

Havant Thicket Reservoir Project

Environmental Statement Volume 4 - Appendices **Appendix A15 Water** **A15.2 Flood Risk Assessment Report**

Portsmouth Water Ltd.

01/09/20

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Foreword

by Bob Taylor, Chief Executive of Portsmouth Water

I feel extremely privileged to be the Chief Executive Officer at Portsmouth Water at this moment as we submit our planning application for the Havant Thicket Reservoir.

It has taken some years to reach this stage and as a Havant-based company we're delighted to be in a position to deliver this vital strategic water resource for the South East and create a unique leisure and wildlife environment for local communities.

Portsmouth Water has a long history since 1857 of supplying water to this area, and we've continually maintained a proud record of some of the highest service standards and the lowest water supply charges of all the water companies in England and Wales.

We believe in 'doing the right thing', a belief further embedded in our Community Pledge, launched earlier this year, and our Community Partnership, launched in the Spring which ensures society benefit is considered in all our decisions. Our aim is to make sure our objectives are in line with those of our communities.

Nowhere is this more important than in the development and delivery of a large infrastructure project like a reservoir, the first to be progressed in the South East since the 1970s.

This project has always had the potential to provide many additional benefits, over and above its core purpose of securing vital water supplies for our customers and those of Southern Water – ensuring we're ready for housing growth, climate change and protecting two world-famous rare chalk streams in Hampshire.

We believe we've captured that potential in this application, not least through the creation of a wetland for threatened bird species, extensive tree, hedgerow and wildflower planting, drought resilience and the educational, training and employment opportunities these also bring. Our plans also create a green leisure space with paths and play areas – offering priceless benefits for health and wellbeing – with a visitor centre aimed at bringing communities together.

And with our eye on future challenges, we've also considered the role we'll play in meeting our unique commitment as an industry to become carbon net zero by 2030.

We've listened carefully to the ideas and thoughts of our customers, communities, partners and regulators – held discussion groups, drop-in sessions, delivered talks, chatted on Facebook, spent time with young people and engaged tirelessly with our regulators, environmental organisations and the planning authorities. We believe this is part of the reason we have such strong support.

We think this application embodies not only our hopes and aspirations for the reservoir, but also those of the communities who will benefit and be part of its delivery, and we look forward to continuing to work in partnership with our neighbours over the months and years to come.



Bob Taylor, Chief Executive Officer, Portsmouth Water

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List of abbreviations

AP	Annual Probability
CH	Chainage
CIRIA	Construction Industry Research and Information Association
EHDC	East Hampshire District Council
EIA	Environmental Impact Assessment
FAS	Flood Alleviation Scheme
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
FRAP	Flood Risk Activity Permit
HBC	Havant Borough council
HCC	Hampshire County Council
HTR	Havant Thicket Reservoir
ICE	Institute Civil Engineers
ICM	Integrated Catchment Model
LLFA	Lead Local Flood Authority
LPA	Local Planning Authority
NGR	National Grid Reference
NPPF	National Planning Policy Framework
NRFA	National River Flow Archive
NTL	Nominal Tidal Limit
PMF	Probably Maximum Flood
PUSH	Partnership for Urban South Hampshire
ReFH	Revitalised Flood Hydrograph
RoFSW	Risk of Flooding from Surface Water
SFRA	Strategic Flood Risk Assessment
SuDS	Sustainable Drainage Systems
TWL	Top Water Level

1. Introduction

- 1.1. This Flood Risk Assessment (FRA) report supports the planning application for the Havant Thicket reservoir (HTR) and should be read in conjunction with the suite of documents that accompany the application for planning permission.
- 1.2. This report forms a quantitative assessment of the risks arising to the site and elsewhere as a result of both its location and development proposals.
- 1.3. The planning application will be submitted to Havant Borough Council (HBC), who are also reviewing on behalf of East Hampshire District Council (EHDC).

Background

- 1.4. The responsible organisation for the project is Portsmouth Water.
- 1.5. The aim of the HTR is to provide a new resource to supply drinking water in the water-stressed South East of England, while maximising the community and environmental benefits of a new raw water reservoir.
- 1.6. The Havant Thicket reservoir site and associated pipeline is allocated in the HBC and EHDC Local Plans to ensure that there will be adequate provision to meet the water supply needs during the plan period and to secure sub-regional supplies in the medium and longer term.
- 1.7. The reservoir will provide improved resilience for existing Portsmouth Water customers and support additional housing growth in the Portsmouth Water supply area. The development will create new habitats, including a wetland, woodland and parkland habitat and enhance existing habitats through woodland management.
- 1.8. The scheme includes the construction of an underground pipeline linking the reservoir with the existing pumping station at Bedhampton (approximately 4.5km long). The pipeline will also be used as an emergency draw down pipeline if the reservoir needs to be drained down in emergency conditions in accordance with Environment Agency safety guidance¹.

Site location

Havant Thicket reservoir Location

- 1.9. The site for the Havant Thicket reservoir is located to the north of the town of Havant in the county of Hampshire. The reservoir site is split along a north-south alignment between two local planning authorities: Havant Borough Council and East Hampshire District Council.
- 1.10. The reservoir site lies within a shallow valley located between the residential settlements of Leigh Park and Rowlands Castle and covers an area of approximately 160 hectares.
- 1.11. The current site is predominantly open grassland with some woodland, hedgerows and ditches.
- 1.12. A site location map is shown in Figure 1-1 below.

¹ Environment Agency, 2017. Guide to drawdown capacity for reservoir safety and emergency planning. [Online] Available at: <https://www.gov.uk/government/publications/guide-to-drawdown-capacity-for-reservoir-safety-and-emergency-planning>

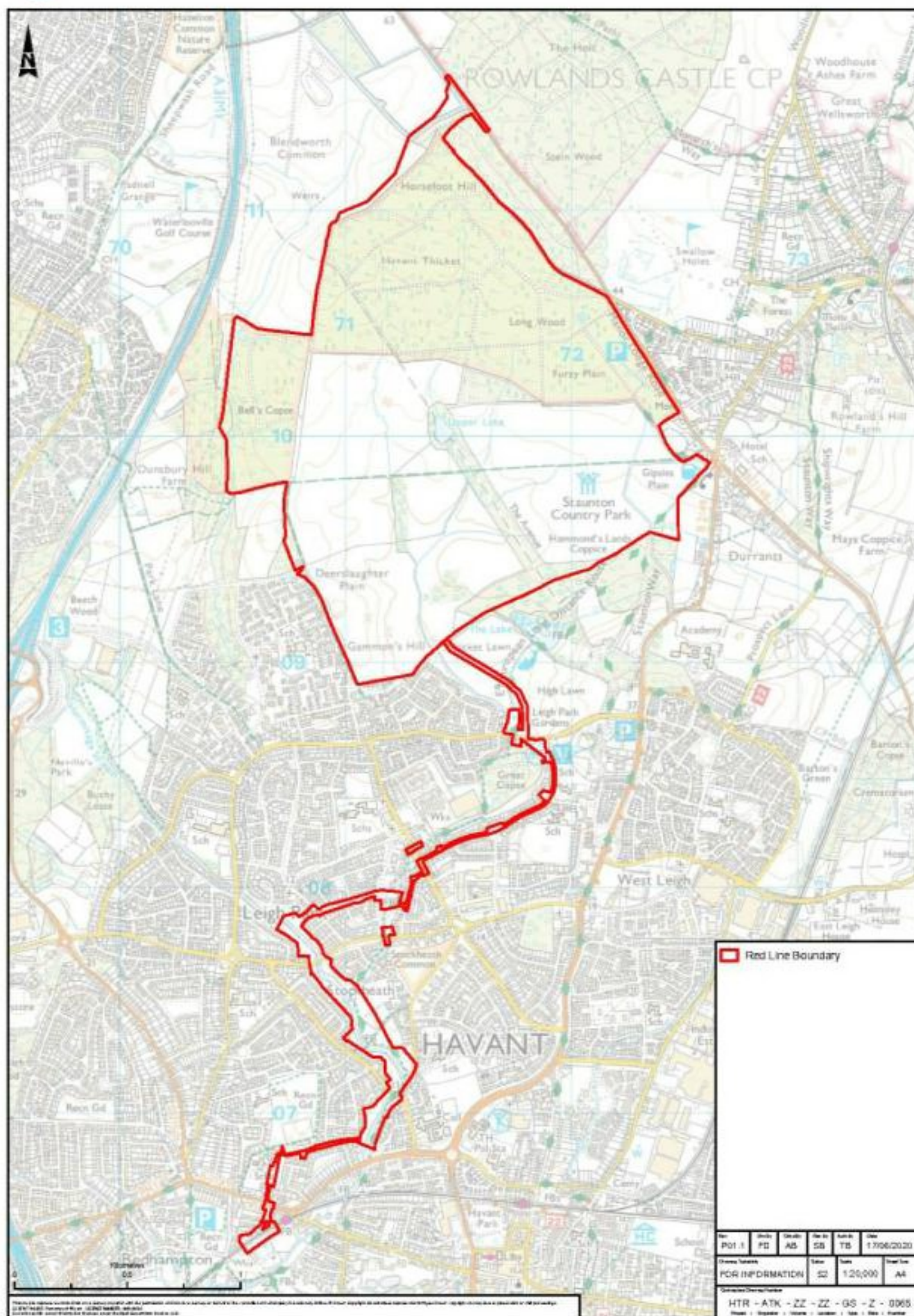


Figure 1-1 - Scheme location map

Site overview

- 1.13. Within the study area there are two Environment Agency defined main rivers, the Hermitage Stream and the Riders Lane stream, as shown on Figure 1-2.
- 1.14. Main rivers are usually larger rivers and streams, on which, the Environment Agency are responsible for carrying out maintenance and improvement works. Smaller rivers are referred to as Ordinary Watercourses, and the flood risk management of these watercourses is carried out by Lead Local Flood Authorities (LLFAs) and / or District Councils.
- 1.15. The Hermitage Stream has an approximate catchment size of 20 km², however 4.5 km² of that area is attributed to the Riders Lane Stream catchment. The upper catchment of the Riders Lane Stream includes five ordinary watercourses, which are ephemeral in nature and often run dry. The Riders Lane Stream also has a pond feature called 'Upper Lake' at National Grid Reference (NGR) SU 7142110030.
- 1.16. To the west of the reservoir site are the headwaters of Park Lane Stream (part of the Hermitage Stream catchment). The Park Lane Stream is classed as Ordinary Watercourse until it is culverted under the urban area, where it becomes a main river. The confluence of the Park Lane Stream with the Hermitage Stream is just upstream of the confluence of the Hermitage Stream with the Riders Lane Stream.
- 1.17. Within the reservoir footprint (considered the area within the top water level of the reservoir), towards the eastern edge, there are some small unnamed watercourses. These are mainly ditch-like still-water headwaters that drain eastwards into the River Lavant.
- 1.18. The Nominal Tidal Limit (NTL) is at NGR SU 70511 06237 on the Hermitage Stream, just south of Meyrick Road. The Hermitage Stream flows over a weir at this point, after which it becomes a transitional waterbody, Langstone Harbour.
- 1.19. Figure 1-2 also shows the location of two National River Flow Archive (NRFA) gauges; Hermitage Stream at Havant (NRFA no. 42017) and Lavant Stream at Leigh Park (NRFA no. 42025). These will be discussed later within the FRA.



Study Sections

- 1.20. For the purpose of this FRA and to ensure consistency with the Environmental Statement², the surface waterbodies have been divided up into study sections, an approach which will be used to describe the baseline and undertaking the assessment. The eight sections are set out in Figure 1-3 and Table 1-1 below.
- 1.21. It should be noted that the hydraulic model used to define the baseline flood risk in Section 3 only covers study sections 3-6.

Table 1-1 - Study Sections used for flood risk analysis

Study section	Description	Grid reference - upstream	Grid reference - downstream
1	Headwater Streams 1, 2 and 3 up to Reservoir footprint	HS1 - SU 71178 10789 HS2 - SU 71520 11000 HS3 - SU 71743 10398	HS1 - SU 71160 10368 HS2 - SU 71421 10346 HS3 - SU 71713 10253
2	Headwater Streams 1, 2, 3, 4, 5, Upper Lake and short section of Riders Lane Stream (RLS) all under Reservoir footprint	HS1 - SU 71160 10368 HS2 - SU 71421 10346 HS3 - SU 71713 10253 HS4 - SU 71828 09793 HS5 - SU 71621 09697 RLS - SU 71314 09436	HS1 - SU 71314 09436 HS2 - SU 71537 09816 HS3 - SU 71537 09816 HS4 - SU 71621 09697 HS5 - SU 71475 09570 RLS - SU 71319 09440
3	RLS between Reservoir footprint up to Hermitage Stream confluence. The Lake Stream (TLS) joining Riders Lane Stream from Leigh Lake.	RLS - SU 71319 09440 TLS - SU 71836 09095	RLS - SU 71007 07503 TLS - SU 71747 08787
4	Hermitage Stream – between A3(M) and confluence with Riders Lane Stream	SU 69631 09096	SU 71007 07503
5	Hermitage Stream – between confluence with Riders Lane Stream and Nominal Tidal Limit (NTL)	SU 71007 07503	SU 70511 06237
6	Langstone Harbour	SU 70511 06237	SU 68652 00138
7	Headwater Streams draining east towards River Lavant	SU 71947 09871	SU 72613 09497
8	Park Lane Stream	SU 70563 10531	SU 70513 09001

² Atkins reference HTR-ATK-ZZ-ZZ-RP-Z-0115

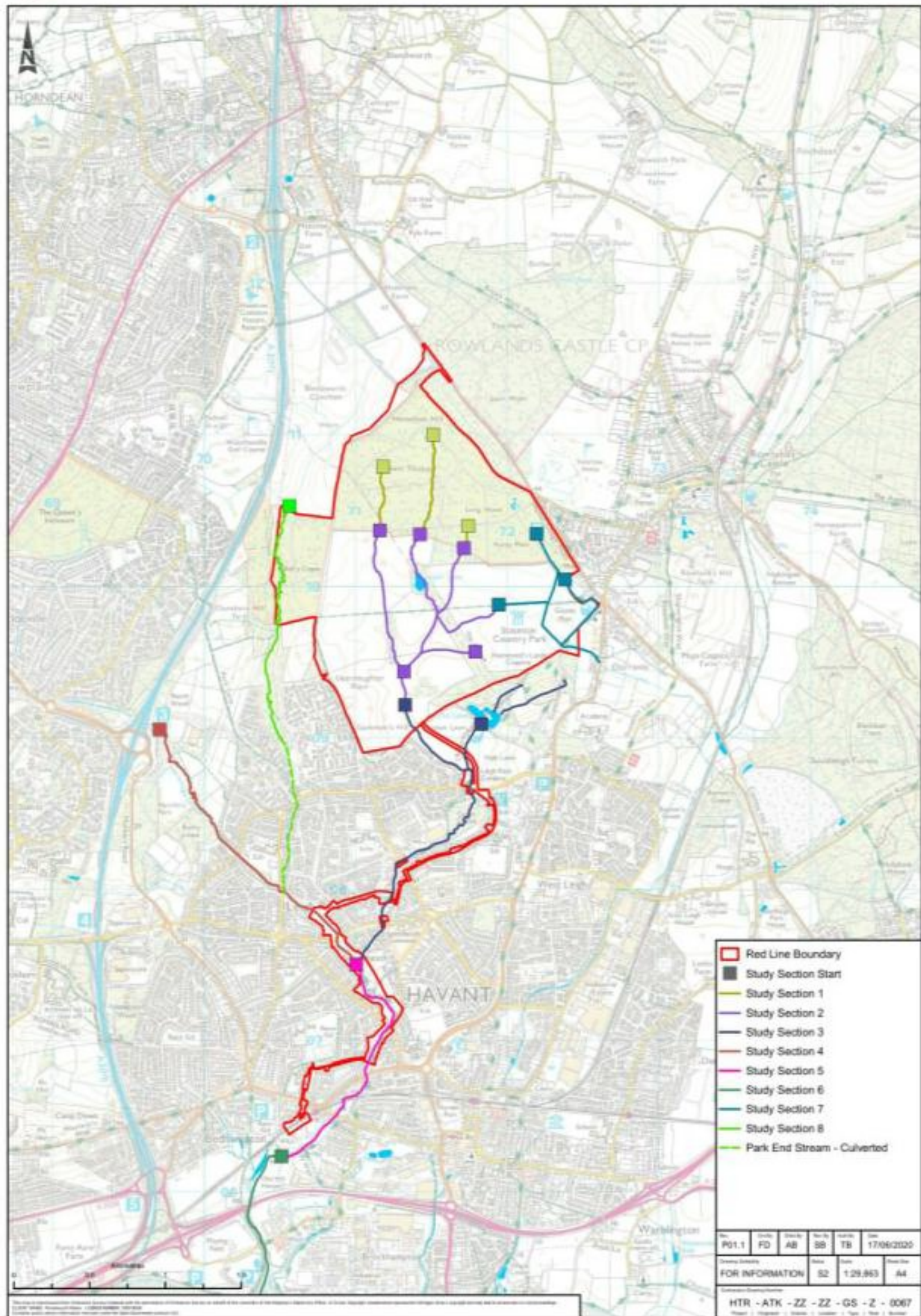


Figure 1-3 - Map showing Study Sections used for analysis

The scheme

- 1.22. The scheme is the construction of a reservoir, including a raised embankment and associated pipeline in Havant, Hampshire. The site will include vehicle and pedestrian accesses, an operational control house and visitor facilities.
- 1.23. The outline planning application comprises the following components:
- Embankment;
 - The buildings (visitor centre, control house and bird hides) and the land use layout for these buildings in the north-west corner, including the car park;
 - Wetland retaining structure and islands;
 - Viewing area; and,
 - The Bedhampton to Reservoir pipeline.
- 1.24. The development proposal will be discussed further in Section 4.

Previous studies

- 1.25. In 2019, a Flood Risk Management Inception Report was undertaken by Atkins³ for the Havant Thicket reservoir project. This aimed to summarise the work undertaken previously, gather information relating to flood risk management, assess its usefulness for the design development and make recommendations for further studies or investigations required.
- 1.26. As part of this study, existing assessments and studies were reviewed for relevance to flood risk. In summary, the assessment concluded the need for future work to include:
- Embankment failure assessment;
 - Impacts of emergency drawdown procedure;
 - A FRA in line with current flood risk planning policy, National Planning Policy Framework⁴;
 - An operational Flood Plan as required to satisfy The Reservoirs Act⁵;
 - An assessment of extreme events including the Probable Maximum Flood (PMF) and the 0.01% (1 in 10,000) Annual Probability (AP) event in line with current reservoir safety requirements⁵; and,
 - Consideration of the impacts of the reservoir, including emergency drawdown on the urban drainage network.
- 1.27. The most relevant documents with findings relevant to flood risk are presented in Table 1-2 below.

Table 1-2 - Previous studies

Report	Year	Finding
Environment Agency Flood Report Information	2008	Details the (now superseded) Havant InfoWorks RS flood model reports and results.
Public consultation	2008	Records that the general public were concerned about flood risk and climate change, specifically noting concerns at Whichers Gate area of Rowland's Castle and Leigh Park.
Panel of Experts Report	2008	Refers to the need for embankment failure to be assessed and for a Flood Plan.
Local Planning Authority (LPA) Environmental	2009	Identifies the need for a FRA in compliance with PPS25 (now superseded by the NPPF), and additionally considering:

³ Atkins reference HTWSR-ATK-XX-XX-RP-Z-0010

⁴ Ministry of Housing, Communities and Local Government, 2012. National Planning Policy Framework. [Online] Available at: <https://www.gov.uk/government/publications/national-planning-policy-framework-2> [Accessed 15 June 2020].

⁵ UK Government, 1975. Reservoirs Act. [Online] Available at: <http://www.legislation.gov.uk/ukpga/1975/23> [Accessed June 2020].

Report	Year	Finding
Impact Assessment (EIA) Scoping Opinion		- The testing and operation of the reservoir emergency drawdown system; - Embankment failure; - Applying the Sequential Test to demonstrate no suitably available alternatives at lower flood risk; and - Satisfying the Havant Borough Council (BC) Local Plan policies on development and flood risk. Subsequent documents indicate that a formal Sequential Test is not required, as discussed below.
Arup Environmental Handover Note	2009	Refers to the need for a FRA in compliance with PPS25 (now superseded by NPPF), and additionally considering: - The testing and operation of the reservoir emergency drawdown system; and - Embankment failure. Appendix B of the handover note provides further detail with areas at possible flood risk identified as Middle Park Way, Great Copse, Durrants Road, Redhill Road/College Close, Finchdean Road, Woodberry Lane, Dean Lane End, Ashcroft Lane.
Arup Engineering Handover Note	2009	Refers to the need for embankment failure to be assessed and for a Flood Plan.
Arup Planning Handover Note	2009	Refers to the need for a FRA with reference to PPS25 (now NPPF), CIRIA C624 and ICE 'Floods and Reservoir Safety'. This states that a formal Sequential or Exception Test would not need to be undertaken as HTR is completely within Flood Zone 1, although evidence should be provided of alternatives considered and rejected.
Portsmouth Water Feedback Reports	2009	These note that the 2009 FRA scoping is largely complete. Key remaining issues were as follows: - Need for summary text in relation to the Sequential Test. The Environment Agency at the time indicated that the Sequential Test would need to be applied, but that this could be covered by reference to the historic 1960s-2000s range of studies rather than formal application of the Test. - The Environment Agency also noted the need to refer to the Probable Maximum Flood (PMF) and the 0.01% (1 in 10,000) AP event and seepage/lining failure events relating to reservoir safety and design. - Portsmouth Water needed clarity on future testing time scale (6 or 12 months) of emergency drawdown system; why low flows (less than 7m³/s at Hermitage Stream) are noted as a concern; whether the emergency drawdown discharge can be contained by Riders Lane and/or Hermitage Streams.
Arup Alternatives and Needs Assessment, and Portsmouth Water Feedback	2011	Notes the Environment Agency's request for the greater use of the historic options studies since the 1960s to set out the reasoning for selecting the Havant Thicket site. The document also notes that the Environment Agency requested the Sequential Test information prior to the FRA being submitted.

Report	Year	Finding
Portsmouth Water HTR Risk Register and Vision	2018	Note the need for a FRA and to consider the risk of embankment failure.
Havant Borough Council Local Plan	2019	
Atkins Lavant Link Pipe and Extreme Tidal Levels Study	2019	Findings described within FRA.
Atkins Emergency Drawdown Study	2019	

Assessing flood risks

- 1.28. A FRA should consider all types of flooding to satisfy the following three key objectives:
- to assess flood risk to the proposed development and to demonstrate that any residual risks to the development and its users would be acceptable;
 - to assess the potential impact of the proposed development on flood risk elsewhere and to demonstrate that the development would not increase flood risk elsewhere; and,
 - to satisfy the requirements of the National Planning Policy Framework (NPPF).
- 1.29. Flood risk should be considered alongside other spatial planning issues such as transport, housing, economic growth, natural resources, regeneration, biodiversity, the historic environment and the management of other hazards.
- 1.30. Throughout this document, the risk of flooding will be described using Annual Probability (AP). For example, a 1% AP, has a 1 in 100 chance of occurring in any given year.

Scope

- 1.31. This report for the scheme provides a quantitative assessment of the risks arising to the site as a result of both its location and development proposals, which seeks to address the following questions:
- Is the site likely to be at risk of flooding from: a watercourse, the sea, an estuary, groundwater, surface water flow, an artificial drainage system, infrastructure failure?
 - Is the proposed development likely to increase flood risk elsewhere through changes to river flow regimes, increases in surface water runoff rates and volumes and / or artificial risk caused by infrastructure failure?
 - Given the above and the nature of the development, is continued promotion of a possible development at the site appropriate?
- 1.32. Through consultation with the relevant key stakeholders, the scope has been designed to include a thorough assessment of baseline flood risk, the potential impact on flood risk of the operation of the scheme, the potential impact on flood risk of the construction of the scheme, and an assessment of the residual risk posed, namely by failure of the reservoir embankment and by emergency rapid drawdown of the reservoir.
- 1.33. Formal consultation meetings have been held with:
- The Environment Agency;
 - Havant Borough Council (LPA); and
 - Hampshire County Council (LLFA).
- 1.34. The purpose of consultation has been to agree the scope of the Flood Risk Assessment, and agree specific approaches regarding:
- Assessment of the baseline flood risk – determining the required return periods for modelling;
 - Emergency Drawdown Strategy – agreeing the parameters by which it can be determined whether the Drawdown Strategy is considered acceptable; and
 - Assessment of the with-scheme impacts – determining the required return periods for modelling, and the approach to breach assessment.

2. Planning policy

Background

- 2.1. The NPPF⁶, published in 2012 and updated in 2019, is the overarching document in relation to development and flood risk and sets out the Government's policy on development relating to flood risk. The aim of the NPPF is to ensure that development is not at an unacceptable risk of flooding. Where development is unavoidable in areas at risk from flooding, the NPPF ensures that the development is safe without increasing flood risk elsewhere and where possible reducing flood risk overall.
- 2.2. In accordance with the NPPF, a site-specific FRA should be provided for all development in Flood Zones 2 and 3. In Flood Zone 1, a FRA should accompany all proposals involving sites of 1 hectare or more; land which has been identified by the Environment Agency as having critical drainage problems; land identified in a Strategic Flood Risk Assessment (SFRA) as being at increased flood risk in the future; or land that may be subject to other sources of flooding, where its development would introduce a more vulnerable use.

Flood zones

- 2.3. The Environment Agency's Flood Map is divided into three separate Flood Zones. These Flood Zones are used by NPPF in determining the appropriateness of proposed developments when considering flood risk through the application of the Sequential Test. They represent the probability of flooding without flood defences in place.
- 2.4. The Flood Zones are defined in Table 2-1.

Table 2-1 - Environment Agency Flood Zones

Flood Zone	Definition
Flood Zone 1: Low Probability	Land where the annual chance of flooding is lower than 0.1% for either fluvial or sea flooding.
Flood Zone 2: Medium Probability	Land where the annual chance of flooding is between 0.1 and 1.0% for fluvial flooding. Or, land where the annual chance of flooding is between 0.1 and 0.5% for flooding from the sea.
Flood Zone 3a: High Probability	Land where the annual chance of flooding is 1.0% or greater for fluvial flooding. Or, land where the annual chance of flooding is 0.5% or greater for flooding from the sea
Flood Zone 3b: Functional Floodplain	Land where water has to flow or be stored in times of flooding. Local planning authorities identify in their Strategic Flood Risk Assessments areas of functional floodplain and its boundaries accordingly, in agreement with the Environment Agency.

- 2.5. The Environment Agency's Flood Map also defines Areas Benefitting from Defences within Flood Zone 3, however this category is not expressly determined within NPPF or the Sequential Test process.
- 2.6. The assessment of Flood Zone for the site is discussed in Section 3.

⁶ Ministry of Housing, Communities and Local Government, 2012. National Planning Policy Framework. [Online] Available at: <https://www.gov.uk/government/publications/national-planning-policy-framework-2> [Accessed 15 June 2020].

Vulnerability classification

- 2.7. The NPPF assigns vulnerability type to land use in terms of vulnerability to the impact of flooding. The vulnerability types range from water-compatible to essential infrastructure. The definitions for vulnerability type, and Flood Zone compatibility, is available on the gov.uk website.
- 2.8. More vulnerable development, for example residential, would be unsuitable for construction in areas at risk from flooding, however water-compatible development types such as water-based recreation might be considered acceptable (if the development does not increase flood risk elsewhere).
- 2.9. NPPF provides guidance on assessing the vulnerability of land uses in relation to flood risk and classifies new developments into one of five categories. These are described on the gov.uk website:
- Essential Infrastructure;
 - Highly Vulnerable;
 - More Vulnerable;
 - Less Vulnerable; and,
 - Water Compatible.
- 2.10. The land use vulnerability changes across the site, due to different infrastructure types being constructed. The main developments are classed as the following:
- Reservoir embankment = Essential Infrastructure
 - Pipeline = Essential Infrastructure
 - Control house = Essential Infrastructure
 - Visitor centre = Less vulnerable
 - Educational Hides = Less vulnerable
- 2.11. A development is always classified based on the most vulnerable elements and for this reason the Havant Thicket reservoir is therefore classed as Essential Infrastructure.

Compatibility

- 2.12. Table 2-2 sets out the NPPF flood risk vulnerability and flood zone compatibility assessment, as taken from Table 3 of the NPPF Planning Practice Guidance.
- 2.13. The table indicates which development types are appropriate within each Flood Zone. Consideration of the flood risk vulnerability and flood zone compatibility is given in Table 2-2.

Table 2-2 - Flood Risk Vulnerability Classification

Flood Zones	Flood Risk Vulnerability Classification				
	Essential infrastructure	Highly vulnerable	More vulnerable	Less vulnerable	Water compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test required	✓	✓	✓
Zone 3a †	Exception Test required†	X	Exception Test required	✓	✓
Zone 3b *	Exception Test required*	X	X	X	✓*

† In Flood Zone 3a essential infrastructure should be designed and constructed to remain operational and safe in times of flood.

* In Flood Zone 3b (functional floodplain) essential infrastructure that has to be there and has passed the Exception Test, and water-compatible uses, should be designed and constructed to:

- Remain operational and safe for users in times of flood;
- Result in no net loss of floodplain storage; and,
- Not impede water flows and not increase flood risk elsewhere.

Flood Risk Assessment

- 2.14. In accordance with the NPPF, in order to reduce the risk of flooding, all developments greater than 1 hectare in size, or sites within Flood Zones 2 or 3, must submit a FRA or comply with the Environment Agency's Flood Risk Standing Advice where relevant.
- 2.15. The red line boundary of the reservoir site is approximately 395 hectares, and the red line boundary of the Bedhampton to Havant Pipeline is approximately 21 hectares. This, and the fact that there are areas of Flood Zones 2 and 3 within the red-line boundary, means that a FRA is required for this development.
- 2.16. If required, dependent on the proposed development type and vulnerability classification, the FRA should demonstrate that the site design passes the Sequential Test and/or Exception Test.

Sequential Test

- 2.17. The reservoir development is allocated in both HBC's and EHDC's Local Plans^{7, 8}, with alternative locations having previously been considered and Havant Thicket being determined as the most appropriate and safe location for the new infrastructure. It was confirmed in a stakeholder meeting⁹ with HBC, Hampshire County Council (HCC) and Portsmouth Water that the Sequential Test is considered satisfied.

Exception Test

- 2.18. It is noted in the Havant SFRA Local Plan Sites document¹⁰, that the Havant Thicket reservoir does not require an Exception Test owing to the nature of the development.

Other policy/legislation

- 2.19. In Table 2-3 below is a summary of National and Local Policies that have been considered as part of the scheme development and through the assessment as part of this FRA.

Table 2-3 - National and Local Policies

Guidance	Region	Description
Floods Directive (2007/60/EC)	UK & Europe	The aim of this Directive is to reduce and manage the risks that floods pose to human health, the environment, cultural heritage and economic activity. It sets the strategic level for flood risk that any development will need to comply with.
Flood and Water Management Act ¹¹	National	The key areas covered by this Act are: Roles and responsibilities for flood and coastal erosion risk management; and, improving reservoir safety.
National Planning Policy Framework (NPPF)	National	The NPPF (as discussed in previously in this section) sets strict tests to protect people and property from flooding which all local planning authorities are expected to follow. It forms the basis of assessment of flood risk for projects.
Reservoirs Act	National	In England, reservoirs that are capable of holding 25,000m ³ or more of water are regulated under the Reservoirs Act 1975. The undertakers (owners or operators) of these reservoirs are required to register them with the Enforcement Authority (the Environment Agency) following which a designation of 'high-risk' or 'not high-risk' will be given. Those designated high-risk will be subject to inspection and supervision by reservoir

⁷ Havant Borough Council, 2019. Havant Borough Local Plan 2036 Pre-Submission Draft.

⁸ East Hampshire District Council, 2019. Draft Local Plan 2017-2036.

⁹ Meeting held on Microsoft Teams on 26/03/2020.

¹⁰ Havant Borough Council, 2019. Strategic Flood Risk Assessment (Local Plan Sites).

¹¹ UK Government, 2010. Flood and Water Management Act 2010. [Online] Available at: <http://www.legislation.gov.uk/ukpga/2010/29/contents> [Accessed June 2020].

Guidance	Region	Description
		engineers. Havant Thicket Reservoir will be designated as 'high-risk' and therefore will fall under the requirements of the Reservoirs Act (1975).
Environment Agency Guide to risk assessment for reservoir safety management ¹²	National	This guide provides a tool for the management of reservoir safety, using a proven and robust framework, supported by science, with which to manage the risk of an uncontrolled release of the contents of a reservoir.
Environment Agency Guide to drawdown capacity for reservoir safety and emergency planning	National	This report outlines the preferred drawdown capacity for reservoirs in the UK. This guidance is intended for reservoir inspection engineers, required by UK reservoir legislation to review the drawdown capacity of reservoirs.
Environmental Permitting Regulations (Defra / EA, 2016)	National	Permitting guidelines for activities that are regulated by the Environment Agency. An Environmental Permit is required if you do an activity that could: • pollute the air, water or land • increase flood risk • adversely affect land drainage
Havant Borough Local Plan 2036 (Pre-Submission)	Local	This has not yet been adopted as is still undergoing consultation. However, it sets out the vision and a framework for the future development, growth and prosperity of Havant Borough. It is influenced by the National Planning Policy Framework (NPPF). The Local Plan outlines Havant Thicket Reservoir as a <i>Key Project (KP9)</i> . However, it does set out criteria for planning permission to be granted. For example, sustainable, cost-effective, providing environmental and community benefits, conserving natural, rural and historic character, assessing impacts on watercourses etc. Policies of note within this Local Plan include: • Policy E19: Managing flood risk in a new development • Policy E20: Drainage infrastructure in new development.
Havant Borough Local Plan (Core Strategy) ¹³	Local	The core strategy forms part of the Havant Borough Local Development Framework (HBLDF) by outlining the broad detail and direction of development within the borough. Havant Thicket Reservoir is an allocated strategic site within Policy CS18. It also identified criteria for planning permission to be granted. For example, sustainable, cost-effective, deliver environmental and community benefits, assess risk of flooding within storm events etc. Another policy of note within this Local Plan is Policy CS15: Flood and Coastal Erosion Risk.
East Hampshire District Council	Local	A new Local Plan (2017-2036) is currently undergoing consultation and thereby still in draft form.

¹² Environment Agency, 2013. Guide to risk assessment for reservoir safety management. [Online] [Accessed June 2020].

¹³ Hampshire County Council, 2011. Hampshire County Council Preliminary Flood Risk Assessment.

Guidance	Region	Description
Local Plan 2017-2036 (Draft)		This allocates Havant Thicket Reservoir site (Policy S31) and identifies it as key infrastructure. It also identifies criteria for planning permission to be granted. For example, sustainable, cost-effective, providing environmental and community benefits, assessing risk of flooding during storm events within a flood risk assessment etc. Policies of note within this Local Plan include: • Policy S25: Managing Flood Risk • Policy S24: Planning for climate change
East Hampshire District Council Local Plan: Joint Core Strategy ¹⁴	Local	The Joint Core Strategy purpose is to provide a policy framework for new development within the District Council area as well as allocating development areas. Policies of note within this Local Plan include: • CP25: Flood Risk • CP26: Water resources/ Water quality – this safeguards the site identified for the Havant Thicket Reservoir from development.
Hampshire County Council Local Flood Risk Management Strategy (LFRMS) ¹⁵	Local	The Flood and Water Management Act 2010 requires LLFAs (Hampshire County Council) to develop a strategy for local flood risk management as well as being responsible for flood risk management relating to groundwater, surface water or ordinary watercourses. This document aids individuals, communities, businesses and authorities to understand the flood risk within Hampshire and how to manage it. The main focus is on local sources of flooding for example, surface water or ordinary watercourses (e.g. streams and ditches).
Hampshire County Council Preliminary Flood Risk Assessment (PFRA) ¹⁶	Local	The PFRA provides an overview of flood risk within Hampshire. This includes flood risk from multiple sources, for example, surface water, groundwater, main rivers, ordinary watercourses, tidal flooding and reservoirs. This informs Local Flood Risk Management Strategies and Surface Water Management Plans. The report also includes the impact of climate change on future flooding and the impact on long term developments.
Partnership for Urban South Hampshire (PUSH) Strategic Flood Risk Assessment (SFRA) ¹⁷	Local	A SFRA is an assessment which aims to inform Local Planning Authorities (E.g. Havant Borough Council) on flood risk within the area in order for site allocations and strategic planning to be undertaken.
East Hampshire District Council Strategic Flood Risk Assessment (SFRA) ¹⁸	Local	This assessment assesses the flood risk within East Hampshire in order to inform the Local Plan, and site-specific flood risk assessments.

¹⁴ East Hampshire District Council, 2014. East Hampshire District Local Plan: Joint Core Strategy.

¹⁵ Hampshire County Council, 2013. Hampshire Local Flood Risk Management Strategy.

¹⁶ Hampshire County Council, 2011. Hampshire County Council Preliminary Flood Risk Assessment.

¹⁷ PUSH, 2016. Strategic Flood Risk Assessment 2016 Update.

¹⁸ EHDC, 2018. East Hampshire Level 1 SFRA.

3. Baseline flood risk

- 3.1. The NPPF states that all types of flooding should be considered in the development framework. The extent to which these should be considered will vary and depends on whether they are considered as significant at the spatial planning scale, and in setting constraints on development in certain areas.
- 3.2. This section of the report scopes the risk of flooding to the proposed development and identifies those sources of flooding that may require further consideration.

Data

- 3.3. To inform the baseline assessment of flood risk, the following available information has been used (accessed March 2020):
 - Environment Agency Flood Zones¹⁹
 - Environment Agency Surface Water Mapping²⁰
 - Environment Agency Statutory main river mapping²¹
 - OS detailed river network²²
 - InfoWorks ICM (Integrated Catchment Model) Hydraulic River Model
 - EA Borehole Groundwater Level data.

Historical flooding

- 3.4. Historically, within the catchment there have been a series of large rainfall events recorded in: Winter 2002-2003; Autumn/Winter 2000; Summer 1997; Autumn 1995; Autumn 1993; Winter 1966; and 1955.
- 3.5. In 1966 a severe flood event with an estimated peak flow of 22 m³/s caused considerable local flooding and prompted the Hermitage Stream Flood Alleviation Scheme (FAS) of 1971-75, in which the Hermitage Stream was straightened, concreted and the bed was lowered, to accommodate a flow of 22-24 m³/s.
- 3.6. In September 2000, 20.7 m³/s was recorded at the Hermitage Stream gauging station (shown on Figure 1-2) which was estimated to have a 1.5% (1 in 65) AP. The event caused damage throughout the catchment compounded by blockages caused by fly-tipping and debris within the channel reach. At the Lavant Stream gauge (also shown on Figure 1-2) the December 2000 storm was estimated to have a 1.2% (1 in 83) AP, however this was estimated as a 33% (1 in 3) AP storm for the Hermitage Stream. This shows that the two catchments flood in different storm event types. A flood occurring on one of the two streams will not usually result in a flood of similar magnitude on the other stream. This is caused by the varying catchment response times caused in part by characteristics such as the surface type, geology and topography.
- 3.7. In general, the catchment has experienced relatively high annual probabilities (low return periods) for the observed storm events, with events other than the September 2000 event estimated at higher the 8% (lower than 1 in 12) AP. Consequently, there has not been significant flooding experienced within the Hermitage Stream Catchment since the 1970s. The large capacity of the watercourse as a result of the Hermitage FAS, also contributes to the reduced frequency of flooding in the catchment since the early 1970s.

Fluvial and tidal flooding

- 3.8. Flooding from rivers (fluvial flooding) occurs following exceedance of the flow capacity of river channels, leading to overtopping of the riverbanks and inundation of the surrounding land.

¹⁹ Environment Agency, 2020. Flood Map for Planning (Rivers and Sea) - Flood Zones 2 & 3.

²⁰ Environment Agency, 2020. Risk of Flooding from Surface Water.

²¹ Environment Agency, 2020. Statutory Main River Map.

²² Ordnance Survey, 2020. OS MasterMap Water Network Layer. [Online]

Available at: <https://www.ordnancesurvey.co.uk/business-government/products/mastermap-water> [Accessed March 2020].

Inundation by high tides, storm surges and waves along coastal regions is described as coastal flooding. The propagation of high tides and storm surges up estuarine channels can lead to overtopping of the riverbanks and inundation of the surrounding land. This is referred to as tidal flooding.

- 3.9. As discussed in Section 2, the Environment Agency publish mapping showing Flood Zones. It is important to note that this mapping does not cover watercourses designated by DEFRA as Ordinary Watercourses, but only Environment Agency defined main rivers. Main rivers are usually larger streams or rivers but can also include smaller watercourses of local significance. Main rivers are shown on the statutory main river maps held by the Environment Agency and they have permissive powers to carry out maintenance works and / or improvements on these rivers. As such, a site not indicated at risk of flooding on the Environment Agency Flood Zones may well be at risk of flooding from an Ordinary Watercourse.
- 3.10. As shown on Figure 3-1, for the Riders Lane Stream, the fluvial flooding occurs mostly in bank, or within areas of open land. There is limited property flooding despite flowing through a highly urbanised area. This same trend is seen downstream of the confluence with the Hermitage Stream.
- 3.11. The large area of Flood Zone 3 upstream on the Hermitage Stream indicates an area of property flooding, however, this is likely to be related to an historic flood event prior to the 1970s Hermitage FAS. The abrupt change in flood risk along a straight line in this location, suggests some uncertainty in the reliability of the Flood Zones for the Hermitage Stream.
- 3.12. The risk of fluvial flooding for the Hermitage Stream has therefore been refined through the development of a detailed hydraulic model for this FRA and the extent of flooding predicted by this modelling is shown in Figure 3-3. This model has been reviewed by the Environment Agency and deemed acceptable for defining the baseline flood risk for this FRA.
- 3.13. As discussed in section 1, the NTL in the current climate is located at a series of weirs situated just south of Meyrick Road, at NGR SU 70511 06237 (As shown in Figure 3-1). Upstream of this point the flood risk is fluvial, however below this point the Flood Zones also include tidal influences. There is no significant tidal flood risk, with flows staying largely in bank, or within areas of open land. There is an increased flood risk at the NTL location. The cascade of weirs has a top crest level of 4.3 mAOD and this current level restricts a 0.1% (1 in 1000) AP event tide, however once current climate change predictions²³ are taken into account, the predicted tide levels exceed the crest of the top weir. Under changing climate predictions therefore, it is expected that the tidal limit will migrate upstream to between the bridges located at Meyrick Road and West Street.

²³ Environment Agency, 2016. Flood risk assessments: climate change allowances. [Online] Available at: <https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances> [Accessed 15 June 2020].

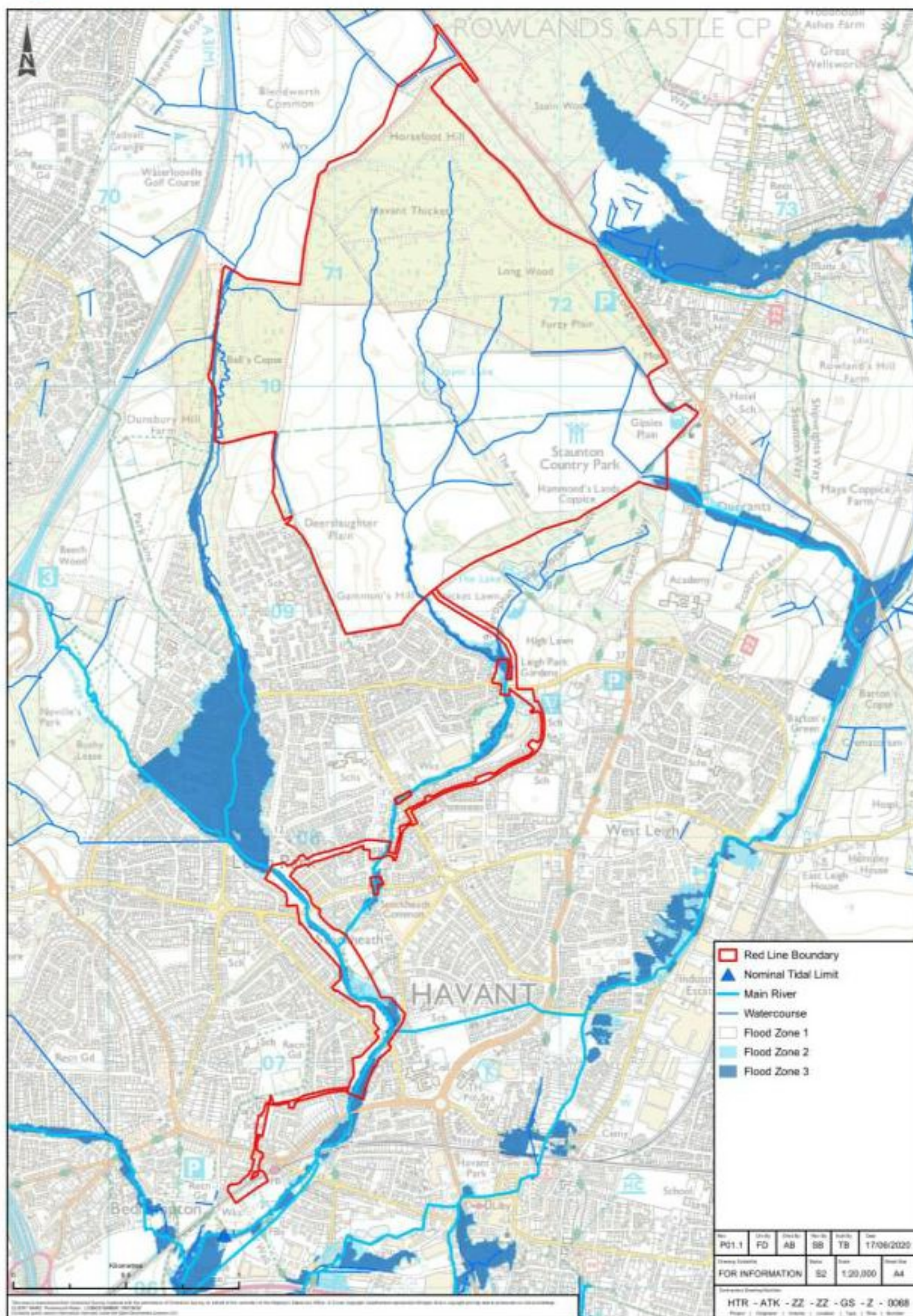


Figure 3-1 - Flood risk from main river and the sea (Flood Zones)

Hydraulic modelling

- 3.14. To refine the baseline understanding of flood risk in the catchment, an InfoWorks ICM (v9.0.2) has been built, allowing for an updated assessment of fluvial and tidal flood risk and to quantify the impacts on flood risk of the reservoir and associated infrastructure.
- 3.15. An InfoWorks CS/RS model, created in 2008 for the Havant Flood Mapping Study²⁴ was taken and updated to InfoWorks ICM for use in this FRA and EIA. The updated model was reviewed and approved by the Environment Agency in 2019. The model upstream boundaries are located at the bridge on Middle Park Way over the Hermitage Stream and at the site of the proposed reservoir on the Riders Lane Stream. The downstream boundary of the model is the point of outflow of the Hermitage Stream into Langstone Harbour. The model covers the extent of the Hermitage Stream and Riders Lane Stream, which are potentially affected by the development of the proposed reservoir.
- 3.16. An overview of the model is shown in Figure 3-2 .

²⁴ Atkins, 2008. Havant Flood Mapping Study.

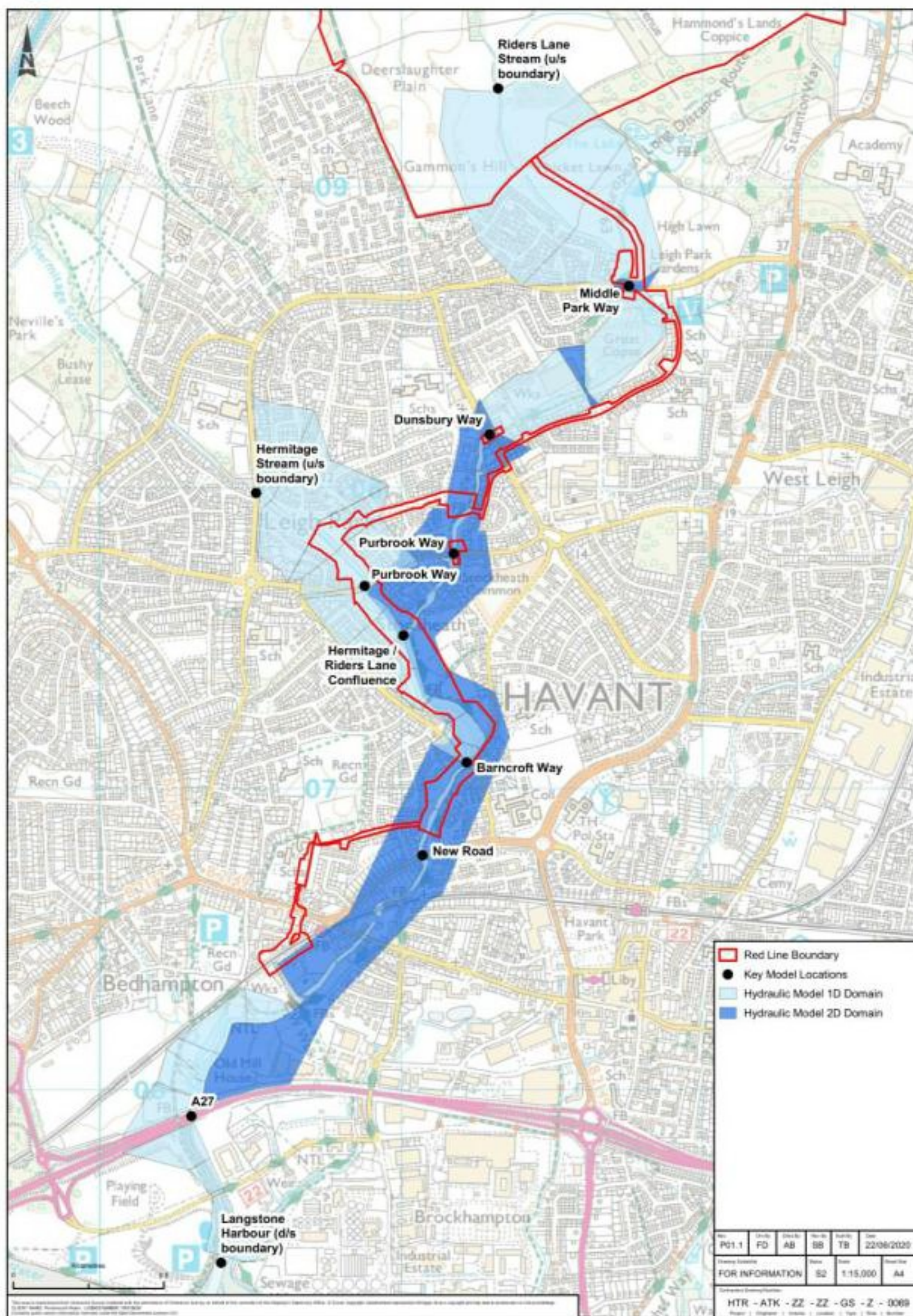


Figure 3-2 - Model overview

- 3.17. As described in Section 1, and shown on Figure 1-2, the catchment is gauged at the downstream extent of the study reach (National River Flow Archive gauge number 42017, Hermitage Stream at Havant).
- 3.18. To define the inflows for the hydraulic model, a hydrological assessment was undertaken, using the gauge data and the latest hydrometric methods and software. A ReFH2.3 (Revitalised Flood Hydrograph) hydrological model at the gauge was calibrated using four high flow events, resulting in a significant shortening of the time to peak from the software suggested default. Design flows at the gauge were estimated, ranging from the 50% (1 in 2) AP event to the 0.1% (1 in 1000) AP event. In all cases, the design ReFH flows routed through the hydraulic model give very good agreement with those estimated from the FEH (Flood Estimation Handbook) statistical method and with the calibration event of September 2000.
- 3.19. Table 3-1 below shows the estimated peak flows from the ReFH2.3 method at the gauge location. For full details on the hydrological derivation, the reader is referred to the Hydrology Proforma (Appendix C). Consultation with the Environment Agency occurred in January 2020 to confirm the required return periods for baseline assessment, which are also shown in Table 3-1.

Table 3-1 - Estimated peak flows at the 'Hermitage Stream at Havant' Gauge

Return Period	Peak Flow (m³/s)		
	Total	Hermitage Stream	Riders Lane Stream
50% (1 in 2) AP	10.7	7.8	2.9
20% (1 in 5) AP	13.9	10.1	3.8
10% (1 in 10) AP	16.2	11.8	4.4
3.33% (1 in 30) AP	19.9	14.5	5.5
1% (1 in 100) AP	24.6	17.8	6.8
0.1% (1 in 1000) AP	40.4	29.1	11.3

- 3.20. Climate change scenarios were created following the latest Environment Agency guidance²⁵. The uplift values applied to the fluvial inflows were taken from the South East River Basin guidance for the 2080s epoch, and two scenarios were used as shown in Table 3-2, as agreed in consultation with the EA.

Table 3-2 - Environment Agency climate change guidance for fluvial flows

South East River Basin	Total potential change anticipated for the '2080s' (2070 to 2115)
Upper end	105%
Central	35%

- 3.21. Tide curves for the 50% (1 in 2) AP, 0.5 % (1 in 200) AP and 0.1% (1 in 1000) AP events were derived for both the present day (2020) and one future epoch (2125). The design tide curves have been generated in accordance with guidance provided in Coastal Flood Boundary Conditions for the UK: update 2018 (Environment Agency, 2018) . Table 3-3 shows the peak tidal levels for each of the return periods and climate change scenarios. Further information is provided in Appendix D Tidal boundary generation.

Table 3-3 - Peak tidal levels

Epoch	Peak tidal levels (mAOD)		
	50% (1 in 2) AP	0.5% (1 in 200) AP	0.1% (1 in 1000) AP

²⁵ Environment Agency, 2016. Flood risk assessments: climate change allowances. [Online] Available at: <https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances> [Accessed 15 June 2020].

2020	2.72	3.19	3.35
2125 (Higher Central)	3.81	4.28	4.44
2125 (Upper)	4.18	4.65	4.81

- 3.22. For more information on the hydraulic modelling method and validation, the reader is referred to Appendix B Hydraulic modelling report.

Model Outputs

- 3.23. The baseline flood risk, defined using the ICM results is shown in Figure 3-3 for the 1% AP and the 1% AP with a central estimate for climate change.
- 3.24. The results from the ICM have been compared with the original Environment Agency Flood Zone map (Figure 3-1). A fluvial risk of 0.1% (1 in 1000) AP has been used to compare to Environment Agency Flood Zone 2 (Figure 3-4), and 1% (1 in 100) AP used to compare results to Environment Agency Flood Zone 3 (Figure 3-5).
- 3.25. Figure 3-4 and Figure 3-5 show that the detailed ICM provides a more refined extent of flood risk within the catchment, when compared to the Environment Agency Flood Zones, which removes the anomaly in the floodplain extent at the northern end of Leigh Park on the Hermitage Stream. The flood extent produced by the ICM is considered a more robust estimate of flood risk within the catchment as it is calibrated to recorded high flows at the catchment gauge and derived from an updated hydraulic model. Notable refinements to the outline, in addition to Leigh Park include the area downstream of the NTL on the Hermitage Stream. The predicted flood extent along the Riders Lane Stream largely remains the same at the Flood Zones.



Figure 3-3 - Existing Flood Risk (Baseline 1 in 100 and 1 in 100 +cc)

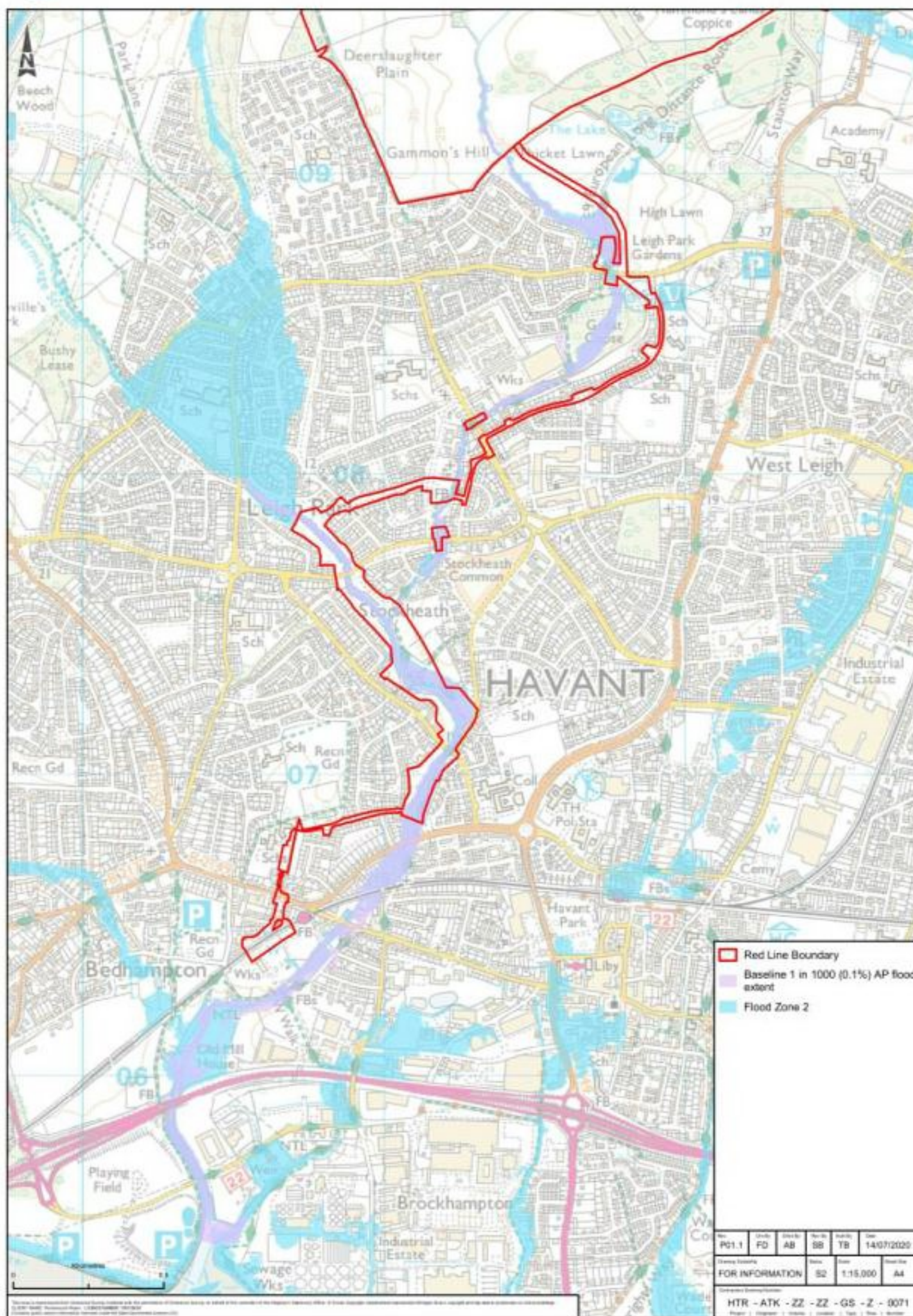


Figure 3-4 - Comparison of Environment Agency Flood Zone 2 and modelled 0.1% (1 in 1000) AP

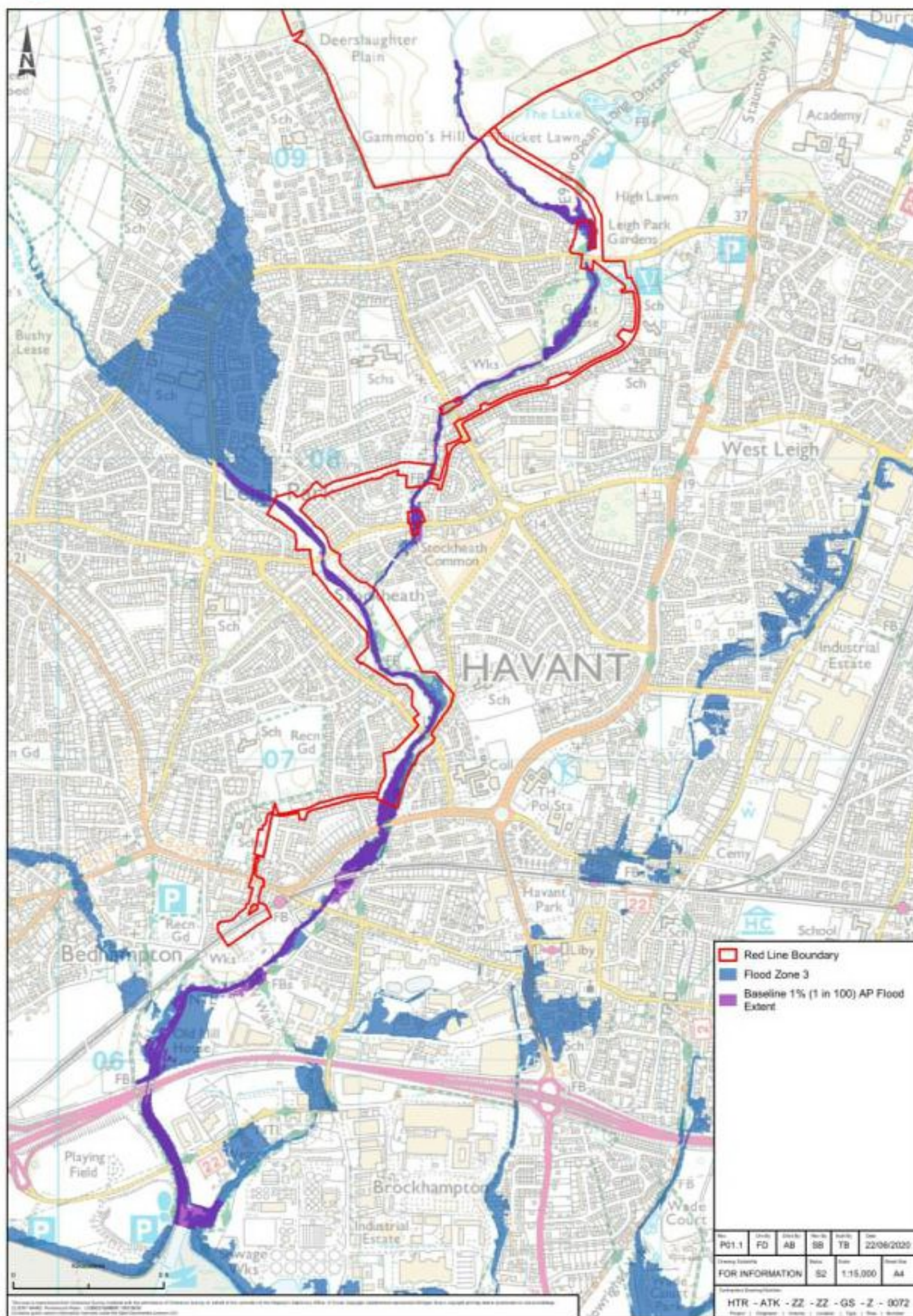


Figure 3-5 - Comparison of Environment Agency Flood Zone 3 and modelled 1% (1 in 100) AP

- 3.26. As discussed above in Section 1, the waterbodies have been divided into study sections. Eight sections have been identified and these have been used to present baseline fluvial flood risk data.
- 3.27. The model outputs in Table 3-4 will be used as a baseline for comparison with the 'with scheme' scenario results.

Table 3-4 - Model outputs used as a baseline (climate change values used are the central estimate)

Study Section	Annual Probability	u/s peak level (m AOD)	d/s peak level (m AOD)	u/s peak flow (m³/s)	d/s peak flow (m³/s)	Floodplain extent (km²)
1	No model data available, assessment will be qualitative					
2	No model data available, assessment will be qualitative					
3	3.33% (1 in 30)	20.80	7.86	3.09	5.45	0.02
	1% (1 in 100)	20.86	8.06	3.84	6.62	0.03
	1% (1 in 100) + CC	20.94	8.37	5.18	8.23	0.04
	0.1% (1 in 1000)	21.01	8.52	6.38	8.25	0.05
4	3.33% (1 in 30)	9.99	7.86	11.77	13.52	0.01
	1% (1 in 100)	10.10	8.06	14.53	16.68	0.01
	1% (1 in 100) + CC	10.28	8.37	19.62	22.51	0.01
	0.1% (1 in 1000)	10.42	8.52	23.90	27.31	0.01
5	3.33% (1 in 30)	7.86	2.95	18.93	18.15	0.03
	1% (1 in 100)	8.06	3.04	23.26	22.70	0.04
	1% (1 in 100) + CC	8.37	3.22	30.74	31.45*	0.06
	0.1% (1 in 1000)	8.51	3.31	37.02	36.52	0.08
6	3.33% (1 in 30)	2.95	2.72	18.15	25.83	0.04
	1% (1 in 100)	3.04	2.72	22.70	29.91	0.04
	1% (1 in 100) + CC	3.22	2.72	31.45	39.13	0.04
	0.1% (1 in 1000)	3.31	2.72	36.52	44.28	0.04
7	No model data available, assessment will be qualitative					
8	No model data available, assessment will be qualitative					

*This follows a different pattern as seen within other AP events for this section. This is due to an increased flow from the Lavant Link Pipe within the climate change scenario.

Surface water flooding

- 3.28. Surface water flooding (also referred to as pluvial flooding) can be caused by overland flow / runoff, and includes water flowing over the ground that has not reached a natural or artificial drainage channel. This can occur when intense rainfall exceeds the infiltration capacity of the ground due to;
- Rainfall which has fallen on ground so highly saturated that it cannot accept any more water,
 - Rainfall which has fallen on ground so hard and dry that runs straight off, and / or
 - Rainfall which has fallen in a built environment which exceeds the urban drainage capacity, such that ponding and overland flow occurs. This can also be referred to as surface water sewer flooding.
- 3.29. Surface water flooding can be caused by water originating from either on-site or from adjacent sites.
- 3.30. The Risk of Flooding from Surface Water (RoFSW) maps, licensed by the Environment Agency (Environment Agency, 2020), indicate the estimate of risk of flooding from surface water (Figure 3-6).
- 3.31. When comparing Figure 3-1/Figure 3-3 and Figure 3-6, surface water flood risk generally follows the same flow path and trend as the Environment Agency Flood Zones, remaining in bank, or within areas of open space. There are, however, localised flow routes through the urbanised roads in the area and localised areas of surface water ponding caused by variations in topography.
- 3.32. Within the study area there is varying probability of surface water flooding with some areas at High probability, equivalent to an AP of flooding each year of greater than 3.3%, and some areas of Low probability correlating to an AP between 0.1% and 1%.
- 3.33. The proposed reservoir will not impact on the volume or rate of surface water flows throughout the majority of the study area (1, 3-8), so quantitative baseline data for each of these study sections has not been derived. The changing flows in the river channel has the potential to impact on the discharge of the surface water drainage network and this will be described qualitatively in Sections 5 and 6.
- 3.34. There will be increased surface water flows from the infrastructure that supports the new reservoir, i.e. the visitor centre and car park, located in study section 2. A separate drainage assessment to support the design will be undertaken at detailed design stage, and will follow the principles of sustainable drainage and will not increase peak flows and volumes into the system.

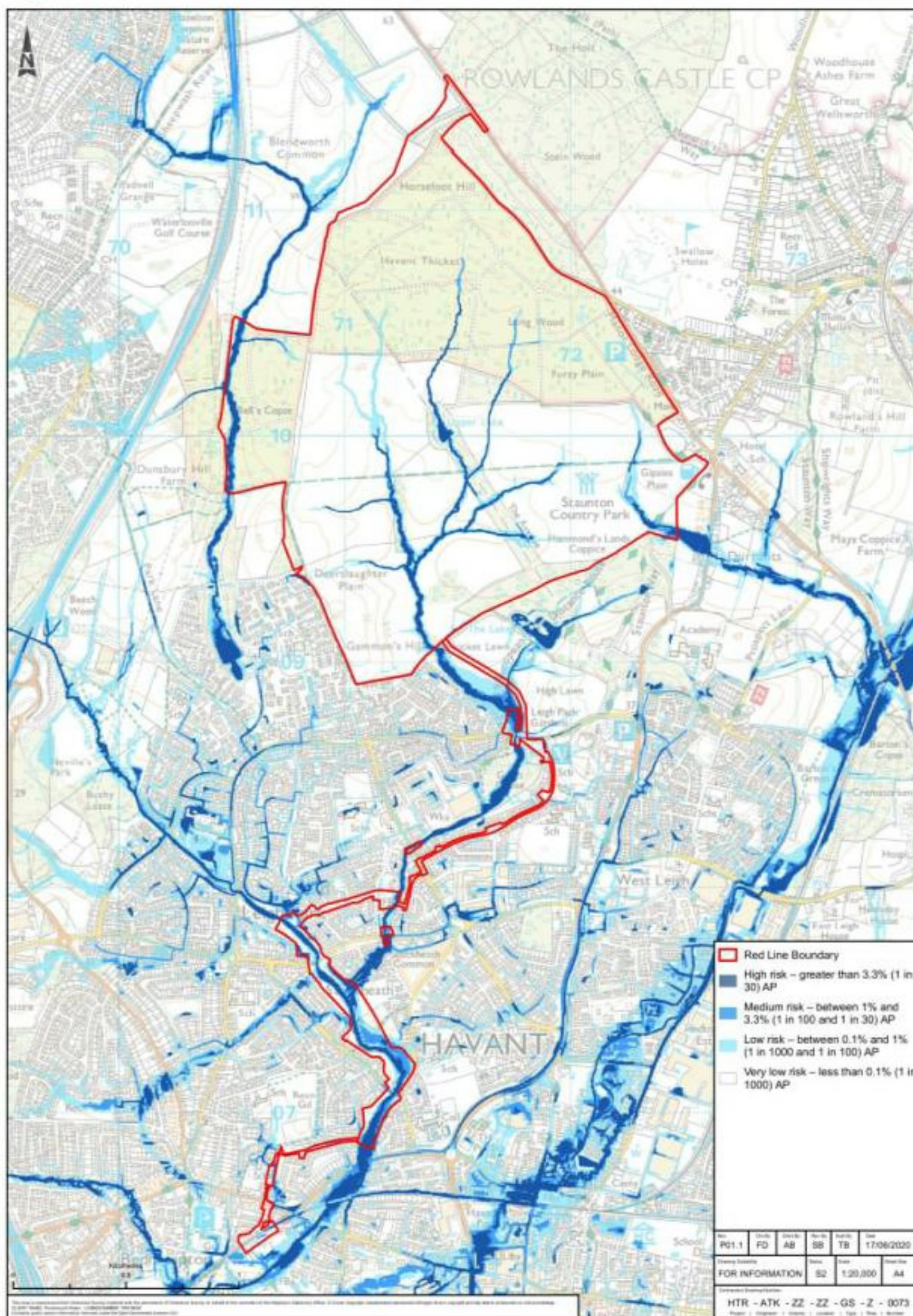


Figure 3-6 - Flood Risk from Surface Water (RoFSW)

Flooding from groundwater

Geology and hydrogeology

- 3.35. The study area lies within the Wessex Basin which covers most of Southern England. The regional basin structure causes the oldest rocks to outcrop in the north and dip to the south. In the northern part of the Havant Thicket study area, the Chalk is at outcrop and then dips towards the coast beneath the younger Palaeogene formations (including the Lambeth Group and the London Clay).
- 3.36. The geological structure of the area is significant; the Chalk has been subject to folding and faulting, resulting in the presence of the Portsdown Anticline and associated Chichester Syncline which stretches along the entire coastal margin.
- 3.37. These geological features are key to understanding the groundwater flow in this area. As the Chalk dips south, it forms a basin filled with the Palaeogene formations (in this case, Lambeth Group and London Clay) and then outcrops again on the other edge of the basin as the Chalk is tilted upwards.
- 3.38. This structure channels flow in the Chalk beneath the Chichester Syncline towards major spring flows which occur as the Chalk nears the surface closer to the coast. These major spring flows include the Havant and Bedhampton Springs.

Groundwater levels

- 3.39. Chalk groundwater levels are monitored by the Environment Agency at five observation boreholes within 5 km of the proposed site²⁶. The locations of the observation boreholes are shown in Figure 3-7. The data provided by the Environment Agency have been summarised in Table 3-5.

Table 3-5 - Groundwater level data

Borehole	Ground level (mAOD)	Length of record (years)	Location relative to reservoir central point	Groundwater levels (mAOD) (2009-2019)		Approx. depth of groundwater below ground (m)
				Min	Max	
Catherington	80.32	15	~5.0km_NNW	27.84	59.69	21 to 53 m
Wood Copse Forest	115.64	51	~4.5 km NE	30.80	45.27	70 to 85 m
Pyle Farm Cottages	56.36	53	~2.7 km N	21.66	43.48	13 to 25 m
Carpenters Finchdean	43.10	34	~4.0 km NNE	22.25	42.40	1 to 21 m
Day Lane Lo vedean**	87.42	56	~4.6 km NW	28.22*	50.07*	37 to 59 m

*min and max from 2009-2019 **limited data therefore min max from 2005-2010

Groundwater summary

- 3.40. As shown by the groundwater levels in Table 3-5, the approximate depth of groundwater below ground varies between 1-85 m. Carpenters Finchdean is situated 4.0 km away and although it has a higher maximum groundwater level recorded, it is not in close proximity to the site.
- 3.41. Furthermore, the Partnership for Urban South Hampshire (PUSH) SFRA identified that although Havant has been susceptible to groundwater flooding historically, these events have been focussed around the upper parts of the Lavant Stream. This is due to highly permeable geology meeting lower permeability geology. The document therefore only recommended that site specific FRAs for sites within the upper reaches of the Lavant Stream should consider high groundwater flows.
- 3.42. It is concluded that groundwater flood risk is defined as negligible risk and therefore not commented on further within this FRA.

²⁶ Environment Agency, 2020. Data request on behalf of Portsmouth Water [received on 02/03/2020] Reference 200302 SSD160159.

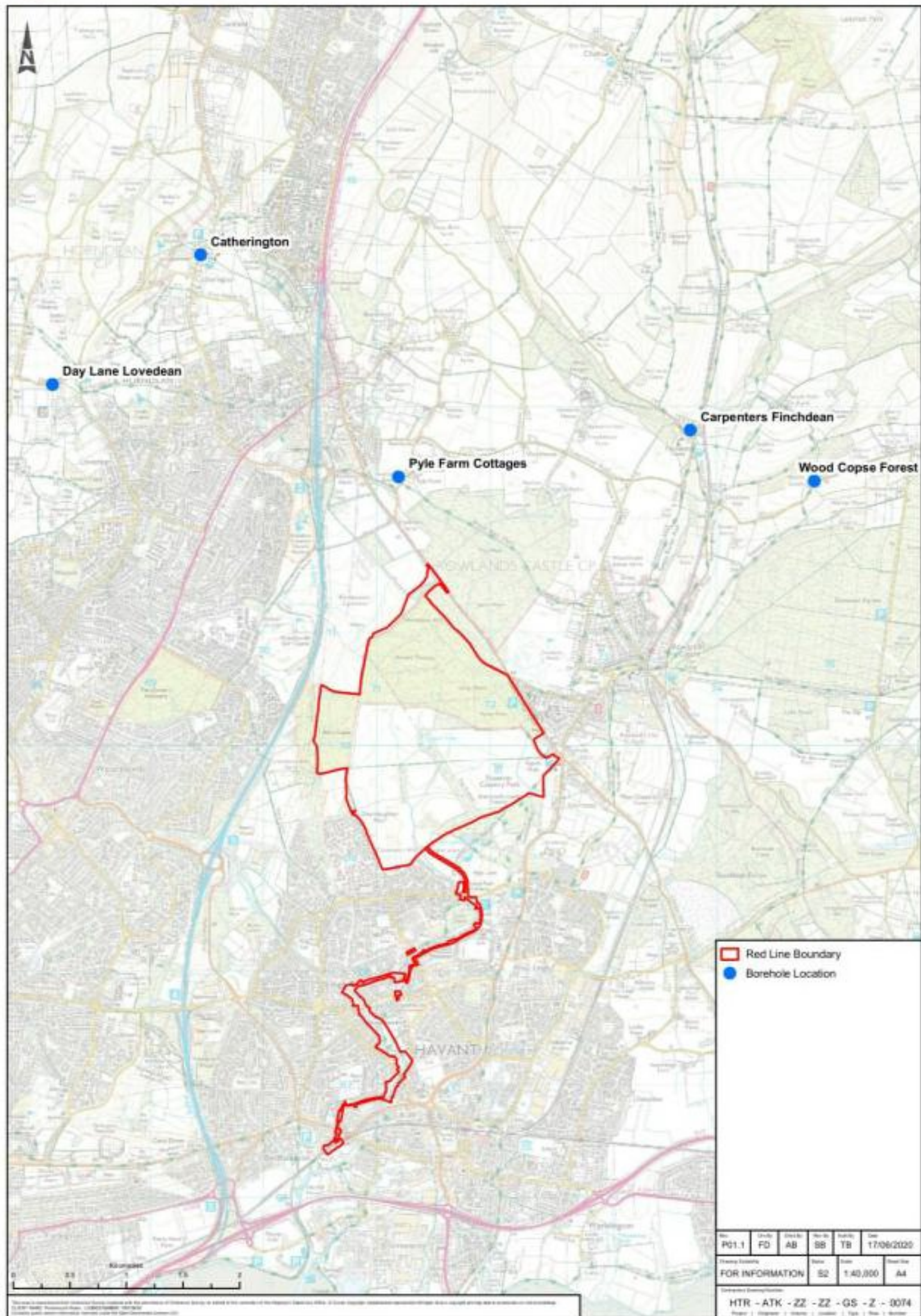


Figure 3-7 - Borehole locations relative to reservoir central point

Other sources of flood risk/ artificial sources

- 3.43. Flooding from artificial sources, in this case reservoirs, occurs when the design standard of a reservoir is exceeded and / or there is a failure of the asset causing downstream flooding.
- 3.44. At present there are no reservoirs within the proximity to the study area and consequently no risk of flooding from reservoirs exceeding their capacity and / or asset failure.
- 3.45. There are no other artificial sources of flood risk (canals, lakes etc) within the study area.
- 3.46. Although there is no baseline artificial flood risk, the operation of the reservoir creates a new, residual flood risk associated with the remote possibility of failure of the embankment. This residual risk is assessed in Section 8.

Summary of flood risk sources

- 3.47. A summary of the flood risk sources is given in Table 3-6. The table also identifies which sources of flood risk have the potential for change as a result of HTR. These will be discussed in sections 5 and 6 of this report.

Table 3-6 - Summary of flood risk sources

Source	Summary	Potential for change as a result of HTR
Fluvial flooding	Fluvial flooding occurs mostly in bank, or within area of open land on both the Riders Lane Stream and downstream on the Hermitage Stream after the confluence of the two. The model results, when compared to the Environment Agency Flood Zones, show that the detailed hydraulic modelling has further refined the flood extents, with noticeable decreases on the Hermitage Stream in Leigh Park and downstream of the NTL.	Yes
Tidal flooding	The NTL is situated on the Hermitage Stream at GR SU 70511 06237. Upstream of this point the flood risk is fluvial however below this point flood risk also includes tidal influences. There is no significant tidal flood risk, with flows staying largely in bank, or within areas of open land.	Yes
Surface water flooding	Surface Water flood risk generally follows the same path and trend as the Environment Agency fluvial Flood Zones, remaining in bank, or within areas of open space. There are, however, localised flow routes through the urbanised roads in the area and localised areas of surface water ponding caused by variations in topography.	Yes
Ground-water flooding	Groundwater levels and information from the PUSH SFRA indicate that groundwater flood risk is small within the catchment, with higher risk in the Lavant catchment to the East. Therefore, groundwater will not be commented on further in this FRA.	No
Other	There is the potential for residual risk to increase with the operation of the reservoir, due to failure of the embankment.	Yes

4. The Scheme

- 4.1. This chapter describes the key design information for the elements of the Scheme, which includes:
- Embankment, wetland retaining structure and control house;
 - Visitor centre, car park and access road; and
 - Bedhampton to Havant Pipeline.
- 4.2. For each section, both the permanent and construction works will be outlined.
- 4.3. The site layout is shown on Figure 4-1.
- 4.4. Full details of the development proposal are given in the Preliminary Design Report²⁷.

²⁷ Atkins reference HTR-ATK-ZZ-ZZ-RP-Z-0122

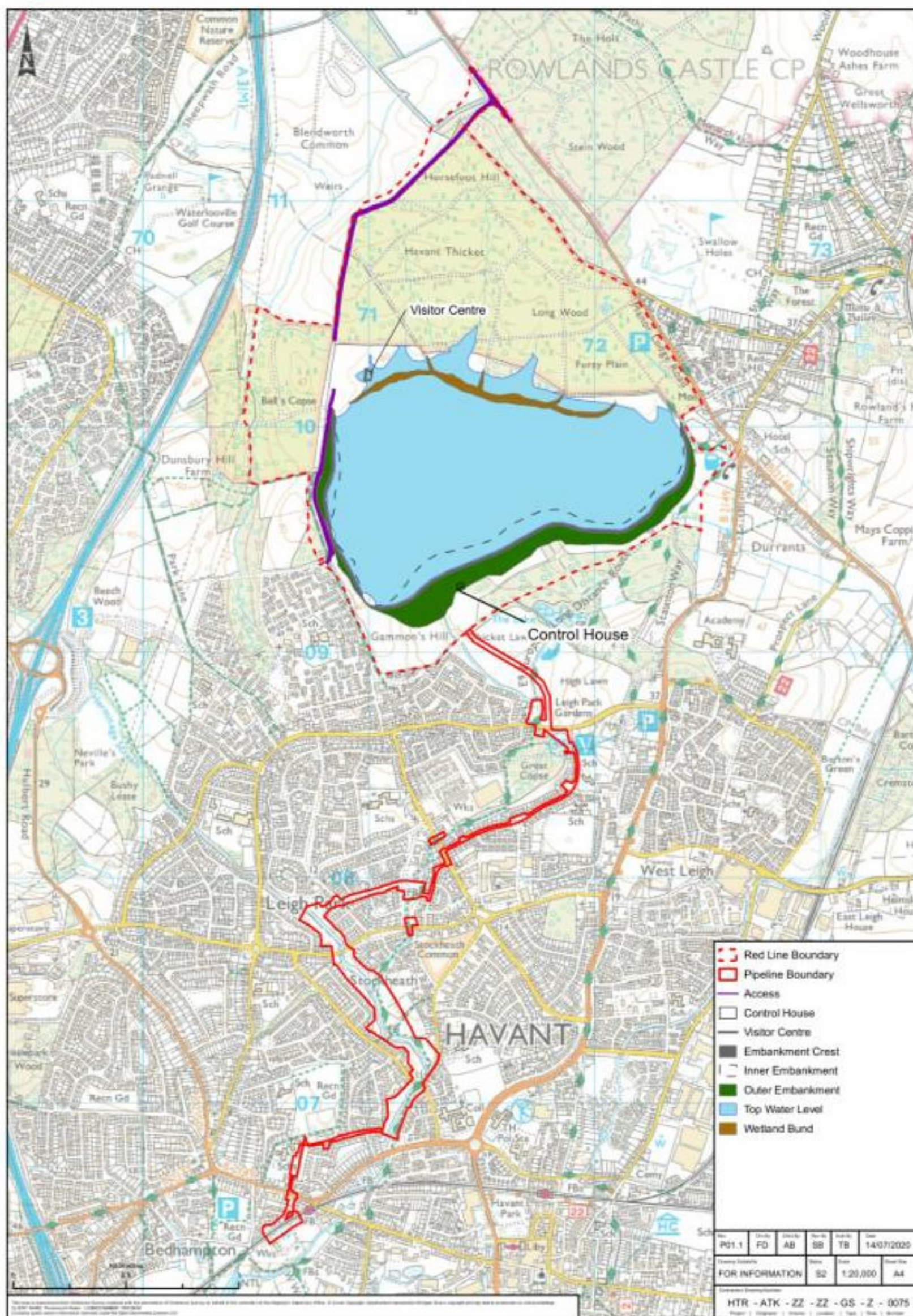


Figure 4-1 - Site layout

Embankment, wetland retaining structure and control house

Permanent scheme elements

Reservoir

- 4.5. The total capacity of the reservoir is 9,320 ML including borrow pits but not including water below lowest draw off level or retained in the wetland.
- 4.6. The top water level (TWL) is 39.5 m AOD, and the lowest operating level is 27.5 m AOD. The lowest level is 20% of the total volume and considered dead storage, i.e. the minimum level before the reservoir operationally fails.
- 4.7. The embankment is 2,770 m long with a height of 20 m and an embankment crest level of 40.5 m AOD. This is 1 m above the maximum water level and therefore there is 1 m of freeboard as demonstrated in the Reservoir Flood Study²⁸.
- 4.8. The spillway is at 39.5 m AOD and is designed to be a 3 m long weir, with a 700 mm diameter pipe, which gives a 1.3 cumecs discharge at a head of 0.4 m. This pipe feeds back into the Riders Lane Stream.
- 4.9. The embankment will have a toe drain around the southern edge, which will collect the surface water run-off from the embankment and channel it into either the Riders Lane Stream or the River Lavant. A small section of the Riders Lane Stream catchment (0.03 km²) will be diverted out of the catchment and into the Lavant Headwaters by the toe drain. The Lavant Headwaters catchment area is still reduced, however, as the embankment will intercept a proportion of the baseline catchment, reducing the overall area from approximately 0.50 km² to 0.38 km².

Wetland retaining structure

- 4.10. The wetland retaining structure extends for approximately 1.3 km along the northern shoreline at a distance 60-80 m away from where the TWL would cover.
- 4.11. This is planned to facilitate wetland habitat development, provide resilience to reservoir level changes and to ensure viable habitat that meets biodiversity net gain requirements.
- 4.12. At TWL (39.5 mAOD) the wetland area will be approximately 10.2 ha, however at retained water level (38.8 mAOD) it will be approximately 6.8 ha. The retained wetland level is the level at which the retaining structure starts to hold back water within the wetland area as reservoir levels fall.
- 4.13. The water level in the wetland will be actively managed through the provision of piped controls at strategic locations along the retention structure, that connect the retained wetland to the main body of the reservoir at naturally occurring low points in the topography.

Control House

- 4.14. The Control House, located to the edge of Round Wood, is a single-story building which will be partially embedded into the southern side of the embankment. It is designed to have level access.
- 4.15. This will control the culvert running through the embankment which will be used to discharge water from the reservoir, and also control the valves associated with drawdown the reservoir in an emergency.

Construction works

- 4.16. The key programme phases for construction are shown below in Table 4-1.

Table 4-1 - Key programme phases

Dates	Key activity / milestone
Spring 2021	Planning permission expected
Spring 2021 – November 2022	Preparation for earthworks outside of woodlands

²⁸ Atkins reference HTR-ATK-RZ-RE-RP-Z-0006.

Spring 2021 – November 2022	Species relocation, tree felling, enabling works including access road construction. Construction of embankment culvert structure
March 2023 – September 2024	Construction of embankment and reservoir control house
Autumn 2024 – Spring 2028	Filling of reservoir in stages with pumped water from Bedhampton Springs

4.17. During construction, there will be disturbance to the downstream flow within the Riders Lane Stream Headwaters. Mitigation measures have therefore been proposed and include:

- The proposal is to construct the wetland retaining structure first, in order to intercept and manage inflows from the three ephemeral streams. The flow within the wetland will be managed by a series of culverts from and between each 'compartment'. These will have valve/slucice controls.
- Three cofferdams will be constructed on HS1-3 while the retaining structure is built. Where these are installed, the stream flow will need to be transported downstream beyond the footprint of the reservoir to re-join Riders Lane Stream.
- The flow of water will need to be managed to ensure that flows downstream do not increase (up to and including the 1% AP) or decrease and cause adverse effects. This will be confirmed at the detailed design stage however, it has been assumed that these flows and volumes will be conveyed downstream to the reservoir embankment and into the Riders Lane Stream, consistent with existing conditions.
- The River Lavant headwaters will also be temporarily diverted to the spillway location where it will be drained into the Riders Lane Stream via a culvert.
- In order to facilitate a construction access road, two culverts on the headwater streams (HS1 and HS2) will need to be replaced. These will be confirmed during detailed design but will be sized appropriately for the anticipated flood flows and include a trash screen to allow vegetation clearance from the track.
- The control house will be built at the same time as the embankment.

Visitor centre, car park and access road

Permanent scheme elements

- 4.18. At the north west of the site, a visitor centre, educational hide and car park will be constructed.
- 4.19. The visitor centre has an approximate internal area of 620 m² and will include a reception, shop, toilets, staff facilities (office, staff room, staff toilets), a café and kitchen, and educational facilities including a classroom. To maximise the visitor experience and the connection with water and nature, the centre is positioned in the water and surrounded by the wetland environment.
- 4.20. The educational bird hide has an approximate internal area of 70 m².
- 4.21. The drainage design for this area of the development site will be developed at the detailed design stage, but will include SuDS principles and will ensure that there is no increase in surface water runoff up to and including the 1% (1 in 100) AP with climate change allowance.
- 4.22. In order to access this public area, an access road is required. There are two routes for the reservoir access road, a south and a north option. Both routes are being developed at preliminary design stage, on the basis that both will be progressed at detailed design.
- 4.23. There are two supporting documents:
- Access Route Option Evaluation²⁹
 - Access Route Basis of Design³⁰.

²⁹ Atkins reference HTR-ATK-XX-XX-RP-Z-0021

³⁰ Atkins reference HTR-ATK-XX-ZZ-RP-Z-0002

- 4.24. The southern option consists of two links of new carriageway connected by a section of existing carriageway (Swanmore Road and Calshot Road). To the west there is a link between Park Lane, and to the east there is a link from Swanmore Road, alongside the embankment, north towards the visitor car park. The north option connects the car park to the B2149 to the North.

Construction works

- 4.25. The construction of the visitor centre, car park and access roads, will increase the impermeable surfaces on the site. The detailed design will ensure no increase in runoff rates during construction. The construction of the visitor centre and associated facilities is scheduled for whilst the reservoir is being filled.
- 4.26. The southern access road crosses the Park Lane Stream. If the construction works are within 8 m of the Ordinary Watercourse or within the floodplain of the Park Lane Stream, then consent will be required.

Pipeline corridor

Permanent scheme elements

- 4.27. The Havant Thicket reservoir will operate as a pumped storage reservoir utilising surplus water from the Havant & Bedhampton Spring source. A bi-directional pipeline (4.4 km) is therefore required to allow for the transfer and discharge of water between the Bedhampton Springs and the reservoir. The location of this is shown on Figure 4-2.
- 4.28. The same pipeline also provides a conduit to convey part of the drawdown flow that is not discharged into the Riders Lane Stream. This flow is to be conveyed over halfway along the route, into the Hermitage Stream near its confluence with the Riders Lane Stream. This discharge facility can reduce the water level in the reservoir in line with the requirements of the Environment Agency Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning (Environment Agency, 2017).
- 4.29. The sizing of the conduit has been analysed, in relation to the flow rate passed down it, within a study on the Emergency Drawdown scenario³¹. The maximum emergency draw-down rate from the reservoir in accordance with Environment Agency guidance has been calculated as 10.9 m³/s (Atkins Preliminary Design Report³²), to allow the reservoir top water level to be drained down by 900 mm in depth over a 24-hour period, allowing the reservoir to be lowered by a third of its water depth in 6 days.
- 4.30. The total discharge rate from the reservoir is then split to allow a 7.05 m³/s to the Riders Lane Stream directly downstream of the reservoir embankment, and 3.85 m³/s to the Hermitage Stream approximately 2 km downstream from the reservoir.
- 4.31. In order for the Riders Lane Stream to receive these emergency drawdown flows without increasing flood risk, three changes to culverts along the river are required as part of the planning application. The locations are shown in Figure 4-2.

Table 4-2 - Alterations to existing culverts

Location	Existing configuration	Proposed configuration
Middle Park Way	36 m long, 1x1540 mm circular concrete culvert	1x1540 mm + 1x1200 mm circular culverts
Dunsbury Way	1x1450 mm circular culvert	1x1450 mm + 1x1200 mm circular culverts
Purbrook Way	3x900 mm circular culverts	5x900 mm circular culvert

³¹ Atkins reference HTR-ATK-XX-XX-RP-Z-0062

³² Atkins reference HTR-ATK-ZZ-ZZ-RP-Z-0122

- 4.32. The pipeline corridor crosses 3 watercourses, as shown on Figure 4-2.
- Hermitage Stream (Study section 4);
 - Riders Lane Stream (Study section 3); and
 - The Lake Stream (Study section 3).
- 4.33. The pipeline includes a 700 mm diameter pipe (1,650 m in length) for normal pump-and-return flows with this increased to 1200 mm diameter (2,750 m in length) for the section where it is to accommodate the emergency drawdown flow from the reservoir site to the Emergency Drawdown Valve. The location of the Emergency Drawdown Valve is not (yet) fixed but will be located on the Hermitage Stream upstream of the Riders Lane Stream confluence and Purbrook Way (Study section 4) and will be subject to geomorphology modelling, flood modelling and aquatic/ecological assessment. It is expected that the valve will be housed in a concrete structure adjacent to the watercourse.

Construction works

- 4.34. The red line boundary of the pipeline is shown in Figure 4-2.
- 4.35. The appointed contractor will confirm the methods to be adopted for the construction of the pipeline however, for much of the pipeline, it is assumed that open cut trench construction methodology will be used.
- 4.36. Whilst the red line boundary for the pipeline includes the main river, it has been assumed that, for the most part, works will be outside of the 8 m consenting boundary. Should works need to take place within the 8 m consenting boundary, a Flood Risk Activity Permit (FRAP) will be required.
- 4.37. At the three river crossings, the channels are relatively narrow and hence a dry open cut trench methodology has been assumed at this stage. This will entail the temporary damming of the watercourses. The construction design has assumed installation of a flume to ensure the downstream conveyance of flood flows up to and including the 1 in 100 (1%) AP. This methodology will require environmental permitting (FRAP) and will be developed in consultation with the Environment Agency. The alternative construction approach would be directional drilling beneath the watercourse, which would be preferable to avoid watercourse disruption and, subject to meeting Exemption No.3³³, would not need a FRAP.
- 4.38. A construction compound is also planned adjacent to the Riders Lane Stream (Study section 3). Each individual pipe section will have a secured (fenced) area and welfare office for the duration of the works within that section. For the purpose of the EIA, it has been assumed that there is no stockpiling of any materials at this construction compound or within any of the pipeline sections and that this is contained within the footprint of the reservoir construction area. All material excavated as part of the trench construction will be immediately removed from site and reinstated when required. Three additional "areas for construction use" are discussed in Section 6.
- 4.39. Surface water management during construction will need to follow normal guidelines for managing surface water during construction. Due to construction materials needed, there could be a temporary increase in hardstanding, and therefore increase in run-off.
- 4.40. The exact construction method for the valve house is unknown at this stage, but it has been assumed that this would only result in limited in-channel works around the location of the valve itself. This will however require an environmental permit.

³³ <https://www.gov.uk/government/publications/environmental-permitting-regulations-exempt-flood-risk-activities/exempt-flood-risk-activities-environmental-permits>

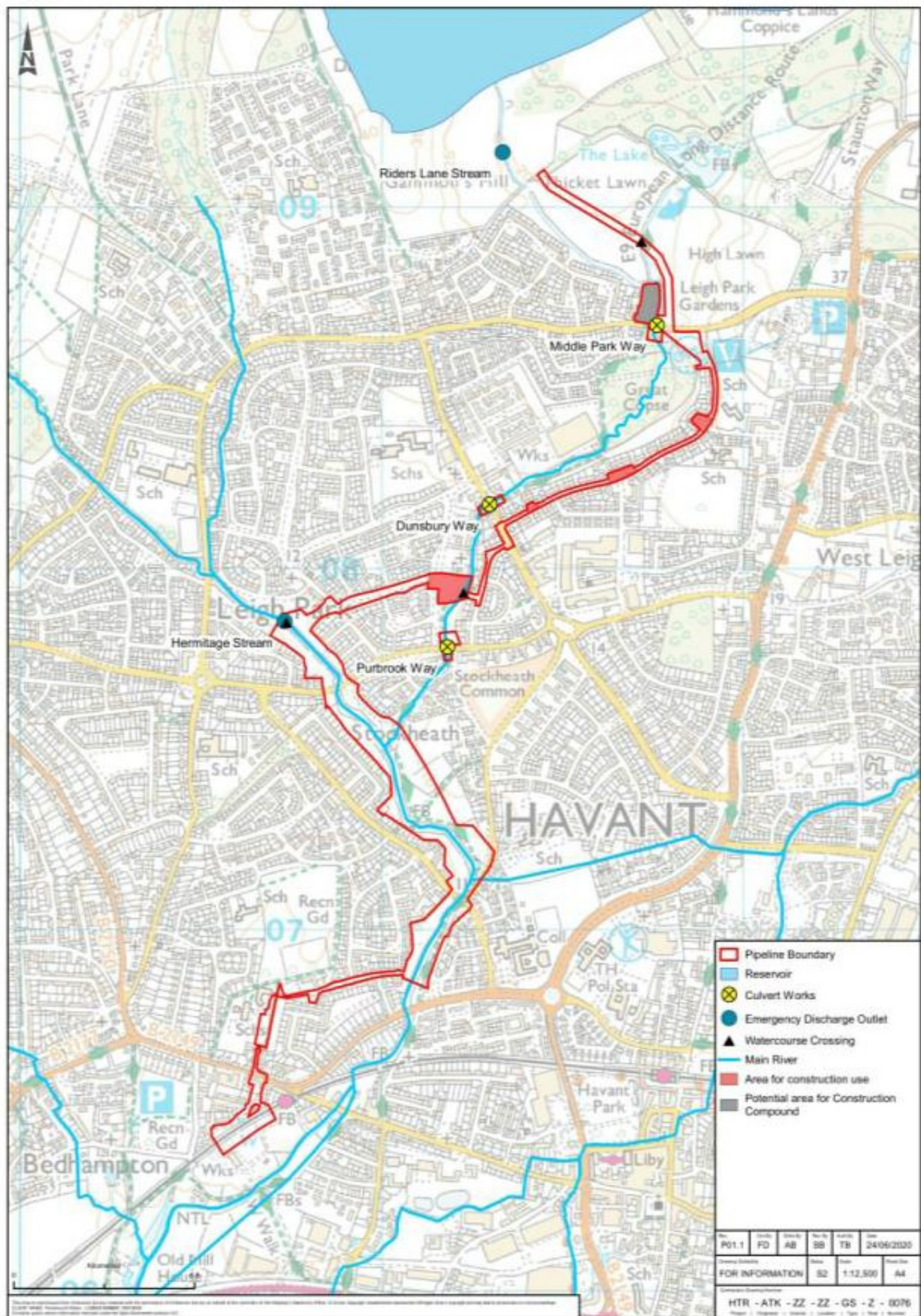


Figure 4-2 - Pipeline

5. Operation flood risk impacts

- 5.1. This chapter describes the flood risk impacts to and from the permanent scheme elements during the operation of the HTR. These will be discussed in the following sections:
- HTR – which includes:
 - Embankment and wetland retaining structure
 - Control House
 - Visitor centre, car park and access road
 - Bedhampton to Havant pipeline.
- 5.2. Construction impacts will be discussed in Section 6.
- 5.3. The impacts will be discussed for fluvial, tidal and surface water flood risk. Artificial flood risk will be discussed in Section 8 as residual risk.
- 5.4. The different elements of the scheme fall within different Flood Zones as shown in Table 5-1. Although the pipeline corridor is in Flood Zone 3, it is underground when operational and therefore there is no associated flood risk. The embankment crosses Flood Zones 2 and 3 where the Riders Lane Stream is situated. Once built however, the Riders Lane Stream will be culverted through the embankment and the embankment will be designed to impound water, so there will be no risk to the structure. All other elements of the proposed scheme are located within Flood Zone 1 and will not be at risk of flooding.

Table 5-1 - Flood Zone of Scheme elements

Scheme element	Flood Zone
Reservoir	Flood Zone 1
Embankment	Flood Zones 1, 2 and 3
Control House	Flood Zone 1
Visitor centre, car park and access road	Flood Zone 1
Pipeline corridor	Flood Zones 1, 2 and 3

Havant Thicket Reservoir

Fluvial flooding

- 5.5. As with the Baseline ICM results, the Operational results have been analysed by study section. The results are shown in the following figures:
- Figure 5-1 shows the post-scheme fluvial flood outlines for the 1% AP both under the current climate, and with the Central Estimate for climate change as discussed in section 3.
 - Figure 5-2 shows the post-scheme change in fluvial flood risk from the baseline for a 1% AP.
 - Figure 5-3 shows the post-scheme change in fluvial flood risk from the baseline for a 0.1% AP.
- 5.6. A quantitative summary for comparison with the baseline is provided in Table 5-2. The fluvial modelling has been undertaken using a 50% (1 in 2) AP downstream tidal condition.

Table 5-2 - Summary of change in fluvial flood risk as a result of the embankment and wetland retaining structure.

Study Section	Summary of change in fluvial flood risk
1	There is no change in flood risk in Study Section 1 as it is upstream of the reservoir.
2	Post construction, within Study Section 2, the ephemeral narrow watercourses will be replaced by a wide expanse of stored water. The watercourses in this study area have been replaced with the reservoir. The visitor centre is built within the wetland, and is therefore at risk of flooding resulting in a rise in the water level of the reservoir. The visitor centre is classified as "Less Vulnerable" development, and as such should be designed with a finished floor level above the 1% (1 in 100 year) AP event, plus an allowance for climate change (35% and 45% for the "central" and "higher central" estimates respectively). The 1% (1 in 100 year) AP event gives reservoir peak water levels of 39.65 m AOD and 39.67 m AOD for the "central" and "higher central" estimates. When an allowance for wave height is included (0.1m, due to the limited fetch distance from the wetland retaining structure) and freeboard (0.3 m), the recommended finished floor level for the visitor centre is 40.1 m AOD. The southern access road is within Study Section 2 and crosses into the Park Lane Stream catchment (Study Section 8). The northern access road does not cross into any other waterbody areas. The areas of construction are not within Flood Zone 2 or 3 and therefore neither access road is expected to alter fluvial flood risk.
3	Directly downstream of the reservoir. During a 1% AP flood, there is a reduction in peak flow of 3.64 m³/s and a reduction in peak level of 0.64 m at the upstream end, and 3.37 m³/s and 0.15 m at the downstream end. Across the Study Section the flood extent for this event is reduced by 0.03 km², which is an 87% reduction. During a 0.1% AP, there is a reduction in peak flow of 5.99 m³/s and a reduction in peak level of 0.66 m at the upstream end, and 2.82 m³/s and 0.10 m at the downstream end. Across the Study Section the flood extent for this event is reduced by 0.04 km², which is an 82% reduction. There is no predicted change in flood risk to The Lake Stream. A small section of the embankment will drain to the Riders Lane Stream via a toe drain, however, this will not increase fluvial flows compared to baseline levels. The location of the control house is not within Flood Zone 2 or 3 and during normal operation of the reservoir the, will not be at risk of flooding. If the reservoir were to breach, then the Control House would be at risk of flooding. This will be discussed in Section 8.
4	The operation of the reservoir and embankment is not predicted to have any impact on fluvial flood risk in Study section 4 as it is upstream of the confluence with the Riders Lane Stream
5	During a 1% AP, there is a reduction in peak flow of 3.50 m³/s and a reduction in peak level of 0.16 m at the upstream node, and 3.20 m³/s and 0.07 m at the downstream node. Between the two nodes the floodplain extent has reduced by 0.01 km², which is a 25% reduction. During a 0.1% AP, there is a reduction in peak flow of 4.48 m³/s and a reduction in peak level of 0.09 m at the upstream node, and 3.87 m³/s and 0.07 m at the downstream node. Between the two nodes the floodplain extent has reduced by 0.01 km², which is a 13% reduction.

	During a 1% AP, there is a reduction in peak flow of 3.20 m³/s and a reduction in peak level of 0.07 m at the upstream node, and 2.78 m³/s and 0 m at the downstream node. Between the two nodes there is no change in the floodplain extent.
6	During a 0.1% AP, there is a reduction in peak flow of 3.87 m³/s and a reduction in peak level of 0.07 m at the upstream node, and 3.95 m³/s and 0 m at the downstream node. Between the two nodes there is no change in the floodplain extent. The effect of the tidal influence reduces the impact of a reduction in flood level and extent.
7	As discussed in previous sections, the Lavant Headwaters will be intercepted by the new embankment. This reduces the catchment area from 0.50 km² to 0.31 km² (38%). This will result in a decrease in flow and level within the Lavant, thereby reducing flood risk. The embankment drains into a toe drain around the southern perimeter, some of which flows into the Lavant. This has been included in the reduction of catchment area calculation and will be discussed further in the surface water section.
8	The southern access road is within Study Section 2 and crosses into the Park Lane Stream catchment (Study Section 8). At this point it is an Ordinary Watercourse. The areas of construction are not within Flood Zones 2 nor 3 and are not expected to alter fluvial flood risk. There is therefore no change expected for the Park Lane Stream.

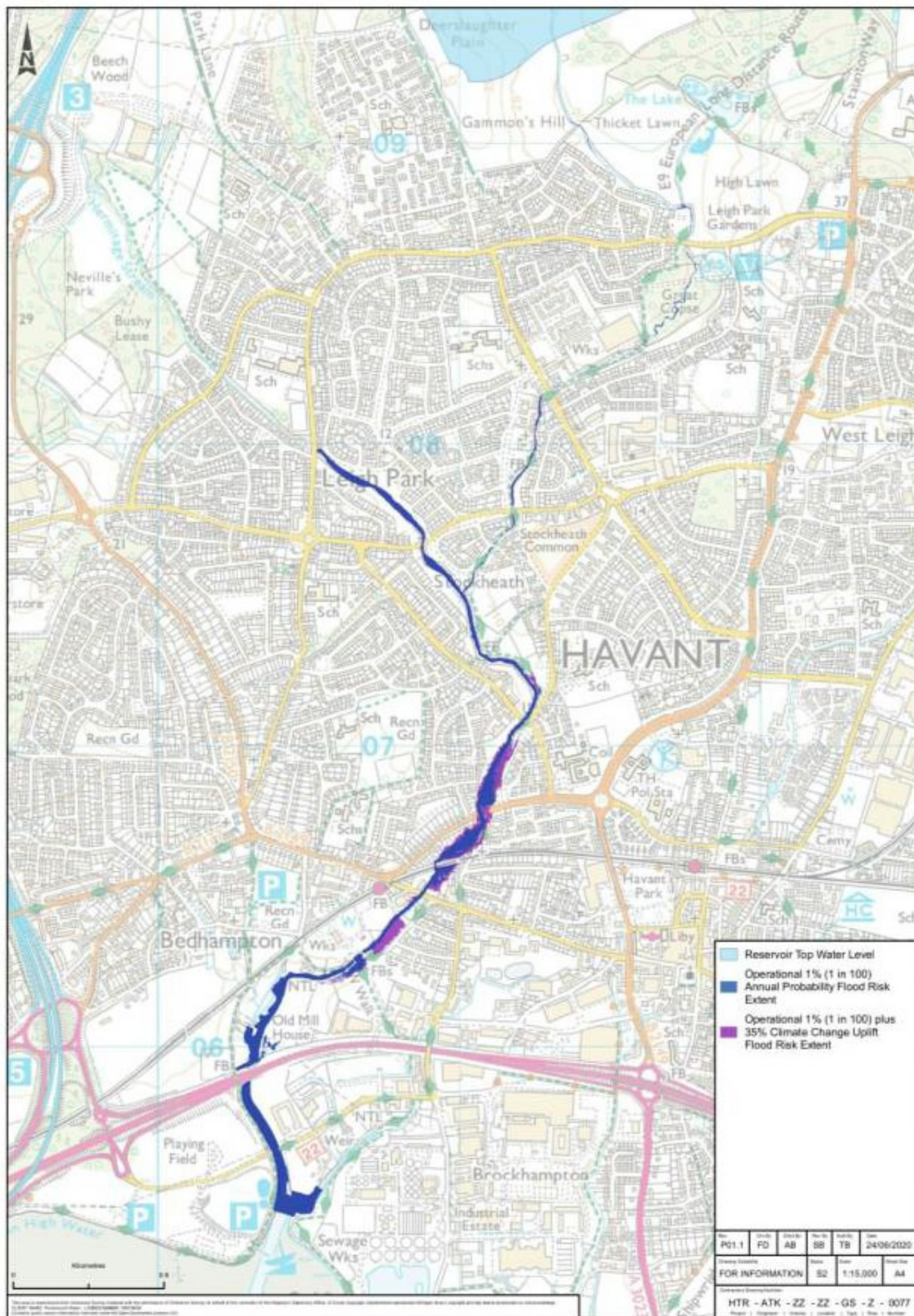


Figure 5-1 - Proposed Flood Risk during operation (1 in 100 and 1 in 100 + CC)

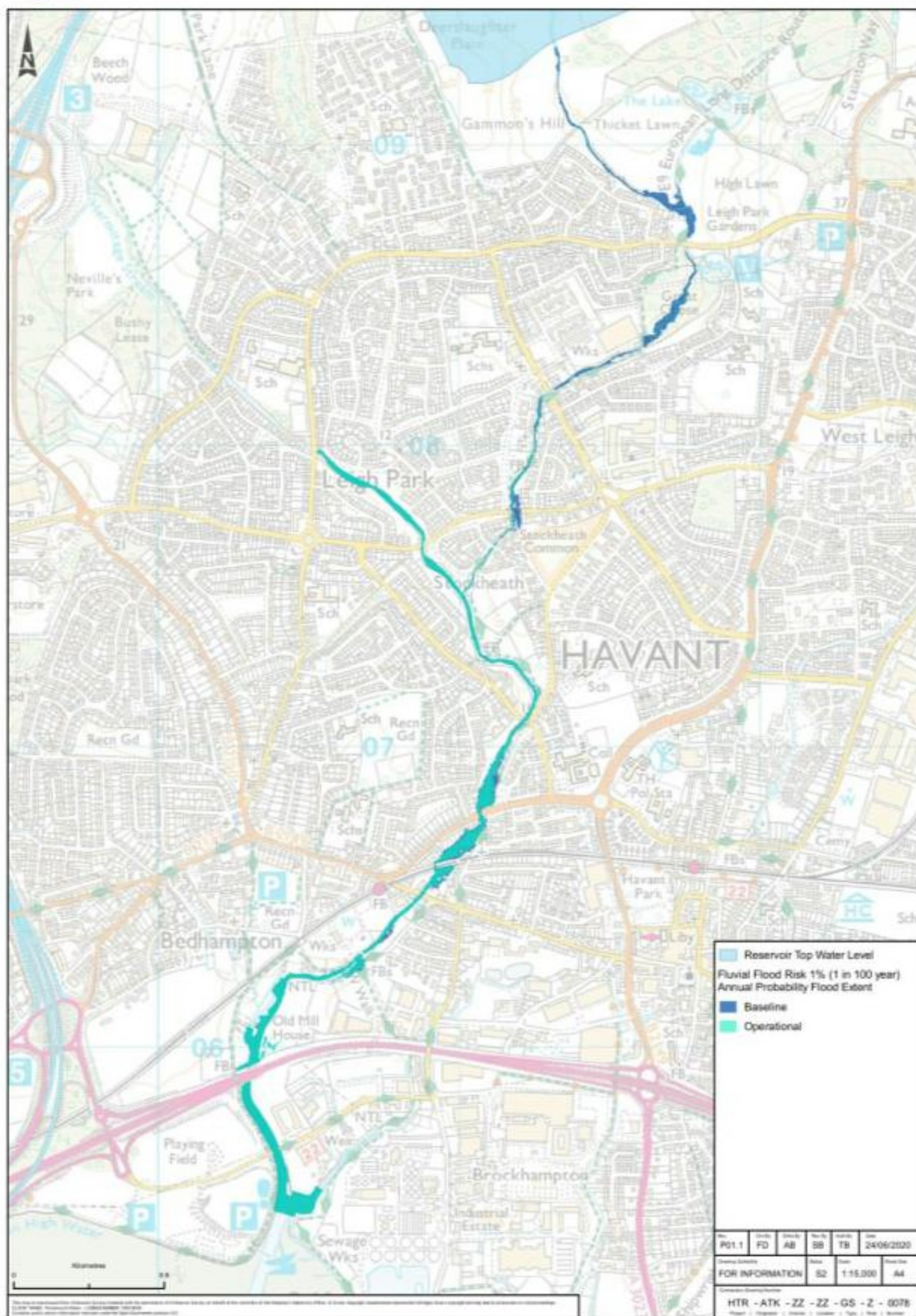


Figure 5-2 - Comparison of 1% fluvial baseline and with scheme

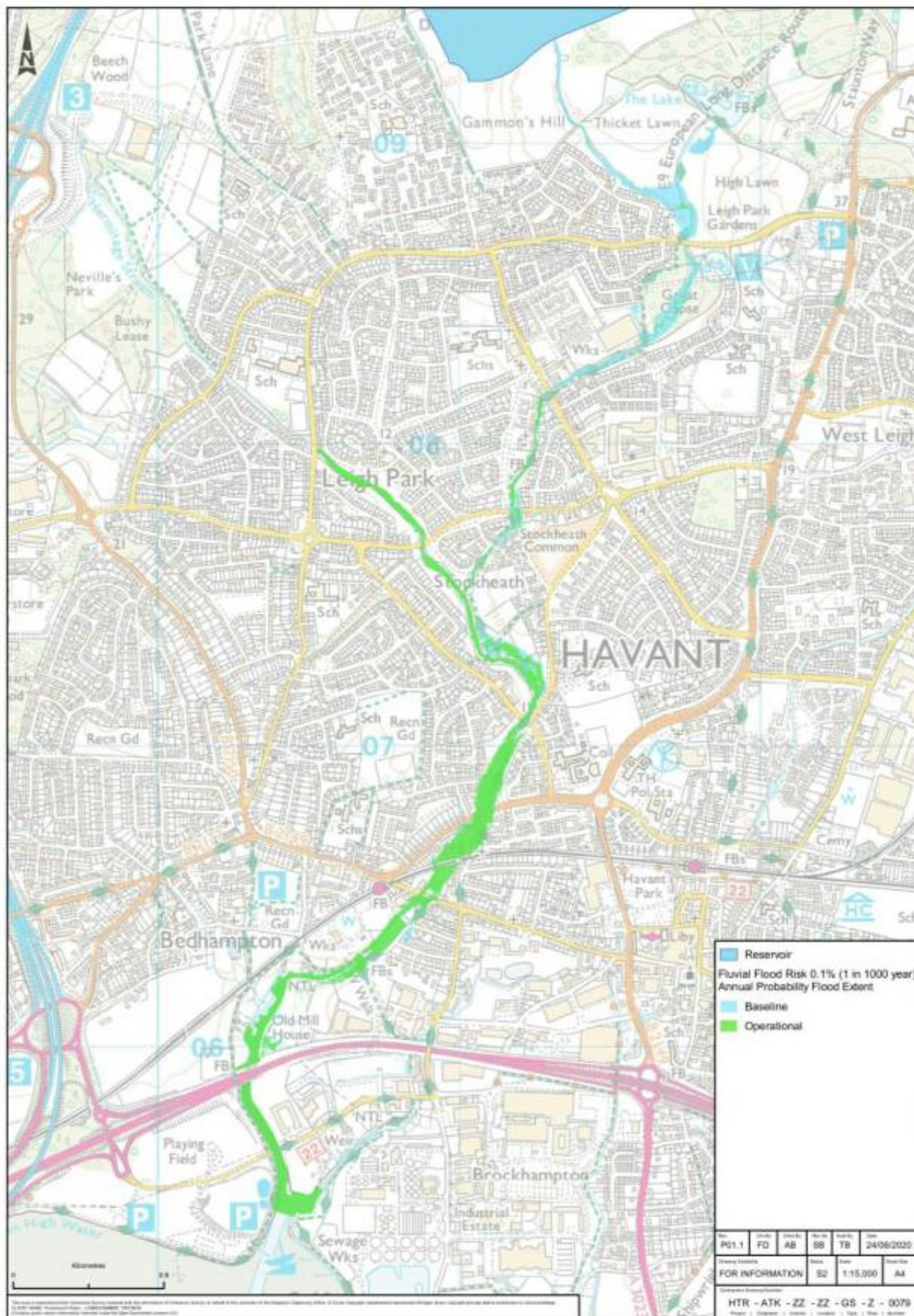


Figure 5-3 - Comparison of 0.1% fluvial baseline and with scheme

With scheme ICM Results

5.7. The following table records the difference from the baseline of the with scheme ICM results. This information has been summarised above within the discussion of With-scheme flood risk impacts.

Table 5-3 - With scheme flood risk and difference from baseline (CC is the central estimate)

Study Section	AP	With Scheme					Difference from baseline					
		u/s peak level (mAOD)	d/s peak level (mAOD)	u/s peak flow (m3/s)	d/s peak flow (m3/s)	Floodplain extent (km2)	u/s peak level (mAOD)	d/s peak level (mAOD)	u/s peak flow (m3/s)	d/s peak flow (m3/s)	Floodplain extent (km2)	Floodplain extent (% change)
1	No river data available, assessment will be qualitative											
2	No river data available, assessment will be qualitative											
3	3.33% (1 in 30)	20.20	7.71	0.16	2.64	0.003	-0.60	-0.15	-2.93	-2.81	-0.02	-85%
	1% (1 in 100)	20.22	7.91	0.20	3.25	0.004	-0.64	-0.15	-3.64	-3.37	-0.03	-87%
	1% (1 in 100) + CC	20.30	8.26	0.31	4.42	0.01	-0.64	-0.11	-4.87	-3.81	-0.03	-85%
	0.1% (1 in 1000)	20.35	8.42	0.39	5.43	0.01	-0.66	-0.10	-5.99	-2.82	-0.04	-82%
4	3.33% (1 in 30)	9.99	7.71	11.77	13.52	0.01	0.00	-0.15	0.00	0.00	0.00	0%
	1% (1 in 100)	10.10	7.90	14.53	16.68	0.01	0.00	-0.16	0.00	0.00	0.00	0%
	1% (1 in 100) + CC	10.28	8.26	19.62	22.52	0.01	0.00	-0.11	0.00	0.01	0.00	0%
	0.1% (1 in 1000)	10.41	8.42	23.90	27.32	0.01	-0.01	-0.10	0.00	0.01	0.00	0%
5	3.33% (1 in 30)	7.71	2.91	16.02	16.45	0.02	-0.15	-0.04	-2.91	-1.70	-0.01	-33%
	1% (1 in 100)	7.90	2.97	19.76	19.50	0.03	-0.16	-0.07	-3.50	-3.20	-0.01	-25%
	1% (1 in 100) + CC	8.26	3.14	26.77	27.87	0.05	-0.11	-0.08	-3.97	-3.58	-0.01	-17%
	0.1% (1 in 1000)	8.42	3.24	32.54	32.65	0.07	-0.09	-0.07	-4.48	-3.87	-0.01	-13%
6	3.33% (1 in 30)	2.91	2.72	16.45	24.51	0.04	-0.04	0.00	-1.70	-1.32	0.00	0%
	1% (1 in 100)	2.97	2.72	19.50	27.13	0.04	-0.07	0.00	-3.20	-2.78	0.00	0%
	1% (1 in 100) + CC	3.14	2.72	27.87	35.19	0.04	-0.08	0.00	-3.58	-3.94	0.00	0%
	0.1% (1 in 1000)	3.24	2.72	32.65	40.33	0.04	-0.07	0.00	-3.87	-3.95	0.00	0%
7	No river data available, assessment will be qualitative											
8	No river data available, assessment will be qualitative											

Tidal flooding

- 5.8. As discussed in Section 3, various tidal scenarios were tested to observe the impact of the scheme on tidal flood risk.
- Figure 5-4 shows the tidal flood risk in the baseline and with-scheme for the 0.5% (1 in 200) AP tidal scenario.
 - Figure 5-5 shows the tidal flood risk in the baseline and with-scheme for the 0.5% (1 in 200) AP tidal scenario for 2125 Upper End estimate climate change.
- 5.9. See Appendix A Mapping (figures A21-A22) for the 0.1% AP tidal outlines for both under current climate, and with climate change allowances.
- 5.10. These figures show that there is no change in extent between the baseline and with-scheme results under the current climate or with inclusion of climate change. The construction of the scheme therefore does not affect tidal flood risk.

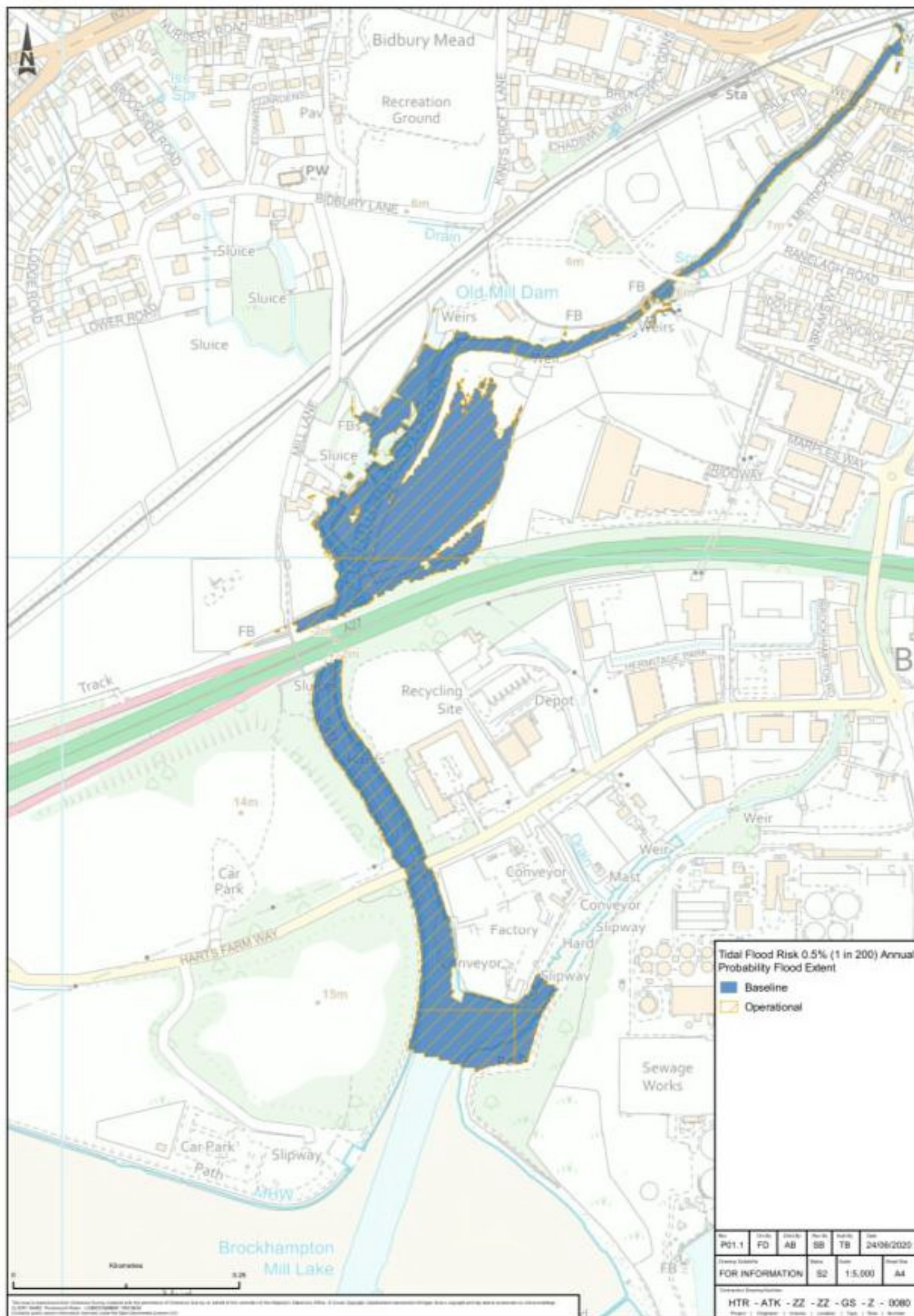


Figure 5-4 - Comparison of 0.5% AP Tidal baseline and with scheme

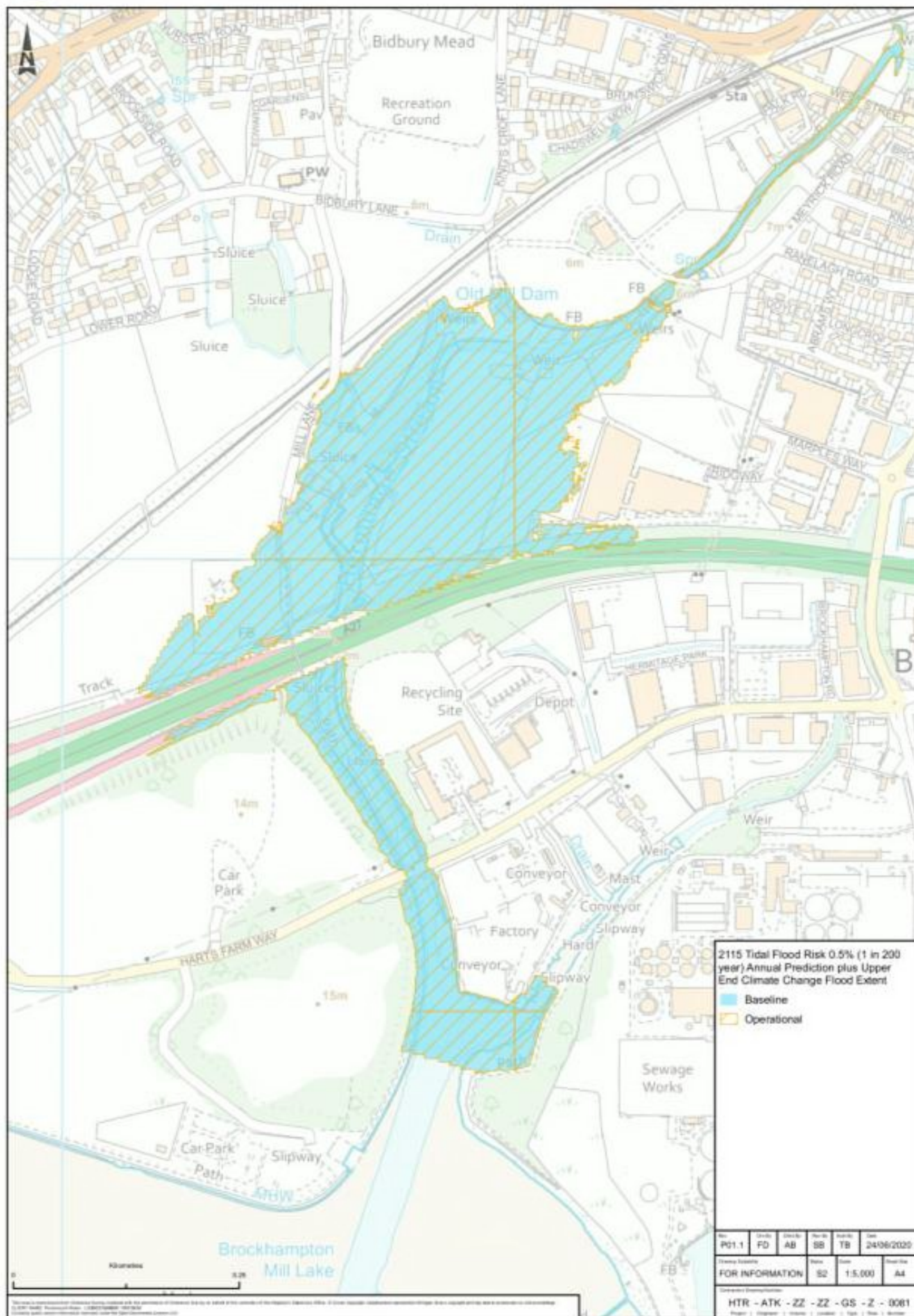


Figure 5-5 - Comparison of 0.5% AP + CC tidal baseline and with scheme

Surface Water flooding

- 5.11. The changes in surface water flood risk will be described qualitatively. A summary of the change in surface water flood risk is given in Table 5-4.

Table 5-4 - Summary of change in surface water flood risk as a result of the embankment and wetland retaining structure.

Study Section	Summary of change in surface water flood risk
1	There is no change in flood risk in Study Section 1 as it is upstream of the reservoir. The surface water flow routes will discharge into the existing streams and into the wetland and / or reservoir.
2	There is no surface water flood risk due to the embankment or wetland retaining structure. The visitor centre will increase the area of hardstanding with a corresponding increase in surface water runoff rates and volumes. Even though these facilities are on the boundary between Study Sections 2 and 8, it is assumed that the surface water flows will be discharged in the Riders Lane Catchment (Study Section 2). The design of the drainage will be finalised at the detailed design stage, however SuDS principles will be adopted where possible and the drainage design will follow the SuDS manual and guidance from the Local Plans/SFRAs. It is therefore assumed that the Visitor Centre will not increase surface water runoff and downstream flood risk. Both the southern access road and the northern access road will increase hardstanding. It is assumed and required that drainage is designed and installed following appropriate policy and guidance so as not to alter surface water flood risk.
3	There is no predicted change in surface water flow routes as a result of the reservoir. Localised surface water flood risk, however, is likely to reduce due to the reduction in peak levels and flow within the watercourse providing more capacity for the urban surface water network to discharge into the channel. Surface water drainage from part of the embankment will be routed into the toe drain and into the Riders Lane Stream. The surface water management from the control house will be managed as part of the overall drainage from the embankment and this will be finalised during the detailed design stage. As part of the design, surface water management techniques will be incorporated, for example, Sustainable Drainage Systems (SuDS), in order to ensure no increase in surface water flood risk from the site of the Control House.
4	There are no changes in surface water flood risk within Study Section 4.
5	There is no predicted change in surface water flow routes as a result of the reservoir. Localised surface water flood risk, however, is likely to reduce due to the reduction in peak levels and flow within the watercourse providing more capacity for the surface water network to discharge into the channel.
6	There is no predicted change in surface water flow routes as a result of the reservoir. The tidal influence in this section reduces the benefit of reduced flows and levels in the channel having a beneficial effect on the discharge of urban surface water.
7	As discussed in relation to fluvial risk, the Lavant Headwaters catchment is reduced by 38%. This calculation includes the surface water drainage from the embankment which is routed into the toe drain and into the River Lavant. There is therefore a reduction in surface water flood risk for Study Section 7.
8	It is assumed that all drainage from the visitor center and car park will discharge to Study section 2 and therefore, there is no predicted change to surface water flood risk to Study Section 8. The southern access road will include drainage management which will not increase surface run off rates or volumes into the catchment.

Pipeline corridor

- 5.12. The pumping operation of the pipeline, due to its construction underground, is not expected to have any impact on Fluvial/Tidal or Surface Water flood risk within any Study Sections.
- 5.13. The pipeline however is also used as part of the drawdown operation. It is expected that the 'emergency drawdown' system will be tested on commissioning up to the capacity of the watercourses. This is discussed further in Section 8.
- 5.14. For routine drawdown for maintenance, the system will be tested at a reservoir water level lower than the Top Water Level. A testing programme will be developed to measure head and discharges at various openings, and at various stages of filling, and these can be compared with the manufacturer's performance data. Discharges during these tests could be less than the channel capacity. It is assumed that submerged valves will be tested to an opening corresponding to 10% of the Emergency Drawdown flow³⁴. This would be around 0.7 m³/s for Riders Lane Stream (via the emergency spill weir and/or the compensation flow channel) and 0.38 m³/s for Hermitage Stream (at the Emergency Drawdown Valve location). For context, the 50% (1 in 2) AP flows on the Riders Lane Stream and Hermitage Stream are approximately 3 m³/s and 8 m³/s respectively, demonstrating that these flows are comparatively small.
- 5.15. The arrangement of maintenance/control valves upstream will still allow the valves to be tested under full head but giving only a partial discharge. The valves would be tested individually, for example, testing the Hermitage Emergency Drawdown, and then after closing the valves, testing the Riders Lane Stream Emergency Drawdown. It is likely that the testing would occur twice per year, following Environment Agency guidance (Environment Agency, 2017). Careful planning would be required to test at the correct time, e.g. the system would not be tested if the additional discharge would cause flooding, or if the river was unusually low. It would also be unreasonable to expect a test to be run for a long period, since this would constitute a waste of water which has been pumped into the reservoir with associated energy costs.
- 5.16. The culvert improvement works (shown on Figure 4-2) to facilitate the Emergency Drawdown Strategy are designed to enhance discharge along the Riders Lane Stream, and thus have no impact on fluvial flood risk during normal operation, other than potentially improving conveyance at key pinch points and providing additional contingency for culvert or trash screen blockage.

³⁴ Panel of Experts comments on the Emergency Drawdown System, 23/04/2020

6. Construction flood risk impacts

- 6.1. This chapter describes the flood risk impacts as a result of the construction elements of the scheme. These will be discussed in the following sections:
- The reservoir site (which includes):
 - Embankment and wetland retaining structure;
 - Control House; and,
 - Visitor centre, car park and access roads.
 - Pipeline corridor
- 6.2. There are no construction impacts on tidal flood risk and therefore this is not assessed in this section.

The reservoir site

Fluvial flooding

- 6.3. During construction there is an increased risk to the ephemeral streams upstream of the reservoir embankment (study sections 1 and 2). As discussed in Section 4, the wetland retaining structure will be constructed in the first phase of construction, and the area will be used to intercept and manage inflows from the three headwater streams of the Riders Lane Stream. This will prevent flooding in the working area and control surface water flow.
- 6.4. While the wetland retaining structure is being built, cofferdams will be constructed in these three headwater streams. This will cause an increase in fluvial flood risk due to the obstruction of the channels and water levels will increase locally, upstream of the cofferdams. This is a rural area with no receptors at risk of increased flood risk. It has been assumed that these flows will be attenuated and conveyed downstream to the reservoir embankment, in order to maintain downstream flows.
- 6.5. Once operational, the wetland retaining structure will reduce flood risk to the area under construction. It has also assumed that the water stored behind this structure will be attenuated and conveyed downstream to the reservoir embankment.
- 6.6. The Lavant headwaters will also be re-routed towards the embankment.
- 6.7. The mechanism of attenuating and transporting the stored flows into the Riders Lane Stream will be confirmed at the detailed design stage. For the purpose of the FRA and EIA however, it has been assumed this will not increase flow or flood risk to the reservoir footprint or other downstream watercourses.
- 6.8. It has also been assumed that the flows will be diverted to the embankment in the proposed spillway location, where they will be drained into the Riders Lane Stream via a culvert. This culvert has been sized accordingly to accommodate all of the headwaters within Study Section 2, including the Lavant headwaters. The risk of exceedance during the construction period is 10%. For a construction period of 3 years, the flood corresponding to a 10% risk has an Annual Probability (AP) of 3.33%. The conveyance capacity of the temporary diversion pipes in the embankment culvert will need to convey a peak flow of the 3.33% (1 in 30) AP flood (4.6m³/s).
- 6.9. Downstream of the embankment the existing flow regime should therefore be maintained for the duration of construction up to and including a 3.33% (1 in 30) AP flood event. Flood flows in excess of this will be attenuated within Study section 2 and will not be passed downstream. As there are no properties within Study Section 2, this should be acceptable mitigation.
- 6.10. The replacement of two culverts to accommodate the temporary construction access road (across HS1 and HS2) will be sized to ensure no obstruction to flood flows up to and including a 1% (1 in 100) AP and are therefore not expected to have any impacts on fluvial flood risk.
- 6.11. There is no change in fluvial flood risk as a result of the construction of the visitor centre, car park and access roads as they are not within Flood Zone 2 or 3.

Surface Water flooding

- 6.12. Within the reservoir footprint, surface water follows the same flow paths as fluvial flows and so the same mitigation measures will apply. Even though there may be an increase in hardstanding as a result of constructing the embankment and wetland retaining structure, surface water flood risk downstream will not increase.
- 6.13. The construction of the permanent access roads, visitors centre, educational hides and car park will impact surface water flood risk by increasing runoff rates and volumes due to increased areas of hardstanding. These facilities are on the boundary between Study sections 2 and 8. It has been assumed that surface water flows would discharge into Study section 2.
- 6.14. The temporary and permanent drainage design for these facilities is currently being developed. It is assumed however, that the design will include sustainable drainage principles in order not to increase run-off rates and volumes to the surrounding areas.
- 6.15. A drainage management plan will include details of how increased surface water runoff from new areas of hardstanding (i.e. access roads, visitor centre, car parks etc) will be managed to ensure no increase in flood risk.
- 6.16. Construction techniques will follow industry standard surface water management guidelines such as
- Pollution Prevention Guidelines (PPGs); and,
 - Guidance for Pollutions Prevention (GPPs)
- 6.17. PPG were a series of documents developed by the Environment Agency for England and Wales, the Northern Ireland Environment Agency for Northern Ireland and the Scottish Environmental Protection Agency for Scotland. Each PPG was targeted at a different business or activity. The PPGs were withdrawn from use in 2015, but are still considered good practice as they have, to date, not been replaced with other formal guidance.
- 6.18. In Scotland, GPP have been developed by the Scottish Environmental Protection Agency as a replacement for PPG. Although they only apply in Scotland and Northern Ireland, GPPs provide environmental good practice guidance for the whole of the UK, accepting there may be some exceptions. Relevant PPG and GPP to this scheme include:
- PPG 1: Understanding your environmental responsibilities - good environmental practices;
 - PPG 3: Use and design of oil separators in surface water drainage systems;
 - GPP 4: Treatment and disposal of wastewater where there is no connection to the public foul sewer; and,
 - GPP 5: Works and maintenance in or near water.

Pipeline corridor

- 6.19. Figure 4-2 shows the location of the pipeline and the red line boundary for the construction works.

Fluvial flooding

- 6.20. The appointed contractor will confirm the methods to be adopted for the construction of the pipeline however, for much of the pipeline, it has been assumed that open cut trench construction methodology will be used.
- 6.21. The pipeline crosses the watercourses in three locations (shown on Figure 4-2).
- Riders Lane Stream.
 - The Lake Stream.
 - Hermitage Stream.
- 6.22. An open trench approach would entail the temporary damming of the watercourses. The construction design has assumed installation of a flume to ensure the downstream conveyance of flood flows up to and including the 1% (1 in 100) AP. This correlates to flows of 6.7 m³/s, 0.7m³/s and 14.5 m³/s at the Riders Lane Stream, The Lake Stream, and Hermitage Stream crossings respectively. No change in fluvial flood risk is therefore expected.
- 6.23. Material from the trenches will not be stored on site (i.e. within the pipeline red line boundary) and therefore there are no predicted impacts on flood flow paths.

6.24. The pipeline red line boundary crosses into both Flood Zone 2 and 3. The following are within Flood Zone 3 and therefore have a high risk of flooding.

- Potential area for construction compound north of Middle Park Way;
- Area for construction use between Ellisfield Road and Riders Lane; and,
- All three culvert improvement work areas.

6.25. The construction compound at Middle Park Way has an area of approximately 5,400 m². Flood Zone 3 covers approximately 2,500 m² of the north eastern section of this area, and the ICM FRA model for the 1% (1 in 100 year) AP event covers a reduced area of approximately 1,600 m² of the compound. As the compound borders the Riders Lane Stream, flood depths are predicted to reach up to 0.5 m in the north east corner of the proposed area. Usage of the construction compound will be directed towards the south western area, which is on higher ground and in Flood Zone 1. Figure 6-1 below shows the construction compound area and the modelled flood depths.

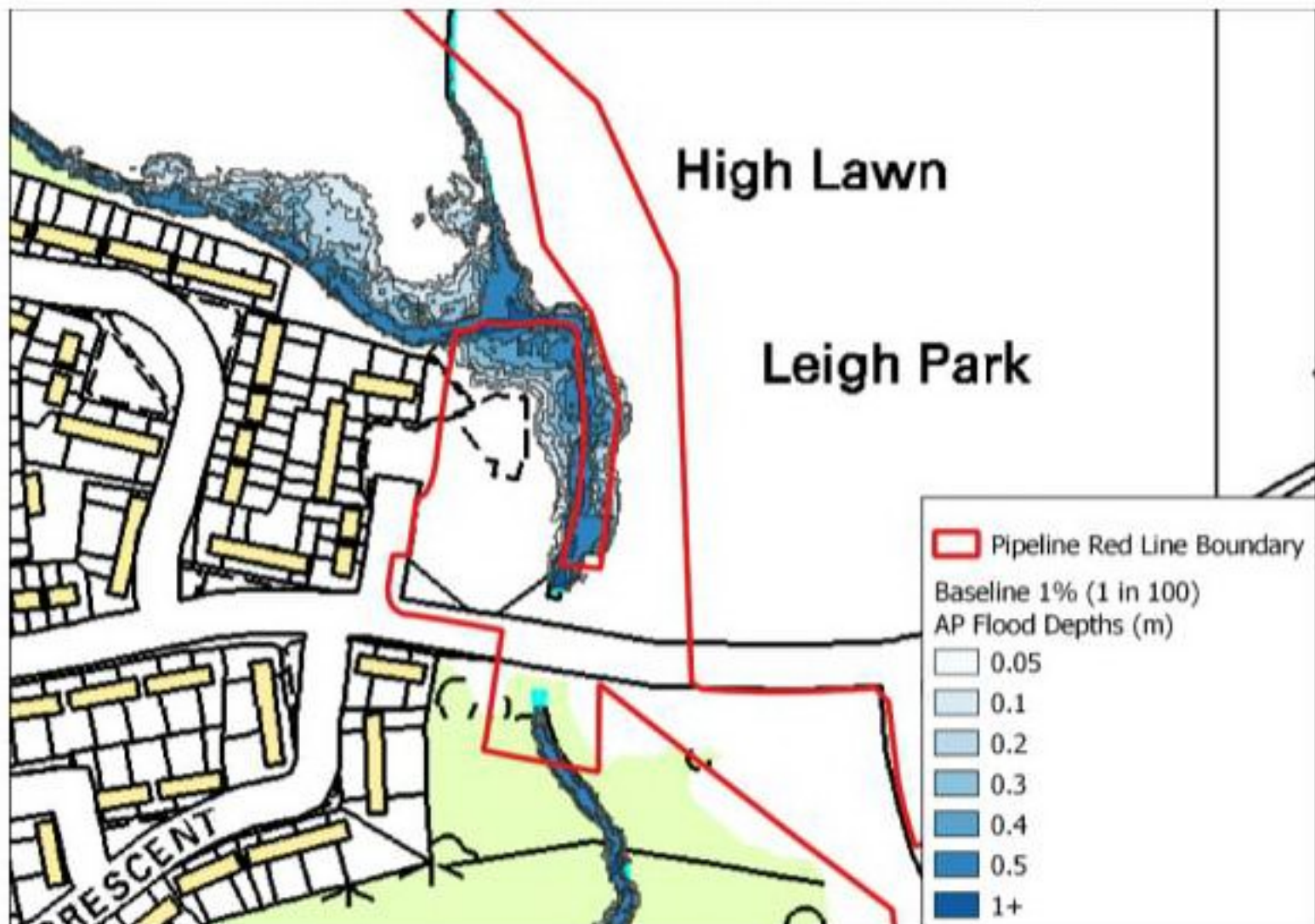


Figure 6-1 - Middle Park Way Construction Compound - flood depths

6.26. The area proposed for construction use between Ellisfield Road and Riders Lane spans the Riders Lane Stream at this location. The floodplain is very constrained at this point, and Flood Zones 2 and 3, and the ICM FRA model outlines are similarly narrow as well as being tree lined. Construction use will be limited to the large area outside of the flood outlines. Figure 6-2 below shows the route of the Riders Lane Stream through the area for construction use, and the associated flood depths.

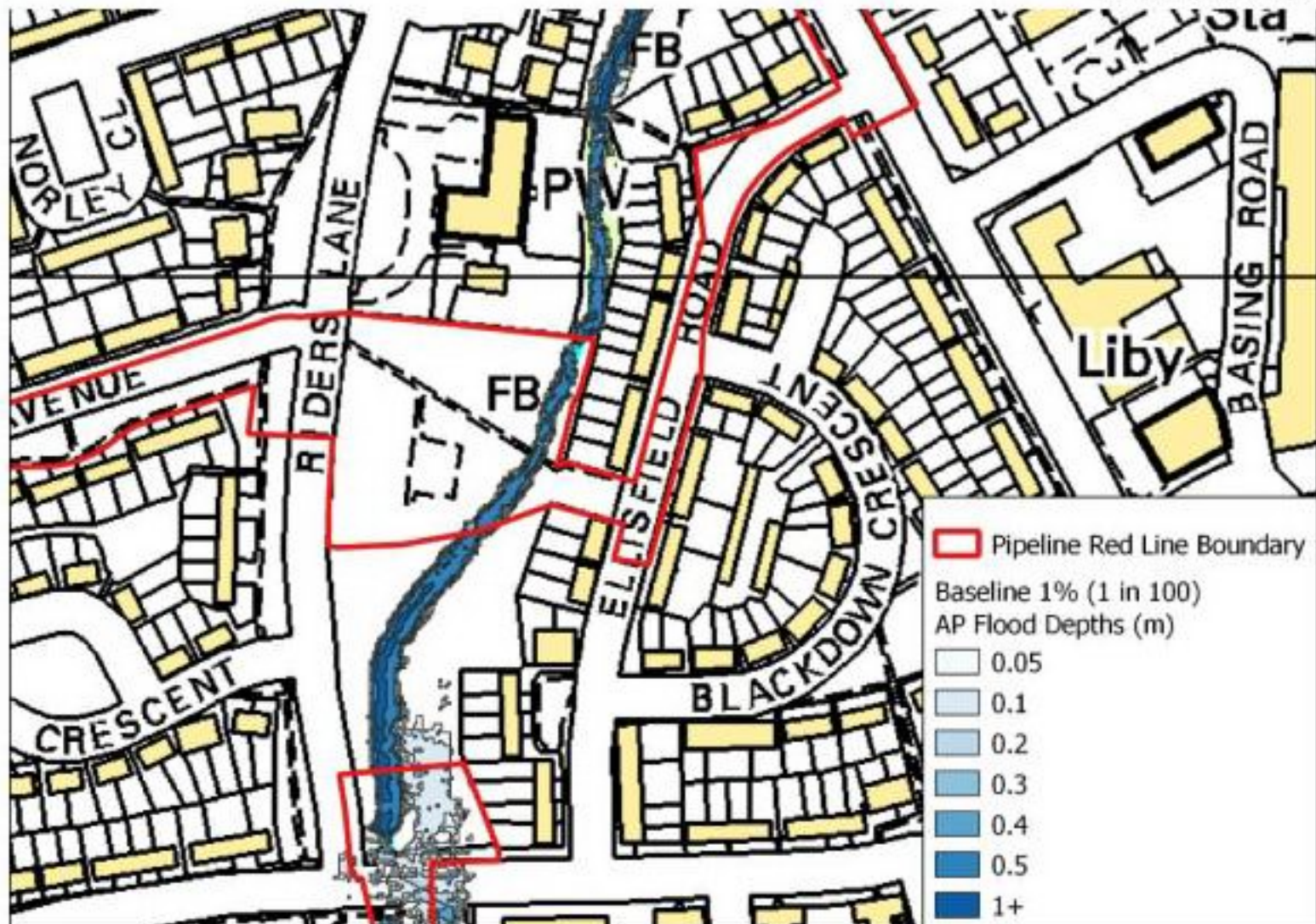


Figure 6-2 - Ellisfield Road/Riders Lane Construction Compound - flood depths

- 6.27. The culvert improvement work areas at Middle Park Way, Dunsbury Way and Purbrook Way also have sections within Flood Zone 3 and the ICM FRA 1% (1 in 100) AP flood outline. Middle Park Way and Dunsbury Way are not expected to flood during the 1% (1 in 100) AP event, although Purbrook Way has modelled flood depths of up to 0.2 m.
- 6.28. A Flood Risk Activity Permit (FRAP) will be required for these works and any works within 8 m of an Environment Agency main river (Hermitage Stream and Riders Lane Stream). An Ordinary Watercourse consent will be required for the culvert works which cross the Lake Stream.

Surface Water flooding

- 6.29. The construction of the pipeline and construction compounds may temporarily increase areas of hardstanding. This could result in minor changes to localised surface water runoff rates and flow paths and will need to be mitigated in order not to increase flood risk elsewhere.
- 6.30. A Drainage Management Plan will include details of how the construction compounds will safely manage surface water flows in line with Environment Agency PPG guidance.

7. Managing flood risk

- 7.1. Most of the mitigation required to safely manage flood risk are already embedded in the construction and scheme design. This includes measures such as pipes, culverts and flumes to ensure the continued pass forward of flows downstream in line with current flow regimes. These will all be sized to ensure there is no increase in fluvial flood risk.
- 7.2. As part of the detailed design, the mechanism for transporting and attenuating stored fluvial flows behind the cofferdams for the wetland retaining structure will need to be designed to ensure no increase in downstream flood risk.
- 7.3. The following flood-related management plans will be drafted and implemented.

Drainage management plan

- 7.4. A drainage management plan will be prepared for the scheme. This will include details such as design standards, pollution control/interception, outfall locations, earthworks, access roads and haulage routes and surface water drains.
- 7.5. It will also include details of how increased surface water runoff from new areas of hardstanding (i.e. access roads, visitor centre, car parks etc) will be managed to ensure no increase in flood risk. Sustainable Drainage Systems (SuDS) will be included in the drainage design where possible. The drainage design should follow the SuDS manual and guidance from the Local Plans/SFRAs should be incorporated.
- 7.6. It should also include details of how the construction compounds will safely manage surface water flows in line with Environment Agency PPG guidance.

Construction environment management plan

- 7.7. A construction environment management plan (CEMP) will also be prepared for the scheme. This will include an agreed process for managing the volume and quality of discharge of water for the construction compound, areas used for construction use, other areas of hard standing and haulage roads, which would discharge into the site water management systems via shut-off sluice valves.
- 7.8. It will also make provision for attenuating surface water run-off generated as part of the reservoir and pipeline construction, which has additional benefits of better management of water quality. The appointed contractor will be expected to attenuate flows during construction up to and including a 3.33% (1 in 30) AP.
- 7.9. Mitigation in line with best practicable measures and construction environmental management procedures is described in the Outline Environmental Management Plan (OEMP), which will form the basis for the production of the CEMP. A Record of Environmental Actions and Commitments (REAC) identifying the commitments to address potential flood risk effects of the construction of the Scheme is included in the OEMP in Appendix A2.1 in ES Volume 4.

Environmental permits

- 7.10. A flood risk activity permit (FRAP) will be required if construction works are:
 - On or near a main river (within 8m for main river and 16m for tidal rivers); or
 - In a floodplain.
- 7.11. As the red line boundary includes Environment Agency main rivers and Flood Zones 2 and 3, a FRAP will be required to permit the construction of the pipeline corridor, culvert enlargement works and construction compounds within the floodplain.
- 7.12. Should the construction method for the pipeline change to directional drilling to cross the watercourses, this activity may be exempt from the need for a FRAP (exemption activity No.3³⁵), assuming the standard conditions can be met.

³⁵ <https://www.gov.uk/government/publications/environmental-permitting-regulations-exempt-flood-risk-activities/exempt-flood-risk-activities-environmental-permits>

- 7.13. Works within 8 m of an Ordinary Watercourse, for example, the pipeline construction crossing the Lake Stream and / or the culvert improvements under the construction access road for the headwaters to the Riders Lane Stream, will require an Ordinary Watercourse consent from the LLFA.
- 7.14. An Environmental Permit from the Environment Agency will also be required to allow any controlled discharges to the Riders Lane Stream, Hermitage Stream and Lavant Stream (for the toe drain).

Operational maintenance

- 7.15. Havant Thicket reservoir will be regulated under the Reservoirs Act 1975 as amended by the Flood and Water Management Act 2010, which was set to promote the safety of large raised reservoirs.
- 7.16. As required by the legislation, the reservoir is being designed and will be constructed under the supervision of an All Reservoirs Panel Engineer (as the Construction Engineer) and during operation a Supervising Engineer will do regular visits (twice a year typically) and prepare an annual statement. An Inspection Engineer will inspect the reservoir at least every 10 years, or when requested by the Supervising Engineer. Following the inspection, the Inspection Engineer in his report can identify "measures in the interests of safety" which must be addressed by Portsmouth Water.
- 7.17. As described in Section 5, testing of the emergency discharge valves will be undertaken twice per year, in line with the requirements of the Reservoirs Act (1975), and the Environment Agency Guidance for Drawdown Capacity (Environment Agency, 2017). The configuration of the control and submerged valves permits testing of each under full head, without a discharge at design capacity. This regular maintenance and testing is required to ensure the continued safe operation of the reservoir and Emergency Drawdown Strategy.
- 7.18. Full testing of the 'emergency drawdown' system, up to the capacity of the watercourses, will be tested on commissioning of the reservoir. Further details of this emergency drawdown scenario are presented in Section 8.

8. Residual risk

Embankment failure

- 8.1. The Scheme will be operated and managed by Portsmouth Water in line with the requirements of the Reservoirs Act 1975. The Act ensures that reservoir operators are legally responsible for ensuring that the stringent safety measures are met through regular maintenance of the reservoir assets (i.e. embankment, spillway, control system and drawdown infrastructure). With these measures in place, the risk of embankment failure and reservoir beach are highly unlikely.
- 8.2. The consequence of a breach, however, has been modelled to understand the flood risk associated with a failure of the reservoir embankment. It has been modelled using a bespoke 2D hydraulic model built in InfoWorks ICM. The approach follows guidance as outlined below:
 - Environment Agency Reservoir Flood Mapping Specification³⁶;
 - Environment Agency Guide to risk assessment for reservoir safety management³⁷; and
 - Panel of Experts recommendations, 2019³⁸.
- 8.3. A single breach location has been considered on the southern face of the embankment, approximately at the natural/existing path of the Riders Lane Stream. This location is selected at the lowest section in the surrounding topography (the valley for the stream), and thus at this location the embankment is at its highest. This provides the worst-case scenario by releasing the entire volume of the reservoir, excluding the volume retained in the wetland. This is also considered the most likely point of failure, since it is the location of the control structure/outfall.
- 8.4. Full details on the breach modelling approach and of the results have been provided to the Environment Agency as a separate report. The method is described in outline below, with breach flood extents only presented in the FRA.
- 8.5. Three scenarios were modelled, as per the Reservoir Flood Mapping Specification:
 - Wet day with embankment breach: Breach scenario in combination with a 0.1% (1 in 1000) AP fluvial event (including on the Hermitage Stream), and an initial reservoir water level taken from the Probable Maximum Flood (PMF) event (maximum water level of 39.96 mAOD);
 - Wet day without embankment breach: For the HTR, this is equivalent 0.1% (1 in 1000) AP fluvial event since the embankment does not overtop (including in the PMF event); and,
 - Dry day with embankment breach: No fluvial inflow, and an initial water level taken as Top Water Level (39.5 mAOD).
- 8.6. The "Dry Day" breach flood extent is shown below in Figure 8-1, with further breach mapping shown in Appendix A Mapping.
- 8.7. The results show that if a breach were to occur then there will be a wide-ranging flood extent that covers most of the urban centre of Havant, Leigh Park and Langstone. This scenario is never anticipated as the reservoir is designed and maintained never to breach.
- 8.8. Following a reservoir inspection by a Panel Engineer, if a breach is considered likely, an 'Emergency Drawdown Strategy' will be implemented to avoid a breach of the embankment and significantly reduce the volume held in the reservoir.

³⁶ Environment Agency, 2016. Reservoir Flood Mapping Specification

³⁷ Environment Agency, 2013. Guide to risk assessment for reservoir safety management. [Online] [Accessed June 2020].

³⁸ Panel of Experts meeting, 11/12/2019



Figure 8-1 - "Dry day" breach flood extent

Emergency Drawdown Strategy

- 8.9. As discussed in Section 4, the pipeline that pumps surplus water from the Havant & Bedhampton spring source, will also be used to provide an Emergency Drawdown facility to safely reduce the water level in the reservoir in line with the requirements of the Environment Agency Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning.
- 8.10. If early signs of internal erosion are identified, the emergency drawdown system provides a means to lower a reservoir's level quickly. This may be a precautionary measure while the problem is investigated, or an emergency measure. In either case, the primary objective would be to reduce the load on the dam, and thereby arrest the internal erosion which has already initiated, or is at high risk of initiating, and prevent it from developing. If this objective cannot be achieved then partial drawdown may at least buy time to make repairs, or evacuate downstream, or employ other techniques to avert failure. In the very worst outcome the intervention of drawdown may at least reduce the consequences of failure by reducing the volume of water released in a breach.
- 8.11. The maximum draw-down rate from the reservoir in accordance with Environment Agency guidance has been calculated as 10.9 m³/s (Preliminary Design Report³⁹) to allow the reservoir top water level to be drained down by a depth of 900mm over a 24-hour period for a Category A reservoir.
- 8.12. The Emergency Drawdown will be accommodated through outfall structures into the Riders Lane Stream at the toe of the dam structure and into the Hermitage Stream via a pipe discharging in the vicinity of Purbrook Way, approximately 2 km downstream from the reservoir site.
- 8.13. The total discharge of 10.9 m³/s from the reservoir will be split to allow:
 - 7.05 m³/s down the Riders Lane Stream; and,
 - 3.85 m³/s down the Hermitage Stream.
- 8.14. The ICM described in section 3 was used to determine the flood risk associated with releasing the drawdown flows. To determine if the capacity of the downstream channels are sufficient to accommodate the Emergency Drawdown without causing flooding to properties, the proposed flows were simulated in combination with an existing flow condition equivalent to the 10% exceedance flow (Q10) in each channel, and a 50% (1 in 2) AP tidal condition. This approach was discussed and agreed with the Environment Agency⁴⁰. In addition to this design test scenario the following sensitivity scenarios were also tested:
 - Modelling of a range of fluvial event flows to identify the thresholds for property flooding at key locations;
 - Testing of model sensitivity to roughness and headloss; and,
 - Testing of Emergency Drawdown scenario against extreme tidal events, including with an allowance for climate change.
- 8.15. Sensitivity testing for the Emergency Drawdown scenario is presented in Appendix B Hydraulic modelling report.
- 8.16. The model results indicate that the emergency drawdown flows into the Riders Lane Stream potentially resulted in 4 properties upstream of Middle Park Way being at risk of flooding, as well as causing some inundation of Dunsbury Way and Purbrook Way roads/bridges. To mitigate this, enhancements to culverts along the Riders Lane Stream (described in Table 4-2) have been included in the scheme design. Middle Park Way had been identified in the Arup Environmental Handover Note (2009) as being at risk of flooding, and this risk is mitigated by the culvert enhancements. The additional locations identified as being at risk in the Arup report are not considered at risk of flooding from the Emergency Drawdown scenario.
- 8.17. These enhancements to culvert capacity were represented in the ICM to create a new 'Emergency Drawdown' flood extent map for the 7.05 m³/s discharge, plus an inclusion of the Q10 flows as per the Environment Agency guidance.
- 8.18. The flood extent is shown on Figure 8-2. There are no residential properties/dwellings at risk of flooding in this design scenario, however a small number of gardens/sheds (2-3) and one industrial

³⁹ Atkins reference HTR-ATK-ZZ-ZZ-RP-Z-0115

⁴⁰ Atkins reference HTR-ATK-XX-XX-MI-Z-0006.

unit are within the modelled extent, showing depths between 0.1-0.2 m. There is no inundation of roads due to the enhanced capacity at the culvert constrictions.

- 8.19. The strategy currently meets the requirement set by the LPA of causing no flooding to residential properties, but does result in the limited flooding described above to gardens, sheds and industrial units. It is recommended that PW work with the EA and LPA to determine whether a plan for flood warning and / or temporary flood resistant / resilience measures should be drafted for deployment as part of the Emergency Drawdown Strategy.

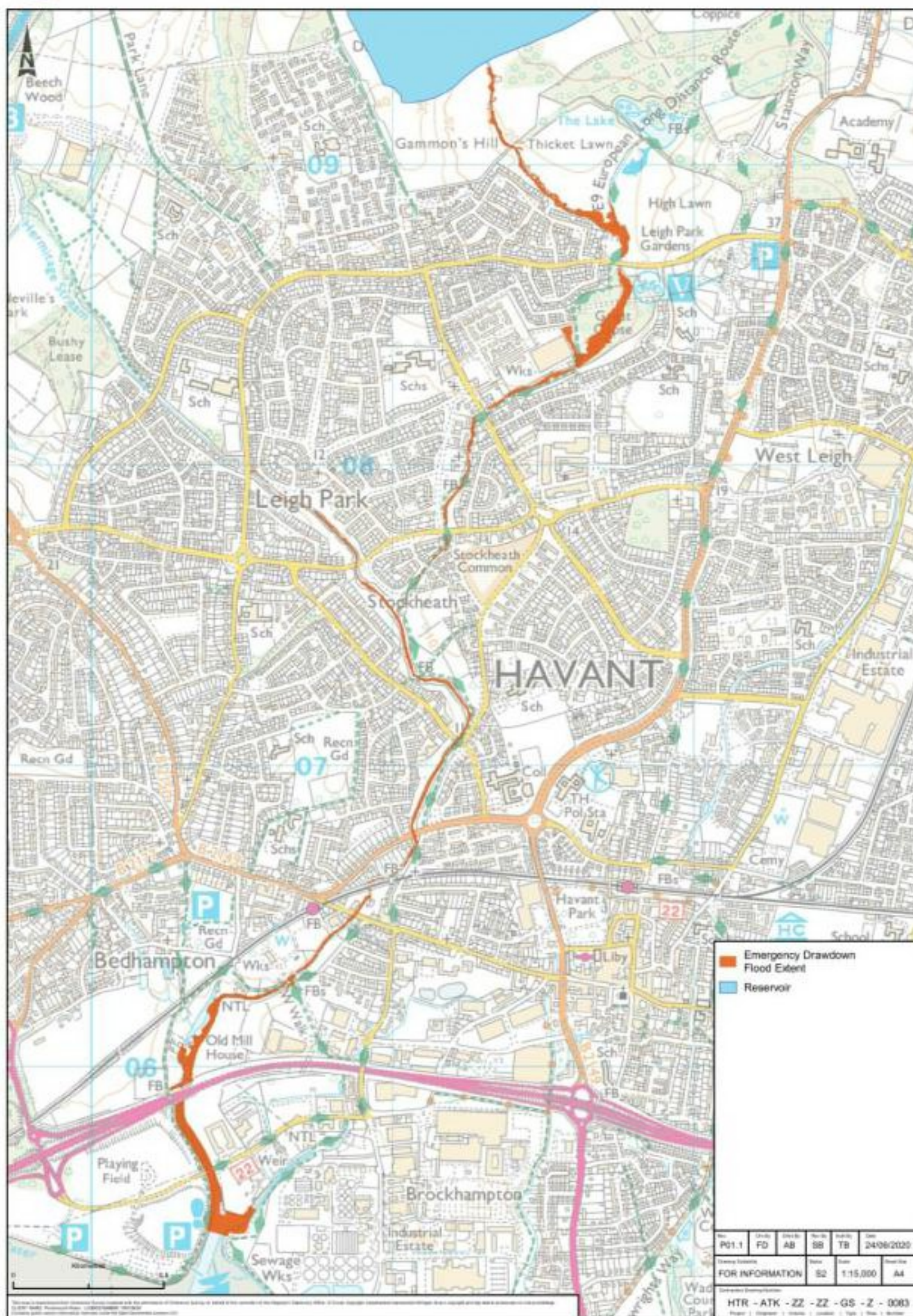


Figure 8-2 - Risk of flooding during an emergency drawdown

- 8.20. The peak flow within the Riders Lane Stream and the Hermitage Stream during an emergency drawdown scenario has been compared to baseline results and shown for each study section in Table 8-1. It should be noted that the emergency drawdown flows only affect study sections 3-6.
- 8.21. This identifies that, for the Hermitage Stream (study sections 4-6), the Emergency Drawdown flows are similar to those greater than a 3.33% (less than a 1 in 30) AP event currently.
- 8.22. For the Riders Lane Stream (study section 3), the flows have a lower comparable AP, and therefore represent a higher magnitude event. However, the modelling shows that this does not cause residential property flooding, and it is a residual risk, rather than an operational risk.

Table 8-1 - Peak flow during emergency drawdown scenario

Study Section		Peak flow (m ³ /s)	AP within baseline scenario
3	U/S	7.19	less than a 0.1% (>1 in 1000) AP
	D/S	7.19	between a 1% (>1 in 100) AP and a 0.1% (<1 in 1000) AP
4	U/S	4.28	greater than a 3.33% (<1 in 30) AP
	D/S	4.28	greater than a 3.33% (<1 in 30) AP
5	U/S	11.47	greater than a 3.33% (<1 in 30) AP
	D/S	9.29	greater than a 3.33% (<1 in 30) AP
6	U/S	9.29	greater than a 3.33% (<1 in 30) AP
	D/S	17.49	greater than a 3.33% (<1 in 30) AP

- 8.23. It was agreed in discussion with HCC⁴¹ that the impact of Emergency Drawdown on surface water drainage in Havant would be qualitatively considered by a comparison of flows with baseline conditions, shown on Table 8-1 above. It should be noted that under normal conditions, the operation of the reservoir is expected to have no impact on surface water drainage, and likely improve drainage due to reduced peak flows and water levels in the Riders Lane and Hermitage Streams.
- 8.24. In the unlikely event of an Emergency Drawdown at full capacity, coinciding with a storm event in Havant, the storm water network discharging to the Hermitage Stream would respond in a similar manner to its current performance as the Emergency Drawdown flows in the watercourse would be greater than a 3.33% (<1 in 30). Impact on surface water flood risk are therefore not anticipated.
- 8.25. The storm water network discharging to the Riders Lane Stream may be restricted however, as the ED flow in the channel is predicted to be between a 1% (1 in 100) and 0.1% (1 in 1000) AP. Although, the likelihood of Emergency Drawdown combining with a storm event is very low, there is a possibility of increased surface water flood risk in areas that discharge to the Riders Lane Stream. It is recommended that Portsmouth Water work with the LLFA and Southern Water to determine whether the potential increase in surface water flood risk would require a plan for mitigation during an Emergency Discharge scenario.
- 8.26. The Emergency Drawdown strategy was also simulated in conjunction with higher magnitude fluvial events, to determine the approximate fluvial event which, if coinciding with the requirement for emergency drawdown, would cause flooding. The model results show that for the Riders Lane Stream, the 50% (1 in 2) AP event would cause some minor inundation of up to 3 residential properties, and for the Hermitage Stream, the onset of flooding is the 10% (1 in 10) AP, with up to 3 residential properties also potentially affected. Map 24 in Appendix A shows the flood outline for the Emergency Drawdown paired with a 3.33% (1 in 30 year) AP fluvial event.

Access and egress conditions, flood warning and evacuation

- 8.27. There are no concerns with regards to access and egress during the Emergency Drawdown scenario, with no residential properties at risk. Although a small section of the industrial unit is

⁴¹ Meeting held on Microsoft Teams on 26/03/2020.

potentially at risk of shallow flooding, there is an alternative route for access and egress via Dunsbury Way.

- 8.28. As discussed, a breach scenario is never anticipated and would only result from an unexpected failure of the embankment, in combination with the inability to invoke the Emergency Drawdown strategy. As such, there would be very limited warning and ability for evacuation. The breach modelling demonstrates that the following key roads may remain out of the breach extent and thus be available to use for evacuation:
- Petersfield Road (B2149);
 - Western section of Purbrook Way, leading to the M3;
 - Park Lane, leading south-west and away from the breach extent; and,
 - Bedhampton Lane, leading west and away from the breach extent.
- 8.29. Safe access and egress to/from the Visitor Centre is provided by the Southern access road.
- 8.30. If there is a requirement to activate the Emergency Drawdown strategy, and it is unsafe to do so from the Control House, the valves can be operated remotely.
- 8.31. Piping failure of the reservoir is known to start slowly, and therefore if the embankment structural integrity was comprised, it is likely that there would be significant warning to the reservoir operator. This would not only allow the Emergency Drawdown strategy to be activated, reducing the hydraulic load on the embankment by drawing down the reservoir level (by a third over 6 days), but also give the reservoir operator time, if decided necessary, to evacuate areas of Havant at risk if breach occurred.
- 8.32. As per DEFRA guidance⁴², an Off-site Plan will need to be developed prior to reservoir operation, which provides a framework of procedures to coordinate a multi-agency response to the off-site consequences of a potential or actual breach. If a dam breach is deemed imminent, the operator raises an alert to the Police, who in turn activate the Plan and instigate the coordination of emergency services, local authorities and other key partners. The Local Authority is responsible for the maintenance of the off-site plan, which will in part be informed by the breach modelling undertaken as part of this FRA.

⁴² DEFRA, 2009. DEFRA Guidance on Reservoir Emergencies.

9. Summary and conclusions

- 9.1. This FRA has been undertaken in accordance with the NPPF to assess flood risk from all sources for the scheme. Based on the scheme design, the geography of the site, information made available or created by the Environment Agency and the FRA hydraulic modelling work, it is concluded that:
- Once operational, the scheme will reduce the existing risk of fluvial flooding along the Riders Lane Stream and further downstream on the Hermitage Stream after the confluence;
 - Elements of the scheme, namely the visitor centre, car park and access roads will increase the hardstanding area, and as such without adequate drainage design, the scheme has the potential to increase this surface water flood risk. The drainage design will be finalised during the detailed design stage and it is assumed that SuDS principles will be applied to ensure there is no increase in surface water flood risk up to and including the 1% (1 in 100) AP, including an allowance for climate change;
 - There is existing fluvial flood risk along the Riders Lane Stream and Hermitage Stream corridors, including where some of the scheme construction elements are proposed, such as at the temporary construction compounds for the pipeline and areas for culvert improvement works;
 - The risk of flooding from groundwater sources was assessed as low in the baseline assessment, and not considered further in the FRA;
 - The risk of flooding from tidal sources was defined as low, but with the potential to change as a result of the scheme. The FRA has demonstrated that there is no change to tidal flood risk as a result of scheme;
 - There is an increase in the residual risk of flooding to one industrial unit and 2-3 garden shed/out builds as a consequence of the Emergency Drawdown Strategy; and
 - In the very unlikely event of an embankment breach, there is an increased residual risk of flooding to the urban area of Havant downstream of the reservoir.

Fluvial flood risk

- 9.2. The assessment of fluvial flood risk along the Hermitage and Riders Lane Streams has been informed by a hydraulic model, developed for this study. Fluvial flood risk in general is limited along these watercourses, in particular the Hermitage Stream, which has substantial capacity due to conveyance improvement works in the 1970s.

Operation

- 9.3. Under normal operation of the reservoir, fluvial flood risk along the Riders Lane Stream, and the Hermitage Stream after their confluence, is reduced due to the attenuation effect of the reservoir. Peak flows on the Riders Lane Stream (Study Section 3) in the 1% (1 in 100 year) AP event, are reduced from 3.84 m³/s to 0.20 m³/s, with a corresponding reduction in floodplain extent of 87%. For the Hermitage Stream downstream of the confluence (Study Section 5), the attenuation effect of the reservoir is reduced with peak flows reducing from 23.26 m³/s to 19.76 m³/s and a corresponding reduction in floodplain extent of 25%.
- 9.4. The River Lavant headwaters will be intercepted by the embankment. This has the effect of reducing the catchment size by 38%, resulting in a decrease in both flow and level within the watercourse downstream and an overall reduction in flood risk.
- 9.5. There are no changes to fluvial flood risk predicted to the Park Lane Stream or the Hermitage Stream upstream of the confluence with the Riders Lane Stream.

Construction

- 9.6. During construction of the wetland retaining structure and the main reservoir embankment, there is a need to capture fluvial flows from the headwater streams and divert them downstream into the Riders Lane Stream. This is required to remove fluvial flows from the construction area and to manage the normal flow regime of the river. Flows from the headwater streams will be captured, using cofferdams and pipes/culverts will be used to route the flow into the Riders Lane Stream

downstream of the reservoir embankment. The culvert through the main embankment will be sized to allow forward flows, unchanged up to and including a 3.3% (1 in 30) AP. Flows during events in excess of this will be stored in the construction area, resulting in an attenuation effect and a reduction of downstream flows and flood risk.

- 9.7. The construction of the pipeline will cross two main rivers and one ordinary watercourse:
- Hermitage Stream (main river)
 - Riders Lane Stream (main river)
 - The Lake Stream (ordinary watercourse)
- 9.8. The construction method at this stage has not been confirmed, but for the purpose of this FRA and EIA, an open trench technique has been assumed. This will have the impact of the rivers being temporarily dammed in these locations for a short period of time while the pipe is laid. To ensure the risk of flooding is managed, a flume will be used on site to ensure the continued pass forward of flows up to and including the 1% (1 in 100) AP.
- 9.9. There are two temporary construction compounds and culvert improvement works proposed within parts of the modelled 1% (1 in 100) AP flood extent. The risk will need to be managed through careful planning of site set up and proposed works. It will be necessary to make the appointed contractors aware of this risk, such that a safe system of working can be set up, that materials are safe and that construction does not increase flood risk elsewhere. For all construction activity with 8 m of the main river and / or within the floodplain, a FRAP will be required to ensure the risks can be safely managed.

Surface water flood risk

- 9.10. During normal operation, the reservoir will not impact on the risk of surface water flood risk throughout the study area.
- 9.11. Though not quantitatively assessed, it is likely that the reduced peak flows and hence levels in the Riders Lane Stream and Hermitage Stream may improve the discharge of surface water flows from the urban areas of Havant.
- 9.12. There is the potential for the scheme to increase surface water flood risk through the addition of hardstanding areas at the reservoir embankment, visitor centre, car park and access roads. The design of the drainage will be confirmed during the detailed design stage and it is assumed that SuDS principles will be adopted to ensure there is no increase in surface water runoff rates up to and including the 1% (1 in 100) AP with climate change allowance.
- 9.13. The reservoir embankment drains into a toe-drain around the southern perimeter of the embankment. This will drain into both the Riders Lane Stream and River Lavant catchments. It is not expected that this will cause a substantial increase in runoff flows and volumes to either watercourse, or both will still have a decreased flood risk as a result of the development.

Residual risk

- 9.14. The operation of the reservoir has the potential to increase the residual risk of flooding to the urban area of Havant, caused by a breach in the embankment.
- 9.15. Whilst the risk of breach is considered extremely low, due to the design of the reservoir and the regular inspection and maintenance regime in line with the Reservoirs Act and Environment Agency guidance, the consequences would be significant, with much of the urban area inundated with deep, fast flowing water.
- 9.16. To further mitigate the unlikely possibility of embankment failure, an Emergency Drawdown strategy has been developed and modelled, in which an emergency high discharge rate is routed into the Riders Lane Stream and Hermitage Stream. This enables the reservoir level to be lowered by 0.9 m over 24 hours, in line with the Environment Agency Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning⁴³, reducing hydraulic load on the embankment to both reduce the

⁴³ Environment Agency, 2017. Guide to drawdown capacity for reservoir safety and emergency planning. [Online] Available at: <https://www.gov.uk/government/publications/guide-to-drawdown-capacity-for-reservoir-safety-and-emergency-planning> [Accessed June 2020].

risk of breach and potentially enable repair works. The emergency discharge will be routed into the Riders Lane Stream directly downstream of the embankment and into the Hermitage Stream via the same pipeline that is normally used to supply the reservoir with excess water from the Bedhampton Springs.

- 9.17. Modelling of the Emergency Discharge Strategy has demonstrated that with the suggested culvert improvements in Table 4-2, the emergency drawdown can be enacted without risk of flooding to residential properties, however, one non-residential property does become at risk of shallow inundation (0.1 - 0.2 m).

Recommendations

- 9.18. At this preliminary design stage, the following recommendations are made to ensure flood risk is minimised and the scheme remains compliant with the NPPF:
- The temporary construction compounds and culvert improvement works locations, and any other pipeline associated works within 8 m of the Environment Agency designated main rivers will require Flood Risk Activity Permits (FRAPs). The pipeline crossing of the ordinary watercourse 'The Lake Stream' will require Ordinary Watercourse Consent from Hampshire County Council.
 - At the locations where the pipeline crosses any watercourse, it is recommended where possible that a trenchless method is applied, for example directional drilling, to remove the requirement for temporary damming of the watercourse.
 - Fluvial flood risk at the temporary construction compounds should be minimised by directing construction compound use away from the floodplain where possible.
 - To mitigate fluvial flood risk to the wetland retaining structure during construction, an appropriately sized means of attenuating and passing forward flow to the embankment will need to be designed. The River Lavant headwaters and stream HS4 (both on the reservoir footprint) will also need to have a drainage design allowing the attenuation and passing forward of flow to the embankment.
 - The culvert replacements to accommodate construction access roads under the reservoir footprint must be sized to ensure no obstruction to flood flows up to and including the 1% (1 in 100) AP, to ensure no impact on fluvial flood risk.
 - Drainage strategies, in line with NPPF, the HBC and EDHC Local Plans and following guidance from the East Hampshire SFRA, will need to be developed to ensure surface water at the visitor centre, car park and access roads is adequately managed. The drainage strategies should include SuDS principles and will ensure that there is no increase in surface water runoff up to and including the 1% (1 in 100) AP with climate change allowance.
 - The visitor centre is built within the wetland, and is therefore at risk of flooding if the water level of the reservoir were to increase. The visitor centre is classified as "Less Vulnerable" development, and as such should be designed with a finished floor level above the 1% (1 in 100 year) AP event, plus an allowance for climate change (35% and 45% for the "central" and "higher central" estimates respectively). The 1% (1 in 100 year) AP event gives reservoir peak water levels of 39.65 m AOD and 39.67 m AOD for the "central" and "higher central" estimates. When an allowance for wave height is included (0.1 m, due to the limited fetch distance from the wetland retaining structure) and freeboard (0.3 m), the recommended finished floor level for the visitor centre is 40.1 m AOD.
 - Prior to the operation of the reservoir, it is recommended that Portsmouth Water develop On-site and Off-site Plans for Reservoir Dam Incidents, in line with DEFRA Guidance on Reservoir Emergencies. The On-site Plan aims to minimise the probability of dam failure in the event of an emergency or incident, and minimise the actual impact of a breach through measures which could reduce or delay the discharge through the breach. The Off-site Plan provides a framework of procedures to coordinate a multi-agency response to the off-site consequences of a potential or actual breach. If a dam breach is deemed imminent, the operator raises an alert to the Police, who in turn activate the Plan and instigate the coordination of emergency services, local authorities and other key partners. The Local Authority is responsible for the maintenance of the off-site plan, which will in part be informed by the breach modelling undertaken as part of this FRA.

Application of planning policy

- 9.19. The scheme is classified as "Essential Infrastructure", with permanent infrastructure in areas of Flood Zones 1 and 3, and temporary construction in areas of Flood Zone 3. It is considered that the scheme can satisfy the requirements of NPPF, is appropriate for the level of flood risk documented in this FRA, and will not increase flood risk at the site or to surrounding areas. Subject to the development of approved detailed drainage designs and obtaining consent for works in proximity to the watercourses, the scheme is considered appropriate and in line with the principles of NPPF.
- 9.20. In accordance with the NPPF, the visitor centre should be constructed with a finished floor level above the 1% (1 in 100) AP level, plus an allowance for freeboard, giving a minimum level of 40.1 mAOD. Safe access and egress during the same AP event is provided by the access road to the south.

Appendices to A15.2 Flood Risk Assessment

Appendix 15.2A: Mapping (provided in separate document due to size)

Drawing title	Drawing number
Figure A1 Baseline Flood Extents	HTR-ATK-ZZ-ZZ-GS-Z-0088
Figure A2 Baseline Flood Depth - 1 in 2 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0089
Figure A3 Baseline Flood Depth - 1 in 5 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0090
Figure A4 Baseline Flood Depth - 1 in 10 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0091
Figure A5 Baseline Flood Depth - 1 in 30 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0092
Figure A6 Baseline Flood Depth - 1 in 100 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0093
Figure A7 Baseline Flood Depth - 1 in 1000 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0094
Figure A8 Baseline Flood Depth - 1 in 100 year Annual Probability + 35% climate change	HTR-ATK-ZZ-ZZ-GS-Z-0095
Figure A9 Baseline Flood Depth - 1 in 100 year Annual Probability + 105% climate change	HTR-ATK-ZZ-ZZ-GS-Z-0096
Figure A10 Operational Flood Extents	HTR-ATK-ZZ-ZZ-GS-Z-0097
Figure A11 Operational Flood Depth - 1 in 2 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0098
Figure A12 Operational Flood Depth - 1 in 5 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0099
Figure A13 Operational Flood Depth - 1 in 10 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0100
Figure A14 Operational Flood Depth - 1 in 30 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0101
Figure A15 Operational Flood Depth - 1 in 100 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0102
Figure A16 Operational Flood Depth - 1 in 1000 year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0103
Figure A17 Operational Flood Depth - 1 in 100 year Annual Probability + 35% climate change	HTR-ATK-ZZ-ZZ-GS-Z-0104
Figure A18 Operational Flood Depth - 1 in 100 year Annual Probability + 105% climate change	HTR-ATK-ZZ-ZZ-GS-Z-0105
Figure A19 Baseline and Operational Flood Extents - 1 in 100 Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0106
Figure A20 Baselines and Operational Flood Extents - 1 in 100 year Annual Probability + 35% climate change	HTR-ATK-ZZ-ZZ-GS-Z-0107
Figure A21 Baseline and Operational Tidal Flood Extents - 1 in 1000 Year Annual Probability	HTR-ATK-ZZ-ZZ-GS-Z-0108
Figure A22 Baseline and Operational Tidal Flood Extents - 1 in 1000 Annual Probability + Climate Change	HTR-ATK-ZZ-ZZ-GS-Z-0109
Figure A23 Emergency Discharge + Q10 + 1 in 2 year Annual Probability Tidal Flood Extent	HTR-ATK-ZZ-ZZ-GS-Z-0110
Figure A24 Emergency Discharge + 1 in 30 Annual Probability Fluvial Event Flood Extent	HTR-ATK-ZZ-ZZ-GS-Z-0111
Figure A25 Breach Extent - Dry Day with Embankment Breach	HTR-ATK-ZZ-ZZ-GS-Z-0112
Figure A26 Breach Extent - Wet Day with and without Embankment Breach	HTR-ATK-ZZ-ZZ-GS-Z-0113

Appendix A15.2B Hydraulic Modelling Report

Havant Thicket Reservoir Project

Environment Statement Volume 4 - Appendices

Appendix A15.2 Water

Appendix B Hydraulic Modelling Report

Portsmouth Water Ltd.

04/09/20

Atkins Doc No.: HTR-ATK-ZZ-ZZ-RP-Z-0151

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Notice

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1. Introduction

Havant Thicket Winter Storage Reservoir (HTWSR), with planned storage of approximately 9.3 Mm³, will operate as a pumped storage reservoir utilising surplus water from the Havant & Bedhampton Spring source. The design of the reservoir includes provision for an emergency discharge facility to reduce the water level in line with the requirements of the Environment Agency (EA) Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning (Environment Agency, 2017).

A hydraulic model is required to inform the Flood Risk Assessment for the HTWSR, to provide an understanding of the baseline fluvial and tidal flood risk against which the impacts of the scheme can be compared. In addition, the FRA will assess the proposed strategy for Emergency Drawdown of the reservoir (using the FRA model), and the likely impact of an embankment breach (using a separate bespoke breach model).

The maximum draw-down rate from the reservoir in accordance with EA guidance has been calculated as 10.9m³/s (Preliminary Design Report, HTR-ATK-ZZ-ZZ-RP-Z-0122) to allow the reservoir top water level to be drained down by a depth of 900mm over a 24 hour period for a Category A reservoir.

It is proposed to accommodate the emergency discharge through outfall structures into the Riders Lane Stream at the toe of the dam structure, and into the Hermitage Stream via a pipe discharging in the vicinity of Purbrook Way, approximately 2km downstream from the reservoir site.

The total discharge rate of 10.9m³/s from the reservoir has been split to allow a 7.05m³/s discharge rate to the Riders Lane Stream directly downstream of the reservoir embankment, and 3.853/s to the Hermitage Stream. This baseline flow split is referred to within the report as "Option 1".

The lower emergency discharge rate of 10.9m³/s now used in the flood risk assessment is due to changes made to the EA guidance since 2009, where the draw down level over 24 hours for a Category A reservoir is now reduced to 900mm.

The use of the emergency drawdown facility may create a potential flood risk downstream of the proposed discharge structures. An InfoWorks ICM flood model of the Hermitage Stream and Riders Lane Stream has been developed to identify the potential impacts of the operation of the emergency drawdown on downstream flood risk. This model has been developed from a previous survey and the model developed for the Environment Agency (EA) Havant Flood Mapping Study (HFMS) (Atkins, 2008). The previous modelling also considered the Lavant Stream, which flows through the east of Havant but is not impacted by the proposed HTWSR.

The modelling carried out for this study has been verified against the 1% Annual Probability (AP) and 0.1% AP outputs from the previous model study and the December 2000 event previously used for calibration of the HFMS model, as set out in the project scope. Additional events have occurred since the HFMS model was developed, but there are no additional level or flood records to support further calibration.

The verified InfoWorks HTWSR ICM model has been used to determine if the capacity of the downstream channels is sufficient to accommodate the emergency discharge, considered in conjunction with an existing flow condition equivalent to the 10% exceedance flow (Q10) in each channel. To achieve this, the following has been undertaken:

- Modelling of Option 1 – 7.05 m³/s to the Riders Lane Stream and 3.85 m³/s to the Hermitage Stream;
- Modelling of a range of flows to identify the thresholds for flooding at key locations; and
- Testing of model sensitivity to roughness and headloss.

In all cases, the modelling has been undertaken using a 50% Annual Probability (AP) downstream tidal condition.

The analysis within this report has been based on the full emergency discharge flow occurring in the event of an emergency condition. During routine testing of the emergency discharge structures to the Riders Lane Stream and Hermitage Stream, pipework valves would be tested sequentially, and hence test flows would be limited to around 10% of the full capacity. Routine testing will be required twice annually to ensure that the pipework and valve system is in operational condition at all times.

Routine valve testing would only be planned to be undertaken at times of low stream flow (below Q50 and hence would not lead to any increase in flood risk.

2. Review of previous modelling

The Environment Agency provided copies of the reports and models relating to the Havant Flood Mapping Study (HFMS) [1] and the Warren Dam Discontinuance Study [2]. These have been reviewed to identify issues that are relevant to the modelling of the Riders Lane Stream and Hermitage Stream for the purposes of the Emergency Discharge Study. A site visit has also been undertaken, which identified other relevant issues. Based on the above a review document has been developed [3], and key points are summarised below (Figure 2-1 below shows key locations referred to).

- The Hermitage Stream catchment has a primarily clay geology, and a rapid response to rainfall events;
- The Lavant Stream catchment is a chalk, groundwater influenced catchment with a slow response to rainfall events and typically with high flows during the winter months;
- The most severe observed flood events on the Hermitage Stream have generally occurred in response to heavy summer rainfall;
- Flows in the Lavant Stream above a threshold (thought to be approximately $0.28 \text{ m}^3/\text{s}$) result in a transfer of flow to the Hermitage Stream via the Lavant Link Pipe;
- Recorded flows in the Hermitage Stream incorporate flow from the Lavant Link Pipe;
- Channels are heavily modified, particularly along the Hermitage Stream, with artificial alignments, numerous structures and outfalls from the urban areas;
- There is significant urban trash, with high blockage potential at a number of trash screens;
- There is a Flood Relief Culvert, which bypasses the lower section of the Hermitage Stream between the railway and A27;
- The downstream boundary is tidal, where the Hermitage Stream outflows into Langstone Harbour. Tidal penetration into the Hermitage Stream is limited by the cascade of weirs adjacent to the Portsmouth Water pumping station which have a top crest level of 4.26 m;
- The approach adopted for the rainfall-runoff modelling in the HFMS produced a good representation of flows in the Hermitage Stream catchment for the December 2000 event (the second largest flow on record);
- The HFMS used a combination of rainfall-runoff modelling (to define the distribution of inflows) and analysis of peak flows to determine the modelled design events;
- The maximum flow transfer (design capacity) of the Lavant Link Pipe is $3 \text{ m}^3/\text{s}$. The HFMS modelling identified that a peak flow of $2.2 \text{ m}^3/\text{s}$ was transferred in the 0.1% AP event in response to a peak flow in the Lavant Stream of approximately $6 \text{ m}^3/\text{s}$;
- Only the 1% AP and 0.1% AP events were modelled during the HFMS, primarily due to issues with model stability;
- Peak modelled flows for the 0.1% AP event were $32.8 \text{ m}^3/\text{s}$ at the Havant gauging station (New Road) with a peak flow upstream of the Hermitage Stream and Riders Lane Stream confluence of $20.7 \text{ m}^3/\text{s}$ in the Hermitage Stream and $9.6 \text{ m}^3/\text{s}$ in the Riders Lane Stream;
- Peak modelled flows for the 1% AP event were $24.2 \text{ m}^3/\text{s}$ at the Havant gauging station (New Road) with a peak flow upstream of the Hermitage/Riders confluence of $15.3 \text{ m}^3/\text{s}$ in the Hermitage Stream and $6.6 \text{ m}^3/\text{s}$ in the Riders Lane Stream;
- The Environment Agency has replaced screens on a number of culverts in the Hermitage Stream and Riders Lane Stream since the 2008 study, and discontinued the Warren Dam;
- Some minor changes to the urban extent (approximately 2% increase in urban extent by area) have occurred during the period since the HFMS was completed;
- The InfoWorks RS / InfoWorks CS / OpenMI approach applied in the HFMS is no longer supported and is effectively superseded by the InfoWorks ICM software; and
- The InfoWorks RS model incorporated a large number of modelling fixes to achieve model stability, including interpolated cross-sections and simplifications of structures.



3. Modelling of the Hermitage Stream and Riders Lane Stream

3.1. Model build

The model for the current study has been developed using the Innovyze InfoWorks ICM software (v9.0.2). This software was selected to provide compatibility with the InfoWorks RS/CS models developed for the HFMS, and because it has a significantly improved capability over InfoWorks RS for modelling the following features that are present in the Havant catchment:

- Steep channels;
- Bridges;
- Culverts;
- Screens; and
- Floodplain and surface flows (using a 2D unstructured mesh and incorporating buildings).

The upstream model boundaries are located at the bridge on Middle Park Way over the Hermitage Stream and at the site of the proposed reservoir on the Riders Lane Stream. The downstream boundary of the model is the point of outflow of the Hermitage Stream into Langstone Harbour. This area covers the full extent of the channels and floodplain potentially affected by the proposed emergency discharges from the proposed reservoir and is represented by the area shaded in blue in Figure 3-1.

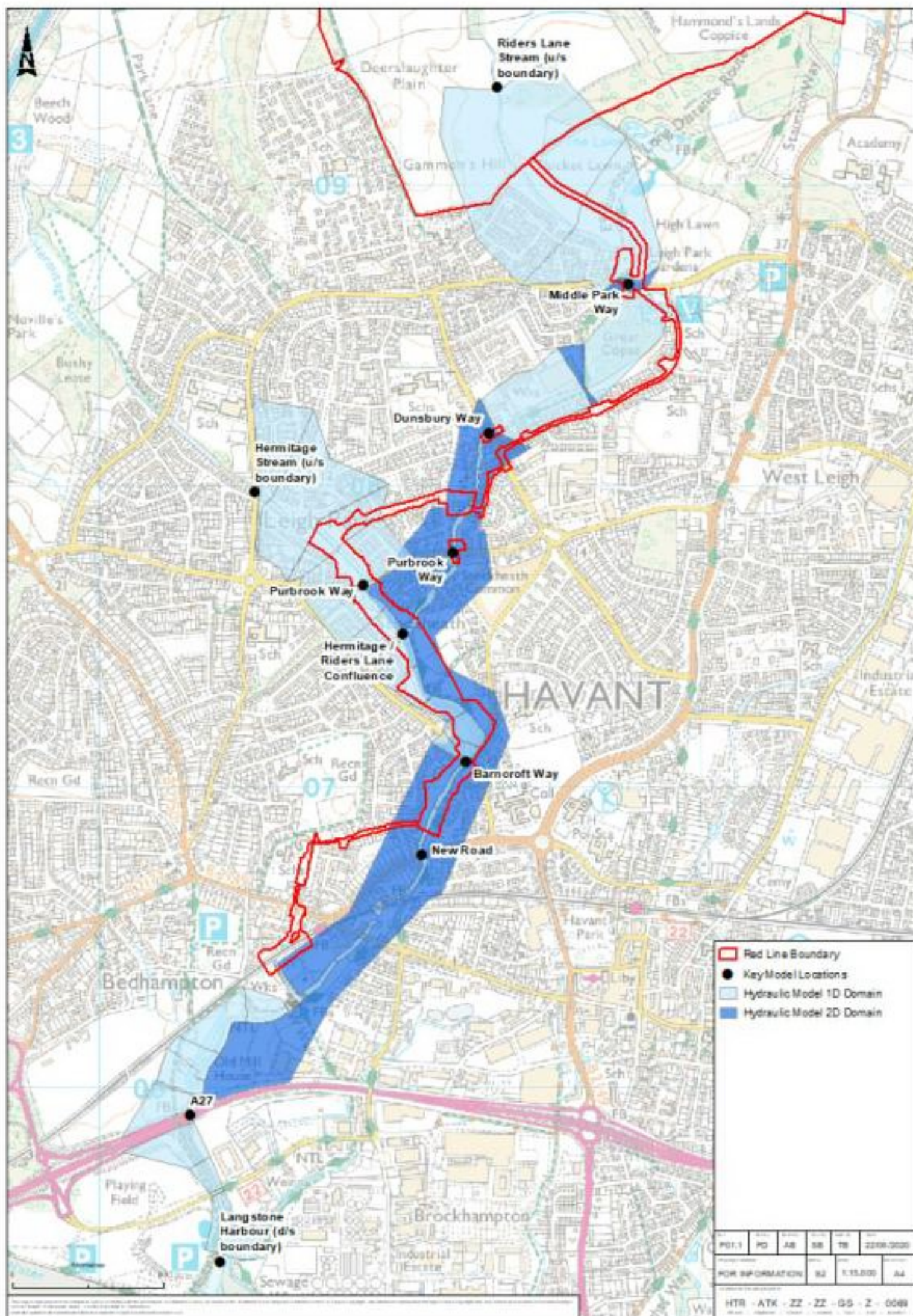


Figure 3-1 - InfoWorks ICM Model Extent

The model build process included the following:

- Import of channel sections, structures and associated parameters from the Infoworks RS model (HFMS 2008);
- Import of the Flood Relief Culvert from the InfoWorks CS model (HFMS 2008);

- Updating of cross section data where the floodplain is incorporated with the channel section using the latest Environment Agency 1m LiDAR (2016, accessed 2020);
- Removal of interpolated sections where these were no longer required for model stability;
- Review and remodelling of all bridge structures and culverts. A number of these structures were previously simplified in the InfoWorks RS model to orifice units due to issues with model stability. These have been fully represented in InfoWorks ICM either as bridges or conduits as appropriate;
- Trash screens have been explicitly included upstream of culverts where present. Data for these has been inferred from a combination of survey and site observations as a number of them have been renewed since the 2005 survey;
- The floodplain has been incorporated using a combination of extended channel sections and 2D mesh areas. This provides a more detailed representation of floodplain flows adjacent to, and away from, the channel, and of surface flows where culvert capacity is exceeded, and;
- Floodplain roughness in 2D modelled areas has been set using OS MasterMap data to inform the surface type and roughness value selection (shown on Table 3-1 below).
- Incorporate new survey data commissioned in response to the Early Start Studies, which identified areas that the model was potentially sensitive to parametrisation and inundation.

Table 3-1 - Manning's N Roughness Values

Land use Type	Final Manning's N
Manmade land	0.025
Manmade road/track/path	0.018
Rail track	0.035
Manmade roadside	0.018
Property land	0.050
Natural land - general	0.040
Water	0.016
Natural land - coniferous trees	0.055
Natural land - scrub	0.070
Natural land - Marsh	0.050
Natural land - Non-coniferous trees	0.055
Natural land - Orchard	0.045
Natural land - rough grass	0.040
Natural roadside	0.040
Natural road/track/path	0.035
Buildings	1.000

In addition to the above, specific issues were identified with the representation of the following structures in the InfoWorks RS model:

- Flood Relief Culvert – the location of the outfall was changed to downstream of the A27 in line with survey drawings and other supporting information. The downstream invert of the culvert was also adjusted from 2.14m AOD to -0.15m AOD in line with the survey data;
- Flood Relief Culvert Inlet Weir – the weir structure upstream of the culvert inlet was not included in the InfoWorks RS model. Dimensions for this have been inferred based on the 2005 survey data and site observations; and
- Dunsbury Way Culvert (Riders Lane Stream) – the culvert dimension has been reduced to a circular pipe of diameter 1.5m. This was previously modelled as a rectangular culvert of 3.74m x 1.07m, which is assumed to be an error in the original model build. A sediment depth of 0.44m has also been assumed in this culvert, based on the downstream survey section and photographic evidence.

3.2. InfoWorks ICM model verification

3.2.1. 1% Annual Probability and 0.1% Annual Probability events

The InfoWorks ICM model has been verified through comparison of the flood extents for the 1% AP and 0.1% AP flood events with the Environment Agency Flood Zone 2 and Flood Zone 3 Flood Maps for Planning. The fluvial Flood Zone 2 and Flood Zone 3 maps at Havant are assumed to be based on the outputs of the HFMS, as they are identical to the HFMS 1% AP and 0.1% AP extent mapping. The outlines differ in the tidal area, where the Flood Zone mapping shows tidal flood extents, and in the area of Warren Park where the flood mapping has been updated using the outputs of the Warren Park Discontinuance Study. Neither of these differences are significant in terms of the current study, as the model verification is being undertaken in relation to the 20% AP tidal event (as used in the HFMS) and Warren Park is upstream of the area of interest on the Hermitage Stream.

The 1% AP and 0.1% AP flows were extracted from the InfoWorks RS model at all points of inflow and were duplicated in the InfoWorks ICM model. For each event, the inflows to both models are effectively identical and any differences in levels and between the flood maps (outside of the tidal area) are attributable to differences in the representation of the channel, floodplain and structures between the models.

The 1% AP outlines are presented in Figures B1 to B8, and the 0.1% AP outlines in Figures B9 to B16, compared with the Flood Zone 3 and Flood Zone 2 flood outlines respectively.

Peak levels and flows upstream of the main structures are given in Table 3-2 for the 1% AP and

Table 3-3 for the 0.1% AP events. Figure 3-2 shows the locations referred to in the tables.

Table 3-2 - Comparison of peak levels and flows - 1% AP event

Location	Model section	Peak Level (mAOD)			Peak Flow (m³/s)	
		EA HFMS	HTWSR ICM	Difference (mm)	EA HFMS	HTWSR ICM
Riders Lane Stream						
Middle Park Way (upstream) (u/s)	RLS4326	16.37	16.67	300	5.7	5.7
High Lawn Way Footbridge (u/s)	RLS3828	15.10	15.51	410	6.2	6.2
Dunsbury Way (u/s)	RLS3480	12.88	14.72	1840	6.3	6.3
Riders Lane Footbridge (u/s)	RLS3329	12.25	12.21	-40	6.6	6.6
Purbrook Way (u/s)	RLS3036	10.46	10.46	0	6.6	6.7
Ditcham Crescent (u/s)	RLS2885	9.23	9.07	-160	6.7	6.6
Hermitage Stream						
Chalton Crescent Footbridge (u/s)	HS3179	9.68	9.55	-130	15.4	15.4
Purbrook Way (u/s)	HS2909	8.90	8.70	-200	15.4	15.4
Barncroft Way (u/s)	HS2135	7.67	7.49	-180	24.4	24.4
New Road Bridge (u/s)	HS1780	7.27	7.23	-40	24.4	24.0
Railway Bridge (u/s)	HS1509	6.59	6.76	170	24.6	24.7
West Street Bridge (u/s)	HS1404	6.25	6.20	-50	15.7	20.9
Meyrick Road Bridge (u/s)	HS1119	5.52	5.41	-110	15.7	20.9
A27 (u/s)	HS0515	2.82	2.78	-40	18.7	24.6
Langstone Harbour	HS0000	2.80	2.73	-70	29.8	32.5

Table 3-3 - Comparison of peak levels and flows - 0.1% AP event

Location	InfoWorks ICM Model section	Peak Level (mAOD)			Peak Flow (m³/s)	
		EA HFMS	ED Study	Difference (mm)	EA HFMS	ED Study
Riders Lane Stream						
Middle Park Way (upstream) (u/s)	RLS4326	17.38	17.98	600	8.3	8.2
High Lawn Way Footbridge (u/s)	RLS3828	15.29	15.77	480	9.1	8.9
Dunsbury Way (u/s)	RLS3480	13.31	14.94	1630	9.2	9.0
Riders Lane Footbridge (u/s)	RLS3329	12.43	12.36	-70	9.7	9.5
Purbrook Way (u/s)	RLS3036	10.58	10.58	0	9.7	9.7
Ditcham Crescent (u/s)	RLS2885	9.52	9.33	-190	9.7	9.6
Hermitage Stream						
Chalton Crescent Footbridge (u/s)	HS3179	9.92	9.75	-170	20.6	20.6
Purbrook Way (u/s)	HS2909	9.16	8.93	-230	20.7	20.7
Barncroft Way (u/s)	HS2135	8.00	7.80	-200	33.0	32.9
New Road Bridge (u/s)	HS1780	7.58	7.61	30	33.1	27.4
Railway Bridge (u/s)	HS1509	7.06	7.16	100	33.4	33.8
West Street Bridge (u/s)	HS1404	6.63	6.47	-160	23.6	29.4
Meyrick Road Bridge (u/s)	HS1119	5.74	5.53	-210	23.6	29.5
A27 (u/s)	HS0515	2.84	2.82	-20	27.9	34.3
Langstone Harbour	HS0000	2.80	2.73	-70	40.1	42.2

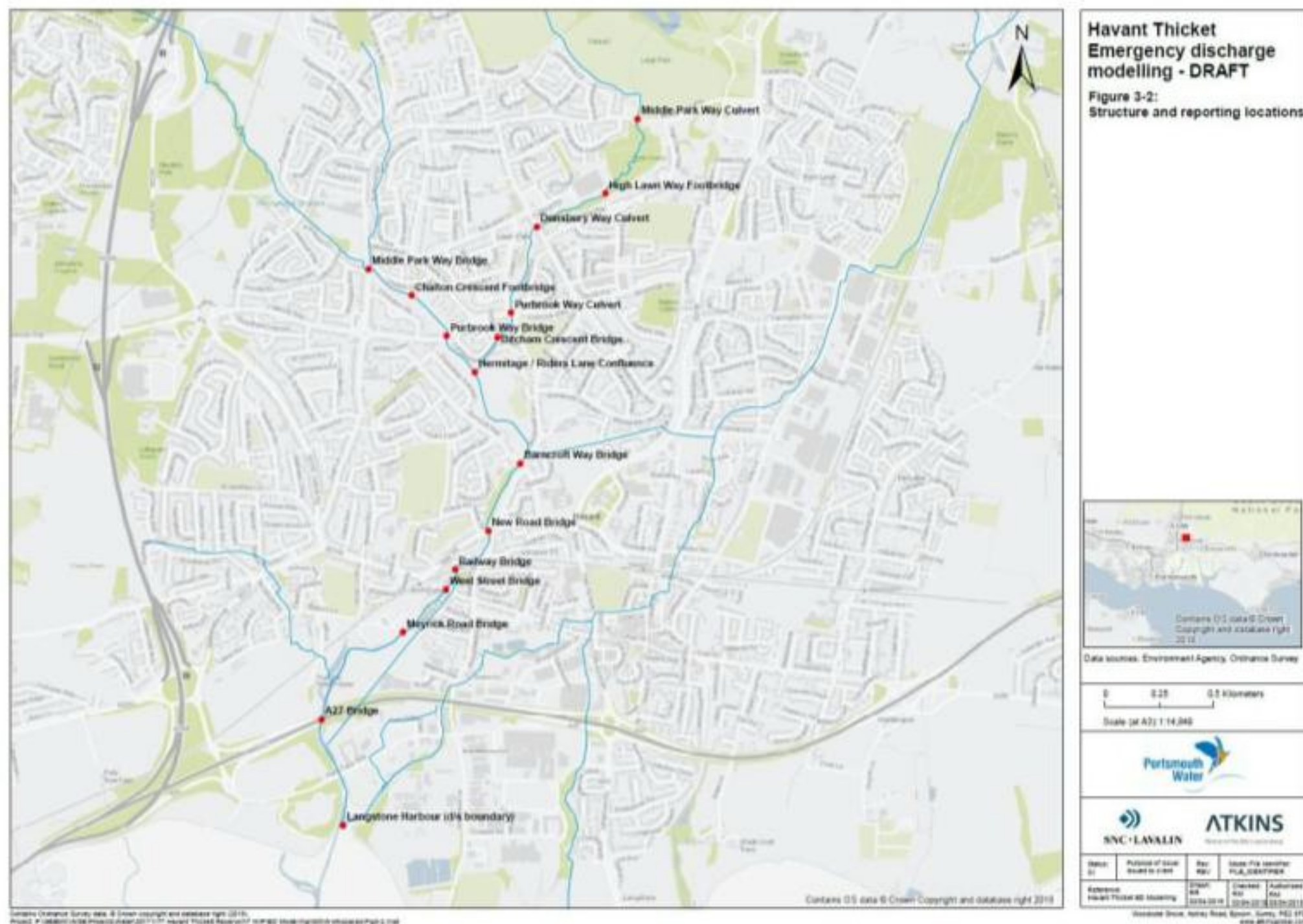


Figure 3-2 - Structure and reporting locations

The following observations on the comparison can be made:

- The results throughout the InfoWorks ICM model area are generally comparable in terms of extent with those from HFMS in areas where the water levels are not significantly affected by head losses at structures;
- Minor differences in extent may occur between the two models due to differences in the way the water surface and the ground model (LiDAR) are intersected between InfoWorks RS and InfoWorks ICM. Differences may also result from the use of the revised 2016 LiDAR data;
- The outlines on the Hermitage Stream in the area downstream of the railway bridge show some differences due to use of a combined 1D/2D approach and the representation of the channel banks in the InfoWorks ICM model;
- Flows through the Flood Relief Culvert in the InfoWorks ICM model are reduced as a result of the inclusion of the upstream weir structure, with flows correspondingly increased in the Hermitage Stream downstream of the inlet to the Flood Relief Culvert;
- The lower peak flow at New Road in the InfoWorks ICM model (27.4 m³/s compared to 33.1 m³/s) is the result of flow bypassing the bridge on the floodplain;
- The use of InfoWorks ICM "BRIDGE" structures generally gives a reduction in the headloss across most of the bridge structures in comparison with the orifice representation used in InfoWorks RS. The use of orifice units is likely to have resulted in an artificially high headloss across many of the bridges in the InfoWorks RS model as in reality the bridges provide very little constriction to flow until levels are above the bridge soffit;
- The use of a 2D approach in the InfoWorks ICM model allows detailed representation of surface flows around the culverts, most notably at Dunsbury Way, and in the reach downstream of the confluence of the Hermitage Stream and Riders Lane Stream;
- The most significant differences on the Riders Lane Stream are observed in the reach upstream of the Dunsbury Way culvert, where the reduction in size of the culvert results in an increase in water levels and flood extent upstream. The 1% AP peak level increases by 1.84 m and the 0.1% AP peak level by 1.63 m immediately upstream of this culvert;
- The 0.1% AP extent upstream of Middle Park Way is greater than in the InfoWorks RS modelling, with the level immediately upstream of the Middle Park Way culvert increasing by 0.61 m; and
- The Flood Zone 2 and Flood Zone 3 outlines in the area of the Middle Park Way bridge on the Hermitage Stream include the mapping developed from the Warren Dam Discontinuance Study which show overland flooding due to surcharging of the Warren Stream culvert. These flows are included in InfoWorks ICM model as a point inflow and the overland component is not modelled.

3.2.2. December 2000 flood event

The December 2000 flood event has also been simulated using the InfoWorks ICM model. As with the design events, the input flows were extracted from the InfoWorks RS model and directly applied at the relevant locations in the InfoWorks ICM model. This event is generally contained within the channel, so flood maps are not presented for this event in this report. Figure 3-3 shows levels at the Havant gauging station for this event, which show a close match with the previous modelling, although these show a systematic overprediction of level against the observed data throughout the event. This is most pronounced during the early part of the event, where the flows developed in the previous study are also overpredicted when compared with the observed flow, as shown in Figure 3-4.

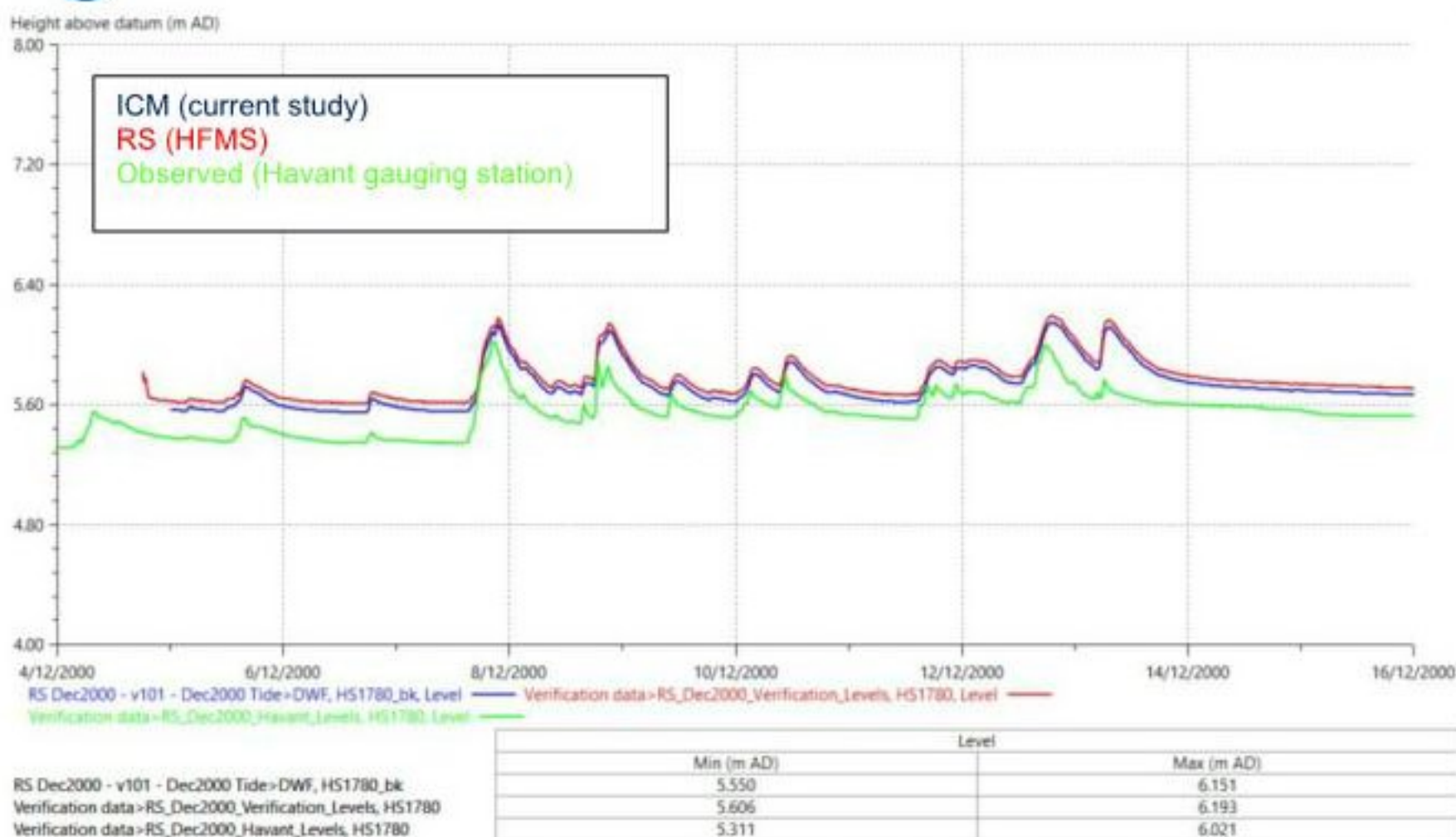


Figure 3-3 – Comparison of levels at Havant gauging station – December 2000

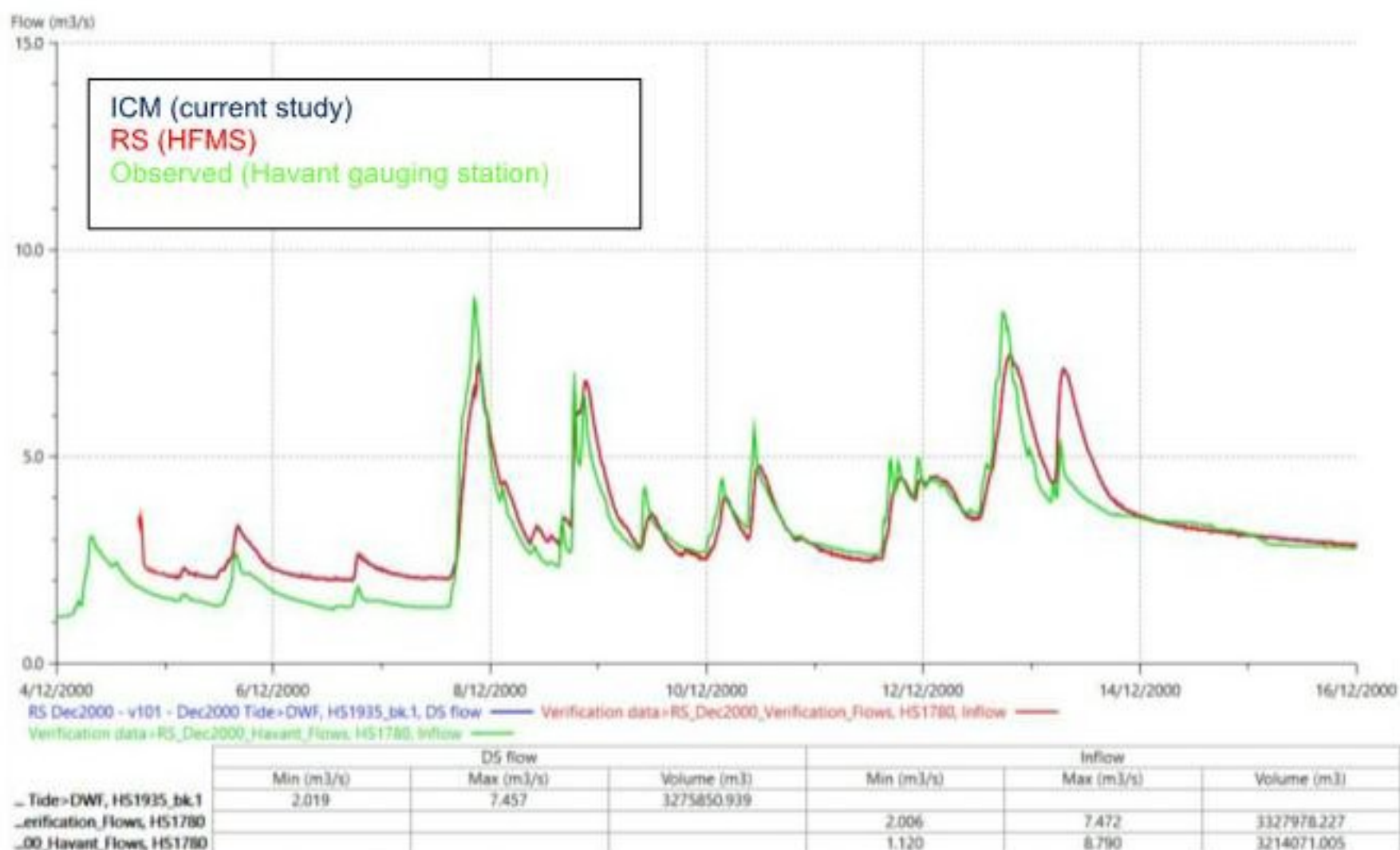


Figure 3-4 – Comparison of flows at Havant gauging station – December 2000

In general, the InfoWorks ICM model produces flood extents that are generally consistent with the earlier modelling for the 1% AP and 0.1% AP events. However, there are some significant differences related to changes in the modelling of the individual structures, most notably for the 0.1% AP event in the upper part of the Riders Lane Stream. A number of model simplifications have been removed, and banks specifically modelled in conjunction with the use of a 2D floodplain approach. As a result, the results of the InfoWorks ICM model can be expected to be more physically realistic.

The InfoWorks ICM model is therefore considered to verify well against the previous 1% AP and 0.1% AP modelling.

3.2.3. Hydrology sensitivity

As described in the FRA hydrological assessment (Appendix C of the main FRA), a test was undertaken to determine the sensitivity of the model to the hydrological approach. This was assessed using averaged Tp and CMAX parameters, calibrated using a ReFH2.3 model in which the 4 highest recent AMAX events were applied. This resulted in a significant reduction in Tp (from 4.8 to 1.4) and an increase to CMAX (From 227 to 277).

The default ReFH2.3 parameters were used for the design modelling as, when routed through the hydraulic model, the results matched very closely to the FEH Statistical Method flow estimates.

However, for comparison, the 1% AP flood extent is plotted on Figure A.1 in Appendix A to demonstrate the sensitivity to changes in the hydrological approach. The hydraulic model shows limited sensitivity to the variation in hydrological approach, with the flood extent through the urban areas of Havant largely identical.

3.3. Emergency discharge flow modelling

3.3.1. Base flow condition

The proposed emergency discharges have been modelled in combination with a range of flow conditions derived from the flow exceedance curve for the Hermitage Stream at Havant gauging station [4]. The range of Q10, Q30, Q50, Q70 and Q90 flows were used, representing the flows that are exceeded at the gauging station for 10%, 30%, 50%, 70% and 90% of the time, based on mean daily flow data. As the Q10 represents the largest of these flows, it has been accepted by the EA as sufficiently precautionary for the purposes of the emergency discharge modelling (email from Laura Lax, EA, 08/03/2019). Hence at this stage only the results for the emergency discharge flows in combination with the Q10 flow are presented in this report.

The Q10 flow at Havant gauging station is 0.57 m³/s and represents combined flow from both the Hermitage Stream and Riders Lane catchments. Flows at Havant may also be augmented by a transfer into the catchment from the Lavant Link Pipe, which discharges from the Lavant into the Hermitage Stream upstream of the Havant gauging station. However, it is difficult to quantify any component from the Lavant, given that this is dependent solely on flow conditions in the Lavant and not those in the Hermitage Stream.

Flows in the Lavant show a distinctly different response to those in the Hermitage Stream catchment due to a significant groundwater component. As the proportion of flow coming from the Lavant Link Pipe is unknown, a very precautionary approach has been adopted where the total flow is assumed to originate in the Hermitage Stream and Riders Lane Stream catchments. This assumption leads to a higher estimate of the Q10 flow at the upstream model boundaries with no flow contribution being assumed from the Lavant Link Pipe. The total flow at Havant remains unchanged.

The exceedance flows at Havant have been proportioned on the basis of the areas of the Hermitage and Riders Lane catchments to their confluence. As the catchments are homogenous, splitting the flow on the basis of catchment area is assumed to be appropriate. The calculated flow is also applied at the upstream end of each modelled reach, which again is precautionary as it would be expected that the total flow in each channel upstream of the confluence would be less than this.

The applied Q10 flows are 0.43 m³/s at the upstream end of the Hermitage Stream, and 0.14 m³/s at the upstream end of the Riders Lane Stream, giving a total of 0.57 m³/s at the Havant gauging station.

The hydraulic model was reviewed externally (July 2019) to verify its use for assessing the impacts of emergency draw down, and future use in supporting a FRA. The review noted that the model contained changes to the representation of structures as noted in Section 3.1, and that the hydrology was the same used in the 2008 study. The changes in structure representation are improvements on the initial model making use of the additional functionality of the newer modelling software, and the 2008 hydrology was used in order to verify the model against previous results, but have since been updated for use within this FRA. Otherwise, the model review demonstrated that the model is appropriate for use, and the consideration of the recommendations can be found in Appendix D.

3.3.2. Emergency discharge condition

It is proposed to accommodate the emergency discharge through outfall structures into the Riders Lane Stream at the toe of the dam structure, and into the Hermitage Stream via a pipe discharging in the vicinity of Purbrook Way, approximately 2km downstream from the reservoir site. The total discharge rate of 10.9m³/s from the reservoir has been split to allow a 7.05m³/s discharge rate to the Riders Lane Stream directly downstream of the reservoir embankment, and 3.85m³/s to the Hermitage Stream, discharged approximately 2km downstream from the reservoir site. This baseline flow split is referred to within the report as "Option 1".

3.3.3. Sensitivity testing

Tests have been undertaken to review the sensitivity of the InfoWorks ICM model to increases in either roughness or headloss coefficient. This approach was adopted as levels in the model are generally seen to be either conveyance or structure controlled. The following tests were undertaken, in conjunction with the Q10 flow:

- All roughness values increased by 20%; and
- All headloss coefficients increased by 20%.

For the roughness tests, the modelled roughness was adjusted for all channels, floodplain, bridges and culverts. For the Riders Lane Stream and Hermitage Stream upstream of the confluence, these values will approach the expected maximum roughness values that may occur due to in-channel and floodplain vegetation. For the reaches of the Hermitage Stream with concrete channel then these values are likely to exceed the maximum expected roughness. The roughness sensitivity test indicates some sensitivity to roughness at key locations, discussed in Section 3.3.6 below.

For headloss sensitivity, headloss coefficients for all bridges and culverts were increased by 20%, for both the inlet and outlet losses. The headloss sensitivity testing indicated that no structure showed an increase in water level of over 10 mm from the baseline and therefore these results are not presented in detail in this report.

3.3.4. Estimated onset of flooding

As advised by the Environment Agency, the Emergency Discharge should be paired with a Q10 flow and 50% AP downstream tidal condition. With these conditions, the FRA demonstrates that Emergency Drawdown can be safely achieved, causing no inundation to residential property.

It is important to understand, however, the fluvial event which, when paired with the Emergency Discharge, causes inundation to residential property.

Figure A3 in Appendix A (of Appendix B Flood modelling report in Appendix A15.2 of ES Volume 4) shows the flooding outlines for the Emergency Discharge when paired with fluvial events ranging from the 50% AP to the 1% AP.

The modelling shows that along the Riders Lane Stream, there is the potential for inundation of a small number of gardens and properties near Ditcham Crescent, in the event of Emergency Discharge coinciding with the 50% AP event. When paired with the 1% AP event, there is some limited additional inundation to gardens downstream of Dunsbury Way, but otherwise no additional risk to property.

Along the Hermitage Stream, residential buildings are not affected until the Emergency Discharge event is paired with the 10% AP event, near New Road. There is more extensive inundation to the same properties when paired with the 1% AP event, but otherwise no additional risk to property from the larger fluvial event.

3.3.5. Interaction with tidal boundary

As advised by the Environment Agency, the interaction of the Emergency Discharge flows with the tidal boundary should be considered, through modelling the drawdown rate with extreme tidal event boundaries and including climate change.

The drawdown scenario was paired with the 0.1% AP downstream tidal condition, plus the Upper End allowance for climate change to 2125, considered the most extreme tidal boundary developed in this study. In this scenario, there is no additional flood risk than shown in either the ED event + 50% AP tidal, nor the 2125 Upper End 0.1% AP tidal event. This demonstrates that the ED event

3.3.6. InfoWorks ICM model results and outputs

The results of the modelling are presented as flood extent maps and tabulated levels at key locations.

Table 3-4 gives the predicted levels at various key locations. The outputs of the sensitivity testing for roughness are also included.

Mapping of the flood extents for the proposed emergency discharge for Option 1 and with the roughness sensitivity scenario is provided in Figure A.2 in Appendix A, with Figure A.3 showing the Emergency Discharge flood extents when paired with high flow events.

Table 3-4 - Emergency discharge Option 1 and sensitivity at key locations

Location	Water level (mOD)	
	Option 1	
	Base	Sensitivity Manning's Roughness+20%
Riders Lane Stream		
Middle Park Way (u/s)	16.21	16.29
High Lawn Way Footbridge (u/s)	15.46	15.46
Dunsbury Way (u/s)	13.17	13.34
Riders Lane Footbridge (u/s)	12.25	12.28
Purbrook Way (u/s)	10.25	10.41
Ditcham Crescent (u/s)	9.10	9.21
Hermitage Stream		
Chalton Crescent Footbridge (u/s)	8.97	9.04
Purbrook Way (u/s)	8.14	8.19
Barncroft Way (u/s)	6.48	6.59
New Road Bridge (u/s)	6.22	6.32
Railway Bridge (u/s)	6.01	6.07
West Street Bridge (u/s)	5.67	5.75
Meyrick Road Bridge (u/s)	5.11	5.13
Langstone Harbour	2.73	2.73

The results indicate the following:

- No flooding of property or infrastructure is indicated on the Hermitage Stream upstream of the confluence with the Riders Lane Stream;
- With increased roughness, there is the potential for shallow (< 0.1m depth) inundation of up to two residential properties near Purbrook Way/Ditcham Crescent. Residential flooding is not indicated anywhere for the baseline Emergency Discharge modelling. Minor inundation at the industrial unit near High Lawn Way is indicated in the baseline modelling;
- No flooding of property is indicated on the Hermitage Stream downstream of the confluence with the Riders Lane Stream, including when the roughness sensitivity test is applied.

The results include the incorporation of additional survey of the culvert and banktop at Purbrook Way, to improve model confidence at this location owing to the sensitivity to roughness.

4. Conclusions

The following conclusions are made based on the current model outputs:

1. The previous model has been updated to InfoWorks ICM, and the representation of floodplain areas and structures improved and updated;
2. The ICM has been verified through comparison with the previous 1% AP and 0.1% AP event outputs, as captured in the Environment Agency Zone 2 and Zone 3 Flood Map for Planning. The comparisons indicate that the ICM model produces results that are comparable with the previous modelling, though significant increases in flood extent are observed upstream of some of the structures;
3. No flooding of residential property or infrastructure is indicated on the Hermitage Stream upstream of the confluence with the Riders Lane Stream under the Option 1 Emergency Discharge Condition;
4. No flooding of property is indicated on the Hermitage Stream downstream of the confluence with the Riders Lane Stream;
5. No flooding of residential property is indicated at any location on the Riders Lane Stream under Option 1 baseline modelling. Minor external inundation is indicated for the industrial unit near High Lawn Way in the event of Emergency Discharge;
6. The model results show sensitivity to roughness in the areas around Purbrook Way/Ditcham Crescent on the Riders Lane Stream, indicating potential shallow inundation of gardens and properties at this location under the increased roughness scenario;

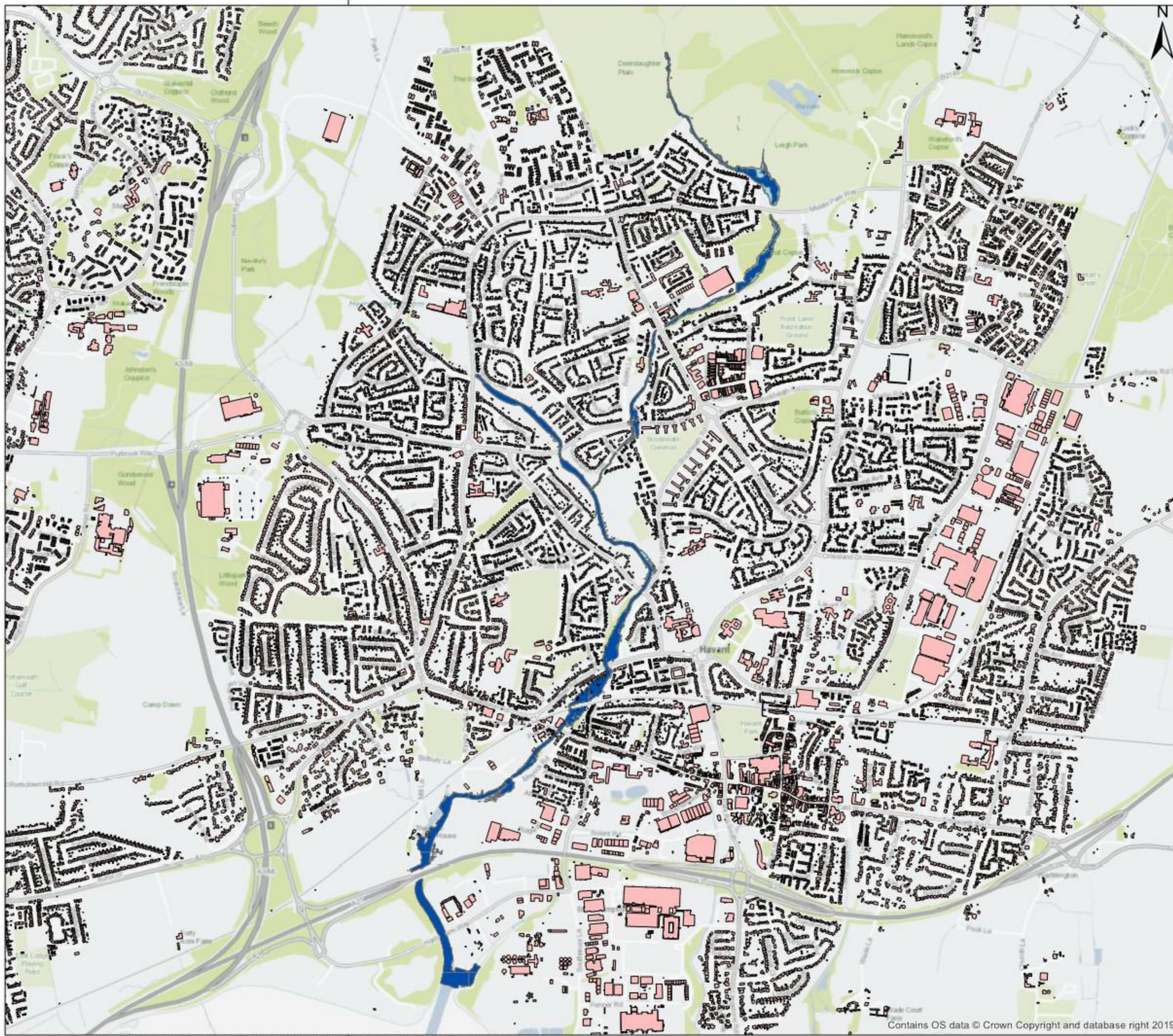
5. References

- [1] Atkins, "Havant Flood Mapping Study," Environment Agency, 2008.
- [2] Atkins, "Havant Modelling Update - Modelling Technical Note," Environment Agency, 2010.
- [3] Atkins, "Review of Environment Agency Havant Model," 2019.
- [4] Centre for Ecology and Hydrology, "National River Flow Archive," 2019. [Online].
- [5] Environment Agency, "Guide to Drawdown Capacity for Reservoir Safety and Emergency Planning," 2017.
- [6] Atkins, "Havant Thicket Winter Storage Reservoir - Emergency Discharge Study," 2019.
- [7] L. Lax, "Email communication," 2019.

Appendices to Appendix A15.2 Hydraulic Modelling Report

Appendix A15.2B Hydraulic Modelling Report

Appendix A Flood extent mapping



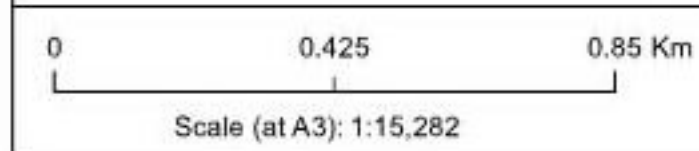
Havant Thicket Emergency discharge modelling

A.1 Hydrology sensitivity test for the 1% AP event

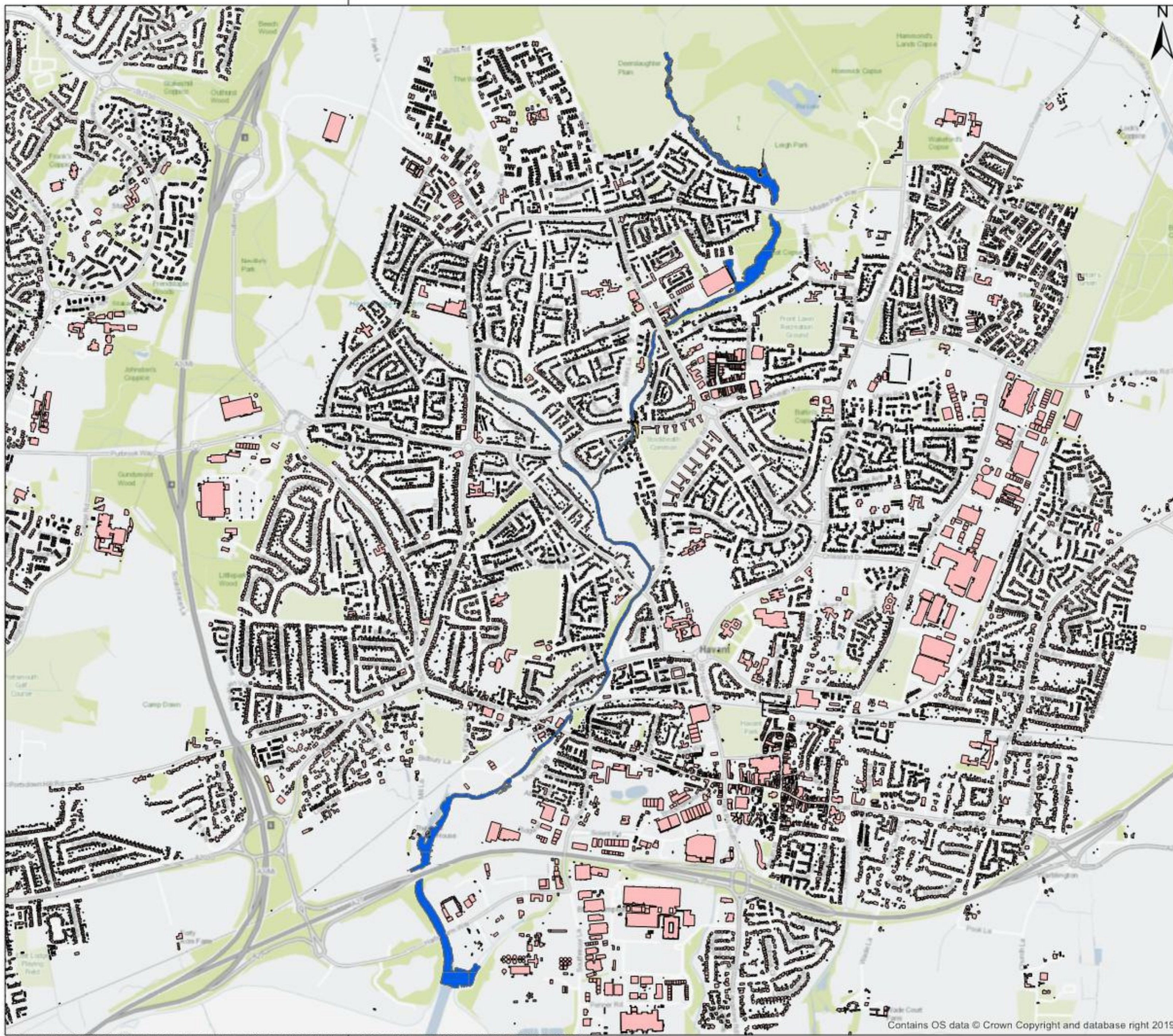
- Key**
- OS Master Map - Buildings
 - Baseline - 1% AP - default ReFH2.3 parameters
 - Hydrology sensitivity - 1% AP - calibrated Tp and CMAX



Data sources: Environment Agency, Natural England, English Heritage, Ordnance Survey



Status: S1	Purpose of issue: Issue to client	Rev: REV	Model File Identifier: FILE_IDENTIFIER
Reference: Havant Thicket Flood Modelling	Drawn: SB 30/06/2020	Checked: AB 30/06/2020	Authorised: 30/06/2020



Havant Thicket Emergency discharge modelling

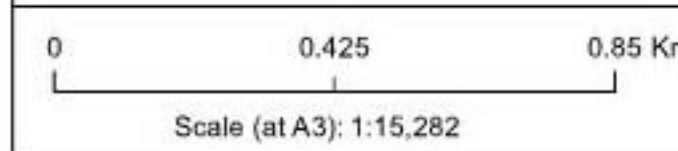
A.2 Riders Lane Stream - Flood Extent for Option 1 + Q10

- Key**
- OS Master Map - Buildings
 - Emergency Discharge + Q10
 - Sensitivity - n+20%

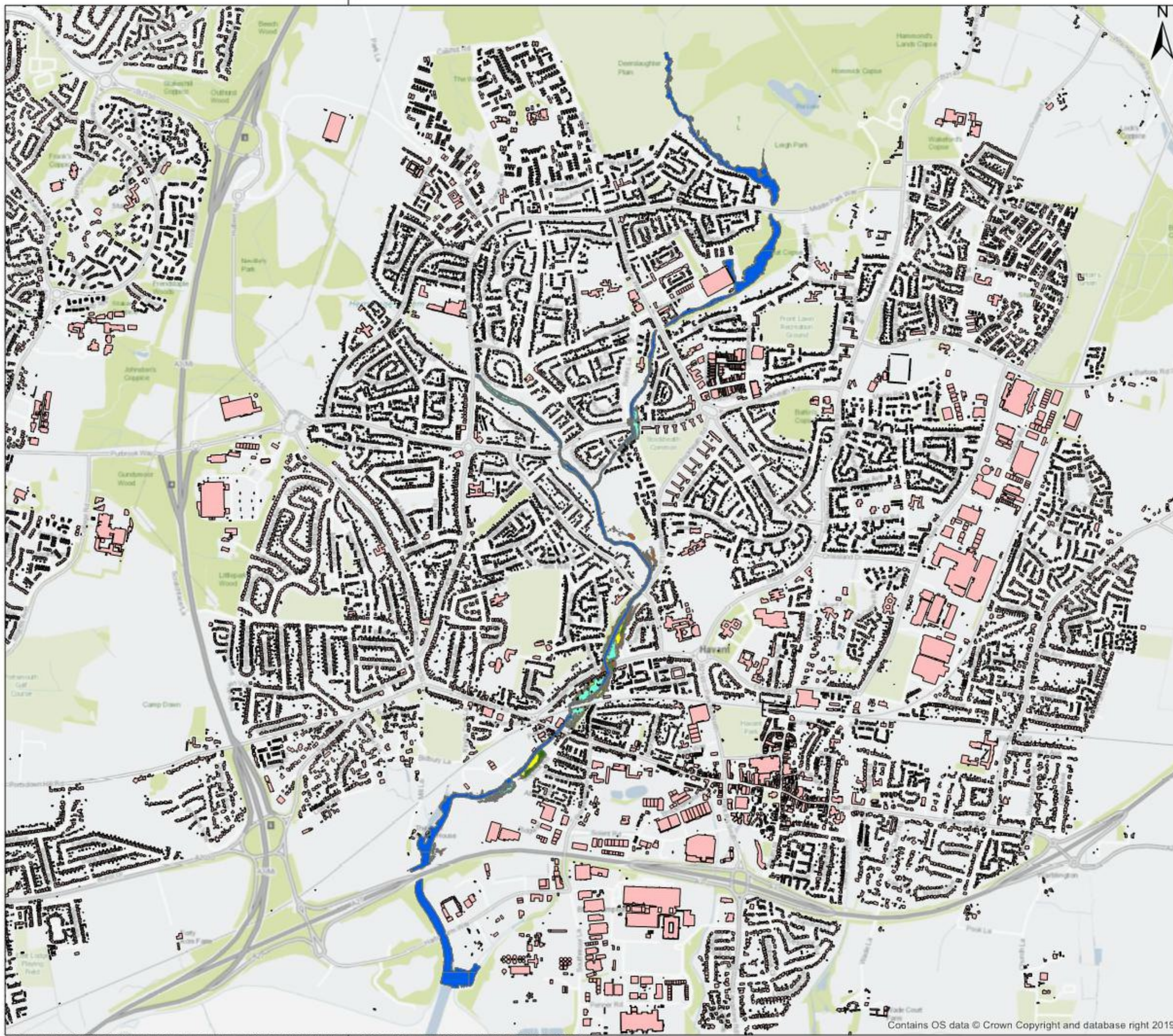
Note:
Emergency Discharge Option 1 -
Riders Lane Stream 7.05m³/s &
Hermitage Stream 3.85m³/s



Data sources: Environment Agency, Natural England,
English Heritage, Ordnance Survey



Status: S1	Purpose of issue: Issue to client	Rev: REV	Model File Identifier: FILE_IDENTIFIER
Reference: Havant Thicket Flood Modelling	Drawn: SB 30/06/2020	Checked: AB 30/06/2020	Authorised: 30/06/2020



Havant Thicket Emergency discharge modelling

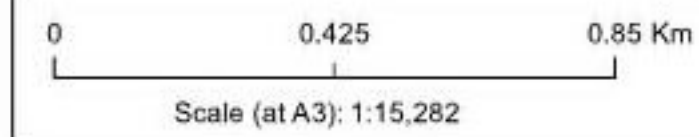
A.3 Riders Lane Stream - Flood Extents for Option 1 + Fluvial events

- Key**
- OS Master Map - Buildings
 - Emergency Discharge + Q10
 - Emergency Discharge + 50% AP
 - Emergency Discharge + 10% AP
 - Emergency Discharge + 3.33% AP
 - Emergency Discharge + 1% AP

Note:
Emergency Discharge Option 1 -
Riders Lane Stream 7.05m3/s &
Hermitage Stream 3.85m3/s



Data sources: Environment Agency, Natural England,
English Heritage, Ordnance Survey



Status: S1	Purpose of issue: Issue to client	Rev: REV	Model File Identifier: FILE_IDENTIFIER
Reference: Havant Thicket Flood Modelling	Drawn: SB 30/06/2020	Checked: AB 30/06/2020	Authorised: 30/06/2020

Appendix A15.2B Hydraulic Modelling Report

Appendix B Model review response 2019

Technical Note

Project:	Havant Thicket Emergency Discharge Study		
Subject:	Model review response		
Author:	Sam Bray		
Date:	11/09/2019	Project No.:	5165942

Document history

Revision	Purpose description	Origin-ated	Checked	Reviewed	Author-ised	Date
Rev 1.0	Issue for client comment	SB	WR	JL	JL	18/09/2019

Client signoff

Client	Portsmouth Water
Project	Havant Thicket Emergency Discharge Study
Project No.	5165942
Client signature / date	

Havant Thicket Emergency Discharge Study – Review Response

This technical note considers the Major comments resulting from the Jeremy Benn Associates (JBA) review of the Havant Thicket Reservoir Model hydrology and hydraulics, detailing the likely impacts and the proposed actions to address the queries.

JBA's review was conducted on 12/07/2019 and assessed the InfoWorks ICM v9.0.2 Havant Thicket Winter Storage Reservoir (HTWSR) emergency discharge model, used to assess the potential for downstream flooding in the event of operation of the drawdown facility.

The review identified one Major issue related to the hydrology, and one related to the hydraulic model. These are addressed in turn as follows.

Hydrology

JBA Review Comment:

"The 2019 study uses hydrology from 2008, which is based on now superseded methods and incomplete hydrometric records. It is recommended that the hydrology is reworked using latest best practice methods and hydrometric data. Given the difference in focus between the 2008 work and the current study, a simpler approach to flow estimation maybe sufficient, although due consideration should still be given to the flow transfer via the Lavant Link Pipe."

Atkins Comment Response:

It is correct that the 2019 study uses hydrology from 2008, which is now out of date and may provide different inflows in comparison to the latest methods with up to date hydrometric records.

Both the 1% AP (Annual Probability) and 0.1% AP events were used as part of an exercise in the 2019 study to verify the updated hydraulic model against the Environment Agency Flood Map for Planning. The fluvial flood zones at Havant were taken from the 2008 study, and as such the same inflows were applied in the verification exercise, in order that any differences in the results could be attributed to the amendments made to the model.

A rework of the hydrology will be undertaken as part of the FRA and is expected to change the inflows to the model and thus the flood outlines. Initial (estimate only) hydrological assessment indicates that the change in inflows is likely to be within 10% for the 1% and 0.1% APs, to be confirmed through a full update for the FRA.

It is important to note that although the hydrology should be updated as part of the FRA to assess the design events (including the 1% and 0.1% APs described), the Emergency Discharge test uses 2019 estimates of the Q10 baseflow at the Havant gauging station, and is not affected by the out of date hydrology used to verify the model.

Hydraulics

JBA Review Comment:

"The model conversion and set-up is sound for the assessment of Flood Zones 2 and 3, however there is some inconsistency between the original imported model and the updated model that should be clarified (e.g. bridge dimension)."

Based on the reporting and model results provided the inflows require updating to assess the impact of the discharge from the proposed reservoir and clarification is required on the use of the use of the +50% roughness scenario for the impact assessment."

Atkins Comment Response:

It is correct that the structures within the updated model do not in all cases match the original imported model. The original InfoWorks RS model included some simplifications of structures in order to achieve model stability. These structures have been fully represented in the updated model, as either bridges or conduits as appropriate, making use of the improved hydraulic structure options in ICM. Specific structure issues identified and corrected include the following:

- Flood Relief Culvert – the location of the outfall was changed to downstream of the A27 in line with survey drawings and other supporting information. The downstream invert of the culvert was also adjusted from 2.14m AOD to -0.15m AOD in line with the survey data;
- Flood Relief Culvert Inlet Weir – the weir structure upstream of the culvert inlet was not included in the InfoWorks RS model. Dimensions for this have been inferred based on the 2005 survey data and site observations; and
- Dunsbury Way Culvert (Riders Lane Stream) – the culvert dimension has been reduced to a circular pipe of diameter 1.5m. This was previously modelled as a rectangular culvert of 3.74m x 1.07m, which is assumed to be an error in the original model build. A sediment depth of 0.44m has also been assumed in this culvert, based on the downstream survey section and photographic evidence (shown in the Model Build Report, Appendix E of the Emergency Drawdown Modelling Report).

For clarity, these specific improvements and justifications are included within the Flood Modelling Report. It is therefore considered that these alterations to structure dimensions are a more realistic representation.

The review comment relating to the inflows is expanded on in the review document as follows:

"The inflow for the reservoir discharge test scenario has been applied as single lumped flows at the upper end of the 2 watercourses; the lateral inflows haven't been included."

It is correct that for the Emergency Discharge test, single flows were applied to the top of the Hermitage Stream and Riders Lane Stream, representing a sustained, high-flow release from the reservoir, coupled with the Q10 baseline flow, as agreed with the Environment Agency on 08/03/2019. The Q10 baseflow was chosen as it represents the largest of the base flows and taken from the Havant gauging station thus capturing flow from both the Hermitage Stream and Riders Lane catchments.

Q10 flows at NRFA (National River Flow Archive) Station 42017, Hermitage Stream at Havant, are 0.568m³/s (accessed 12/09/2019, and the same value used in the Emergency Discharge Test). This was apportioned to the Hermitage and Riders Lane Streams as flows of 0.43m³/s and 0.14m³/s respectively, in line with their relative catchment sizes.

These were added to the emergency discharge flows from the reservoir, calculated to be 11.1m³/s in total, and would in reality be applied as a lumped inflow at the top of each Stream. This total was apportioned to the Hermitage and Riders Lane Streams as flows of 8.6m³/s and 2.5m³/s respectively.

The combined Q10 and Emergency Discharge flows were then applied as constant inflows at the top of the model for a time period of 1 week, alongside a tidal boundary including a 50% AP level to ensure any interaction with or limitation on discharge from a high tidal boundary were captured.

Although the baseflows could be applied laterally to the Streams as opposed to lumped with the Emergency Discharge flows, the impact of such a change would not affect the results, given that the baseflow only represents a small (5%) proportion of the inflows applied at the upstream boundaries.

The final review comment suggests clarification is required on the use of the +50% roughness sensitivity scenario. The +50% roughness scenario was used as a "book-end" in the flood modelling to identify areas of the model that may be sensitive to change, highlighting areas for specific focus for modelling as part of the

FRA. It is therefore not intended necessarily for use for impact assessment, but to identify areas/properties potentially sensitive to an adjustment in model parameters.

Suggested actions

The following actions are suggested to improve the modelling following the items captured within the review:

- Update the hydrology as part of the FRA, using the latest methods and updated hydrometric records. This will ensure that the model is providing the best estimate of flooding resulting from the various design events;
- Ensure that the improvements to hydraulic structure representation are fully captured within the Model Build Report;
- Ensure that the derivation of the inflows applied in the Emergency Discharge test is clearly captured within the Model Build Report; and
- Clarify the purpose of the +50% roughness sensitivity scenario, and review the areas highlighted by the sensitivity test to confirm the thresholds for flooding and model parameters as part of the FRA.

Appendix A15.2B Hydraulic Modelling Report

Appendix C. Model build report

Integrated Catchment Model

MODEL BUILD REPORT

Project	Havant Thicket Emergency Discharge Study	Job Number	5165942
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Model Type	1D/2D	Software	InfoWorks ICM
Revision	1.1	Date	21/05/2019
Originator	Rob Millington	Reviewer	Will Rust

PROJECT BACKGROUND AND APPROACH

PROJECT DESCRIPTION & REQUIREMENTS

Build an integrated surface and fluvial model to model the impact of emergency discharges downstream of the proposed Havant Thicket Winter Storage Reservoir (HTWSR), covering the Riders Lane Stream and the Hermitage Stream downstream of Middle Park Way Bridge, to the confluence of the Hermitage Stream and Langstone Harbour.

Develop the model taking outputs from the Environment Agency Havant Flood Mapping Study (EAHFMS) undertaken in 2008, including porting elements of the combined InfoWorks RS / InfoWorks CS model developed for that study.

Update the model to remove simplifications and other model artefacts (eg interpolated sections) not required by InfoWorks ICM. Remodel structures using the appropriate representations in InfoWorks ICM. Update floodplain to 2D where appropriate.

Verify the InfoWorks ICM model against the 1% and 0.1% design events, and the December 2000 event modelled during the EAHFMS.

Simulate a range of high and low flows to inform an understanding of the thresholds for flooding of property and infrastructure in terms of flow and annual probability (including review of the results of the 0.1% event modelling).

Simulate the option for the emergency discharges identified during the initial phase of the ED study namely:

Option 1 - 2.5 m³/s downstream of the dam into the Riders Lane Stream through Pipe A and 8.6 m³/s into the Hermitage stream through Pipe B (vicinity of Purbrook Way Bridge).

MODEL APPROACH & SUMMARY

Software used, including versions.	InfoWorks ICM v9.0.2
Node Summary and river length.	Nodes Total: 100 Nodes Manholes: 0 Pipes: 7 Rivers: 43 Bridges: 13 Weir Fixed: 1 Irregular weirs: 8 Bar screens: 4 Culvert inlets: 7 Culvert outlets: 7 Inline banks: 12 2D zones: 4

Integrated Catchment Model

MODEL BUILD REPORT

	2D elements: 27469
Latest Model Version (Atkins WDS):	Model: snumbat://SGBD016371:40000/PW_Havant_Thicket_ICM_v9.0.2 Model Network: Havant_RS_Build_1 (v118)
Transportable database:	Havant_ICM_v902_050619.icmt (includes 1%AP, 0.1% AP, Dec 2000 and Option 1 runs)
Does the model run successfully?	Yes.

Integrated Catchment Model

MODEL BUILD REPORT

Model Schematisation

MODEL CONCEPTUALISATION

Model Summary	Channel and floodplain network built using 1D river reaches and 2D mesh, linked by banklines. 2D mesh is applied in patches to represent above ground flows over some long culverts.
Model boundary conditions	Normal Depth or Vertical wall boundary conditions have been applied on the 2D zones. The zones are extended significantly beyond the extents of any modelled flooding and show no glass walling. A tidal downstream boundary condition is applied at the outflow of the Hermitage Stream into Langstone Harbour as a level file at the outfall. The observed tide curve from Portsmouth is applied for the December 2000 event. A 2 year surge tide event developed using the EA "Coastal Design Sea Levels – Coastal Flood Boundary datasets" is applied in conjunction with the 1% AP, 0.1% AP and Emergency Discharge flows. The tidal surge peak is synchronised with the peak flow downstream of Bedhampton weirs. Point inflows are included at model break nodes at upstream boundaries and intermediate locations.

HYDROLOGY

Inflows	All inflows to the model are provided as inflow files. 1% AP and 0.1% AP inputs were extracted from EAHFMS InfoWorks RS model. Low flows based on data downloaded from National River Flow Archive flow duration curves.
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DRAINAGE

Foul / Combined Network	Not currently included
Storm Network	Imported from EAHFMS InfoWorks RS model, with all data reviewed and updated against original survey data and site visits. Flood Relief Culvert imported from InfoWorks CS model.

1D SCHEMATISATION

Rivers & channels	Interpolated and duplicated sections removed except where required at junctions or structures. River reaches built from survey cross-sections. Break nodes added where required for inflows or to terminate reaches at structures or junctions. Floodplain sections updated from 2016 Environment Agency (resolution?) LiDAR outside of surveyed parts of sections.
Storage nodes	Storage nodes have been included to represent the following: - The confluence of the Hermitage Stream and Riders Lane Stream - The split in the Hermitage Stream at the Flood Relief Culvert - The area between the Flood Relief Culvert Inlet Weir and the culvert inlet.
Roughness values	Roughness values were taken from InfoWorks RS model. Values in channels are unlikely to have changed significantly, particularly in reaches with concrete lined channels. Potential for roughness increase due to lack of maintenance on banks and in channels has been sensitivity tested (+20% and +50%)
Channel conveyance	Conveyance curves were checked at all locations for discontinuities in the conveyance data, and any found were eliminated using panel markers as required.

Integrated Catchment Model

MODEL BUILD REPORT

Bridges	Bridges have been represented using ICM Bridge links, with appropriate sections, openings and bridge deck. Headloss coefficients left at default. Most bridges are single span across concrete lined channels so only have an impact on channel flows when levels exceed the bridge soffit levels.
Culverts	Culverts are represented using a series of Inlet -> Conduit -> Outlet links between the upstream and downstream river reaches. Overtopping flows at 5 culverts are represented using a combination of Inline Banks and 2D mesh zones to simulate flow from the upstream channel onto the surface, then returning to the downstream channel. Overtopping of the footpath culvert downstream of RLS3329 is represented using an irregular weir. Overtopping of the A27 culvert is represented using an inline bank link, though this is not overtopped in any of the modelled scenarios.
Trash screens	Bar Screens were added on 4 culverts upstream of the ICM Inlet links to represent trash screens. The bar width, angle and bar shape were determined from site visits and inference from photographs.
Weirs	A Weir has been included to represent the structure across the Hermitage Stream adjacent to the inlet to the Flood Relief Culvert. The weir upstream of the culvert is represented as an Irregular weir. The crest levels for this have been inferred from observations made on site and the survey of the adjacent weir across the Hermitage Stream. The cascade of weirs downstream of Meyrick Road (adjacent to Bedhampton pumping station) is represented by a series of Irregular weirs.

2D SCHEMATISATION	
2D Zone	4 x 2D zones were included to represent overland flows at the following locations: • Middle Park Way Culvert (Riders Lane Stream) • Footbridge d/s of Great Copse (Riders Lane Stream) • Dunsbury Way (Riders Lane Stream) • Purbrook Way (Riders Lane Stream) to the A27 Maximum triangle area and minimum element area set to 10m². Vertical wall or normal depth boundary (Model shows no intersection of flood levels with the boundary of the 2D zones) Lowering element ground levels to adjacent bank levels Roughness from roughness zones
Mesh Zones	Mesh zones have been used to set the 2D mesh level to ground level at Dunsbury Way and at the footbridge downstream of Great Copse. This prevents the mesh from using the LiDAR data at these locations as this does not represent ground level accurately.
Roughness	Roughness zones were created using OSMM data and used to represent buildings. See "P:\GBEMC\WGE\Projects\Water\20171177 Havant Thickett Reservoir\7 WIP\ED Modelling\OSMM_Roughnesses.xlsm"
Buildings	Represented as roughness zone (value 1.000) and also as porous polygons. Porous polygons have infinite height and porosity of 0.1.
Other urban fabric (roads etc)	Not explicitly included in mesh. Roughness values have been varied using roughness zones.
Banklines	Banklines were added to support the following: • Creation of Inline banks to represent flow from 1D areas of the model to 2D. At culverts, these banklines represent the survey profile at ground level above the culvert on both the upstream and downstream headwalls.

Integrated Catchment Model

MODEL BUILD REPORT

- Linkage between river reaches and adjacent floodplain modelled using 2D zones. Banklines are derived either from LiDAR data or from assessment of the survey cross-sections, or a combination of both.

VERIFICATION

The InfoWorks ICM model has been verified through comparison of the flood extents for the 1% AP and 0.1% AP flood events with the Environment Agency Flood Zone 2 and Flood Zone 3 Flood Maps for Planning.

The 1% AP and 0.1% AP flows were extracted from the InfoWorks RS model at all points of inflow and were duplicated in the InfoWorks ICM model. For each event, the inflows to both models are effectively identical and any differences in levels and between the flood maps (outside of the tidal area) are attributable to differences in the representation of the channel, floodplain and structures between the models.

The 1% AP outlines are presented in the Main Report in Figures B1 to B8, and the 0.1% AP outlines in Figures B9 to B16, compared with the Flood Zone 3 and Flood Zone 2 flood outlines respectively.

Peak levels and flows upstream of the main structures are given in Table 1 for the 1% AP and Table 2 for the 0.1% AP event.

Table 1 Comparison of peak levels and flows – 1% AP event

Location	Model section	Peak Level (mAOD)			Peak Flow (m³/s)	
		EA HFMS	ED Study	Difference (mm)	EA HFMS	ED Study
Riders Lane Stream						
Middle Park Way (u/s)	RLS4326	16.37	16.67	300	5.7	5.7
High Lawn Way Footbridge (u/s)	RLS3828	15.10	15.51	410	6.2	6.2
Dunsbury Way (u/s)	RLS3480	12.88	14.72	1840	6.3	6.3
Riders Lane Footbridge (u/s)	RLS3329	12.25	12.21	-40	6.6	6.6
Purbrook Way (u/s)	RLS3036	10.46	10.46	0	6.6	6.7
Ditcham Crescent (u/s)	RLS2885	9.23	9.07	-160	6.7	6.6
Hermitage Stream						
Chalton Crescent Footbridge (u/s)	HS3179	9.68	9.55	-130	15.4	15.4
Purbrook Way (u/s)	HS2909	8.90	8.70	-200	15.4	15.4
Barncroft Way (u/s)	HS2135	7.67	7.49	-180	24.4	24.4
New Road Bridge (u/s)	HS1780	7.27	7.23	-40	24.4	24.0
Railway Bridge (u/s)	HS1509	6.59	6.76	170	24.6	24.7
West Street Bridge (u/s)	HS1404	6.25	6.20	-50	15.7	20.9
Meyrick Road Bridge (u/s)	HS1119	5.52	5.41	-110	15.7	20.9
A27 (u/s)	HS0515	2.82	2.78	-40	18.7	24.6
Langstone Harbour	HS0000	2.80	2.73	-70	29.8	32.5

Flows in highlighted cells differ due to changes in the representation of the entrance to the Flood Relief Culvert. This results in reduced flows in the FRC and a corresponding increase in the main channel.

Table 2 Comparison of peak levels and flows - 0.1% AP event

Location	InfoWorks ICM Model section	Peak Level (mAOD)			Peak Flow (m³/s)	
		EA HFMS	ED Study	Difference (mm)	EA HFMS	ED Study
Riders Lane Stream						
Middle Park Way (u/s)	RLS4326	17.38	17.98	600	8.3	8.2
High Lawn Way Footbridge (u/s)	RLS3828	15.29	15.77	480	9.1	8.9

Integrated Catchment Model

MODEL BUILD REPORT

Dunsbury Way (u/s)	RLS3480	13.31	14.94	1630	9.2	9.0
Riders Lane Footbridge (u/s)	RLS3329	12.43	12.36	-70	9.7	9.5
Purbrook Way (u/s)	RLS3036	10.58	10.58	0	9.7	9.7
Ditcham Crescent (u/s)	RLS2885	9.52	9.33	-190	9.7	9.6
Hermitage Stream						
Chalton Crescent Footbridge (u/s)	HS3179	9.92	9.75	-170	20.6	20.6
Purbrook Way (u/s)	HS2909	9.16	8.93	-230	20.7	20.7
Barncroft Way (u/s)	HS2135	8.00	7.80	-200	33.0	32.9
New Road Bridge (u/s)	HS1780	7.58	7.61	30	33.1	27.4
Railway Bridge (u/s)	HS1509	7.06	7.16	100	33.4	33.8
West Street Bridge (u/s)	HS1404	6.63	6.47	-160	23.6	29.4
Meyrick Road Bridge (u/s)	HS1119	5.74	5.53	-210	23.6	29.5
A27 (u/s)	HS0515	2.84	2.82	-20	27.9	34.3
Langstone Harbour	HS0000	2.80	2.73	-70	40.1	42.2

Flows in highlighted cells differ due to changes in the representation of the entrance to the Flood Relief Culvert. This results in reduced flows in the FRC and a corresponding increase in the main channel. Flow differs at New Road Bridge as flow bypasses this bridge on the 2D mesh in the InfoWorks ICM Model.

The following observations on the comparison can be made:

- The results throughout the InfoWorks ICM model area are generally comparable in terms of extent with those from the previous modelling in areas where the water levels are not significantly affected by headlosses at structures;
- Minor differences in extent may occur between the two models due to differences in the way the water surface and the ground model (LiDAR) are intersected between InfoWorks RS and InfoWorks ICM. Differences may also result from the use of the revised 2016 LiDAR data;
- The outlines on the Hermitage Stream in the area downstream of the railway bridge show some differences due to use of a combined 1D/2D approach and the representation of the channel banks in the InfoWorks ICM model;
- Flows through the Flood Relief Culvert in the InfoWorks ICM model are reduced as a result of the inclusion of the upstream weir structure, with flows correspondingly increased in the Hermitage Stream downstream of the inlet to the Flood Relief Culvert;
- The lower peak flow at New Road in the InfoWorks ICM model (27.4 m³/s compared to 33.1 m³/s) is the result of flow bypassing the bridge on the floodplain;
- The use of InfoWorks ICM "BRIDGE" structures generally gives a reduction in the headloss across most of the bridge structures in comparison with the orifice representation used in InfoWorks RS. The use of orifice units is likely to have resulted in an artificially high headloss across many of the bridges in the InfoWorks RS model as in reality the bridges provide very little constriction to flow until levels are above the bridge soffit;
- The use of a 2D approach in the InfoWorks ICM model allows detailed representation of surface flows around the culverts, most notably at Dunsbury Way, and in the reach downstream of the confluence of the Hermitage Stream and Riders Lane Stream;
- The most significant differences on the Riders Lane Stream are observed in the reach upstream of the Dunsbury Way culvert, where the reduction in size of the culvert results in an increase in water levels and flood extent upstream. The 1% AP peak level increases by 1.84 m and the 0.1% AP peak level by 1.63 m immediately upstream of this culvert;
- The 0.1% AP extent upstream of Middle Park Way is greater than in the InfoWorks RS modelling, with the level immediately upstream of the Middle Park Way culvert increasing by 0.61 m; and
- The Flood Zone 2 and Flood Zone 3 outlines in the area of the Middle Park Way bridge on the Hermitage Stream include the mapping developed from the Warren Dam Discontinuance Study which show overland flooding due to surcharging of the Warren Stream culvert. These flows are included in InfoWorks ICM model as a point inflow and the overland component is not modelled.

Integrated Catchment Model

MODEL BUILD REPORT

The December 2000 flood event has also been simulated using the InfoWorks ICM model. As with the design events, the input flows were extracted from the InfoWorks RS model and directly applied at the relevant locations in the InfoWorks ICM model. This event is generally contained within the channel, so flood maps are not presented for this event in this note. Figure 1 shows levels at the Havant gauging station for this event, which show a close match with the previous modelling, though these show a systematic overprediction of level against the observed data throughout the event. This is most pronounced during the early part of the event, where the flows developed in the previous study are also overpredicted when compared with the observed flow, as shown in Figure 2.

The systematic error is most probably related to the roughness value (manning 'n' of 0.03) applied for this section of concrete channel. It is recommended that the calibration of the InfoWorks ICM model at this location is reviewed as part of additional verification for the purposes of the HTWSR Flood Risk Assessment (FRA).

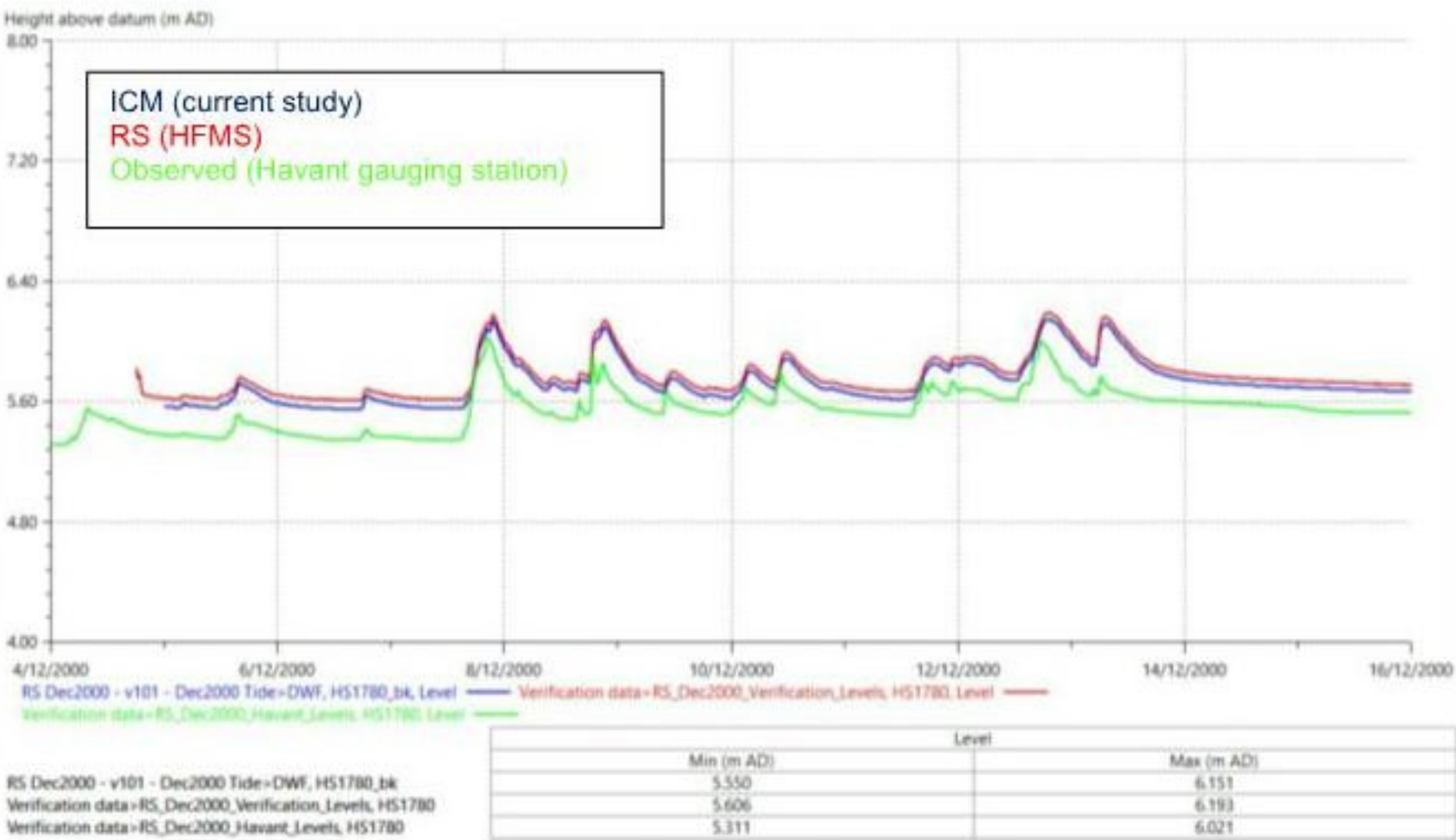


Figure 1 Comparison of levels at Havant gauging station - December 2000

Integrated Catchment Model

MODEL BUILD REPORT

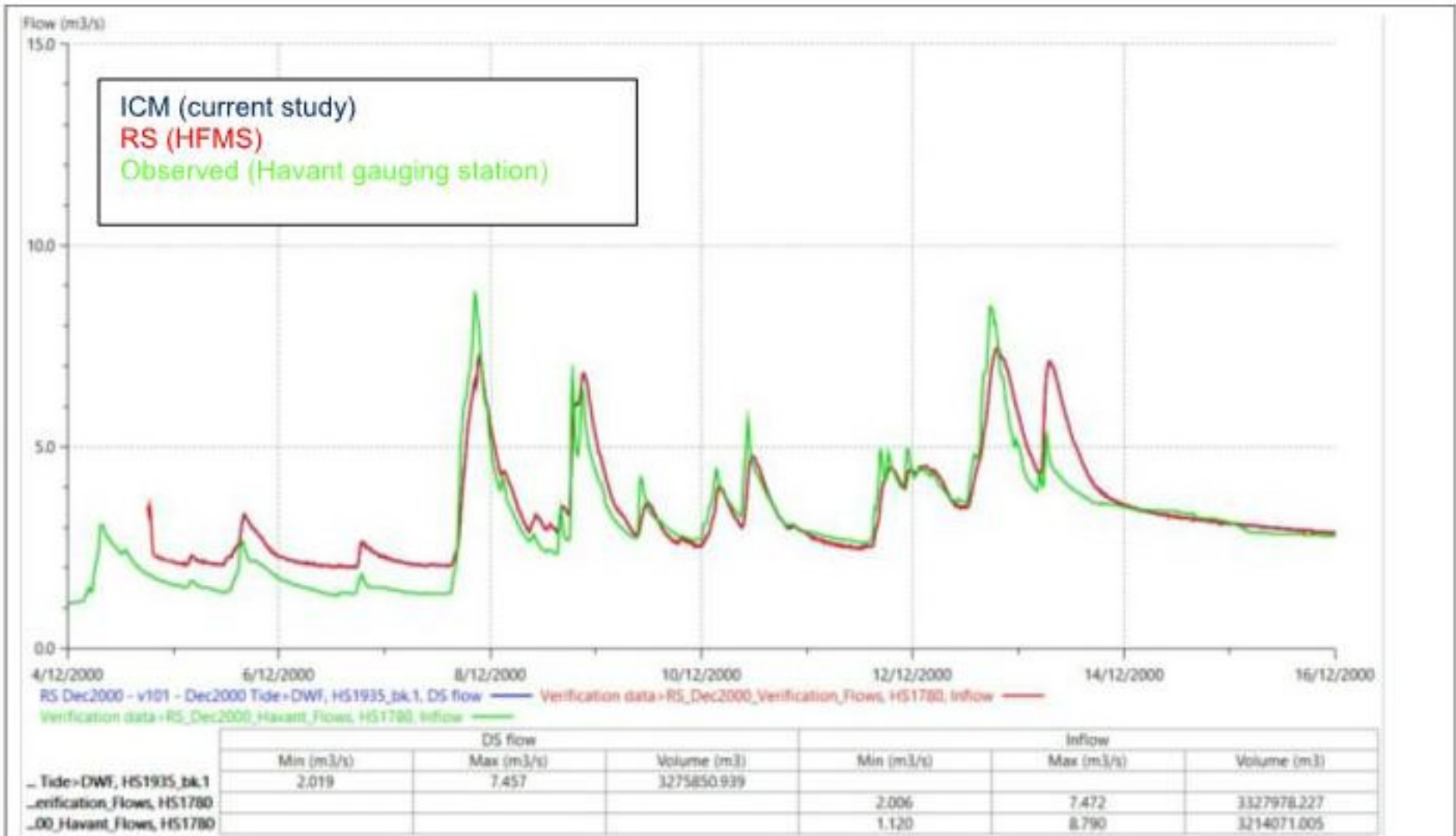


Figure 2 Comparison of flows at Havant gauging station - December 2000

In general, the InfoWorks ICM model produces flood extents that are generally consistent with the earlier modelling for the 1% AP and 0.1% AP events. However, there are some significant differences related to changes in the modelling of the individual structures, most notably for the 0.1% AP event in the upper part of the Riders Lane Stream. A number of model simplifications have been removed, and banks specifically modelled in conjunction with the use of a 2D floodplain approach. As a result, the results of the InfoWorks ICM model can be expected to be more physically realistic. The InfoWorks ICM model is therefore considered to verify well against the previous 1% AP and 0.1% AP modelling.

SIMULATION PARAMETERS

The model is provided as an InfoWorks ICM transportable database, and includes the Runs including the modelled parameters applied. Key parameters are summarised below:

Timestep:	1s
Output timestep:	900s
GPU setting:	If available
1D/2D:	Link 1d/2D calculations at minor timestep
GM mods	Adjust bank levels based on adjacent ground levels

MODEL STABILITY

Volume balance:
1% AP: 0.000% in all 2D zones
0.1% AP: 0.000% in all 2D zones
Option 1: 0.002% in all 2D zones

Integrated Catchment Model

MODEL BUILD REPORT

Model Stability:	Minor oscillations in flow at Section HS1935 during 0.1% AP event. Approx. 5% of peak flow equating to 40mm variation in level at peak of 0.1% AP event.
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Appendix A15.2C Hydrology - FEH Proforma

Flood estimation calculation record

Introduction

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

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3	STATISTICAL METHOD	11
4	REVITALISED FLOOD HYDROGRAPH (REFH2) METHOD	14
5	DISCUSSION AND SUMMARY OF RESULTS	19
6	ANNEX - SUPPORTING INFORMATION	23

Approval

	Signature	Name and qualifications	For Environment Agency staff: Competence level (see below)
Calculations prepared by:	Tracey Ashworth	Tracey Ashworth BSc (Hons), MSc, MCIWEM, C.WEM, C.Sci	3
Calculations checked by:	Charlie Bleasdale	Charlie Bleasdale BSc Graduate member of CIWEM	1
Calculations approved by:	Clare Hodgson	Clare Hodgson BA (Hons), MCIWEM, C.WEM, C.Sci	3

Environment Agency competence levels are covered in [Section 2.1](#) of the flood estimation guidelines:

- Level 1 – Hydrologist with minimum approved experience in flood estimation
- Level 2 – Senior Hydrologist
- Level 3 – Senior Hydrologist with extensive experience of flood estimation

ABBREVIATIONS

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

1 Method statement

1.1 Overview of requirements for flood estimates

Item	Comments
Give an overview which includes: • Purpose of study • Approx. no. of flood estimates required • Peak flows or hydrographs? • Range of return periods and locations • Approx. time available	Portsmouth Water are proposing a new impounding dam and associated reservoir (Havant Thicket Winter Storage Reservoir [HTWSR]) in the upstream reaches of the Riders Lane Stream near Havant. A Flood Risk Assessment (FRA) is required to assess the impact of a dam breach and an Emergency Discharge (ED) from the reservoir. Flow hydrographs for return periods up to and including the 1000-year event are required as hydraulic model inputs so that the dam breach and ED impacts can be calculated.

1.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	The Hermitage Stream drains a total catchment of just over 17 km². The catchment is mainly impervious and is underlain by Reading Beds and London Clay. Intensive suburban development has occurred since the early 1960s, and the lower catchment is now heavily urbanised. The catchment has a flashy and responsive regime. There is a flood relief culvert which transfers flow from the River Lavant catchment, which is to the east; flow is diverted via a pipe into the Hermitage Stream when the flow in the River Lavant exceeds 0.28 m³/s.

1.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – NRFA peak flows dataset, Version 8, September 2019
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1.4 Gauging stations (flow or level)

(at the sites of flood estimates or nearby at potential donor sites)

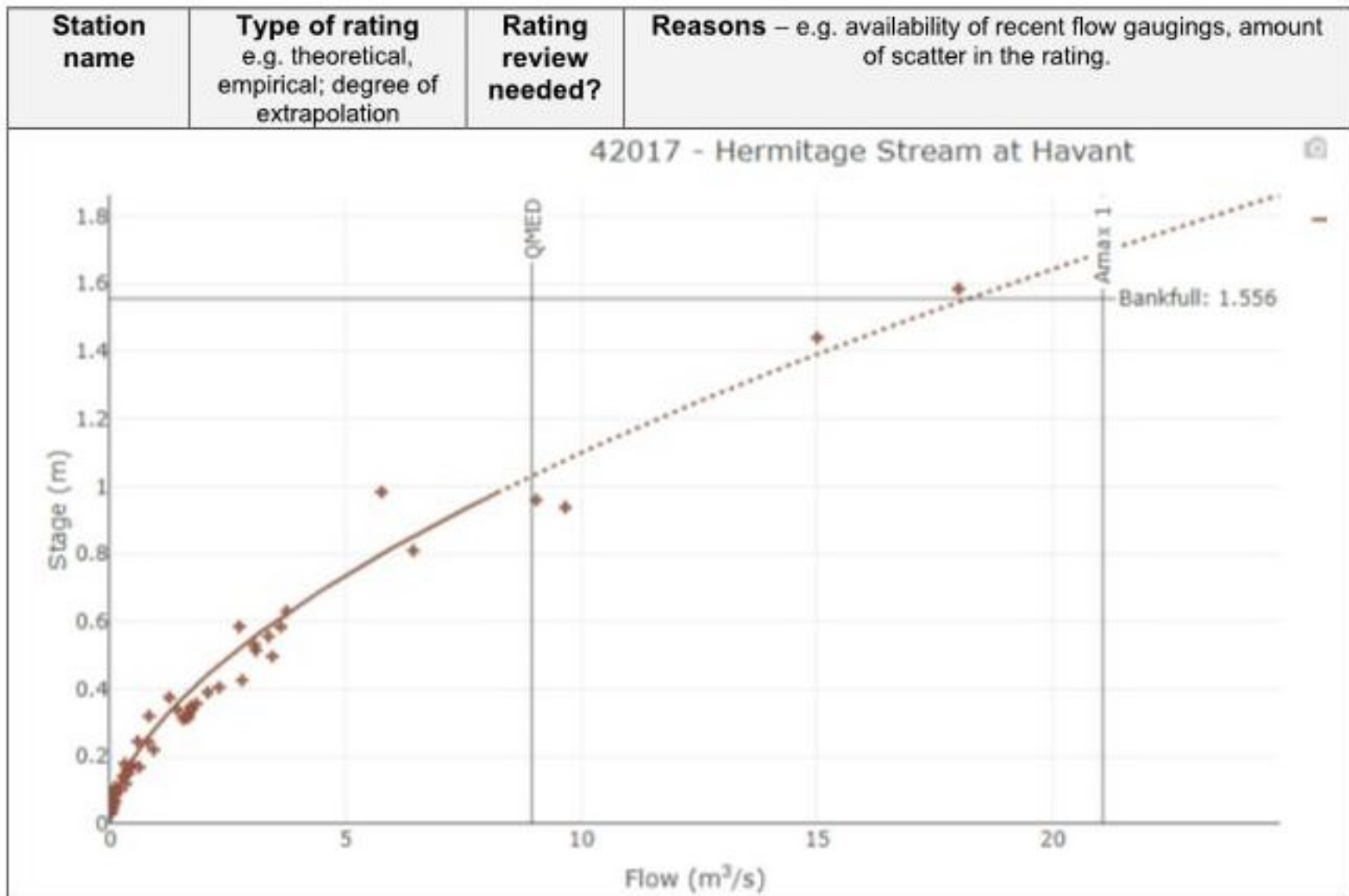
Water-course	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km ²)	Type (rated / ultrasonic / level...)	Start and end of flow record
Hermitage Stream	Havant	42017	42017	SU710067	17	Velocity-area	1953 - present

1.5 Data available at each flow gauging station

Station name	Start and end of data on NRFA	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	Other comments on station and flow data quality – e.g. information from NRFA, trends in flood peaks, outliers.
Havant	1953 - present	No	Yes	No	No	Velocity area station, 5.36m wide in a concrete trapezoidal section. Formalised reach with grass berms beneath a bridge; concrete block wall higher on outside of bend on right bank. Chart stage record extends back to 1953 providing peak flows pre-1979. June-August 2000 flows considered suspect. Station was possibly abandoned for a short duration around 1974. In winter of 1961, there existed a long period of high baseflow where the contribution from the Lavant catchment may have been greater. Structure designed to measure up to 14 m ³ /s; all but exceptional flows contained and structure only exceeded a few times since installation. Bridge downstream may cause some backing up. Trolleys often thrown into stream here. One rating applied across period of record. Rating theoretically based but numerous current meter gaugings for this site.
Give link/reference to any further data quality checks carried out			N/A			

1.6 Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Reasons – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Havant	Rated section	No	A detailed rating review was not included in the scope of the study. A simple check of the rating has been undertaken. The rating appears reasonable up to bank full for high flows. The highest check gauging is 18 m ³ /s, which has an estimated return period of 25 years. The data from the gauge has been considered suitable for use in this study.
Give link/reference to any rating reviews carried out		No rating review undertaken. A plot of the rating from the NRFA is below.	



1.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)	No				
Historic flood data – give link to historic review if carried out.	No				
Flow data for events	Yes	Yes	Environment Agency. Reference number SSD160639	Feb 2020	15-minute flow and rainfall data received from 2000 to present (Havant at Hermitage Stream, grid reference SU7106506790)
Rainfall data for events	Yes	Yes	Environment Agency.		15-minute rainfall data at received from 1998 to 2018 (Havant RG, station number 322336, grid reference SU7109606274 and WALDERTON TBR, station number 265123502, grid reference SU7861810340).
Potential evaporation data	No				

Results from previous studies	No				
Other data or information (e.g. groundwater, tides)	None				

1.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes
Outline the conceptual model, addressing questions such as: - Where are the main sites of interest? - What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides...) - Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? - Is there a need to consider temporary debris dams that could collapse?	The catchment will be divided into sub-catchments to provide inflows to the hydraulic model. High peak flows will be the likely cause of any flooding. For the baseline study, there are no significant reservoirs in the catchment and no need to consider temporary debris dams.
Any unusual catchment features to take into account? e.g. - highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% - highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments - pumped watercourse – consider lowland catchment version of rainfall-runoff method - major reservoir influence (FARL<0.90) – consider flood routing - extensive floodplain storage – consider choice of method carefully	The catchment is urbanised and there is an inflow from the River Lavant via the flood relief culvert during high flows. In the hydraulic model, the Lavant contribution will be added as a constant flow.
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	FEH statistical method at the flow gauge (42017) and ReFH2 method for the hydraulic model sub-catchment inflows. The flows in the hydraulic model at the gauge will be compared with the FEH statistical estimates, and if appropriate, the inflows will be scaled.
Software to be used (with version numbers)	FEH online WINFAP-FEH v4 ReFH2.3

2 Locations where flood estimates required

2.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH CD-ROM (km ²)	Revised AREA if altered
A	Hermitage Stream	Upstream extent of hydraulic model	470550	108000	10.82	10.85
B	Hermitage Stream	Lateral inflow from site A to confluence with Riders Lane Stream	470950	107600	-	1.34
C	Riders Lane Stream	Catchment to location of proposed dam	471300	109450	1.91	2.05
D	Riders Lane Stream	Tributary inflow downstream of proposed dam	471750	108850	0.91	0.48
E	Riders Lane Stream	Lateral inflow to Hermitage Stream confluence	471750	107600	-	1.55
F	Hermitage Stream	Lateral inflow downstream of the Riders Lane Stream and Hermitage Stream confluence	470950	107600	-	0.86
42017	Hermitage Stream	Havant flow gauging station	471050	106800	17.33	17.14
*	Hermitage Stream	Catchment used to provide descriptors for site B	470500	107950	0.73	-
Reasons for choosing above locations		A to F are the model inflow catchments. 42017 is the flow gauging station at the downstream extent of the study reach. * This catchment was used to provide descriptors for catchment B, which is the lateral inflow to Hermitage Stream upstream of the confluence with Riders Lane Stream. D was used to provide catchment descriptors for E and F.				

2.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FAR L	PRO PWE T	BFIHOST1 9	DPLBA R (km)	DPSBA R (m/km)	SAA R (mm)	SPRHOS T	URBEX T 2000 (FEH value)	FPEX T
A	1	0.34	0.275	3	35.8	792	45.77	0.2538	0.0709
B	1	0.34	0.252	1.32	36.7	759	47.53	-	0.0655
C	0.973	0.34	0.251	0.99	34.4	793	46.53	0.0046	0.0459
D	0.896	0.34	0.312	0.671	31.4	793	43.72	0.0412	0.1209
E	0.896	0.34	0.312	1.23	31.4	793	43.72	-	0.1209
F	0.896	0.34	0.312	1.23	31.4	793	43.72	-	0.1209

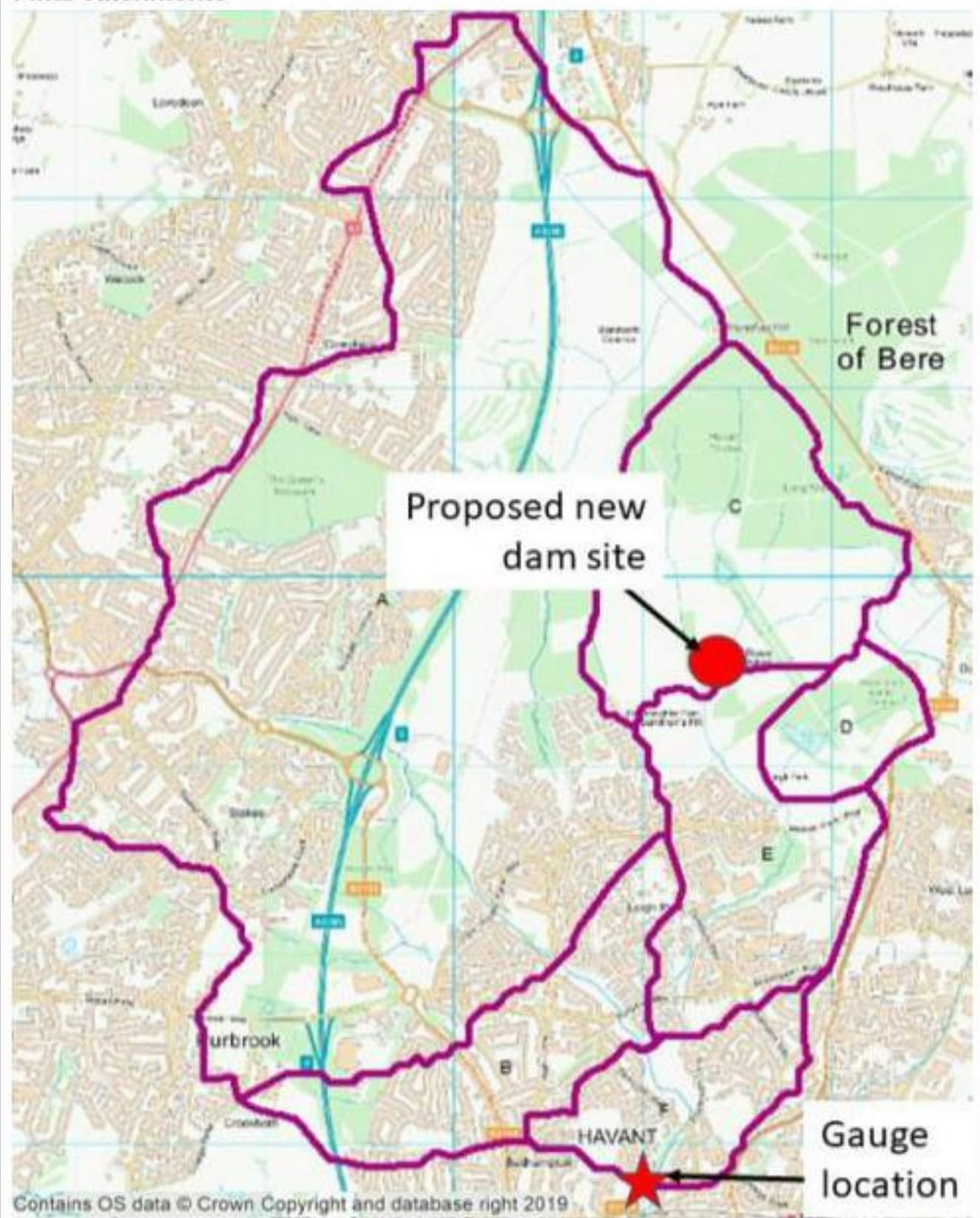
Site code	FAR L	PRO PWE T	BFIHOST1 9	DPLBAR (km)	DPSBAR (m/km)	SAA R (mm)	SPRHOS T	URBEXT 2000 (FEH value)	FPEX T
42017	0.991	0.34	0.273	4.04	32.7	785	46	0.2376	0.0749

2.3 Checking catchment descriptors

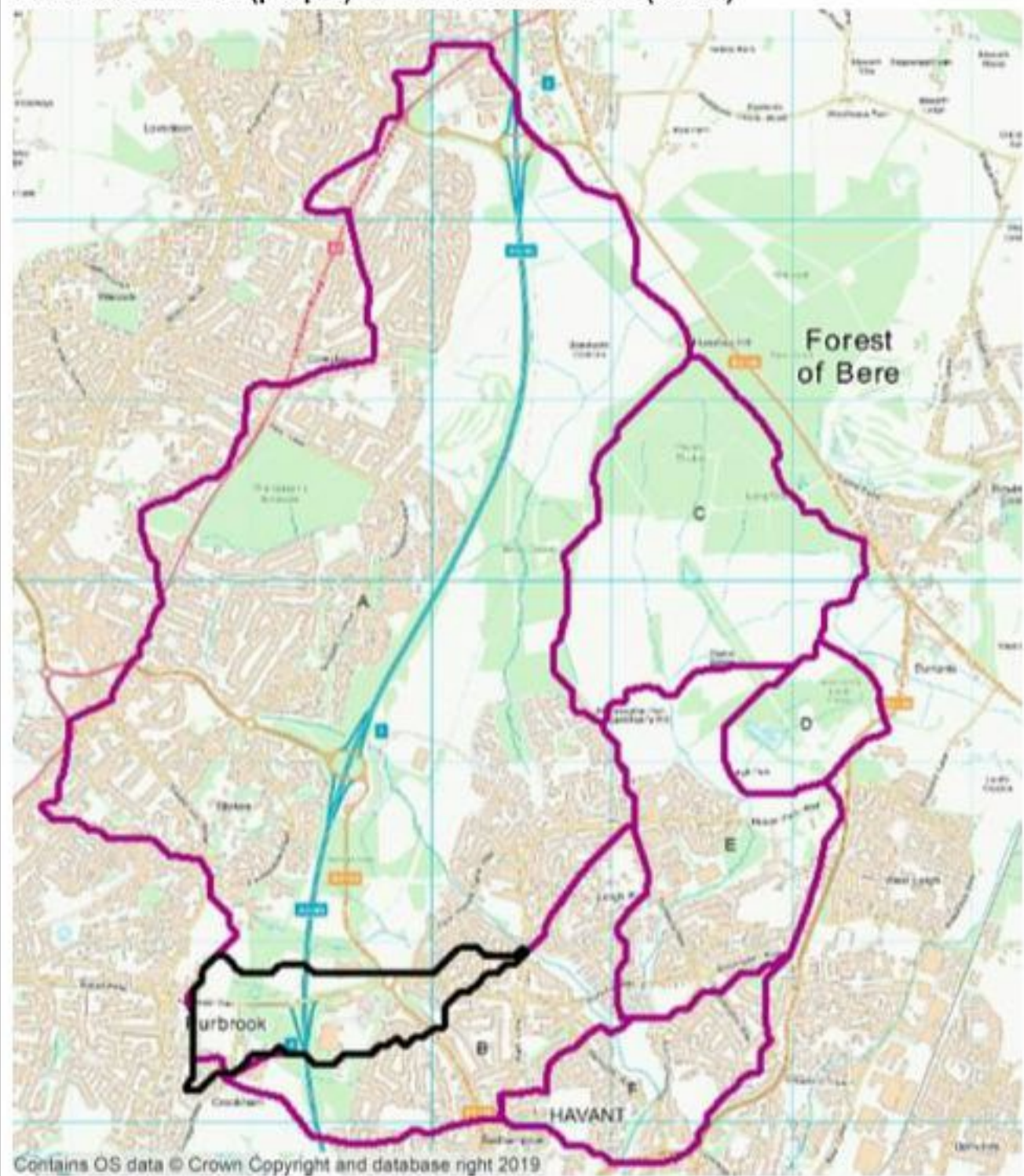
Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Catchment boundaries were checked using OS mapping and LiDAR. An area in the upstream extent of catchment D was removed. Other changes to catchment boundaries were small. FEH catchments and final catchments are shown at the end of this section of the report.																																																						
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	DPLBAR for catchment D was adjusted using the equation $DPLBAR=AREA^{0.548}$ because the catchment area was reduced significantly. Changes to other catchment areas and associated DPLBAR values were small and therefore no other adjustments were made. Catchment D was used to provide descriptors for catchments E and F. It is noted that D has a low FARL value of 0.896 and E and F contain no reservoirs. However, the ReFH2 method is being used to estimate flows and FARL is not used in the ReFH2 calculations. For catchment D this means that the flows will be conservative because no attenuation will be accounted for.																																																						
Source of URBEXT	Urban areas were delineated using Mastermap, and it was confirmed with Wallingford Hydrosolutions (via email on the 26/02/2020, reference number (#1300) that this is acceptable. For the ReFH2 method, the urban areas in km² were added into the models. For the FEH statistical method at the 42017 gauge, URBEXT was obtained using the equation $URBEXT2000 = URBAN \text{ fraction} * 0.629$. The urban areas are as follows: <table><tr><th></th><th></th><th>Updated_area</th><th>Urban_area</th><th></th><th></th></tr><tr><th>Name</th><th>FEH area</th><th>km2</th><th>km2</th><th>Urban Area %</th><th>URBEXT2000</th></tr><tr><td>A</td><td>10.8225</td><td>10.85</td><td>5.26</td><td>48%</td><td>0.305</td></tr><tr><td>B</td><td>lateral</td><td>1.34</td><td>0.979</td><td>73%</td><td>0.461</td></tr><tr><td>C</td><td>1.905</td><td>2.05</td><td>0.008</td><td>0%</td><td>0.002</td></tr><tr><td>D</td><td>0.91</td><td>0.48</td><td>0.011</td><td>2%</td><td>0.014</td></tr><tr><td>E</td><td>lateral</td><td>1.55</td><td>0.731</td><td>47%</td><td>0.297</td></tr><tr><td>F</td><td>lateral</td><td>0.86</td><td>0.585</td><td>68%</td><td>0.427</td></tr><tr><td>TOTAL at 42017 gauge</td><td>17.33</td><td>17.14</td><td>7.574</td><td>44%</td><td>0.278</td></tr></table>			Updated_area	Urban_area			Name	FEH area	km2	km2	Urban Area %	URBEXT2000	A	10.8225	10.85	5.26	48%	0.305	B	lateral	1.34	0.979	73%	0.461	C	1.905	2.05	0.008	0%	0.002	D	0.91	0.48	0.011	2%	0.014	E	lateral	1.55	0.731	47%	0.297	F	lateral	0.86	0.585	68%	0.427	TOTAL at 42017 gauge	17.33	17.14	7.574	44%	0.278
		Updated_area	Urban_area																																																				
Name	FEH area	km2	km2	Urban Area %	URBEXT2000																																																		
A	10.8225	10.85	5.26	48%	0.305																																																		
B	lateral	1.34	0.979	73%	0.461																																																		
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D	0.91	0.48	0.011	2%	0.014																																																		
E	lateral	1.55	0.731	47%	0.297																																																		
F	lateral	0.86	0.585	68%	0.427																																																		
TOTAL at 42017 gauge	17.33	17.14	7.574	44%	0.278																																																		
Method for updating of URBEXT	None because urban areas were delineated using recent mastermap information.																																																						

Catchment boundaries

Final catchments



Final catchments (purple) and FEH catchments (black)



3 Statistical method

The FEH statistical method was used to estimate flows at the Havant at Hermitage Stream flow gauge only (gauge no. 42017). Single site, enhanced single site and pooled growth curves were used and the results compared.

3.1 Search for donor sites for QMED (if applicable)

Comment on potential donor sites Mention: - Number of potential donor sites available - Distances from subject site - Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors - Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	QMED was estimated directly from the subject site flow gauge AMAX data; therefore, no donor site was used.
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3.2 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
QMED was estimated directly from the subject site gauge AMAX data.						

3.3 Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m ³ /s)	Data transfer						Final estimate of QMED (m ³ /s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d _{ij} (km)	Power term, a	Moderated QMED adjustment factor, (A/B) ^a	If more than one donor		
							Weight	Weighted average adjustment factor	
42017	AM	8.9	-	-	-	-	-	-	8.9
Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?						N/A			
Which version of the urban adjustment was used for QMED, and why?						Urban areas were obtained from mastermap as described in section 2.3 above. No urban adjustment as QMED calculated from gauge AMAX data.			
Notes Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone. When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added. When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 ^{Error! Bookmark not defined.} should be used. If the original FEH equation has been used, say so and give the reason why. The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data. The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the									

Site code	Method	Initial estimate of QMED (m ³ /s)	Data transfer						Final estimate of QMED (m ³ /s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d _{ij} (km)	Power term, a	Moderated QMED adjustment factor, (A/B) ^a	If more than one donor		
							Weight	Weighted average adjustment factor	

centroids of the subject catchment and the donor catchment. The final estimate of QMED is (A/B)^a times the initial estimate from catchment descriptors.

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

3.4 Derivation of pooling groups

The composition of the pooling groups is given in the Annex. Several subject sites may use the same pooling group.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons Note also any sites that were investigated but retained in the group.	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
42017 SS	42017 (single site)	Yes	N/A	L-CV = 0.242 L-skew = 0.193
42017 ESS	42017 (enhanced single site)	Yes	The following sites were removed from the default pooling group: 26802 – Chalk catchment 49005 – only 8 years of data 36010 – highest flow is an estimate, further gaugings required 7011 – only 6 years of data 2601 – Chalk catchment 44013 – Chalk catchment 47022 – China clay workings have altered hydrology in the catchment	L-CV = 0.272 L-skew = 0.189 (At site L-moments de-urbanised)
42017 P	42017 (pooled)	No	The following sites were removed from the default pooling group: 26802 – Chalk catchment 49005 – only 8 years of data 36010 – highest flow is an estimate, further gaugings required 7011 – only 6 years of data 2601 – Chalk catchment 44013 – Chalk catchment 47022 – China clay workings have altered hydrology in the catchment	L-CV = 0.227 L-skew = 0.207

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons Note also any sites that were investigated but retained in the group.	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
Notes Pooling groups were derived using the revised procedures from Science Report SC050050 (2008). The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.				

3.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (3.4)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustments	Growth factor for 100-year return period
42017 SS	SS	42017 SS	GL (LMED), recommended in the FEH	N/A	Location: 1.0 Scale: 0.246 Shape: -0.193	2.821
42017 ESS	ESS	42017 ESS	GL, lowest absolute value of Z	UAF=1.23	Location: 1.00 Scale: 0.230 Shape: -0.238	2.918
42017 P	P	42017 P	GL, lowest absolute value of Z	UAF=1.23	Location: 1.00 Scale: 0.187 Shape: -0.256	2.644
Notes Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters. Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010). Growth curves were derived using the revised procedures from Science Report SC050050 (2008).						

3.6 Flood estimates from the statistical method

Site code	Flood peak (m ³ /s) for the following return periods (in years)								
	2	5	10	25	30	50	75	100	200
42017 SS	8.9	12.4	15.0	17.7	19.4	21.7	23.7	25.2	29.2
42017 ESS	8.9	12.3	14.9	18.7	19.7	22.1	24.4	26.1	30.8
42017 P	8.9	11.7	13.9	17.2	18.3	20.1	22.1	23.6	27.8

The flows estimated using the 3 methods are all very similar. The FEH statistical method is not being used directly to derive the hydraulic model inflows, but to compare modelled (with ReFH2 inflows) and FEH statistical peaks at the flow gauge (42017) location. It is recommended that the ESS peak flow estimates are used for the comparison because it is the more conservative estimate, and also makes more use of the flow gauge data than the pooled analysis. The flow gauge (42017) is suitable for QMED only on the NRFA but the rating looks to provide reasonable estimates up to bank full. It should be noted that the gauge flows will include any contribution from the River Lavant via the flood relief culvert.

4 Revitalised flood hydrograph (ReFH2) method

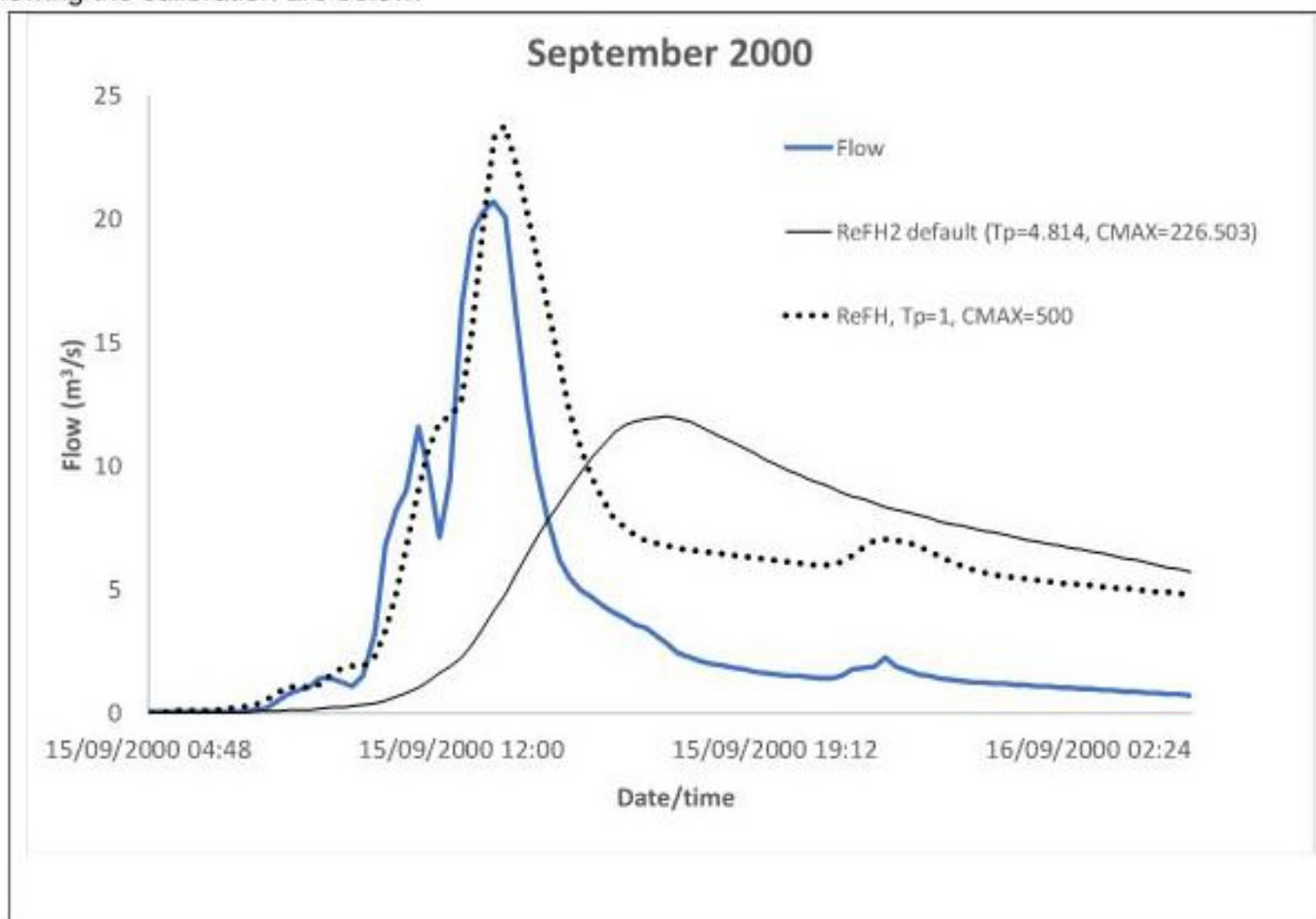
4.1 Calibration event analysis

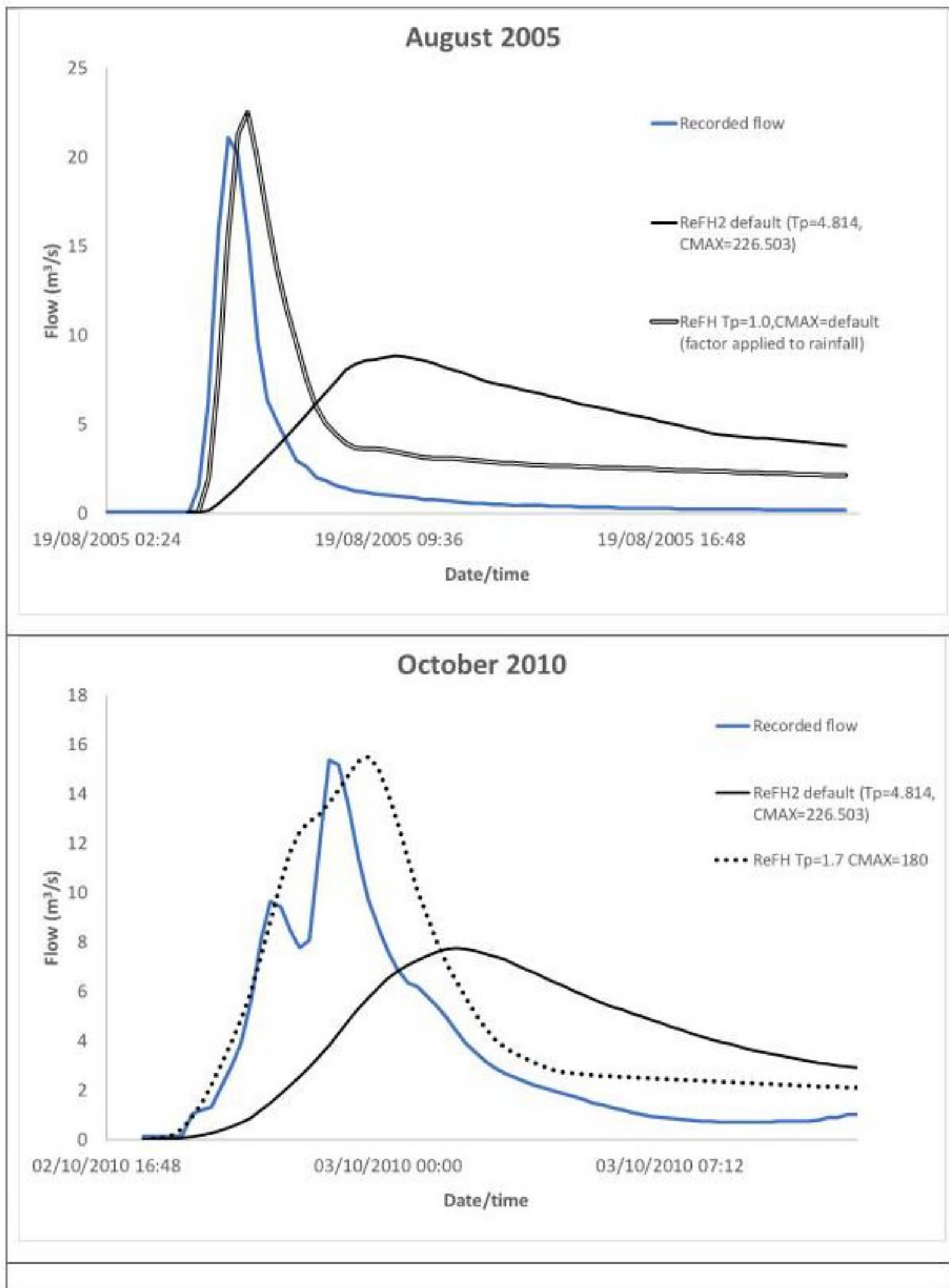
The ReFH2.3 method enables recorded event and antecedent rainfall to be used to estimate event flow hydrographs. The 4 highest recent AMAX events recorded at the Havant flow gauge (42017) were selected and used to calibrate a ReFH2.3 model. Rainfall data recorded at the Havant TBR was used and compared with catchment daily rainfall (CDR) available on the NRFA to check that it was representative. Adjustments to T_p and CMAX were made so that the recorded and modelled hydrographs matched.

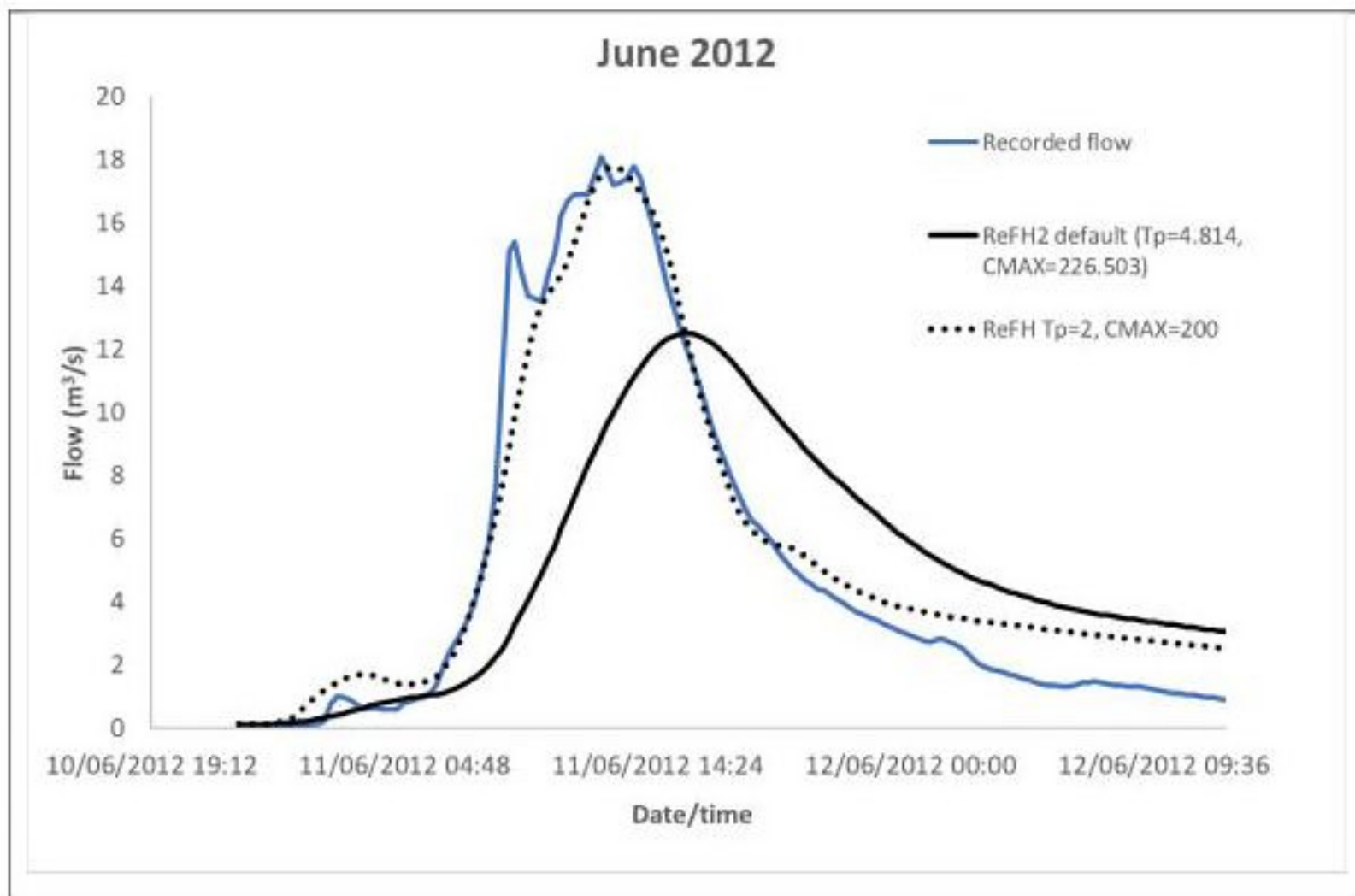
The events are as follows:

Date of peak	Peak flow m^3/s	Havant TBR event rainfall total mm	Rainfall duration (hours)	Calibrated T_p (hours)	Calibrated CMAX (mm)
15/09/2000	20.7	64.4	9	1	500
19/08/2005	21.1	42 (reduced to 29.4 after comparison with CDR)	4	1	226.5
02/10/2010	15.4	33.6	5.75	1.7	180
11/06/2012	18.1	59.6	19	2	200
Average				1.43	276.6
Default				4.814	226.5

Plots showing the calibration are below:







4.2 Parameters for ReFH2 model

Site code	Method: OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours) Time to peak	C _{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR 100-year Baseflow recharge
A	CD	3.945	227.683	29.5	0.42
B	CD	2.447	214.478	23.392	0.296
C	CD	2.119	213.922	21.916	0.287
D	CD	2.47	250.652	26.219	0.633
E	CD	2.47	250.652	26.219	0.635
F	CD	2.47	250.652	26.219	0.634
42017	CD	4.814	226.503	31.34	0.411
42017	OPT	1.43	276.6	31.34	0.411
Brief description of any flood event analysis carried out (further details should be given below or in a project report)			See Section 4.1 above. Flows for sub-catchments A to F were estimated using the default parameters above, and also using adjustments based on the calibration at the flow gauge (42017): • Tp adjustment = 0.297 (1.43/4.814) • CMAX adjustment = 1.22 (276.6/226.503)		

4.3 Design events for ReFH method

Site code	Season of design event (summer or winter)*	Storm duration (hours)	Storm area for ARF (if not catchment area)
A	Winter	8.5	

Site code	Season of design event (summer or winter)*	Storm duration (hours)	Storm area for ARF (if not catchment area)
B	Winter	8.5	Area of catchment to the flow gauge (42017) was used, which gives: - ARF=0.953 - SCF=0.731
C	Winter	8.5	
D	Winter	8.5	
E	Winter	8.5	
F	Winter	8.5	
42017	Winter	8.5	
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?			No Catchment wide storms were applied, with the same storm duration, profile, ARF and SCF used for all the sub-catchments. * The criteria for using winter or summer storms in ReFH2 is that Summer storms are selected by default if: - URBEXT₂₀₀₀ is ≥ 0.30, or $0.15 \leq \text{URBEXT}_{2000} < 0.30$ and BFIHOST19 is ≥ 0.65 - Winter storms are selected in all other cases. The catchment at the flow gauge (42017) has an URBEXT ₂₀₀₀ < 0.3 and BFIHOST < 0.65, therefore winter storms were used.

4.4 Flood estimates from the ReFH method

Site code	Flood peak (m ³ /s) for the following return periods (in years)								
	<i>Default parameters used</i>								
	2	5	10	30	50	75	100	200	1000
A	6.3	8.2	9.6	11.8	12.9	13.8	14.5	16.5	23.9
B	1.1	1.5	1.7	2.1	2.3	2.5	2.6	3.0	4.2
C	1.6	2.1	2.5	3.1	3.4	3.6	3.8	4.4	6.4
D	0.29	0.38	0.45	0.55	0.61	0.65	0.69	0.78	1.1
E	1.1	1.4	1.6	2.0	2.2	2.3	2.5	2.8	4.1
F	0.62	0.81	0.95	1.2	1.3	1.4	1.4	1.6	2.4
42017	8.7	11.4	13.3	16.3	17.8	19.1	20.1	22.8	32.9

Site code	Flood peak (m ³ /s) for the following return periods (in years)								
	<i>Adjustment made to Tp and CMAX based on model calibration</i>								
	2	5	10	30	50	75	100	200	1000
A	10.7						24.6		40.2
B	1.6						3.7		6.0
C	2.4						5.4		8.9
D	0.43						1.0		1.7
E	1.5						3.5		5.8
F	0.90						2.1		3.2
42017	16	22	27	34	37	40	42	48	67

The ReFH2 flows estimated using default parameters were routed through the hydraulic model and the table below shows the modelled peaks at the flow gauge (42017). The flow at the gauge also includes the contribution from the Lavant flood relief culvert – a flow of 2 m³/s was added to the hydraulic model for return periods up to and including the 100-year event, and 2.2 m³/s was added for the 1,000-year event.

Site code	Flood peak (m ³ /s) for the following return periods (in years)								
	<i>Default parameters used</i>								
	2	5	10	30	50	75	100	200	1000
42017 routed flows	10.6	14.0	16.3	20.3			24.3		32.9 (39.4 including all bypassed flow)

5 Discussion and summary of results

5.1 Comparison of results from different methods

This table compares peak flows from various methods with those from the FEH Statistical method at example sites for two key return periods. Blank cells indicate that results for a particular site were not calculated using that method.

The ReFH2 flows are for the lumped catchment rather than the routed sub-catchment flows in the hydraulic model.

Site code	Ratio of peak flow to FEH Statistical peak (ESS method)					
	Return period 2 years			Return period 100 years		
	ReFH2 default parameters	ReFH2 calibrated parameters	Other method	ReFH2 default parameters	ReFH2 calibrated parameters	Other method
42017	0.98	1.80	N/A	0.77	1.61	N/A

To help determine which to use as the final design flows, the return period of recent recorded flow peaks was calculated. The results are in the table below. Given the 52-year data set and the flows recorded, the FEH single site analysis provides a reasonable assessment of the design flows (which would be expected). This suggests that the FEH statistical method provides the most appropriate flows to use as the final design flows.

Adding the flow from the Lavant link pipe to the ReFH2 design peak flows gives return period estimates which are more similar to the those calculated using the FEH statistical method. A flow of 2 m³/s from the Lavant was added for return periods up to and including the 200-year event, and 2.2 m³/s was added for the 1,000-year event

Return period estimate (years) at 42017 using the methods below					
Date of peak	Peak flow m ³ /s	FEH statistical (single site)*	ReFH2.3 with default parameters	ReFH2.3 with default parameters, with Lavant link pipe flows included	ReFH2.3 with average calibrated Tp and CMAX
19-Aug-05	21.1	44	130	75	4
15-Sep-00	20.7	40	120	50 to 75	4
11-Jun-12	18.1	22	55	30	Between 2 and 3
14-Sep-95	18.0	22	50	30	Between 2 and 3
22-Oct-66	15.6	12	25	10 to 30	<2
02-Oct-10	15.4	11	20	10 to 30	<2

* a single site analysis has been used here to estimate the return periods because the calculations are easily undertaken using WINFAP, note that the SS and ESS flows are very similar.

The ReFH2 flows estimated using default parameters were routed through the hydraulic model and the table below shows the modelled peaks at the flow gauge (42017) compared to the estimates from the FEH statistical method. These ReFH2 flows in the hydraulic model are similar to the FEH statistical flows.

Site code			Flood peak (m ³ /s) for the following return periods (in years)								
			<i>Routed flows in the hydraulic model – default Tp and CMAX parameters used</i>								
			2	5	10	30	50	75	100	200	1000
42017	routed	ReFH2 flows	10.6	14.0	16.3	20.3	-	-	24.3	-	32.9 (39.4 including all bypassed flow)
42017	FEH statistical	SS	8.9	12.4	15.0	19.4	21.7	23.7	25.2	29.2	

Site code	Flood peak (m ³ /s) for the following return periods (in years)								
	<i>Routed flows in the hydraulic model – default Tp and CMAX parameters</i>								
	<i>used</i>								
	2	5	10	30	50	75	100	200	1000
42017 statistical ESS	8.9	12.3	14.9	19.7	22.1	24.4	26.1	30.8	
42017 FEH statistical pooled	8.9	11.7	13.9	18.3	20.1	22.1	23.6	27.8	

5.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The FEH statistical method should be the preferred flows estimation method as it makes best use of gauged data. When the ReFH2 flows (with default parameters) from the sub-catchments are routed through the hydraulic model, and the estimated inflow from the Lavant flood relief channel is added, then the peaks at the gauge (42017) are very similar to the FEH statistical peak flows. It is therefore proposed that the ReFH2 method be used to calculate the sub-catchment flow hydrographs, with no adjustment to Tp and CMAX made. It is recommended that as a sensitivity, the ReFH2 flows estimated using the averaged adjusted Tp and CMAX parameters are used to estimate flows for the 100 and 1000-year events. The impact of the sensitivity will be reported in the Hydraulic Modelling Appendix of the Flood Risk Assessment, (HTR-ATK-XX-XX-RP-Z-0042, Atkins 2020).
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5.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are: • The catchment boundaries are representative • The flows recorded at the gauge (42017) are representative of the catchment flood response and suitable for QMED and ESS analysis. • The urban area on OS maps can be used to estimate URBEXT2000 accurately.																											
Discuss any particular limitations , e.g. applying methods outside the range of catchment types or return periods for which they were developed	The inflow from the Lavant along the flood relief culvert is not gauged. The impact of the Lavant inflow will be discussed in the Hydraulic Modelling Appendix of the Flood Risk Assessment, (HTR-ATK-XX-XX-RP-Z-0042, Atkins 2020). In general the model is insensitive to the impact of the Lavant inflow due to the very high conveyance capacity of the Hermitage Stream.																											
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).	At the flow gauge (42017) the confidence intervals estimated using WINFAP for the single site analysis are as follows: <table><tr><th>Return Period</th><th>Magnitude</th><th>Confidence Intervals</th></tr><tr><td>2</td><td>8.9</td><td>8.945 - 8.945</td></tr><tr><td>5</td><td>12.4</td><td>11.852 - 13.224</td></tr><tr><td>10</td><td>15.0</td><td>13.768 - 16.653</td></tr><tr><td>25</td><td>18.6</td><td>16.418 - 21.913</td></tr><tr><td>50</td><td>21.7</td><td>18.361 - 26.863</td></tr><tr><td>75</td><td>23.7</td><td>19.666 - 30.226</td></tr><tr><td>100</td><td>25.2</td><td>20.589 - 32.855</td></tr><tr><td>200</td><td>29.2</td><td>22.863 - 39.940</td></tr></table>	Return Period	Magnitude	Confidence Intervals	2	8.9	8.945 - 8.945	5	12.4	11.852 - 13.224	10	15.0	13.768 - 16.653	25	18.6	16.418 - 21.913	50	21.7	18.361 - 26.863	75	23.7	19.666 - 30.226	100	25.2	20.589 - 32.855	200	29.2	22.863 - 39.940
Return Period	Magnitude	Confidence Intervals																										
2	8.9	8.945 - 8.945																										
5	12.4	11.852 - 13.224																										
10	15.0	13.768 - 16.653																										
25	18.6	16.418 - 21.913																										
50	21.7	18.361 - 26.863																										
75	23.7	19.666 - 30.226																										
100	25.2	20.589 - 32.855																										
200	29.2	22.863 - 39.940																										

Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	The design peak flow estimates and hydrographs were derived for the purposes of this modelling study. If peak flow estimates and hydrographs are required for a different purpose it is recommended that, at a minimum, a review of the results is carried out.
Give any other comments on the study, for example suggestions for additional work.	None

5.4 Checks

Are the results consistent, for example at confluences?	Yes																
What do the results imply regarding the return periods of floods during the period of record?	See Section 5.1.																
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	2.918 (ESS) and 2.821 (SS) for the FEH statistical method, which are both realistic.																
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	The ReFH2 method has been used. The ReFH2 ratio 1000/100 is $32.9/20.1 = 1.64$ Therefore, for the FEH statistical method (ESS), the 1000-year flow can be estimated as $26.1 \times 1.64 = 42.7 \text{ m}^3/\text{s}$. This is reduced to a 1000-year flow of $38.7 \text{ m}^3/\text{s}$ if the lower FEH statistical (P) estimate of the 100-year flow is used. As a comparison, the 1000-year flow in the hydraulic model (including bypassing) is $39.4 \text{ m}^3/\text{s}$, which lies between these two statistical estimates.																
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	<table border="1"> <thead> <tr> <th>Catchment</th><th>Peak flow (l/s/ha) estimated using ReFH2 and default parameters</th></tr> </thead> <tbody> <tr> <td>A</td><td>13</td></tr> <tr> <td>B</td><td>19</td></tr> <tr> <td>C</td><td>19</td></tr> <tr> <td>D</td><td>14</td></tr> <tr> <td>E</td><td>16</td></tr> <tr> <td>F</td><td>16</td></tr> <tr> <td>42017</td><td>15</td></tr> </tbody> </table>	Catchment	Peak flow (l/s/ha) estimated using ReFH2 and default parameters	A	13	B	19	C	19	D	14	E	16	F	16	42017	15
Catchment	Peak flow (l/s/ha) estimated using ReFH2 and default parameters																
A	13																
B	19																
C	19																
D	14																
E	16																
F	16																
42017	15																
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	N/A																
Are the results compatible with the longer-term flood history?	Yes																
Describe any other checks on the results	None																

5.5 Final results

The peak flows for catchments A to F were estimated using the ReFH2 method with default parameter values. The flows at the gauge 42017 are from the FEH statistical ESS analysis.

Site code	Flood peak (m ³ /s) for the following return periods (in years)								
	2	5	10	30	50	75	100	200	1000
A	6.3	8.2	9.6	11.8	12.9	13.8	14.5	16.5	23.9
B	1.1	1.5	1.7	2.1	2.3	2.5	2.6	3.0	4.2
C	1.6	2.1	2.5	3.1	3.4	3.6	3.8	4.4	6.4
D	0.29	0.38	0.45	0.55	0.61	0.65	0.69	0.78	1.1
E	1.1	1.4	1.6	2.0	2.2	2.3	2.5	2.8	4.1
F	0.62	0.81	0.95	1.2	1.3	1.4	1.4	1.6	2.4
42017	8.9	12.3	14.9	19.7	22.1	24.4	26.1	30.8	42.7

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	
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6 Annex - supporting information

6.1 Final pooling group composition

Enhanced single-site

Station	Distance	Years of d	QMED AM	L-CV	L-SKEW	Discordan
42017 (Hermitage Stream @ Havant)	0	52	8.945	0.285	0.146	0.129
25019 (Leven @ Easby)	0.656	40	5.384	0.343	0.378	2.64
27010 (Hodge Beck @ Bransdale Weir)	0.865	41	9.42	0.224	0.293	0.359
41020 (Bevern Stream @ Clappers Bridge)	1.043	49	13.66	0.203	0.181	0.599
72014 (Conder @ Galgate)	1.082	50	16.465	0.233	0.162	0.1
73015 (Keer @ High Keer Weir)	1.084	27	12.33	0.205	0.281	0.569
27051 (Crimple @ Burn Bridge)	1.256	46	4.539	0.219	0.148	0.212
28058 (Henmore Brook @ Ashbourne)	1.264	13	8.838	0.188	-0.109	3.078
28041 (Hamps @ Waterhouses)	1.329	33	26.126	0.223	0.291	1.314
27032 (Hebden Beck @ Hebden)	1.351	52	3.923	0.207	0.244	0.476
48004 (Warleggan @ Trengoffe)	1.364	49	9.983	0.255	0.257	0.472
72007 (Brock @ Upstream of a6)	1.376	40	28.964	0.199	0.218	1.306
24006 (Rookhope Burn @ Eastgate)	1.417	20	24.62	0.152	0.117	1.267
49004 (Gannel @ Gwills)	1.443	49	15.022	0.257	0.104	1.478
Total		561				
Weighted means				0.272	0.189	

Pooled

Station	Distance	Years of d	QMED AM	L-CV	L-SKEW	Discordan
25019 (Leven @ Easby)	0.656	40	5.384	0.343	0.378	2.537
27010 (Hodge Beck @ Bransdale Weir)	0.865	41	9.42	0.224	0.293	0.325
41020 (Bevern Stream @ Clappers Bridge)	1.043	49	13.66	0.203	0.181	0.534
72014 (Conder @ Galgate)	1.082	50	16.465	0.233	0.162	0.11
73015 (Keer @ High Keer Weir)	1.084	27	12.33	0.205	0.281	0.514
27051 (Crimple @ Burn Bridge)	1.256	46	4.539	0.219	0.148	0.215
28058 (Henmore Brook @ Ashbourne)	1.264	13	8.838	0.188	-0.109	2.857
28041 (Hamps @ Waterhouses)	1.329	33	26.126	0.223	0.291	1.197
27032 (Hebden Beck @ Hebden)	1.351	52	3.923	0.207	0.244	0.421
48004 (Warleggan @ Trengoffe)	1.364	49	9.983	0.255	0.257	0.433
72007 (Brock @ Upstream of a6)	1.376	40	28.964	0.199	0.218	1.227
24006 (Rookhope Burn @ Eastgate)	1.417	20	24.62	0.152	0.117	1.16
49004 (Gannel @ Gwills)	1.443	49	15.022	0.257	0.104	1.47
Total		509				
Weighted means		509		0.227	0.207	

6.2 Additional supporting information

Appendix A15.2D Tidal Boundary Generation

Technical Note

Project:	HTR Havant Thicket Outline Design		
TN Ref. No:	HTR-ATK-ZZ-ZZ-RP-Z-0153		
Subject:	Flood Risk Assessment Appendix D Tidal Boundary Generation		
Author:	AB		
Date:	28/08/20	Project No.:	5169117
Distribution:	With Havant Thicket Flood Risk Assessment	Representing:	N/A

Document history

Revision	Status	Purpose description	Originated	Checked	Reviewed	Authorised	Date
P01	S2	First Issue	AB	SB	TB	---	28/08/20

ATKINS BASELINE - LOW RISK

This memo summarises the derivation of design tide curves for the 1 in 2 year, 200 year and 1000 year (T2, T200 and T1000) tidal events in the present day (2020) and one future epoch (2125). The curves are to be used as tidal boundary conditions in a hydraulic model for Havant Reservoir FRA.

The design tide curves have been generated in accordance with guidance provided in Coastal flood boundary conditions for the UK: update 2018 (Environment Agency, 2019), from here on referred to as CFB18.

Extreme sea levels were taken from the CFB18 dataset at Chainage 4610 which is the closest point to Langstone Harbour as shown in Figure 1.

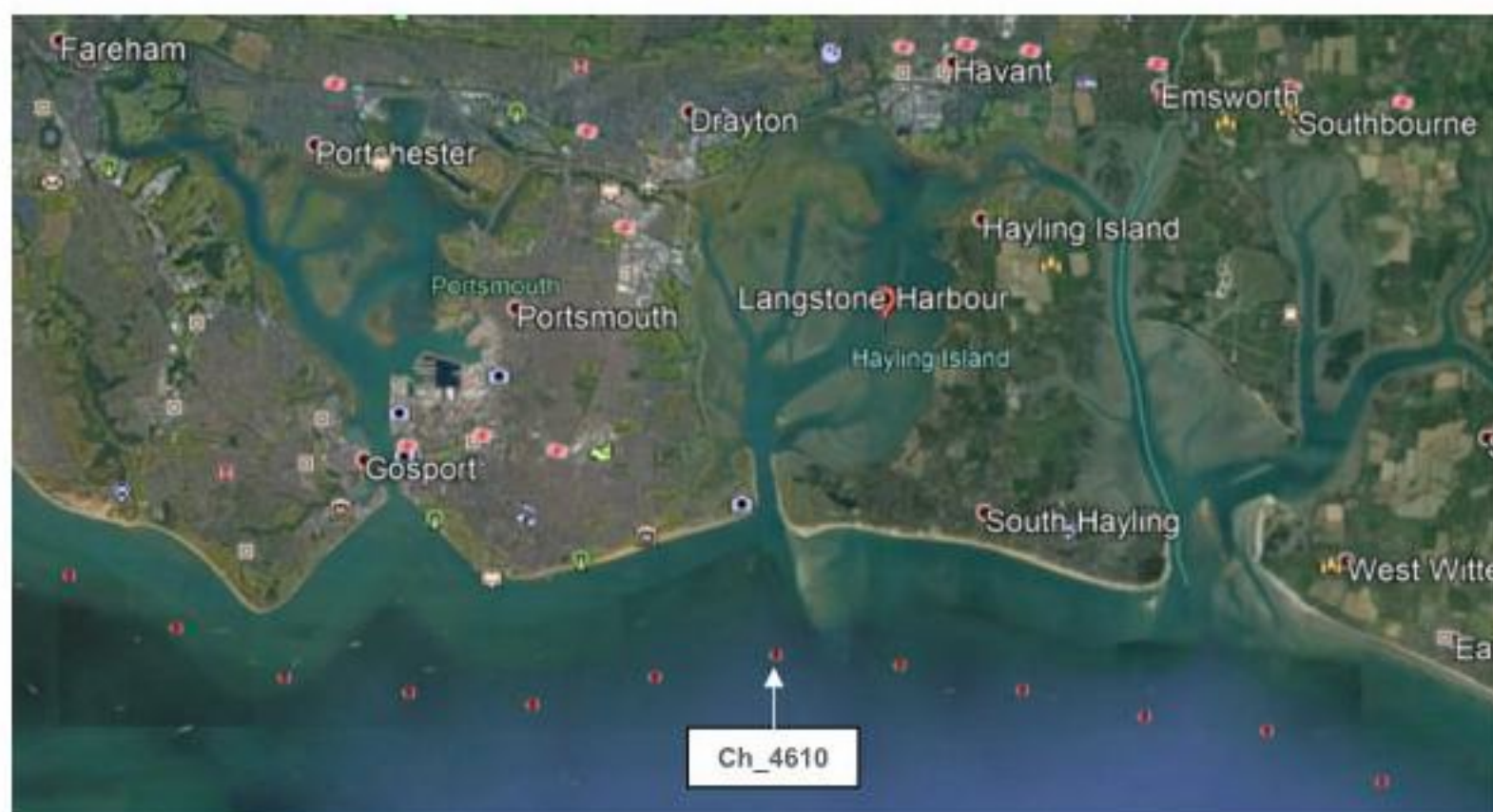


Figure 1 - Location Plan for Langstone Harbour (Google Earth, 2020)

Table 1 – Selected Extreme Sea Levels at Ch_4610

Return period	T2	T200	T1000
Sea level (mAOD)	2.70	3.17	3.33

It is noted that the extreme levels provided are for a base year of 2017 and while the levels are quoted to 2 decimal places, they are only considered accurate to 1 decimal place.

The base astronomical tide curve was generated using our standard TIDSIM software, which has been developed in-house and used for over 20 years, based on harmonic constants given in the Admiralty Tide Tables ⁽¹⁾ for Langstone Harbour. A tide curve was generated for the year 2018 at a 15-minute interval throughout the year. Levels were reduced by -2.74m to convert from Chart to Ordnance datum and the tide curve interrogated to identify a high tide level between MHWS and HAT suitable for uplifting. The date and time of the selected peak is on 20/03/2018 at 01:15. The event duration was set to 99 hours to match the surge profile and provides approximately 1.5 days before and 2.5 days after the selected peak.

ATKINS BASELINE - LOW RISK

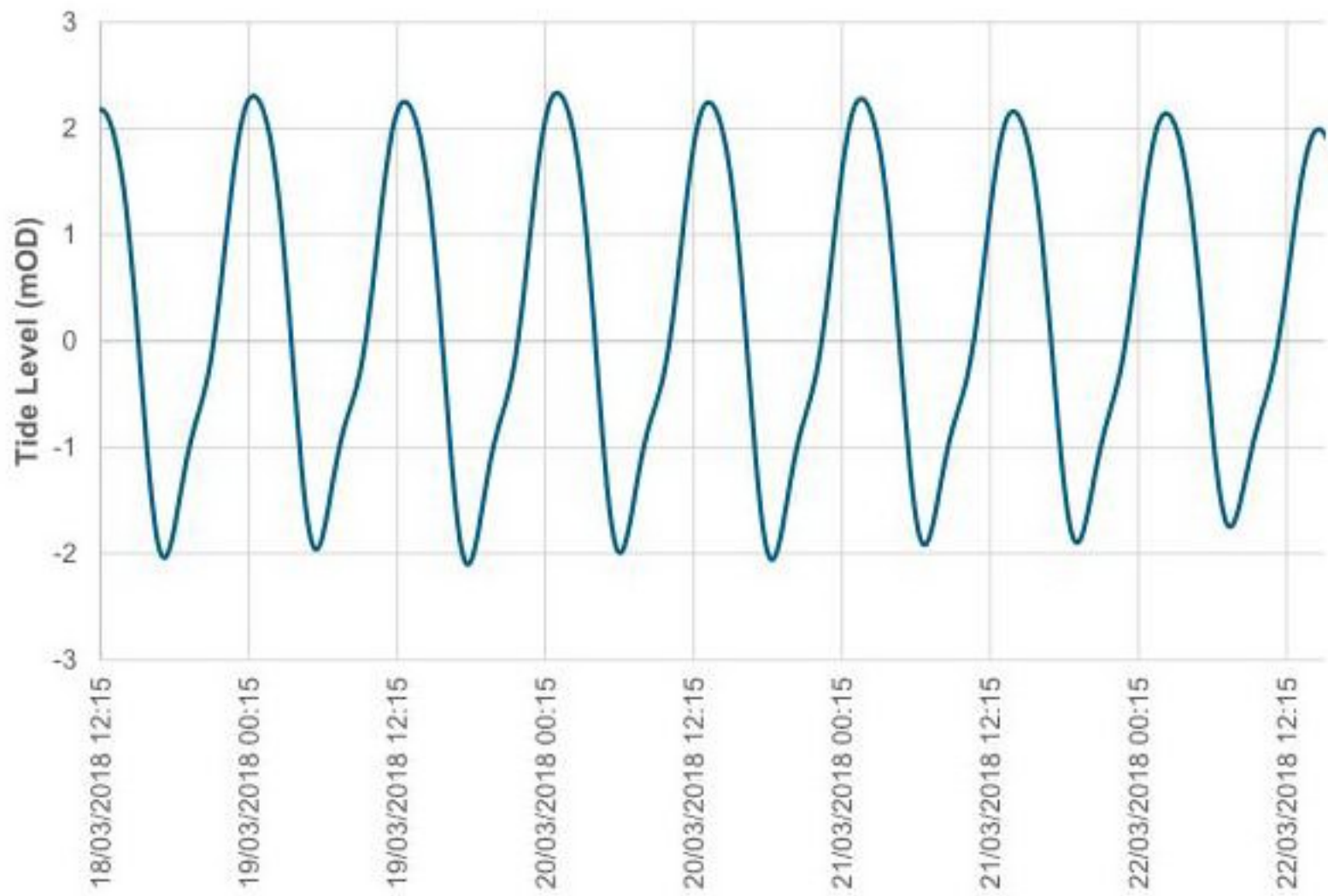


Figure 2 - Base astronomical tide curve

Donor surge shape 13 (Portsmouth) from the CFB dataset, as shown in Figure 3, was used to uplift the base tide to extreme levels. Peak astronomical high tide was selected to coincide with the peak of the surge, which in this case is after 37 hours.

ATKINS BASELINE - LOW RISK

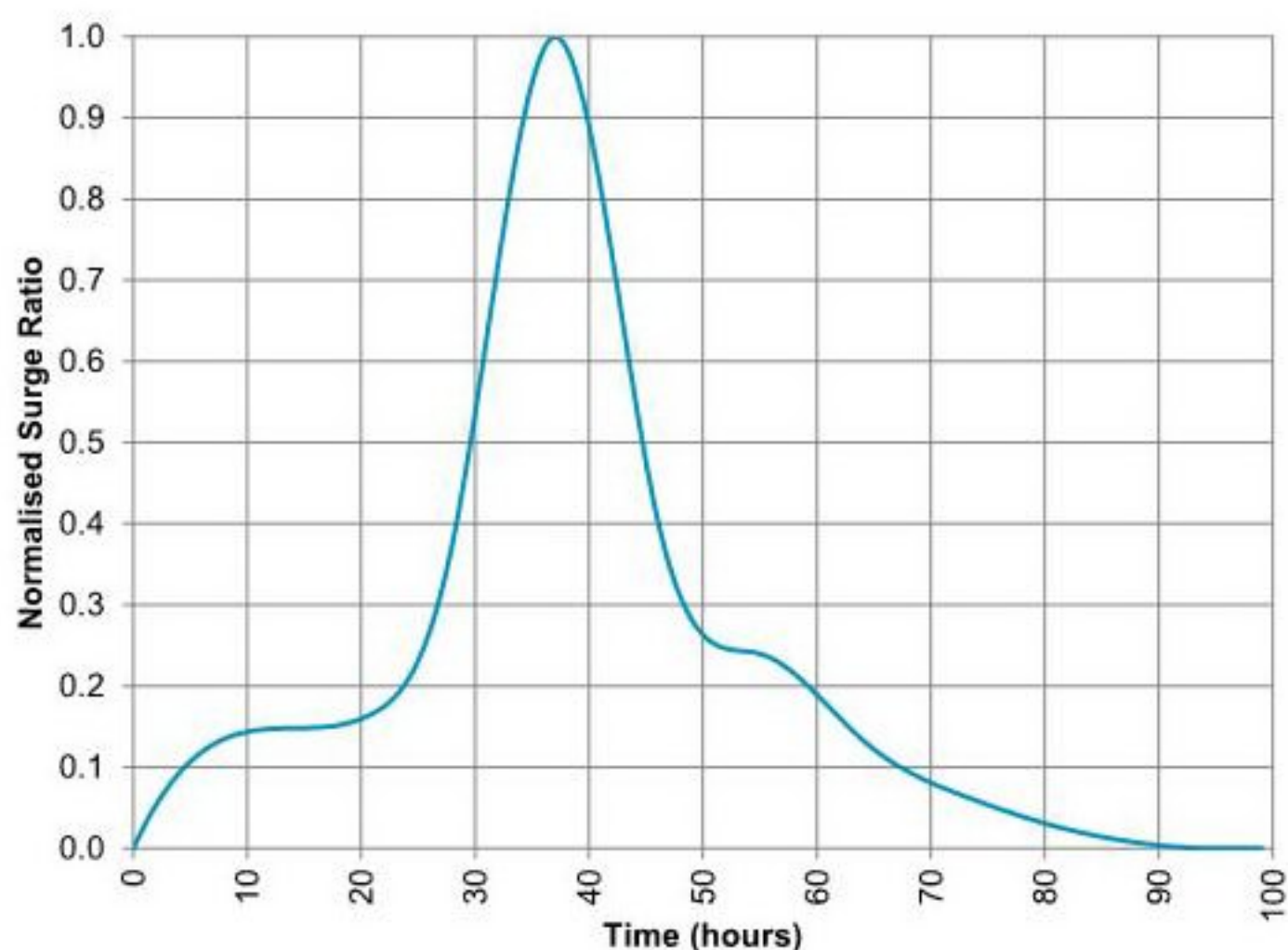


Figure 3 - Surge shape for Portsmouth

The above 3 components (extreme sea level, base astronomical tide curve and surge shape) were combined to produce the design tide curves. The base astronomical tide curve was uplifted to peak at the selected extreme water levels based on the surge shape. At each timestep, the design tide level is calculated based on the difference between the base astronomic tide level and the extreme sea level multiplied by the normalised surge ratio. The equation used is as follows:

Design tide level = base astronomic tide level + (normalised surge ratio *(extreme sea level – peak level of base astronomic tide))

Figure 4 shows the design tide curves in the base year of 2017.

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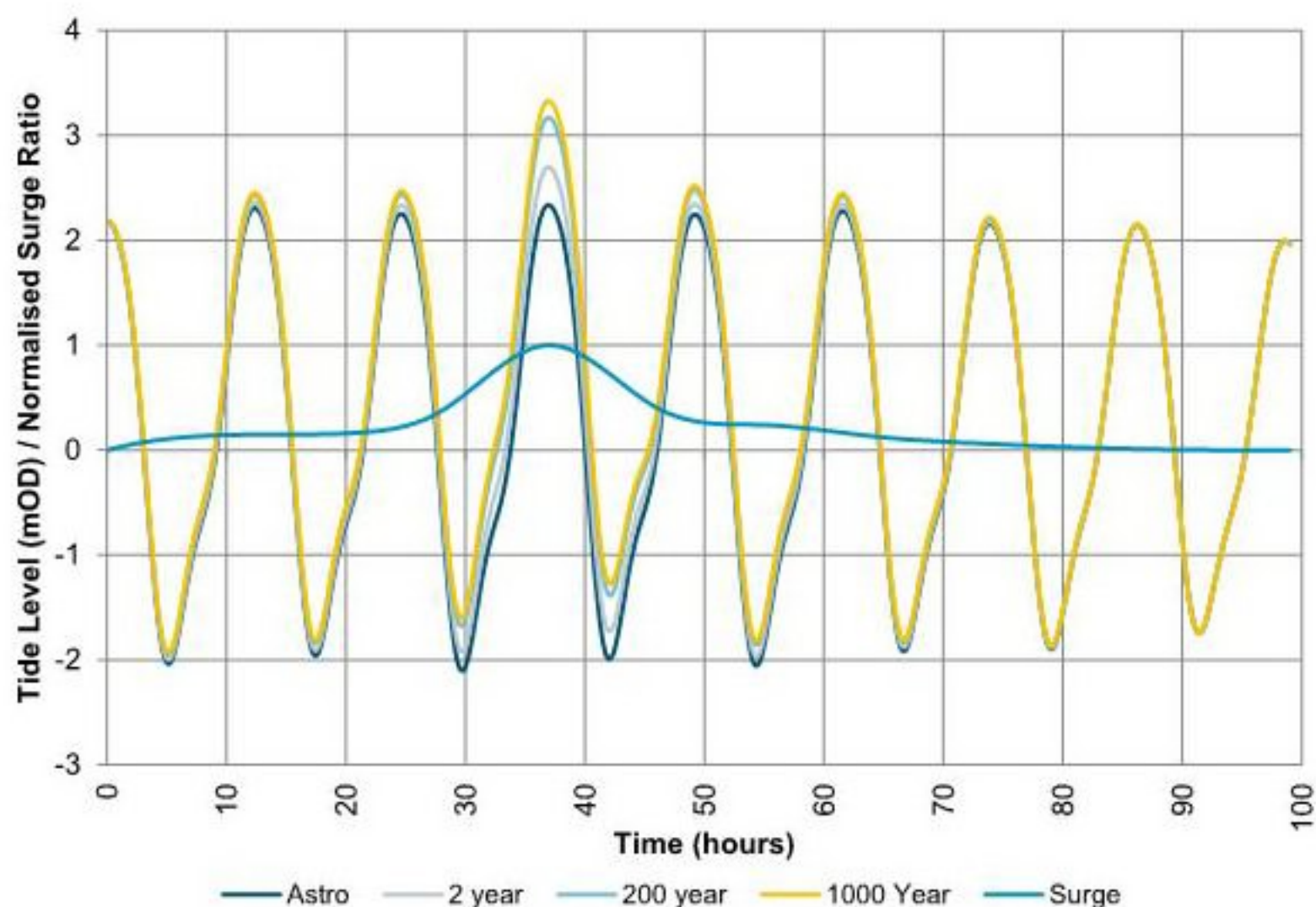


Figure 4 – Design Tide Curves in 2017

Two different climate change scenarios were applied to produce design tide curves in 2020 and 2125 based on the Higher Central and Upper End sea level rise allowances for the South East region given in Table 3 of Flood Risk Assessments: Climate Change Allowances (GOV.UK, 2020).

Table 2 – Climate Change allowances used (beyond 2017 base year)

Year	Higher central	Upper end
2020	0.0171	0.0207
2125	1.1046	1.4832

The design tide curves for each climate change scenario (Higher central and Upper end) are shown in Figure 5 and Figure 6 below.

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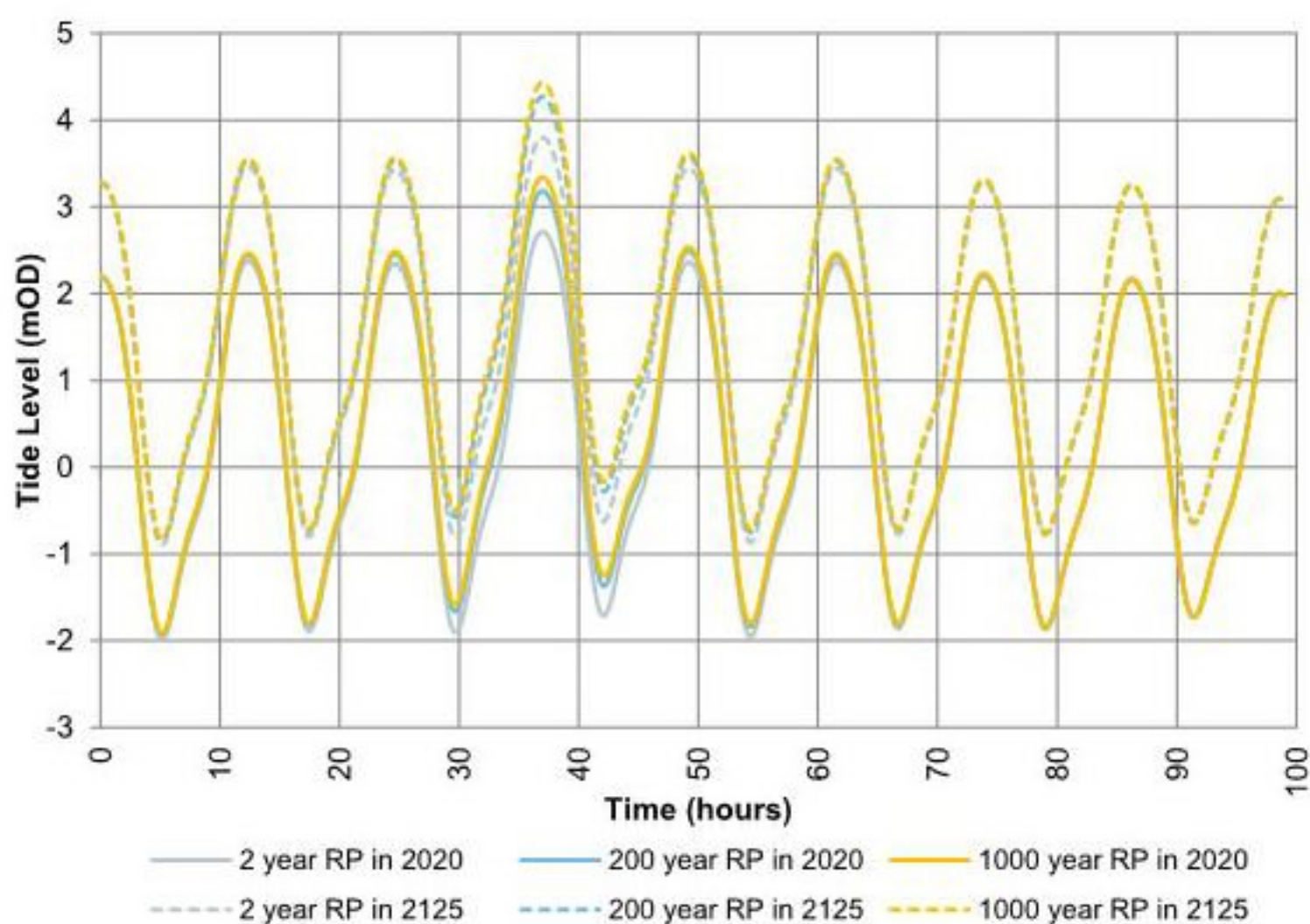


Figure 5 - Design Tide Curves for Higher Central scenario

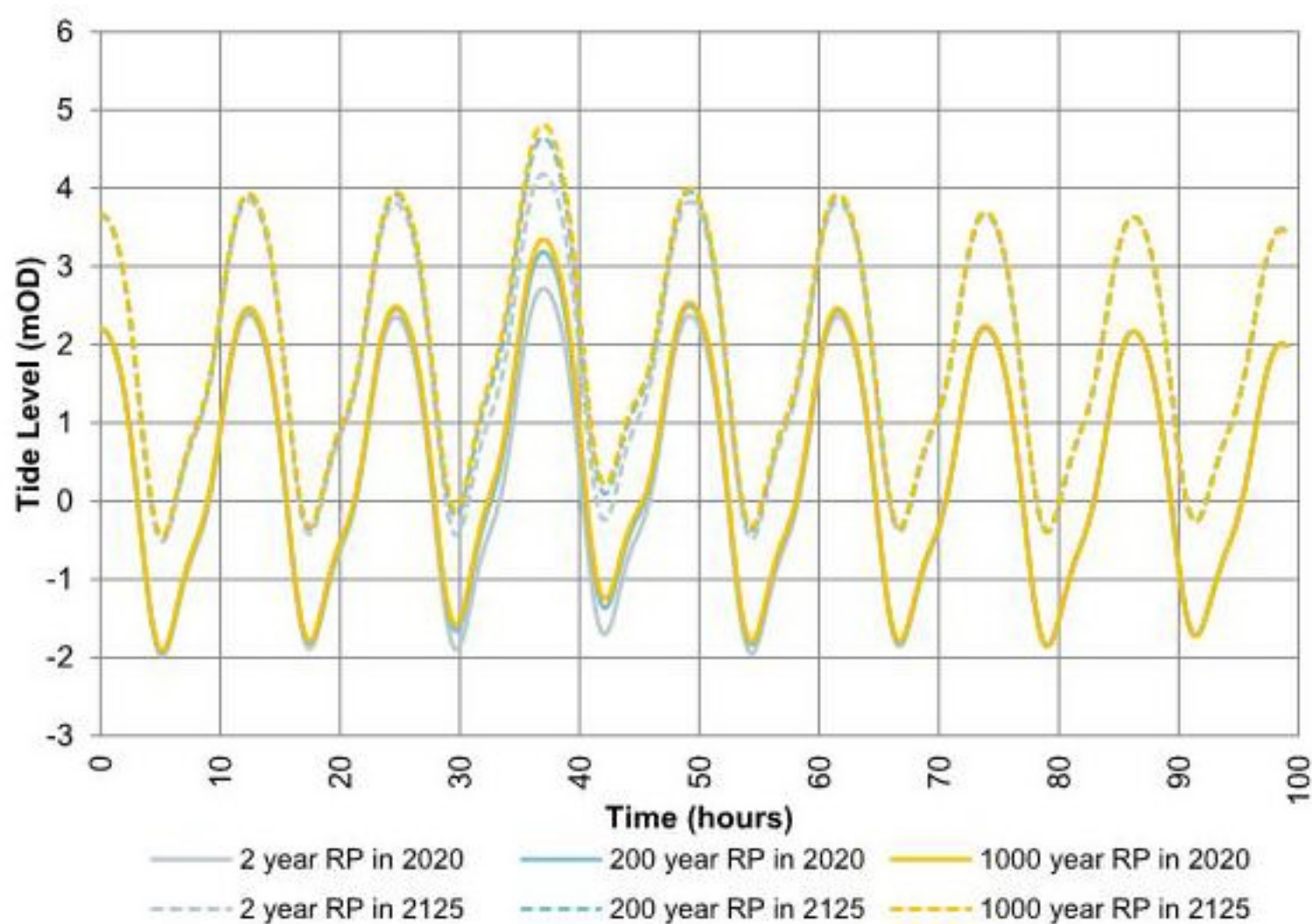


Figure 6 - Design Tide Curves for Upper End scenario

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ATKINS BASELINE - LOW RISK

Finally, it is noted that the base astronomical tide is for Langstone Harbour, while extreme sea levels used are seaward of the harbour entrance (extremes not available within the harbour). Comparing MHWS and HAT levels inside and outside the harbour, within the harbour MHWS is 0.2m higher and HAT is 0.38m higher. Extreme tide levels within the harbour are likely to be higher than the levels used; this is a data limitation. It is recommended that sensitivity testing is carried out within the hydraulic model to consider the impact of higher extreme water levels within the harbour.

References

- (1) United Kingdom Hydrographic Office (2017). *Admiralty Tide Tables United Kingdom and Ireland, Volume 1B. NP201B.*
- (2) Environment Agency (2019). *Coastal Flood Boundary Conditions for the UK: Update 2018. Technical Summary Report.*
- (3) GOV.UK (2020). *Flood Risk Assessments: Climate Change Allowances.*



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