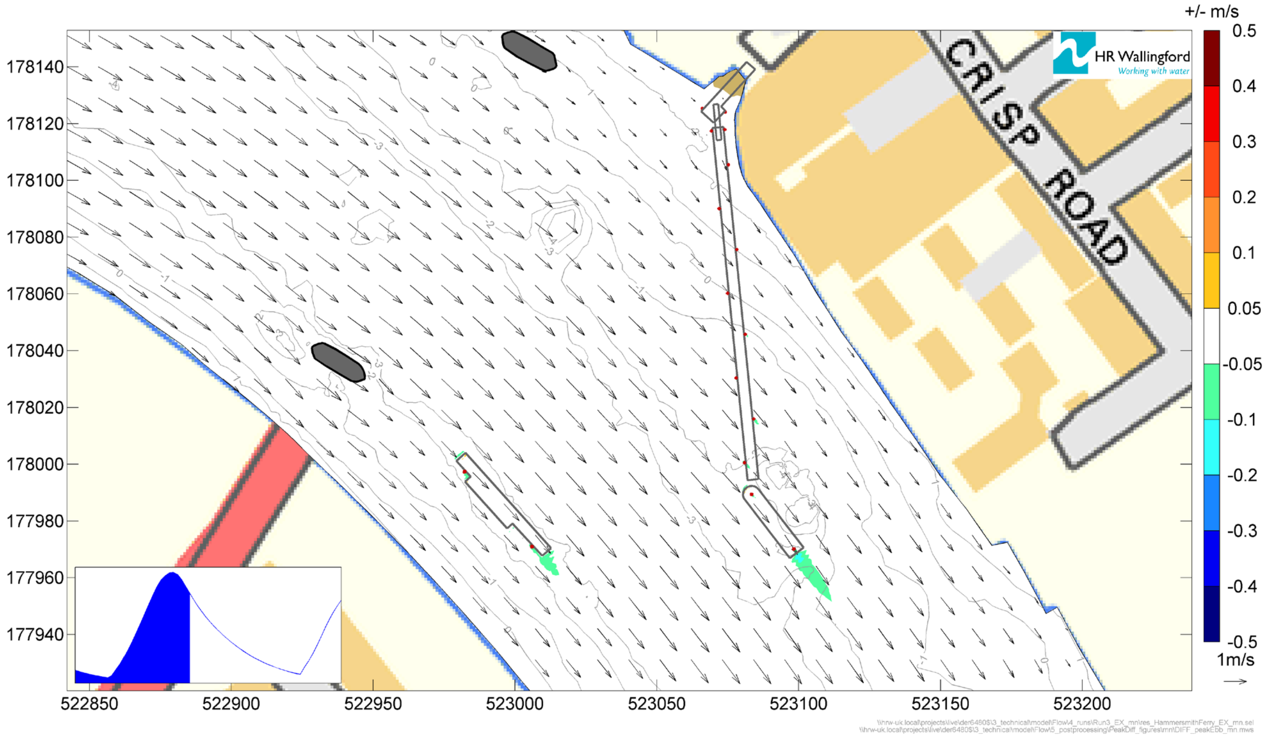

Figure 3.9: Difference in peak ebb depth averaged currents associated with the proposed changes

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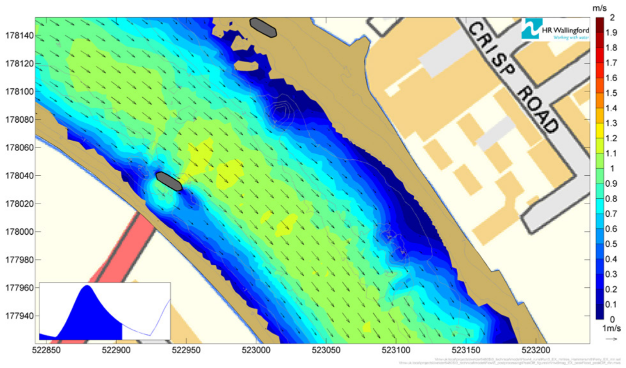

Figure 3.10: Baseline conditions, later ebb depth averaged currents – MN scenario

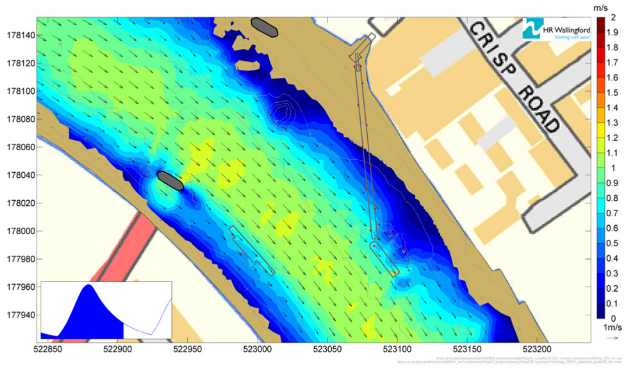

Figure 3.11: With proposed changes, later ebb depth averaged currents – MN scenario

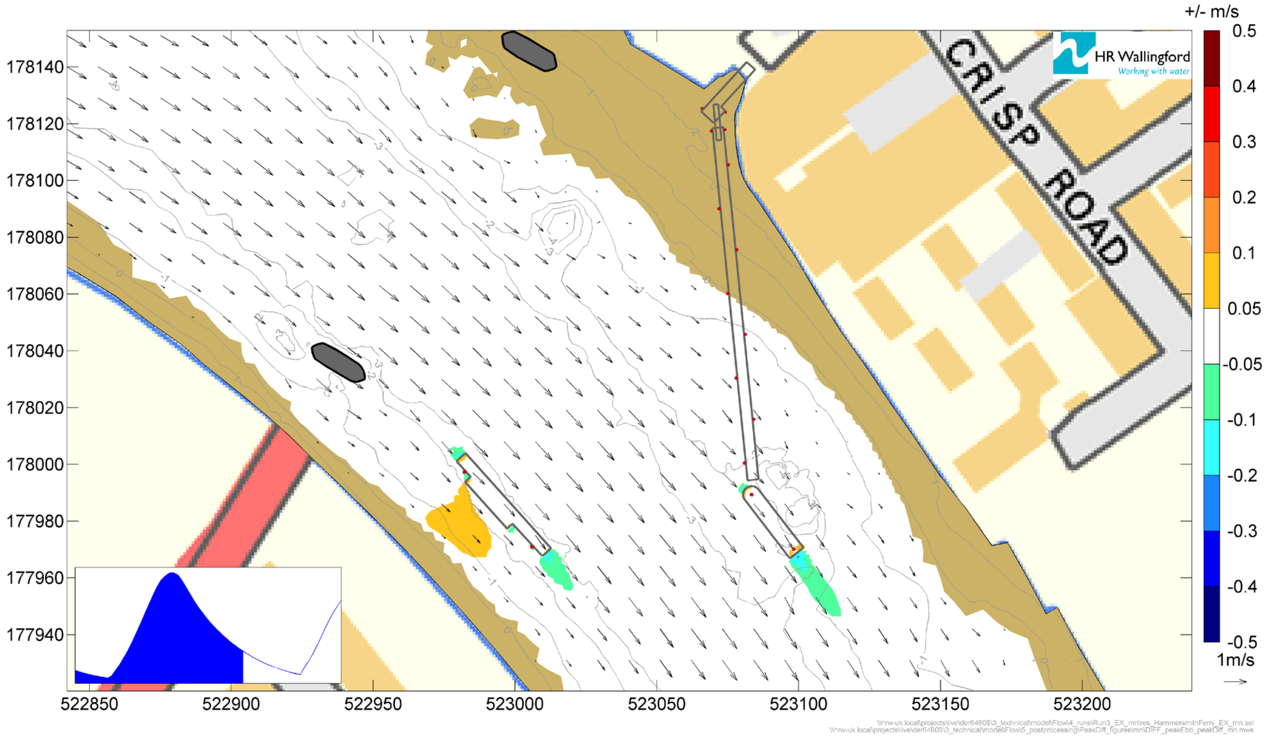

Figure 3.12: Difference in later ebb depth averaged currents associated with the proposed changes – MN scenario

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3.4. Impact on hydrodynamics – time series

Time series of currents taken over a tidal cycle provide further information regarding the impact of the proposed works on the local hydrodynamics. Figure 3.13 to Figure 3.20 show the through tide speed and direction of flow for baseline and proposed cases at the eight locations noted in Figure 3.1.

Location 1 at the Hammersmith bridge pier on the Barnes Temporary Pier side shows a very small decrease in peak flood tide flows, demonstrating a negligible effect on this third-party structure. Locations 2 and 3 up and downstream of the Barnes Temporary Pier show the effect of the piles and pier to reduce the flow speeds in the lee of the structure, but again below any significant magnitude of changes. Location 4 on the foreshore at Barnes shows a slight speed increase as flow is diverted around the pier structure, with the effect more pronounced on the later ebb tide.

Position 5 underneath the floating walkway on the upper foreshore shows the effect of the reduced water depth underneath the temporary pier sections around peak flood tide, with no other changes apparent. Positions 6 and 7, up and downstream of the Hammersmith Temporary Pier show similar speed reductions as for the Barnes Temporary Pier . Position 8 at the outfall discharge site shows there is no effect of the structure at this location.

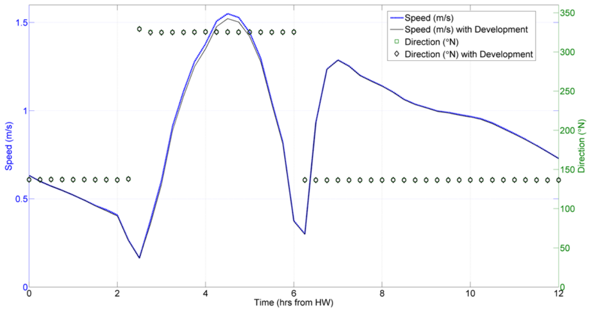

Figure 3.13: Position 1: Temporal variation in current speed and direction for baseline and proposed cases

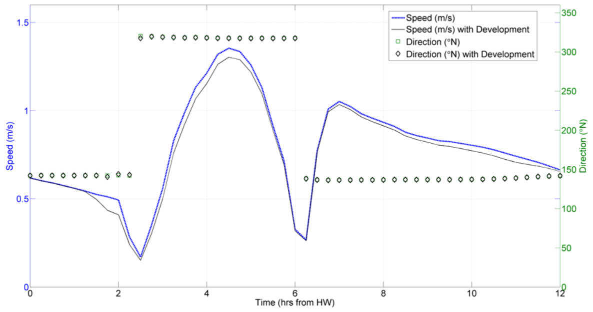

Figure 3.14: Position 2: Temporal variation in current speed and direction for baseline and proposed cases

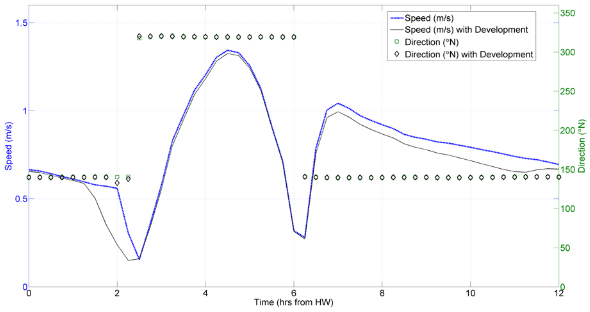

Figure 3.15: Position 3: Temporal variation in current speed and direction for baseline and proposed cases

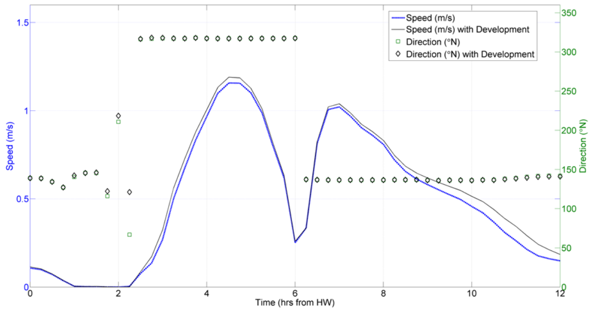

Figure 3.16: Position 4: Temporal variation in current speed and direction for baseline and proposed cases

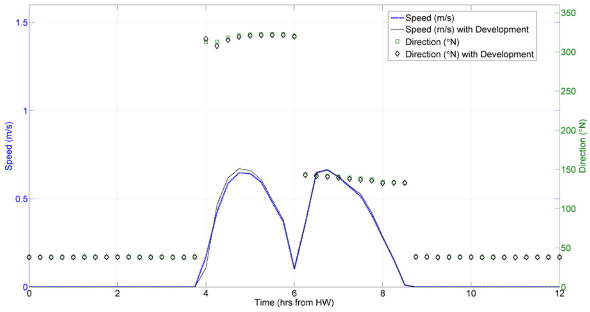

Figure 3.17: Position 5: Temporal variation in current speed and direction for baseline and proposed cases

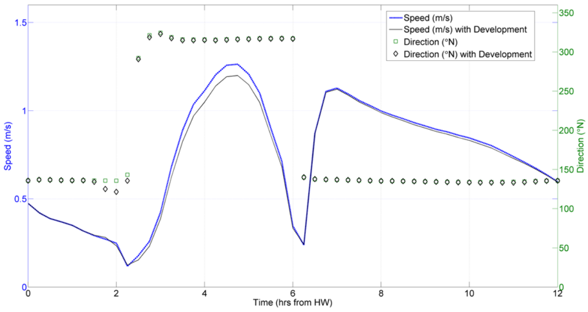

Figure 3.18: Position 6: Temporal variation in current speed and direction for baseline and proposed cases

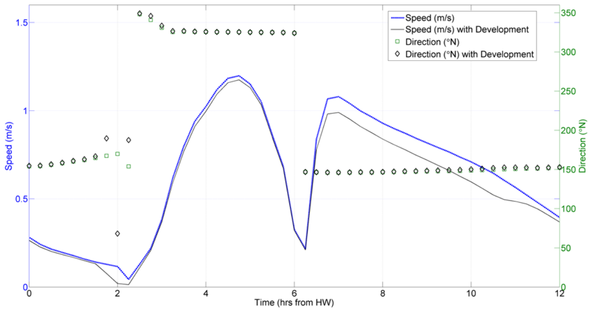

Figure 3.19: Position 7: Temporal variation in current speed and direction for baseline and proposed cases

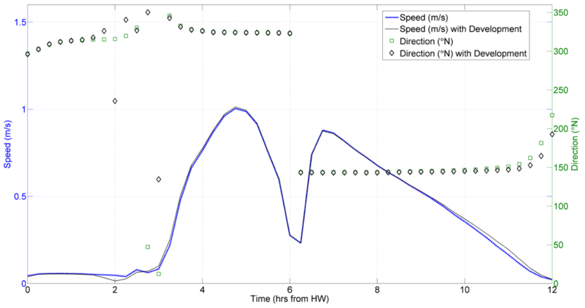

Figure 3.20: Position 8: Temporal variation in current speed and direction for baseline and proposed cases

3.5. Impact on morphology

The implications of the predicted currents for the bed sediments and morphology of the area were assessed by analysis of the type of bed material that would be expected for the bed shear stress generated by the currents alone (see Section 3.1 for further explanation). Figure 3.21 shows the results of this analysis for baseline conditions while Figure 3.22 shows the change in expected bed material with the proposed works associated with the temporary piers and walkway.

The results show that the bed material is presently generally be made up of sand and gravels up to 5 mm on the foreshores, increasing to 10 mm in main channel areas that are subject to the fastest speed magnitudes. These predictions align with photographs of the foreshore taken during a previous visit to the site (Photograph 3.1). The holes in the riverbed related to the two outfall sites cause the area of smaller gravel across the northern foreshore to extend riverward due to reduced bed shear stresses in the deeper water. The bed sediment in these area would also be affected by the discharges emerging from the outfalls.

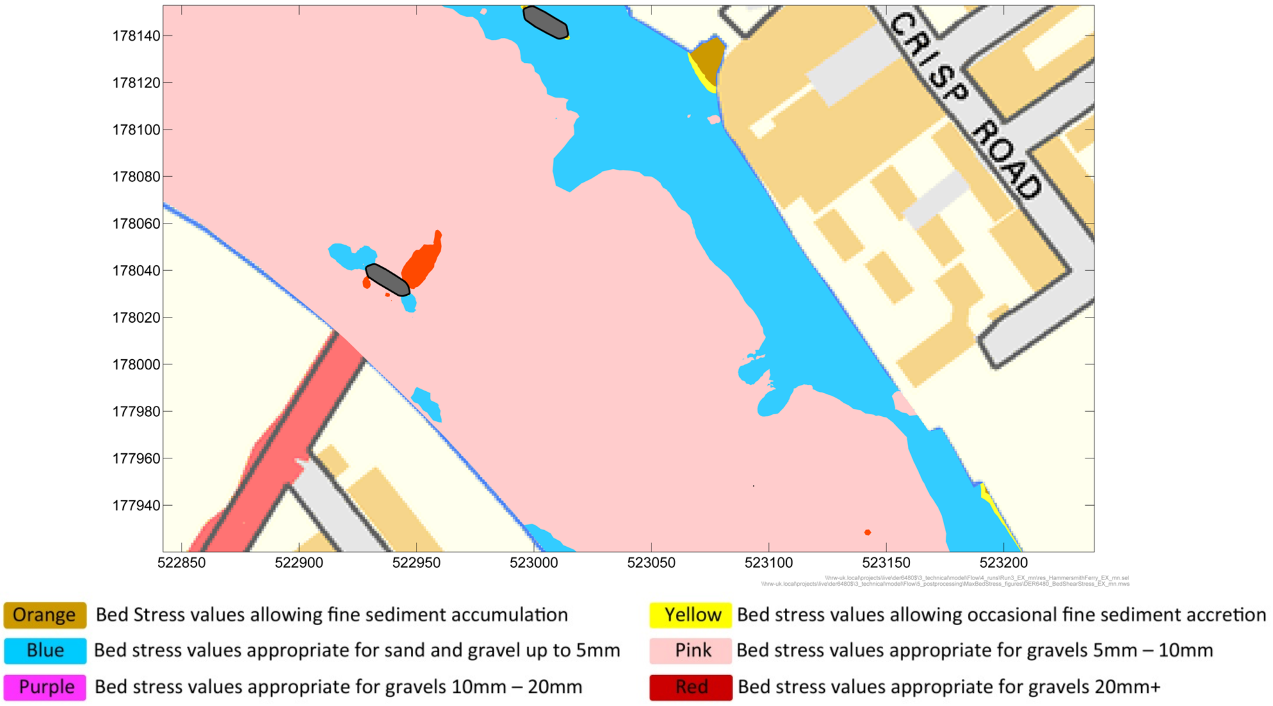

Figure 3.21: Baseline conditions: bed material summary based on peak bed shear stress – MN scenario

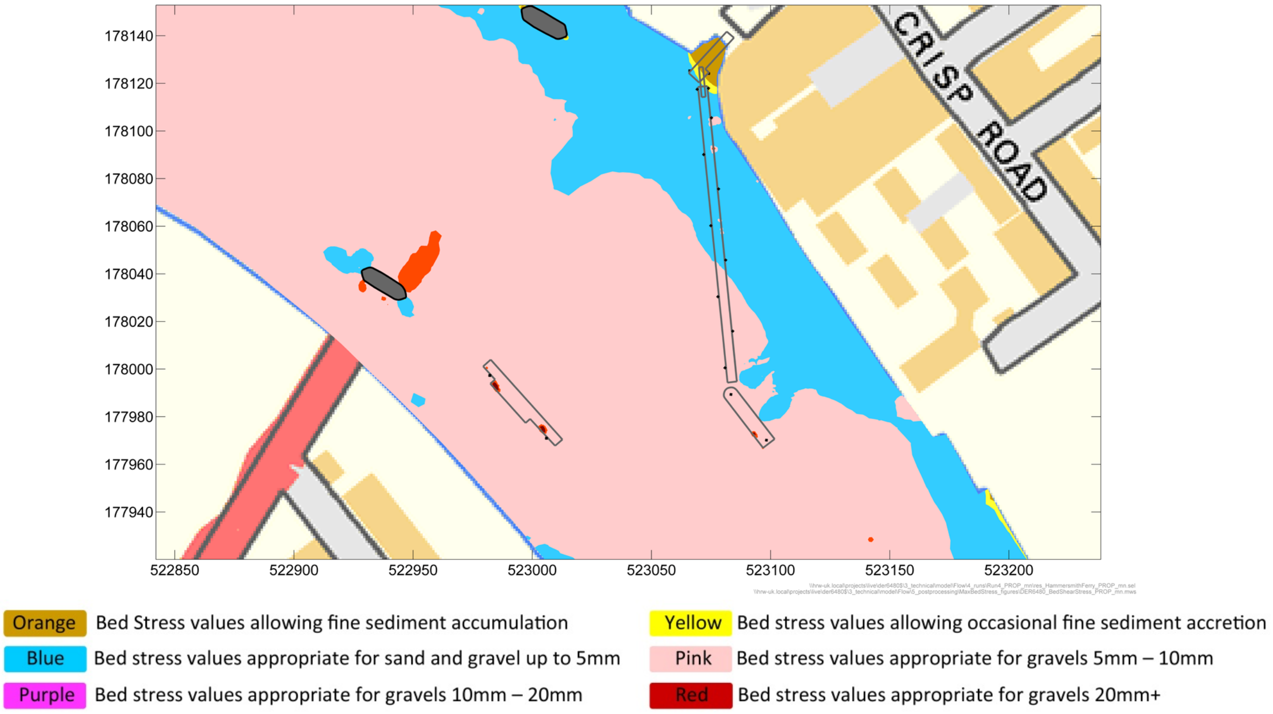

Figure 3.22: With proposed layout: bed material summary based on peak bed shear stress - MN scenario

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Comparing Figure 3.21 and Figure 3.22, the effects of the proposed works are very localised:

- small patches of increased maximum grain size from 5 to 10 mm related to slight increases in the maximum bed shear stress underneath the temporary piers, indicating that in these very localised areas some bed material coarsening possibly leading to a small amount of erosion may occur.
- a small area of increase in maximum grain size to 5 to 10 mm on the Barnes foreshore suggesting some coarsening the sediment in this area - removing some of the finer fraction material, if present.
- a very small 2 m^2 patch visible in Figure 3.22 as the small pink circle around one of the walkway piles on the upper Hammersmith foreshore. This decrease in the maximum grain size suggests that there may be some changes to the substrate composition in the immediate vicinity of the piles.

Photograph 3.1: Hammersmith foreshore: sand and gravel foreshore composition

Source: *HR Wallingford*

3.6. Consideration of episodic outfall discharge

The proposed layout was run with two representations of a discharge event from the two outfalls adjacent to the Hammersmith Temporary Pier, each with a duration of two hours, one across typical low water and one across typical high water, using the set-up described in Section 2.1.2.

The combined effect of the representative discharge events in combination with the proposed Hammersmith Temporary Pier layout is shown for typical high and low water conditions in Figure 3.23 and Figure 3.24, respectively.

The effect of the representative discharge event across high water is almost indiscernible (small footprint of speed differences < 0.1 m/s) from typical conditions, due to the ambient flow speeds, as well as the increased dispersion of the flow from the outfalls in the deeper water. For the low water event, the effect of the discharge is localised, rapidly dissipating into the scour holes that have formed in the riverbed during previous discharge events and reducing in speed as a result.

These results indicate that there is the possibility for speed differences of +/- 0.1 m/s to occur at and around the Hammersmith Temporary Pier and restraining piles during low water discharge events. Otherwise the potential effect of the outfall on the proposed structures is concluded to be extremely small.

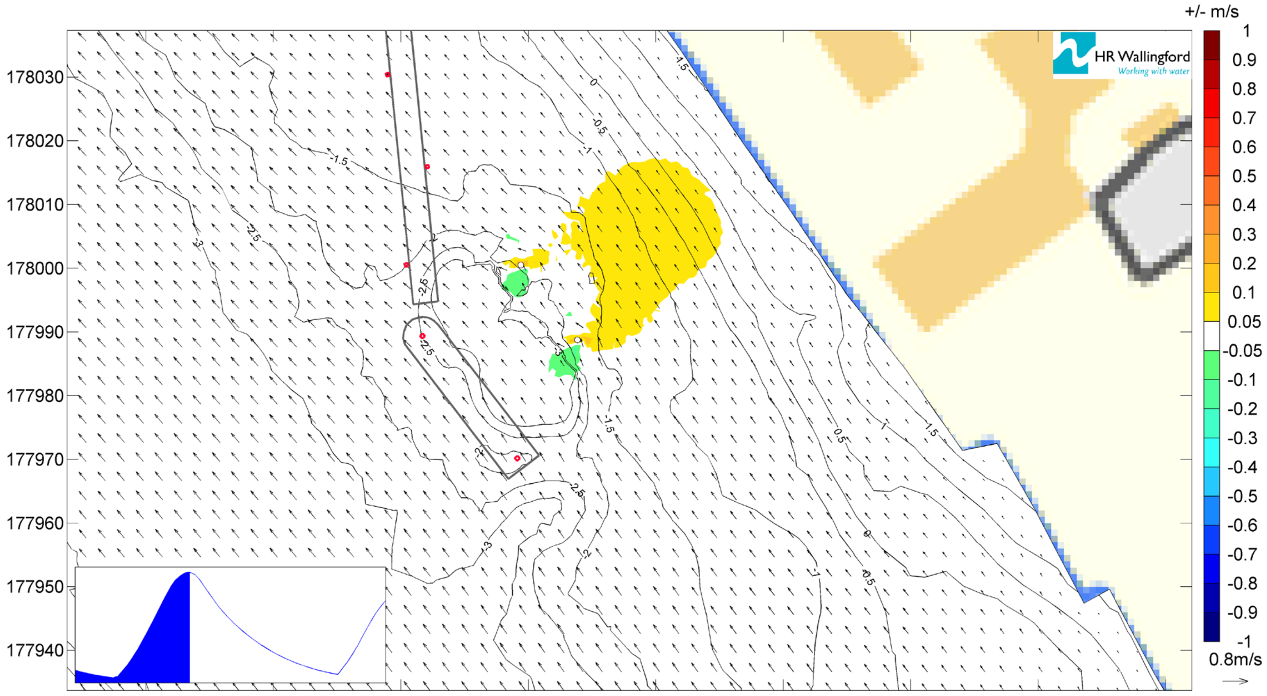

Figure 3.23: Difference in depth averaged currents associated with the proposed changes during a representative outfall discharge event at typical high water

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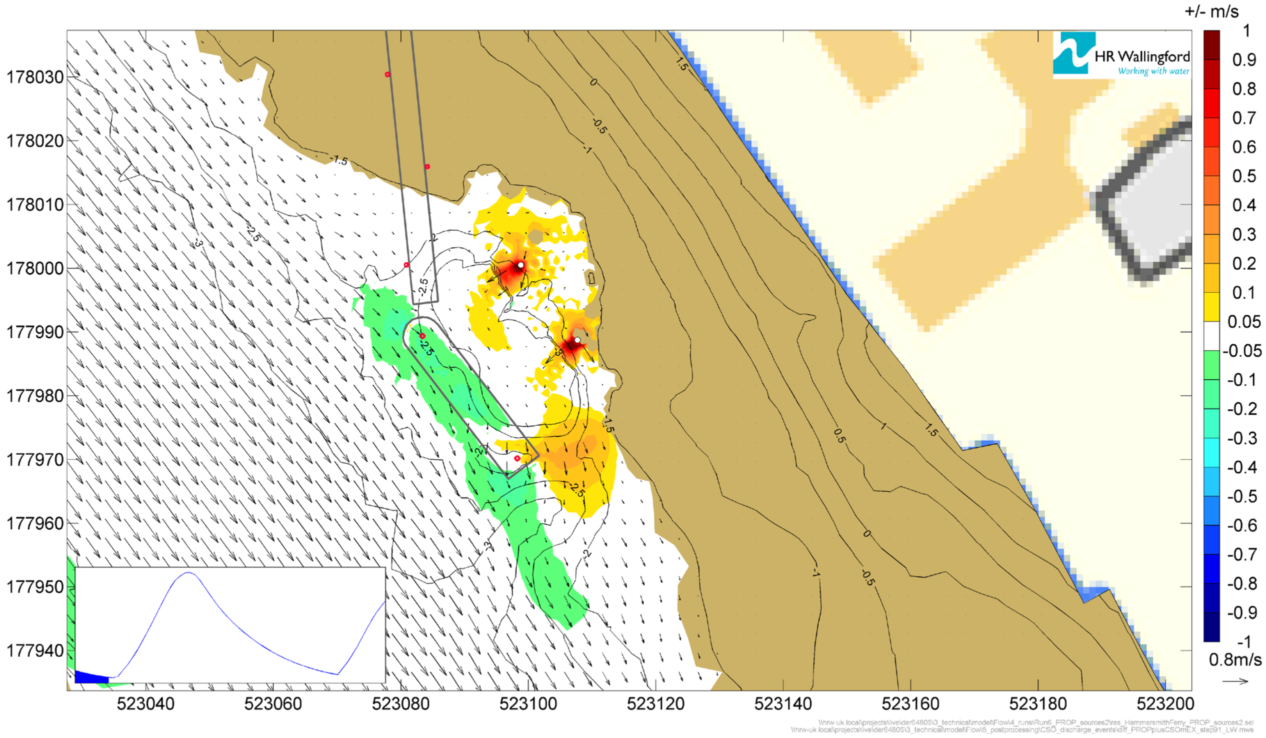

Figure 3.24: Difference in depth averaged currents associated with the proposed changes during a representative outfall discharge event at typical low water

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4. Scour assessment

Scour assessments can generally be subject to large uncertainties associated with the variability in bed material. Therefore it is recommended that information on the sequence and properties of the surface and underlying soils be acquired within the programme of geotechnical surveys planned for the bridge (e.g. grab samples, boreholes, Cone Penetration Testing, laboratory analysis for sedimentological and geotechnical parameters). While it is preferable that the scour assessment is undertaken following the collection of the required data, the timeline for the temporary pedestrian and cycle bridge is such that geotechnical sampling is not available at the time of this assessment. Therefore, it is necessary to consider a number of bed composition scenarios that are known to be representative of the conditions at the bridge site.

Scour is a physical process related to the movement of seabed sediment by the flow of water away from a structure. With regard to the geotechnical nature of scour, the ground conditions are described by geotechnical parameters and the flow of water by hydraulic parameters. The interface between these two domains is termed “loose boundary hydraulics” and hence scour is of a geotechnical nature as it relates to the reduction in ground level around a structure. In non-cohesive soils, scour can be considered to be a combined hydraulic and geotechnical process in that the flow interacts with the geotechnical properties of the soil such as grain size, shape and density, which have an influence on the scour and erosion processes.

An assessment of the scour potential has been undertaken for the proposed completed temporary pedestrian and cycle bridge piers using three empirical methods. These three approaches are standard methods for estimating scour depth and have been applied in this study to assess the scour potential at the piers. The standard methods apply to non-cohesive soils and are:

- the approach of Richardson and Davis (2001) which forms one of the empirical methods given within the current version of the US Department of Transport, Federal Highway Administration (FHWA) Hydraulic Engineering Circular (HEC) No.18 [referred to here as HEC-18];
- the method of Tavouktsoglou *et al*. (2017) which is based on the depth-averaged Euler number as a means of representing the pressure gradient down the face of the structure; and,
- the empirical method of Sheppard *et al*. (2011).

In the present case there is a nearby analogy for scour given the proximity of the bridge piers of Hammersmith Bridge to the temporary pedestrian and cycle bridge. An assessment of the scour present at the existing bridge piers is included in this section to ground-truth the predictions.

4.1. Grain size scenarios

The geotechnical desk-study report for the temporary pedestrian and cycle bridge project (Pell Frischmann, 2019) includes historical BGS borehole data, of which one location is located within the estuary. This borehole log collected in 1924 indicates around 0.4 m of ‘dirty gravel and sand’ overlying brown and blue clay.

Grab sample data is available for the nearby Putney Bridge, which is in a similar river bend location approximately 2 km downstream of the Hammersmith site. Interrogation of recent photographs of the foreshore down to MLWS at both locations demonstrates the bed material composition is dominated by sandy gravel (compare Photograph 3.1 with Photograph 4.1).

Particle Size Distribution (PSD) analysis of the Putney grab samples provides three grain size scenarios that can be considered applicable to the Hammersmith site. These are summarised in Table 4.1. The bed material is considered to be non-cohesive to ensure that the results are conservative. An additional coarse sand scenario is included as a sensitivity test.

Table 4.1: Grain size scenarios selected for the scour assessment informed by grab samples at Putney Bridge (Scenarios 1 to 3) plus a coarse sand as a sensitivity test (Scenario 4)

Scenario	Description	d_{50} (mm)	d_{10} (mm)	d_{90} (mm)
1	Coarse gravel in the main channel	16	0.4	60
2	Medium gravel on the lower foreshore	12	0.4	29
3	Smallest d_{50} of the Putney grab samples on the upper foreshore	4	0.1	25
4	Coarse sand sensitivity test	0.6	0.1	2.5

Source: HR Wallingford

Photograph 4.1: South foreshore at Putney Bridge

Source: HR Wallingford

4.2. Empirical scour predictions

Peak flow speeds and corresponding water depths across a typical spring tide were extracted for the flood and ebb phases from the flow model results as presented in Section 3 at the locations shown in Figure 4.1.

Timeseries of the extracted data are shown in Figure 4.2, Figure 4.3 and Figure 4.4; with the full spectrum of potential scouring conditions listed in Table 4.2. Empirical scour predictions have been made for those cases highlighted in bold in Table 4.2, avoiding unnecessary repetition of similar flow speed and water depth combinations.

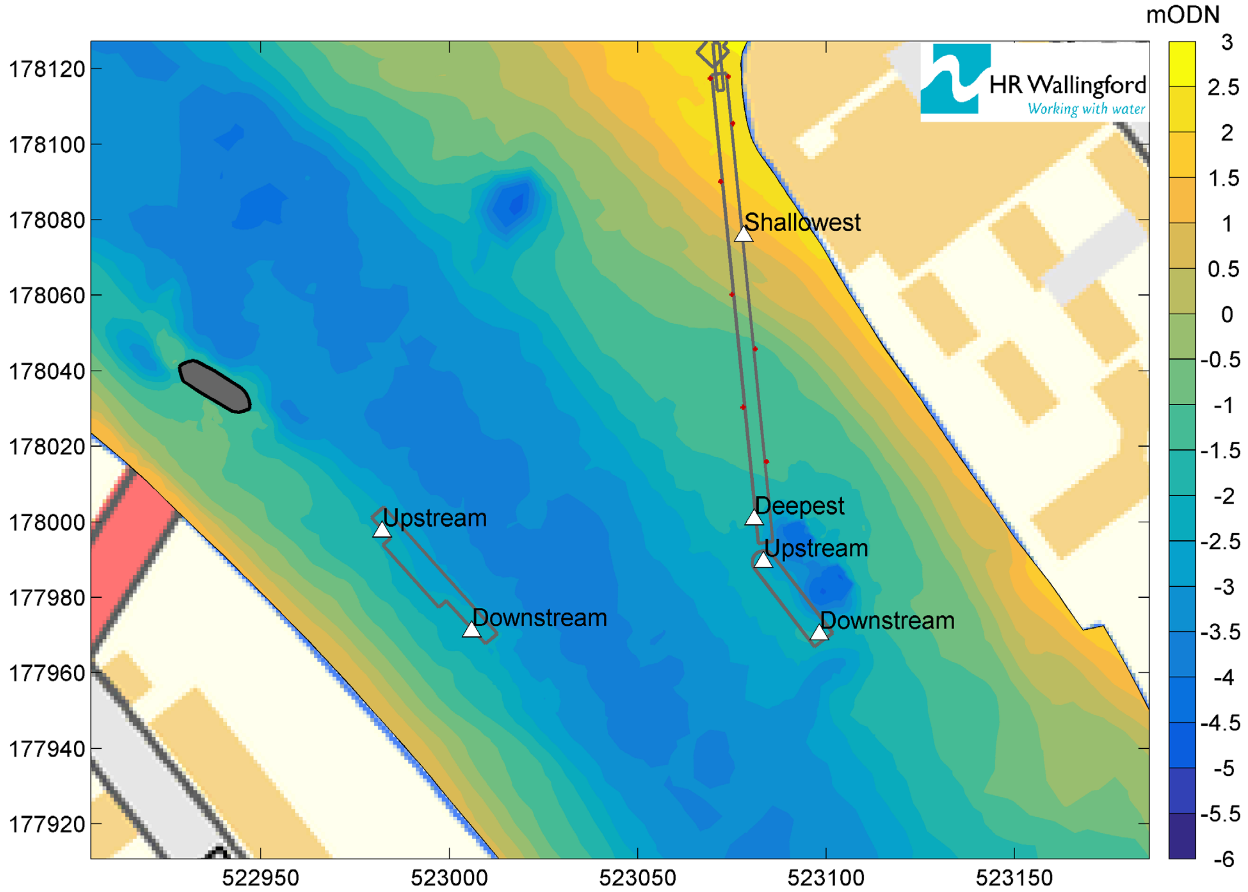

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Figure 4.1: Pile locations considered for the scour assessment. The 'shallowest' pile on the floating walkway is the shallowest to still experience significant flow speeds

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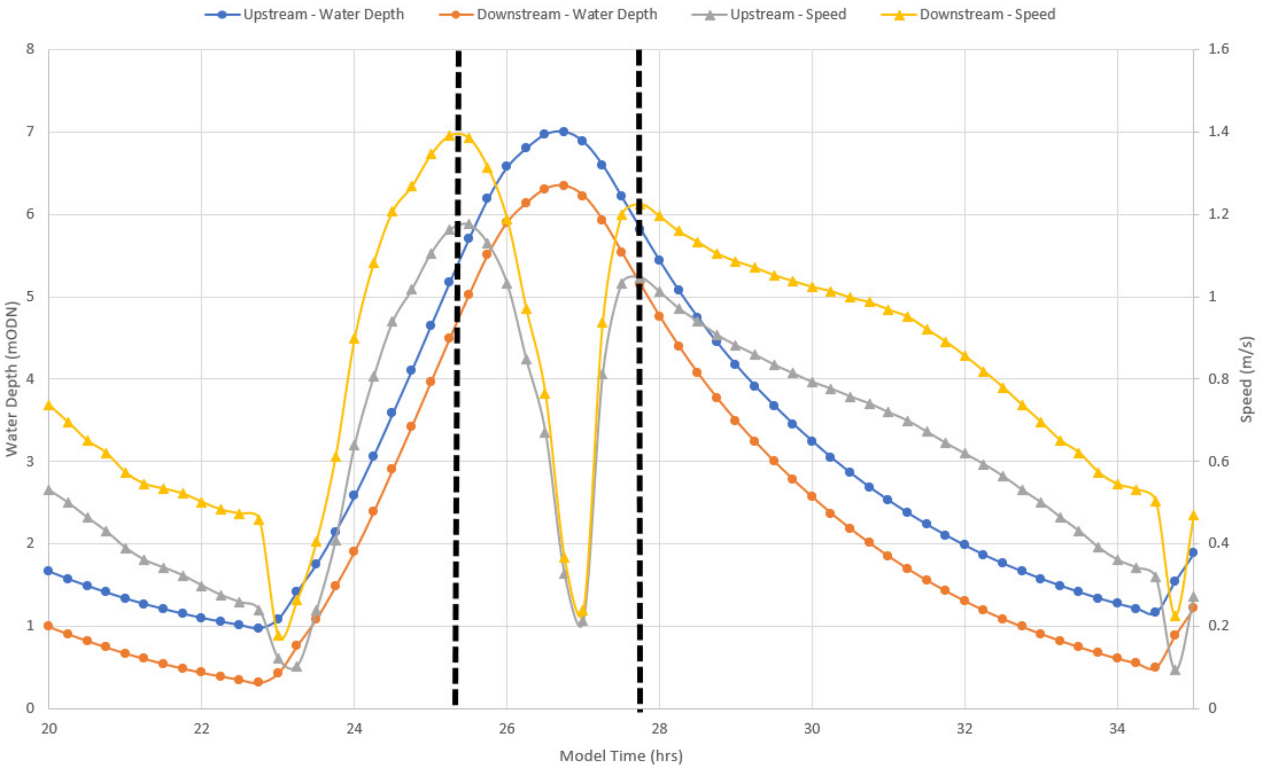

Figure 4.2: Speeds and depths at the pile locations considered at Hammersmith Temporary Pier. Vertical dashed black lines indicate the times of peak speeds on the flood and the ebb

Source: HR Wallingford using the Thames 2D Base Model

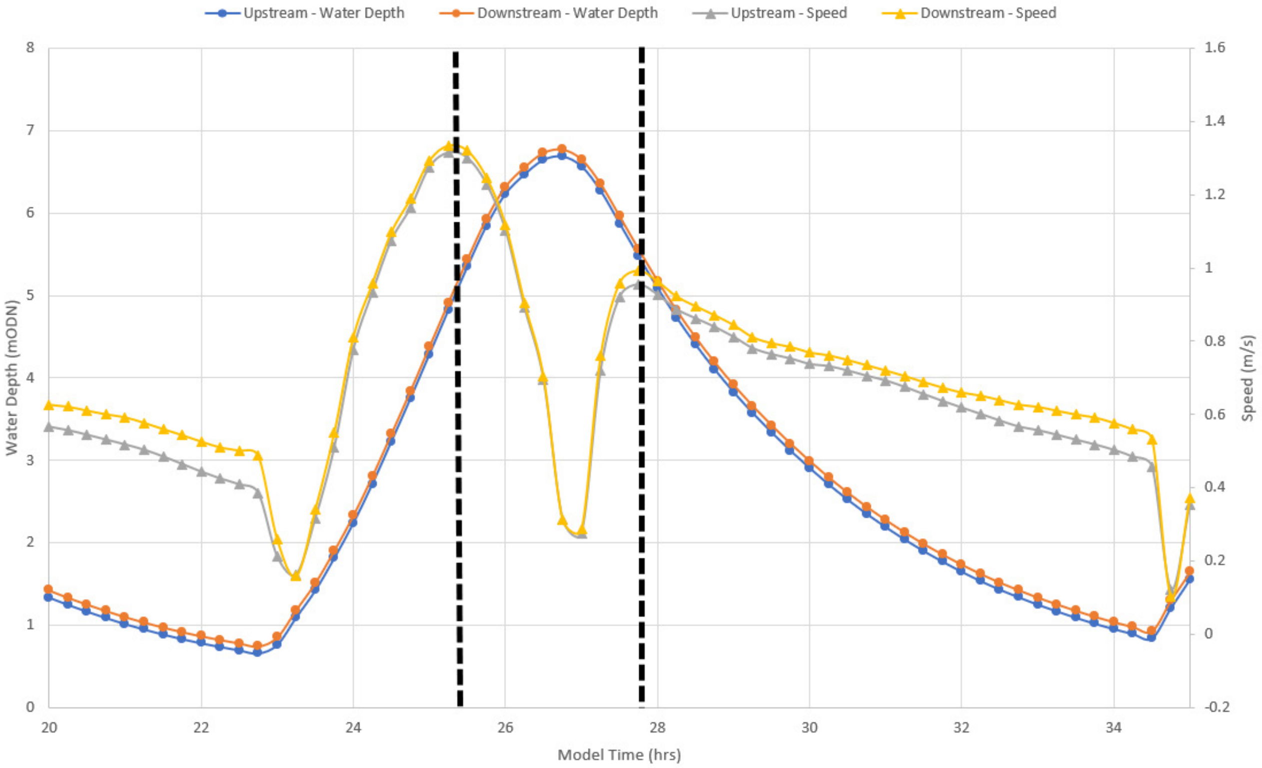

Figure 4.3: Speeds and depths at the pile locations considered at Barnes Temporary Pier. Vertical dashed black lines indicate the times of peak speeds on the flood and the ebb

Source: HR Wallingford using the Thames 2D Base Model

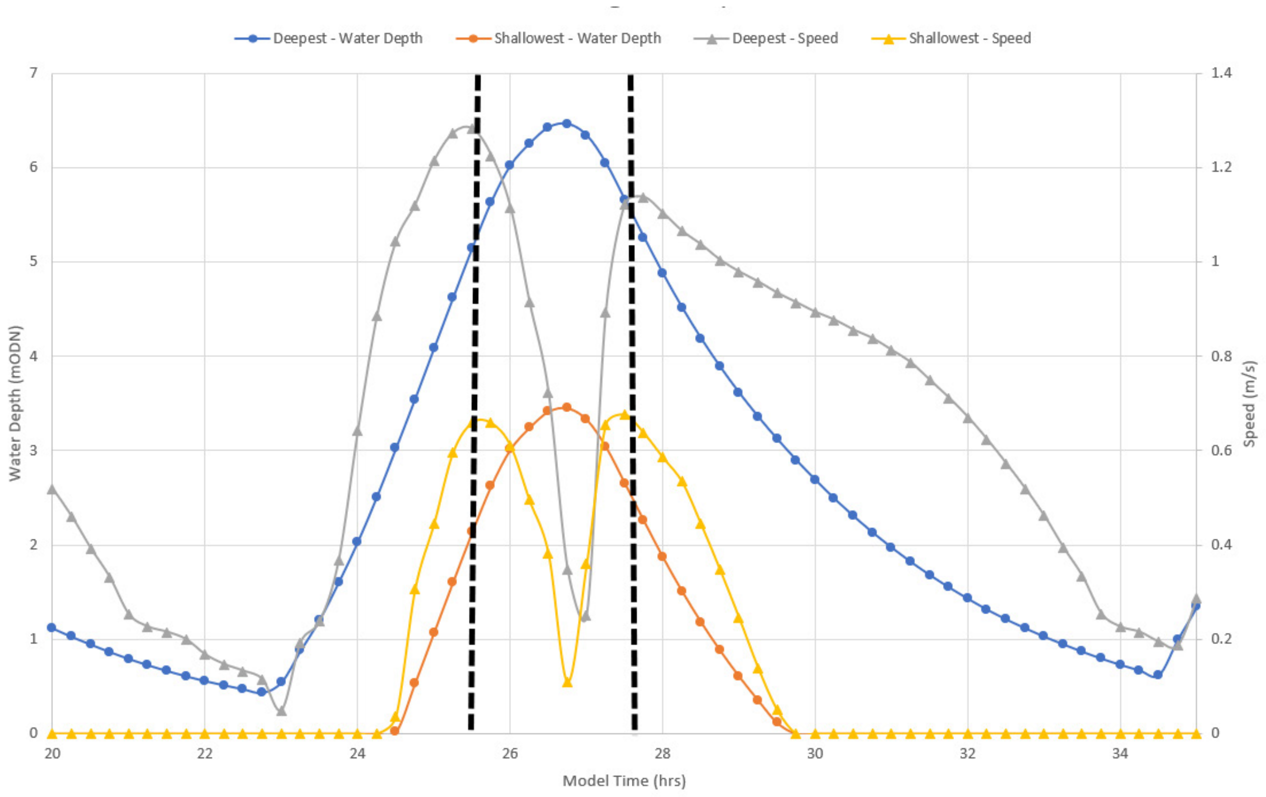

Figure 4.4: Speeds and depths at the pile locations considered at the Floating Walkway. Vertical dashed black lines indicate the times of peak speeds on the flood and the ebb

Source: HR Wallingford using the Thames 2D Base Model

Table 4.2: Full spectrum of potential scouring conditions, based on the timeseries shown in Figure 4.2 to Figure 4.4. Empirical scour predictions have been made for those cases highlighted in bold

Location	Phase	Pile Position	Depth-averaged velocity (m/s)	Water depth (mODN)
Hammersmith Temporary Pier	**Flood**	**Upstream**	**1.39**	**5.18**
	Flood	**Downstream**	**1.16**	**4.49**
	Ebb	Upstream	1.22	5.81
	Ebb	Downstream	1.04	5.14
	LW during outfall discharge event	**Upstream**	**0.15**	**0.47**
Barnes Temporary Pier	**Flood**	**Upstream**	**1.33**	**4.82**
	Flood	Downstream	1.33	4.91
	Lowest Ebb	Upstream	0.48	0.89
	Lowest Ebb	**Downstream**	**0.55**	**0.98**
Floating Walkway	**Flood**	**Deepest**	**1.28**	**5.15**
	Flood	**Shallowest with flow**	**0.66**	**2.14**
	Ebb	Deepest	1.14	5.26
	Ebb	Shallowest with flow	0.67	2.25

Source: HR Wallingford using the Thames 2D Base Model

The scour depths predicted for the grain size scenarios presented in Table 4.1 are presented for the HEC-18 method in Table 4.3, for Tavouktsoglou (2018) in Table 4.4, and for Sheppard et al (2011) in Table 4.5.

Of the three representative grain size scenarios which are similar to the sediments found at the site (Scenarios 1-3), the largest scour depths of around 1.1 m are predicted for grain size scenario 3 for all piling locations except for the Floating Walkway ‘shallowest’ pile, which has a maximum predicted scour depth of 0.69 m.

The sensitivity test with coarse sand, which can be considered an almost unrealistic worst case given the known presence of armouring gravel at the site, predicts maximum scour depths of around 1.2 m.

It is assumed that the presence of a strong well-consolidated clay layer beneath the sandy gravel will provide a geological control on scour development, such that scour depths will be limited to the thickness of the layer overlying the clay.

Based on these empirical predictions and the site conditions, it is therefore estimated that local scour depths will not exceed 1 m at any of the piling locations, and will more than likely be limited to less than 0.5 m. In particular the Hammersmith Temporary Pier piles are sited within the long-standing scour holes associated with the outfall, such that the bed here will be composed of larger immobile gravels. The localised increase in bed strength at this location will be a further limitation on scour development.

Table 4.3: Scour predictions for the four grain size scenarios using the HEC-18 method

	Hammersmith Temporary Pier			Barnes Temporary Pier		Floating Walkway	
Grain Size Scenario	Peak flood (upstream)	LW during discharge event (upstream)	Peak flood (downstream)	Peak flood (upstream)	Lowest Ebb (downstream)	Peak flood (deepest)	Peak Flood (shallowest)
1	0.52	0.14	0.45	0.49	0.26	0.44	0.30
2	0.49	0.14	0.45	0.48	0.26	0.44	0.30
3	0.49	0.22	0.45	0.48	0.26	0.44	0.30
4	1.12	0.59	1.06	1.20	0.66	1.19	0.80

Source: HR Wallingford

Table 4.4: Scour predictions for the four grain size scenarios using the Tavouktsoglou (2018) method

	Hammersmith Temporary Pier			Barnes Temporary Pier		Floating Walkway	
Grain Size Scenario	Peak flood (upstream)	LW during discharge event (upstream)	Peak flood (downstream)	Peak flood (upstream)	Lowest Ebb (downstream)	Peak flood (deepest)	Peak Flood (shallowest)
1	0.92	0.87	0.08	0.90	0.45	0.90	0.61
2	0.92	0.87	0.09	0.91	0.46	0.91	0.63
3	0.95	0.91	0.11	0.94	0.53	0.94	0.69
4	1.00	0.97	0.16	0.99	0.66	0.99	0.80

Source: HR Wallingford

Table 4.5: Scour predictions for the four grain size scenarios using the Sheppard et al (2011) method

	Hammersmith Temporary Pier			Barnes Temporary Pier		Floating Walkway	
Grain Size Scenario	Peak flood (upstream)	LW during discharge event (upstream)	Peak flood (downstream)	Peak flood (upstream)	Lowest Ebb (downstream)	Peak flood (deepest)	Peak Flood (shallowest)
1	0.44	n/a	0.15	0.37	n/a	0.24	n/a
2	0.73	n/a	0.36	0.65	n/a	0.47	n/a
3	1.12	n/a	1.12	1.12	0.30	1.03	0.39
4	0.87	n/a	0.86	0.87	0.74	0.79	0.74

Source: HR Wallingford. Note that 'n/a' indicates that the combined grain size and flow speed scenario is not valid for this method.

4.3. Scour at existing Hammersmith Bridge pier

The proximity of the existing Hammersmith Bridge piers to the proposed temporary ferry piers allows ground-truthing of the scour predictions in the absence of project-specific geotechnical data. Recent (2019) bathymetry data for the site (Figure 4.6) shows that scour around the northern existing Hammersmith Bridge pier is limited, due to its position on the upper foreshore where flow speeds are lower. The southern existing bridge pier is considered to be the best analogy to the possible scour at the proposed piles; however it should be borne in mind that the bathymetry indicates that there is likely some scour protection in place.

Scour at the existing southern Hammersmith Bridge pier is offset towards the foreshore on the upstream end of the pier and offset towards the main channel on the downstream end. This is thought to be due to a combination of the presence of scour protection, but more importantly the misalignment of the structure to the main flow causing turbulent eddy shedding off the sides of the structure rather than from the more streamlined nose of the pier (see Section 3.2 and 3.3) leading to more bluff body flow and the resulting additional turbulence associated with this.

Four profiles have been extracted through the upstream and downstream scour holes to quantify the scour depths, locations shown in Figure 4.6, extracted depths shown in Figure 4.7 to Figure 4.10. These profiles demonstrate a scour depth of about 1.6 m in the larger upstream scour hole which is higher on the foreshore than the downstream scour hole, which correspondingly is not as deep, with scour depths of around 0.6 m. The downstream scour hole is considered to be more indicative of the potential for scour around the piles proposed as part of the temporary piers, due to their position on the downstream side of the bridge.

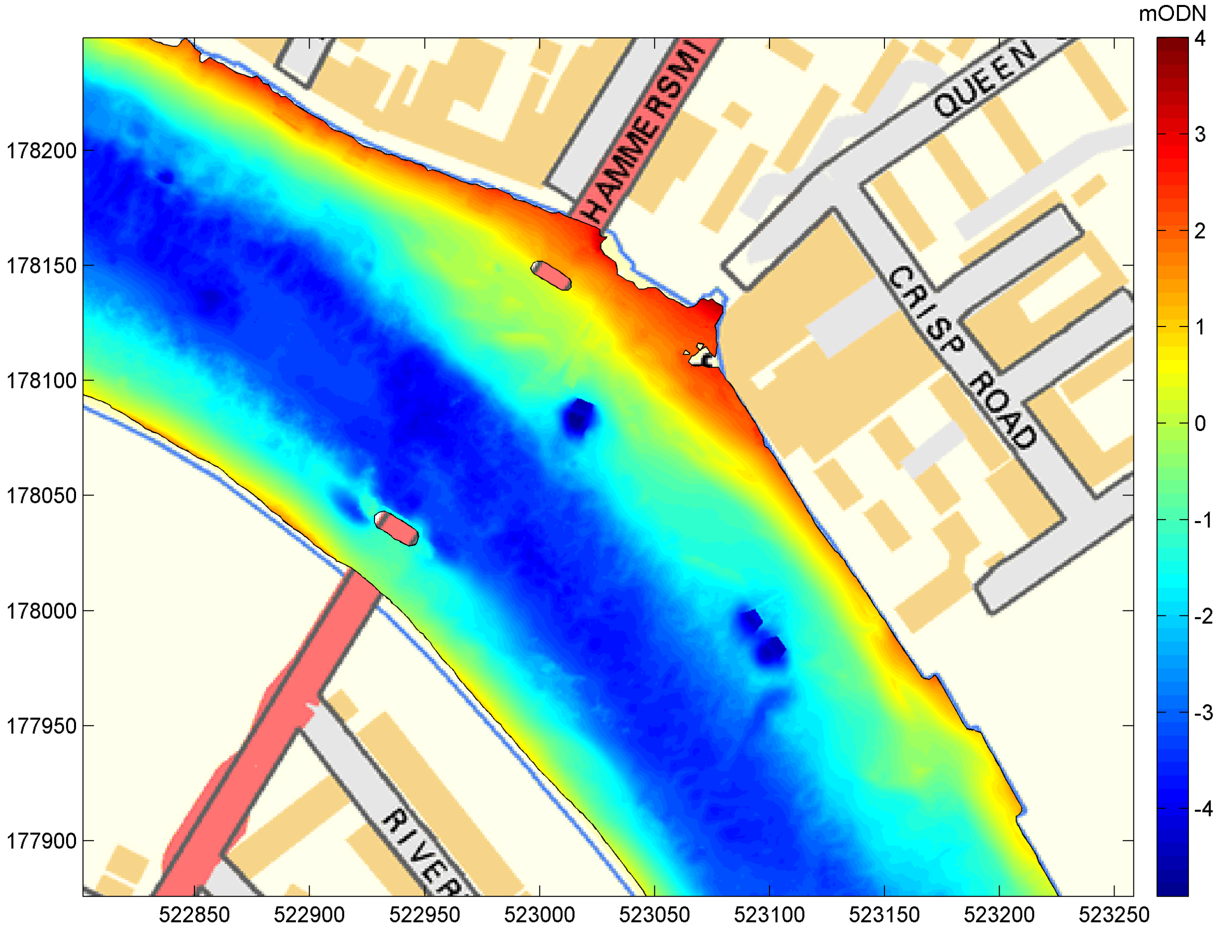

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Figure 4.5: Detailed bathymetry data at the project site

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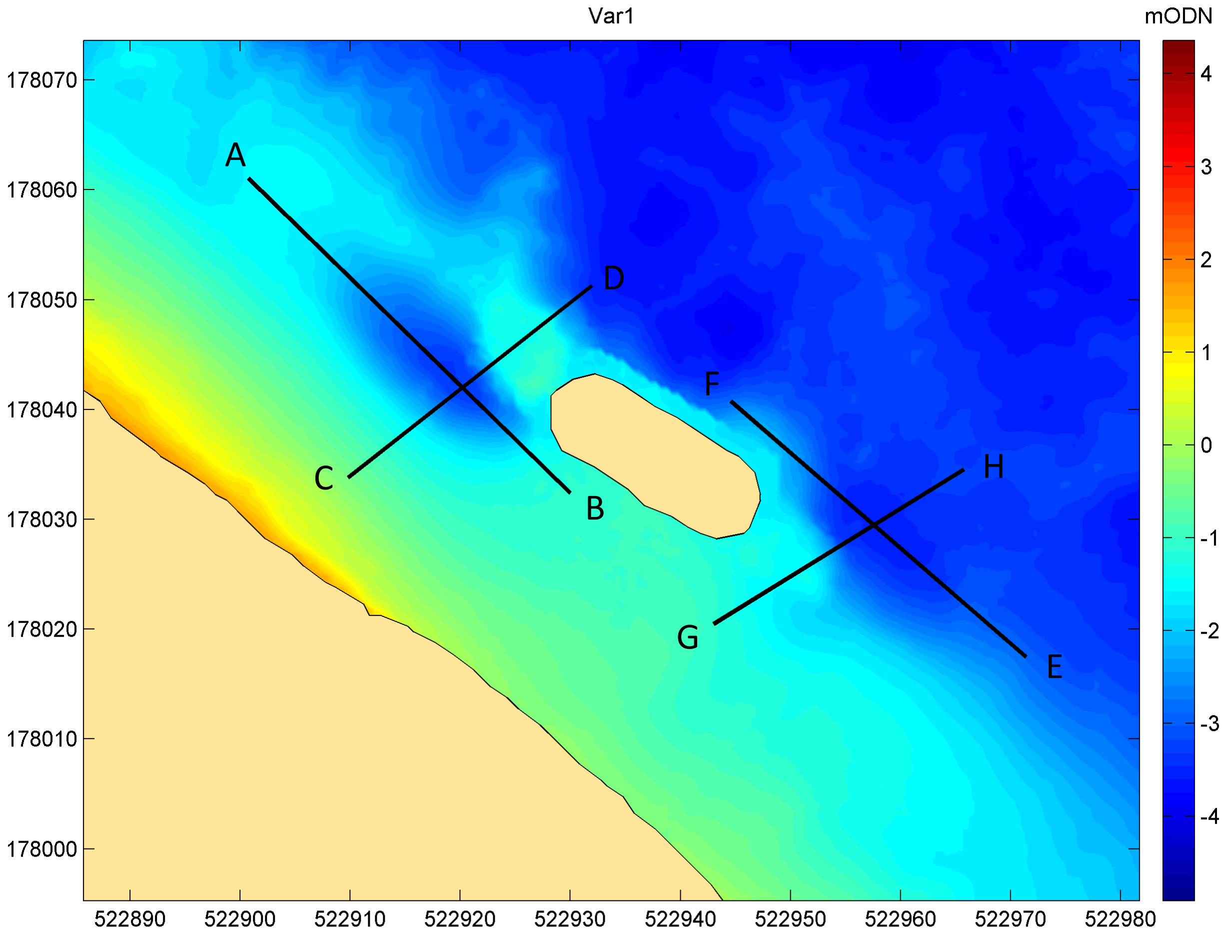

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Figure 4.6: Profile locations at the southern existing Hammersmith Bridge pier

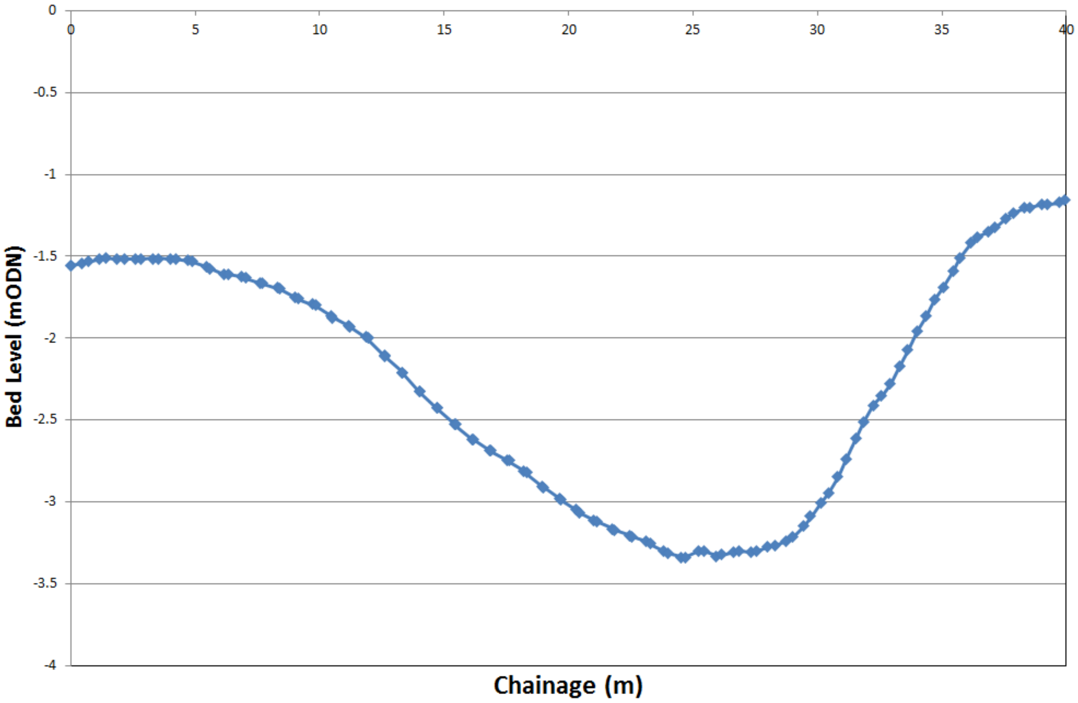

Figure 4.7: Profile AB, location shown in Figure 4.6

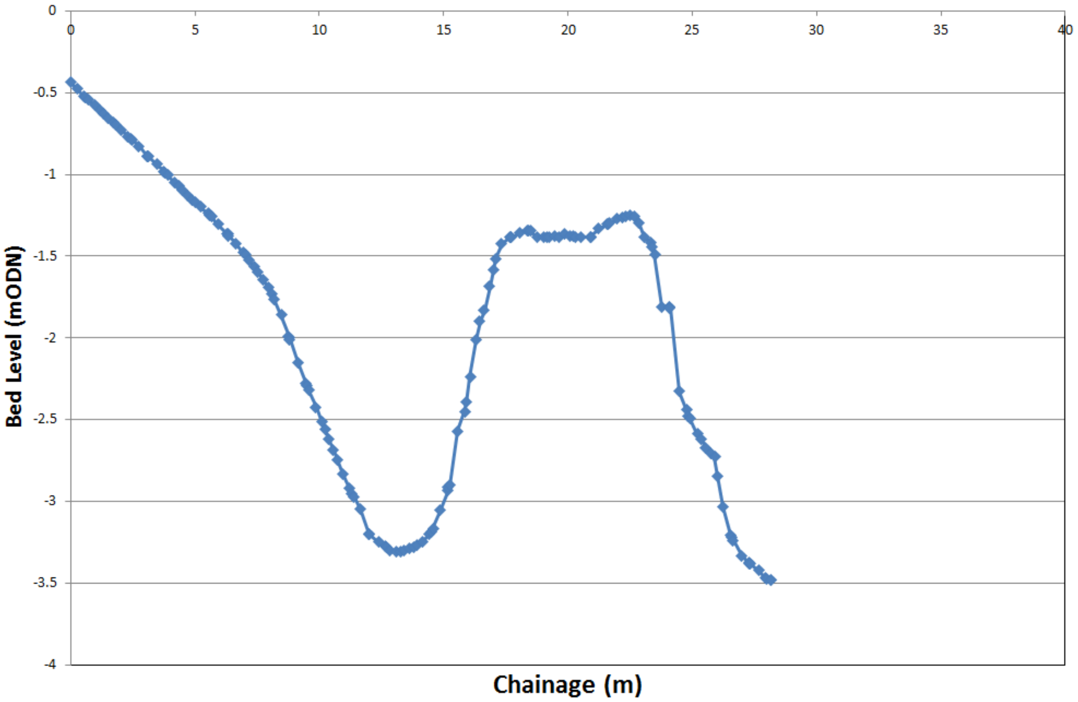

Figure 4.8: Profile CD, location shown in Figure 4.6

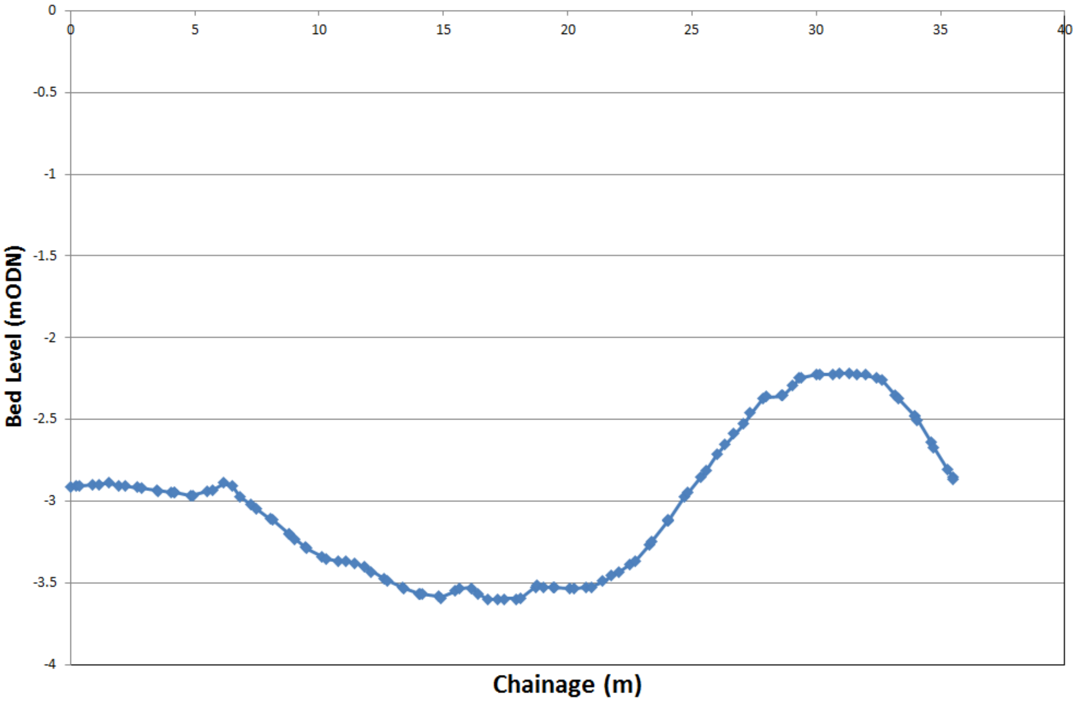

Figure 4.9: Profile EF, location shown in Figure 4.6

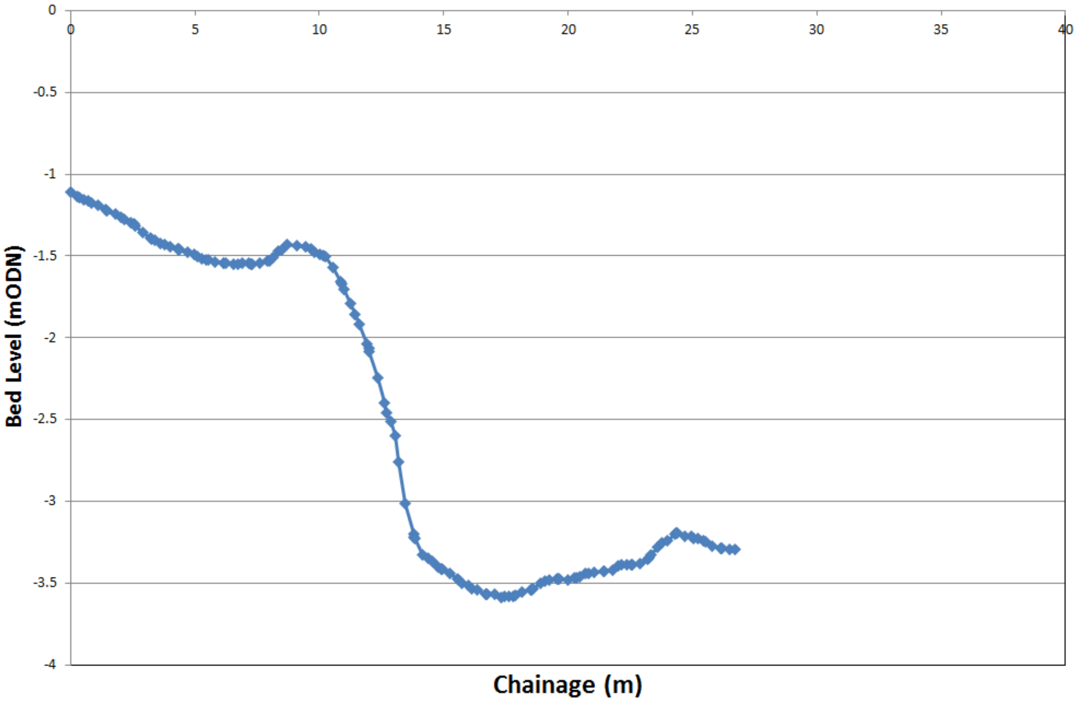

Figure 4.10: Profile GH, location shown in Figure 4.6

4.4. Potential for scour at the grounded floating walkway

There has been some regulatory concern expressed that there will be scour of the foreshore around the floating walkway during grounding. To investigate the potential for scour to occur under this scenario, the bathymetry of the foreshore is shown relative to the water line at LW in Figure 4.11. The duration over which the grounding walkway will be subject to scouring forces is considered to be very short. There are however small drainage channels evident on the foreshore at this location (see Figure 4.12), passing beneath and perpendicular to the floating walkway, which can also be seen as small recesses in the bathymetry in Figure 4.11. Therefore, it is useful to consider the cross-shore component of velocity to give an indication for the flooding and draining speeds that can be expected to occur in these drainage channels.

For simplification, the U-component of the velocity (that is movement in an east-west direction) was used as an indicator for cross-shore velocity speeds. The U-component of velocity is shown for ebbing conditions in Figure 4.13, and for flooding conditions in Figure 4.14. When the foreshore is draining, speeds remain < 0.1 m/s for the length of the floating walkway at the grounding point. The flooding foreshore experiences speeds of up to 0.2 m/s for the length of floating walkway subject to shallow water as it is re-floated. The rapid deepening of the water however as the walkway is re-floated along its length will quickly stop any removal of sediment by scouring processes as water depths increase.

Based on this assessment, the risk of local scour occurring of the grounded floating walkway is low. Any sediment movement that does occur during flooding and draining around the walkway is expected to be simply a redistribution of surficial sediments to a depth of a few centimetres, well within the bounds of natural variability.

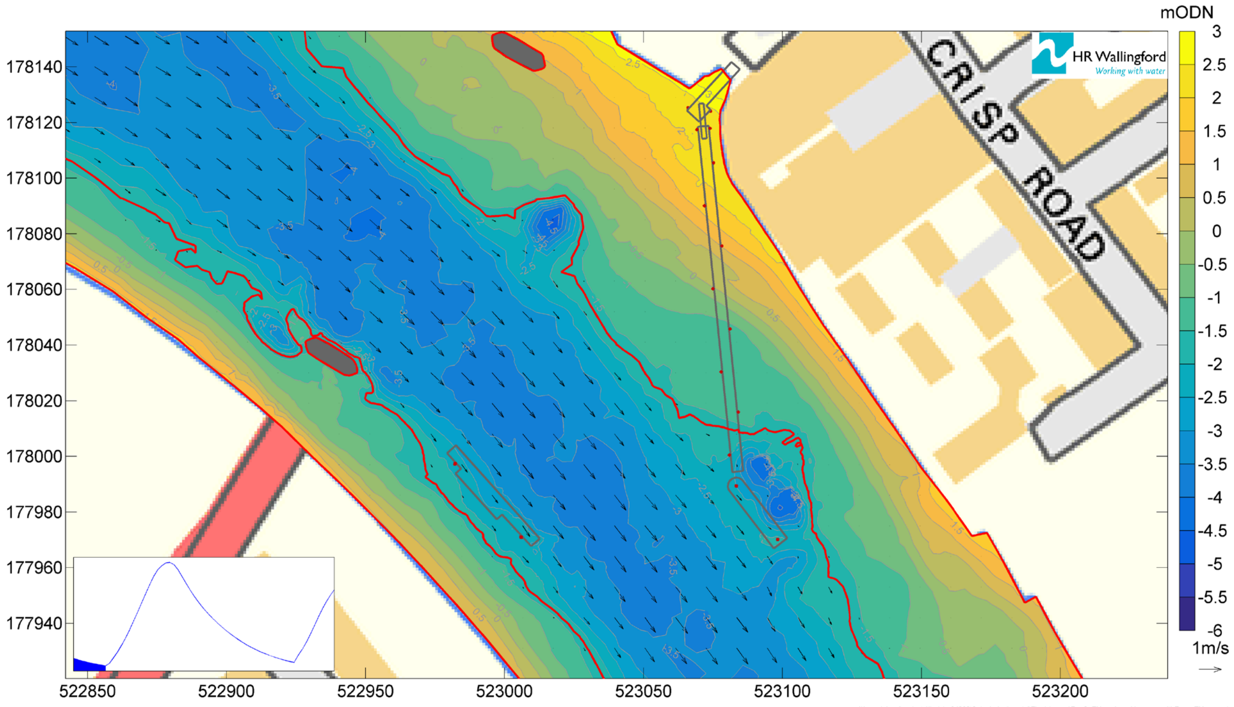

Figure 4.11: Foreshore bathymetry, with the typical low water line indicated approximately as the red contour

Background contains OS data © Crown Copyright (2019)

Figure 4.12: Drainage channels evident on the foreshore beneath the floating walkway

Source: HR Wallingford using Google Earth

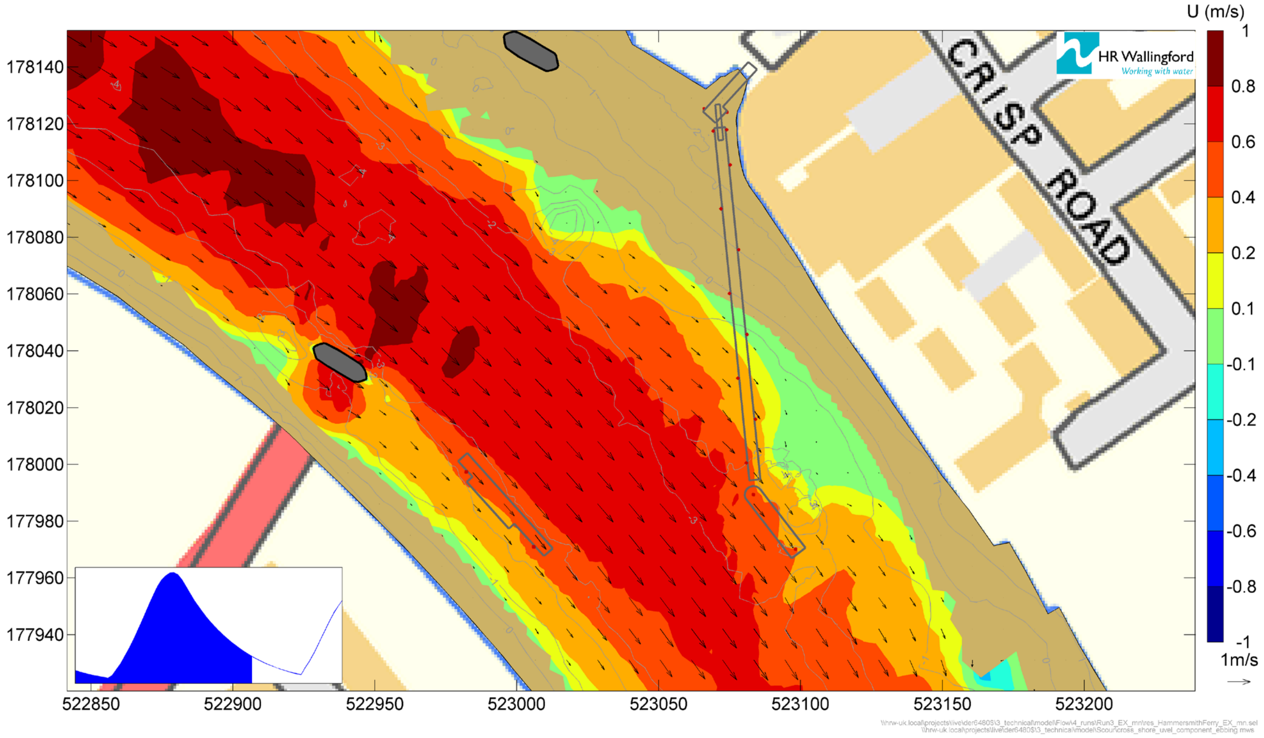

Figure 4.13: U-component of velocity as an indicator for speeds close to the grounding walkway as the foreshore drains

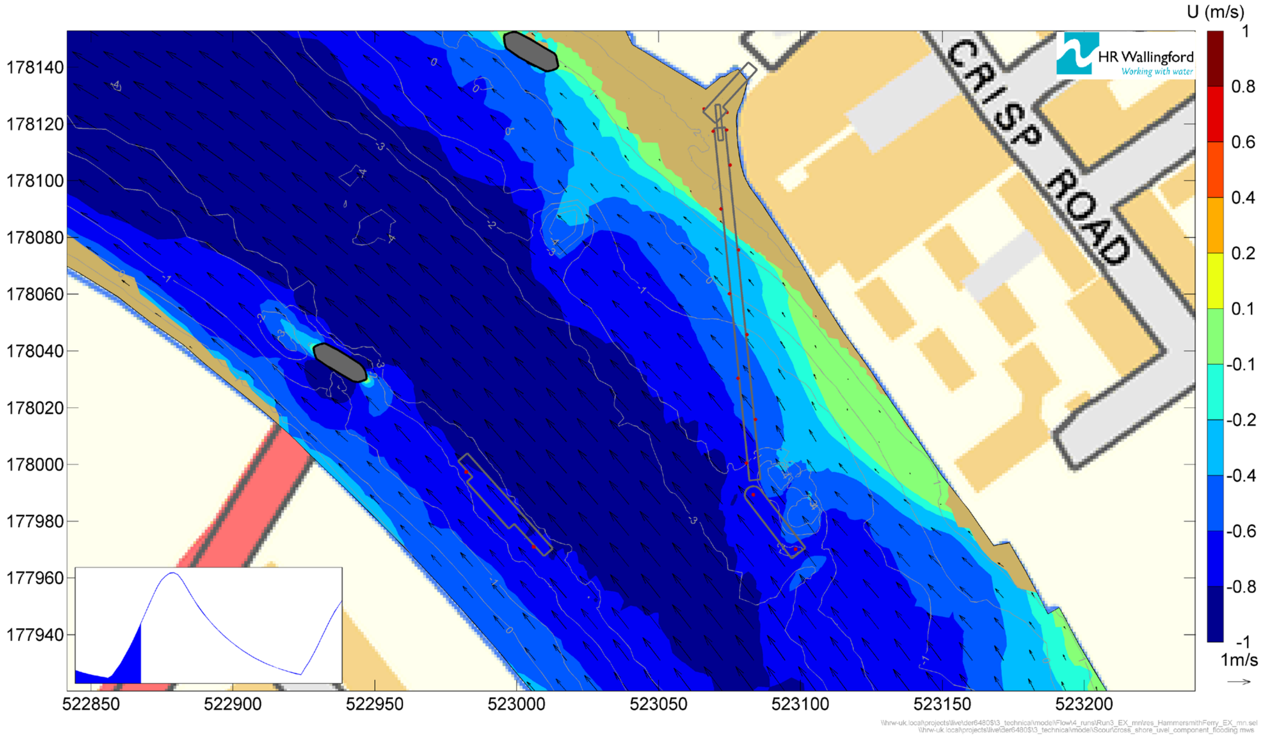

Figure 4.14: U-component of velocity as an indicator for speeds close to the grounding walkway as the foreshore drains

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4.5. Scour assessment discussion and conclusions

Local scour may occur around the proposed piles at the Hammersmith and Barnes Temporary Pier, and the piles restraining the floating walkway, to depths no deeper than 1 m, but more than likely restricted to less than 0.5 m.

This predicted scour depth is unlimited by the presence of a stronger underlying layer of clay, which is known to be present in the tidal Thames with varying thicknesses of overlying mobile material. It is the thickness of this mobile material (sandy gravel at the Hammersmith site) that will ultimately control the scour depths that develop around the piles. The limited available geotechnical data defining this layer indicates that it is less than 1 m thick in the vicinity of the works, which would limit scour depths to a similar level. It is recommended that scour predictions are updated if and when site specific geotechnical data becomes available.

The scour observed at the existing southern Hammersmith Bridge pier is observed to occur to depths of 0.6 m on the downstream side, which provides an analogy for the maximum scour depths that can be expected for the conditions at the site. The observed scour depths here help support the predictions made above.

Consideration has been given to the potential flow speed increases at the Hammersmith Temporary Pier piles during a outfall discharge event. The results show that there is limited increased risk of scour due to the proximity to the outfall.

The risk of local scour occurring of the grounded floating walkway is considered to be low. Any scour that does occur during flooding and draining is expected to be within the bounds of natural variability.

5. Summary and conclusions

The collated summary and conclusions are provided in the Executive Summary at the start of this report and for brevity are not repeated here.

6. References

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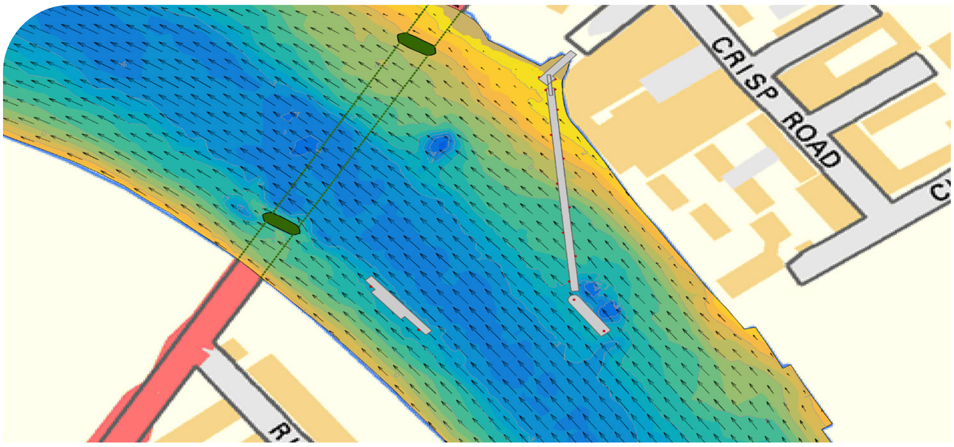

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