Annex B2.1: Physical Environment Assessment Report

Standard Gate two submission for London Water Recycling SRO

Notice – Position Statement

This document has been produced as the part of the process set out by RAPID for the development of the Strategic Resource Options (SROs). This is a regulatory gated process allowing there to be control and appropriate scrutiny on the activities that are undertaken by the water companies to investigate and develop efficient solutions on behalf of customers to meet future drought resilience challenges.

This report forms part of suite of documents that make up the 'Gate 2 submission.' That submission details all the work undertaken by Thames Water in the ongoing development of the proposed SRO. The intention at this stage is to provide RAPID with an update on the concept design, feasibility, cost estimates and programme for the schemes, allowing decisions to be made on their progress.

Should a scheme be selected and confirmed in the Thames Water final Water Resources Management Plan (WRMP), in most cases it would need to enter a separate process to gain permission to build and run the final solution. That could be through either the Town and Country Planning Act 1990 or the Planning Act 2008 development consent order process. Both options require the designs to be fully appraised and, in most cases, an environmental statement to be produced. Where required that statement sets out the likely environmental impacts and what mitigation is required.

Community and stakeholder engagement is crucial to the development of the SROs. Some high-level activity has been undertaken to date. Much more detailed community engagement and formal consultation is required on all the schemes at the appropriate point. Before applying for permission Thames Water will need to demonstrate that they have presented information about the proposals to the community, gathered feedback and considered the views of stakeholders. We will have regard to that feedback and, where possible, make changes to the designs as a result.

The SROs are at a very early stage of development, despite some options having been considered for several years. The details set out in the Gate 2 documents are still at a formative stage.

Disclaimer

This document has been written in line with the requirements of the RAPID Gate 2 Guidance and to comply with the regulatory process pursuant to Thames Water's statutory duties. The information presented relates to material or data which is still in the course of completion. Should the solutions presented in this document be taken forward, Thames Water will be subject to the statutory duties pursuant to the necessary consenting process, including environmental assessment and consultation as required. This document should be read with those duties in mind.

LONDON EFFLUENT REUSE SRO

Annex B.2.1.

Aquatic Physical Environment Assessment Report

Report for: Thames Water Utilities Ltd

Ref. **4700399659**

Ricardo ref. ED13591 | Issue: 1.2 | 12/10/2022

Version Control

Version 1.0 – 27/07/2022		First Draft
Version 1.1 – 26/09/2022		Updates to include Teddington DRA 100/150 and estuarine modelling Incorporation of L2 Assurance Comments Incorporation of NAU Comments.
Version 1.2 – 12/10/2022		Second round of L2 Assurance Comments

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Thames Water Utilities Ltd

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1. INTRODUCTION

This report is part of series of Environmental Assessment Reports which catalogue the set of environmental assessments of the London Effluent Reuse Strategic Resource Option (SRO) through RAPID Gate 2: *Detailed feasibility, concept design and multi-solution decision making* and onward to RAPID Gate 3: *Developed design, finalised feasibility, pre-planning investigations and planning applications*. The reports set out the environmental assessments, which will in turn support regulatory assessment requirements proportionate to RAPID Gate 2 and onward to RAPID Gate 3. The scope and approach to the environmental evidence provided in these reports was set out in the Gate 2 Scoping Report and consulted on with the National Appraisal Unit (NAU) in November 2021.

This document has been produced as the part of the process set out by RAPID for the development of the Strategic Resource Options (SROs). This is a regulatory gated process allowing there to be control and appropriate scrutiny on the activities that are undertaken by the water companies to investigate and develop efficient solutions on behalf of customers to meet future drought resilience challenges.

This report forms part of suite of documents that make up the 'Gate 2 submission.' That submission details all the work undertaken by Thames Water (TWUL) in the ongoing development of the proposed SRO. The intention at this stage is to provide RAPID with an update on the concept design, feasibility, cost estimates and programme for the schemes, allowing decisions to be made on their progress.

Should a scheme be selected and confirmed in the TWUL final Water Resources Management Plan (WRMP), in most cases it would need to enter a separate process to gain permission to build and run the final solution. That could be through either the Town and Country Planning Act 1990 or the Planning Act 2008 development consent order process. Both options require the designs to be fully appraised and, in most cases, an environmental statement to be produced. Where required that statement sets out the likely environmental impacts and what mitigation is required.

Community and stakeholder engagement is crucial to the development of the SROs. Some high-level activity has been undertaken to date. Much more detailed community engagement and formal consultation is required on all the schemes at the appropriate point. Before applying for permission TWUL will need to demonstrate that they have presented information about the proposals to the community, gathered feedback and considered the views of stakeholders. We will have regard to that feedback and, where possible, make changes to the designs as a result.

The SROs are at a very early stage of development, despite some options having been considered for several years. The details set out in the Gate 2 documents are still at a formative stage.

1.1. LONDON EFFLUENT REUSE STRATEGIC RESOURCE OPTIONS

For Gate 2, the London Effluent Reuse SRO is set out as four source options and a range of sizes. One option is in east London, utilising final effluent from Beckton STW. The other three options are in west London, utilising crude sewage or final effluent from Mogden sewage treatment works (STW) to a maximum total reduction of 200 Ml/d, with differing London Effluent Reuse scheme discharge locations in the freshwater River Thames.

Full details of the conceptual design of the four schemes are provided in the Conceptual Design Reports[1] (CDR). For assessment purposes no specific mitigation is allowed for unless included as part of option design as set out in CDR (other than the Habitats Regulations Assessment (HRA) Stage 2 and Initial Environmental Appraisal (IEA)) which has regard for additional mitigation as per the All Company Working Group (ACWG) methodology). A DRA intake would include appropriate fish screening and all new outfalls would include appropriate eel management measures.

High level summaries of each option are provided below.

1.1.1. Beckton water recycling scheme

Final effluent from Beckton STW would be treated at a new advance water recycling plant (AWRP) within Beckton STW for advanced treatment. Recycled water would be conveyed via a new tunnel from the Beckton

[1] Jacobs (2022) London Reuse Strategic Resource Option, Gate 2 Conceptual Design Reports.

AWRP to Lockwood Pumping Station and then a Thames-Lee Tunnel (TLT) extension from Lockwood Pumping Station to a proposed new outfall located on a side channel of the freshwater Lee Diversion, known as the Enfield Island Loop, upstream of the existing Thames Water Enfield intake to the King George V Reservoir. Additional abstraction for public water supply on a put/take basis would be through existing intakes in the lower Lee, to supplement the raw water supply to the Lee Valley reservoirs. The option reduces the final effluent at the extant Beckton STW outfall to the estuarine Thames Tideway.

The Beckton water recycling scheme has been assessed for Gate 2 independently at 100 Ml/d, 200 Ml/d, and 300 Ml/d.

1.1.2. Mogden water recycling scheme

Final effluent from Mogden STW would be pumped in a new pipeline to a new water recycling plant located at a site near Kempton water treatment works (WTW)) for advanced treatment via a new AWRP. Recycled water would be transferred in a new pipeline for discharge into the freshwater River Thames at a new outfall upstream of the existing Thames Water Walton intake. Additional abstraction for public water supply on a put-take basis would be through existing downstream intakes on the River Thames. AWRP wastewater and reverse osmosis (RO) concentrate would be conveyed back to Mogden STW inlet works via a return pipeline(s). There is an option that the AWRP wastewater could be discharged to the South Sewer for return to Mogden STW, but it is not possible to return the RO concentrate by this means. The scheme reduces the final effluent at the extant Mogden STW outfall to the estuarine Thames Tideway.

The Mogden water recycling scheme has been assessed for Gate 2 independently at 50 Ml/d, 100 Ml/d, 150 Ml/d and 200 Ml/d.

1.1.3. Mogden South Sewer scheme

Crude sewage would be diverted from the South Sewer of the sewerage catchment of Mogden STW. The South Sewer runs close to Kempton Park WTW and the diverted sewage would be pumped to a new AWRP located at a site near Kempton WTW for advanced treatment. Recycled water would be transferred in a new pipeline for discharge into the freshwater River Thames at an outfall upstream of the existing Thames Water Walton intake. Additional abstraction for public water supply on a put-take basis would be through existing downstream intakes on the River Thames. Waste streams from the AWRP would be conveyed by a new pipeline and treated at Mogden STW. The scheme reduces the final effluent at the extant Mogden STW outfall to the estuarine Thames Tideway.

The Mogden South Sewer scheme has been assessed for Gate 2 at 50 Ml/d.

During Gate 2, Thames Water took the decision to pause development of the Mogden South Sewer scheme due to limitations on available flow within the sewer, cost of the scheme and regional modelling not selecting the scheme under any water resources planning horizon scenario. The Gate 1 concept design is therefore used in Gate 2, with the exception where scheme elements are shared with the Mogden water recycling scheme (certain conveyance routes, AWRP and discharge location) which have been further developed through Gate 2.

The Mogden South Sewer scheme has not been progressed through Gate 2 environmental assessments, and so a dedicated assessment section is not included within this report. However, due to the similarities with the 50 Ml/d Mogden water recycling scheme (AWRP, discharge location and volume), the outcomes of that assessment can be considered representative of a physical environment assessment of a 50 Ml/d Mogden South Sewer scheme.

1.1.4. Teddington DRA scheme

Final effluent from Mogden STW would be subject to further treatment at a new tertiary treatment plant (TTP) at Mogden STW. The treated water would be transferred in a new pipe-jacked tunnel for discharge into the freshwater River Thames at a new outfall upstream of the tidal limit at Teddington Weir. Additional abstraction for public water supply on a take-put basis would be through a new intake from the freshwater River Thames, upstream of the new outfall. Abstracted water would be pumped into the nearby TLT for transfer to Lockwood pumping station, part of Thames Water's Lee Valley reservoirs in North London. The option reduces the final effluent at the extant Mogden STW outfall to the estuarine Thames Tideway.

The Teddington DRA scheme has been assessed for Gate 2 independently at 50 Ml/d, 75 Ml/d, 100 Ml/d and 150 Ml/d.

1.2. LONDON EFFLUENT REUSE SRO OPERATING PATTERN

To support the environmental assessments at Gate 2, an indicative operating pattern has been developed. The approach uses the 19,200-year stochastic flow series developed for the River Thames catchment for the Water Resources South East group (WRSE). The stochastic flow series represent contemporary climate conditions and provide information on the return frequency, or regularity, of both the likely river flow conditions and London Effluent Reuse SRO operation. The stochastic years have been made available as 48-year continuous periods. CEH flow bands were derived for each individual day from the full modelled 19,200 years and from these each year was assigned a return frequency. All 400 of the 48-year periods were reviewed for the pattern of return frequencies within those 48-years, and one of those has been selected as having representative flow characteristics to inform the environmental assessments. The selected 48-year series[2] includes a suitable range of regular low and moderate low flow periods. It does not include extreme low flows that are considered to be less regular than once every fifty years. It should be noted that this operating pattern is for the London Effluent Reuse SRO solution used on its own for Thames Water, without conjunctive use with other Thames Water SROs (such as the South East Strategic Reservoir Option (SESRO)). It also uses the controlling triggers developed by Thames Water for current strategic resource options (such as Thames Gateway Water Treatment Plant) based on lower River Thames flows and Thames Water's total London reservoir storage. The indicative pattern is shown in Figure 1-1, noting that outside the normal operating pattern the Gate 2 engineering design includes a plant and conveyance maintenance flow at all times, with the recycled/treated water being discharged at the reuse outfall but not re-abstracted. The rate of the maintenance flow discharge variers with London Effluent Reuse scheme.

Within these patterns, selected return frequencies have been selected for the detailed assessment including modelling used extensively in the assessments presented for Gate 2. These are a 1:5 return frequency year with moderate-low flows in the River Thames at Teddington with a 1:5 return frequency operating pattern in terms of duration and season (model reference A82). Also a 1:20 return frequency year with very low flow years in the River Thames at Teddington with a 1:20 return frequency operating pattern in terms of duration and season (model reference M96). Noting the scheme would only be used on a 1:2 return frequency, these capture a suitable range of circumstances and have been discussed and reviewed with the regulators during Gate 2. In addition, a 1:50 return frequency year of extremely low flows in the River Thames at Teddington and with a 1:50 return frequency operating pattern in terms of duration and season (model reference N17), has been prepared and reviewed for consideration of scheme resilience. Such a low return frequency is outside the regularity of occurrence included in WFD assessments and is not described further in this report. It is noted that the three model years A82, M96 and N17 were selected from the full 19,200 year dataset from comparison with other years of the same return frequency and have been swapped in to the selected 48-year series to improve confidence in the series selected for environmental assessment.

As shown on Figure 1-2, expected London Effluent Reuse SRO usage would typically be in the months August to November, peaking at 37% of days in September. Outside this period, there would be less regular usage in July and December, with usage very rare in June and January and not anticipated in February, March, April or May.

Specifically for the assessments in this report, the modelled A82 1:5 year return frequency moderate-low flow year includes a period of operation of 99 consecutive days between 6 August and 12 November. The modelled M96 1:20 year return frequency very low flow year includes a period of operation of 161 consecutive days between 11 July and 18 December and following a brief period of higher river flows increasing total London reservoir storage, an additional period of 5 days between 7 January and 11 January.

[2] Note these are 48 calendar years. The environmental assessment period has been selected as a water resources year (1 April to 31 March) and as such the selected period includes 47 water resources years from the 48 calendar years,

Figure 1-1 Representation of the operational pattern of London Effluent Reuse SRO schemes as used in the Gate 2 environmental assessments

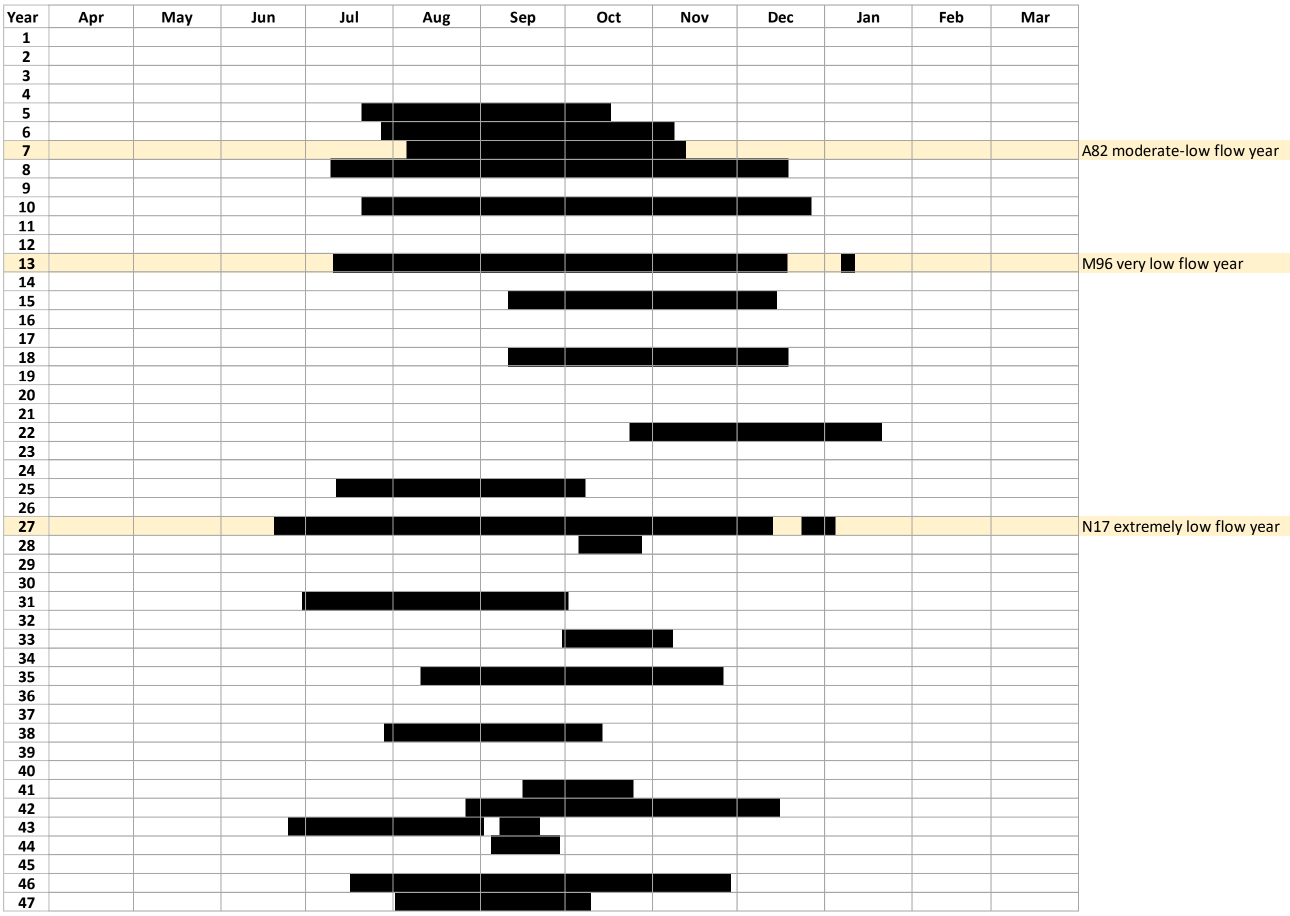
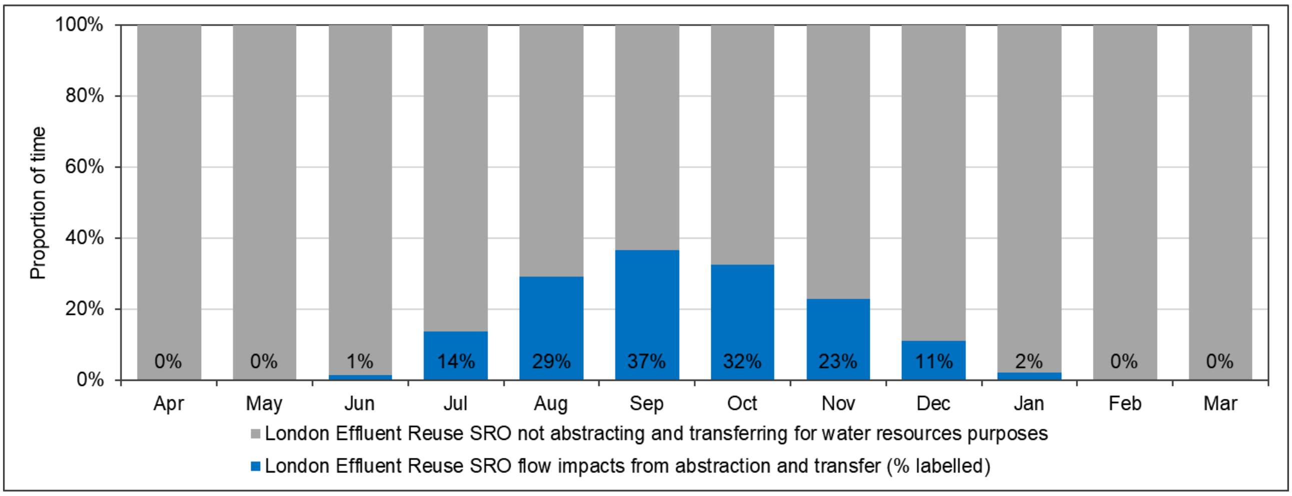


Figure 1-2 Representation of the per calendar month operational pattern of London Effluent Reuse SRO schemes as used in the Gate 2 environmental assessments



1.3. THE PURPOSE OF THIS REPORT

The purpose of this series of Assessment Reports (Annex B.2.) is to set out the environmental baseline for each reach of the full study area to identify the source of greatest potential magnitude of change that a London Effluent Reuse SRO might cause within that reach, and then assess the potential for change to environmental pathways (physical environment and water quality) and receptors (aquatic ecology). The report identifies where additional data and/or more detailed analysis is required in Gate 3 as the London Effluent Reuse SRO designs are developed and operating regimes refined. The findings of these reports provide the evidence base to inform the HRA, Water Framework Directive (WFD) and IEA assessments.

This report provides the assessment for the Gate 2 Physical Environment topic. As per the Gate 2 Physical Environment Evidence Report, Table 1-1 outlines the task and approach to assessment for the physical environment assessment for Gate 2 of the London Reuse SRO. It also outlines the evidence base that has been used to undertake the assessment for each of the tasks.

The study area for the London Effluent Reuse SRO has been divided into the following water courses:

- The freshwater River Thames from Shepperton Weir to the tidal limit at Teddington, noting the 1D river model boundary is Cricklade in the upper catchment of the River Thames
- Channels of the freshwater Lee from Newman's Weir on the Enfield Island Loop to the tidal limit at Three Mills Lock
- The estuarine Thames Tideway from the tidal limit at Teddington, including the Richmond Pound, to 3km seawards of Beckton STW outfall, noting the estuarine model boundary is at Southend-on-Sea.
- The estuarine Bow Creek (tidal Lee) from Three Mills Lock to the Thames Tideway.

The findings of the report are used to support the WFD assessments (Annex B.4.), HRAs (Annex B.3.), Natural Capital (NC) assessments (Annex B.6.) and IEA (Annex B.5.) of the London Effluent Reuse SRO.

Section 2 of this report provides a high-level overview of the reference conditions for the zone of influence of the London Effluent Reuse SRO sub-options. The full reference conditions are presented in Appendix 1. Sections 3, 4 and 5 provide the Physical Environment assessment for each SRO sub-option included in the Gate 2 submission. Section 6 provides a summary of the additional data and assessment requirement required during Gate 3.

Table 1-1 Tasks and assessment approach to the aquatic physical environment assessment for London Effluent Reuse SRO

Task item	Scope of assessment	Approach to assessment	Evidence Base for Task
a. Flow change	• Assessment of discharge, level and velocity patterns throughout the study area (both river and estuary) for the range of reference conditions and scenarios with Reuse scheme	• Develop and interrogate fluvial flow series at key locations for Gate 2 reference conditions and scenario sets • Use time-series outputs from the 1D hydraulic model of the freshwater River Thames • Use fixed scenario outputs from the 3D hydraulic model of selected reaches the freshwater River Thames • Interrogate TELEMAC model time-series outputs for Thames Tideway.	• Hydraulic models development data • Hydraulic models scenario parameterisation data
b. Review of outfall (and DRA intake) design including screening together with local velocity effects	• Assess effects of operation of outfalls and DRA intake within the river and provide any need to alter the design based on evidence	• Interrogate 3D fluvial modelling outputs to describe significance of changes in flow velocity field plume and changes in depth.	• Hydraulic models development data • Hydraulic models scenario parameterisation data • Outfall design schematics.
c. River mainstem, weir pool and estuarine wetted habitat change	• Assess effects on level, velocity and wetted habitat change including at Sunbury, Molesey and Teddington weir pools	• Interrogate 3D fluvial modelling outputs together with hydromorphological survey data (Lower Thames areas of marginal habitat and the high sensitivity habitats around River Roding/ Creekmouth) to describe significance of changes in flow velocity and wetted area to provide information for change for key species in the ecological assessments.	• Bathymetry and ADCP survey data for hydraulic model development. • Habitat preferences for identified fish species. • Location of specific habitats of importance throughout river and tideway.
d. Fish pass and barrier passability (freshwater River Thames)	• Assess effect on passability of fish passes at weirs (Sunbury, Molesey and Teddington) and weirs in the Enfield Island Loop	• Confirm critical levels for fish pass operation. • Review river level model outputs calculated under varying scenarios and compare these with critical levels for fish pass operation to identify any potential impacts and their magnitude. • Interrogate 3D fluvial modelling outputs to describe significance of changes in flow velocity and wetted area to provide information for change for key species in fisheries assessment.	• River 3D TELEMAC model outputs. • Weir and fish pass schematics. • Bathymetry and ADCP survey data for hydraulic model development.
e. Richmond Pound drawdown Physical Environment assessment	• Asses the specific effects of planned annual maintenance drawdown on the physical habitats within Richmond Pound	• Interrogate estuarine TELEMAC modelling outputs to describe changes in habitat availability during those periods (baseline) and with Reuse scheme.	• Tideway TELEMAC modelling outputs. • Bathymetry and ADCP survey data. • Location of specific habitats of importance within Richmond Pound.
f. Estuarine sediment assessment	• Develop and agree key assessment points to understand any sediment changes in the estuary	• Interrogate modelled sediment dynamics output (estuarine TELEMAC model) to • describe variability in sediment dynamics during reference conditions and with Reuse scheme scenarios.	• Tideway TELEMAC modelling outputs.

2. REFERENCE CONDITIONS

2.1. INTRODUCTION

In order to inform the assessment for each of the tasks set out in Table 1-1, this section establishes the reference conditions for each task as per the relevant study area. The study area for each task has been set out per task as it is not consistent across tasks.

The physical environment assessments of London Effluent Reuse SRO at Gate 2 have been undertaken to assess change from a range of different appropriate reference conditions at times when a London Effluent Reuse SRO could be utilised. These reference conditions are different patterns of river flow and STW final effluent flow (see Section 1.2): a 1:5 return frequency moderate-low flow year; and a 1:20 return frequency very low flow year. Some selected reference conditions are specific to selected flow conditions. These are identified for the specific areas of the study area and physical environment tasks as described in Table 2-1. The comparison of reference conditions with individual London Effluent Reuse SRO options is described in Sections 3-5 below.

Table 2-1 Physical environment reference conditions

Task	Freshwater River Thames	Estuarine Thames Tideway	Freshwater Lee Diversion Channel
General hydrodynamic conditions in the study area	• Modelled 1:5 return frequency moderate low flow year (A82) • Modelled 1:20 return frequency very low flow year (M96)	• Modelled 1:5 return frequency moderate low flow year (A82) • Modelled 1:20 return frequency very low flow year (M96)	• Representative moderate low flow year (1/4/2016-31/3/2017) • Representative very low flow year (1/4/2011-31/3/2012)
Local hydrodynamic conditions around potential SRO in-river structures	• Selected flows at Walton Bridge: 600 Ml/d (Q99.5), 780 Ml/d (Q97), 950 Ml/d (Q91) • Selected flows at Teddington Weir aligning with TTF: 300 Ml/d, 400 Ml/d, 700 Ml/d	-	• ADCP surveys in Enfield Island Loop at 260 Ml/d (Q64) and 430 Ml/d (Q35)
River mainstem, weir pool and estuarine wetted habitat	• Selected flows at Walton Bridge: 600 Ml/d (Q99.5), 780 Ml/d (Q97), 950 Ml/d (Q91) • Selected flows at Teddington Weir aligning with TTF: 300 Ml/d, 400 Ml/d, 700 Ml/d	• Modelled 1:5 return frequency moderate low flow year (A82) • Modelled 1:20 return frequency very low flow year (M96)	• ADCP surveys in Enfield Island Loop at 260 Ml/d (Q64) and 430 Ml/d (Q35)
Fish pass and barrier passability	• Selected flows at Walton Bridge: 600 Ml/d (Q99.5), 780 Ml/d (Q97), 950 Ml/d (Q91)	-	• ADCP surveys in Enfield Island Loop at 260 Ml/d (Q64) and 430 Ml/d (Q35)
Richmond Pound drawdown physical environment	-	• Modelled 1:5 return frequency moderate low flow year (A82) • Modelled 1:20 return frequency very low flow year (M96)	-
Estuarine sediment	-	• Modelled 1:5 return frequency moderate low flow year (A82) • Modelled 1:20 return frequency very low flow year (M96)	-

The data used for establishing the reference conditions has been outlined in the Gate 2 Physical Environment Evidence Report and in Table 1-1. The supporting evidence of physical environment reference conditions is presented in Appendix 1 for each of the physical environment tasks presented in Table 2-1.

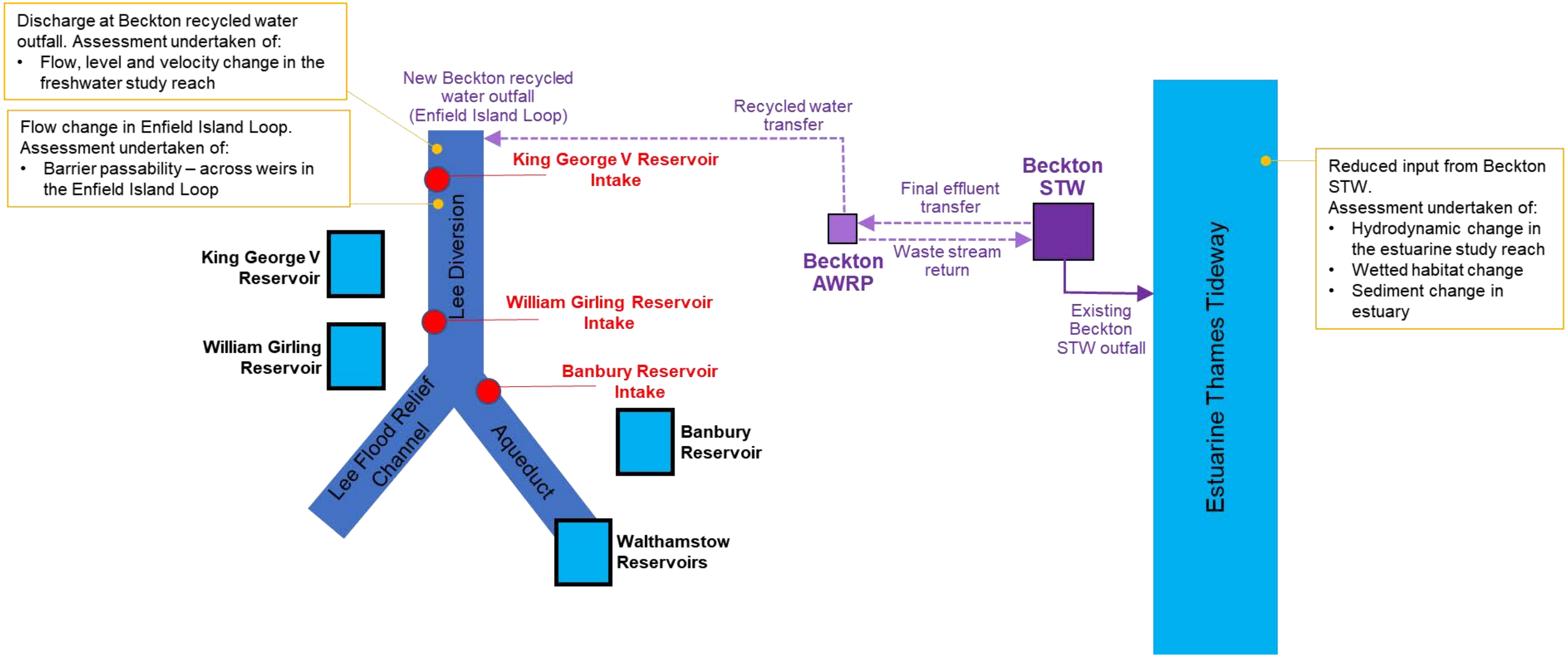
3. PHYSICAL ENVIRONMENT ASSESSMENT OF BECKTON WATER RECYCLING SCHEME

3.1. INTRODUCTION

Specific to the Beckton water recycling scheme, the assessment for each of the tasks set out in Table 1-1 is set out in this section. As set out spatially in the conceptualisation of physical environment effects in Figure 3-1, the specific assessments of the Beckton water recycling schemes are:

- Flow changes from Beckton water recycling scheme.
- Review of Beckton water recycling outfall design including screening
- Wetted habitat change in freshwater channels of the Lee, estuarine Bow Creek and estuarine Thames Tideway
- Enfield Island Loop barrier passability
- Thames Tideway estuarine sediment assessment
- Summary of physical environment assessment of Beckton water recycling scheme.

Figure 3-1 Representation of the Beckton water recycling aquatic study area with conceptualisation of physical environment effects and listing of assessment undertaken for Gate 2



To support the environmental assessments at Gate 2, an indicative operating pattern has been developed, as described in Section 1.2. Outside the normal operating pattern the Gate 2 engineering design includes a 15 Ml/d tunnel maintenance flow, with the recycled water being discharged to the Enfield Island Loop of the Lee Diversion Channel. A tunnel maintenance flow would not be discharged to the Enfield Island Loop at times of flood risk in the Lower Lee.

3.2. FLOW CHANGES FROM BECKTON WATER RECYCLING SCHEMES

3.2.1. Overview

A Beckton water recycling scheme would increase flows in the Enfield Island Loop of the Lee Diversion Channel upstream of Thames Water's Enfield intake by 100-300 Ml/d (dependent on option assessed) when in use for water resources purposes, and at 15 Ml/d at other times. When operational for water resources purposes, flow augmented by recycled water by a Beckton water recycling scheme would typically be re-abstracted at Thames Water's Enfield intake to King George V Reservoir or at Thames Water's Chingford South intake to William Girling Reservoir, 3.4 km downstream on the Lee Diversion Channel. There may be some operational circumstances where a Beckton water recycling scheme would also enable increased abstraction rates at Thames Water's Chingford Supply Channel intake to the Lower Lee Reservoir's Walthamstow Reservoir Group.

Final effluent flows from Beckton STW discharged to the estuarine Thames Tideway at Beckton would reduce by the corresponding amount to the amount transferred to the Enfield Island Loop of the Lee Diversion Channel.

3.2.2. Freshwater Channels of the Lee

Selected representative years have been used to show an indicative flow pattern along the Enfield Island Loop of the Lee Diversion Channel in Figure 3-2. Key locations in the Lee Diversion Channel and Enfield Island Loop are shown on Figure 3-3. It is important to note that due to the way these flows have been derived they specifically relate to flow in the Lee Diversion Channel prior to flow splitting at Newman's Sluice. Under low and normal range flows all of these flows are routed over Tumbling Bay Weir and into the Enfield Island Loop. At high and very high flows the peak flow component is routed through Newman's Sluice along the Lee Diversion Channel through Newman's Sluice. Therefore, the peak flows shown on Figure 3-2 are not a feature of the flow regime of the Enfield Island Loop. It is also important to note that these flows are appropriate for assessment of the point of discharge of a Beckton water recycling option and for the ~100m of channel to the intake to King George V Reservoir. Due to significant abstraction at the intake to King George V Reservoir (licensed daily maximum rate 818 Ml/d) flows shown on Figure 3-2 are not representative of reference condition flows in the Enfield Island Loop in the ~400m of channel between the intake to King George V Reservoir and the reconnection with the Lee Diversion Channel or downstream. Further, the partitioning of the re-abstraction regime for augmented flows from a Beckton water recycling scheme would be subject to operational decisions relating to raw water storage management for Coppermills WTW and is not something that can be incorporated into environmental assessment. As such, at one extreme, all of the reference condition flow together with all of the Beckton water recycling scheme augmented flow could be abstracted at the intake to King George V Reservoir with zero flow passed forward; at the other extreme all of the reference condition flow together with all of the Beckton water recycling scheme augmented flow could be conveyed to and along the Chingford Supply Channel aqueduct to the Walthamstow Reservoir group, with zero abstraction at the intake to King George V Reservoir or William Girling Reservoir.

The selected periods include a prolonged period through the summer and autumn months of modelled river flow in the Lee Diversion Channel around 100-200 Ml/d, reducing to c.40 Ml/d. It is noted that the flows shown for the Lee Diversion Channel are constantly changing, without a stable flow regime.

The extent of flow change from a Beckton water recycling scheme is contextualised through use of flow duration statistics for the 12 year derived series for the Lee Diversion Channel and through comparison with Environment Agency information on naturalised flows locally in the middle Lee, for CAMS Assessment Point AP14[3]. It is noted that although the CAMS information relates to the natural flow rates, the geomorphology of

[3] Environment Agency (2020) CAMS: London abstraction licensing strategy: A strategy to manage water resources sustainably. January 2020.
https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/865039/CAMS-London-abstraction-licensing-strategy.pdf

the Enfield Island Loop is heavily modified and not contiguous with natural habitat. For an illustrative flow condition of Q95 very low flow from the derived flow series for the Lee Diversion Channel of 126 Ml/d, Figure 3-2 provides contextual comparison for the extent of flow change from the Beckton water recycling scheme that would occur in the ~100m of the Enfield Island Loop between the Beckton water recycling scheme outfall and the existing intake to King George V Reservoir, and could occur downstream in the remaining ~500m of the Enfield Island Loop and downstream into the Lee Diversion Channel. This contextualises that flow changes local are not necessarily adverse, as described further in Section 3.4.2 for wetted habitat change.

Figure 3-2 Flow in the Enfield Island Loop of the Lee Diversion Channel used for assessment of Beckton water recycling scenarios

a) Locally at EIL; for A82 1:5 moderate-low flow scenario

b) Locally at EIL; for M96 1:20 very-low flow scenario

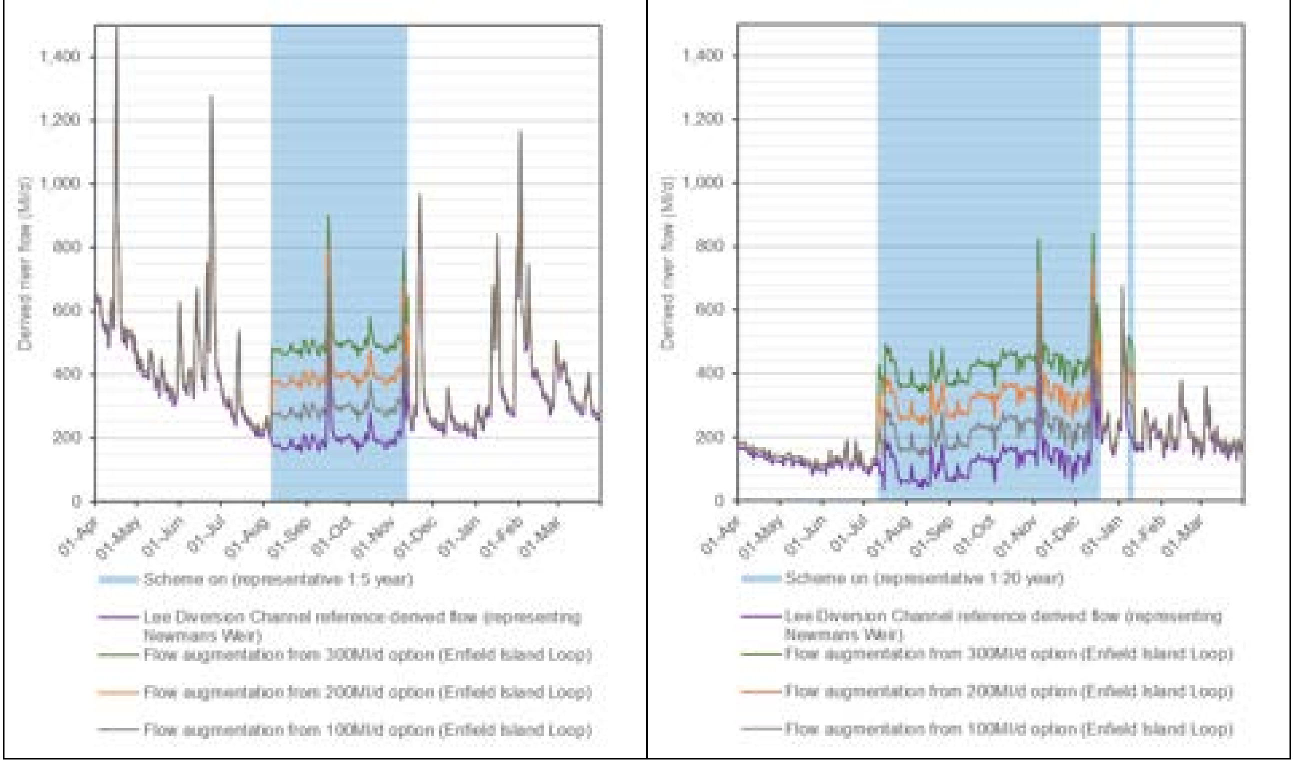
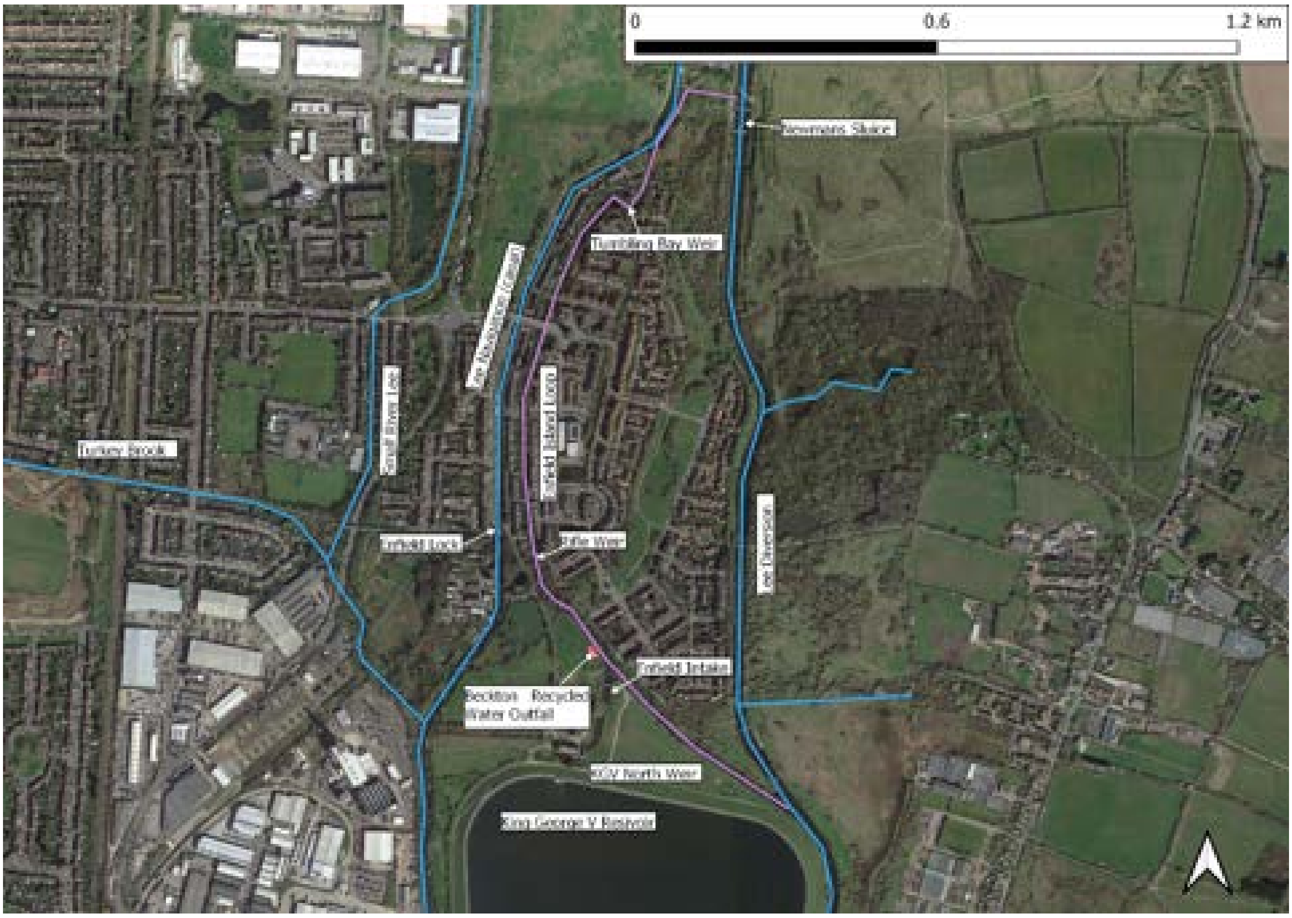


Table 3-1 Contextualisation of an illustrative flow without Beckton water recycling scheme (126 Ml/d) with flow addition from Beckton water recycling scheme

	Without Beckton water recycling scheme	100 Ml/d Beckton water recycling scheme	200 Ml/d Beckton water recycling scheme	300 Ml/d Beckton water recycling scheme
Illustrative flow	126 Ml/d	226 Ml/d	326 Ml/d	426 Ml/d
Flow statistic based on derived flow series	Q95	Q72	Q50	Q35
Flow statistic based on CAMS naturalised series at AP14	<Q100	Q99	Q85	Q65

Figure 3-3 Key locations in the Lee Diversion Channel and Enfield Island Loop



3.2.3. Estuarine Bow Creek

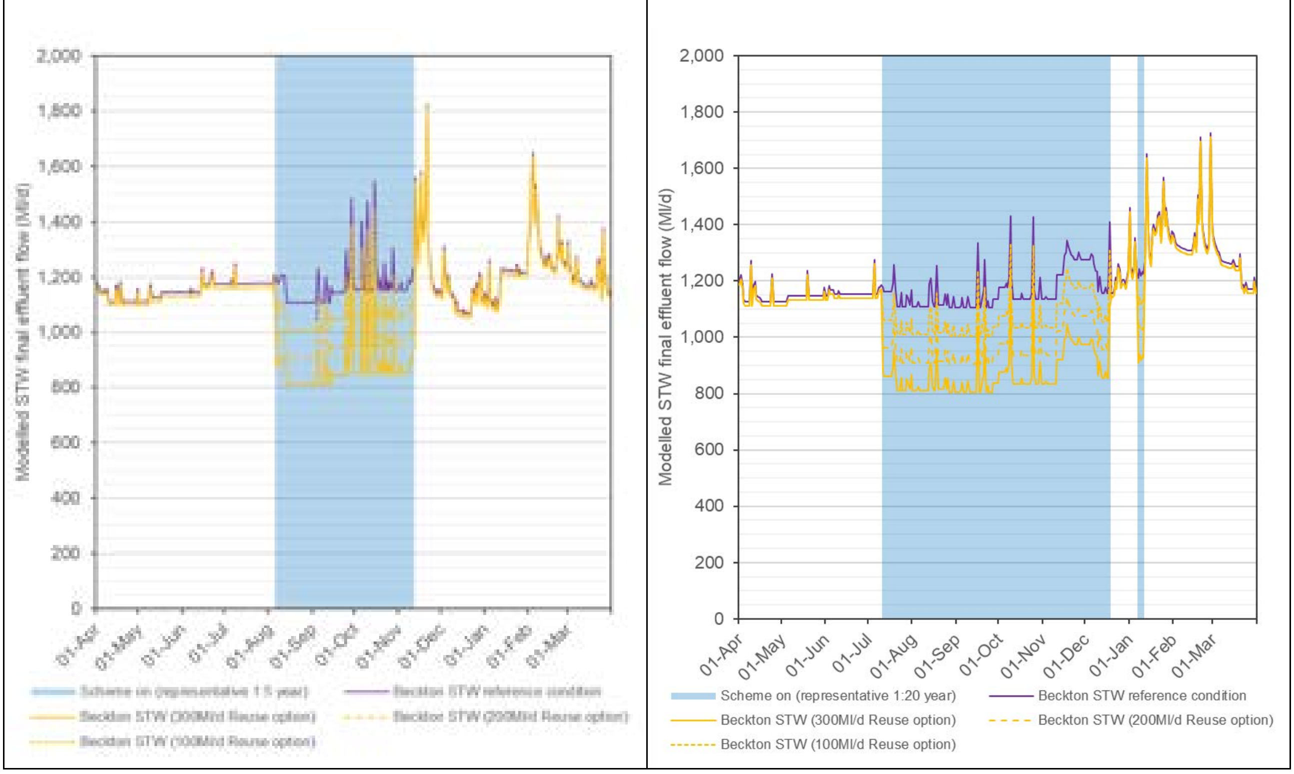
A Beckton water recycling scheme would not impact flows passed forward from the freshwater reaches of the Lee to the tidal Lee in Bow Creek.

3.2.4. Estuarine Thames Tideway

Estuarine hydrodynamics assessment has been undertaken for both the A82 and M96 representative model years with a 300 Ml/d Beckton water recycling scheme. This represents a maximum case of effluent contribution from Beckton STW to the middle Thames Tideway. A flow series has been derived for Beckton STW final effluent based off measured effluent flow rates at the STW and the daily flow characteristics locally in west London in the model years. Modelled effluent flow rates are shown in Figure 3-4 for the sizes of Beckton water recycling scheme.

Figure 3-4 Beckton STW final effluent flow rates used for modelled assessment of Beckton water recycling scenarios

a) Beckton STW final effluent; A82 1:5 moderate-low flow scenario | b) Beckton STW final effluent; M96 1:20 very-low flow scenario



In the A82 scenario during the scheme on period, modelled Beckton STW reference condition flows are 1,170 Ml/d (daily mean). A 300 Ml/d Beckton water recycling scheme would reduce these flows by 300 Ml/d, a 26% reduction. For the other sizes of Beckton water recycling scheme, final effluent flow reductions into the middle Thames Tideway would be 17% for a 200 Ml/d scheme; and 9% for a 100 Ml/d scheme. In the M96 scenario during the scheme on period, modelled Beckton STW reference condition flows are also 1,170 Ml/d (daily mean). A 300 Ml/d Beckton water recycling scheme would reduce these flows by 300 Ml/d, a 26% reduction. For the other sizes of Beckton water recycling scheme, final effluent flow reductions into the middle Thames Tideway would be 17% for a 200 Ml/d scheme; and 9% for a 100 Ml/d scheme.

In addition to the Beckton STW final effluent flow rates, the 2D/3D Thames Tideway hydrodynamic model was parameterised with a representative daily variable flow series for each of the following tributaries of the Thames Tideway: River Thames, River Crane, River Brent, Beverley Brook, River Wandle, River Ravensbourne, River Lee, River Roding, River Beam, River Ingrebourne, Running Water Brook / Rainham Marshes, River Cray and River Darent, Mar Dyke; and Mogden STW and Crossness STW. The Gate 2 Thames Tideway hydrodynamic modelling did not include conjunctive use with Thames Gateway Desalination Plant.

Key modelled hydrodynamic output in the Thames Tideway for assessment of the Beckton water recycling schemes is the effect on water levels. Figure 3-5 and Figure 3-6 show the modelled minimum water levels, at spring tide low water slack, between downstream of Teddington Weir to the QE2 Bridge for A82 and M96 reference conditions and a 300 Ml/d Beckton water recycling scheme.

Figure 3-5 Minimum water level along the Thames Tideway thalweg during A82 flows for reference condition and 300 Ml/d Beckton water recycling scheme during 6 August to 12 November period of operation. (B300 line sitting behind baseline line where no change is shown)

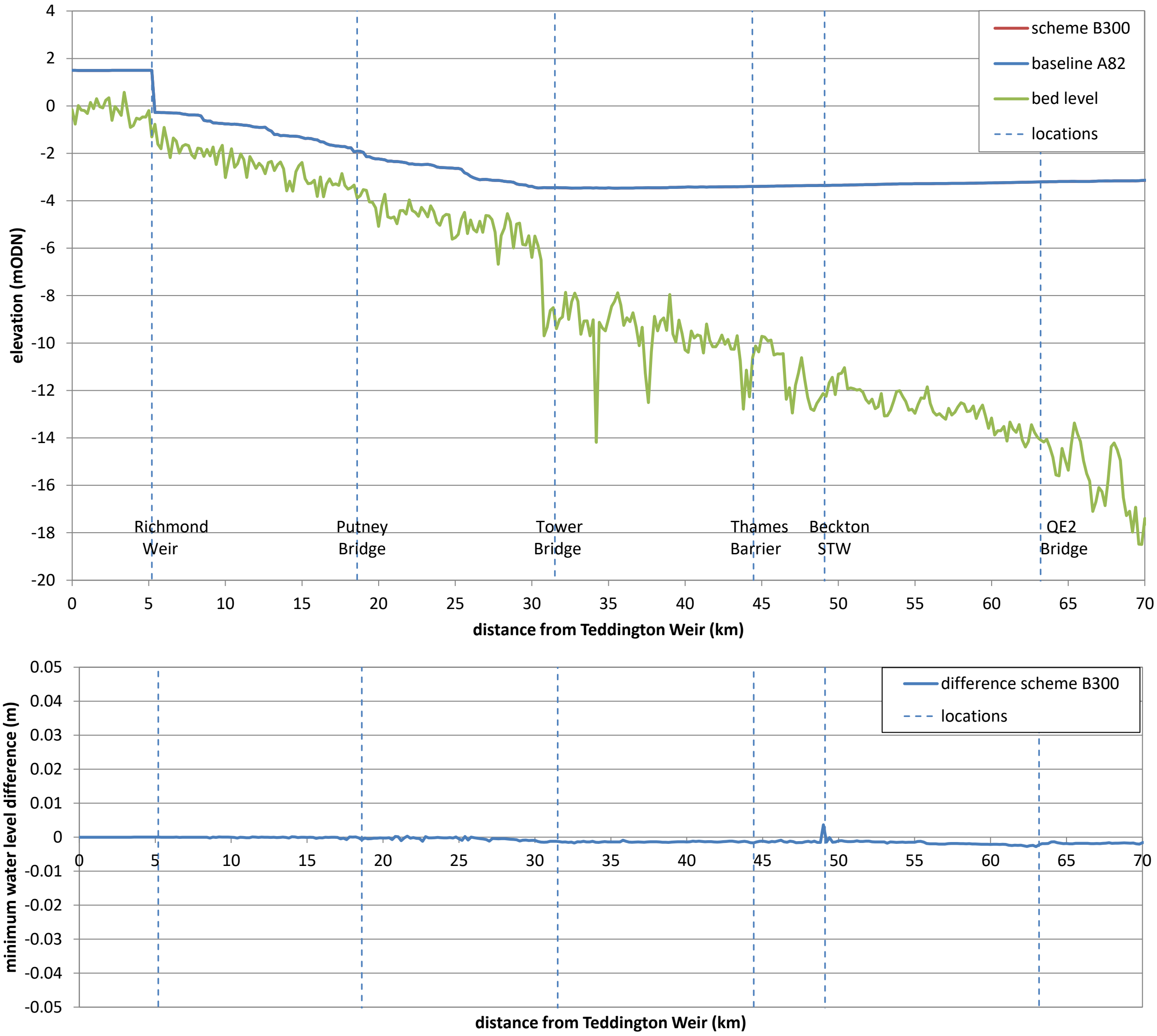
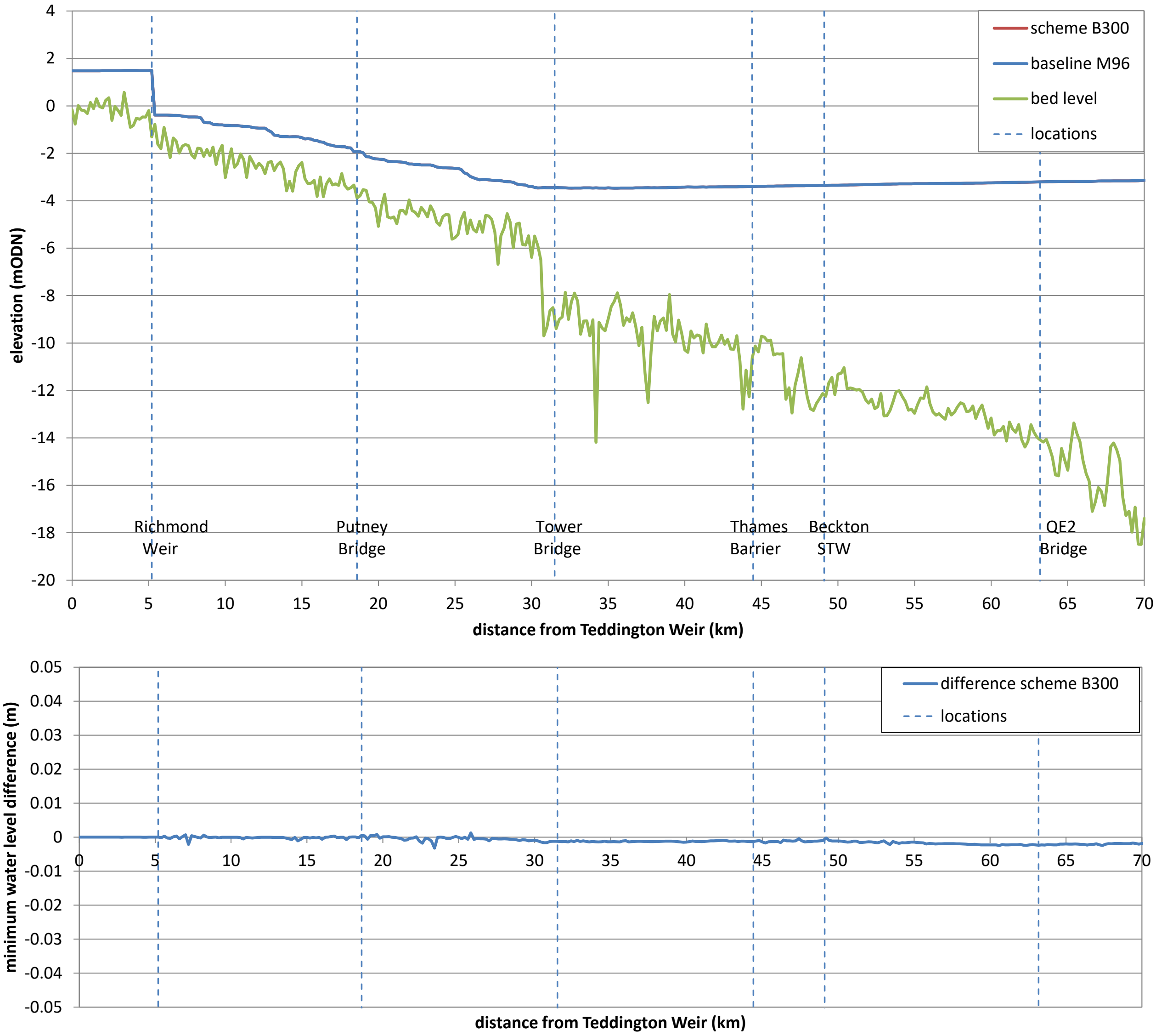


Figure 3-6 Minimum water level along the Thames Tideway thalweg during M96 river flows for reference condition and 300 Ml/d Beckton water recycling scheme during 1 August to 30 November period of operation. (B300 line sitting behind baseline line where no change is shown)



Modelling shows no discernible change in low tide water level for a 300 Ml/d Beckton water recycling scheme compared with reference conditions for A82 or M96 scenarios. It is noted for context, that the volume of estuarine water identified from the 2D/3D Telemac modelling as passing the point on the Thames Tideway at Beckton STW is 80Mm^3 on each flood tide and each ebb tide on a spring tide; and 50Mm^3 on each flood tide and each ebb tide on a neap tide. Both a 1,200 Ml/d reference condition flow contribution from Beckton STW and a 300 Ml/d effluent flow reduction from a 300 Ml/d Beckton water recycling scheme are very small proportions of that tidal exchange.

In order to understand how water surface elevation varied on a diurnal basis during a complete spring-neap-spring tidal cycle, model data for the A82 and M96 baselines and the associated Beckton 300 Ml/d scheme between 15 October and 1 November, extracted from a point in the estuary adjacent to Beckton STW, were plotted alongside baseline-scheme changes. These data are presented in Figure 3-7 for the A82 scenarios and Figure 3-8 for M96 scenarios.

Figure 3-7 Diurnal water surface elevation and change for A82 baseline and 300 Ml/d Beckton water recycling scheme between 15 October and 1 November

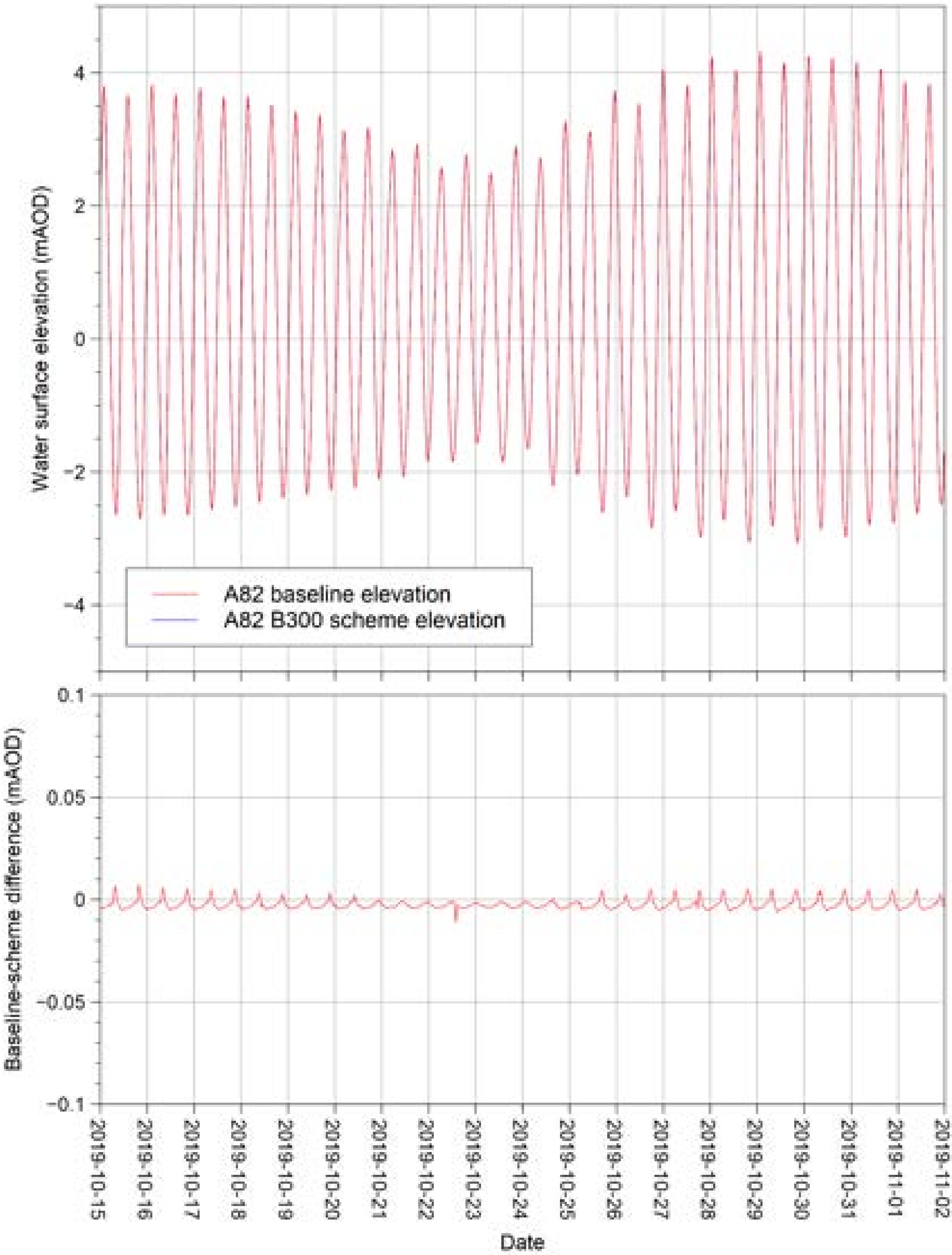
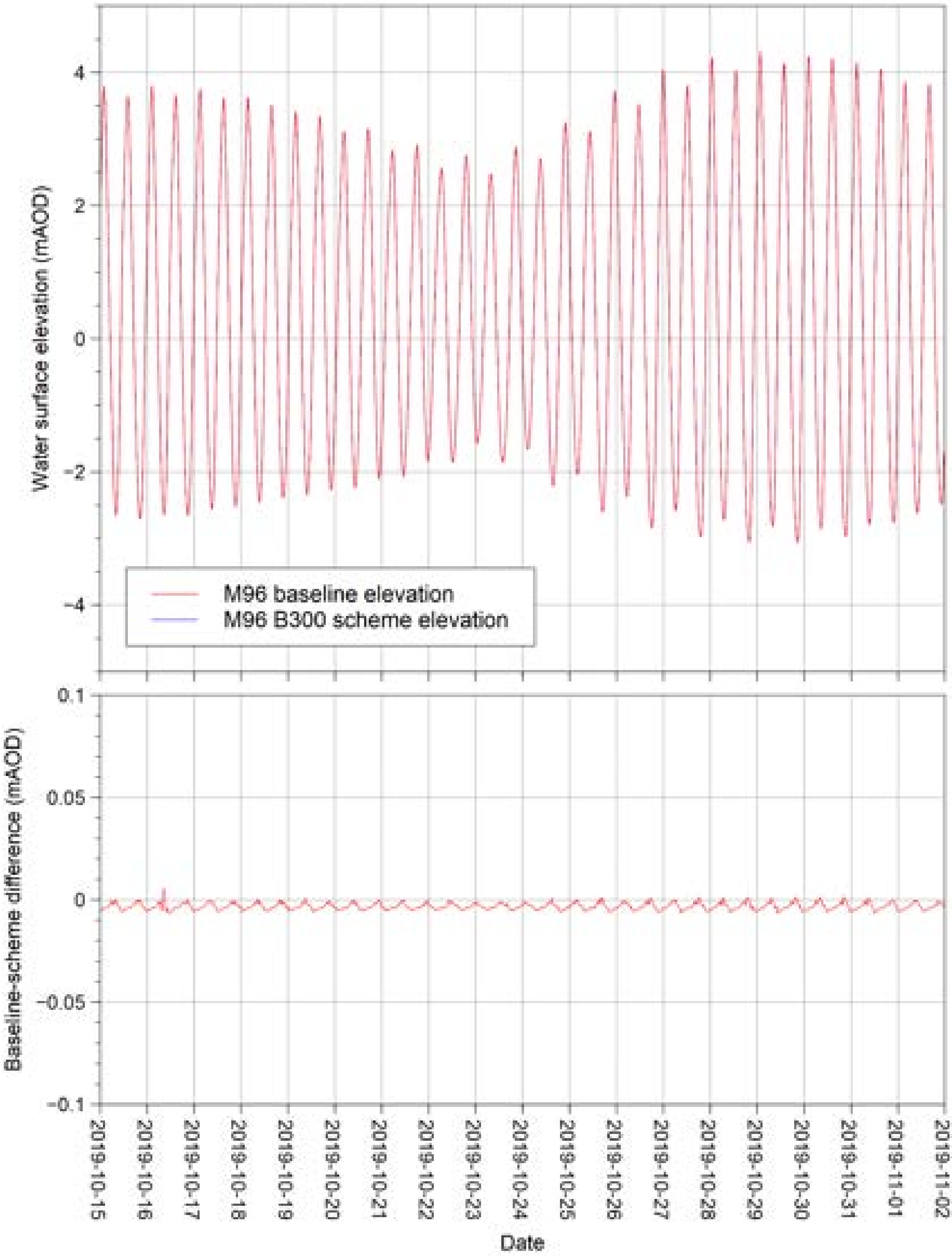


Figure 3-8 Diurnal water surface elevation and change for M96 baseline and 300 Ml/d Beckton water recycling scheme between 15 October and 1 November



Changes between the baseline and 300 Ml/d scheme water surface level for both A82 and M96 show daily differences of not more than 0.01mAOD.

3.3. REVIEW OF BECKTON WATER RECYCLING OUTFALL DESIGN INCLUDING SCREENING

3.3.1. Overview

The Beckton water recycling scheme would include a recycled water outfall on the Enfield Island Loop of the Lee Diversion, at an indicative location shown on Figure 3-3. Details of the design of the outfall are at early stage in Gate 2, noting the basic design includes an exit velocity of 0.3m/s and a lipped physical barrier to prevent the passage of eel or other fish into the outfall structure. As such, at this stage there is no bespoke assessment of the nature of a velocity plume into the Enfield Island Loop. Noting the channel cross-section profile (wide/shallow) and the proportion of water which is recycled water at time of discharge, it is assumed that the plume would extend across the full width of the channel within metres of the discharge point.

3.4. WETTED HABITAT CHANGE IN FRESHWATER LEE CHANNELS, ESTUARINE BOW CREEK AND ESTUARINE THAMES TIDEWAY

3.4.1. Overview

Across the study area, modelled and limited measured information are available from which to describe the level, velocity and wetted habitat within the freshwater reaches around the River Lee and the Thames Tideway.

3.4.2. Freshwater Enfield Island Loop

A full analysis of existing MoRPh data (surveyed on 20 April 2022), River Habitat Survey (RHS) data, LiDAR data and recent photography of the reach (13 September 2022) is presented in Appendix 1, Section 4.4. In summary, the MoRPh data surveyed between the proposed outfall location and the reservoir intake, showed a highly modified channel (Figure 3-9), characterising it as “fairly poor”, with extensive smooth flow and no physical features, except for some submerged macrophytes. The other data supported the observations of a heavily modified channel with modified and reinforced steep to vertical banks, no bank and marginal habitat and very limited in-channel habitat. A longitudinal section of the entire Enfield Island Loop channel shows the great influence of the weirs on flow dynamics within the system. More recently images of the channel downstream of the intake channel identified extensive macrophyte coverage in the channel. It was concluded that any releases associated with the scheme are very unlikely to lead to any impacts given the poor quality and scant available habitat between the outfall and intake channel. The habitat potential of the macrophytes downstream of the intake channel were noted, however these lie outside the potential impacted reach and there currently exists no information to characterise pass forward flows past the intake channel when the scheme is operating and therefore what influence, if any, flows may exert on these macrophytes.

Figure 3-9 Enfield Island Loop of the Lee Diversion Channel from right bank looking downstream to the fenced area of the intake to King George V Reservoir

Representative ADCP cross-section data for Enfield Island Loop is provided in Appendix 1 Section 3.3. Site 10 is most representative for understanding changes due to the proposed discharge, as this cross-section is located adjacent to it (and upstream of the King George V Reservoir intake). Under low flows (262 Ml/d) average cross-section depth and velocity are 1.07m and 0.19m/s. Under high flows (428 Ml/d) average cross-section depth declines slightly to 0.99m and velocity increases to 0.29m/s. Noting that this reflects that repeat surveys are not at exactly the same location, the response to increased flow is clearly an increase in velocity and not a change in depth,

For an illustrative flow condition of Q95 very low flow from the derived flow series for the Lee Diversion Channel of 126 Ml/d, Table 3-2 provides contextual comparison for the extent of wetted habitat change from flow change from the Beckton water recycling schemes that would occur in the ~100m of the Enfield Island Loop between the Beckton water recycling scheme outfall and the existing intake to King George V Reservoir. Using the understanding that the channel between the Beckton water recycling scheme outfall and King George V Reservoir intake is c18m in width and at all relevant flows fully occupying the vertically-sided channel. For a constant water level and constant wetted area of 15m^2 this indicates mean velocity changes as shown in Table 3-2.

Table 3-2 Contextualisation of likely changes in mean velocity in the Enfield Island Loop for an illustrative flow without Beckton water recycling scheme (126 Ml/d) and with flow additions from Beckton water recycling scheme

	Without Beckton water recycling scheme	100 Ml/d Beckton water recycling scheme	200 Ml/d Beckton water recycling scheme	300 Ml/d Beckton water recycling scheme
Illustrative flow	126 Ml/d	226 Ml/d	326 Ml/d	426 Ml/d
Mean velocity	0.10 m/s	0.18 m/s	0.25 m/s	0.33 m/s

3.4.3. Freshwater Lee Diversion

The Lee Diversion Channel around and downstream of Enfield Island Loop is an artificial channel designed to convey flood flows during periods of high flows, although it does convey some limited flows under low and normal flow conditions, particularly between Enfield Island Loop and Chingford. The Lee Diversion Channel is, for most of its length, composed of vertical to near vertical concrete banks and a concrete bed and does not contain any appreciable habitat of note which could be impacted by the proposed released flows.

3.4.4. Estuarine Bow Creek

A Beckton water recycling scheme would not impact flows passed forward from the freshwater reaches of the Lee to the tidal Lee in Bow Creek.

3.4.5. Middle Thames Tideway (Battersea to Tower Bridge)

The 2D/3D Thames Tideway Telemac model has been used to provide predictions of intertidal area exposure and duration of exposure. Exposure and changes against the baseline (outlined in Appendix 1 Table A-1, Figure 3-10 and Figure 3-11) for the 300 Ml/d Beckton water recycling scheme for A82 and M96 runs is shown in Table 3-3.

Table 3-3 Beckton water recycling 300 Ml/d scheme intertidal area exposure

Reach	Max exposed area (ha) – difference from baseline		Average exposed area (ha) – difference from baseline		Average duration of exposure (hours) – difference from baseline	
	A82	M96	A82	M96	A82	M96
Tower Bridge to Beckton STW outfall	0	0	0	0	0	0
Beckton to Dagenham (3km seaward)	0.2	0.1	0	0	0	0.1
Dagenham to QE2 Bridge	0.3	0.4	0	0	0	0
Total	0.4	0.5	0.1	0	-	-

Visual representation of the distribution of the percentage of time of intertidal exposure for the 300 Ml/d scheme A82 and M96 model runs are presented in Figure 3-10 and Figure 3-11 respectively.

Figure 3-10 Beckton 300 Ml/d scheme A82 percentage of time intertidal exposure change against baseline (15 October to 1 November)

Figure 3-11 Beckton 300 Ml/d scheme M96 percentage of time intertidal exposure change against baseline (15 October to 1 November)

Modelled minimum water levels along the Thames Tideway (Teddington Weir to seaward of the QE2 Bridge) for the baseline and 300 Ml/d Beckton water recycling scheme model runs are presented in Figure 3-5 and Figure 3-6. There is no discernible change in water level.

The modelled data (Table 3-3, Figure 3-10 and Figure 3-11) indicate that between Beckton STW and QE2 Bridge there is very limited change in exposure between the baseline and the scheme, ranging between a maximum of 0.4ha for the A82 scenario and 0.5ha for the M96 scenario. There is generally very limited change in the duration of exposure, at some points reducing by only a few minutes compared to the baseline.

3.5. ENFIELD ISLAND LOOP BARRIER PASSABILITY

3.5.1. Overview

There is only limited measured information available from which to describe the water level at barriers within Enfield Island Loop. This limited information is expanded below to provide an understanding of potential changes in barrier passibility.

3.5.2. Lower weir

As noted in Appendix 1 Section 5.3 there are three weirs on the Enfield Island Loop (see also Figure 3-3), however, as the proposed outfall discharge is immediately upstream of the King George V Reservoir intake and below both Tumbling Bay Weir and Rifle Weir only the low height weir (KGV North Weir), located immediately prior to the confluence with the Lee Diversion Channel will be affected by changes in flow. KGV North Weir is downstream of the intake to King George V Reservoir, and its primary function is to maintain water level at the intake itself for abstraction (see Table 6-1f).

There is limited information to characterise the impact of the proposed discharge on passibility of this weir noting that both the reference flow regime at the weir and effect of re-abstraction on the flow regime are dependent on operational circumstances. However, level data collected using fixed pressure transducers has indicated that the average difference of water level upstream and downstream of the reservoir intake is 0.15m. While not directly applicable to the change in weir passibility, this information could indicate that increased flows in the Enfield Island Loop downstream of the Beckton water recycling scheme outfall could lead to increased levels over the low-level weir. LiDAR data presented in Appendix 1 Figure 6-2 indicates the water surface difference across the weir to be c1.0m. Site visit photography (Appendix 1 Table 6-1f) identifies a straight crested, low slope transverse weir without a fish pass. Recognising that the CAMS assessment has identified unsuitably low flow in the Lee Diversion Channel prior to the Enfield Island Loop (CAMS AP14), any flow increase that may be associated with the scheme is considered beneficial to connectivity. An increase in flow would: increase the water depth over the weir crest; increase the depth of water on the weir face; decrease the head difference between the water level at the tail of the weir and the crest - all three of which are considered beneficial for connectivity.

3.6. THAMES TIDEWAY ESTUARINE SEDIMENT ASSESSMENT

An assessment of sediment flux within the Thames Tideway at four specific transects, Tower Bridge, Thames Barrier, Halfway Reach and Long Reach (Figure 3-12) between the baseline and with the 300 Ml/d Beckton water recycling scheme has been modelled, and the results for a single spring tide for the A82 and M96 model runs are presented in Figure 3-13 and Figure 3-14 respectively. This data represents the changes over a single spring tide only and does not indicate long term cumulative changes in sediment flux over multiple tides, river flows and fluvial, estuarine and marine sediment supply conditions.

Figure 3-12 Thames Tideway sediment flux transect locations

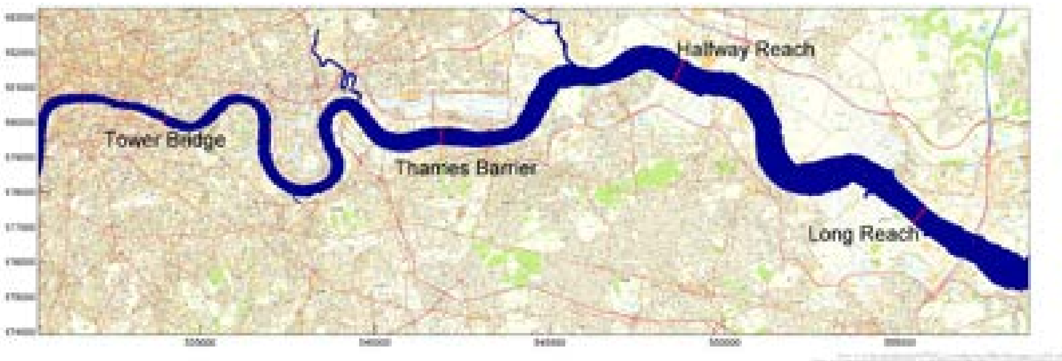


Figure 3-13 Sediment flux at each of the four transects over a spring tide for A82 baseline and 300Ml/d Beckton water recycling scheme

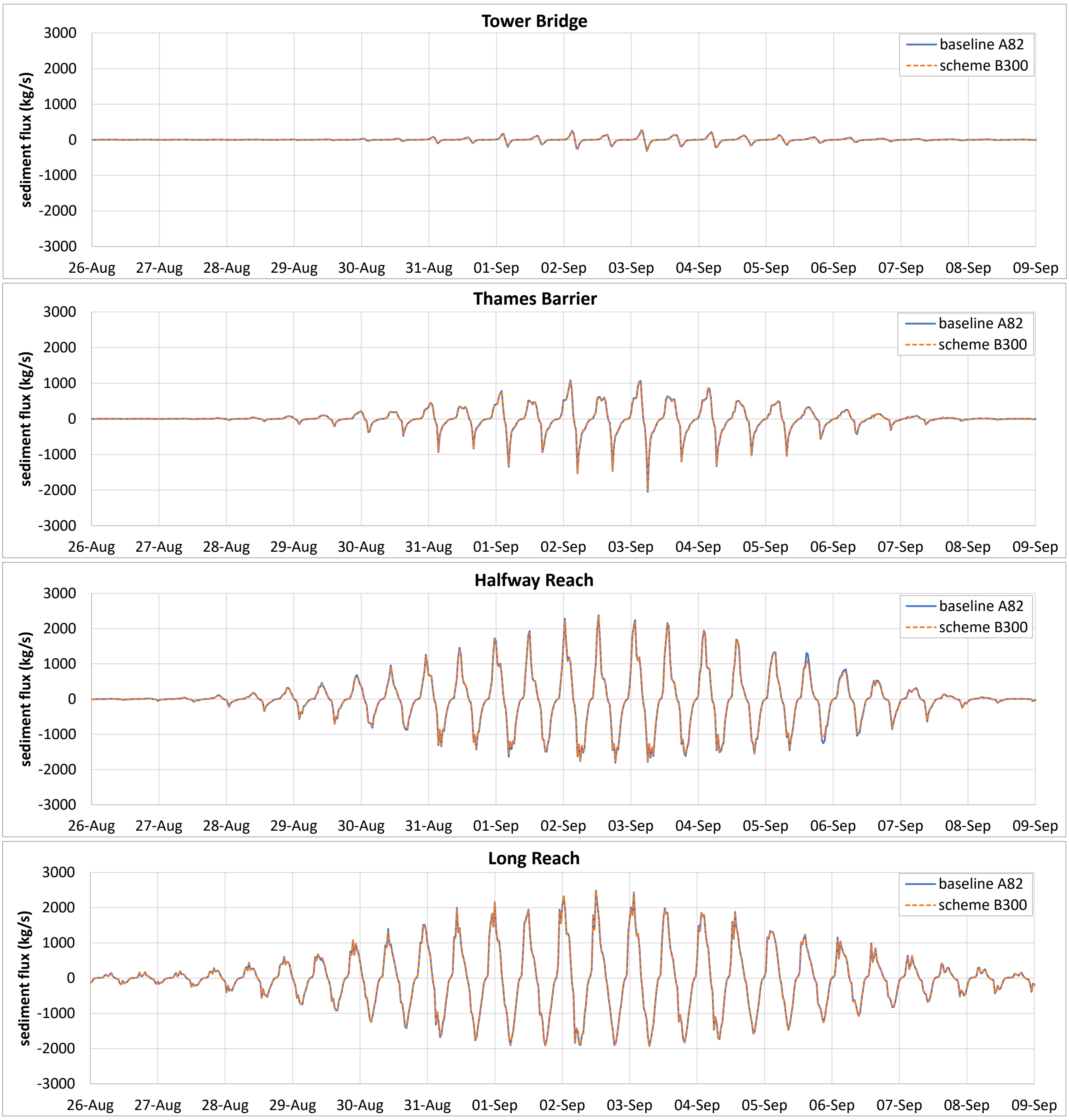
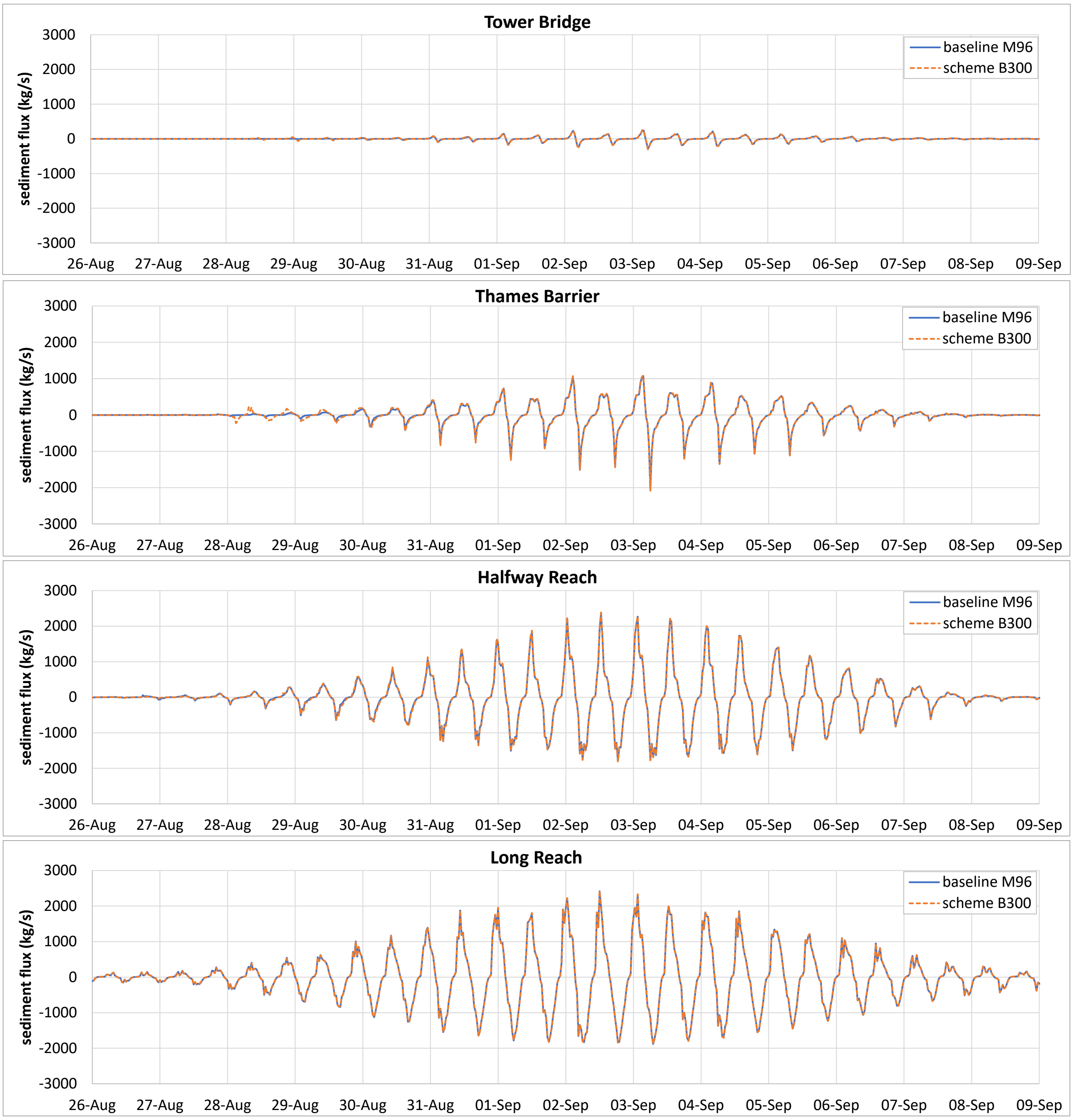


Figure 3-14 Sediment flux at each of the four transects over a spring tide for M96 baseline and 300Ml/d Beckton water recycling scheme



For both the A82 and M96 schemes there are essentially negligible changes in sediment flux during a spring tide, with only the visible changes in flux noted at tidal maxima and minima on 5 September for the Halfway Reach transect under the A82 scenario only.

Table 3-4 and Table 3-5 show the numerical differences in total ebb and flood sediment transport over the spring tide for the A82 and M96 scenarios respectively.

Table 3-4 Total ebb and flood sediment transport over a spring tide for A82 baseline and 300Ml/d Beckton water recycling scheme

Transect	Total ebb sediment transport (tonnes)			Total flood sediment transport (tonnes)		
	Baseline	Scheme	Difference	Baseline	Scheme	Difference
Tower Bridge	2,541.7	2,458.5	-83.2 (-3.3%)	2,825.3	2,732.1	-93.3 (-3.3%)
Thames Barrier	17,711.3	17,351.4	-359.9 (-2.0%)	18,349.5	17,928.1	-421.4 (-2.3%)
Halfway Reach	39,007.8	38,310.6	-697.2 (-1.8%)	39,616.8	38,855.4	-761.4 (-1.9%)
Long Reach	49,511.2	49,173.7	-337.4 (-0.7%)	48,507.7	48,350.6	-157.1 (-0.3%)

Considering baseline sediment transport across the transects during ebb and flood periods for the A82 scenario (Table 3-4) there is a net landward movement of sediment for the Tower Bridge, Thames Barrier and Halfway Reach transects (283.6t, 638.2t and 609.0t respectively), with a net seaward movement at the Long Reach transect (1,003.5t). The modelled data indicate negligible change in sediment transport between the baseline and the scheme for each of the transects, with reductions of between -3.3% and -0.3% (declining in a seawards direction), with the net movement of sediment at each transect under the scheme remaining unchanged from the baseline.

Table 3-5 Total ebb and flood sediment transport over a spring tide for M96 baseline and 300Ml/d Beckton water recycling scheme

Transect	Total ebb sediment transport (tonnes)			Total flood sediment transport (tonnes)		
	Baseline	Scheme	Difference	Baseline	Scheme	Difference
Tower Bridge	2,337.1	2,421.7	+84.6 (+3.6%)	2,648.7	2,731.0	+82.3 (+3.1%)
Thames Barrier	16,697.0	17,026.2	+329.2 (+2.0%)	17,592.8	17,930.9	+338.1 (+1.9%)
Halfway Reach	37,947.5	38,444.5	+497.0 (+1.3%)	38,927.5	39,465.6	+538.2 (+1.4%)
Long Reach	48,073.8	48,119.4	+45.6 (+0.1%)	46,521.1	46,703.5	+182.3 (+0.4%)

Considering baseline sediment transport across the transects during ebb and flood periods for the M96 scenario (Table 3-5) there is a net landward movement of sediment for the Tower Bridge, Thames Barrier and Halfway Reach transects (311.6t, 895.8t and 980.0t respectively), with a net seaward movement at the Long Reach transect (1,552.7t). The modelled data indicate negligible change in sediment transport between the baseline and the scheme for each of the transects, with reductions of between +3.6% and +0.1% (declining in a seawards direction), with the net movement of sediment at each transect under the scheme remaining unchanged from the baseline.

3.7. SUMMARY OF PHYSICAL ENVIRONMENT ASSESSMENT OF BECKTON WATER RECYCLING SCHEMES

Table 3-6 summarises the potential physical environment impacts for each of the sizes of a Beckton water recycling scheme.

Table 3-6 Summary of potential physical environment impacts for Beckton water recycling schemes

Size	Flow	Outfall design	Wetted habitat	Barrier passability	Estuarine sediment
100 Ml/d	Major. 80% increase in very low flows(Q95) in ~100m reach of Enfield Island Loop, with 0-80% increase in flows downstream in ~500m reach of Enfield Island Loop and downstream Lee Diversion. Zero change beyond Flanders Weir.	Negligible. Not set out in detail at Gate 2 but due to extent of flow increase, a 0.3m/s exit velocity and the shallow channel depth would result in full dispersal of plume within metres of the outfall in a heavily modified channel.	Minor. No change in water level or water width, 0.08m/s increase in mean flow velocity in ~100m reach of heavily modified channel of the Enfield Island Loop at very low flow conditions. Unknown change downstream in a largely artificial channel without aquatic habitat. Indiscernible change in intertidal exposure in the estuarine Thames Tideway	Negligible; fisheries conclusions are included in the B.2.3 Fish Assessment Report. One low barrier, KGV North Weir, in the Enfield Island Loop with potential for increase in depth of water over crest and reduction in head difference both of which reduce any barrier effect.	Negligible. Negligible changes in sediment transport within the Thames Tideway from final effluent flow reductions at Beckton STW.
200 Ml/d	Major. 160% increase in very low flows (Q95) in ~100m reach of Enfield Island Loop, with 0-160% increase in flows downstream in ~500m reach of Enfield Island Loop and downstream Lee Diversion. Zero change beyond Flanders Weir.		Moderate. No change in water level or water width and 0.15m/s increase in mean flow velocity in ~100m reach of heavily modified channel of the Enfield Island Loop at very low flow conditions. Unknown change downstream in a largely artificial channel without aquatic habitat. Indiscernible change in intertidal exposure in the estuarine Thames Tideway		
300 Ml/d	Major. 240% increase in very low flows (Q95) in ~100m reach of Enfield Island Loop, with 0-240% increase in flows downstream in ~500m reach of Enfield Island Loop and downstream Lee Diversion. Zero change beyond Flanders Weir.		Moderate. No change in water level or water width and 0.23m/s increase in mean flow velocity in ~100m reach of heavily modified channel of the Enfield Island Loop at very low flow conditions. Unknown change downstream in a largely artificial channel without aquatic habitat. Indiscernible change in intertidal exposure in the estuarine Thames Tideway		

For the Beckton water recycling scheme, physical environment impacts are described in the ~100m reach of heavily modified channel of the Enfield Island Loop between a Beckton water recycling outfall and the existing intake to King George V Reservoir. There may also be impacts in the remaining ~500m heavily modified reach of the Enfield Island Loop downstream to the confluence with the Lee Diversion Channel but the effects cannot be quantified as they are entirely dependent on the abstraction regime operated for the Thames Water intakes. Where there are physical environment impacts, these relate to the major increase in flow, of 100 Ml/d, 200 Ml/d or 300 Ml/d. This is in the context that derived reference conditions Q95 very low flow is 126 Ml/d. However, 126 Ml/d is a non-natural flow in the middle reaches of the Lee and Q95 is considered by the Environment Agency as 280 Ml/d. Those major increases in flow would increase velocities within the channel. The channel is observed with steep banks and from ADCP survey is with limited bed variability in the ~100m impacted reach. Where the bank is not constraining the hydraulic response to increased flow, there is also potential for increases in wetted channel width.

The Beckton water recycling scheme would not associate with effects on the Thames Tideway from reductions in Beckton STW final effluent input into the middle Tideway. Hydrodynamic modelling has identified negligible changes in low water spring tide water levels and therefore negligible change in intertidal habitat exposure. The effects on modelled suspended sediment concentration within the Thames Tideway for Beckton water recycling scheme are indiscernible from reference conditions and therefore there would be no change in sediment deposition and mud habitats in the Thames Estuary.

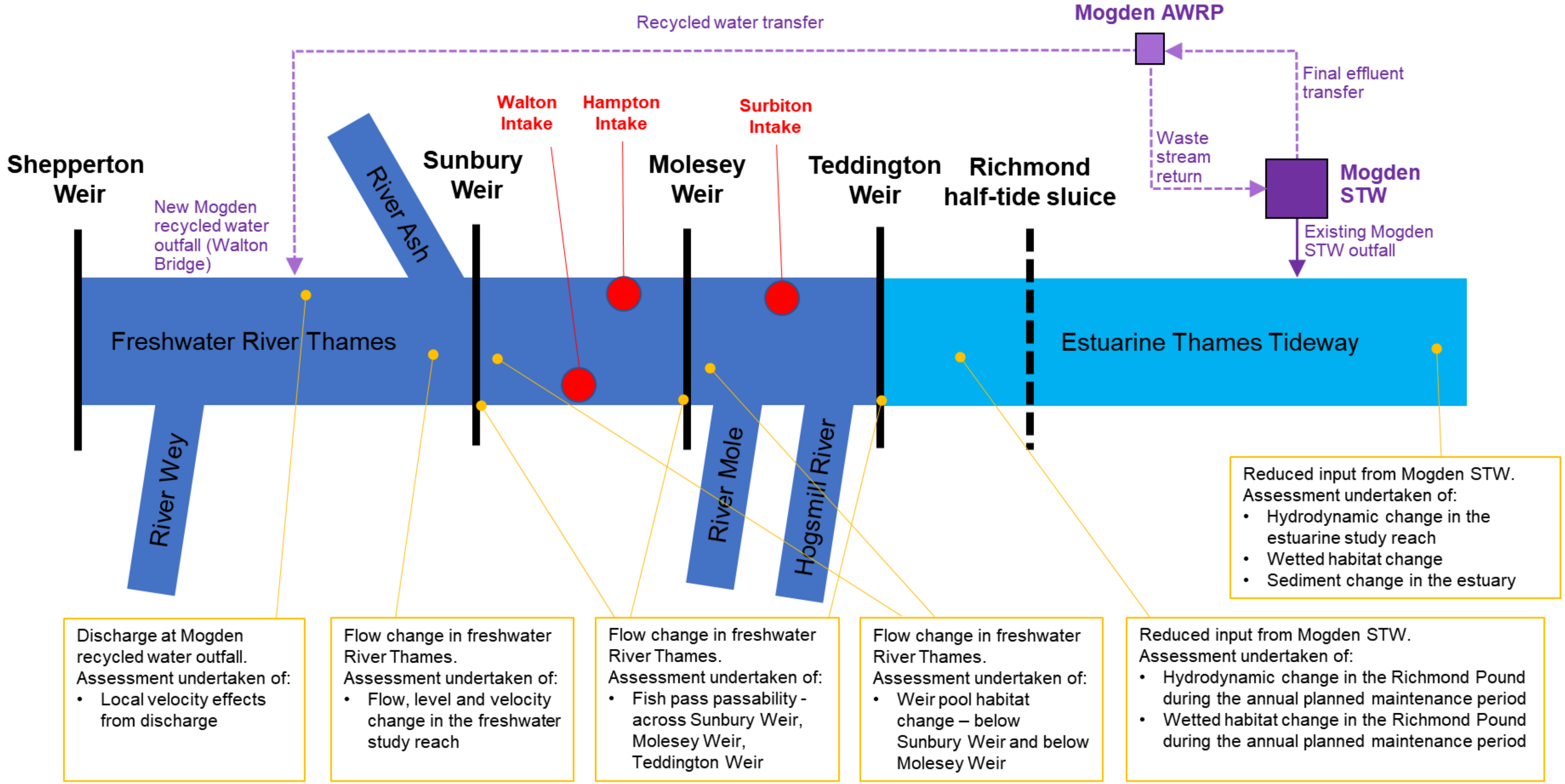
4. PHYSICAL ENVIRONMENT ASSESSMENT OF MOGDEN WATER RECYCLING SCHEME

4.1. INTRODUCTION

Specific to the Mogden water recycling schemes, the assessment for each of the tasks set out in Table 1-1 is set out in this section. As set out spatially in the conceptualisation of physical environment effects in Figure 4-1, the specific assessments of the Mogden water recycling schemes are:

- Flow changes from Mogden water recycling schemes
- Review of Mogden water recycling outfall design including screening
- Wetted habitat change in freshwater River Thames and estuarine Thames Tideway
- Sunbury Weir, Molesey Weir and Teddington Weir fish pass and barrier passability
- Richmond Pound drawdown physical environment assessment
- Thames Tideway estuarine sediment assessment
- Summary of physical environment assessment of Mogden water recycling schemes.

Figure 4-1 Representation of the Mogden water recycling aquatic study area with conceptualisation of physical environment effects and listing of assessment undertaken for Gate 2



4.2. FLOW CHANGES FROM MOGDEN WATER RECYCLING SCHEMES

4.2.1. Overview

A Mogden water recycling scheme would increase flows in the River Thames at the Walton Bridge outfall by 50-200 Ml/d (dependent on scheme assessed) when in use for water resources purposes, and at 25% of the scheme rate at all other times. The Gate 2 engineering design includes a 25% plant maintenance flow at all times, with the treated water being discharged to the River Thames at Walton Bridge but not re-abstracted.

When operational for water resources purposes, flow augmented by a Mogden water recycling scheme would typically be re-abstracted at Thames Water's Walton intake, although there may be some operational circumstances where a Mogden water recycling scheme would also enable increased abstraction rates at Thames Water's Hampton intake to the TLT. It is considered unlikely that a Mogden water recycling scheme would associate with increased abstraction at Thames Water's Surbiton intake as, typically, at times of operation of a Mogden water recycling scheme, the Surbiton intake would be abstracting at maximum rate anyway in order to manage abstraction rates through Thames Water's M2 licence against Teddington Target Flows.

Final effluent flows from Mogden STW discharged to the estuarine Thames Tideway at Isleworth Ait would reduce by the corresponding amount to the amount transferred to the freshwater River Thames at Walton Bridge.

4.2.2. Freshwater River Thames

Although the WRSE water resources model is effective at describing the operational pattern of a London Effluent Reuse scheme and the flows at Teddington Weir it is not specifically designed to represent abstraction rates at individual Thames Water' intakes and as such it is not a precise tool for describing River Thames flows at Walton Bridge. The WRSE water resources model representation of River Thames flows at Walton Bridge for reference conditions and with a 200 Ml/d Mogden water recycling scheme as used in the River Thames 1D modelling are shown in Figure 4-2. The magnitude of model reference condition flows correspond well with the 30 year of gauge data at the Thames at Walton river flow gauge, which is for the same section of river. The Walton flow gauge has informed the flows used in 2D/3D hydraulic modelling of the study area presented in Appendix 1 Section 2.2. The scheme would only be triggered for operation at Teddington Target Flows of 700 Ml/d or lower and as such the scheme would only augment flows under low to very low flow periods in the River Thames, and would not operate continuously at those discharge volumes (with the exception of a 25% maintenance flow).

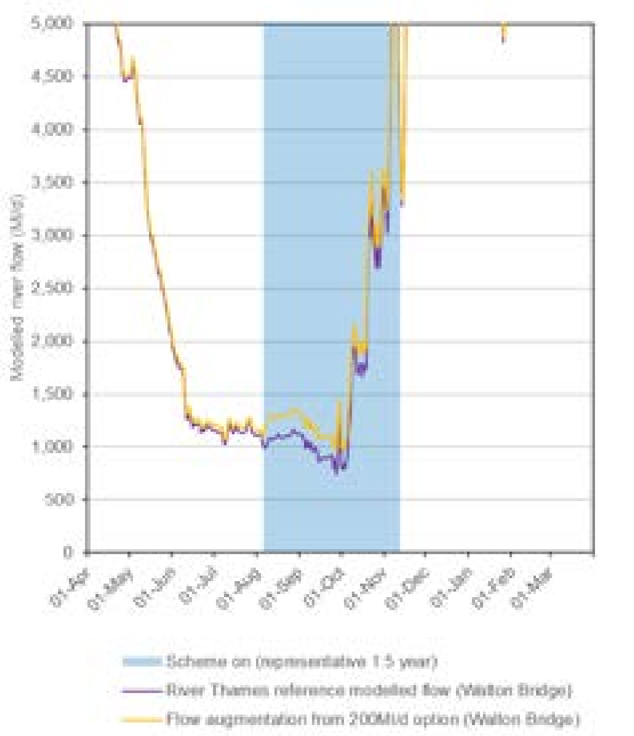
At Walton Bridge both the selected 1:5 year return period and the selected 1:20 year return period modelled reference conditions include a prolonged period through the summer and autumn months of modelled river flow at Walton Bridge around 1,000 Ml/d, reducing to c.750 Ml/d. A 200 Ml/d Mogden water recycling scheme would increase these flows as shown in Figure 4-2, including a 200 Ml/d increase during the representative scheme on period and a 50 Ml/d increase outside those periods. Reference condition flows in the River Thames at Walton Bridge are lowest during the representative scheme on periods, noting that there are also periods of low flow as flows in the River Thames recede in late spring/ early summer prior to the representative scheme on periods. However, in general, outside the representative scheme on periods river flows are much higher – to a peak of 17,000 Ml/d in the A82 scenario and 22,000 Ml/d in the M96 scenario, noting the flow axis is truncated in Figure 4-2. Other scheme sizes would increase flows during the same periods as follows: a 150 Ml/d Mogden water recycling scheme would increase flows by 150 Ml/d during the representative scheme on period and by 37.5 Ml/d outside those periods; a 100 Ml/d Mogden water recycling scheme would increase flows by 100 Ml/d during the representative scheme on period and by 25 Ml/d outside those periods; and a 50 Ml/d Mogden water recycling scheme would increase flows by 50 Ml/d during the representative scheme on period and by 12.5 Ml/d outside those periods.

A selected representative date has been used to show an indicative flow pattern along the River Thames from Walton Bridge to Teddington Weir in Figure 4-3 for a 200 Ml/d Mogden water recycling scheme. The representative date shows the typical pattern of flow change in the freshwater River Thames study area for a typical operation date with river flows at Walton Bridge of c.950 Ml/d (Q91) and pass-forward flows to the estuarine Thames Tideway at Teddington Weir of 700 Ml/d. At the Mogden water recycling outfall at Walton Bridge, river flow would be augmented by 200 Ml/d for the sized scheme shown; this represents a 21% increase in river flow. For the other sizes of Mogden water recycling scheme (not shown) flow increases locally at Walton

Bridge would be 16% for a 150 Ml/d scheme; 11% for a 100 Ml/d scheme; and 5% for a 50 Ml/d scheme. For lower river flows at Walton Bridge the proportional flow increase from augmentation release from a Mogden water recycling scheme would be higher. The augmented river flow conditions would remain the same over Sunbury Weir. At Thames Water's existing Walton and Hampton intakes a significant proportion of the augmented flow would be abstracted, in addition to river flows abstracted under reference conditions. In the representative date shown, all augmented flow is re-abstracted at the Walton intake and there are no flow differences downstream of there as consequence of the Mogden water recycling scheme. In total in the representation shown, 3.4km of freshwater River Thames would be subject to flow augmentation. Where there is some re-abstraction at the Hampton intake, a further 2.0km of freshwater River Thames would be subject to flow augmentation. Regardless of whether re-abstraction occurs at Walton or Hampton, it is unlikely that there would be flow change over Molesey Weir. It is also recognised that Molesey Weir has a legally binding minimum flow rate of 168 Ml/d to maintain the fish passes there. Under low river flow conditions, lowest flows typically occur at Molesey Weir which is located between the Hampton intake and the confluence with the River Mole. Downstream of Molesey Weir there is flow addition from the River Mole, abstraction at the Surbiton intake and flow addition from the Hogsmill River. The Surbiton intake and Hogsmill River are not specifically included in the 1D water quality model and are therefore not individually shown as amending flow on Figure 4-3.

Figure 4-2 Flow downstream of Walton outfall in the freshwater River Thames used for modelled assessment of Mogden water recycling scenarios

a) Downstream Walton outfall; A82 1:5 moderate-low flow scenario



b) Downstream Walton outfall; M96 1:20 very-low flow scenario

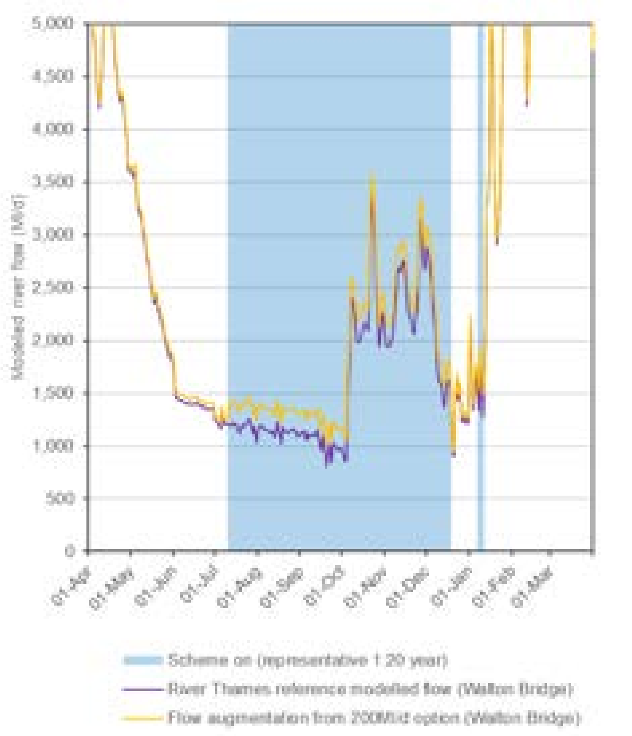
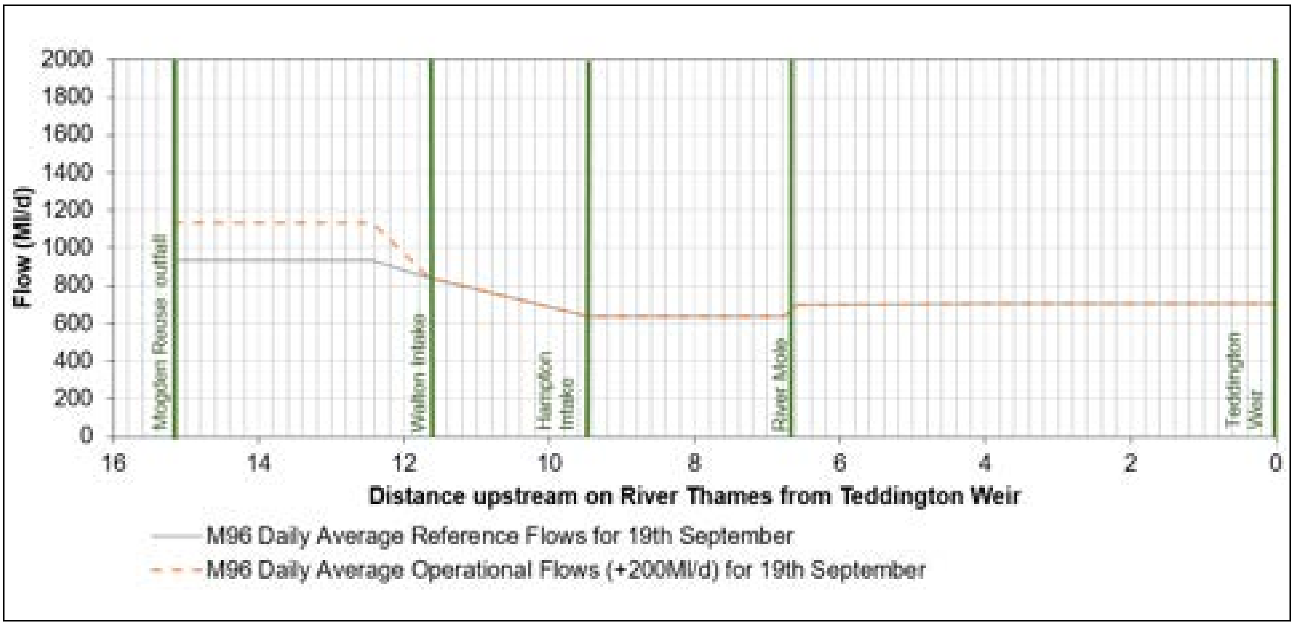


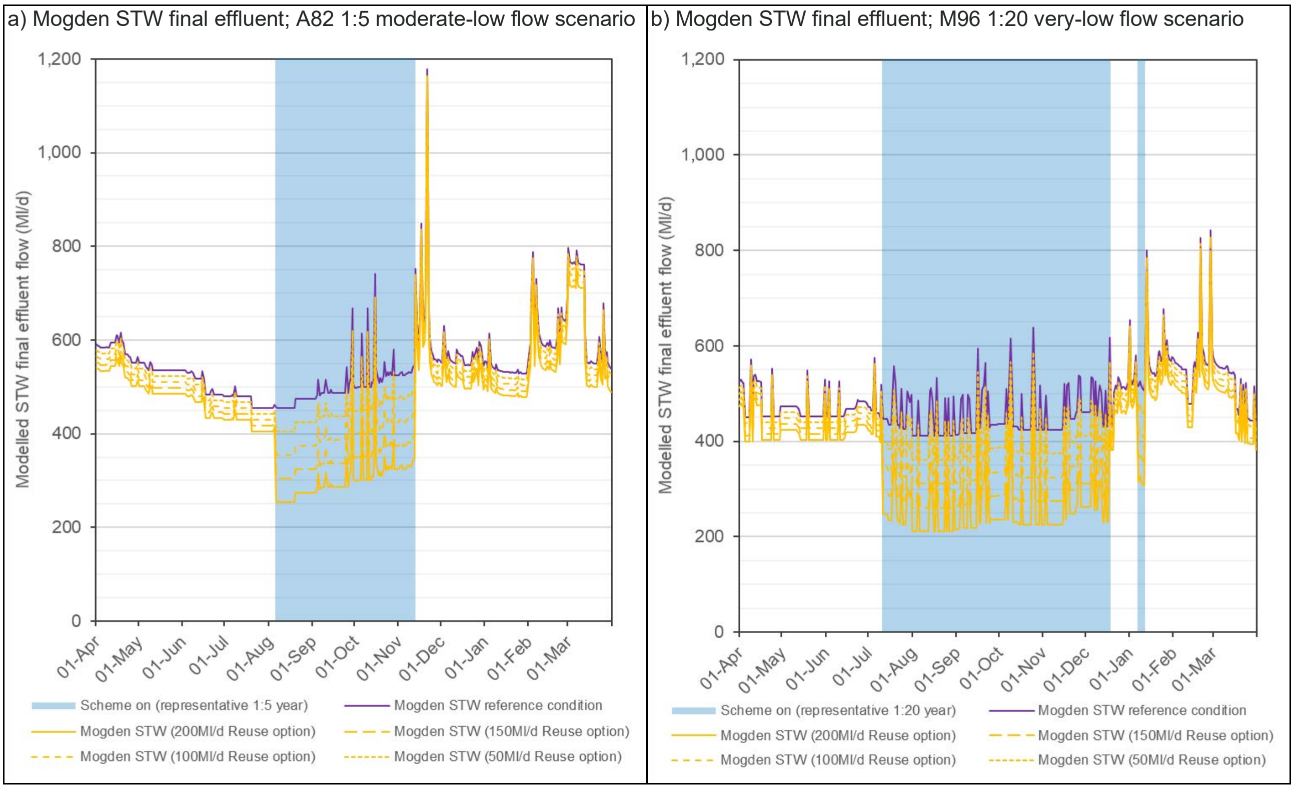
Figure 4-3 Flow along the study reach of the freshwater River Thames for assessed Mogden water recycling scenarios



4.2.3. Estuarine Thames Tideway

Estuarine hydrodynamics assessment has been undertaken for both the A82 and M96 representative model years with a 200 Ml/d Mogden water recycling scheme. This represents a maximum case of effluent contribution from Mogden STW to the upper Thames Tideway. A flow series has been derived for Mogden STW final effluent based off measured effluent flow rates at the STW and the daily flow characteristics locally in west London in the model years. Modelled effluent flow rates are shown in Figure 4 for the sizes of Mogden water recycling scheme.

Figure 4 Mogden STW final effluent flow rates used for modelled assessment of Mogden water recycling scenarios



In the A82 scenario during the scheme on period, modelled Mogden STW reference condition flows are 504 Ml/d (daily mean). A 200 Ml/d Mogden water recycling scheme would reduce these flows by 200 Ml/d, a 40% reduction. For the other sizes of Mogden water recycling scheme, final effluent flow reductions into the upper Thames Tideway at Isleworth Ait would be 30% for a 150 Ml/d scheme; 20% for a 100 Ml/d scheme; and 10% for a 50 Ml/d scheme. In the M96 scenario during the scheme on period, modelled Mogden STW reference condition flows are 458 Ml/d (daily mean). A 200 Ml/d Mogden water recycling scheme would reduce these flows by 200 Ml/d, a 44% reduction. For the other sizes of Mogden water recycling scheme, final effluent flow reductions into the upper Thames Tideway at Isleworth Ait would be 33% for a 150 Ml/d scheme; 22% for a 100 Ml/d scheme; and 11% for a 50 Ml/d scheme.

In addition to the Mogden STW final effluent flow rates, the 2D/3D Thames Tideway hydrodynamic model was parameterised with a representative daily variable flow series for each of the following tributaries of the Thames Tideway: River Thames, River Crane, River Brent, Beverley Brook, River Wandle, River Ravensbourne, River Lee, River Roding, River Beam, River Ingrebourne, Running Water Brook / Rainham Marshes, River Cray and River Darent, Mar Dyke; and Beckton STW and Crossness STW. The Gate 2 Thames Tideway hydrodynamic modelling did not include conjunctive use with Thames Gateway Desalination Plant.

Key modelled hydrodynamic output in the Thames Tideway for assessment of the Mogden water recycling schemes is the effect on water levels. Figure 4-5 and Figure 4-6 show the modelled minimum water levels, at spring tide low water slack, between downstream of Teddington Weir to immediately seaward of Putney Bridge for A82 and M96 reference conditions and a 200 Ml/d Mogden water recycling scheme.

Figure 4-5 Minimum water level along the Thames Tideway thalweg during A82 flows for reference condition and 200 Ml/d Mogden water recycling scheme during 6 August to 12 November period of operation.

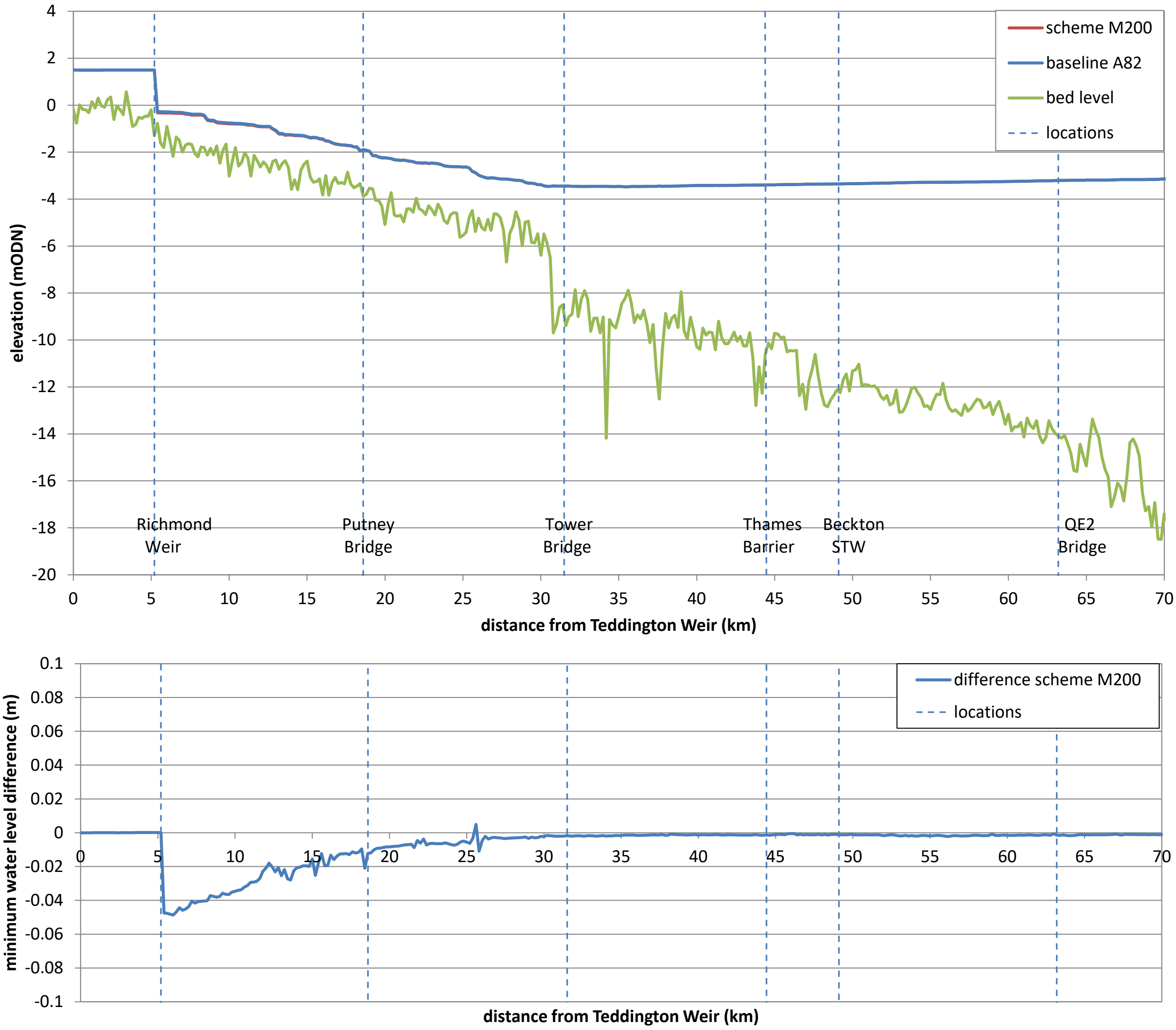
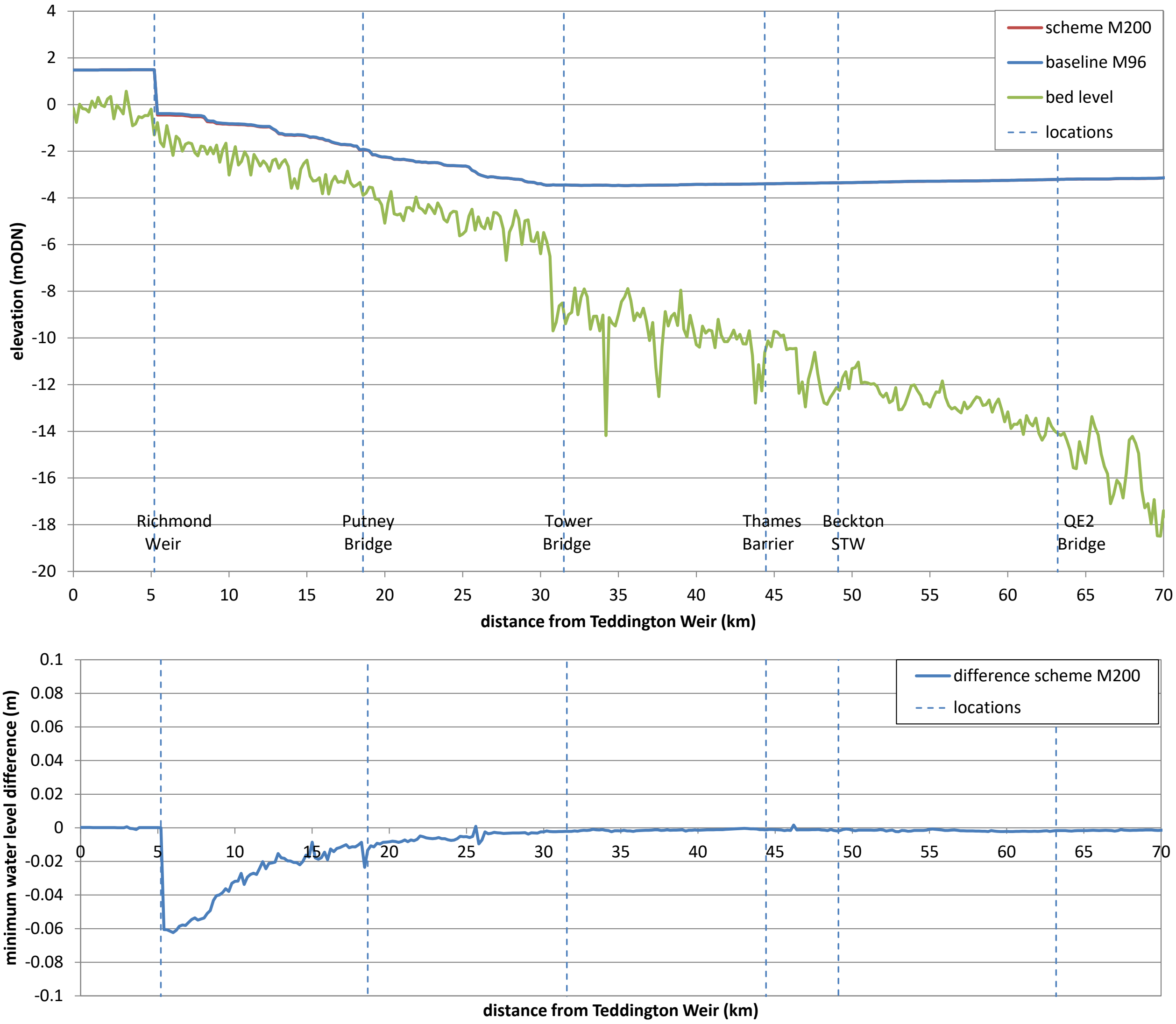


Figure 4-6 Minimum water level along the Thames Tideway thalweg during M96 river flows for reference condition and 200 Ml/d Mogden water recycling scheme during 1 August to 30 November period of operation.



These show the greatest reduction in low tide water level of 5cm for A82 and 6cm for M96 – with greatest effect centred around Isleworth Ait, and no effect extending into the Richmond Pound at times of operation of the Richmond half-tide sluice in all months except November.

In order to understand how water surface elevation varied on a diurnal basis during a complete spring-neap-spring tidal cycle, model data for the A82 and M96 baselines and the associated 200 Ml/d Mogden water recycling scheme between 15 October and 1 November, extracted from a point in the estuary immediately downstream of Richmond Sluice, were plotted alongside baseline-scheme changes. These data are presented in Figure 4-7 for the A82 scenarios and Figure 4-8 for M96 scenarios.

Figure 4-7 Diurnal water surface elevation and change for A82 baseline and 200 Ml/d Mogden water recycling scheme between 15 October and 1 November

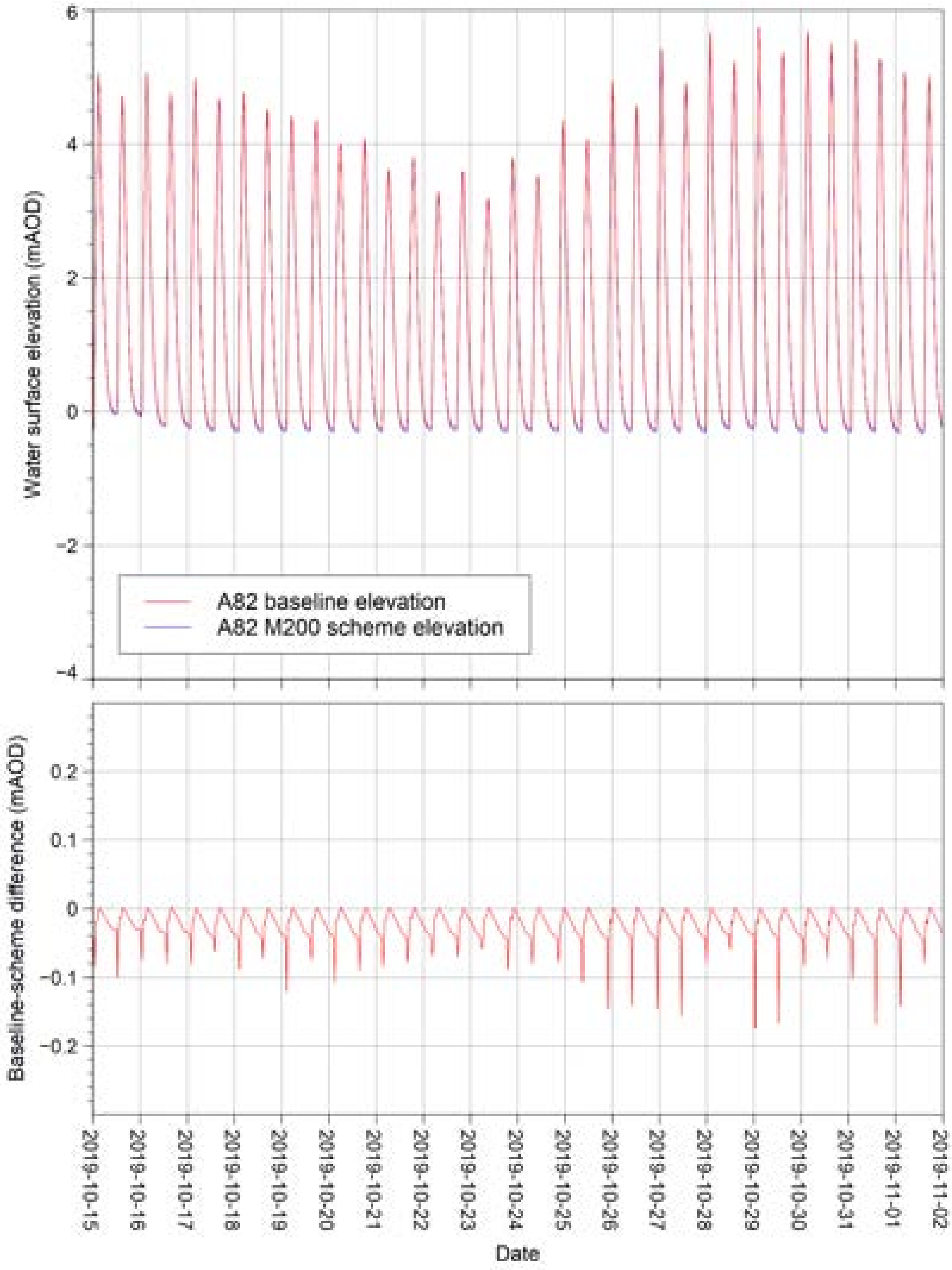
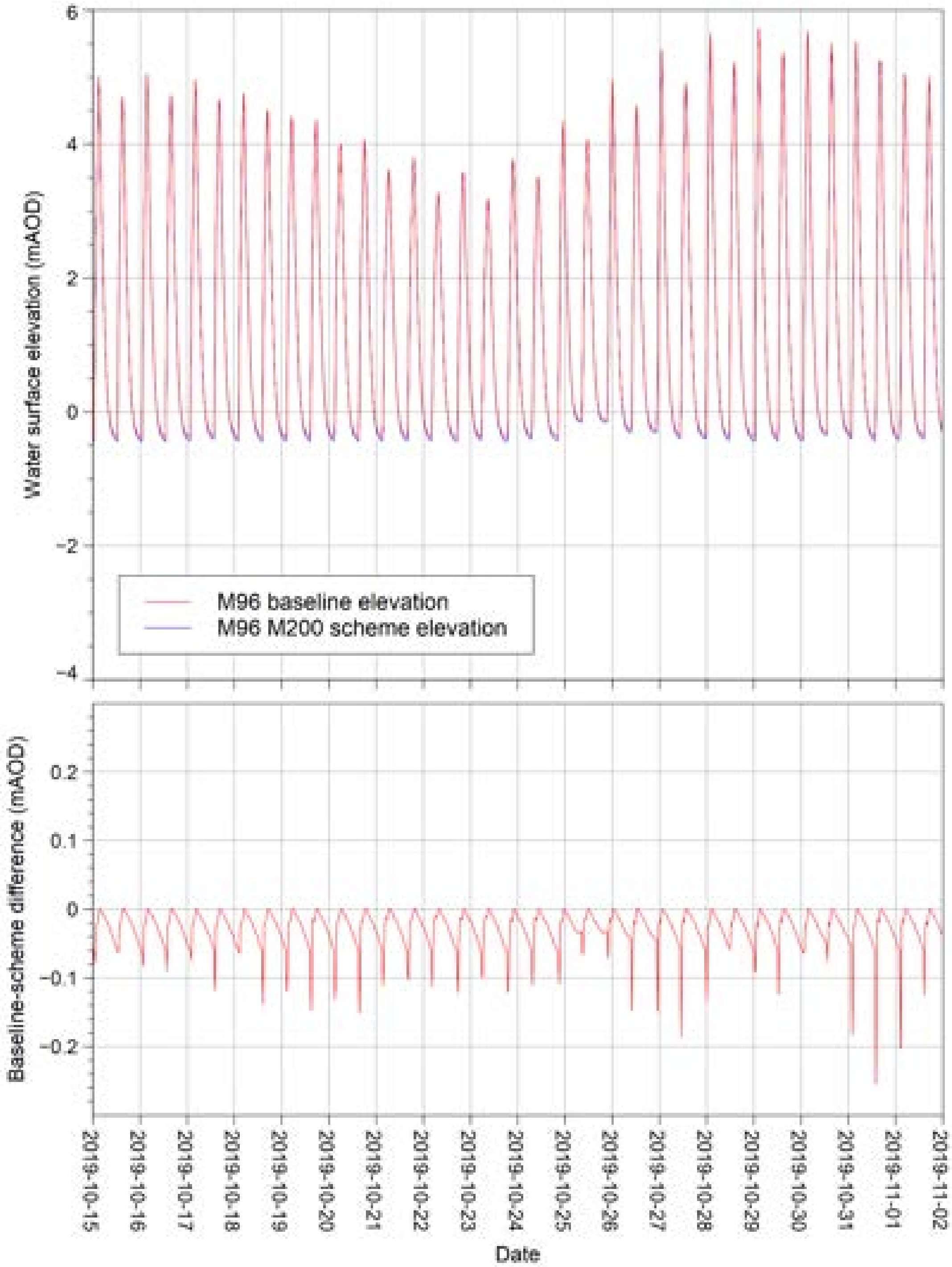


Figure 4-8 Diurnal water surface elevation and change for M96 baseline and 200 Ml/d Mogden water recycling scheme between 15 October and 1 November



The data for A82 and M96 baseline and scheme water surface levels show that all of the change in level with the scheme is associated with the lowest water levels at low tide. For the A82 scheme, differences in water surface level between the baseline and scheme are around 0.1mAOD lower, sometimes being up to 0.19mAOD lower. For M96, the level is slightly lower at around 0.12mAOD, sometimes up to 0.23mAOD lower.

4.3. REVIEW OF MOGDEN WATER RECYCLING OUTFALL DESIGN INCLUDING SCREENING

4.3.1. Overview

In accordance with the approach set out in in Table 1-1, the change in velocity pattern at the Mogden water recycling outfall has been assessed through 3D modelling.

4.3.2. Mogden water recycling Outfall in the Freshwater River Thames

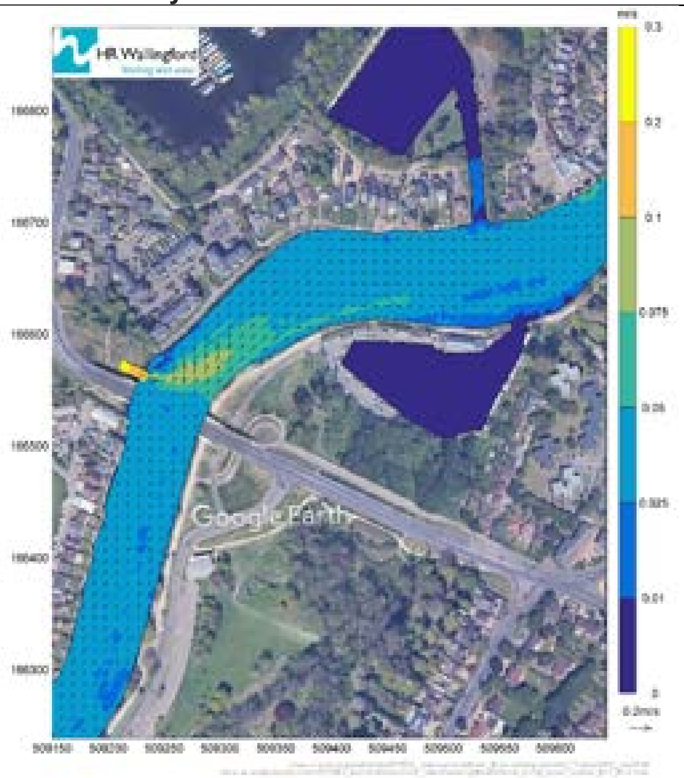
The effects on the hydrodynamics of the River Thames around the location of the proposed Mogden water recycling outfall at Walton have been simulated using a 3D TELEMAC model. The modelling uses the scenarios (Appendix 1 Section 3.2): 600 Ml/d river flow (Scenario 1), 780 Ml/d river flow (Scenario 2) and 950 Ml/d river flow (Scenario 3), with an outfall discharge of 200 Ml/d moving at 0.3m/s discharged at a 90˚ angle to the riverbank. The result of the model runs for each scenario are presented below. The baseline model flow velocity predictions are outlined in Appendix 1 Section 3.2.

Scenario 1: 600 Ml/d river flow, extremely low river flow conditions

The depth-average velocity around the Walton outfall under Scenario 1 conditions and the velocity differences between this and the baseline are presented in Figure 4-9.

Figure 4-9 Depth-average velocity at Walton outfall, 600 Ml/d, Scenario 1 (200 Ml/d outfall discharge)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

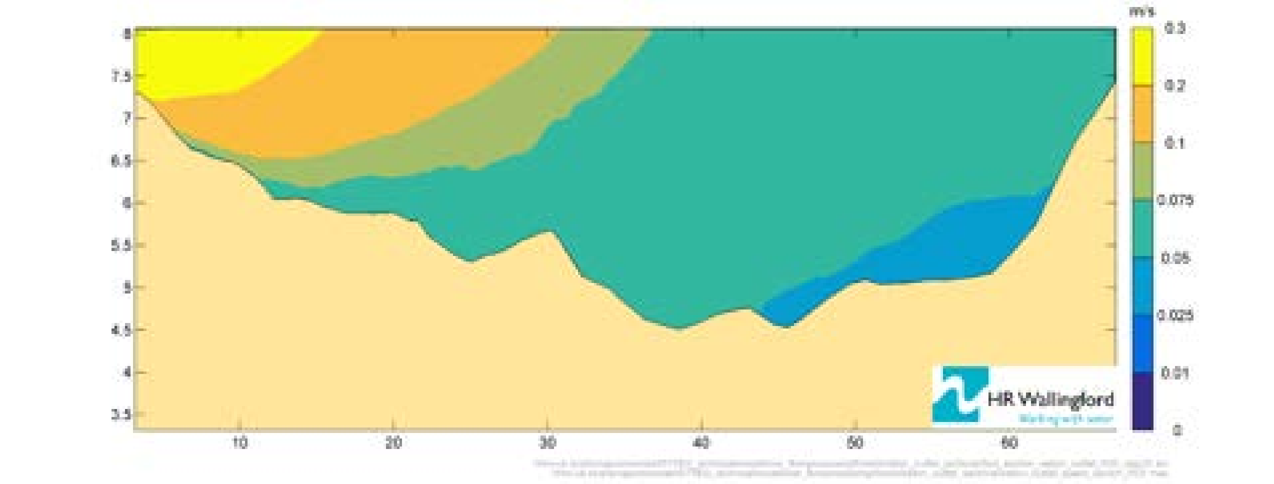
Under the developed Scenario 1 simulation the discharge leads to a plume which increases flow velocities in channel by 0.05-0.1m/s. The area of increased velocity is spatially restricted to the outfall area and in the thalweg. The model suggests the increased velocity of the plume rapidly declines by ~150m downstream of the discharge, with remaining flow velocities in the channel range from 0.025-0.05m/s, similar to upstream of the proposed discharge point, although a small tongue of higher velocities (0.05-0.075m/s) persists for ~250m downstream on the right bank. With the exception of the outfall, most velocity vectors remain in a downstream direction, although there is some localised flow movement towards the left bank downstream of the outfall.

The modelled difference shows that velocities in the majority of the channel downstream of the outfall increase by 0.005-0.05m/s, with higher velocities between 0.05-0.3m/s in very close proximity to the outfall. There are some areas of reduced velocity (-0.01 - -0.05m/s) immediately upstream and downstream of the outfall, particularly between the bank the plume.

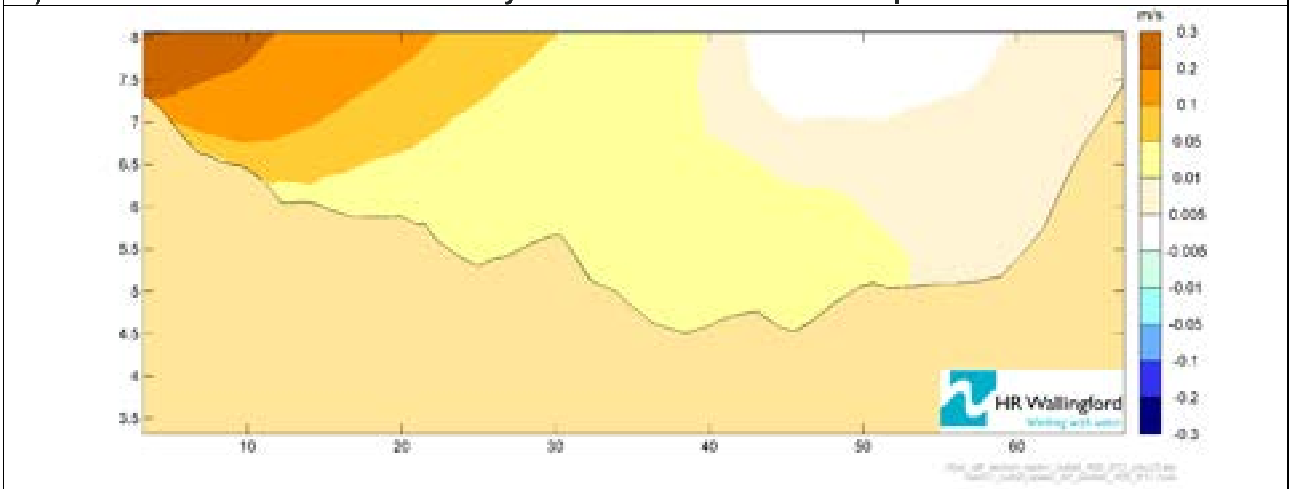
Figure 4-10 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Walton outfall under the Scenario 1 flows.

Figure 4-10 Depth-average velocity at Walton outfall cross-section, 600 Ml/d, Scenario 1 (200 Ml/d outfall discharge)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed



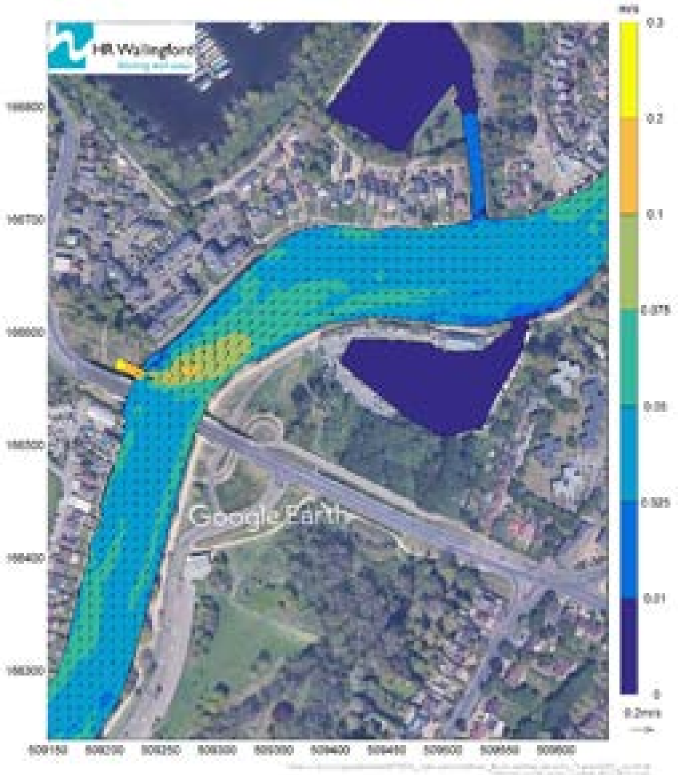
The difference data show that across most of the cross-section there is a velocity gradient, reducing from 0.2-0.3m/s close to the outfall to 0.01-0.05m/s and lower for the channel cross-section from around 20m away from the outfall. It should be noted that higher velocities of the outfall discharge are concentrated towards the surface of the river, the higher velocities rapidly declining with depth towards the channel bed.

Scenario 2: 780 Ml/d river flow, extremely low river flow conditions

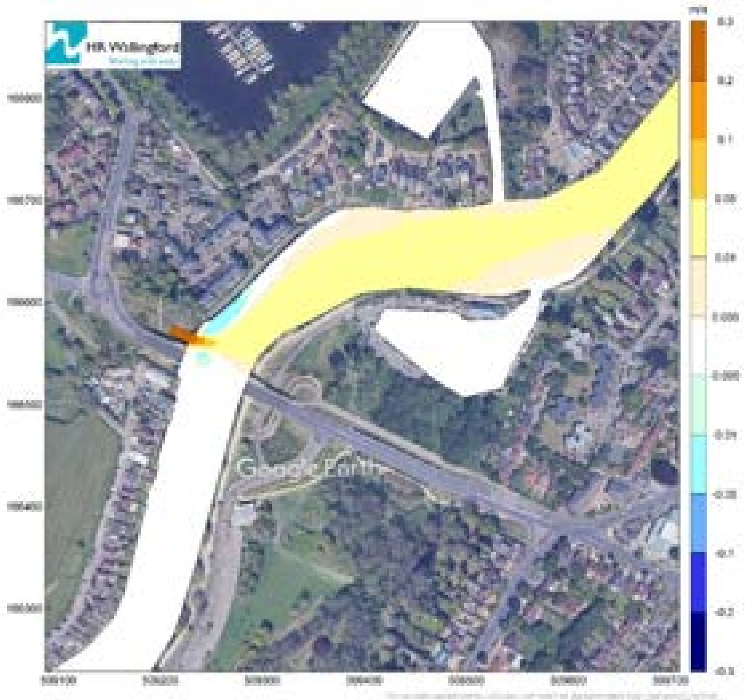
The depth-average velocity around the Walton outfall under Scenario 2 conditions and the velocity differences between this and the baseline are presented in Figure 4-11.

Figure 4-11 Depth-average velocity at Walton outfall, 780 Ml/d, Scenario 2 (200 Ml/d outfall discharge)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed



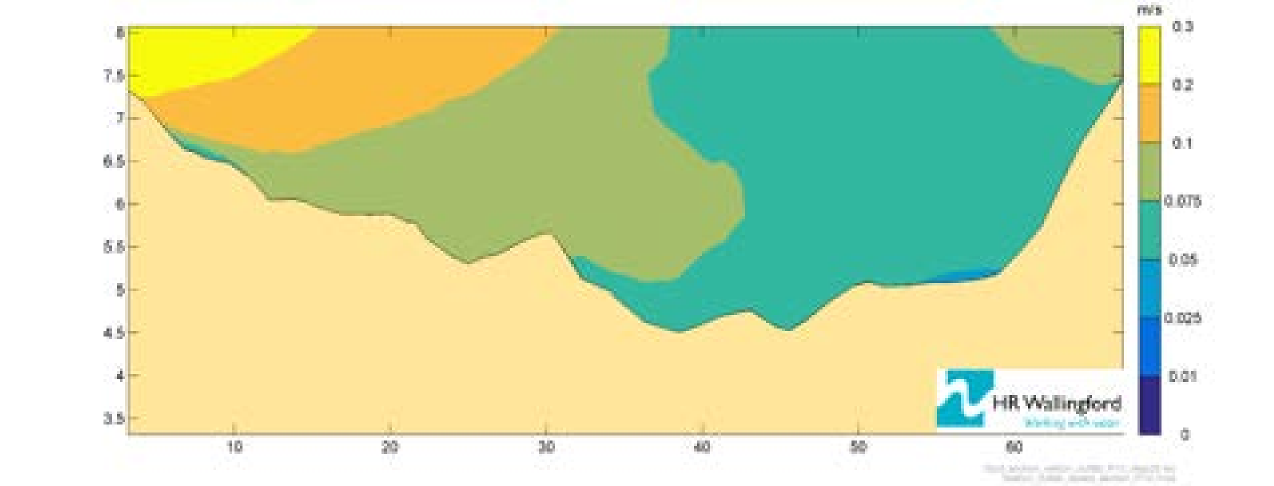
The spatial distribution and magnitude of channel velocities under the Scenario 2 run are similar to the Scenario 1 run. The discharge plume is present, with flow velocities in channel of ~0.05-0.1m/s, with slightly higher peak velocities of ~0.1-0.2m/s towards the centre of the channel adjacent to the outfall. The area of increased velocities (0.05-0.075m/s) also stretches further downstream to around 250m. With the exception of the outfall, most velocity vectors remain in a downstream direction, although there is some localised flow movement towards the left bank downstream of the outfall.

The modelled difference shows that velocities in the majority of the channel downstream of the outfall increase by 0.005-0.05m/s, with higher velocities between 0.05-0.3m/s in very close proximity to the outfall. There are only limited velocity increases (0.005-0.01m/s) around the left and right banks downstream of the outfall. As for Scenario 1, there are spatially limited areas of reduced velocity (-0.01 - -0.05m/s) immediately upstream and downstream of the outfall, particularly between the bank and the plume. These appear to be larger in extent when compared to the Scenario 1 run.

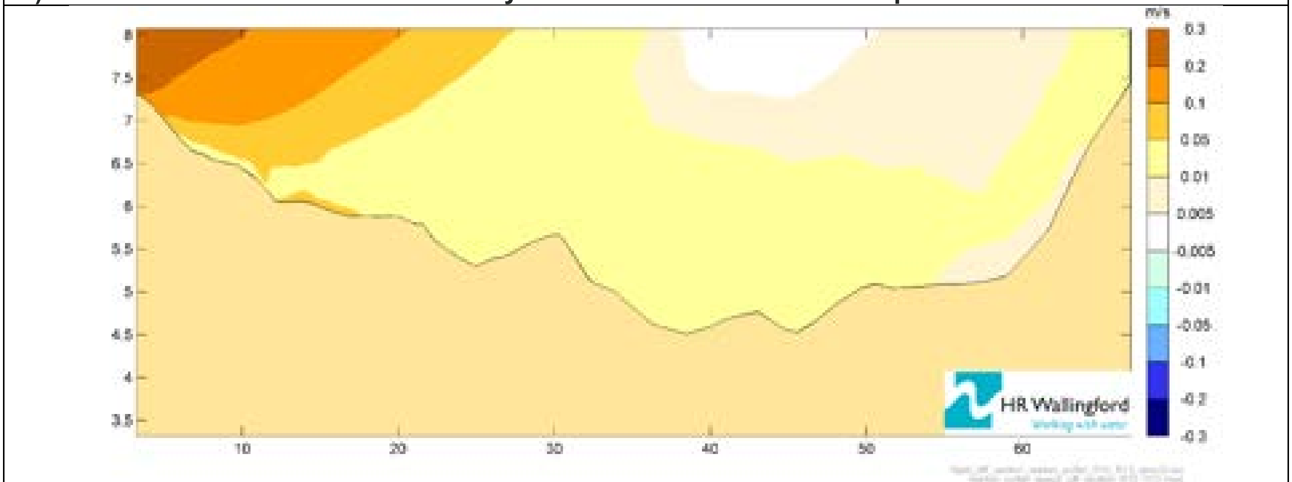
Figure 4-12 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Walton outfall under the Scenario 2 flows.

Figure 4-12 Depth-average velocity at Walton outfall cross-section, 780 Ml/d, Scenario 2 (200 Ml/d outfall discharge)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed



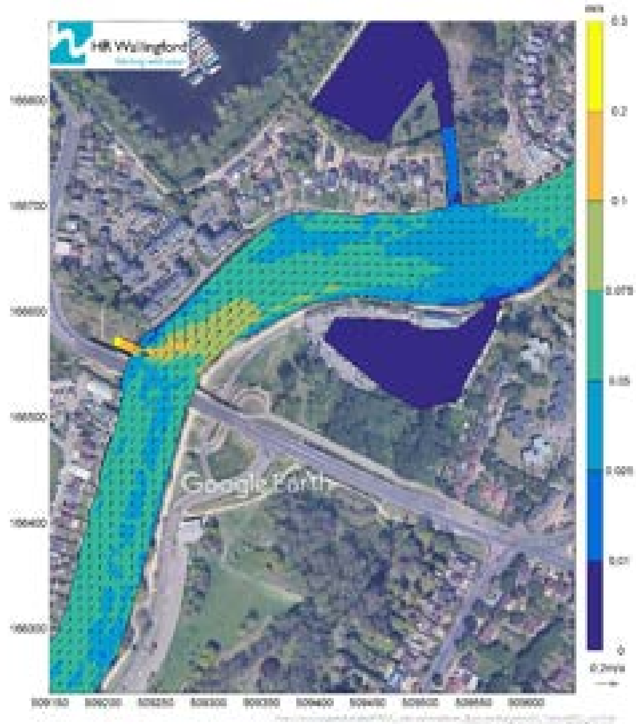
The difference data show that across most of the cross-section there is a velocity gradient, reducing from 0.2-0.3m/s close to the outfall to 0.01-0.05m/s and lower for the channel cross-section from around 25m away from the outfall. It should be noted that higher velocities of the outfall discharge are concentrated towards the surface of the river. The higher velocities decline rapidly with depth towards the channel bed, however, when comapred to Scenario 1, there is a slight increase in bed velocities of 0.05-0.1m/s noted around 15m from the outfall.

Scenario 3: 950 Ml/d river flow, low river flow conditions

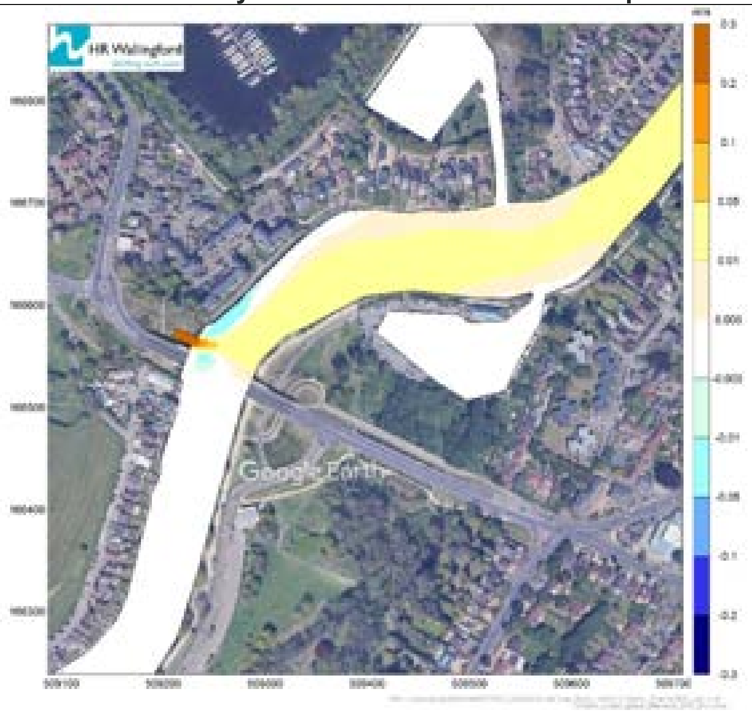
The depth-average velocity around the Walton outfall under Scenario 3 conditions and the velocity differences between this and the baseline are presented in Figure 4-13.

Figure 4-13 Depth-average velocity at Walton outfall, 950 Ml/d, Scenario 3 (200 Ml/d outfall discharge)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed



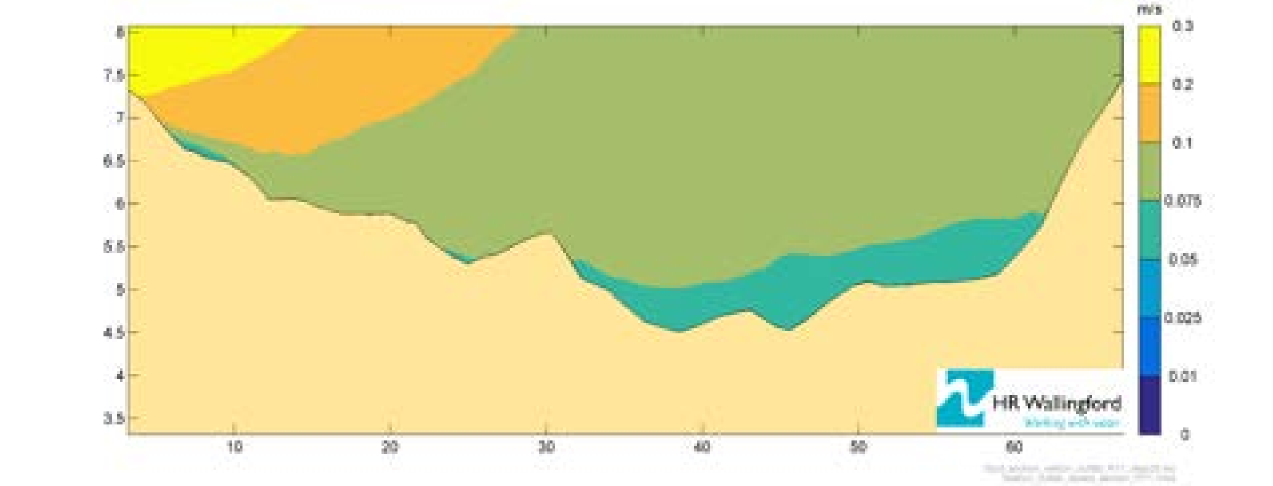
The spatial distribution and magnitude of channel velocities under the Scenario 3 run are similar to the Scenario 2 run. The discharge plume is present, with flow velocities in channel of ~0.05-0.1m/s, with slightly higher peak velocities of ~0.1-0.2m/s towards the centre of the channel adjacent to the outfall. The area of increased velocities (0.05-0.075m/s) also stretches further downstream to around 260m. For this scenario there are notably higher velocities adjacent to the left bank (0.05-0.075m/s) than for previous runs. With the exception of the outfall, most velocity vectors remain in a downstream direction, although there is some localised flow movement towards the left bank downstream of the outfall.

The modelled difference shows that velocities in the majority of the channel downstream of the outfall increase by 0.005-0.05m/s, with higher velocities between 0.05-0.3m/s in very close proximity to the outfall. In comparison to Scenario 2, velocities on the left bank decline slightly to -0.005 – 0.005m/s, although velocities are slightly higher (0.005-0.01m/s) on the left and right banks downstream of the outfall. As for Scenario 2, there are spatially limited areas of reduced velocity (-0.01 - -0.05m/s) immediately upstream and downstream of the outfall, particularly close to the bank between the plume, with the downstream area being larger in extent than for Scenario 2.

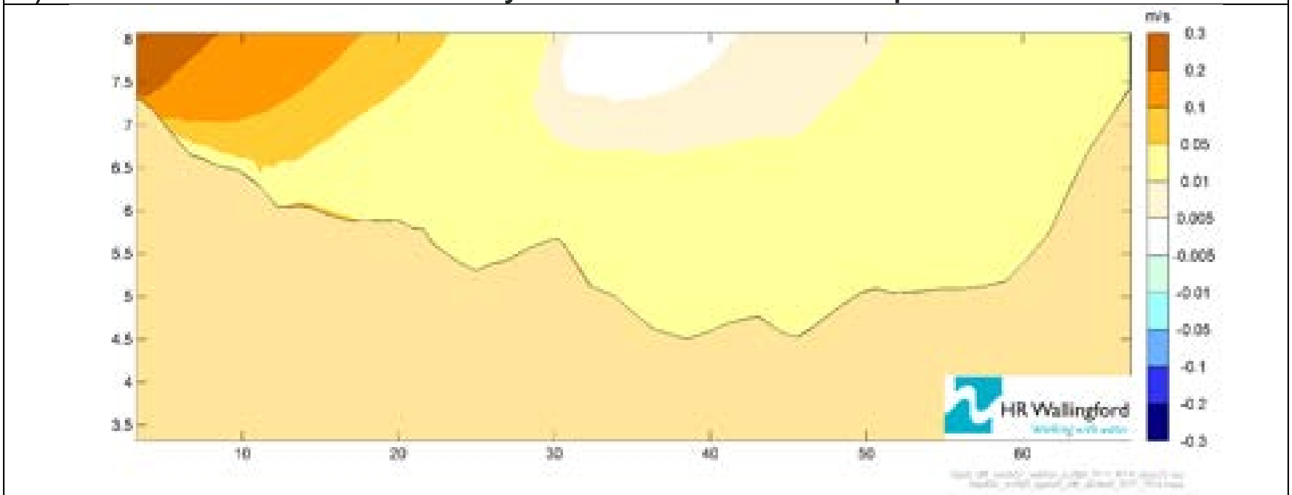
Figure 4-14 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Walton outfall under the Scenario 3 flows.

Figure 4-14 Depth-average velocity at Walton outfall cross-section, 950 Ml/d, Scenario 3 (200 Ml/d outfall discharge)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed



The difference data show that across most of the cross-section there is a velocity gradient, reducing from 0.2-0.3m/s close to the outfall to 0.01-0.05m/s and lower for the channel cross-section from around 25m away from the outfall. The higher velocities of the outfall discharge are concentrated towards the surface of the river and decline rapidly with depth towards the channel bed. The slight increase in bed velocities of 0.05-0.1m/s noted for Scenario 2 are reduced for Scenario 3 (likely due to the effect of the higher incipient river flows).

4.4. WETTED HABITAT CHANGE IN FRESHWATER RIVER THAMES AND ESTUARINE THAMES TIDEWAY

4.4.1. Overview

HR Wallingford[4] have produced a report presenting model outputs from simulations of a proposed outfall at Walton on the River Thames. Modelling included three scenarios:

- Scenario 1: 600 Ml/d river flow, discharge; representative of extremely low flow conditions at Walton Bridge of Q99.5 (based on gauged flow data)
- Scenario 2: 780 Ml/d river flow, discharge; representative of extremely low flow conditions at Walton Bridge of Q97 (based on gauged flow data)
- Scenario 3: 950 Ml/d river flow, discharge; representative of low flow conditions at Walton Bridge of Q90 (based on gauged flow data)

In all scenarios the discharge was simulated as 200 Ml/d with an exit velocity of 0.3 m/s as a 'developed' output, 'baseline' simulations, without discharge, were also conducted to assess the impact on hydrodynamics. Thames Water's abstractions at Walton and Hampton were included providing an additional combined abstraction of 200 Ml/d to scenarios. Results from the modelling are presented below for Sunbury Weir pool and Molesey Weir pool.

4.4.2. Sunbury Weir pool

Flow velocity data were extracted for three cross-sections downstream of Sunbury Weir as part of the Sunbury weir pool assessment. The locations of these three cross-sections are presented in Appendix 1 Section 4.2 (Figure A7) along with an assessment of the baseline flow velocities. The results of modelling for the three flow scenarios (outlined in Appendix 1 Section 4.2 and above in Section 4.4.1) and the changes in flow velocity between the baseline and modelled conditions are presented below.

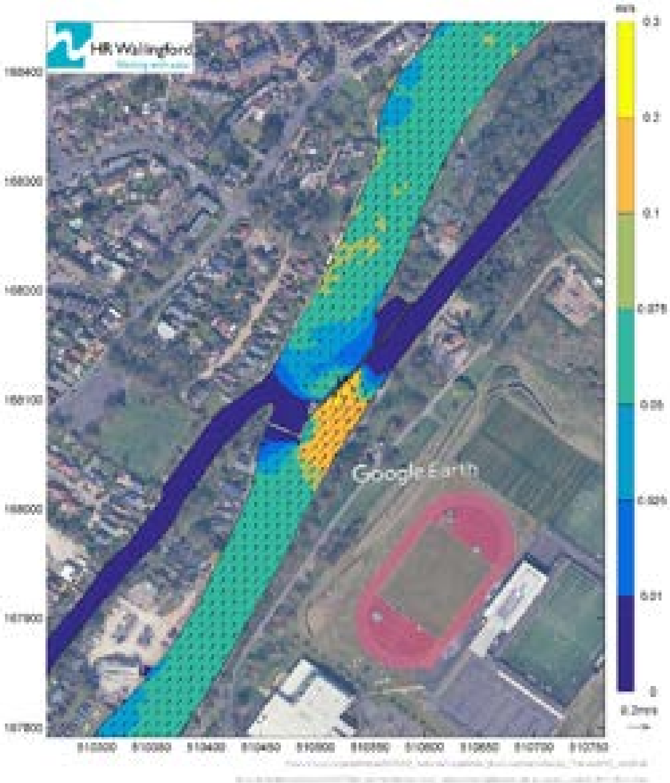
Scenario 1: 600 Ml/d river flow, extremely low river flow conditions

The depth-average velocity for Sunbury Weir under Scenario 1 conditions and the velocity differences between this and the baseline are presented in Figure 4-15.

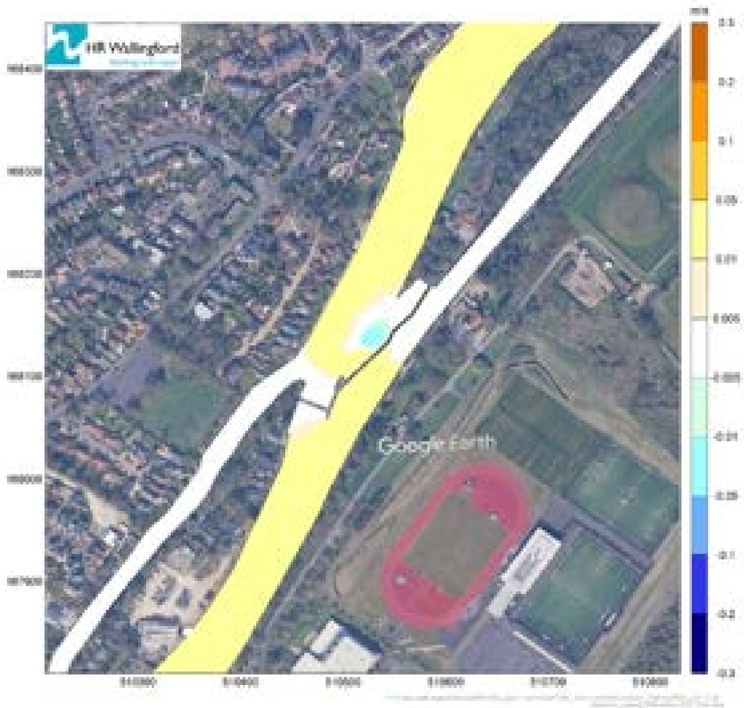
[4] HR Wallingford (2022). DER6575 London Reuse SRO - West London Options: Modelling for Gate 2 - Walton discharge

Figure 4-15 Depth-average velocity at Sunbury weir, 600 Ml/d, Scenario 1

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

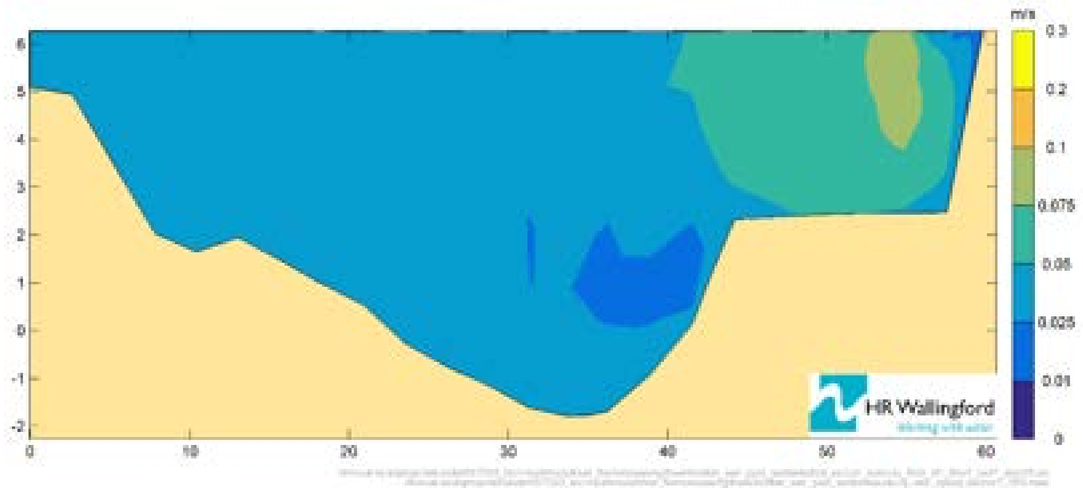


Under the developed Scenario 1 simulation only a minimal change of between 0.01-0.05m/s is expected both upstream and downstream of Sunbury Weir under Scenario 1 conditions. A small and spatially localised reduction of -0.01- -0.05m/s is predicted to the right of the weir pool downstream of the weir.

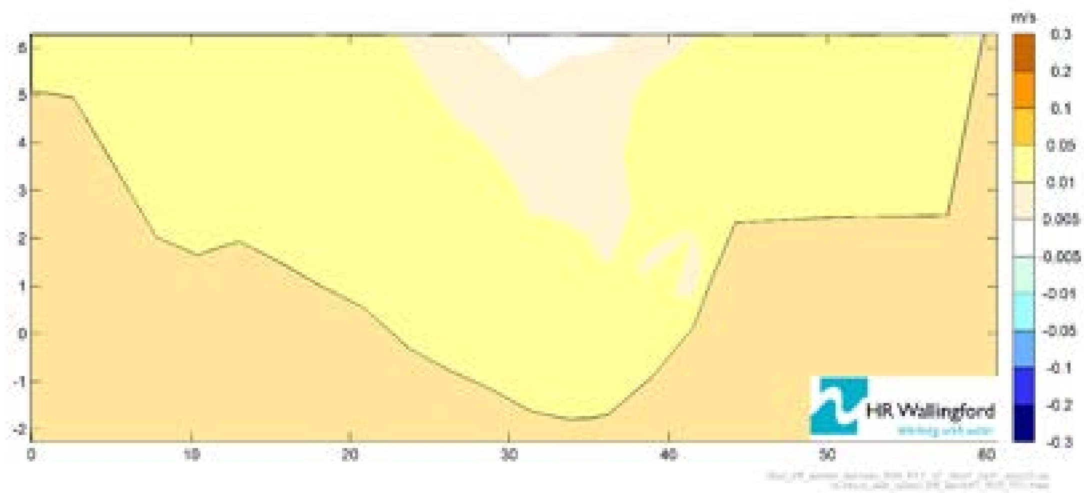
Figure 4-16 shows modelled changes in flow velocity for cross-section 1 under the Scenario 1 flows.

Figure 4-16 Section 1 flow velocities at Sunbury weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

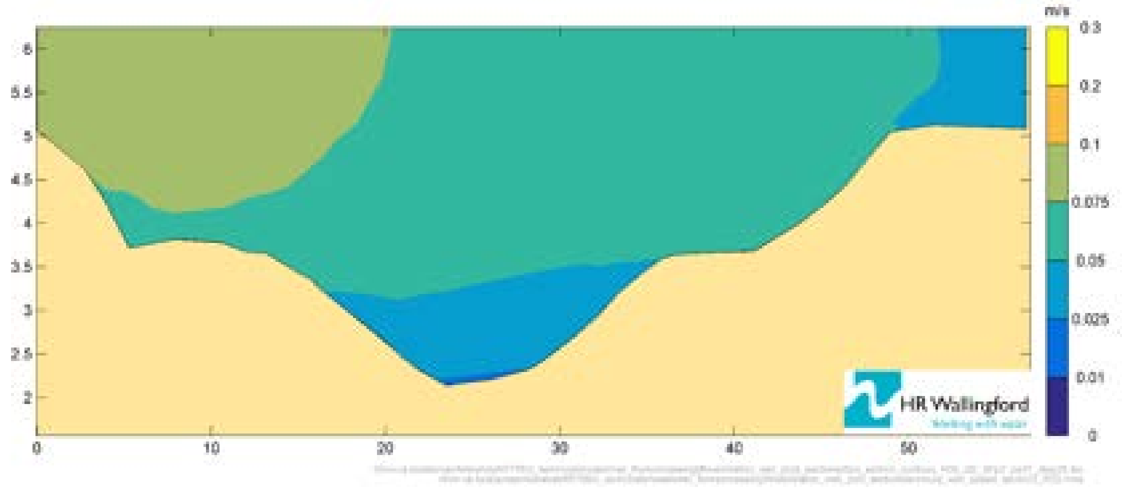


The difference data show that across most of the cross-section there is a maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs. A smaller increase of 0.005-0.01m/s is noted in the centre of the channel above the weir pool.

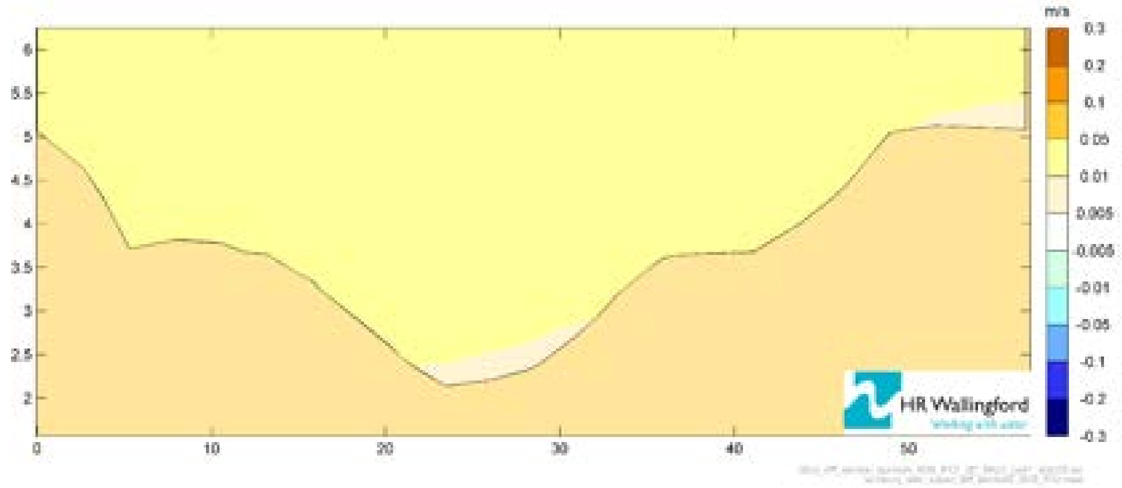
Figure 4-17 shows modelled changes in flow velocity for cross-section 2 under the Scenario 1 flows.

Figure 4-17 Section 2 flow velocities at Sunbury weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

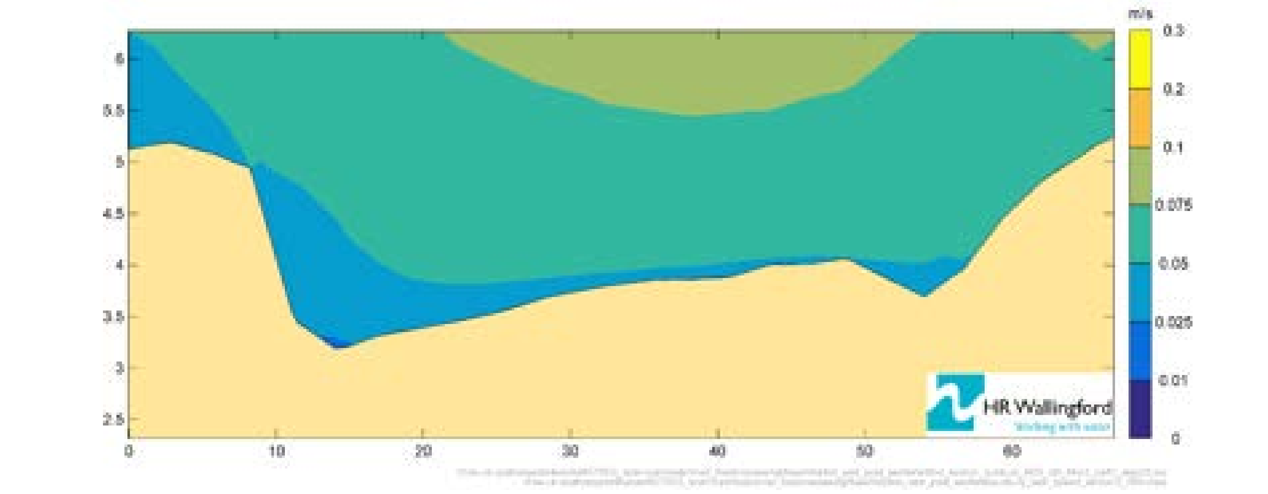


The difference data show that across most of the cross-section there is maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs across the majority of the cross-section. There are some spatially limited increases of 0.005-0.01m/s noted at the channel bed at the centre and the right bank.

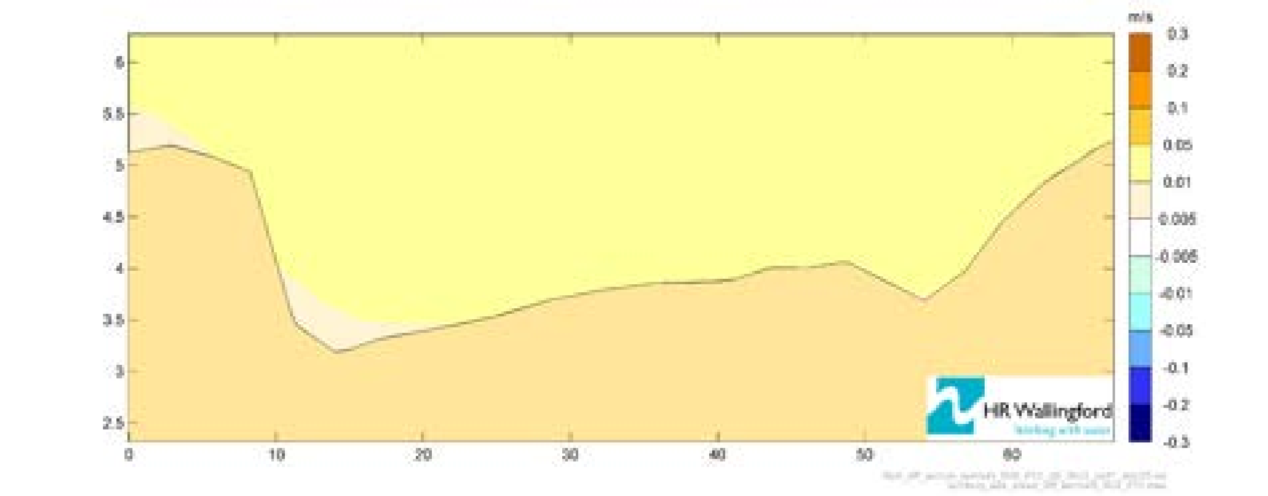
Figure 4-18 shows modelled changes in flow velocity for cross-section 3 under the Scenario 1 flows.

Figure 4-18 Section 3 flow velocities at Sunbury weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed



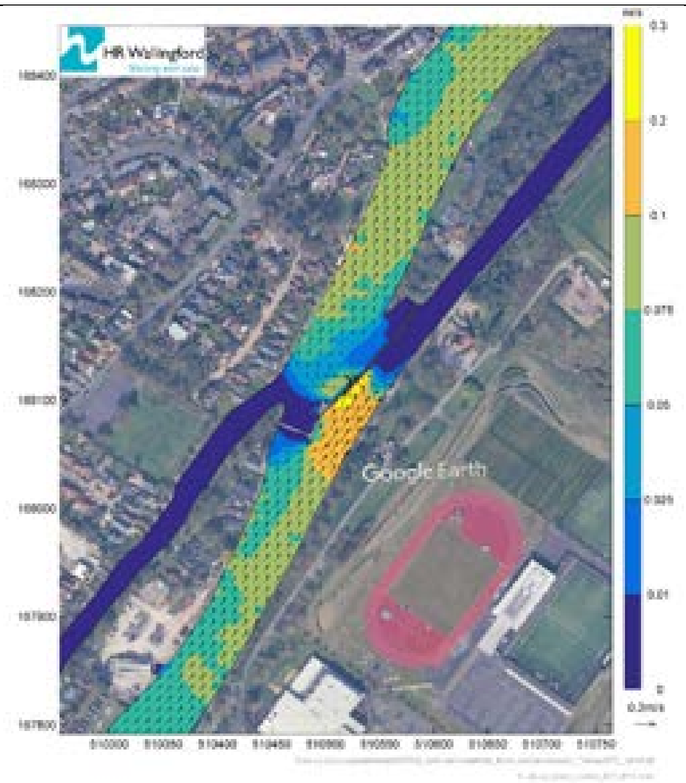
The difference data show that across most of the cross-section there is maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs across the majority of the cross-section. There are some spatially limited increases of 0.005-0.01m/s noted at the channel bed towards and at the left bank.

Scenario 2: 780 Ml/d river flow, extremely low river flow conditions

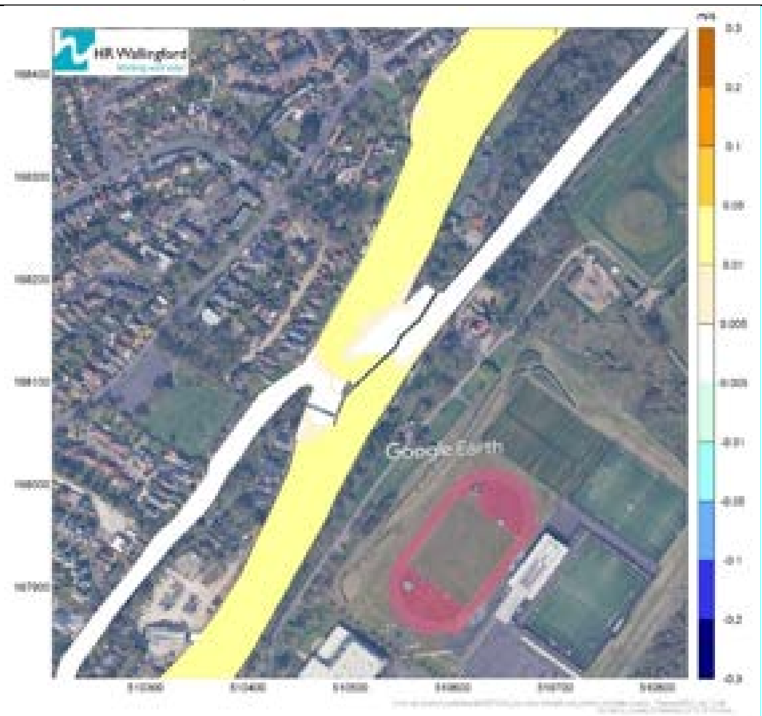
The depth-average velocity for Sunbury Weir under Scenario 2 conditions and the velocity differences between this and the baseline are presented in Figure 4-19.

Figure 4-19 Depth-average velocity at Sunbury weir, 780 Ml/d, Scenario 2

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

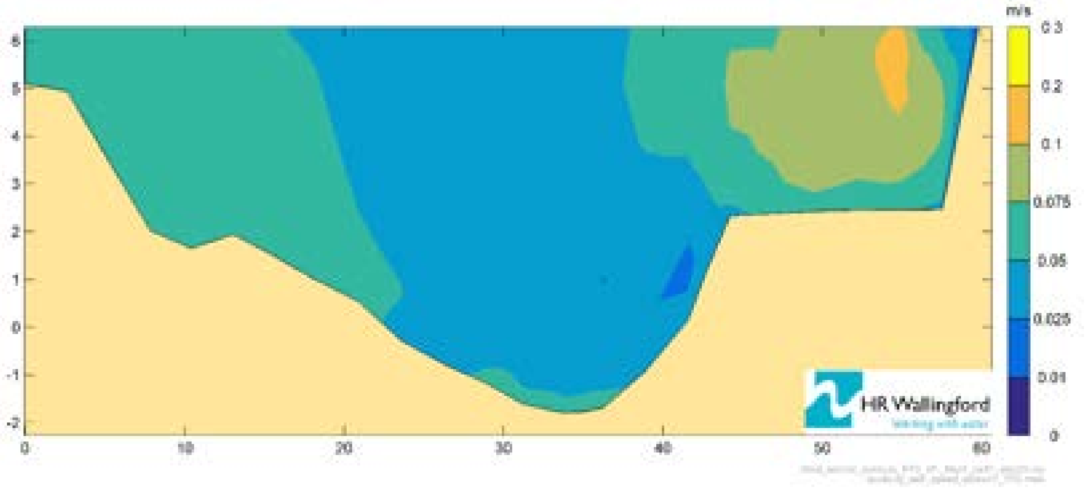


Under the developed Scenario 2 simulation only a minimal change of between 0.01-0.05m/s is expected both upstream and downstream of Sunbury Weir. There is a change of between -0.005 – 0.005m/s predicted to the right of the weir pool downstream of the weir, slightly elevated when compared to Scenario 1.

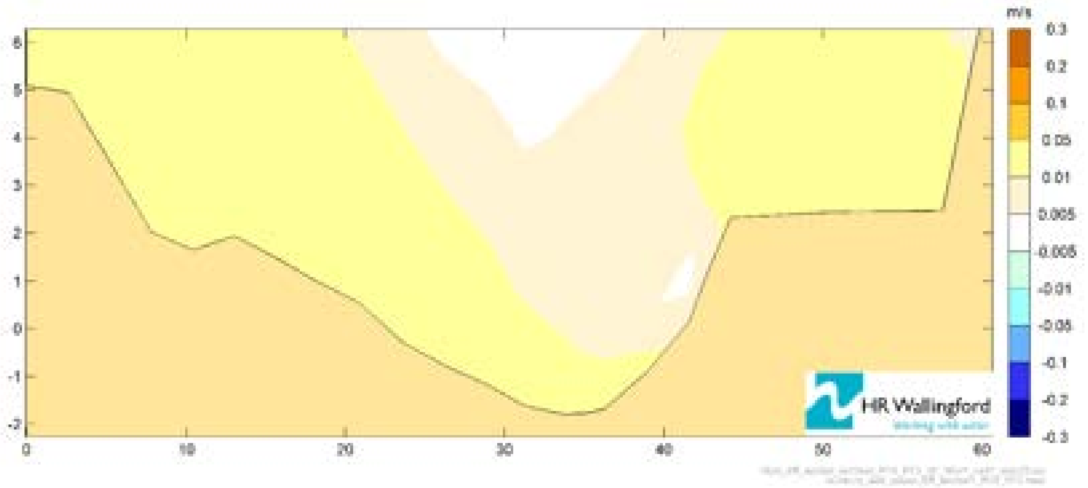
Figure 4-20 shows modelled changes in flow velocity for cross-section 1 under the Scenario 2 flows.

Figure 4-20 Section 1 flow velocities at Sunbury weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

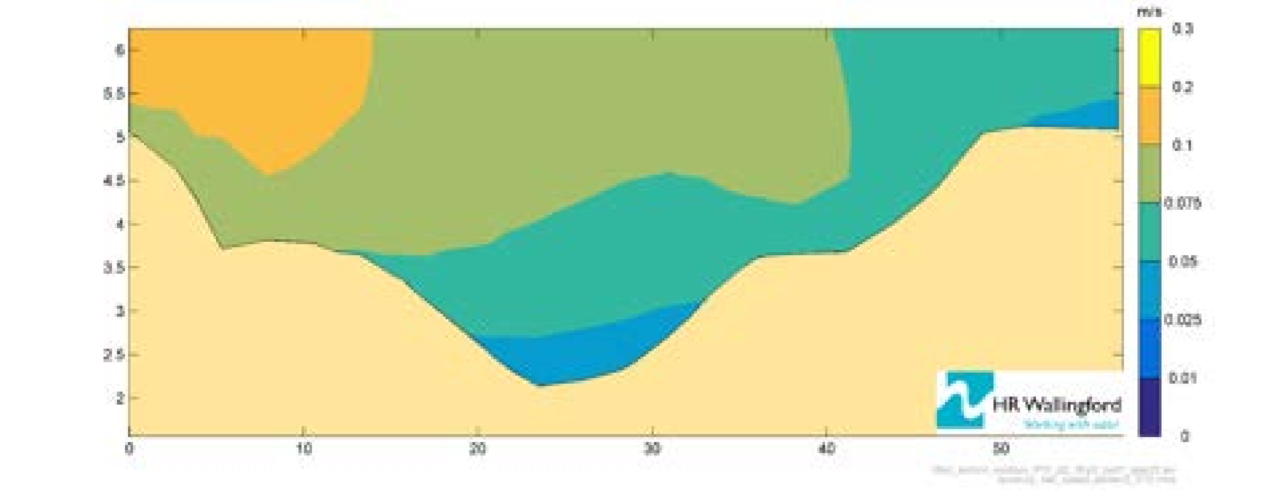


The difference data show that across most of the cross-section there is a maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs, mostly around the left and right sides of the channel. A smaller increase of 0.005-0.01m/s is noted predominantly within the centre of the channel above the weir pool.

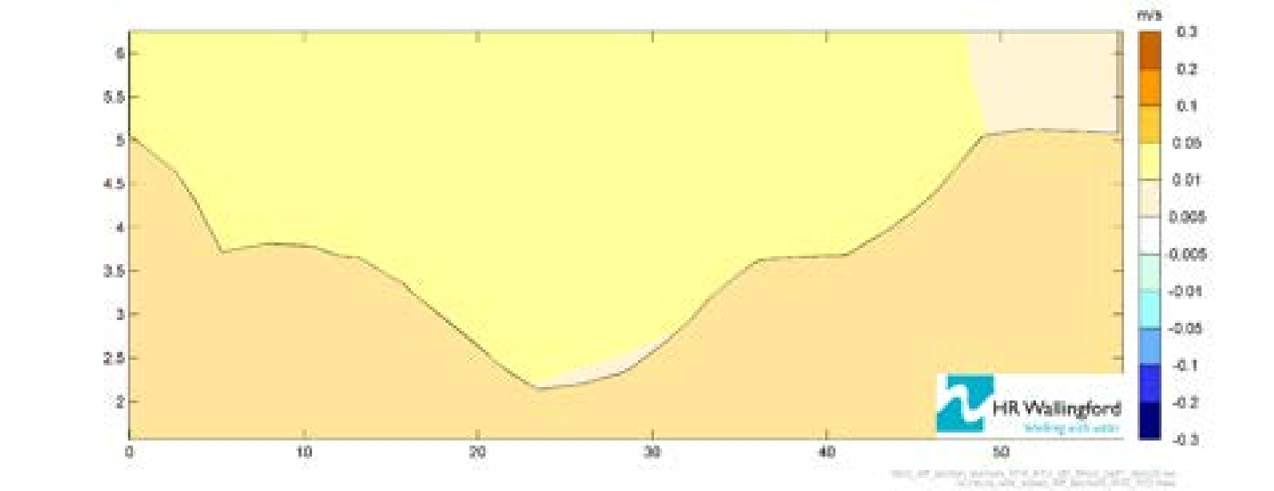
Figure 4-21 shows modelled changes in flow velocity for cross-section 2 under the Scenario 2 flows.

Figure 4-21 Section 2 flow velocities at Sunbury weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

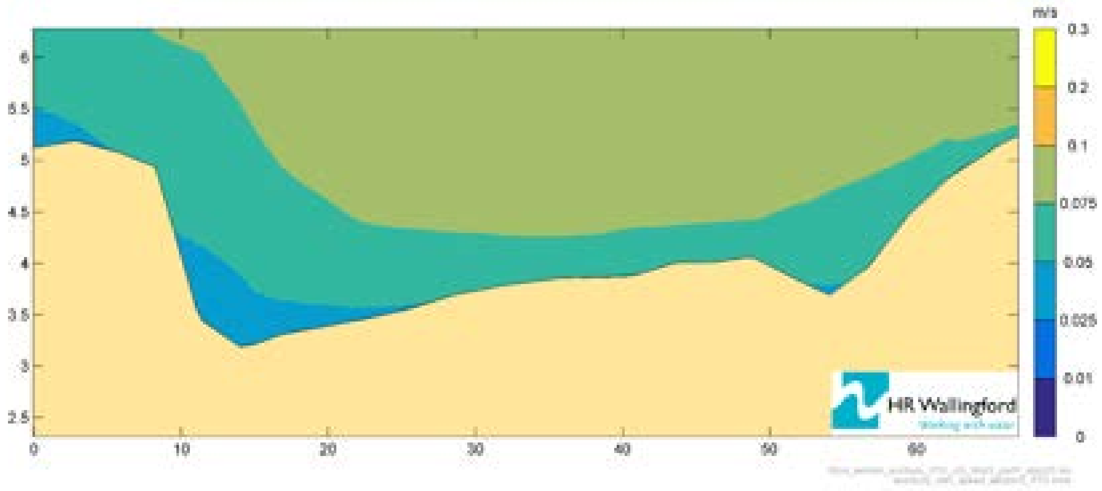


The difference data show that across most of the cross-section there is a maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs. A smaller increase of 0.005-0.01m/s is noted predominantly at the channel margins on the right bank and a very smal area of change in the centre of the channel just above the channel bed.

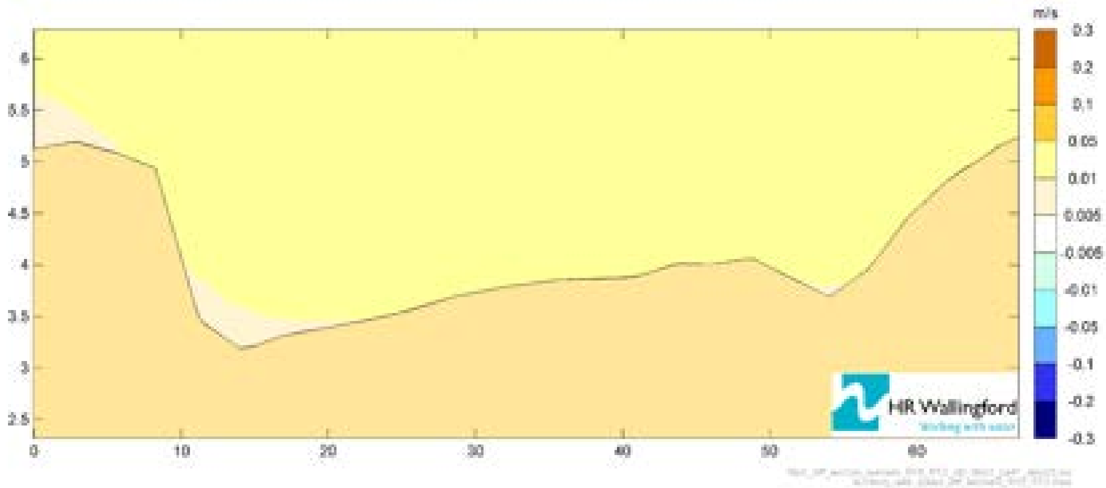
Figure 4-22 shows modelled changes in flow velocity for cross-section 3 under the Scenario 2 flows.

Figure 4-22 Section 3 flow velocities at Sunbury weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed



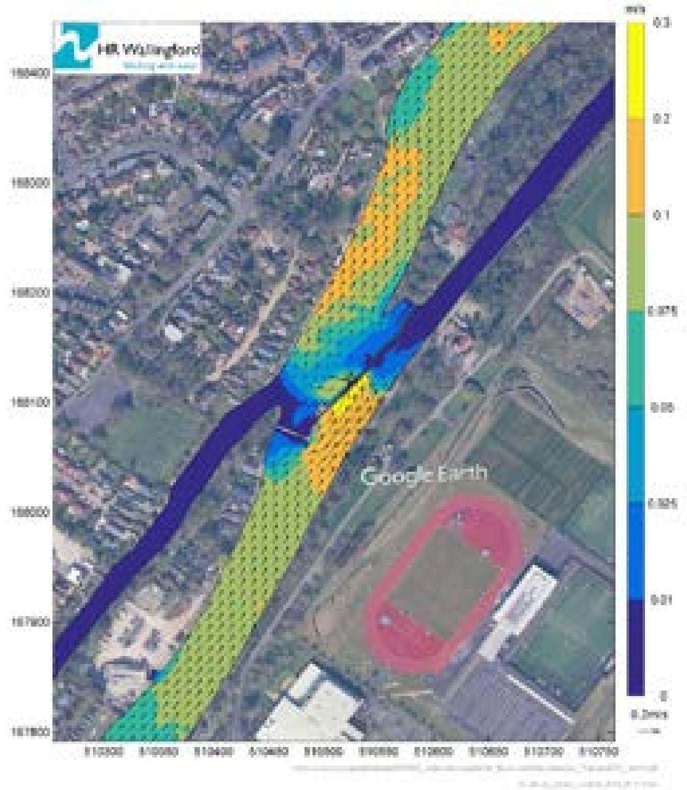
The difference data show that across most of the cross-section there is a maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs. A smaller increase of 0.005-0.01m/s is noted predominantly at the channel bed towards the left bank.

Scenario 3: 950 Ml/d river flow, low river flow conditions

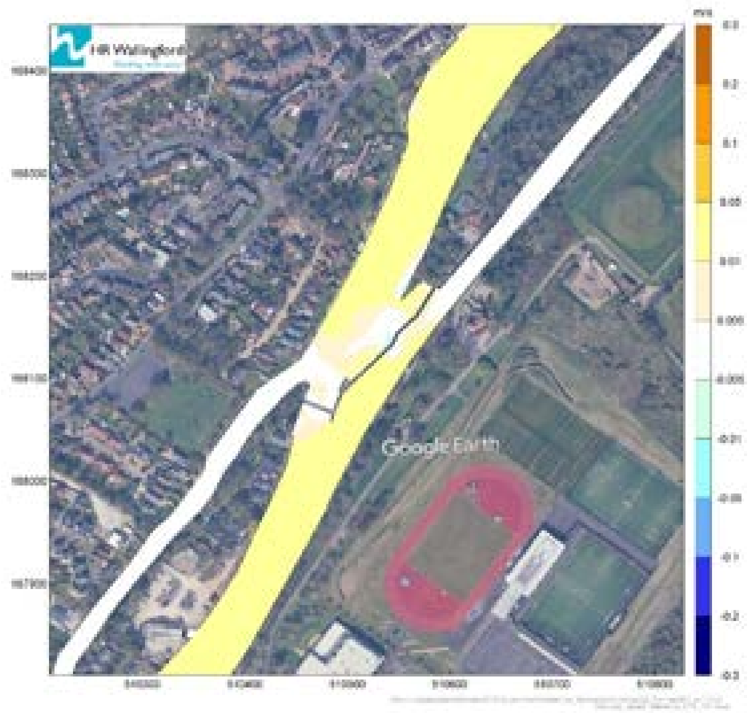
The depth-average velocity for Sunbury Weir under Scenario 3 conditions and the velocity differences between this and the baseline are presented in Figure 4-23.

Figure 4-23 Depth-average velocity at Sunbury weir, 950 Ml/d, Scenario 3

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

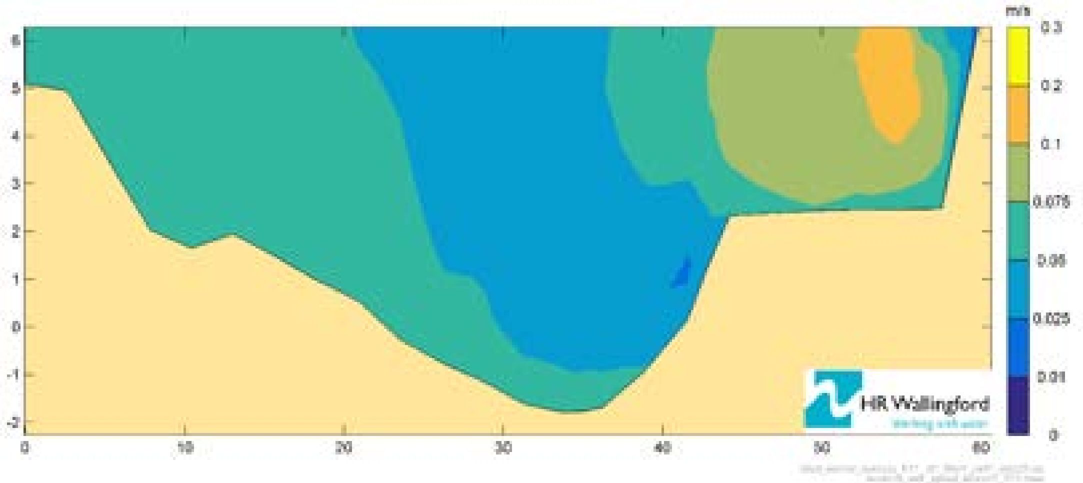


Under the developed Scenario 3 simulation only a minimal change of between 0.005-0.01m/s local and immediately downstream of the weir, with a predominant 0.01-0.05m/s increase both upstream and downstream of Sunbury Weir.

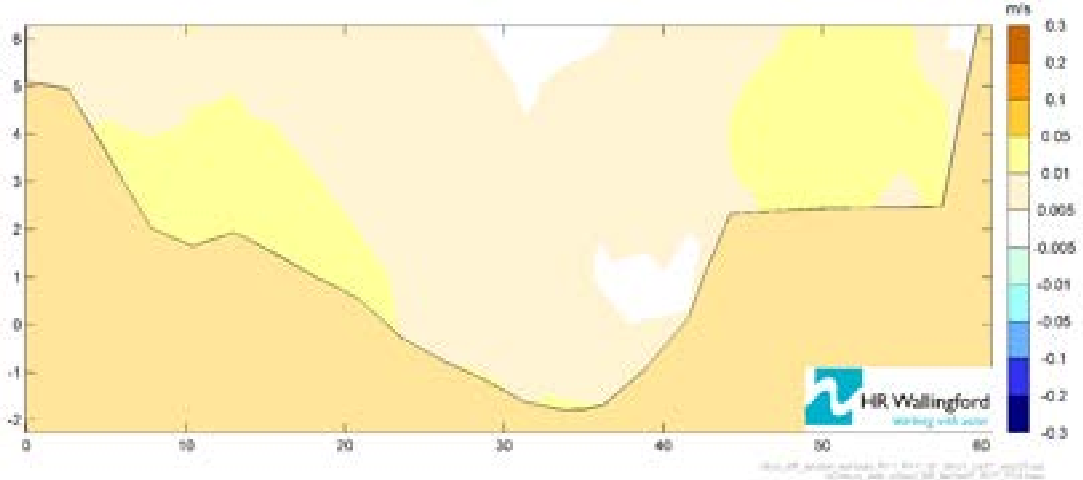
Figure 4-24 shows modelled changes in flow velocity for cross-section 1 under the Scenario 3 flows.

Figure 4-24 Section 1 flow velocities at Sunbury weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

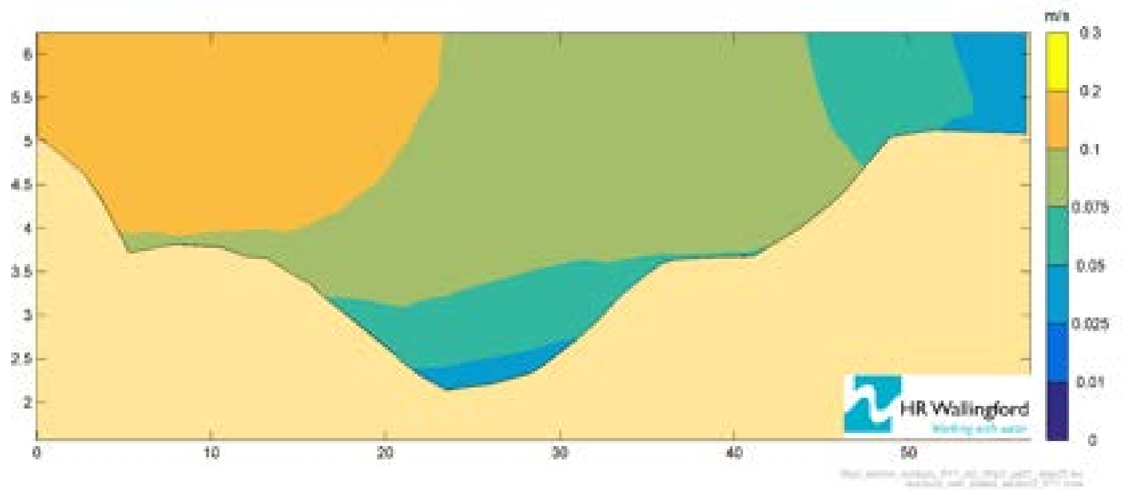


The difference data show that across most of the cross-section there is a 0.005-0.01m/s increase in flow velocity between the baseline and the developed runs, mostly around the centre and left bank of the channel. A larger increase of 0.01-0.05m/s is noted predominantly on the right bank and around the bed of the channel towards the left bank.

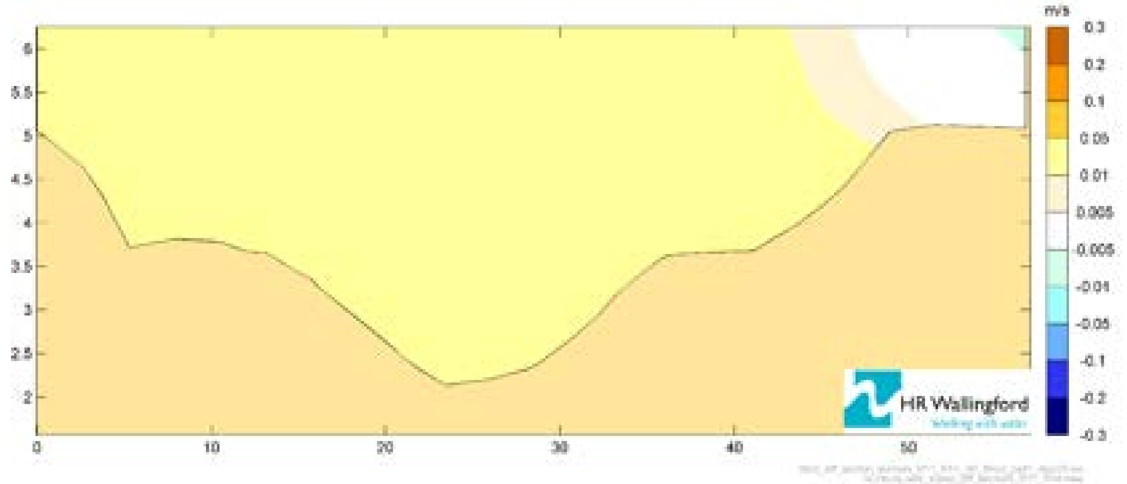
Figure 4-25 shows modelled changes in flow velocity for cross-section 2 under the Scenario 3 flows.

Figure 4-25 Section 2 flow velocities at Sunbury weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

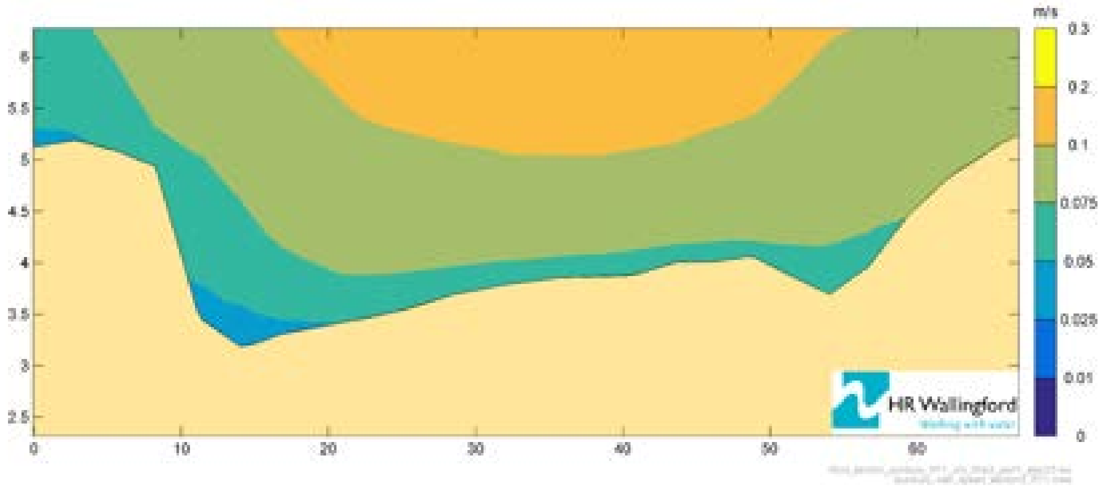


The difference data show that across most of the cross-section there is a 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs. A reduction in flow from 0.005-0.01m/s to -0.01 - -0.005m/s is noted to occur around the left bank.

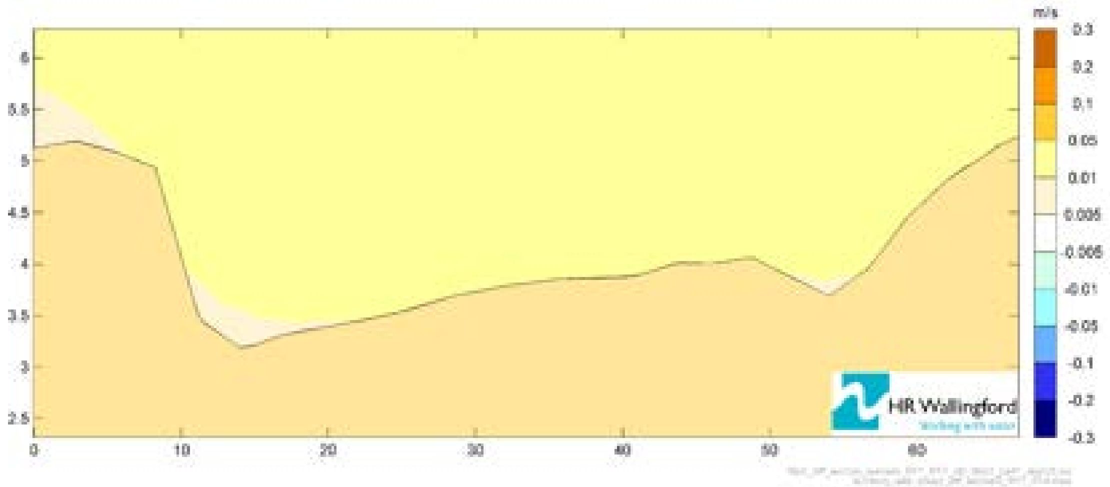
Figure 4-26 shows modelled changes in flow velocity for cross-section 3 under the Scenario 3 flows.

Figure 4-26 Section 3 flow velocities at Sunbury weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed



The difference data show that across most of the cross-section there is a maximum 0.01-0.05m/s increase in flow velocity between the baseline and the developed runs. Some spatially limited increases of 0.005-0.01m/s are noted close to the channel bed.

4.4.3. Molesey Weir pool

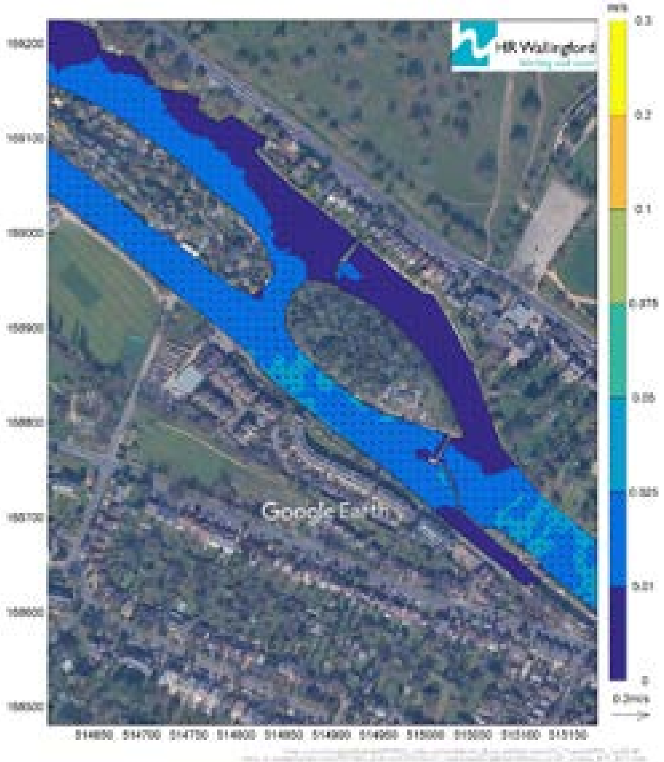
Modelled data from six river cross-sections were extracted from the baseline modelled data for the Molesey weir pool assessment, focusing on two weir pool areas for the northernmost weir and southernmost weir. The location of each of these cross-sections is presented in Appendix 1 Section 4.2. In addition, two longitudinal sections were extracted. Their locations are presented in Appendix 1 Section 4.2. An assessment of the baseline flow velocities has been outlined in Appendix 1 and the changes in flow velocity between the baseline and modelled conditions are presented below.

Scenario 1: 600 Ml/d river flow, extremely low river flow conditions

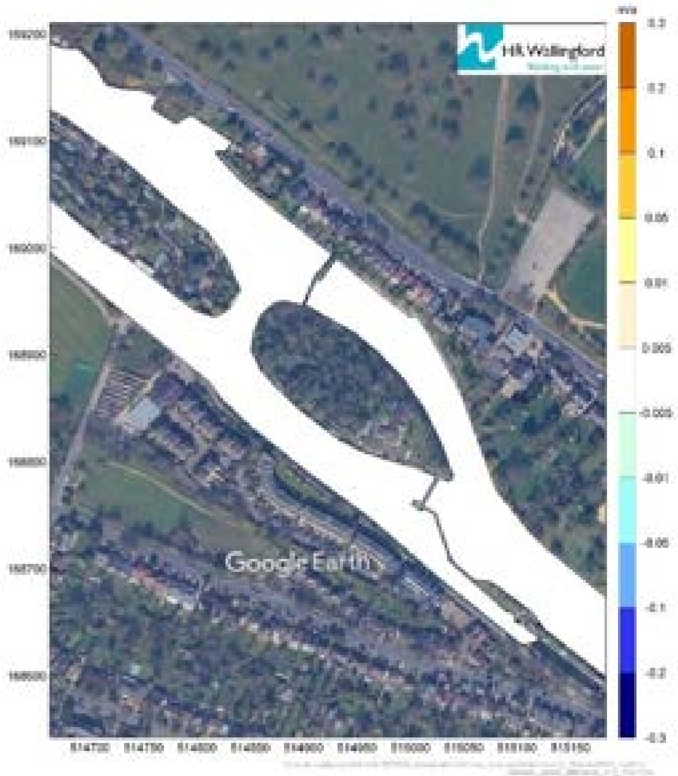
The depth-average velocity for Molesey Weir under Scenario 1 conditions and the velocity differences between this and the baseline are presented in Figure 4-27.

Figure 4-27 Depth-average velocity at Molesey weir, 600 Ml/d, Scenario 1

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

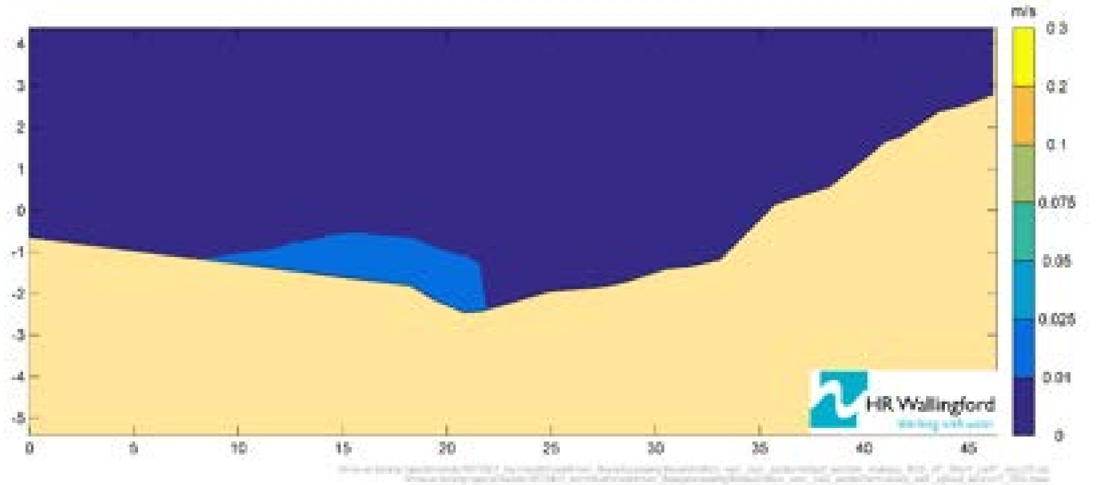


Under the developed Scenario 1 simulation a change of between -0.005 – 0.005m/s is predicted throughout the reach upstream and downstream of both weirs.

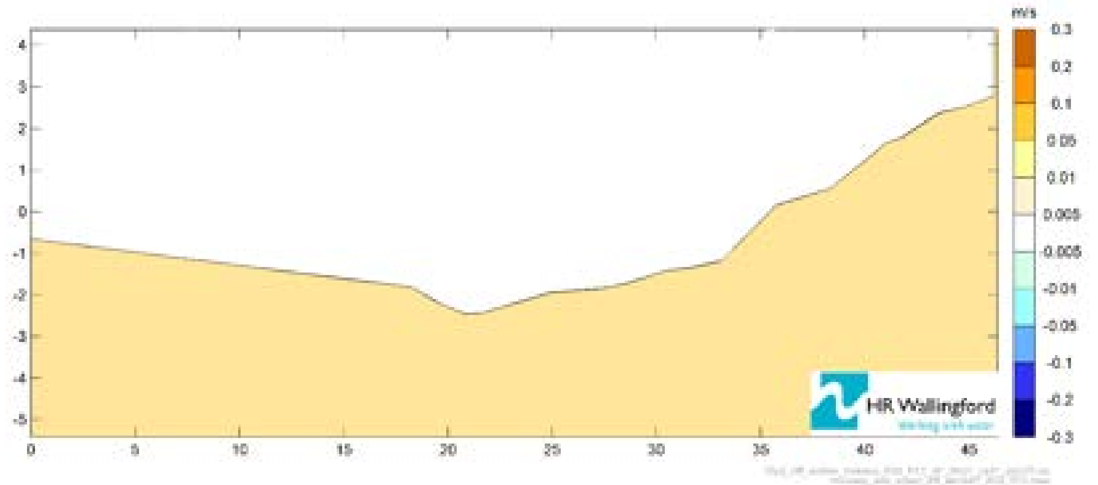
Figure 4-28 shows modelled changes in flow velocity for cross-section 1 under the Scenario 1 flows.

Figure 4-28 Section 1 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

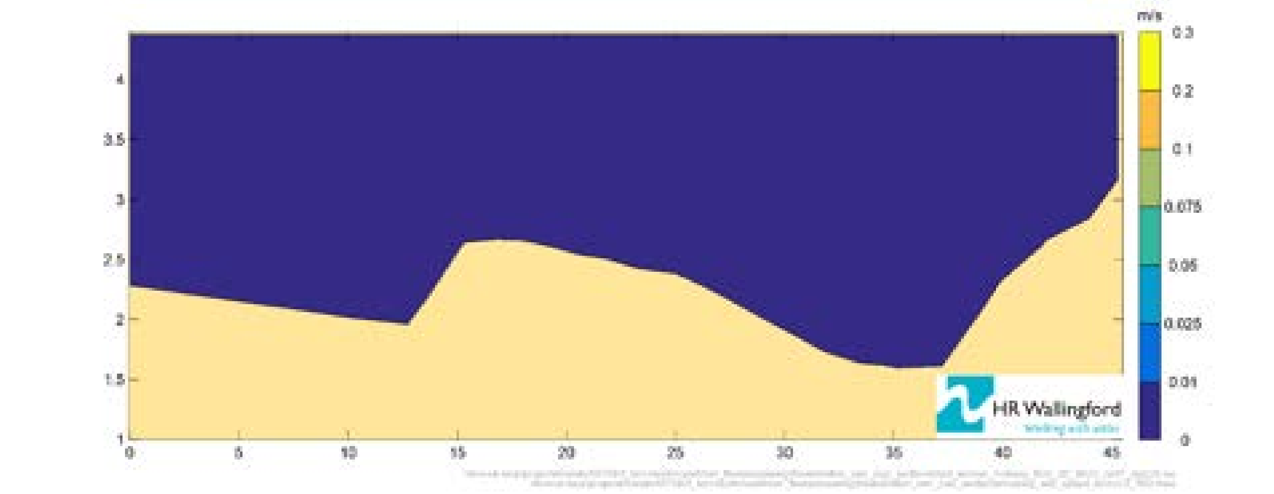


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

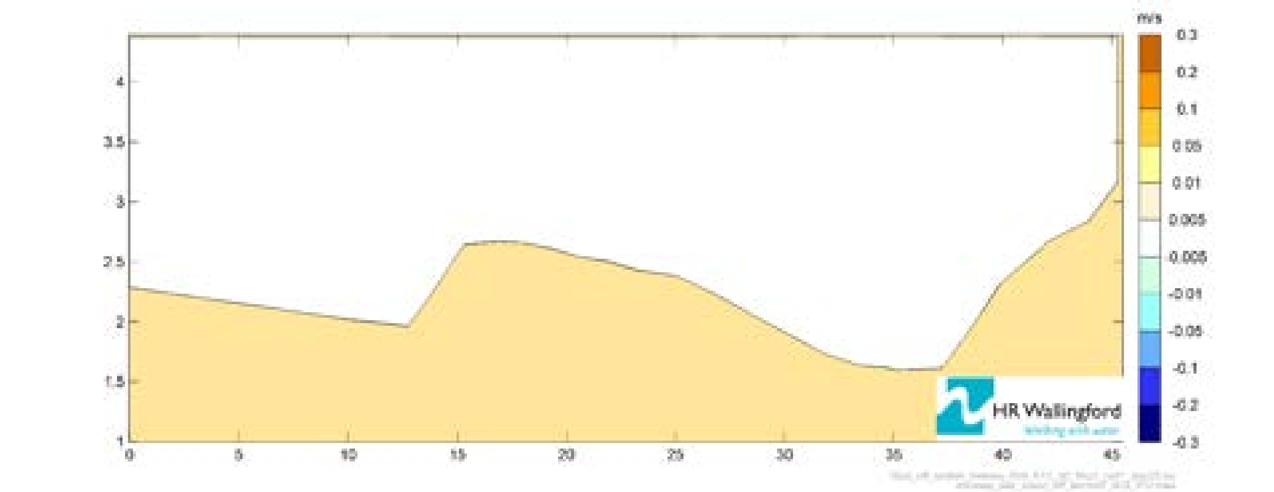
Figure 4-29 shows modelled changes in flow velocity for cross-section 2 under the Scenario 1 flows.

Figure 4-29 Section 2 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

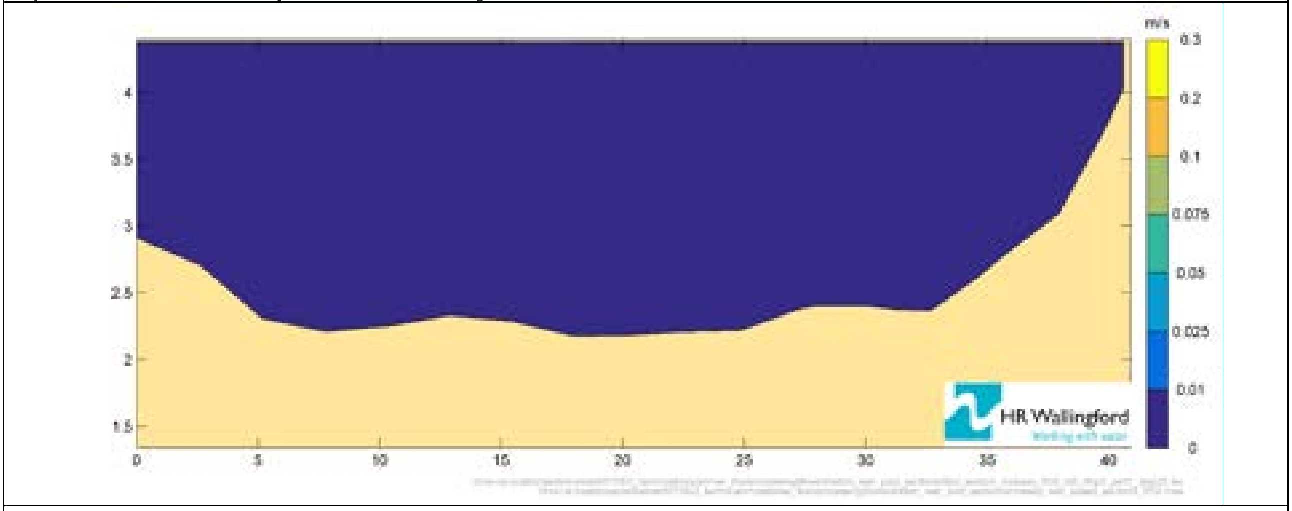


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

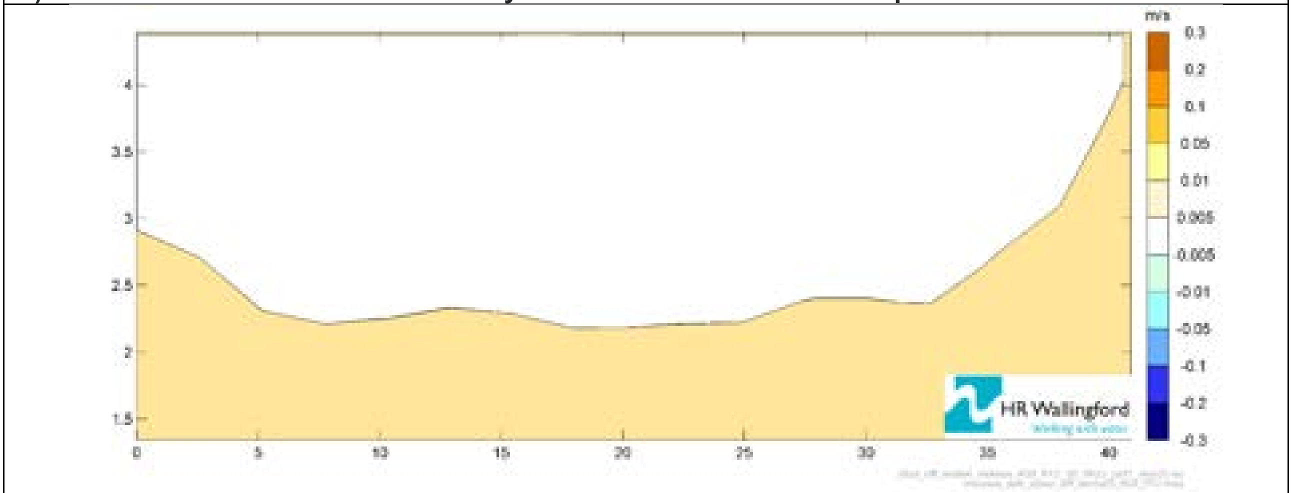
Figure 4-30 shows modelled changes in flow velocity for cross-section 3 under the Scenario 1 flows.

Figure 4-30 Section 3 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

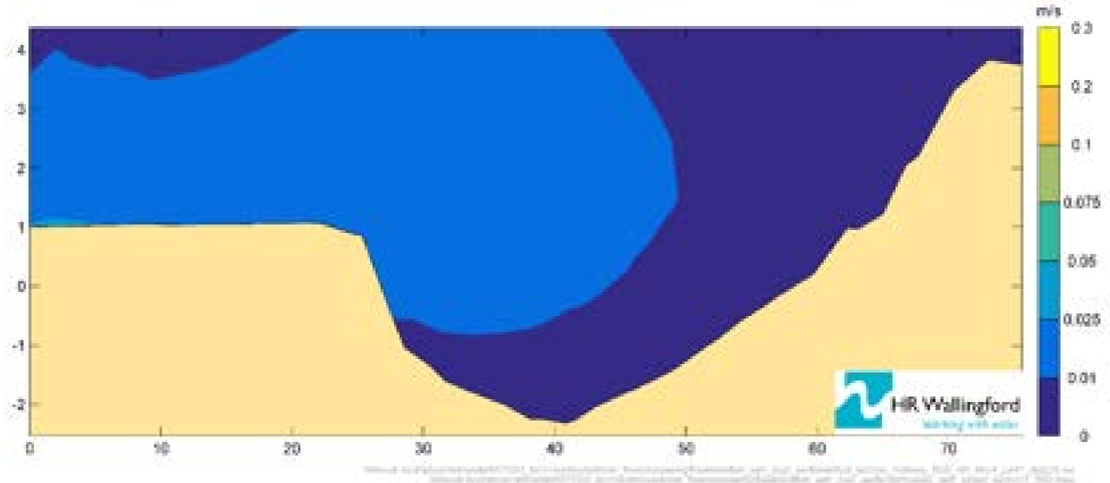


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

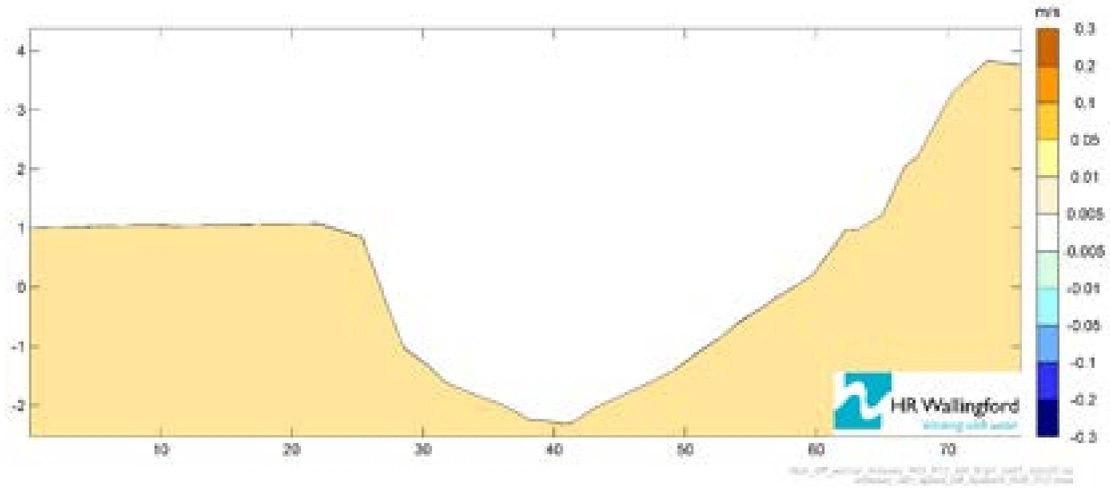
Figure 4-31 shows modelled changes in flow velocity for cross-section 4 under the Scenario 1 flows.

Figure 4-31 Section 4 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

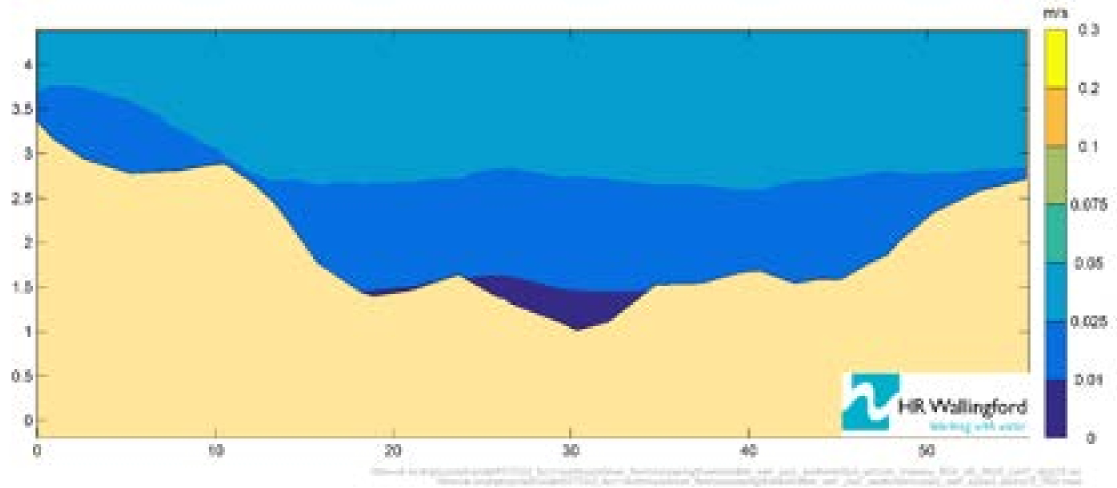


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

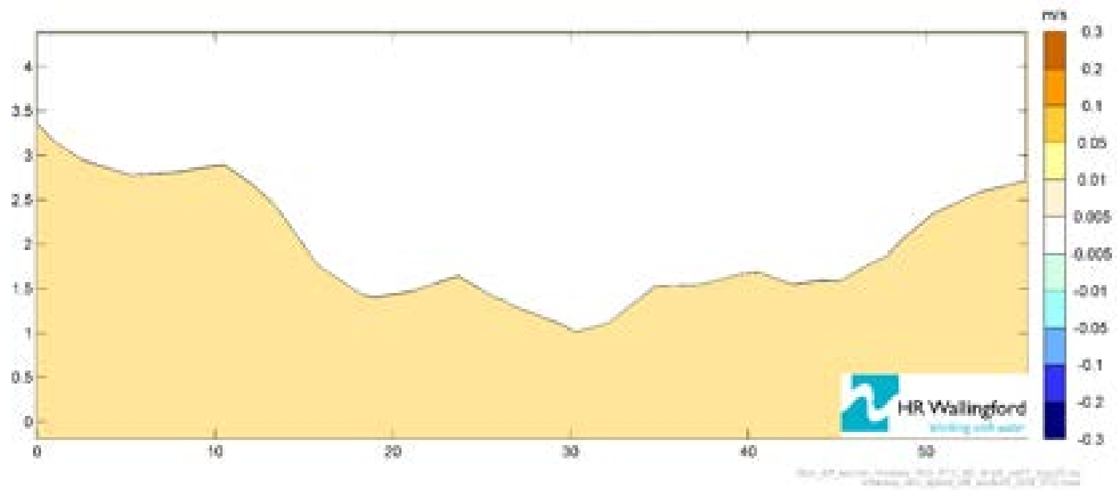
Figure 4-32 shows modelled changes in flow velocity for cross-section 5 under the Scenario 1 flows.

Figure 4-32 Section 5 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

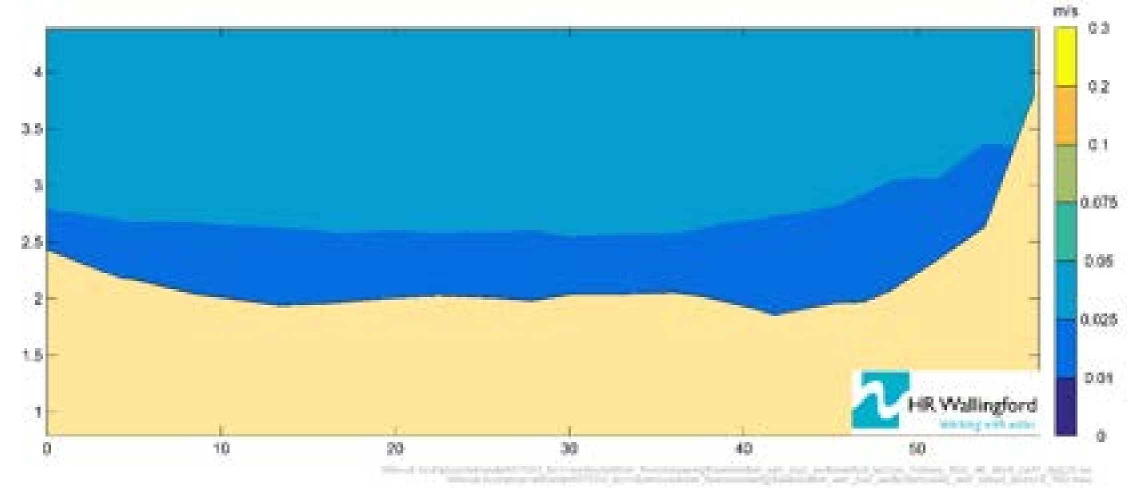


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

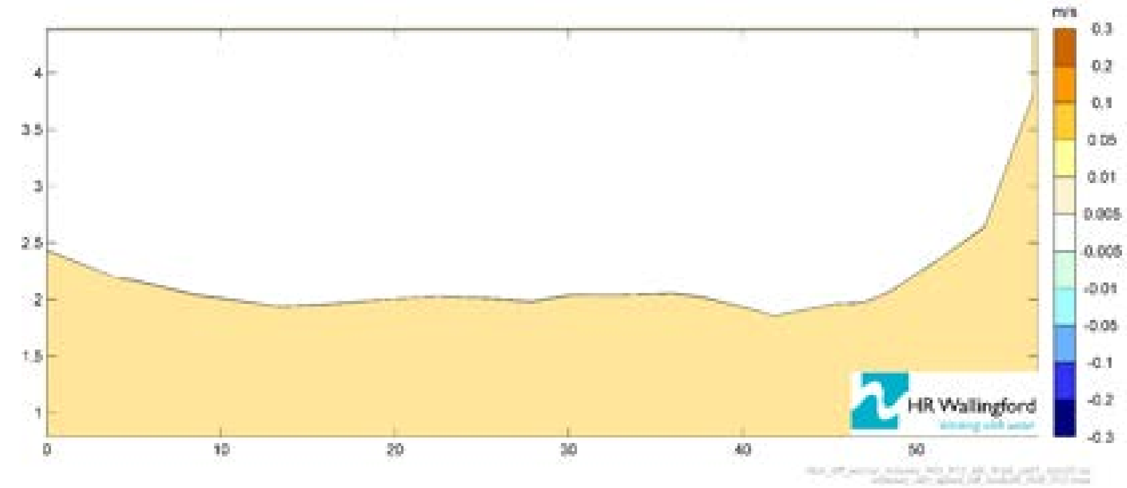
Figure 4-33 shows modelled changes in flow velocity for cross-section 6 under the Scenario 1 flows.

Figure 4-33 Section 6 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

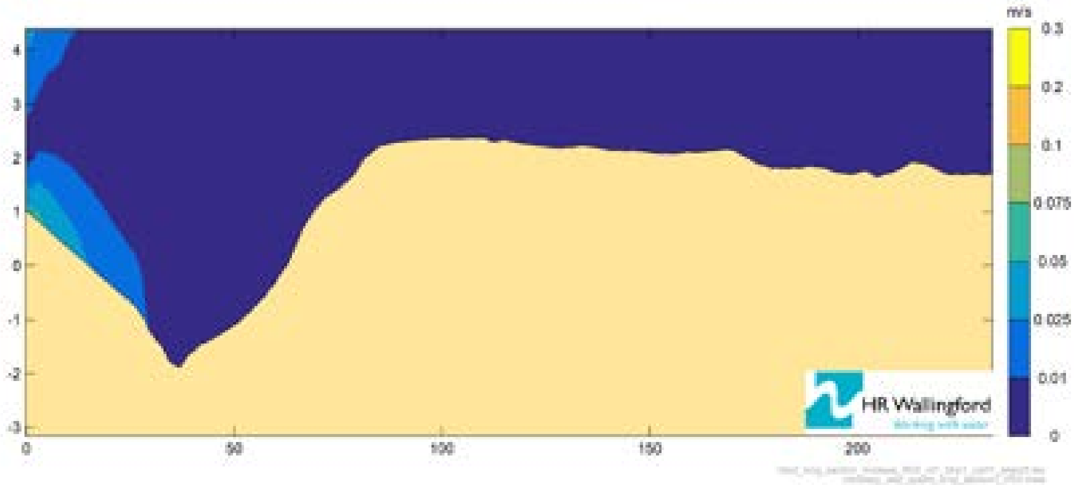


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

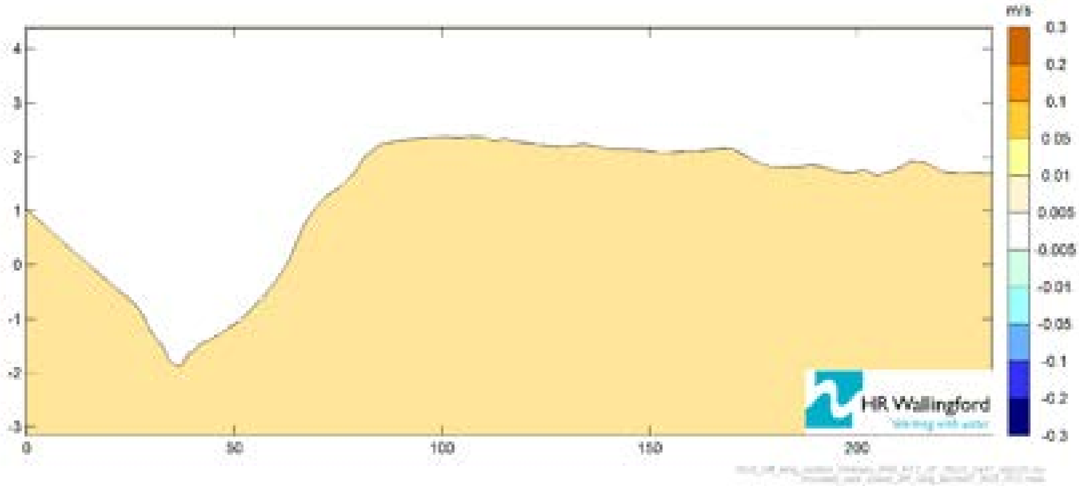
Figure 4-34 shows modelled changes in flow velocity for longitudinal section 1 under the Scenario 1 flows.

Figure 4-34 Longitudinal section 1 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed

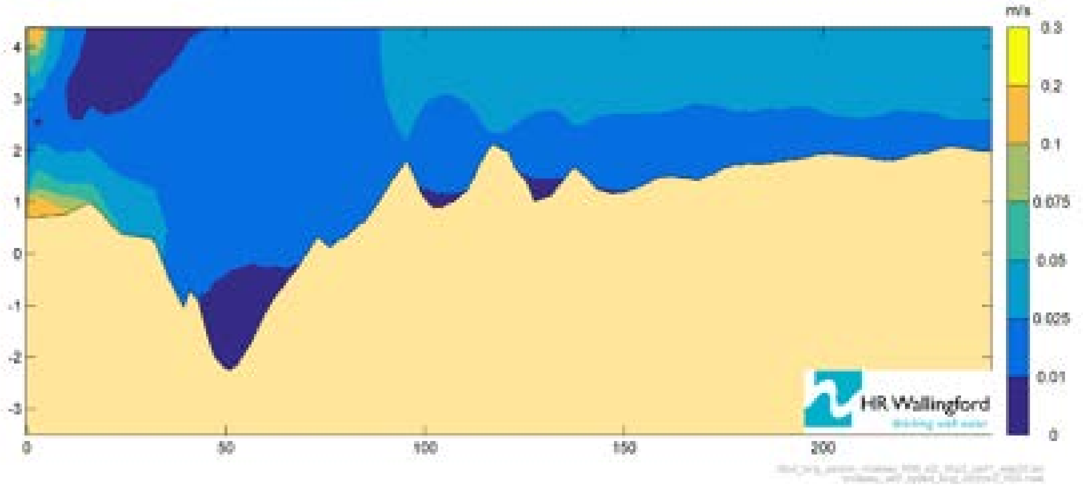


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across the entire longitudinal section.

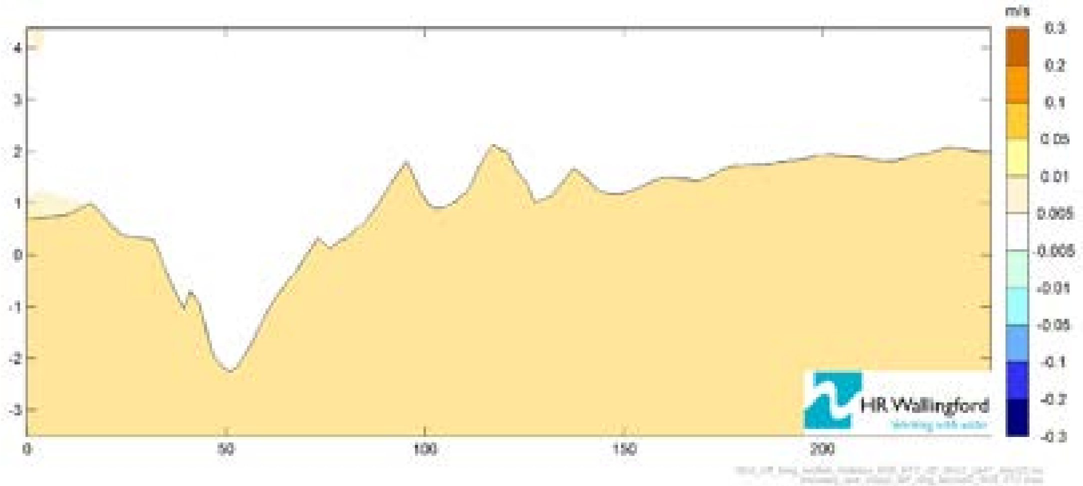
Figure 4-35 shows modelled changes in flow velocity for longitudinal section 2 under the Scenario 1 flows.

Figure 4-35 Longitudinal section 2 flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed



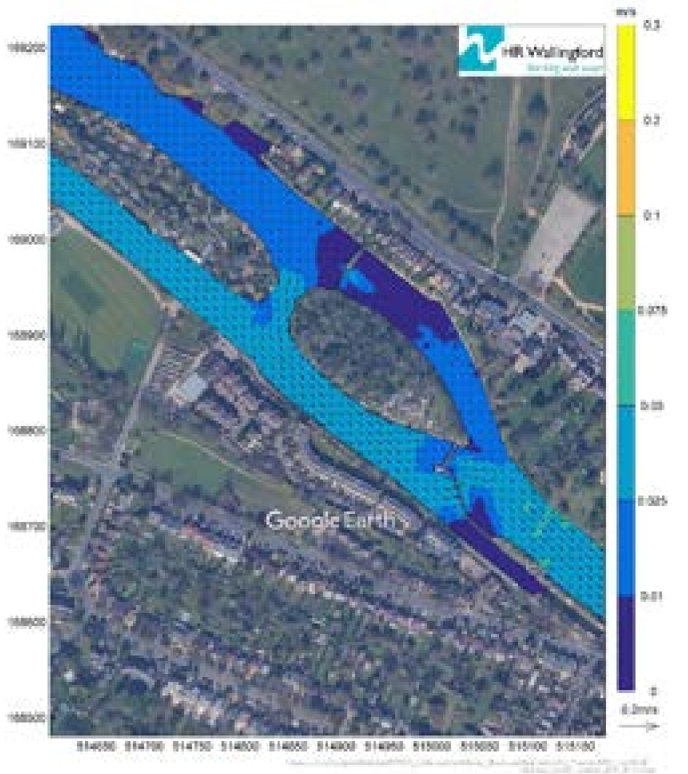
The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across most of the longitudinal section, with some increases of 0.005-0.01m/s noted at the base and top of the depth profile at the start of the section adjacent to the weir.

Scenario 2: 780 Ml/d river flow, extremely low river flow conditions

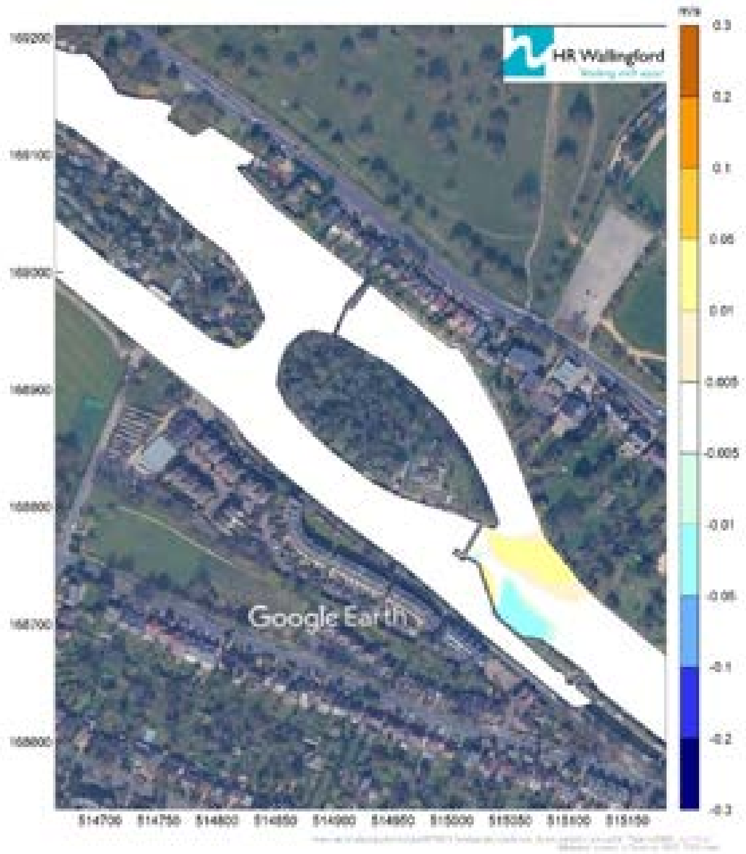
The depth-average velocity for Molesey Weir under Scenario 2 conditions and the velocity differences between this and the baseline are presented in Figure 4-36.

Figure 4-36 Depth-average velocity at Molesey weir, 780 Ml/d, Scenario 2

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed



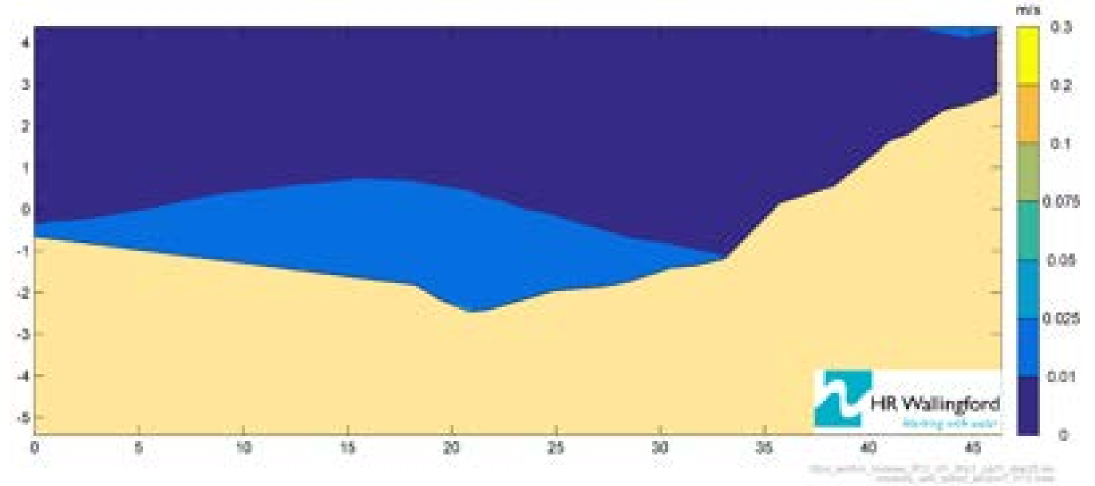
Under the developed Scenario 2 simulations much of the reach upstream and downstream of the weirs shows very little change in velocity of between -0.005 – 0.005m/s. There are small, localised changes in flow velocity downstream of the southernmost weir, with a reduction in flow velocity of -0.05 - -0.01m/s adjacent to the weir and an increase in velocity of 0.01-0.05m/s at the confluence with the channel bifurcation on the left bank.

Under the developed Scenario 2 simulation a change of between -0.005 – 0.005m/s is predicted throughout the reach upstream and downstream of both weirs.

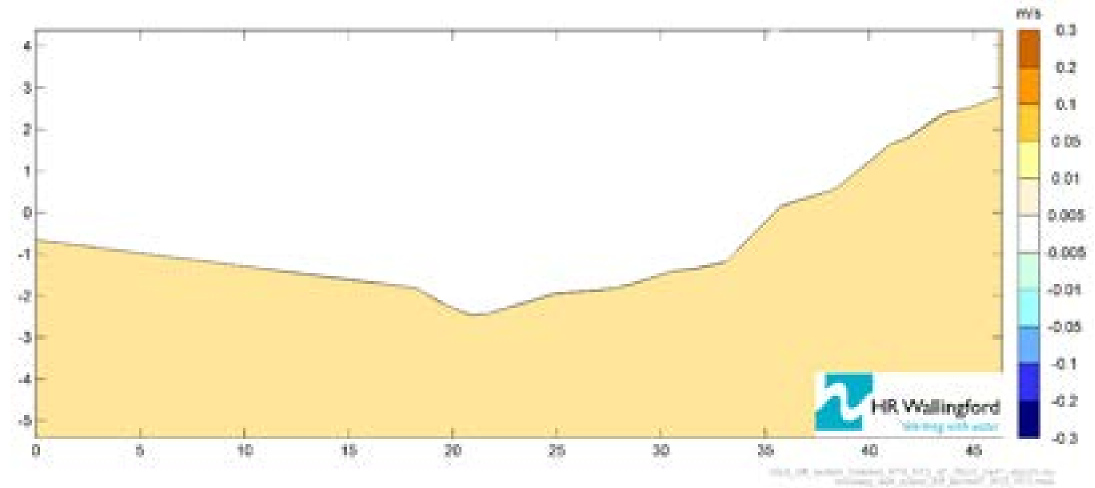
Figure 4-37 shows modelled changes in flow velocity for cross-section 1 under the Scenario 2 flows.

Figure 4-37 Section 1 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

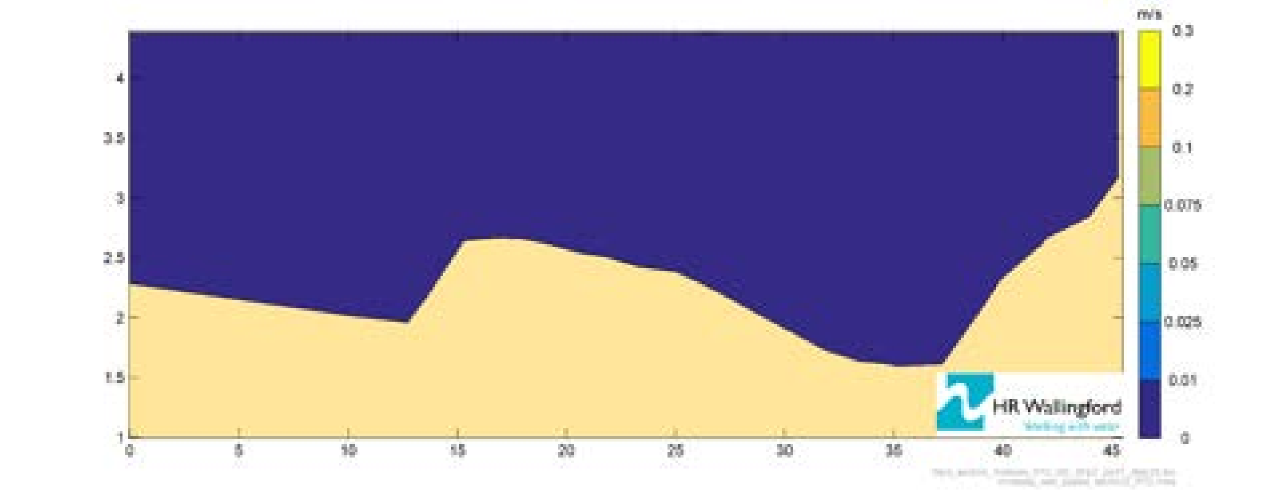


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

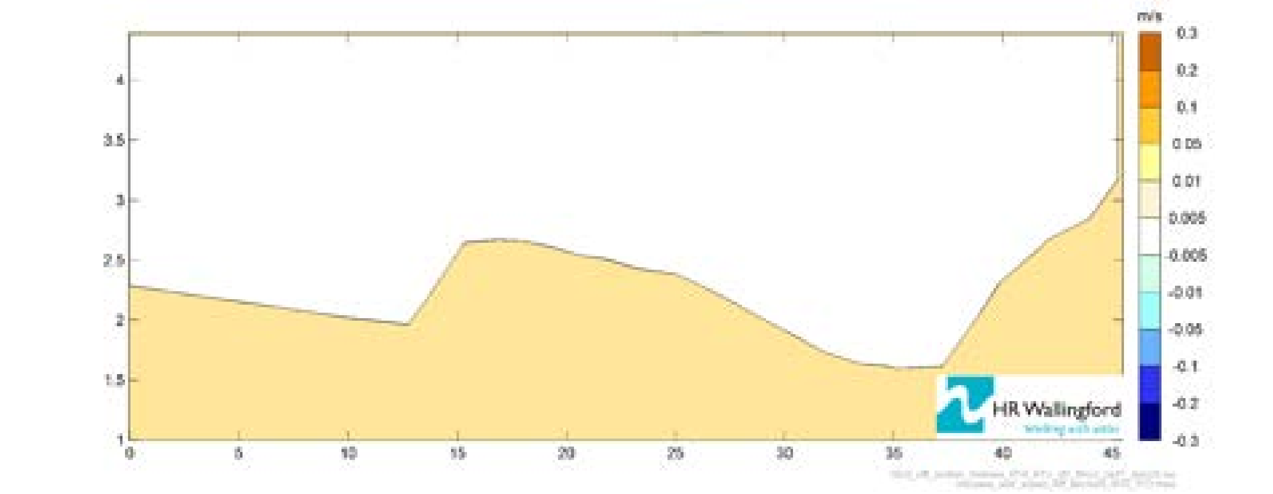
Figure 4-38 shows modelled changes in flow velocity for cross-section 2 under the Scenario 2 flows.

Figure 4-38 Section 2 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

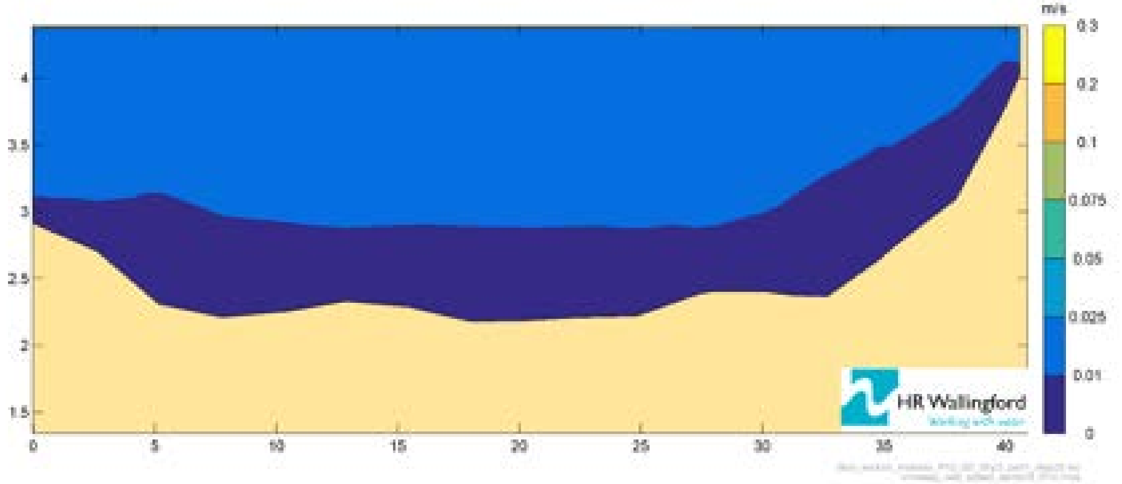


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

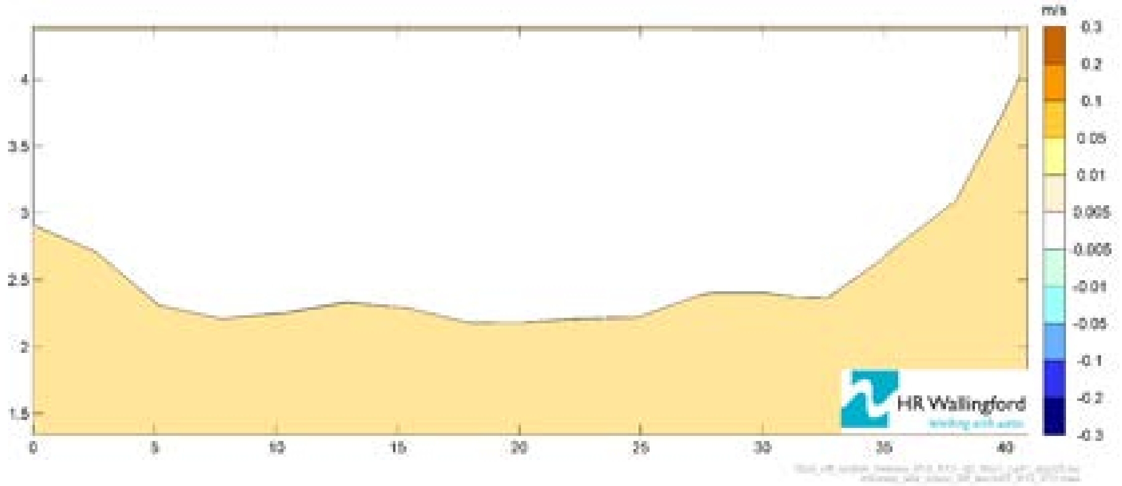
Figure 4-39 shows modelled changes in flow velocity for cross-section 3 under the Scenario 2 flows.

Figure 4-39 Section 3 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

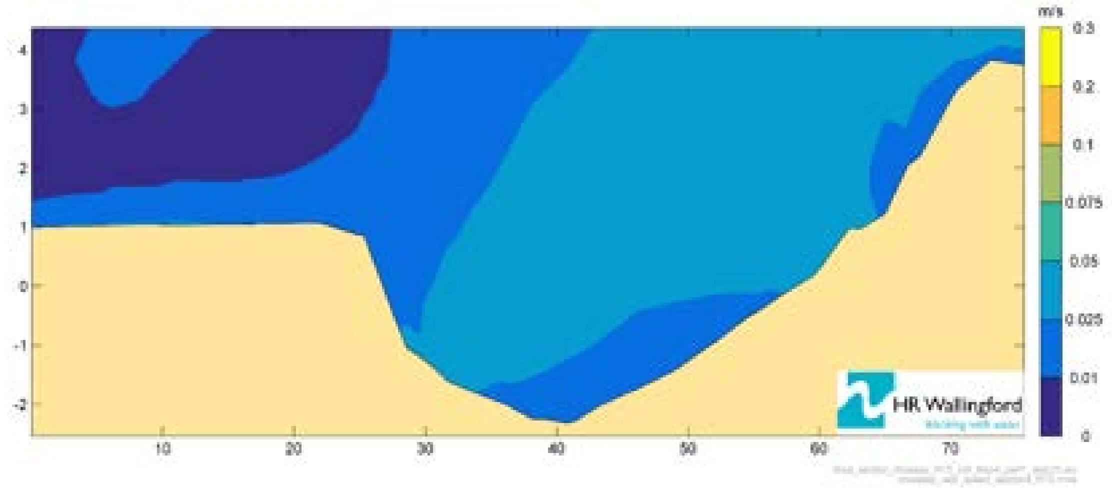


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

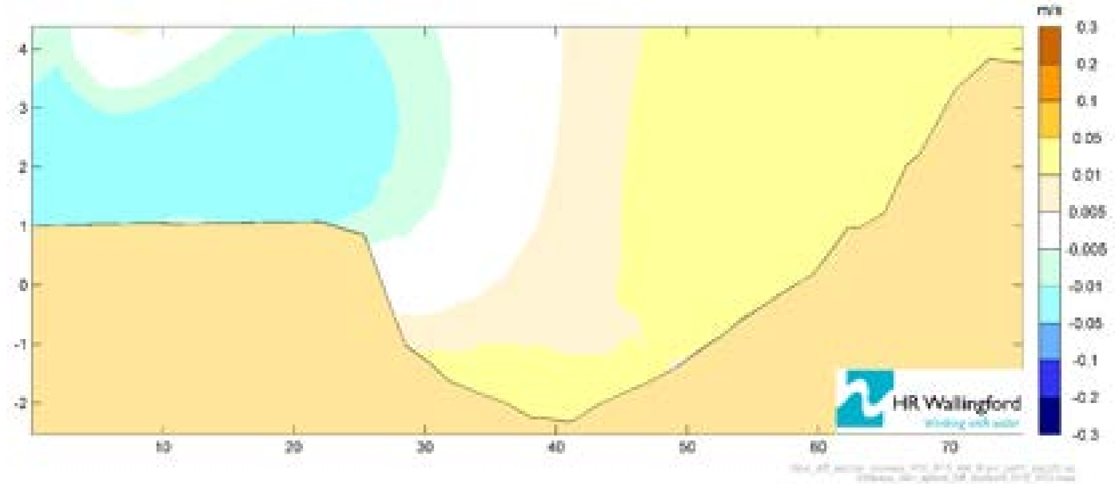
Figure 4-40 shows modelled changes in flow velocity for cross-section 4 under the Scenario 2 flows.

Figure 4-40 Section 4 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

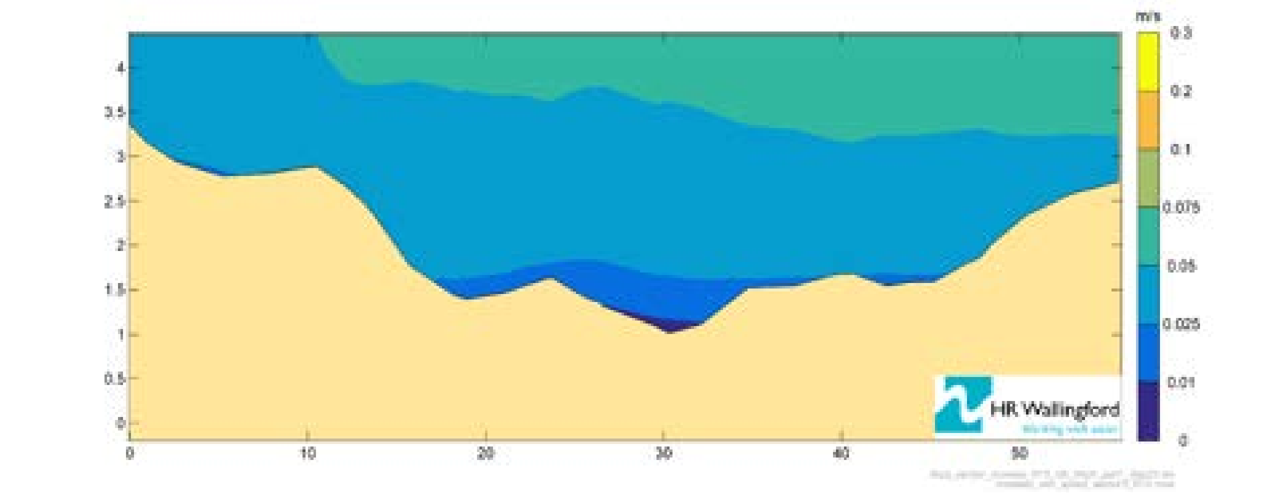


The difference data indicate that there is general gradient of velocity change from the left to right side of the cross-section. Reductions in velocity of -0.05 - -0.01m/s are seen at the left side of the section, increasing to 0.01-0.05m/s towards the right side of the channel and within the base of the weir pool.

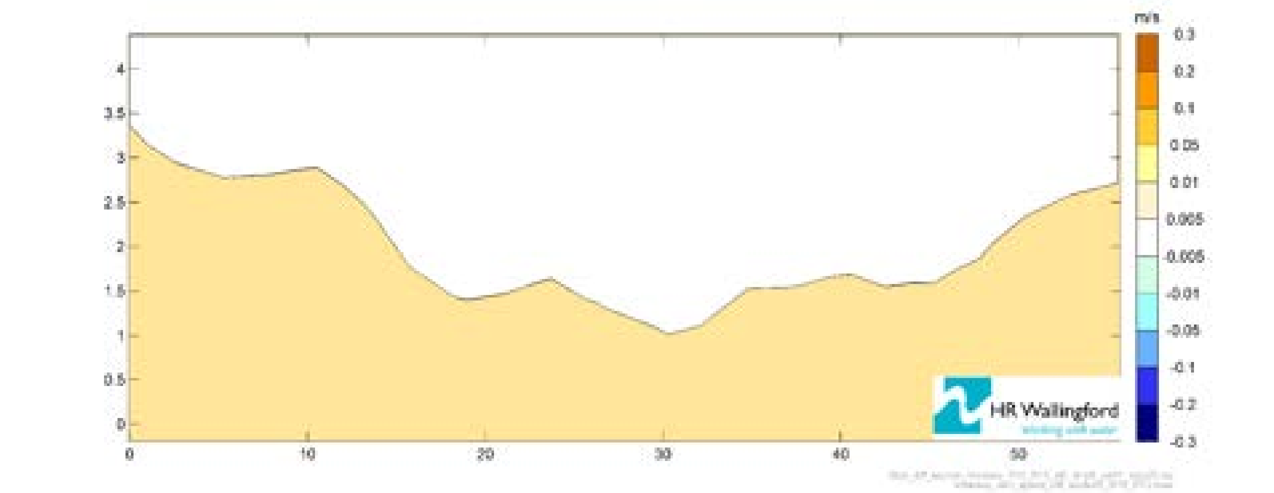
Figure 4-41 shows modelled changes in flow velocity for cross-section 5 under the Scenario 2 flows.

Figure 4-41 Section 5 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

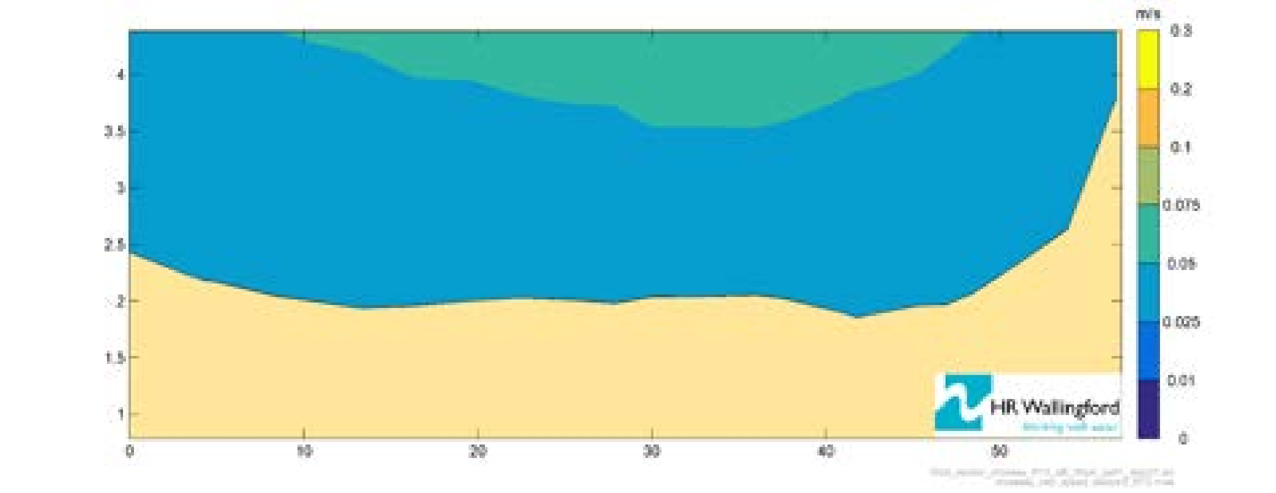


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

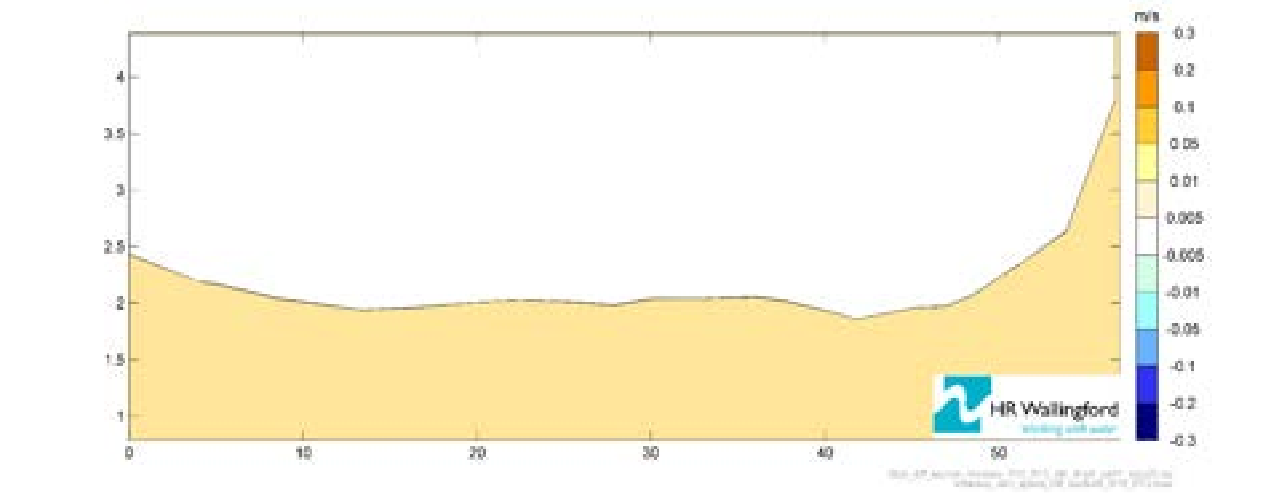
Figure 4-42 shows modelled changes in flow velocity for cross-section 6 under the Scenario 2 flows.

Figure 4-42 Section 6 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

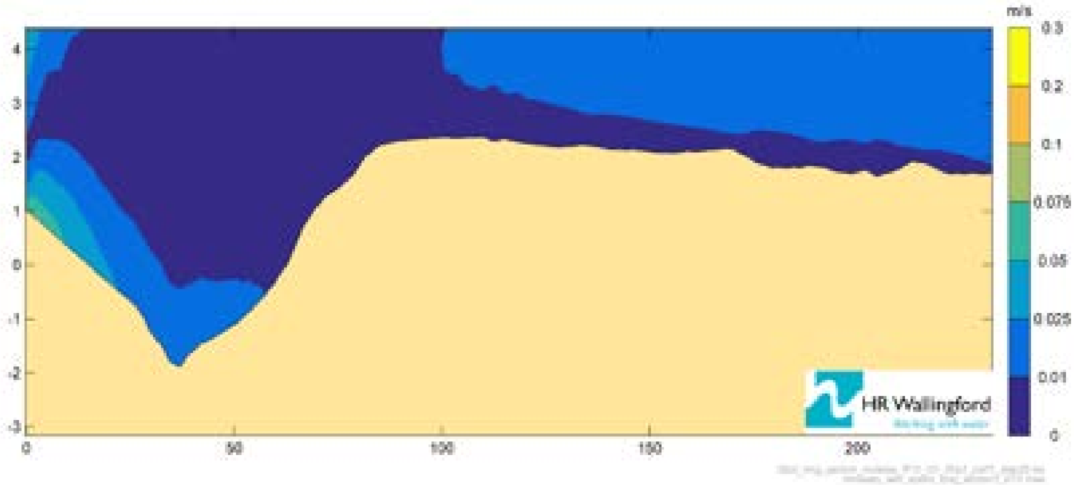


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

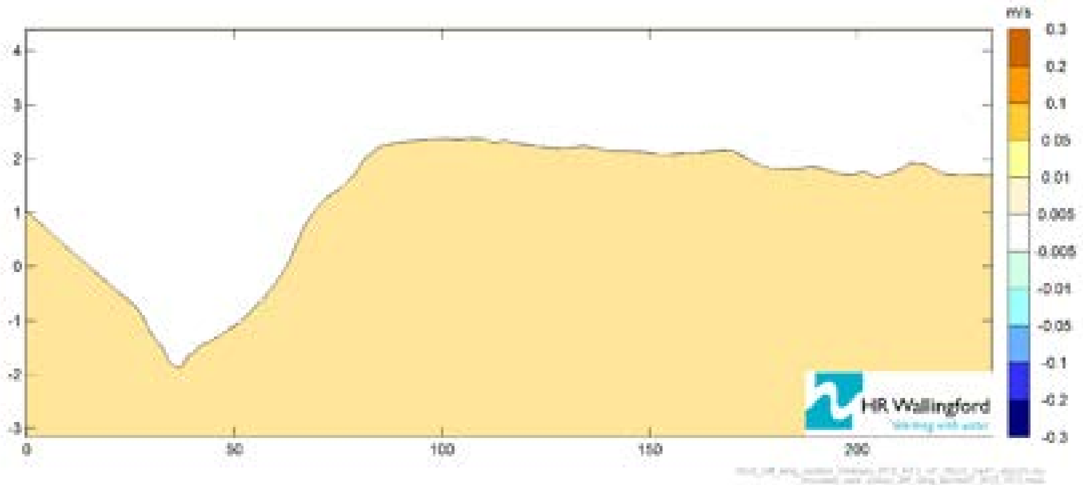
Figure 4-43 shows modelled changes in flow velocity for longitudinal section 1 under the Scenario 2 flows.

Figure 4-43 Longitudinal section 1 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

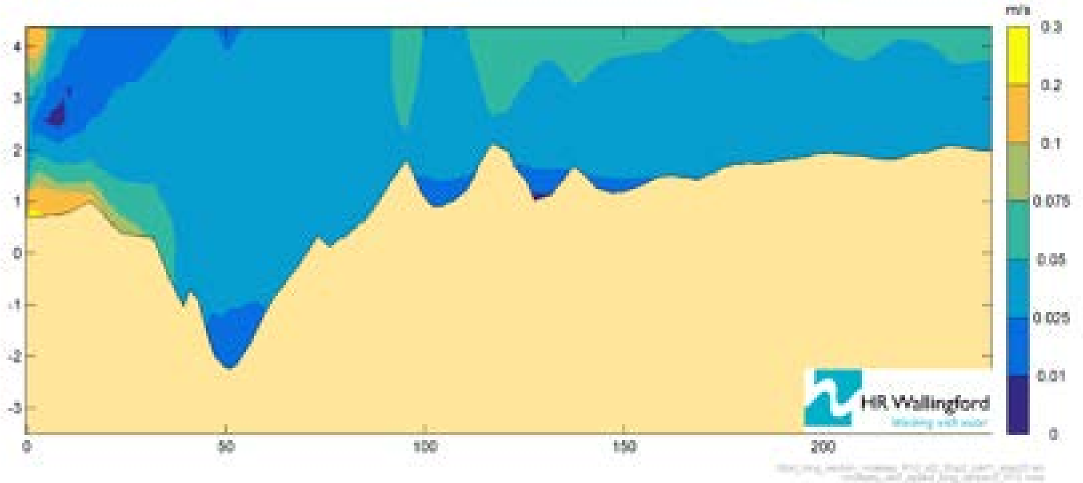


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across the entire longitudinal section.

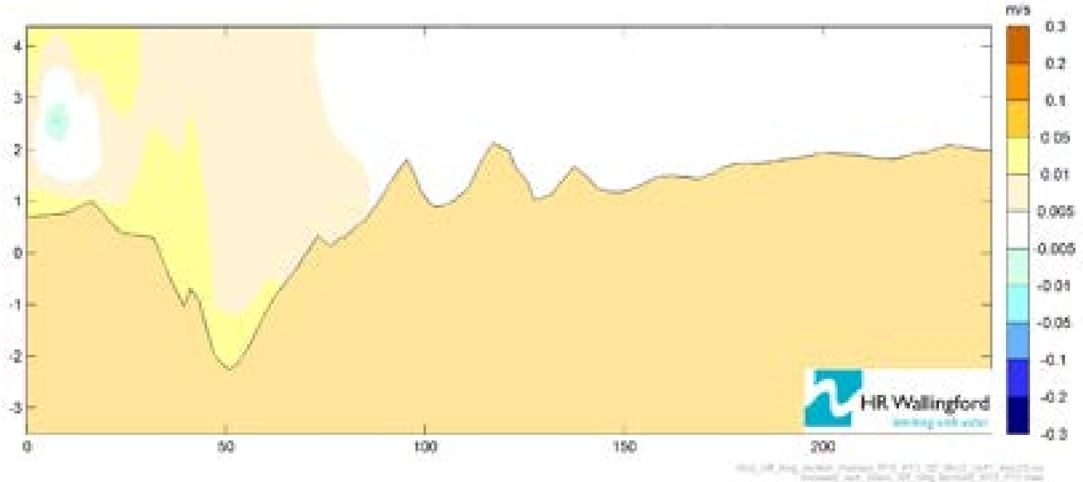
Figure 4-44 shows modelled changes in flow velocity for longitudinal section 2 under the Scenario 2 flows.

Figure 4-44 Longitudinal section 2 flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed



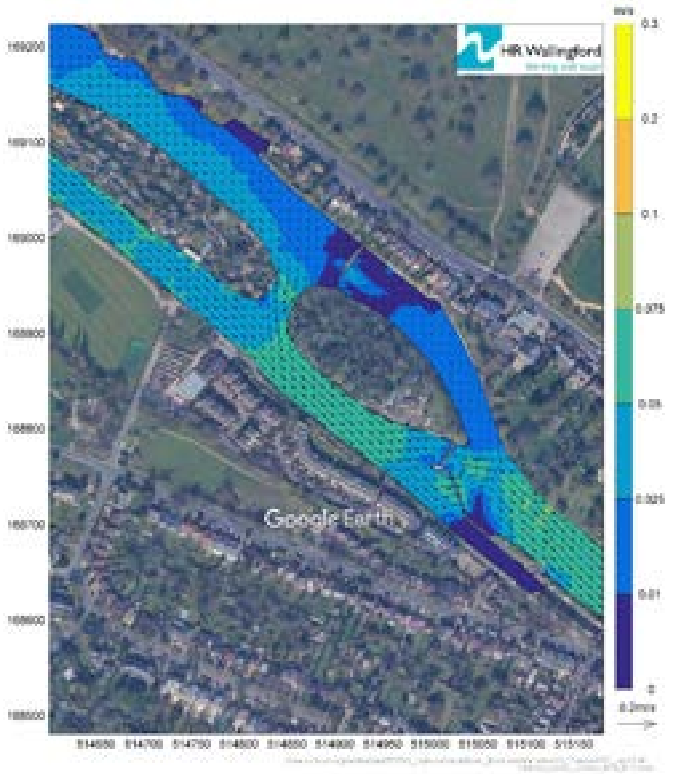
The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across the longitudinal section from around 80m chainage downstream of the weir. Velocity increases around the weir and in the weir pool itself of between 0.005-0.05m/s down to around 80m chainage downstream are noted, with slightly higher increases in velocity simulated at the upstream edge and base of the weir pool.

Scenario 3: 950 Ml/d river flow, low river flow conditions

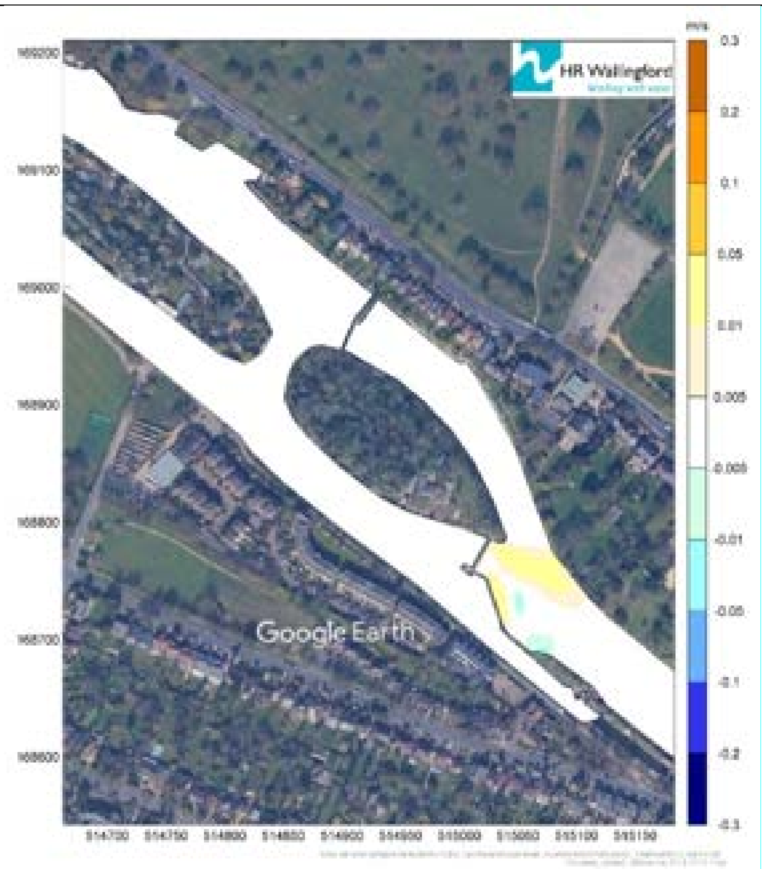
The depth-average velocity for Molesey Weir under Scenario 3 conditions and the velocity differences between this and the baseline are presented in Figure 4-45.

Figure 4-45 Depth-average velocity at Molesey weir, 950 Ml/d, Scenario 3

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed



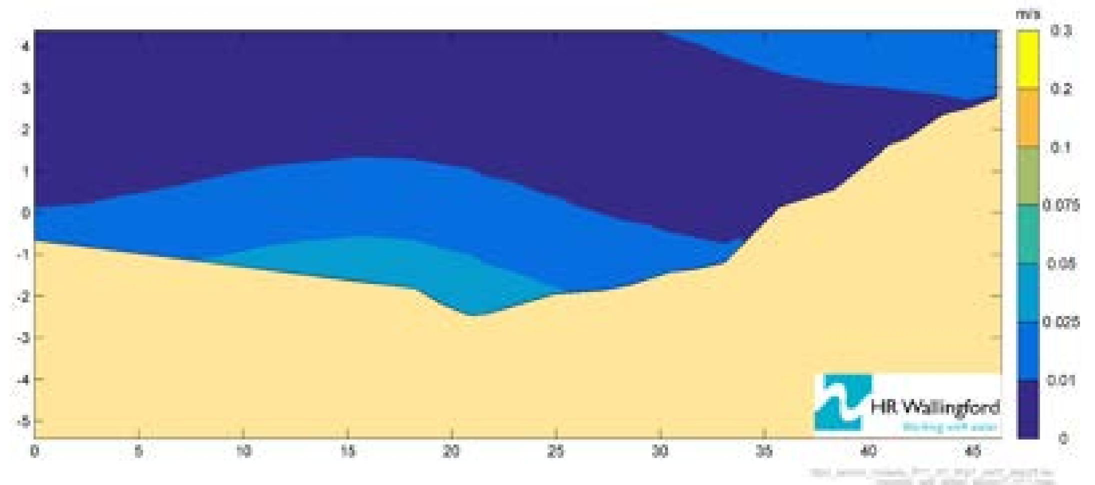
Under the developed Scenario 3 simulations much of the reach upstream and downstream of the weirs shows very little change in velocity of between -0.005 – 0.005m/s. There are small, localised changes in flow velocity downstream of the southernmost weir, with a reduction in flow velocity of -0.05 - -0.01m/s adjacent to the weir and an increase in velocity of 0.01-0.05m/s adjacent to the weir and at the confluence with the channel bifurcation on the left bank.

Under the developed Scenario 3 simulation a change of between -0.005 – 0.005m/s is predicted throughout the reach upstream and downstream of both weirs.

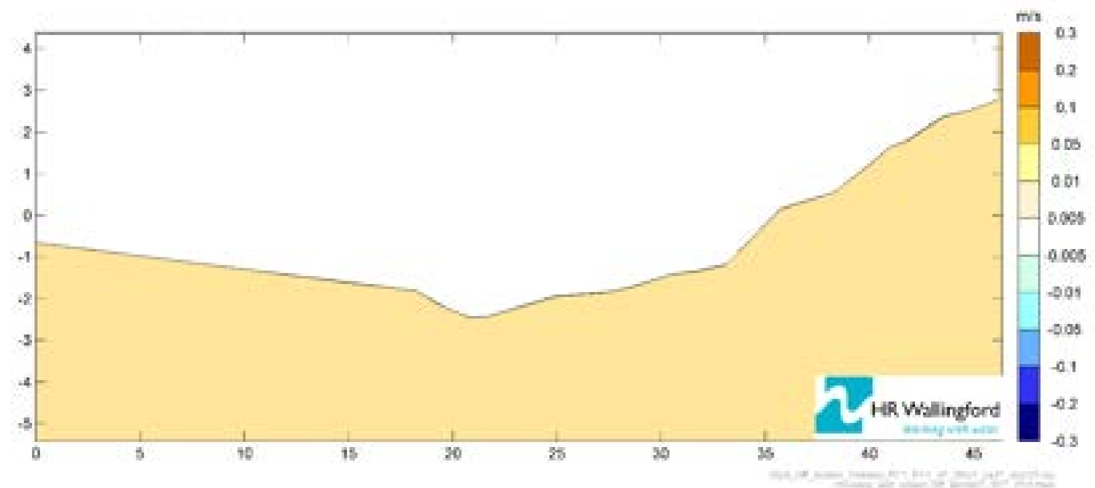
Figure 4-46 shows modelled changes in flow velocity for cross-section 1 under the Scenario 3 flows.

Figure 4-46 Section 1 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

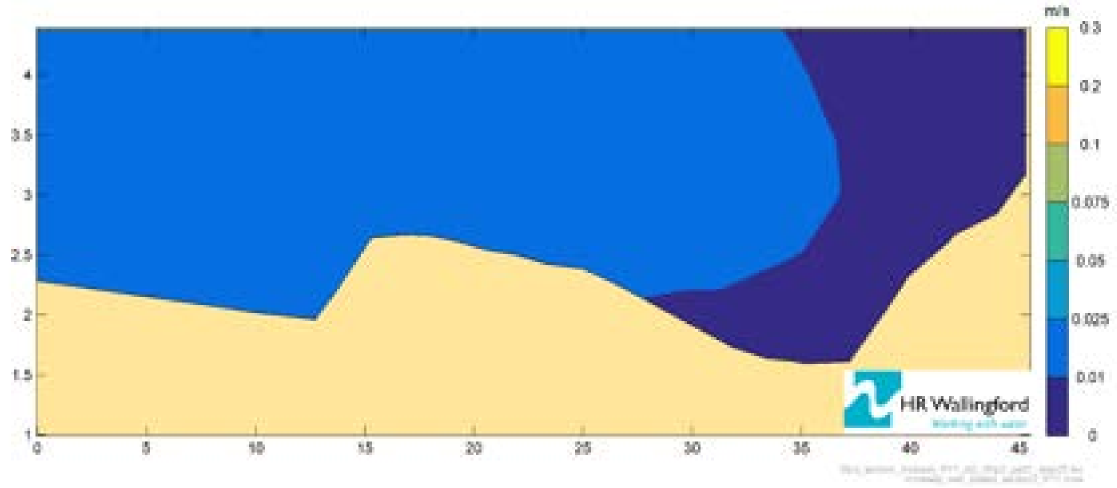


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

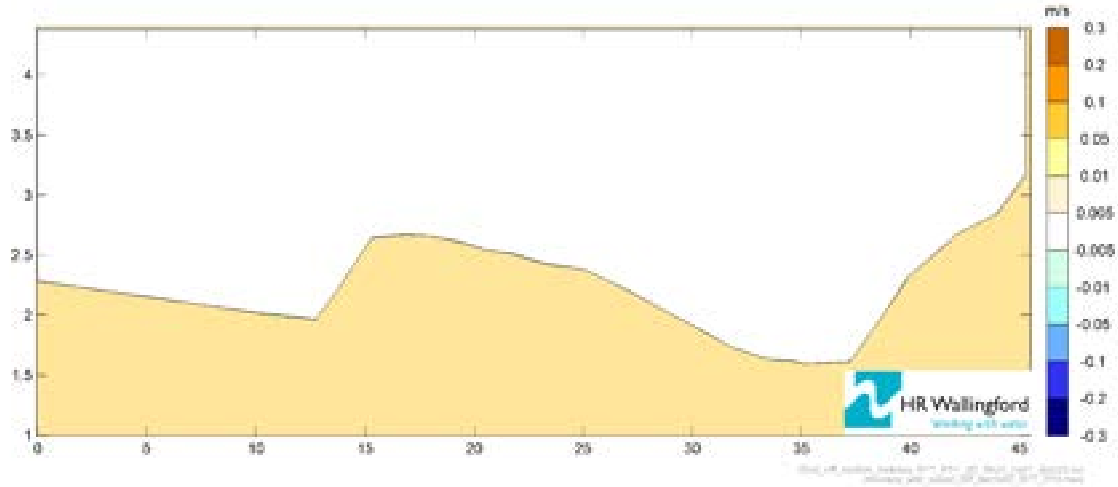
Figure 4-47 shows modelled changes in flow velocity for cross-section 2 under the Scenario 3 flows.

Figure 4-47 Section 2 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

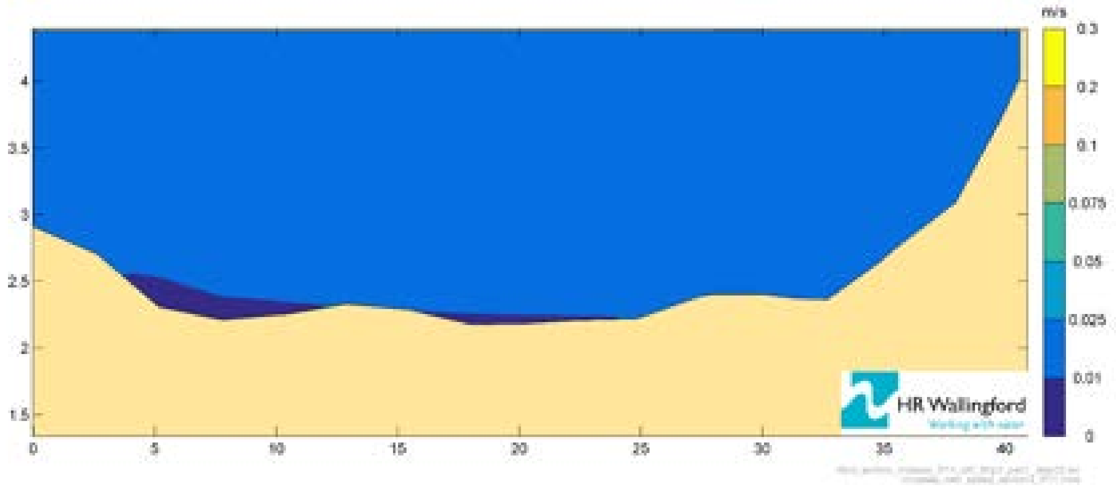


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

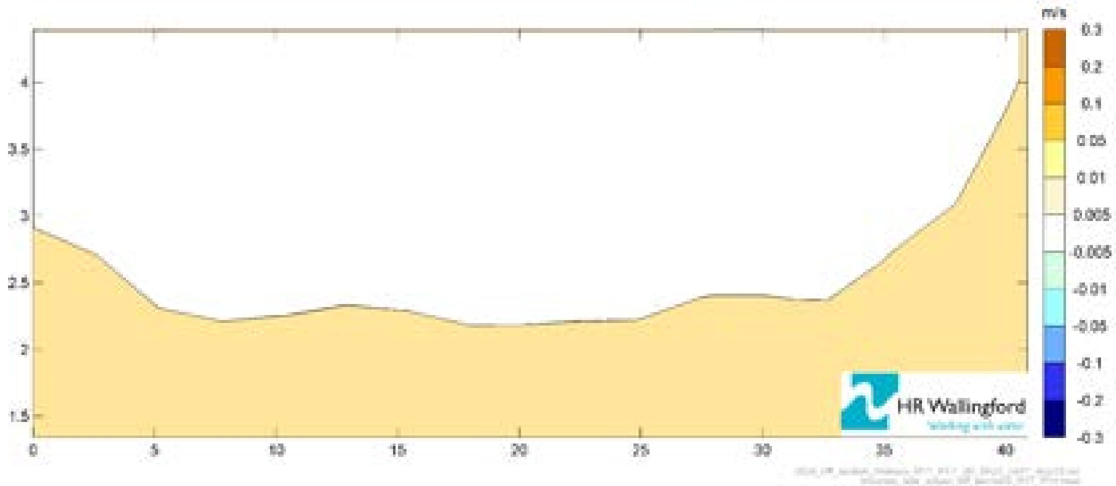
Figure 4-48 shows modelled changes in flow velocity for cross-section 3 under the Scenario 3 flows.

Figure 4-48 Section 3 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

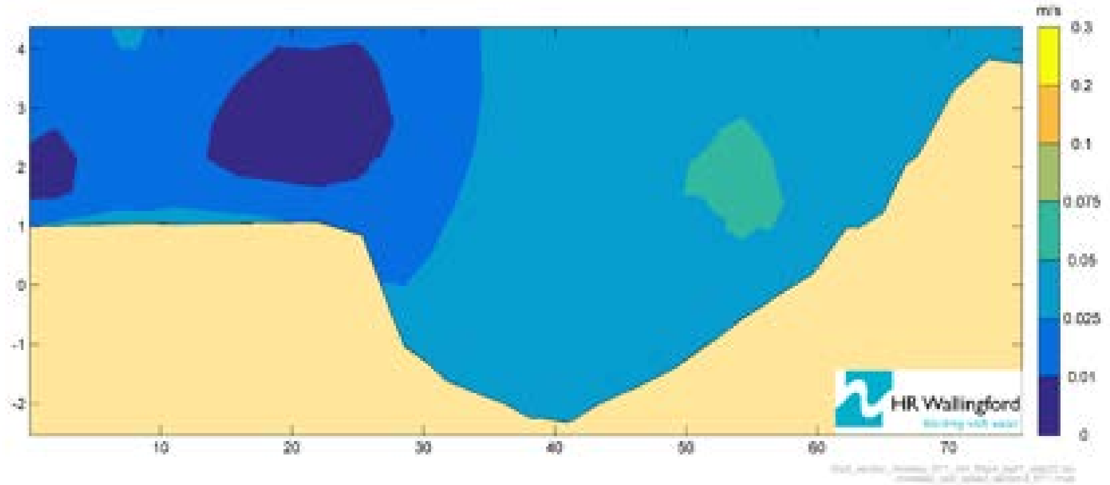


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

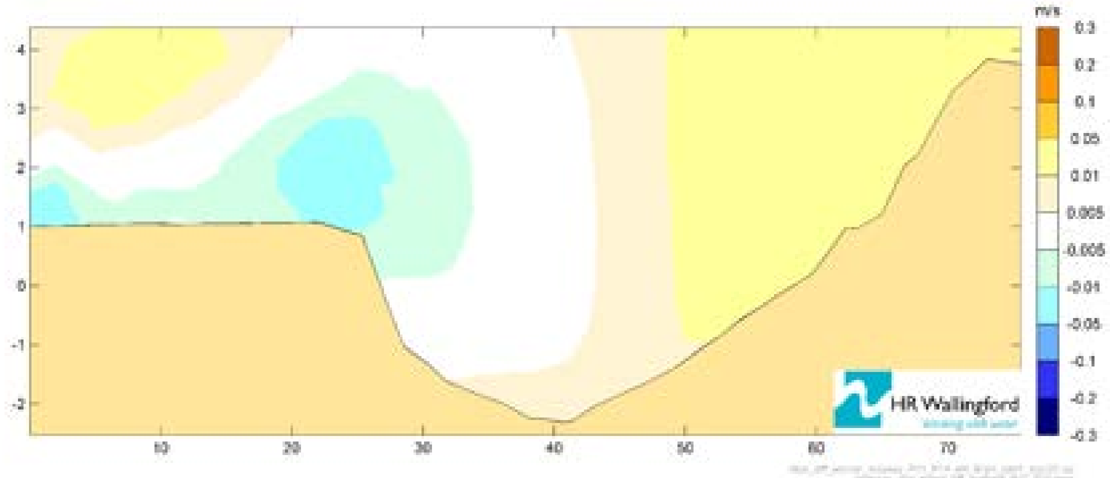
Figure 4-49 shows modelled changes in flow velocity for cross-section 4 under the Scenario 3 flows.

Figure 4-49 Section 4 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

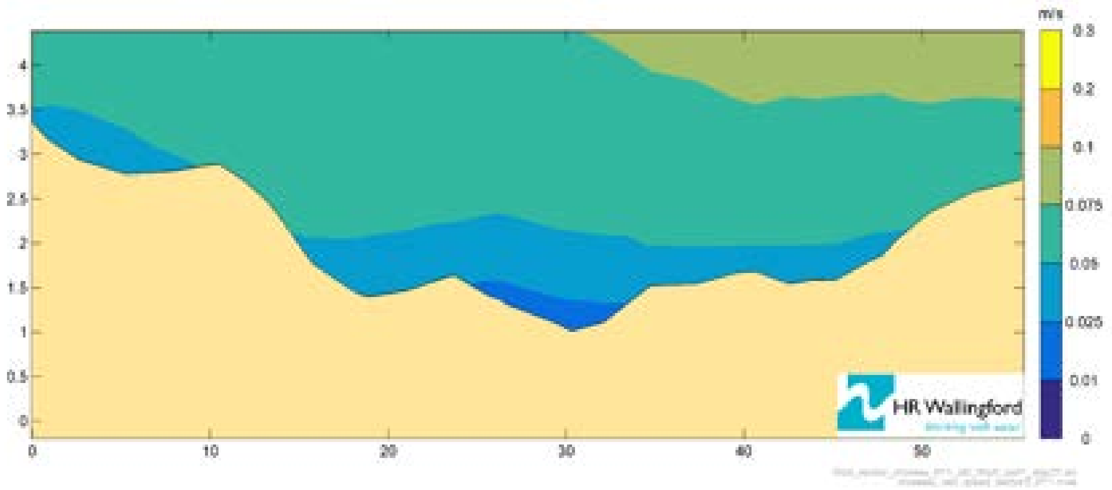


The difference data indicate that there is general gradient of velocity change from the left to right side of the cross-section. Reductions in velocity of -0.05 - -0.01m/s are seen at the left side of the section (mostly at the base of the channel towards the bed), increasing to 0.01-0.05m/s towards the right side of the channel and within the base of the weir pool. Additionally, increasing flow velocity to 0.005-0.05m/s at the surface towards the left bank is also simulated.

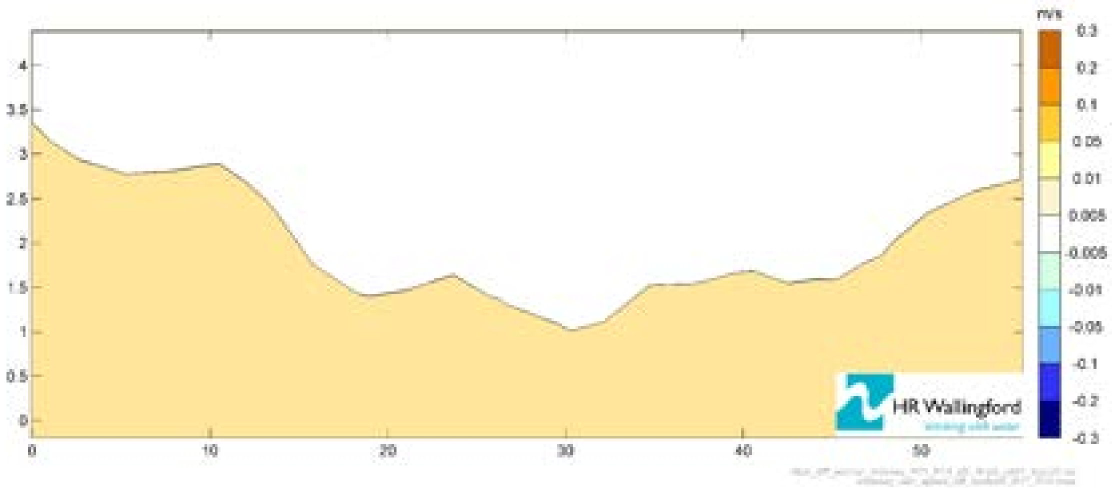
Figure 4-50 shows modelled changes in flow velocity for cross-section 5 under the Scenario 3 flows.

Figure 4-50 Section 5 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

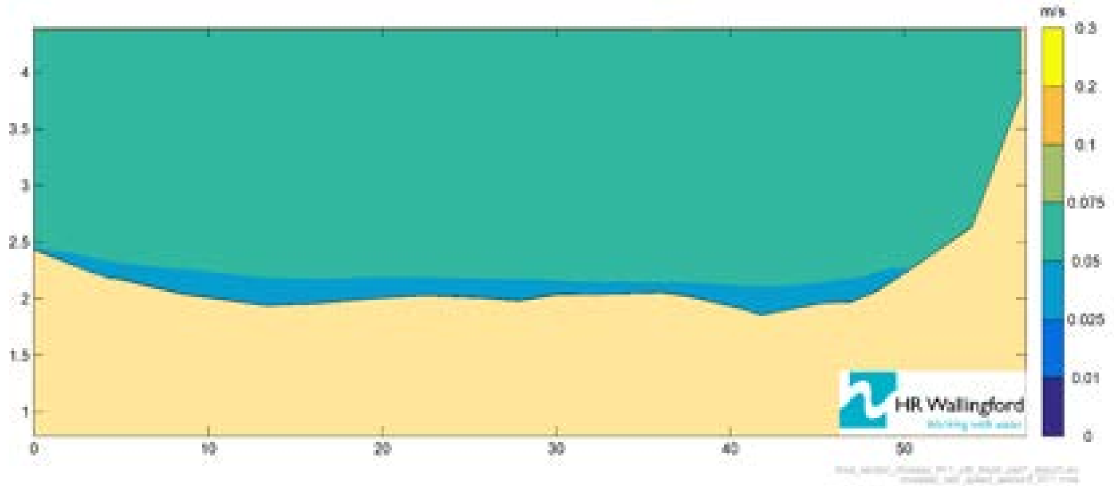


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

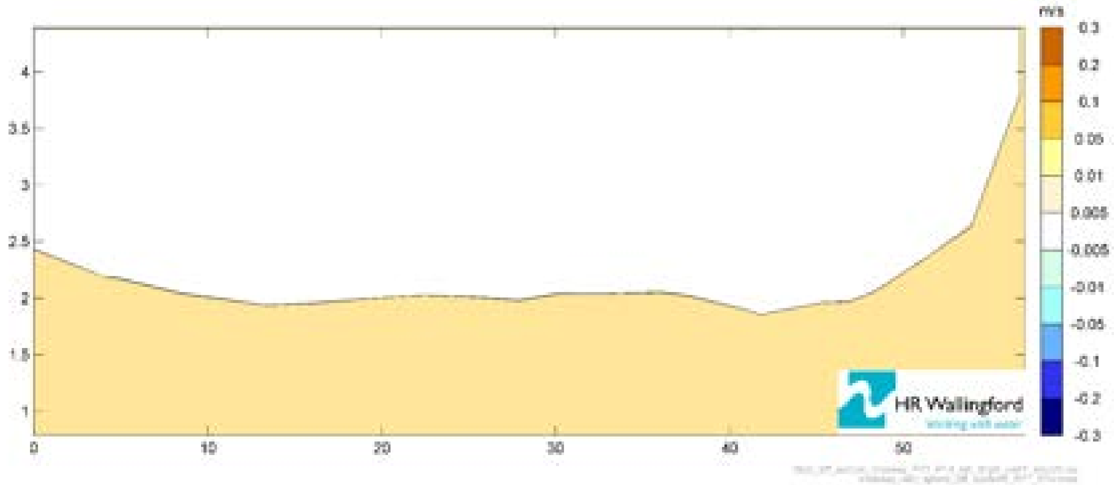
Figure 4-51 shows modelled changes in flow velocity for cross-section 6 under the Scenario 3 flows.

Figure 4-51 Section 6 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

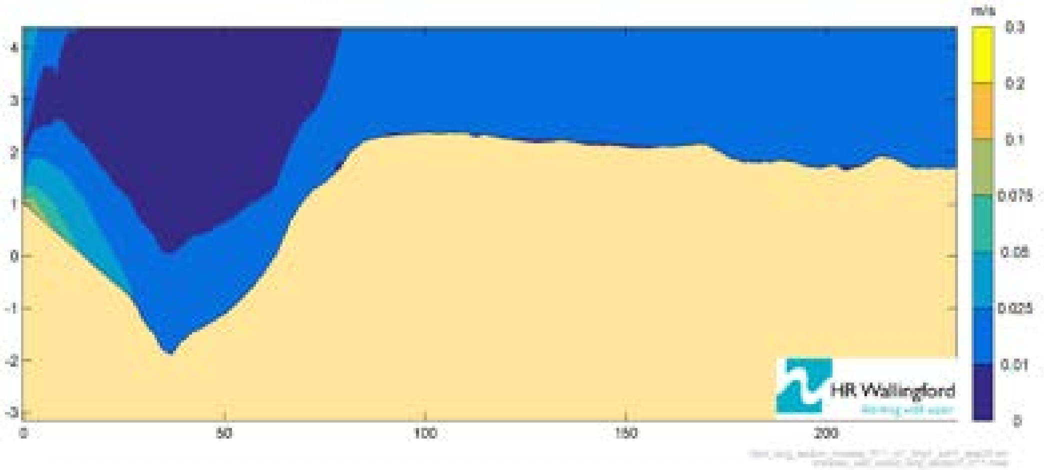


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across all of the cross-section.

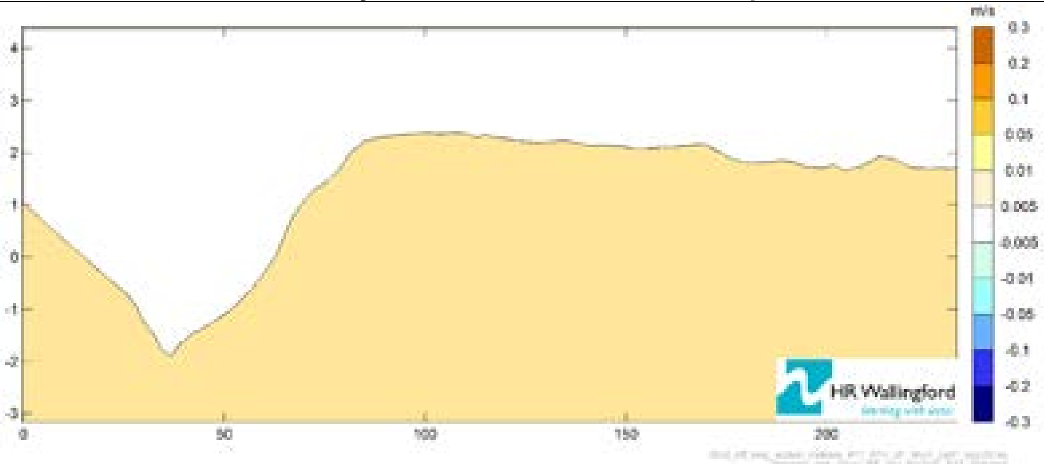
Figure 4-52 shows modelled changes in flow velocity for longitudinal section 1 under the Scenario 3 flows.

Figure 4-52 Longitudinal section 1 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

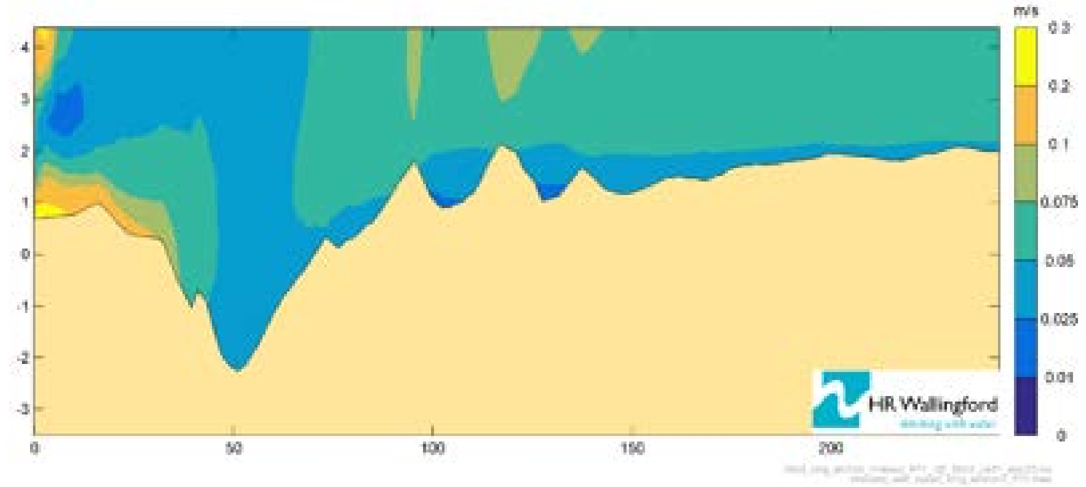


The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across the entire longitudinal section.

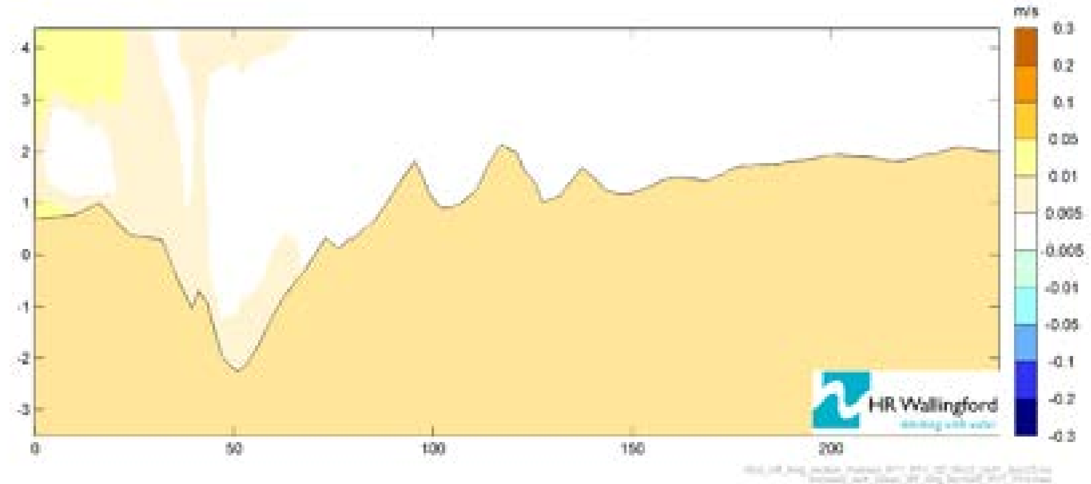
Figure 4-53 shows modelled changes in flow velocity for longitudinal section 2 under the Scenario 3 flows.

Figure 4-53 Longitudinal section 2 flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed



The difference data indicate that there is limited velocity change of between -0.005 – 0.005m/ across the longitudinal section from around 70m chainage downstream of the weir. Velocity increases around the weir and in the weir pool itself of between 0.005-0.05m/s down to around 70m chainage downstream are noted. In comparison to the Scenario 2 simulations, the slightly higher increases in velocity are located towards the flow surface rather than at the upstream edge and base of the weir pool.

4.4.4. Teddington Weir pool

Figure 4-5 and Figure 4-6 show the modelled minimum water levels along the Thames Tideway study reach between downstream of Teddington Weir to Tower Bridge for A82 and M96 baseline flows as well as 200 Ml/d Mogden water recycling scheme release scenarios respectively. Teddington Weir pool is located immediately downstream of Teddington Weir at 0km distance and within Richmond Pound.

Both Figure 4-5 and Figure 4-6 illustrate that there is no change in water levels at the weir pool when the baseline water level is compared with that of the Mogden water recycling scheme.

The 2D/3D Thames Tideway Telemac model has been used to provide predictions of intertidal area exposure and duration of exposure. Exposure and changes against the baseline (outlined in Appendix 1 Table A-1, Figure 4-54 and Figure 4-55) for the 200 Ml/d Mogden water recycling scheme for A82 and M96 runs is shown in Table 4-1.

Table 4-1 200 Ml/d Mogden water recycling scheme change in intertidal area exposure

Reach	Max exposed area (ha) – difference from baseline		Average exposed area (ha) – difference from baseline		Average duration of exposure (hours) – difference from baseline	
	A82	M96	A82	M96	A82	M96
Teddington Weir to Richmond Half-tide Sluice	0	0	0	0	-0.1*	-0.5*
Richmond Half-tide Sluice to Kew Bridge	0.6	0.4	0.2	0.2	-0.1	0
Kew Bridge to Hammersmith Bridge	1.4	1.2	0.2	0.2	-0.1	0
Hammersmith Bridge to Wandsworth Bridge	0.5	0.5	0	0.1	0	-0.1
Wandsworth Bridge to Vauxhall Bridge	-0.2	0.4	0	0	0	0
Vauxhall Bridge to Tower Bridge	0	0	0	0	0	0
Total	2.3	2.5	0.4	0.5	-	-

*Difference in hours due to rounding of significant figures in the exposed area calculations.

Visual representation of the distribution of the percentage of time of intertidal exposure for the M200 scheme A82 and M96 model runs are presented in Figure 4-54 and Figure 4-55 respectively.

Figure 4-54 200 Ml/d Mogden water recycling scheme A82 percentage of time intertidal exposure change against baseline (15 October to 1 November)

Figure 4-55 200 Ml/d Mogden water recycling scheme M96 percentage of time intertidal exposure change against baseline (15 October to 1 November)

The modelled data (Table 4-1, Figure 4-54 and Figure 4-55) indicate that around Teddington Weir there is no change in exposure between the baseline and the scheme, although there is a slight reduction in the duration of exposure by a several minutes.

4.4.5. Richmond Pound

Modelled minimum water levels along the Thames Tideway (Teddington Weir to Richmond Half-tide Sluice) for the baseline and 200 Ml/d Mogden water recycling scheme model runs are presented in Section 4.2.3.

Richmond Pound covers the study reach from the downstream end of Teddington Weir (0km) out to ~5.5km downstream. Both Figure 4-5 and Figure 4-6 (Section 4.2.3) illustrate that there are no changes in water levels under either of the flow scenarios in the pound when water level is compared with that of the 200 Ml/d Mogden water recycling scheme.

The modelled data (Table 4-1, Figure 4-54 and Figure 4-55) indicate that within the pound there is no change in exposure between the baseline and the scheme, although there is a slight reduction in the duration of exposure by a several minutes.

4.4.6. Upper Thames Tideway (Richmond Half-tide Sluice to Battersea)

Modelled minimum water levels along the Thames Tideway (Teddington Weir to Battersea) for the baseline and 200 Ml/d Mogden water recycling scheme model runs are presented in Section 4.2.3.

For the A82 scenario (Figure 4-5, Section 4.2.3), a maximum reduction in level between the baseline and scheme of ~0.05m is seen starting at the downstream side of Richmond Pound. The reduction in water declines with distance along the tideway and is mostly ameliorated to zero change by the end of the reach at ~21km.

For the M96 scenario (Figure Figure 4-6, Section 4.2.3), a maximum reduction in level between the baseline and scheme of ~0.06m is seen starting at the downstream side of Richmond Pound. The reduction in water declines with distance along the tideway and is mostly ameliorated to zero change by the end of the reach at ~21km.

The modelled data (Table 4-1, Figure 4-54 and Figure 4-55) indicate that between Richmond Half-tide Sluice and Wandsworth Bridge there is a limited change in intertidal exposure between the baseline and the 200 Ml/d Mogden water recycling scheme, ranging between a maximum of 0.5-1.4ha for the A82 scenario and 1.2ha for the M96 scenario compared to a maximum baseline intertidal exposure (see Appendix 1 Table A-1) of 62.6ha and 65.3ha respectively. There is generally very limited change in the duration of exposure, at some points reducing by only a few minutes compared to the baseline.

4.4.7. Middle Thames Tideway (Battersea to Tower Bridge)

Modelled minimum water levels along the Thames Tideway (Teddington Weir to Tower Bridge) for the baseline and 200 Ml/d Mogden water recycling scheme model runs are presented in Section 4.2.3.

Both Figure 4-5 and Figure 4-6 illustrates that there is no change in water levels in this reach (between ~21km to ~31km) when the baseline water level is compared with that of the 200 Ml/d Mogden water recycling scheme.

The modelled data (Table 4-1, Figure 4-54 and Figure 4-55) indicate that between Wandsworth Bridge and Tower Bridge there is a very limited change in exposure between the baseline and the scheme, ranging between a maximum of -0.2ha (reduction) for the A82 scenario and 0.4ha change for the M96 scenario. Towards the end of the reach there is no change in exposure. There is no change in the duration of exposure in the reach compared to the baseline.

4.5. SUNBURY WEIR, MOLESEY WEIR AND TEDDINGTON WEIR FISH PASS AND BARRIER PASSABILITY

4.5.1. Overview

Changes in water level within the River Thames can affect the operation of fish passes at Sunbury Weir, Molesey Weir and Teddington Weir. Modelled water level data at each of these weirs under varying river flows and respective Mogden water recycling schemes have been extracted and used to understand changes against the baseline and how these could impact barrier passibility.

4.5.2. Sunbury Weir Fish Pass

Modelled water level at Sunbury Weir under varying river flows and a 200 Ml/d Mogden water recycling scheme release from the Walton discharge are given in Table 4-2. Levels are given as a developed level which represents the water levels under the 200 Ml/d Mogden water recycling scheme release at Walton and the difference in level when compared to the baseline water level. Baseline level data for changes in modelled water level upstream and downstream of Sunbury Weir are presented in Appendix 1 Section 5.2 (Sunbury Weir water levels).

Table 4-2 Modelled changes in water levels at Sunbury Weir under varying river flows for a 200 Ml/d Walton discharge

Sample location	600 Ml/d river flow water level (mAOD)		780 Ml/d river flow water level (mAOD)		950 Ml/d river flow water level (mAOD)	
	Developed level	Difference from baseline	Developed level	Difference from baseline	Developed level	Difference from baseline
S1 (upstream)	8.06	+0.04	8.09	+0.03	8.11	+0.02
S2 (upstream)	8.06	+0.04	8.09	+0.03	8.11	+0.02
S3 (downstream)	6.26	+0.00	6.30	+0.00	6.33	+0.00
S4 (downstream)	6.26	+0.00	6.30	+0.00	6.33	+0.00

The data show that under the three different river flows and the 200 Ml/d outfall release there is minor change in water level of between 0.04m to 0.02m upstream of the weir, with no recorded change downstream of the weir. Assessment of fisheries effects is included in the Fish Assessment Report.

4.5.3. Molesey Weir Fish Pass

Modelled water level at Molesey Weir under varying river flows and a 200 Ml/d release from the Walton discharge are given in Table 4-3. Levels are given as a developed level which represents the water levels under the Walton release and the difference in level when compared to the baseline water level. Baseline level data for changes in modelled water level upstream and downstream of Molesey Weir are presented in Appendix 1 Section 5.2 (Molesey Weir water levels).

Table 4-3 Modelled changes in water levels at Molesey Weir under varying river flows for a 200 Ml/d Walton discharge

Sample location	600 Ml/d river flow water level (mAOD)		780 Ml/d river flow water level (mAOD)		950 Ml/d river flow water level (mAOD)	
	Developed level	Difference from baseline	Developed level	Difference from baseline	Developed level	Difference from baseline
M1 (upstream)	6.26	+0.00	6.30	+0.00	6.33	+0.00
M2 (upstream)	4.38	+0.00	4.38	+0.00	4.38	+0.00
M3 (downstream)	6.26	+0.00	6.30	+0.00	6.33	+0.00
M4 (downstream)	4.38	+0.00	4.38	+0.00	4.38	+0.00

The data show that under the three different river flows and the 200 Ml/d outfall release there is no predicted change in water level upstream or downstream of the weir. Assessment of fisheries effects is included in the Fish Assessment Report.

4.5.4. Teddington Weir Fish Pass

For the Mogden water recycling scheme there would be no change in flows at Teddington Weir as consequence of operation of the scheme for water resources purposes. As there would also be no change in flows at Molesey Weir, there would be no influence of the Mogden water recycling scheme on water level management or water level in the Molesey-Teddington reach of the freshwater River Thames. As such there would be no change in the flow or level at Teddington Weir fish passes. Assessment of fisheries effects is included in the Fish Assessment Report.

4.6. RICHMOND POUND DRAWDOWN PHYSICAL ENVIRONMENT ASSESSMENT

Data for the characterisation of the Richmond Pound physical environment due to changes from the operation of the 200 Ml/d Mogden water recycling scheme have been addressed in several areas in the document, namely wetted habitat change in Section 4.4 (Table 4-1), hydrodynamic changes in Appendix 1 (Section A6) and suspended sediment changes in Section 4.7 (Figure 4-62). These data show that there are very limited changes in wetted habitat, water level and suspended sediment concentration in the pound.

During the November period, tidal level management in Richmond Pound is withdrawn at Richmond half-tide sluice. In order to understand the physical environment within Richmond Pound under these conditions, specific hydrodynamic modelling of the November period has been completed. The results of this modelling for hydrodynamics, wetted habitat and suspended sediment concentration for the Mogden A82 and M96 scenarios are presented in the following sections.

4.6.1. Flow changes and water level

Key modelled hydrodynamic output in the Richmond Pound for assessment of the Mogden water recycling schemes is the effect on water levels. Figure 4-56 and Figure 4-57 show the modelled minimum water levels between downstream of Teddington Weir to immediately seaward of Putney Bridge for A82 and M96 reference conditions and a 200 Ml/d Mogden water recycling scheme during the November drawdown period for Richmond Pound.

Figure 4-56 Minimum water level along the Thames Tideway thalweg between Teddington Weir (0km) and Putney Bridge during A82 flows for reference condition and 200 Ml/d Mogden water recycling scheme during 1 – 30 November period of operation when Richmond Pound is drawndown.

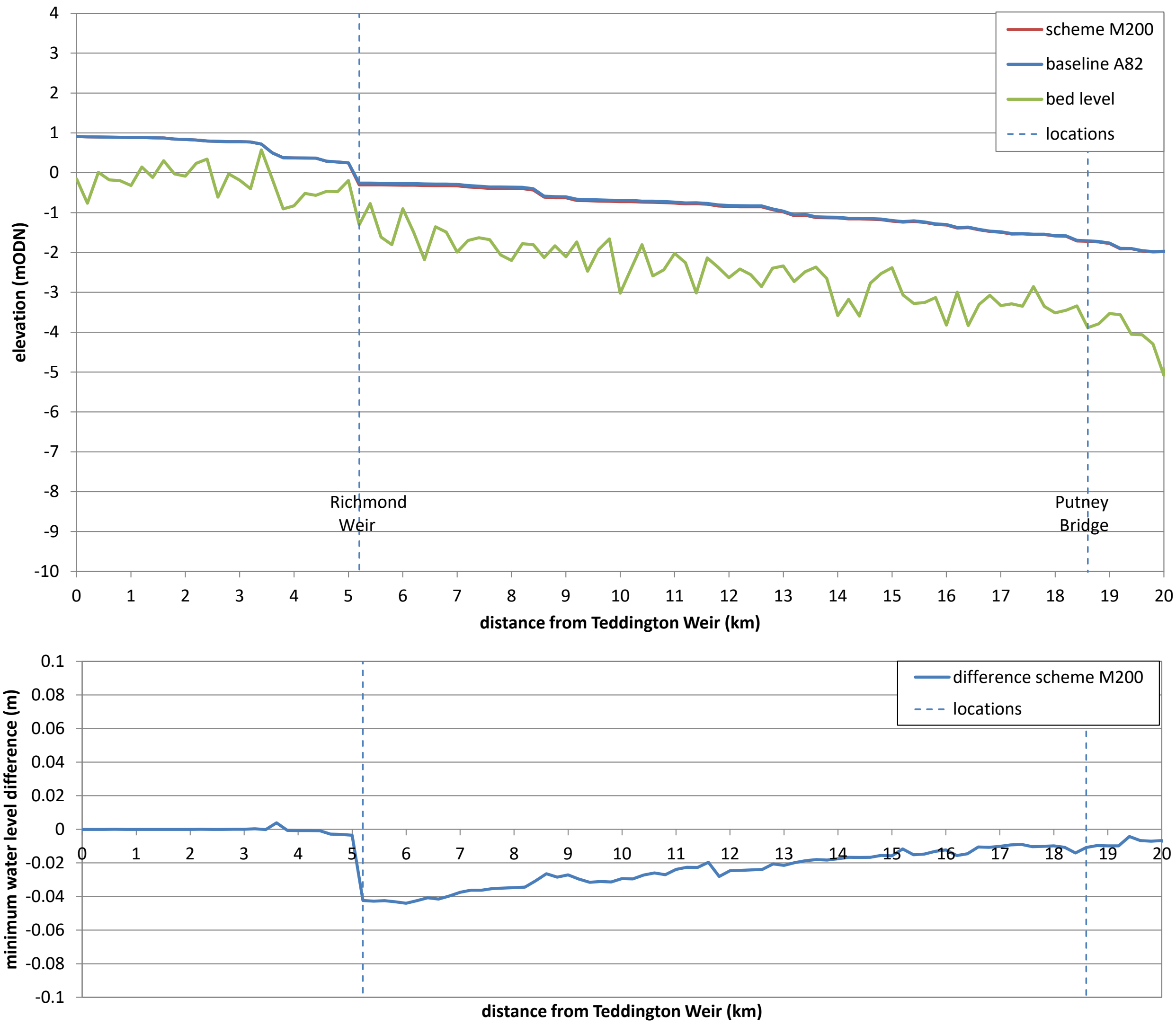
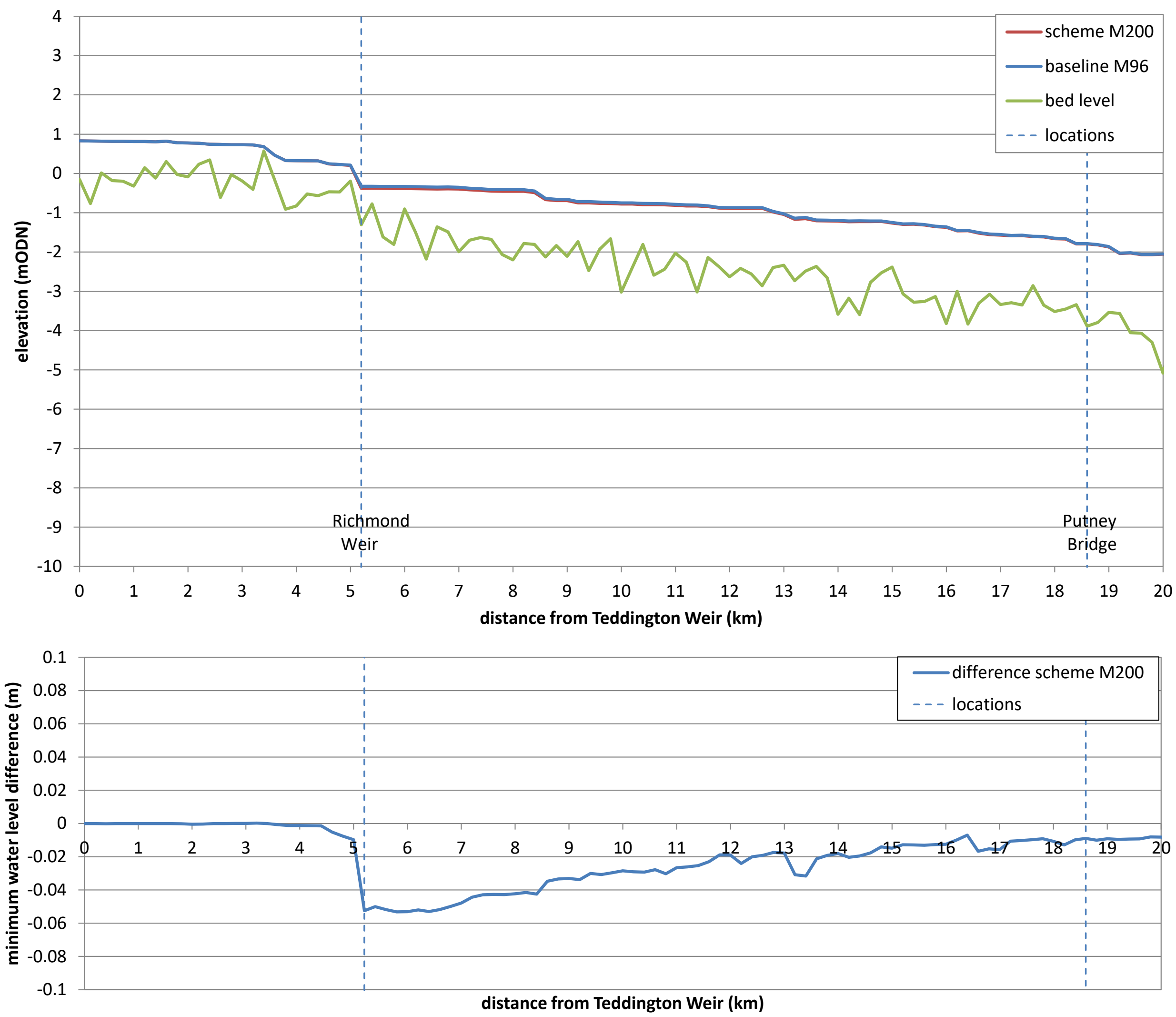


Figure 4-57 Minimum water level along the Thames Tideway thalweg between Teddington Weir (0km) and Putney Bridge during M96 flows for reference condition and 200 Ml/d Mogden water recycling scheme during 1 – 30 November period of operation when Richmond Pound is drawndown.



For the A82 scenario there is a small increase in scheme-baseline difference minimum water level of <0.01m at ~3.6km downstream in the reach. Comparing the A82 and M96 water levels immediately prior to the Richmond sluice (~4.0-5.2km) both show declines to ~0.04m and ~0.06m respectively, however the decline for the M96 scheme begins slightly further upstream (~4.5km compared to ~5.0km for the A82 scheme). These data show that, within the drawndown Richmond Pound, there is essentially no change in baseline and scheme water levels during the November period.

For both the A82 and M96 schemes, the difference between baseline and scheme water levels reach their lowest immediately downstream of Richmond sluice levels (~0.04m and ~0.06m for A82 and M96 respectively), increasing thereafter until beyond 20km downstream. This pattern is seen for A82 and M96 schemes when Richmond Pound is operating (Figure 4-5, Figure 4-6 and Section 4.2.3).

4.6.2. Wetted habitat and exposure

The 2D/3D Thames Tideway Telemac model has been used to provide predictions of the location of intertidal area exposure and duration of exposure. Exposure and changes against the baseline for the November drawdown period (outlined in Appendix 1, Section 6) for the 200 Ml/d Mogden water recycling scheme for A82 and M96 runs are shown in Figure 4-58 and Figure 4-59.

Figure 4-58 Mogden scheme A82 percentage of time intertidal exposure change against baseline (1 – 16 November) during Richmond Pound November drawdown period

Figure 4-59 Mogden scheme M96 percentage of time intertidal exposure change against baseline (14 – 29 November) during Richmond Pound November drawdown period

Apart from a small area of change (~1%) in exposure time in the left channel bifurcation of Eel Pie Island for M96, there is no change when compared to the baseline A82 and M96 time of exposure.

4.6.3. Suspended sediment concentrations

Figure 4-60 and Figure 4-61 illustrate the 95th percentile suspended sediment concentrations (SSC) and change in SSC against the baseline along the thalweg of the Thames Tideway between Teddington Weir and Putney Bridge for the A82 and M96 river flows and the Mogden 200 Ml/d scheme respectively.

Figure 4-60 95th percentile thalweg suspended sediment concentration between Teddington Weir and Putney Bridge for A82 flows and 200 Ml/d Mogden scheme during the 1 - 30 November Richmond Pound drawdown period

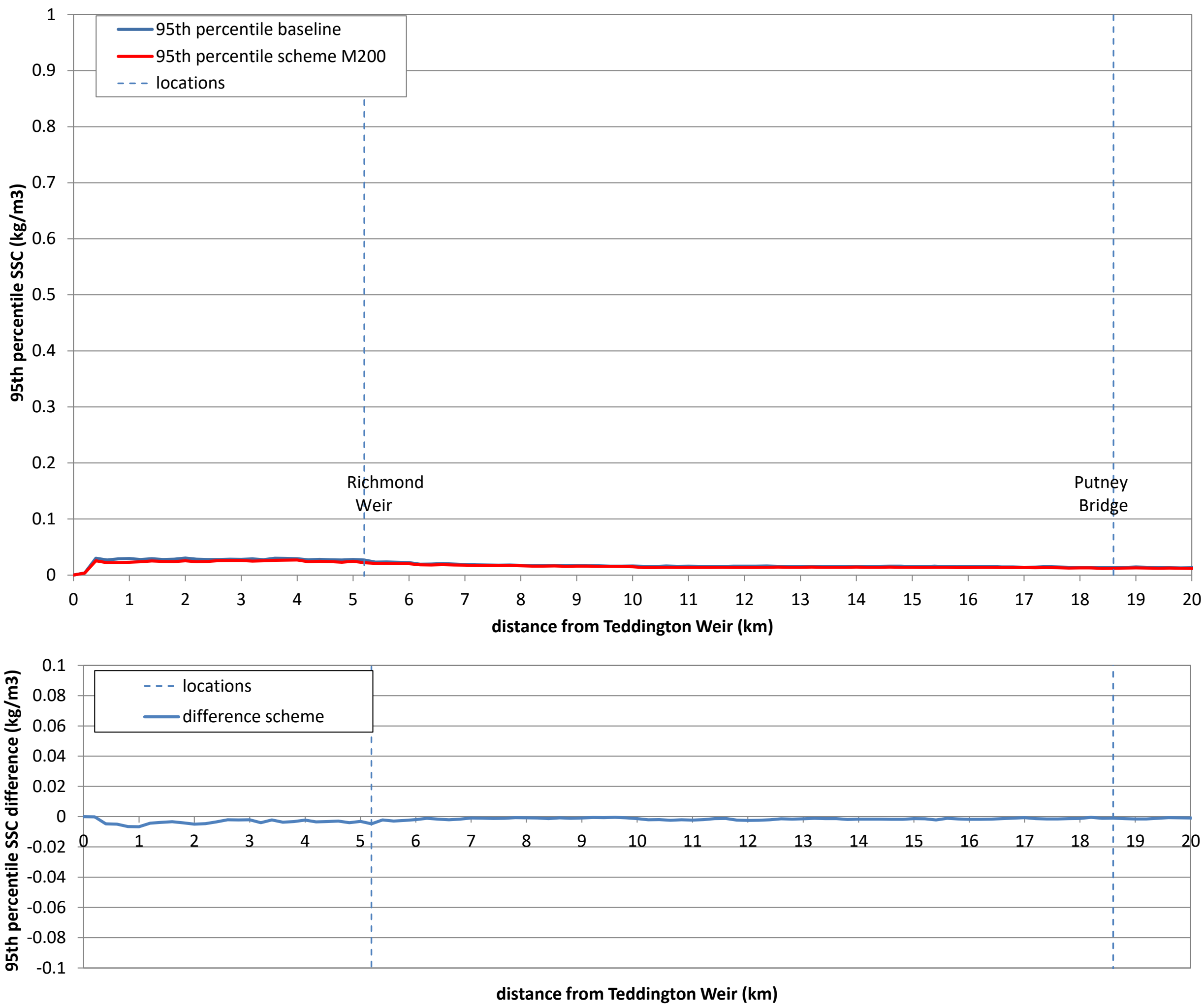
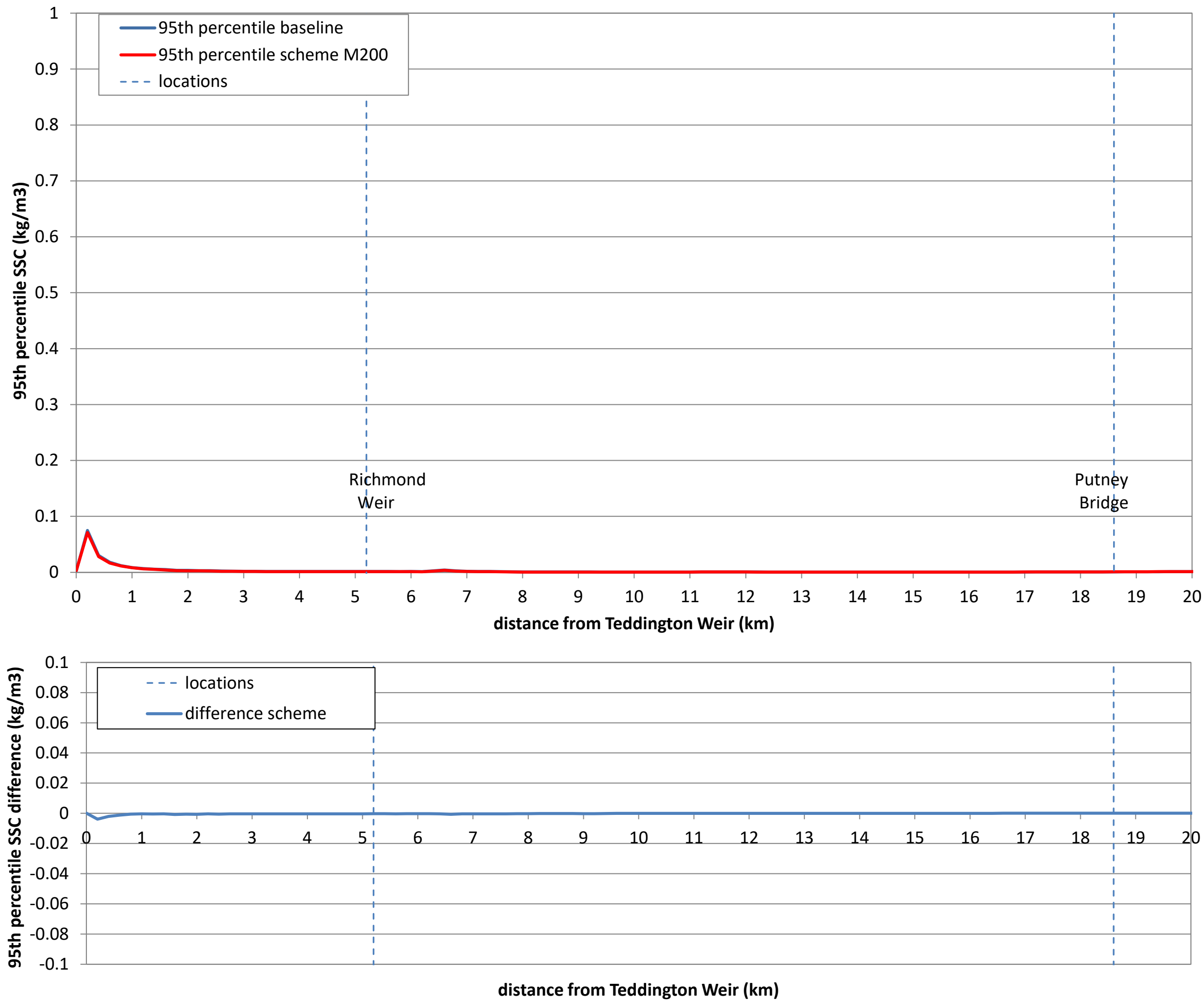


Figure 4-61 95th percentile thalweg suspended sediment concentration between Teddington Weir and Putney Bridge for M96 flows and 200 Ml/d Mogden scheme during the 1 - 30 November Richmond Pound drawdown period



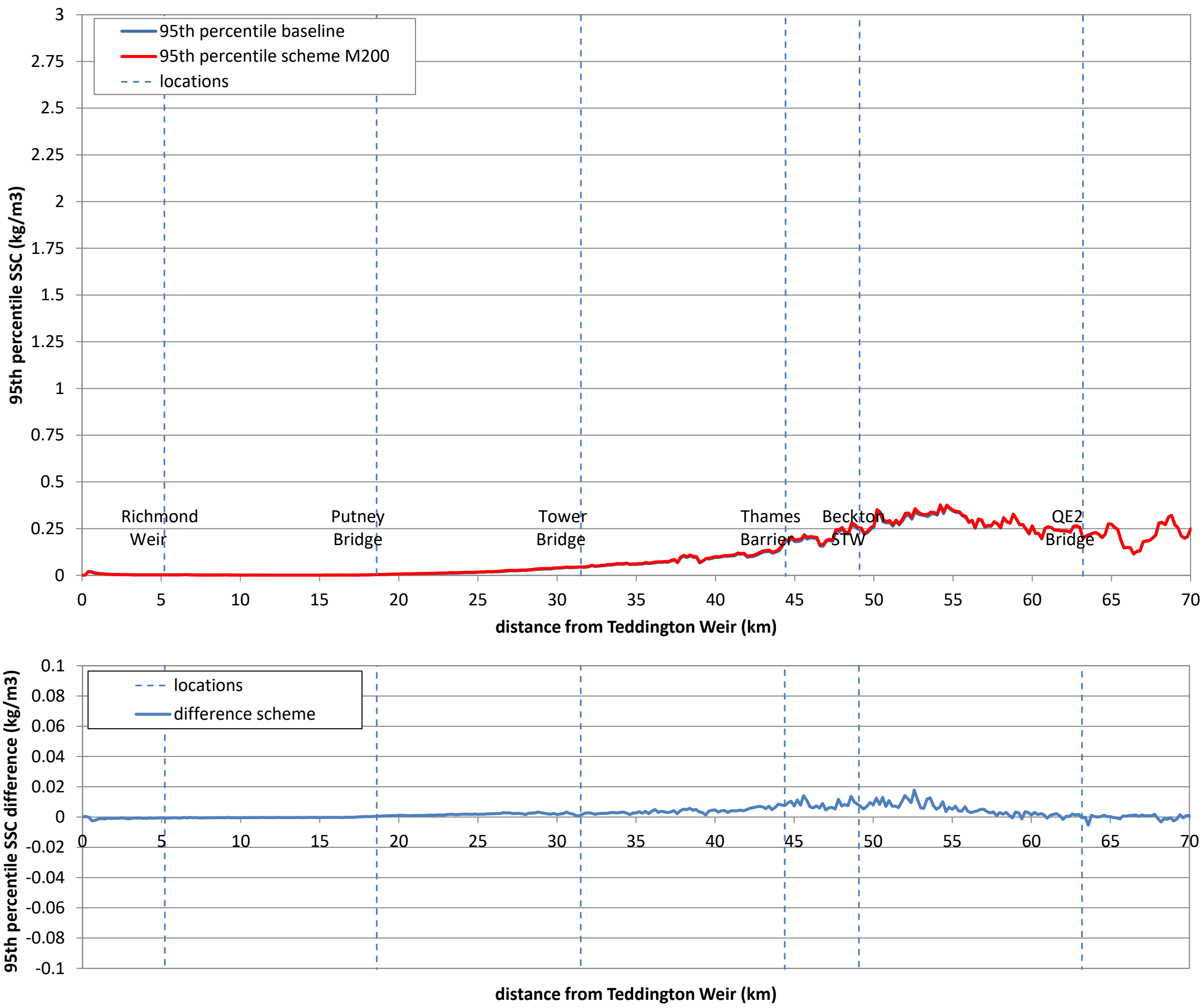
Under the A82 flows, there is an initial increase in SSC immediately downstream of Teddington Weir (concentrations not greater than 0.05kg/m^3, likely related to scour downstream of the weir) followed by a gradual decline in SSC out to Putney Bridge. On the whole the A82 scheme flows sees a slight decrease in SSC of <0.01kg/m3 when compared to baseline within the Pound and out to about 7km. SSC is therefore essentially unchanged against the baseline.

For the M96 flows there is an rapid increase from zero to just less than 0.1kg/m^3 at the base of Teddington Weir (likely related to scour downstream of the weir), followed by an rapid decline to near zero rapidly over the next 1.5km. Generally, M96 sees a slight decrease in SSC of <0.01kg/m3 when compared to baseline just after Teddington Weir and essentially no change in Richmond Pound and out to Putney Bridge thereafter.

4.7. THAMES TIDEWAY ESTUARINE SEDIMENT ASSESSMENT

Figure 4-62 illustrates the 95th percentile SSC and change in SSC against the baseline along the thalweg of the Thames Tideway between Teddington Weir and 3km seaward of the QE2 Bridge for the M96 river flow and the 200 Ml/d Mogden water recycling scheme.

Figure 4-62 95th percentile thalweg suspended sediment concentration for M96 flows for 200 Ml/d Mogden water recycling scheme



The data show that SSC under the 95th percentile baseline and scheme do not exceed 0.5kg/m³ across the entire study reach, with SSC beginning to increase from around Putney Bridge (~18km) until the end of the reach. The highest concentrations are to be found between the Thames Barrier and the end of the reach, which is reflective of the increasing tidal dominance as the estuary nears the North Sea. The data show there is essentially no perceptible change when the 200 Ml/d Mogden water recycling scheme is operating compared to baseline between Teddington Weir (0km) and 40km.

In order to understand how SSC varied on a diurnal basis during a complete spring-neap-spring tidal cycle, model data for the A82 and M96 baselines and the associated 200 Ml/d Mogden water recycling scheme between 15 October and 1 November, extracted from a point in the estuary immediately upstream of Richmond Sluice, were plotted alongside baseline-scheme changes. These data are presented in Figure 4-63 for the A82 scenarios and Figure 4-64 for M96 scenarios.

Figure 4-63 Diurnal SSC and change for A82 baseline and 200 Ml/d Mogden water recycling scheme between 15 October and 1 November

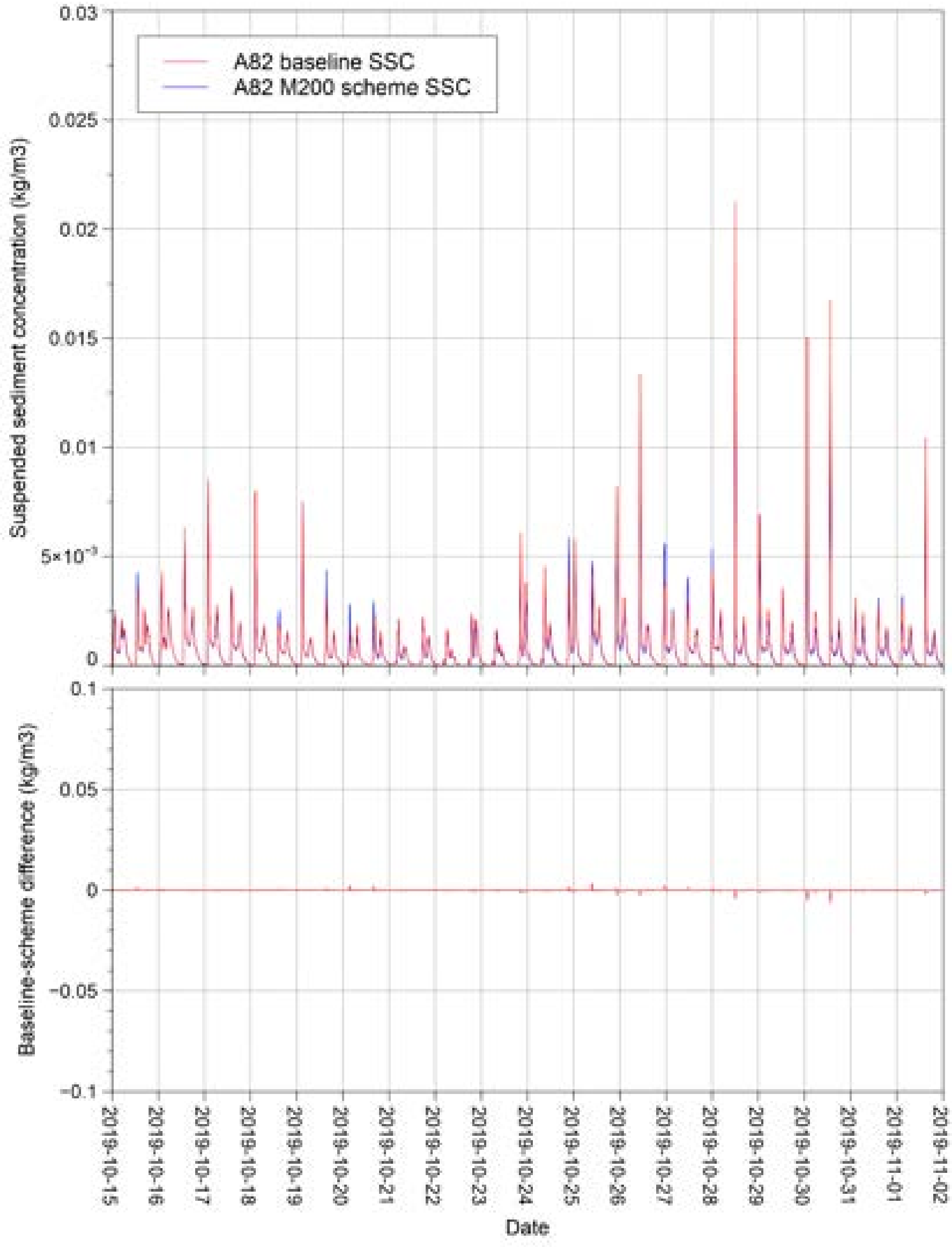
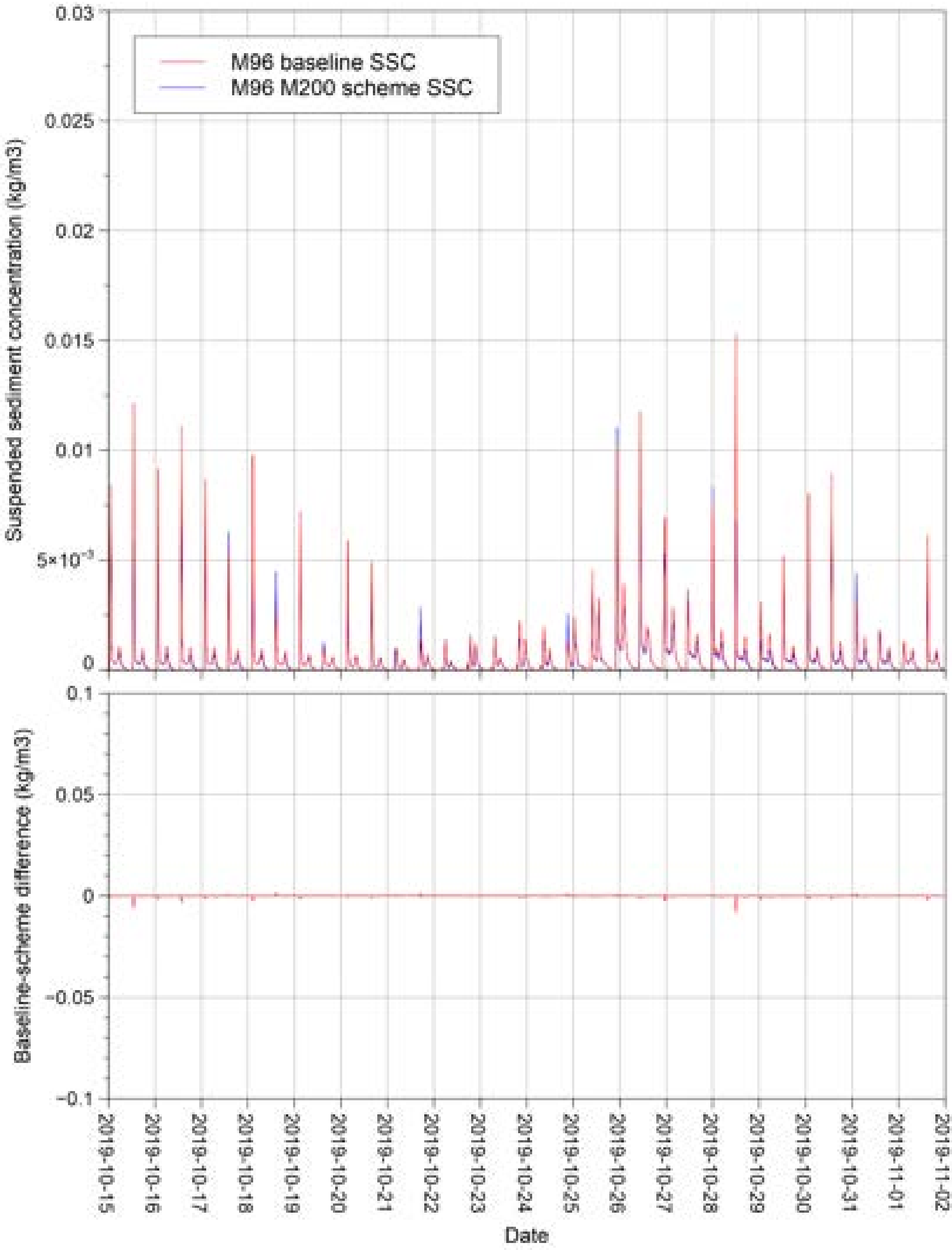


Figure 4-64 Diurnal SSC and change for M96 baseline and 200 Ml/d Mogden water recycling scheme between 15 October and 1 November



For both the A82 and M96 schemes, most change in SSC occurs at the around the highest tide levels. There is effectively negligible change in SSC for both A82 and M96 schemes with generally zero difference between the baseline and scheme SSC. The few changes in SSC which do occur are much less than 0.01kg/m^3.

4.8. SUMMARY OF PHYSICAL ENVIRONMENT ASSESSMENT OF MOGDEN WATER RECYCLING SCHEMES

Table 4-4 summarises the potential physical environment impacts for each of the sizes of a Mogden water recycling scheme.

Table 4-4 Summary of potential physical environment impacts for Mogden water recycling schemes

Size	Flow	Outfall design	Wetted habitat	Fish pass and barrier passability	Richmond Pound drawdown	Estuarine sediment
50 Ml/d	Minor 5% increase in very low flows (Q95) with main flow increase affecting 3.4km reach (Walton Bridge outfall to Walton intake) and no change 5.4km downstream of outfall (Hampton intake)	Negligible Plume velocity characteristics inferred from larger schemes modelling.	Very minor increase in flow velocities in Sunbury Weir pool inferred from larger schemes modelling. No change in wetted habitats modelled in Molesey Weir pool as no expected change in flows over Molesey Weir. Negligible changes in exposure of estuarine wetted habitat inferred from larger schemes modelling.	Negligible change in river levels for scheme when compared to baseline; fisheries conclusions are included in the B.2.3 Fish Assessment Report.	Negligible changes in physical environment within Richmond Pound.	Negligible changes in suspended solids concentration within the estuary.
100 Ml/d	Minor 11% increase in very low flows (Q95) with main flow increase affecting 3.4km reach (Walton Bridge outfall to Walton intake) and no change 5.4km downstream of outfall (Hampton intake)					
150 Ml/d	Moderate 16% increase in very low flows (Q95) with main flow increase affecting 3.4km reach (Walton Bridge outfall to Walton intake) and no change 5.4km downstream of outfall (Hampton intake)					
200 Ml/d	Moderate 21% increase in very low flows (Q95) with main flow increase affecting 3.4km reach (Walton Bridge outfall to Walton intake) and no change 5.4km downstream of outfall (Hampton intake)	Negligible. Increased velocities from plume of (0.05-0.075m/s) stretches downstream to around 260m for discharge into 970Ml/d (Q91) scenario.	Very minor increase in flow velocities in Sunbury Weir pool modelled. No change in wetted habitats modelled in Molesey Weir pool as no expected change in flows over Molesey Weir. Negligible changes in exposure of estuarine wetted habitat.	Negligible change of between 0-0.04m in river levels for scheme when compared to baseline; fisheries conclusions are included in the B.2.3 Fish Assessment Report.		

In conclusion, the Mogden water recycling schemes may lead to up to moderate impacts on flows when compared to the baseline conditions in the River Thames. However, these changes are negligible when considering impacts to water level depth and average flow velocities. Additionally, the data indicates that there are negligible impacts on fish pass barrier passibility, negligible impacts on the Richmond Pound and on wetted habitat, water level and suspended sediment concentration in the Thames Tideway.

As stated in Section 1.1.3, the Mogden South Sewer scheme design and assessment has not been progressed through Gate 2. However, due to the similarities with the 50 Ml/d Mogden water recycling scheme (AWRP, discharge location and volume), the outcomes presented above for the 50 Ml/d scheme assessment can be considered representative of a physical environment assessment of a 50 Ml/d Mogden South Sewer scheme.

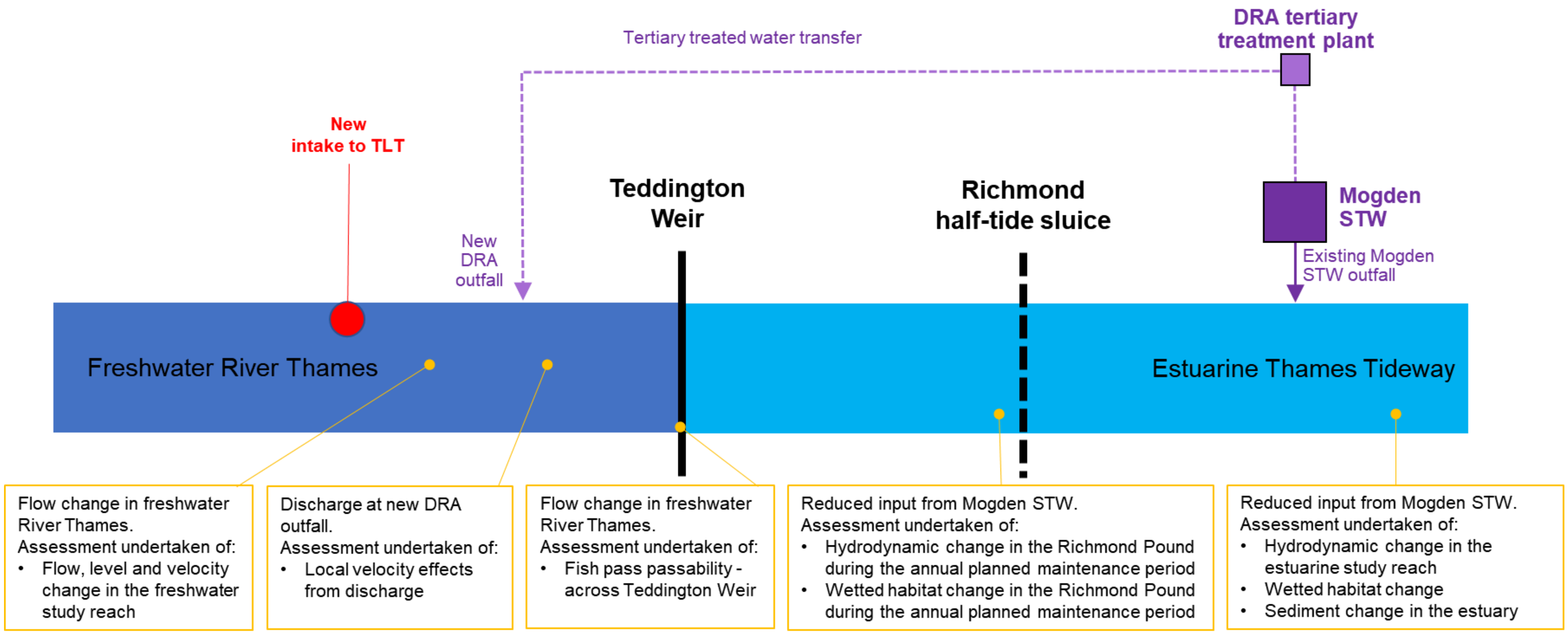
5. PHYSICAL ENVIRONMENT ASSESSMENT OF TEDDINGTON DRA SCHEME

5.1. INTRODUCTION

Specific to the Teddington DRA scheme, the assessment for each of the tasks set out in Table 1-1 is set out in this section. As set out spatially in the conceptualisation of physical environment effects in Figure 5-1, the specific assessments of the Teddington DRA scheme are:

- Flow changes from Teddington DRA scheme
- Review of Teddington DRA outfall and intake design including screening
- Wetted habitat change in freshwater River Thames and estuarine Thames Tideway
- Teddington Weir fish pass and barrier passability
- Richmond Pound drawdown physical environment assessment
- Thames Tideway estuarine sediment assessment
- Summary of physical environment assessment of Teddington DRA scheme.

Figure 5-1 Representation of the Teddington DRA aquatic study area with conceptualisation of physical environment effects and listing of assessment undertaken for Gate 2



To support the environmental assessments at Gate 2, an indicative operating pattern has been developed. For Teddington DRA schemes this mirrors the patterns described for Mogden water recycling schemes in Section 4.1. Outside the normal operating pattern the Gate 2 engineering design includes a 25% tunnel maintenance flow at all times, with the treated water being discharged to the River Thames at Teddington but without corresponding abstraction.

5.2. FLOW CHANGES FROM TEDDINGTON DRA SCHEMES

5.2.1. Overview

A Teddington DRA scheme would abstract flows from the freshwater River Thames locally upstream of Teddington Weir by 50, 75, 100 or 150 Ml/d (dependent on scheme assessed) when in use for water resources purposes. Correspondingly, the scheme would discharge at the same rate, 250m downstream, and at 25% of the scheme rate at all other times. The scheme would only be triggered for operation at Teddington Target Flows of 700 Ml/d or lower and as such the scheme would only affect flows under low to very low flow periods in the River Thames.

Final effluent flows from Mogden STW discharged to the estuarine Thames Tideway at Isleworth Ait would reduce by the corresponding amount to the amount transferred to the freshwater River Thames at Teddington Weir.

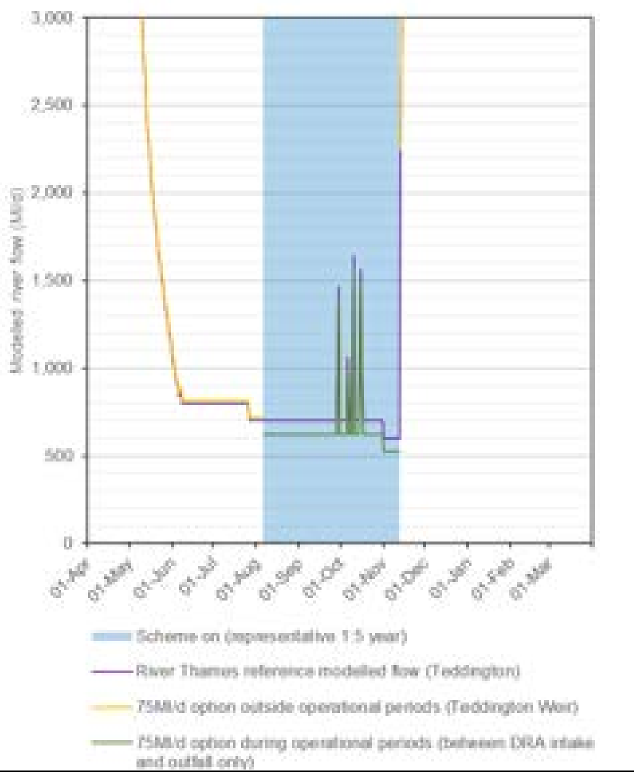
5.2.2. Freshwater River Thames

Selected representative years have been used to show an indicative flow pattern along the River Thames from Walton Bridge to Teddington Weir in Figure 5-2 with a 75 Ml/d Teddington DRA scheme. It is important to note that when operational for water resources purposes (scheme on period) flow changes associated with a Teddington DRA scheme (i.e. 150 Ml/d for a 150 Ml/d Teddington DRA scheme; 100 Ml/d for a 100 Ml/d Teddington DRA scheme; 75 Ml/d for a 75 Ml/d Teddington DRA scheme; and 50 Ml/d for a 50 Ml/d Teddington DRA scheme) would be exclusively within the ~250m reach between the intake and outfall, with no change at Teddington Weir. When the scheme is not on for water resources purposes, the 25% maintenance flow (i.e. 37.5 Ml/d for a 150 Ml/d Teddington DRA scheme; 25.0 Ml/d for a 100 Ml/d Teddington DRA scheme; 18.75 Ml/d for a 75 Ml/d Teddington DRA scheme; and 12.5 Ml/d for a 50 Ml/d Teddington DRA scheme) flow changes associated with a Teddington DRA scheme would be exclusively flow increases downstream of the outfall to Teddington Weir, with no change between the intake and outfall.

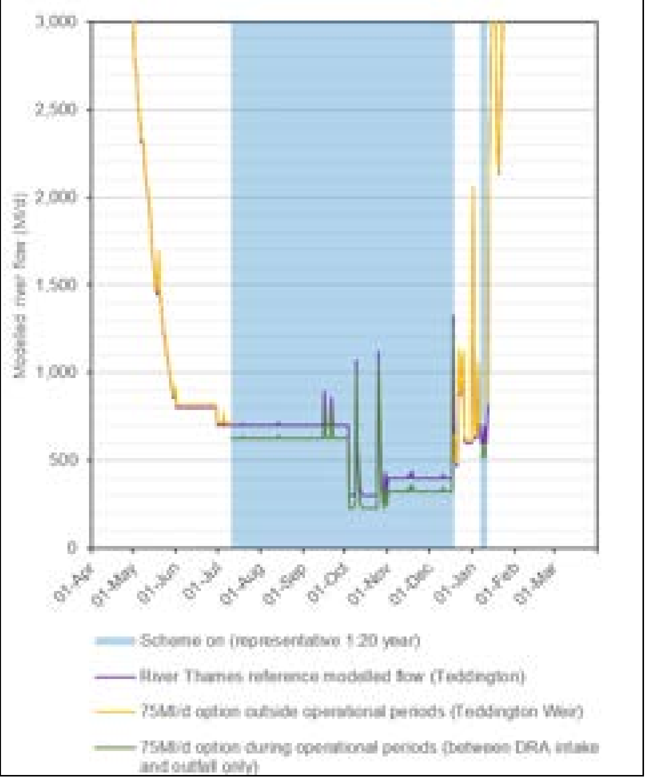
Reference condition flows in the River Thames at Teddington Weir are lowest during the representative scheme on periods of summer and autumn. For the selected 1:5 year return period the lowest modelled flows at Teddington Weir are 600 Ml/d for 12 dates in November. For the selected 1:20 year return period the lowest modelled flows at Teddington Weir are 300 Ml/d, for 17 dates in October. There are also periods of low flow as flows in the River Thames recede in late spring/ early summer prior to the representative scheme on periods. However, in general, outside the representative scheme on periods river flows are much higher – to a peak of 15,000 Ml/d in the A82 scenario and 25,000 Ml/d in the M96 scenario, noting the flow axis is truncated in Figure 5-2.

Figure 5-2 Flow in the freshwater River Thames used for modelled assessment of 75 Ml/d Teddington DRA scenarios

a) Locally at Teddington Weir; A82 1:5 moderate-low flow scenario



b) Locally at Teddington Weir; M96 1:20 very-low flow scenario

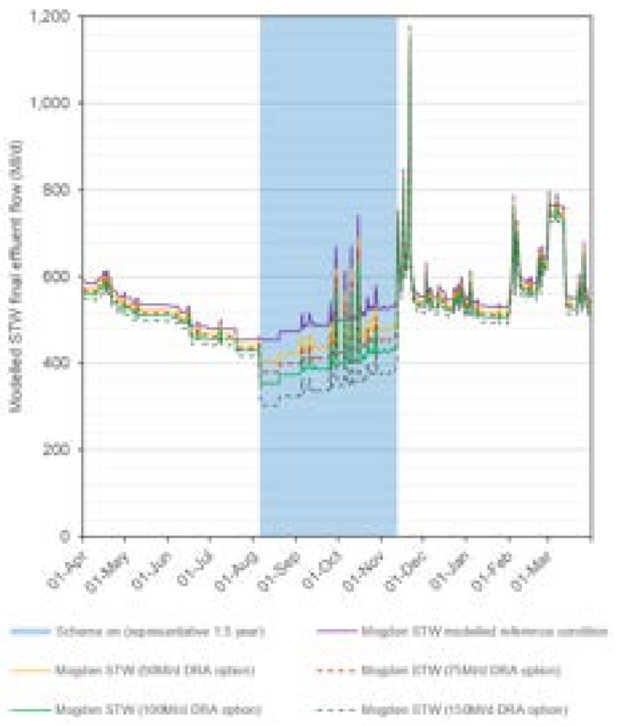


5.2.3. Estuarine Thames Tideway

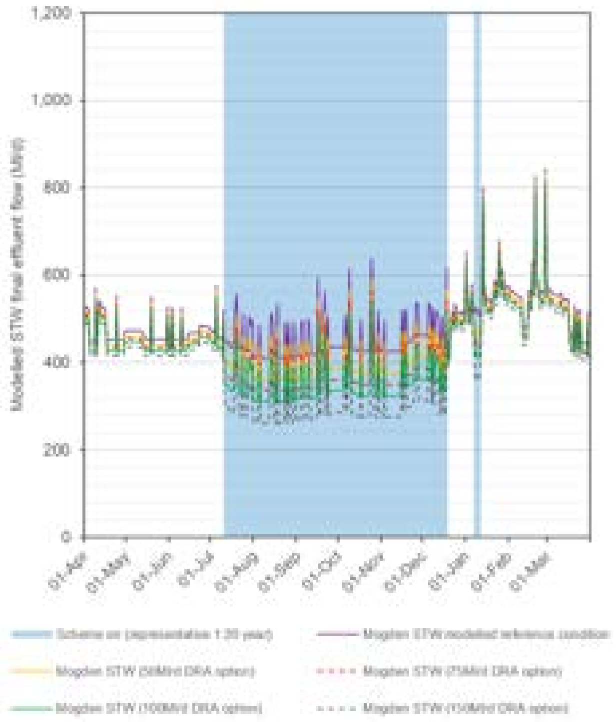
Estuarine hydrodynamics assessment has been undertaken for both the A82 and M96 representative model years with a 200 Ml/d Mogden water recycling scheme in Section 4.2.3. This represents a greater reduction in effluent contribution from Mogden STW to the upper Thames Tideway than for the sizes of the Teddington DRA schemes (max of 150 Ml/d). A flow series has been derived for Mogden STW final effluent based off measured effluent flow rates at the STW and the daily flow characteristics locally in west London in the model years. Modelled effluent flow rates are shown in Figure 5-3 for the sizes of Teddington DRA scheme.

Figure 5-3 Mogden STW final effluent flow rates used for modelled assessment of Teddington DRA scenarios

a) Mogden STW final effluent; A82 1:5 moderate-low flow scenario



b) Mogden STW final effluent; M96 1:20 very-low flow scenario



In the A82 scenario during the scheme on period, modelled Mogden STW reference condition flows are 504 Ml/d (daily mean). A Teddington DRA scheme would reduce these flows as follows:

- A 150 Ml/d Teddington DRA scheme would reduce these flows by 150 Ml/d, a 30% reduction.
- A 100 Ml/d Teddington DRA scheme would reduce these flows by 100 Ml/d, a 20% reduction.
- A 75 Ml/d Teddington DRA scheme would reduce these flows by 75 Ml/d, a 15% reduction.
- A 50 Ml/d Teddington DRA scheme would reduce these flows by 50 Ml/d, a 10% reduction.

In the M96 scenario during the scheme on period, modelled Mogden STW reference condition flows are 458 Ml/d (daily mean). A Teddington DRA scheme would reduce these flows as follows:

- A 150 Ml/d Teddington DRA scheme would reduce these flows by 150 Ml/d, a 33% reduction.
- A 100 Ml/d Teddington DRA scheme would reduce these flows by 100 Ml/d, a 22% reduction.
- A 75 Ml/d Teddington DRA scheme would reduce these flows by 75 Ml/d, a 16% reduction.
- A 50 Ml/d Teddington DRA scheme would reduce these flows by 50 Ml/d, a 11% reduction.

With regards potential changes in water level in the Thames Tideway as consequence of Mogden STW final effluent flow reductions of 50 Ml/d to 150 Ml/d from a Teddington DRA scheme, these have not been directly modelled. Information from the modelling of the larger effluent flow reduction, a 200 Ml/d effluent flow reduction, described in Section 4.2.3 is used to describe that any changes in minimum (low tide) water levels resulting from a Teddington DRA scheme would be considerably less than 6cm and with greatest effect centred around Isleworth Ait, and no effect extending into the Richmond Pound at times of operation of the Richmond half-tide sluice in all months except November.

5.3. REVIEW OF TEDDINGTON DRA OUTFALL AND INTAKE DESIGN INCLUDING SCREENING

5.3.1. Overview

In accordance with the approach set out in in Table 1-1, the change in velocity pattern at the Teddington DRA outfall has been assessed through 3D modelling.

5.3.2. Teddington DRA Outfall and Intake in the Freshwater River Thames

The effects on the hydrodynamics of the River Thames around the location of the proposed Teddington DRA outfall have been simulated using a 3D TELEMAC model. The modelling uses the scenarios outlined in Appendix 1 (Section 3.2): 300 Ml/d river flow (Scenario 1), 400 Ml/d river flow (Scenario 2) and 700 Ml/d river flow (Scenario 3), with an outfall discharge of 75 Ml/d moving at 0.3m/s and the intake abstracting 75 Ml/d. The results of the model runs for each scenario are presented below. The baseline model flow velocity predictions are outlined in Appendix 1 Section 3.2.

Additional modelling of increased abstraction and discharge volumes for the Teddington DRA were undertaken for a Scenario 2 river flow of 400 Ml/d using the 3D TELEMAC model. For both of these additional scenarios an outfall discharge and intake abstraction of either 100 Ml/d or 150 Ml/d, moving at 0.3m/s, were simulated. The results of these model runs are presented separately from the 75 Ml/d results, below. The baseline model flow velocity predictions for the 400 Ml/D river flow are outlined in Appendix 1 Section 3.2.

Scenario 1: 300 Ml/d river flow

The depth-average velocity around the Teddington DRA outfall and intake under Scenario 1 conditions and the velocity differences between this and the baseline are presented in Figure 5-4.

Figure 5-4 Depth-average velocity at the Teddington DRA outfall and intake, 300 Ml/d, Scenario 1 (75 Ml/d outfall discharge and 75 Ml/d intake)

a) Scenario 1 developed flow velocity

b) Scenario 1 difference in flow velocity between baseline and developed

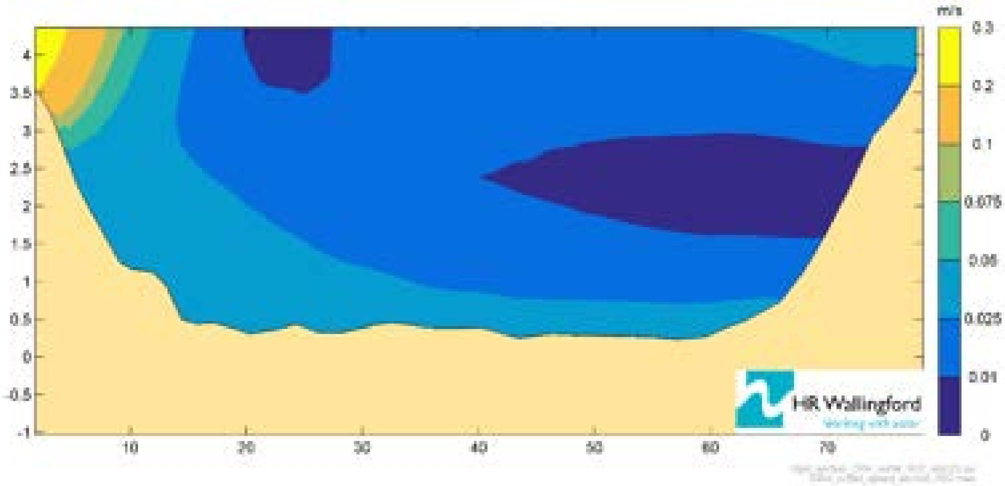

Under the developed Scenario 1 simulation a spatially limited increase in flow velocities occurs at the point of discharge (0.1-0.3m/s), with flow velocities increasing by 0.05-0.075m/s immediately downstream of the outfall and concentrated against the right bank for ~100m downstream. Generally, the velocities across the channel range from 0-0.025m/s, indicating still to very slow-moving flow. Apart from at the intake itself, there are no significant changes in flow velocities here. Velocity vectors remain predominantly in a downstream direction, although show a slight deflection towards the outfall as upstream flow passes by.

The modelled difference shows that velocities in the majority of the channel upstream and downstream of the outfall vary by around -0.005 – 0.005m/s. Generally, in a small area around the outfall (~10m upstream and ~100m downstream), flow velocity increase by around 0.01-0.05m/s. Decreases of between -0.05 - -0.01m/s are noted over a similar area on the left bank adjacent to the outfall.

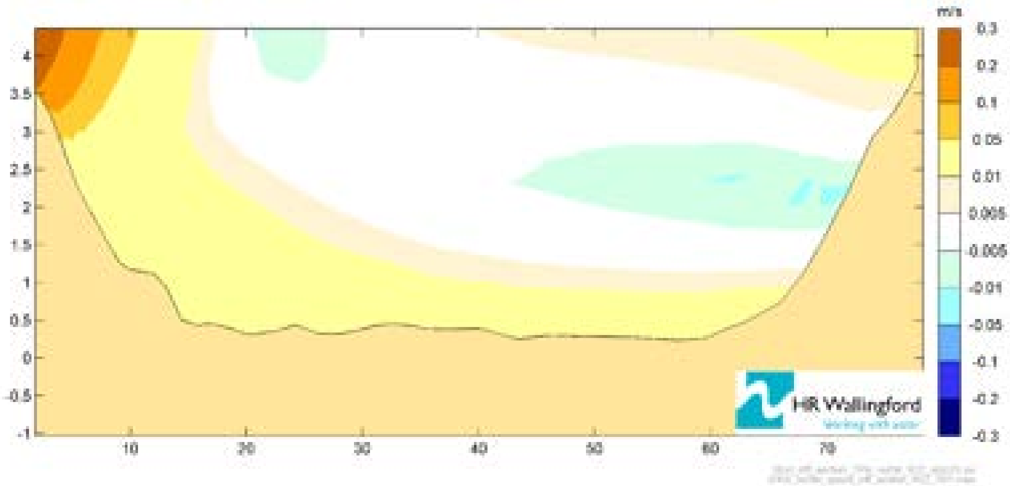
Figure 5-5 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Teddington DRA outfall under the Scenario 1 flows.

Figure 5-5 Depth-average velocity at the Teddington DRA outfall cross-section, 300 Ml/d, Scenario 1 (75 Ml/d outfall discharge and 75 Ml/d intake)

a) Scenario 1 developed flow velocity



b) Scenario 1 difference in flow velocity between baseline and developed



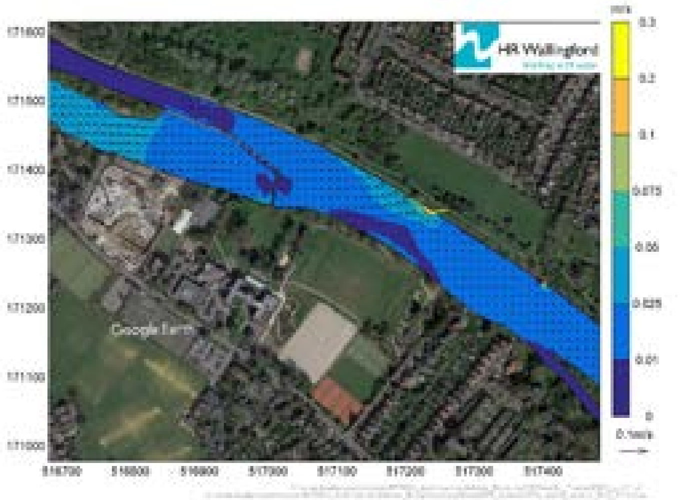
The difference data show that velocities peak around the outfall (left side of the cross-section) at around 0.05-0.3m/s and extend to a maximum of ~10m out into the channel. The modelling suggests that there is an increase in flow velocity of between 0.01-0.05m/s out to around 20m, across much of the channel bed and at the water surface towards and at the left bank (40-80m chainage). The remainder of the channel cross-section shows a range of minimal decreases and increases in velocity from -0.05 – 0.005m/s, with decreases in flow concentrated in the middle of the flow cross-section on the left bank (40-70m chainage).

Scenario 2: 400 Ml/d river flow

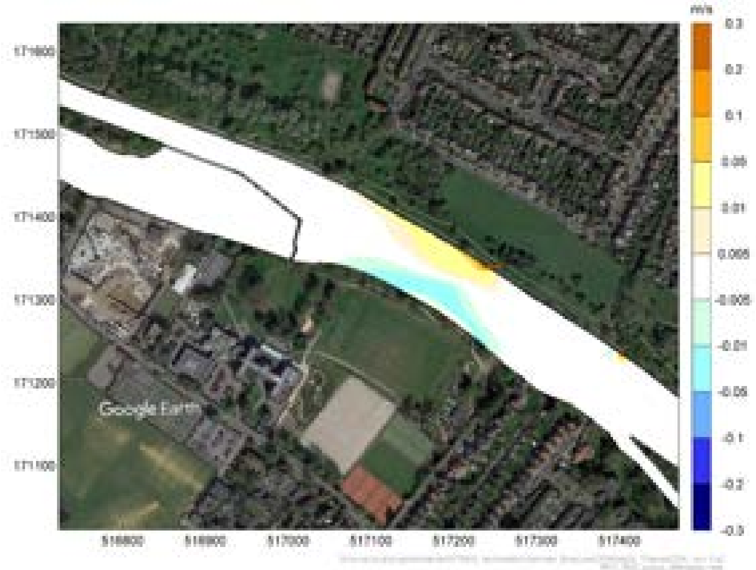
The depth-average velocity around the Teddington DRA outfall under Scenario 2 conditions and the velocity differences between this and the baseline are presented in Figure 5-6.

Figure 5-6 Depth-average velocity at the Teddington DRA outfall and intake, 400 Ml/d, Scenario 2 (75 Ml/d outfall discharge and 75 Ml/d intake)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed

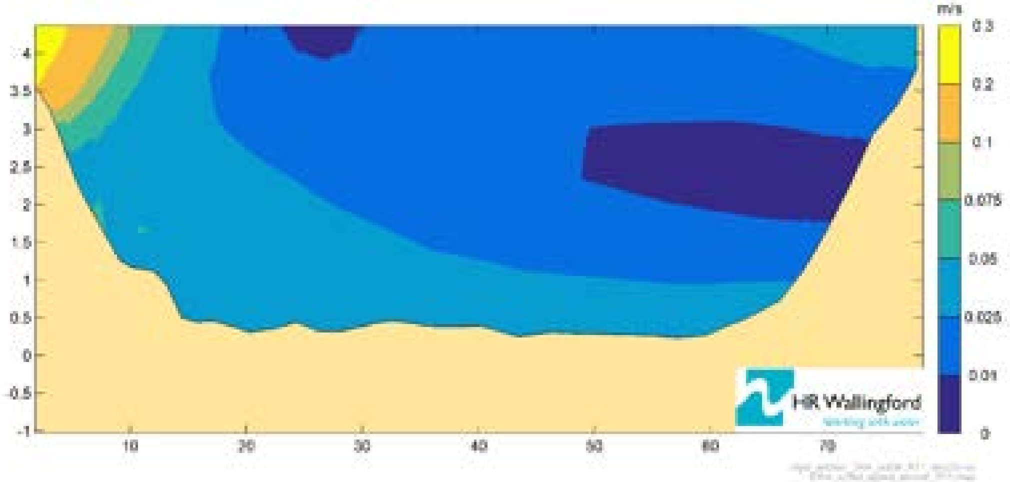


Under the developed Scenario 2, flow velocities and patterns remain essentially unchanged from those simulated in Scenario 1.

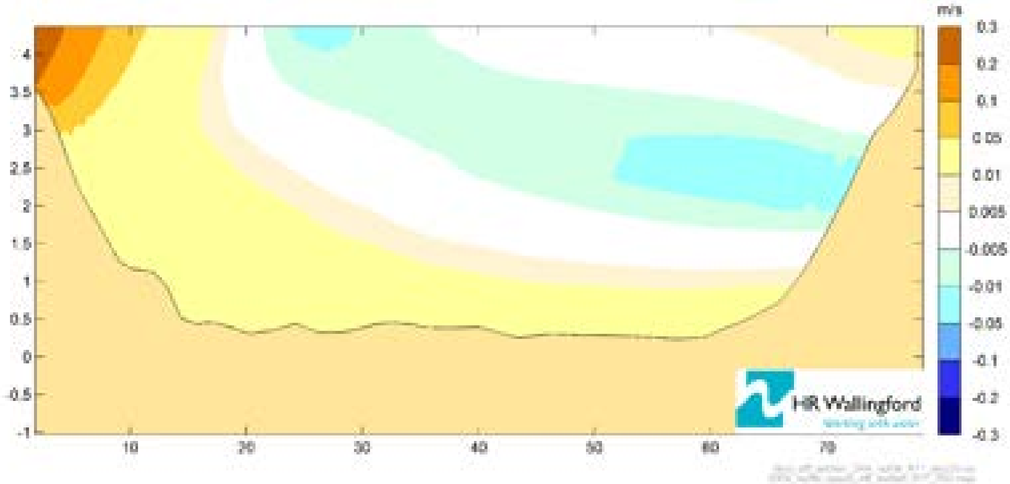
Figure 5-7 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Teddington DRA outfall under the Scenario 2 flows.

Figure 5-7 Depth-average velocity at the Teddington DRA outfall cross-section, 400 Ml/d, Scenario 2 (75 Ml/d outfall discharge and 75 Ml/d intake)

a) Scenario 2 developed flow velocity



b) Scenario 2 difference in flow velocity between baseline and developed



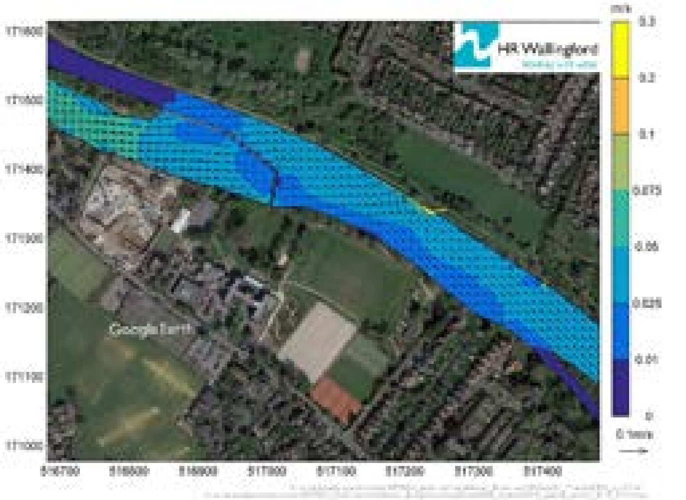
The difference data remain relatively unchanged from that of Scenario 1, showing velocities peaking around the outfall (left side of the cross-section). The higher velocities noted around the channel bed encroach further up the vertical channel profile, while the reduction in velocities noted at the left bank in Scenario 1 become more prevalent through the flow cross-section. On the whole, flow velocities remain very low.

Scenario 3: 700 Ml/d river flow

The depth-average velocity around the Teddington DRA outfall under Scenario 3 conditions and the velocity differences between this and the baseline are presented in Figure 5-8.

Figure 5-8 Depth-average velocity at the Teddington DRA outfall and intake, 700 Ml/d, Scenario 3 (75 Ml/d outfall discharge and 75 Ml/d intake)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed

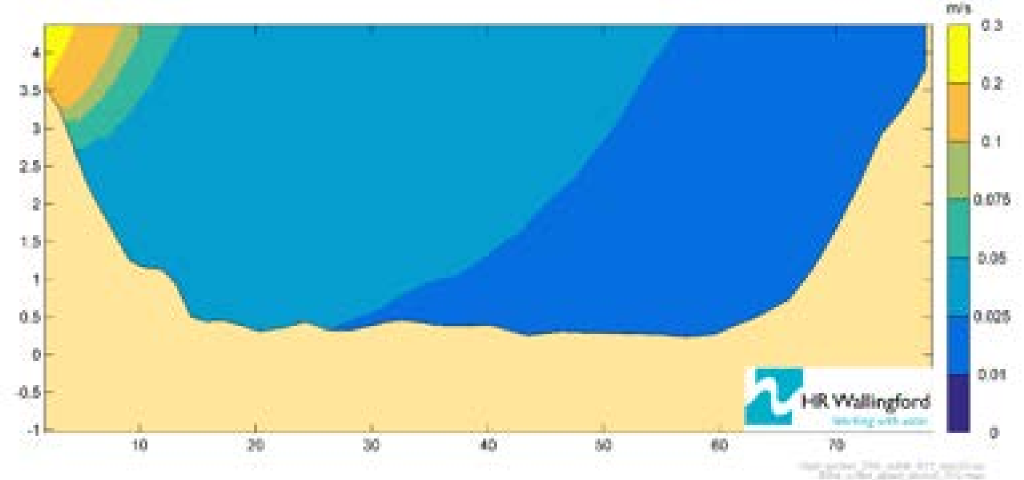
Under the developed Scenario 3 simulation river flow velocities have increased to between 0.025-0.05m/s, when compared to the earlier scenarios. Higher flow velocities (0.05-0.075m/s) are noted on the right bank than the left (0.025-0.05m/s). The higher velocities seen around and downstream of the outfall in previous scenarios are not present here, likely due to the higher flow velocities of the main channel, although locally elevated velocities next to the outfall (0.1-0.3m/s) remain present. Velocity vectors remain predominantly in a downstream direction, although some slight deflection towards the outfall as upstream flow passes by remains.

The modelled differences in velocity remain similar to those in Scenario 1 and Scenario 2, although the reduced flow velocities on the left bank appear to cover more of the channel laterally and longitudinally than in previous scenarios.

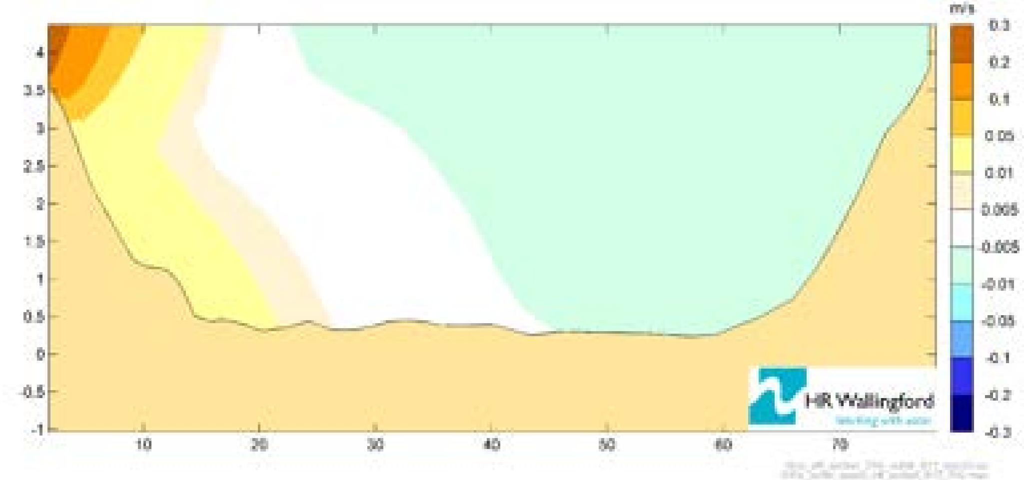
Figure 5-9 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Teddington DRA outfall under the Scenario 3 flows.

Figure 5-9 Depth-average velocity at the Teddington DRA outfall cross-section, 700 Ml/d, Scenario 3 (75 Ml/d outfall discharge and 75 Ml/d intake)

a) Scenario 3 developed flow velocity



b) Scenario 3 difference in flow velocity between baseline and developed



As for the earlier scenarios the difference data show that velocities peak around the outfall (left side of the cross-section) at around 0.05-0.3m/s and extend to a maximum of ~10m out into the channel. The modelling suggests that there is an increase in flow velocity of between 0.01-0.05m/s out to around 20m across most of the vertical channel profile. In contrast to earlier scenarios, the remainder of the channel shows either limited change in velocity (-0.005 – 0.005m/s) or a reduction in velocity (-0.01 - -0.005m/s).

Scenario 2: 400 Ml/d river flow with 100Ml/d and 150Ml/d schemes

The depth-average velocity and the velocity differences between this and the baseline around the Teddington DRA outfall and intake under Scenario 2 river flow conditions of 400 Ml/d, simulating a 100 Ml/d abstraction and 100 Ml/d discharge, are presented in Figure 5-10.

Figure 5-10 Depth-average velocity at the Teddington DRA outfall and intake, 400 Ml/d river flow, 100 Ml/d outfall discharge and 100 Ml/d intake

a) Developed flow velocity

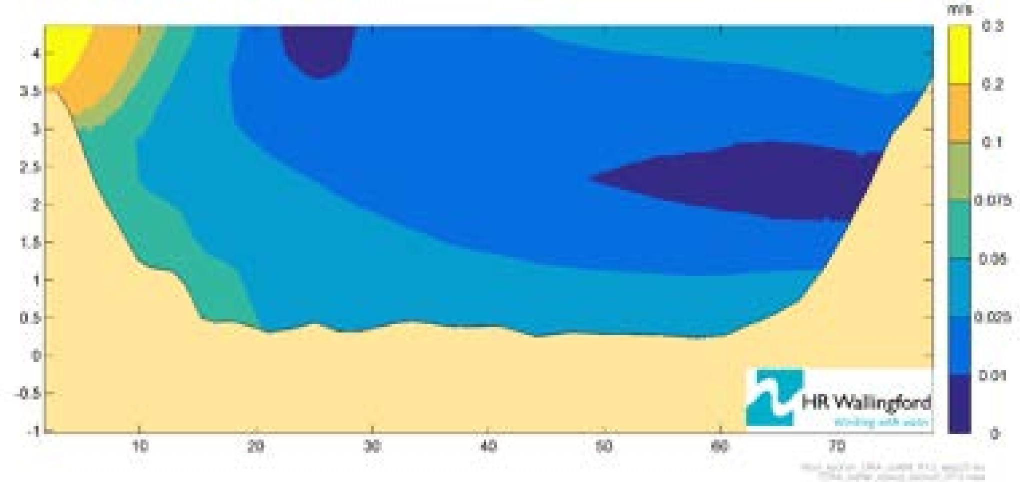
b) Difference in flow velocity between baseline and developed

Under the developed simulation a spatially limited increase in flow velocities occurs at the point of discharge (0.1-0.3m/s), with flow velocities increasing by 0.05-0.075m/s immediately downstream of the outfall and concentrated against the right bank for ~200m downstream. Flow velocities are lower on the left bank directly opposite the outfall between 0-0.01m/s. Generally, the velocities across the channel range from 0-0.025m/s, indicating still to very slow-moving flow. Velocity vectors remain predominantly in a downstream direction, although show a slight deflection towards the outfall as upstream flow passes by. There is very little difference when compared to the Scenario 2 75 M/d scheme, with the exception of very slightly elevated velocities in the channel downstream of the outfall (c.f. the longer velocity vectors), although these remain within the same velocity group (0.01-0.025m/s).

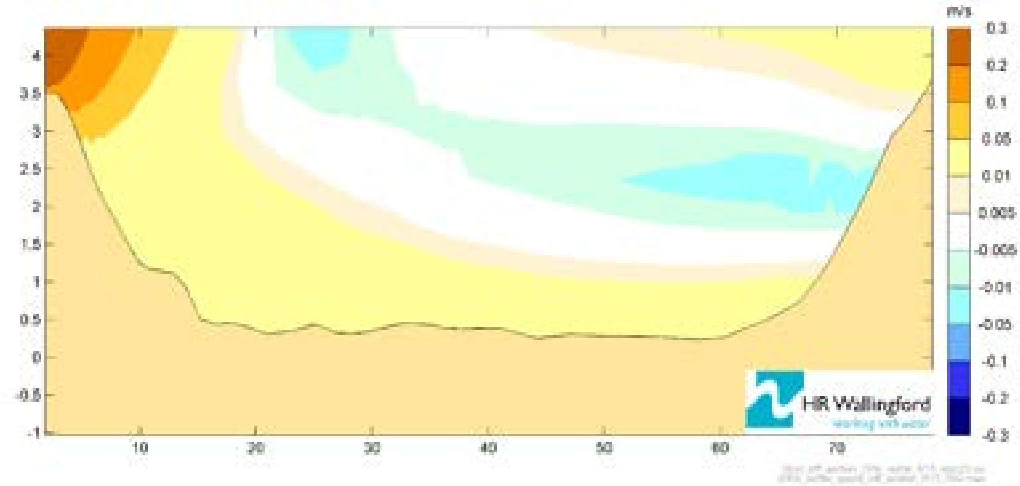
Figure 5-11 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Teddington DRA outfall under the Scenario 2 flows with 100 Ml/d abstraction and discharge.

Figure 5-11 Depth-average velocity at the Teddington DRA outfall cross-section, 400 Ml/d river flow, 100 Ml/d outfall discharge and 100 Ml/d intake

a) Developed flow velocity



b) Difference in flow velocity between baseline and developed

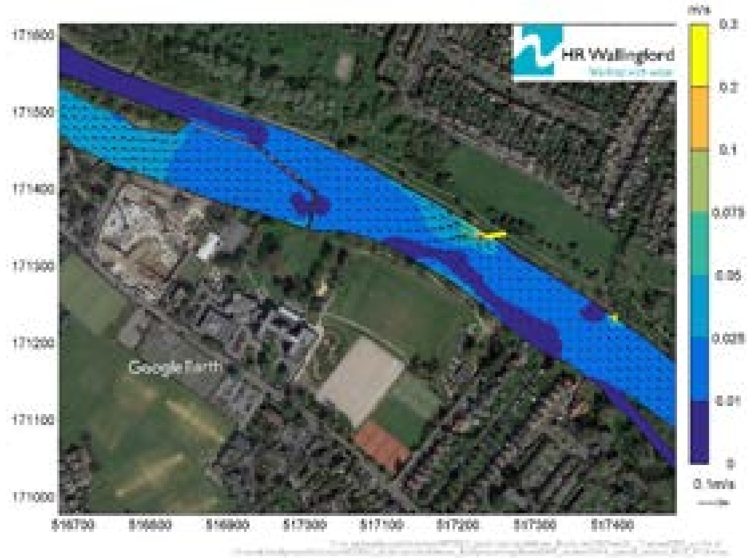


The difference data show that velocities peak around the outfall (left side of the cross-section) at around 0.05-0.3m/s and extend to a maximum of ~13m out into the channel. The modelling suggests that there is an increase in flow velocity of between 0.01-0.05m/s out to around 20m, across much of the channel bed and at the water surface towards and at the left bank (60-80m chainage). The remainder of the channel cross-section shows a range of minimal decreases and increases in velocity from -0.05 – 0.005m/s, with decreases in flow concentrated in the middle of the flow cross-section on the left bank (50-70m chainage). There are minimal differences in velocities when compared to the Scenario 2 75 Ml/d scheme.

The depth-average velocity and the velocity differences between this and the baseline around the Teddington DRA outfall and intake under Scenario 2 river flow conditions of 400 Ml/d, simulating a 150 Ml/d abstraction and 150 Ml/d discharge, are presented in Figure 5-12.

Figure 5-12 Depth-average velocity at the Teddington DRA outfall and intake, 400 Ml/d river flow, 150 Ml/d outfall discharge and 150 Ml/d intake

a) Developed flow velocity



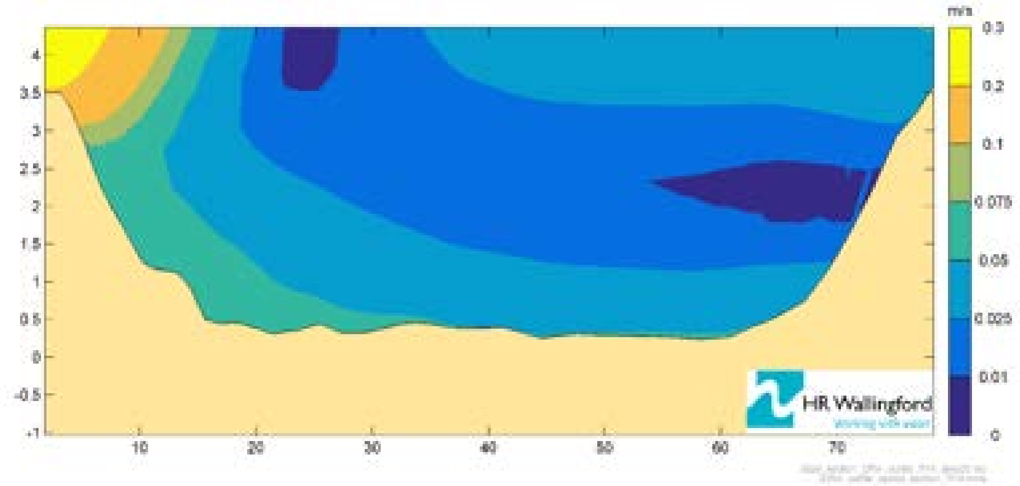
b) Difference in flow velocity between baseline and developed

Under the developed simulation a spatially limited increase in flow velocities occurs at the point of discharge (0.1-0.3m/s), with flow velocities increasing by 0.05-0.075m/s immediately downstream of the outfall and concentrated against the right bank for ~200m downstream, very similar to the 100 Ml/d scheme. Flow velocities are lower on the left bank directly opposite the outfall between 0-0.01m/s, however, in comparison to the 75 Ml/d and 100 Ml/d schemes, the area of reduced velocities extends further upstream and across the channel (as seen on the velocity difference plot with reductions of -0.0005 - -0.01m/s). Generally, the velocities across the channel range from 0-0.025m/s, indicating still to very slow-moving flow. Velocity vectors remain predominantly in a downstream direction, although show a deflection towards the outfall as upstream flow passes by. There is very little difference when compared to the Scenario 2 75 M/d and the 100 Ml/d scheme, with the exception of very slightly elevated velocities in the channel downstream of the outfall (c.f. the longer velocity vectors), although these remain within the same velocity group (0.01-0.025m/s), and the upstream reduction in velocities.

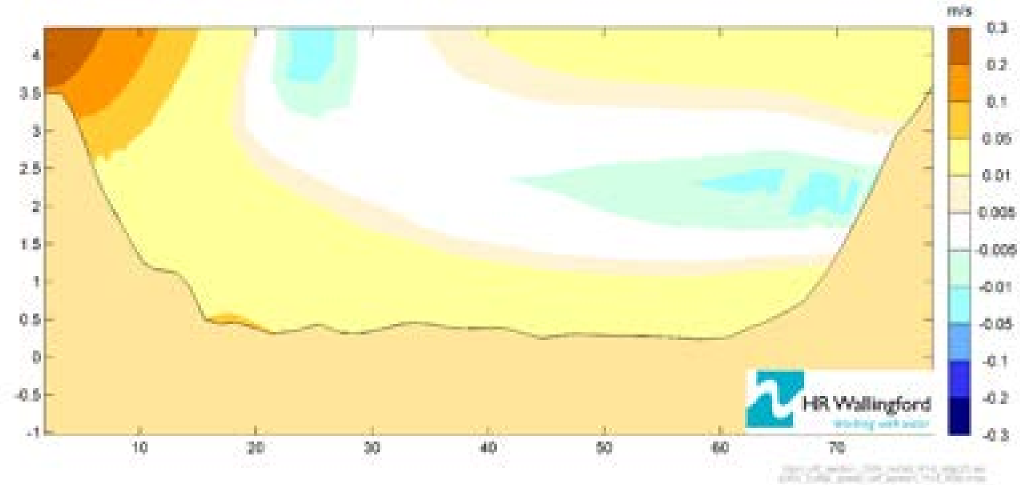
Figure 5-13 shows modelled changes in flow velocity for the river cross-section perpendicular to the location of the Teddington DRA outfall under the Scenario 2 flows with 150 Ml/d abstraction and discharge.

Figure 5-13 Depth-average velocity at the Teddington DRA outfall cross-section, 400 Ml/d, 150 Ml/d outfall discharge and 150 Ml/d intake

a) Developed flow velocity



b) Difference in flow velocity between baseline and developed



The difference data show that velocities peak around the outfall (left side of the cross-section) at around 0.05-0.3m/s and extend to a maximum of ~15m out into the channel. The modelling suggests that there is an increase in flow velocity of between 0.01-0.05m/s out to around 20m, across much of the channel bed and at the water surface towards and at the left bank and extending into the centre of the channel (35-80m chainage). The remainder of the channel cross-section shows a range of minimal decreases and increases in velocity from -0.05 – 0.005m/s, with decreases in flow concentrated in the middle of the flow cross-section on the left bank (40-70m chainage). There are minimal differences in velocities when compared to the Scenario 2 75 Ml/d and 100 Ml/d scheme.

5.4. WETTED HABITAT CHANGE IN FRESHWATER RIVER THAMES AND ESTUARINE THAMES TIDEWAY

5.4.1. Overview

In accordance with the approach set out in in Table 1-1, the change in velocity pattern at the Teddington DRA intake and between the intake and outfall has been assessed through 3D modelling.

5.4.2. Mainstem River Thames (DRA intake to Teddington Weir)

The outputs from modelled data for the reach covering the DRA intake to Teddington Weir have been outlined in Section 5.3 for three river flow scenarios under the 75 Ml/d outfall discharge and 75 Ml/d intake abstraction, as well as a 100 Ml/d and 150 Ml/d outfall discharge and intake abstraction for a 400 Ml/d river flow. These data show that there are predicted to be minimal changes in velocities throughout the reach, with velocities on the whole remaining in the 0-0.05m/s range.

In the area of the reach immediately around the outfall, modelling predicts only minimal increases in velocity, predominantly 0.005-0.05m/s, with velocities at the discharge point up to 0.1-0.2m/s (predominantly reflecting the velocity of the released water as it mixes into the river water). The modelling also indicates very minor and spatially limited reductions in flow velocities on the left bank opposite the outfall of around -0.005 - -0.05m/s, although for the 150 Ml/d scheme these velocity reductions occur further upstream towards the intake (though stopping prior to it) and also in a thin band across the channel. Flow velocity vectors remain relatively unchanged, although there is a slight deflection of flow towards the operating outfall as upstream flow passes which reduces in magnitude as incipient river flow increases (c.f. Scenario 1 to Scenario 3 and the 100 Ml/d and 150 Ml/d schemes). The modelled cross-section data indicate that flow level and width do not change over the modelled scenarios.

In the few metres immediately around the intake, modelling predicts only minimal changes in velocity, predominantly 0.005-0.05m/s immediately upstream of the intake and a small reduction of -0.05 - -0.01m/s immediately downstream of the intake. Velocities at the intake point are up to 0.1-0.2m/s, this reflects the velocity of the abstracted water. There are no significant changes in velocity vectors. It is inferred from the intake cross-section that there are unlikely to be any significant changes in river level at or adjacent to the intake.

5.4.3. Teddington Weir pool

Section 4.4.4 describes the changes in Teddington Weir pool for the modelled minimum baseline and scheme water levels and wetted habitat changes under the A82 and M96 flows immediately downstream of Teddington Weir under the 200 Ml/d Mogden water recycling scheme. These data are deemed relevant as they are representative of the flows being passed forward over Teddington Weir under the Teddington DRA scheme.

These data illustrate that there is no change in water levels at the weir pool when the baseline water level is compared with that of the Mogden water recycling scheme, and that there is no change in exposure between the baseline and the scheme, although there is a slight reduction in the duration of exposure by a several minutes. Given the suitability of these data for assessing changes due to the operation of the Teddington DRA, it is concluded that there will be negligible change in wetted habitat around the weir pool during operation.

5.4.4. Richmond Pound

Section 4.4.5 describes the changes in Richmond Pound for the modelled minimum baseline and scheme water levels and wetted habitat changes under the A82 and M96 flows under the 200 Ml/d Mogden water recycling scheme.

These data illustrate that there is no change in water levels in the pound when the baseline water level is compared with that of the Mogden scheme, and that there is no change in exposure between the baseline and the scheme, although there is a slight reduction in the duration of exposure by a several minutes.

5.4.5. Upper Thames Tideway (Richmond Half-tide Sluice to Battersea)

Section 0 describes the changes in the Upper Thames Tideway between Richmond Pound and Wandsworth Bridge for the modelled minimum baseline and scheme water levels and wetted habitat changes under the A82 and M96 flows under the 200 Ml/d Mogden water recycling scheme.

These data indicate that there is a limited change in exposure between the baseline and the scheme, ranging between a maximum of 0.5-1.4ha for the A82 scenario and 1.2ha for the M96 scenario, with a generally very limited change in the duration of exposure, at some points reducing by only a few minutes compared to the baseline.

5.4.6. Middle Thames Tideway (Battersea to Tower Bridge)

Section 4.4.7 describes the changes in the Middle Thames Tideway between Wandsworth Bridge and Tower Bridge for the modelled minimum baseline and scheme water levels and wetted habitat changes under the A82 and M96 flows under the 200 Ml/d Mogden water recycling scheme.

These data indicate that there is a very limited change in exposure between the baseline and the scheme, ranging between a maximum of -0.2ha (reduction) for the A82 scenario and 0.4ha change for the M96 scenario.

Towards the end of the reach there is no change in exposure. There is no change in the duration of exposure in the reach compared to the baseline.

5.5. TEDDINGTON WEIR FISH PASS AND BARRIER PASSABILITY

Modelled water level at Teddington Weir under varying river flows and a 75 Ml/d release from the Teddington discharge (north bank outfall) are given in Table 5-1. Levels are given as a developed level which represents the water levels under the Teddington DRA release and the difference in level when compared to the baseline water level. Baseline level data for changes in modelled water level upstream of Teddington Weir are presented in Appendix 1 Section 5.2 (Teddington Weir water levels).

Table 5-1 Modelled changes in water levels at Teddington Weir under varying river flows for a 75 Ml/d Teddington DRA discharge (north bank outfall)

Sample location	300 Ml/d river flow water level (mAOD)		400 Ml/d river flow water level (mAOD)		600 Ml/d river flow water level (mAOD)	
	Developed level	Difference from baseline	Developed level	Difference from baseline	Developed level	Difference from baseline
T1 (upstream)	4.37	0.00	4.38	0.00	4.41	0.00
T2 (upstream)	4.37	0.00	4.38	0.00	4.41	0.00

The data show that under the three different river flows and the 75 Ml/d outfall release there is no predicted change in water level upstream of the weir. There is also no change showed for the 100 Ml/d or 150 Ml/d outfall release. Assessment of fisheries effects is included in the Fish Assessment Report.

5.6. RICHMOND POUND DRAWDOWN PHYSICAL ENVIRONMENT ASSESSMENT

Data for the characterisation of the Richmond Pound physical environment due to changes from the operation of the 200 Ml/d Mogden water recycling scheme have been addressed in several areas in the document, namely wetted habitat change in Section 4.4 (Table 4-1), hydrodynamic changes in Appendix 1 (Section 6) and suspended sediment changes in Section 4.7 (Figure 4-62). Although these data are for the 200 Ml/d Mogden water recycling scheme, they are deemed relevant as they are representative of the flows being passed forward over Teddington Weir under the smaller sized Teddington DRA scheme. These data show that there are no to very limited changes in wetted habitat, water level and suspended sediment concentration in the pound.

During the November period, tidal level management in Richmond Pound is withdrawn at Richmond half-tide sluice. In order to understand the physical environment within Richmond Pound under these conditions, specific hydrodynamic modelling of the November period has been completed. The results of this modelling for hydrodynamics, wetted habitat and suspended sediment concentration for the Mogden A82 and M96 scenarios are presented in full in Section 4.6.

5.7. THAMES TIDEWAY ESTUARINE SEDIMENT ASSESSMENT

Section 4.7 and Figure 4-62 illustrates the 95th percentile SSC and change in SSC against the baseline along the thalweg of the Thames Tideway between Teddington Weir and 3km seaward of the QE2 Bridge for the M96 river flow and the 200 Ml/d Mogden water recycling scheme.

In summary, the data show that SSC under the 95th percentile baseline and scheme do not exceed 0.5kg/m^3 across the entire study reach. SSC increases from around Putney Bridge (~18km) until the end of the reach, with the highest concentrations being present between the Thames Barrier and the end of the reach. The data show there is no perceptible change between Teddington Weir (0km) and 40km, with only a very limited change in SSC from 40km to the end of the reach. Although these data are for the 200 Ml/d Mogden water recycling scheme, they are deemed relevant as they are representative of the reduction of final effluent from Mogden STW under the smaller sized Teddington DRA scheme.

5.8. SUMMARY OF PHYSICAL ENVIRONMENT ASSESSMENT OF TEDDINGTON DRA SCHEME

Table 5-2 summarises the potential physical environment impacts for each of the sizes of a Teddington DRA scheme.

Table 5-2 Summary of potential physical environment impacts for Teddington DRA schemes

Size	Flow	Outfall and intake design	Wetted habitat	Fish pass and barrier passability	Richmond Pound drawdown	Estuarine sediment
50 Ml/d	Moderate 17% reduction in exceptionally low flows for 250m between intake and outfall (300 Ml/d upstream of intake)	Negligible change in velocities at intake or outfall inferred from larger scheme modelling assessment of negligible	Negligible change in water level or velocities between intake and outfall inferred from larger scheme modelling assessment of negligible	Negligible water level change inferred from larger scheme modelling assessment of negligible; fisheries conclusions are included in the B.2.3 Fish Assessment Report.	Negligible change in wetted habitat, water level and suspended sediment concentration.	Negligible change in wetted habitat, water level and suspended sediment concentration.
75 Ml/d	Moderate 25% reduction in exceptionally low flows for 250m between intake and outfall (300 Ml/d upstream of intake)	Negligible change in velocities at intake or outfall modelled.	Negligible change in water level or velocities between intake and outfall modelled. Negligible change in wetted habitat.	Negligible water level change modelled; fisheries conclusions are included in the B.2.3 Fish Assessment Report.		
100 Ml/d	Major 33% reduction in exceptionally low flows for 250m between intake and outfall (300 Ml/d upstream of intake)	Negligible change in velocities at intake or outfall modelled.	Negligible change in water level or velocities between intake and outfall modelled. Negligible change in wetted habitat.	Negligible water level change modelled; fisheries conclusions are included in the B.2.3 Fish Assessment Report.		
150 Ml/d	Major 50% reduction in exceptionally low flows for 250m between intake and outfall (300 Ml/d upstream of intake)	Negligible change in velocities at intake or outfall modelled.	Negligible change in water level or velocities between intake and outfall modelled. Negligible change in wetted habitat.	Negligible water level change modelled; fisheries conclusions are included in the B.2.3 Fish Assessment Report.		

In conclusion, the Teddington DRA schemes may lead to up to major reduction in flows when compared to the baseline conditions in the ~250m of the River Thames between the intake and outfall. However, these changes are negligible when considering impacts to water level depth and flow velocities. Additionally, the data indicates that there are negligible impacts on fish pass barrier possibility, negligible impacts on the Richmond Pound and on wetted habitat, water level and suspended sediment concentration in the Thames Tideway.

6. CURRENT KNOWLEDGE GAPS AND FUTURE INVESTIGATIONS AT GATE 3

6.1. PREVIOUSLY IDENTIFIED GAPS IN PHYSICAL ENVIRONMENT UNDERSTANDING

At Gate 1 the physical environment assessment identified evidence gaps which required addressing for Gate 2. A list of these gaps in Gate 1 were:

1. Better understanding of the bathymetry, flow conditions and links to water quality of weir pools at Sunbury Weir, Molesey Weir and Teddington Weir. This could be collected using ADCP and water quality sonde surveys of weir pools under multiple varying river flow conditions in order to better quantify the morphology, hydrodynamics and water quality of the current pools and how these are affected by changes in flow. This would be reinforced by a more quantitative understanding of the species which inhabit and utilise the weir pools.
2. The need to develop and better understanding of the locations, extent and sensitivity of existing marginal habitat within the River Thames. This should be augmented with ADCP surveys of selected key high sensitivity habitats under different flows, to better understand how hydrodynamics and exposure risk change with flows and how this impacts these locations. These surveys would include linked assessments of species present e.g. marginal macrophytes, macroinvertebrates and fish.
3. Consideration of any potential impacts on the Richmond Sluice when this reverts to fully tidal during the annual draw off between September to November.

These gaps in evidence collection and modelling have been filled as part of the Gate 2 assessment.

ADCP surveys were undertaken of the weir pools at Sunbury Weir and Molesey Weir, together with around the Gate 2 Mogden water recycling scheme outfall at Walton Bridge and the Gate 2 Teddington DRA intake and outfall locally upstream of Teddington Weir. These data informed the 3D scenario modelling of the effects of outfall and flow augmentation at these key points.

ADCP surveys and UK Hab River Morph surveys at key locations and reaches in the freshwater River Thames, channels of the Lee and in the upper and middle Thames Tideway have been incorporated into the Gate 2 physical environment assessment and form evidence for the fisheries and aquatic ecology assessments.

The licensed removal of the tidal control at Richmond Half-tide Sluice by Port of London Authority annually for a period of November was included as specific model conditions in the 2D/3D Telemac hydrodynamic modelling at Gate 2.

6.2. KNOWLEDGE GAPS IDENTIFIED DURING GATE 2

The comprehensive physical environment assessment at Gate 2 for the London Effluent Reuse schemes has identified the magnitude of physical environment effect in both the freshwater and estuarine study areas of the schemes. These assessments have assessed negligible impacts in the estuarine environment and as such there are no gaps in knowledge of further hydrodynamic-linked pathways not explored at Gates 1 or 2.

In the Enfield Island Loop of the Lee Diversion Channel the major flow changes from flow augmentation from a Beckton water recycling scheme are for a ~100m length of heavily modified channel. There may be an additional zone of influence for the downstream ~500m of the Enfield Island Loop, but the flow regime in that reach is determined by operation of the intake to the King George V Reservoir, which may abstract no water, or abstract all of the flow, including all of the augmented flow from a Beckton water recycling scheme. High-spec ADCP surveys of river depth and flow velocity through the water column have been repeatedly undertaken in the reach and provide context of change from flow change. River condition surveys have also been undertaken. Fisheries assessment for Gate 2 identified that no further physical environment surveys are required to further clarify the fisheries assessments.

In the freshwater River Thames there are notable flow changes from flow augmentation associated with the Mogden water recycling scheme in the reach between the Gate 2 Walton Bridge outfall and Thames Water's extant Walton intake. The flow increases, always at exceptionally low to low river flow conditions, are assessed as with negligible or very minor impacts on: river velocity; general channel wetted habitat; weir pool wetted habitat; or fish pass passibility. No additional evidence collection is considered to be required to further verify these assessments.

In the freshwater River Thames the flow changes from flow reduction associated with the Teddington DRA schemes is exclusively in the ~250m reach between the Gate 2 intake and outfall locally upstream of Teddington Weir. The flow reductions, always at exceptionally low to low river flow conditions, are assessed as with negligible impacts on: river level; river velocity; and wetted habitat. The outfall plume is modelled with negligible effects on river velocities. With no net flow change downstream of the outfall there are no impacts downstream of any plume. No additional evidence collection is considered to be required to further verify these assessments.

6.3. FUTURE INVESTIGATIONS AT GATE 3

As the engineering design and operational triggers of the London Effluent Reuse schemes are progressed in Gate 3, further specificity can be added to the Gate 2 assessments.

As engineering design progresses, Gate 2 tools can be re-used to assess variants in outfall velocities or discharge angle for discharge in the 3D Telemac model of the River Thames. A 2D hydrodynamic model of the Enfield Island Loop locally between Rifle Weir and the Lee Diversion Channel may assist with detailed design of a Beckton water recycling outfall.

The use of water resources modelling at Gate 2 has provided the best available information on likely patterns of scheme use available at the time. However, with WRSE and other Regional Groups WRMP24 Plan reconciliation, the pattern of use of London Effluent Reuse SRO and other SROs will develop. New variants on operating patterns and cumulatives can be readily tested through scenarios using the Gate 2 river and estuary modelling tools. These include variants in standby and ramp-up/ ramp-down patterns within the 1D model of the River Thames.

Appendix 1 Supporting Evidence of Physical Environment Reference Conditions

1. CONTENTS

The reference conditions for each of the following tasks has been set out in the following sections:

- General hydrodynamic conditions in the study area – Appendix 1 Section 2
- Local hydrodynamic conditions around potential SRO in-river structures – Appendix 1 Section 3
- River mainstem, weir pool and estuarine wetted habitat – Appendix 1 Section 4
- Fish pass and barrier passability – Appendix 1 Section 5
- Richmond Pound drawdown physical environment – Appendix 1 Section 6
- Estuarine sediment – Appendix 1 Section 7.

2. GENERAL HYDRODYNAMIC CONDITIONS IN THE STUDY AREA

2.1. Overview

This section sets out the reference conditions for the discharge, level and velocity patterns throughout the study area:

- Freshwater River Thames – Appendix 1 Section 2.2
- Estuarine Thames Tideway – Appendix 1 Section 2.3
- Freshwater Lee Diversion Channel – Appendix 1 Section 2.4

The evidence available, the general patterns observed in the data and any particular pressures are outlined and where to view this evidence have been set out for each of these reaches.

2.2. Freshwater River Thames

This section draws on the following data sources to establish the general hydrodynamic reference conditions for the freshwater River Thames:

- 1D hydraulic modelling of selected one-year scenarios in the reach between Walton Bridge and Teddington Weir
- 3D hydraulic modelling of selected scenarios between Walton Bridge and Sunbury Weir; and locally upstream of Teddington Weir
- 2D hydraulic modelling of selected scenarios around Sunbury Weir and Molesey Weir, including the weir pools downstream of each weir.

The hydrological assessments at Gate 2 have a preference for alignment with flow series from water resources modelling. The developing WRSE water resources stochastic modelling series provides opportunities to better describe SRO potential operating patterns alongside River Thames river flow conditions. As such the WRSE water resources modelling has been interrogated to provide reference flow conditions for the River Thames. Although these series are considered robust at the water resources model end point at Teddington Weir, the validity of the series at other locations in the lower River Thames is unproven. Until the water resources model is developed further, the Gate 2 assessment of flows in the lower River Thames has been positioned against the 30-year gauged record at the Thames at Walton flow gauge. Gauged data (1991-2020) describe an extremely low flow statistic Q_{99} of 700 Ml/d, very low flow statistic Q_{95} of 850 Ml/d, low flow statistic Q_{90} of 976 Ml/d.

The time series representation of the reference condition River Thames flows for the selected model scenarios are incorporated within the assessments in Main Report Section 4.2.2 for Walton Bridge and in Section 5.2.2 for Teddington Weir. The reference conditions have been selected to be representative of low flow conditions in the lower River Thames as described in Section 4.1. These include a prolonged period through the summer and autumn months of modelled river flow at Walton Bridge around 1,000 Ml/d, reducing to c.750 Ml/d. Between Walton Bridge and Teddington Weir river flows are influenced (sequentially) by: abstraction at Thames Water's Walton intake; abstraction at Thames Water's Hampton intake; flow inputs from the River Mole; abstraction at Thames Water's Surbiton intake; and flow inputs from the Hogsmill River. For the selected

1:5 year return period the lowest modelled flows at Teddington Weir are 600 Ml/d for 12 dates in November. For the selected 1:20 year return period the lowest modelled flows at Teddington Weir are 300 Ml/d, for 17 dates in October.

An overview of the hydrodynamic conditions within the freshwater River Thames are provided below in Section 3.2.

2.3. Estuarine Thames Tideway

This section draws on the following data sources to establish the general hydrodynamic reference conditions for the estuarine Thames Tideway:

- 2D/3D hydraulic modelling of selected one-year scenarios in the reach between Teddington Weir and Southend -on-Sea

The time series representation of the reference condition Thames Tideway estuarine hydrodynamics, in particular level/depth, for the selected model scenarios are incorporated within the assessments in Main Report Sections 3.4, 4.4 and 5.4.

2.4. Freshwater Lee Diversion Channel

This section draws on the following data sources to establish the general hydrodynamic reference conditions for the freshwater Lee Diversion Channel:

- Gauged flow data locally in the River Lee based on Lee at Rammey Marsh flow gauge and Cobbins Brook at Sewardstone Road flow gauge.
- ADCP data measured by Ricardo between 1 November 2018 and 18 January 2022.

With respect to this channel reach there are three suitable ADCP survey locations:

- River Lee downstream of Newmans sluice (Site 9, Lee Diversion Channel, TQ3763098296).
- River Lee upstream of the King George V intake (Site 10, Enfield Island Loop, TQ 37272 98177).
- River Lee downstream of the King George V intake (Site 12, Enfield Island Loop, TQ 36992 97746).

Basic hydrodynamic reference conditions for the three sites are:

- **River Lee downstream of Newmans sluice (Site 9)** – Measured discharge between November 2018 and January 2022 averaged 342 Ml/d with a standard deviation of 996 Ml/d. A minimum flow of 0.35 Ml/d was noted during dry periods (30 May 2019), to a peak of 5,078 Ml/d during wet periods (14 January 2021). This variation in flow is due to the nature of the channel and its purpose as a flood conveyance structure. It should be noted that there are no flow data to understand the relative proportion of flows within the Lee Diversion Channel which is bypassed down the Enfield Island Loop although it is assumed under low flow conditions that all flows are routed into the Enfield Island Loop.
- **River Lee upstream of the King George V intake (Site 10)** – Measured discharge between November 2018 and January 2022 averaged 291 Ml/d with a standard deviation of 90.1 Ml/d. A minimum flow of 121 Ml/d and a maximum flow of 429 Ml/d was noted. The site is located immediately upstream of the King George V Reservoir intake.
- **River Lee downstream of the King George V intake (Site 12)** - Measured discharge between November 2018 and January 2022 averaged 256 Ml/d with a standard deviation of 101 Ml/d. A minimum flow of 35.4 Ml/d and a maximum flow of 432 Ml/d was noted. The differences in average and minimum flow are due to the site being located immediately downstream of the King George V Reservoir intake. Maximum flows at the site are essentially the same as those at Site 10 immediately upstream.

Water resources modelling in the Lee Valley is not currently sufficiently spatially accurate to provide a reference condition flow series for the Enfield Island Loop. The 12-year flow series for the Rammey Marsh flow gauge (2010-2021) forms the basis of the flow understanding, Gauged data (1991-2020), with the daily flow series for the Cobbins Brook flow gauge at Sewardstone Road added, describe an extremely low flow statistic Q_{99} of 63 Ml/d, very low flow statistic Q_{95} of 125 Ml/d, low flow statistic Q_{90} of 153 Ml/d.

The time series representation of the reference condition Lee Diversion Channel flows for the selected scenarios are incorporated within the assessments in Section 3. The reference conditions have been selected from the measured dataset to be representative of low flow conditions at times of scheme operation: 1/4/2016-

31/3/2017 selected as representative of 1:5 flow conditions and 1/4/2011-31/3/2012 as representative of 1:20 flow conditions. The selected periods include a prolonged period through the summer and autumn months of modelled river flow in the Lee Diversion Channel around 100-200 Ml/d, reducing to c.40 Ml/d. Flow partitioning at Newmans Weir on the Lee Diversion Channel is understood to be exclusively into the Enfield Island Loop under low flow conditions, with flow only passing through Newmans Sluice along the main Lee Diversion Channel under high flow conditions.

An overview of the hydrodynamic conditions within the Enfield Island Loop are provided below in Section 3.3.

3. LOCAL HYDRODYNAMIC CONDITIONS AROUND POTENTIAL SRO IN-RIVER STRUCTURES

3.1. Overview

This section sets out the reference conditions for the discharge, level and velocity patterns which are adjacent to potential SRO in-river structures within the study area:

- Freshwater River Thames – Appendix 1 Section 3.2.
- Freshwater Lee Diversion Channel – Appendix 1 Section 3.3.

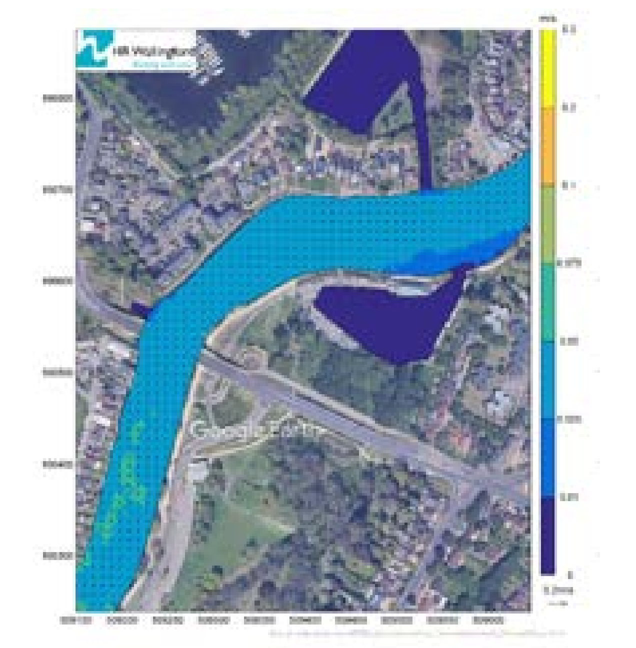
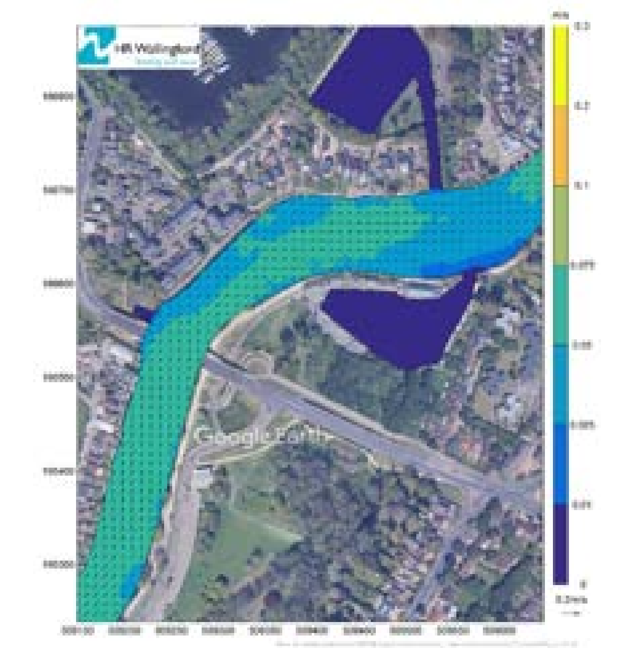
The evidence available, the general patterns observed in the data and any particular pressures are outlined and where to view this evidence have been set out for each of these reaches.

3.2. Freshwater River Thames

The results of 2D/3D river modelling undertaken by HR Wallingford for the River Thames between Walton Bridge and Teddington Weir is used to understand the baseline hydrodynamic conditions, specifically flow velocity and direction, for areas around the proposed Walton Bridge outfall for Mogden water recycling schemes and Teddington intake and outfall for Teddington DRA schemes.

For the Walton Bridge outfall, three baseline flow conditions were simulated, namely 600 Ml/d river flow (Scenario 1), 780 Ml/d river flow (Scenario 2) and 950 Ml/d river flow (Scenario 3). The 950Ml/d river flow scenario is considered from the Thames at Walton flow gauge record as an average river flow at Walton Bridge at times of a Mogden water recycling scheme operation, albeit a Q_{91} flow statistic, with the other flows for 2D/3D modelling representative of more extreme conditions: $Q_{99.5}$ and Q_{97} respectively.

Figure A-1 Baseline depth average flow velocity around the proposed location of the Walton outfall

a) 600 Ml/d flow (baseline Scenario 1)	b) 780 Mld/ flow (baseline Scenario 2)
	
c) 950Ml/d flow (baseline Scenario 3)	

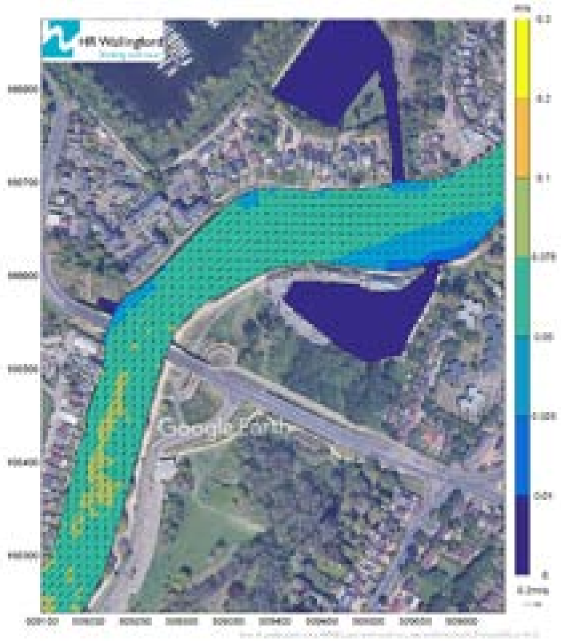


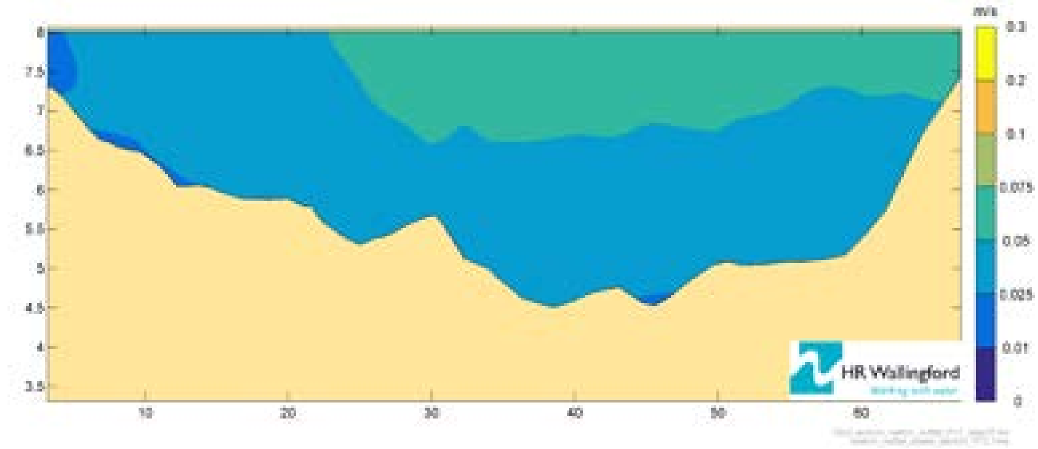
Figure A-1 shows the distribution of baseline depth averaged flow velocity immediately upstream and downstream of the proposed location of the Walton outfall for the three baseline river flow conditions. The data show that:

- **600 Ml/d** – Baseline flow velocities are predominantly between 0.025-0.05m/s throughout the simulated channel reach, with flow velocity vectors directed predominantly downstream. There are some localised areas of increased flow velocities predicted in the centre of the channel ~0.23km upstream of the proposed outfall (at Walton Bridge), measuring up to 0.05-0.075m/s, while there is a localised area at ~0.35km downstream on the right bank where flow velocities drop to between 0.01-0.025m/s.
- **780 Ml/d** – Baseline flow velocities show and increase to 0.05-0.075m/s for the 780 Ml/d flow baseline scenario through much of the study reach, with flow velocity vectors directed predominantly downstream. Around 0.35km downstream flow velocities remain 0.025-0.05m/s, with the lower flow velocities on the right bank remaining but covering a reduced spatial extent.
- **950 Ml/d** – Baseline flow velocities of 0.05-0.75m/s are present over most of the study reach, with higher velocities of 0.1-0.2m/s occurring in the centre of the channel upstream of Walton Bridge and the proposed intake. As for previous baseline flows, there is a localised area at ~0.35km downstream on the right bank where flow velocities remain low however the spatial extent of these velocities have declined when compared to the lower baseline flows.

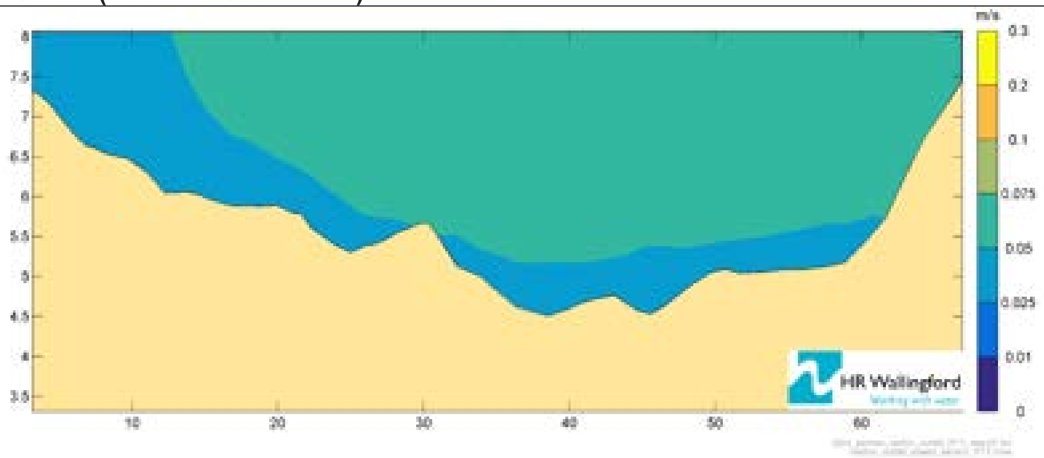
Baseline flow velocities for a cross-section drawn perpendicular to the proposed Walton outfall (outfall to be located at the left side of the cross-section) are presented in Figure A-2 for the three flow scenarios.

Figure A-2 Baseline depth average flow velocity at a cross-section perpendicular to the proposed Walton outfall

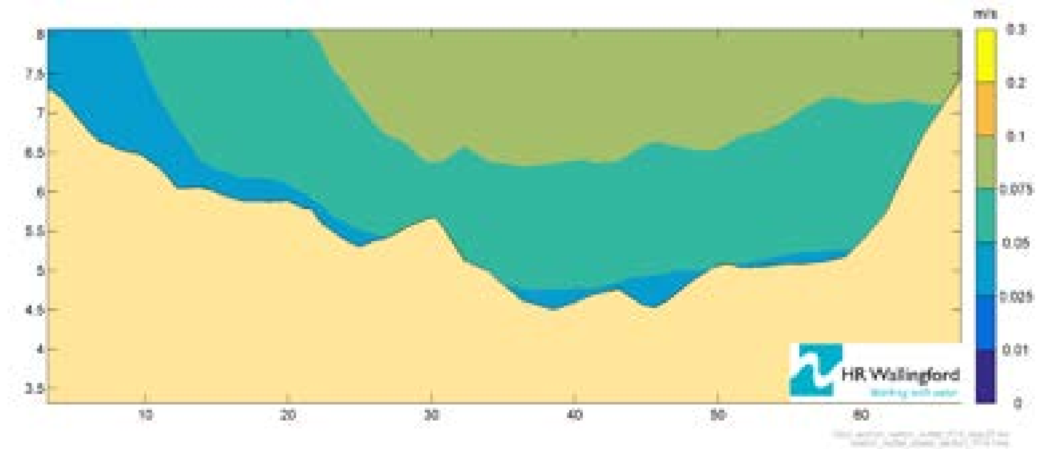
a) 600 Ml/d flow (baseline Scenario 1)



b) 780 Mld/ flow (baseline Scenario 2)



c) 950 Ml/d flow (baseline Scenario 3)



The baseline cross-section data show that:

- **600 Ml/d** – Baseline flow velocities are predominantly between 0.025-0.05m/s, with an area of increased velocity of 0.05-0.075m/s towards the right-hand bank. There are very spatially limited areas of slower flows, 0.01-0.025m/s, located at the left bank and at some points on the channel bed.
- **780 Ml/d** – Baseline flow velocities have increased when compared to Scenario 1 and are predominantly between 0.05-0.075m/s, with an area of lower flow velocities of 0.025-0.05m/s towards the left bank and along the base of the channel.

- **950 Ml/d** – Baseline flow velocities have increased when compared to Scenario 2. Much of the channel cross-section on the right bank and to the centre of the channel shows velocities of 0.075-0.1m/s. Velocities decline with depth and towards the left bank to around 0.05-0.075m/s for most of the cross-section. Flows of 0.025-0.05m/s are noted at the base of the channel.

For the Teddington outfall, three baseline flow conditions were simulated, namely 300 Ml/d river flow (Scenario 1), 400 Ml/d river flow (Scenario 2) and 700 Ml/d river flow (Scenario 3). The 700 Ml/d river flow scenario is considered from the WRSE water resources modelling as an average river flow at Teddington Weir at times of a Teddington DRA scheme operation, with the other flows representing more extreme conditions linked to specific Teddington Target Flow values in the Lower Thames Control Diagram.

Figure A-3 Baseline depth average flow velocity around the proposed location of the Teddington outfall

a) 300 Ml/d flow (baseline Scenario 1)

b) 400 Mld/ flow (baseline Scenario 2)

c) 700 Ml/d flow (baseline Scenario 3)

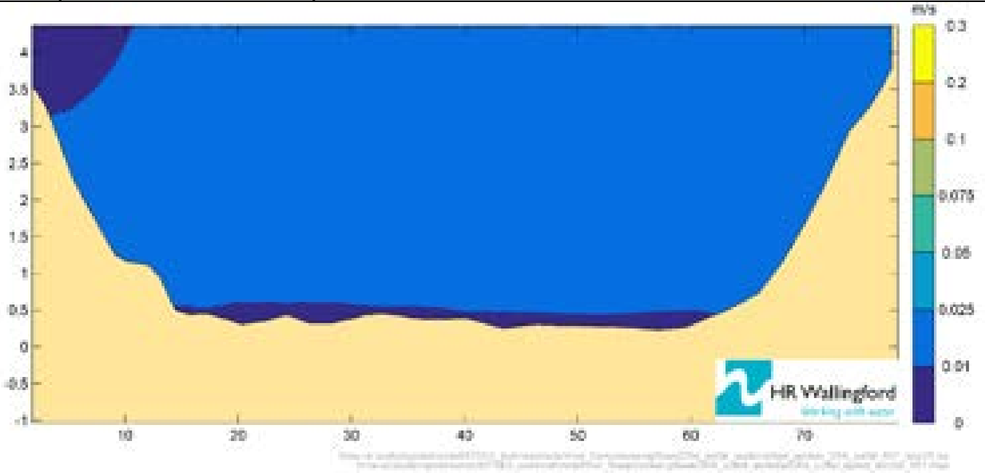
Figure A-3 shows the distribution of baseline depth averaged flow velocity immediately upstream and downstream of the proposed location of the Teddington outfall. The data show that:

- **300 Ml/d** – The data show that baseline flow velocities are predominantly between 0.01-0.025m/s throughout the simulated channel reach, with flow velocity vectors directed predominantly downstream. There is a localised reduction of flow velocities from 0-0.01m/s at the crest of Teddington Weir.
- **400 Ml/d** – The magnitude, direction and spatial distribution of flow velocities upstream of Teddington Weir remain predominantly similar to those for the 300 Ml/d flow. The most significant change is located downstream of the weir in where flows increase to 0.025-0.05m/s, with a reduction in the area of lower flow velocities at the crest of the weir.
- **700 Ml/d** – The data show that there is an increase in flow velocity across the reach up to 0.05-0.075m/s, though flow directions remain largely unchanged. There is a localised increase in flow velocities to 0.01-0.025m/s around the weir crest and to 0.05-0.075m/s downstream of the weir.

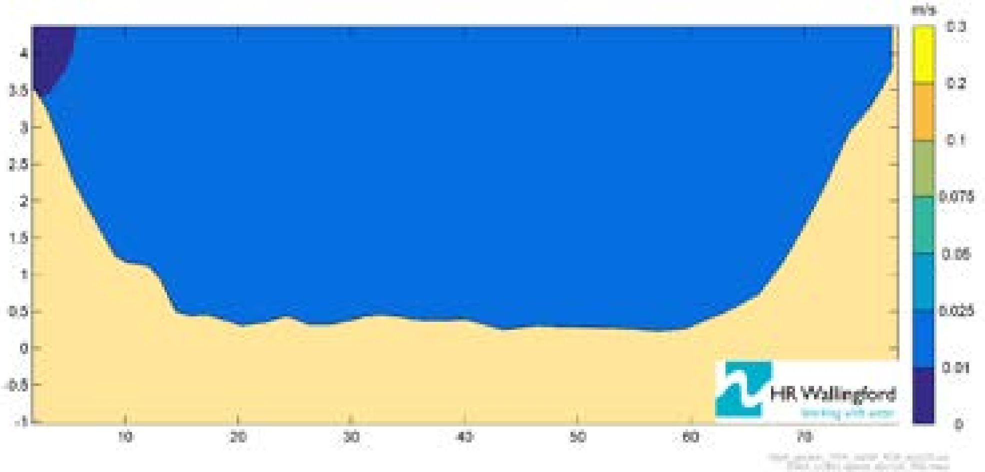
Baseline flow velocities for a cross-section drawn perpendicular to the proposed Teddington outfall (outfall to be located at the right side of the cross-section) are presented in Figure A-4 for the three flow scenarios.

Figure A-4 Baseline depth average flow velocity at a cross-section perpendicular to the proposed Teddington outfall

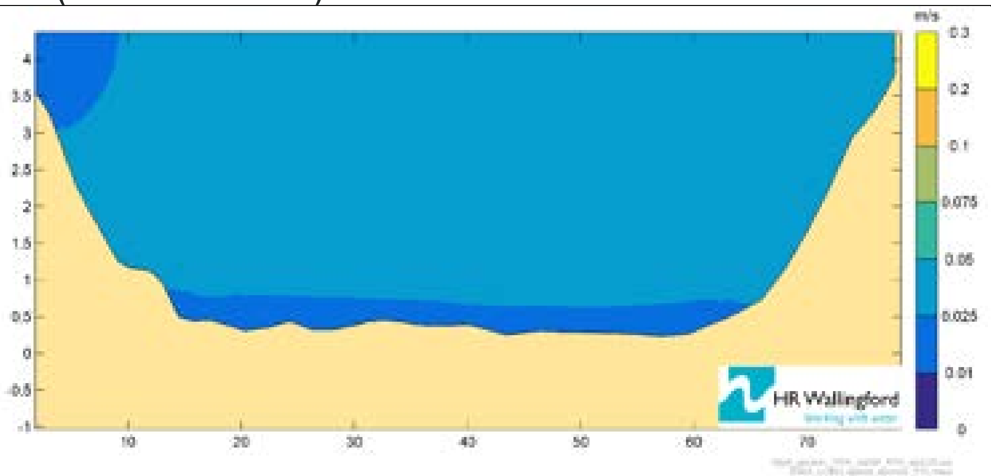
a) 300 Ml/d flow (baseline Scenario 1)



b) 400 Mld/ flow (baseline Scenario 2)



c) 700 Ml/d flow (baseline Scenario 3)



The baseline cross-section data show that:

- **300 Ml/d** – Baseline flow velocities are predominantly between 0.01-0.025m/s over most of the cross-section, with an area of essentially still to very slow-moving water of 0-0.01m/s predicted at the base of the channel and at the right bank (left of the section).

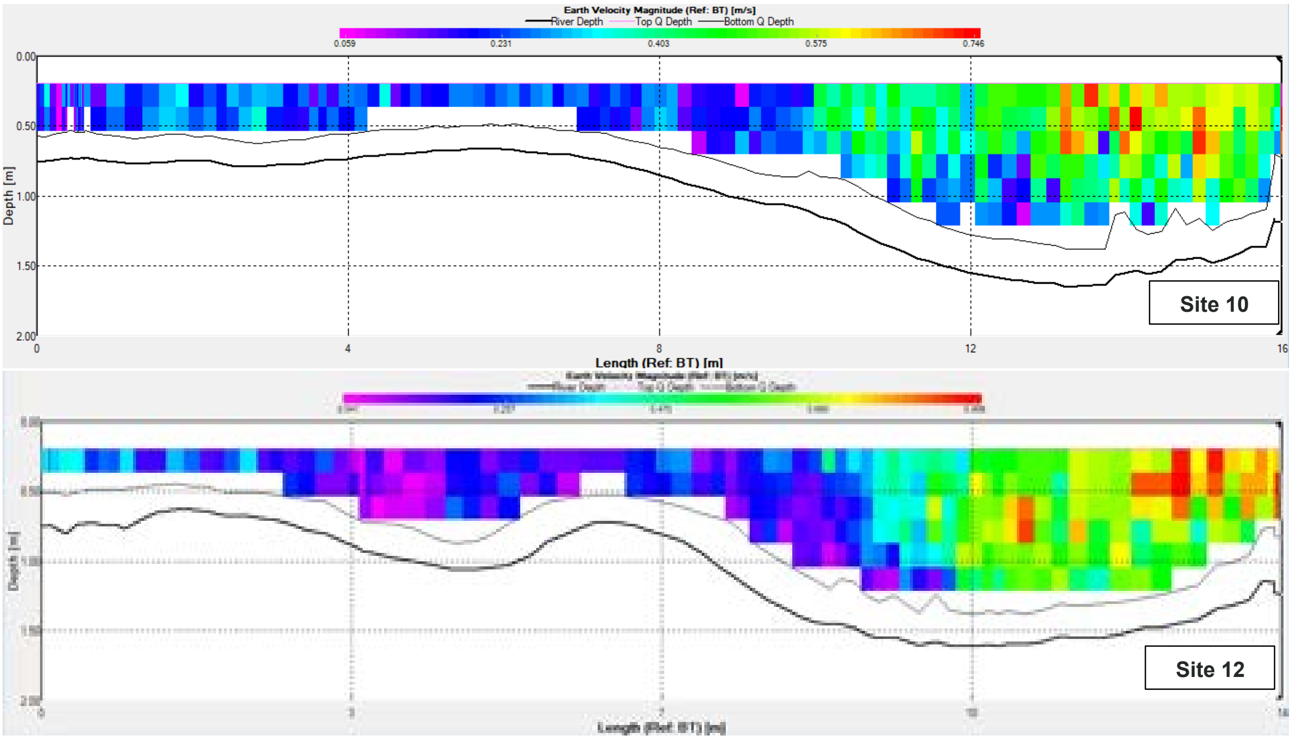
- **400 Ml/d** – Baseline flow velocities remain similar to Scenario 1, being predominantly between 0.01-0.025m/s over most of the cross-section. The area of essentially still to very slow-moving water of 0-0.01m/s predicted at right bank remains, however similarly still to very slow-moving water at the base of the channel is now moving at 0.01-0.025m/s.
- **700 Ml/d** – Baseline flow velocities mirror those in Scenario 1 but are replaced by increased flow velocities. Flow velocities are predominantly between 0.025-0.05m/s over most of the cross-section, with an area of slow-moving water of 0.01-0.025m/s predicted at the base of the channel and at the right bank (left of the section).

3.3. Freshwater Enfield Island Loop

As noted in Section 2.4, there are two ADCP sites on the Enfield Island Loop. ADCP cross-section data for Site 10 and Site 12 on the Enfield Island Loop have been provided for a representative high flow event (29 January 2021) during the measurement period and a representative low flow event (9 September 2021) during the measurement period.

Figure A-5 shows the hydrodynamics measured at the two sites on Enfield Island Loop under the representative high flows of 29 January 2021 (Site 10: 428 Ml/d and Site 12: 432 Ml/d).

Figure A-5 ADCP cross-sections showing depth and velocity profiles in Enfield Island loop under representative high flows of 29 January 2021

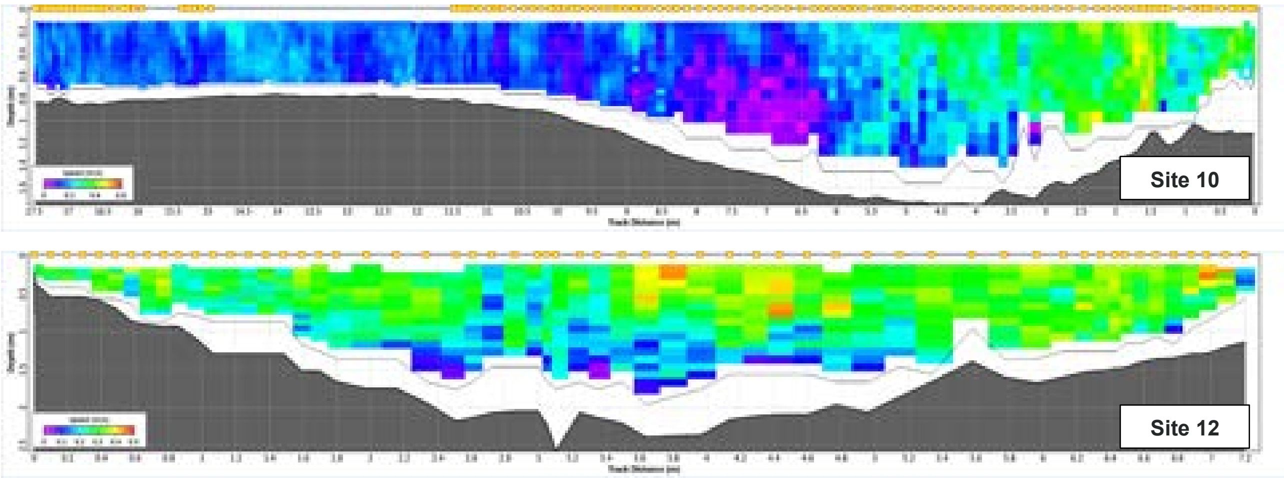


The data show that flow velocity for Site 10 ranges from 0.06-0.75m/s, with an average of 0.36m/s. Flow depth ranges from 0.79-1.64m. The higher flow velocities and depths are located towards the channel thalweg on the right side of the channel.

For Site 12, flow velocity ranges from 0.04-0.91m/s, with an average of 0.82m/s. Flow depth ranges from 0.62-1.64m.

Figure A-6 shows the hydrodynamics measured at the two sites on Enfield Island Loop under the representative low flows of 9 September 2021 (Site 10: 262Ml/d and Site 12: 229Ml/d).

Figure A-6 ADCP cross-sections showing depth and velocity profiles in Enfield Island Loop under representative low flows of 9 September 2021



The data show that flow velocity for Site 10 ranges from 0.02-0.42m/s, with an average of 0.19m/s. Flow depth ranges from 0.76-1.74m and averages 1.07m/s. The higher flow velocities and depths are located towards the channel thalweg on the right side of the channel.

For Site 12, flow velocity ranges from 0-0.36m/s, with an average of 0.17m/s. Flow depth ranges from 0.20-2.56m and averages 1.09m/s. The flow velocities are distributed over much of the channel cross-section and are reduced compared to Site 10, likely due to the effect of abstraction.

4. RIVER MAINSTEM, WEIR POOL AND ESTUARINE WETTED HABITAT

4.1. Overview

Across the study area, modelled and measured information are available from which to describe the level, velocity and wetted habitat within the freshwater reaches and the Thames Tideway.

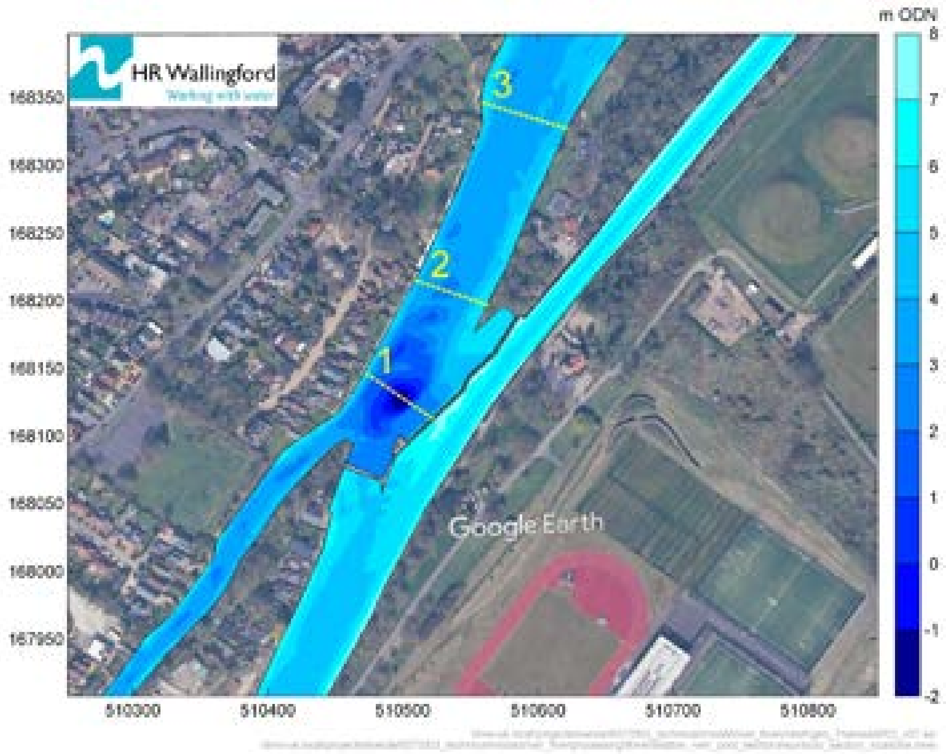
4.2. Freshwater River Thames

The results of 2D/3D river modelling undertaken by HR Wallingford for the River Thames between Walton Bridge and Teddington Weir is used to understand the baseline hydrodynamic conditions for weir pool and wetted habitat at the Sunbury Weir pool and Molesey Weir pool. For both weir pools, modelling was undertaken for three different baseline flow conditions, namely 600 Ml/d river flow (Scenario 1), 780 Ml/d river flow (Scenario 2) and 950 Ml/d river flow (Scenario 3).

Sunbury Weir pool

Modelled data from three cross-sections were extracted from the baseline modelled data for the Sunbury weir pool assessment, their locations are presented in Figure A-7.

Figure A-7 Sunbury weir cross-section locations

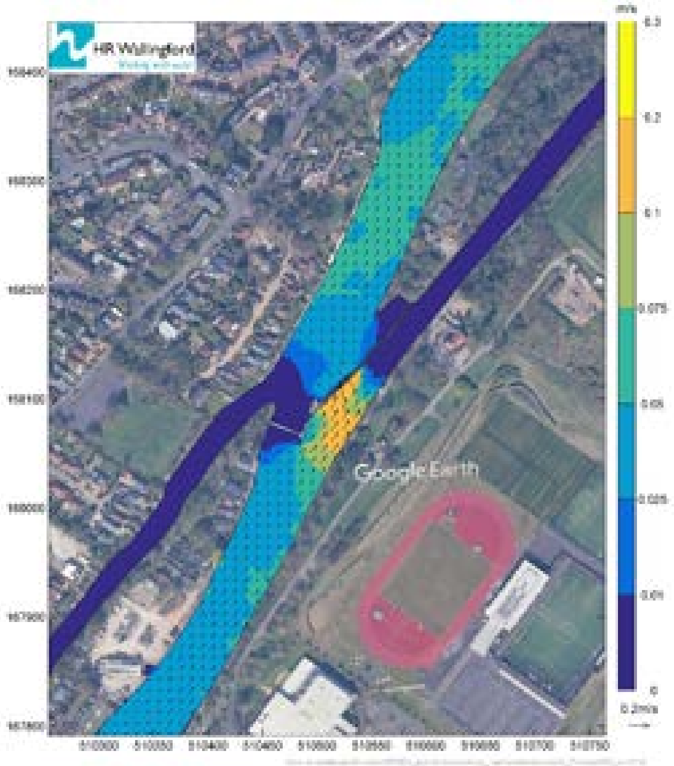


A brief overview of the baseline velocity data at Sunbury Weir pool for Scenarios 1 to Scenario 3 are presented below.

Scenario 1: 600 Ml/d river flow, extremely low river flow conditions

Figure A-8 illustrates the baseline depth average velocity at Sunbury Weir under 600 Ml/d river flow (Scenario 1).

Figure A-8 Baseline depth-average velocity at Sunbury weir, Scenario 1

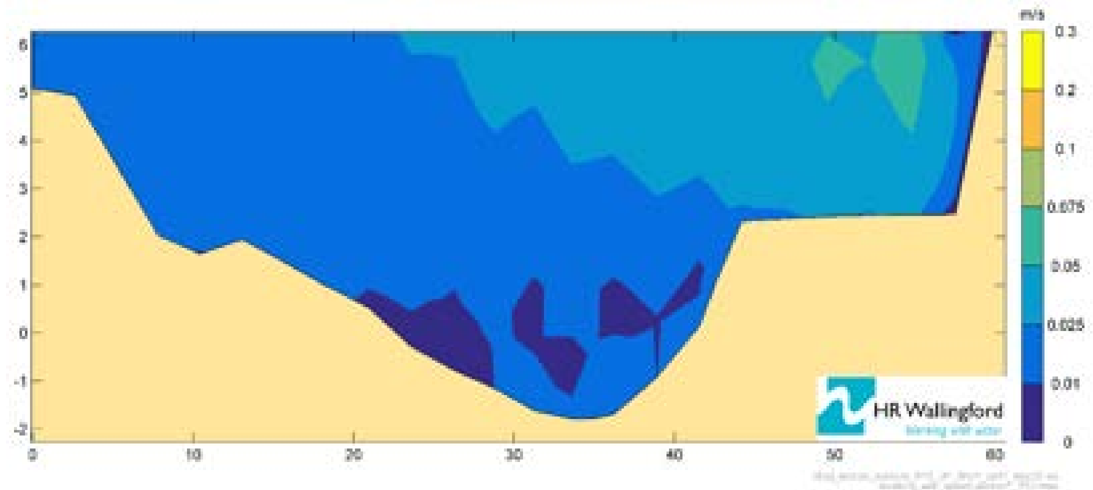


The data indicate velocities adjacent to the upstream face of the weir peak at around 0.1-0.2m/s. Immediately downstream of the weir in the area of the weir pool velocities decline to 0.025-0.05m/s, increasing slightly to 0.05-0.075m/s ~100-150m downstream of the weir pool as the channel bed begins to shallow away from the weir pool.

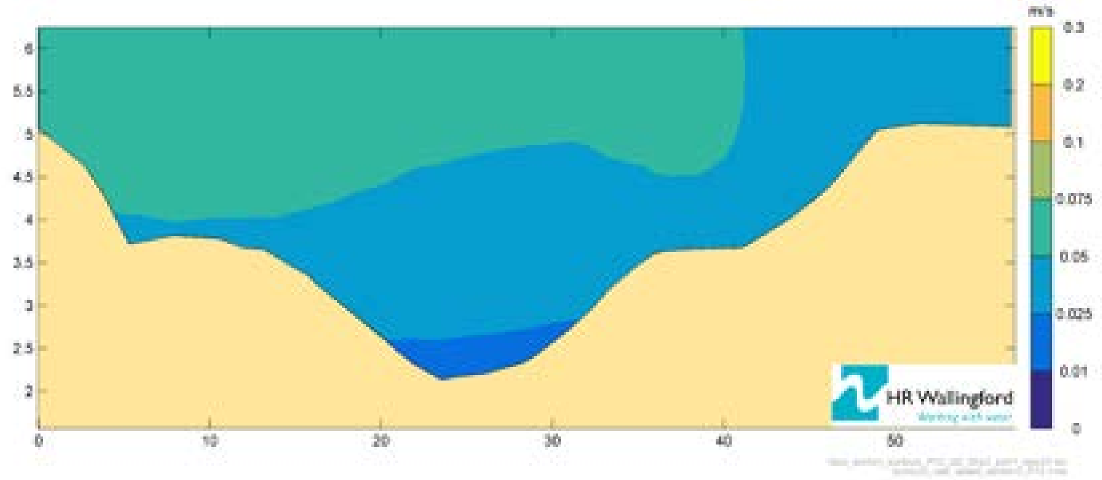
Figure A-9 shows the distribution of baseline depth averaged flow velocity at each of the three cross-sections at Sunbury Weir pool under the 600 Ml/d flow scenario (Scenario 1).

Figure A-9 Cross-sections of baseline flow velocities at Sunbury Weir pool, 600 Ml/d (Scenario 1)

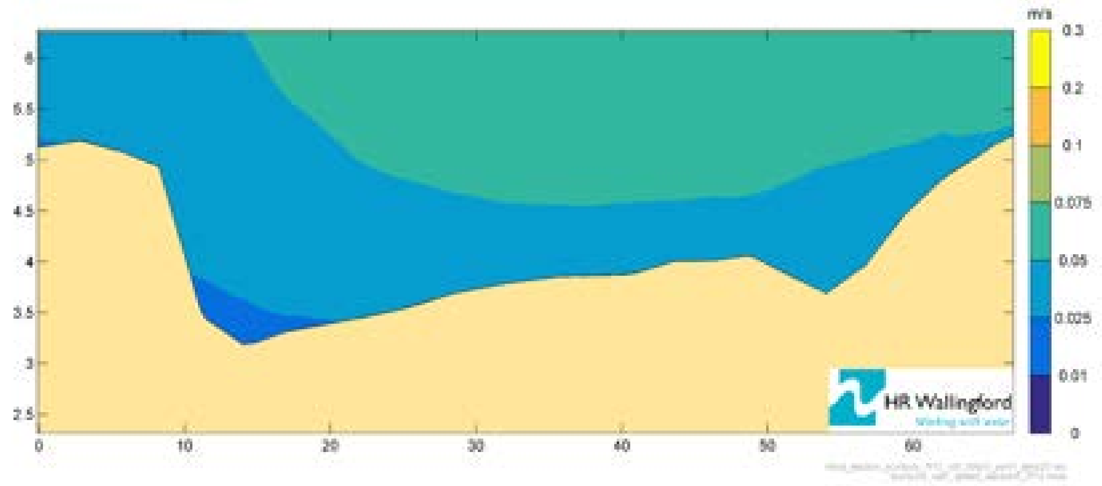
a) Baseline flow velocity, Section 1



b) Baseline flow velocity, Section 2



c) Baseline flow velocity, Section 3



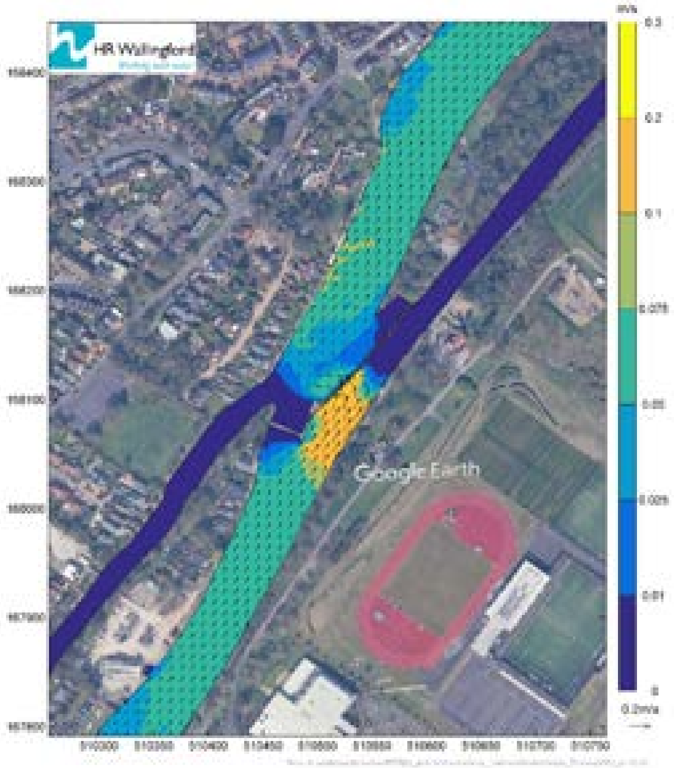
The baseline cross-section velocity data show that:

- **Section 1** – The cross-section is located directly within the weir pool and ~56m downstream from the weir. The data show that baseline flow velocities are predominantly between 0.01-0.05m/s across the cross-section, with higher velocities on the right side adjacent to the weir (0.05-0.075m/s) and declining towards the left bank (0.01-0.025m/s) as the influence of the weir declines. Towards the base of the weir pool flow velocities are very low, between 0-0.01m/s.
- **Section 2** – The cross-section is located ~85m downstream of the weir pool (Section 1) and ~150m downstream of the weir. The channel bed has shallowed by ~4m when compared to Section 1. The velocity data indicate that the right of the channel has lower flow velocities of around 0.025-0.05m/s, and these increase towards the left bank to 0.05-0.075m/s. Flow velocities across the deepest part of the channel bed are reduced to 0.01-0.025m/s.
- **Section 3** – The cross-section is located ~227m downstream from the weir pool (Section 1) and ~290m downstream from the weir. The data show that the shallower depths of the section have velocities of 0.05-0.075m/s, declining to 0.025-0.05m/s towards the bed. There is a slight asymmetry in flow velocities indicated by the section, with higher flow velocities on the right of the channel when compared to the left.
-

Scenario 2: 780 Ml/d river flow, extremely low river flow conditions

Figure A-10 illustrates the baseline depth average velocity at Sunbury Weir under 780 Ml/d river flow (Scenario 2).

Figure A-10 Baseline depth-average velocity at Sunbury weir, Scenario 2

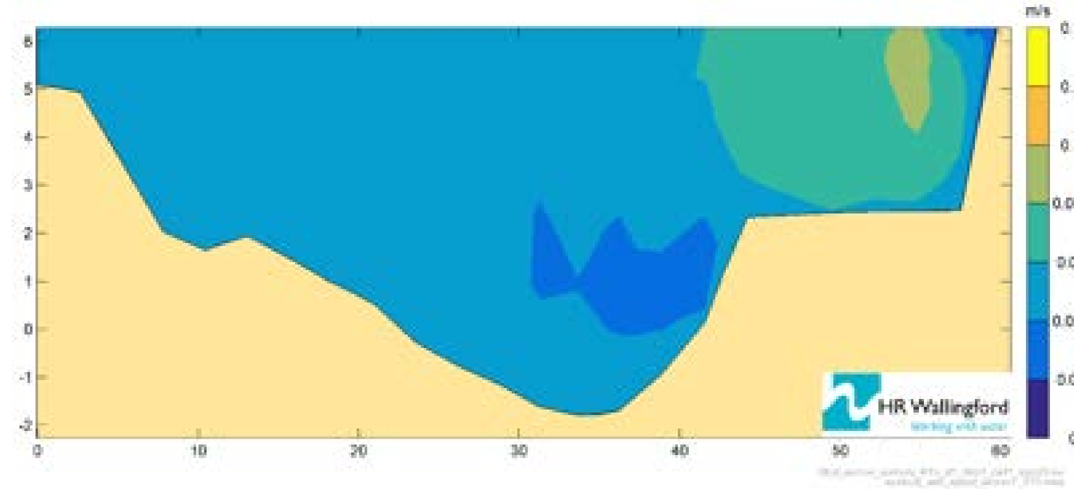


Similar to Scenario 1, the data indicate velocities adjacent to the upstream face of the weir peak at around 0.1-0.2m/s, although there are some localised velocities between 0.2-0.3m/s. On the leading edge of the weir pool velocities are between 0.05-0.075m/s, declining to between 0.01-0.05m/s over the weir pool before increasing to 0.05-0.075m/s immediately downstream of the weir pool as the channel bed begins to shallow away from the weir pool.

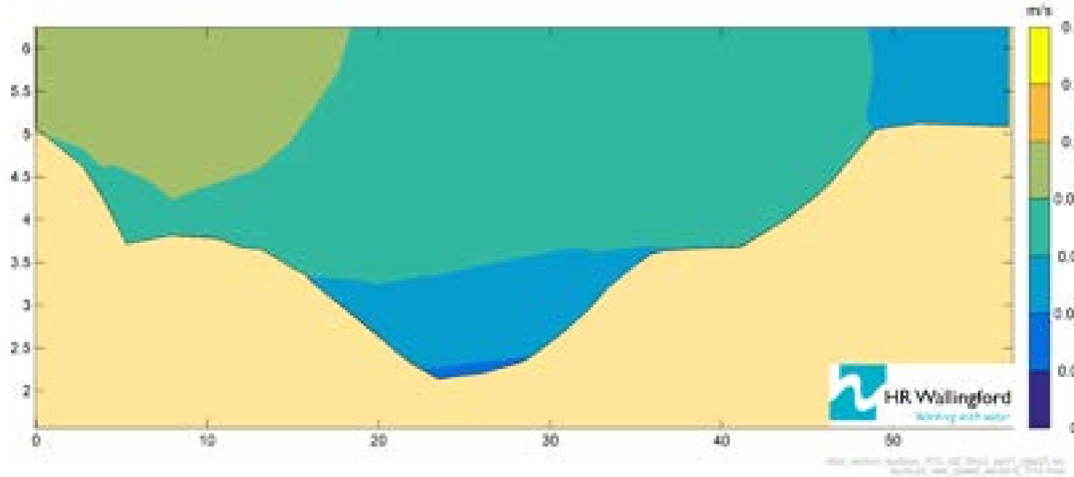
Figure A-11 shows the distribution of baseline depth averaged flow velocity at each of the three cross-sections at Sunbury Weir pool under the 780 Ml/d flow scenario (Scenario 2).

Figure A-11 Cross-sections of baseline flow velocities at Sunbury Weir pool, 780 Ml/d (Scenario 2)

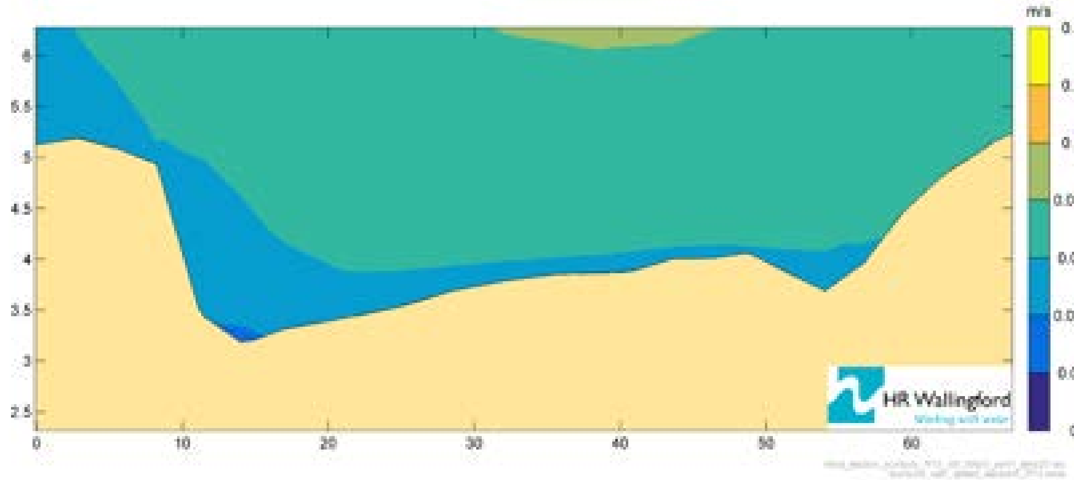
a) Baseline flow velocity, Section 1



b) Baseline flow velocity, Section 2



c) Baseline flow velocity, Section 3



The baseline cross-section velocity data show that:

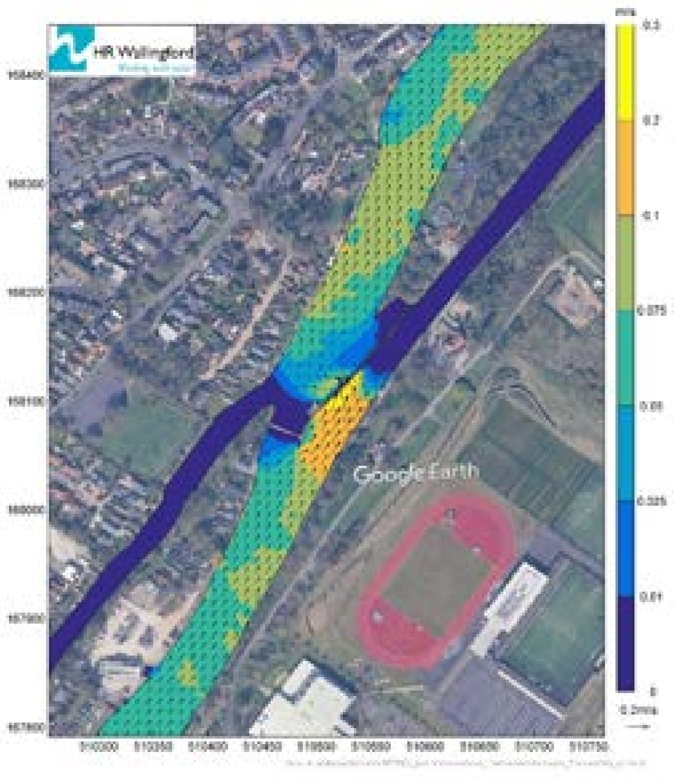
- **Section 1** – The cross-section is located directly within the weir pool and ~56m downstream from the weir. The data show that baseline flow velocities are predominantly uniform between 0.025-0.05m/s across most of the cross-section, with higher velocities on the right side adjacent to the weir (0.05-0.01m/s). The reduction in flow velocity towards the left bank (0.025-0.05m/s) is related to the declining influence of the weir and the deepening channel. Within the weir pool flow velocities remain similar to that in the rest of the cross-section with only a small area of slight reduction to 0.01-0.025m/s indicated adjacent to the point where the bed rapidly deepens away from the weir.

- **Section 2** – The cross-section is located ~85m downstream of the weir pool (Section 1) and ~150m downstream of the weir. The channel bed has shallowed by ~4m when compared to Section 1. The velocity data indicate that the right of the channel has lower flow velocities of around 0.025-0.05m/s, however, higher velocities of 0.05-0.075m/s are prevalent over most of the cross-section. There is a notable increase in flow velocities at the left bank to 0.075-0.1m/s. Flow velocities across the deepest part of the channel bed are reduced to around 0.01-0.05m/s (higher than in Scenario 1).
- **Section 3** – The cross-section is located ~227m downstream from the weir pool (Section 1) and ~290m downstream from the weir. The data show that flow velocities over the cross-section are predominantly 0.05-0.075m/s, with velocities reducing to 0.025-0.05m/s only very close to the channel bed and towards the left bank. Higher velocities of 0.075-0.1m/s are indicated towards the surface of the centre of the channel. When compared to Scenario 1 flows, there remains a slight asymmetry in flow velocities in the section, with higher flow velocities on the right of the channel when compared to the left.
-

Scenario 3: 950 Ml/d river flow, low river flow conditions

Figure A-12 illustrates the baseline depth average velocity at Sunbury Weir under 950 Ml/d river flow (Scenario 2).

Figure A-12 Baseline depth-average velocity at Sunbury weir, Scenario 3

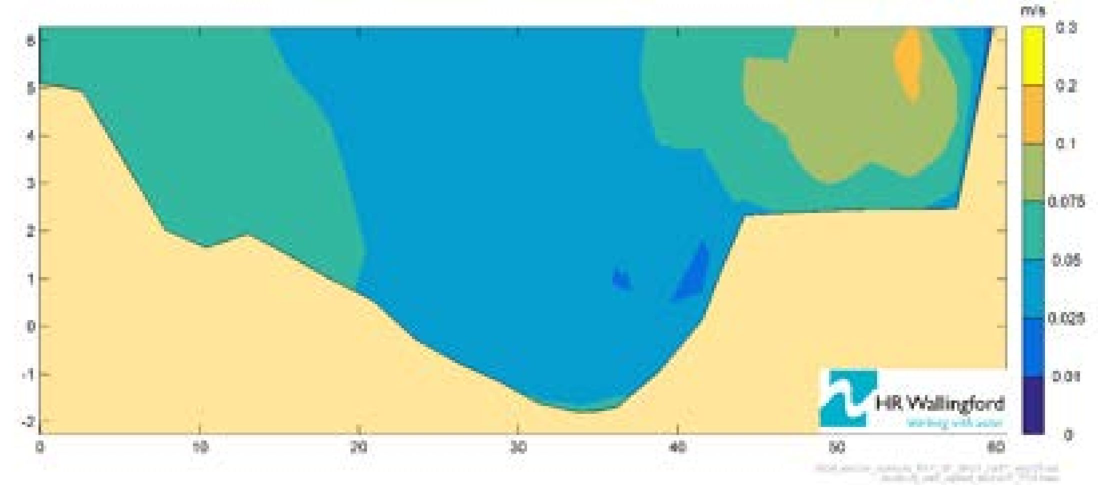


The higher flows of Scenario 3 show a similar velocity distribution to Scenario 2. The data indicate velocities adjacent to the upstream face of the weir peak at around 0.1-0.2m/s, although there is a greater spatial increase in velocities between 0.2-0.3m/s when compared to Scenario 2. On the leading edge of the weir pool velocities are between 0.075-0.1m/s, declining to between 0.01-0.05m/s over the weir pool before increasing to 0.05-0.075m/s immediately downstream of the weir pool as the channel bed begins to shallow away from the weir pool. 150m downstream of the weir pool velocities in the channel are higher than for previous scenarios, ranging up to 0.075-0.1m/s.

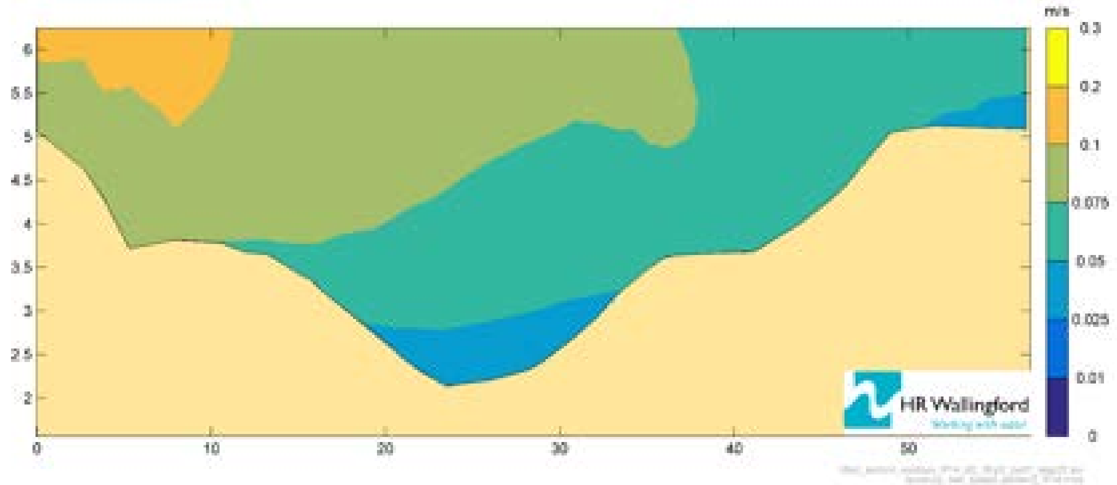
Figure A-13 shows the distribution of baseline depth averaged flow velocity at each of the three cross-sections at Sunbury Weir pool under the 950 Ml/d flow scenario (Scenario 3).

Figure A-13 Cross-sections of baseline flow velocities at Sunbury Weir pool, 950 Ml/d (Scenario 3)

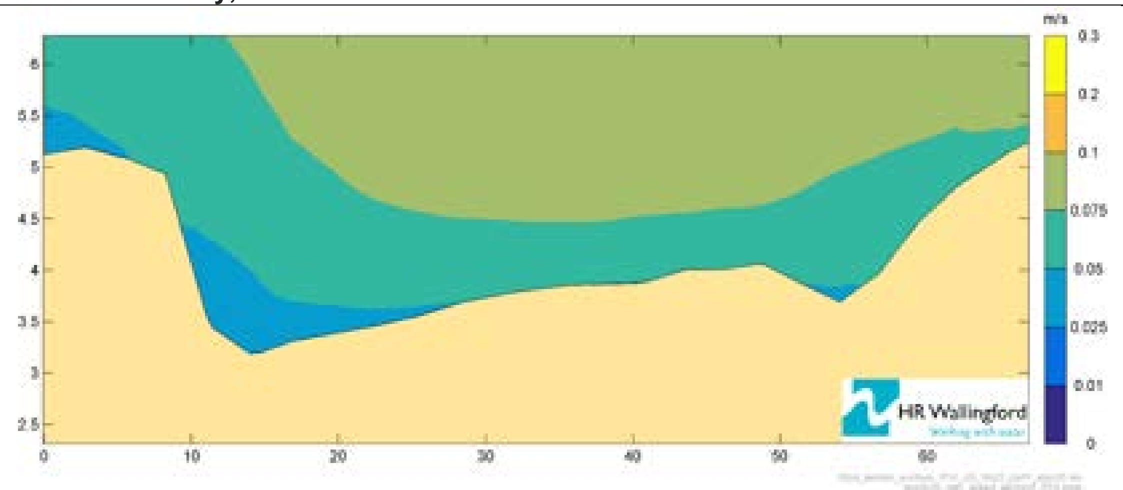
a) Baseline flow velocity, Section 1



b) Baseline flow velocity, Section 2



c) Baseline flow velocity, Section 3



The baseline cross-section velocity data show that:

- **Section 1** – The cross-section is located directly within the weir pool and ~56m downstream from the weir. The data show that baseline flow velocities are predominantly uniform between 0.025-0.05m/s across most of the central areas of the cross-section. As for previous lower flows (Scenario 1 and Scenario 2), there are higher velocities on the right side adjacent to the weir (0.05-0.02m/s), however there are increased velocities (0.05-0.075m/s) on the left bank. Within the weir pool flow velocities remain similar to that in the rest of the cross-section with only a small area of slight reduction to 0.01-0.025m/s indicated adjacent to the point where the bed rapidly deepens away from the weir but a slight increase in flow velocities to 0.05-0.075m/s located in the area immediately around the bed of the weir pool.

- **Section 2** – The cross-section is located ~85m downstream of the weir pool (Section 1) and ~150m downstream of the weir. The channel bed has shallowed by ~4m when compared to Section 1. The velocity data indicate that the right of the channel has lower flow velocities of around 0.05-0.075m/s, increasing to 0.075-0.1m/s towards the centre of the channel and towards the channel bed, and 0.1-0.2m/s towards the left bank. There is a slight decline in flow velocity to 0.025-0.05m/s towards the channel bed.
- **Section 3** – The cross-section is located ~227m downstream from the weir pool (Section 1) and ~290m downstream from the weir. The data show that flow velocities over the cross-section are predominantly 0.05-0.1m/s (an increase from Scenario 2), with velocities reducing to 0.05-0.075m/s close to the channel bed and also the left bank. When compared to previous velocities for Scenario 1 and Scenario 2, there remains a slight asymmetry in flow velocities in the section, with higher flow velocities on the right of the channel when compared to the left.

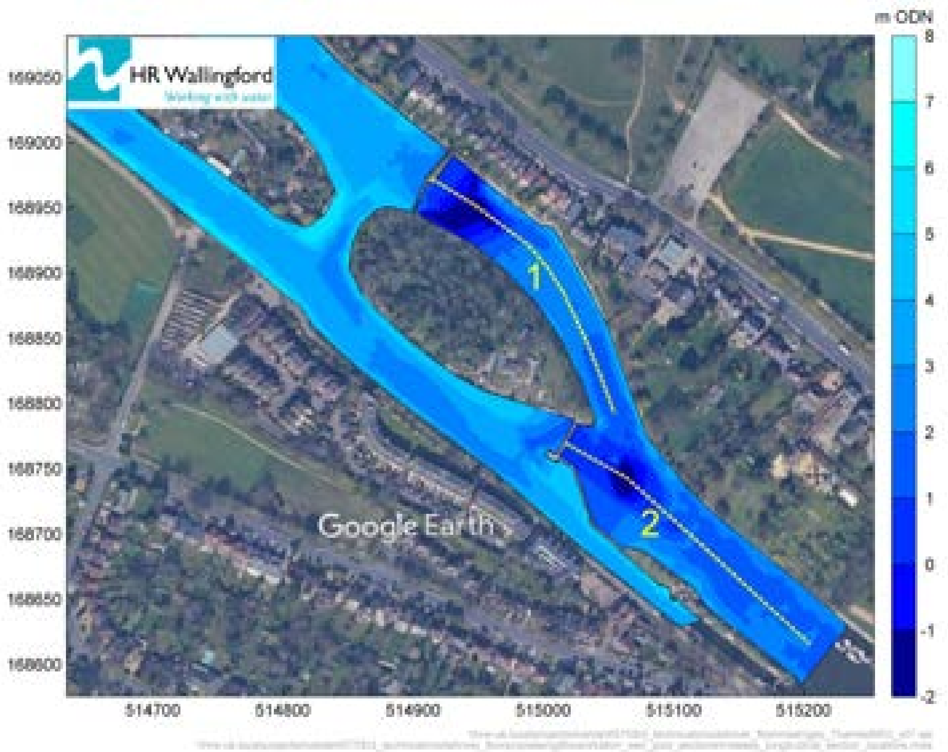
Molesey Weir pool

Modelled data from six river cross-sections were extracted from the baseline modelled data for the Molesey weir pool assessment, focusing on two weir pool areas for the northernmost weir and southernmost weir, these are presented in Figure A-14.

Figure A-14 Molesey weir cross-section locations

In addition to the river cross-sections, modelled data for two longitudinal sections were extracted at Molesey weir, as presented in Figure A-15.

Figure A-15 Molesey weir longitudinal section locations



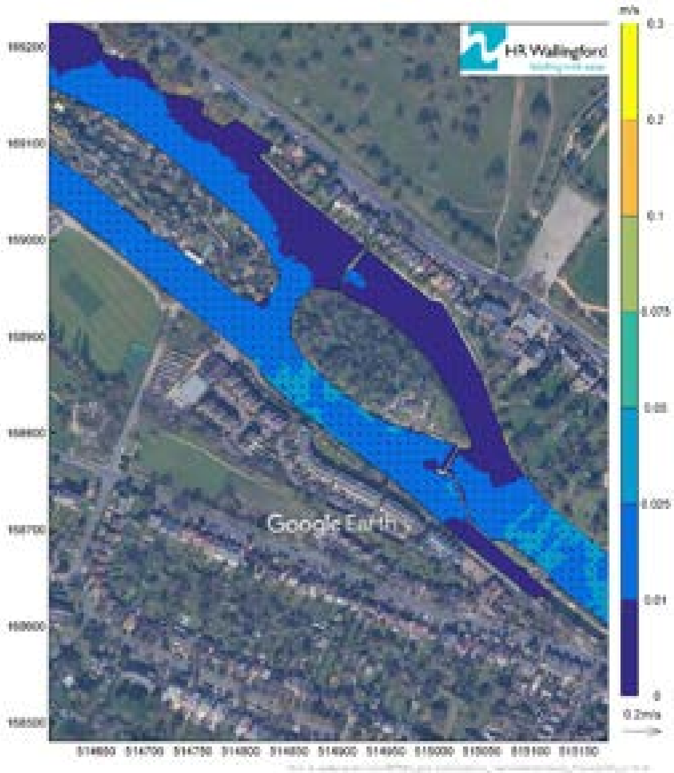
A brief overview of the baseline velocity data at Molesey Weir pool for Scenarios 1 to Scenario 3 are presented below.

Scenario 1: 600 Ml/d river flow, extremely low river flow conditions

Figure A-16

illustrates the baseline depth average velocity at Molesey Weir under 600 Ml/d river flow (Scenario 1).

Figure A-16 Baseline depth-average velocity at Molesey weir, Scenario 1

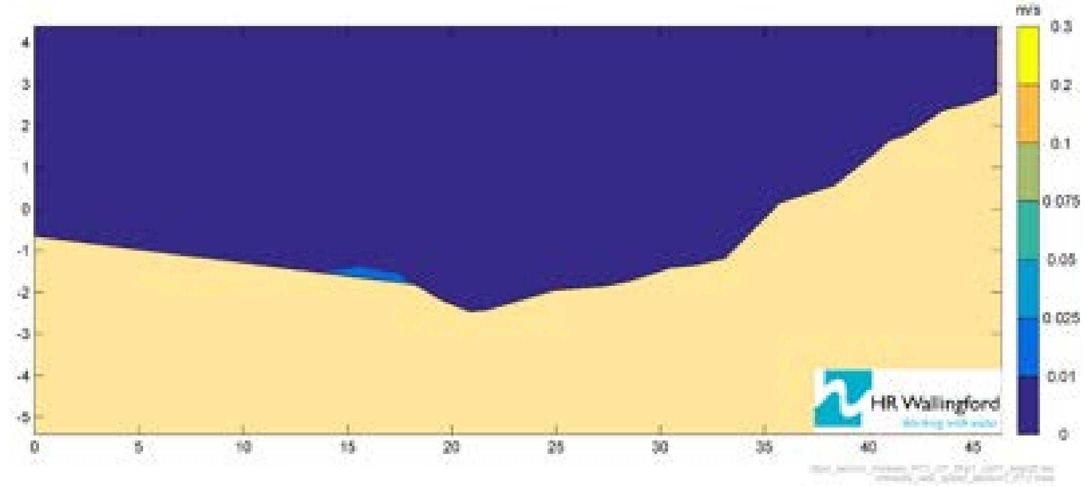


For the northernmost weir, the data indicate velocities are very low, around 0-0.01m/s. For the southernmost weir, flow velocites are predominantly 0.01-0.025m/s, although there are some areas of increased flow velocity (up to 0.025-0.05m/s) in the channel after the confluence of the bifurcated arms of the River Thames.

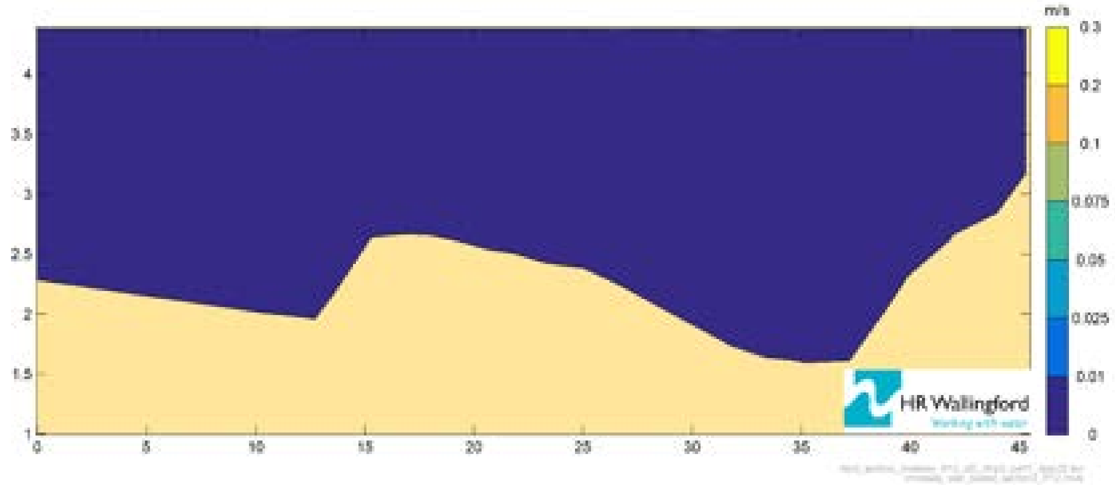
Figure A-17 shows the distribution of depth averaged flow velocity at each of the six cross-sections at Molesey Weir under the 600 Ml/d flow (Scenario 1).

Figure A-17 Cross-sections of baseline flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

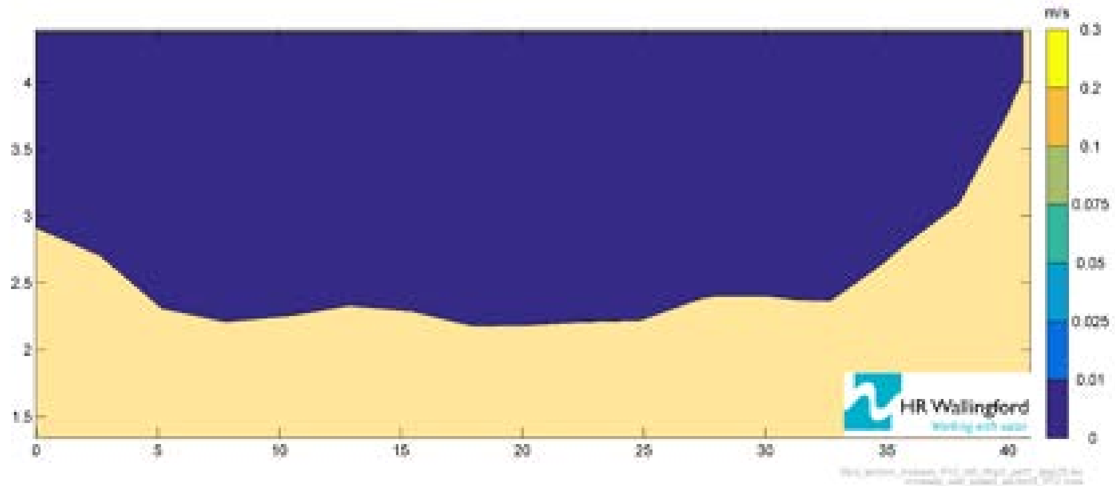
a) Baseline flow velocity, Section 1, northernmost weir



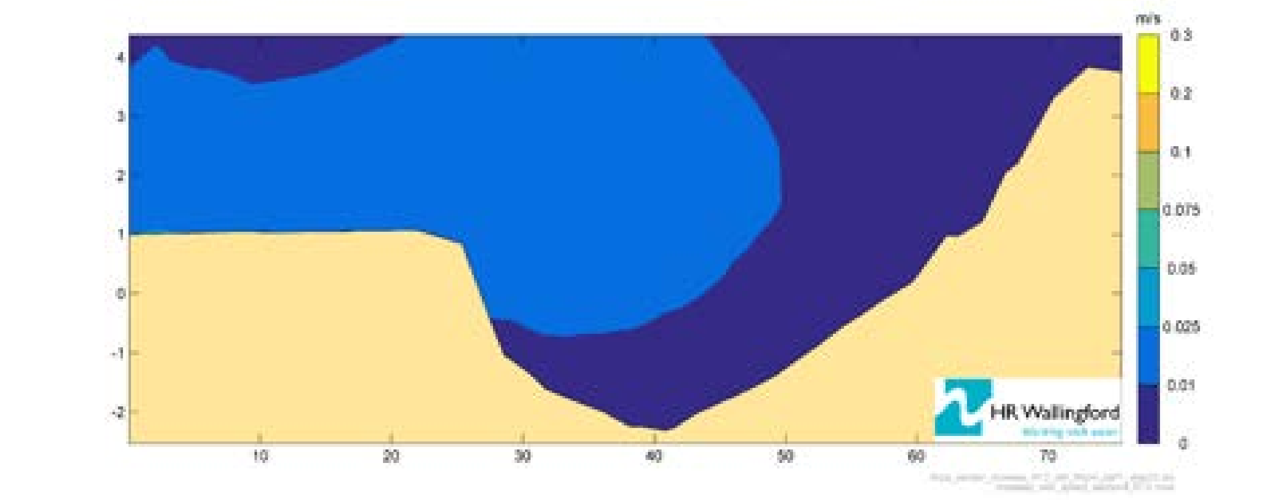
b) Baseline flow velocity, Section 2, northernmost weir



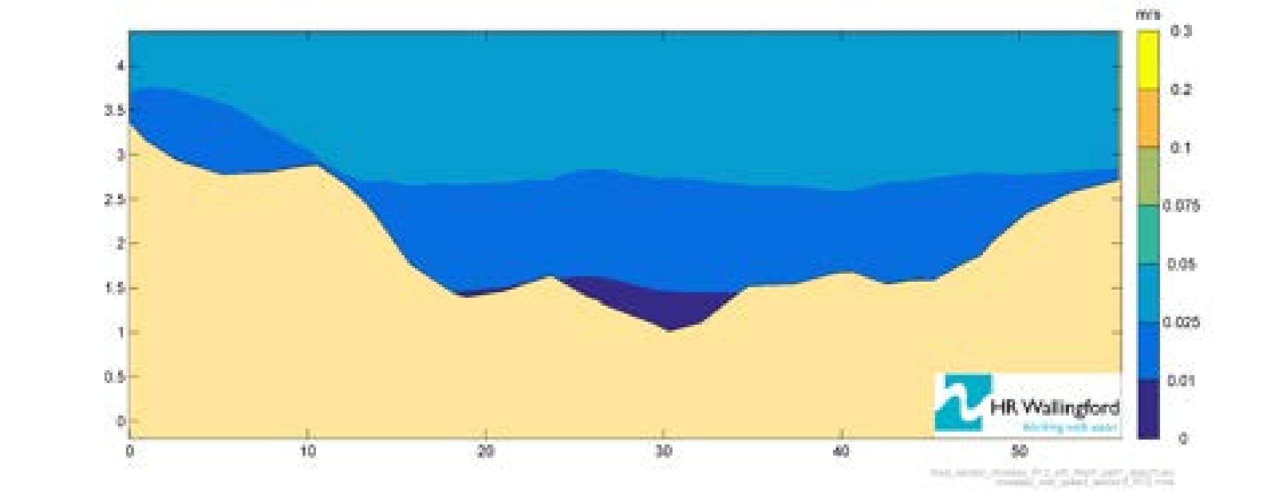
c) Baseline flow velocity, Section 3, northernmost weir



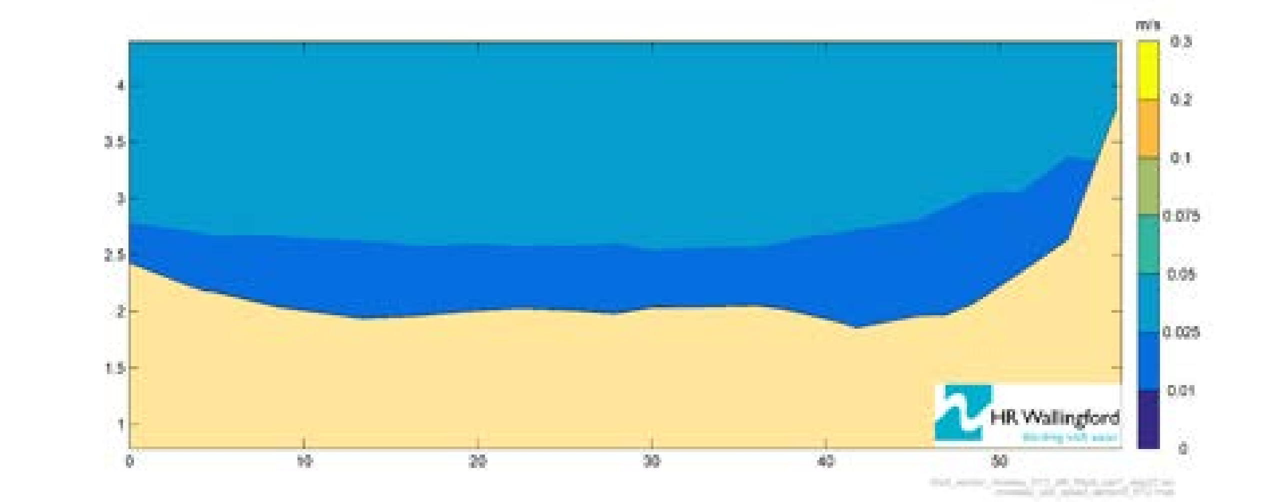
d) Baseline flow velocity, Section 4, southernmost weir



e) Baseline flow velocity, Section 5, southernmost weir



f) Baseline flow velocity, Section 6, southernmost weir



The baseline cross-section velocity data show that:

- **Section 1** – The cross-section is located directly within the weir pool of the northernmost weir and ~40m downstream from the weir. The data show that baseline flow velocities are near wholly 0-0.01m/s across the entire cross-section, except for a very small area on the bed which measures 0.01-0.025m/s.
- **Section 2** – The cross-section is located ~50m downstream of the weir pool (Section 1) and ~90m downstream of the northernmost weir. The channel bed has shallowed by between 2-4m when

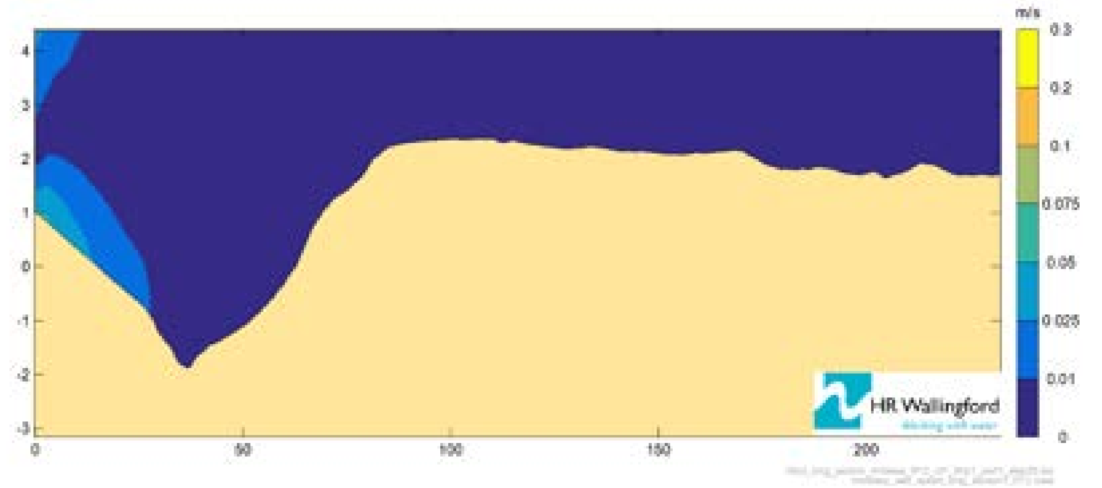
compared to Section 1. The velocity data indicate that the entire cross-section is characterised by essentially still to very slow flow of 0-0.01m/s.

- **Section 3** – The cross-section is located ~95m downstream from the weir pool (Section 1) and ~135m downstream from the northernmost weir. The velocity data indicate that the entire cross-section is characterised by essentially still to very slow flow of 0-0.01m/s.
- **Section 4** – The cross-section is located directly within the weir pool of the southernmost weir and ~54m downstream from the southernmost weir. The data show that velocities of between 0.01-0.025m/s are present on the left bank, declining to 0-0.01m/s on the right bank and in the weir pool.
- **Section 5** – The cross-section is located ~76m downstream from the weir pool (Section 1) and ~129m downstream from the southernmost weir. The data show that there is a vertical velocity gradient at the cross-section, declining from 0.025-0.05m/s towards the water surface to 0.01-0.025m/s to the bottom of the cross-section with a region of 0-0.01m/s at the very bed.
- **Section 6** – The cross-section is located ~213m downstream from the weir pool (Section 1) and ~267m downstream from the southernmost weir. The data show a vertical velocity gradient, with most of the channel cross-section moving at 0.025-0.05m/s and declining to 0.01-0.025m/s towards the channel bed.

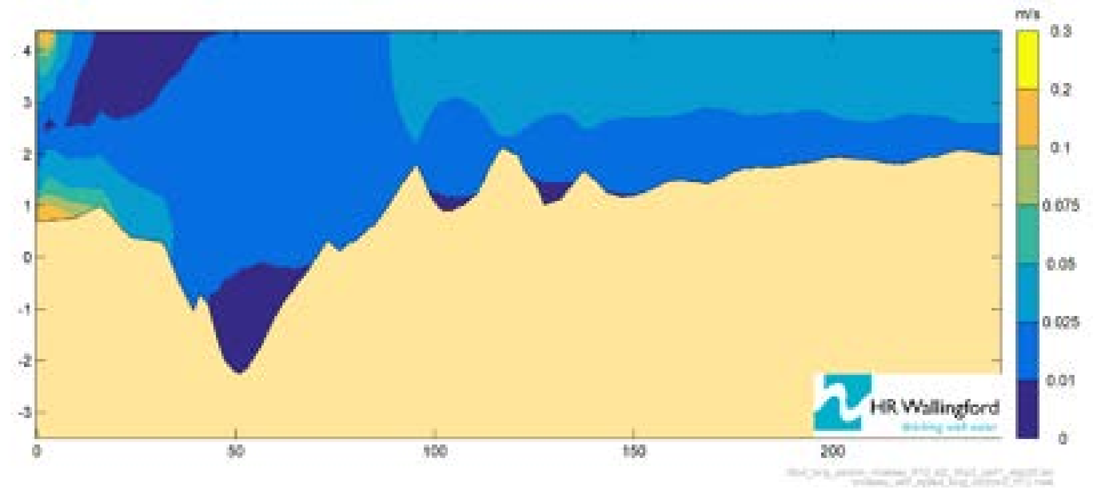
Figure A-18 shows the distribution of depth averaged flow velocity at each of the two longitudinal sections at Molesey Weir under the 600 Ml/d flow (Scenario 1).

Figure A-18 Longitudinal sections of baseline flow velocities at Molesey weir pool, 600 Ml/d (Scenario 1)

a) Baseline flow velocity, longitudinal section 1, northernmost weir



b) Baseline flow velocity, longitudinal section 2, southernmost weir



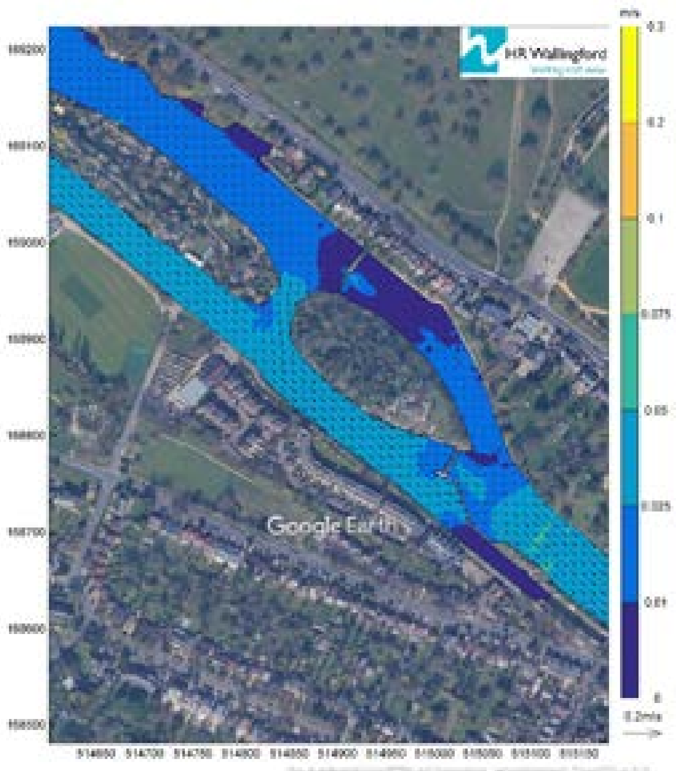
The baseline longitudinal velocity data show that:

- **Section 1** – The data show that baseline flow velocities are predominantly 0-0.01m/s throughout the longitudinal section and the weir pool. There are some localised increases in flow velocity between 0-25m downstream of the weir of between 0.01-0.05m/s, with a small area of this increased velocity encroaching on the upstream side of the weir pool.
- **Section 2** – The data show a more complex velocity profile than for Section 1. It should be noted that the peaks in channel bed between 100-150m are due to large bedform features (possibly dunes or antidunes). Over much of the section velocities range from 0.01-0.025m/s, with surface velocity increasing slightly to 0.025-0.05m/s from 100m downstream (noting the control on flow velocities exerted by the bedforms). Flow velocities in the weir pool are relatively uniform at 0.01-0.025m/s, although these decline to 0-0.01m/s at the base of the of the pool. Higher velocities, between 0.025-0.2m/s are recorded immediately downstream of the weir.

Scenario 2: 780 Ml/d river flow, extremely low river flow conditions

Figure A-19 illustrates the depth-average velocity at Molesey weir under 780 Ml/d river flow (Scenario 2).

Figure A-19 Baseline depth-average velocity at Molesey weir, Scenario 2

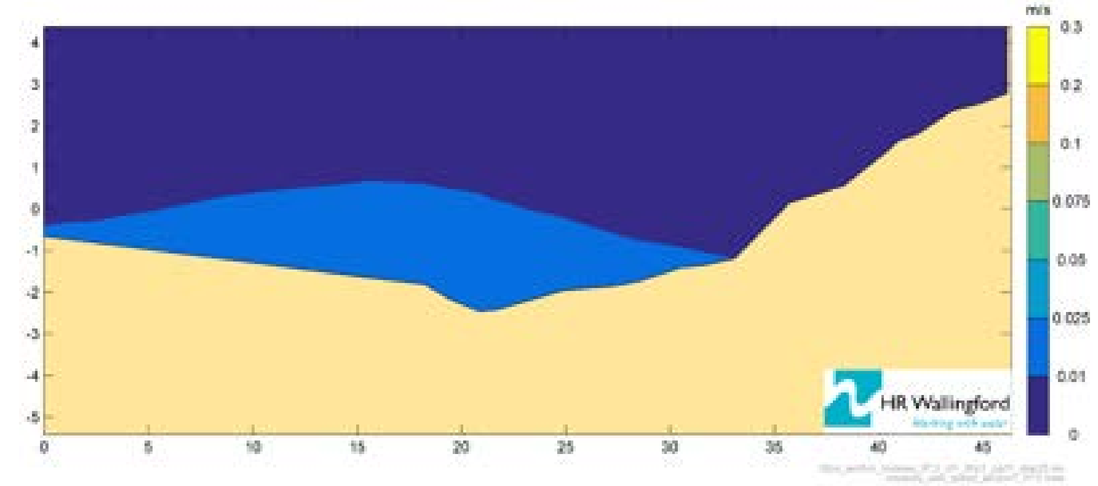


For the northernmost weir, the data indicate velocities are very low, around 0-0.01m/s, although are shown to increase to 0.01-0.025m 190m downstream. For the southernmost weir, flow velocites are predominantly 0.01-0.025m/s downstream of the weir, increasing to 0.025-0.05m/s throughout the channel after the confluence of the bifurcated arms of the River Thames.

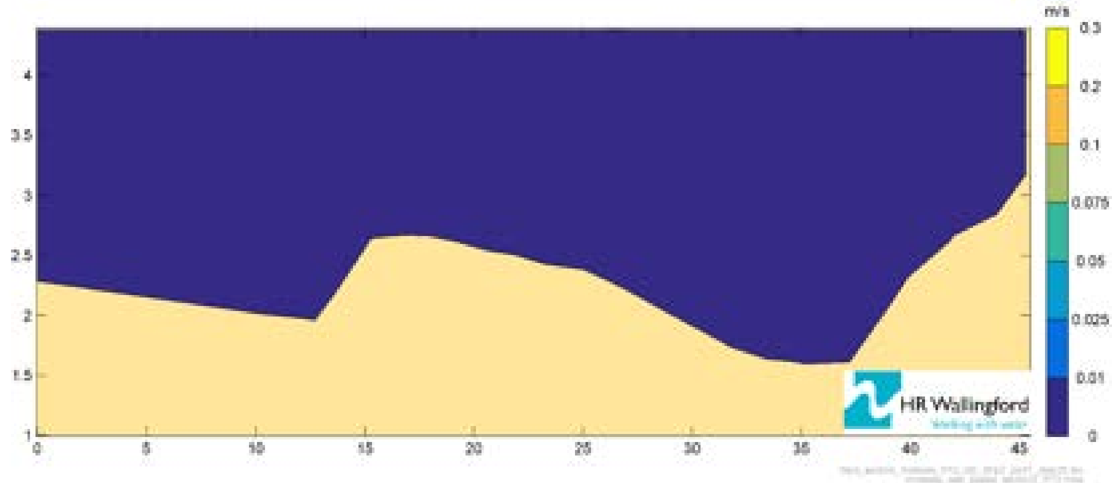
Figure A-20 shows the distribution of depth averaged flow velocity at each of the six cross-sections at Molesey Weir under the 780 Ml/d flow (Scenario 2).

Figure A-20 Cross-sections of baseline flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

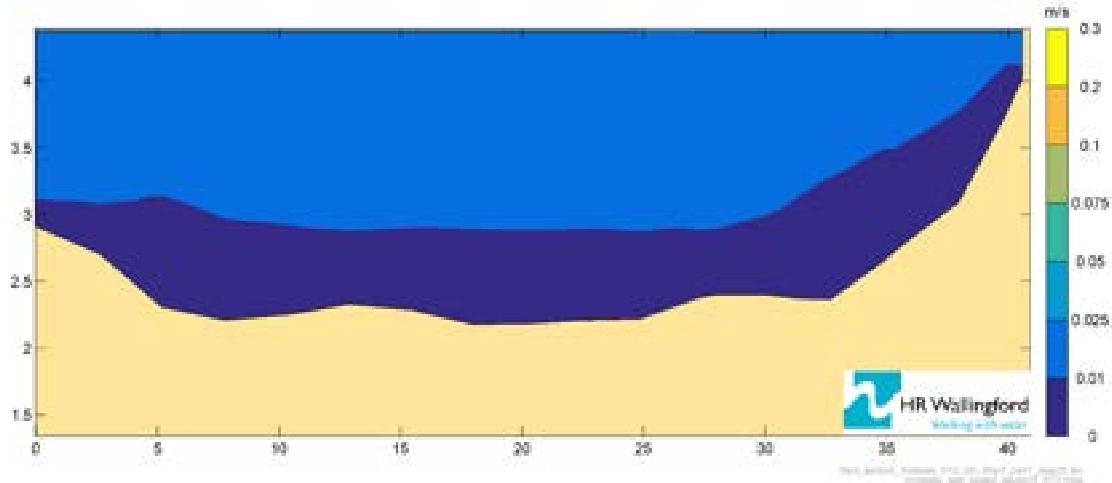
a) Baseline flow velocity, Section 1, northernmost weir



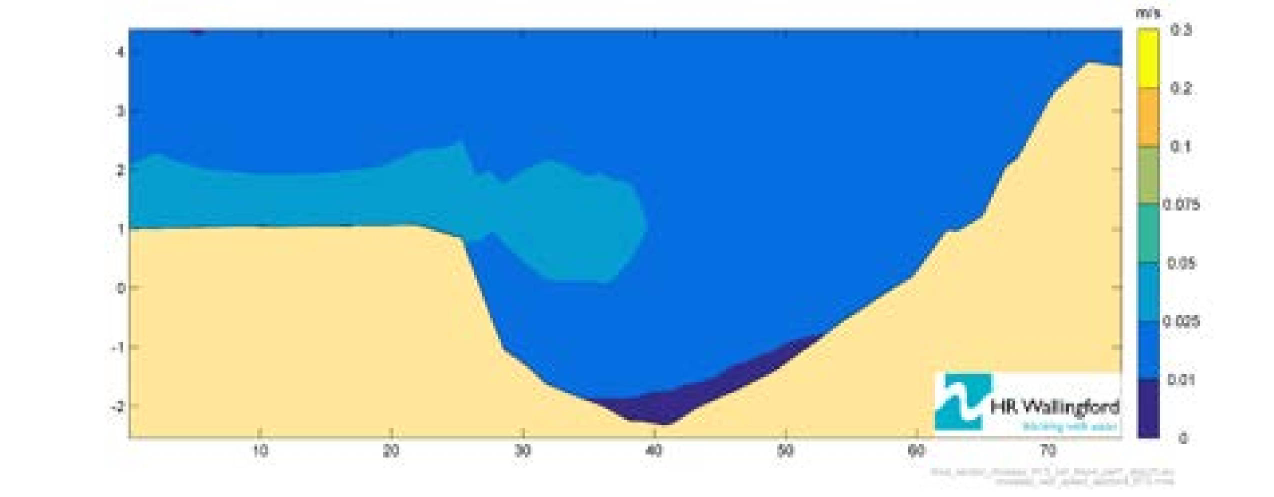
b) Baseline flow velocity, Section 2, northernmost weir



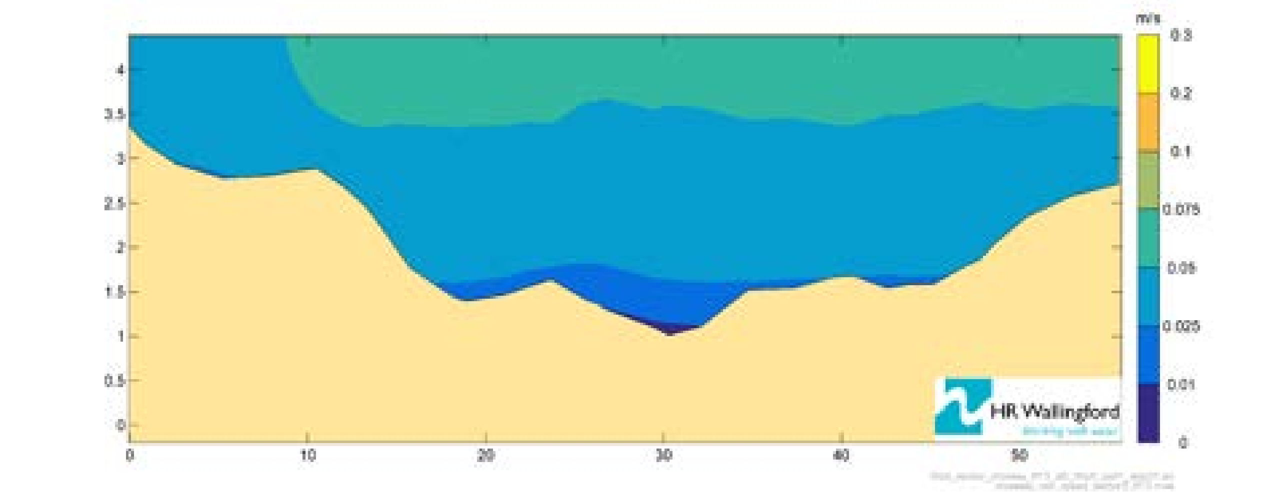
c) Baseline flow velocity, Section 3, northernmost weir



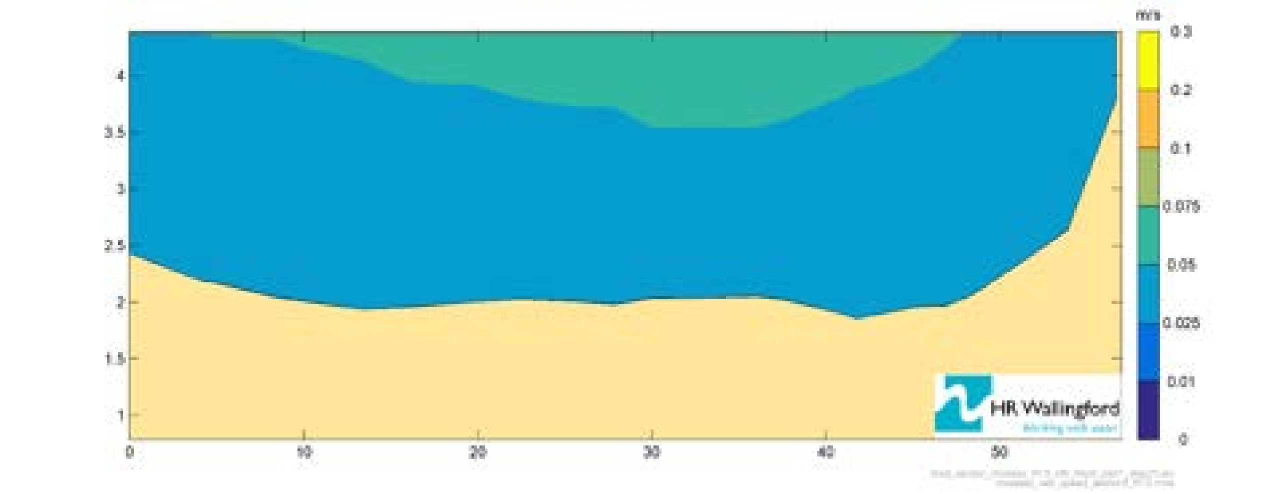
d) Baseline flow velocity, Section 4, southernmost weir



e) Baseline flow velocity, Section 5, southernmost weir



f) Baseline flow velocity, Section 6, southernmost weir



The baseline cross-section velocity data show that:

- **Section 1** – The cross-section is located directly within the weir pool of the northernmost weir and ~40m downstream from the weir. The data show that baseline flow velocities are near wholly 0-0.01m/s across the entire cross-section, with an expansion of the area of flow close to the bed with elevated velocities of 0.01-0.025m/s.
- **Section 2** – The cross-section is located ~50m downstream of the weir pool (Section 1) and ~90m downstream of the northernmost weir. The channel bed has shallowed by between 2-4m when

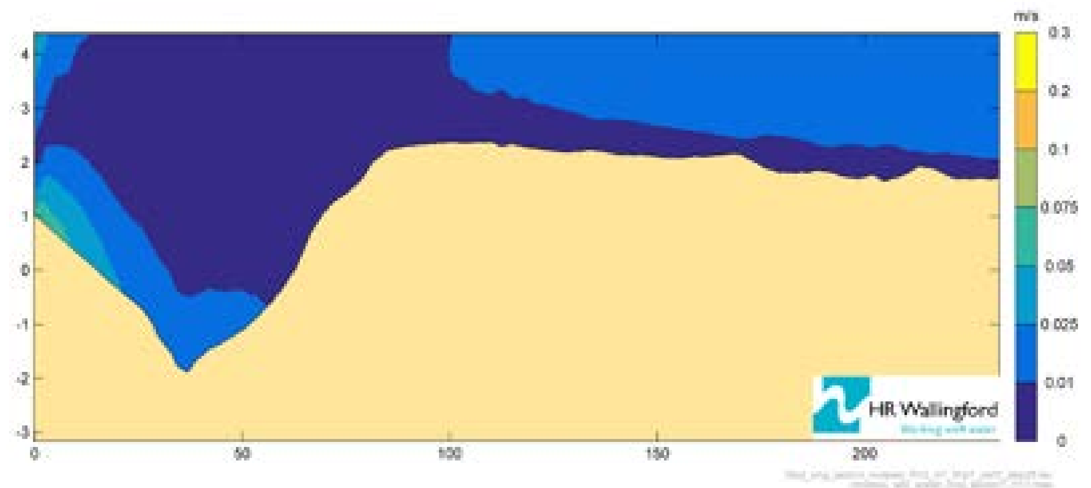
compared to Section 1. The velocity data indicate that the entire cross-section is characterised by essentially still to very slow flow of 0-0.01m/s.

- **Section 3** – The cross-section is located ~95m downstream from the weir pool (Section 1) and ~135m downstream from the northernmost weir. The velocity data indicate an increase of flow velocities to 0.01-0.025m/s over much of the upper and middle cross-section depths. Flow close to the bed remains essentially still to very slow flow of 0-0.01m/s.
- **Section 4** – The cross-section is located directly within the weir pool of the southernmost weir and ~54m downstream from the southernmost weir. The data show that most of the majority of the cross-section is moving at 0.01-0.025m/s, including most of the weir pool, with increased velocities of between 0.025-0.05m/s being present on the left bank. At the very base of the weir pool flow is still to very slow, at around 0-0.01m/s.
- **Section 5** – The cross-section is located ~76m downstream from the weir pool (Section 1) and ~129m downstream from the southernmost weir. The data show that there is a vertical velocity gradient at the cross-section, declining from 0.05-0.075m/s towards the water surface to 0.025-0.05m/s over much of the middle and lower parts of the cross-section to 0.01-0.025m/s towards the bottom of the cross-section. There is a region of 0-0.01m/s at the very bed.
- **Section 6** – The cross-section is located ~213m downstream from the weir pool (Section 1) and ~267m downstream from the southernmost weir. The data show a vertical velocity gradient, with the upper most portion of the flow moving at 0.05-0.075m/s and declining to 0.025-0.05m/s across the remaining flow cross-section.

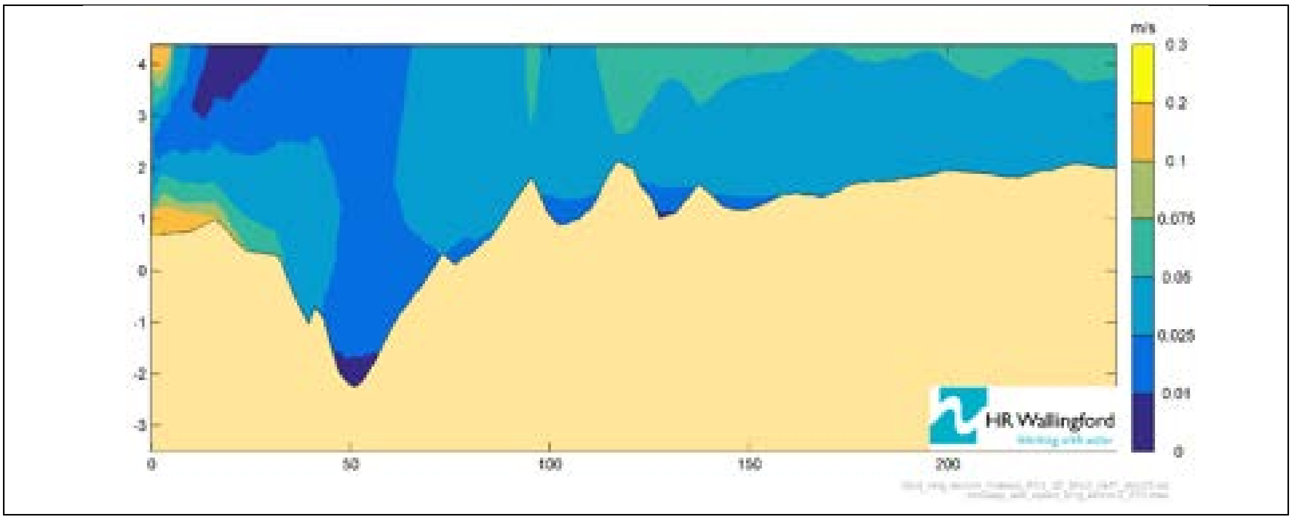
Figure A-21 shows the distribution of depth averaged flow velocity at each of the two longitudinal sections at Molesey Weir under the 780 Ml/d flow (Scenario 2).

Figure A-21 Longitudinal sections of baseline flow velocities at Molesey weir pool, 780 Ml/d (Scenario 2)

a) Baseline flow velocity, longitudinal section 1, northernmost weir



b) Baseline flow velocity, longitudinal section 2, southernmost weir



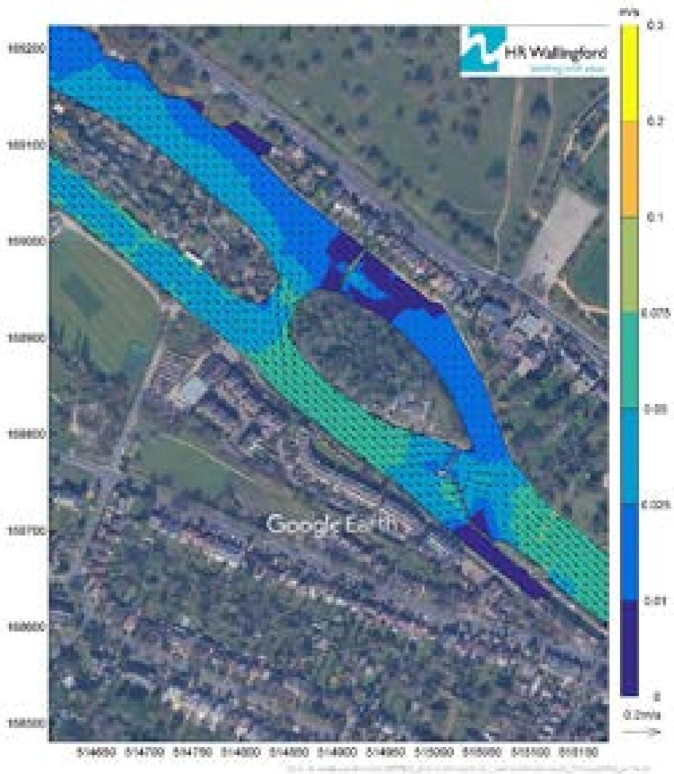
The baseline longitudinal velocity data show that:

- **Section 1** – The data show that baseline flow velocities are predominantly 0-0.01m/s throughout the longitudinal section and the weir pool, although, compared to Scenario 1, there are increases in flow velocity to 0.01-0.025m/s at the upstream face and base of the weir pool and in the main channel from 100m downstream. There are some localised increases in flow velocity between 0-25m downstream of the weir of between 0.01-0.075m/s, with a small area of this increased velocity encroaching on the upstream side of the weir pool.
- **Section 2** – Over much of the section velocities range from 0.01-0.05m/s, with surface velocity increasing to 0.05-0.075m/s from 100m downstream. There remains some control of flow velocity due to the presence of the bedforms and there are reductions in flow velocity on the lee sides of these bedforms. Flow velocities in the weir pool are relatively uniform at 0.01-0.025m/s, however higher velocities from the weir are encroaching on the upstream margins of the weir. There remains a small area of still to very slow flow (0-0.01m/s) at the base of the of the pool. Higher velocities, between 0.05-0.2m/s are recorded immediately downstream of the weir.

Scenario 3: 950 Ml/d river flow, low river flow conditions

Figure A-22 illustrates the depth-average velocity at Molesey weir under 950 Ml/d river flow (Scenario 3).

Figure A-22 Baseline depth-average velocity at Molesey weir, Scenario 3

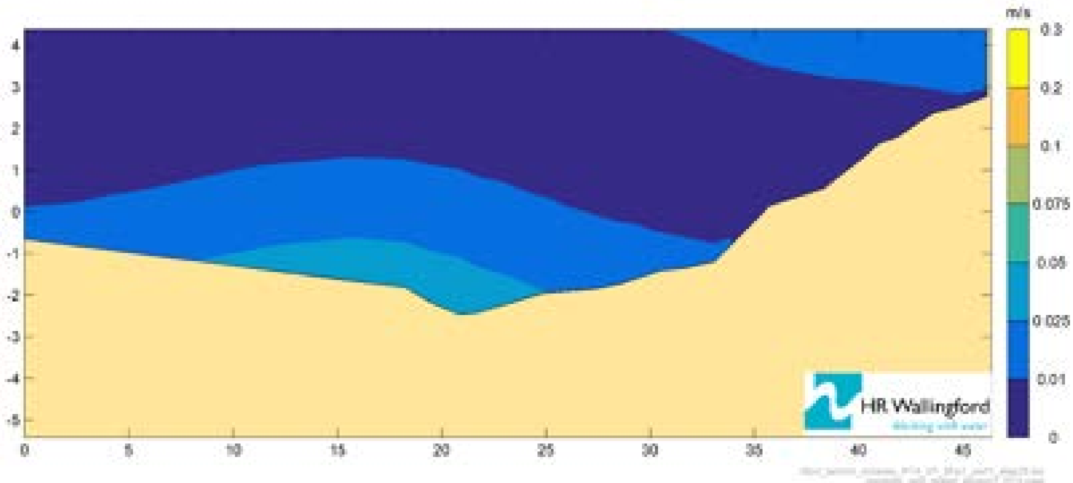


For the northernmost weir, the data indicate velocities are very low, around 0-0.01m/s, although are shown to increase to 0.01-0.025m 190m downstream. For the southernmost weir, flow velocites increase to 0.025-0.05m/s downstream of the weir, increasing to 0.05-0.075m/s throughout the channel after the confluence of the bifurcated arms of the River Thames, although there are spatially discrete areas of higher flow velocities in this section (0.075-0.1m/s).

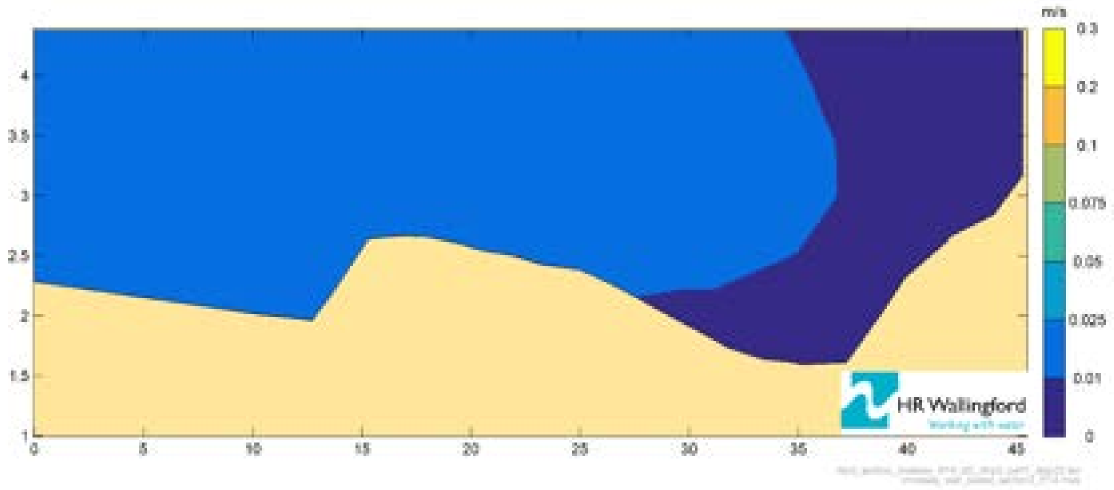
Figure A-23 shows the distribution of depth averaged flow velocity at each of the six cross-sections at Molesey Weir under the 950 Ml/d flow (Scenario 3).

Figure A-23 Cross-sections of baseline flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

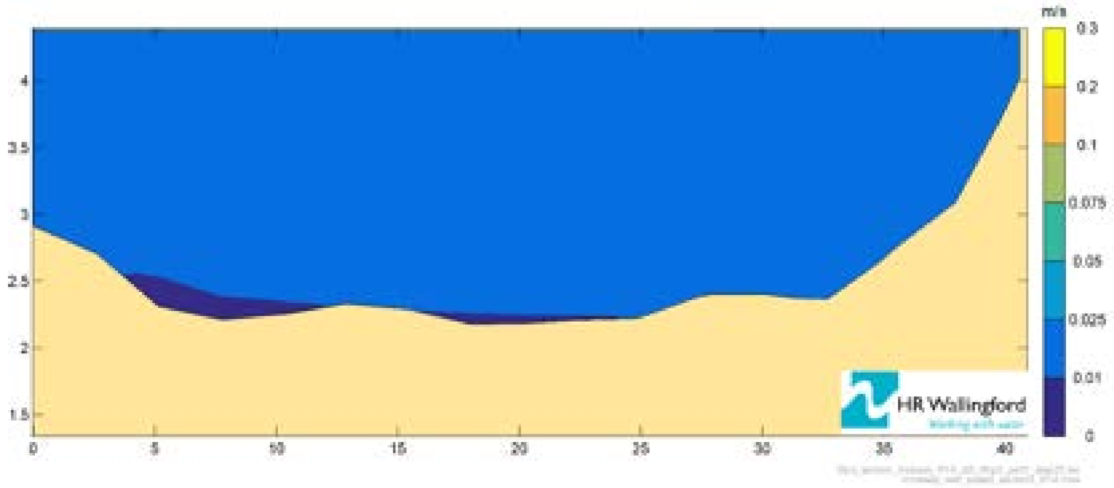
a) Baseline flow velocity, Section 1, northernmost weir



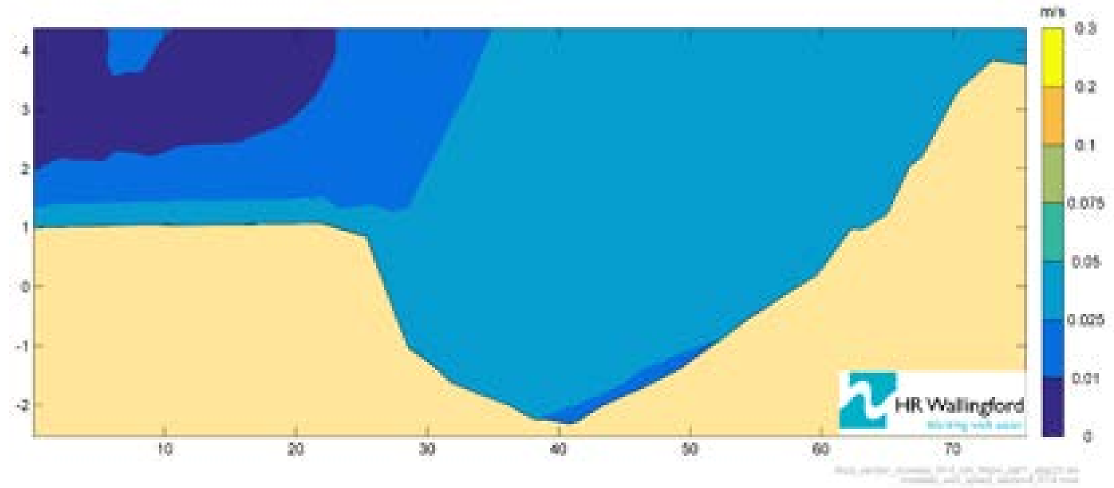
b) Baseline flow velocity, Section 2, northernmost weir



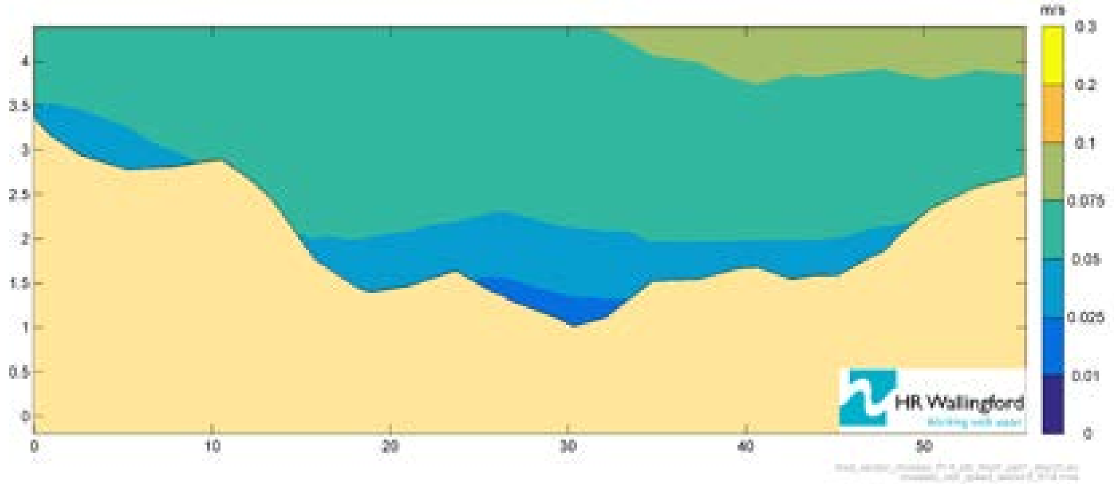
c) Baseline flow velocity, Section 3, northernmost weir



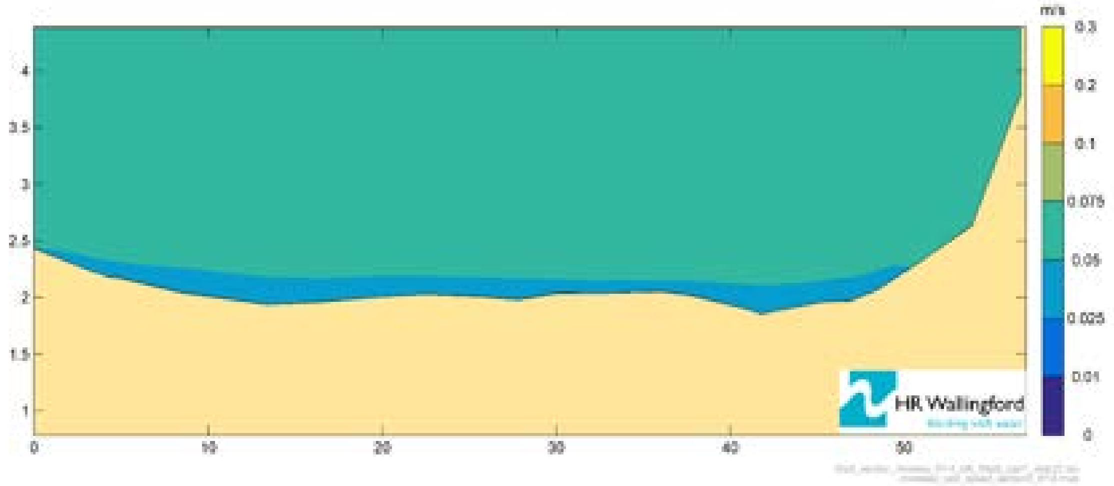
d) Baseline flow velocity, Section 4, southernmost weir



e) Baseline flow velocity, Section 5, southernmost weir



f) Baseline flow velocity, Section 6, southernmost weir



The baseline cross-section velocity data show that:

- **Section 1** – The cross-section is located directly within the weir pool of the northernmost weir and ~40m downstream from the weir. The data show that baseline flow velocities are 0-0.01m/s across much of the cross-section, with an expansion of the area of flow close to the bed with elevated velocities of 0.01-0.05m/s and an area of increased flow velocity to 0.01-0.025m/s towards the right bank.
- **Section 2** – The cross-section is located ~50m downstream of the weir pool (Section 1) and ~90m downstream of the northernmost weir. The channel bed has shallowed by between 2-4m when compared to Section 1. The velocity data indicate that flows have increased from previous scenarios, with much of the cross-section characterised by 0.01-0.025m/s flow velocities. A portion of the flow depth on the right banks remains still to very slow flowing at 0-0.01m/s.
- **Section 3** – The cross-section is located ~95m downstream from the weir pool (Section 1) and ~135m downstream from the northernmost weir. The velocity data indicate that flow velocity has increased over the majority of the cross-section to 0.01-0.025m/s, with only a small area of still to very slow flow (0-0.01m/s) being present at the channel bed.
- **Section 4** – The cross-section is located directly within the weir pool of the southernmost weir and ~54m downstream from the southernmost weir. The data show that velocities across most of the cross-section, including the weir pool, have increased to 0.025-0.05m/s when compared to Scenario 2. The left bank has noted a decrease in velocities to 0-0.025m/s, when compared with Scenario 2.
- **Section 5** – The cross-section is located ~76m downstream from the weir pool (Section 1) and ~129m downstream from the southernmost weir. The data show that there is a vertical velocity gradient at the cross-section, with velocities increasing when compared to Scenario 2. A high of 0.075-0.1m/s is noted at the surface close to the right bank, declining to 0.05-0.075m/s over most of the cross-section. Flow

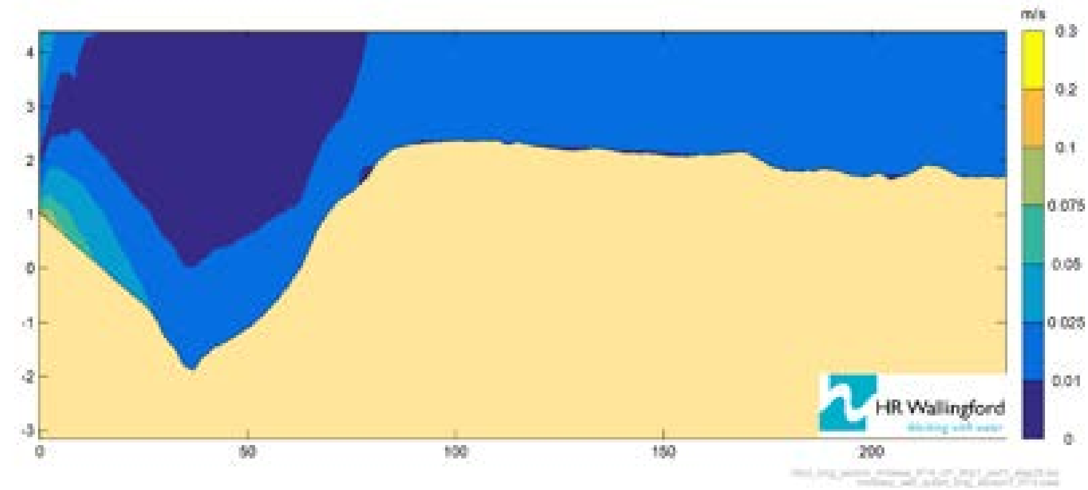
velocity declines further to 0.025-0.05m/s over a narrow section covering most of the bed area, with a spatially limited area of 0.025-0.05m/s at the very base of the channel.

- **Section 6** – The cross-section is located ~213m downstream from the weir pool (Section 1) and ~267m downstream from the southernmost weir. When compared to Scenario 1 and Scenario 2, the data show that the channel cross-section is mostly of a uniform flow velocity of 0.05-0.075m/s, with a thin section of flow of 0.025-0.05m/s covering most of the channel bed.

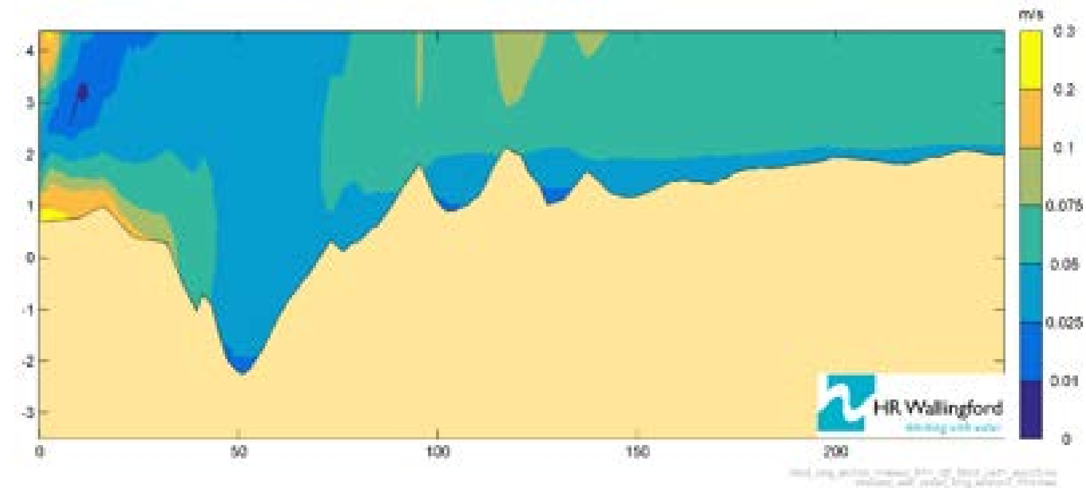
Figure A-24 shows the distribution of depth averaged flow velocity at each of the two longitudinal sections at Molesey Weir under the 950 Ml/d flow (Scenario 3).

Figure A-24 Longitudinal sections of baseline flow velocities at Molesey weir pool, 950 Ml/d (Scenario 3)

a) Baseline flow velocity, longitudinal section 1, northernmost weir



b) Baseline flow velocity, longitudinal section 2, southernmost weir



The baseline longitudinal velocity data show that:

- **Section 1** – The data show that baseline flow velocities are predominantly 0.01-0.025m/s throughout the longitudinal section and within the weir pool. Above the weir pool flow velocities remain still to very slow at 0-0.01m/s. Flow velocities remain highest downstream of the weir pool at 0.025-0.1m/s. The data suggest that the increased flow velocities from the weir could be moving along the channel bed into the weir pool, although it should be noted these velocities are very small.
- **Section 2** – Over much of the section velocities range from 0.025-0.075m/s, with surface velocity occasionally increasing to 0.075-0.1m/s from 100m downstream. There is reduced control of flow from the bedforms, although there are reductions in flow velocity on the lee sides of the bedforms. Flow velocities in the weir pool have increased from Scenario 2 up to 0.025-0.05m/s, with higher velocity flow from the weir encroaching on the upstream margins of the weir. There remains a small area of

very slow flow (0.01-0.25m/s) at the base of the of the pool, this being higher than for previous scenarios. Higher velocities, between 0.05-0.3m/s are recorded immediately downstream of the weir.

4.3. Estuarine Thames Tideway

Changes in habitat within the estuarine Thames Tideway seawards of Teddington Weir are presented below as relative changes in exposure along the Tideway.

Mogden scheme

The results of 3D tideway modelling undertaken by HR Wallingford for the estuarine Thames Tideway between Teddington Weir and Southend on Sea is used to understand the baseline exposure for marginal wetted habitat under the A82 and M96 flows (Table A-1).

Table A-1 Mogden scheme baseline intertidal area exposure

Reach	Max exposed area (ha)		Average exposed area (ha)		Average duration of exposure (hours)	
	A82 baseline	M96 baseline	A82 baseline	M96 baseline	A82 baseline	M96 baseline
Teddington Weir to Richmond Half-tide Sluice	0.4	0.4	0.0	0.1	1.8	4.4
Richmond Half-tide Sluice to Kew Bridge	9.7	10.6	3.6	3.8	9.0	8.7
Kew Bridge to Hammersmith Bridge	20.8	22.4	4.7	4.9	5.4	5.2
Hammersmith Bridge to Wandsworth Bridge	32.1	32.3	6.8	6.9	5.1	5.2
Wandsworth Bridge to Vauxhall Bridge	33.2	33.0	4.0	4.2	2.9	3.0
Vauxhall Bridge to Tower Bridge	15.1	15.1	1.3	1.3	2.0	2.1
Total	111.3	113.7	20.4	21.2	---	---

Visual representation of the distribution of the percentage of time of intertidal exposure for the baseline A82 and M96 model runs are presented in Figure A-25 and Figure A-26 respectively.

Figure A-25 Mogden scheme A82 baseline percentage of time intertidal exposure (15 October to 1 November)

Figure A-26 Mogden scheme M96 baseline percentage of time intertidal exposure (15 October to 1 November)

Baseline habitat exposure data for the Mogden scheme shows up to a total of 111.3ha and 113.7ha of exposure between Teddington Weir and Tower Bridge (over a total distance of 31km). The data show that the largest area of exposure lies between Kew Bridge and Vauxhall Bridge. There is limited difference in baseline exposure area under the A82 and M96 scenarios. Average duration of exposure is greatest between Richmond Weir and Kew Bridge (9.0h), declining in a seaward direction to 2.0h between Vauxhall Bridge to Tower Bridge as the tidal influence increases.

Beckton scheme

The results of 3D tideway modelling undertaken by HR Wallingford for the estuarine Thames Tideway between Teddington Weir and Southend on Sea and is used to understand the baseline exposure for marginal wetted habitat (Table A-2).

Table A-2 Beckton scheme baseline intertidal area exposure

Reach	Max exposed area (ha)		Average exposed area (ha)		Average duration of exposure (hours)	
	A82 baseline	M96 baseline	A82 baseline	M96 baseline	A82 baseline	M96 baseline
Tower Bridge to Beckton STW outfall	126.4	126.4	24.3	24.9	4.6	4.7
Beckton to Dagenham (3km seaward)	44.2	44.0	16.0	16.1	8.7	8.8
Dagenham to QE2 Bridge	181.0	180.7	56.5	57.1	7.5	7.6
Total	351.6	351.1	96.7	98.1	---	---

Visual representation of the distribution of the percentage of time of intertidal exposure for the baseline A82 and M96 model runs are presented in Figure A-27 and Figure A-28 respectively.

Figure A-27 Beckton scheme A82 baseline percentage of time intertidal exposure (15 October to 1 November)

Figure A-28 Beckton scheme A82 baseline percentage of time intertidal exposure (15 October to 1 November)

Baseline habitat exposure for the Beckton scheme shows up to a total of 351.6ha and 351.1ha of exposure between Tower Bridge to QE2 Bridge (over a total distance of 32km). The data show that the largest area of exposure lies between Dagenham to QE2 Bridge. There is a slight reduction in baseline exposure area between the A82 and M96 scenarios. Average duration of exposure is greatest between Beckton and to Dagenham.

4.4. Freshwater Lee Diversion Channel

An understanding of the channel morphology and habitats present has been undertaking using extant data. The location of key data points within the reach (namely photo locations and cross-sections) have been provided in Figure 6-1.

Figure 6-1 Location of key data points within the study reach.

Character photos detailing the short study reach of the Enfield Island Loop covering the location of the proposed outfall and existing intake and taken on 13 September 2022, are presented in Table 6-1.

Table 6-1 Enfield Island Loop character photos

a) Photo 1 – looking upstream around the location of the proposed Beckton water recycling outfall.	b) Photo 2 – looking downstream toward the proposed Beckton water recycling outfall location on right bank.
	
c) Photo 3 – looking downstream at the King George V Reservoir intake channel and intake structure on the right of the image.	d) Photo 4 – looking upstream towards reservoir intake. Note willow over channel and extensive in-channel macrophytes.
	
e) Photo 5 – looking downstream towards Lee Flood Diversion Channel with King George V Reservoir to right. Note extensive in-channel macrophytes.	f) Photo 6 – looking upstream at weir located at end of the Enfield Island Loop.
	

A MoRPh survey of the reach between the proposed outfall and the King George V Reservoir intake undertaken on 20 April 2022 provides information to characterise part of the reach. The survey indicates that both bank tops are characterised by extensive grasses and short herbs with deciduous trees present. A footpath runs along much of the right bank. Bank faces are characterised as being steep and obviously reshaped with extensive artificial concrete reinforcement of the banks. Channel substrate was identified as being composed of silt and sand towards the upper portions of the reach and increasing in size to gravel-pebble substrate towards the bottom of the reach. No physical features (such as bars, berms, eroding banks etc.) were identified in the reach, although submerged macrophytes were present, and flow types were dominantly smooth. These data are illustrated on the relevant character photos, namely photos 1, 2 and 3

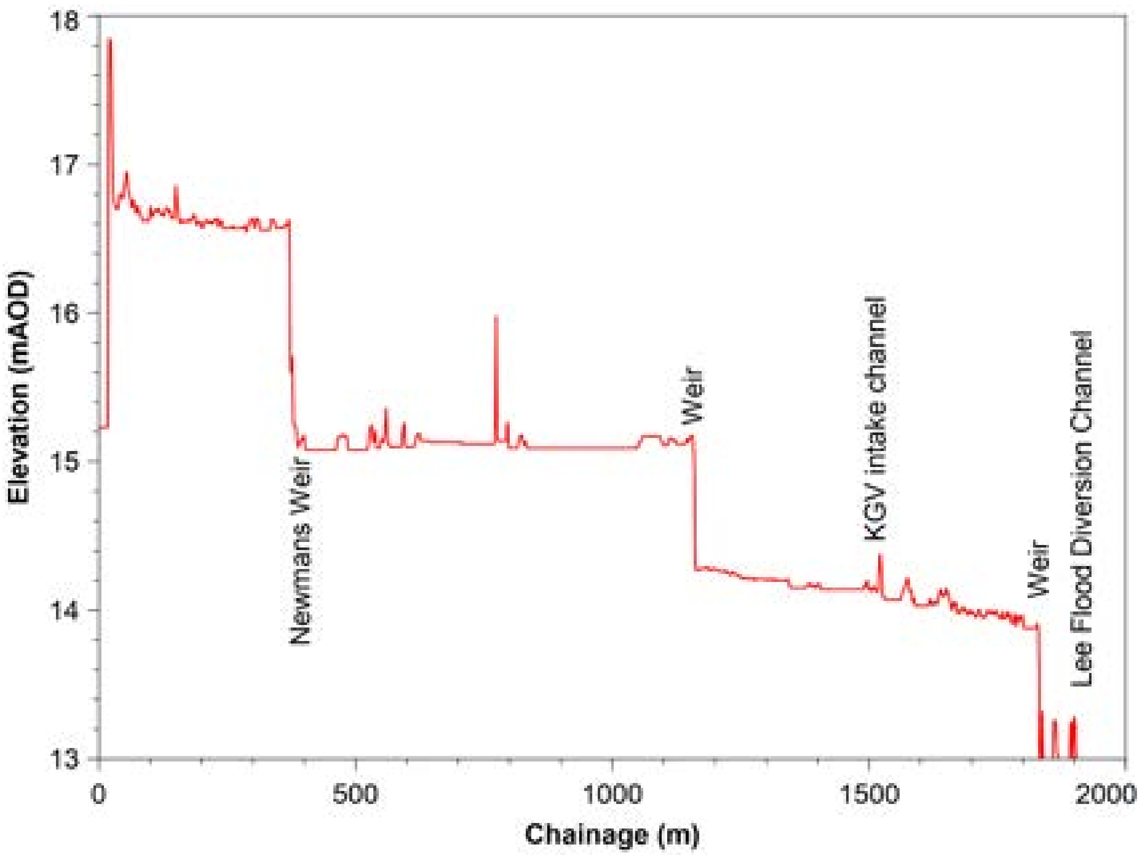
(Table 6-1a, b and c). An overview of hydrodynamic conditions within the Enfield Island Loop is presented in Appendix 1 Section 2.4.

Downstream of the MoRPh reach, and the reservoir intake channel and structure, photos 4, 5 and 6 (Table 6-1d, e and f) indicate that the high degree of anthropogenic modification of the channel continues, clearly showing steep to vertical concrete banks. Where visible substrates appear to be relatively fine and not likely to have differed greatly from that indicated by the MoRPh surveys. The most significant difference is the presence of extensive in-channel emergent macrophytes downstream of the reservoir intake channel, these comprising over 70% of the channel bed along the left bank, and concentrating flow mostly down the right bank. These macrophytes terminate at the weir at the end of the reach (Table 6-1f).

The high level of anthropogenic modification is supported by an RHS survey of the reach undertaken in September 2003 (RHS ID 30485), with a Habitat Modification Class of 5 (with five being the lowest class), predominantly due to the presence of extensive resectioned and reinforced banks and bridges. The survey indicated a low habitat quality assessment score of 23, suggesting that there was a distinct lack of any suitable habitat linked features at the time of the survey.

Further characterisation of the morphology of the channel has been undertaken using existing Environment Agency 1m LiDAR from the National LiDAR programme[5] to determine a longitudinal section of the channel (Figure 6-2, with location of section shown on inset map in Figure 6-1) and cross-sections of the channel at five locations between the proposed outfall and the end of the Enfield Island Loop (Table 6-2, with locations on Figure 6-1). For each cross section, the left of the section represents the left bank of the channel.

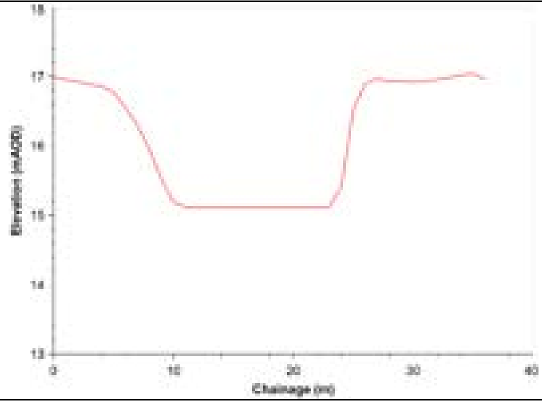
Figure 6-2 Enfield Island Loop longitudinal section



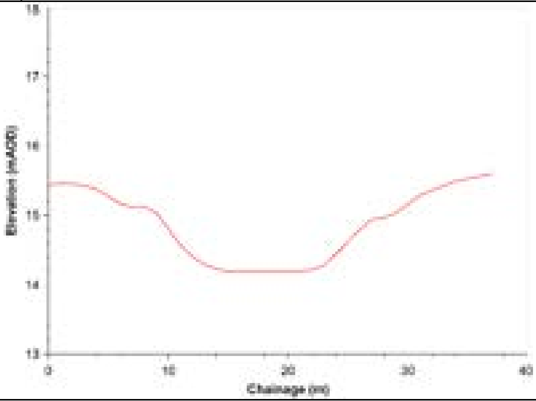
[5] https://www.data.gov.uk/dataset/f0db0249-f17b-4036-9e65-309148c97ce4/national-lidar-programme. Accessed 12 September 2022.

Table 6-2 Enfield Island Loop channel cross sections

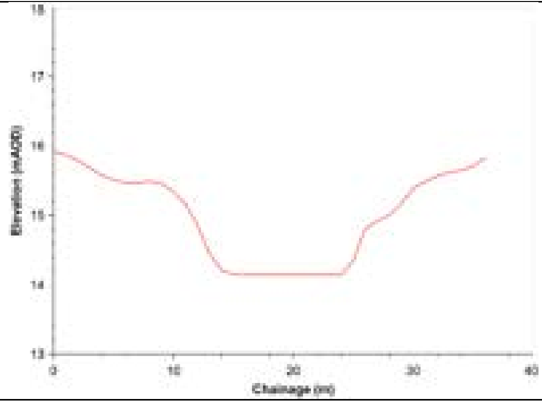
a) Cross section 1



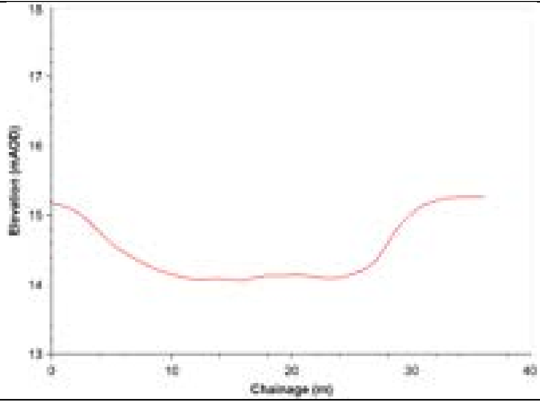
b) Cross section 2



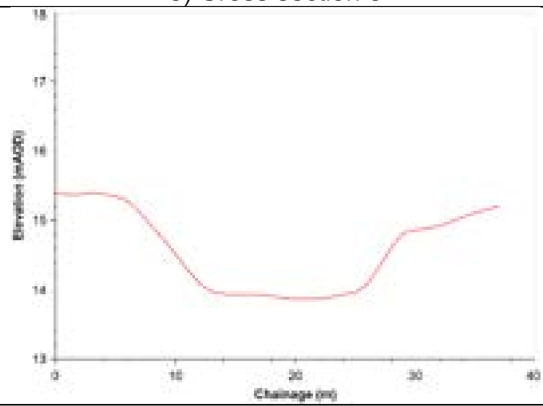
c) Cross section 3



d) Cross section 4



e) Cross section 5



The longitudinal section (Figure 6-2) covering the entire Enfield Island Loop from the bifurcation from the Lee Flood Diversion Channel to the confluence with the Flood Diversion Channel clearly shows the low gradient, extensive ponded flow and heavily modified nature of the longitudinal section of the channel due to the presence of three significant weirs along the reach. The lower most weir at ~1.83km downstream is that represented in (Table 6-1f). The individual cross sections also highlight the relatively steep and vertical nature of the reinforced banks (commonly rising by 1.5m over 1m), acknowledging that the spatial resolution of the LiDAR is only 1m and therefore the banks are likely to be much steeper, as evidenced by photographic evidence in Table 6-1.

The data clearly show that the reach of the River Lee around the Enfield Island Loop is heavily modified with no physical features, and contains several significant barriers to flow, sediment and ecology. Photo and cross-section data indicate a largely box-section shaped channel, which given ADCP data seems likely to respond to changes in flow with increases in velocity rather than depth. Given the highly modified nature of the channel between the proposed Beckton water recycling outfall and the King George V Reservoir intake, its box-section form and the lack of any in-channel habitat features changes in flow are not likely to exert any influence on the very limited habitat present. The data indicate the presence of extensive macrophytes within the channel downstream of the reservoir intake. While these likely provide some in-channel habitat potential these macrophytes fall outside the potential area of impact of the scheme and there currently exists no knowledge of how much flow bypasses the current reservoir intake and passes forward into the Lee Flood Diversion Channel and therefore what impacts any pass forward flow exerts on these macrophytes.

5. FISH PASS AND BARRIER PASSABILITY

5.1. Overview

Across the freshwater study area, modelled and measured information are available from which to describe the water level at fish passes and barriers.

5.2. Freshwater River Thames

Baseline water levels have been extracted from the 2D river model at points upstream and downstream of each of the three weirs. Locations of these sample points and baseline water levels for each are presented below. There are extant fish passes and separate eel passes at each of Sunbury Weir, Molesey Weir and Teddington Weir. There are no fish passes on weirs in the Enfield Island Loop.

Sunbury Weir water levels

Baseline water levels were extracted from the 2D model for upstream (S1 and S2) and downstream (S3 and S4) of Sunbury Weir (Figure A-29).

Figure A-29 Water level extraction points at Sunbury Weir

Baseline water levels at the weir are presented in Table A-3.

Table A-3 Modelled baseline water levels at Sunbury Weir under varying river flows

Sample location	Water level (mAOD)		
	600 Ml/d river flow	780 Ml/d river flow	950 Ml/d river flow
S1 (upstream)	8.02	8.06	8.08
S2 (upstream)	8.02	8.06	8.08
S3 (downstream)	6.26	6.30	6.33
S4 (downstream)	6.26	6.30	6.33

The data show that levels upstream and downstream of Sunbury Weir remain relatively invariant under changing river flows, with upstream levels increasing by 0.06m for a 350 Ml/d increase in flow and downstream levels increasing by 0.07m for a 350 Ml/d increase in flow.

Molesey Weir water levels

Baseline water levels were extracted from the 2D model for upstream (M1 and M2) and downstream (M3 and M4) of Molesey Weir (Figure A-30).

Figure A-30 Water level extraction points at Molesey Weir

Baseline water levels at the weir are presented in Table A-4.

Table A-4 Modelled baseline water levels at Molesey Weir under varying river flows

Sample location	Water level (mAOD)		
	600 Ml/d river flow	780 Ml/d river flow	950 Ml/d river flow
M1 (upstream)	6.26	6.30	6.33
M2 (upstream)	4.38	4.38	4.38

Sample location	Water level (mAOD)		
	600 Ml/d river flow	780 Ml/d river flow	950 Ml/d river flow
M3 (downstream)	6.26	6.30	6.33
M4 (downstream)	4.38	4.38	4.38

The data show that levels upstream and downstream of Molesey Weir remain relatively invariant under changing river flows. Upstream and downstream levels are shown to increase by 0.07m for a 350 Ml/d increase in flow.

Teddington Weir water levels

Baseline water levels were extracted from the 2D model for upstream (T1 and T2) of Teddington Weir (Figure A-31) only.

Figure A-31 Water level extraction points at Teddington Weir

Table A-5 Modelled baseline water levels at Teddington Weir under varying river flows

Sample location	Water level (mAOD)		
	300 Ml/d river flow	400 Ml/d river flow	600 Ml/d river flow
S1 (upstream)	4.37	4.38	4.41
S2 (upstream)	4.37	4.38	4.41

The data show that levels upstream of Teddington Weir remain relatively invariant under changing river flows, with upstream levels increasing by 0.04m for a 300 Ml/d increase in flow.

5.3. Freshwater Enfield Island Loop

There are three weirs located on Enfield Island Loop, Newmans Weir at the top of the loop, a larger weir located immediately upstream of the intake into King George V Reservoir and a small low weir at the bottom of the loop prior to the inflow into the Lee Diversion Channel (see photo of weir in Table 6-1f).

The ADCP sites surveyed on the loop (Site 10 and Site 12) are located between the latter two weirs and therefore do not provide any significant information on fish passage and barrier passibility at these weirs. However, associated level gaugings taken using fixed pressure transducer sensors at Site 10 and Site 12 at 15-minute intervals indicates an average difference in level between the two sites of 0.15m, ranging between a minimum level difference of -0.213m and a maximum level difference of 0.452m. This information, while indicating that levels downstream of the King George V Reservoir intake are lower than upstream does not provide any information to better understand passibility of the weirs. The photo of the weir at the end of the Enfield Island Loop (Table 6-1f) indicates that there is no fish pass present on the weir.

6. RICHMOND POUND DRAWDOWN PHYSICAL ENVIRONMENT

Baseline hydrodynamics and sediment concentrations for Richmond Pound during the November drawdown period are presented in combination with modelled changes in hydrodynamics are presented in Figure 4-56 and Figure 4-57 while suspended sediment concentrations are presented in Figure 4-60 and Figure 4-61.

The results of 3D tideway modelling undertaken by HR Wallingford for the estuarine Thames Tideway is used to understand the baseline exposure for marginal wetted habitat under the A82 and M96 flows during the November drawdown of the Richmond Pound. Visual representations of the distribution of the percentage of time of intertidal exposure for the baseline A82 and M96 model runs are presented in Figure A-32 and Figure A-33 respectively.

Figure A-32 Mogden scheme A82 baseline percentage of time intertidal exposure (1 – 16 November) during Richmond Pound November drawdown period

Figure A-33 Mogden scheme M96 baseline percentage of time intertidal exposure (14 – 29 November) during Richmond Pound November drawdown period

Baseline habitat exposure data for the Mogden scheme during the Richmond Pound November drawdown shows only spatially limited areas of marginal change within the reach between Teddington Weir and Richmond Sluice. Most change in these areas is around 20% or less, although there are some areas of greater change during both A82 and M96 scenarios, particularly around East Twickenham on the left bank. The areas of greatest time of exposure, up to ~50% for A82 and ~60% for M96, are seen around the narrower bifurcated left bank channel around Eel Pie Island (larger than during the normal level controlled operation of Richmond Pound, c.f. Figure A-32 and Figure A-33), as expected given the drawdown) and Isleworth Ait (downstream of the Richmond Sluice).

7. ESTUARINE SEDIMENT

Baseline sediment flux and transport for the Beckton water recycling scheme are presented in Figure 3-13 and Figure 3-14 and Table 3-4 and Table 3-5, while sediment concentrations for the Mogden water recycling scheme are presented in Figure 4-62 to Figure 4-64.

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