Thurrock Flexible Generation Plant

Environmental Statement Volume 6

Appendix 17.2: Hydrodynamic Modelling and Sediment Assessment

Date: February 2020

Environmental Impact Assessment

Environmental Statement

Volume 6

Appendix 17.2

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Summary

This document provides details of the numerical hydrodynamic modelling undertaken in support of the coastal process assessment in Volume 3, Chapter 17: Marine Environment. The document describes the modelling undertaken and the results obtained which have provided the basis for the EIA. It also provides details of the supporting sediment transport analysis.

Qualifications

The document has been prepared by Helen Godwin, a consultant numerical modeller at ABPmer. Helen has an MSc in Engineering in the Coastal Environment and three years' experience in hydrodynamic and sediment modelling at ABPmer.

It has been checked by Peter Whitehead an Applied Marine Scientist at ABPmer with over 40 years' experience in estuary processes, sediment dynamics and EIA.

1. Introduction

1.1.1 ABPmer has been commissioned by Thurrock Power Ltd to provide estuary physical process input to the Volume 3, Chapter 17: Marine Environment of the Thurrock Flexible Generation Power Station Environmental Statement (ES). This includes the production of a baseline environmental impact assessment (EIA) of the potential effects of a temporary Abnormal Indivisible Load (AIL) Facility, a causeway, on the hydrodynamics and sediment transport processes of the Thames Estuary. The AIL Facility is required to deliver abnormal indivisible loads (AILs) to the construction site via barge.

1.1.2 To assist with the EIA, a hydrodynamic model has been set up and calibrated. This technical appendix provides a description of the hydrodynamic model that was applied for the assessment and details the setup and calibration process undertaken to demonstrate that the model produces a representative simulation of the existing processes within the domain. The results of the modelling have been analysed and interpreted using the baseline understanding and expert geomorphological analysis to determine the extent and magnitude of hydrodynamic change expected as a result of the AIL Facility.

1.1.3 The numerical model results have been used to support an empirical analysis of the effects of the AIL Facility on sediment mobilisation and sediment transport. The analysis methods and results are presented in the subsequent sections of this report.

1.1.4 Finally, the AIL Facility will require the removal of circa 16,000 m^3 of material. A plume assessment has been completed using the results of the numerical modelling undertaken for the adjacent Tilbury2 development.

2. Hydrodynamic Modelling

2.1 Introduction

2.1.1 The hydrodynamic modelling has been completed using the Danish Hydraulic Institute (DHI) software package MIKE21 FM (Flexible Mesh), which has been developed specifically for applications within oceanographic, coastal and estuarine environments. MIKE21 FM HD is the 2D *hydrodynamic* module, allowing assessment of temporal and spatial variations in water levels and depth-averaged currents. The MIKE FM software utilises an unstructured grid approach, providing the optimal degree of flexibility in the representation of complex geometries and enabling smooth representation of boundaries. In this way, small mesh elements may be used in areas where more detail is required, and larger elements in areas where less detail is needed.

2.2 Boundary Conditions

2.2.1 The model has two boundaries, at the eastern and western extent of the model. The eastern boundary is forced with water-levels from Total Tide (TT) at Coryton. The western boundary is forced with both water-levels and flows from TT at Erith.

2.3 Grid

2.3.1 The model grid spans a section of the Thames between Erith to the west and Coryton to the east. The model grid utilises the flexible mesh feature of the MIKE 21 software allowing the grid resolution to vary, with areas of interest typically covered with a higher resolution to increase the accuracy and level of detail. The more distant reaches are represented at a coarser resolution to aid computational efficiency. The outer extents of the model have a resolution of about 60 m, increasing to 30 m in the middle section of the River. The main area of interest is at 10 m resolution (red box in Figure 2.1).

Figure 2.1: Extent and resolution of model grid.

2.4 Bathymetry

2.4.1 Model bathymetry was obtained from the UK Hydrographic Office (UKHO) and Environment Agency LiDAR. The coverage, source and resolution of each dataset is summarised in Table 2.1. All datasets have a horizontal datum of British National Grid, and vertical reference datum of Ordnance Datum Newlyn (ODN). The datasets were interpolated onto the model grid to form the bathymetry within the model mesh (Figure 2.2).

Table 2.1: Model bathymetry source data.

Coverage	Source	Resolution (m)
Greenwich Reach to Coalhouse Point	UKHO, 2017	30–50
Oaze to Coalhouse point	UKHO, 2017	30–50
Site area	Environment Agency LiDAR tile, 2016	1

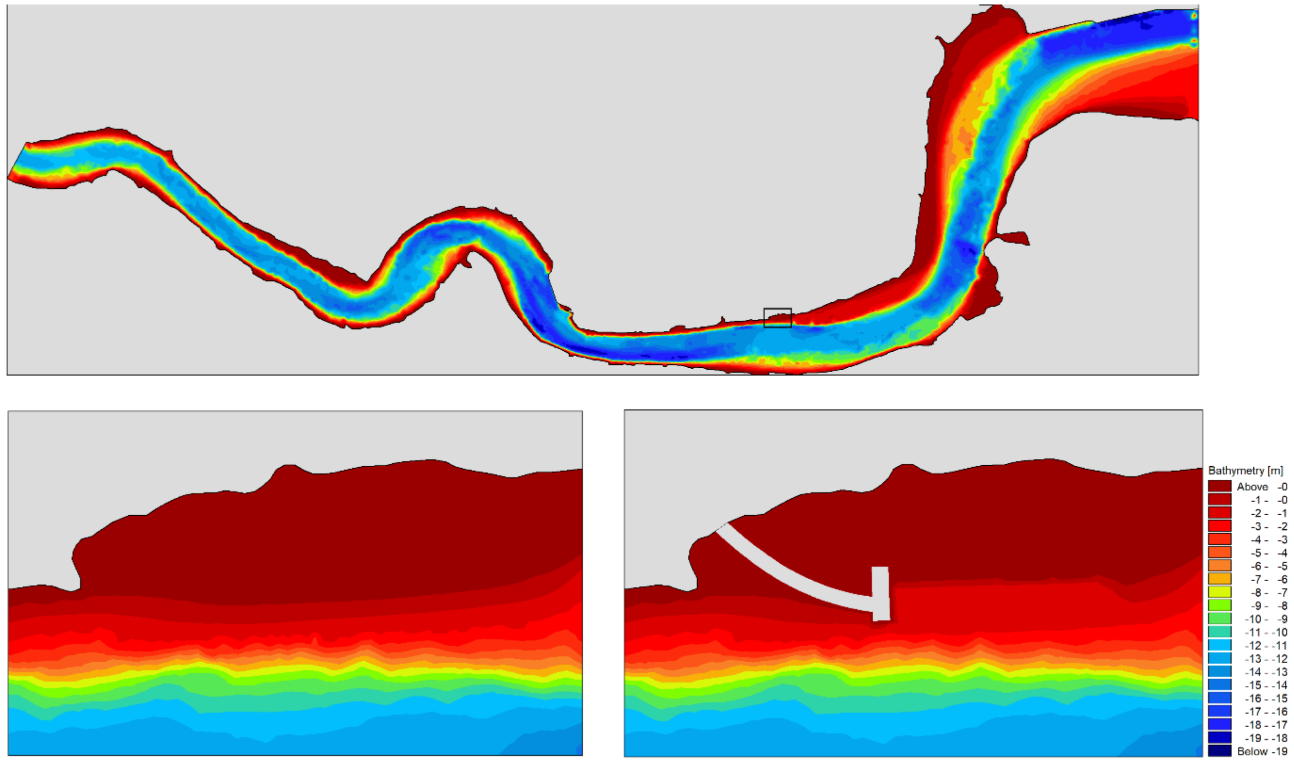

Figure 2.2: Interpolated bathymetry over the whole model domain (top) and across the site (bottom left). Bottom right shows the mesh bathymetry as a result of the addition of the AIL Facility.

2.5 Bed Roughness

2.5.1 Bed roughness in the model describes the friction from the seabed 'felt' by moving water. Changing the magnitude of bed roughness locally affects the rate at which water moves in that area and so can affect tidal water level, range and phasing, and the speed of tidal currents. As such, bed roughness is a key variable in the model that optimises the model performance in comparison to measured data. Following a series of sensitivity tests, the final bed roughness map varies between 20 and 60 $m^{(1/3)}$/s, dependant on depth.

2.6 Bed Shear Stress

2.6.1 The model calculates the bed shear stress (BSS) that is exerted on the bed sediments. To assess these effects with respect to the potential for sediment accretion or erosion, approximate thresholds of deposition and erosion have been derived using data provided in Whitehouse *et al*. (2000) for the fine mud sediments that are present on the intertidal mudflats at the location of the proposed AIL Facility. To provide a worst-case assessment for sediment movement a bed density of 1080 kg/m³ associated with 'fresh', recently settled sediment (i.e. before consolidation) has been used to determine the BSS threshold for erosion. The derived parameters are:

i. Threshold for deposition – 0.1 N/m²; and
ii. Threshold for erosion – 0.24 N/m²

2.7 Scheme for Assessment

2.7.1 The AIL Facility modelled has a surface width of 12.5 m and length of 181 m. The causeway ties into the existing ground level (approximately 4 mODN), to approximately 0.88 mODN at the crane platform adjacent to the proposed dredged vessel berthing area. The edges of the causeway are formed of rock revetments at a slope of 1:3 (v:h) to merge with the existing bed levels. The vessel berthing and approach areas have been levelled to -1.77 mODN, with a near slope of 1:5 to intersect with the existing bathymetry. The causeway concept design is provided as Figure 2.3.

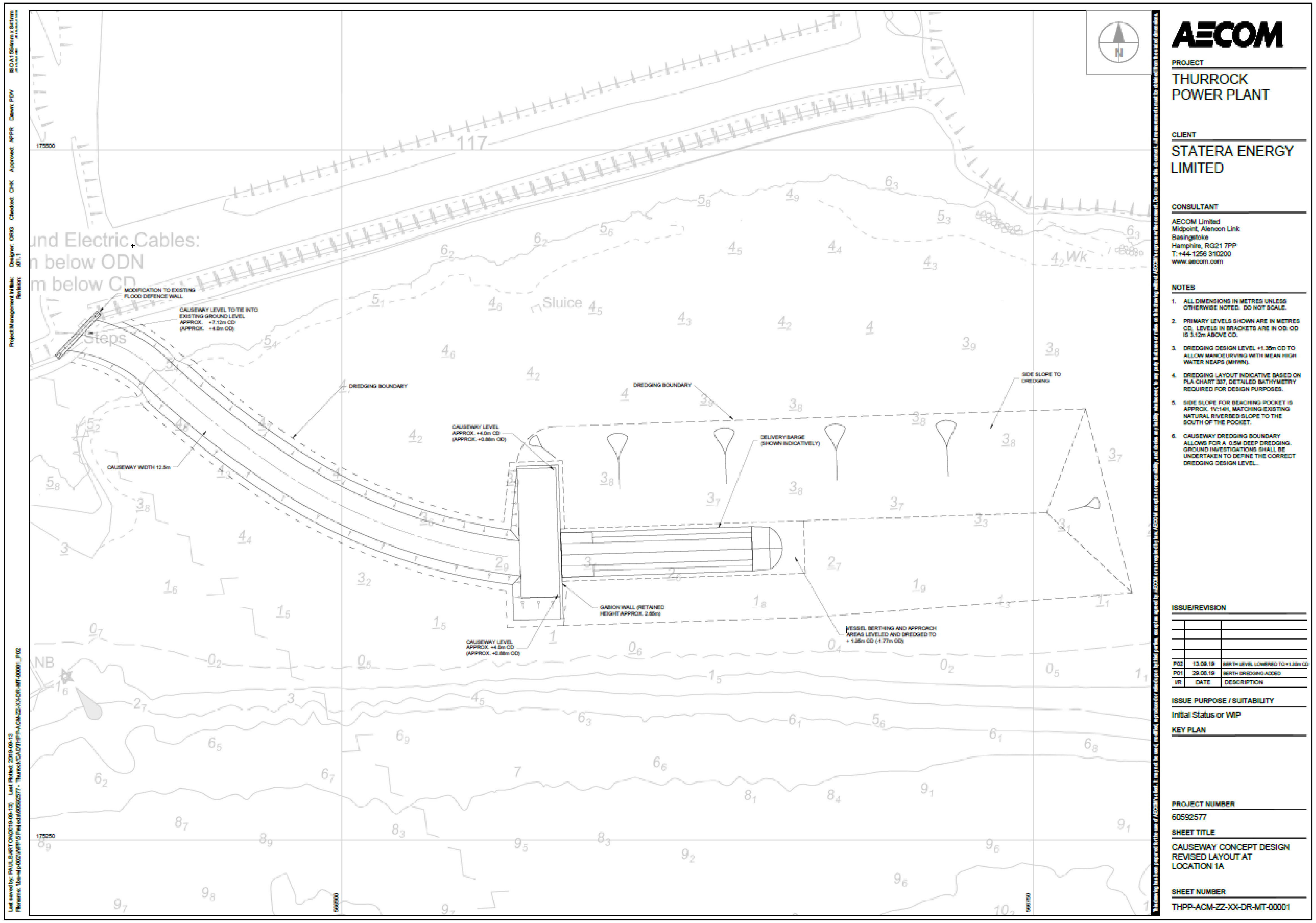

Figure 2.3: AIL Facility concept design.

2.7.2 Four scenarios have been modelled to assist with the EIA:

i. Baseline – "current" hydrodynamic conditions to assess the impact of the AIL Facility on the hydrodynamics;
ii. AIL Facility – causeway, crane platform structure and berth pocket added to the model;
iii. AIL Facility with Roll on Roll off (RoRo) vessel – as above, however with the addition of a RoRo barge of 3.5 m berth at the end of the AIL Facility (Figure 2.3); and
iv. Cumulative Assessment – AIL Facility with RoRo vessel modelled in combination with the Tilbury2 development, with vessels moored alongside as assessed in HR Wallingford (2017) for the ES of that development.

2.8 Calibration

2.8.1 Due to the lack of measured data available the model calibration was carried out against TT data. TT data is available for 3 sites across the model domain; Coryton on the eastern boundary, Tilbury near to the site in the centre of the model, and Erith on the western boundary (Figure 2.4).

2.8.2 Figure 2.5 shows TT and model water levels at Coryton, Tilbury and Erith over a 5-day period. When comparing the model water levels to the TT water levels, it is evident that both show the same pattern of propagation through the river, with a lag time in high water of approximately 15 minutes and 25 minutes between Coryton and Tilbury, and Tilbury and Erith respectively.

Figure 2.4: Erith, Tilbury and Coryton calibration sites in relation to the site (red box).

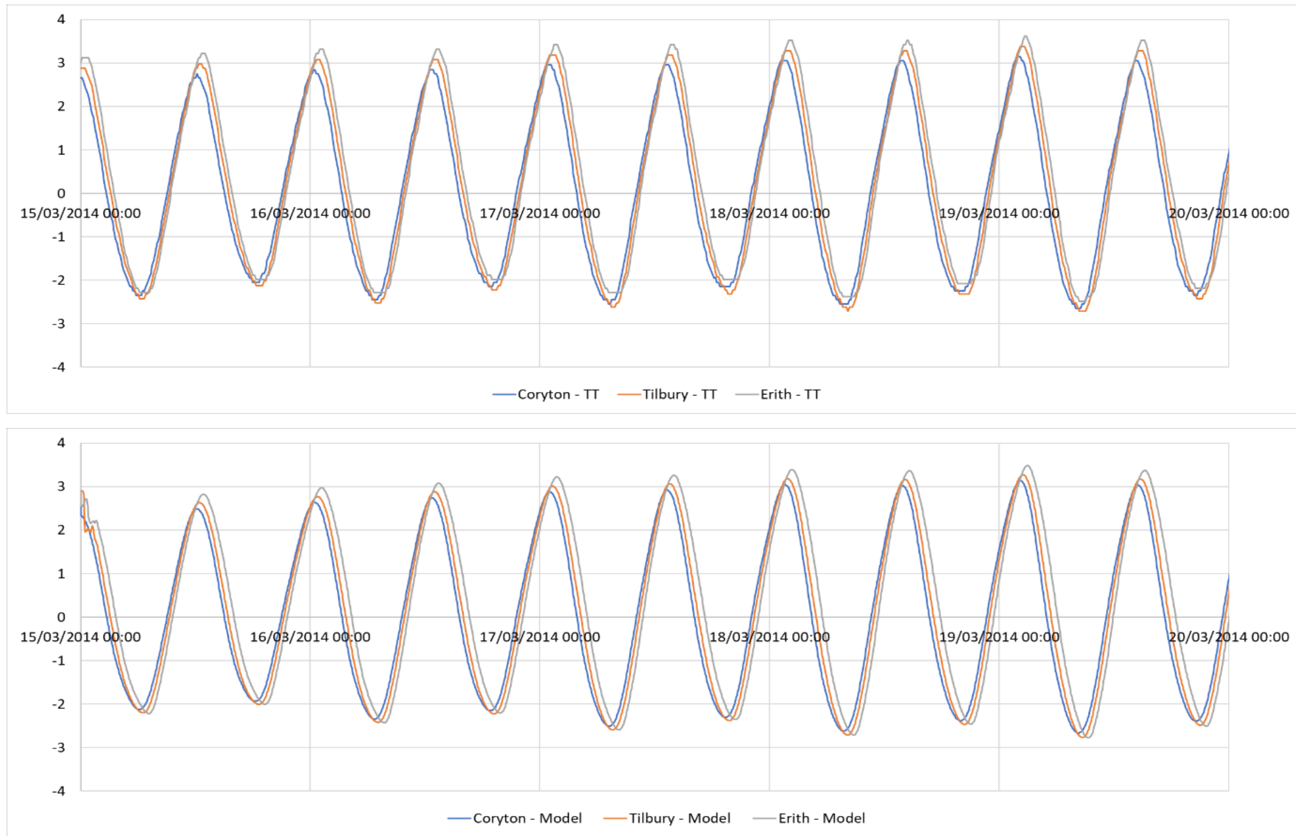

Figure 2.5: Water level at Coryton, Tilbury and Erith over a 5-day period of TT data (top) and model data (bottom).

2.8.3 Comparison with TT water level data at Coryton (Figure 2.6) shows the model to be performing well, with no discernible difference in phasing and only a slight under/overprediction in levels on a few tides. When looking at data for the same period at Tilbury, again the model is performing well (Figure 2.7). There is a slight difference in phasing, with the model approximately 15 minutes behind the TT data. TT flow speed data was not available.

2.8.4 When considering the flows at Tilbury, the model is capturing the magnitudes well, particularly on the ebb tide. Generally, on the flood there is a slight underprediction of flows, however this is no more than approximately 0.05 $m.s^{-1}$.

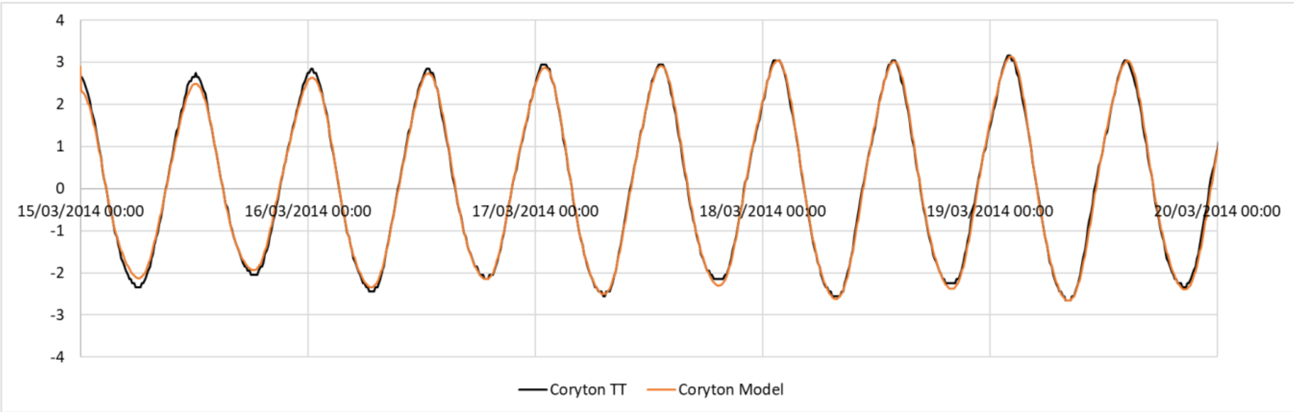

Figure 2.6: Comparison between modelled and TT water level data at Coryton.

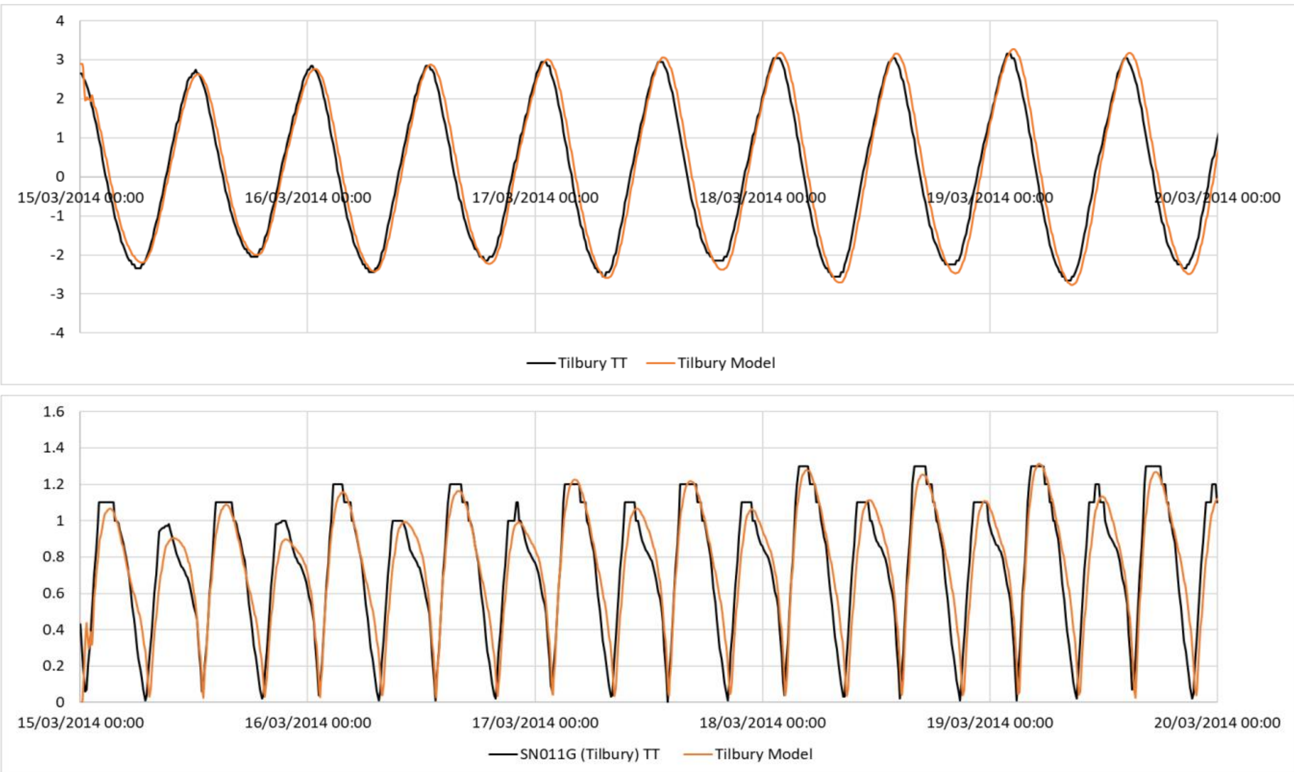

Figure 2.7: Comparison between modelled and TT water level (top) and flow speed data (bottom) at Tilbury.

2.8.5 Figure 2.8 presents the flow speeds and direction across the whole channel during peak flood and peak ebb. Once again this shows the model to be behaving as expected, with higher flows in the main channel, reducing towards the banks on either side. There is an evident divergence and reduction in flows around the existing jetty structures.

2.8.6 The model is therefore shown to correctly represent the propagation of the tide through the area, whilst closely simulating the flow speed and directional distribution, including localised jetty effects. The model therefore sufficiently reproduces the hydrodynamics to allow EIA of the effects of the AIL Facility on the flow regime of the local Thames Estuary.

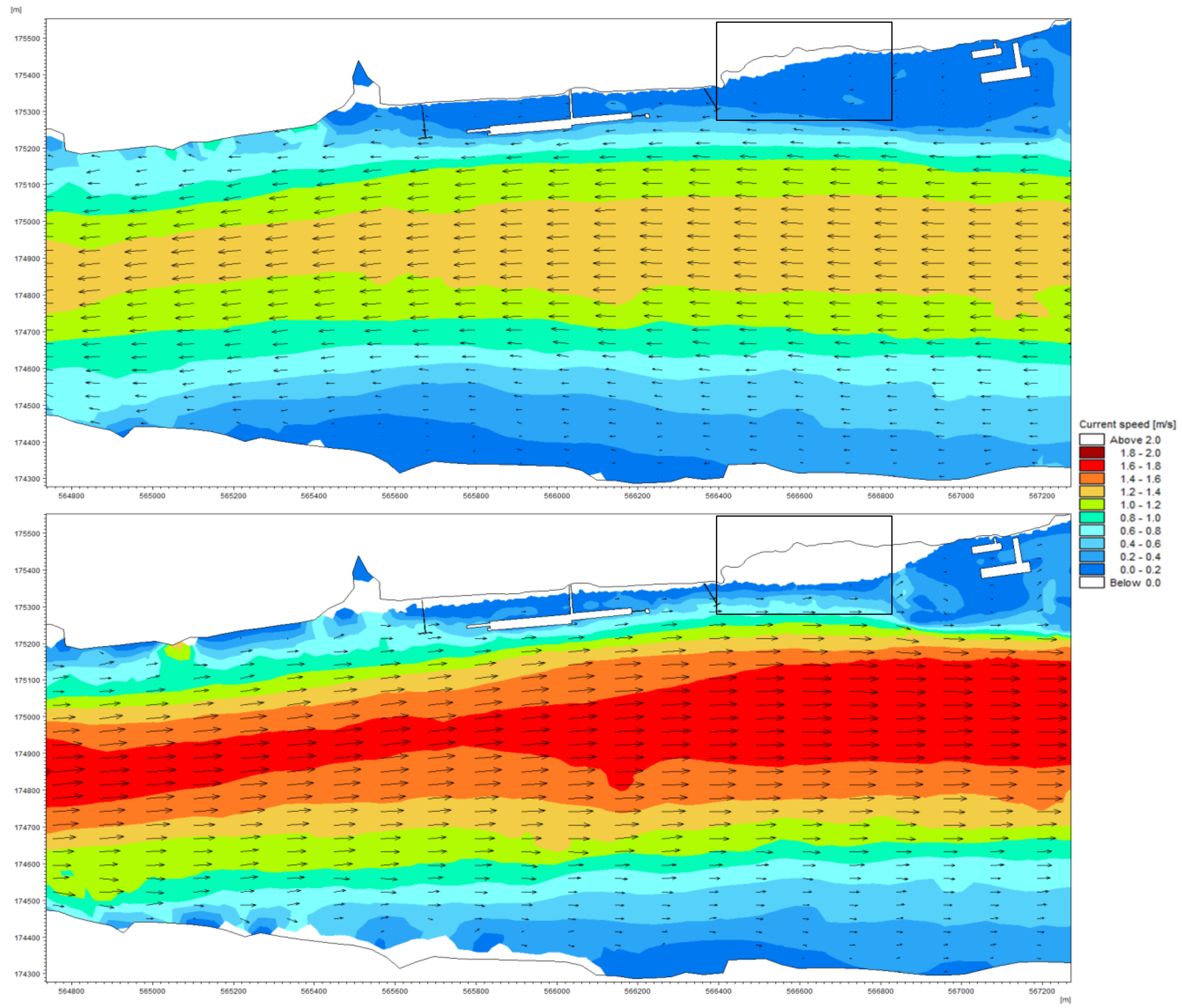

Figure 2.8: Local flow speeds during peak flood (top) and peak ebb (bottom). Black box indicates the site.

3. Hydrodynamic Model Results

3.1 Introduction

3.1.1 The results of the hydrodynamic modelling are presented as follows:

i. AIL Facility;
ii. AIL Facility with RoRo Vessel; and
iii. Cumulative Assessment.

3.1.2 Three maps plots are provided for the time when maximum changes are observed during flood and ebb tides compared to the baseline to show the likely maximum spatial extent of effect of the scenarios. The plots show the flow speeds and direction through the area without the AIL Facility in place (baseline), the difference from the baseline in the context of the estuary reach and at increased resolution to show the detail of the change around the development.

3.1.3 A further 6 timeseries plots are provided for locations within the area of change to indicate the temporal variation of effects through the tide. The plots compare the spring tide current speed, direction and bed shear stress (BSS) as a result of the three scheme runs.

3.2 AIL Facility

3.2.1 Figure 3.1 and Figure 3.2 show baseline peak flow speeds through the area are generally less than 0.2 $m.s^{-1}$. The effect of the AIL Facility on both the maximum flood and ebb tide flows is minimal and confined to the immediate area surrounding the proposed structure, with no effects of note extending into the subtidal area.

3.2.2 The overall effects of the AIL Facility, down to a difference of less than ± 0.01 $m.s^{-1}$ from baseline flows (circa 5% change) is shown to extend 215 m and 250 m up and down estuary.

3.2.3 The greatest reduction in flow occurs shoreward over the intertidal from the seaward edge of the structure with the extent of change extending for circa 50 m towards the shallow subtidal. Within this area, on the flood tide the combined effect of the causeway and berth enhance an existing flow eddy over the berth area. This reduces the flows at the eddy centroid and increases flows at the edges. The flow is concentrated around the causeway. Elsewhere, flows are generally reduced.

3.2.4 Figure 3.2 shows the greatest changes occur to the ebb flows, where flow speeds are generally reduced by up to 0.12 $m.s^{-1}$ (circa 30%). The causeway deflects flow around the head, which is then drawn towards the outer edge of the berth, increasing flows by up to 0.04 $m.s^{-1}$ at the down river end of the berth as the flow move back towards the shore.

3.2.5 The vector plots on both stages of the tide also show the impacts on current directions are minimal, with only small changes in direction in the wake of the structure.

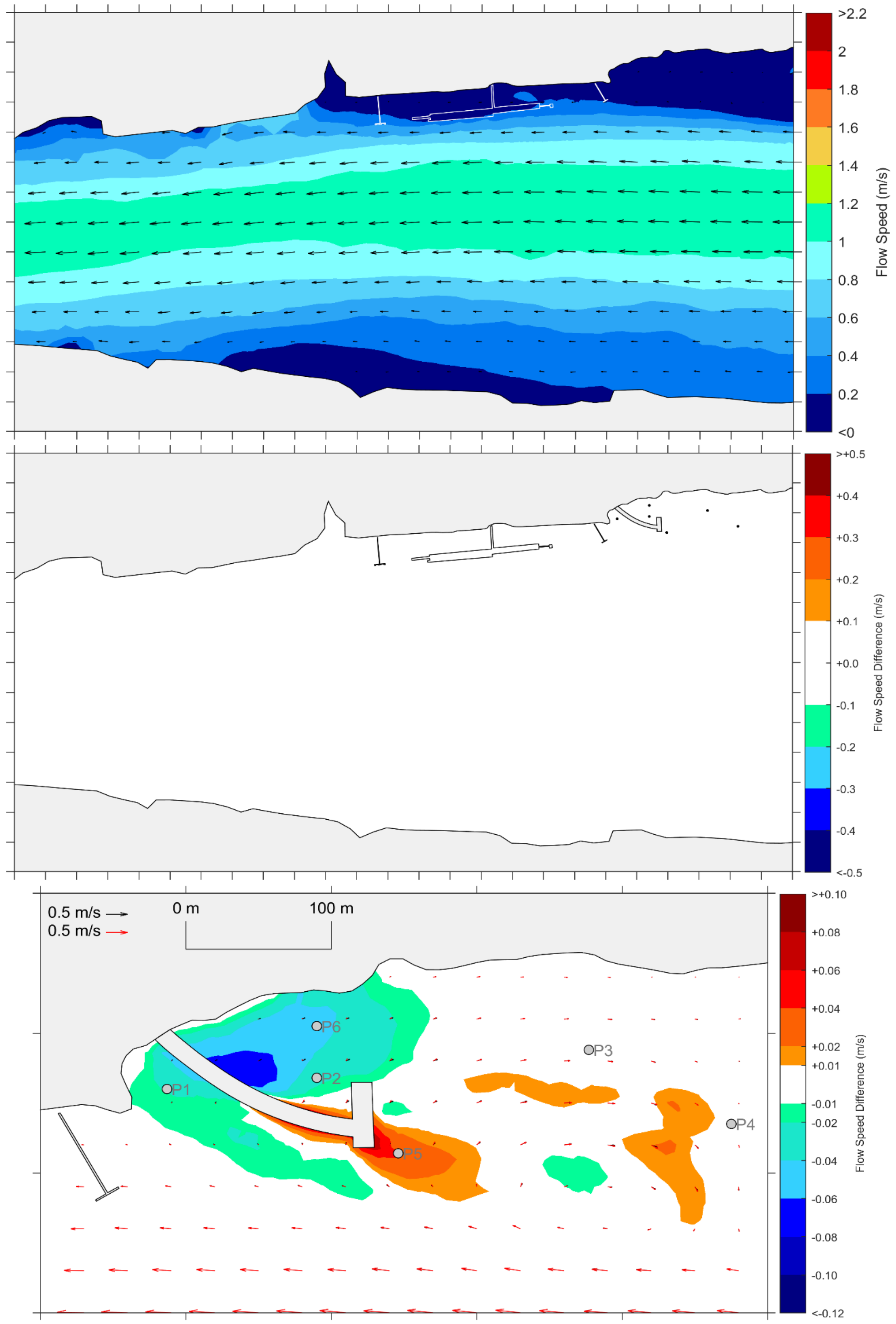

Figure 3.1: Baseline flow speeds during flood tide (top). Effect of AIL Facility (middle and bottom).

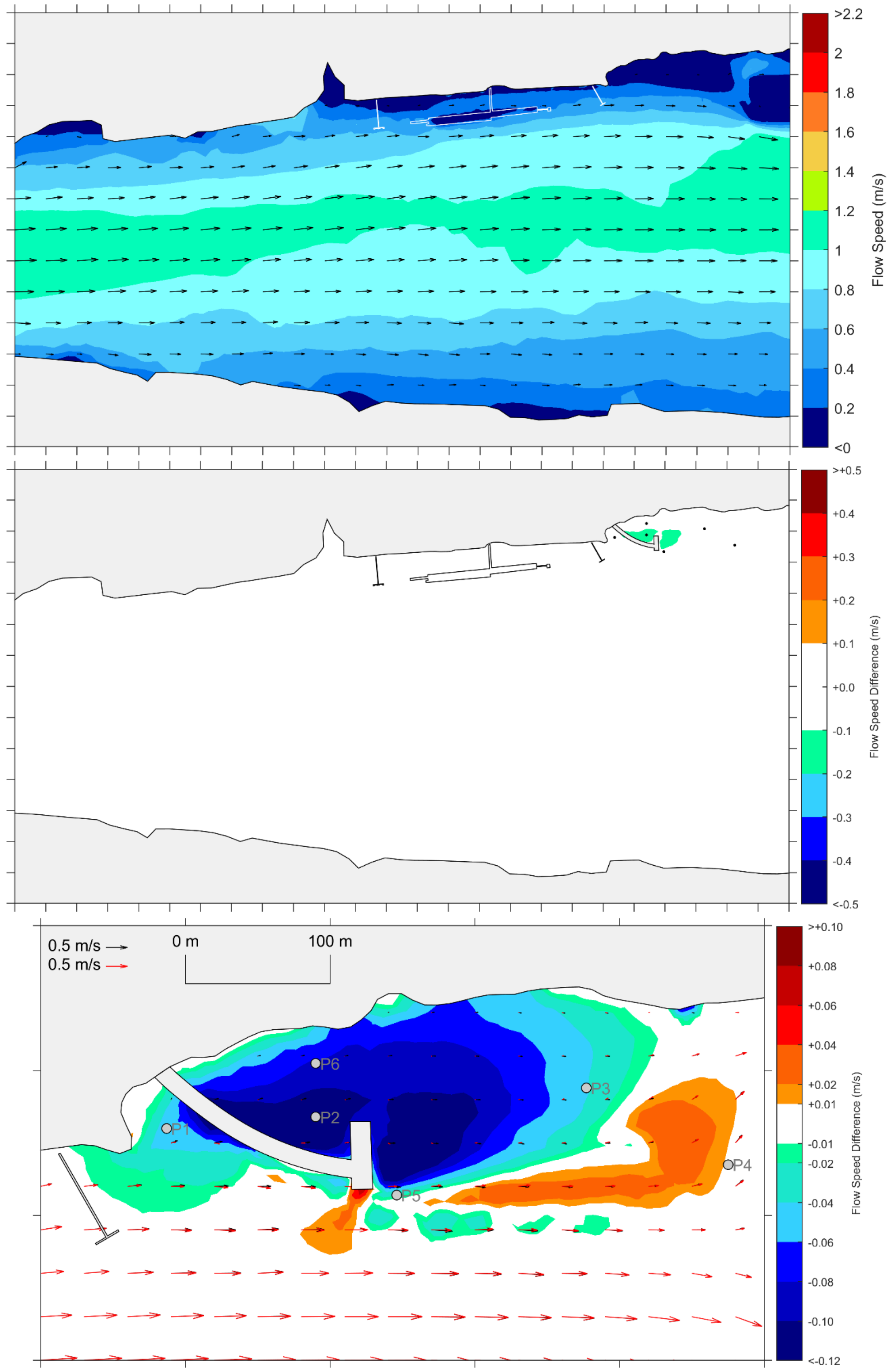

Figure 3.2: Baseline flow speeds during ebb tide (top). Effect of AIL Facility (middle and bottom).

3.2.6 In general, the timeseries plots, inshore of the causeway level, both up and down the river show a similar pattern of reduced flows, whilst the intertidal is inundated for circa 5 hours on each tide (Figure 3.3, Figure 3.4, Figure 3.5).

3.2.7 At these locations, flow speed reductions of up to 50% are evident, for the most part reducing absolute flows to less than 0.1 $m.s^{-1}$. Baseline flow directions are generally in a north to northeast direction both on the flood and ebb, reflecting the flood tide eddy in this area up river of the AIL Facility (location P1, Figure 3.3).

3.2.8 A small direction change is evident, but the AIL Facility does not materially change the existing eddy in this area. Down river the flow directions become more rotary, varying from north as the tide initially floods, then swings through east over high water and towards the south as the mudflat drains.

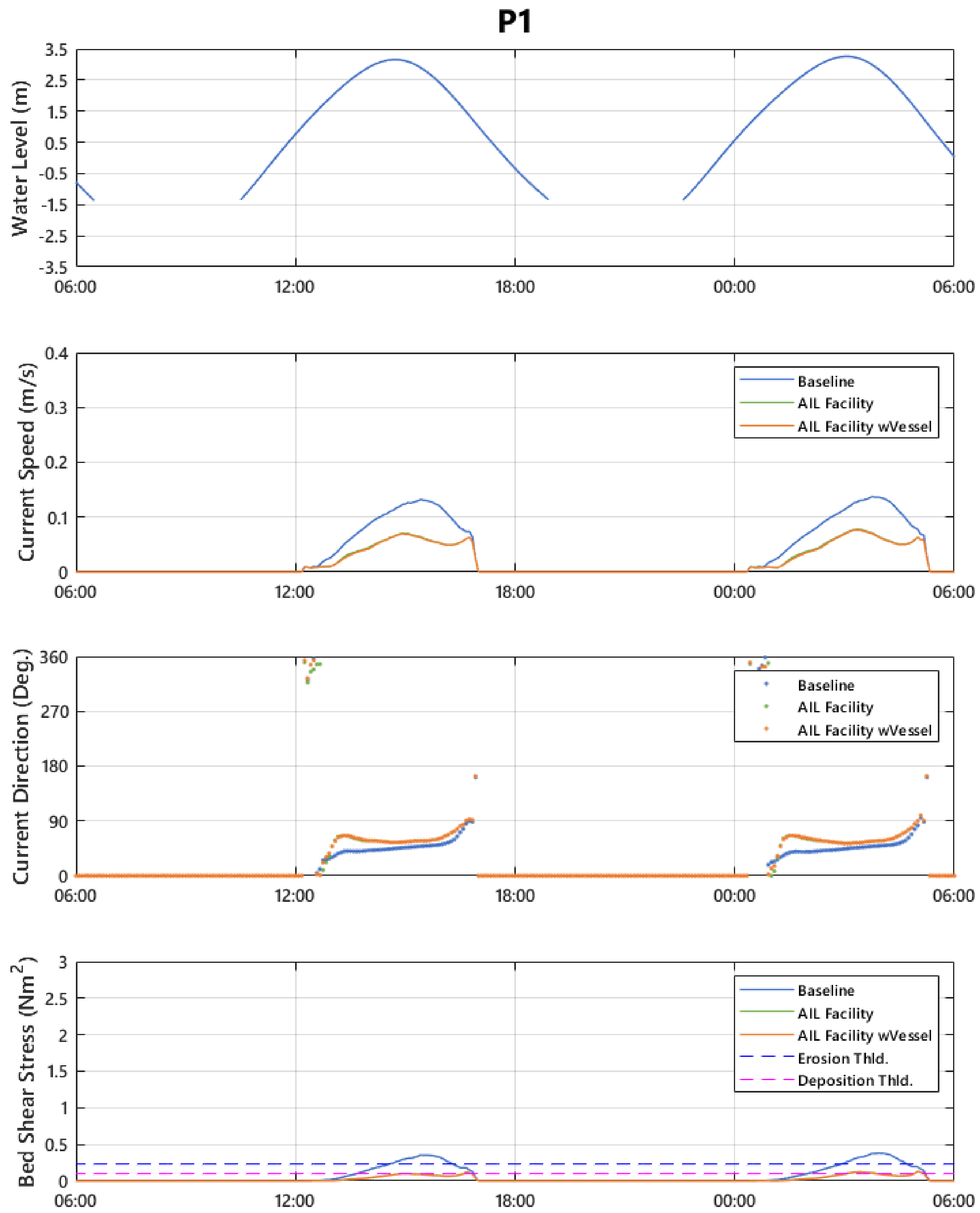

Figure 3.3. Comparison of current speed, direction and BSS as a result of the AIL Facility and AIL Facility with RoRo vessel compared to the baseline at P1.

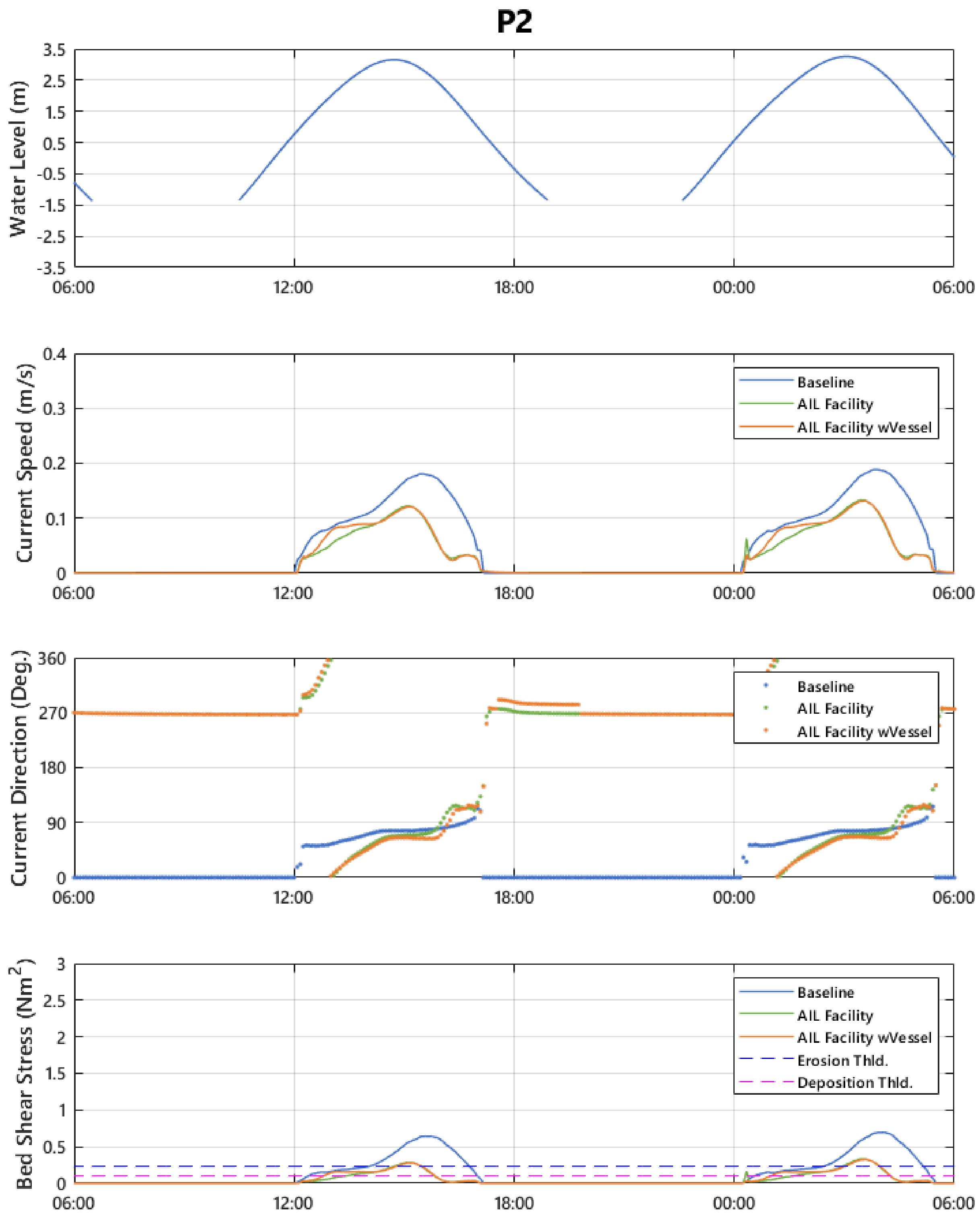

Figure 3.4: Comparison of current speed, direction and BSS as a result of the AIL Facility and AIL Facility with RoRo vessel compared to the baseline at P2.

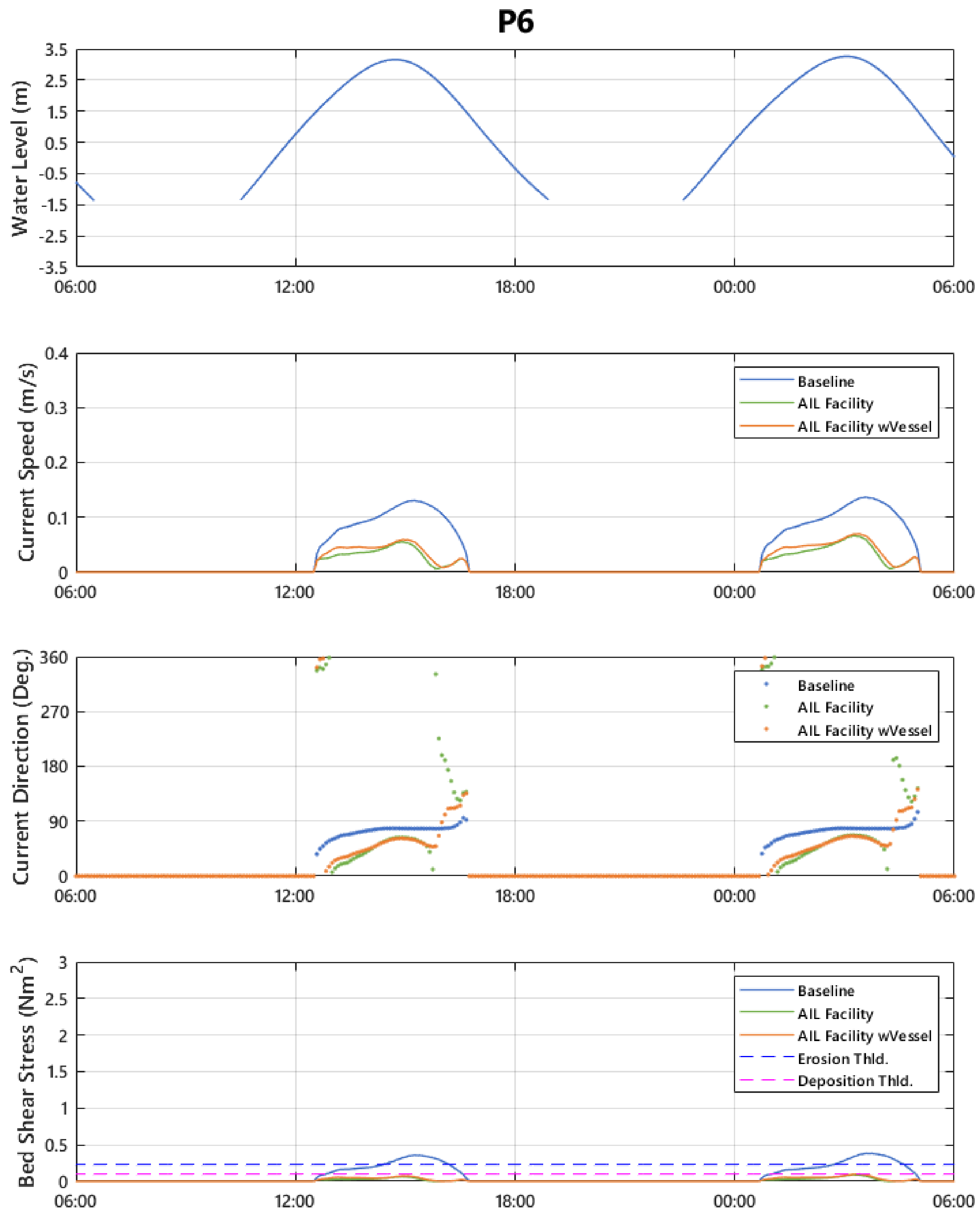

Figure 3.5: Comparison of current speed, direction and BSS as a result of the AIL Facility and AIL Facility with RoRo vessel compared to the baseline at P6.

3.2.9 Location P4 (Figure 3.6) at the very eastern end of the dredged area shows an anticlockwise rotation of the tide from west to east during the period of mudflat inundation, with the greatest flows on the ebb, just before the site drains (i.e. in very shallow water). The flow characteristics here are not changed by either scenario. The maximum change shown at P5 (Figure 3.1) is shown by Figure 3.7 to change in character throughout the tide this reflects the local effects created by the corner of the crane platform which changes in local characteristics through the tide. Overall however there is very little difference in the general tidal character at this location.

3.2.10 The changes to the bed shear stresses (BSS) generally show that locations immediately in the 'shelter' of the causeway for the most part will become predominantly depositional with BSS not substantially increasing above the likely threshold deposition. Under existing baseline conditions, the depositional tendency is generally 'outweighed' by the erosional tendency on the ebb tide, which would provide a status quo or marginal erosional tendency for the mudflat overall. This will change to an accretional tendency with the AIL Facility.

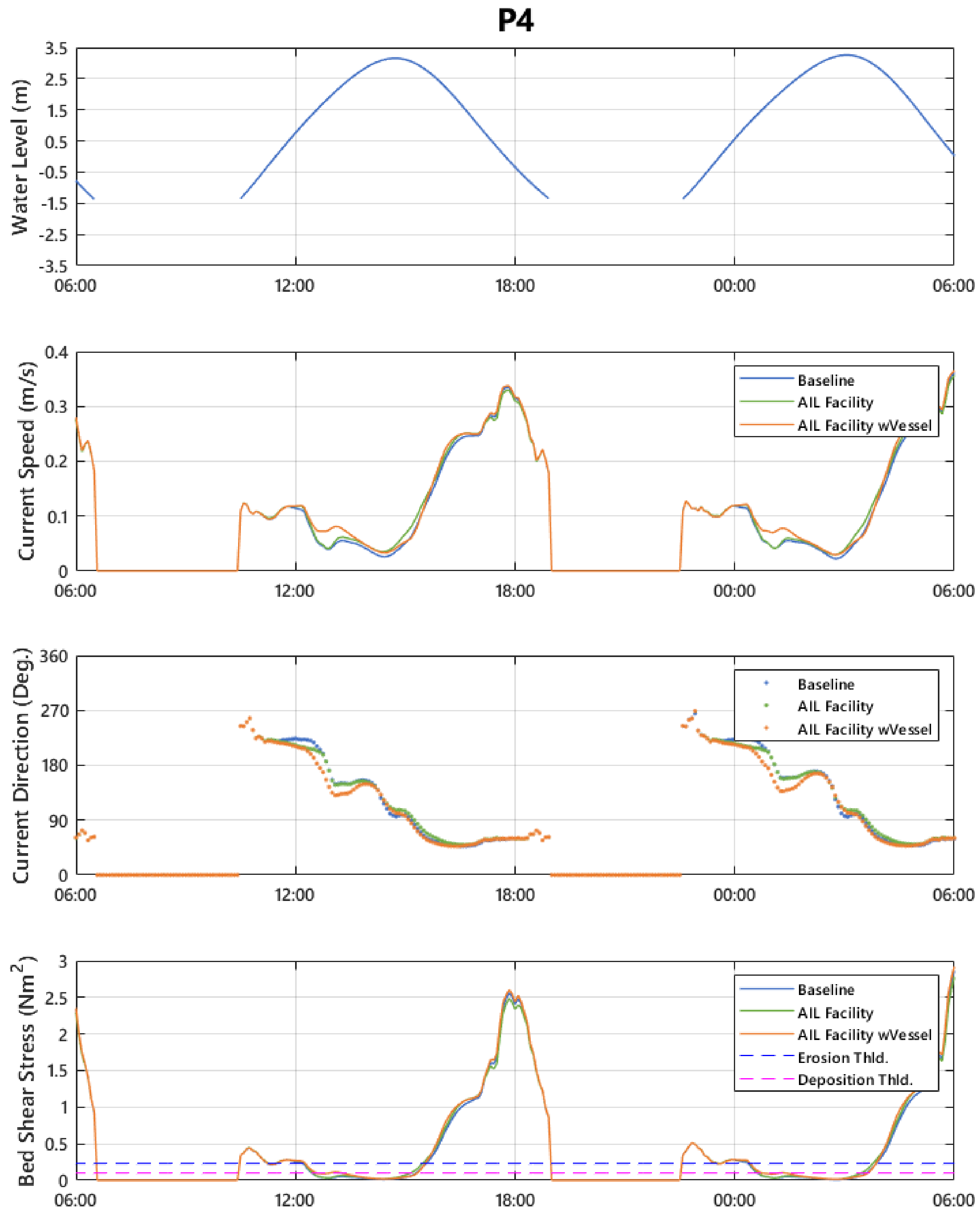

Figure 3.6: Comparison of current speed, direction and BSS as a result of the AIL Facility and AIL Facility with RoRo vessel compared to the baseline at P4.

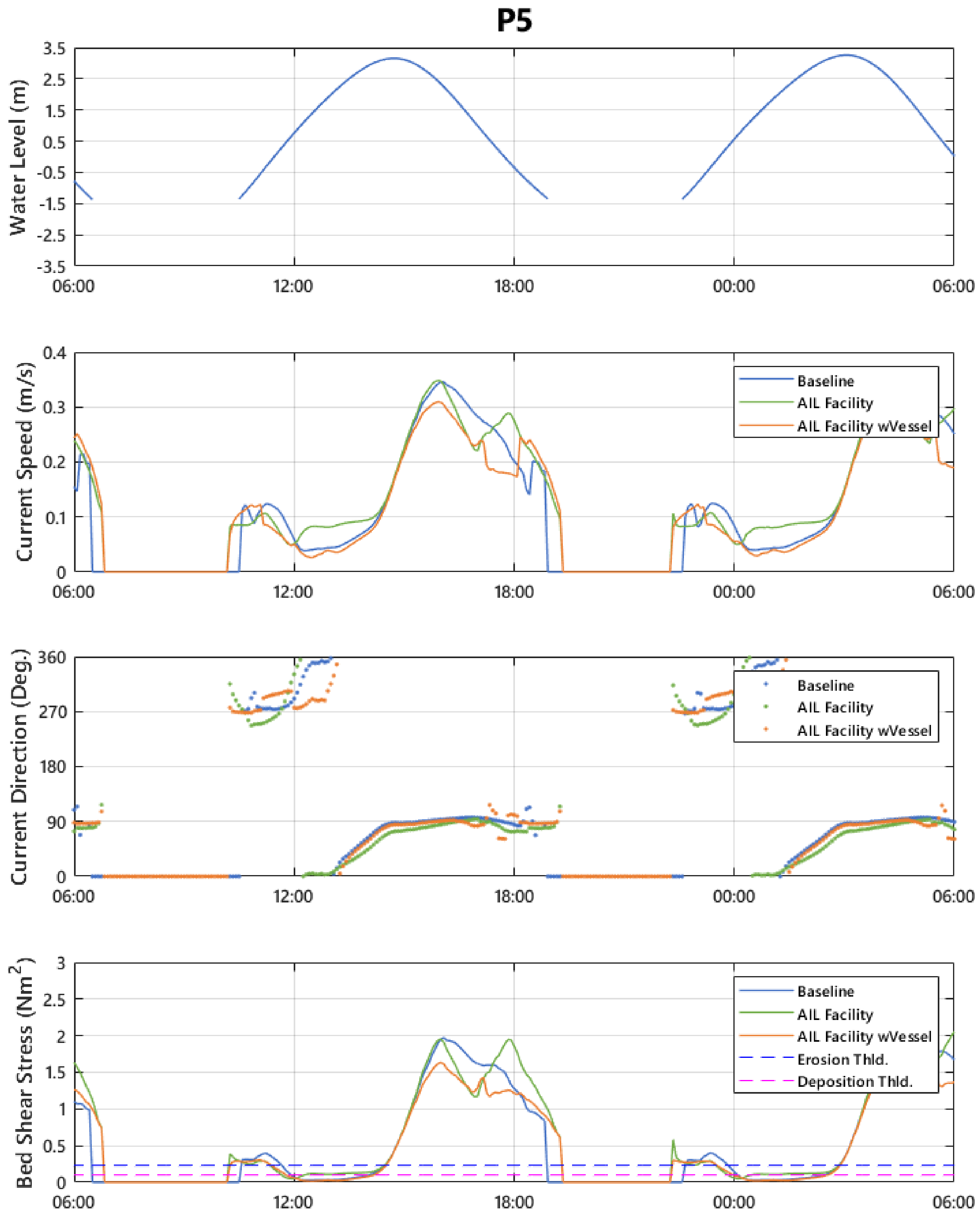

Figure 3.7: Comparison of current speed, direction and BSS as a result of the AIL Facility and AIL Facility with RoRo vessel compared to the baseline at P5.

3.3 AIL Facility with RoRo Vessel

3.3.1 Comparison of the flows around the AIL Facility with and without the RoRo vessel in the broad scale show very little difference. This is demonstrated when comparing Figure 3.1 and Figure 3.2 with Figure 3.8 and Figure 3.9. There is no effect on the main estuary flows or flood flows towards Tilbury2. The detailed difference plots show there are effects caused by the introduction of the vessel, however these are predominantly within the berth under the vessel and shoreward of the vessel.

3.3.2 On the flood, flows are increased above the baseline by up to 0.08 $m.s^{-1}$ below the vessel which increase flows further around the head of the causeway compared to the no vessel case. The vessel also 'blocks' the eddy flow from reaching the 'zone' inshore of the vessel, hence reducing the flow speeds in this area compared to both the baseline and AIL Facility alone. This change illustrated at timeseries location P3 (Figure 3.10). On the ebb, the vessel effect maintains the reduced flow created by the dredge and increases the effect marginally to the east. East of the causeway head, the vessel has little additional effects on the ebb flows. This is illustrated, for example by no difference between the scenarios, on timeseries plots for locations P1 and P2 (Figure 3.3 and Figure 3.4).

3.3.3 The BSS data infers that the vessel may cause slight scour of the berth pocket on the flood tide, but sedimentation will remain on the ebb. Whilst the vessel is in place, there is slightly more potential for sedimentation shoreward of the vessel than without the vessel. These effects however are unlikely to be noticeable, given the short periods of time and frequency of vessel movements.

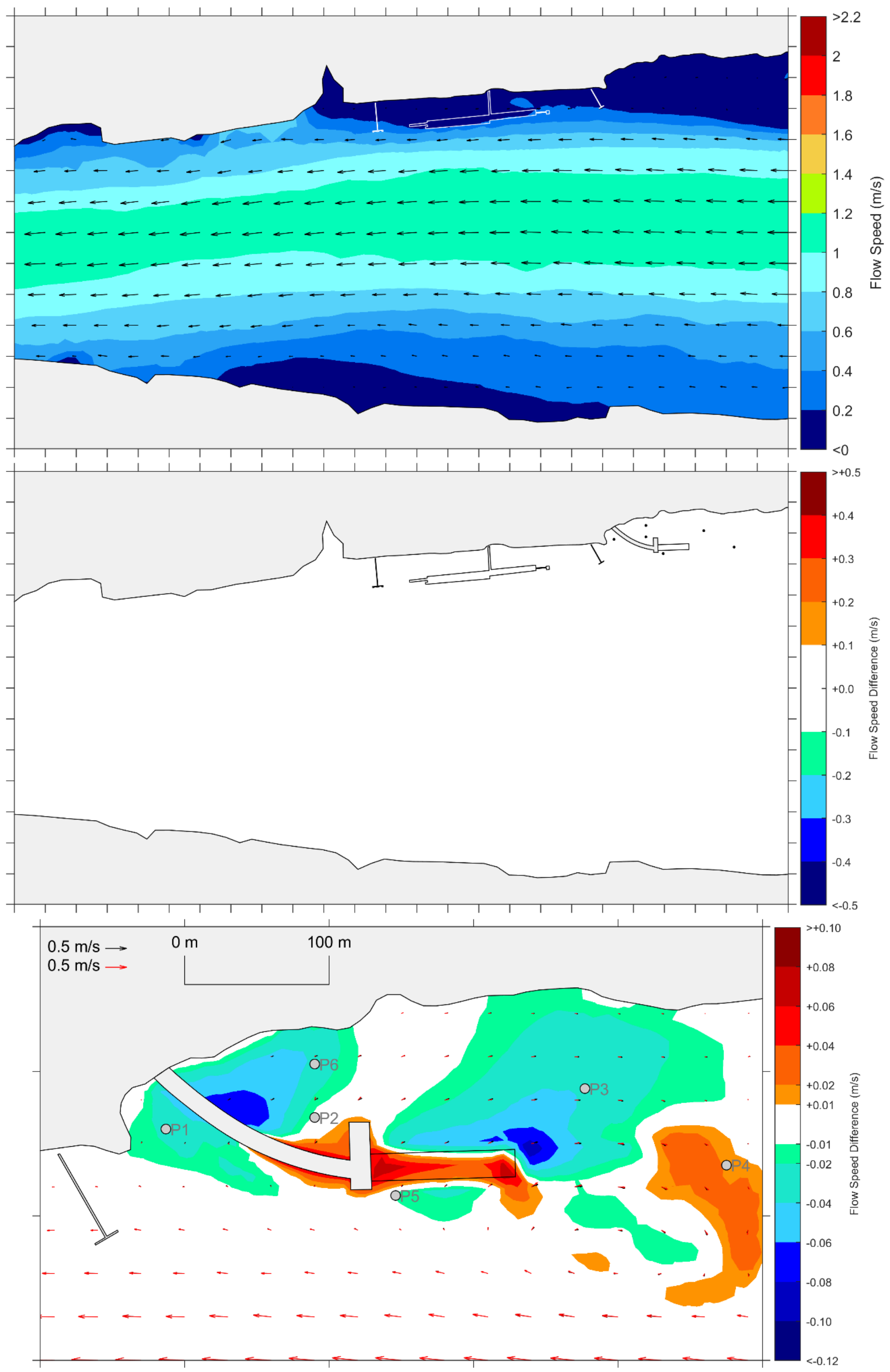

Figure 3.8: Baseline flow speeds during flood tide (top). Effect of AIL Facility with RoRo vessel on baseline flows (middle and bottom).

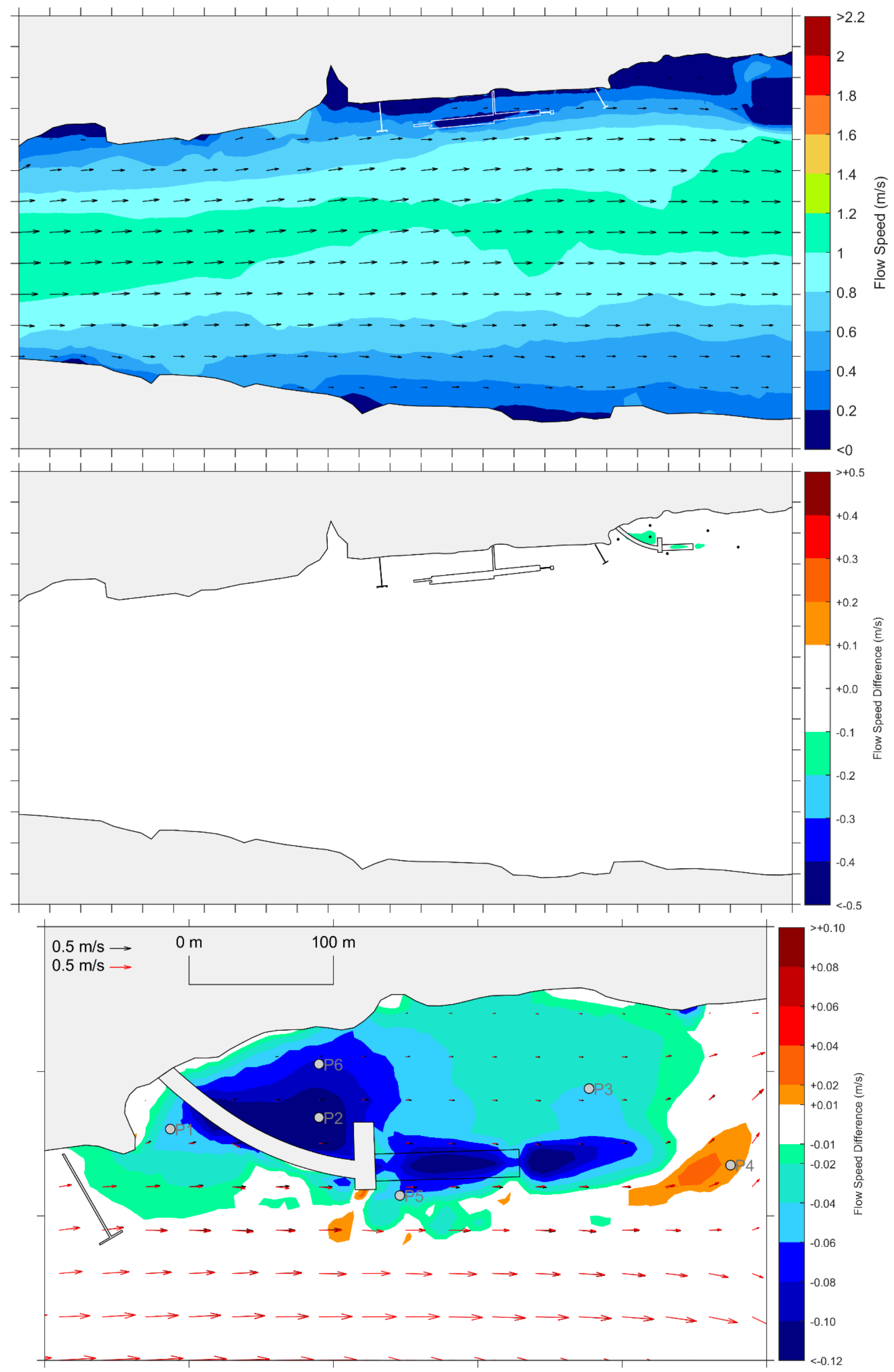

Figure 3.9: Baseline flow speed during ebb tide (top). Effect of AIL Facility with RoRo vessel on baseline flows (middle and bottom).

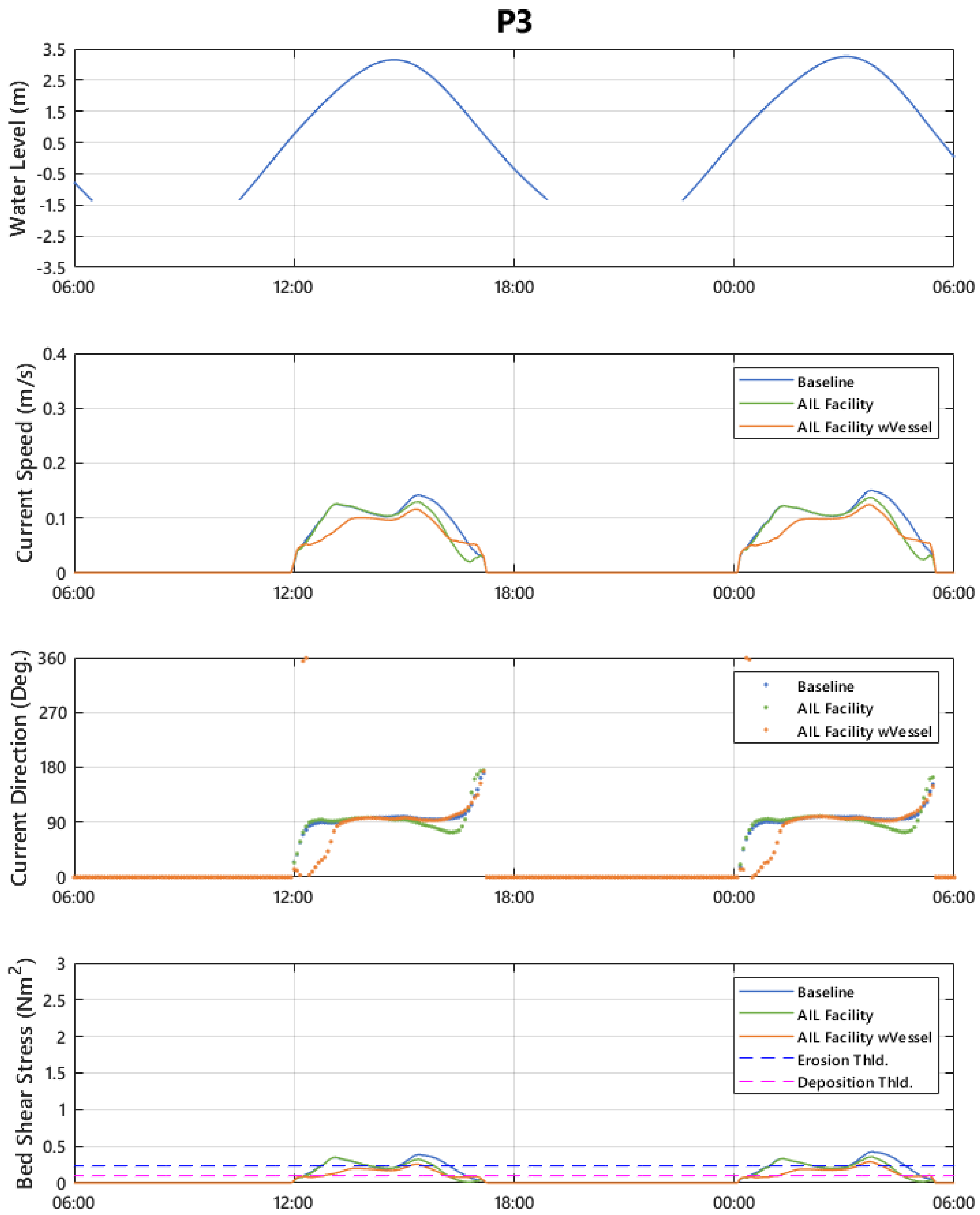

Figure 3.10: Comparison of current speed, direction and BSS as a result of the AIL Facility and AIL Facility with RoRo vessel compared to the baseline at P3.

3.4 Cumulative Assessment

3.4.1 The AIL Facility will be constructed in relatively close proximity to the Tilbury2 development up river (under construction) and the existing offloading facility for Tideway at Goshems Farm down river. There is therefore a requirement to determine whether the effects of the AIL Facility will interact with these developments. For this cumulative assessment, a combined worst case scenario has been modelled. This incorporates the Tilbury2 development in operation with 3 vessels at the terminal, along with the RoRo vessel (barge) at the AIL Facility.

3.4.2 The modelling undertaken for Tilbury2 (HR Wallingford, 2017), showed that the operation of this facility would create changes to the hydrodynamic environment above 0.1 m.s^{-1} which extended in front of the AIL Facility at the time of peak ebb flows (Figure 3.11).

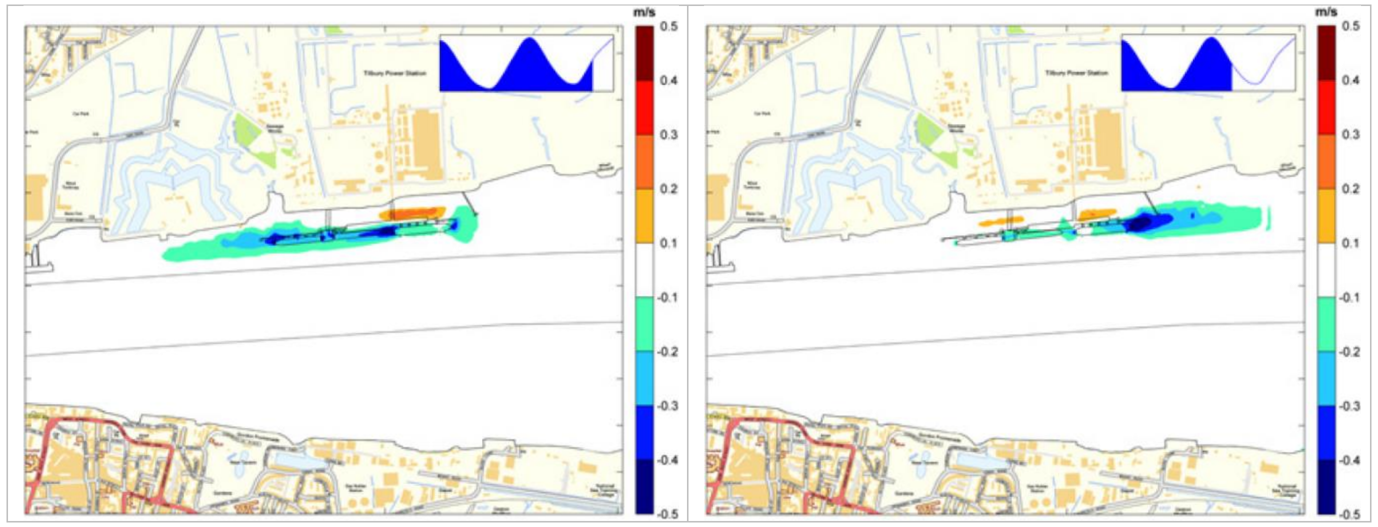

Figure 3.11: Effect of Tilbury2 scheme with 3 vessels on flow speeds during peak flood (left) and peak ebb (right). HR Wallingford, 2017.

3.4.3 The middle plots on Figure 3.1 and Figure 3.2 shows that the AIL Facility will not affect the baseline flows to interact with Tilbury2 of the Goshem Farm jetty at the flow speed resolution used for the Tilbury2 ES. The bottom plots also show this is still the case at a higher resolution.

3.4.4 The combined (cumulative assessment) scenario for this ES, showing the difference from baseline flows, is shown in Figure 3.12 and Figure 3.13 for the flood and ebb tides respectively. These plots confirm the above inference made from the modelling of the individual developments, that used two different models and methods of representing the developments at the lower presentation resolution.

3.4.5 At the higher resolution comparison of Figure 3.12 and Figure 3.13 with Figure 3.1 and Figure 3.2 respectively shows that the fully operational Tilbury2 jetty will cause a small interactive effect with the AIL Facility. This is shown as a reduction in flood and ebb flows (up to 0.05 $m.s^{-1}$, <10%) passing in front of the AIL berth. This can be seen in the timeseries plot for location P5 (Figure 3.14), where both flows and BSS are reduced. However, the small magnitude of change will not affect the sedimentary patterns in this area. Additionally, on the flood, the 'splitting' of the flow around Tilbury2, 'pulls' more flow inshore, accelerating flows by up to 0.04 $m.s^{-1}$ outside the causeway (Figure 3.12).

3.4.6 There are no cumulative effects different to that of the AIL Facility with vessel alone indicated by the plan plots. This is confirmed by the timeseries locations in the area where the changes are almost identical (less than 0.01 $m.s^{-1}$) to those of the AIL Facility. Consequently, these profiles have not been individually plotted.

3.4.7 These results show that the combined effects are negligible in magnitude and are local to the vicinity of the AIL Facility on the flood tide.

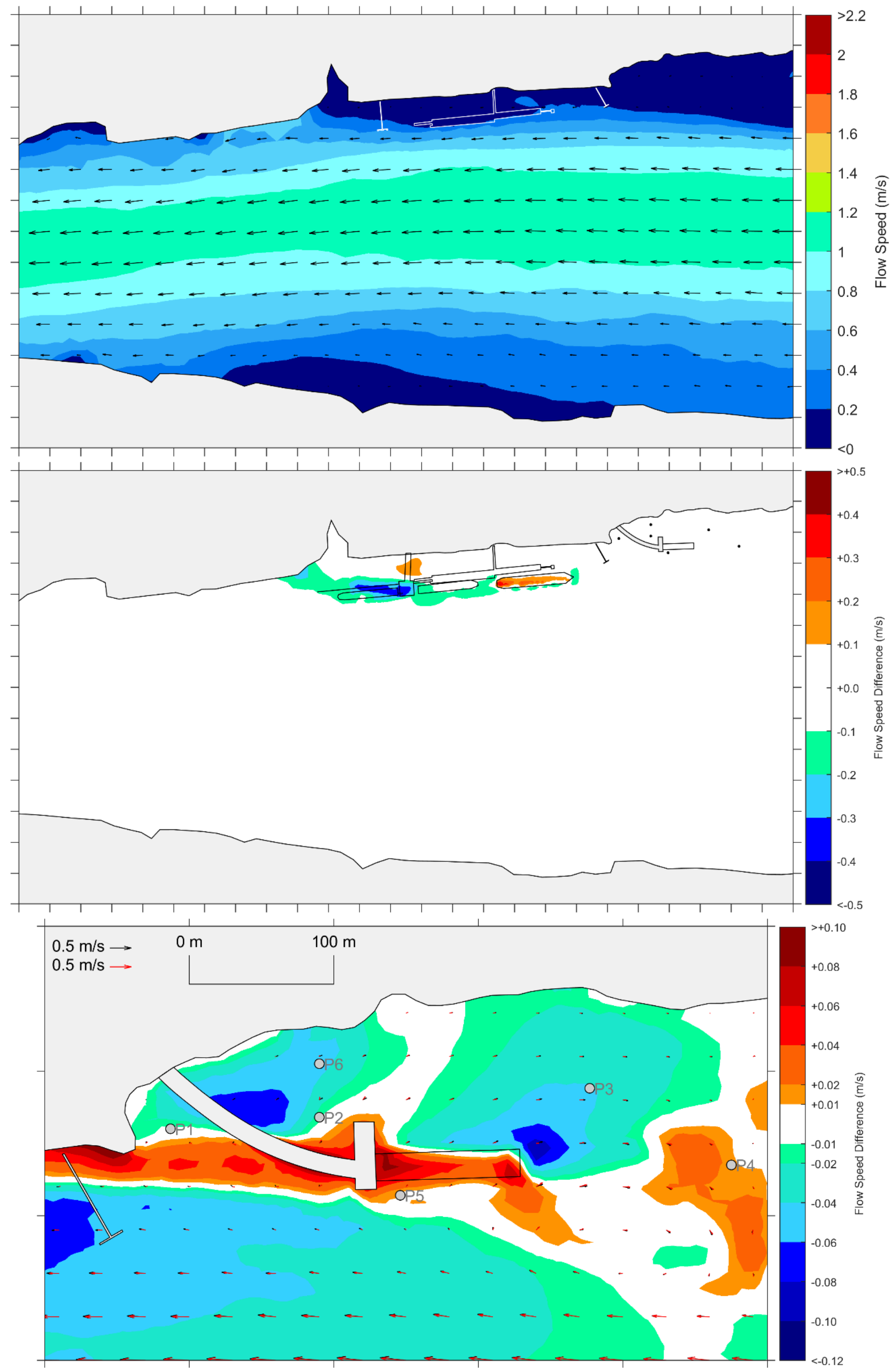

Figure 3.12: Baseline flows speeds during flood tide (top). Effect of Cumulative scenario on baseline flows (middle and bottom).

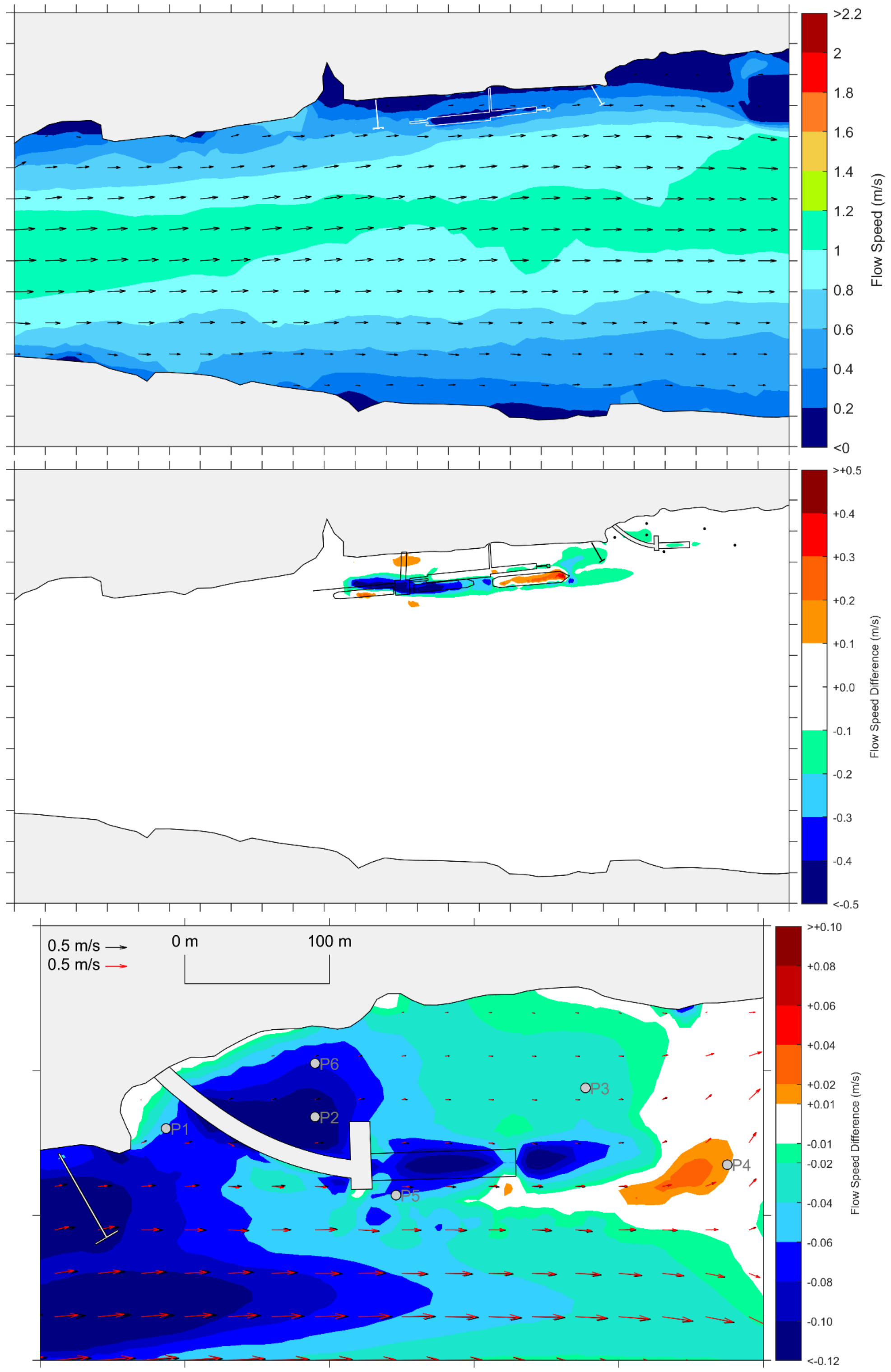

Figure 3.13: Baseline flows speeds during ebb tide (top). Cumulative scenario on baseline flows (middle and bottom).

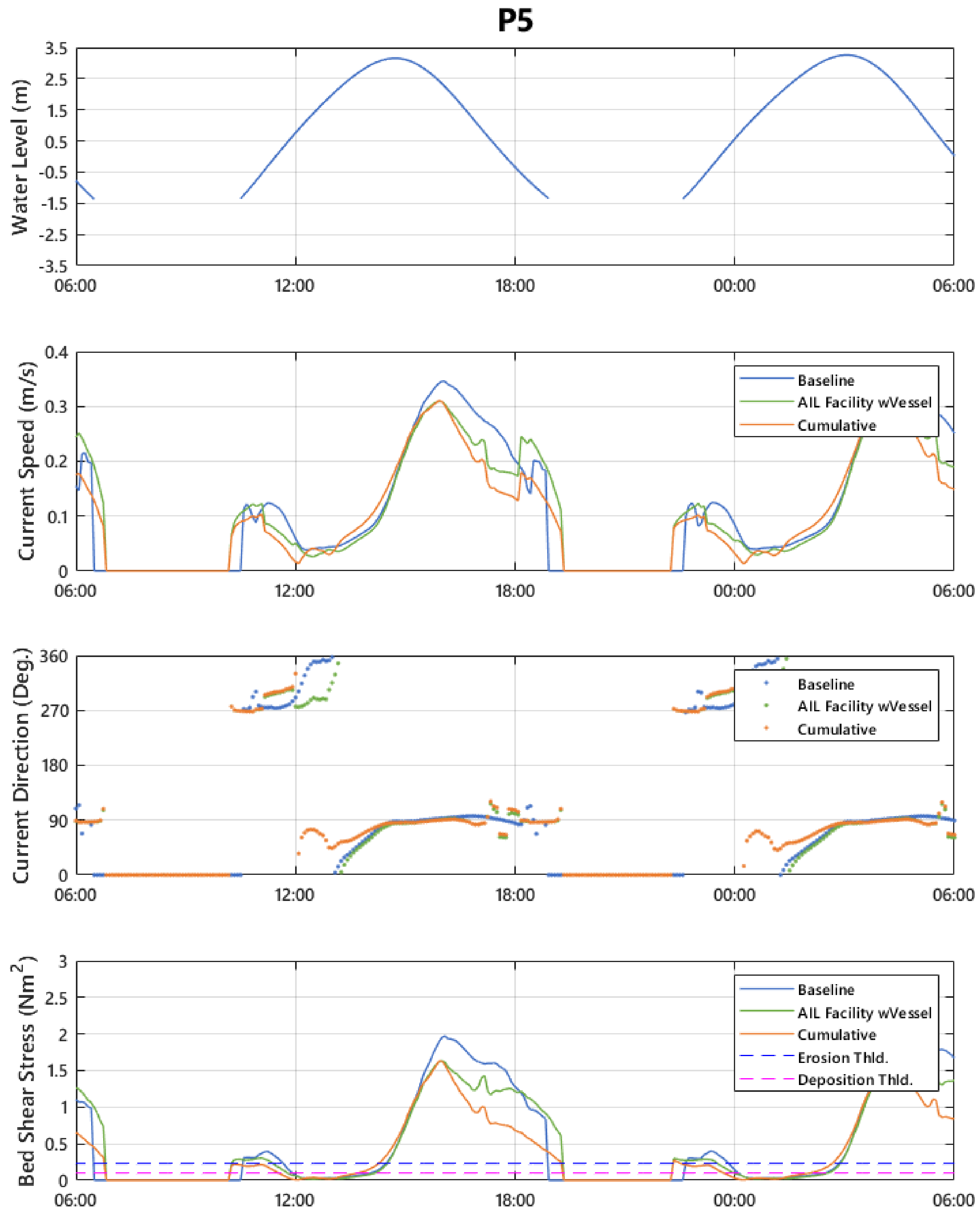

Figure 3.14: Comparison of current speed, direction and BSS as a result of the AIL Facility with RoRo vessel, and Cumulative scenario compared to the baseline at P5.

4. Sediment Transport Assessment

4.1 Introduction

4.1.1 The construction of the AIL Facility will be entirely within the intertidal mudflat. The dredged pocket (volume 13,000 m³) will cause a small direct local increase in the tidal prism of the River Thames. This, however, will be partially compensated by the rock volume in the causeway.

4.1.2 Numerical modelling of the AIL Facility on the hydrodynamic regime of the River Thames has indicated there will be no/negligible change to the river flows. As a result, there is no change likely to occur to the sediment regime in this area. The main effects are reductions in flow speeds that reduce the BSS occurring over the intertidal mudflat areas 'enclosed' by the causeway and behind the dredged berth, and vessel when moored.

4.1.3 As noted in the previous section, the BSS (presented in the bottom panel of the timeseries plots) in these areas of lower flows will be reduced, such that any sediment entering the area in the tidal prism, as a worst case would all settle to the bed, except under wave events. In the baseline case, there is potential for erosion of this material thus creating the existing relatively stable or slowly eroding mudflat.

4.2 Discussion

4.2.1 The modelled changes to the flow regime indicate that sedimentation will occur behind the causeway and berth until a new equilibrium is established. This would further compensate the change in tidal prism and may have the potential to cause a small net reduction in the tidal prism over time. Any measurable changes to the river bed will be confined to the immediate development area, however these changes will be negligible in the context of the River Thames as a whole.

4.2.2 Suspended sediment concentration (SSC) data is not available for the intertidal area. Measurements of the fine sediment SSC from 2002 are available near the "Power Station Jetty" in HR Wallingford, 2017 (Tilbury2 jetty). The diagram from this report is repeated as Figure 4.1. The absolute location, however is not shown but is most likely to be from the subtidal channel.

4.2.3 The figure indicates that SSC around the AIL Facility could vary between 100 mg/l and 2,000 mg/l. A more detailed analysis for the period when the mudflat is tidally inundated suggests that at the outer edge of the mudflat and berth location, when depths are shallow, the SSC could be between 1,500 mg/l and 2,000 mg/l for a short period of time. As the tide rises to HW and then falls on the ebb, concentrations over the mudflat are likely to average in the order of 250 mg/l to 500 mg/l (HR Wallingford, 2002).

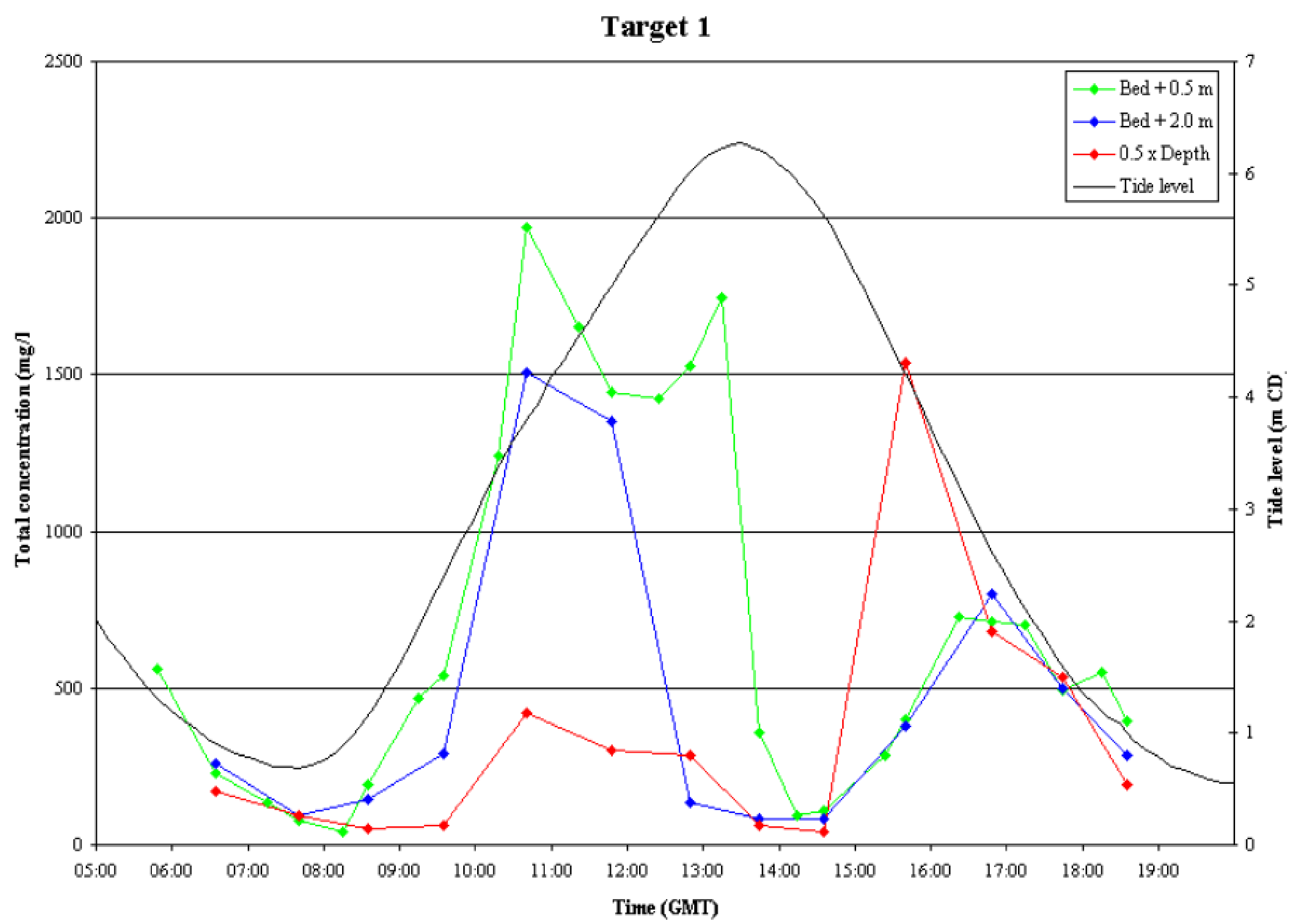

Figure 4.1: Suspended fine sediment concentration observed near the Power Station Jetty in July 2002 (HR Wallingford, 2002).

4.3 Results

4.3.1 The existing mudflat level behind the AIL Facility is around +0.8 m ODN, which gives a depth of water of approximately 2 m to Mean High Water (MHW). Assuming all sediment deposits and the bulk (wet) density of the settled sediment varies between 1,300 kg/m³ and 1,500 kg/m³ (wet), then the initial sedimentation rate would vary between about 1.6 m/yr and 0.5 m/yr respectively. At such rates, however, the depth of water over the mudflat would quickly reduce; the BSS would marginally increase and because erosion is unlikely, self-weight consolidation would occur. All these processes act to reduce the overall sedimentation rate with time.

4.3.2 Accounting for these changes at 3-month intervals, shown on Figure 4.2, indicates that the actual depth of accumulation over a year from the completion of the AIL Facility would be in the range 0.4 – 1.2 m, with sedimentation rates reduced to between 0.4 – 0.8 m/yr. The graph indicates that the new equilibrium will take a minimum of at least three years without sediment consolidation and longer as consolidation occurs.

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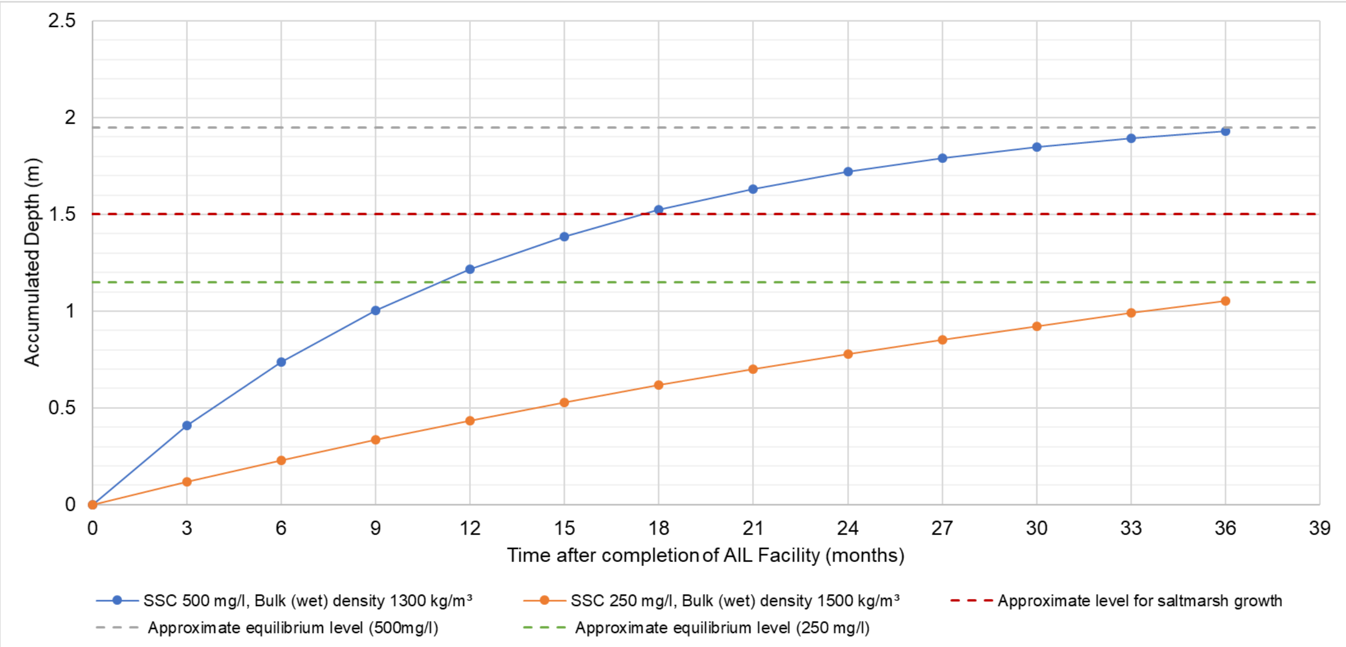

Figure 4.2: Accumulated depth over a 3-year period for SSC of 250 mg/l, bulk (wet) density of 1500 kg/m³; and SSC of 500 mg/l, bulk (wet) density of 1300 kg/m³.

4.3.3 Pioneer saltmarsh is generally considered to start to grow at elevations around Mean High Water Neaps (MHWN), i.e. about +2.3 m ODN (circa 1.5 m above existing bed level). The reduction in sedimentation rates with time indicates that, at the higher SSC of 500 mg/l (and lower bed density), the sediment accumulation thickness could reach an equilibrium at a higher level than MHWN. The graph indicates saltmarsh will start to grow around 18 months following completion of the AIL Facility. Under this scenario sedimentation would continue as would the development of the salt marsh, particularly behind the AIL Facility.

4.3.4 At the lower SSC of 250 mg/l (with the higher realistic bed density) however, sedimentation rates indicate that the sediment accumulation thickness will not have reached an equilibrium in the first three years. Also, the projected equilibrium level may not occur at a level where Pioneer saltmarsh could grow. Taking account of the likely consolidation and variability in the existing profile this suggests that any saltmarsh development is likely to be restricted to behind the AIL Facility and the rest of the area of flow speed change will remain mudflat at elevations 1 – 1 m above the existing bed level.

4.3.5 Sedimentation is likely to occur over the area of the berth pocket, particularly over the inshore half. Berth clearance may be required if there are long periods, circa 3 months plus, between vessels. Figure 4.2 indicates the annual accumulated depth of sedimentation is likely to be in the range 0.4 – 1.2 m/yr towards the rear of the berth. Based on the berth area to be maintained of about 5,000 m², the worst-case maintenance dredge commitment is estimated to be 2,000 – 6,000 m³/yr.

5. Plume Dispersion Assessment

5.1 Introduction

5.1.1 To construct the causeway and dredge the berth pocket a total of 16,100 m³ of *in situ* material is likely to be dredged. Just under 13,000 m³ of this will be from the berth pocket. For the purposes of the assessment it is assumed a (worst case) method of Water Injection Dredging (WID). The remainder will be dug out 'in the dry' by land - based plant which will not-contribute to dredge dispersion.

5.1.2 A comprehensive modelling assessment for the WID method was undertaken for the nearby Tilbury2 development (HR Wallingford, 2017) for a significantly larger dredge volume. The results of the assessment have been used as an analogue for the AIL Facility.

5.2 Discussion

5.2.1 The dredging for the AIL Facility will induce a near bed sediment plume of fine mud sediment, that will move down the channel side slope under gravity into the same, faster flowing streamlines that determined the plumes from the Tilbury2 development. As the modelled WID dredged material contained a proportion of mud similar to that to be dredged for the AIL Facility, the plume extents from the modelling are likely to be very similar to those modelled for Tilbury2. These extents for dredging plume on both the flood and ebb tides are shown in Figure 5.1 (reproduced from HR Wallingford, 2017). This plot shows the maximum depth average SSC modelled within the plume extent at any time in a 14 day spring neap cycle run with continuous dredging. The plot indicates that sediment from the AIL Facility dredge could move in the water column (predominantly near bed) *circa* 20 km up and down estuary of the dredge location.

5.2.2 The model scenario represented a total volume of dredging of the order of five times that which will be dredged for the AIL Facility. Also, due to the shallow water location for the AIL Facility, the dredging will be undertaken by a smaller dredger (slower rate) and for a considerably restricted period of operation over high water. Consequently, the sediment discharge rate will be significantly lower than that used for the Tilbury2 modelling.

5.2.3 From understanding of dredge projects elsewhere with similar tidal restrictions, a realistic daily rate of removal for the AIL Facility is estimated at about 750 m³/day compared to the 4,000 - 4,500 m³/day used in the model simulation. The dredge will therefore take *circa* 17 days to complete, which is similar to the overall time considered in the modelling.

Figure 5.1: Maximum increase in depth averaged suspended sediment concentration during 14 days of WID - Scenario 1 (HR Wallingford, 2017).

5.2.4 The lower rate of dredging will reduce the maximum concentration in the plume to below *circa* 100 mg/l at any location, except immediately adjacent to the dredge. The distribution of the SSC in the plume will remain similar to that shown in Figure 5.1. The maximum values will only occur for short periods controlled by the changing tidal flow distribution. Further modelling outputs from HR Wallingford (2017) indicate that the SSC, when averaged through the spring/neap cycle, are only to exceed 10 mg/l above background within about 1 km up and down river of the dredge.

5.3 Results

5.3.1 The AIL Facility dredge, if undertaken on both the flood and ebb tides could produce a sediment plume extending 20 km up and down river, and across the full width of the river. The dredge is-estimated to take of the order of 17 days. Average SSC increases are unlikely to exceed 10 mg/l above background levels beyond about 1 km up and down river of the dredged berth, although short period (<1-hour duration) episodic maxima of around 100 mg/l could occur over a wider area.

5.3.2 The sediment dispersed is likely to be fine grained and will move predominantly within the area of faster river flows. Most is therefore likely to be incorporated into the background suspended sediment within the river.

5.3.3 During the dredge there will be a local increase in SSC through the berths at Tilbury2, however any increased sedimentation will be very small and only last for the period of the dredge. Should this be of concern, it can be mitigated by restricting the dredge to the ebb flows, which would increase the dredge lapse time to around 1 month. Any permanent accretion arising from the dredge over the wider area is likely to occur on the lower intertidal, within the extent of the plume, however depths of accumulation will be low (of the order of 1 mm) and therefore unmeasurable in the river.

5.3.4 Overall, these changes will be considerably less than those indicated for the recently consented Tilbury2 development.

6. References

HR Wallingford (2002) Tilbury Power Station jetty improvement works, Hydrodynamic and sediment issues. Report EX4626.

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