HyNet North West Hydrogen Pipeline

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Appendix 7F

Flood Risk Assessment

October 2024

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Annex 7F3 Gale Brook Hydraulic Model Report
Annex 7F4 River Mersey Model User Report (To be provided for Final ES)
Annex 7F5 Puddinglake Brook Hydraulic Model Report
Annex 7F6 Outline Drainage Strategy

1. Introduction

1.1 Context

1.1.1 This Flood Risk Assessment (FRA) accompanies **Chapter 7: Water Environment**, of the Draft Environmental Statement (ES) for the HyNet North West Hydrogen Pipeline Project ('the Project').

1.1.2 The FRA should also be read in conjunction with the full description of the Project provided in **Chapter 2: The Project**.

1.1.3 The Project is a Nationally Significant Infrastructure Project (NSIP) under the Planning Act 2008 (the 'Act')[1]. Under Section 31 of the Act, development consent is required for development to the extent that it is or forms part of an NSIP. Development consent is granted by the making of a Development Consent Order (DCO) for which an application may be made under Section 37 of the Act to the Secretary of State (delegated to the Planning Inspectorate (PINS)). This FRA has been prepared to support the DCO application.

1.1.4 The FRA explores potential sources of flood risk to the Project from tidal, fluvial, surface water, groundwater and artificial sources. It also considers any potential impacts on flood risk arising from the Project on other external receptors. The potential impact of climate change on flood risk is considered throughout the assessment.

1.1.5 This FRA has been prepared in accordance with the overarching National Policy Statements (NPS) for Energy EN-1[2] and NPS for Gas Supply Infrastructure and Gas and Oil Pipelines EN-4[3].

[1]UK Government (2008) The Planning Act 2008. (online) Available at: https://www.legislation.gov.uk/ukpga/2008/29/contents (Accessed September 2024).
[2] Department of Energy Security & Net Zero (2023). Overarching National Policy Statement for Energy (EN-1). (Online) Available at: https://www.gov.uk/government/publications/overarching-national-policy-statement-for-energy-en-1 (Accessed June 2024).
[3] Department of Energy Security & Net Zero (2023). National Policy Statement for Gas Supply Infrastructure and Gas and Oil Pipelines (EN-4). (Online) Available at: https://www.gov.uk/government/publications/national-policy-statement-for-natural-gas-supply-infrastructure-and-gas-and-oil-pipelines-en-4 (Accessed June 2024).

1.1.6 Reference has also been made to the National Planning Policy Framework (NPPF)[4] and associated Planning Practice Guidance[5], where relevant, for additional guidance regarding flood risk and development, as appropriate. The development of this FRA has also been informed by consultation and engagement with key stakeholders (**Appendix 7A**), including the Environment Agency (EA), and the following Lead Local Flood Authorities (LLFAs) which overlap the project area: Cheshire East Council (CEC); Cheshire West and Chester Council (CWCC); Halton Borough Council (HBC); St Helens Borough Council (SHBC); Trafford Council (TC); and Warrington Borough Council (WBC).

1.2 Scope

1.2.1 This FRA considers the flood risks associated with the construction and operation /maintenance phases of the Project. Both flood risks to the Project from external sources and flood risks arising from the Project to external receptors are considered. The FRA covers all proposed infrastructure and activities within the Draft Order Limits, which define the anticipated maximum extent of land in which the Project, including construction works, would take place (as shown in **Figure 1.1, Chapter 1: Introduction**). This FRA follows a source-pathway-receptor led approach to the assessment of flood risk. Sources are defined as the source of the flood risk, such as direct rainfall, watercourses, the sea, groundwater or artificial sources. Pathways define the means by which the source of flood risk can impact receptors. A specific combination of sources and pathways is referred to as a flood mechanism, such as tidal overtopping of the sea defences as a result of high tides and storm surge. Receptors comprise those persons or assets that could be vulnerable to the flood mechanisms identified.

1.2.2 As a result of climate change, it is predicted that the climate conditions will change. The potential increases in peak rainfall intensities and magnitude of flood events have been taken into account over the Project's expected lifetime. With due consideration of the temporary nature of many of the elements of the Project, which are only required during construction of the pipeline. The approach taken in this assessment is considered to be proportionate to the risk and appropriate to the scale, nature and location of the Project.

[4] Ministry of Housing, Communities and Local Government (2021). National Planning Policy Framework (updated 20 December 2023). (Online) Available at: https://www.gov.uk/government/publications/national-planning-policy-framework--2. (Accessed June 2024).

[5] Ministry of Housing, Communities and Local Government (2021). Planning Practice Guidance on Flood Risk and Coastal Change Framework (updated 25 August 2022). (Online) Available at https://www.gov.uk/guidance/flood-risk-and-coastal-change. (Accessed June 2024).

1.3 Sources of Information and Consultation

1.3.1 Sources of existing data and information used in this FRA are provided in **Table 7F1.1**.

Table 7F1.1 Sources of existing data and information

Data	Source	Purpose
Aerial Imagery	Google Maps	Aerial views of the Draft Order Limits and surrounds to inform baseline conditions
Ordnance Survey (OS) maps	Ordnance Survey	Baseline information on the hydrological context, including topography, drainage and water features
Environment Agency Statutory Main River Map	Gov.uk Open Data[6]	Definition of watercourses, in relation to the development
Lidar data • **1m resolution** • **2m resolution**	Gov.uk Open Data[7]	Baseline information on topography and ground elevations. This data source also used in flood models
Topographic survey	Warner Surveys	Detailed cross section survey of the Gale Brook watercourse, incorporated in detailed flood modelling
Historic Flood Map	Environment Agency[8]	Mapping shows the maximum extent of individual recorded flood outlines from River, sea and groundwater sources
Flood Map for Planning	Gov.uk[9]	For assessment of fluvial and tidal flood risk

[6]Gov UK Open data (Online) Available at: https://data.gov.uk/
[7] Gov UK Open data (Online) Available at Gov.uk Open Data
[8] EA, Historic Flood Map (Online) Available at: https://www.data.gov.uk/dataset/76292bec-7d8b-43e8-9c98-02734fd89c81/historic-flood-map (accessed 02/02/23)
[9] Flood Map for planning (Online) Available at https://flood-map-for-planning.service.gov.uk/

Data	Source	Purpose
Long term flood risk map	Environment Agency[10]	Maps providing baseline information on the flood risk from rivers and sea, surface water and artificial sources
Risk of Flooding from Surface Water (RoFSW) Mapping	Gov.uk[11]	For assessment of surface water flood risk
Susceptibility to Groundwater Flooding Map (StGWF)	British Geological Survey (BGS)	For assessment of groundwater flood risk
Flood Risk from Reservoirs Mapping	Gov.uk[12]	For assessing reservoir flood risk
LLFA and Local Planning Authority (LPA) planning documents	Various online sources including websites of county councils	Includes plans and guidance such as Strategic and Preliminary Flood Risk Assessments (SFRAs and PFRAs), Catchment Management Plans and SuDs guidance
Bedrock and superficial geological mapping	British Geological Survey (BGS) Geology of Britain Viewer[13] BGS Onshore GeoIndex[14] BGS Hydrogeological mapping[15]	Geological Mapping

[10] Environment Agency, 2023, Long term flood risk map, https://check-long-term-flood-risk.service.gov.uk/map
[11] Risk of Flooding from surface Water mapping, (Online) Available at: https://flood-warning-information.service.gov.uk/long-term-flood-risk/map
[12] Gov.uk. Check the long term flood risk for an area in England. (Online) Available at: https://flood-warning-information.service.gov.uk/long-term-flood-risk/map
[13] BGS Geology of Britain. (Online) Available at: https://mapapps.bgs.ac.uk/geologyofbritain/home.html
[14] BGS. GeoIndex Onshore. (Online) Available at: http://mapapps2.bgs.ac.uk/geoindex/home.html
[15] BGS. Hydrogeological mapping. (Online) Available at: http://www.largeimages.bgs.ac.uk/iip/hydromaps.html?id=south-downs.jp2

Data	Source	Purpose
Aquifer Designations	Multi-Agency Geographic Information for the Countryside[16]	To characterise the underlying aquifers and hydrogeology
Environment Agency flood model data	Environment Agency modelled flood extents for the various flood models within the Draft Order Limits Environment Agency flood models: • Lower Weaver, 2020[17] • Mersey Estuary, 2018[18] • River Mersey (Warrington), 2020[19] • Manchester Ship Canal, 2018[20] • Stanlow Refinery, 2019[21]	For assessment of fluvial and tidal flood risks

1.3.2 Flood risk modelling identified as being required as a result of the Preliminary Flood Risk Assessment (PFRA) in 2022 has been undertaken (see **Annexes 7F1 to 7F5**) and the results are used to inform this FRA. All models are being shared with the Environment Agency for review. Some further work is ongoing following comments from the Environment Agency. All models are listed in **Table 7F1.2.**

Table 7F1.2 Status of flood models used in the assessment

Model	Source	Status
Mersey Estuary Model	Adapted from EA Flood Model	Final

[16] Defra. MAGIC map. (Online) Available at: https://magic.defra.gov.uk/home.htm
[17] JBA on behalf of the Environment Agency, 2020, Lower Weaver Flood Mapping Study Final Report
[18] JBA on behalf of the Environment Agency, 2018, Mersey Estuary, Ditton Brook and River Gowy Final Report
[19] Jacobs on behalf of the Environment Agency, 2020, Warrington FRMS Design Stage Hydraulic Model Report
[20] CH2M on behalf of the Environment Agency, 2018, Modelling 1, 15-16 (Inland Modelling): Manchester Ship Canal Model Report)
[21] Jacobs on behalf of the Environment Agency, 2019, Stanlow and Tranmere Flood Risk Management Scheme: Stanlow and Tranmere Hydraulic Modelling Report

Model	Source	Status
Lower Weaver Model	Adapted from EA Flood Model	Minor change outstanding on tidal boundary for Upper End fluvial scenario following EA review.
Gale Brook Model	Created by WSP by expanding on the EA model for the Stanlow Oil Refinery	Clarifications and justifications to be added to report following EA review.
River Mersey	Adapted from EA Flood Model	Internal WSP model review ongoing.
Puddinglake Brook Model	Created by WSP	Guidance on hydrology has changed during model development. New set of inflows is being prepared and model will be updated with these.

1.3.3 An Outline Drainage Strategy (**Annex 7F6**) considers all the permanent above ground installations including all HAGIs and BVIs. It also considers the drainage strategies to be applied to the greenfield pipe laydown areas along the pipeline route and provides the drainage strategy principles for temporary construction areas. Detailed drainage design will be carried out with reference to the National Planning Policy Framework (NPPF) and Building Regulations Part H, CIRIA C753 SuDS Manual, DEFRA/EA guidance and the relevant LLFA's where appropriate.

1.4 Terminology

Flood event probability and flood zone definition

1.4.1 Throughout this FRA, the terminology 'Annual Exceedance Probability' (AEP) is used to describe the magnitude and likelihood of a flood event occurring. AEP expresses the probability of a flood occurring in any given year. For example, what can sometimes be referred to as a '1 in 100 year flood event', is a flood with a 1 in 100 or 1% probability of occurring in any given year. Use of the AEP terminology makes it clearer that there is a probability of this magnitude of flooding occurring in any one year, not just once every 100 years.

1.4.2 **Table 7F1.3** provides to clarify the use of the AEP terminology as well a description of the Flood Zone definitions as set out in the NPPF[4] and its associated PPG[5] on Flood Risk and Coastal Change, and which are shown on the Environment Agency's Flood Map for Planning[9].

Table 7F1.3 Flood Zone definitions and associated annual exceedance probability

Probability of flooding	AEP	Definition
Flood Zone 1: Low probability	<0.1% AEP of river or sea flooding	Land with less than 1 in 1,000 probability of flooding from rivers or the sea, in any given year.
Flood Zone 2: Medium probability	1% - 0.1% AEP of river flooding 0.5% – 0.1% AEP of sea flooding	Land with between a 1 in 100 and 1 in 1,000 of river flooding; or land having between a 1 in 200 and 1 in 1,000 probability of sea flooding.
Flood Zone 3a: High Probability	>1% AEP of river flooding >0.5% AEP of sea flooding	Land having a 1 in 100 or greater probability of river flooding in any year; or Land having a 1 in 200 probability or greater of sea flooding in any year.
Flood Zone 3b: Functional Floodplain	Functional floodplain will normally comprise: land having a 3.3% or greater probability of flooding, with any existing flood risk management infrastructure operating effectively; or land that is designed to flood (such as a flood attenuation scheme), even if it would only flood in more extreme events (such as 0.1% annual probability of flooding)	This zone comprises land where water has to flow or be stored in times of flood. The identification of functional floodplain should take account of local circumstances and not be defined solely on rigid probability parameters. Local planning authorities should identify in their Strategic Flood Risk Assessments areas of functional floodplain and its boundaries accordingly, in agreement with the Environment Agency.

1.4.3 **Table 7F1.4** provides a summary of flood risk category definitions used in the Environment Agency's Risk of Flooding from Surface Water (RoFSW) mapping, which differ from those used to describe fluvial and tidal flood zones.

Table 7F1.4 Surface water flood risk category definitions and associated annual exceedance probability

Probability of flooding	AEP	Definition
Very low risk	<0.1% AEP of surface water flooding	Land with less than 1 in 1,000 probability of flooding from surface water, in any given year.
Low risk	0.1 – 1% AEP of surface water flooding	Land with between 1 in 1,000 and 1 in 100 probability of flooding from surface water, in any given year.
Medium risk	1 – 3.3% AEP of surface water flooding	Land with between 1 in 100 and 1 in 30 probability of flooding from surface water, in any given year.
High risk	>3.3% AEP of surface water flooding	Land with more than 1 in 30 probability of flooding from surface water, in any given year.

1.5 Structure of FRA

1.5.1 The remainder of this FRA is structured as follows:

- **Section 2** establishes the planning policy context for the assessment;
- **Section 3** provides an overview of the relevant characteristics of the Draft Order Limits and a description of the Project;
- **Section 4** comprises a screening assessment to consider the potential risk from all sources of flooding prevailing across the Draft Order Limits;
- **Section 5** presents the assessment of flood risks associated with the Project construction phase;
- **Section 5.8** presents the assessment of flood risks associated with the Project operational phase;
- **Section 7** specifies flood risk management measures to address the risks identified in **Section 5** and **Section 5.8**;
- **Section 8** sets out how the specific planning requirements have been / will be addressed in the final FRA, including the Sequential and Exception Tests; and
- **Section 9** presents summary and conclusions.

1.5.2 The FRA contains the following annexes:

- **Annex 7F1: Mersey Estuary Model User Report, for Rocksavage, Stanlow and Higher Walton HAGIs**;
- **Annex 7F2: Lower Weaver Model User Report, for Rocksavage HAGI**;
- **Annex 7F3: Gale Brook Hydraulic Model Report, for Stanlow HAGI**;
- **Annex 7F4: River Mersey Model User Report, for Higher Walton HAGI**;
- **Annex 7F5: Puddinglake Brook Hydraulic Model Report, for Inovyn HAGI**; and
- **Annex 7F6: Outline Drainage Strategy**.

2. Planning Policy Context and Requirements

2.1 Introduction

2.1.1 This section identifies the key planning policy documents that define the scope of this assessment and concludes by summarising the flood risk requirements applicable. This Section is structured in a hierarchical order, from national policy down to local guidance.

2.2 National policies and guidance

Planning Act 2008

2.2.1 The Project is defined as a Nationally Significant Infrastructure Project (NSIP) under Section 14(1)(f) of the Act[1] .Therefore, a DCO is required to authorise the Project.

2.2.2 The Act[1] requires that DCO applications must be determined in line with the requirements of the relevant NPSs, which provide the overarching principles relevant to major energy infrastructure and the policies against which applications for NSIPs should be determined.

2.2.3 In a hierarchical context, the FRA will be prepared in accordance with the Act[1], NPS EN-1[2], which sets out planning policy with regard to NSIPs in the energy sector, and NPS EN-4[3], which covers gas supply and storage. Reference will also be made to NPPF[4] and its associated PPG[5] as set out in the next sections.

National Policy Statement (NPS) for Energy (EN-1)

Overarching NPS for Energy (EN-1)

2.2.4 The NPSs set out UK Government planning policy for NSIPs in England and Wales. The Overarching NPS for Energy (EN-1)[2] establishes national policy for energy infrastructure and informs the decisions by the Secretary of State (SoS) for Business, Energy and Industrial Strategy (BEIS) on applications (DCOs) for energy developments that fall within the scope of the NPSs.

2.2.5 Section 4.10 of NPS EN-1 discusses climate change adaption.

2.2.6 The minimum requirements for all FRAs, irrespective of the development type, are set out in paragraph 5.8.15 of NPS EN-1. Section 5.8 also informs on the application of the Sequential and Exception Tests. These are set out in **Table 7F 2.1**, together with the location in which they are addressed in this assessment.

2.2.7 In addition to these, there are a number of other references to flood risk made throughout the NPS EN-1.

Table 7F 2.1 NPS EN-1 and minimum FRA requirements

FRA Element	EN-1 Minimum FRA Requirements	Section of this FRA
Policy	The development proposal should be in line with any relevant national and local flood risk management strategies. (Section: Applicant assessment, paragraph 4.6.12).	**Section 2**
Scope of FRA	Be proportionate to the risk and appropriate to the scale, nature and location of the Project. (Section: Applicant assessment, paragraph 5.8.15).	**Section 1.2**
Assessment	Consider the risk of flooding arising from the project in addition to the risk of flooding to the Project. (Section: Applicant assessment, paragraph 5.8.15).	**Sections 5 & 5.8**
Climate change	Take the impacts of climate change into account, across a range of climate scenarios, clearly stating the development lifetime over which the assessment has been made. (Section: Applicant assessment, paragraph 5.8.15).	**Section 2.6**
Climate change	Applicants should demonstrate that proposals have a high level of climate resilience built-in from the outset and should also demonstrate how proposals can be adapted over their predicted lifetimes to remain resilient to a credible maximum climate change scenario. These results should be considered alongside relevant research which is based on the climate change projections. (Section: Applicant assessment, paragraph 4.10.11).	**Section 2.2**, from **paragraph 2.2.16**
Approach	Be undertaken by competent people, as early as possible in the process of preparing the proposal. (Section: Applicant assessment, paragraph 5.8.15).	All
Baseline	Be supported by appropriate data and information, including historical information on previous events. (Section: Applicant assessment, paragraph 5.8.15).	**Table 7F1.1 Sources of existing data and information**
Vulnerability and safe access	Consider the vulnerability of those using the site, including arrangements for	**Section 7**

FRA Element	EN-1 Minimum FRA Requirements	Section of this FRA
	safe access and escape. (Section: Applicant assessment, paragraph 5.8.15).	**Chapter 7: Water Environment**, **Appendix 7D – Assessment of water environment effects**
Assessment	Consider and quantify the different types of flooding (whether from natural and human sources and including joint and cumulative effects) and include information on flood likelihood, speed-of-onset, depth, velocity, hazard and duration. (Section: Applicant assessment, paragraph 5.8.15).	**Sections 5 & 5.8**
Assessment	Identify and secure opportunities to reduce the causes and impacts of flooding overall, making as much use as possible of natural flood management techniques as part of an integrated approach to flood risk management. (Section: Flood Risk, paragraph 5.8.12).	**Section 7** **Chapter 7: Water Environment, Appendix 7D – Assessment of water environment effects**
Assessment	Consider the effects of a range of flooding events including extreme events on people, property, the natural and historic environment and river and coastal processes. (Section: Applicant assessment, paragraph 5.8.15).	**Sections 5 & 5.8** **Chapter 7: Water Environment, Appendix 7D – Assessment of water environment effects**
Assessment	Detail those measures that will be included to ensure the development will be safe and remain operational during a flooding event throughout the development's lifetime without increasing flood risk elsewhere (Section: Applicant assessment, paragraph 5.8.15).	**Section 7**
Assessment	The Project is designed and constructed to remain safe and operational during its lifetime, without increasing flood risk elsewhere (subject to the exceptions set out in Section: Secretary of State decision making, paragraph 5.8.42). The Project includes safe access and escape routes where required, as part of an agreed emergency plan, and that any residual risk	**Section 7**

FRA Element	EN-1 Minimum FRA Requirements	Section of this FRA
	can be safely managed over the lifetime of the development (paragraph 5.8.36)	
Residual risks	Include the assessment of the remaining (known as 'residual') risk after risk reduction measures have been taken into account and demonstrate that these risks can be safely managed, ensuring people will not be exposed to hazardous flooding; (Section: Applicant assessment, paragraph 5.8.15).	**Chapter 7: Water Environment, Appendix 7D – Assessment of water environment effects**
Functional floodplain	Development should be designed to ensure there is no increase in flood risk elsewhere, accounting for the predicted impacts of climate change throughout the lifetime of the development. There should be no net loss of floodplain storage and any deflection or constriction of flood flow routes should be safely managed within the site. Mitigation measures should make as much use as possible of natural flood management techniques. (Section: Applicant assessment, paragraph 5.8.15)	**Section 7**
Flood risk management infrastructure	Consider both the potential adverse and beneficial effects of flood risk management infrastructure, including raised defences, flow channels, flood storage areas and other artificial features, together with the consequences of their failure and exceedance; (Section: Applicant assessment, paragraph 5.8.15).	N/A flood protection for the Project does not rely on existing flood defences
Vulnerability and safe access	Consider the vulnerability of those using the site, including arrangements for safe access and escape; (Section: Flood Risk, paragraph 5.8.12)	**Section 7** **Chapter 7: Water Environment, Appendix 7D – Assessment of water environment effects**
Flood warning and evacuation plan	The applicant should take advice from the local authority emergency planning team, emergency services and, where appropriate, from the local resilience forum when producing an evacuation plan for a manned energy project as part of the FRA. Any	**Section 7** **Chapter 7: Water Environment, Appendix 7D – Assessment of water environment effects**

FRA Element	EN-1 Minimum FRA Requirements	Section of this FRA
	emergency planning documents, flood warning and evacuation procedures that are required should be identified in the FRA. (Section: Mitigation, paragraph 5.8.34)	
Surface water runoff	Consider how the ability of water to soak into the ground may change with development, along with how the proposed layout of the Project may affect drainage systems. (Section: Applicant assessment, paragraph 5.8.15)	**Annex 7F6: Outline Drainage Strategy**
Drainage and Sustainable Drainage Systems (SuDS)	The applicant should give priority to the use of SuDS and make provision for their adoption and maintenance SuDS (as required in the next paragraph on National Standards) have been used unless there is clear evidence that their use would be inappropriate (Section: Secretary of State decision making, paragraph 5.8.36)	**Annex 7F6: Outline Drainage Strategy**
Drainage and SuDS	For energy Ppjects which have drainage implications, approval for the Project's drainage system, including during the construction period, will form part of the development consent issued by the Secretary of State. The Secretary of State will therefore need to be satisfied that the proposed drainage system complies with any National Standards published by Ministers under paragraph 5(1) of Schedule 3 to the Flood and Water Management Act 2010. (Section: Secretary of State decision making, paragraph 5.8.37)	**Annex 7F6: Outline Drainage Strategy**
Drainage and SuDS	Site layout and surface water drainage systems should be designed to cope with events that exceed the design capacity of the system, so that excess water can be safely stored on or conveyed from the site without any adverse impacts (Section: Mitigation, paragraph 5.8.26).	**Annex 7F6: Outline Drainage Strategy**
Drainage and SuDS	The surface water drainage arrangements for any Project should, accounting for the predicted impacts of climate change throughout the development's lifetime, be	**Annex 7F6: Outline Drainage Strategy**

FRA Element	EN-1 Minimum FRA Requirements	Section of this FRA
	such that the volumes and peak flow rates of surface water leaving the site are no greater than the rates prior to the proposed Project, unless specific off-site arrangements are made and result in the same net effect (Section: Mitigation, paragraph 5.8.27).	
Sequential Test	The Sequential Test and sequential approach should be applied. If essential energy infrastructure has to be located in such areas for operational reasons, they should only be consented if the development will not result in a net loss of floodplain storage and will not impede water flows. (Section: Secretary of State decision making, paragraph 5.8.41)	**Section 8.1 The Sequential Test**
Exception Test	The Exception Test, where necessary, should be applied. (Section: Flood Risk, paragraph 5.8.10)	**Section 8.2** The Exception Test

National Policy Statement (NPS) for Oil and Gas Supply and Storage (EN-4)

2.2.8 The NPS for Oil and Gas Supply Infrastructure and Gas and Oil Pipelines (EN-4[3]) establishes national policy for certain energy infrastructure and informs the decisions by the Secretary of State for BEIS on applications for energy developments that fall within the scope of the Act.

2.2.9 Sections of NPS EN-4 that are relevant to this assessment are as follows:

- Section 2.3 of NPS EN-4 Climate change adaptation refers to flood risk, stating that applicants should set out how the proposal will be resilient to increased risk of flooding.
- Section 2.9.35 of NPS EN-4 on Water quality states that in the case of aquifer storage the applicant should assess the impact of the displacement of groundwater with respect to its potential interference with groundwater flow pathways and flood risk.
- Section 2.22.0 of NPS EN-4 refers to the use of sustainable drainage systems to control runoff as part of mitigation measures.

National Planning Policy Framework (and associated Planning Practice Guidance)

2.2.10 The NPPF[4] sets out the UK Government's planning policies for England and how these should be applied. It provides a framework for local planning authorities and

decision makers, both for drawing up plans and making decisions about planning applications. This is supported by Planning Practice Guidance[5].

2.2.11 Although the NPPF and the associated Planning Practice Guidance (PPG) are not directly applicable to NSIP developments, they do provide additional relevant guidance on a range of issues, including the definition of flood zones, development vulnerability classifications, compatibility of development types and flood zones, and appropriate allowances for the effects of climate change. Furthermore, the 2023 update of the energy infrastructure NPSs has brought the flood risk requirements of NPS EN-1 into alignment with those within NPPF.

2.2.12 The NPPF sets out requirements for FRA for new developments and describes how the Sequential (Paragraph 167 and 168) and Exception Tests (Paragraph 169 and 170) should be applied, depending on the Flood Zone that the Project is located in, and its Flood Vulnerability classification. Paragraph 170 states that:

> *"To pass the exception test it should be demonstrated that:*
>
> *a) the development would provide wider sustainability benefits to the community that outweigh the flood risk; and*
>
> *b) the development will be safe for its lifetime taking account of the vulnerability of its users, without increasing flood risk elsewhere, and, where possible, will reduce flood risk overall."*

2.2.13 Paragraph 173 of the NPPF requires that new development should not increase flood risk elsewhere, and that opportunities should be sought to reduce flood risk, where possible. Paragraph 173 states:

> *"…Development should only be allowed in areas at risk of flooding where, in the light of this assessment (and the sequential and exception tests, as applicable) it can be demonstrated that:*
>
> *a) within the site, the most vulnerable development is located in areas of lowest flood risk, unless there are overriding reasons to prefer a different location;*
>
> *b) the development is appropriately flood resistant and resilient such that, in the event of a flood, it could be quickly brought back into use without significant refurbishment;*
>
> *c) it incorporates sustainable drainage systems, unless there is clear evidence that this would be inappropriate;*
>
> *d) any residual risk can be safely managed; and*
>
> *e) safe access and escape routes are included where appropriate, as part of an agreed emergency plan."*

2.2.14 Paragraph 175 of the NPPF states:

> *"Major developments should incorporate sustainable drainage systems unless there is clear evidence that this would be inappropriate. The systems used should:*
>
> *a) take account of advice from the lead local flood authority;*

b) have appropriate proposed minimum operational standards;

c) have maintenance arrangements in place to ensure an acceptable standard of operation for the lifetime of the development; and

d) where possible, provide multifunctional benefits."

2.2.15 There is a presumption for the use of sustainable drainage systems (SuDS) within any development, except in rare instances that it can be demonstrated that SuDS principles cannot be feasibly incorporated within a development, as agreed with the planning authority.

Climate change allowances for FRA

2.2.16 NPS EN-1 and NPS EN-4 give advice on accounting for climate change, to the effect that developments should be resilient and adaptive to the latest climate change projections. EN-1 states that the most up to date UK climate projections should be used in the assessment of climate change effects on flood risk over the development lifetime.

2.2.17 Current Environment Agency Flood Risk Assessments: climate change allowances guidance[22] has been used to determine appropriate climate change allowances to determine future flood hazard for the FRA. The climate change allowances in the latest version of this guidance are based on the UKCP18 Climate Projections. Relevant allowances for the Project location are summarised in **Table 7F 2.2** to **Table 7F 2.4**.

2.2.18 The Environment Agency guidance[22] provides climate change allowances for extreme rainfall and river flows by individual operational management catchment. The Project crosses three Environment Agency Management Catchments which are listed below (from north to south).

- Lower Mersey;
- Upper Mersey; and
- Weaver Gowy.

2.2.19 The climate change allowance relevant to the Management Catchments listed below are set out in **Table 7F** 2.2 for river flows and **Table 7F** 2.3 for rainfall.

Table 7F 2.2 Climate change allowances – peak river flow allowances

Allowance category	Total potential change anticipated for the '2020s' (2015 to 2039)	Total potential change anticipated for the '2050s' (2040 to 2069)	Total potential change anticipated for the '2080s' (2070 to 2115)
Peak river flows – Lower Mersey Management Catchment			

[22] Environment Agency (2022) Flood Risk Assessments: climate change allowances guidance. Available at: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances (Accessed August 2024).

Allowance category	Total potential change anticipated for the '2020s' (2015 to 2039)	Total potential change anticipated for the '2050s' (2040 to 2069)	Total potential change anticipated for the '2080s' (2070 to 2115)
Upper end	32%	55%	90%
Higher central	22%	35%	57%
Central	18%	27%	44%
Peak river flows – Upper Mersey Management Catchment			
Upper end	27%	51%	85%
Higher central	17%	31%	53%
Central	13%	22%	41%
Peak river flows – Weaver Gowy Management Catchment			
Upper end	36%	64%	106%
Higher central	24%	40%	67%
Central	19%	30%	52%

(Based on a 1981 – 2000 baseline)

Table 7F 2.3 Climate change allowances – peak rainfall allowances

Allowance category	Total potential change anticipated for the '2050s' (to 2060)	Total potential change anticipated for the '2070s' (2061 to 2125)
Peak rainfall – Lower Mersey Management Catchment – 3.33% AEP		
Upper end	35%	40%
Central	20%	30%
Peak rainfall – Lower Mersey Management Catchment – 1% AEP		
Upper end	40%	45%
Central	25%	30%
Peak rainfall – Upper Mersey Management Catchment – 3.33% AEP		
Upper end	35%	40%
Central	20%	30%

Allowance category	Total potential change anticipated for the '2050s' (to 2060)	Total potential change anticipated for the '2070s' (2061 to 2125)
Peak rainfall – Upper Mersey Management Catchment – 1% AEP		
Upper end	40%	45%
Central	25%	30%
Peak rainfall – Weaver Gowy Management Catchment – 3.3% AEP		
Upper end	35%	40%
Central	20%	25%
Peak rainfall – Weaver Gowy Management Catchment – 1% AEP		
Upper end	40%	45%
Central	25%	30%

(Based on a 1981 – 2000 baseline)

2.2.20 The Project is classified as Essential Infrastructure, therefore the guidance[4] states that for Essential Infrastructure in Flood Zones 2 or 3 the Higher Central allowance should be applied to river flows.

2.2.21 Additionally, as the Project is classed as a NSIP, the Environment Agency guidance[23] suggests sensitivity testing any design using a 'credible maximum' future scenario. For river flows, this corresponds to the 2080s Upper End allowance.

2.2.22 The design life of the pipeline and HAGIs is 40 years but it is anticipated that the actual life of the pipeline could extend well over 40 years depending on its condition, as over time parts are likely to be refurbished or replaced.

2.2.23 For the FRA, it is assumed the lifetime of the Project is not expected to extend beyond 2100. When considering surface water flooding, any development with a lifetime up to 2100 must be designed to be safe and ensure no increase in flood risk elsewhere for the central rainfall allowance in the 1% AEP event. There is no requirement to assess a credible maximum surface water event.

2.2.24 Future uplifts in sea level are specified regionally. Those for North West England are detailed in **Table 7F 2.4**. FRAs must assess both Upper End and Higher Central when considering future tidal flood risk.

[23] Environment Agency (2022b) Flood Risk Assessments: climate change allowances guidance. Available at: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances#credible-maximum-scenarios (Accessed August 2024).

2.2.25 Again, as the Project is classed as an NSIP, a maximum credible sensitivity test applying the H++ scenario of 1.9m for the total sea level rise to 2100 is required to demonstrate the resilience of infrastructure to tidal flooding over its lifetime.

Table 7F 2.4 Climate change allowances – sea level (North West)

Allowance	2000 to 2035 (mm)	2036 to 2065 (mm)	2066 to 2095 (mm)	2096 to 2125 (mm)	Cumulative rise 2000 to 2125 (metres)
Upper End	5.7 (200)	9.9 (297)	14.2 (426)	16.3 (489)	1.41
Higher Central	4.5 (158)	7.3 (219)	10 (300)	11.2 (336)	1.01

2.3 Local Planning Policy

2.3.1 The Project traverses the areas of six local planning authorities (LPAs), Cheshire East (CEC) Council, Cheshire West and Chester Council (CWCC), Halton Borough Council (HBC), St Helens Borough Council (SHBC), Trafford Council (TC) and Warrington Borough Council (WBC). Although, as an NSIP, the Project is not subject to LPA consent, LPAs are Statutory Consultees in the DCO process, and the Secretary of State may consider local planning policy to be relevant to the examination process. Local planning policies of relevance to flood risk and the Project are summarised below. Local planning policies should be aligned with the requirements of NPPF at national level, and this has generally been found to be the case.

Cheshire East Council

2.3.2 Cheshire East Council Local Plan Strategy (2017)[24], Policy SE13 (Flood Risk and Water Management) requires all developments at risk of flooding should be supported by an appropriate FRA to reduce the risk of flooding, taking into account the impacts of climate change in line with the Cheshire East Strategic Flood Risk Assessment (SFRA). In addition, improvements to the current surface water drainage network are sought and to be designed to manage surface water. This should include appropriate SuDS and must not result in an increase in runoff. Developments should enhance and protect surface and ground water quality and comply with the Water Framework Directive by ensuring suitable mitigation measures are in place.

[24] Cheshire East Council Local Plan Strategy (2017). Available at: local-plan-strategy-web-version-1 (cheshireeast.gov.uk) (Accessed August 2024)

Cheshire West and Chester Council

2.3.3 Cheshire West and Chester Adopted Development Plan (2015)[25], Policy ENV1 (Flood Risk and Water Management) seeks to reduce flood risk and to protect and enhance water quality through an appropriate FRA, mitigation. The drainage of the new development shall be designed to reduce surface water run-off rates to include the implementation of SuDS.

Halton Borough Council

2.3.4 Halton Borough Council Core Strategy (2013)[26], Policy CS19 (Sustainable Development and Climate Change) requires development to incorporate appropriate climate change resilience including passive design features, conservation measures, and the management of surface water run-off. Policy CS23 (Managing Pollution and Risk) requires management of flood risk and not to exacerbate existing levels of flood risk nor place residents or property at risk.

St Helens Borough Council

2.3.5 St Helens Local Plan Core Strategy (2012[27]), Policy CP 1 (Ensuring Quality Development in St Helens) seeks to avoid adding to the causes and sources of flood risk and ensuring no adverse impact on other sites. Where this cannot be achieved, but there is an overriding need for the development and a suitable alternative site cannot be identified, protection from the risk of flooding, through the use of SuDS and/or other appropriate measures and/or flood defences will be required.

Trafford Council

2.3.6 Trafford Local Plan: Core Strategy (2012)[28], Policy L5 (Climate Change) states that any development that has potential to cause adverse pollution of water or ground will not be permitted unless it includes mitigation measures. Trafford Council will also control development in areas at risk of flooding considering the level of risk, requiring an appropriate FRA at the planning application stage. In addition, surface water run-off should be reduced through the use of appropriate SuDS based on mapping within Trafford Council's SFRA.

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[25] Cheshire West and Chester Adopted Development Plan (2015). Available at: Cheshire West and Chester Local Plan (Part One) Strategic Policies - Keystone (Accessed August 2024)

[26] Halton Borough Council Core Strategy (2013). Available at CS Final Front Cover 2013.pdf (halton.gov.uk) (Accessed August 2024)

[27] St. Helens Local Plan Core Strategy (2012). Available at: St.Helens Local Plan Core Strategy (sthelens.gov.uk) (Accessed August 2024)

[28] Trafford Local Plan: Core Strategy (2012). Available at: trafford.gov.uk/planning/strategic-planning/docs/core-strategy-adopted-final.pdf (Accessed: August 2024)

Warrington Borough Council

2.3.7 Warrington Borough Council Local Plan (2021/2023)[29] Policy ENV2 (Flood Risk and Water Management) outlines the requirements to manage surface water run-off, to ensure that flood risk is not increased, the use of SuDS, floodplain compensatory storage and safe design within development proposals. Any development identified as at intermediate and high risk of surface water flooding (greater than 0.5 hectares) should be supported by a site-specific FRA, which considers information in Warrington's SFRA and preliminary Flood Risk Assessment (pFRA).

2.4 Other Relevant Plans and Consenting Requirements

2.4.1 A number of bodies with responsibility for management and regulation of the water environment have produced plans that are of relevance to this assessment. These bodies also have responsibilities for the regulation of activities in and around watercourses that could affect flood risk. Within the Draft Order Limits, these comprise the EA and LLFAs[30]. Relevant plans produced by these bodies and their consenting powers are summarised below.

Environment Agency

2.4.2 The EA is the lead statutory body with responsibility for protection of the water environment. It is also responsible for flood defence and drainage for Main Rivers (Main River is a statutory designation which is usually applied to larger watercourses) and estuarine and coastal areas. The EA has produced regional management plans and policies for the water environment; the key documents of relevance to this FRA are:

- Lower Mersey Catchment Flood Management Plan (CFMP);
- Upper Mersey Catchment Flood Management Plan; and
- Weaver Gowy Catchment Flood Management Plan.

2.4.3 **Table 7F 2.5** shows the relevant sub-areas and policies for each CMFP. The Draft Order Limits span five policy areas, as defined within the plans, as follows:

- Policy 2 – "*areas of low to moderate flood risk where we [the Environment Agency] can generally reduce existing flood risk management actions.*"

[29] Warrington Borough Council Local Plan (2021/2023). Available at: Adopted Local Plan 2021/22 – 2038/39 | warrington.gov.uk (Accessed: August 2024)

[30] Internal Drainage Boards (IBDs) are responsible for maintaining drainage networks in low-lying agricultural areas of many parts of England and Wales. They produce management plans, and exercise Ordinary Watercourse Consenting powers for their operational areas. However, there are no IDBs in the Order Limits for the Project, so they are not discussed further in this document.

- Policy 4 – "*areas of low, moderate or high flood risk where we are already managing the flood risk effectively but where we may need to take further actions to keep pace with climate change.*"
- Policy 5 – "*areas of moderate to high flood risk where we can generally take further action to reduce flood risk.*"
- Policy 6 – "*areas of low to moderate flood risk where we will take action with others to store water or manage run-off in locations that provide overall flood risk reduction or environmental benefits.*"

Table 7F 2.5 Catchment Flood Management Plan sub-areas and policies

Catchment Flood Management Plan	Sub-area	Policy Option
Upper Mersey	Bollin	4
	Lower Sinderland	5
Mersey Estuary	Warrington (Lower Sankey, Padgate and Woolston)	5
	Middle Mersey Estuary	4
	Widnes and Penketh	2
	St Helens	2
Weaver Gowy	Frodsham and Runcorn	4
	Northwich	5
	Stanlow	4
	Rural Areas	6

2.4.4 The Environment Agency is a Statutory Consultee in the DCO process and is also responsible for regulating engineering works liable to affect Main Rivers through the issuing of Flood Risk Activities Permits (FRAPs). Under Schedule 25 of the Environmental Permitting Regulations 2016, any works within 8m of a Main River banktop (or 8m of the landward side of a flood defence structure or culvert), increasing to 16m for a tidal Main River, and also some works within Main River floodplains, would require FRAP from the Environment Agency.

Lead Local Flood Authorities (LLFAs)

2.4.5 LLFAs were defined as risk management authorities under the Flood and Water Management Act 2010[31]. LLFAs are responsible for the management of local flood risk (i.e. all other flood risk except that from Main Rivers and the sea). This is done though the process of developing Preliminary Flood Risk Assessments (pFRAs) and Local Flood Risk Management Strategies (LFRMS). LLFAs are also responsible for regulating engineering works likely to affect Ordinary Watercourses through issuing Ordinary Watercourse Consents (OWCs) outside of IDB areas. Ordinary watercourses are all streams, drains, ditches and passages through which water flows that are not classified by the Environment Agency as a Main River.

2.4.6 The Draft Order Limits traverses six LLFA areas, which align with the LPA areas outlined in **paragraph 2.3.1** including: CEC, CWCC, HBC, SHBC, TC and WBC. A summary of relevant assessments and guidance produced by each of the LLFAs is provided below. OWCs would need to be applied for any works in and around Ordinary Watercourses within the standoff distances specified by the individual LLFAs.

2.4.7 Five of the six LLFAs which the Draft Order Limits traverse are part of the Cheshire and Mid-Mersey (CMM) sub-regional LLFA working group which agrees to follow the same principles in producing each individual Local Flood Risk Management Strategy (LFRMS) with the aim of ensuring a catchment wide approach. The catchment partners in the CMM group are CEC, CWCC, HBC, SHBC and WBC.

Cheshire East Council

2.4.8 Cheshire East Council's LFRMS was published in 2017, building on the high-level findings of the pRFA (2011) and SFRA (2013). The LFRMS provides principles for managing flood risk within Cheshire East. Cheshire East requires OWC for any structure, permanent or temporary, within 8m of an Ordinary Watercourse.

Cheshire West and Chester Council

2.4.9 The Cheshire West and Chester Council LFRMS (2016) indicates key sources of flooding, including from the Rivers Weaver and Dane and surface water runoff. The SFRA (2016) also indicates that surface water flooding is a risk in the region in areas with flat terrain.

2.4.10 CWCC have produced guidance from Ordinary Watercourse consenting which requires OWC for any structure, permanent or temporary, within 8m of an Ordinary Watercourse.

[31] Flood and Water Management Act 2010. Available at: Flood and Water Management Act 2010 (legislation.gov.uk) (Accessed: September 2024)

Halton Borough Council

2.4.11 The Halton Borough Council LFRMS (2015) details the sources of flood risk in the Borough, including fluvial, surface water and canals. The SFRA (2011) also discusses the residual risks associated with the canals present in the Borough. No information on OWC is available from HBC.

St Helens Borough Council

2.4.12 The St Helens Borough Council LFRMS (2014) details the potential flood risk in the Borough and the effect of climate change on future flood risk. The position of St Helens at the top of the Mersey sub-catchment means that the fluvial flood risk is generally low. The pFRA (2017) identifies surface water flooding as a major source of flood risk in the Borough.

Trafford Council

2.4.13 Trafford Council are responsible for local flood risk management within their region, which involves developing and maintaining a strategy for the management of local flood risk. The LFRMS (2014) details the various sources of flood risk including surface water, groundwater, fluvial and canal. The SFRA (2011) highlights that surface water poses the most significant risk in the region.

2.4.14 Trafford Council uses the North West SuDS Pro-forma, which states that if development proposals are within 8m of the top of the banks of an Ordinary Watercourse, or make changes to such a watercourse, then a OWC may be required in addition to planning permission.

Warrington Borough Council

2.4.15 Warrington Borough Council LFRMS (2017) sets out their position in relation to managing, regulating and coordinating the local flood risk, including flooding from other sources, including surface water, groundwater and the sewer network. The SFRA (2011) highlights that the River Mersey and its tributaries that run through the borough are the primary source of flood risk. Tidal flooding is also highlighted as a risk due to the tidal influence on the River Mersey extending to Howley Weir in Warrington.

2.4.16 Warrington Borough Council works with the Environment Agency and United Utilities on the Sankey Brook Flood Risk Management Scheme. The Scheme is located east of Warrington, north of the proposed Higher Walton HAGI (HWH 1). The Scheme does not intersect with the Draft Order Limits of the Project, however further consultation will be undertaken to determine whether changes to flow rates in the River Mersey may affect the Project.

2.4.17 The Scheme aims to reduce tidal and fluvial flood risk to more than 1,700 residential and business properties currently at risk of flooding with a 1% chance of occurring in any given year. The Scheme comprises linear defences (maximising use of natural floodplain), conveyance improvements at Old Liverpool Road (Sankey Bridges), a tidal gate on Dallam Brook, refurbishment of Longford Barrage (UU asset), residual flood risk mitigation measures (SuDs, attenuation, localised pumping), and local flood resilience measures. The scheme is expected

to gain approval in November 2024, and construction is planned for June 2029 to November 2031.

2.5 Other Technical Guidance

SuDS guidance

CIRIA SuDS manual (C753)

2.5.1 The Construction Industry Research and Information Association (CIRIA) SuDS (C753) is the most up-to-date industry standard containing revised principles and technical advice for the planning, design, construction, management, and maintenance of effective SuDS. The drainage systems for new developments should be designed to align with the SuDS manual.

Defra Non-statutory technical standards for sustainable drainage systems, 2015

2.5.2 The Department for Environment, Food and Rural Affairs (Defra) Non-Statutory Technical Standards for Sustainable Drainage Systems is a national guidance document that provides a set of standards to be applied when designing SuDS systems for new developments. Standards include controls on peak flow and volume of run-off, plus flood risk internal to the development and downstream. These comprise a common national set of basic standards for England, which are used as the basis of more specific local guidance prepared by individual LLFAs, as discussed below.

Lead Local Flood Authority surface water drainage advice

2.5.3 Each LLFA produces guidance with regards to their requirements for drainage design and runoff estimation, which are all principally based upon the non-statutory guidance set out above. They each set out and promote the same drainage hierarchy as follows:

- Prevention of runoff by good site design and reduction of impermeable areas.
- Source Control: Dealing with water where and when it falls (e.g. infiltration techniques).
- Site Control: Management of water in the local area (e.g. swales, detention basins).
- Regional Control: Management of runoff from sites (e.g. balancing ponds, wetlands).

2.5.4 The methods used to discharge surface water should be prioritised in the following order: infiltration to ground; watercourse; and combined/surface water sewer.

2.5.5 Local SuDS standards and drainage requirements for each of the LLFAs affected by the Project are summarised in **Table 7F 2.6**.

Table 7F 2.6 Local SuDS standards per LLFA

	Cheshire East	Cheshire West and Chester	Halton*	St Helens*	Trafford	Warrington
Consider connections to highways and drains	Consent is needed, as well as consent from the owner of the drain or sewer pipes.	Not required, unless solely water from highway gullies	N/A	N/A	Not Specified	Consultation is required withthe Highways Authority if SuDS would impact on adopted public highways or discharge surface water to highways and drainage systems.
Minimum Brownfield Discharge Rate reduction	Discharge rates will need to be reduced in line with the non-Statutory Technical Standards for Sustainable Drainage Systems (2015).	Must be designed to 30% reduction	N/A	N/A	Must be designed to greenfield site rates	Existing brownfield rates will need to be taken into consideration.
Greenfield Discharge Rate to be used	Greenfield rate or less will need to be used	Use of IH124 method appropriate	N/A	N/A	Must be designed to greenfield rate or less	Must be designed to greenfield rate or less

	Cheshire East	Cheshire West and Chester	Halton*	St Helens*	Trafford	Warrington
Climate Change Allowance	Not specified.	Must use the Environment Agency climate change rainfall intensity allowance.	N/A	N/A	Must use 40%	Must use 20% allowance on rainfall intensity for non-residential developments.
Urban Creep Allowance	Not specified.	Must use 10% urban creep allowance	N/A	N/A	Must use 10% urban creep allowance	Urban creep allowance depends on number of dwellings per ha: If 25 or fewer dwellings/ha, use 10% 26-30 dwellings/ha, use 8% 31-35 dwellings/ha, use 6% 36-45 dwellings/ha, use 4% If 50 or more dwellings/ha, use 2%

	Cheshire East	Cheshire West and Chester	Halton*	St Helens*	Trafford	Warrington
						For Flats and Apartments, use 0%
Identified Special Areas of Drainage	Development proposals for critical drainage areas must address and mitigate known risks in the area.	All development in designated Critical Drainage Areas (CDAs) would be required to incorporate measures to alleviate surface water flood risk through the layout and form of the development, including the appropriate application of SuDS to intercept and attenuate overland flow and drained water.	N/A	N/A	Developments in a CDA must submit a Site-Specific FRA, Sustainable Drainage Strategy and a Sustainable Drainage Strategy Pro Forma in the planning application.	Recommended that within CDAs, a reduction of 50% in surface water discharge rates from new development on brownfield sites and a reduction to greenfield rates on all other development sites is achieved.
Local SuDS Guidance	All developments in Cheshire East	Must consider biodiversity and water quality	N/A	N/A	N/A	Must design SuDS so that they are easy to maintain

	Cheshire East	Cheshire West and Chester	Halton*	St Helens*	Trafford	Warrington
	are required to have SuDS.	benefits into SuDS design.				and minimise risk of blockage. Must demonstrate how pollutants are prevented or controlled by SuDS scheme. Must follow principles of the SuDS Management Train.

* Local SuDS guidance for Halton and St. Helens has not been made available.

2.6 Summary of Design Standards for Assessment

2.6.1 Design standards for flood protection measures for flood vulnerable critical elements of project infrastructure are specified **Table 7F 2.7.**

2.6.2 Use of the fluvial 1% AEP event and the 0.5% AEP tidal event as the design standard for flood protection measures is specified in the NPPF PPG. In line with National Grid's approach to use the fluvial 0.5% AEP event as a design standard for electricity and gas transmission infrastructure, the Applicant has made the decision to also use the fluvial 0.5% AEP event as the design standard for the Project.

2.6.3 The latest Environment Agency climate change guidance makes it clear that assessment of surface water flood risks should be assessed up to the 1% AEP event.

2.6.4 Given the short construction period proposed for the pipeline, it has been agreed in principle with the Environment Agency[32] that construction phase elements are to be assessed using the present day flood risk estimates only, with no allowance for climate change.

2.6.5 With regards to allowances for future climate change for flood protection and drainage during the operational phase of the Project, these have been based on the current guidance provided by the EA for FRA[33], assuming the lifetime of the infrastructure does not extend beyond 2100.

2.6.6 However, some LLFAs require a climate change uplift for surface water higher than that reflected in the EA guidelines. Therefore, to consider a single criterion across all LLFAs, and in view of the significant nature of this project, it is proposed that the higher upper end allowances are used when considering the effects of climate change on the peak rainfall intensities.

2.6.7 It is also recognised that NPS EN-1[2] and EA guidance on climate change[23] advise that further assessment should be made of residual risks to this type of infrastructure associated with 'credible maximum' events beyond the required design standard. For fluvial flood risk, the 0.5% AEP plus Upper End allowance for climate change has been assessed; and for tidal flood risk the Upper End and H++ allowance for climate change has been assessed as credible maxima events. There is no requirement to assess a credible maximum event for surface water flood risk.

[32] As agreed during a meeting with the Environment Agency on 13/06/2022. Minuted in 807733-WOOD-IA-CE-RP-LPA-616047

[33] Flood risk assessments: applying for planning permission - GOV.UK (www.gov.uk)

Table 7F 2.7 Design Standards for Assessment

Development phase	Source of flooding	Design standard
Construction	Tidal	0.5% AEP (present day)
	Fluvial	0.5% AEP (present day)
	Surface Water	1% AEP (present day)
Operation	Tidal	0.5% AEP (higher central allowance up to 2095)
	Fluvial	0.5% AEP (higher central allowance for the '2080s')
	Surface Water	1% AEP (upper end allowance for the '2070s')

3. Site Characteristics

3.1 Location

3.1.1 The Project is located in the North West of England in the area between Stanlow, St Helen's, Partington and Northwich and consists of four corridors: West Corridor; North Corridor; East Corridor and South Corridor (as shown in **Figure 7.1**, **Chapter 7: Water Environment**).

3.1.2 The West Corridor extends from the Stanlow Refinery north-east to Rocksavage, crossing the River Weaver and Weaver Navigation, then south-east through Aston before crossing the Trent and Mersey Canal and joining the Central Hub north of the A553 (Northwich Road).

3.1.3 The North Corridor extends from the Central Hub north crossing the M56, Bridgewater Canal, Manchester Ship Canal, then north-west across the River Mersey between Widnes and Warrington and then broadly north to Ravenhead, St Helen's.

3.1.4 The East Corridor extends from the Central Hub broadly east, crossing the M6 and M56, up to Partington.

3.1.5 The South Corridor extends from the Central Hub broadly south, crossing the Trent and Mersey Canal and Weaver Navigation and then around Northwich to the west, finishing approximately 250m to the east of the A530.

3.1.6 This FRA assesses the flood risks within the Draft Order Limits. The Draft Order Limits are defined for the Project in **Chapter 2: The Project**. A detailed description of the water environment baseline for the Draft Order Limits is given within **Section 7.5** of **Chapter 7: Water Environment**, relevant aspects of which are briefly summarised below.

3.2 Land Use

3.2.1 Land use within the Draft Order Limits is predominantly rural and is characterised by open fields with hedgerows and tree lines. However, it also includes some industrial areas and runs adjacent to some residential areas. The Draft Order Limits also traverse several significant transport infrastructure features including motorways, A-roads and railway lines, in addition to minor roads and access routes.

3.3 Climate

3.3.1 The average annual total rainfall is 729mm, based on the Hawarden Airport[34] climate station, located 13km north-west of the Draft Order Limits. This is lower than the average for the North West of England. The average monthly rainfall is 61mm, with the highest levels of rainfall observed in October (76.24mm) the lowest levels observed in March (48.22mm). The Woodford (Greater Manchester) climate station, located approximately 12km to the east, has higher average annual rainfall (868mm) than the Hawarden Airport climate station, located west of the Draft Order Limits.

3.4 Topography

3.4.1 Elevations within the Draft Order Limits range from 5mAOD to 70mAOD, with the lowest lying land located in areas adjacent to the Mersey Estuary, lower River Mersey, Weaver Navigation and Manchester Ship Canal in the western of part of the Draft Order Limits. The areas of highest elevation within the Draft Order Limits are areas located within Runcorn up to 70mAOD on Runcorn Hill, and in the South Corridor which continues south-east towards Northwich and rises to 50mAOD in parts. The areas surrounding Northwich levels rise up to between 40mAOD and 50mAOD. The highest points in the Higher Walton area range between 40mAOD and 50mAOD.

3.5 Geology, Hydrogeology and Soils

3.5.1 The Draft Order Limits are predominantly located on triassic bedrock geology consisting of sandstone, mudstone, siltstone and conglomerates with some coal formations in the north.

3.5.2 These bedrock formations are obscured in the majority of locations by superficial deposits of till, sand, gravel, and alluvium, with some isolated pockets of peat along the East Corridor and West Corridor. The alluvium is associated with the route of the River Weaver and River Mersey. Overlying soils are a mix of wet loamy and clayey soils and free-draining sandy soils with some pockets of peat soils.

3.5.3 Sections of the West, North and East Corridors are underlain by principal bedrock aquifers, the remainder of the Draft Order Limits is underlain by secondary bedrock aquifers. In addition, large parts of the Draft Order Limits are also underlain by secondary superficial aquifers.

[34] Met Office, (2022). Hawarden (Flintshire) UK climate averages. (online) Available at: https://www.metoffice.gov.uk/research/climate/maps-and-data/uk-climate-averages/gcmys019j (Accessed 16 May 2022).

3.6 Hydrological Setting

3.6.1 The Project is located across three EA management catchments: Weaver Gowy; Lower Mersey; and Upper Mersey. As a result, there is potential for the Project to interact with major Main Rivers, including the River Mersey, River Weaver, Weaver Navigation and Manchester Ship Canal, as well as numerous smaller Main Rivers, tributaries and agricultural drains.

3.6.2 Whilst in most parts of England the Main Rivers tends to be confined only to larger watercourses, there are a large number of Main Rivers within the Draft Order Limits, many of which are actually very small minor watercourses. This is the legacy of the different approach that was historically taken to water level management in low-lying agricultural areas in the North West compared to the rest of England. Instead of establishing IDBs, small watercourses in rural areas were designated as Main Rivers to bring water level management under the responsibility of the EA and predecessor bodies.

3.7 Flood Defence Assets

3.7.1 There are EA maintained flood defence assets located along sections of the River Weaver at its downstream reach, the River Mersey through Warrington and along the Mersey Estuary coastline.

3.7.2 Those flood defence assets along the River Weaver directly interact with the Project around Rocksavage where there is a flood defence asset located on the left bank of the River Weaver and an area benefiting from defences stretching from Stanlow to Rocksavage.

3.7.3 Those flood defence assets along the River Mersey result in areas benefiting from defences through Warrington, some of which intersect with the Draft Order Limits.

3.8 Project Description

Overview

3.8.1 A summary of the different elements of the Project are provided below, however, a more detailed project description is provided in **Chapter 2: The Project**. For the FRA, optionality remains for various elements such as HAGIs, as described in **Chapter 2: The Project**.

3.8.2 The Project includes the construction and operation of up to 100 km of new pipelines to distribute hydrogen to industry, power stations and for blending with the existing natural gas network in North West England. The pipeline network would connect to a newly constructed Hydrogen Production Plant at Stanlow site as the source of hydrogen for onward distribution through the network. A Central Hub HAGI connects the four main pipeline corridors. Branching off from each main corridor there would be spurs, which are smaller pipelines that form the connections to users identified as part of the Project.

3.8.3 The pipelines that make up the network would vary in diameter between approximately 6” and 42”, dependent on which corridor or spur they are on. The

pipeline would be buried along the entire route apart from short sections within secure compounds at locations where HAGIs and BVIs are required. It is currently envisaged that nine HAGIs and two BVIs would be required at points along the network.

3.8.4 The pipeline, HAGIs, BVIs, Distribution Network Operator (DNO) connections and pigging areas make up the permanent operational infrastructure associated with the Project. A HAGI site will include a range of permanent above ground components such as above ground pipework, pressure reduction units, instrumentation and control local equipment rooms. BVIs will be in-line with the pipeline and a part of the valve would extend about 1.5m above ground. DNO connections will be required between the HAGIs and the electrical distribution network and telecommunication systems. Pigging areas are inspection facilities from which an inspection and maintenance device (a pipeline inspection gauge or 'pig') as well as operating equipment, including valves, can be launched and received through the pipeline. The facilities include above ground pipework from which to launch and receive the pig as well as operating equipment including valves.

3.8.5 During construction, there would be other temporary activities and infrastructure associated with the Project including:

- pipe laying;
- pipeline crossing works;
- material storage areas;
- trenchless crossing compounds;
- HAGI and BVI construction compounds; and
- temporary access routes with watercourse crossings.

Vulnerability classification

3.8.6 As set out in **Section 2**, the NPPF[6]and the associated PPG[5] provide guidance on vulnerability classifications for development types, based on their use/function and the compatibility of development with flood zones.

3.8.7 A matrix is provided in **Table 7F 3.1** that applies the NPPF flood risk vulnerability to the various elements of the Project and their compatibility with the Flood Zones (FZ). The Project is defined as 'Essential Infrastructure' as such is compatible with Flood Zone 1 and 2 and in Flood Zone 3a and 3b if it passes the Exception Test. As the Project is classed as Essential Infrastructure (NPPF[6], Annex 3), construction activities have in turn been classified as Essential Infrastructure, with the exception of pipeline watercourse crossings which are classified as 'Water Compatible' due to the nature of construction techniques used.

Table 7F 3.1 Application of the flood risk vulnerability and Flood Zone 'compatibility' matrix to the Project

Project type	Flood risk vulnerability classification	Flood Zone(s)	Flood risk vulnerability and flood zone 'compatibility'
Construction Phase			
Pipeline crossings	Water Compatible	1, 2 and 3	✓
Pipe laying	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
Material storage locations	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
Trenchless crossing compounds	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
HAGI temporary compounds	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
Temporary access routes	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
Operational Phase			
Pipeline	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
DNO connections	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
HAGIs	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3
Pigging areas	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3

Project type	Flood risk vulnerability classification	Flood Zone(s)	Flood risk vulnerability and flood zone 'compatibility'
Block Valve Installations	Essential Infrastructure	1, 2 and 3	✓ Exception Test required within FZ3

- "✓" symbol shows flood zone compatibility

4. Flood Risk Screening

4.1 Introduction

4.1.1 The FRA has been split into two parts. Firstly, a screening of all potential sources of flood hazard in terms of source, pathway and receptor. For a source to be screened in, a receptor must be identified. The presence of a source is initially screened in **Section 4**. Following the screening, a more detailed assessment of flood risk to and arising from the specific elements of the Project is carried out for construction (**Section 5**) and operational phases (**Section 5.8**). Where necessary, appropriate flood risk management measures are set out in **Section 7** to address the identified risks.

Screening approach

4.1.2 The aim of this initial screening assessment is to identify which sources of flooding pose a risk to and from the Project, both during construction and associated with those elements of permanent operational infrastructure. The screening assessment has been carried out by assessing those sources of data identified in **Table 7F1.1** and **Section 4.2** and where they intersect with the Project's construction and operational infrastructure.

4.1.3 The data used in the screening assessment does not include an allowance for future climate change, however the screening assessment of sources of risk has been undertaken with future increases as a consequence of climate change in mind.

4.2 Flood Risk Datasets

4.2.1 The following sections describe the key sources of flood risk data that have been used in the screening.

Fluvial and Tidal Flood Risk

4.2.2 The Environment Agency's (EA) Flood Map for Planning (FMfP)[9] has been used in the screening as it provides an indication of the likelihood of flooding from fluvial / tidal sources and the location of flood defences. Flood Zones 1 to 3 indicate a Low, Medium, and High likelihood of flooding respectively, as defined in **Table 7F1.4.** Flood Zones and locations of defences are shown on **Figure 7.7** which accompanies **Chapter 7: Water Environment** (any area not highlighted/coloured blue on these maps is Flood Zone 1).

4.2.3 To understand fluvial and tidal flood risk in more detail, hydraulic models can be used to understand factors such as depth, water level, velocity and hazard as well as time of inundation and time to peak. The EA holds a number of existing hydraulic models across the Draft Order Limits (see **Table 7F1.1**).

4.2.4 Hydraulic modelling has been carried out to inform this FRA for four sites: Hydrogen Storage Facility (HSF), Hydrogen Production Plant (HPP), Rocksavage

HAGI and Higher Walton HAGIs. Where appropriate, the existing EA models were updated (see **Table 7F1.1**) to bring them in line with the latest climate change allowances and update other key elements such as topography or land use.

Surface Water Flood Risk

4.2.5 The Environment Agency's Risk of Flooding from Surface Water (RoFfSW) Map[11] has been used to provide an understanding of likely surface water flow paths and areas of surface water ponding/flooding for the 3.33%, 1% and 0.1% AEP. The dataset also contains some depth banding to provide an indication as to how deep water could get in these areas of ponding and along identified flow paths but for this initial screening assessment only the extents have been used. The RoFfSW mapping is shown on **Figure 7.8** which accompanies **Chapter 7: Water Environment**.

Groundwater Flood Risk

4.2.6 The British Geological Society (BGS) Areas Susceptibility to Groundwater Flooding (AStGWF) map (**Figure 7.9** which accompanies **Chapter 7: Water Environment**) provides a regional understanding of the likelihood that groundwater flooding (defined by the BGS as "*defined as the emergence of groundwater at the ground surface or the rising of groundwater into man-made ground under conditions where the normal range of groundwater levels is exceeded*")[35].

4.2.7 The map is split into four categories:

- **Category A**: Limited potential for groundwater flooding to occur based on rock type and estimated groundwater level during periods of extended intense rainfall;
- **Category B**: Potential for groundwater flooding of property situated below ground level;
- **Category C**: Potential for groundwater flooding to occur at surface; and
- **Elsewhere**: Not considered to be prone to groundwater flooding.

4.2.8 Categories A to C are based on rock type and estimated groundwater level during periods of extended intense rainfall. The 'Elsewhere' category is based solely on rock type.

4.2.9 Where a development falls within Category B or C, it is recommended that an investigation occurs in to whether groundwater emergence is known to have occurred in the past and/or ensure measures are put in place as part of any development to reduce the impact of groundwater flooding.

[35] BGS, Definition of groundwater flooding available at: Susceptibility to Groundwater Flooding | British Geological Survey (BGS)

Artificial Sources

4.2.10 The Environment Agency's Reservoir Flood Maps (RFM, **shown in Figure 7.12 of Chapter 7: Water Environment**)[12] show where water may go in the unlikely event of a dam or reservoir failure. The Reservoir Flood Maps include two scenarios: a 'dry-day' and a 'wet-day'. The 'dry-day' scenario is defined as "*the flooding that would occur if the dam or reservoir failed when rivers are at normal levels*". The 'wet day' scenario assesses "*how much worse the flooding might be if a river is already experiencing an extreme natural flood*". For this screening assessment, both scenarios were used.

4.2.11 The nature of sewer flooding means there are no publicly available mapping datasets available to inform an assessment. Preliminary Flood Risk Assessments prepared by the LLFAs can include known hotspot areas prone to historic sewer flooding and have been used where available. United Utilities highlighted through the Statutory Consultation that the town centre in St Helens has experienced sewer flooding in the past, however, the Project does not coincide with this area.

4.2.12 Flooding from network service reservoirs, that hold treated drinking water have not been considered as they tend to be of much less capacity than the raw water reservoirs assessed in the EA's Reservoir Flood Maps.

4.2.13 There is no known available mapping or data regarding the potential flood risk associated with a failure of the canal network. Therefore, topographic information was reviewed to understand whether this residual risk is likely.

Historic Flooding

4.2.14 The Environment Agency Recorded Flood Outlines[8] show a number of historic flood events in the Draft Order Limits, notably through Warrington, Northwich and Acton Bridge. The majority of these recorded historic events fall outside of the Draft Order Limits with the exception of the flood events in Acton Bridge in 2019 and 2021 associated with the River Weaver (**Figure 7.10** which accompanies **Chapter 7: Water Environment**), and one near Higher Walton in 1990 associated with the River Mersey (**Figure 7.11** which accompanies **Chapter 7: Water Environment**).

4.2.15 Given the rural nature of the land use within the Draft Order Limits, it is likely there have been other incidents of localised flooding associated with intense rainfall and minor watercourses over the years but, due to the lack of receptors, these have not been recorded.

4.2.16 As discussed in **Section 7.5** of **Chapter 7: Water Environment**, surface water is a potential risk across the Draft Order Limits, but no specific hot spots of historic flooding have been identified. Equally no historic reports are available for groundwater or artificial sources.

4.3 Screening of All Potential Sources of Flood Risk

4.3.1 **Table 7F 4.1** provides an initial screening of all potential sources of flood risk across the Project Draft Order Limits.

Table 7F 4.1 Screening of all potential sources of flood risk

Source of flooding	Risk posed	Potential connection to the Project Draft Order Limits	Screened in?
Tidal	Medium risk	The Environment Agency's FMfP indicates the pipeline in the West Corridor runs parallel to the Mersey Estuary and crosses the River Weaver and Weaver Navigation within the tidal limit. The North Corridor crosses the River Mersey downstream of the tidal limit, which is at Howley Weir in Warrington. There are tidal flood defences in place to protect against a 0.5% AEP present day tidal flood event. However, in the future the standard of protection of these is likely to decrease with climate change. Areas of the pipeline, HAGIs and associated construction activities intersect with areas of tidal flood risk. The risk to the Project's permanent infrastructure and construction from tidal flooding is discussed further in **Section 5** and **Section 5.8**.	Yes
Fluvial	High risk	The Environment Agency's FMfP indicates the primary source of flood risk to the Project is from fluvial sources. The Project is within the River Mersey catchment and traverses several major Main Rivers including the River Weaver, Weaver Navigation, Manchester Ship Canal and the River Mersey as well as a large number of other smaller watercourses and their associated areas of Flood Zones 2 and 3. Areas of the pipeline, HAGIs and associated construction activities intersect with areas of fluvial flood risk. Those watercourses which pose a risk to the Project permanent infrastructure and construction are discussed further in **Section 5** and **Section 5.8**.	Yes
Surface water run-on	Medium risk	The Environment Agency's RoFfSW Map shows a risk of flooding from surface water run-on and ponding across the Draft Order Limits for a range of events from high risk to very low risk. The majority of areas are shown to be at very low risk of surface water flooding. However, there are some locations of ponding or flow paths which coincide with existing smaller	Yes

Source of flooding	Risk posed	Potential connection to the Project Draft Order Limits	Screened in?
		watercourses and drainage ditches. Some of these areas of surface water risk are also within fluvial Flood Zones. Areas of the pipeline, HAGIs and associated construction activities intersect with areas of surface water flood risk. The risk to the Project's permanent infrastructure and construction from surface water run-on is discussed further in **Section 5** and **Section 6**.	
Groundwater	High	The BGS's AStGWF maps indicate the Draft Order Limits contain a number of areas where there is potential for groundwater flooding at surface to occur as a result of extended periods of high rainfall. These areas of potential groundwater emergence generally coincide with watercourse valley floors, productive superficial deposits and soils. Additionally, there are a number of other areas where there is a potential risk of groundwater flooding to below ground construction activities. Areas of the pipeline, HAGIs, Block Valves and associated construction activities intersect with areas susceptible to groundwater flood risk. The risk posed by groundwater flooding is discussed further in **Section 5** and **Section 5.8**.	Yes
Artificial - Reservoir	Low risk	The Environment Agency's RFM shows that the main River Mersey and River Weaver could convey floodwater originating from the failure of upstream reservoirs. In a 'dry' scenario, where river flows are typical at the time of reservoir failure the risk of flooding from reservoir extents are generally smaller than the fluvial Flood Zones. Only in a 'wet' scenario, where a river is in flood at the time of reservoir failure do the risk of flooding from reservoir extents in some places exceed the Flood Zone 2 extent. These reservoirs are highly managed, so the overall risk of failure is very low and highly dependent on the river flow condition. Therefore, mitigation against this residual risk would be commensurate with mitigation against an extreme fluvial food risk and is addressed	Yes – as part of fluvial.

Source of flooding	Risk posed	Potential connection to the Project Draft Order Limits	Screened in?
		under fluvial flood risk (second item in this table).	
Artificial - Sewer	Negligible	A review of the SFRAs for the Draft Order Limits determines that there are no concentrated areas of high sewer flood risk within the Draft Order Limits. Above ground permanent infrastructure is predominantly proposed in rural (greenfield) or industrial (brownfield) area. Also, no areas of permanent above ground infrastructure coincide with areas of known sewer flooding and the Project would not require integration into any existing sewer networks. Therefore, the risk posed by sewer flooding is considered to be negligible. In the event that sewer flooding occurs, it is anticipated that the flooding would follow the natural topography of the landscape and accumulate in low-lying areas, therefore patterns would be similar to that associated with surface water flooding. **Therefore, the risk of sewer flooding is not considered further in this FRA.**	No
Artificial - Canal	Low risk	Five canals are present within the Draft Order Limits: Bridgewater Canal, Manchester Ship Canal, Sankey Canal, Trent and Mersey Canal, and Weaver Navigation. Canals and their associated structures are maintained and regulated to maintain a constant water volume so do not pose a risk of flooding the event of a storm event. They do however pose a residual risk as a consequence of a breach if the Canal is impounded and raised above ground. For the permanent above ground infrastructure, located near a canal, the FRA includes an assessment of the residual risk of canal breach where applicable. **Chapter 7: Water Environment Section 7.14** contains further details of canals in the Draft Order Limits.	Yes, as a residual risk

5. Assessment of Flood Risk – Construction Phase

5.1 Introduction

5.1.1 The potential sources of flood risk within the Draft Order Limits have been summarised in **Section 4**. This Section assesses specific flood risk to receptors associated with the construction phase of the Project. This includes the flood risk posed to construction activities and infrastructure as well as potential flood risks to third-party receptors as a consequence of the construction of the Project.

5.1.2 In addition to the Draft Order Limits, Limits of Deviation (LoD) have also been defined. The Limits of Deviation indicate the maximum area of land within each of the permanent infrastructure may be located.

5.1.3 **Figure 1** shows the LoD of the elements of the Project in the context of flood risk during construction.

5.1.4 Appropriate flood risk management measures are specified to address the identified risks in **Section 7**, all of which have been incorporated into the design of the Project through the embedded environmental measures specified in **Chapter 7: Water Environment Table 7.16**.

5.2 Pipe Laying

5.2.1 The laying of the main pipeline would traverse areas of flood risk and existing floodplains. The pipeline would be buried and therefore an element of excavation is required to lay the pipeline. The pipeline routing has been optimised to avoid areas of flood risk wherever possible, but by its nature it is inevitable that there would be some locations where it intersects with areas of flood risk.

5.2.2 The West Corridor flows through a significant expanse of defended floodplain of the Mersey Estuary and associated unnamed main river channels. It then traverses the River Weaver floodplain and some smaller Main Rivers. Approximately 45% of the LoD area for West Corridor is in Flood Zone 3, mainly associated with the tidal River Mersey. The floodplain stretches along the M56, impacting the area from Frodsham to Elton. This part of the West Corridor is also associated with potential groundwater flooding at surface as it is a low-lying, marshland area.

5.2.3 The North Corridor flows through significant expanses of floodplain associated with the River Mersey and Manchester Ship Canal and some smaller Main Rivers. Approximately 15% of LoD of the North Corridor is in Flood Zone 3 and Flood Zone 2. The majority of the North Corridor is at some risk of groundwater flooding at surface, however the flooding is limited and would likely only occur during periods of extended high rainfall. The area around Clock Face, north of M62, is more prone to surface flooding and has the potential for groundwater flooding at surface to occur. Two options for the pipeline routes are considered for a section

between the River Mersey and the A5080 (**Figure 1**, sheet 15). Both options are in Flood Zone 1 and have limited potential for groundwater flooding. Surface water flood risk is more significant for the northern option, where half of area of the LoD intersects with a surface water flow path.

5.2.4 The East Corridor flows through expanses of floodplain in Flood Zone 2 and 3, associated with the River Bollin and some smaller Main Rivers, which impact approximately 5% of the LoD area for the East Corridor. Potential for groundwater flooding at surface spreads across the entire LoD of the East Corridor and impacts about 60% of the area. The potential for groundwater flooding to occur at surface is found in pockets of areas near Partington, Warburton and in low-lying areas associated with the Gale Brook.

5.2.5 The South Corridor flows through limited areas of floodplain associated with the River Weaver, Weaver Navigation and some smaller Main Rivers. Approximately 5% of the LoD area is in Flood Zone 2, and circa 2% is within Flood Zone 3. The South Corridor stretches through various areas at risk of groundwater flooding. For a section of the pipeline located south of A533, two route options are considered (**Figure 1**, sheet 28). Both options are in Flood Zone 1, however the pipeline closer to A49 intersects with a surface water flow path.

5.2.6 Excavated topsoil from the pipe trench will be stored at the edge of the works, typically in bunds in the order of 2m in height. They will be sited to avoid risk of topsoil loss and impeding areas of surface water flow and gaps left between bunds to avoid waterlogging. The resulting loss of floodplain storage would be temporary and minimal. Bunds would be moved out of the floodplain wherever possible and those remaining would be aligned with the main flow direction to avoid impeding flows during a flood. Temporary access tracks may be raised above the floodplain would have the potential to impact flood pathways and take up floodplain storage.

5.2.7 The excavated route would be reinstated to ensure no change in topography and therefore no change in floodplain storage or conveyance. So, risk from the pipe laying would be low. Risk to the pipe laying of flooding throughout the construction would be managed through embedded environmental measures discussed further in **Section 7**.

5.2.8 Flood risk at pipeline watercourse crossings is discussed in **Section5.4**.

5.3 Material Storage Areas

5.3.1 There are 35 potential material storage areas identified as part of the Project (**Figure 1**). Of these:

- One is located in Flood Zone 2 and 3 (Figure 1, Sheet 12) ;
- One is located in Flood Zone 2 (Figure 1, Sheet 28);
- The remaining 33 material storage areas are in Flood Zone 1;
- Thirty sites have some element of groundwater flooding, and nine of those sites are at risk of groundwater flooding at surface in the event of prolonged high rainfall;

- Eleven sites are potentially at risk of surface water ponding on site; and
- Two sites along the North Corridor are in the vicinity of a main river so a FRAP may be required.

5.3.2 Material storage areas are where lengths of pipe to construct the pipeline would be stored for a short time period during construction. In terms of flood risk, the greatest risk is to those sites within a fluvial or tidal flood zone or those which intersect a surface water flow route. In the event that these areas are utilised during construction, there would be a risk that lengths of pipe being stored there could be washed away, or that they could interrupt existing flow paths.

5.3.3 If materials are stored in an area of undefended floodplain there is also a risk that it could displace floodplain storage, thereby increasing flood risk elsewhere. One particular location coincides with the floodplain of the River Weaver in Acton Bridge. More than 50% of the area of this pipeline storage area intersects Flood Zone 2. It is also an area of surface water flood risk and historic flooding has been recorded here. This location is highlighted as high risk.

5.3.4 The material storage area at Eastford Road, associated with the Novelis Spur is in Flood Zone 3. This location is at risk of flooding from the River Mersey. This location is highlighted as high risk.

5.3.5 A small part of the Sutton Manor material storage area interacts with fluvial flood zones, but through sequential planning of the site it could be possible to avoid the areas of highest risk.

Table 7F 5.1 Flood risk overview – Material Storage Areas

Material Storage Area	Flood Zone	Tidal Flooding	Fluvial Flooding	Benefitting from defences	Surface Water Flooding	Reservoir Inundation	Groundwater flooding
Stanlow	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Central Hub	1	No risk	No risk	n/a	Very low risk	No risk	No risk
Acton Bridge	2	No risk	Yes, from River Weaver	No	Ponding at 3.33% AEP	Yes, at wet day extend from Bosley Reservoir	Potential for groundwater flooding to occur at surface
Bryn	1	No risk	No risk	n/a	Ponding at the NW corner at 0.1% AEP	No risk	Potential for groundwater flooding of property situated below ground level and at surface
Hydrogen Storage Facility	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding of property situated below ground level
Antrobus	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding

Material Storage Area	Flood Zone	Tidal Flooding	Fluvial Flooding	Benefitting from defences	Surface Water Flooding	Reservoir Inundation	Groundwater flooding
							of property situated below ground level
Sworton Heath 1	1	No risk	No risk	n/a	Very low risk	No risk	No risk
Broomedge	1	No risk	No risk	n/a	Flow path at 0.1% AEP	No risk	Potential for groundwater flooding to occur at surface
Cuerdley 1	1	No risk	No risk	n/a	Ponding along the North border at 0.1% AEP	No risk	Limited potential for groundwater flooding to occur
Bostock Green	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding to occur at surface
Sworton Heath 2	1	No risk	No risk	n/a	Very low risk	No risk	No Risk
Warburton 2	1	No risk	No risk	n/a	Ponding at 3.33% AEP	No risk	Potential for groundwater flooding of property situated below ground level and at surface

Material Storage Area	Flood Zone	Tidal Flooding	Fluvial Flooding	Benefitting from defences	Surface Water Flooding	Reservoir Inundation	Groundwater flooding
Warburton 1	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Cuerdley 2	1	No risk	No risk	n/a	Flow path at 0.1% AEP	No risk	Limited potential for groundwater flooding to occur
Union Bank	1	No risk	No risk	n/a	Ponding at 0.1% AEP	No risk	Limited potential for groundwater flooding to occur
Sutton Manor	1, except small part in FZ 2	No risk	Yes, from Pendlebury Brook	No	Ponding at 0.1% AEP	No risk	Potential for groundwater flooding of property situated below ground level
Preston on the Hill 1	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding of property situated below ground level
Preston on the Hill 3	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding of property situated below ground level

Material Storage Area	Flood Zone	Tidal Flooding	Fluvial Flooding	Benefitting from defences	Surface Water Flooding	Reservoir Inundation	Groundwater flooding
Sutton Weaver	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding to occur at surface
Eastford Road	2 and 3, fluvial/ tidal	Yes (Flood Zone 2 and 3)	Yes (Flood Zone 2 and 3)	No	Ponding at the NW corner at 0.1% AEP	Yes, multiple reservoirs (Fiddler's Ferry, Lymm Dam, Prescot)	Limited potential for groundwater flooding to occur
Louchers Lane	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Wash Lane	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Knutsford Road	1	No risk	No risk	n/a	Very low risk	Yes, at wet day extends (reservoirs Greenbooth and Lymm Dam)	Potential for groundwater flooding of property situated below ground level
St Helens	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding

Material Storage Area	Flood Zone	Tidal Flooding	Fluvial Flooding	Benefitting from defences	Surface Water Flooding	Reservoir Inundation	Groundwater flooding
							of property situated below ground level
Station Road	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Cuerdley 3	1	No risk	No risk	n/a	Flow Path at 0.1% AEP	No risk	Limited potential for groundwater flooding to occur
Preston on the Hill 2	1	No risk	No risk	n/a	Very low risk	No risk	partially at Potential for groundwater flooding of property situated below ground level
Sandiway 1	1	No risk	No risk	n/a	Flow path at 0.1% AEP	No risk	Potential for groundwater flooding to occur at surface
Sandiway 2	1	No risk	No risk	n/a	Very low risk	No risk	Potential for groundwater flooding to occur at surface
Warrington 1	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur

Material Storage Area	Flood Zone	Tidal Flooding	Fluvial Flooding	Benefitting from defences	Surface Water Flooding	Reservoir Inundation	Groundwater flooding
Warrington 2	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Sherdley Road 1	1	No risk	No risk	n/a	Very low risk	No risk	No Risk
Sherdley Road 2	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Sherdley Road 1a	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur
Sherdley Road 2a	1	No risk	No risk	n/a	Very low risk	No risk	Limited potential for groundwater flooding to occur

5.4 Pipeline Watercourse Crossings

5.4.1 The pipeline route traverses many watercourses along its length, resulting in 113 watercourse crossing locations. The pipeline would be buried and where there is a requirement to cross watercourse the following construction techniques are proposed:

- Open cut;
- Micro-tunnel (trenchless);
- Auger Bore (trenchless); and
- Horizontal Directional Drilling (HDD) (trenchless).

5.4.2 **Chapter 2: The Project** provides additional detail on what is involved for each of these pipeline watercourse crossing techniques.

5.4.3 The majority (87) of pipeline watercourse crossings (32 at rivers and 55 at stream/ditches) are proposed to be constructed using the open cut trenched technique, which would involve isolation of the watercourse at the crossing location with fluming or overpumping during pipeline installation and reinstatement following completion of works. The open cut technique is proposed for smaller watercourses only.

5.4.4 Auger bore (5 crossings), micro-tunnel (11 crossings) and HDD (10 crossings) are all trenchless techniques and are proposed for where larger watercourses (mainly those classed as Main Rivers) are crossed. At some watercourse crossings multiple options are being considered at this stage.

5.4.5 Ordinary Watercourse Consent from the LLFA will be required for all open cut crossings of Ordinary Watercourses. A Flood Risk Activity Permit (FRAP) from the EA will be required for all open cut crossings over Main Rivers.Trenchless crossings of Main Rivers may be carried out as an Exemption from FRAP, provided the conditions set out under FRAP Exemption FRA3 can be met[36].

5.4.6 Asopen-cut crossings of watercourses are required, the construction of these areas would be inherently at risk of flooding. The construction technique involves temporary disruption of flows in the watercourse which are being crossed, which could potentially increase flood risk to external receptors. However, the construction period for each open-cut crossing is short and therefore the risk of a flood coinciding with this is minimal. Construction would be timed to heed any flood warnings. Measures regarding watercourse crossings during construction phase are presented in **Table 7F 7.1**.

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[36] Exempt flood risk activities: environmental permits, Available at: https://www.gov.uk/government/publications/environmental-permitting-regulations-exempt-flood-risk-activities/exempt-flood-risk-activities-environmental-permits#service-crossing-below-the-bed-of-a-main-river-not-involving-an-open-cut-technique-fra3 (Accessed 27 August 2024)

5.5 Trenchless Crossing Compounds

5.5.1 There are 104 trenchless compounds proposed to support the construction of pipeline crossings of watercourses, roads and utilities. At this stage, 24 of these compounds remain as options. Further information on these is provided in **paragraph 5.5.7** and **Table 7F 5.2** below.

5.5.2 Out of the total 104 trenchless crossing compounds, 22 would be located in Flood Zone 3, and four in Flood Zone 2. One is located in Acton Bridge on the left bank of the River Weaver in an area where recent historic flooding has been experienced. The boundaries of 11 trenchless crossing compound sites are within 8m of a non-tidal Main River or 16m tidal Main River and may require a full FRAP. Wherever possible, opportunities have been sought to relocate those trenchless crossing compounds at locations deemed to be at lower flood risk. However, as trenchless crossings are being used to cross watercourses, it may not be possible to site trenchless crossing compounds outside of the floodplain, especially of the large rivers in the Draft Order Limits (e.g. River Weaver; River Mersey), due to their need for a location close to a watercourse crossing. This is the case for the three trenchless crossing compound options associated with crossing the River Mersey near Rocksavage HAGI, which intersect with Flood Zone 3.

5.5.3 Around 50 of the trenchless crossing compounds have some element of surface water flood risk associated with them, although this generally consists of minor areas of ponding across the sites. Eight trenchless crossing compounds have some areas of the site at risk of surface water flooding even in a 3.33% AEP event. There are 28 trenchless crossing compounds in areas with a risk of groundwater flooding at surface and 17 at risk of groundwater flooding below ground level (which may affect HDD launch and reception pits, for example).

5.5.4 Where fluvial and surface water flood risks have been identified they do not necessarily cover the entire site. As such it may be possible to sequentially plan any temporary compound to manage this risk to ensure no loss of storage or interruption to conveyance pathways and thereby minimise impacts on external receptors.

5.5.5 There is the potential for excavated materials, soil from the soil processing plant, materials from settlement pits or drilling mud/ slurry processing, stored materials and equipment, fuel from the fuelling area, and waste from waste disposal to become mobilised in case of the trenchless crossing compound becoming inundated during times of flooding. Deposition with flood waters of such materials elsewhere could lead to damage or obstruction downstream, thus increasing the flood and pollution risk to non-project receptors. There is a potential to increase the flood risk from discharges from dewatering of launch and reception pits.

5.5.6 The length of time a trenchless crossing site would be in use varies depending on the technique, complexity, depth and length of crossing. This may vary between one to six months. Where sources of flood risk cannot be avoided, the timing of construction would be between March and September so it occurs at a time of year with less risk of flooding. Some continuous (24 hour) working may be required as defined in **paragraph 2.5.15** of **Chapter 2: The Project**.

5.5.7 The risk of flooding for the trenchless crossing compounds options is summarised in **Table 7F 5.2**.

Table 7F 5.2 Flood risk overview – Options for temporary crossing compounds

Compound ID	Associated element	Crossing type	Flood Zone	Surface water flooding	Groundwater flooding
73	West Corridor: Option for crossing at A5117	HDD Exit; Microtunnel Entry	1	No	Limited potential for groundwater flooding to occur
69		Microtunnel Exit	1	No	Limited potential for groundwater flooding to occur
68		Microtunnel Entry/ HDD Entry	1	Ponding 0.1% AEP	Limited potential for groundwater flooding to occur
75	West Corridor: Option for crossing River Weaver to Rocksavage HAGI	HDD Entry	3	No	No risk
34		Microtunnel Exit	3	No	No risk
52		Microtunnel Entry/ HDD Exit	3	No	Potential for groundwater flooding to occur at surface
32	West Corridor: Option for crossing Sutton Weaver	Microtunnel Entry	3	0.1% AEP	Potential for groundwater flooding to occur at surface
70		Microtunnel Exit/ HDD Entry	1		Potential for groundwater flooding to occur at surface
101	South Corridor: Option pipeline route SP6	Microtunnel Entry	1	No	Potential for groundwater flooding of property situated below ground level
102		Microtunnel Exit	1	No	Potential for groundwater flooding to occur at surface

Compound ID	Associated element	Crossing type	Flood Zone	Surface water flooding	Groundwater flooding
19	South Corridor: Option pipeline route SP7	HDD Entry/ Microtunnel Entry	1	Flow path at 3.33% AEP	Potential for groundwater flooding of property situated below ground level
18		Microtunnel Entry/ HDD Exit	1	No	Potential for groundwater flooding to occur at surface
58	East Corridor: Options for crossing M6	HDD Exit/ Microtunnel Entry	1	Ponding at 0.1% AEP	Potential for groundwater flooding to occur at surface
78		Microtunnel Entry	1	Ponding at 0.1% AEP	Potential for groundwater flooding to occur at surface
59		Microtunnel Entry/ HDD Exit	3	0.1% AEP	No risk
50	North Corridor: Options for Higher Walton	Microtunnel Exit	3	No	Limited potential for groundwater flooding to occur
100		Microtunnel Exit	3	No	Limited potential for groundwater flooding to occur
99		Microtunnel Entry	3	No	Limited potential for groundwater flooding to occur
97	North Corridor: Option for pipeline route NP7	HDD Entry	1	No	Limited potential for groundwater flooding to occur
96		HDD Exit	1	No	Limited potential for groundwater flooding to occur

Compound ID	Associated element	Crossing type	Flood Zone	Surface water flooding	Groundwater flooding
95		Microtunnel Exit	1	No	Limited potential for groundwater flooding to occur
94		Microtunnel Entry	1	No	Limited potential for groundwater flooding to occur
84	North Corridor: Option for pipeline route NP6	Microtunnel Exit	1	0.1 AEP	Limited potential for groundwater flooding to occur
83		Microtunnel Entry	1	Ponding at 0.1% AEP	Limited potential for groundwater flooding to occur

5.6 Temporary Access Route Watercourse Crossings

5.6.1 To facilitate construction of the pipeline and the transportation of equipment materials and personnel, a network of temporary access routes has been proposed within the Draft Order Limits.

5.6.2 Where possible, these temporary access routes make use of the existing road network. But in some areas, new temporary access routes would need to be constructed.

5.6.3 Where new temporary access routes are proposed to be constructed, some would need to cross watercourses, requiring temporary bridges or culverts to be installed..

5.6.4 The Crossing Schedule (**Figure 2.3** and **Appendix 2B**) proposed plans show the need to cross watercourses designated as Main Rivers. Clear span bridges would be provided for larger and/or more environmentally sensitive watercourses to minimise effects on watercourse conveyance capacity. These new access routes are generally through rural areas, so receptors are minimal with the exception of agricultural land. The detailed design of all temporary access watercourse crossings will be subject to either Ordinary Watercourse Consent from the relevant LLFA or FRAP from the EA post DCO consent.

5.7 HAGI Temporary Compounds

5.7.1 HAGI temporary construction compounds are required to support the construction of the HAGI options listed in **Table 7F 6.1**. The HAGI temporary construction compounds are within the Draft Order Limits.

5.7.2 Four of the 14 HAGI construction compounds intersect with Flood Zone 3, these are the three compounds associated with the Higher Walton Option HWH2 and Option HWH8 and the compound at the Hydrogen Storage Facility (HSF).

5.7.3 Results from the updated River Mersey (Warrington) model (**Figure 8** and **Annex 7F4**) suggest that there is no fluvial flood risk to the Higher Walton compounds. Results from the updated Mersey Estuary modelling (see **Figure 7** and **Annex 7F4**) show that:

- Parts of the Higher Walton HWH2 East (ID 3) compound may be at risk from tidal flooding, depending on the compound ground levels;
- The Higher Walton (ID 6) compound would remain free of tidal flooding for all relevant scenarios, with the exception of the north-eastern corner of the compound footprint being at risk of tidal flooding in the H++ scenario;
- The Higher Walton HWH2 West (ID 7) compound 3 would remain free of tidal flooding for all relevant scenarios.

5.7.4 The Hydrogen Storage Facility (HSF) compound is predominantly located in Flood Zone 1, with the southern boundary in the fluvial Flood Zones 2 and 3 of the Puddinglake Brook. Results from the Puddinglake Brook Hydraulic Model (**Figure**

9 and **Annex 7F5**) show that the compound footprint is not at risk of fluvial flooding.

5.7.5 The construction compound associated with Hydrogen Storage Facility (HSF) and Partington PH1 (compound ID 10) also have a risk of groundwater flooding at surface. The construction compound associated with the Warburton HAGI has potential for groundwater flooding of development below ground level.

5.7.6 Four of all compounds, associated with Clockface, Central Hub, Higher Walton HWH2 (compound ID 7) and Partington (compound ID 5) HAGIs, have areas of surface water ponding within the compound footprint.

Table 7F 5.3 Flood Risk Overview – HAGI Temporary Construction Compound

Associated HAGI	Where to find on Figure 1	Flood zone	Watercourse	Surface water flooding	Reservoir inundation	Groundwater flooding
Clock Face	Sheet 17	1	No	Ponding in the north east of the site at 3.33% AEP	No	Potential for groundwater flooding of development below ground level
Hydrogen Storage Facility (HSF)	Sheet 34	Predominantly located in Flood Zone 1, with the southern boundary in Flood Zones 2 and 3 Modelling shows that compound footprint not at risk of fluvial flooding (see **Figure 9**)	Puddinglake Brook	No	No	Potential for groundwater flooding to occur at surface
Central Hub	Sheet 8	1	No	Minor ponding at 0.1% AEP	No	Not considered to be prone to groundwater flooding
Rocksavage	Sheet 5	1	River Weaver and Weaver Navigation	No	Yes – but only when river in flood*	Not considered to be prone to

Associated HAGI	Where to find on Figure 1	Flood zone	Watercourse	Surface water flooding	Reservoir inundation	Groundwater flooding
						groundwater flooding
Runcorn	Sheet 5	1	Weaver Navigation	No	No	Not considered to be prone to groundwater flooding
Hydrogen Production Plant (HPP)	Sheet 1	1	Gale Brook (Gowy Catchment)	No	No	Limited potential for groundwater flooding to occur
Higher Walton HWH2 East (ID 3)	Sheet 12	2 and 3; detailed modelling shows that parts of the compound may be at risk from tidal flooding, depending on the ground levels (see **Figure 7**)	Manchester Ship Canal	No	Yes – but only when river in flood*	Limited potential for groundwater flooding to occur
Higher Walton HWH2 West (ID 7)	Sheet 12	3, however detailed modelling suggests that the compound would remain free of fluvial and tidal flooding for all relevant	Manchester Ship Canal	Some ponding within compound footprint	Yes – but only when river in flood*	Limited potential for groundwater flooding to occur

Associated HAGI	Where to find on Figure 1	Flood zone	Watercourse	Surface water flooding	Reservoir inundation	Groundwater flooding
		scenarios (see **Figure 7**)				
Higher Walton (ID 6)	Sheet 12	3, however detailed modelling suggests that the compound would remain free of fluvial and tidal flooding for all relevant scenarios, with the exception of the north-eastern corner of the compound footprint being at risk of tidal flooding in the H++ scenario (see **Figure 7**)	Manchester Ship Canal	No	No	Limited potential for groundwater flooding to occur
Higher Walton HWH8 (ID 13)	Sheet 12	1	Manchester Ship Canal	No	Yes – but only when river in flood*	Limited potential for groundwater flooding to occur
Warburton	Sheet 25	1	No	No	No	Potential for groundwater flooding of development below ground level

Associated HAGI	Where to find on Figure 1	Flood zone	Watercourse	Surface water flooding	Reservoir inundation	Groundwater flooding
Partington (ID 5)	Sheet 27	1	No	Minor ponding at 0.1% AEP	No	Not considered to be prone to groundwater flooding
Partington PH1 (ID 10)	Sheet 27	1	No	No	No	Potential for groundwater flooding to occur at surface
Partington Option PH2 (ID 12)	Sheet 27	1	No	No	No	Not considered to be prone to groundwater flooding

*The Environment Agency publish two reservoir inundation maps – one ‘dry’ assuming normal flow in the river and one ‘wet’ assuming high river flows.

5.8 Block Valve Installations, DNO Connections and Pigging Areas

5.8.1 The construction of Block Valve Installations, DNO Connections and Pigging Areas will take place within the Draft Order Limits at the designated locations. The risk of flooding to these locations has been considered in Sections 6.4 Block Valve Installations (BVIs), 6.5 DNO Connections and 6.6 Pigging Areas. In line with the design standards specified in Section 2.6, the impact of climate change can be disregarded for the construction phase.

6. Assessment of Flood Risk- Operational Phase

6.1 Introduction

6.1.1 **Figure 2** shows the operational phase elements of the Project in the context of flood risk.

6.2 Pipeline

6.2.1 The pipeline would be buried for its entire length, apart from short sections within secure compounds at locations where HAGIs, BVIs and pigging areas are required, and therefore not be at risk from fluvial, tidal, surface water or artificial sources of flood risk. It would also not increase the risk of flooding from these sources to third party receptors as there would be no above ground sections of pipeline and therefore no displacement of floodplain storage.

6.2.2 The flood risk associated with the operational pipeline is limited to risks associated with the interruption of shallow groundwater flow paths. The pipeline would be at low risk from groundwater itself, but through potentially interrupting a groundwater flow path it could result in an increase in groundwater emergence and flooding elsewhere without appropriate mitigation.

6.2.3 The target excavation depth of the pipeline would be on average a 2.5m to 3m deep trench. However, it may be a greater depth particularly where the pipeline is 42” in diameter, and where the pipeline passes beneath services and other crossings it may be 4m to 6m deep, with HDD crossings being up to 15m deep.

6.2.4 Therefore, the risk of groundwater flooding occurring is greatest in those areas where the pipeline would run through perched water tables or superficial aquifers. However, all areas along the pipeline route were screened out of having major interaction with shallow groundwater, except for the Frodsham area along the West Corridor (refer **to Appendix 7B: Detailed water environment baseline information**, **Table 7B.8** for further details).

6.2.5 The EA has advised that “*although the Ince and Frodsham Marshes are served by land drainage pumping stations that manage water levels across the lower lying parts of the catchment (i.e. north of the M56 corridor), the pumping stations have a relatively low discharge rate and are not designed to pump large flood flows. As a consequence, there are periods of time when the marsh areas resemble exactly that and it takes some time (along a period of drier weather) for normal water levels to resume.*”[37]

[37] Email communication from Environment Agency to WSP, received 20/09/2023 at 10:26. Subject: RE: Flooding north of M56.

6.2.6 The flow direction here would mostly be to the north-west, perpendicular to the pipeline, from Frodsham towards the marshes and on to the estuary. The pipeline could potentially cause shallow groundwater flows to back up and increase the risk for groundwater emergence upgradient (i.e. south-east, of the pipeline). There is already the potential for groundwater flooding at the surface in this area. In theory, this potential could increase and the area at risk could expand towards the south-east into the settlements of Frodsham and Helsby as a result of the presence of the pipeline.

6.2.7 However, horizontal groundwater flow within the surface will be minimal due to low permeability sediments and shallow hydraulic gradients in the area where flow is largely controlled by surface drainage. Furthermore, in the Frodsham area the pipeline runs mostly parallel to the M56, whose foundations are expected to already provide a barrier to shallow groundwater flow upstream of the pipeline. Subsurface flows intersecting with the pipeline are therefore expected to be slowed down already and the pipeline would therefore be unlikely to significantly change the current flow patterns.

6.2.8 It is however recognised that buried channels in this area may influence groundwater flow and any localised impacts will be mitigated through further ground investigation and appropriate design.

6.2.9 Embedded environmental measures to mitigate this risk are discussed in **Section 7.3**.

Pipeline crossings

6.2.10 The pipeline is to be buried for its entire length; this includes where the pipeline route crosses watercourses. Therefore, all watercourse crossings would traverse the watercourses below the existing bed level to ensure no impact on the conveyance capacity of the watercourse and therefore no impact of flood risk to third party receptors.

6.3 HAGIs

6.3.1 **Table 7F 6.1** provides an overview of the flood risk associated with the nine HAGI locations, and the additional two HAGI location options. It highlights the flood risk present at the HAGI location and indicates where a FRAP would be required as a consequence of proximity to a Main River or associated flood defence.

6.3.2 Seven of the nine HAGIs listed in **Table 7F 6.1** are sited in Flood Zone 1. Runcorn HAGI and Hydrogen Storage Facility HAGI are both in Flood Zone 1 but close to the edge of the existing Flood Zone 2 or 3.

6.3.3 Six of these seven HAGIs have small, localised areas of surface water flooding within their boundary. The exception is Hydrogen Storage Facility HAGI, which only has a very low risk of flooding from surface water. It may, however, be susceptible to groundwater flooding at surface following prolonged heavy rainfall. Warburton, Clock Face and St Helens HAGI have potential for groundwater flooding of infrastructure below ground level. Groundwater flooding is unlikely to adversely impact the operational development, providing appropriate drainage is

provided, but could be an issue for construction where excavation of the ground is required (as mentioned in **Section 5.7**).

6.3.4 Irrespective of the nature and extent of flood risk from external sources to each HAGI site, outline drainage strategies have been developed for all sites to support the FRA (**Annex 7F6**), detailing how drainage would be provided at each site to ensure that surface water runoff from impermeable surfaces does not increase flood risk elsewhere. These drainage strategies make allowance for future climate change on extreme rainfall in line with the latest EA guidance (as summarised in **Table 7F 2.3**) and will incorporate appropriate SuDS elements (discussed further in **Section 7.4**).

Rocksavage

6.3.5 Rocksavage HAGI LoD is predominantly located in Flood Zone 1, with only the northern corner of the LoD located in Flood Zones 2 and 3 (**Figure 2**, Sheet 5). The HAGI has been positioned on higher ground above the floodplain on a 'dry island' between the River Weaver in the south-west and the Weaver Navigation in the north-east.

6.3.6 The modelling undertaken by the EA for the Mersey Estuary 2016 study indicates that the Rocksavage HAGI LoD remains free from tidal flooding in the present day 0.5% AEP tidal event.

6.3.7 Modelling undertaken by the EA for the Lower River Weaver in 2019 assessed the impact of climate change on fluvial flood risk by modelling up to a 70% increase in river flows. Even with this increase in flow the Rocksavage HAGI LoD remained dry for the 1% AEP event.

6.3.8 The climate change allowances applied for the existing EA modelling studies are different to the recent EA guidance outlined in **Table 7F 2.2** and **Table 7F 2.4.** As a consequence, further modelling analysis was undertaken to fully understand the impacts of future climate change and residual risk to the Rocksavage HAGI LoD.

6.3.9 As the Project is an NSIP, maximum credible scenarios for both tidal and fluvial flooding were investigated to ensure the resilience of the Rocksavage HAGI LoD in line with the design standards detailed in **Section 2.6**.

6.3.10 The model used in the EA Mersey Estuary 2018 study[18] was run for the design event (0.5% AEP (2100) Higher Central), as well as the more extreme 0.5% AEP (2100) Upper End, and the 0.5% AEP H++ events. The H++ event was considered as the credible maximum scenario. A model report is included in **Annex 7F1** and describes the checks undertaken to ensure that the model is fit for the purpose of this study. The results in **Figure 3** show that the Rocksavage HAGI LoD remains free from flooding for all three tidal events with climate change events.

6.3.11 The model used in the EA Lower River Weaver 2019 study[17] was run for the design event (0.5% AEP (2100) Higher Central), as well as the more extreme 0.5% AEP (2100) Upper End event, which is considered the fluvial credible maximum event. A model report is included in **Annex 7F2** and describes the checks undertaken to ensure that the model is fit for the purpose of this study. The results in **Figure 4** show that the Rocksavage HAGI LoD remains free from fluvial flooding for both the design event and the credible maximum scenario.

6.3.12 Rocksavage LoD HAGI is not at risk from surface water and there is no potential for groundwater flooding at surface. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Annex 7F6**).

6.3.13 The Weaver Navigation Canal is located about 200m south-east of the Rocksavage LoD HAGI at an elevation of about 10m below the HAGI LoD. The Canal is owned and maintained by the Canal & River Trust. According to the data presented in Cheshire West and Chester Level 1 SFRA[38], there are no historical events of canal breach. Given that the Rocksavage HAGI LoD is on higher elevation and based on the historical information the likelihood of canal flooding from canal bank failure is considered to be low.

HPP HAGI

6.3.14 The HPP HAGI LoD is located within the Gale Brook catchment, a tributary to the River Gowy (**Figure 2**, Sheet 1). The southwest corner of the initially proposed HPP HAGI location was shown to be at risk of flooding from the Gale Brook. Through the design process, the location was adapted and moved further east, away from the watercourse, in line with the embedded environmental measures C-22 and C-9 (**Table 7F 7.1**), and D-1 and D-2 (**Table 7F 7.2**).

6.3.15 The results of the modelling undertaken by the Environment Agency for the Mersey Estuary 2018 study[18] indicate that the HPP HAGI LoD remains free from flooding in the present day 0.5% AEP tidal event.

6.3.16 As the Project is an NSIP, maximum credible scenarios for both tidal and fluvial flood were investigated to ensure the resilience of the HAGI in line with the design standards detailed in **Section 2.6**.

6.3.17 The EA Mersey Estuary 2018 model[18] with updated inflows (see **paragraph 6.3.10** & **Figure 5**) shows that the HPP HAGI is outside the influence of tidal flooding from the Mersey Estuary. The peak tidal design event (0.5% AEP, higher central allowance up to 2095) flood levels closest to the site rise to 6.10m AOD, compared to a minimum site level of 9.43m (based on 1m EA LiDAR (2019) and 0.1m Cadent DSM[39]). The design flood extent is at a distance of 795m from the HAGI site. Even the H++ scenario flood extent does not get closer than 690m, with a peak flood level of 7.85mAOD.

6.3.18 The existing EA owned model of the Gale Brook, the Stanlow Oil Refinery model[21], was developed by Jacobs in 2019, which was itself based on the River Gowy Model developed by JBA in 2016. For the purpose of this FRA, the model was further refined using channel survey for the Gale Brook commissioned by the Applicant. Furthermore, the 2D area was extended to more accurately define the

[38] Cheshire West and Chester Level 1 Strategic Flood Risk Assessment Available at: flood-risk-assessment-final-report (cheshirewestandchester.gov.uk) (Accessed: August 2024)

[39] Sensat Survey Report (2023) for Cadent: S23314 - HyNet North West Hydrogen Pipeline

flood risk from this watercourse, and the most recent climate change allowances were incorporated to evaluate the impact of future climate change on the flood risk. The resultant model has been named Gale Brook model and is documented in **Annex 7F3**. The results in **Figure 6** show that the site remains free from flooding for both the design event and the credible maximum scenario.

6.3.19 EA RoFSW mapping shows a surface water flow paths along the northern part of the LoD of the HPP HAGI at 0.1% AEP. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Annex 7F6F**).

6.3.20 Stanlow HPP HAGI is proposed in an area of limited potential for groundwater flooding to occur, therefore groundwater flooding is unlikely to be an issue.

6.3.21 There are no canals or reservoirs in the Gale Brook catchment that could pose a residual risk of flooding to the site in case of infrastructure failure.

Higher Walton Option HWH2

6.3.22 Higher Walton Option HWH2 HAGI LoD (HWH2) is predominantly located in Flood Zone 1 with just the north-west corner intersecting with Flood Zone 3 and the north-eastern and south-western corners intersecting with Flood Zone 2 (**Figure 2**, Sheet 12). However, it is located within a 'dry island' surrounded by the Flood Zone 2/3 associated with the River Mersey in the north, and with the Manchester Ship Canal in the south. The existing site level exceeds 10.2mAOD, except for a circa 50 x 7m rectangle in the north-east corner of the HWH2 LoD that drops to 9.2mAOD (based on the Sensat survey[39]) and 8.6mAOD (based on EA LiDAR from 2022).

6.3.23 Results of the modelling undertaken by the Environment Agency for the Mersey Estuary 2016 study indicate that the HWH2 LoD remains free from flooding in a 0.5% AEP present day tidal event.

6.3.24 As the Project is an NSIP, maximum credible scenarios for both tidal and fluvial flooding were investigated to ensure the resilience of the HAGI in line with the design standards detailed in **Section 2.6**.

6.3.25 The EA Mersey Estuary 2018 model[18] with updated inflows (see **paragraph 6.3.10** and **Figure 7**) shows that the HWH2 LoD is not at risk of flooding from the tidal design event (0.5% AEP (2100) Higher Central). The north-east corner of the LoD is shown to experience peak flood levels of 8.1mAOD during the Upper End climate change scenario. In the model, this flood level equates to a maximum depth of 2.2m. However, the LiDAR data informing the model is now superseeded, and the land here actually exceeds the peak modelled flood level of the 0.5% AEP event with the Upper End climate change scenario. The maximum credible scenario, H++, shows flooding to 9.3mAOD on the eastern sections of the LoD, equating to calculated maximum depths of 0.1 to 0.7m in the north-east corner (using the Sensat and EA LiDAR ground levels, respectively).

6.3.26 The Environment Agency River Mersey (Warrington) model[19] (2019) was updated, including with recent topography and land use data, and then run for the fluvial design event (0.5% AEP (2100) Higher Central). The more extreme 0.5% AEP (2100) Upper End event was considered as the credible maximum scenario.

However, the model was unable to cope with the high volume of inflows and no results could be obtained for this event.

6.3.27 A model report is included in **Annex 7F4**. The results in **Figure 8** show that the HWH2 LoD (elevated to at least to 9.2mAOD) remains free from flooding for the design event. The closest flood extents are in the marshes approximately 480m north-east of the site and rise to c. 6.3mAOD. The north bank of the Manchester Ship Canal at Lower Walton 1,370m east of the site gets flooded to 7.93mAOD.

6.3.28 While no flood levels and extents could be generated for the credible maximum scenario, the design event suggests that there is a freeboard of at least 2.2m for the majority of the HWH2 LoD. It is unlikely that the credible maximum flood level would be more than 2.2m higher than the design level. In the east and west of the HWH2 LoD, there is some potential for surface water ponding, but a low risk of flooding from groundwater. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Annex 7F6**).

6.3.29 The HWH2 LoD is located on the north bank of the Manchester Ship Canal. The Canal is cut into the ground here, rather than embanked, therefore it is unlikely to breach near the HWH2 LoD. One flood event associated with the Manchester Ship Canal has been reported in the Warrington Local Flood Risk Management Strategy[40] which occurred upstream of the HWH2 LoD. Based on the information, the risk of canal breach is low. Risk of flooding from the Canal exceeding its banks, has been considered as part of fluvial flood risk in paragraphs 6.3.26 to 6.3.28.

Higher Walton Option HWH8

6.3.30 The Higher Walton HAGI Option 8 (HWH8) LoD is located in the floodplain (Flood Zone 2 and 3) of an unnamed watercourse. The watercourse discharges into the Manchester Ship Canal approximately 90m north of the HWH8 LoD. At the time of writing the Draft Environmental Statement a modelling study is in progress to refine the information of fluvial flood risk for this option and will be provided for the final FRA supporting the DCO application.

6.3.31 The site is not at risk of surface water or groundwater flooding. Given the HWH8 LoD is at risk of fluvial flooding, in flood risk terms, the HWH8 LoD does not appear to be an improvement over the HWH2 LoD presented in **paragraph 6.3.22**.

6.3.32 The HWH8 LoD is located on the south bank of the Manchester Ship Canal. The Canal is cut into the ground here, rather than embanked, therefore it is unlikely to breach near the HWH8 LoD. Furthermore, the land to the south of the Canal continually rises and inundation from the Canal would therefore be routed away from HWH8 LoD and the risk of flooding from a canal breach is low.

[40] Warrington Borough Council, Flood Risk Management Strategy 2017, Warrington Borough Council

Hydrogen Storage Facility (HSF)

6.3.33 The HSF HAGI LoD is inland and not at risk of tidal flooding. The HSF HAGI LoD is located adjacent to the Puddinglake Brook, which is a potential source of fluvial flooding.

6.3.34 The EA Flood Map shows the southern boundary of the HSF HAGI LoD to slightly intersect with Flood Zones 2 and 3 (**Figure 2**, sheet 34). A simple hydraulic model was developed to delineate this potential risk in more detail. The model report is provided in **Annex 7F5**. The model results in **Figure 9** show that the HAGI LoD is outside of the design flood extent (0.5% AEP with higher central allowance for the '2080s').

6.3.35 As the Project is an NSIP, a maximum credible scenario of fluvial flood risk was investigated to ensure the resilience of the HAGI in line with the design standards detailed in **Section 2.6**.

6.3.36 The more extreme 0.5% AEP (2100) Upper End event was considered as the credible maximum scenario. Model results demonstrated that the HAGI LoD remains outside of the flood extent of this credible maximum scenario.

6.3.37 EA RoFSW mapping suggests that the LoD of the HSF HAGI is at risk of surface water flooding at 0.1% AEP along the southern boundary. This area of surface flooding is associated with the Puddinglake Brook, which has been modelled and shown not to pose a risk to the site. In addition, there is a small area of surface water ponding at 0.1% AEP in the south-west corner of the HSF HAGI LoD. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Appendix 7F6**).

6.3.38 The HSF HAGI LoD is located in areas with potential for groundwater flooding to occur at surface.

6.3.39 The closest canal is Trent and Mersey Canal, located at more 1.5km east of the HSF HAGI LoD. There are few recorded events of breaching and overtopping that happened in different sections of the Canal[41]. Due to the distance from the Canal, the HSF HAGI is not considered to be affected by a potential canal breach.

Central Hub

6.3.40 Available flood risk mapping shows that the location of Central Hub HAGI is in Flood Zone 1, nor is it at risk of notable surface and groundwater flooding (**Figure 2**, Sheet 8). The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Appendix 7F6**)

6.3.41 The Central Hub HAGI is located approximately 1km north of the Trent and Mersey Canal. There are few recorded events of breaching and overtopping that happened in different sections of the canal. Due to the distance from the Canal, the Central Hub HAGI is not considered to be affected by a potential canal breach.

[41] Cheshire West and Chester, Flood Risk assessment Available at: flood-risk-assessment-final-report (cheshirewestandchester.gov.uk)

Clockface

6.3.42 Available flood risk mapping (**Figure 2**, Sheet 17) shows that the location of the Clockface HAGI LoD is in Flood Zone 1. A localised area in the north of the Clockface HAGI LoD is susceptible to ponding from surface water flooding during 3.33% AEP event.

6.3.43 The Clockface HAGI is at low risk of groundwater flooding and has potential for groundwater flooding to occur below ground level. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Appendix 7F6**)

6.3.44 The closest canal is Sankey Canal, at about 5km distance from the Clockface HAGI's LoD. The distance is significant, and the area is considered not to be affected by a potential canal breach.

Partington Option PH1

6.3.45 Partington Option PH1 HAGI LoD is located in Flood Zone 1 (**Figure 2**, Sheet 27). An area of surface water ponding occurs during 0.1% AEP event, in the centre of the LoD. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Appendix 7F6**).

6.3.46 The Partington HAGI Option PH1 is located in an area where there is potential groundwater flooding to occur at surface.

6.3.47 Partington HAGI Option PH1 LoD is located at about 1km east of the Manchester Ship Canal, at an elevation 2-3m higher than the Canal. No major flood events associated with the Manchester Ship Canal have been reported in this area in the Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA[42]. Based on the information, the risk of canal breach is low given that the Canal is heavily controlled and the canal flooding is considered as very low likelihood residual risk.

Partington Option PH2

6.3.48 The Partington HAGI LoD Option PH2 is located in Flood Zone 1. The location is not at risk of surface and groundwater flooding.

6.3.49 Partington Option HAGI PH2 LoD is located at about 1.3km south-east of the Manchester Ship Canal. No major flood events associated with the Manchester Ship Canal have been reported in this area in the Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA[42]. Based on the information, the risk of canal breach is low given that the canal is heavily controlled, and the canal flooding is considered as very low likelihood residual risk.

6.3.50 Overall, in terms of flood risk, Partington HAGI Option PH2 LoD appears to be more favourable than Partington HAGI LoD Option PH1.

[42] Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA, 2010 Strategic Flood Risk Assessment (SFRA) | Manchester City Council

Runcorn

6.3.51 The Runcorn HAGI LoD is located in Flood Zone 1, approximately 250m north of the Weaver Navigation canal and the Flood Zones 2 and 3 associated with it (**Figure 2**, Sheet 5). The Runcorn HAGI is at situated 20m higher than the bank of the Canal, and therefore despite its proximity to Flood Zones 2 and 3, there is no risk of flooding fluvial or tidal flooding.

6.3.52 A ditch runs through the Runcorn HAGI LoD north of the proposed HAGI layout. It appears to enter a culvert near the western LoD boundary, which discharges into a larger ditch further west, outside the Runcorn HAGI LoD. There is no surface water flooding associated with the ditches. None of the proposed HAGI elements intersect with the ditch or culvert.

6.3.53 An area of surface water ponding at 3.33% AEP coincides with about a third of the proposed footprint of Runcorn HAGI. The surface water drains towards the east and joins a surface water flow route. The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Annex 7F6**).

6.3.54 The Runcorn HAGI LoD is partially at risk of groundwater flooding, with the majority of the area intercepting an area with potential for groundwater flooding to occur at surface.

6.3.55 Based on the available information about past canal floods[43], the likelihood of canal breach and failure associated with the Weaver Navigation is low. The Weaver Navigation is cut at this location and the Runcorn HAGI LoD is elevated above the Canal. The risk of canal flooding from failure is therefore considered to be low.

Warburton

6.3.56 The Warburton HAGI LoD is in Flood Zone 1 and is at low risk of surface water flooding (**Figure 2**, Sheet 25). The allowable surface water discharge volume and the attenuation provisions required have been addressed as part of the Outline Drainage Strategy (**Appendix 7F6**).

6.3.57 BGS groundwater flood mapping[35] shown in **Figure 7.9** of **Chapter 7: Water Environment** indicates that the site may be susceptible to flooding of subsurface structures.

6.3.58 Warburton HAGI LoD is located between two canals, the Manchester Ship Canal on the west and the Bridgewater Canal on the east. Both canals are at distances larger than 1km and at elevations lower than the Warburton HAGI LoD location. The Bridgewater Canal is owned and operated by the Manchester Ship Canal Company in conjunction with the Bridgewater Canal Trust. Few historical flood events have been reported[42] and none of these events affected the Warburton

[43] Halton Borough Council, Preliminary Flood Risk Assessment Review, 2017 Microsoft Word - PFRA Update 2017_v9 final - FINAL (halton.gov.uk)

HAGI area. Therefore, it is considered that the likelihood of canal flooding and breach is low.

HAGI Summary

6.3.59 In summary:

- Rocksavage HAGI LoD is located in Flood Zone 1, the site remains free from tidal and fluvial flooding for the design events and credible maximum scenario and groundwater flooding is unlikely to be an issue.
- HPP HAGI LoD is located in Flood Zone 1, the site remains free from tidal and fluvial flooding for the design events and credible maximum scenario and groundwater flooding is unlikely to be an issue.
- Higher Walton HWH2 HAGI LoD is mainly in Flood Zone 1, remains free from flooding in a 0.5% AEP tidal event. There is a residual flood risk for the H++ tidal scenario. Groundwater flooding is unlikely to be an issue.
- Higher Walton HWH8 LoD is in Flood Zones 2 and 3 and detailed modelling to determine the fluvial and tidal flood risk is pending. The location is not at risk from surface and groundwater flooding.
- HSF HAGI LoD is not at risk of tidal/fluvial flooding, however there is a potential for groundwater flooding to occur at surface. The Site is close to the edge of Flood Zones 2 and 3. Modelling shows that site remains outside of flood zones, even when the influence of climate change over the project lifetime is considered.
- Central Hub HAGI LoD is not at risk of any type of flooding.
- Clockface HAGI LoD is in Flood Zone 1, ponding occurs due to surface flooding and there is potential for groundwater flooding.
- Partington HAGI PH1 LoD is in Flood Zone 1 (no fluvial or tidal flood risk) and it is at risk of surface water ponding and potential groundwater flooding.
- Partington HAGI PH2 LoD is in Flood Zone 1(no fluvial or tidal flood risk) and has potential for groundwater flooding and surface water ponding.
- Runcorn HAGI LoD is in Flood Zone 1, however there is potential for surface water ponding and groundwater flooding. A minor watercourse runs through the LoD and into a culvert on the eastern boundary of the LoD, but avoids the proposed HAGI layout.
- Warburton HAGI LoD is in Flood Zone 1 (no fluvial or tidal flood risk) and the site may be susceptible to flooding of subsurface structures.

Table 7F 6.1 Flood Risk Overview – HAGI Limits of Deviation (LoDs)

HAGI	Flood Zone	Watercourse catchment	NPPF compatible	Distance to nearest Main River/flood defences	Surface Water Flooding	Reservoir Inundation	Canal flooding from failure	Groundwater flooding	Comments
Rocksavage	Predominantly located in Flood Zone 1, with only the northern corner of the LoD located in Flood Zones 2 and 3 (Fluvial) , however detailed modelling shows that the LoD remain free of fluvial and tidal flooding for all relevant scenarios	River Weaver	Yes, as detailed modelling shows LoD not at risk of fluvial or tidal flooding	>200m	No	Yes – but only when river in flood*	Low risk from the Weaver Navigation Canal	Not considered to be prone to groundwater flooding	Tidally defended to present day 0.5% AEP
		Mersey Estuary		>2.9km					
Hydrogen Production Plant (HPP)	1	Gale Brook (Gowy Catchment)	Yes	>120m	Surface water flow route with 0.1% AEP	No	No canals in the vicinity	Limited potential for groundwater flooding to occur	
Higher Walton (Option HWH2)	Predominantly located in Flood Zone 1, with north-west corner in Flood Zone 3 (fluvial & tidal), north-eastern and south-western corners in Flood Zone 2 (fluvial & tidal)	Manchester Ship Canal	Yes, as detailed modelling shows LoD not at risk of fluvial or tidal flooding, except in most extreme tidal H++ scenario	<10m	Ponding 3.33% AEP	No	Low risk from Manchester Ship Canal	Limited potential for groundwater flooding to occur	Majority of site within a ‘dry island’. Detailed modelling shows LoD not at risk of fluvial or tidal flooding, except in most extreme tidal H++ scenario.
		River Mersey		>800m					
Higher Walton (Option HWH8)	3 (Fluvial)	Unnamed tributary to the	If Exception Test is passed	<10m	No	No	Low risk from Manchester Ship Canal	Not considered to be prone to groundwater flooding	

HAGI	Flood Zone	Watercourse catchment	NPPF compatible	Distance to nearest Main River/flood defences	Surface Water Flooding	Reservoir Inundation	Canal flooding from failure	Groundwater flooding	Comments
		Manchester Ship Canal							
Hydrogen Storage Facility (HSF)	Predominantly located in Flood Zone 1, with the southern boundary in Flood Zones 2 and 3	Puddinglake Brook	Yes, as modelling shows LoD not at risk of fluvial flooding	30m	Minor ponding 0.1% AEP	No	Low risk from Trent and Mersey Canal	Potential for groundwater flooding to occur at surface	Site close to edge of flood zone. Modelling shows that site remains outside of flood zone under influence of climate change.
Central Hub	1	Whitely Brook	Yes	>500m	Minor ponding 0.1% AEP	No	Low risk from Trent and Mersey Canal	Not considered to be prone to groundwater flooding	
Clock Face	1	Whittle Brook	Yes	>750m	Ponding North side 3.33% AEP	No	Low risk from Sankey Canal	Potential for groundwater flooding of development below ground level	
Partington (Option PH1)	1	Manchester Ship Canal	Yes	>1km	Minor ponding 0.1% AEP	No	Low risk from Manchester Ship Canal	Not considered to be prone to groundwater flooding	
		Red Brook		>1km					
Partington (Option PH2)	1	River Mersey	Yes	>1km	No	No	Low risk from Manchester Ship Canal	No	
		Sinderland Brook		>900m					
Runcorn	1	Weaver Navigation	Yes	30m	Ponding 3.33% AEP	No	Low risk from Weaver Navigation Canal	Not considered to be prone to groundwater flooding	Mapping indicates a minor watercourse stops north of site. Currently unknown if it sinks or is culverted under site.

HAGI	Flood Zone	Watercourse catchment	NPPF compatible	Distance to nearest Main River/flood defences	Surface Water Flooding	Reservoir Inundation	Canal flooding from failure	Groundwater flooding	Comments
Warburton	1	River Bollin Tributary	Yes	>375m	Ponding NE corner 3.33% AEP	No	Low risk from Manchester Ship Canal and Bridgewater Canal	Potential for groundwater flooding of development below ground level	

*The Environment Agency publish two reservoir inundation maps: one ‘dry’ assuming normal flow in the river, and one ‘wet’ assuming high river flows.

6.4 Block Valve Installations (BVIs)

6.4.1 Two BVIs are proposed for the Project, at Sworton on the East Corridor and near Hartford on the South Corridor. Both BVIs are in Flood Zone 1 and more than 16m away from a Main River. The locations are not at risk of surface water flooding. Only one of the location (Hartford BVI) along the South Corridor has the potential for groundwater flooding to occur at surface. Any changes to surface water runoff rates and pathways as part for the development have been addressed as part of the Outline Drainage Strategy (**Annex 7F6**).

6.4.2 The Hartford BVI is located at about 3km west from the Weaver Navigation Canal. Given the great distance and the elevation difference, the BVI being higher, the risk of canal flooding is low and is considered as residual.

6.4.3 The Sworton BVI is located 3km south of the Bridgewater Canal and 9km northeast from the Trent and Mersey Canal. The BVI LoD lies between 73mAOD and 69mAOD, making this location on of the highest points in the Project. Considering the distance and elevation difference between the BVI and the Canal with the BVI being higher, the location of the Sworton BVI is considered at low risk of canal flooding.

6.5 DNO Connections

6.5.1 The HAGIs sites will require connection to the local distribution network operator's (DNO) electrical distribution network. The DNO connections are permanent overground structures. Nine DNO connections are located in the vicinity or within the HAGIs areas and one is near Hartford BVI.

6.5.2 Eight of the ten DNO connections are in Flood Zone 1.

6.5.3 The DNO connection adjacent to Higher Walton HAGI Option HWH2 intercepts Flood Zones 2 and 3, but detailed modelling (see **paragraphs 6.3.25** and **6.3.26**, and **Figures 7** and **8**) shows that it remains free from flooding for the fluvial and tidal design events, as well as the credible maximum tidal event and H++. It is at limited risk of groundwater flooding to occur during period of extended intense rainfall.

6.5.4 The DNO connection at Rocksavage HAGI is in Flood Zone 2, but detailed modelling (see **paragraphs 6.3.10** and **6.3.11**, **Figures 3** and **4**) shows that only the far north-eastern corner of the DNO connection area is at risk of fluvial flooding from the design and maximum credible event. The site is not at risk of surface or groundwater flooding.

6.5.5 Three DNO connections are at risk of surface water flooding. Areas of ponding are identified at the DNO connections at Partington Option PH1, Higher Walton Option PH2 HAGIs and Hartford BVI.

6.5.6 Six locations have some element of potential groundwater flooding risk. Two locations, associated with Higher Walton Option HWH2 and Clockface HAGIs, are in areas with limited potential for groundwater flooding to occur during periods of extended intense rainfall. Four DNO connections at Runcorn HAGI, Warburton

HAGI, HSF HAGI and Hartford BVI, are located in areas with potential for groundwater flooding of property situated below ground level.

The DNO connections are accompanying elements to the HAGIs and BVI, and thus share the same risk of canal flooding. The DNO connections associated with Runcorn, Central Hub, Stanlow HPP, Higher Walton Option HWH2, Rocksavage, Partington Option PH1 HAGIs and Hartford BVI are located close to a canal. Based on the analysis undertaken for the risk of flooding from canals for the HAGI and BVI sites above, it is considered that the likelihood of canal flooding and canal failure is low

6.6 Pigging Areas

6.6.1 Two permanent pigging areas are being considered: one near Partington Option PH1 HAGI and one near Higher Walton Option HWH2 HAGI.

6.6.2 The pigging LoD area at Partington Option PH1 HAGI is in Flood Zone 1 and the risk of surface water flooding is low, only the north-west corner of the area is at risk of flooding at 0.1% AEP. This location is not at risk of groundwater flooding.

6.6.3 The entire pigging area at Higher Walton Option HWH2 is in Flood Zone 2 and 3 associated with the Manchester Ship Canal and River Mersey. However, more detailed modelling (see paragraphs **6.3.25** and **6.3.26**, and **Figures 7** and **8**) shows that the pigging area remains outside the modelled peak flood extents for the tidal and fluvial design events. The Higher Walton pigging area sits at 9mAOD and thus 1.24m above the peak fluvial design water level of 7.76mAOD. It also remains dry for the tidal maximum credible scenario of the 0.5% AEP event (closest peak flood level of 7.93mAOD) and even in the most extreme H++ scenario (closest peak flood level of 8.91mAOD).

6.6.4 Even though no model results are available for the credible maximum fluvial event, the Higher Walton Option HWH2 pigging area is likely to remain free from flooding due to its elevation of at least 9mAOD, which is 1.52m above the closest peak design flood level of 6.02mAOD at approximately 500m north and 1.52m above the next closest peak design flood level of 7.48mAOD at 790m east. This pigging area is not at risk of surface or groundwater flooding.

6.6.5 As described in **paragraphs 6.3.22 and 6.3.47** of this FRA, the Manchester Ship Canal is in vicinity of the Partington Option PH1 and Higher Walton Option HWH2 HAGIs and their accompanying elements, such as the pigging areas. Based on the information about canal flooding and breach, both pigging areas are considered to be at low risk from canal breach and the canal flooding is considered a residual risk.

7. Flood Risk Management

7.1 Introduction

7.1.1 This section sets out the embedded environmental measures and Drainage Strategy design principles that are incorporated/embedded into the Project to ensure that flood risk impacts to and from the Project are minimised and appropriately managed during the construction and operation and maintenance phases.

7.1.2 For all temporary works, this FRA sets out the requirements for flood mitigation and drainage management measures for construction. It is currently envisaged that compliance with the requirements of the outline CEMP (**Draft Environmental Statement Appendix 3B: Draft Outline Construction Environmental Management Plan**) would be secured via relevant DCO Requirements, which would be discharged by the relevant Local Planning Authority in consultation with the EA and LLFA(s) prior to commencement. Further regulatory scrutiny of the details of works in proximity to watercourses (particularly watercourse crossings) would be secured through FRAP or Ordinary Watercourse Consent applications. Further information on this process will be provided in the DCO application.

7.1.3 It is envisaged that flood mitigation and drainage requirements for permanent infrastructure would be secured via relevant DCO Requirements, for instance requiring that detailed drainage design be in general accordance with the measures set out in this FRA and the accompanying Drainage Strategy, within the DCO application, and that this is subject to approval by the relevant LPA in consultation with the EA and/or LLFA(s) prior to commencement.

7.1.4 Embedded environmental measures to address flood risks identified in **Section 5** for the construction phase are presented in **Section 7.2** and **Table 7F 7.1**; those to address the risks to permanent infrastructure identified in **Section 5.8** for the operation and maintenance phase are presented in **Section 7.3** and **Table 7F** 7.2.

7.1.5 All embedded environmental measures relating to flooding follow the outline principles presented in **Section 7.6** of **Chapter 7: Water Environment** and are a sub-set of the wider list of embedded water environment measures in **Table 7.16** relating to the water environment. The Embedded Environmental Measure ID is included in the tables in **Chapter 7: Water Environment**. Any changes to the design between statutory consultation and the DCO application submission will be reassessed as part of the final FRA.

7.2 Flood Risk Management – Construction Phase

7.2.1 Embedded environmental measures for the construction phase are identified by their ID in **Table 7F 7.1** and link to the comprehensive **Table 7.16** of measures for the Water Environment in **Chapter 7.**

Table 7F 7.1 Embedded environmental measures relevant to flood risk during the construction phase

Construction Element	Flood Risk	Mitigation	Embedded Environmental Measure ID
All	All sources	Preferential siting	C-22
		Stand-off distances from watercourses, flood defences and water supply and sewage infrastructure	C-9
		Construction Environmental Management Plan (CEMP*)	C-1
		Flood Emergency Response Plan (FERP)	C-23
	Surface water run-off	Drainage and Water Management Plan (DWMP)	C-2
	Fluvial, groundwater, surface water	Timing of works	C-3
	Fluvial/Tidal	Stand-off distances from watercourses, flood defences and water supply and sewage infrastructure	C-9
Material storage areas	Fluvial, tidal, surface water	Preferential siting	C-22
Pipeline Crossings	Fluvial	Watercourse Crossing Plan (WCP)	C-25
		Trenchless pipeline crossings	C-26
		Trenched crossings of minor watercourses	C-27
Trenchless Crossing compounds	All sources	Minimise raised structures in floodplains	C-24
	Fluvial, tidal, surface water	Preferential siting	C-22

Construction Element	Flood Risk	Mitigation	Embedded Environmental Measure ID
		Construction Environmental Management Plan (CEMP*)	C-1
		Silt management measures	C-8
		Timely reinstatement following completion of works	C-10
		Pollution Prevention Plan (PPP)	C-13
		Storage of hydrocarbons and other chemicals	C-14
		Monitoring and inspection programme	C-19
Temporary Access Routes	Fluvial	Temporary access tracks design	D-4
		Temporary construction access watercourse crossings	C-28
HAGI construction Compounds	All sources	Minimise raised structures in floodplains	C-24
		Development of flood protection and drainage measures	D-3
	Fluvial, tidal, surface water	Preferential siting	C-22

*The construction works would be implemented via a Construction Environmental Management Plan (CEMP), which would cover the management of surface water drainage and flood risk mitigation throughout the duration of the construction works. **Annex 7F6**: **Outline Drainage Strategy** summarises the key methods by which surface water and dewatered groundwater would be managed to avoid flood risk and protect the water quality of receiving water bodies during construction.

7.3 Flood Risk Management – Operation and Maintenance Phase

7.3.1 The general embedded environmental measures for flood risk and drainage for permanent infrastructure during the operation and maintenance phase of the Project are set out by their ID in **Table 7F 7.2** and link to the comprehensive **Table 7.16** of measures for the Water Environment in **Chapter 7**. These have been

applied throughout the design process to minimise flood risk to and from the Project.

7.3.2 The detailed modelling for the sites where potential flood risk from fluvial and/or tidal sources have been identified have demonstrated that these sites are safe from flooding from these sources for the lifetime of the Project and no further mitigation is required. This is provided that the location of the sites remains unchanged and the sites are constructed at least to existing ground levels, without any excavation or digging below the current ground level. However, at this stage of the Project, detailed designs are not all finalised. Any uncertainty regarding the final location of infrastructure is addressed by using Limits of Deviation (LoD), which delineate the area within which the infrastructure will be sited without the exact location having been finalised. Flood risk was assessed for the entirety of the LoD areas in **Sections 5** and **6**.

Table 7F 7.2 Embedded environmental measures relevant to flood risk during the operation and maintenance phase

Project Element	Flood Risk	Mitigation	Embedded Measure ID
All	All sources	Underground pipeline	D-1
		Optimisation of routing	D-2
Pipeline	Groundwater	Management of subsurface flow pathways	D-5
	Fluvial, Surface water	Underground pipeline	D-1
HAGI, Block Valve Installation, DNO connections and pigging areas	All sources	Optimisation of routing	D-2
	Surface Water run-off	Development of flood protection and drainage measures	D-3

7.4 Drainage Strategies

7.4.1 Each of the HAGI and BVI sites have different site characteristics and have therefore been assessed individually to establish preferred drainage solutions to ensure no increase in run off from the sites. Drainage strategies are provided in **Annex 7F6**. They adhere to the LLFA requirements on runoff rates and follow the principles of the SuDS hierarchy.

8. Project Specific Planning Policy Requirements

8.1 The Sequential Test

8.1.1 The primary routeing objective for a pipeline is to join up the supply points with the demand points, in this case connecting the Hydrogen Production Plant with the Hydrogen Storage Facility and connecting both of these with the agreed customers and potential off-take points.

8.1.2 The relevant flood risk datasets described in **Section 4.2** were used in the mapping exercise to identify the most appropriate corridors between these points, with a preference to avoid areas of flood risk wherever possible. Strategic options for connection routes were evaluated and Option A was taken forward as the most appropriate option (see **Appendix 2A**). From this a Route Corridor and HAGI Search Areas were developed. Two rounds of consultation and ongoing engineering and environmental work allowed the corridors and HAGI locations to be refined. These route alignments formed the Preliminary Order Limits at the PEIR stage. Following another consultation exercise and further environmental/engineering work, the feedback and new findings were worked into the Draft Order Limits considered in this FRA supporting the Draft ES. This sequential design process is described in **Section 2** of **Chapter 2: The Project**, and fully documented in **Appendix 2A: Design Evolution**.

8.1.3 In some locations, the requirements of the project design along with consideration of other environmental and engineering constraints has meant that it was not possible to avoid development in an area of flood risk. Therefore, some construction activity would have to occur within flood risk areas. The construction areas affected by flood risk have been quantified in **Section 5**. The assessment identified the following:

- Pipeline: West Corridor areas associated with the tidal River Mersey and areas along M56 from Frodsham to Elton. The North Corridor expanses of floodplain associated with the River Mersey and Manchester Ship Canal and some smaller Main Rivers. The East Corridor is associated with risk from the River Bollin. Along the South Corridor, areas at risk are associated with River Weaver and Weaver Navigation. Embedded environmental measures to address the flood risk during pipeline construction are presented in **Table 7F 7.1**.
- Material storage areas: Acton Bridge material storage area is in Flood Zone 2 in an area prone to recent flooding and Eastford Road material storage area is in Flood Zone 3. Small part of the material storage area at Sutton Manor intersects with fluvial flood zone. In addition to the general mitigation measures for all construction elements, implementing a sequential approach to storage on site will ensure that the pipes are placed away from high flood risk areas and conveyance routes.

- 22 trenchless crossing compounds are located in Flood Zone 3, four in Flood Zone 2 including the compound at Acton Bridge that is in area where recent flooding has been reported. To address the flood risk, the embedded environmental measures C-25, C-26 and C-27 in **Table 7F 7.1** will be incorporated during the construction phase.
- Three temporary HAGI compounds all associated with Higher Walton options HWH2 and HWH8 are in Flood Zone 3. At the north-eastern part of the Clockface HAGI surface water ponding occurs at 3.33%AEP. Incorporating the embedded measures C-24, D-3 and C-22 will help mitigate the flood risk.

8.1.4 Permanent project features were designed out of areas of flood risk wherever possible. Flood risks to permanent above ground infrastructure for the operational phase of the Project are assessed in **Section 5.8**. This assessment demonstrated the following:

- HSF HAGI LoD has potential for groundwater flooding to occur at surface. Clockface HAGI LoD, Partington Option PH1 HAGI LoD, Partington Option PH2 HAGI LoD and Runcorn HAGI LoD have potential for surface and groundwater flooding.
- The location of Hartford BVI has potential for groundwater flooding to occur at surface.
- The DNO adjacent to Higher Walton HAGI Option HWH2 is in Flood Zones 2 and 3 and surface flooding occurs at 0.1%AEP. The DNO at Rocksavage HAGI is in Flood Zone 2. The DNO at Hartford BVI has potential for surface flooding at 0.1% AEP and groundwater flooding. The DNO at Partington Option PH1 has elements of surface water flooding at 3.33% AEP. The DNOs at Clockface (category A), Warburton (category B), HSF (category B and C) and Runcorn (category B) HAGIs are at risk of groundwater flooding.
- The pigging area at Partington Option PH1 HAGI is mainly not at risk, only the north-west corner of the area is at risk of flooding at 0.1% AEP.

8.1.5 To minimise the flood risk on the permanent infrastructure (HAGIs, BVI, DNO connections and pigging areas) the embedded measure D-2 deals with the placement of permanent infrastructure within the LoD to reduce flood risk. The management of surface water run-off is in line with the Drainage Strategy.

8.1.6 At this stage of the Project, optionality in the design has been much reduced, but some options are still being considered (e.g. route options for complex trenchless crossing locations), and alternative location options for the Higher Walton HAGI and the Partington HAGI. Consideration of flood risk will continue to feed into the final choice of siting alongside other environmental and engineering constraints.

8.2 The Exception Test

8.2.1 The requirements of the Exception Test were set out in **Section 2**, along with the flood risk vulnerability and flood zone ‘compatibility’ matrix in **Table 7F 3.1**, which confirmed that the Exception Test needs to be passed for the infrastructure

elements of the Project located in Flood Zone 3a and 3b, and which cannot be moved out of the Flood Zone.

Wider Sustainability Benefits

8.2.2 Part 1 of the Exception Test requires the Project to provide wider sustainability benefits to the community that outweigh flood risk. As stated in NPS EN-1[2], this would include the benefits (including need) related to the infrastructure.

8.2.3 The Project would make a significant contribution to delivering critical energy infrastructure for the UK and reducing wider carbon emissions in accordance with National Policy (**Section 2** of **Chapter 2: The Project**).

Flood Risk

8.2.4 Part 2 of the Exception Test requires that the Project would be safe for its lifetime taking account of the vulnerability of its users, without increasing flood risk elsewhere and, where possible, would reduce flood risk overall. 'Essential infrastructure' in Flood Zone 3a should also be designed and constructed to remain operational and safe in times of flood; and result in no net loss of floodplain storage.

8.2.5 The embedded environmental measures specified in **Table 7F 7.1** for the construction and in **Table 7F 7.2** for the operation and maintenance phase would be applied to the elements in Flood Zone 3 to satisfy the above criteria. Hydraulic modelling has been carried out for the permanent HAGIs identified as having potential flood risks based on flood maps. This modelling has shown that these sites will remain safe from flooding throughout the proposed development's lifetime without the need for additional mitigation measures, as long as existing ground levels are not reduced.

8.2.6 Flood warning and evacuation plans will be prepared for Project elements that remain at residual risk of flooding, to ensure safe emergency site access and egress routes are available. Preparation of these plans is assured through the CEMP.

8.2.7 There would be a small residual risk from the temporary displacement of floodplain storage by materials stored in areas at risk of flooding. There could also be a risk from materials becoming mobilised in flood water and leading to obstructions and/or increased flood risk elsewhere. Such risks are associated with two material storage areas (Acton Bridge (Flood Zone 2) and Eastford Road (Flood Zone 3)), and several trenchless crossing compounds. Impact would be temporary and limited to a few months' duration at a time, with no permanent loss of floodplain storage.

8.2.8 The affected project elements are therefore considered to have passed the Exception Test, with the exception of the material storage area in Flood Zone 3 at Eastford Road, for which further flood risk assessment and design development work is required. This work will be completed within the final FRA submitted with the DCO application.

9. Summary and Conclusions

9.1 Summary

9.1.1 As the Project is classified as an NSIP, the FRA has been carried out in accordance with NPS EN-1[2], which details planning policy in regard to NSIPs in the energy sector, and the NPPF plus associated PPG where relevant, for additional guidance regarding flood risk and development.

9.1.2 This FRA considers the flood risks associated with the permanent operational infrastructure associated with the proposed pipeline as well as the construction elements.

9.1.3 Flood risks associated with tidal, fluvial, groundwater and surface water sources have been identified during the construction and operation/maintenance phases of the Project.

9.1.4 The flood risk management standards, including the appropriate climate change uplifts to be applied, for operation and maintenance phase elements of the Project are set out in **Section 2.6**. During construction and operation phases of the Project, there would potentially be an increase in impermeable area, which result in greater surface water runoff. To manage the increased runoff from these areas, drainage strategies (**Annex 7F6**) have been developed to ensure there is no increase in flood risk elsewhere.

9.2 Conclusions and Recommendations

9.2.1 The FRA demonstrates that the Project is safe from flood risk and does not increase flood risk to third party receptors during its construction, with the following exceptions:

- There are outstanding flood risks to two of the 36 material storage areas: Acton Bridge, Eastford Road.
- Approximately half of the material storage area at Acton Bridge coincides with Flood Zone 2 associated with the River Weaver and is in an area where flooding has been recorded in the past. The material storage area at Eastford Road, associated with the Novelis Spur, is entirely in Flood Zone 3 adjacent to the River Mersey. In the event of a flood event on the River Weaver or the River Mersey respectively, there is the risk that the material storage areas become inundated. The construction materials take up volume that was previously available to store flood waters, therefore potentially raising the overall flood level. Pipe sections could be lifted and moved by the floodwater leading to damage to property or infrastructure. Materials could also be swept away and lead to blockages in watercourse crossings. Further flood risk assessment and design development work is planned to reduce the risk as much as possible for the final FRA supporting the DCO application.

9.2.2 Although the risks would largely be mitigated through the embedded environmental measures (**Table 7F 7.1**):

- C-1: all construction elements to follow a CEMP
- C-2: developing of Drainage and Water Management Plan to support CEMP to address the surface water run-off,
- C-3: Timing of works in known areas of flood risk to heed any flood warnings. Watching brief of flood warnings for areas benefitting from defences,
- C-6: Temporary provision of surface water drainage for construction areas,
- C-22: Sequential approach to positioning of compound infrastructure to avoid areas of high flood risk and conveyance routes and
- C-24: measures to minimise raised structures in floodplains, and mitigate impacts where unavoidable through appropriate alignment, leaving gaps and cross-drainage

9.2.3 Residual risk remains and it is recommended to move the Acton Bridge and Eastford Road material storage areas out of the areas of flood risk.

9.2.4 Overall, the permanent operational infrastructure associated with the Project (pipeline, HAGIs, BVIs, DNO connections and pigging areas) are not considered to be at risk of significant flooding. Hydraulic modelling has demonstrated that those sites where a potential tidal or fluvial flood risk was identified from an initial screening of FMfP are safe from flooding over the lifetime of the proposed development, provided current site ground levels are maintained. Appropriate site-specific drainage measures, as set out in the Drainage Strategy in **Annex 7F6**, will be sufficient to address any localised surface water or groundwater emergence flood risk, as well as ensuring that the operational infrastructure would not increase flood risk to third party receptors. Additionally, with appropriate embedded environmental measures to manage groundwater flowpaths along the pipeline route, the operational pipeline would not increase risk of flooding to third party receptors.

9.2.5 The flood mitigation measures detailed in **Section 7** are deemed to be sufficient in mitigating the risks associated with the Project, during construction and operation, with the exceptions detailed above, for which further flood risk assessment and design development work is required. This work will be completed within the final FRA submitted with the DCO application.

9.2.6 Decommissioning has not been specifically considered at this stage, but associated works are likely similar to those during construction, but of a lesser magnitude and spatial extent, given that much of the subsurface infrastructure would be left in-situ.

HyNet North West Hydrogen Pipeline

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Annex 7F1
Mersey Estuary Model User Report

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Technical note:
Mersey Estuary - Model User Report

1. Introduction

1.1 Context

This model user report accompanies the Hynet Flood Risk Assessment. This report documents the modelling undertaken using the existing Environment Agency (EA) owned model for the Mersey Estuary to provide updated tidal flood extents and levels incorporating the latest Coastal Flood Boundary (CFB) dataset[1] and EA sea level rise allowances (2021) for the North West[2]. The updated flood model results are required for the assessment of tidal flood risk for the Stanlow, Higher Walton and Rocksavage Hydrogen Above Ground Infrastructure (HAGI) infrastructure elements of the wider project.

When the original model was developed, two climate change events were modelled for the 0.5% AEP event considering 2065 and 2115 future epochs (50-year and 100-year), presumably based on applicable guidance at the time of the study.

Updated climate change allowances have been considered and run based on the latest guidance within the online Planning Practice Guidance (PPG)[3]. As the project and associated infrastructure is considered to be a Nationally Significant Infrastructure Project (NSIP), an 'Upper End' climate change allowance has been modelled in addition to the 'Higher Central' allowance. The H++ has also been considered as a sensitivity test, representing a credible maximum scenario. A future epoch of 2100 has been considered for the sea level rise allowance, assuming a 75-year design life.

The model software has been updated from the original study, and hence the model has also been re-run for the 0.5% AEP event to provide a like-for-like comparison to understand any changes to modelled flood risk as a result of software version.

1.2 Purpose and methodology

The purpose and methodology steps of this modelling study are outlined below:

- Review existing EA model to satisfy that it is fit for purpose to assess flood risk to the site;
- Run existing EA model of the Mersey Estuary to provide flood extents and levels for the 0.5% AEP design event;
- Generate new ISIS Event Data (.IED) files for the 0.5% AEP 2100 Higher Central, 0.5% AEP 2100 Upper End and 0.5% AEP 2100 H++ climate change allowance events;

[1] https://www.data.gov.uk/dataset/73834283-7dc4-488a-9583-a920072d9a9d/coastal-design-sea-levels-coastal-flood-boundary-extreme-sea-levels-2018 (Accessed February 2023)
[2] https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances#sea-level-allowances (Accessed February 2023)
[3] https://www.gov.uk/guidance/flood-risk-and-coastal-change (accessed February 2023)

- Generate new ISIS Event Files (.IEF), to output results to appropriate location on the local system, reading in the new IED file described above and existing model elements (DAT file and TCF);
- Run model using updated IEFs above, and;
- Process results files for the area of interest.

1.3 Data – existing model

A data request was made to the EA in August 2022 to provide the most up to date and fit for purpose hydraulic model (and accompanying reports and results) of the Mersey Estuary. The linked 1D-2D model provided by the EA was developed in Flood Modeller (previously ISIS) and TUFLOW software in 2015 by JBA Consulting[4], and the final report was issued in 2018. The model provides the best and most up to date indication of tidal flood risk within the Mersey Estuary.

Regarding the Stanlow HAGI, the EA advised consideration of the separate River Gowy model as the preferred model for assessing tidal flood risk on the lower reaches of the River Gowy given that this model has a finer grid resolution and provides a better representation of the channel and hydraulic structures. However, the Stanlow HAGI site is situated outside of the River Gowy model domain), whereas the Mersey Estuary model includes both the River Gowy and the Stanlow HAGI domain (see Figure 4.3). The Gowy model uses outputs from the Mersey Estuary model to provide tidal boundary conditions. The Mersey Estuary Model was therefore chosen for the assessment of tidal flood risk to the Stanlow, Higher Walton and Rocksavage HAGIs, with the option of using the River Gowy model for a refined assessment of flood risk to the Stanlow HAGI, should it be needed. Further discussion is provided in Section 4.3.

The supplied files and data used in this assessment are summarised in Table 1.1 below.

Table 1.1 Supplied data

Item	Description
Product 4	Hydraulic modelling report and accompanying Hydrology report describing the project background and model updates incorporated. *2015s3059 - Mersey Coastal Final Report.pdf* *2015s3059 - Mersey Coastal ISIS-TUFLOW modelling checkfile v11.pdf*
Product 5	Accompanying model results in GIS and .csv format for the Mersey Estuary, Ditton Brook and River Gowy.
Product 6	Hydraulic model files for the ISIS (now FloodModeller) and TUFLOW model elements.

For the purposes of these climate change model runs, no additional data sources have been used other than the model data files supplied by the EA. The model has been reviewed, as detailed in Section 2, to determine the model's fitness for purpose.

[4] JBA Consulting (2018). Mersey Estuary, Ditton Brook and River Gowy. Final Report.

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2. Model review

The following sub-sections review the key elements of the existing model to satisfy that the model is fit for purpose to assess flood risk to the site.

2.1 Model software

The original JBA modelling study was carried out initially using ISIS (now FloodModeller) version 3.7 and TUFLOW version 2013-12-AD-iSP-w64.

The updated model runs for this assessment have been run using FloodModeller version 6.0 (released in 2022) and TUFLOW version 2020-10-AF-iSP-w64 (released in 2022). The sensitivity of the model results to the updated model software versions has been assessed and detailed in Section 4.1.

2.2 Topography

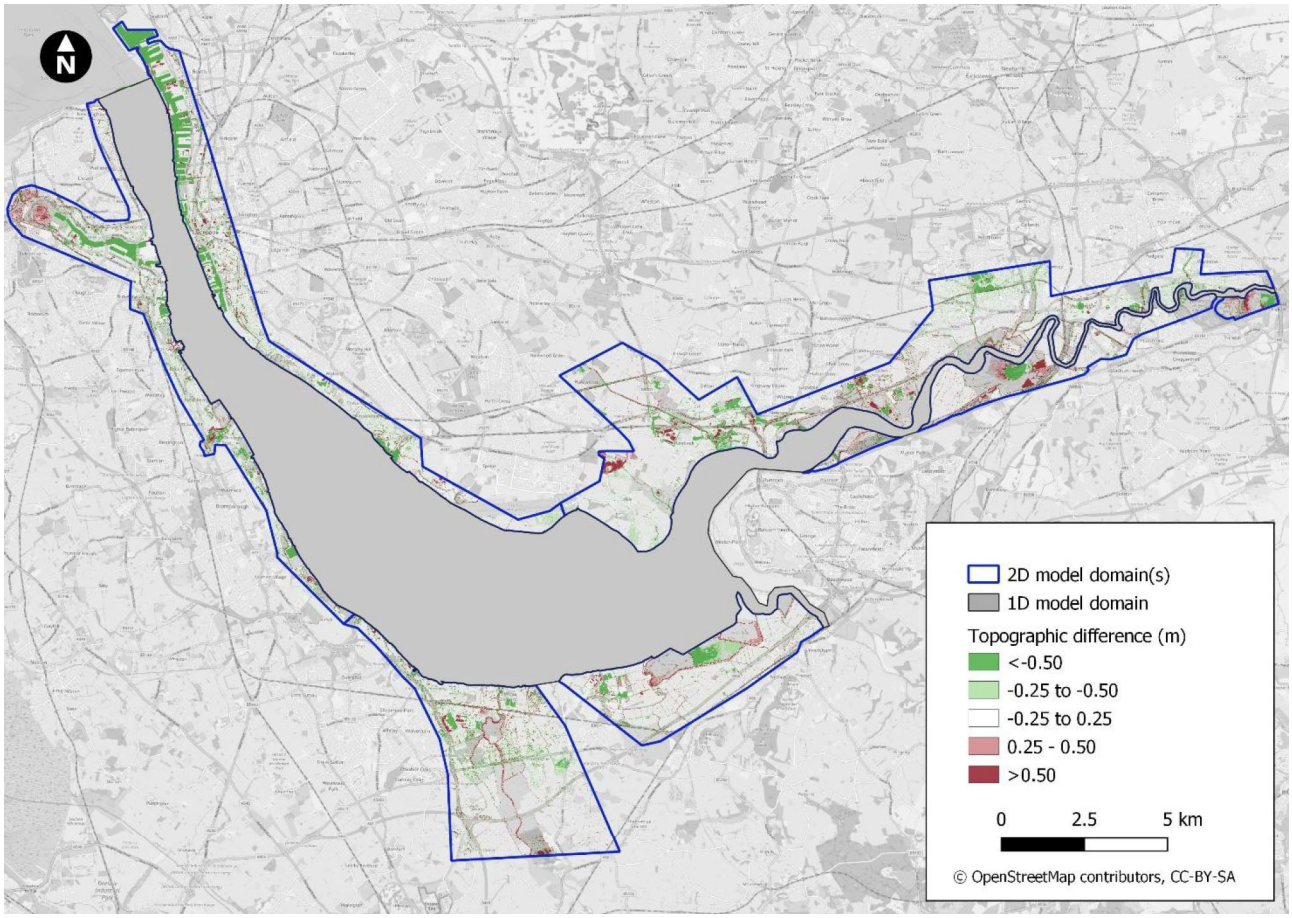
LiDAR

The current best available LiDAR data from the EA for the area of interest has been assessed against the model 'DEM_Z' check file to understand whether there have been any recent developments (that are not captured in the existing model) that may have a bearing on flood risk. The DEM_Z check file represents the final model elevation used and has been used in preference to the input LiDAR data since it accounts for any topographic modifications applied (to represent developments for instance). The LiDAR used in the model is understood to have been flown in 2013.

A raster comparison between these two datasets has been undertaken and shown in Figure 2.1. The comparison indicates that differences are typically minimal (< 0.25m) across the majority of the TUFLOW 2D model domains. Significant differences are noted within the 1D model domain, however, this is to be expected given the model DEM_Z file represents the 1D channel bathymetry whereas the DTM only represents the water surface level. In addition, the tidal channel here is likely subject to significant erosion and deposition, resulting in frequent geomorphological change. For this reason, the 1D model domain has been excluded from the figure.

Most notable topographic changes in the 2D model domain identified in the intervening period relate to developments at Ince Park off Marsh Lane, new developments in the northern portion of the Stanlow Oil Refinery and landscape changes at Arpley landfill site southwest of Warrington. However, none of the changes are deemed to have a significant bearing on flood risk to the area of interest for this study, given their limited scale and magnitude relative to the tidal flood events of concern. Therefore, the existing model topography is deemed to be appropriate for the assessment of flood risk for this study.

Figure 2.1 Topographic comparison - LiDAR



Topographic survey

Topographic drone survey has been undertaken in 2022 as part of the project and is available for the Higher Walton HAGI site, which sits on the southern boundary of the 2D model extent adjacent to Acton Grange Viaduct. The survey data has been processed into a digital elevation model (DEM) for comparison against the model topography DEM_Z, as described above. The recent topographic survey data reveals that since the model LiDAR was flown in 2013, the HAGI site has been reprofiled and 'levelled', to a fairly uniform platform sloping from 11.8mAOD in the west to 9.3mAOD in the east.

For the purposes of this assessment, incorporation of the topographic survey into the 2D model topography has not been deemed necessary. As shown in Figure 2.2, the 0.5% AEP 2100 H++ scenario flood extent is shown to intersect the HAGI layout based on the 2013 LiDAR data, however, the recent topographic survey reveals that the new platform level would remain well above (>1.5m) the peak flood level of this scenario event.

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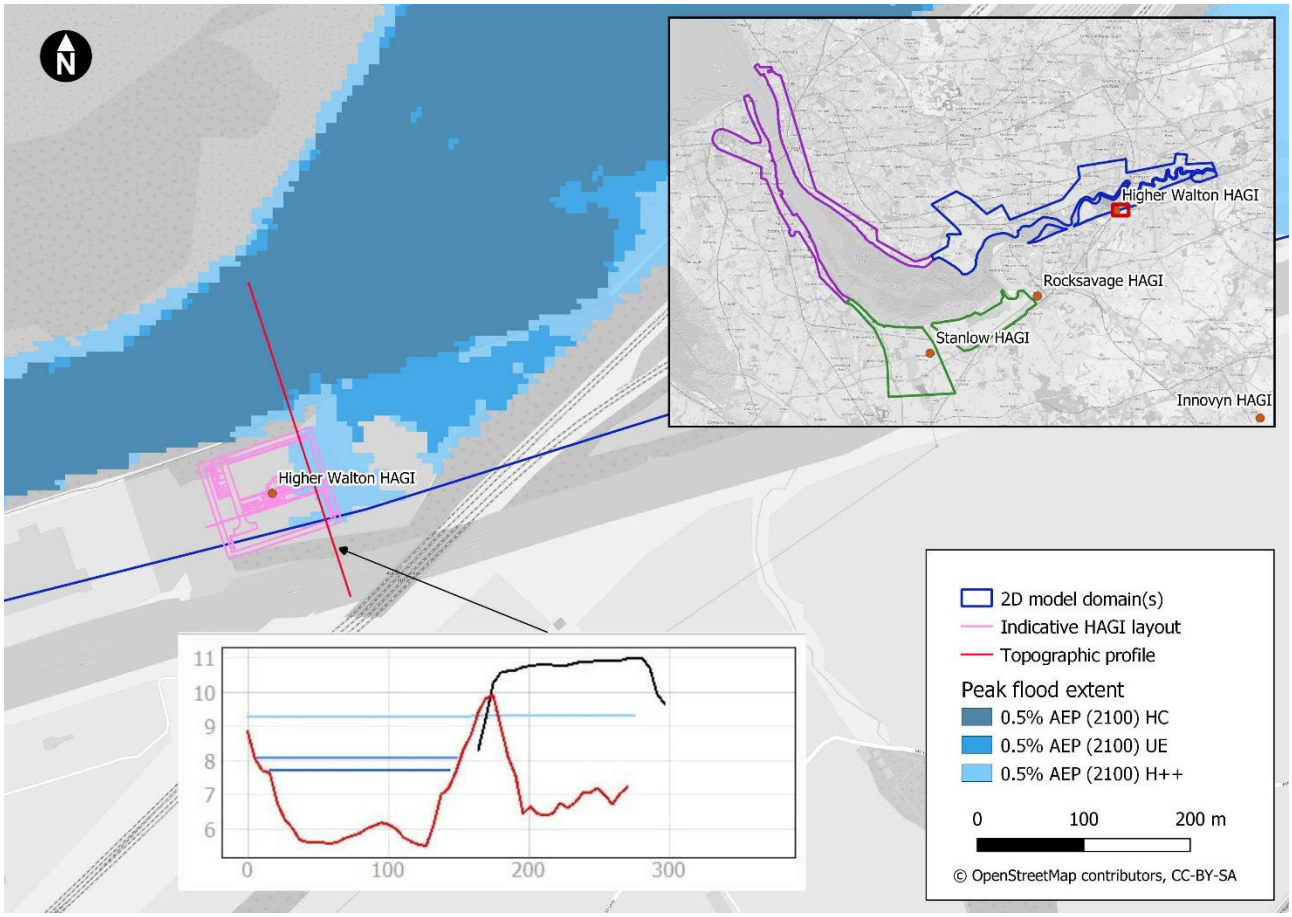
Figure 2.2 Topographic comparison – survey[1]



[1]The topographic profile shows the model 'DEM_Z' topography (based on 2013 LiDAR) in red, the topographic survey data in black, and peak water levels for the climate change event modelled.

2.3 Model schematisation

A high-level review of the model schematisation has been undertaken to satisfy that the model representation is appropriate for assessment of flood risk for the purposes of this study. There are various bridges not explicitly represented within the 1D model such as the Mersey Gateway, however, this is reasonable given the scale of the structures and minimal impediment to flows. No issues were identified with the current model schematisation which would have a significant bearing on flood risk within the area of interest of this study.

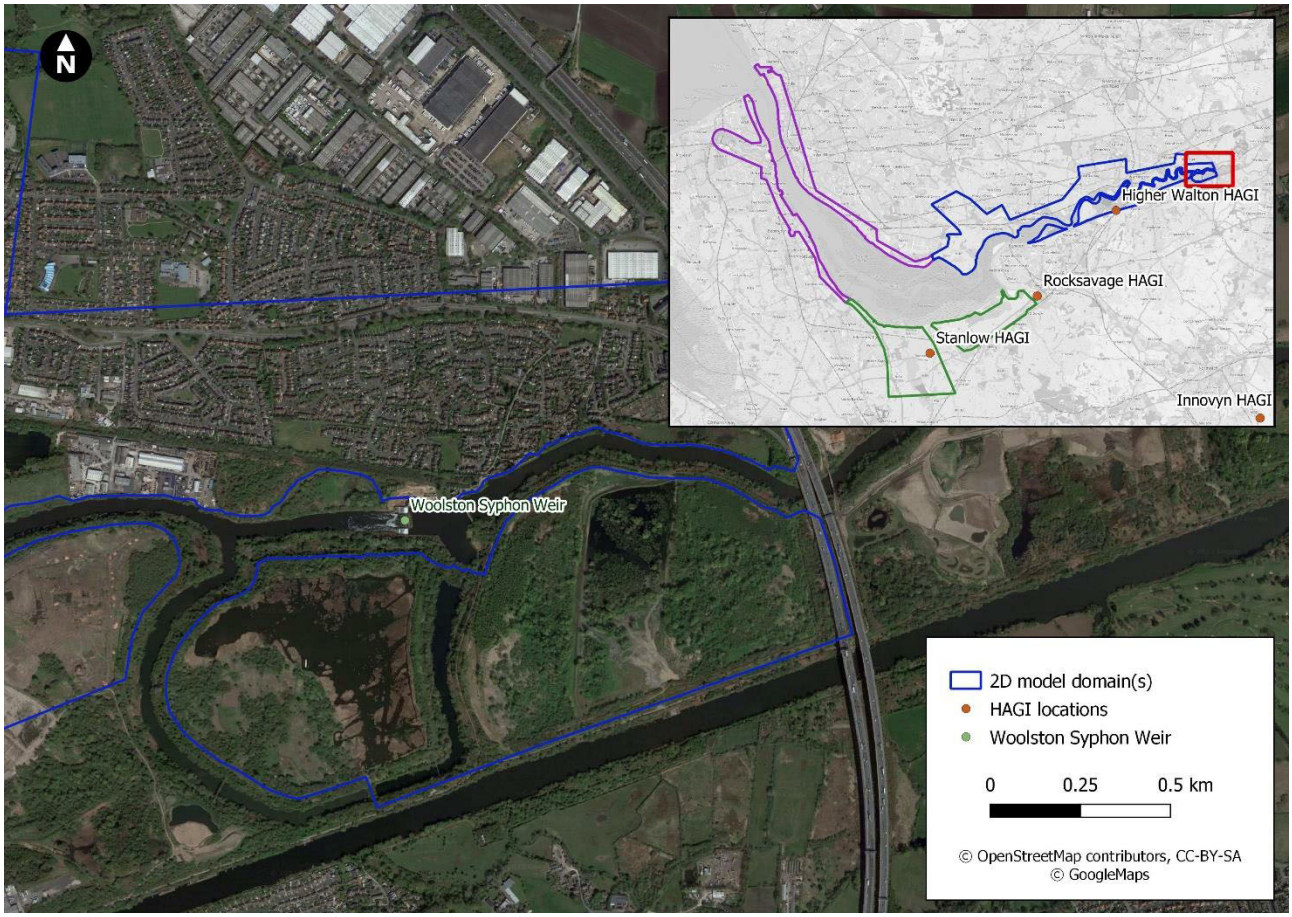
3. Model development

This section describes the new model files developed for the purposes of the updated climate change run. For a full overview of the existing EA model refer to the original model report in Appendix A.

3.1 Flood modeller

For the 0.5% AEP 2100 Higher Central and Upper End scenarios, no updates have been made to the 1D elements of the model. However, the 0.5% AEP 2100 H++ scenario initially failed when run with the existing Flood Modeller 1D network as a result of reverse flow being encountered at the Woolston Weir syphon unit ('WOOLSsyphCu'). The weir is situated on the River Mersey at Woolston, approximately 12km upstream of the Higher Walton HAGI and represented by two weir units and a syphon unit in the original model. The weir is shown in Figure 3.1 below.

Figure 3.1 Woolston Syphon Weir



Flow through the syphon unit makes up only a minor proportion of the total flow at this point, and the structure is a sufficient distance away from the HAGI sites of interest in this study to have no impact on local flood risk. Therefore, for the purposes of the H++ sensitivity test, the syphon unit has been removed from the model network to overcome the reverse flow model failure encountered in this scenario. Review of model results suggests that flow through the syphon unit in the original model is accommodated by the two adjacent weir units, with negligible impact to downstream flow or flood levels.

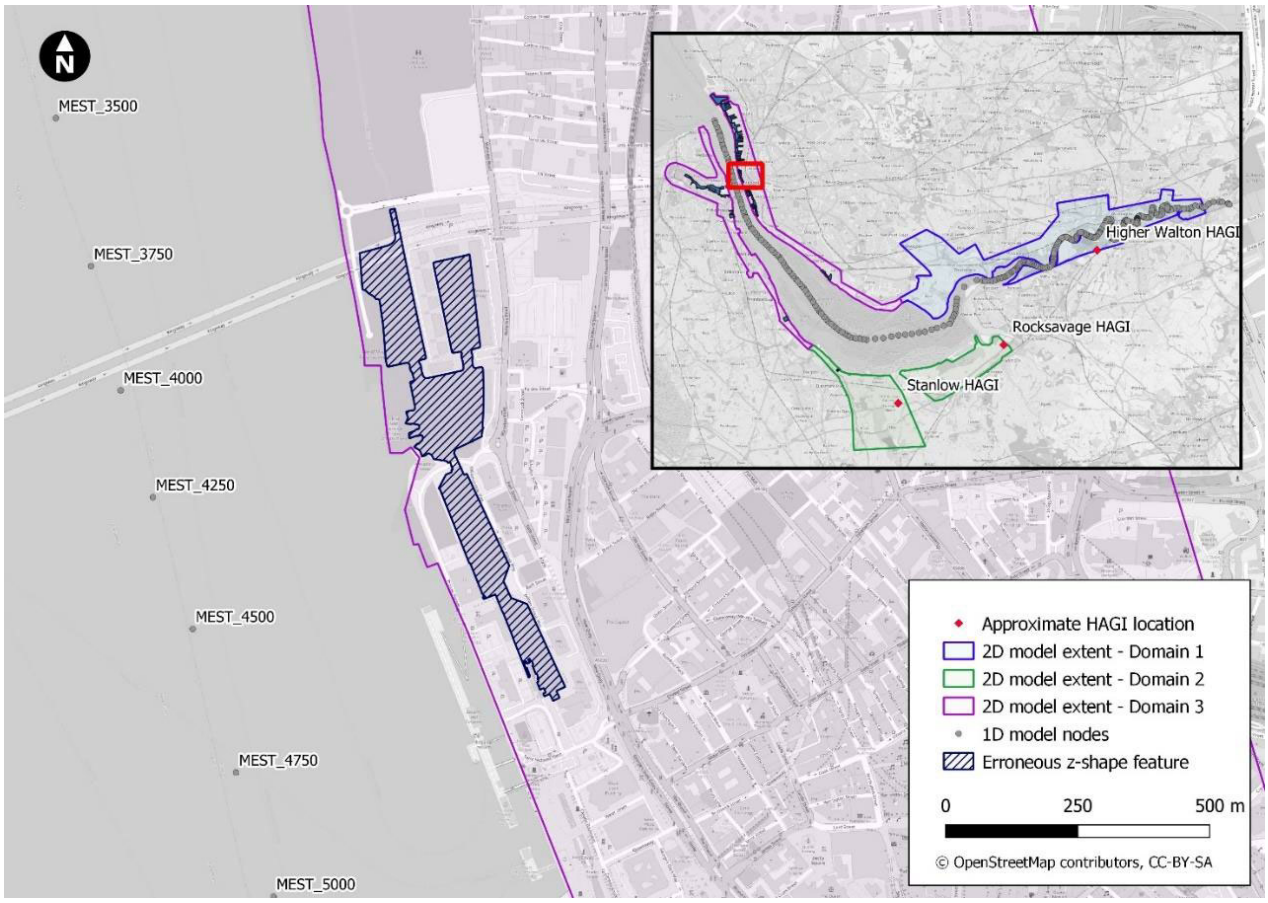
Refer to Appendix A for more information regarding the 1D model including model schematic, details of roughness values, structures, and representation of defences.

3.2 TUFLOW

Running the existing model as supplied without modification resulted in TUFLOW error message 2232[5] (Failed to complete triangulation). Troubleshooting the model pointed to an issue with a feature in the z-shape layer *'2d_zsh_docks_Mersey_001.shp',* being used to define initial water levels within the adjacent docks along the Mersey Estuary.

The feature in question is shown in Figure 3.2, and is believed to have had an invalid geometry type (ie. self-intersections, duplicate vertices etc). In order to resolve this, a 'simplify' GIS processing tool was used to reduce the number of vertices within the feature using a tolerance of 0.1m. The resultant 'simplified' feature therefore diverges from the original feature by a maximum of 0.1m. Considering the underlying 2D cell size of 10m, this is thought to result in negligible difference to the resulting model topography (defined in the 'DEM_Z' check file) and subsequent model results. In addition, the feature is located well away from the area(s) of interest for this study.

Figure 3.2 Erroneous z-shape feature



No further model updates have been made to the model input files. An additional command has however been added to the geometry control files ('Zero Z Point == Warning') to negate TUFLOW error message 2049, to allow an absolute value of zero (metres above ordnance datum, mAOD) to be set within the topography.

It is believed that updates to the TUFLOW software versions and updates to how error messages are handled within TUFLOW since the time of the previous study are responsible for the initial error

[5] https://wiki.tuflow.com/index.php?title=TUFLOW_Message_2232#:~:text=TUFLOW%20was%20unable%20to%20triangulate,image%20for%20a%20larger%20view. (Accessed March 2023)

messages encountered here, rather than any fundamental issues with the existing model configuration.

3.3 Boundary conditions

New Flood Modeller Event Data (.IED) files have been generated to provide the updated 0.5% AEP 2100 Higher Central and Upper End tidal boundaries, for the head-time (HTBDY) boundary unit 'MEST_0000'. The steps used to produce the new tidal boundary conditions are described in the sub-section below. The fluvial inflows have been left unchanged from the original model. It is noted that they are identical for the present day and the climate change runs in the original model.

The model also includes wave overtopping inflows primarily applied into domain 3 of the TUFLOW 2D model (see Figure 3.2), in addition to 1D flow-time boundaries (QTBDY) into the Manchester Ship Canal. The wave overtopping inflows applied in the original model remain unchanged for these updated model runs. No amendments have been made to the overtopping inflows associated with the 0.5% AEP plus climate change events to reflect the revised tidal boundaries modelled, given that any differences are likely to be minor and have no bearing on flood risk to the area of interest for this study in domains 1 and 2.

Tidal boundaries

Revised tidal boundaries have been produced for 0.5% AEP 2100 Higher Central, Upper End and H++ climate change events, in accordance with the CFB user guide[6]. The peak tidal levels for these events are calculated from the updated CFB (2018) extreme sea level of 6.42mAOD plus an uplift as detailed in Table 3.2 below.

The tidal boundary for the 0.5% AEP present day event in the original EA model peaks at 6.127mAOD and is assumed to represent the year 2015, when the EA modelling was undertaken. This tidal boundary was applied for the 0.5% AEP present day event of the current model. Although technically it would have been more appropriate to apply a tidal boundary based on the updated CFB (2018) extreme sea level of 6.42mAOD, the model results show that the HAGI sites remain dry even in the 0.5% AEP events with the climate change allowances, showing that the underestimation of the 0.5% AEP 'present day' peak tide level is inconsequential to the assessment of tidal flood risk at the HAGI sites.

The below sub-sections summarise the various steps and data inputs in producing the boundaries.

CFB data

The CFB 2018 data[1] has been used to provide the extreme sea levels and surge shape applicable for the model. The closest CFB data point to the model downstream boundary is '1168' (328962, 397107), which was also used in the original JBA modelling study. The CFB data set defines an estuary 'cut off' line for the Mersey estuary, upstream of which the CFB 2018 data is not directly applicable. This cut off line approximately coincides with the downstream boundary of the Mersey Estuary model, meaning that the CFB data is suitable for the Mersey Estuary model downstream boundary. The accompanying guidelines indicate to use Surge Shape 29 from donor site Liverpool for this section of coastline.

Sea level rise projections

Climate change sea level rise projections are defined by the EA per river basin district[2], and defined for 'higher central' (70th percentile) and 'upper end' (95th percentile) climate change allowances. The sea level rise allowances are understood to be based on the UK Climate

[6] Environment Agency (2018). Coastal flood boundary conditions for the UK: update 2018. User guide. SC060064/TR7

Projections (UKCP18) Representative Concentration Pathway (RCP) 8.5 scenario. For the H++ sea level rise allowance, +1.9m is used for the total sea level rise to 2100. Relevant sea level rise projections are shown in Table 3.1.

Table 3.1 Sea level rise projections

		Sea level rise (mm/year)				Total uplift (mm)
River basin district	Allowance	2000 - 2035	2036 - 2065	2066 - 2095	2096 - 2125	
North West	Higher Central	4.5	7.3	10	11.2	628
North West	Upper End	5.7	9.9	14.2	16.3	867
UK wide	H++					1900

The CFB 0.5% AEP extreme sea level has been uplifted based on the EA sea level rise allowances, to produce future 2100 extreme sea levels for the Higher Central, Upper End and H++ scenarios. Resultant levels are shown in Table 3.2, with comparison to the original 0.5% AEP 2065 and 2115 levels modelled as part of the original study.

Table 3.2 Uplifted extreme sea levels

	Extreme sea levels (mAOD Newlyn)				
	Existing boundaries (JBA)		Updated boundaries		
AEP	**2065**	**2115**	**2100 (HC)**	**2100 (UE)**	**2100 (H++)**
0.5%	6.4	6.8	7.1	7.3	8.3

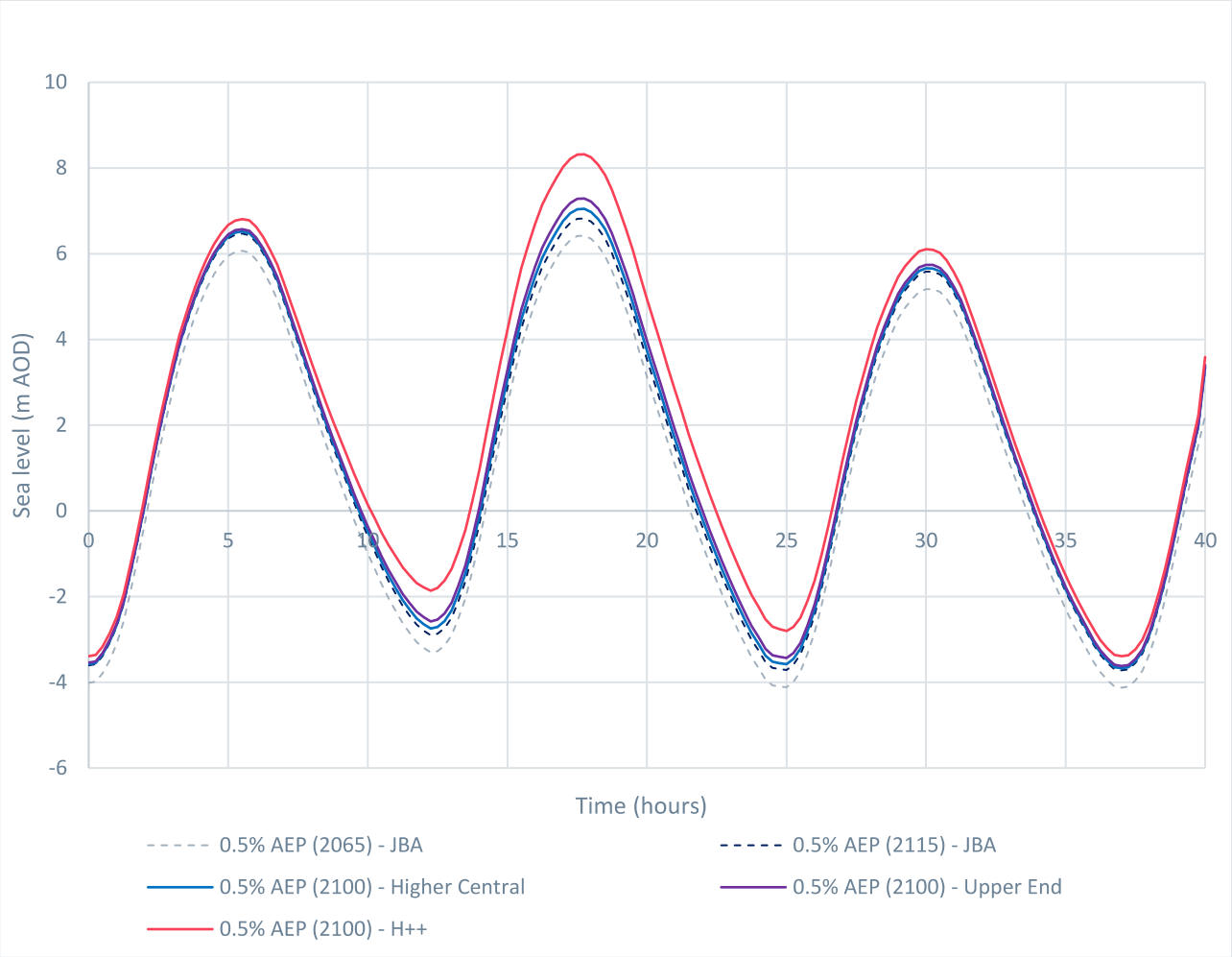
Base tidal curve

In the absence of the base tidal curve used to derive the tidal boundaries in the original modelling study, the existing 0.5% AEP 2115 tidal boundary has been used to provide the base tidal curve. This has been uplifted in accordance with the CFB 2018 surge shape (with the peak surge coincident with the high tide peak) and to match the extreme sea levels in Table 3.2. Using the existing 0.5% AEP 2115 tide data as the base curve ensures consistency with the tidal curve shape used in the original study.

Results

Final tidal boundaries for the 0.5% AEP 2100 Higher Central, Upper End and H++ events are shown in Figure 3.3, with comparison to the existing boundaries.

Figure 3.3 Final tidal boundaries



1d-2d linking

No changes were made to the links between the 1D and the 2D parts of the Mersey Estuary model. It is noted that the Manchester Ship Canal is represented as a 1D channel and not linked to the 2D domain for the 2km lengths upstream or downstream of the Higher Walton site.

In the H++ event the flood extent of the tidal River Mersey extends to the Manchester Ship Canal. As there is no link into the 1D domain of the Canal, glass walling occurs, including adjacent to the site. However, the water level gradient points away from the site in both cases. This means that if there were a link between the Manchester Ship Canal and the floodplain, water is likely to flow away from the site resulting in a lower water level than currently shown in the H++ scenario.

Glass walling occurs elsewhere in the 2D domains during the H++ event, but the locations are far enough away from the HAGI sites and therefore unlikely to affect flood levels near the site.

3.4 Model proving

As there has been no change in the fundamentals of the model geometry or hydrology, just a simple uplift to model climate change, no additional calibration or sensitivity analysis has been carried out on the model. Refer to Appendix A for a discussion on the calibration and model proving undertaken on the EA Mersey Estuary model.

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4. Model runs

4.1 Sensitivity testing

The existing 0.5% AEP event has also been re-run using the updated TUFLOW version 2020-10-AF-iSP-w64 and Flood Modeller version 6.0 to understand the impact of the updated software version on modelled flood results at the site. The depth difference is shown in Figure 4.1, whereby the updated version results have been assessed relative to the original version used.

However, the value of the comparison is limited, as it was noted that there is a discrepancy in the result output resolution between the EA supplied model results (2m) and the outputs from the supplied model (5m, i.e. 10m TUFLOW cell size). This limits the value of the results comparison given that the model version is not the only variable being assessed. It is not known whether the supplied results have been post-processed to 2m or whether those results were produced using a different model configuration.

The updated peak flood extent is broadly comparable to the original model results. However, there are notable differences in the peak flood depths. In the estuary and up the river channel to a point some 3.5km south-west of the Higher Walton HAGI site, at NGR SJ 55405 84560, they are typically 0.10m lower than in the model results provided by the EA. The difference rises to 0.30m in the floodplain closest to the site. Some isolated areas away from the main channel experience between 1.00 – 2.35m lower water depths than in the model results provided by the EA. The largest difference of 2.35m occurs at a water body in Moore Nature Reserve, adjacent to the Higher Walton HAGI (Figure 4.2). It is not clear if these very large differences are caused by the differing model resolution, funnelling effects or something else.

For the purposes of this assessment, it has been assumed that the model supplied by the EA (and with a cell size of 10m) is configured correctly.

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Figure 4.1 Software version sensitivity, with HAGI locations shown in pink

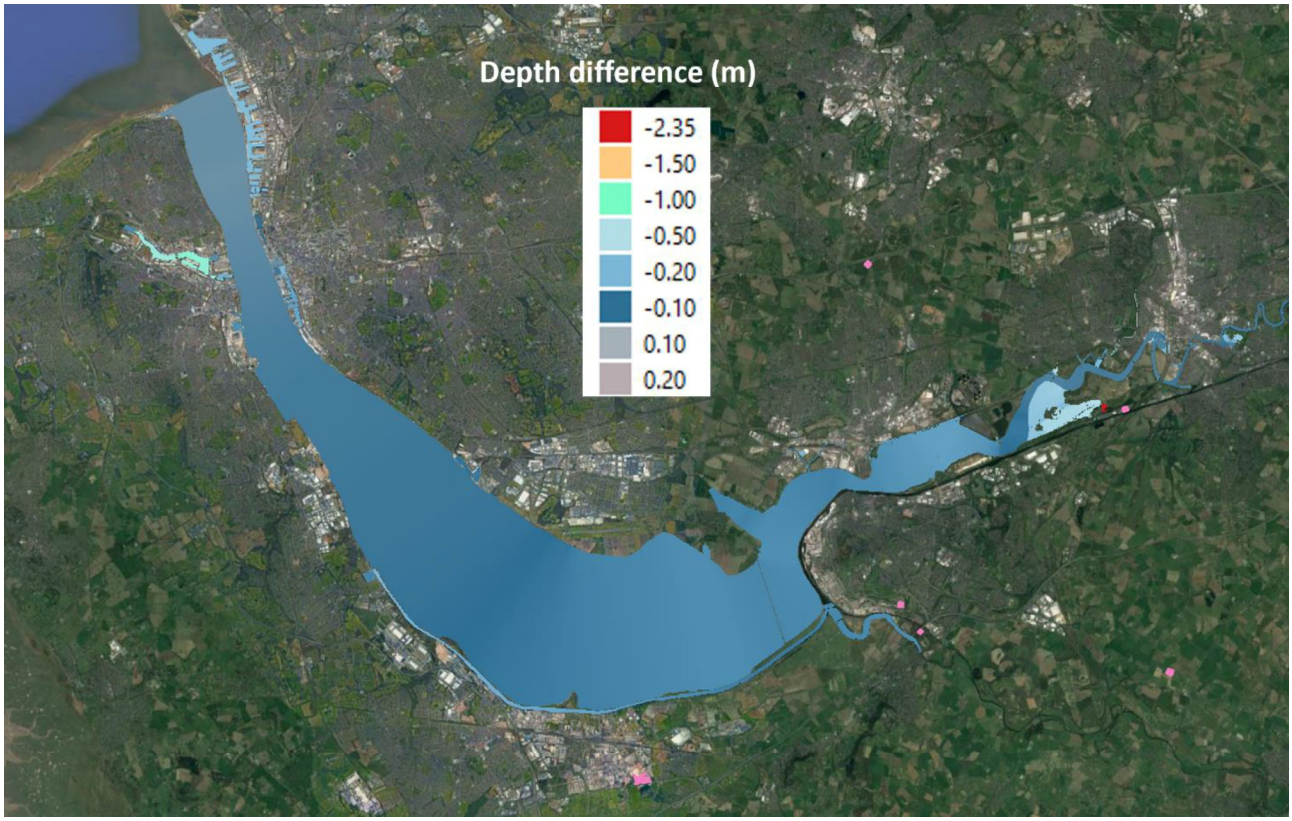
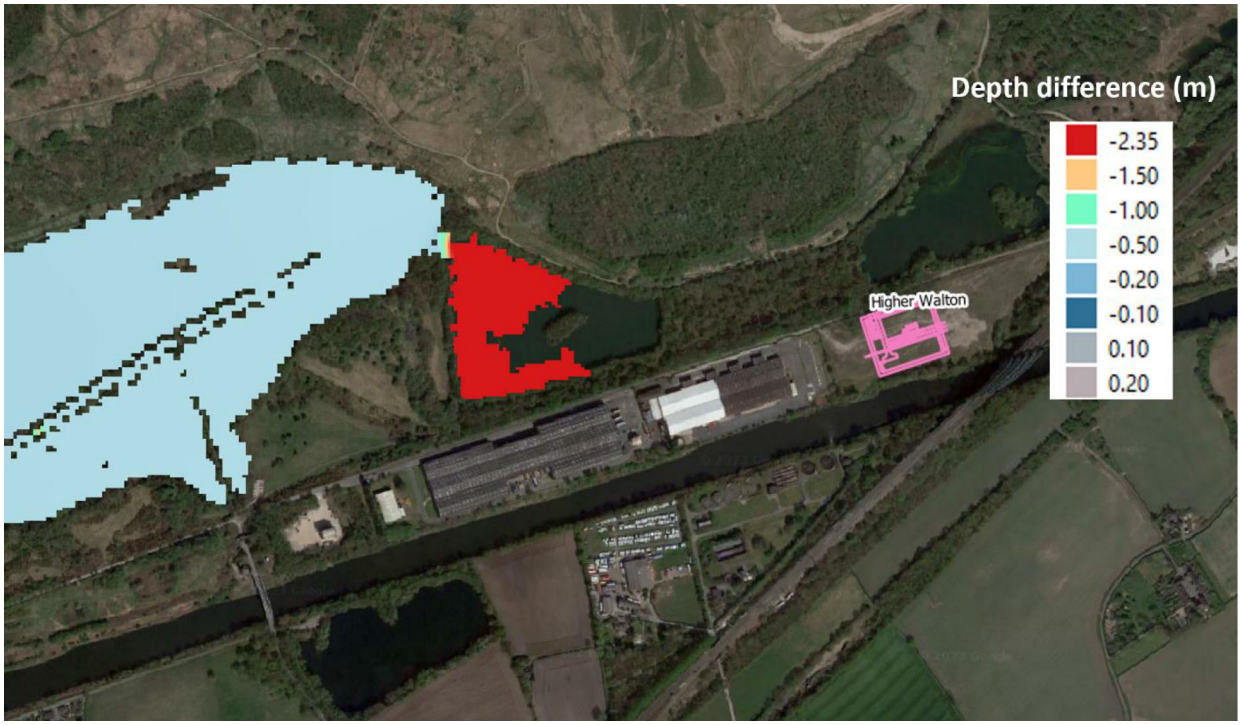


Figure 4.2 Software version sensitivity – Higher Walton HAGI



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4.2 Design run

Model software

The updated model design runs for the two climate change runs have been run using Flood Modeller version 6.0 and TUFLOW version 2020-10-AD-iSP-w64.

Run parameters

Three design model runs have been undertaken using the EA model, as outlined in Table 4.1.

Table 4.1 Model run record and parameters

Item	Comments
Design Events (AEP)	0.5% (2015 – 'present day'), 0.5% 2100 (Higher Central), 0.5% 2100 (Upper End), 0.5% 2100 (H++)
Scenarios	Defended
Model run parameters and settings	Model run parameters have predominantly (with the exception of the H++ scenario) been left unchanged from the original model runs undertaken in the JBA 2018 study. It is noted that the following parameters diverge from the default parameter values: Dflood: 5 Theta: 0.85 (0.9 used for H++ scenario[1]) Alpha: 0.65 (0.5 used for H++ scenario[1]) Maxitr: 19 Weight: 0.05
Flood Modeller network	0.5% AEP present day, 2100 Higher Central and Upper End scenarios: MerseyEstuary_JBA'15_017.DAT 0.5% AEP 2100 H++ scenario: MerseyEstuary_JBA'15_020_H++.DAT
Flood Modeller event data	Fluvial and tidal: Mersey_Est_200yr.IED Mersey_Est_200yrCC_2100_UE_WSP_01.IED Mersey_Est_200yrCC_2100_HC_WSP_01.IED Mersey_Est_200yrCC_2100_H++_WSP_01.IED Defences: Mersey-MSC_Defences.IED
Flood Modeller event file	MerseyEstuary_018_200yr.ief MerseyEstuary_018_200yrCC_UE_2100_WSP.ief MerseyEstuary_018_200yrCC_HC_2100_WSP.ief MerseyEstuary_020_200yrCC_H++_2100_WSP.ief
TUFLOW control data	MerseyEst_200yr_WSP_v18.tcf MerseyEst_200yrCC_2100_UE_WSP_v18.tcf MerseyEst_200yrCC_2100_HC_WSP_v18.tcf MerseyEst_200yrCC_2100_H++_WSP_v20.tcf

[1] Given the magnitude of the H++ scenario tidal peak, it was necessary to relax the theta and alpha model parameters in order to improve model stability. Given that this has been applied as a sensitivity test, these values are deemed to be acceptable.

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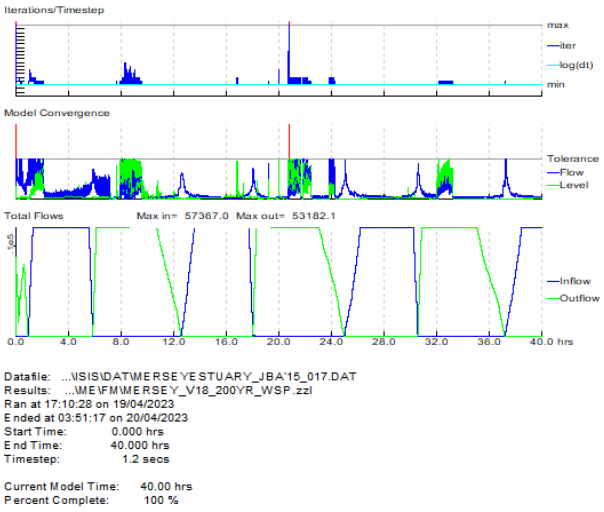
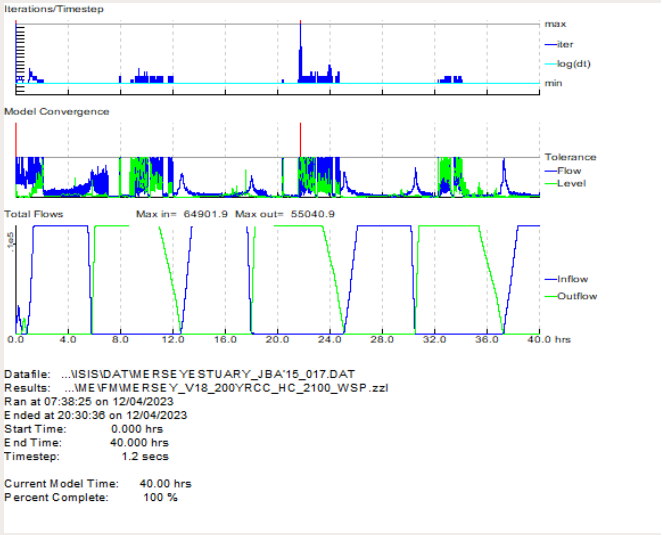
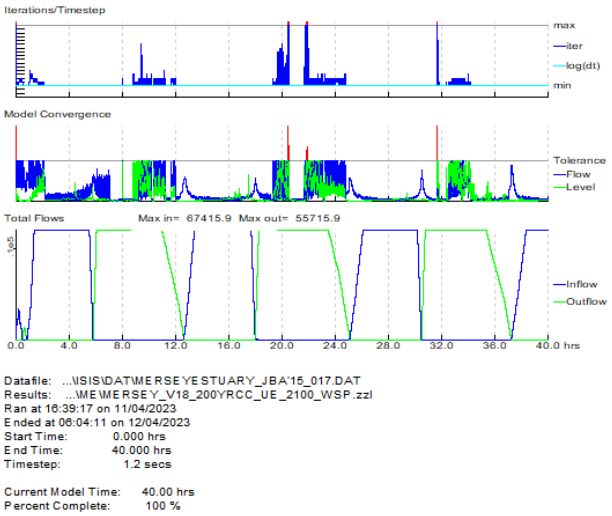
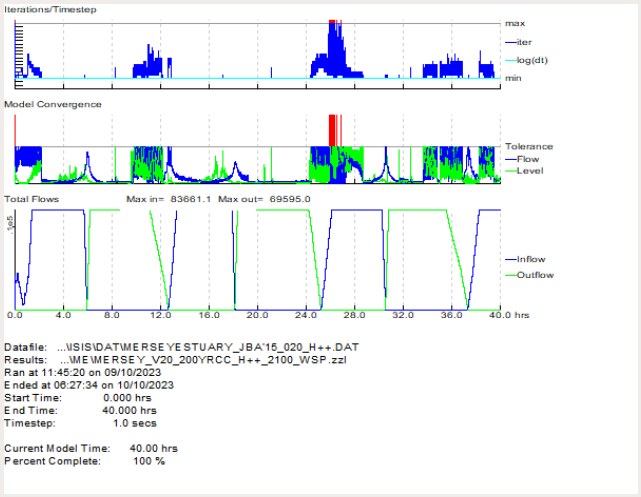
Quality assurance

A summary of the model run performance for the updated runs undertaken is provided in Table 4.2. The mass error in TUFLOW is initially high at the start of the simulation, though reduces to within +/- 1% after the initial wetting period. This is comparable to (and less than) the original model runs of the JBA 2018 study[4]. It is noted that double precision was used for the original model runs but has not been used for the updated climate change runs for the purposes of this assessment. However, this is deemed to be acceptable given the model performance indicators in Table 4.2 and further satisfied by running a sensitivity test using double precision. Resultant peak flood levels between the two simulations (single precision and double precision) were virtually identical (differences < 1mm).

The Flood Modeller model is also within the convergence tolerance. The model performance for these runs is comparable to the existing defended scenario model runs detailed in Appendix A.

Table 4.2 Model run performance

Scenario	AEP	TUFLOW Final Cumulative Mass Error (%)		FloodModeller performance
		Whole simulation	Flow In + Out >5%	
Defended	0.5%	-0.16	-0.13	

Defended	0.5% 2100 (HC)	-0.09	0.17	
Defended	0.5% 2100 (UE)	-0.08	0.19	
Defended	0.5% 2100 (H++)	0.25%	0.45%	

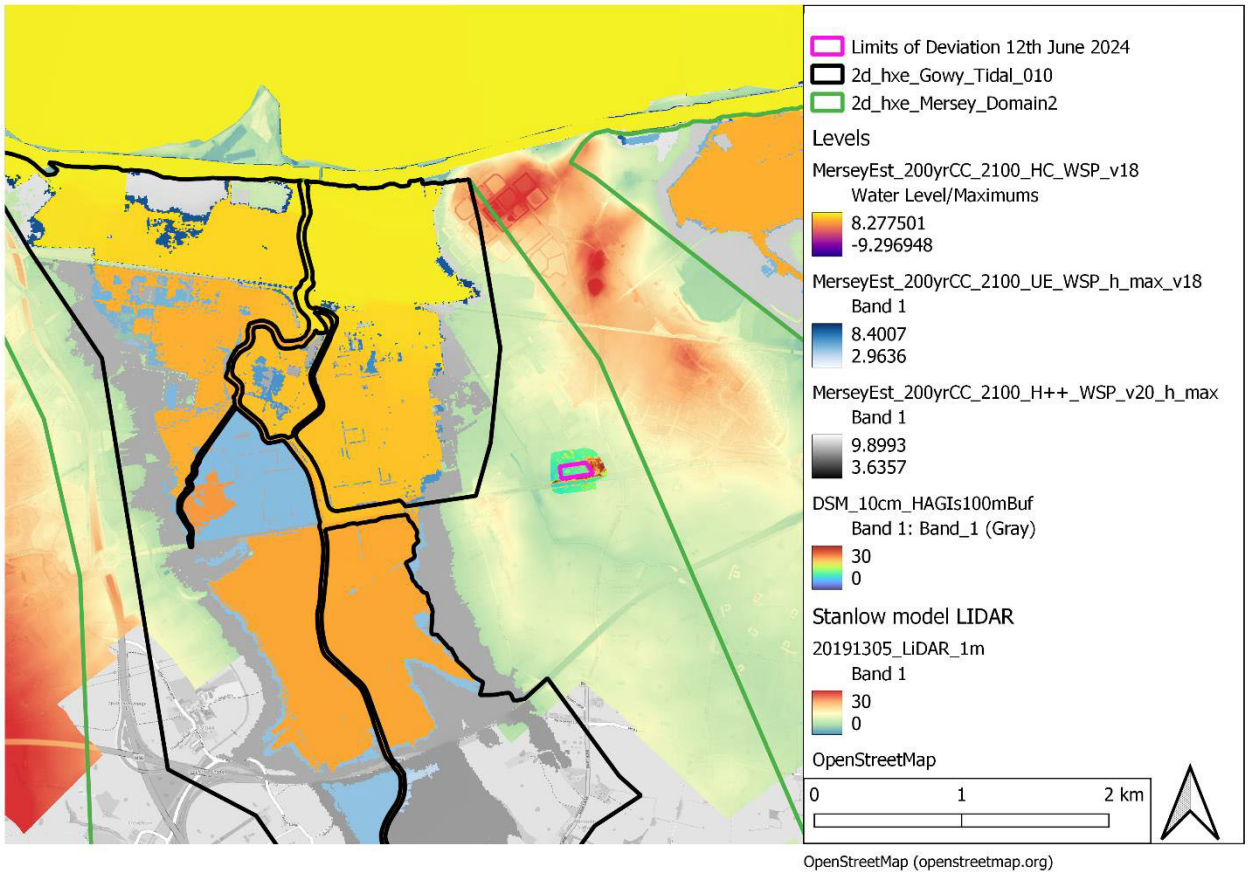
The TUFLOW Simulation File (.tsf) for the 0.5% AEP 2100 Upper End run records a total of 62 checks and 281 warning messages prior to simulation, and 49 warnings and 62 checks recorded during the simulation. Checks primarily relate to coincident points (which TUFLOW ignores) and lowered SX z-points, whilst warnings relate mostly to water level lines not crossing or snapping to

1D channel. Warnings during the simulation all relate to negative depths being encountered. The number of checks and warning messages is comparable to those encountered in the original JBA model runs, and the number of negative depths is notably less.

4.3 Requirement for River Gowy model

The Mersey Estuary model results from the design and sensitivity runs (Section 4.2) in the River Gowy area are displayed in Figure 4.3. The peak design event flood levels closest to the Stanlow HAGI site rise to 6.10mAOD, compared to a minimum site level of 9.43m (based on 1m EA LiDAR (2019) and 0.1m Cadent DSM[7]). The design flood extent is at a distance of 795m from the HAGI site. Even the H++ scenario flood extent does not get closer than 690m, with a peak flood level of 7.85mAOD. Given that the results from the Mersey Estuary model show no tidal flood risk to the Stanlow site from the Mersey Estuary or via the River Gowy, there is no benefit in running the River Gowy model with updated boundary conditions to assess tidal risk to the Stanlow HAGI.

Figure 4.3 Stanlow HAGI in relation to River Gowy model and Mersey Estuary model



[7] Sensat Survey Report (2023) for Cadent: S23314 - HyNet North West Hydrogen Pipeline

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5. Conclusions and recommendations

The existing EA owned hydraulic model for the Mersey Estuary has been deemed fit for the purpose of assessment to support the development of the various HAGI sites within the vicinity of the Mersey Estuary. It has been used to run updated sea level rise scenarios for the Higher Central and Upper End allowances, considering a 2100 future epoch (i.e. 75-year lifetime) and applied to the 0.5% AEP event. The H++ scenario was also modelled. Updated tidal boundaries have been developed in accordance with the latest EA climate change guidance, PPG and CFB dataset. The TUFLOW and Flood Modeller software have been updated.

The climate change scenarios, in particular H++, push the magnitude of flood events this model was originally built for. The model continues to perform well with minor modifications, but glass walling does occur in some locations. Each instance of glass walling was carefully examined with the project sites in mind and found not to impact on the model results. Care should be taken when interrogating model results for other areas.

Resultant model performance for the updated runs is within the acceptable bounds. There are no further recommendations to those made in the original modelling report.

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03	Issue 03 –edits in response to further EA review comments	June 2024

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Annex A
Mersey Estuary, Ditton Brook and River Gowy (JBA 2018)

Mersey Estuary, Ditton Brook and River Gowy

Final Report

December 2018

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Purpose

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Executive Summary

JBA was commissioned by the Environment Agency in 2015 to undertake a package of coastal and fluvial flood modelling, mapping and forecasting. One component of this package specifically concerns the Mersey Estuary and two of its tributaries - Ditton Brook and the River Gowy.

JBA was required to deliver a linked (1D-2D) model of the Mersey Estuary to assess tidal flood risk and update the existing fluvial models of Ditton Brook and the River Gowy. This included updating the channel geometry and developing linked 2D floodplain domains. The results from the updated Mersey Estuary model were used to inform the tidal downstream boundaries for both of the fluvial tributary models. Each model was run for a specified range of AEPs and used to assess the potential implications of future climate change for flood risk.

In addition to the above project requirements, the River Gowy model was used to represent tidal flood risk as well as fluvial flood risk. This was used in preference to the Mersey Estuary model given its finer 2D grid resolution and better representation of the channel and hydraulic structures.

There have been a significant number of previous modelling studies carried out on the Mersey Estuary and its tributaries. This study seeks to update the existing flood risk mapping by both adapting and further developing existing hydraulic models and developing new models where required.

For each of the three study watercourses a linked 1D-2D hydraulic modelling approach was adopted. One-dimensional hydraulic models provide both a computationally efficient and accurate way of representing flow in channels and through structures. However, on the floodplains where flow routes can be complex and multi-directional, 2D models offer significant advantages. Within this study, most main rivers and significant tributaries were represented within the 1D domains and their floodplains were represented using 2D domains.

The following conclusions can be drawn from the flood risk analysis of the Mersey Estuary, Ditton Brook and River Gowy models:

- The Mersey Estuary design runs illustrated the tidal flood risk benefit resulting from the new Warrington FAS. The residential areas behind the two phases of the scheme are modelled to be at risk of tidal flooding of less than 0.1% AEP as a result of the scheme. The undefended scenarios indicate a risk of tidal flooding from an event of 0.5% AEP.
- Comparison of the updated and existing and undefended model outlines for the Mersey Estuary indicates that Warrington and the area of Howley weir marks the boundary between predominantly tidal or fluvial flood risk.
- The Ditton Brook and Mersey Estuary models also highlighted the benefit of the Ditton Brook flood defences. The defences reduced the level of flood risk on the right bank of Ditton Brook (Golden Triangle Industrial Estate) from a 5% AEP to 1% AEP for a tidal flood event and from 0.1% AEP to less than 0.1% AEP for a fluvial event.
- The Ditton Brook hydraulic modelling illustrated the dominance of fluvial and tidal flood risk between the upper and lower reaches, respectively, of the study domain. The lower watercourse was dominated by a tidal signature and extreme fluvial flows had a low impact on flood risk. Nearly all of the modelled extreme flood flows are retained in bank as peak water levels fall across Ditton Junction bridge and Ditton Road bridge. The location where the dominance of fluvial or tidal flood risk changes is the reach between Speke Road and Ditton Junction, which is embanked by high ground. This is consistent across the design and undefended scenarios. Overtopping of the downstream flood wall defences (between Foundry Lane and Stewards Brook) occurs at the <0.1% AEP in the fluvial scenario, but properties are affected on the left bank of Ditton Brook at the 2% tidal scenario and on the right bank at the 1% AEP. The upper watercourse is dominated by fluvial flows and there is a much less prominent tidal signature.
- Much of the River Gowy floodplain was inundated upstream of Stanlow at frequent fluvial flood events (>50% AEP). This regular flooding of fields tied in with anecdotal evidence gathered during the site visit. Little tidal inundation was predicted at higher AEP events due to the significant tidal defences and the impact of the Manchester Ship Canal embankment. Under present day design scenarios, localised flooding occurred downstream of the railway at Stanlow on the right bank of the River Gowy at the 10% AEP and site buildings were affected at the 3.33% AEP. Widespread inundation upstream of the railway line and Folly Gates was observed between the 1% and 0.1% AEP only. The MSC was overtopped from the Mersey Estuary at the 0.1% AEP only.

- Stanlow oil refinery was first flooded from a fluvial event at the 5% fluvial AEP, with water spilling onto the floodplain from Mill Brook. All of the flood risk here was associated with flooding from Mill Brook and inundation was localised and did not affect oil refinery buildings or access roads. At higher magnitude events, up to the 0.5% AEP, flooding in this area was confined only to access roads or undeveloped grassland. It was not until the 0.1% fluvial event that water spills from the right bank of the Gowy onto the site and embankments in the area of Mill Brook around refinery storage units were overtopped.
- In comparison to the design scenario, the River Gowy undefended 5% and 1% fluvial scenarios resulted in greater flooding at Stanlow whilst the extent at the 0.1% was similar.
- The Mersey Estuary and River Gowy tidal modelling highlighted the potential implications of predicated future climate change; in particular, for Stanlow oil refinery. Under conditions of climate change, where inundation at the 0.5% AEP is only localised downstream of the railway and in the region of Folly Gates, the potential disruption to the remainder of Stanlow oil refinery increases dramatically. In the 2065 scenario, most access roads on the right bank of the River Gowy were flooded and in the 2115 scenario, the A5117, the main road that forms the southern boundary of the site, is overtopped.
- The railway downstream of Folly gates and the immediate area between the railway and Folly gates marks the area of transition between tidally and fluvially dominated flood risk. At the undefended 1% AEP, tidal inundation is dominant in eastern areas of the site, whereas in western areas it is fluvial inundation that is dominant.

In addition, it is possible to make some initial recommendations as to how these hydraulic models might be improved or current understanding developed with additional data:

- Confidence in the design flows applied to both the River Gowy and Ditton Brook is reduced because of the lack of high quality flow records. On Ditton Brook, there is currently no river level or flow monitoring; however, with only limited flood risk arising from the watercourse installation of a gauge here is unlikely to be viable. On the River Gowy the situation is very different, especially when the predicated effects of climate change are considered. Whilst there is a flow gauging station on this watercourse it is a rated station which is understood to be heavily influenced by vegetation. Improving the quality of flow gauging at this location would help to improve confidence in the predicted flood risk.
- Consider acquiring topographic/defence height survey of the defence line between the Mersey Estuary and MSC.
- Consider acquiring crest level survey for the new Warrington FAS as built to confirm that the scheme is correctly represented in the Mersey Estuary model.
- Consider acquiring local defence or topographic crest level information for the right bank section between the River Mersey and the railway beneath Warrington Bank Quay station. Although the modelling approach is based on the best available data and a consistent approach has been applied across the study domain, flood risk may be overestimated in this area.
- The MSC and linking structures to the Mersey Estuary were included to improve the representation of overtopping, water storage and water transfer along the impoundment. The operating rules for the Weaver sluice gates were assumed and the Eastham locks is a simple spill linking the MSC to the Mersey Estuary. Further modelling work could benefit from a more detailed understanding of the operating procedures of the Weaver sluice gates and how the dock gates and Eastham Lock gates would realistically behave during an extreme high tide. However, for the purposes of extreme tidal flood modelling, the approach taken here is appropriate and further information would not impact model results.
- Future work could be an opportunity to carry out further study regarding the implications of climate change at more frequent AEPs to test the design limitations of current defences at Stanlow and Warrington.
- Joint probability of tidal and fluvial events has not been considered as part of this study. Future work may consider the implications of this and a concurrent high fluvial flood event and high tide for the Mersey Estuary, Ditton Brook and River Gowy.
- Little is known about the internal dimensions or drainage capacity of the syphon under the Mersey Estuary. This information would be best established by internal survey and inspection by a specialised team.

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Abbreviations

%	percent
1D	One dimensional
2D	Two dimensional
AEP	Annual Exceedance Probability
AIMS	Asset Information Management System
ASCII	American Standard Code for Information Interchange (a text based file format)
CEH	Centre for Ecology and Hydrology
CFBD	Coastal Flood Boundary Dataset
DRN	Detailed River Network
DSM	Digital Surface Model
DTM	Digital Terrain Model
ESTRY	1D modelling package within TUFLOW software
FAS	Flood alleviation scheme
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
FWFRAs	Flood Warning Flood Risk Areas
FZ	Flood Zone
GIS	Geographic Information System
HT	Head-time (hydrological boundary)
ISIS	1D flood modelling software (superseded by Flood Modeller)
JBA	Jeremy Benn Associates Limited
km	kilometres
km^2	kilometres squared
LIDAR	Light Detection and Ranging
m	metres
m^3/sec	metres cubed per second or cumec
mAOD	metres above Ordnance datum
MHWS	Mean High Water Spring tide
MSC	Manchester Ship Canal
OS	Ordnance Survey
QT	Flow-time (hydrological boundary)
ReFH	Revitalised Flood Hydrograph method for flow estimation
TUFLOW	Two dimensional unsteady flow (software)
UK	United Kingdom

1 Introduction

1.1 Package overview

Jeremy Benn Associates (JBA) Limited was commissioned by the Environment Agency in 2015 to undertake a package of coastal and fluvial flood modelling, mapping and forecasting. One component of this package specifically concerns the Mersey Estuary and two of its tributaries - Ditton Brook and the River Gowy.

1.2 Project requirements

JBA was required to deliver a linked (1D-2D) model of the Mersey Estuary to assess tidal flood risk and update the existing fluvial models of Ditton Brook and the River Gowy. This included updating the channel geometry and developing linked 2D floodplain domains. The results from the updated Mersey Estuary model were used to inform the tidal downstream boundaries for both fluvial tributary models. Each model was run for a specified range of annual exceedance probabilities (AEPs) (as outlined in section 7.1) and used to assess the potential implications for flood risk of future climate change.

In addition to the above project requirements, the River Gowy model was used to represent tidal flood risk as well as fluvial flood risk. This was used in preference to the Mersey Estuary model given its finer 2D grid resolution and better representation of the channel and hydraulic structures.

1.3 Study area

The study area is illustrated in Figure 1-1 and the extent of each model is described below.

1.3.1 Mersey Estuary

The Mersey Estuary, downstream of Widnes and Runcorn, is wide (almost four kilometres (km) at the widest extent), deep and artificially embanked in many places by concrete and steel walls or piling. Running parallel to, and just to the south of, the Mersey Estuary is the Manchester Ship Canal (MSC), which diverges from the River Mersey at Lymm and discharges via lock gates into the Estuary at Eastham. At Weston, a short distance downstream of Runcorn, the River Weaver discharges into the Mersey Estuary via the MSC and the River Weaver sluice gates, which open to allow fluvial flood flows into the Mersey Estuary and maintain the statutory water level in the MSC. Upstream of Widnes, the River Mersey narrows considerably and is much more sinuous through Warrington. In contrast to the lower Mersey Estuary, which contains no in-channel structures, this upper reach contains several large bridges, weirs and a siphon.

The urbanised landscape surrounding the lower Mersey Estuary is an area of expansive industrial and residential redevelopment with many of the old docklands and associated buildings undergoing conversion to housing or commercial property.

The upstream extent of the Mersey Estuary model is the point of diversion of the Mersey from the MSC, near Lymm and approximately two kilometres east of Thelwall Viaduct and the outskirts of Warrington. The Mersey Estuary model extends from this point to approximately 50km downstream, through Warrington, Runcorn and Widnes, to the mouth of the Mersey Estuary at Bootle.

1.3.2 Ditton Brook

Ditton Brook is a largely defended tributary of the Mersey Estuary and discharges at Widnes on the Mersey Estuary's right/northern bank. The defences along Ditton Brook consist of flood walls between Ditton and Hale Bank and embankments upstream of Speke Road. Where not formally defended, Ditton Brook is flanked by high ground.

Ditton Brook has a total catchment area of 68km^2 and flows through primarily open grassland with few isolated dwellings before bypassing the industrialised outskirts of Ditton and Halebank. The upstream extent of the Ditton Brook model is Cart Bridge near Halewood Village. The study reach is approximately 5km long and extends to the downstream confluence with the Mersey Estuary.

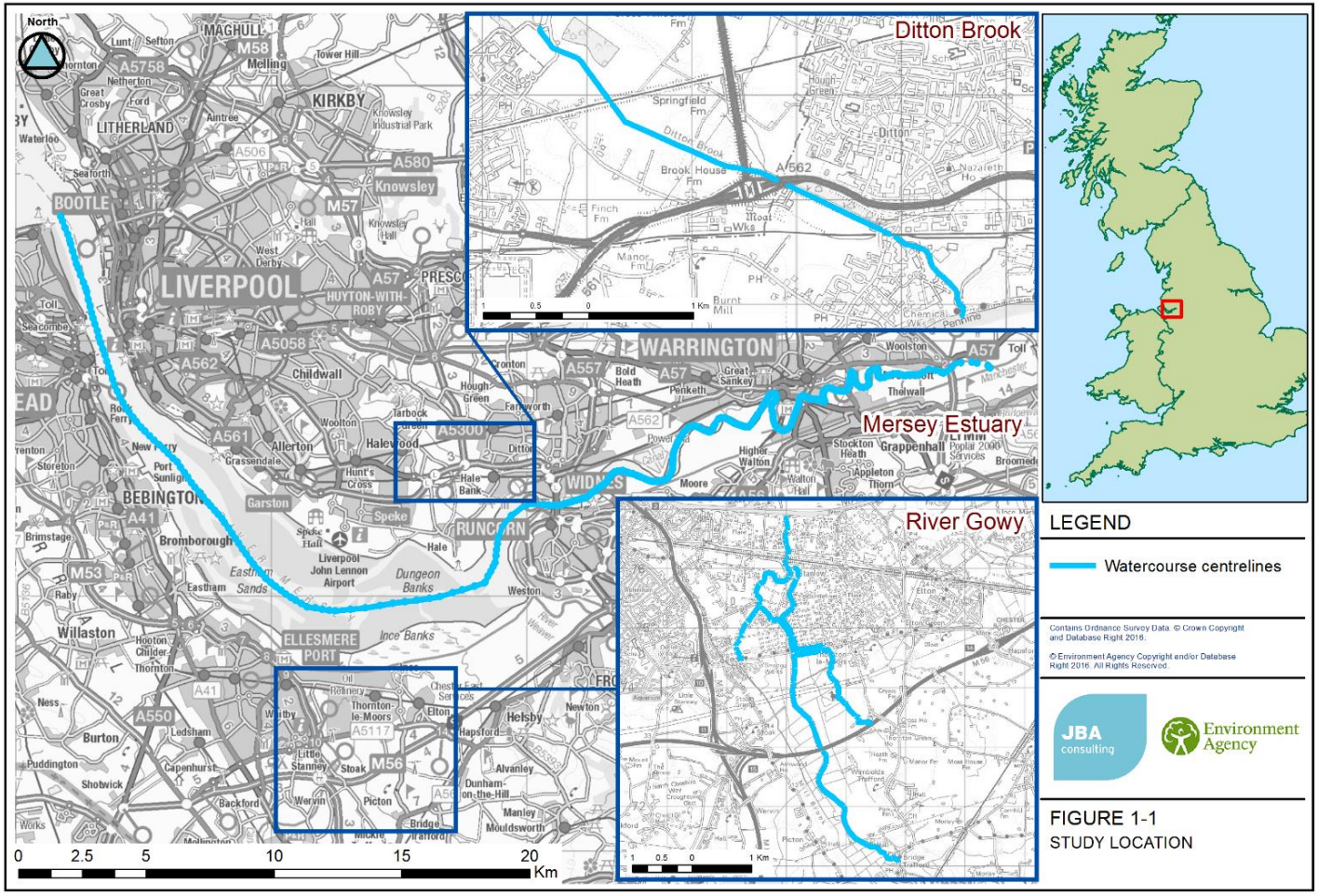
1.3.3 River Gowy

The River Gowy is a much larger watercourse than Ditton Brook, draining approximately 180km^2 of Cheshire, and joins the Mersey Estuary on its south bank at Stanlow Point (Ellesmere Port) via a siphon below the MSC. The upstream extent of the River Gowy model is Bridge Trafford. However, Mill Brook and Thornton Brook (tributaries to the River Gowy) are also included to represent additional flow routes across the wide, flat floodplains of the Gowy Meadows. The study reach is approximately 8km in length (from Bridge Trafford) and extends downstream to the upstream opening of the Gowy-MSC siphon.

The study reach of the River Gowy is largely embanked and flows between open mixed-use agricultural land and is joined by several straightened field drains before flowing through the Stanlow oil refinery and discharging into the Mersey Estuary. Stanlow oil refinery is a "regionally important economic asset"[1] and the potential consequences, either financial, social or environmental, because of tidal or fluvial flooding at the site could be significant. The watercourse through this reach has been artificially straightened, embanked and modified to efficiently convey flow whilst protecting the site from fluvial and tidal flooding. The River Gowy is protected from the propagation of a high tide by Folly Gates, a flapped multi-barrel culvert structure that ties into the downstream tidal defence line.

1 Environment Agency (2009) Weaver Gowy Flood Management Plan: https://www.gov.uk/government/uploads/system/uploads/attachment_data/file/293779/Weaver_Gowy_Catchment_Flood_Management_Plan.pdf

Figure 1-1: Study location



1.4 Previous modelling studies

There have been a significant number of previous modelling studies carried out on the Mersey Estuary and its tributaries. This study seeks to update the existing flood risk mapping by both adapting and further developing existing hydraulic models and developing new models where required.

1.5 Required outputs

The following outputs were required by the Environment Agency for this project:

- New or updated 1D-2D hydrodynamic models of the Mersey Estuary, Ditton Brook and the River Gowy;
- Analysis of present day fluvial flood risk along Ditton Brook and the River Gowy;
- Analysis of present day tidal flood risk along the Mersey Estuary and lower River Mersey including wave overtopping modelling;
- Tidal flood mapping outputs for the Mersey Estuary and fluvial flood mapping outputs for Ditton Brook and the River Gowy for the following AEPs:
  - 50%
  - 20%
  - 10%
  - 5%
  - 4%
  - 3.33%
  - 2%
  - 1.33%
  - 1%
  - 0.5%
  - 0.1%
- Flood mapping outputs for climate change scenarios for 2065 and 2115 as specified by the Environment Agency (see section 7.1);
- Flood mapping outputs for undefended modelling scenarios;
- Depth, level and hazard grids; hazard grids are classified using the Flood Hazard rating as described in the Defra/Environment Agency guidance document FD2321[2].
- Maximum velocity vectors;
- Spreadsheet and point shapefile of in-channel model results;
- Animations of 1% and 4% AEP flood events (depth) and velocity vectors;
- National Flood and Coastal Defence Database (NFCDD) data population;
- Modelling and Decision Support Framework (MDSF) 2 system ready data;
- Update to existing flood warning areas.

1.6 Report structure

This main project report is intended to provide a non-technical overview of the Mersey Estuary study. The technical calculations and justification of the decisions made during the modelling process are provided as appendices.

The structure and content of this report has been based on the Environment Agency Strategic Flood Risk Management Hydraulic Modelling and Risk Mapping Model Report Performance Scope (2010). This report is structured under the following headings:

1. Introduction
2. Qualitative description of flood response
3. Model approach and justification
4. Input data plan
5. Technical method and implementation
6. Model proving
7. Model results

2 DEFRA\Environment Agency. 2006. *Flood Risks to People, Guidance document FD2321/TR2*. Prepared by HR Wallingford, Flood Hazard Research Centre and Risk & Policy Analysts Ltd.

8. Assumption and limitations
9. Conclusions and recommendations.

Two technical appendices are included that provide an audit trail of the hydrology and modelling methods used. These are:

- The Flood Estimation Handbook (FEH) calculation record (Appendix A);
- The hydraulic modelling check file (Appendix B);
- A discussion of the wave overtopping modelling including sensitivity testing (Appendix C).

1.7 Draft deliverables

The following draft deliverables have been supplied to accompany this draft report:

- Edited flood outlines for design, climate change and undefended scenarios in shapefile format.
- Maximum depth, water level, velocity and hazard grids in American Standard Code for Information Interchange (ASCII) format.

2 Qualitative description of flood response

The source-pathway-receptor conceptual model is often employed to highlight the individual processes and components that influence flood risk in a given area. This chapter summarises how the components of this concept apply to the Mersey Estuary, Ditton Brook and River Gowy.

2.1 Sources

The source of flood water along the Mersey Estuary and its associated tributaries include both tidally and fluvially driven high water levels.

The Mersey Estuary hydraulic model was developed for this study to assess the risk of flooding during a large tidal event (i.e. surging high water levels in the Irish Sea). The River Gowy model was utilised (in addition for modelling fluvial events) to model the impact of these surging high tides at the Stanlow oil refinery. Fluvial flood risk to the upper reaches of the Mersey Estuary has been assessed previously as part of the 2013 Halcrow Warrington Scheme Flood Risk Assessment (FRA) study.

The Ditton Brook and the River Gowy models were built to simulate extreme fluvial events and adopted a Mersey Estuary mean high water spring (MHWS) tidal series at the downstream boundary of each. However, water levels on both watercourses can be affected by tidal conditions in the Mersey Estuary. The River Gowy has flapped outfalls (at Folly Gates) that prevent water from the Mersey Estuary flowing back up the river. During high tide, discharge through these structures will be impeded (or "tide locked") resulting in elevated water levels upstream. No such features exist on Ditton Brook, allowing tidal ingress into the lower reaches of the watercourse.

2.2 Pathways

In the Mersey Estuary catchment, the main flood pathways (or mechanisms) are overtopping of channel sides resulting in floodplain inundation. As many of the Mersey Estuary floodplains are typically small (relative to the size of the estuary), the volume of overtopping is rarely a significant factor in determining flood extent. However, this will not be the case in all locations, such as the lower Gowy at Stanlow, where floodplains are large and flat. In addition, the MSC provides an available volume of storage and flooding of land on the opposite banks of the Mersey Estuary will flood when this volume is exceeded and any adjacent high ground is overtopped.

The MSC also acts to convey flow if the high ground or defences between it and the Mersey Estuary are overtopped. Overtopping flood water may spill into the MSC and convey along the impoundment (the MSC thereby acting as a flood defence, as a barrier to flow and through the provision of additional storage) or, if there is sufficient volume of overtopping from the Mersey Estuary, result in local exceedance of channel capacity and overtop the opposite side of the canal and inundate the opposing floodplains. In some cases, the landward MSC channel side is higher than the spill level from the Mersey Estuary. This therefore provides additional defence from flooding in the event of tidal overtopping.

Flooding on both Ditton Brook and the River Gowy is driven by exceedance of channel capacity and impeded outflow into the Mersey Estuary during high tides. Rising water levels in each watercourse overtop either embanked or natural channel sides before conveying through the adjacent floodplains. On both watercourses, the afflux generated by hydraulic structures also contributes to locally elevated water levels in some locations.

Flooding on the River Gowy catchment is also influenced by the Thornton Brook diversion scheme whereby the route of the watercourse beneath the A5117 was blocked and flow diverted towards the main channel of the Gowy upstream of this embankment. These works were undertaken by the Environment Agency circa 2003.

In the case of Mill Brook, a tributary of the River Gowy, upstream and downstream flapped outfalls mean that, during a fluvial flood event on the River Gowy, the only flow route to enter Mill Brook and flow towards Stanlow would be through overtopping of the River Gowy left bank defence and flow across the floodplain.

2.3 Receptors

The primary receptors at risk of flooding in the study area include densely populated urban areas such as Liverpool, Birkenhead and Warrington, smaller settlements such as Hale Village and areas of high economic value such as the large oil refinery at Stanlow.

Possible impacts on such receptors from flooding range from loss of productivity or property to injury or even, in the most severe cases, loss of life. In the case of the Stanlow oil refinery there is also the potential for significant environmental damage.

This study addresses how the level of risk posed to these receptors may change under scenarios of predicted future climate change.

3 Model approach and justification

3.1 Aims of study

This study has been commissioned to update the existing hydraulic models and improve the current understanding of tidal flood risk from the Mersey Estuary as well as fluvial flood risk arising from both Ditton Brook and the River Gowy. The models developed for this study will be used to assess how flood risk and flood warning will be affected by the predicted impacts of climate change.

3.2 In-channel domain

For each of the three study watercourses a linked 1D-2D hydraulic modelling approach was adopted. One-dimensional hydraulic models provide both a computationally efficient and accurate way of representing flow in channels and through structures. However, on the floodplains where flow routes can be complex and multi-directional, 2D models offer significant advantages. Within this study, most main rivers and significant tributaries were represented within the 1D domains and their floodplains were represented using 2D domains. There were however some exceptions; for example, where the floodplains were narrow and constrained by topography it was felt they were best represented in the 1D domain. Conversely floodplain culverts were represented using 1D hydraulic structures linked directly to the 2D floodplain domain. Tributary watercourses in the Mersey Estuary model, such as Ditton Brook and Sankey Brook, were also represented in the 2D domain. However, where possible, their geometry was informed by previous modelling or survey.

The choice of modelling approach was initially less obvious for the Mersey Estuary. The Mersey Estuary is in places over 4km wide and could potentially be represented in either a 1D or 2D domain. However, for the purposes of this study, the in-channel area of the Mersey Estuary was modelled in a 1D domain with sections in the lower estuary derived from the most recently surveyed bathymetric data. The main reasons were the lack of hydrometric data to either demonstrate the need for, or calibrate, a 2D model, the additional run time and data production of a 2D model, and that the previous 1D model calibrated well at the available gauges.

3.3 Two-dimensional model resolution

The choice of spatial resolution for 2D models is always a compromise between detail and practicality. Whilst a fine resolution model will undoubtedly represent more detail it will also take considerably longer to process, produce considerably more data and may not significantly impact model results.

The specific requirements of each study domain were considered before selecting a suitable 2D model resolution. For the floodplains of Ditton Brook, a 4m resolution was preferred whereas on the River Gowy the broad flat upstream floodplains and wide flow paths through Stanlow Refinery allowed a 6m resolution to be adopted. The sheer scale of the Mersey Estuary model necessitated adopting a coarser, 10m, resolution.

3.4 Hydraulic modelling check file

Further discussion on the modelling approach and justification for decisions made during the model build is provided in the hydraulic modelling check file (Appendix B).

4 Input data plan

4.1 Summary of supplied project data

The data made available by the Environment Agency for use in this study are summarised in Table 4-1.

Table 4-1: Summary of supplied project data

Data Type	Format	Description	Use for this study
Coastal Erosion model (2010). Developed by Royal Haskoning.	MIKE21	A wave transformation model of the Lower Mersey Estuary.	Reviewed and utilised as part of wave overtopping modelling (see Appendix C).
Warrington Flood Hazard Mapping 1D-2D model (2010) (Mersey, Penketh-Whittle and Sankey). Developed by Halcrow.	ISIS-TUFLOW	1D-2D model of upper study domain. Built for purposes of fluvial modelling.	Used to inform representation of tributary watercourses in the Mersey Estuary 2D model (section 5.2.1.1).
Warrington Scheme FRA 1D-2D model (2013). Developed by Halcrow	ISIS-TUFLOW	Most recently updated 1D-2D Warrington model with flood alleviation scheme (FAS) defences as z-lines.	In channel domain for River Mersey used in 1D modelling. Modelling of other tributaries used as survey to inform 2D model. See section 5.2.1.1.
Flood Forecasting 1D model for Mersey Estuary (2010). Originally developed by JBA Consulting.	ISIS	Extensively calibrated 1D model of lower Mersey Estuary.	1D domain used for Mersey Estuary component of new 1D model. Trimmed to bank top and largely connected to new 2D domain or MSC model. See section 5.2.1.1.
Ditton Brook 1D model (2008). Developed by JBA Consulting.	ISIS	1D only fluvial model utilised for existing flood mapping.	Updated for current project with 2014 survey. See section 5.2.1.2)
Lower Gowy 1D model (2009). Developed by Scott-Wilson in 2003 and adapted with updated hydrology by B&V Group.	ISIS	1D only fluvial model of River Gowy from MSC to Bridge Trafford.	New survey commissioned for River Gowy, Thornton Brook and Mill Brook. See section 4.2. Existing model not utilised and new model developed from scratch using new topographic data (section 5.2.1.3).
River Gowy and Thornton Brook Improvements, Stanlow, Scott-Wilson 2003.	Hard copy ring binders	Three ring binders of information regarding the physical improvements to the River Gowy and Thornton Brook drainage arrangements. This specifically addressed the diversion channel across the upstream end of the A5117.	Used survey or as-built drawings to inform dimensions of flapped surface water outfalls where available.
MSC model (2013), developed by JBA Consulting.	ISIS	1D only ship canal model. Designed for modelling extreme fluvial events. Also includes lower reach of River Weaver and Weaver Navigation.	Utilised to represent available storage following overtopping from the Mersey Estuary. Some structures adapted to model tidal flood event. Spill links between Mersey and MSC retained in new modelling

Table 4-1: Summary of supplied project data			
Data Type	Format	Description	Use for this study
			as best available data. See section 5.2.1.1.
1 m and 2m filtered Digital Terrain Models (DTM) and unfiltered Digital Surface Models (DSM).	ASCII	1m and 2m DTM and DSM (LIDAR) data.	Filtered LIDAR was used to inform topographic components of each 2D model. No null values were identified. Any modifications to the data has been detailed in Appendix B.
Ordnance Survey (OS) 1:10,000, 1:25,000 and 1:50,000 mapping and OS MasterMap.	GIS	High quality mapping of all of study area of different resolutions.	Flood mapping and reporting
Existing Flood Warning Areas shapefiles.	GIS Shapefile	A representation of the existing flood warning areas that were updated by this study.	Comparison of model results and identifying where flood warning areas may need to be updated.
Flood history shapefiles, reports and photographs for areas of the Mersey Estuary and Ditton Brook. See section 4.2.	GIS, word processing documents, PDFs and JPEGs	Scattered flood history information for the Mersey Estuary. Tidal flood events only.	Sensibility checking of model results. See section 6.4.
Existing Flood Zone (FZ) maps.	GIS Shapefile	FZ mapping supplied for whole study domain.	Referenced when sensibility checking model results and used for report mapping.
Detailed river network (DRN) data.	GIS Shapefile	DRN data supplied for whole study domain.	Report mapping.
Topographic survey for Ditton Brook and Stewards Brook.	PDF, DWG and ISIS	Topographic survey collected in 2014.	Used to update the existing 1D model of Ditton Brook where 2014 survey was available.
Hourly wind data from Speke.	Excel	Hourly wind peak data covering 1990-2015. Some data omissions early in life of gauge.	Calibration of Mersey Estuary model. See section 6.3.
Mersey Estuary bathymetry data (2013).	ASCII	Bathymetry data supplied for lower Mersey Estuary between North Sea and Widnes/Runcorn bridge. Confirmed to be most recently surveyed data.	Used to verify 1D model and produce detailed water level lines.
Environment Agency Asset Information Management System (AIMS)	GIS Shapefile	AIMS data supplied for whole study domain.	Used in each of the hydraulic models to inform defence crest elevations. See section 5.2.2 for description of method for modelling defences.
Surveyed defence crest levels for the Fiddler's Ferry	XYZ data (text)	Recent XYZ survey data of the defence crest. Used to inform the defended model	Informed the crest elevations of the Fiddlers Ferry defence in TUFLOW. Recognised by

Table 4-1: Summary of supplied project data			
Data Type	Format	Description	Use for this study
embankment.		runs	AIMS as a formal defence. See section 5.2.2.
Stage and level gauge data for the River Mersey and Mersey Estuary.	Microsoft Excel / Text file	Variable time frames with some missing data points. Generally, a very thorough record.	Calibration of Mersey Estuary model. See section 6.3.

4.2 Historical flood information

JBA was supplied with the following historical flooding information for the Mersey Estuary:

- Drawn flood outlines in Warrington and Howley from a tidal flood event (1990);
- Drawn maximum extent flood outline at the Golden Triangle Industrial Estate (adjacent to Hale Road) on Ditton Brook from a tidal event (1990);
- Flood history photographs and shape files for Warrington and Fiddler's Ferry (2013);
- Drawn flood outline and photographs of flooding at Hale Village and access roads (Carr Lane and Town Lane) (no reported property flooding) (December 2013);
- Photographs of defence embankment at Fiddler's Ferry (February 2014) (no visible overtopping);
- Hydrometric (level) data for Gladstone Dock, Fiddler's Ferry and Westy gauges;
- Anecdotal evidence for frequency and locations of historical flooding for the Stanlow oil refinery and River Gowy.

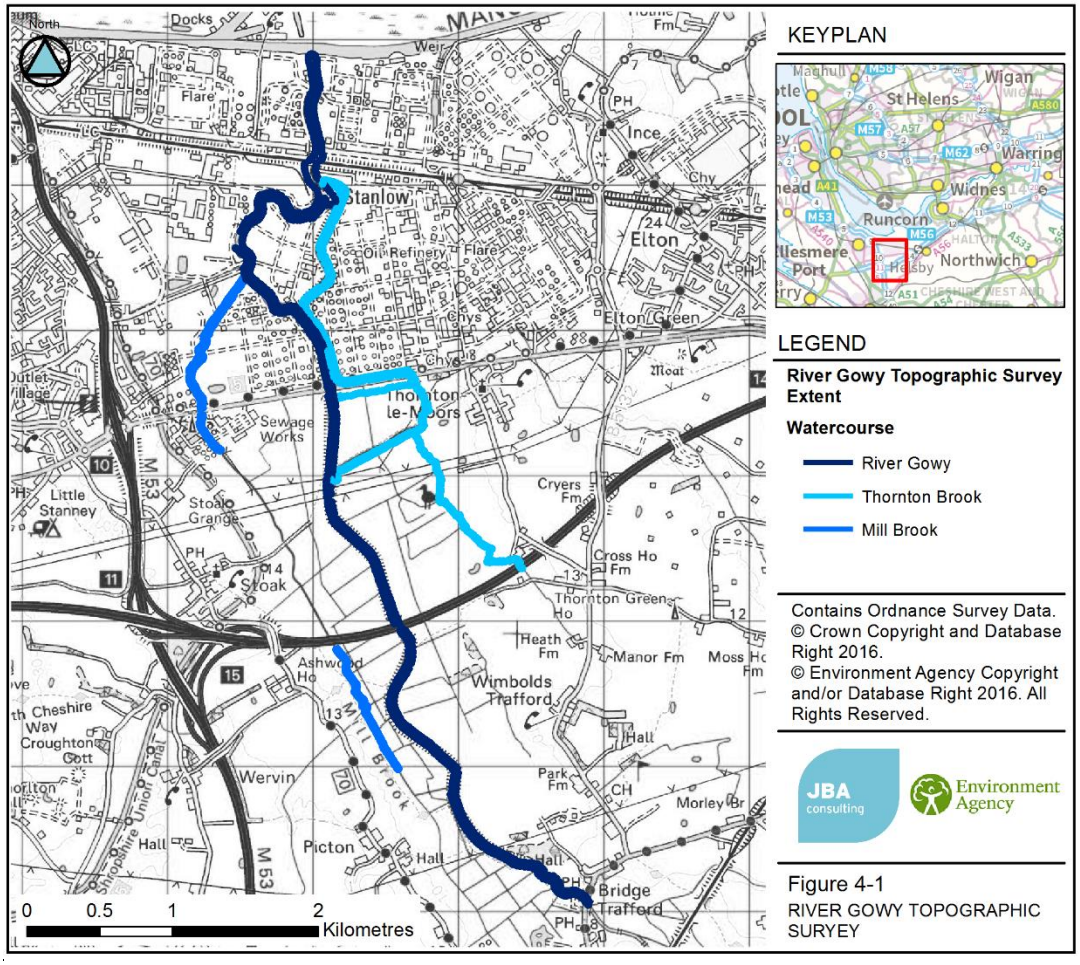
4.3 Additional topographic survey

The existing 1D model of the River Gowy was deemed to be no longer fit for purpose. Firstly, most structures were represented as Bernoulli loss units and not structure specific units that employ tailored equations as opposed to more generic loss functions. Secondly, some of the more recent topographic changes to the watercourse (such as the Thornton Brook diversion) were not included in the existing model and would need to be included in this study.

JBA was therefore commissioned to survey most the study reach of the River Gowy, through the Stanlow oil refinery and the Gowy Meadows, to inform the new hydraulic model build (Figure 4-1). The survey included 23 new cross sections along the River Gowy between the MSC and Bridge Trafford, 15 cross sections along Thornton Brook, 18 cross sections along Mill Brook and bank-top survey for most the study domain.

There is a section in the middle of Mill Brook which was not surveyed. It was unclear from the site visit in which direction Mill Brook would flow in normal conditions, but it was clear that the flood mechanism during a fluvial event into Mill Brook would be embankment overtopping and flow across the floodplain. Therefore, it was agreed with Environment Agency that this reach of Mill Brook was best represented within the TUFLOW domain and, on that basis, did not require detailed topographic survey.

Figure 4-1: River Gowy topographic survey



5 Technical method and implementation

5.1 Flood estimation

This section presents a summary of the FEH calculations undertaken for estimating boundary conditions applied to the Mersey Estuary, Ditton Brook and River Gowy hydraulic models. A more detailed account of the calculations is provided in the FEH calculation record in Appendix A.

5.1.1 Mersey Estuary

The Mersey Estuary model was, for the purposes of this study, only used to assess tidal flood risk. Therefore, FEH methodologies do not apply. The approach for representing the model boundaries is described below.

5.1.1.1 Model inflows

An assessment of joint probability was not included in the study so a steady inflow at an estimated mean flow has been applied to the upstream fluvial inflow boundary. A mean flow was applied at the upstream node (MSY25869) that utilised CEH (Centre for Ecology and Hydrology) Hydrometric Register mean flow data.

Under normal flow conditions the flow control structures on the MSC at Latchford Locks remain closed effectively diverting all flow into the Lower Mersey. For this reason, inflows have been applied directly to the Mersey upstream boundary.

5.1.1.2 Tidal boundary condition

To model the complete range of required design tidal flood events, an astronomical tidal boundary of specified AEP was extracted from the Coastal Flood Boundary Dataset (CFBD) (at node 1168) and applied at the 1D downstream boundary (node MEST_0000). This included a 0.5% AEP event with allowances for climate change.

5.1.2 Ditton Brook

5.1.2.1 Catchment summary

Ditton Brook is, by comparison to the Rivers Mersey and Gowy, a small watercourse with a total catchment area of 68km^2 as it discharges into the Mersey estuary (Figure 5-1). The watercourse is fed by a complex network of tributaries draining south eastern Liverpool, culminating in a relatively wide and embanked channel from Halewood to the confluence with the Mersey Estuary. Immediately upstream of the outfall the watercourse is joined by the urbanised Steward's Brook, a 5km^2 catchment draining Widnes.

The predominant catchment characteristic is the dense urban land cover in the upper reaches and at Widnes. The catchment contains major suburbs of Liverpool including Childwall, Halewood and Prescot. It is anticipated such heavy urbanisation (with an URBEXT2000 value of 0.25) will significantly influence the shape and magnitude of the flood hydrographs.

Underlain by low permeability and seasonally wet loams and clays, the catchment has BFIHOST and SPRHOST values of 0.417 and 36.88 respectively.

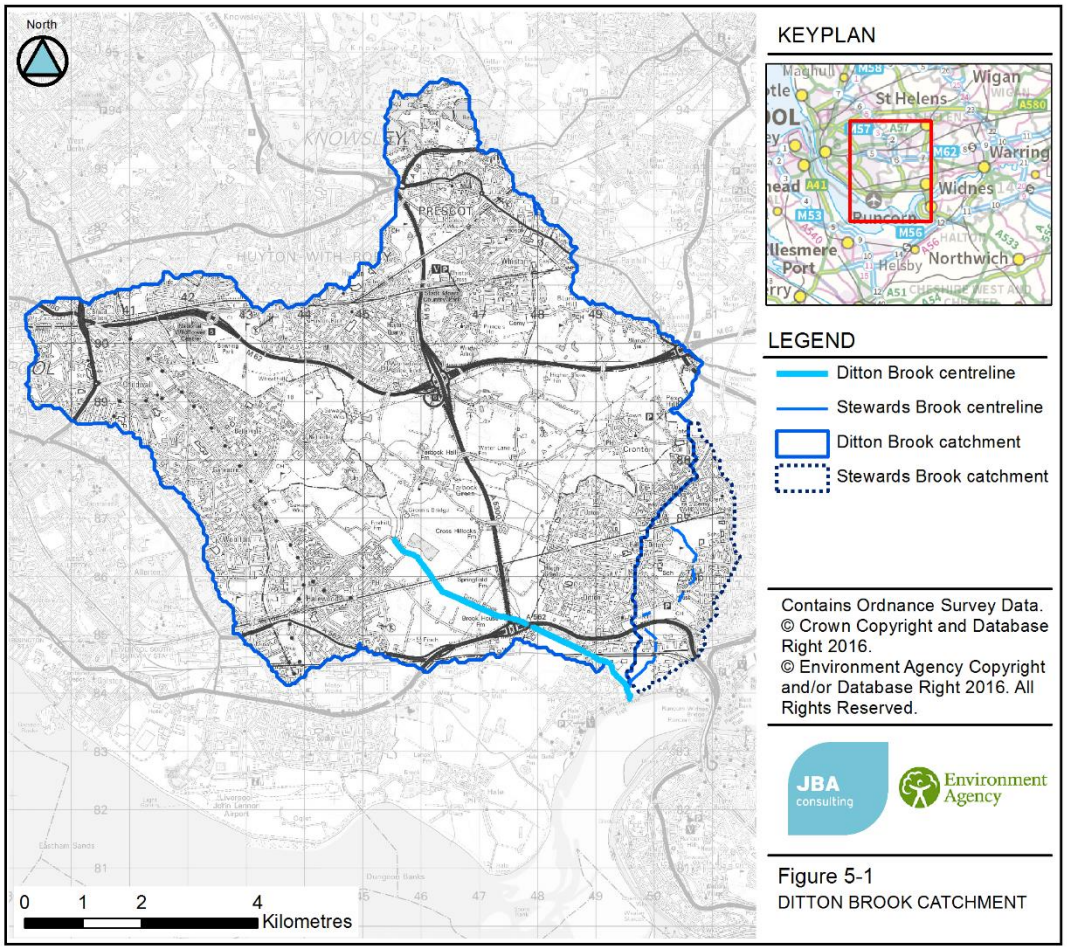
Annual average rainfall over the Ditton Brook catchment is 845mm.

5.1.2.2 Choice of method

A rainfall-runoff approach using the urban variant of ReFH provided an estimate of flood volume as well as flood peaks. The urban component allowed a faster routing time of flood waters from urban portions of the catchment compared to the undeveloped rural areas. It was assumed that all paved areas within the Ditton Brook catchment area drain towards the channel, and not away via sewer systems. A consistent storm duration was used across all inflows, with the durations determined via testing of the hydraulic model to establish which provided the highest flood water levels.

The effect of storm duration was tested at the 1% AEP event. Nine durations (ranging from 5.5 to 21.5 hours at an interval of 2 hours) were tested. The critical duration was demonstrated to be 19.75 hours.

Figure 5-1: Ditton Brook catchment



5.1.3 River Gowy

5.1.3.1 Catchment summary

The River Gowy (Figure 5-2) rises in the village of Lower Bunbury in the Pockton Hills to the west of Crewe, with a maximum altitude of 222mAOD. For much of its upper course the river runs alongside the Shropshire Union Canal in a constrained valley before entering a wider floodplain as it conveys to the east of Chester. The lower reaches are embanked as the channel passes below the M56 motorway and the MSC, discharging into the Mersey Estuary at the Ellesmere Port (Stanlow) oil refinery.

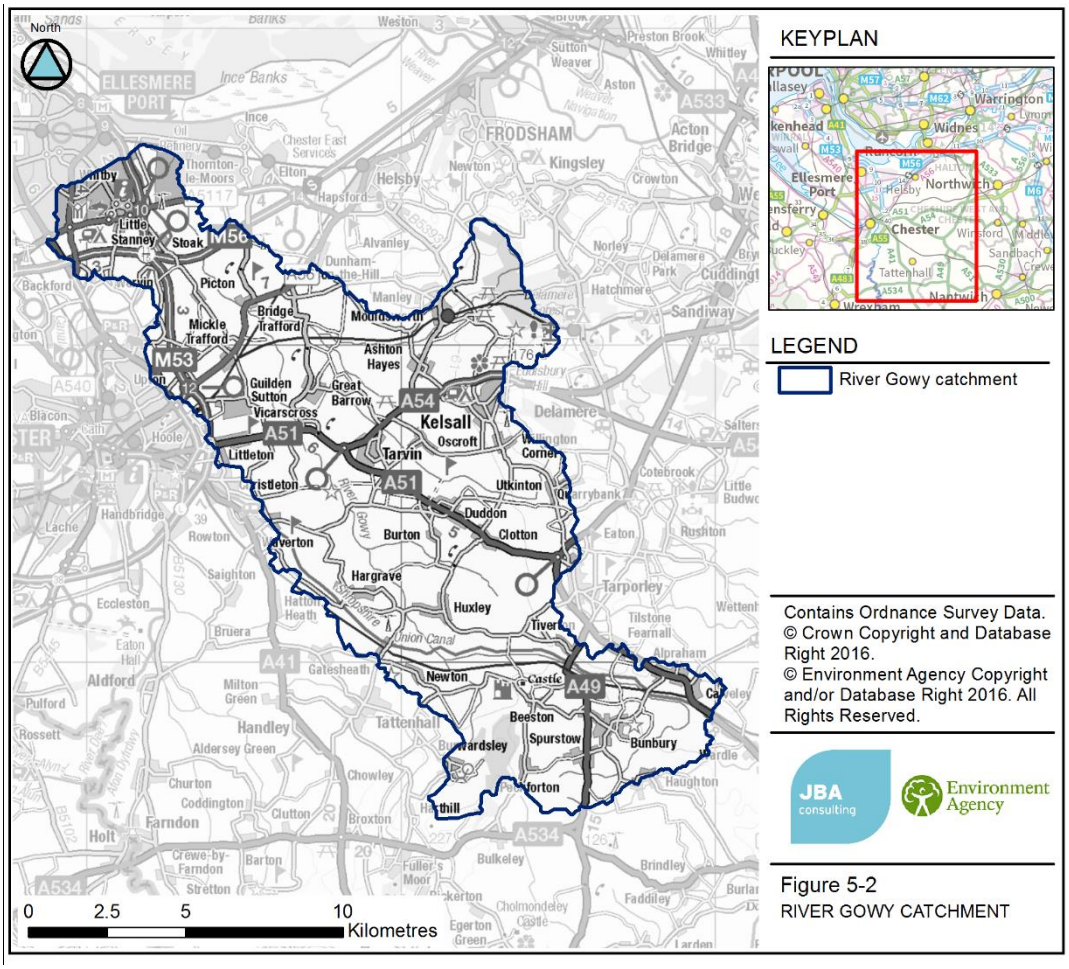
During high tides the watercourse's lower reaches can fill with tidal water conveyed through a syphon beneath the MSC. This tide can propagate no further than Folly Gates, near the centre of the Stanlow oil refinery, where there are flapped outfall structures and flood defences. Tide locking occurs as a result, therefore an appreciation of flood volume as well as flood peak is important. The total catchment area at the outfall is 178km^2.

Predominantly rural, the catchment includes large areas of pastoral agriculture; the only developed areas of note are Bunbury, Kelsall, Tarvin, Mickle Trafford and Stanlow oil refinery, resulting in an URBEXT2000 value of 0.03.

The catchment geology is made up of Triassic sandstones and marls overlain with glacial drift deposits, which results in a partially permeable catchment reflected in a BFIHOST value of 0.526 and SPRHOST of 32.6%.

Annual average rainfall over the River Gowy catchment is 724mm.

Figure 5-2: River Gowy catchment



5.1.3.2 Choice of method

Peak flow estimates on the River Gowy were estimated using both the FEH Statistical and Revitalised Flood Hydrograph (ReFH). The ReFH estimates were taken forward as the preferred flows for the following reasons:

- Given the tide locking characteristics of the lower catchment (where the major receptors are located) it was important to include an estimate of flood volume in addition to flood peak. Use of ReFH (with a user defined time to peak (Tp) value) offers a better representation of volume than FEH statistical.
- ReFH allows testing of the critical storm duration to be undertaken, an important consideration where flood volume is important.
- Using ReFH does not place reliance on the quality of the Bridge Trafford gauge. Estimating Tp uses water levels records, but only in terms of the hydrograph timing in relation to rainfall.

The effect of storm duration was tested at the 1% AEP event. Eleven durations (ranging from 10 hours to 49 hours) were tested. The critical duration was demonstrated to be 45.5 hours.

5.2 Hydraulic modelling

This section summarises the hydraulic modelling approach specifically for each of the Mersey Estuary models.

5.2.1 Method and modelling software

A 1D-2D ISIS-TUFLOW modelling approach has been utilised to deliver the required outcomes of the study. The TUFLOW components of each model were developed in ArcGIS, using an ASCII grid of Light Detection and Ranging (LIDAR) elevations as the ground model, with alterations to this undertaken in shapefile format. The supplied bathymetry data were used to inform the 1D cross section geometry and water level lines for the Mersey Estuary model. Included below is a summary of the model build for each of the three models developed. A detailed account of the process and methods used is provided in the accompanying hydraulic modelling check file (Appendix B).

5.2.1.1 Mersey Estuary

The Mersey Estuary model utilised a combination of the existing 1D components of the Mersey Estuary Flood Forecasting, Warrington flood risk assessment (FRA), Warrington flood hazard and MSC models. This model has been built specifically for assessing tidal flood risk.

During extreme flood events, the embanked MSC forms a barrier between the Mersey Estuary and its southern floodplains; this was included in the new model to represent this blockage to flow and to represent both storage and conveyance in the canal in the event of overtopping. The existing MSC ISIS model was linked to the Mersey model using lateral spills that allowed the transfer of water between the two watercourses. The River Weaver sluice gates were retained and connected to the Mersey Estuary via an open junction unit. These sluice gates allow water to discharge to the Mersey Estuary from the River Weaver if water levels in the MSC exceed the statutory water level (4.15mAOD). The operating rules for these gates were estimated to maintain this statutory water level and offset in increments of 100mm to promote model stability. Although the survey that informed the MSC model is old (circa 1960), modelling in this way provides a much more realistic representation of how tidal overtopping from the Mersey Estuary would behave within an impoundment and where flooding on the landward channel side is likely to occur.

A 10m TUFLOW/2D cell size was chosen for the Mersey Estuary model to achieve a compromise between sufficient detail from the hydraulic model and model run time.

The docks in the floodplain on the lower Mersey Estuary were connected directly to the 1D ISIS channel domain via 1D ESTRY uni-directional (flapped) rectangular culverts to represent the lock systems. No bathymetry was available to determine the dock floor levels. However, an approximate water surface was available in the DTM. The bed levels within the docks were therefore lowered to an estimated level (3m below the water surface when the LIDAR was flown). They were then filled back to the LIDAR elevation using an initial water level to represent a realistic water surface. The uni-directional culverts meant that the gates were forced open during high tide and the locks stayed full, or the gates closed, when the tide in the Mersey Estuary receded.

The lower River Gowy, up to Folly Gates (the tidal defence), was originally included in the Mersey Estuary model as a 'gully line' and connected to the Mersey Estuary via two ESTRY culverts that represent the siphon. A gully line is a GIS layer in TUFLOW that applies surveyed bed level points to a line to modify the LIDAR data and appropriately represent the elevation and long section of the channel centreline. However, due to the importance of conveyance in the lower reaches of the River Gowy it was considered more appropriate to represent tidal flood risk to Stanlow using the same 1D-2D approach as adopted for the fluvial Gowy model, using both a finer 2D grid resolution and a more detailed 1D representation of the channel and hydraulic structures. The mapped extent also included any overtopping from the MSC extracted from the Mersey Estuary model. This is described further in section 5.2.1.3.

An upstream flow-time (QT) series was included as a constant mean flow derived from the CEH UK Hydrometric Register for the Irlam Weir (River Mersey) gauging station. This was deemed to be appropriate for the purposes of modelling a tidal only flood event.

An astronomical tidal boundary of specified AEP was extracted from the CFBD (at node 1168) and applied at the 1D downstream boundary.

In addition to the inundation resulting from the astronomical tidal downstream model boundary, the Mersey Estuary model also included provision for additional inflow into the 2D model caused by locally wind driven wave overtopping.

A discussion of the wave overtopping modelling is presented in Appendix C.

5.2.1.2 Ditton Brook

The new Ditton Brook 1D-2D model utilised a small number of sections and structures from the existing Ditton Brook 1D model. However, where recently acquired (2014) survey was available, existing cross sections and structures were replaced and remodelled using the new survey. Stewards Brook was represented as a hydrological inflow only and was not included in the 1D or 2D model geometries. It was observed that results in this area of Ditton Brook were almost exclusively controlled by the tide it was concluded that, due to the bed level difference between Stewards Brook and Ditton Brook and the steepness of the incoming channel, the additional 1D channel would not provide significant storage at high tide.

A 4m cell size was chosen for the Ditton Brook model to allow for the sufficient representation of detailed flow paths through the urban and industrial areas throughout the TUFLOW domain.

Upstream model boundaries are represented as ISIS ReFH units converting catchment rainfall data into runoff and fluvial flow. A time varying MHWS tide, calculated through the Mersey Estuary model, was applied at the downstream boundary.

5.2.1.3 River Gowy (fluvial)

The River Gowy 1D model was developed using topographic survey acquired (February-March 2016) by JBA for the purposes of this study. The 1D channel component included the entire study reach of the River Gowy from Bridge Trafford to the Mersey Estuary, all of Thornton Brook, and the downstream extent of Mill Brook within the Stanlow oil refinery. Upstream of Stanlow, Mill Brook was included only in the TUFLOW domain.

TUFLOW was used to represent the Gowy floodplains and a 6m cell size was chosen to allow sufficient representation of detailed flow paths through the Stanlow site, whilst maintaining manageable simulation times. Culverts beneath major embankments (such as the M56) were included as 1D ESTRY units linked to the 2D floodplain domain.

A different model schematic was adopted for the 0.1% fluvial AEP flood event whereby the floodplains upstream of the M56 were modelled only in 1D, as ISIS reservoir units. This approach was used to minimise model instability whilst reducing run times. During large magnitude events the floodplains in this reach are fully inundated and act simply as a large reservoir behind the motorway (which does not overtop). This characteristic can be accurately represented in 1D only given the lack of complex flow paths.

All fluvial flood flows were added to the Gowy model as ISIS ReFH units converting catchment rainfall data into runoff and fluvial flow. A time varying MHWS tide, extracted from the Mersey Estuary model, was applied at the downstream boundary. For the tidal scenario, a mean flow was specified at Bridge Trafford gauge (and low dummy flows applied elsewhere), with the design tidal levels applied at the downstream boundary, taken from results of the Mersey Estuary model.

5.2.1.4 River Gowy (tidal)

Unlike the Mersey Estuary and Ditton Brook models, the River Gowy model was used to simulate both fluvially and tidally dominated scenarios. This approach was adopted due to the complex interactions between the Mersey Estuary and the River Gowy and the importance of conveyance in the Gowy channel during an extreme tide. Inclusion of the tidal scenario in the Gowy specific model also allowed representation of the flow through the MSC siphon and the interaction with the flapped outfalls at Folly Gates and elsewhere.

An extreme tidal downstream boundary (for each specified return period and climate change scenario) was extracted from the Mersey Estuary model for the nearest node to the MSC siphon outlet. In these model runs the level in the MSC was also extracted and applied as an HT TUFLOW boundary, adding water directly to the floodplain where the MSC southern embankment was shown to overtop in the Mersey Estuary model.

5.2.2 Defended (design) scenarios

Defences were included across each of the Mersey Estuary models; most of these were included either within the 2D domain or along the 1D-2D boundary. Defence lines and elevation

data were extracted from a combination of topographic survey, existing 2D models and Environment Agency AIMS data.

In order to utilise the best available defence data for each respective structure, the following approach was devised:

- To use defence z-lines (including elevations) from the Warrington FRA model for both phase 1 and phase 2 of the Warrington FAS;
- To use defence z-lines (including elevations) from the Warrington FRA model for all other modelled defences that are not part of the new scheme (this will be used in preference to the elevations in AIMS);
- For defences not previously included in a TUFLOW model, upstream and downstream crest levels of formal embankments and walls were used where they were provided in AIMS;
- To use the topographic survey to represent the Fiddler's Ferry, Ditton Brook (where available as part of cross section survey) and River Gowy defences;
- Where raised defences were not included in a previous TUFLOW model and did not have crest level data in AIMS, crest heights were extracted from LIDAR data.

However, there were exceptions to the above rules where crest levels provided in AIMS were either notably lower than some or all of those in the LIDAR data (for instance at defensive embankments) or where the LIDAR gave a more detailed representation of defence crests. In such cases, peak LIDAR elevations were used preferentially over the AIMS data to prevent a potentially undulating defence profile or a conflict in levels.

In the case of the Mersey Estuary model, 1D spills between large parts of the lower Mersey Estuary and MSC were extracted from LIDAR. For the defended model runs, these spills were compared against AIMS data and adjusted so that the minimum spill level was greater than or equal to the published AIMS crest level. The existing spills from the MSC model (in the area of Runcorn and the River Weaver) were not changed as, although defence crests are based on old (approximately 1960s) survey, these were taken to be the best available data of defence crests.

5.2.3 Undefended scenarios

The undefended scenarios seek to understand the flood risk situation if any formal defences simply did not exist.

In TUFLOW, the elevations of 1D-2D boundary cells that align with the crest of embankments (and any adjacent cells associated with that defence) were lowered to represent ground levels behind the defences. Where defences were present within the 2D domain, any raised embankment topography was removed from the ground model (by interpolating ground elevations from either side of the embankment using a z-shape) in TUFLOW.

Where defences were included in the 1D model between the Mersey Estuary and MSC, any spills that were extracted from LIDAR and modified using AIMS data were reset back to the LIDAR elevation. Elsewhere, where spills have been utilised from the existing MSC model, for simplicity these spill elevations were retained. The landward side of the ship canal is almost always higher than the AIMS or modelled spill level. If there were to be any flooding in this area, the defences would therefore be overtopped anyway. As a result, there was no benefit to removing these defences for the undefended model scenario and this avoids any major assumptions regarding the undefended spill crest.

For future modelling work, it would be beneficial to collate new survey of these defence crests between the Mersey Estuary and MSC.

5.2.4 Model schematisation

Figure 5-3, Figure 5-4 and Figure 5-5 illustrate the general schematisation of the Mersey Estuary, Ditton Brook and River Gowy models and include an overview of the 1D and 2D model domains.

Further detail regarding the model schematisation can be found in the hydraulic modelling check file in Appendix B.

5.3 Flood warning

An update to the flood warning areas will be provided following the approval of the draft modelling results and flood outlines.

In accordance with the package scope, this will involve:

- A review of the existing flood warning areas in comparison to the updated undefended 0.1% AEP flood outlines. This will identify whether modifications to these extents are required.
- A review of the first property to flood using the updated and approved flood mapping and modelled level results.
- Splitting the undefended 0.1% AEP) into flood alert, risk and warning areas, or Flood Warning Flood Risk Areas (FWFRAs), based on probability of flooding, flooding mechanism, land use type and number of properties. These areas will then be aggregated into communities that can be provided with a Flood Warning Service.

Figure 5-3: Mersey Estuary schematisation overview

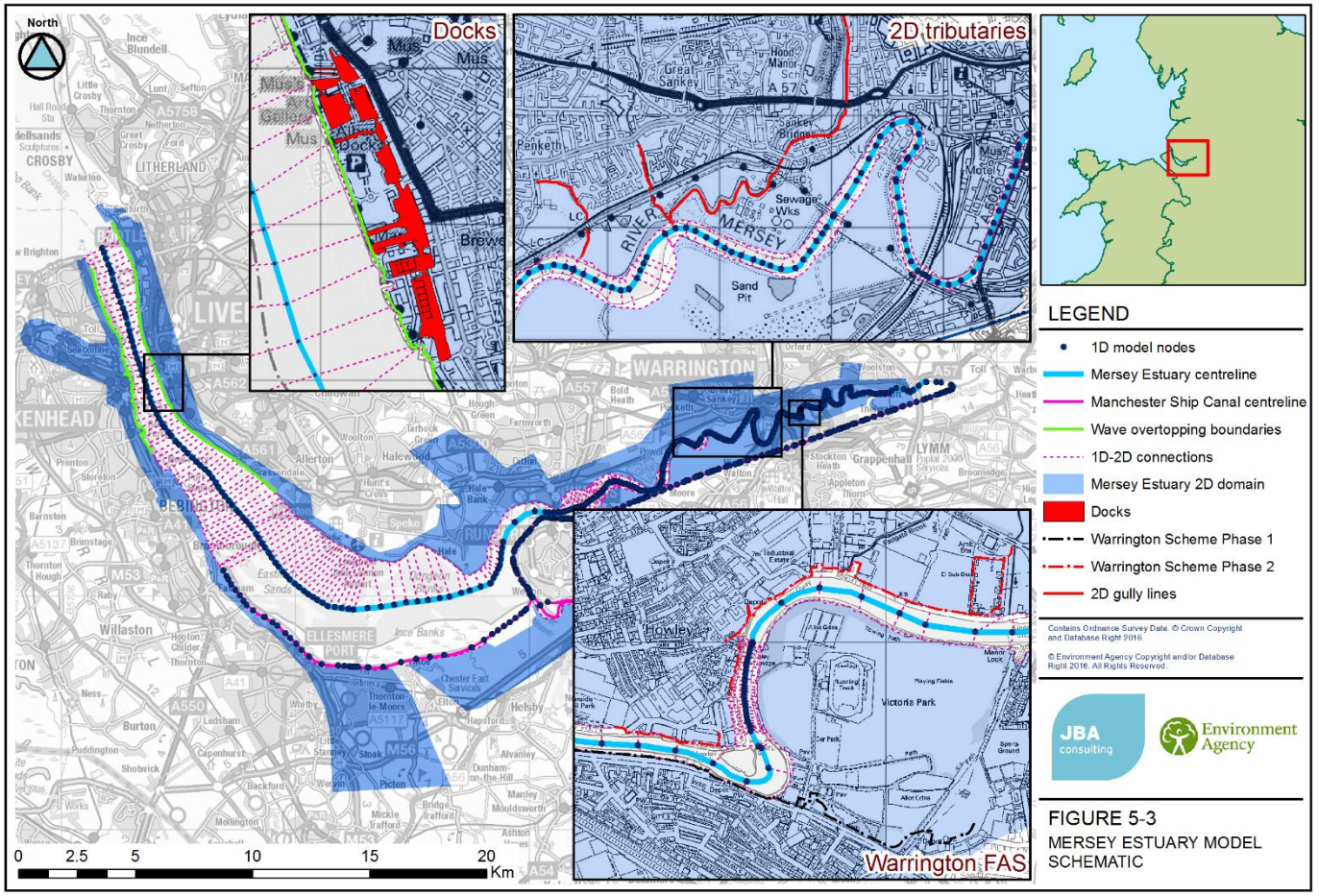


Figure 5-4: Ditton Brook schematisation overview

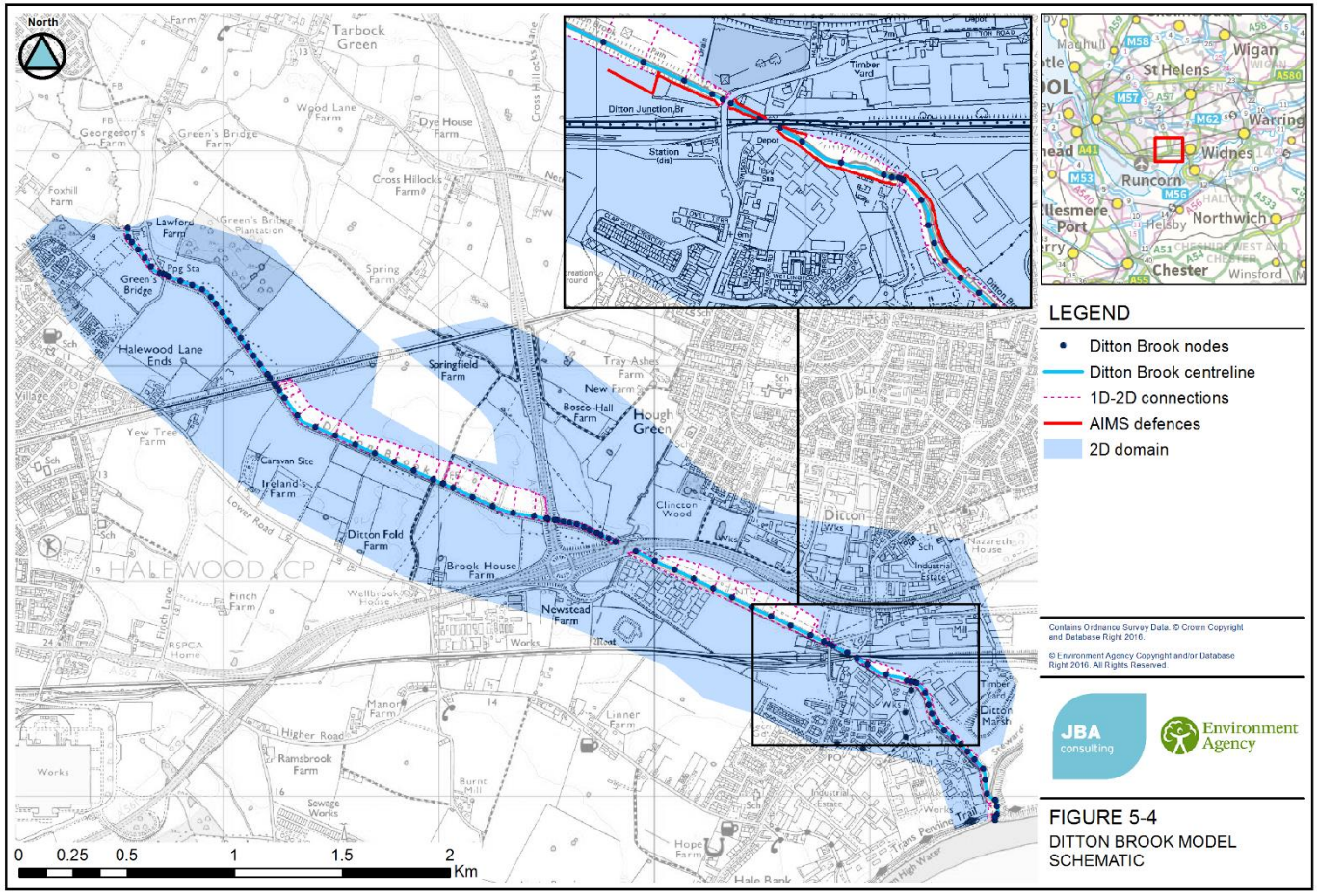
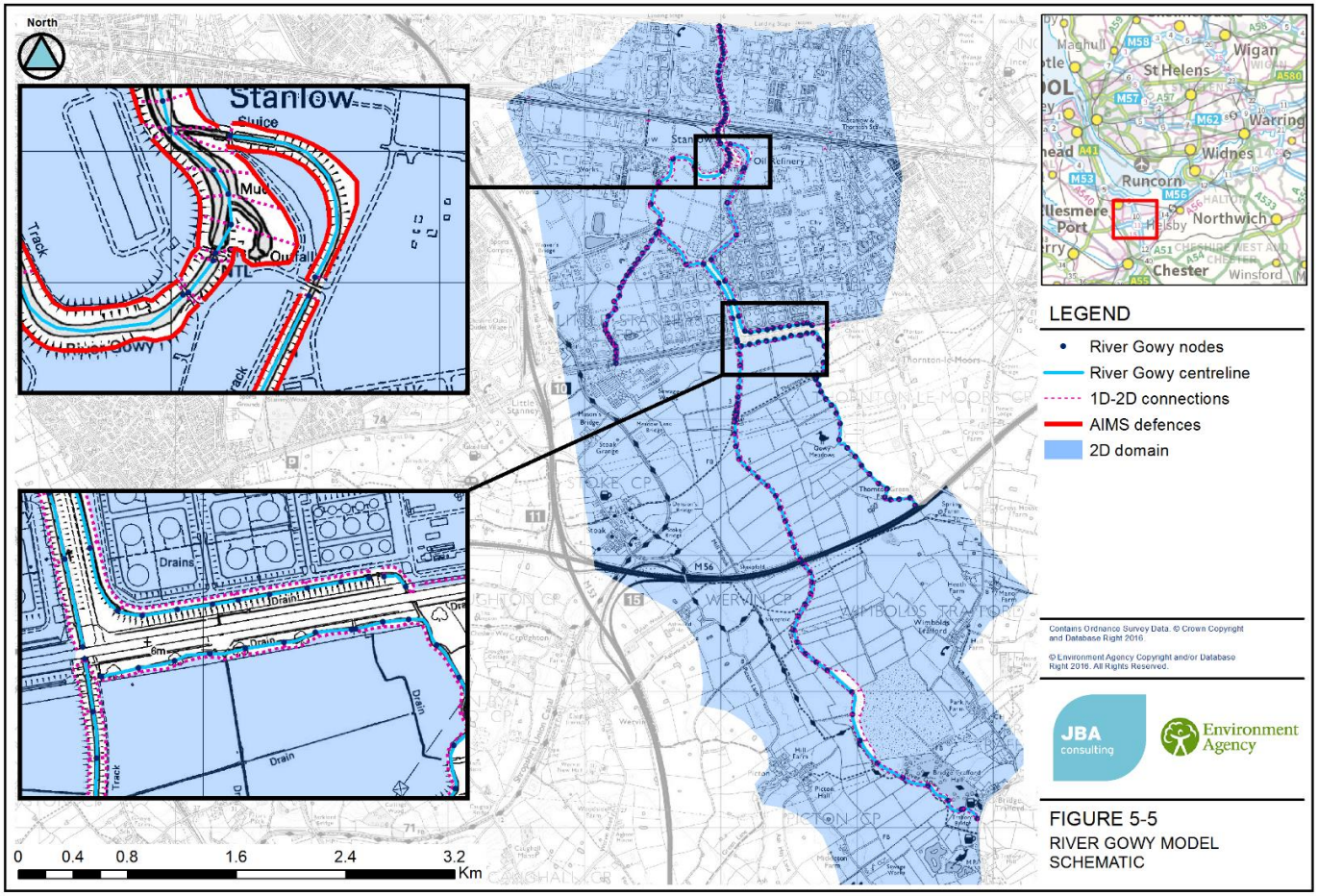


Figure 5-5: River Gowy schematisation overview



6 Model proving

6.1 Introduction

This chapter describes the run performance, calibration, sensibility and sensitivity analysis that has been undertaken on each of the three hydraulic models developed for this study. The results from these have been included in the hydraulic modelling check file (Appendix B). A summary of these results is presented here.

6.2 Run performance

The hydraulic models developed for this study have little in the way of numerical instability. However, simplifications or modifications were carried out in particular instances to smooth the transition between 1D and 2D flow, the switch between positive and negative flow along tidal reaches and to improve the simulation of low flows in the 1D model. Such modifications are small in the context of the wider model and do not notably alter the in-channel water level or impact the volume of flow that can be conveyed across the 2D domain.

For the River Gowy, the upstream extent of the model used a 1D floodplain approach for the fluvial 0.1% AEP only. This approach encouraged stability whilst remaining an appropriate method of representing the local flood hydraulics during such an extreme event. The most extreme tidal events were simulated using a reduced 1D and 2D timestep (no changes were made to model geometry).

6.3 Calibration

There are no historic or active river gauges on Ditton Brook to enable calibration of this model and there is insufficient confidence in the river flow gauge at Bridge Trafford to extract a suitable magnitude flood event for calibration purposes on the River Gowy.

However, on the Mersey Estuary there are sufficient gauged data to check whether or not the model reliably captures the propagation of the flood wave during a tidal event.

The peak (highest on record) tidal flood event that was covered by each of the Gladstone Dock, Fiddler's Ferry and Westy level gauges was selected to check the representation of observed events. This was 30 March 2006 and 20 hours were extracted from the Gladstone Dock gauge either side of the peak to form the downstream boundary (29.03.2006 18:15 - 31.03.2006 08:00). Flow data from upstream gauges for the same time period were extracted to form the upstream boundary. Where flow data were not available, the mean flow was applied from the CEH hydrometric register. The upstream gauges used for calibration are listed below:

- Ashton Weir (Mersey) flow-time record.
- Little Woolden Hall (Glaze Brook) mean flow (flow-time record does not cover this event).
- Partington (Sinderland Brook) flow-time record.
- Dunham Massey (Bollin) flow-time record.

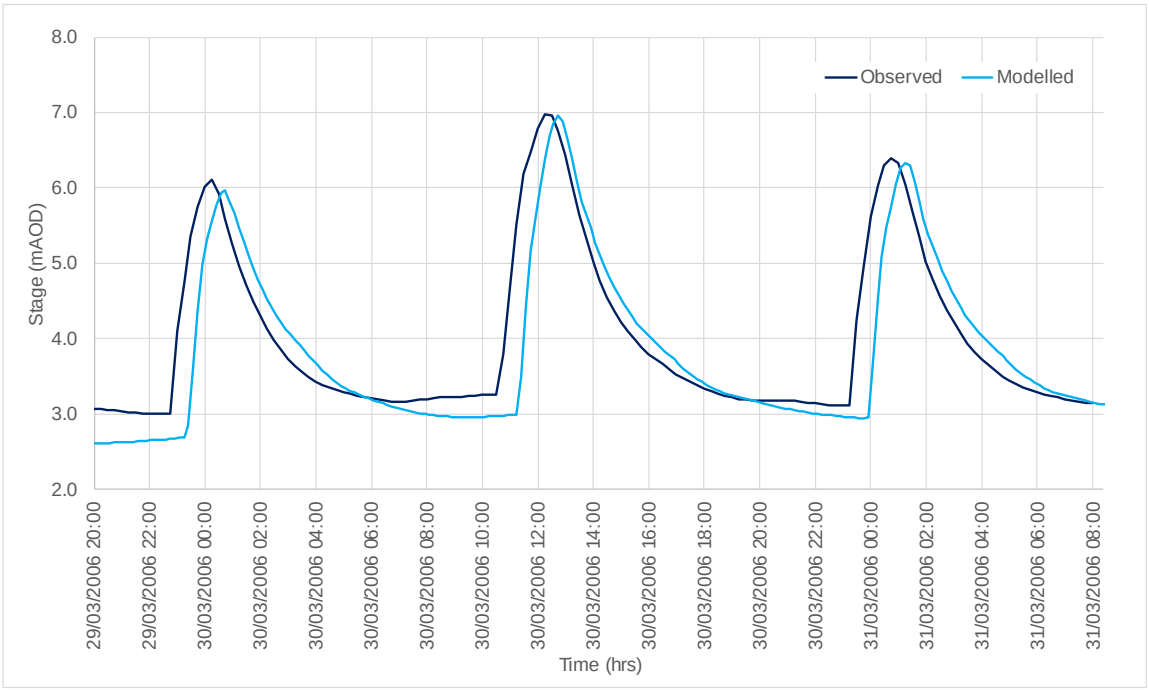
For the March 2006 high tide event, the model was shown to closely represent the observed tidal peaks at both Fiddler's Ferry (within 0.02m) and Westy (within 0.02m) (Figure 6-1). In addition, the general hydrograph shape closely matches the observed hydrograph shape at Fiddler's Ferry. However, at Westy, there is a more erratic hydrograph shape that is not echoed closely in the model. This is likely to be because the modelled channel geometry (in one dimension) is a simplified version of the Mersey Estuary bed and does not fully represent the geometry of sandbars and mudflats.

There is slight lag between the observed and modelled peak water levels at both the Fiddlers Ferry and Westy gauge locations. This lag does not affect peak levels or hydrograph shape and therefore will not affect flood extents and is not significant enough to affect the provision of flood warning. More testing would be needed to fully understand the cause of this delay. However, the lag is consistent between the two gauges and therefore the cause is very likely to originate downstream of Fiddlers Ferry.

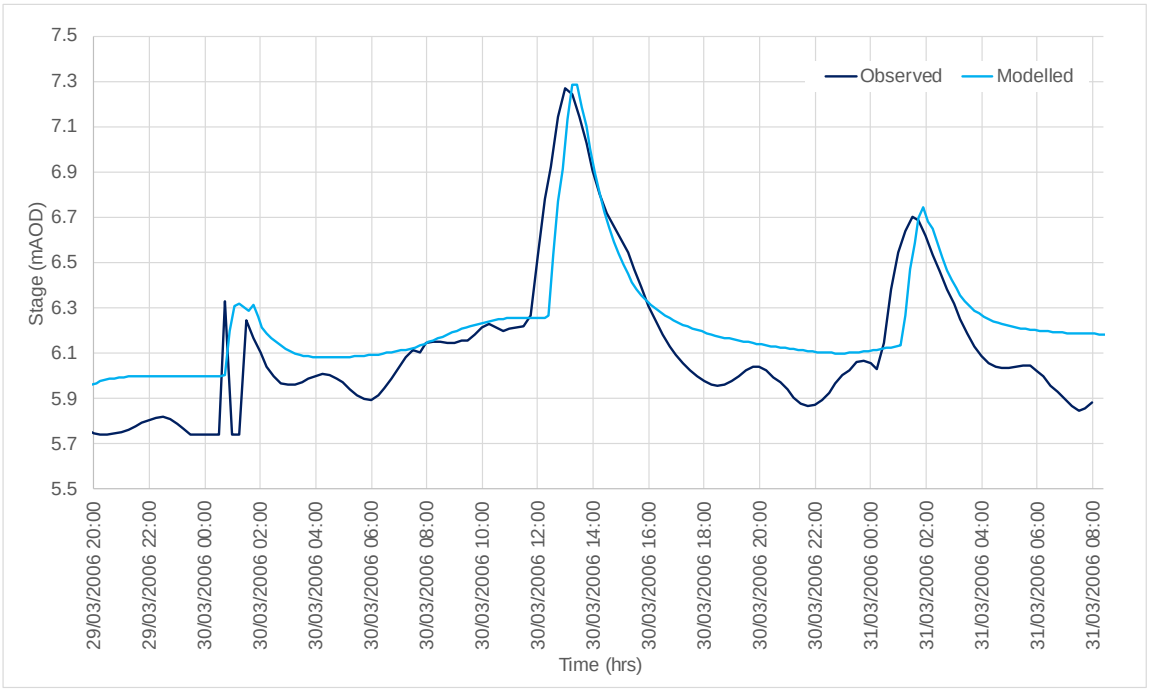
The model was determined to closely reflect the observed data. No changes were made to the model to further improve this.

Figure 6-1: Representation of observed hydrometric data for March 2006 high tide event in the Mersey Estuary model

Fiddler's Ferry:



Westy:



6.4 Sensibility

Sensibility checking of hydraulic model results is valuable to determine, using any available flood related information, whether or not the model is performing within bounds of acceptability. This might be particularly useful in the event where there is insufficient or inappropriate hydrometric data available for calibration. Such flood related information may include recorded flood history, any available hydrometric data, performance of defences, anecdotal evidence of flood frequency or recorded origin or extent of flooding.

6.4.1 Tidal flooding - historic flood events

6.4.1.1 26-27 February 1990:

- JBA was supplied drawn flood outlines for Ditton Brook at the Golden Triangle industrial estate for the February 1990 tidal flood event. The onset of flooding on Ditton Brook from a tidal event occurs in the area of the Golden Triangle Industrial Estate at the 1% AEP. However, the extent of the recorded flooding from 1990 does not occur until the 0.5-0.1% AEP. The supplied AIMS data indicate that the Ditton Brook defences were constructed in 1994. Therefore, the defended outlines cannot be verified against the supplied historical information. However, undefended flood risk at Ditton Brook from the Mersey Estuary echoes the drawn 1990 flood outline at the 5% AEP. The undefended flood outlines therefore appear sensible.
- JBA was also supplied drawn flood outlines at Howley and Warrington for the February 1990 tidal flood event in the area of Victoria Park. The Mersey Estuary model indicated that the onset of flooding at this location is 20% AEP magnitude. The observed flood history shows that Knutsford Road was overtopped in 1990. This road is now defended by the Warrington FAS. This road is modelled to overtop at the 0.5% but not the 5% undefended AEP. We could therefore suggest that the event was between 5% and 0.5% AEP. The peak water level for this event at Westy was 6.987mAOD. This corresponds to a modelled event between 4% and 3.33% AEP.
- JBA was also supplied drawn flood outlines for Moss Side (overtopping of Moore Flood Bank) for the February 1990 tidal flood event. The modelled onset of flooding at the Moore Flood Bank was modelled to be 50% AEP as a result of embankment overtopping. However, overtopping in the Mersey Estuary model does not occur in the locations indicated on the observed flood extent drawing and the general extent is much smaller. This area is a land fill site and the access routes in the present day aerial imagery are not marked on the historic flood extent mapping. It is possible that ground levels have changed since 1990, which would likely affect flood extent and the location of overtopping. Beyond the landfill site, in a downstream direction, the Mersey Estuary model is accurately capturing locations of overtopping. The 5% AEP flood extent would appear to be too large to correspond to this recorded flood outline. However, the loss of available storage upstream at the landfill site may have resulted in this flood risk relocating locally to this location.

6.4.1.2 5 December 2013:

- JBA was supplied a photograph of overtopping from the Mersey Estuary into the MSC at Ince. This location is shown to be start overtopping at the 50% AEP (<1m^3/sec peak flow) and the LIDAR would indicate that this area might overtop regularly. However, this serves to highlight that detailed bank top survey could be beneficial in this area. The level at Fiddler's Ferry on the 5 December 2013 was 7.317mAOD. This would correspond to a modelled event between 0.5% and 0.1%AEP. At the 0.5% AEP, peak flow volume over the modelled spill in this location is almost 170m^3/sec. Without additional information regarding overtopping along this reach, it is difficult to ascertain whether this is sensible.
- JBA was also supplied a drawn historic flood outline for Hale Village for the December 2013 tidal event. It is unknown from the supplied data how frequently that flooding is experienced in this location. However, flooding in 2013 was extensive and did block access roads without reported impact to property. The extent of the modelled flood area does not increase significantly with event magnitude, but does suggest frequent overtopping of the access roads. Flooding in the area of Hale is modelled to occur from the Mersey Estuary at the 50% AEP. This is a result of the low lying floodplains of Ram's Brook and Decoy Marsh. Town Lane and Carr Lane are shown to flood at the 50% AEP. There is, however, no risk shown to property at the 0.1% AEP. Although the 0.5% AEP outline corresponds well to the distance of the recorded flood outline from properties on Hoghton Road, there is little change between overall extent in this location at extreme AEPs. Therefore, the modelled flood extents in this location appear to be sensible, but further understanding of the frequency of access problems due to flooding would be beneficial.

- The data provided shows Fiddler's Ferry to have flooded in 2013 following localised exceedance of the channel side level. Flooding at Fiddler's Ferry does not occur under any of the defended model scenarios. However, the site is shown to be at risk at the 20% AEP with the defence removed. It is understood that this defence was installed only recently and it is not believed to have flooded since. It may be that this defence has not yet been tested to understand more clearly if the model is performing sensibly in this area. Under conditions of climate change, the defence is overtopped at the 0.5% AEP.

6.4.2 Tidal flooding - hydrometric records

Comparison of peak modelled water levels at Liverpool to the gauged record at Gladstone Dock indicates that the downstream boundary data (from the CFBD) are sensible for modelling extreme tide on the Mersey Estuary. For a 17-year tide record, the 50% AEP peak design water level is exceeded 12 times, the 20% AEP four times, and the 10% AEP twice. The tidal level results from the Mersey Estuary model are therefore highly likely to be suitable.

The downstream Folly Gates water level record includes data from 1999 - 2015, although there is a step change in 2004 (where it is assumed the datum was changed); the data recorded prior to this point are not comparable with model results due to the lower values recorded. However, the datum level of this gauge is unknown and has been assumed to be 0mAOD. From 2004 - 2015 there are no exceedances of the modelled 50% AEP tidal water level of 6.4mAOD, although two events came within <0.1m of this and a further three within 0.2m. This implies the model is overestimating tidal levels in comparison with the gauge record, although it is likely this is by no more than 0.2m. There are two likely causes of this; either the datum of the gauge record is not 0mAOD or the dimensions of water levels are attenuated between the Mersey (probably correct) and the gauge. The most likely structure to cause this attenuation would be the MSC siphon - the internal geometry of which is assumed. However, the gauge record for fluvial floods, when the greatest volumes will be conveying beneath the siphon, does not show a clearly shallower falling limb, implying water levels in the lower Gowy fall with the tide and are not controlled by the capacity of the syphon. For that reason, it is suggested the datum is the greatest source of uncertainty and it is recommended that this is surveyed.

6.4.3 Ditton Brook fluvial flooding

No fluvial flooding information is available for Ditton Brook within the study reach.

6.4.4 River Gowy fluvial flooding

No extensive flood history information is available for the River Gowy, besides some local anecdotal knowledge on the frequency of flooding and the presence of an upstream flow gauge (Bridge Trafford) and a water level gauge at Folly Gates.

Firstly, anecdotal evidence suggests that the floodplains upstream of Stanlow are frequently inundated whilst the oil refinery is not known to have ever flooded from a fluvial driven event. The model replicated these trends, with some floodplain inundation predicted at the 50% AEP event, but no flooding predicted at Stanlow until the 5% event.

The Folly Gates upstream level gauge, which records river levels upstream of the tidal outfall, was queried against the model results. In the supplied period of record (1999 - 2015) the 50% AEP flood event was exceeded nine times. However, further analysis suggested an erroneous gauge reading for four of these events, when a spike was observed in the record and no corresponding high flow or level was recorded at Bridge Trafford. Another event was discounted due to a large tidal signature in the downstream Folly Gates gauge, implying this was not a fluvial-only event (as simulated by the model). The modelled 50% AEP level of 4.7mAOD was exceeded four times in 16 years; this may imply a slight underestimate in modelled levels but without a longer gauge record this is difficult to conclude with confidence. No reliable events in the record exceeded either the 10% or 1% AEP water levels.

6.5 Sensitivity

6.5.1 Overview

The Environment Agency's National Flood Risk Mapping (NFRM) Specification states the need for sensitivity testing, in order to quantify some of the uncertainty inherent within hydraulic modelling.

All sensitivity runs were completed using the 1% AEP flows (unless otherwise specified). These runs included:

- Sensitivity to hydraulic roughness: channel and floodplain Manning's *n* values were adjusted globally by ±20%.
- Sensitivity to flow (Ditton Brook and River Gowy only): Inflows were altered by ±20% to investigate the degree of uncertainty resulting from the various methods of hydrological analyses.
- Sensitivity to downstream boundary condition:
    - Mersey Estuary: downstream HT boundary adjusted by ±0.2m for the 0.5% AEP and 0.5m for the 0.1% AEP. These were the pre-determined confidence intervals for the CFBD.
    - Ditton Brook and River Gowy: downstream MHWS tide adjusted by ±0.5m.
- Sensitivity to wave overtopping (Mersey Estuary only): average percentage change for several overtopping profiles that resulted from varying the wave overtopping model parameters.

6.5.2 Sensitivity results

A detailed and quantified description of model sensitivity is included in the hydraulic modelling check file in Appendix B. However, in summary, sensitivity testing on the three Mersey Estuary models has demonstrated the following:

- Although the lower reaches of the Mersey Estuary model demonstrate low sensitivity to hydraulic roughness, the implications are amplified with distance upstream as the tidal wave propagates towards Warrington. The model has been shown, however, to calibrate well to observed peak tide levels and accurately represent the propagation of the upstream tidal wave. We would therefore infer that the roughness values used in this model are appropriate.
- The Ditton Brook model demonstrates low overall sensitivity to hydraulic roughness, particularly in reaches of the model downstream of the railway nearest to Halewood. The dominant controls on water levels in this reach include the capacity of any downstream structure and the tide.
- The River Gowy model also demonstrates low overall sensitivity to hydraulic roughness, particularly in the area of the Stanlow oil refinery where there is the potential for the highest financial, environmental and social losses.
- The 1D model results indicates a greater sensitivity in the Ditton Brook model to flow in the upper reaches of the study domain, away from the dominance of a tidal signature downstream of Hale Road.
- The 1D level results from the downstream boundary sensitivity tests on the Mersey Estuary indicate that the impact of adjustment to tide level reduce with distance upstream. However, the scale of the impact may be locally variable depending on channel geometry (i.e. where the channel narrows or widens).
- Apart from the immediate downstream reach of the Ditton Brook model, the implications of adjustment to the downstream boundary condition reduces rapidly with distance upstream. For Ditton Brook, this is the impact of in-channel structures and channel gradient.
- On the River Gowy, Folly Gates marks a transition between a tidally and fluvially dominant watercourse. However, the sensitivity test illustrates that there is an impact, caused by tide locking of the fluvial inflows, on water levels at Stanlow. In-channel water levels will rise or fall in relation to a rise or fall in tide level. The impact is reduced by Folly Gates, but the impact of tide locking creates an upstream tidal signature that is sensitive to changes in downstream boundary condition.

- A detailed discussion of the wave overtopping modelling (including sensitivity testing) is included in Appendix C.

7 Model results

7.1 Model runs

The range of scenarios simulated by the updated hydraulic modelling are outlined in Table 7-1.

Table 7-1: Model runs

Mersey Estuary	Design	• 50%, 20%, 10%, 5%, 4%, 3.33%, 2%, 1.33%, 1%, 0.5% and 0.1% AEPs.
	Climate change	• 0.5% AEP extreme tidal downstream boundary plus climate change 2065 (+0.292m). Upstream fluvial inflows remained the same as design. • 0.5% AEP extreme tidal downstream boundary plus climate change 2115 (+0.699m). Upstream fluvial inflows remained the same as design.
	Undefended	• 5%, 0.5% and 0.1% AEPs.
Ditton Brook	Design	• 50%, 20%, 10%, 5%, 4%, 3.33%, 2%, 1.33%, 1%, 0.5% and 0.1% AEPs.
	Climate change	• 1% AEP extreme fluvial upstream boundaries plus climate change 2065 (+25% flow). Also included was the equivalent increase in MHWS tide. • 1% AEP extreme fluvial upstream boundaries plus climate change 2115 (+30% flow). Also included was the equivalent increase in MHWS tide.
	Undefended	• 5%, 1% and 0.1% AEPs.
River Gowy	Design	• 50%, 20%, 10%, 5%, 4%, 3.33%, 2%, 1.33%, 1%, 0.5% and 0.1% AEPs.
	Climate change	• 1% AEP extreme fluvial upstream boundaries plus climate change 2065 (+25% flow). Also included was the equivalent increase in MHWS tide. • 1% AEP extreme fluvial upstream boundaries plus climate change 2115 (+30% flow). Also included was the equivalent increase in MHWS tide.
	Undefended	• 5%, 1% and 0.1% AEPs.
	Tidal flood risk	• All Mersey Estuary design, climate change and undefended scenarios using Mersey Estuary results extracted from nearest node to the River Gowy downstream boundary (MEST_20500). • Also included is any overtopping from the MSC; water levels in the canal also extracted from the Mersey Estuary model.

The following sections provide a general description of the flood outlines produced by the hydraulic models, highlighting locations of significant inundation or defence overtopping and where there are properties at risk. The supplied digital flood outlines, grids and animations which accompany this report should be referred to for a more detailed view of areas at risk.

It is important to note that plotting flood outlines on a map can imply a degree of certainty and accuracy. The flood outlines are in fact somewhat uncertain with the largest source of uncertainty likely to be the boundary conditions (inflows and tidal boundaries) used to run the hydraulic model; a full list of the assumptions and limitations associated with this project are provided in chapter 8.

7.2 Flood outlines

The flood outlines for majority of the study domains were mapped using the maximum depth results from TUFLOW. A 1D mapping approach was only used in two instances only; on the MSC upstream of the River Weaver confluence and upstream of the M56 on the Gowy for the 0.1% AEP event. This approach utilised peak modelled water levels at each ISIS node to project a water surface across the floodplain using extended cross sections.

7.2.1 Mersey Estuary

A selection of tidal flood outlines for the design, climate change and undefended scenarios for the Mersey Estuary are shown in Figure 7-1, Figure 7-2 and Figure 7-3 respectively.

7.2.1.1 Design

The defended model runs illustrated the level of tidal flood risk posed to different reaches of the Mersey Estuary under present day conditions. Significant receptors on the Mersey Estuary included the Stanlow oil refinery, industrial works sites throughout Warrington (e.g. Bank Quay), the Tranmere oil terminal (an extension of the Stanlow oil refinery), Liverpool and Birkenhead.

In the defended scenario, areas affected by flooding were predominantly uninhabited floodplain (e.g. near Hale) marshland (e.g. North Marsh) or recreational land (e.g. Victoria Park, near Howley). However, the model indicated localised flood risk to industrial buildings on the left bank of the lower Ditton Brook during a tidal flood event as frequent as the 3.33% AEP because of defence overtopping.

Towards the downstream end of the Mersey Estuary, large potential receptors include Liverpool, Birkenhead and Liverpool John Lennon Airport. The airport was not shown to flood at the 0.1% AEP. Liverpool and Birkenhead, however, were affected by localised flooding adjacent to dock basins, driven directly by levels in this Estuary, and wave overtopping. Any potential flooding, even at the 0.1%, was largely confined to areas adjacent to the locks with some road access potentially restricted. Strand Street (A5036) in Liverpool was inundated at the 2% AEP and overtopped at the 1% AEP.

At the 50% AEP, large areas of uninhabited low lying floodplain at Hale and Moss Side were shown to be at risk of some inundation. Flooding at Moss Side was a result of embankment (formal defence) overtopping at one low spot. However, flood extent was localised to the immediate floodplain. Although the flooding near to Hale may cause access or egress disruption (Town Lane and Carr Lane), there are no implications for damage to property.

However, the 50% AEP flood event also presented a risk to railway infrastructure at Bank Quay. This railway line[3] (running east-west) does not appear to be a main line (such as that running north-south through Warrington Bank Quay station) for use by the public, more of an industrial asset, but may experience disruption during a 50% AEP tidal event. This flooding occurred because of overtopping of high ground (as designated in AIMS) at the upstream end of the flood wall defence. This might be appropriate and the 1D results indicated that the high ground in the LIDAR should be overtopped. However, there was no flood history to sensibility check this. It might have been that the LIDAR was not capturing ground levels accurately or that the site was in fact defended by a scheme that was not included in the Mersey Estuary model. Either way, the understanding of flood risk at this site could benefit from more site-specific flood history or surveyed defence or bank crest information.

At the 10% AEP, industrial buildings on the inside of a large River Mersey meander near to Bank Quay were shown to be at risk from inundation because of high ground overtopping. There was a risk from events as frequent as 2% AEP of serious hindrance to access and egress.

At the 0.1% AEP, properties on Whittle Brook were affected following propagation of the tidal flood wave through the open culvert and on Sankey Brook as the tide propagated along the tributary.

How Stanlow is affected by tidal flood events on the south side of the MSC, downstream of Folly Gates, was mapped using the higher resolution 2D domain in the River Gowy model and included in the Mersey Estuary flood outlines. This included any overtopping flood volume from the MSC. Please see section 7.2.4. The Tranmere oil terminal, however, experienced some

[3] Engineers Line Reference and mileage: SDJ2 12.0440 to SDJ2 11.000

wave overtopping at the 10% AEP but significant inundation of the lower lying area of the site at the 0.1% AEP only.

7.2.1.2 Climate change

The climate change scenarios for the Mersey Estuary highlighted the potential flood risk implications for large receptors on the Mersey Estuary at the 0.5% AEP with allowances for sea level rise over the next 50 and 100 years.

These receptors included Stanlow oil refinery, the industrial sites near Keckwick, Ince Marshes and adjacent works, Liverpool and industrial sites on the left and right banks of Ditton Brook. Climate change scenario modelling indicated that each of these areas may be subject to a considerable future increase in the level of flood risk. However, only at the 0.5% plus 2115 climate change were more central areas of Liverpool affected. The airport was not shown to be at flood risk at the 0.5% 2115 climate change scenario.

7.2.1.3 Undefended

For the purposes of flood zone mapping, undefended modelling of the Mersey Estuary was also carried out. The undefended scenario indicated the flood risk benefit of the Warrington FAS, of the River Weaver defensive embankments and of the Mersey Estuary defensive embankments protecting the large water treatments works near Sankey Brook.

The benefit of the River Gowy tidal defences is described in section 7.2.4.

Figure 7-1: Mersey Estuary defended flood outlines

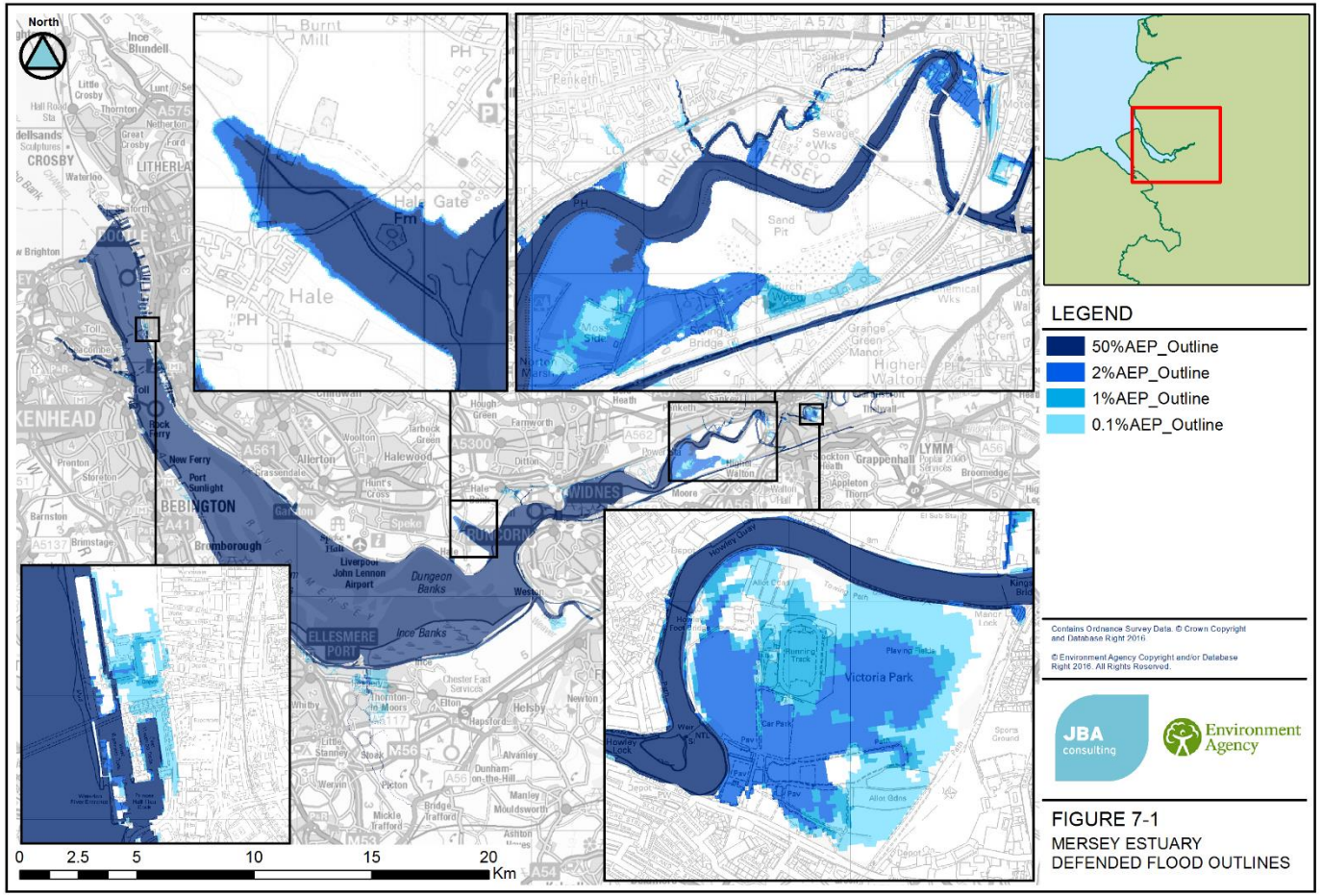


Figure 7-2: Mersey Estuary climate change flood outlines

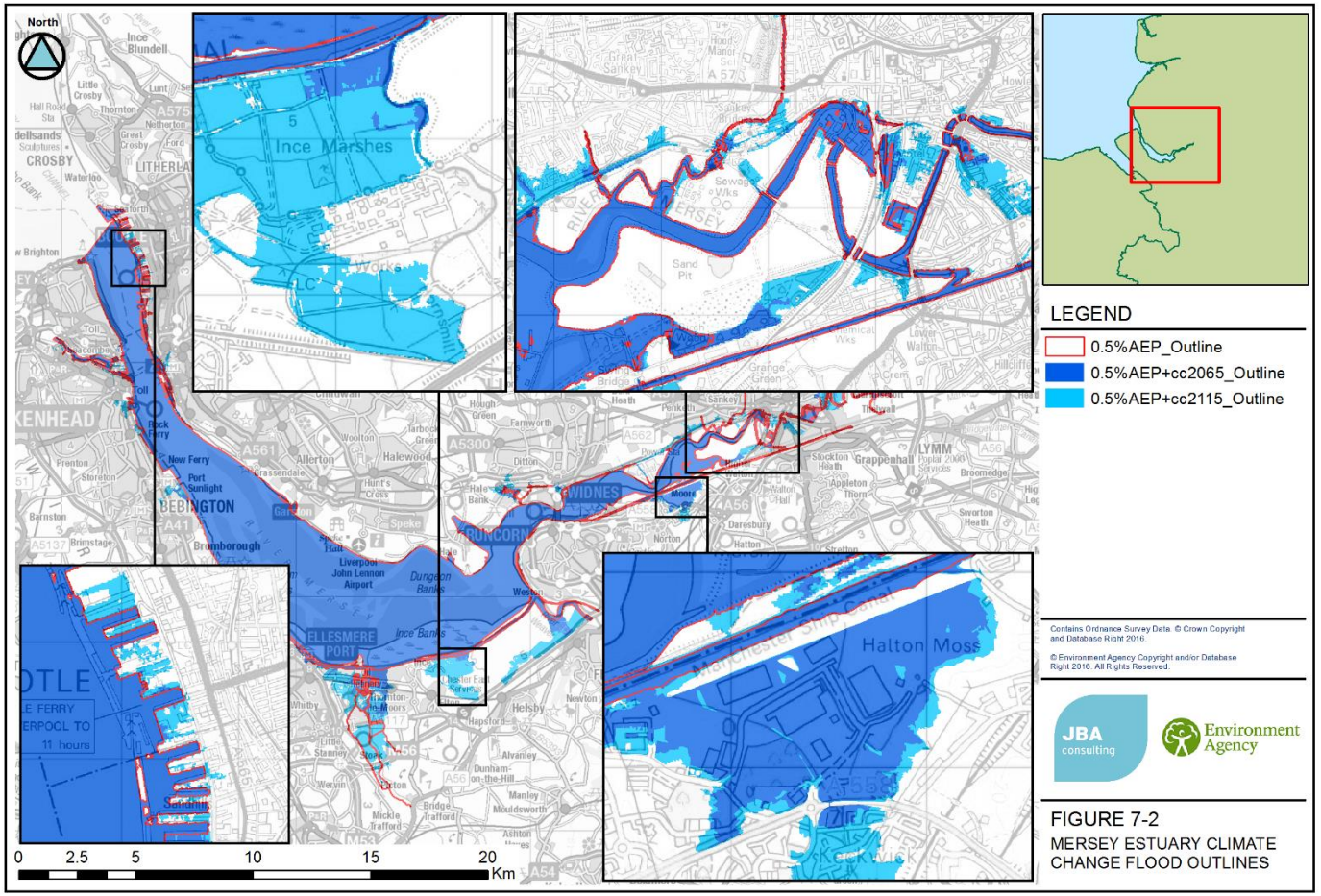
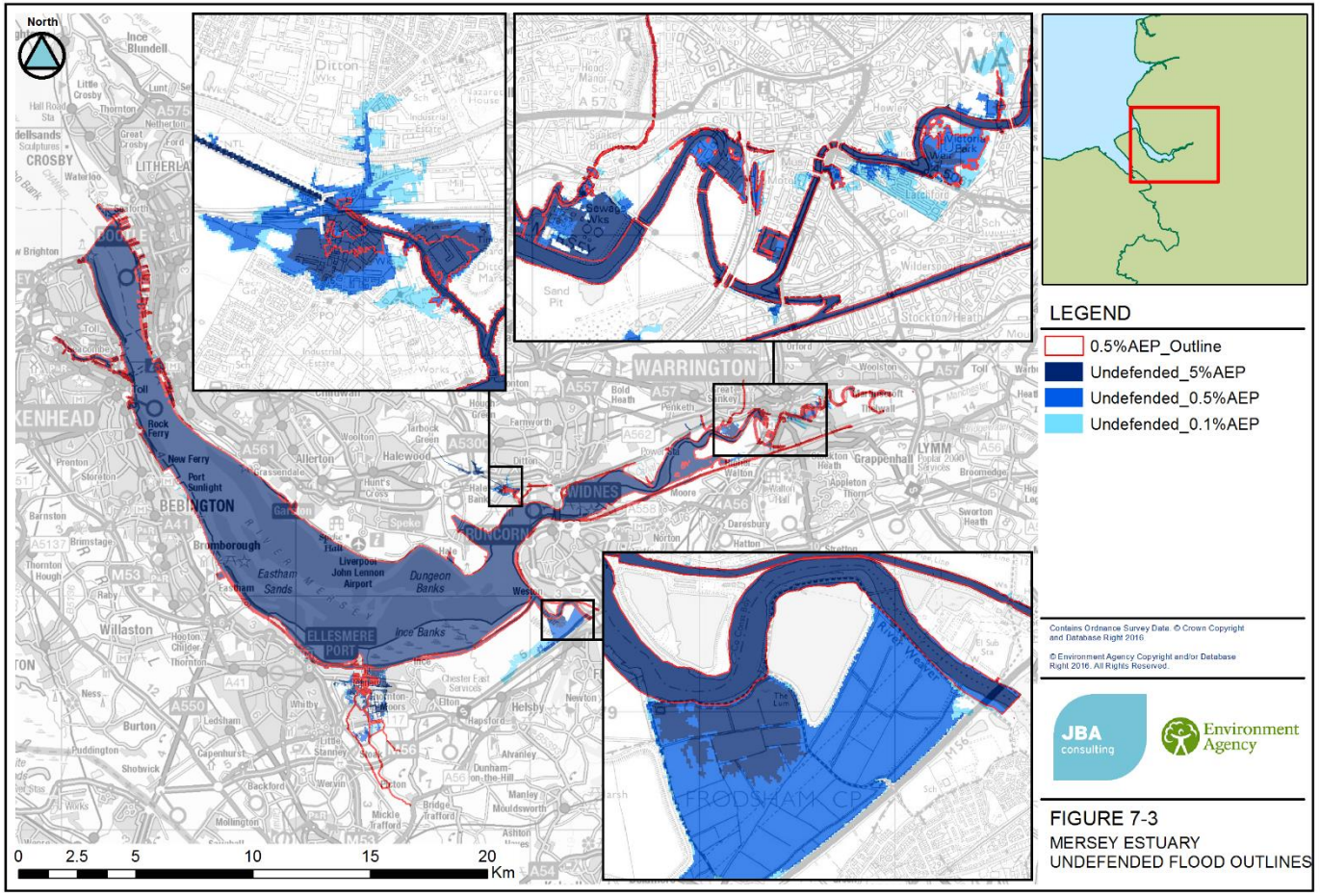


Figure 7-3: Mersey Estuary undefended flood outlines



7.2.2 Ditton Brook

A selection of fluvial flood outlines for the design, climate change and undefended scenarios for Ditton Brook are shown in Figure 7-4, Figure 7-5 and Figure 7-6 respectively.

7.2.2.1 Design

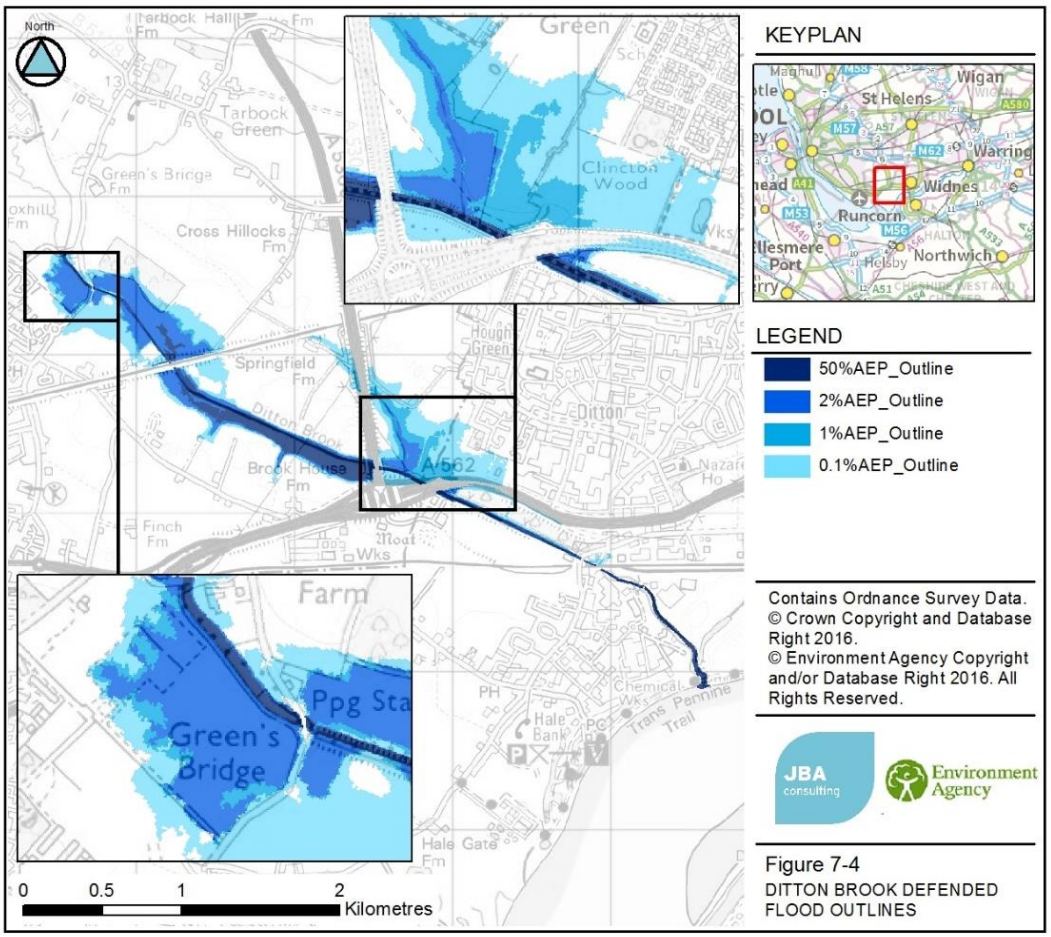
The Ditton Brook design runs (Figure 7-4) highlighted areas at risk of flooding within the lower catchment. Property in Halewood on Cherrywood Avenue (recently constructed housing is at flood risk from a 3.33% AEP fluvial flood event because of water backing up behind Green's Bridge, exceeding channel capacity and spilling over the right bank of Ditton Brook. Properties were surrounded only at the 0.1% AEP. Please note that this housing development is not illustrated on the supplied OS mapping. Flood water in this area of floodplain is retained behind Greensbridge Lane, which is raised approximately 0.5m above the adjacent floodplain and is overtopped only at the 0.1% AEP.

Elsewhere in the model domain, properties were at very low flood risk (less than 0.1% AEP), except for an isolated group of industrial/warehouse structures on the left bank of Ditton Brook at Hale Road, following overtopping of the channel defences, and a small group of properties at the junction of Cartbridge Lane and Greensbridge Lane which were affected at the 0.1% AEP only as water levels exceed the higher channel side on the left bank of Ditton Brook.

Generally, however, downstream of Speke Road, there was little indicated fluvial flood risk. Flooding upstream of this point occurs as a result of water backing up behind the Speke Road bridge and resulting in overtopping of the left bank defence. Downstream of this point, Ditton Brook is bounded by high ground on the left and right banks and there was a large modelled drop in peak water surface level between the Ditton Junction bridge and downstream Ditton Road bridge (approximately 0.5m at the 1% AEP). This was a product of the constriction on upstream flow because of the capacity of the Ditton Junction bridge. This reduction was sufficient so that even extreme AEPs were largely retained within the high channel sides.

Downstream of Ditton Junction Bridge, however, indicates a much clearer tidal signature in the 1D model. Between the Ditton Road bridge and the point of discharge into the Mersey Estuary there is a 2.5m drop in bed level and there are no structures that cause a constriction on flow. At the 1% AEP, there is a further (approximately) 0.5m drop in peak water levels across the Ditton Road bridge. Flood risk along this lower reach of Ditton Brook is therefore tidally dominated.

Figure 7-4: Ditton Brook defended flood outlines

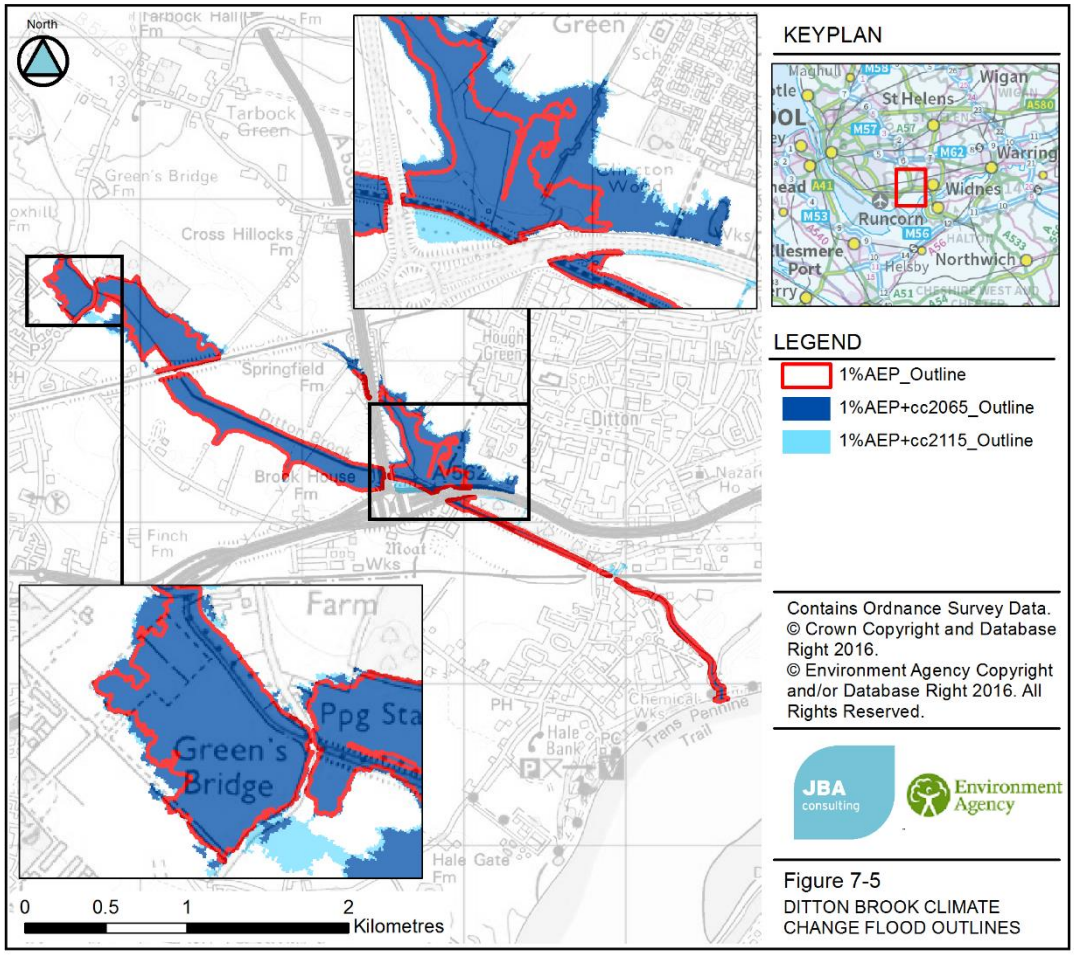


7.2.2.2 Climate change

The Ditton Brook climate change scenarios (Figure 7-5) highlight how the overall flood extent for a 1% AEP event may change over the next 50 or 100-years. Halewood and the Lightoaks Drive housing development was indicated to be at additional flood risk with properties at risk of being surrounded and potential access roads (Cherrywood Avenue) cut off by flood water. Similarly, Greensbridge Lane was overtopped at the 1% AEP 2065 scenario, which might cause further difficulties with access and/or egress.

In addition, properties at Hale Road and the corner of Cartbridge Lane and Greensbridge Lane were also affected under the 2065 scenario. In summary, several properties may see a change in the onset of flooding of flood risk from 0.1%AEP or less to 1% AEP.

Figure 7-5: Ditton Brook climate change flood outlines



7.2.2.3 Undefended

For the purposes of flood zone mapping and flood warning areas, undefended modelling of Ditton Brook was also conducted. Upstream of the Knowsley Expressway, there was a marginal reduction in overall flood extent with flood defences removed. However, this was offset by a significant increase in flood extent downstream of this road bridge.

Properties at particular risk included industrial buildings off Turnall Road on the north side of the A562, the Golden Triangle Industrial Estate near Hale Road and Ditton Junction Bridge. However, the significant risk to property was only evident at the 0.1% undefended AEP. At more frequent AEPs, increased levels of flood risk were largely contained to areas of uninhabited floodplain.

Figure 7-7 illustrates the difference between the modelled undefended 1% and 0.1% AEP scenarios and the existing Flood Zone mapping supplied by the Environment Agency. It is noticeable that the modelled undefended outlines were generally notably smaller. The only exception was where the new 1D-2D approach could highlight the flow route beneath the Knowsley Expressway and illustrate how the flood level may propagate along the tributary.

Figure 7-6: Ditton Brook undefended flood outlines

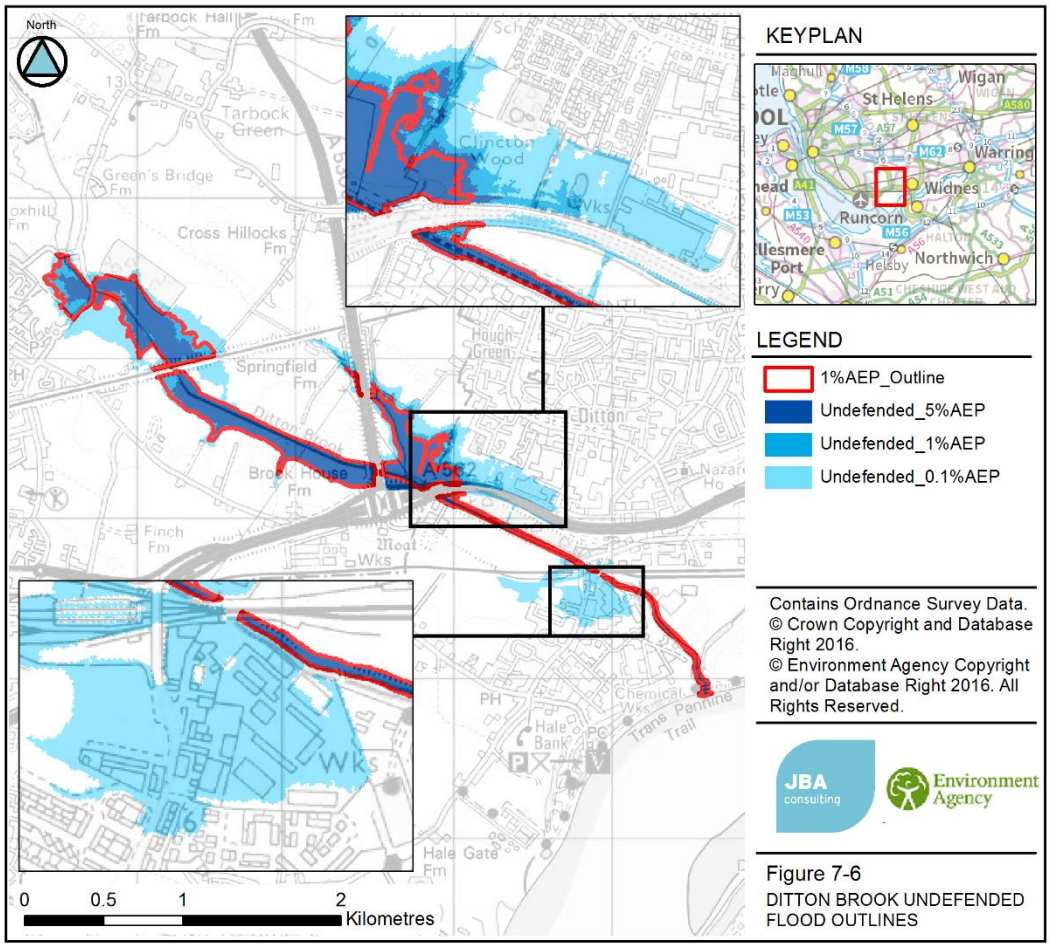
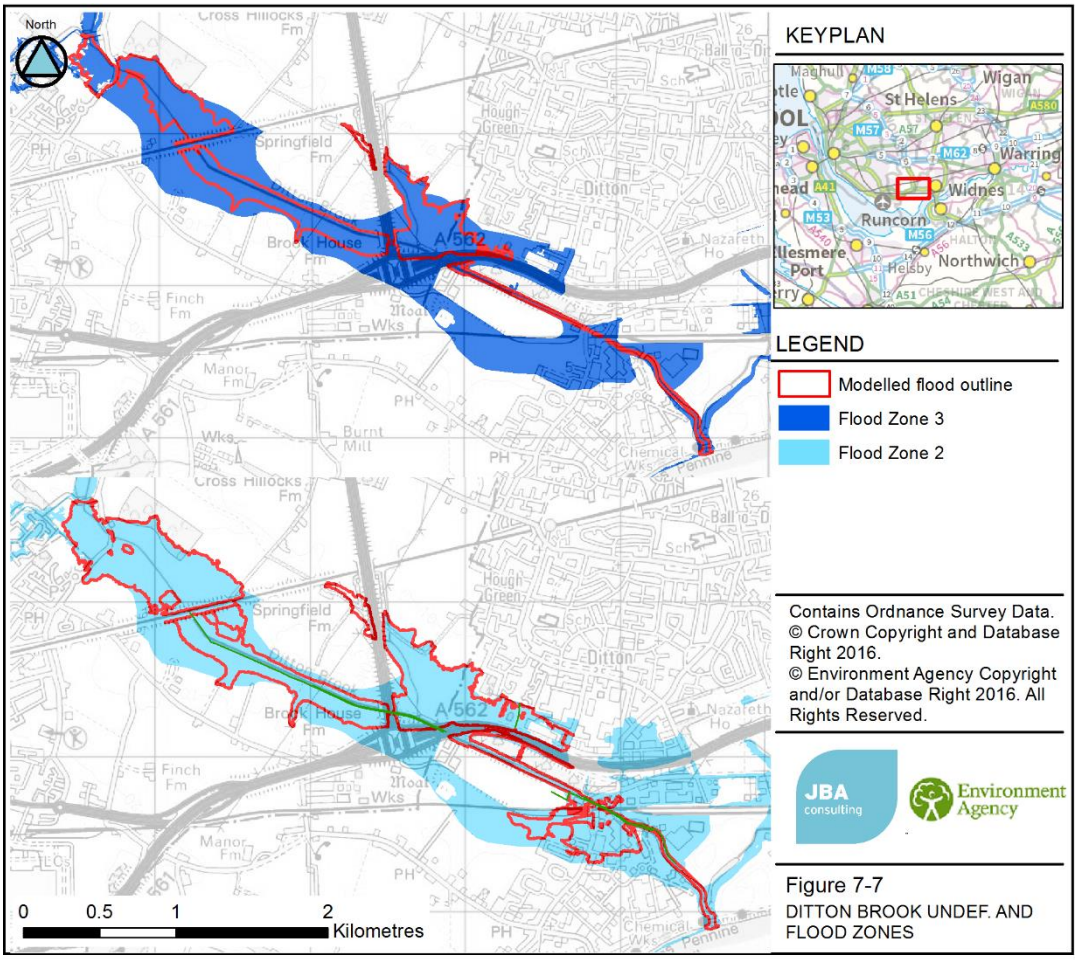


Figure 7-7: Ditton Brook comparison to existing flood zones



7.2.3 River Gowy (fluvial)

A selection of fluvial flood outlines for the design, climate change and undefended scenarios for the River Gowy are shown in Figure 7-8, Figure 7-9 and Figure 7-10 respectively.

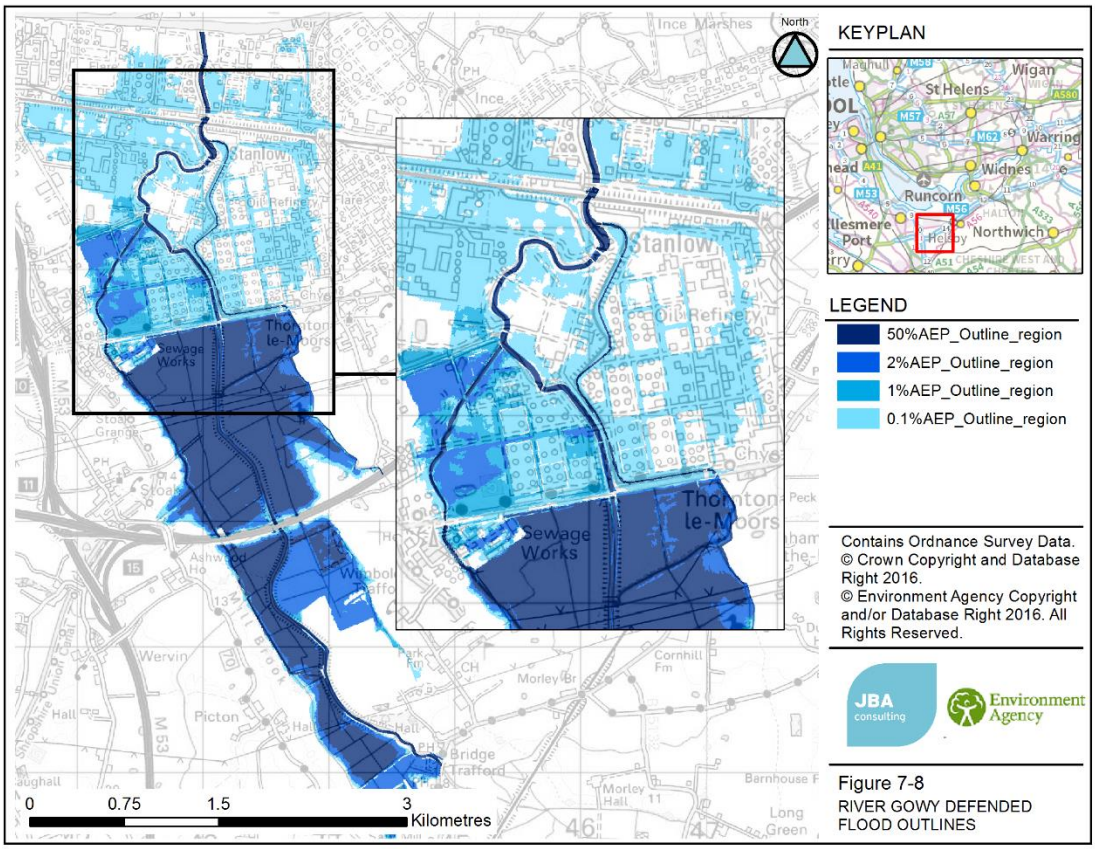
7.2.3.1 Design

The most obvious characteristic of fluvial flooding on the River Gowy is the extensive inundation seen at relatively low magnitude events, such as the 50% AEP. Much of the floodplain upstream of Stanlow oil refinery is inundated, including the left bank upstream of the M56 embankment. This onset of flooding agrees with the anecdotal evidence provided during the site visit, stating that these areas are regularly flooded. Flood extents in these areas are not shown to significantly increase at rare AEPs once the low-lying floodplain is inundated. The sides of the valley are relatively steep; hence a greater depth of water does not result in a significantly greater flood extent. The M56 embankment was not predicted to overtop at any modelled event.

The first flooding seen on the Stanlow site occurs at the 5% AEP event, with a small flooded area along Mill Brook. Significant inundation does not occur until the 2% AEP and the predicted outlines increase across the site for the rarer AEPs. Flooding is isolated to the Mill Brook area at the 1% event, although by the 0.1% water reaches the right bank floodplain and areas close to the railway embankment. These areas are protected up until this event by the flood defences in the area; this is demonstrated by the change in flood extents in the undefended scenarios.

Much of the flooding seen at Stanlow oil refinery is caused by the combination of a large fluvial flow and the inhibited discharge into the Mersey Estuary due to the high tide. Water therefore banks up behind the Folly Gates tidal outfall and spills from the river channels into the floodplains.

Figure 7-8: River Gowy defended flood outlines (fluvial)

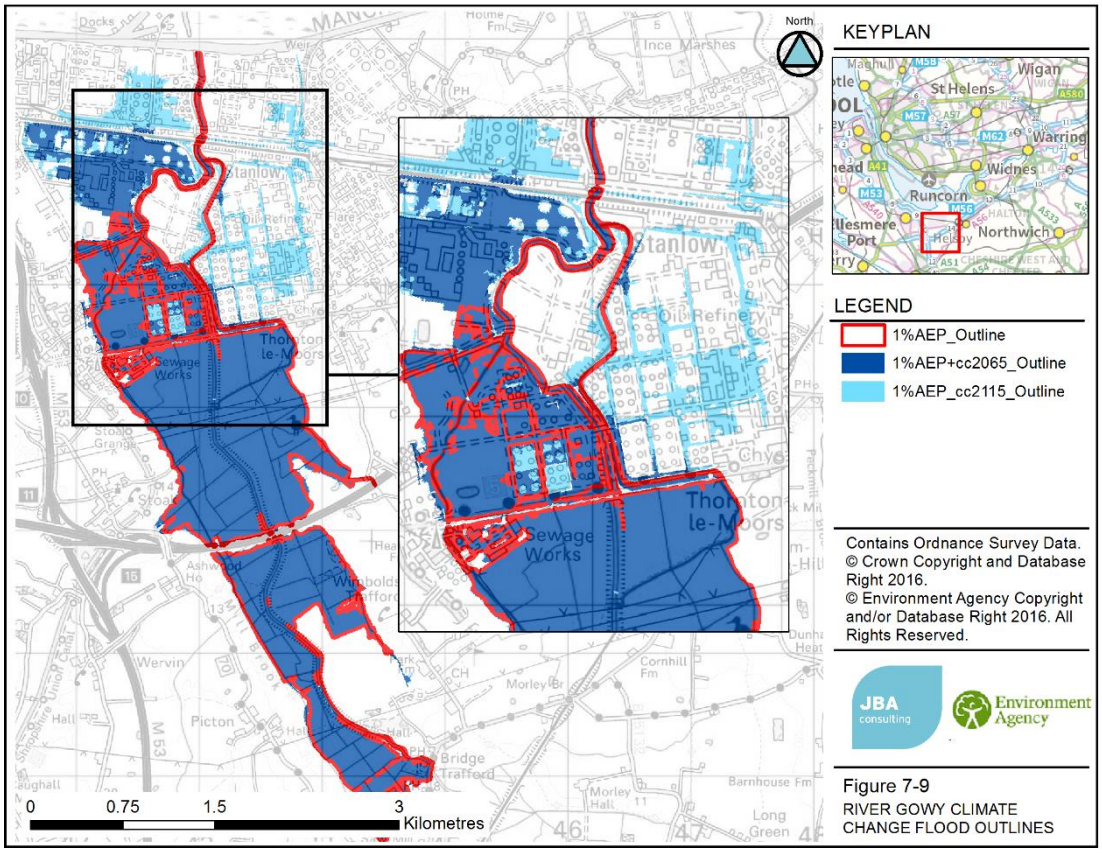


7.2.3.2 Climate change

Both climate change scenarios have a significant impact on the River Gowy fluvial flood outlines. Increasing 1% AEP flows using the 2065 climate change allowance (in combination with an increased MHWS tide) results in flooding on the left bank of the Gowy within Stanlow oil refinery; this is due to channel capacity exceedance and an additional period of tide locking. Upstream of the site flood outlines are relatively unchanged.

The flood extent is increased further if the predicted 2115 flow and tidal allowances are used, with water spilling from the right bank of Thornton Brook and conveying along access roads within the refinery. Flooding extends north of the railway embankment on both the left and right banks of the Gowy, conveying through bridges beneath the railway.

Figure 7-9: River Gowy climate change flood outlines (fluvial)



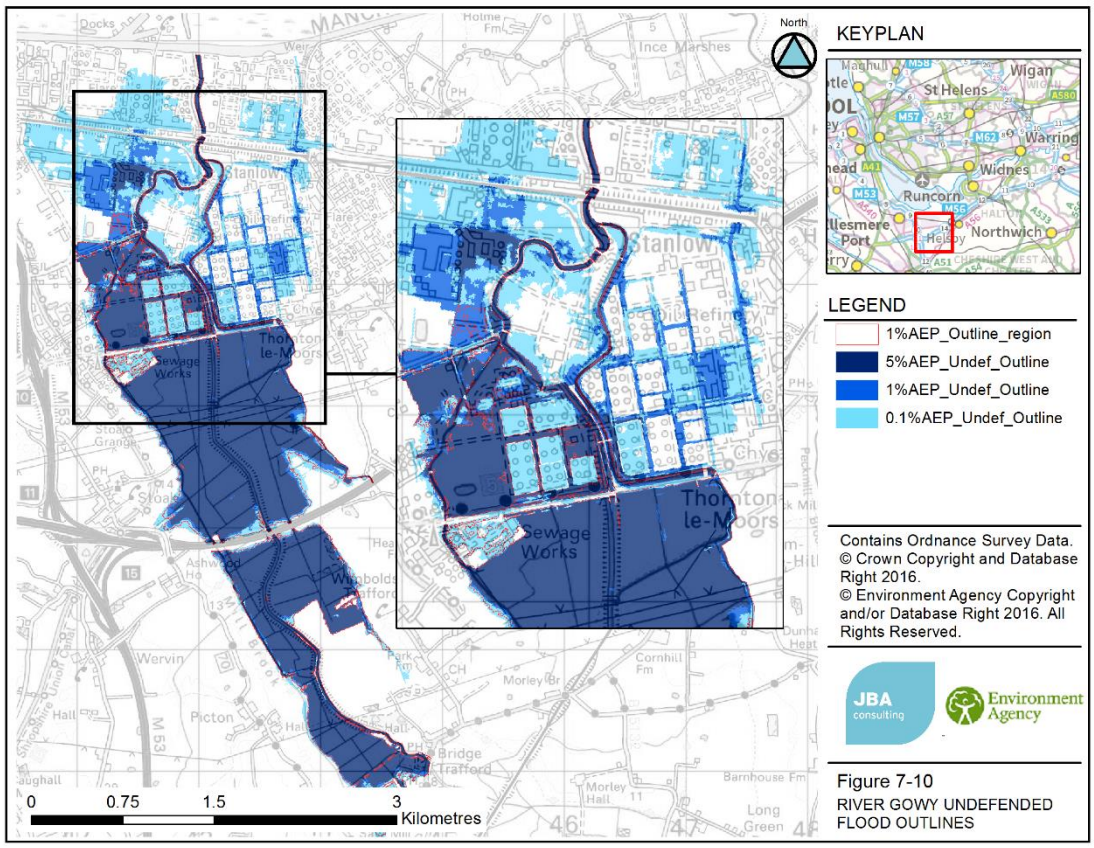
7.2.3.3 Undefended

The impact of defence removal on the River Gowy varies with fluvial AEP event. Upstream of the M56 the 5% AEP sees a slight increase in flood outlines when defences are removed as more water can spill into the floodplain. However, this is not the case for either the 1% or 0.1% AEP events during which the floodplain is full and deep water is ponded behind the M56 embankment; with such large maximum depths of water on the floodplain the defences have no influence.

Between the M56 and Stanlow there is little difference between the defended and undefended outlines of any modelled flood event. Water spills onto the floodplain even in the defended 50% event and therefore the defences do not work to reduce flood extents at the more extreme 5%, 1% and 0.1% floods (although water depths are increased). The only difference of note it at the 1% event on the right bank of Mill Brook (immediately upstream of the A5117 embankment). In the undefended scenario, water does not spill from Mill Brook into the sewage treatment works, whilst it does at the 1% defended. The reason is due to the extra upstream storage afforded by the defence removal.

Stanlow oil refinery sees significantly greater flooding from Mill Brook in the undefended 5% event, with water spilling east and north from the channel and surrounding the storage tanks located here. Additional areas are also inundated at the 1% event near Mill Brook, but more strikingly the right bank Thornton Brook floodplain becomes inundated, whilst remaining dry in defended runs. This is predominantly due to the removal of flood defences between the River Gowy and Thornton Beck, allowing greater transfer of water into the smaller channel. Removal of the right bank Thornton Brook defences creates a flow path into the refinery. Both defended and undefended outlines are similar for the 0.1% AEP.

Figure 7-10: River Gowy undefended flood outlines (fluvial)



7.2.4 River Gowy (tidal)

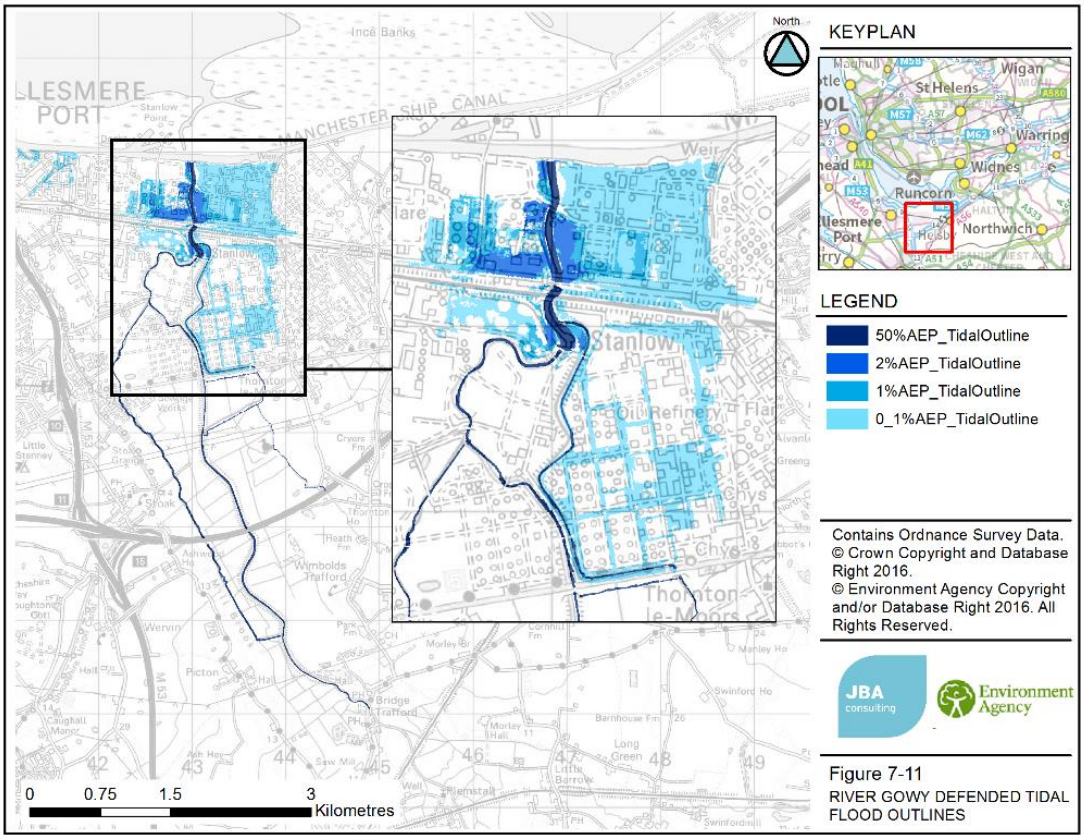
7.2.4.1 Design

No flooding is predicted at regularly occurring events such as the 50% and 20% AEP tidal floods, due to retention of water behind tidal embankments; this is consistent with the local flood history. A small area of the Gowy right bank is inundated by overtopping water at the 10% through to the 3.33% events but no major flooding is experienced until the 1% event.

The 1% AEP event sees some flooding to the north of the railway embankment (see Figure 7-11) due to overtopping of the lower Gowy tidal defences. Flood waters in this event remain relatively well constrained, with only a small area of the oil refinery inundated. During more extreme tidal floods on the Mersey Estuary, greater inundation of the River Gowy floodplains is predicted, with overtopping of the Gowy combining with overtopping on the left bank of the MSC at the 0.5% and 0.1% floods. Once the right bank area to the north of the railway line is submerged, water can convey southward through a bridge below the railway, flooding the roads to the east of Thornton Brook as it flows through Stanlow.

In no modelled design event does tidal flooding extend beyond the A5117 road embankment, or onto the Gowy left bank upstream of the railway.

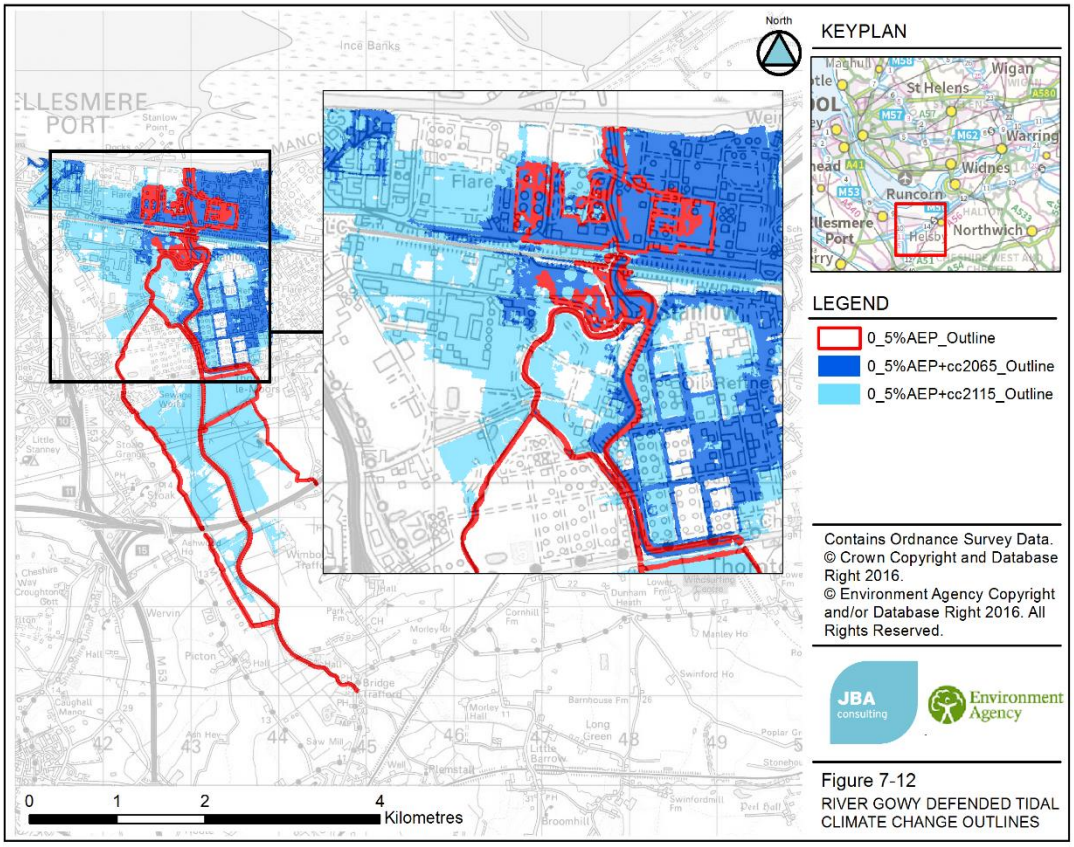
Figure 7-11: River Gowy defended flood outlines (tidal)



7.2.4.2 Climate change

Application of climate change allowances to the 0.5% AEP tidal levels results in significantly greater inundation at Stanlow as shown in Figure 7-12 compared to the design flood outlines. Use of 2065 allowances bring the 0.5% outlines broadly in line with the design 0.1% outline, with much of the right bank of the Gowy flooded. If the more extreme 2115 values are adopted flooding is predicted as far upstream as the M56 embankment, and greater areas of the oil refinery are also inundated, including the left bank around the Mill Brook confluence. This is both due to the greater water levels and the inhibited discharge from the Gowy system in the estuary during low tides.

Figure 7-12: River Gowy climate change flood outlines (tidal)

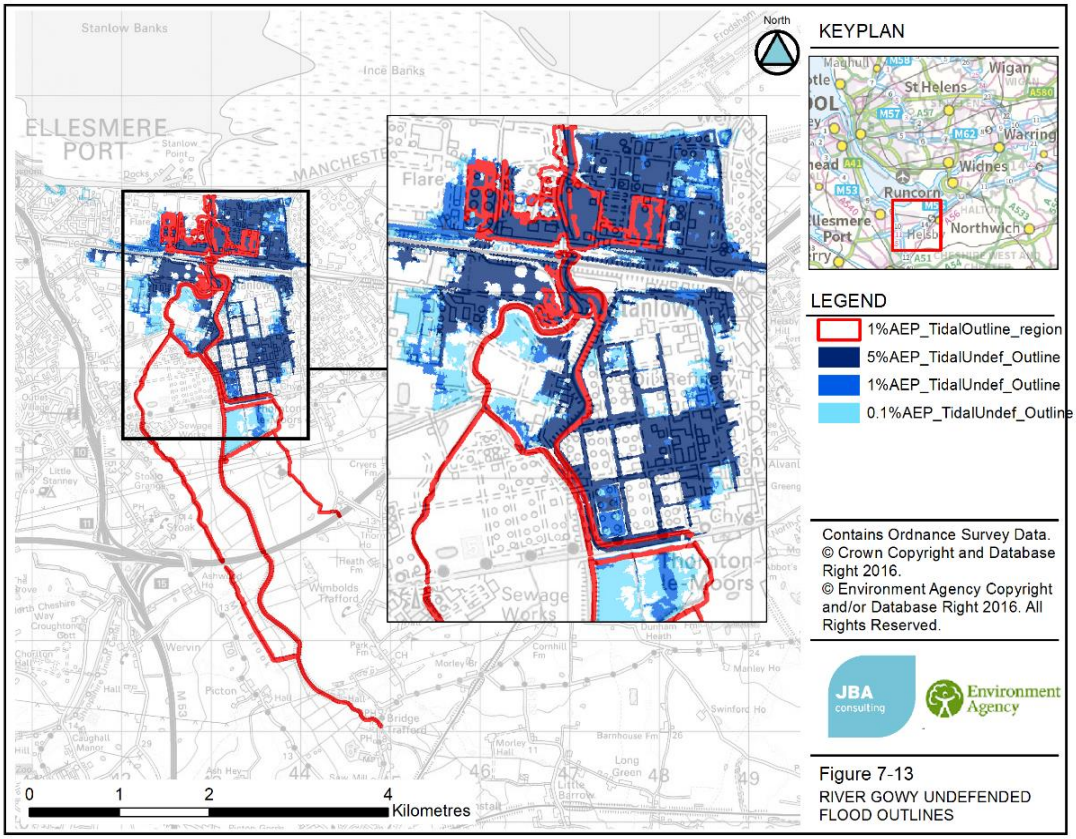


7.2.4.3 Undefended

Figure 7-13 shows the flood extents associated with an undefended River Gowy tidal scenario for a variety of events. For the purposes of this simulation the fluvial defences (upstream of Folly Gates) remain in place, as does the MSC (which is not classed as an official defence asset but acts as a defacto defence structure).

Significant overtopping is predicted at the 5% AEP event with much of the right bank between the MSC and railway embankment flooded; water also was shown to flow southwards towards Thornton Brook along the roads which serve the oil refinery. At the rarer events flood outlines extend beyond the A5117 embankment, although are not predicted to reach the M56. The main area of flooding is located on the right bank of the River Gowy; extents are limited on the left bank due to the presence of higher land behind the removed defences, particularly upstream of the railway.

Figure 7-13: River Gowy undefended flood outlines (tidal)



7.3 Flood hazard mapping

Flood hazard has been calculated using the following formula[4]:

$$HR = D(V+0.5) + DF$$

Where: HR = (flood) hazard rating
D = depth of flooding (m)
V = velocity of floodwater (m/sec)
DF = debris factor.

The conservative land use option to this function has also been applied in TUFLOW. This is consistent with other studies across the coastal modelling package.

The flood hazard mapping presented in this section therefore highlights where flooding may (as a function of depth, velocity and debris) cause damage to property, hinder productivity or potentially cause a threat to life.

This equates to a hazard rating as indicated in Table 7-2.

Table 7-2: Hazard rating descriptions

Hazard rating	Degree of flood hazard	Description
0 - 0.75	Low	Caution "Flood zone with shallow flowing water or deep standing water"

4 Defra/Environment Agency (2006): Flood and Coastal Defence R&D Programme. FD2321 - The Flood Risks to People Methodology. http://randd.defra.gov.uk/Document.aspx?Document=FD2321_3436_TRP.pdf

0.75 -1.25	Moderate	Dangerous for some (i.e. children) “Danger: Flood zone with deep or fast flowing water”
1.25 - 2.50	Significant	Dangerous for most people “Danger: flood zone with deep fast flowing water”
2.50 +	Extreme	Dangerous for all “Extreme danger: flood zone with deep fast flowing water”

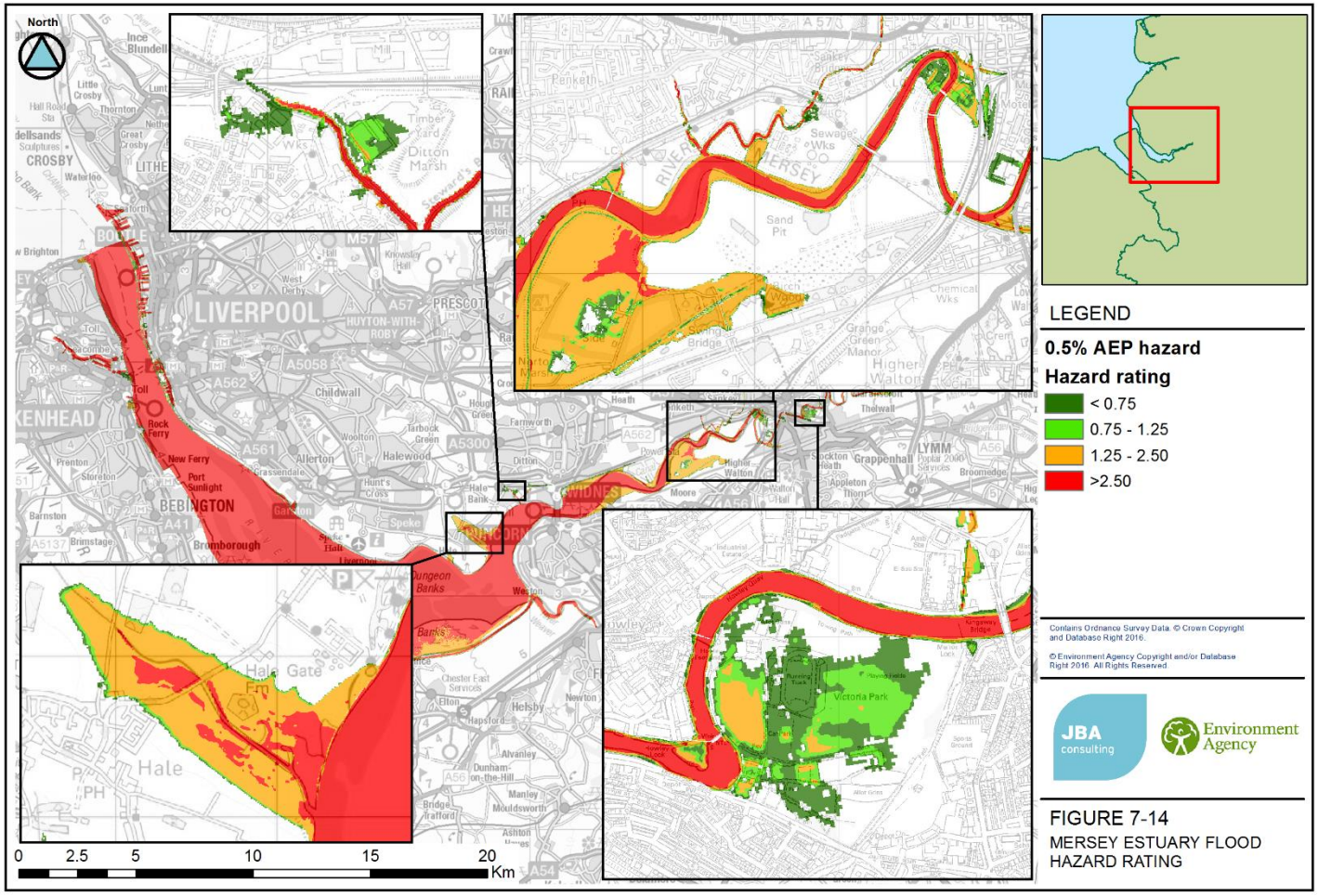
7.3.1 Mersey Estuary

The flood hazard rating mapping for the Mersey Estuary is shown in Figure 7-14.

The flood hazard rating for the Mersey Estuary study domain at the 0.5% AEP is generally significant (or dangerous for most people) across the floodplain domain. Largely, this rating occurs in locations where flooding is therefore unlikely to cause potential damage to property or harm to people. Where flood hazard is designated as extreme, this only regards the Mersey Estuary, River Mersey and tributary watercourses where the tidal flood wave propagates upstream.

Where a risk to people or property might be posed, such as Victoria Park near Howley in Warrington or the industrial sites in the Ditton Brook catchment, the hazard rating is generally moderate to low. Only where there are depressions in the floodplain is the hazard rating higher. The localised flooding near dock basins in Liverpool and Birkenhead is also calculated to be moderate to low. However, in certain locations, such as at the two access roads to the north east of Hale, the model indicates a potentially dangerous risk (or significant hazard) in the event that these roads are overtopped.

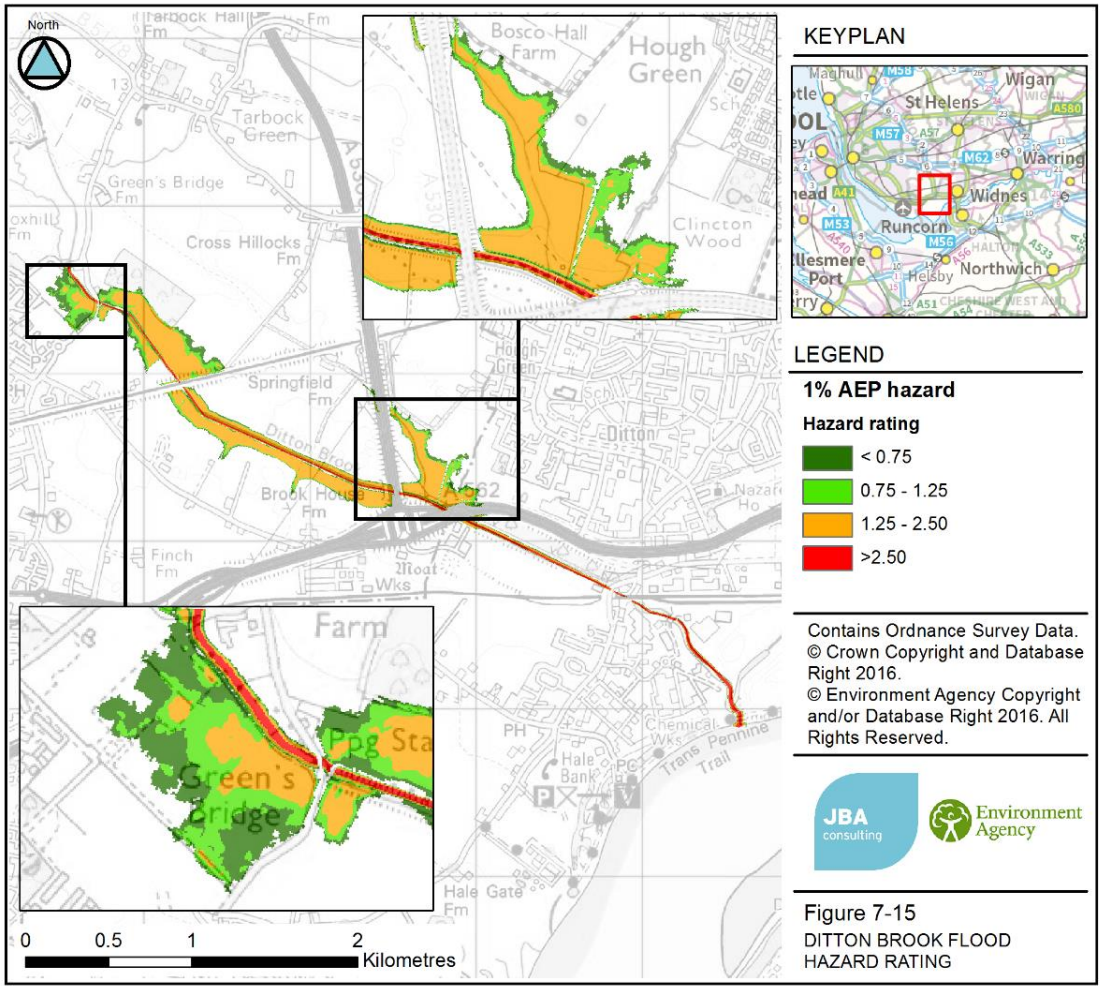
Figure 7-14: Mersey Estuary flood hazard rating mapping



7.3.2 Ditton Brook

The flood hazard rating mapping for Ditton Brook is shown in Figure 7-15.

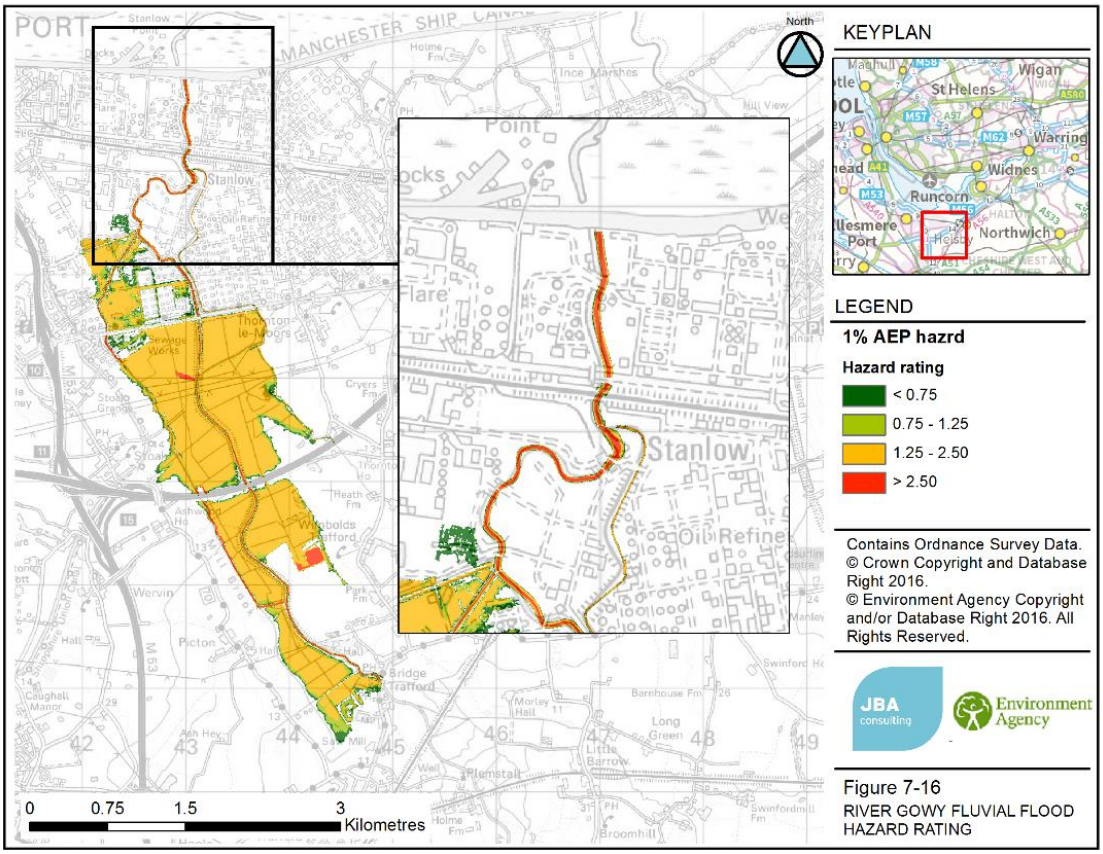
Figure 7-15: Ditton Brook flood hazard rating mapping



Flood hazard rating for the Ditton Brook study domain at the 1% AEP is modelled to be largely significant. However, this rating occurs in locations where flooding is unlikely to cause potential damage to property or harm to people. Where a risk to people or property might be posed, the hazard rating is moderate to low.

7.3.3 River Gowy

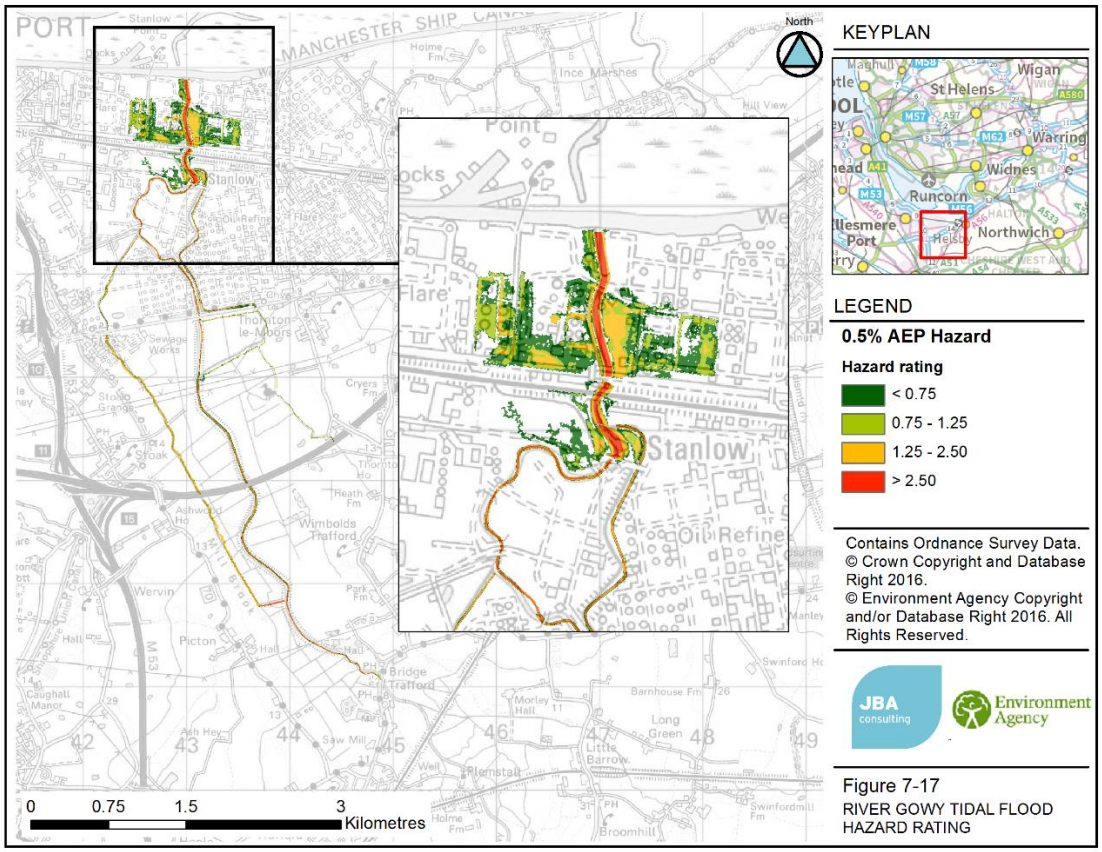
Figure 7-16: River Gowy flood hazard rating mapping (fluvial)



The 1% AEP fluvial flood risk hazard on the River Gowy shows little extreme risk; where this occurs, it is contained in channel and in areas of deep water on the floodplain. One area on the left bank is associated with water spilling from the channel, resulting in high velocities. Most of the study reach is classified as significant risk owing to the deep-water ponding behind large embankments. Some significant risk exists in Stanlow.

Tidal hazard (for the 0.5% AEP) is shown below. Flood outlines for this event are less extensive than for a fluvial event and the propagation of water over the floodplain is relatively slow and shallow, hence the lower hazard ratings shown below. Once again, the highest values are found in-channel, where deeper water is present.

Figure 7-17: River Gowy flood hazard rating mapping (tidal)



7.4 Updates to flood warning areas

Flood warning review and any required updates will be completed following approval of draft modelled flood outlines and water levels.

8 Assumptions and limitations

8.1 Summary

Any hydrological and hydraulic modelling project is subject to a number of assumptions and limitations, given the nature of attempting to represent a real-world scenario with the use of equations and computer software. This chapter seeks to describe any assumptions made during, or notable limitations of, the Mersey Estuary study.

8.2 Assumptions

A number of standard assumptions, such as the subjective approach of deriving Manning's *n* values, are inherent across all modelling projects. Sensitivity tests were undertaken on each of the Mersey Estuary models to demonstrate the impact of these standard uncertainties and are discussed in Appendix B.

Important assumptions made during the project include the following:

- The internal geometry/capacity of the River Gowy siphon is unknown. It is known that the penstocks of the siphon are fixed in the positions that were surveyed and the structures have been modelled as simple orifice units. Nothing is known about the internal geometry of capacity of the structures. However, the approach is appropriate based on the information available for the study. There is no evidence in the hydrometric record to indicate that the siphon provides a significant control on levels and upstream levels fall at approximately the same rate as the tide. An update of the hydraulic models would, however, be recommended if internal survey data of the Gowy siphon became available.
- There was uncertainty regarding the operation and geometry of lock gates and docks during flood events. A simplified uni-directional culvert, initial dock water level and estimated dock depth was applied to represent a realistic 'normal' water surface and the gates being forced open at high tide. It is not known for certain how these gates and water levels in the docks would respond during a flood event. However, although this is an assumption, water levels in the docks and any resulting flooding will be dependent almost exclusively on water levels in the Mersey Estuary. Therefore, the assumptions made for this study will have negligible impact on overall flood extent.
- Similarly, a simplified representation of the Eastham Lock gate (the MSC model downstream boundary) has also been used in the Mersey Estuary model. This was extracted from the existing MSC model and included as a fixed spill representing the top of the lock gates. There are two sets of gates that would prevent flow through the gate compound in either direction. Therefore, this simplified approach was appropriate for this model.
- The operating procedure of the Weaver sluice gates for an extreme tidal scenario has been assumed to regulate the water surface in the MSC to the statutory level (4.15mAOD) and to promote model stability.
- The spills between the MSC and Mersey Estuary from the existing MSC model are based on old (approximately 1960s) survey data. We would recommend acquiring defence crest survey for the 1D spill boundary between the Mersey Estuary and MSC. We have reviewed that these defences will be overtopped before the landward side of the ship canal in Runcorn is affected by tidal flooding. However, an updated understanding of defence crests in this area would be beneficial to the model.
- The defence crests for the Warrington FAS have been extracted from the Warrington FRA model and are not based on as-built drawings or surveyed crest levels. The levels in the FRA model are a constant design crest level modelled as a z-line and does not account for low spots that may be in the defence. Defence crest survey for new Warrington FAS would be beneficial to confirm that they are correctly represented in the model.

8.3 Limitations

In addition, study-specific limitations include the following:

- Representation of the Mersey Estuary 1D as opposed to 2D. The size of the lower Mersey Estuary and the availability of bathymetric data might have warranted the representation of this reach in 2D. However, with agreement from the client, this study opted for a 1D approach. The main reasons were the lack of hydrometric data to either

demonstrate the need for, or calibrate, a 2D model, the additional run time and data production of a 2D model, and that fact that the previous 1D model calibrated well at the available gauges. A 1D approach is a simplified version of the Mersey Estuary bed and does not represent features such as sand bars or mud flats in any degree of detail. However, as shown in this study, a 1D approach performs well and reliably recreates observed flood peaks and flood volumes.

- The data for Gladstone Dock (the local data for Liverpool) was not included in the CFBD. It was therefore assumed that this data set remains suitable for tidal flood modelling for the Mersey Estuary despite the omission of these data.
- A 10m 2D model for the Mersey Estuary meant that detailed flow routes (e.g. between buildings or along roads) were probably not represented. Similarly, due to this coarse 2D resolution, a generalised approach to floodplain Manning's *n* was also applied. A site-specific study, for example an FRA, might require greater localised detail.
- Where it was deemed to be cost effective, topographic survey was commissioned for the purposes of this study. Where survey has not been recently carried out, we have utilised the best available information. These data sources have included previous modelling studies, LIDAR and aerial photography (used to approximate structure dimensions). In some cases, the dimensions of structures in the 2D domain have been estimated from one or a variety of these sources and each have associated limitations. It is thought, however, that this method improves accuracy in the 2D domain as opposed to just utilising the supplied filtered LIDAR/DTM.
- There was an absence of reliable gauged data for both Ditton Brook and the River Gowy. Ditton Brook is ungauged and it was deemed that the record at Bridge Trafford was unreliable (particularly for large events) and therefore not suitable to enable calibration of the River Gowy model. Access to hydrometric data for calibration on Ditton Brook and appropriate data to enable calibration at Bridge Trafford would be beneficial for future studies.
- There was an absence of information regarding several surface water outfalls in Stanlow. Some, but not all, were observed in the recent topographic survey and some geometries were available in the Thornton Brook Improvements documentation. It was unknown whether any of the other structures were flapped and how much each structure contributes to in-channel water levels during a flood event. It was assumed that water cannot exit the River Gowy through these structures before the embankment defences are overtopped. In addition, a generalised hydrological approach was applied and any possible inflow at these locations is included.
- The models produced for this study do not represent any joint probability and the assumption is that they are independent. The fluvial or tidal boundaries developed for the respective fluvial or tidal model have been derived as independent extreme events with a suitable opposing boundary condition (i.e. MHWS tide or mean flow data).

9 Conclusions and recommendations

The Environment Agency commissioned JBA in 2015 to carry out a hydraulic modelling study to improve the understanding of flood risk along the Mersey Estuary, Ditton Brook and River Gowy.

The project was required to deliver a new tidally focused 1D-2D model of the River Mersey, utilising a number of existing models, and update the existing 1D fluvial models of Ditton Brook and the River Gowy, using recently acquired topographic survey, and produce a new 2D domain for each. The results from the Mersey Estuary model will be used to improve the tidal downstream boundaries for any future tributary models and all models built for this study will be used to update the existing flood risk mapping.

9.1 Conclusions

The following conclusions can be drawn from the flood risk analysis of the Mersey Estuary, Ditton Brook and River Gowy models:

- The Mersey Estuary design runs illustrated the flood risk benefit resulting from the new Warrington FAS. The residential areas behind the two phases of the scheme are modelled to be at risk of flooding of less than 0.1% AEP. The undefended scenarios indicate a risk of flooding from an event of 0.5% AEP.
- Comparison of the updated and existing and undefended model outlines for the Mersey Estuary indicates that Warrington and the area of Howley weir marks the boundary between predominantly tidal and fluvial flood risk.
- The Ditton Brook and Mersey Estuary models also highlighted the benefit of the Ditton Brook flood defences. The defences reduce the level of flood risk on the right bank of Ditton Brook (Golden Triangle Industrial Estate) from a 5% AEP to 1% AEP for a tidal flood event and from 0.1% AEP fluvial less than 0.1% AEP for a fluvial event.
- The Ditton Brook hydraulic modelling illustrated the dominance of fluvial and tidal flood risk between the upper and lower reaches, respectively, of the study domain. The lower watercourse was dominated by a tidal signature and extreme fluvial flows had a low impact on flood risk. Nearly all the modelled extreme flood flows are retained in bank as peak water levels fall across Ditton Junction bridge and Ditton Road bridge. The location where the dominance of fluvial or tidal flood risk changes is the reach between Speke Road and Ditton Junction that is embanked by high ground. This is consistent across the design and undefended scenarios. Overtopping of the downstream flood wall defences (between Foundry Lane and Stewards Brook) occurs at the <0.1% AEP in the fluvial scenario, but properties are affected on the left bank of Ditton Brook at the 2% tidal scenario and on the right bank at the 1% AEP The upper watercourse is dominated by fluvial flows where there is a much less prominent tidal signature.
- Much of the River Gowy floodplain was inundated upstream of Stanlow at frequent fluvial flood events (>50% AEP). This regular flooding of fields tied in with anecdotal evidence gathered during the site visit. Little tidal inundation was predicted at higher AEP events due to the significant tidal defences and the impact of the Manchester Ship Canal embankment. Under present day design scenarios, localised flooding occurred downstream of the railway at Stanlow on the right bank of the River Gowy at the 10% AEP and site buildings were affected at the 3.33% AEP. Widespread inundation upstream of the railway line and Folly Gates was observed between the 1% and 0.1% AEP only. The MSC was overtopped from the Mersey Estuary at the 0.1% AEP only.
- Stanlow oil refinery was first flooded from a fluvial event at the 5% fluvial AEP, with water spilling onto the floodplain from Mill Brook. All of the flood risk here was associated with flooding from Mill Brook and inundation was localised and did not affect oil refinery buildings or access roads. At higher magnitude events, up to the 0.5% AEP, flooding in this area was confined only to access roads or undeveloped grassland. It was not until the 0.1% fluvial event that water spills from the right bank of the Gowy onto the site and embankments in the area of Mill Brook around refinery storage units were overtopped.
- In comparison to the design scenario, the River Gowy undefended 5% and 1% fluvial scenarios resulted in greater flooding at Stanlow whilst the extent at the 0.1% was similar.
- The Mersey Estuary and River Gowy tidal modelling has highlighted the potential implications of predicated future climate change, particularly for Stanlow oil refinery. Under conditions of climate change, where inundation at the 0.5% AEP is only localised downstream of the railway and in the region of Folly Gates, the potential disruption to the

remainder of Stanlow oil refinery increases dramatically. In the 2065 scenario, most access roads on the right bank of the River Gowy were flooded and in the 2115 scenario, the A5117, the main road that forms the southern boundary of the site, is overtopped.

- The railway downstream of Folly gates and the immediate area between the railway and Folly gates marks the area of transition between tidally and fluvially dominated flood risk. At the undefended 1% AEP, tidal inundation is dominant in eastern areas of the site, whereas in western areas it is fluvial inundation that is dominant.

9.2 Recommendations

In addition, it is possible to make some initial recommendations as to how these hydraulic models might be improved or current understanding developed with additional data:

- Confidence in the design flows applied to both the River Gowy and Ditton Brook is reduced because of the lack of high quality flow records. On Ditton Brook, there is currently no river level or flow monitoring; however, with only limited flood risk arising from the watercourse installation of a gauge here is unlikely to be viable. On the River Gowy the situation is very different, especially when the predicated effects of climate change are considered. Whilst there is a flow gauging station on this watercourse it is a rated station which is understood to be heavily influenced by vegetation. Improving the quality of flow gauging at this location would help to improve confidence in the predicted flood risk.
- Consider acquiring topographic/defence height survey of the defence line between the Mersey Estuary and MSC.
- Consider acquiring crest level survey for the new Warrington FAS as built to confirm that the scheme is correctly represented in the Mersey Estuary model.
- Consider acquiring local defence or topographic crest level information for the right bank section between the River Mersey and the railway beneath Warrington Bank Quay station. Although the modelling approach is based on the best available data and a consistent approach has been applied across the study domain, flood risk may be overestimated in this area.
- The MSC and linking structures to the Mersey Estuary were included to improve the representation of overtopping, water storage and water transfer along the impoundment. The operating rules for the Weaver sluice gates were assumed and the Eastham locks is a simple spill linking the MSC to the Mersey Estuary. Further modelling work could benefit from a more detailed understanding of the operating procedures of the Weaver sluice gates and how the dock gates and Eastham Lock gates would realistically behave during an extreme high tide. However, for the purposes of extreme tidal flood modelling, the approach taken here is appropriate and further information would not impact model results.
- Future work could be an opportunity to carry out further study regarding the implications of climate change at more frequent AEPs to test the design limitations of current defences at Stanlow and Warrington.
- Joint probability of tidal and fluvial events has not been considered as part of this study. Future work may consider the implications of this and a concurrent high fluvial flood event and high tide for the Mersey Estuary, Ditton Brook and River Gowy.
- Little is known about the internal dimensions or drainage capacity of the syphon under the Mersey Estuary. This information would be best established by internal survey and inspection by a specialised team.

Appendices

A Flood Estimation Handbook calculation record

B Hydraulic modelling check file

C Wave overtopping modelling

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Annex 7F2
Lower Weaver Model User Report

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Technical note:
Lower Weaver Model User Report

1. Introduction

1.1 Context

This model user report accompanies the Hynet Flood Risk Assessment (FRA). This report documents the modelling undertaken using the existing Environment Agency (EA) owned model for the Lower Weaver to provide updated flood extents and levels incorporating the latest (2021) climate change allowances for the Weaver Gowy Management Catchment[1]. Specifically, the Lower Weaver model provides an indication of fluvial flood risk to the Rocksavage Hydrogen Above Ground Infrastructure (HAGI) site (referred to hereafter as 'the site'), which forms part of the wider proposed development.

When the original model was developed, climate change allowances of 30%, 35% and 70% were applied to the 1% AEP flow, respectively, presumably in accordance with relevant guidance at the time. Whilst the 1% AEP plus climate change allowance fluvial event would typically form the design standard for fluvial flood risk in accordance with the online Planning Practice Guidance[2] (PPG), the 0.5% AEP plus climate change event has been chosen as the design event for the purposes of this FRA.

Given that the project and associated infrastructure is considered to be a Nationally Significant Infrastructure Project (NSIP), both the 'Higher Central' and 'Upper End' climate change allowances have been assessed, the latter forming a sensitivity test. The relevant climate change uplifts for the Weaver Gowy Management Catchment (covering the period to 2100) are 67% and 106%, respectively. These allowances have therefore been applied to the 0.5% AEP fluvial event.

The model software version has been updated from the original study, and hence the model has also been re-run for the existing 1% AEP event to provide a like-for-like comparison to understand any changes to modelled flood risk as a result of software version.

1.2 Purpose and methodology

The purpose and methodology steps of this modelling study are outlined below:

- Review existing EA model to satisfy that it is fit for the purpose of assessing flood risk to the site;
- Run existing EA model of the Lower Weaver with 1% AEP event with current versions of software to ensure version update does not impact on model results;
- Generate new Flood Modeller Event Data (.IED) files for the 0.5% AEP plus 67% and 106% climate change events (using multiplication factors of 1.67 and 2.06 to the existing 0.5% AEP IED files);

[1] https://environment.data.gov.uk/hydrology/climate-change-allowances/river-flow (Accessed December 2022)
[2] https://www.gov.uk/guidance/flood-risk-and-coastal-change (Accessed April 2023)

- Generate new Flood Modeller Event Files (.IEF), to output results to appropriate location on the local system, reading in the new IED file described above and existing model elements (DAT file and TCF);
- Run model using updated IEFs above, and
- Process results files for the area of interest.

1.3 Data – existing model

A data request was made to the EA in August 2022 to provide the most up to date and fit for purpose hydraulic model (and accompanying reports and results) of the Lower Weaver. The linked 1D-2D model provided by the EA was developed in Flood Modeller and TUFLOW software and completed in 2019. The supplied files and data used in this assessment are summarised in Table 1.1 below.

Table 1.1 Supplied data

Item	Description
Product 6	Model results in GIS and .csv format.
Product 7	Hydraulic model files for the Flood Modeller and TUFLOW model elements.

For the purposes of these climate change model runs, no additional data sources have been used other than the model data files supplied by the EA. The EA signed the original model off for use in flood studies in general. In addition, Section 2 details the checks undertaken as part of this study to determine the model's suitability for assessing flood risk to the proposed development site.

2. Model review

The following sub-sections review the key elements of the existing model to satisfy that the model is fit for purpose to assess flood risk to the site.

2.1 Model software

The original modelling study was carried out using Flood Modeller version 4.5, and TUFLOW version 2018-03-AE-iDP-w64. The updated model runs for this assessment have been run using Flood Modeller version 6.0 (released in 2022) and TUFLOW version 2020-10-AD (released in 2022).

The sensitivity of the model results to the updated model software versions has been assessed and detailed in Section 4.1.

2.2 Channel geometry

It is noted that the original modelling study was primarily concentrated on updating the anticipated flood risk to Wennington, Acton Bridge and Dutton Locks, upstream of the HAGI site. Whilst floodplain representation of the lower modelled reaches (and adjacent to the HAGI site) was

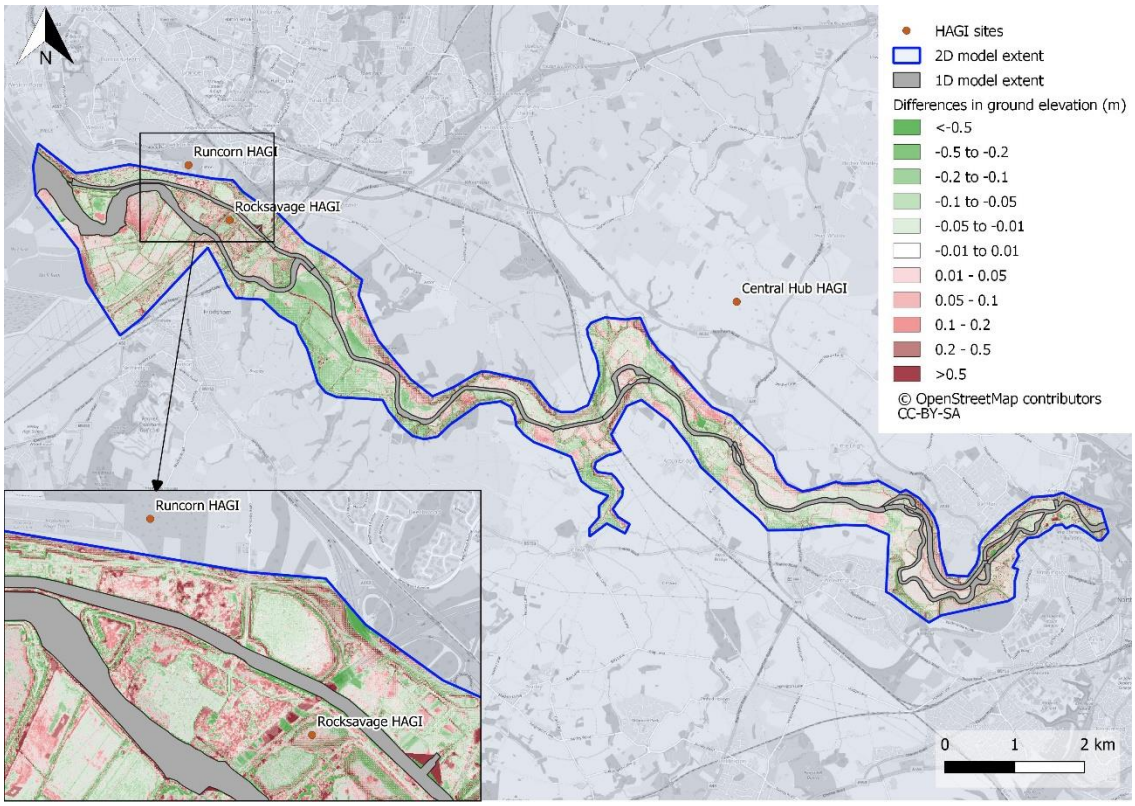
updated to 2D as part of the model update, the 1D channel representation remained largely unchanged from older 2002 survey data. No check survey has been undertaken as part of this study to determine the validity of the channel geometry representation. However, the lower reach of the River Weaver is heavily modified in general with various control structures influencing flow hydraulics. Therefore, it is unlikely that any significant geomorphological changes have occurred since the original channel survey data was captured and which could fundamentally impact the modelled flood risk at the HAGI site. The existing channel representation has therefore been deemed to be acceptable for the purposes of this assessment.

2.3 Topography

The current best available LiDAR data for the site and surrounding area has been assessed against the model 'DEM_Z' check file to understand whether there have been any recent developments (that are not captured in the existing model) that may have a bearing on flood risk to the HAGI site. The DEM_Z check file represents the final model elevation used and has been used in preference to the input LiDAR data since it accounts for any topographic modifications applied (to represent developments for instance).

A raster comparison between these two datasets is shown in Figure 2.1. The 1D model extent has been excluded from the analysis since the DEM_Z file represents the in-channel (sub-surface) geometry based on the 1D cross sections (through the use of TUFLOW water level lines), and hence is not comparable to the LiDAR data which represents the water surface elevation of the river and does not capture the sub-surface geometry. There are widespread differences in topographic levels throughout the 2D model extent typically in the region of +/- 0.20m, though with some more significant differences of over 1m. Based on review of historic satellite imagery, there are no noted developments or excavation works responsible for the differences in elevation between the two datasets, and it is thought that these primarily relate to the different dataset resolutions. The DEM_Z file is reflective of the 6m model cell size, whilst the LiDAR dataset is at a significantly higher resolution of 1m, resulting in widespread minor differences between the datasets. Some of the isolated, more significant differences in elevation are typically where the 1D geometry data has significantly influenced the 1D-2D boundary in the DEM_Z file, resulting in changes along the river channel banks.

Figure 2.1 Topography difference – model vs LiDAR



Therefore, the existing model topography is deemed to be appropriate for the assessment of flood risk to the site. Should the model be updated to incorporate the latest 2020 LiDAR data, it is expected that similar widespread differences would be evident between the resulting model DEM_Z file and the current LiDAR dataset (of higher resolution). Ultimately, the anticipated flood risk at the site is not expected to be affected. Updated model topography would further give rise to additional modelling complications associated with the 1D-2D linking which has been parameterised based on the LiDAR used in the original model.

2.4 Model schematisation

The schematisation of the existing model within the vicinity of the site has been reviewed to satisfy that the model representation is appropriate for assessment of flood risk to the site. In particular, the representation and inclusion of structures and buildings has been considered in detail, given these have the greatest potential to impact the assessment of flood risk to the site.

2.5 Structures

The representation of key hydraulic structures that may have a bearing on flood risk to the site has been reviewed. The adjacent M56 road bridge (immediately downstream of the site) and access bridge (approximately 800m downstream of the site) have not been explicitly represented. However, this is reasonable given the associated deck levels of each crossing and negligible obstruction to flow.

Representation of adjacent structures appears to be justified, and therefore the model representation is deemed to be appropriate for assessing flood risk specific to the HAGI site.

2.6 Buildings

The spatial representation of buildings has been defined based on the Ordnance Survey MasterMap (OSMM) data, presumably current at the time of the original study. The HAGI site and immediate surrounding area is predominantly greenfield with few nearby developments. However, the OSMM data has been assessed against the most recent aerial imagery to satisfy that this is still representative. No recent developments have been built since the time of the original model build (and would therefore be missing in the model representation) that could have a bearing on flood risk to the HAGI site.

2.7 Scenarios

The existing modelling study included defended, undefended, sluice gate failure and roughness sensitivity run scenarios. For the purpose of assessing updated flood risk to the HAGI site, the defended and sluice gate failure model scenarios have been identified. The defended scenario represents formal flood defences throughout the model based on current best understanding and represents the various control and sluice gate structures with a set of automated logical rules (presumably based on current understanding of their operation). The sluice gate failure scenario assumes that all sluices within the model are closed, and has been used as part of this study to understand the residual risk of flooding to the HAGI site should the structures fail during a flood. The undefended and roughness sensitivity scenarios have not been considered further as part of this study.

3. Model development

This section describes the new model files developed for the purposes of the updated climate change run. For a full overview of the existing EA model refer to the original model report in (2018s0738 – Lower Weaver Final Report).

3.1 1D model (Flood modeller)

No updates have been made to the 1D elements of the model. Refer to the original model report for more information regarding the 1D model including model schematic, details of roughness values, structures, and representation of defences.

3.2 2D model (TUFLOW)

No updates have been made to the 2D elements of the model. Refer to the original model report for more information regarding the 2D model including model schematic, details of roughness values, and representation of defences.

3.3 Boundary conditions

New Flood Modeller Event Data (.IED) files have been generated to provide the 0.5% AEP plus 67% climate change allowance and 0.5% AEP plus 106% climate change allowance inflows to the model associated with the River Weaver. The existing 0.5% AEP IED file has been used as a basis, and flow multipliers of 1.67 and 2.06 applied to inflows. It was noted that a scaling factor of 0.59 had been applied to the two lateral inflow ReFH boundaries in the 0.5% AEP baseline event.

Therefore, the scaling factors have been updated by applying the 1.67 and 2.06 multiplication factors, respectively.

The updated 0.5% AEP plus climate change events have been run in combination with the pre-existing (and unchanged) 2117 MHWS downstream boundary and steady 'low flow' boundary for the Manchester Ship Canal. This is in accordance with the existing fluvial climate change runs undertaken.

The IED file for the 1% AEP run has remained unchanged from the original model, in combination with the 2017 MHWS downstream boundary and steady 'low flow' boundary for the Manchester Ship Canal.

3.4 Model proving

As there has been no change in the fundamentals of the model geometry or hydrology, just a simple uplift to model climate change, no additional calibration or sensitivity analysis has been carried out on the model. Refer to the original model report for a discussion on the calibration and model proving undertaken on the EA Lower Weaver model.

4. Model runs

4.1 Software sensitivity

The existing 1% AEP event has been re-run using the updated TUFLOW version 2020-10-AD-iSP-w64 and Flood Modeller version 6.0 to understand the impact of the updated software version on modelled flood results at the site. A 2D peak flood depth difference map is shown in Figure 4.1, whereby the updated version results have been assessed relative to the original version used.

The difference between 2D flood depth and extent outputs for the two software versions is negligible and is within 0.01m (i.e. 1cm) across the entire 2D model domain. The 1D modelled in-channel results have also been reviewed which are similarly comparable. Therefore, re-running the model in the updated software version has no consequence with respect to the pre-existing results.

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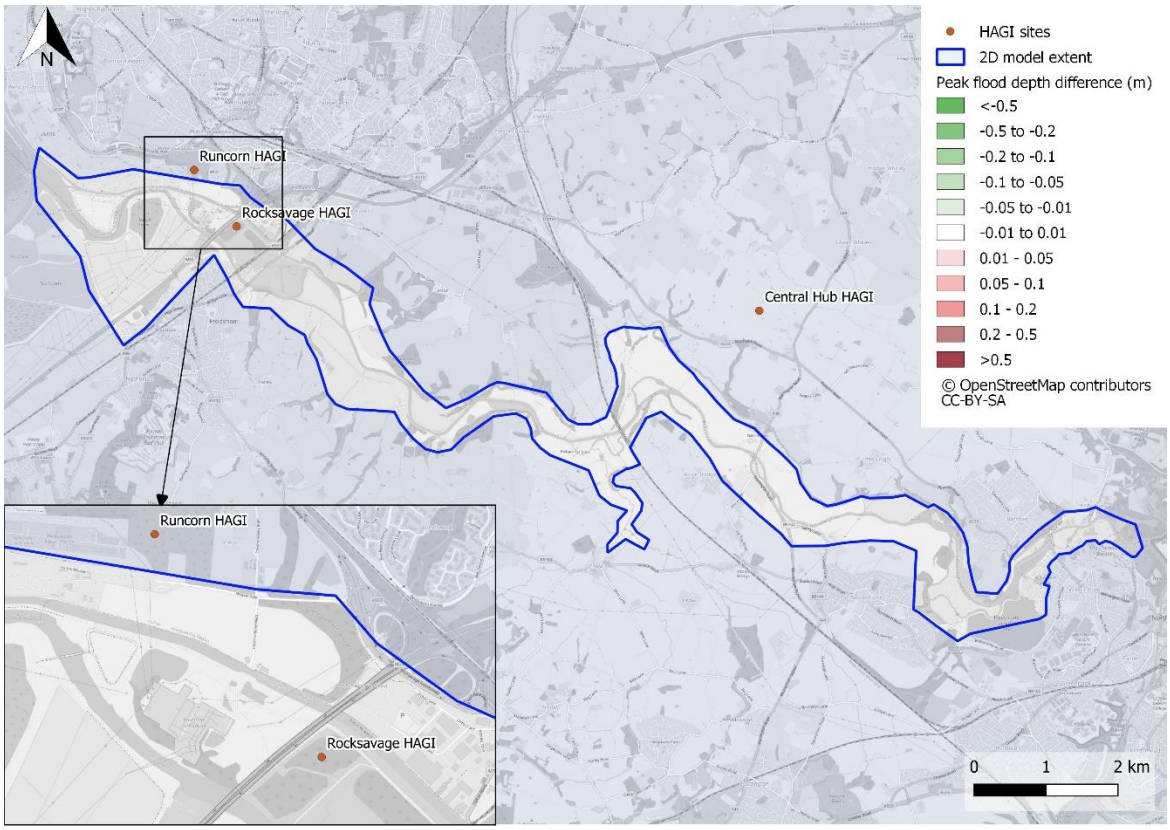
Figure 4.1 Depth and extent difference



4.2 Design runs

Model software

The updated model design runs have been run using Flood Modeller version 6.0 and TUFLOW version 2020-10-AD-iSP-w64.

Run parameters

Four design model runs have been undertaken using the EA model, as outlined in Table 4.1. Both the ‘defended’ and ‘sluice gate failure’ model scenarios have been run.

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Table 4.1 Model run record and parameters

Item	Comments	
Design Events (AEP)	0.5% plus 67% and 0.5% plus 106%	
Scenarios	Defended | Sluice gate failure	
Model run parameters and settings	Model run parameters have been left unchanged from the original EA model runs	
Flood Modeller network	LW_049.DAT	
Flood Modeller event data	Weaver inflows: LW_Q200CC67_SD45_latsc0.59.IED LW_Q200CC106_SD45_latsc0.59.IED	Other: MEST_MHWSCC_(2117)_Repeated_SD45.IED MSC(2018)_(Steady)_LowFlow.IED Sluice gate failure: SGF_CLOSED.IED
Flood Modeller event file	DEF04_6m_Q200CC106_M(LOW)_ME(MHWS)_WSP_02.ief DEF04_6m_Q200CC106_M(LOW)_ME(MHWS)_WSP_02.ief SGF_6m_Q200CC67_M(LOW)_ME(MHWS)_WSP_02.ief SGF_6m_Q200CC106_M(LOW)_ME(MHWS)_WSP_02.ief	
TUFLOW control data	~s1~_~s2~_~e1~_~e2~_~e3~_WSP_02.tcf	

4.3 Quality assurance

A summary of the model run performance for the updated runs undertaken is provided in Table 4.2. The mass error in TUFLOW is within the recommended bounds of +/- 3% mass balance and reduces to within +/- 1% after the initial wetting period. The Flood Modeller model is also within the convergence tolerance. The model performance for these runs is comparable to the existing defended scenario model runs detailed in the original model report.

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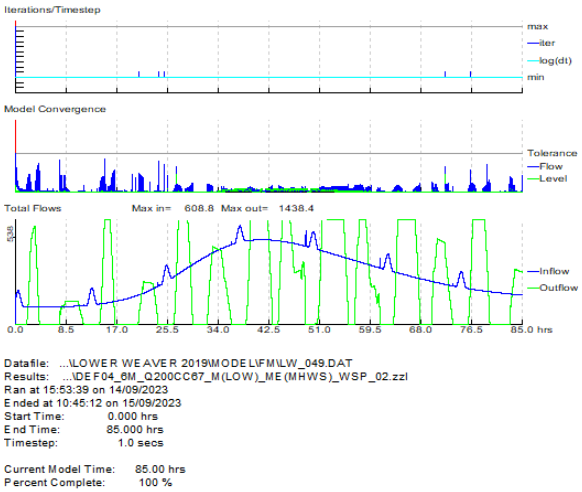
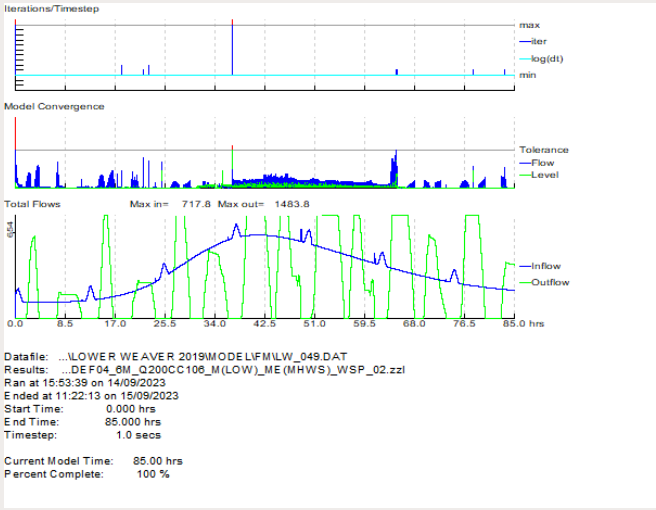
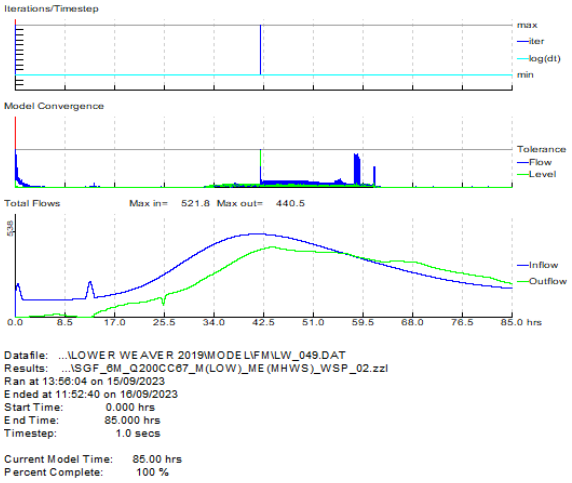
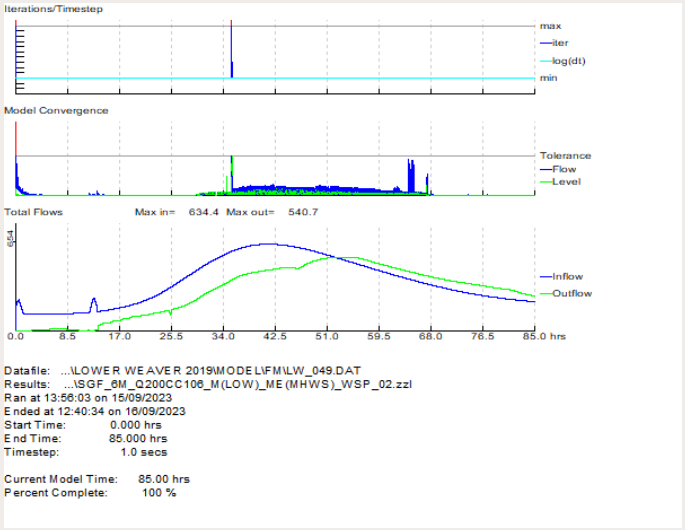
Table 4.2 Model run performance

Scenario	AEP	TUFLOW Peak Cumulative Mass Error (%)		Flood Modeller performance
		Whole simulation	Flow In + Out >5%	
Defended	0.5% (67%)	0.14%	0.16%	
Defended	0.5% (106%)	-0.29%	-0.30%	

Sluice gate failure	0.5% (67%)	-0.34%	-0.34%	
Sluice gate failure	0.5% (106%)	-0.27%	-0.27%	

The TUFLOW Simulation File (.tsf) records a total of two checks and five warning messages prior to simulation. Checks related to two instances of coincident 2d_bc points (which TUFLOW ignores), and the warning messages relate to a manning's n value and three instances where an active cell has no active faces (surrounded by HX link cells). The three latter warning messages were not output from the existing model runs, though it is thought that this is related to the TUFLOW software update since model representation remains unchanged. Each instance of this warning message has been reviewed in the model and confirmed to be inconsequential.

5. Conclusions and recommendations

The existing EA owned hydraulic model for the Lower Weaver has been deemed fit for the purpose of assessment to support the development of the Rocksavage HAGI site. It has been used to run the defended 0.5% AEP event accounting for both the Upper End and Higher Central allowances. Multiplication factors of 1.67 (67%) and 2.06 (106% allowance) have been applied to the existing 0.5% AEP inflow hydrograph, to generate new IEDs and incorporated into new IEFs for the model runs. The TUFLOW and Flood Modeller software versions have been updated, and therefore an additional model run has been carried out for the 1% AEP event to allow for like-for-like

comparison to the new climate change run. Resultant model performance for the updated runs is within the acceptable bounds. There are no further recommendations to those made in the original modelling report.

Issued by

.....
Jack Park

Approved by

.....
Angela Dhaliwal

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Document revisions

No.	Details	Date
P01.01	Issue 1	May 2023
	Issue 2	September 2023

HyNet North West Hydrogen Pipeline

Delivering clean growth

Annex 7F3 Gale Brook Hydraulic Model Report

1. Introduction

1.1 Context and scope

1.1.1 This model user report accompanies the Hynet Flood Risk Assessment (FRA). This report documents the modelling undertaken, using the existing Environment Agency (EA) owned model for the Stanlow Oil refinery to provide updated and improved fluvial flood extents and levels for the Gale Brook. These results have been used to assist in the assessment of fluvial flood risk to the Stanlow Hydrogen Production Plant (HPP) Hydrogen Above Ground Infrastructure (HAGI) and elements of the wider project.

1.1.2 The scope of this modelling study is limited to the single reach of the Gale Brook which flows north, to the west of the Stanlow HPP HAGI, as this is the primary source of flood risk to the HAGI. The primary purpose of the modelling is to quantify and more accurately define the fluvial flood risk presented to the HAGI from the Brook.

1.2 Background

1.2.1 The HyNet North West Hydrogen Pipeline Project includes the construction and operation of approximately 125km of new underground pipelines to distribute hydrogen to a variety of industries across Merseyside, Greater Manchester, and Cheshire. The Stanlow HPP HAGI, which will connect the HPP and the distribution pipeline network, was identified as one of four HAGIs at risk of fluvial or tidal flooding in the Preliminary Environmental Information Report (PEIR). The south-west corner of the Stanlow HAGI is in close proximity to Flood Zones 2 and 3. These Flood Zones are associated with the Gale Brook, an Environment Agency (EA) Main River, within the wider River Gowy catchment.

1.2.2 The existing EA owned model of the Gale Brook was developed by Jacobs in 2019 , which was itself an update of the River Gowy Model developed by JBA in 2016. In the Jacobs 2019 modelling study, the representation of the Gale Brook was extended and updated with more extensive survey of the watercourse incorporated. However, the model representation of the watercourse was predominantly 1D with extended floodplain cross sections, and therefore the model has been updated to improve the representation of flood risk to the Stanlow HAGI site.

1.2.3 The 2019 modelling study applied climate change allowances of 25% and 30% to the 1% AEP flow. These allowances for peak river flow were based on relevant EA 2016 guidance (at the time of the study) for the North West River Basin, and for 50-year and 100-year epochs. Whilst the 1% AEP plus climate change allowance fluvial event would typically form the design standard for fluvial flood risk in accordance with the online Planning Practice Guidance (PPG), the 0.5% AEP plus climate change event has been chosen as the design event for the purposes of the FRA.

1.3 Methodology

1.3.1 The modelling study follows the steps outlined below:

- Review the existing EA Model to satisfy that it is fit for purpose to assess flood risk for the wider area;
- Edit and update the existing EA model, in order to provide improved representation of flood risk from the Gale Brook to the Stanlow HPP HAGI site. Key model updates are outlined below:
  - Extend the 1D Gale Brook model extent upstream of the M56 motorway bridge with new channel survey data to represent any attenuation impacts of the motorway structure;
  - Update the Gale Brook floodplain representation to 2D, based on the latest available LiDAR data, and;
  - Review and update 1D model representation to incorporate the latest survey data (commissioned in 2022) where discrepancies against the previous model representation are noted.
- Generate revised hydrological inflows to the model, taking into account the extended 1D model extent;
- Generate new Flood Modeller Event Data (.IED) files for a range of flood events, and incorporating the 67% and 106% climate change allowances to the 0.5% AEP event;
- Generate new Flood Modeller Event Files (.IEF), to output results to appropriate location on the local system, reading in the new IED file described above and existing model elements (DAT file and TCF);
- Run model using updated IEFs above;
- Process results files for the area of interest, and;
- Establish whether floodplain compensation would be required, which current PPG guidance states is required if floodplain is lost during the 1% AEP event including the higher central allowance for climate change.

1.4 Report Structure

1.4.1 The structure of this model user report is as follows:

- Section 2 – provides an overview of the input data available for the purpose of this study and a detailed review of the existing EA model to determine its suitability for this assessment;
- Section 3 – details the model updates incorporated as part of this study in order to provide improved and updated (incorporating the latest climate change allowances) flood risk outputs to the Stanlow HAGI site;
- Section 4 – provides an overview of the model runs undertaken, run parameters, run performance and sensitivity analysis carried out, and;

- Section 5 – provides conclusions and recommendations from this modelling study.

2. Input data

2.1 Existing model

2.1.1 The principal data source of relevance to this study is the existing EA-owned hydraulic model of the River Gowy and Gale Brook. The latest Stanlow Oil Refinery model was developed by Jacobs in 2019 on behalf of the EA, and is an update to the pre-existing River Gowy model originally developed by JBA in 2016. The model and associated EA data supplied are outlined in **Table 2.1**, and described in more detail in the following sub-sections.

Table 2.1 Supplied EA flood model data

Item	Description
Product 4	Hydraulic modelling report and accompanying Hydrology report describing the project background and model updates incorporated. *ENVIMNW000998-JAC-0-00-RP-HY-0004_P02.pdf* *Stanlow Flood Estimation Calculation Record.docx*
Product 5	Accompanying model results in GIS and .csv format.
Product 6	Hydraulic model files for the ISIS (now FloodModeller) and TUFLOW model elements.

2.1.2 Additional data sources used for the purposes of this model update are provided in **Table 2.2.**

Table 2.2 Additional data sources

Data	Source	Comments
Topographic survey	Warner Surveys	New channel survey taken in 2022 of the Gale Brook, upstream of the M56 motorway (previously not represented in the model) and throughout the modelled reach to check the validity of the existing model representation (based on channel survey undertaken in 2019).
	Sensat survey	Photogrammetric drone survey commissioned by Cadent for the project that covers the entire Red Line Boundary (i.e. including the HAGI sites), DSM with horizontal resolution of 0.1m and vertical accuracy of 0.042m.
LiDAR	EA	2022 EA composite LiDAR data for representation of the Gale Brook floodplain. This is understood to have been flown in 2021 for the area of interest in this study.
Landuse	Ordnance Survey (OS)	OS OpenMap Local data, used for landuse delineation and materials for the updated Gale Brook 2D representation.

2.2 Model review

2.2.1 The Stanlow Oil Refinery model (referred to hereafter as the Stanlow model) was provided by the EA on in December 2022, and is based on the 2016 River Gowy model, originally developed by JBA Consulting1.

2.2.2 The 2016 Gowy model, a Flood Modeller – TUFLOW, 1D-2D linked model, included explicit, detailed representation of the River Gowy between Bridge Trafford and the confluence with the River Mersey, as well as two tributaries: the Thornton Brook and the Mill Brook.

2.2.3 The 2019 JBA Stanlow Oil refinery model was developed in 2019 by Jacobs on behalf of the EA[1], to evaluate the risk of tidal and fluvial flood risk to the Stanlow Oil refinery site. The original Gowy 2016 model was extended to include the Gale Brook and the N38 interceptor channel (which discharges into the Manchester Ship Canal). The key updates concerning the Gale Brook (and of most relevance to this study) are described below:

- Explicit representation of a 3.5km reach of the Gale Brook from the M56 motorway bridge to its outfall to the River Gowy;
- 1D representation of the Gale Brook was provided from the M56 motorway bridge and through the culverted Stanlow Oil Refinery. The open channel of the Gale Brook downstream of the refinery running parallel to the railway line is represented in 2D;
- 1D cross sections were based on channel survey commissioned as part of the study in 2019, and supplemented by 1m 2019 LiDAR for representation of the floodplain; and
- New hydrological inflows were developed for the Gale Brook, based on the ReFH1 model for consistency with the wider model.

2.2.4 The following subsections review the model in more detail.

Model software

2.2.5 The 2019 Stanlow modelling study was carried out using Flood Modeller version 4.4, and TUFLOW version 2018-03-AB-iSP-w64. The updated model runs for this assessment have been run using Flood Modeller version 6.2 (released in 2023) and TUFLOW version 2023-03-AC (released in 2022).

Boundary conditions

Hydrology

2.2.6 In the 2019 Stanlow model, no changes were made to the original hydrology inflows developed as part of the Gowy 2016 model, which consist of ReFH1 boundary units. However, given the explicit representation of Gale Brook, the

[1] JBA Consulting (2016). Mersey Estuary, Ditton Brook and River Gowy

existing 'Stanlow02' hydrological inflow (representing the Gale Brook) was replaced with three inflows along the length of the Gale Brook. These are:

- GaleB1 – upstream point inflow applied to the 1D Flood Modeller model. This inflow represents the upstream catchment area of 2.52km^2;
- GaleRes12 – lateral inflow representing the intervening catchment (2.28km^2) between the M56 and culverted section of the watercourse within the Stanlow Oil Refinery. Flows are distributed equally along this reach, and;
- GaleRes23 – lateral inflow representing the intervening catchment (1.85km^2) between the culverted section of the watercourse and the confluence to the River Gowy. This inflow is applied at the downstream end of the culverted section of the watercourse in the TUFLOW 2D model.

2.2.7 For consistency with the pre-existing hydrological inflows developed in the Gowy 2016 model, the updated Gale Brook inflows developed for the Stanlow model update in 2019 were generated using ReFH1 boundary units. Design flows were developed using a 45.5 hour storm duration, consistent with the River Gowy critical storm duration used in the Gowy 2016 model, and also considering a 3.9 hour storm duration for comparison, representative of the Gale Brook critical duration.

2.2.8 Whilst the ReFH1 boundary units representing the Gale Brook were deemed to be suitable for the purposes of the Stanlow model study in 2019, they do not represent current best practice in accordance with the EA flood estimation guidelines. In addition, the Gale Brook model representation and extent has been revised (as detailed below in **Section 3**). Therefore, the existing Gale Brook hydrological inflows required updating to provide an improved representation of flood risk along the watercourse and conform to the revised model representation. These hydrology updates are described in detail in **3.2**.

Downstream

2.2.9 Downstream boundaries used in the 2019 Stanlow model remained unchanged from the 2016 Gowy model. Fluvial design runs apply a mean high-water spring (MHWS) tidal boundary for the Mersey estuary (into which the River Gowy discharges), and consider a nominal navigation level of 4.71m AOD on the Manchester Ship Canal.

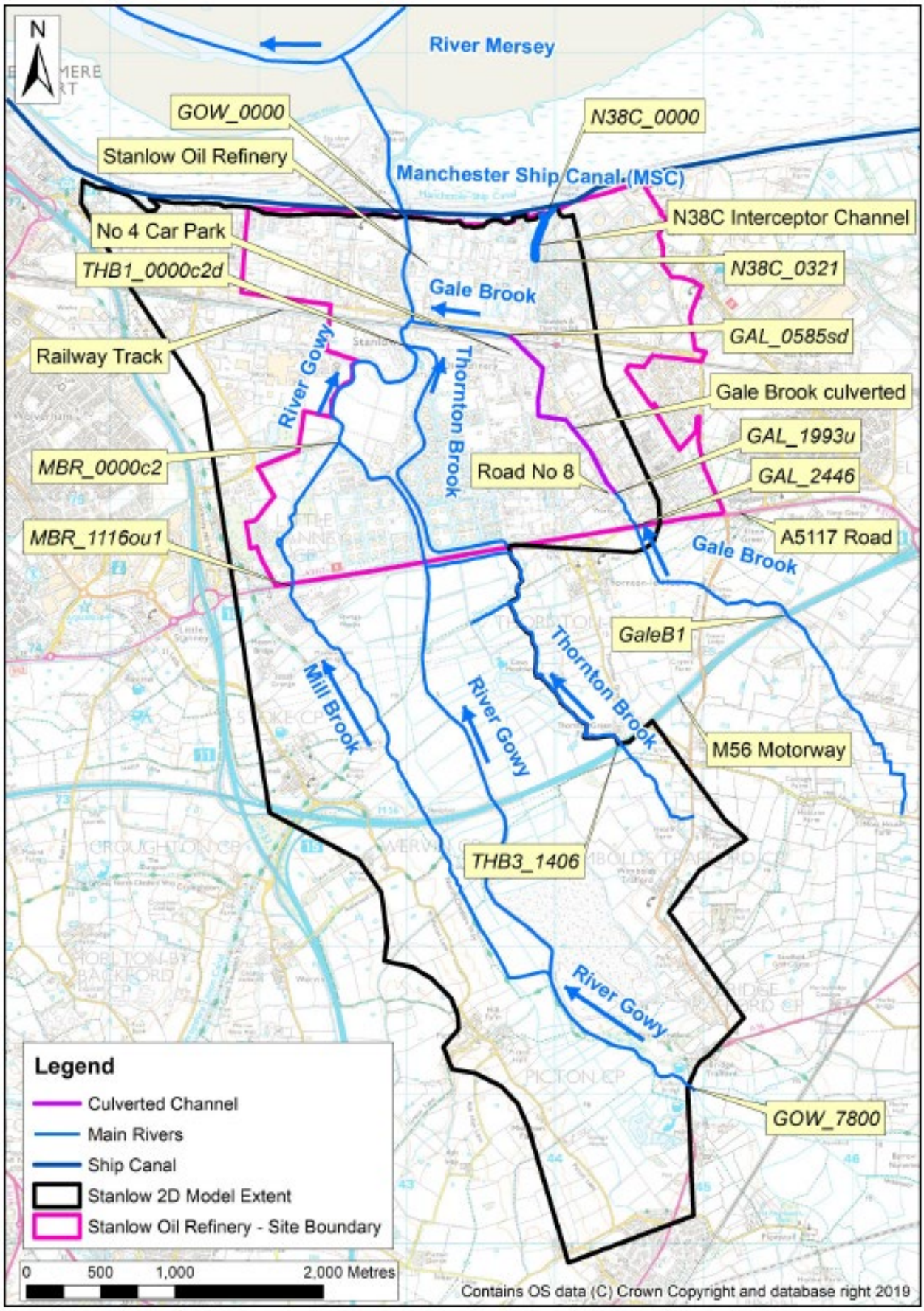
2.2.10 The Gale Brook drains into the River Gowy immediately north of the railway line through a flapped outfall beneath the Gowy tidal flood defence (reinforced in the 2D topography using a z-shape feature). This has been modelled via a '1d_nwke' ESTRY (unidirectional flow) connection into the River Gowy Flood Modeller open channel network.

Model schematisation

1D Model

2.2.11 The 1D model extent in the 2019 Stanlow model representing the Gale Brook stretched from the M56 Motorway (node GaleB1 at NGR SJ 45557 74247) and to the outfall of the culverted section (beneath the Stanlow Oil Refinery) north of the railway line (node GAL_0585sd at NGR SJ 43704 76087). The 1D model of the Gale Brook in the Jacobs 2019 model had a length of 3039m and is shown in **Graphic 2.1**.

Graphic 2.1 Stanlow model overview (Jacobs 2019)



2.2.12 Upstream of the Stanlow Oil Refinery culvert (Road no.8 and node GAL_1993u), the watercourse is predominantly open channel with a total of six structures represented. Channel and structure geometry is based on survey data commissioned as part of the project and taken in 2019. The floodplain for this reach is predominantly represented using extended 1D channel cross sections informed by 1m LiDAR data. The exception is a short (~450m reach) 1D-2D linked section immediately upstream of the Stanlow Oil Refinery and shown by the 2D model extent in **Graphic 2.1**. The Manning's roughness values for the Gale Brook channel were determined using site visit photography, and values ranged between 0.05 to 0.07 for the channel bed and 0.04 to 1.00 for the floodplain.

2.2.13 The culverted section beneath the Stanlow Oil Refinery has been modelled as a rectangular Flood Modeller conduit unit. Whilst this is based on survey taken at the upstream face of the culvert (node GAL_1993u), it is assumed that these dimensions remain representative throughout the 1400m length.

2.2.14 As outlined in **Table 2.2**, additional channel and structure survey for the Gale Brook was commissioned as part of this study to enable the extension of the 1D model domain upstream of the M56 embankment and in order to review the validity of the existing channel survey data. The extended modelled reach length allows for the representation of any attenuation impacts associated with the M56 structure and other crossings upstream, in order to improve the representation of flood risk to the Stanlow HAGI site downstream. The 1D model updates are detailed further in **Section 3.3**.

2D Model

2.2.15 Immediately upstream of the Stanlow Oil Refinery culvert at node GAL_1993u, a 450m reach of the watercourse is modelled as 1D/2D, whereby the floodplain is represented in 2D and based on 2019 LiDAR data.

2.2.16 The open channel section between the refinery culvert outlet (node GAL_0585sd at NGR SJ 43704 76087) and the confluence with the River Gowy (NGR SJ 43034 76201) is modelled within the TUFLOW 2D model component. This open section of channel was represented using a TUFLOW 2d_zsh gully line to enforce the watercourse bed level and was based on 2019 1m horizontal resolution LiDAR data.

2.2.17 Roughness in the 2D model domain was delineated based on OS MasterMap data from 2017, classified into simplified landuse classes and assigned a Manning's n roughness coefficient.

2.2.18 In order to improve the modelled representation of floodplain flow and attenuation along the Gale Brook and upstream of the Stanlow HAGI site, a 2D representation of the floodplain has been incorporated throughout the modelled reach as part of this study. The latest available LiDAR data (flown in 2021) has been used to inform the 2D model extent throughout the Gale Brook modelled reach. This model update is detailed further in **Section 3.3**.

3. Model development

3.1.1 Given the scale of the existing Stanlow model also covering the River Gowy, Mill Brook, Thornton Brook, and N38C interceptor channel, the model has been refined in extent to focus only on the Gale Brook for efficiency. As shown through sensitivity testing of the downstream boundary conditions in **Section 4.5**, flood risk outputs upstream of the Stanlow refinery culvert on the Gale Brook are insensitive to conditions within the River Gowy and the section of the Gale Brook between the refinery culvert and the confluence with the River Gowy. Therefore, exclusion of the wider model has no bearing to anticipated flood risk adjacent to the HAGI site and is justified. The following sub-sections detail the updates made to the model.

3.2 Boundary conditions

Inflow boundaries

3.2.1 As described in **section 2.2**, the existing Gale Brook hydrological inflows within the Stanlow model were deemed to be inadequate for the purposes of this assessment. Revised design inflows have been developed for the Gale Brook and detailed in the accompanying Flood Estimation Handbook (FEH) calculation record, Annex A. The approach is summarised in this section below.

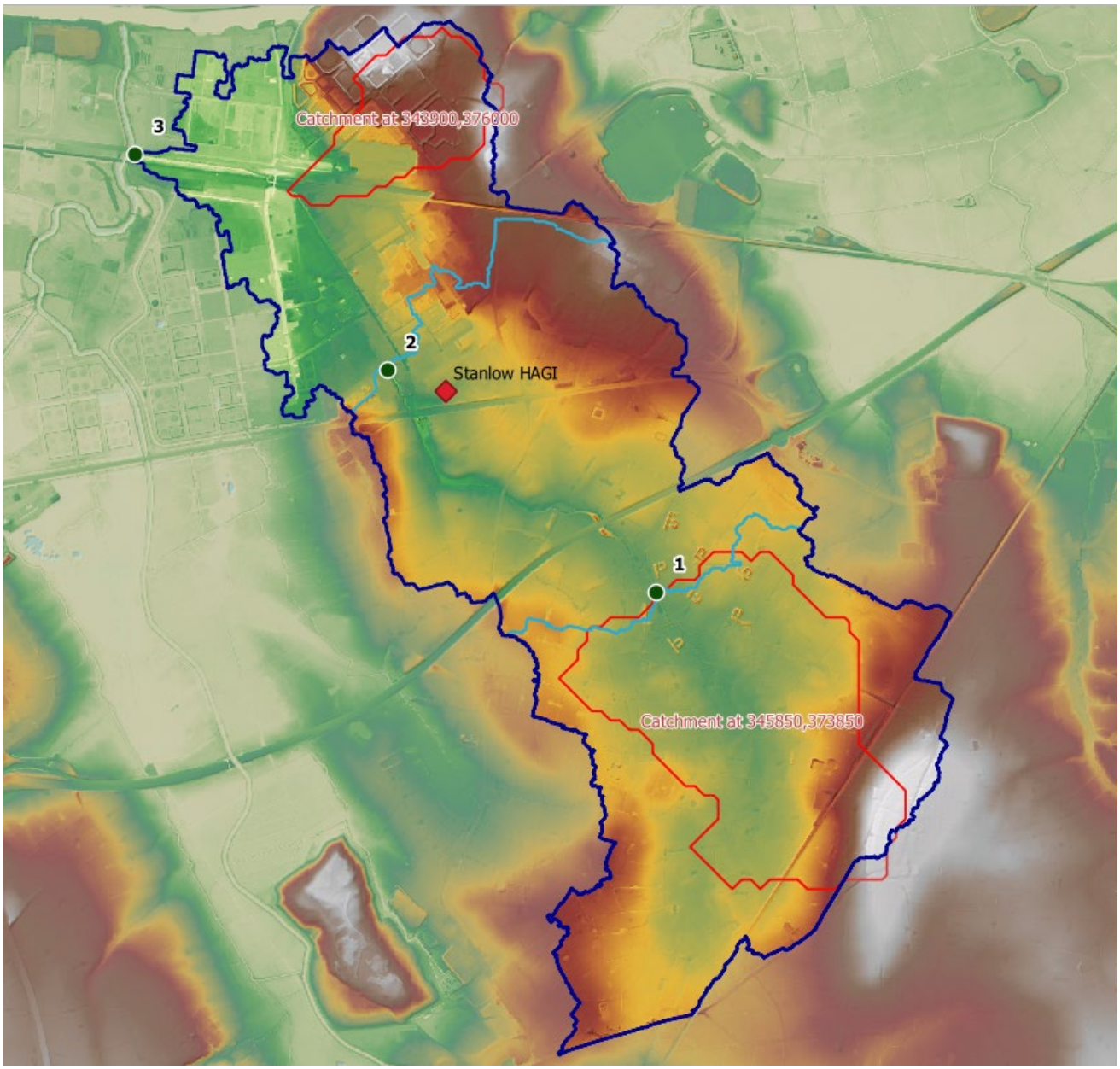
3.2.2 Revised inflows have been developed using updated ReFH2 software which is in line with current best practice in the EA flood estimation guidelines. This has been used in preference to the now superseded ReFH1 software and FEH statistical method, given the small scale of the catchment.

3.2.3 A similar approach to the Stanlow 2019 study has been employed for this assessment in terms of flow estimation points (FEPs). Three FEPs have been used within the catchment, summarised below.

- FEP1 – upstream inflow to the Flood Modeller 1D model representing the upstream catchment of 3.81km^2 (revision of existing GaleB1 unit);
- FEP2 – lateral inflow representing the intervening catchment of 3.15km^2 between the upstream extent of the model and start of the culverted section of the watercourse at the Stanlow Oil Refinery (revision of existing GaleRes12 unit). Flows are distributed equally along this reach, and
- FEP3 – lateral inflow representing the intervening catchment of 2.51km^2 between the culverted section of the watercourse and the confluence to the River Gowy (revision existing GaleRes23). This inflow is similarly applied at the downstream end of the culverted section of the watercourse in the TUFLOW 2D model.

3.2.4 The FEH catchment for FEP1 was reviewed against the catchment delineated from the latest available EA LiDAR data. The FEH catchment was found to significantly underestimate the catchment area by approximately 50%, as demonstrated in **Graphic 3.1**.

Graphic 3.1 Gale Brook Flow Estimation Points



Graphic showing the original FEH catchments in red, revised total catchment (dark blue) and sub-catchments (light blue), and flood estimation points (green) displayed against the underlying LiDAR.

3.2.5 This is a further key difference from the original Stanlow model inflows, which were developed based on the accepted and unchanged FEH catchment boundary (presumably deemed to be acceptable as part of the Stanlow 2019 study).

3.2.6 It is worth noting also that the original Stanlow model was run with a focus on flood risk associated with the River Gowy, and therefore a storm duration of 45.5-hours was used to derive the inflow hydrology for all catchments (assuming a consistent catchment wide storm). Whilst this results in the most conservative flow estimates for the River Gowy, existing peak flows for the Gale Brook are under-predicted and therefore the associated flood risk outputs on this watercourse do not represent a

reasonable worst case. These two factors result in significantly more conservative flow estimates developed as part of this study.

3.2.7 In accordance with the refined model extent, existing ReFH1 boundaries associated with the wider model have been removed.

Downstream boundary

3.2.8 For the updated Gale Brook model, the 1D/2D link representing the confluence to the River Gowy has been adapted and updated to a downstream boundary. The Gale Brook discharges through a flood defence and via a flapped outfall into the River Gowy (shown in **Photo 6** and **Photo 7**, **Annex B**), modelled as a 1D (unidirectional flow) culvert in the baseline model which connects to the adjacent River Gowy Flood Modeller 1D channel unit.

3.2.9 A 1D HT (Head-Time) boundary condition TUFLOW layer has been added to the model to replace the Flood Modeller unit, with a set water level equivalent to the MHWS. An allowance for sea level rise should ideally have been added to account for the rise in sea levels during the assumed lifetime of the Project up to 2100. The model uses present day MHWS instead. However, as described in **section 4.5**, sensitivity test results show that flood risk adjacent to the HAGI site is fluvially dominated and insensitive to the downstream boundary conditions within the River Gowy. Therefore the use of the present day MHWS as a downstream boundary is considered acceptable.

3.2.10 In addition, a 2D Head-Flow (HQ) boundary has been added to the M56 motorway to prevent glasswalling of floodwater that surcharges the M56 culvert (described further in **Section 3.3**). Floodwater that surcharges the culvert flows west along the M56, leaving the Gale Brook catchment and would therefore have no bearing on flood risk downstream at the Stanlow HAGI site of principal concern for this study.

3.2.11 The new and updated model downstream boundary layers associated with this model update are outlined in **Table 3.1.**

Table 3.1 Downstream boundary model layer updates

New layer	Reasoning
1d_nwke_GB_Gowy_089_L.SHP	Feature taken from '1D_nwke_Gowy_010.shp' in the original model to represent outfall structure to the River Gowy (other features in this layer were removed given that they relate to model features outside of the updated Gale Brook model domain). The outfall is modelled as two 1.5m x 1.5m rectangular culverts, which aligns with site observations as shown in Photo 6 and Photo 7, Annex B. The culvert has been modelled with type 'RU', with the 'U' specifying unidirectional flow to model the

New layer	Reasoning
	flapped outfalls and prevent backflow of tidal water.
1d_bc_GB_Gowy_DSBDY_089_P.shp	New 1D boundary condition added to represent downstream boundary (replacing previous connection to the River Gowy Flood Modeller 1D river unit). A Head-Time (HT) boundary has been used, setting a static water level of 5.28mAOD, equivalent to the Mersey Estuary MHWS.
2d_bc_GB_Gowy_089_P.shp	1D/2D SX linking feature taken from '2D_bc_hxi_Gowy_005_P.shp' in the original model (other features in this layer were removed given that they relate to model features outside of the updated Gale Brook model domain).
2d_bc_HQ_M56_072_L.shp	New 2D boundary condition added to allow for floodwater bypassing the Gale Brook catchment along the M56 to leave the model. A slope of 0.002m/m has been used, based on a long profile of the motorway at the boundary location.

3.3 Hydraulic Model Build

3.3.1 The following sub-sections detail the specific updates made to the model along with any associated review and reasoning.

1D model

Channel

3.3.2 The Gale Brook 1D model has been extended to include a 500m long reach upstream of the M56 motorway bridge. A total of eight additional river sections and two new structures have been added to the model to define this new reach, based on surveyed cross sections taken in 2022 and commissioned as part of this study. The additional nodes used for this reach are outlined in **Table 3.2**, and specific details about the structures are provided in **Table 3.4** further below.

Table 3.2 Additional 1D nodes – extended 1D reach upstream of M56

Node ID	Flood Modeller unit type	Source of data
GaleB1	River Section	Revised upstream model section – based on 2022 survey cross section ID 24.
GaleB1In1	Interpolate	Auto-interpolate
GAL_4048	River Section	2022 survey cross section ID 25.
GAL_4048In1	Interpolate	Auto-interpolate
GAL_3983	River Section	2022 survey cross section ID 26.
GAL_3983In1	Interpolate	Auto-interpolate
GAL_3890	River Section	2022 survey cross section ID 27.
GAL_3890In1	Interpolate	Auto-interpolate
GAL_3890In2	Interpolate	Auto-interpolate
GAL_3780	River Section	2022 survey cross section ID 28.
GAL_3780bu	Bridge	2022 survey cross section ID 28.
GAL_3768	River Section	2022 survey cross section ID 29.
GAL_3768In1	Interpolate	Auto-interpolate
GAL_3676	River Section	2022 survey cross section ID 30 (10m upstream of motorway crossing structure).
GAL_3666	River Section	2022 survey cross section ID 30 (10m upstream of motorway crossing structure). Section duplicated and values lowered based on average upstream slope to provide estimate of section data and invert level for the motorway culvert upstream entrance.
GAL_3666	Culvert Inlet	Culvert parameterised based on survey photographs.
GAL_3666cu	Conduit	As mentioned above, upstream invert estimated based on 2022 survey cross section ID 30 and assuming upstream slope.
GAL_3666cd	Conduit	2022 survey cross section ID 31.

Node ID	Flood Modeller unit type	Source of data
GAL_3666cd	Culvert Outlet	Culvert parameterised based on survey photographs and 2022 survey cross section ID 31.

3.3.3 Downstream of the M56 road embankment, the new 2022 survey has been reviewed against the existing model representation (based on 2019 survey data). Where comparisons can be directly made between the two survey datasets at coincident cross sections, the channel geometry typically aligns well (typically within 0.05m) and therefore there is confidence that no significant geomorphological change has occurred to the channel since the time of the 2019 survey.

3.3.4 On the whole, the pre-existing model representation (based on the 2019 survey data) is of higher resolution and deemed to be of better quality than the survey commissioned as part of this study. Given that there is still confidence in the data, it was decided that the existing section data for this reach would predominantly be retained.

3.3.5 In instances where there is noted change between the two datasets (typically where bed levels have increased indicating siltation) or where the new 2022 survey captures additional detail, some minor changes to the model representation have been made. All changes to existing model nodes in this reach are detailed in **Table 3.3**, along with any additional nodes incorporated to improve model representation.

Table 3.3 Section data updates to reach downstream of M56

Node ID	Update(s)	Reasoning
Existing model nodes		
GAL_3631	Raised bed level to 7.032m AOD to match 2022 survey cross section ID 31.	Pre-existing model sections match well with the 2022 survey, though the 2022 bed level is higher than the existing section indicating siltation. The existing bed level has therefore been raised to reflect this, providing a slightly more conservative representation in terms of flow conveyance.
GAL_3470	Raised bed level to 7.265m AOD to match 2022 survey cross section ID 33.	
GAL_3465	Raised bed level to 7.260m AOD to match 2022 survey cross section ID 34.	
GAL_1993	Raised bed level to 5.02m AOD to match 2022 survey cross section ID 54.	

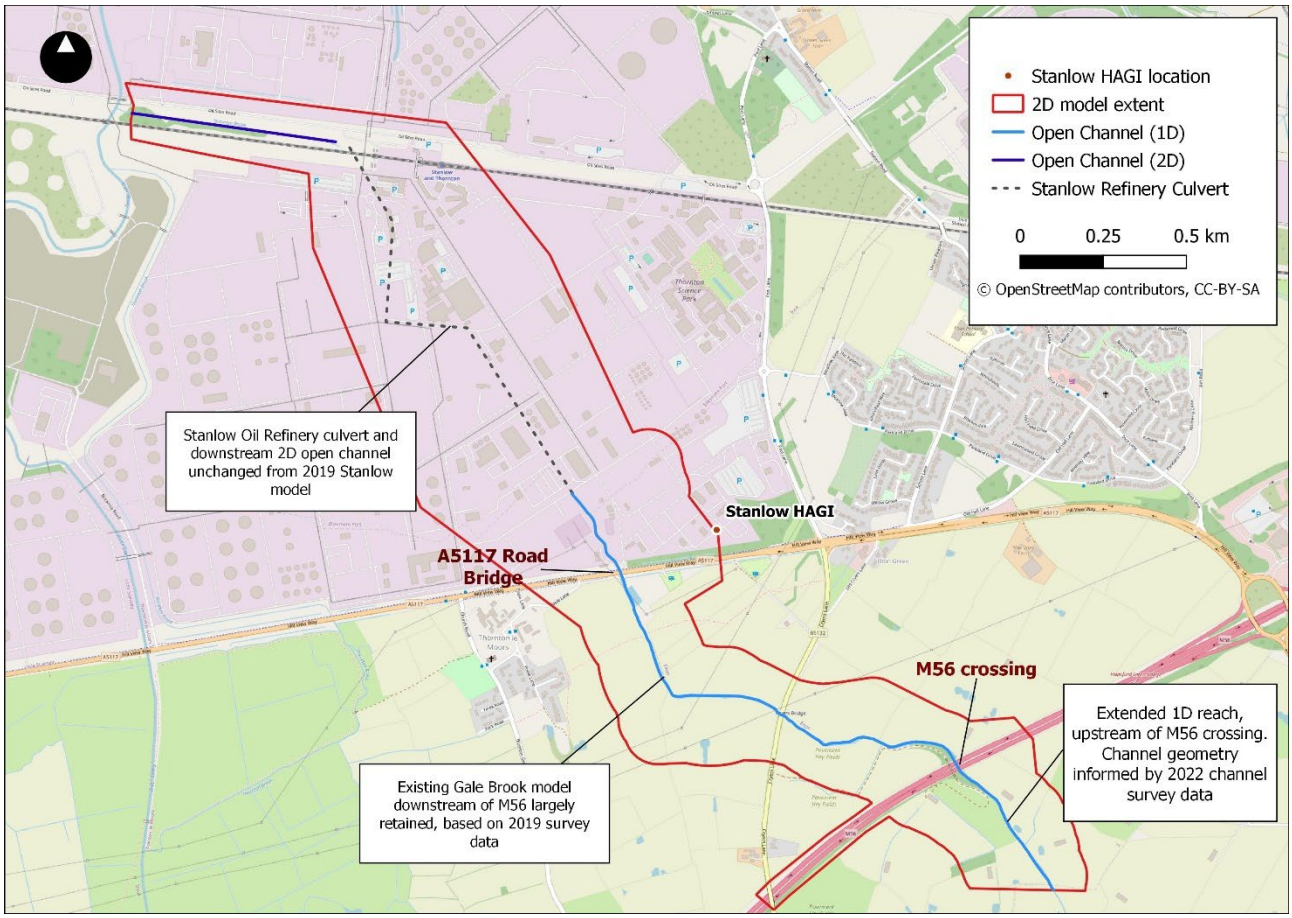
Node ID	Update(s)	Reasoning
GAL_2245	Replaced section data with 2022 survey data	Pre-existing section data used in the model was a duplicate from an upstream section. Cross section ID 49 from the 2022 survey was taken at the downstream face of the culvert and therefore more appropriate.
Removed model nodes		
GAL_2271	Node removed	Model sections representing the short (5m) open channel section downstream of the A5117 road bridge have been removed to improve model stability in extreme events. The open-channel section has been represented by an extension of the pre-existing A5117 culvert, as detailed further in Table 3.4. Sensitivity testing as described in Section 4.5 demonstrates that this open channel section has negligible control on local flooding mechanisms.
GAL_2266	Node removed	
New model nodes		
GAL_3016	New model node based on 2022 survey section ID 41.	Surveyed bed level at 2022 Survey section 41 (17m downstream of the Cryers Bridge) shows a significant 0.6m drop in bed level from existing model section GAL_3033. Downstream of this, cross sections align well between the two datasets and therefore there is confidence in this bed drop.
GAL_2409	New model node, duplicate of GAL_2446. Channel values revised based on slope to GAL_2303, and right bank values revised based on LiDAR and approximated bund	New model sections added to represent the lower right bank floodplain and bund, upstream of the A5117 road bridge.

Node ID	Update(s)	Reasoning
	configuration defined by a z-shape (see section below and Table 3.6).	
GAL_2321	New model node, duplicate of GAL_2446. Channel values revised based on slope to GAL_2303, and right bank values revised based on LiDAR.	
GAL_2210	New model node, based on 2022 cross section ID #50	New section added to provide additional detail to model between nodes GAL_2245 and GAL_2171
GAL_3631In1	Auto-interpolate section added.	Interpolate sections added to ensure smooth gradations between channel properties thus avoiding sudden variations which can cause instability issues in the model.
GAL_3456In1		
GAL_3311In1		
GAL_3186In1		
GAL_3016In1		
GAL_3016In2		
GAL_3016In3		
GAL_2861In1		
GAL_2861In2		
GAL_2861In3		
GAL_2708In1		
GAL_2708In2		
GAL_2708In3		
GAL_2563In1		
GAL_2563In2		
GAL_2409In1		
GAL_2409In2		
GAL_2160In1		

Node ID	Update(s)	Reasoning
GAL_2160In2		

3.3.6 An overview of the updated Gale Brook model 1D (and 2D) extents and schematisation is provided in **Graphic 3.2**. As mentioned above, the wider 1D model networks representing the River Gowy, Mill Brook, Thornton Brook and N38C interceptor channel have been excluded from this updated model.

Graphic 3.2 Updated Gale Brook model overview



Floodplain representation

3.3.7 Throughout the 1D modelled reach from the M56 motorway embankment and the entrance to the Stanlow Oil Refinery culvert, existing model sections have also been edited to deactivate the left and right bank floodplain representation. This is in association with the updated 2D representation of the floodplain, detailed further in the sub-section below. The 1D model sections have been used to inform the de-activated 2D model extent, to ensure no double counting of flow.

Key structures

3.3.8 The majority of structures within the 1D Gale Brook model remain unchanged from the original Stanlow model. Two additional crossings have been incorporated into the model in the new reach upstream of the M56 motorway informed by the 2022 survey data and site photographs (provided in Annex B). In addition, a minor update has been made to the existing farm crossing at node GAL_3470, in association with the section data update detailed in **Table 3.3**.

3.3.9 **Table 3.4** details the new structures represented, and edits made, including a number of changes to the 1D spill units which have been replaced with a 2D representation.

Table 3.4 Model structures

Node ID	Structure description	Unit	Comments
New structures			
GAL_3780	Farm crossing, approximately 115m upstream of the M56 motorway crossing.	Bridge unit	Invert levels taken from 2022 survey cross section IDs 28 and 29. The structure dimensions were not explicitly surveyed. Soffit level has been estimated based on review of survey photographs (see Photo 1, Annex B), assuming a 500mm cover level from the deck level.
GAL_3666	M56 motorway culvert	Conduit (with culvert inlet and outlet)	Upstream invert level estimated based on 2022 survey section ID 30 (10m upstream of culvert) and based on the upstream gradient. Downstream invert level has been taken from 2022 survey section ID 31. Dimensions estimated (deemed to be 1m box culvert) based on survey photographs (see Photo 3, Annex B), given that the structure could not be surveyed given safety concerns and proximity to the motorway.
Updated structures			
GAL_3470	Farm crossing, approximately 160m downstream of the M56	Bridge unit	Section data updated in accordance with 2022 survey data (section ID 33) reflecting a higher bed level, likely indicating siltation since the 2019 survey.

Node ID	Structure description	Unit	Comments
	motorway crossing.		
GAL_3048cu	Cryers Bridge	Conduit (with culvert inlet and outlet)	1D spill units removed and replaced with 2D representation to improve representation of spill flood mechanisms.
GAL_2303cu **GAL_2303cd**	Hill View Way Road (A5117) Road Bridge	Conduit section (with culvert inlet and outlet)	The short (5m) open channel section between the A5117 road bridge and unnamed bridge #1 has been removed and represented with an extension of the A5117 culvert to improve model stability. Chainage has been added to GAL_2303cd to represent this short section. Units GAL_2266cu and GAL_2266cd have been replaced with symmetrical conduit units, to match the dimensions of the A5117 culvert (GAL_2303cu and GAL_2303cd) which is more constricting to flow. In addition, a flat gradient has been assumed across the length of the extended culvert modelled. 1D spill units associated with the A5117 road bridge have been removed and replaced with a 2D representation to improve representation of flooding mechanisms. Spill units associated with unnamed bridge #1 have been removed in accordance with the removal of the open channel section.
GAL_2266cu **GAL_2266cd**	Unnamed bridge #1 (Stanlow Oil Refinery)	Conduit (with culvert inlet and outlet)	
GAL_2171cu	Unnamed bridge #2 (Stanlow Oil Refinery)	Conduit (with culvert inlet and outlet)	1D spill units removed and replaced with 2D representation to improve representation of flooding mechanisms.

Roughness

3.3.10 In-channel Manning's n roughness values remain largely unchanged from the original model. Values of 0.05 to 0.07 were used for channel bed values, which

were deemed to be representative based on reference to standard guidance and site photographs provided in **Annex B**.

2D model

Extent

3.3.11 The extent of the 2D model associated with the Gale Brook in the existing 2019 Stanlow model has been increased to add 2D floodplain representation across the entire modelled reach of the Gale Brook. It comprises of the reach between downstream of the M56 motorway, which was previously mostly represented in 1D only, and the reach upstream of the M56 motorway, that has been added as part of this study.

3.3.12 The 2D model boundary was also extended along the M56 motorway, as shown in **Graphic 3.2**. This enables the representation of floodwater from the Gale Brook surcharging the culvert under the M56 motorway flowing southwest across the natural catchment divide along the motorway which is recessed into the landscape and acts as a flow channel out of the Gale Brook catchment. As stated in Section 3.2, a 2D boundary has been added here to prevent glasswalling of floodwater.,

3.3.13 At the upstream extent of the 2D model, floodwater has been allowed to glasswall against the model boundary, since it would not be appropriate to allow for floodwater to leave the model here.

Grid size

3.3.14 In order to improve the representation of floodplain flow, the cell size was reduced from 6m in the original model to 4m.

1D/2D link

3.3.15 As detailed in the sub-section above, the 1D modelled cross sections have been trimmed throughout the Gale Brook modelled reach to account for the 2D floodplain update. Therefore, 1D/2D linking layers have been updated accordingly, accounting for the refined 1D channel extent modelled where the previous extended cross sections representing the floodplain have been deactivated. Updated layers associated with this update are detailed in **Table 3.5**.

Table 3.5 1D/2D link updates

New layer	Reasoning
1D_isis_nodes_GalB_079_P.shp	Supersedes ‘1d_isis nodes_GalB_004.MIF’, including additional 1D model nodes included in this study.
2d_bc_hxi_GB_079_L.shp	Revised HX and CN link layers associated with the updated 1D/2D representation, superseding ‘2d_bc_hxi_GB_003.MIF’.

New layer	Reasoning
2d_bc_inactive_GB_079_R.shp	Supersedes '2d_bc_inactive_GB_002.MIF', in accordance with the revised 1D/2D extents in this study.

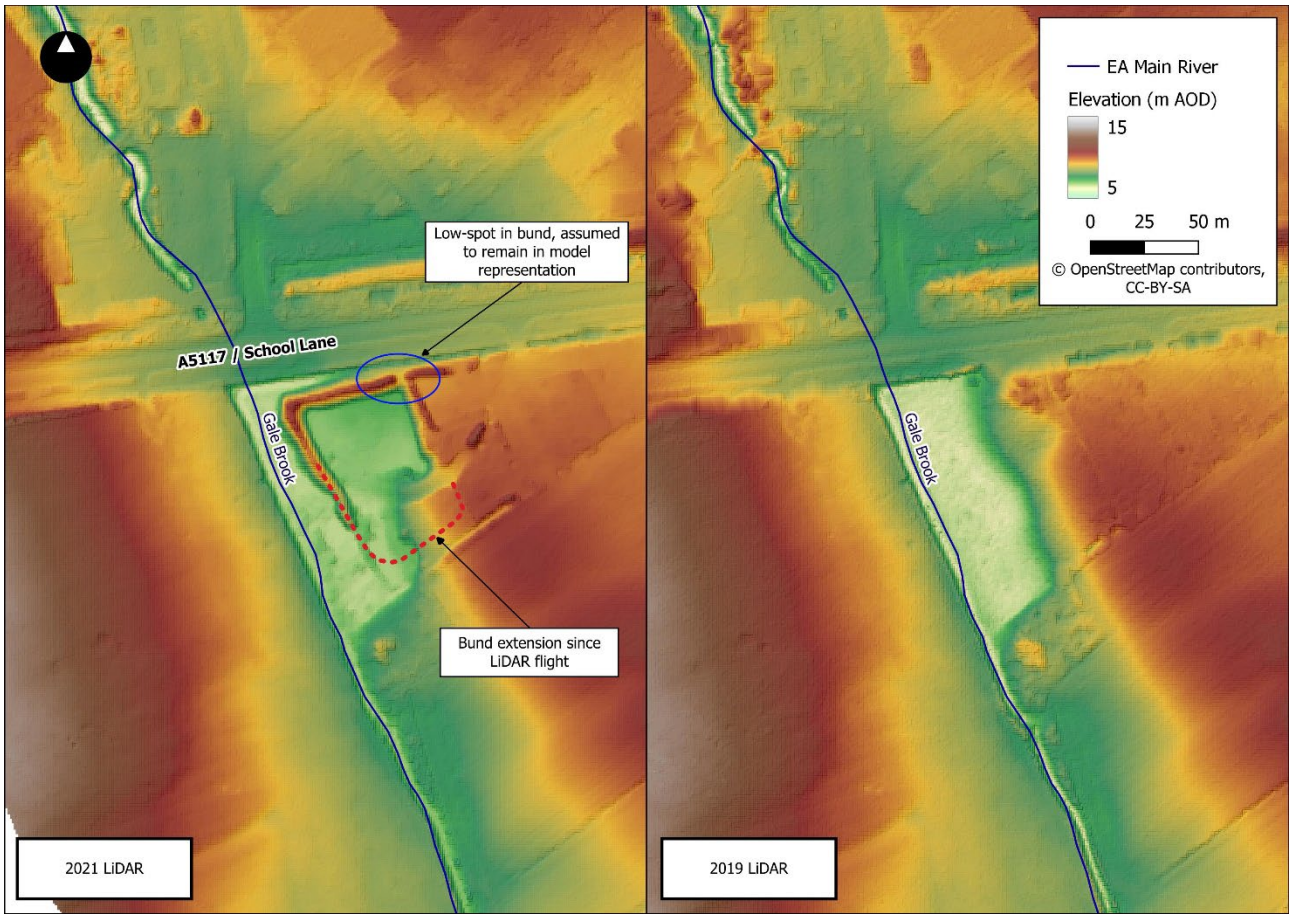
Topography

3.3.16 The 2D model topography informing the Gale Brook modelled section has been updated to be based on the latest available EA LiDAR data, understood to have been flown in February 2021. This ensures that the model is based on the most up to date topographic data reflective of any recent topographic modifications associated with recent developments and excavations, for example.

3.3.17 It was noted through review of historic aerial imagery and recent LiDAR data that there have been recent topographic modifications on the right bank of the Gale Brook immediately south of the A5117 Road Bridge associated with the Little Meadow Caravan Park. Most notably, a bund has been created within the watercourse floodplain, understood to have been built in early 2021. This is partly reflected in the latest LiDAR data shown in **Graphic 3.3**, which was flown mid-way through the works. Based on review of aerial imagery taken later in 2021 (shown in **Graphic 3.4**), the bund appears to have been developed further on the southern boundary to fully encapsulate the land parcel within it. The 2021 LiDAR shows a dip to 9.23mAOD in the bund in the north-eastern corner of the bunded area.

3.3.18 However, as indicated in **Graphic 3.4**, most recent aerial imagery taken in March 2022 indicates what appears to be a cut through the bund (circled red), which would allow for some floodplain storage. It is not possible to discern if the dip in the north-eastern corner remains.

Graphic 3.3 Little Meadow Caravan Park bund – LiDAR



Graphic 3.4 Little Meadow Caravan Park bund – aerial imagery

3.3.19 Given the timescales and scope of this study, it has not been possible to determine any further details or future plans regarding this bund. To assume a worst-case conservative approach, the bund has been assumed to be fully impermeable, without allowance for the cut through noted in the latest satellite imagery. The dip in the north-eastern corner of the bunded area has been assumed to remain.

3.3.20 To represent the complete bund (as seen in the April 2021 aerial imagery, Figure 3.4), new z-shape features have been introduced into the model, detailed in **Table 3.6**.

3.3.21 Several topographic z-shape features were implemented in the original Stanlow 2019 model to reinforce the 1D/2D link (bank) elevation in the model and to define the bridge deck of GAL_2171cu in the Stanlow Oil Refinery. These layers have been removed and are outlined in Table 3.6 with associated reasoning.

3.3.22 Additional z-shape features have been added to remove the low spot in the DTM associated with the short open channel section downstream of the A5117 road bridge, removed from the model as detailed in **Table 3.3** and **Table 3.4**. This is to avoid the double counting of storage between the 1D and 2D model domains.

Table 3.6 Updated z-shape features

Layer name	Reasoning
Removed layer(s)	
2d_zsh_GB_Banktops_001_ptp.MIF	Based on attribute data, banktop elevations appear to be based on 2019 LiDAR (contrary to the geometry control file notes suggesting survey data). Therefore layer was removed to allow for the updated model bank elevations to be based on the latest LiDAR data.
2d_zsh_GAL2171_001.MIF	As indicated in the geometry control file notes, Z-shape feature used to 'smooth' the bridge deck representation. Values appear to be based on 2019 LiDAR values and the bridge deck is represented well within the latest EA LiDAR data used in this model update. Therefore, feature has been removed.
New layer(s)	
2d_zsh_GB_bund_069_L.shp	Z-shape feature to represent the bund noted on the right bank of the Gale Brook upstream of the A5117 road bridge. Crest elevations are based on the LiDAR elevations recorded on the western portion of the bund which is captured in the 2021 LiDAR.
2d_zsh_A5117_open_channel_078_R.shp **2d_zsh_A5117_open_channel_078_P.shp**	Z-shape features to remove localised low spot from the 2D model topography associated with the short open-channel section downstream of the A5117 road

Layer name	Reasoning
	bridge, removed from the model to avoid double counting of storage. Elevations have been interpolated from the perimeter of the polygon to result in a smooth 2D surface.

Sensat survey

3.3.23 The Sensat survey has a higher resolution than the EA LiDAR DTM, but its extent is limited to the project's Red Line Boundary and includes buildings and trees, rather than the bare earth surface of the DTM. It therefore was not used directly in the model.

3.3.24 A comparison between the Sensat DSM and the EA LiDAR DTM for the site showed that elevations correspond well with each other for areas without buildings and trees. This gives additional confidence in the representation of the site within the model. IT confirms that the site elevations range from 9.4mAOD in the north-west to 11.8mAOD in the south-west.

Roughness

3.3.25 Roughness classification remains based on the OS MasterMap data used in the original 2019 Stanlow model, where available. Upstream of the M56, further land cover data was needed to delineate the land cover classification given that this was outside of the previous model and OS MasterMap data extent. Ordnance Survey LocalMap data has been used to delineate 2D landcover in this short reach, and using the same roughness classifications as in the existing 2019 Stanlow model for consistency.

Miscellaneous updates

3.3.26 Various miscellaneous updates have been made to the 2D model control files, as listed in **Table 3.7.** In addition, output folders within each control file have been updated as necessary, configured to the local IT system.

Table 3.7 Miscellaneous 2D model updates

Item / Command	Comments
Additions	
SHP Projection == ..\model\shp\Projection\Projection.shp	New command added to set the projection for new shapefile (.shp) layers to British National Grid (same as existing .MIF files).

Item / Command	Comments
GIS Projection Check == WARNING	New command to overcome any errors associated with projection issues using MIF and SHP formats combined.
Snap Tolerance == 0.01	New command to overcome snapping issues associated with projections of MIF and SHP formats.
SX FMP Unit Type Error == OFF	New command to allow for Flood Modeller interpolate sections to be linked to the 2D TUFLOW domain.
Boundary Viscosity Factor == 2	New command added to increase the eddy viscosity coefficient along external boundaries to stabilise the model.
Removed	
1d_WLLp_Gowy_005.shp **1d_WLL_Gowy_005.shp**	Water level lines (WLL) removed due to encountering TUFLOW error 1229 (thought to be associated with the newer model version used for this study)
GIS Format == MIF	Command removed given a combination of MIF and SHP format layers are used for this updated model

Modelling assumptions made

3.3.27 The reader should refer to the original Jacobs 2019 Stanlow modelling report for a full overview of assumptions associated with the underlying model. Of key relevance to this study, is the assumption that the Stanlow refinery culvert dimensions remain consistent throughout its length.

3.3.28 The key modelling assumptions introduced as part of this study and of key relevance are outlined below:

- It is assumed that the bund identified on the right bank of the Gale Brook upstream of the A5117 acts as a complete barrier to floodplain flow for conservatism. Crest elevations of the northwest portion of the bund represented in the 2021 LiDAR are assumed to be representative of the final crest elevation of the bund.
- The A5117 road bridge culvert (and associated constriction to flow) is assumed to act as the main control to local flooding mechanisms, and therefore, the removal of the short open-channel section downstream to improve model stability is justified. This assumption has been assessed further and justified as discussed in **Section 4.5**.

- Based on review of latest survey data collected in 2022 in support of this study, it is assumed that the pre-existing 2019 channel survey still provides a good representation of the channel geometry. Where direct comparisons can be made between the 2019 and 2022 survey data, higher bed elevations captured in the 2022 survey are assumed to represent siltation and have therefore been used to update the 2019 survey.
- The dimensions of the M56 culvert are un-surveyed, and therefore dimensions (1m box culvert) have been estimated based on available photographs and upstream invert level has been assumed based on the nearest upstream surveyed section and assuming the average upstream slope. Model sensitivity to the culvert has been assessed further in **Section 4.5**.

4. Model proving

4.1 Run parameters

4.1.1 Updated model runs have been carried out using Flood Modeller version 6.0 and TUFLOW version 2020-10-AF-iSP-w64. Design model runs and associated parameters are outlined in **Table 4.1**.

Table 4.1 Model run record and parameters

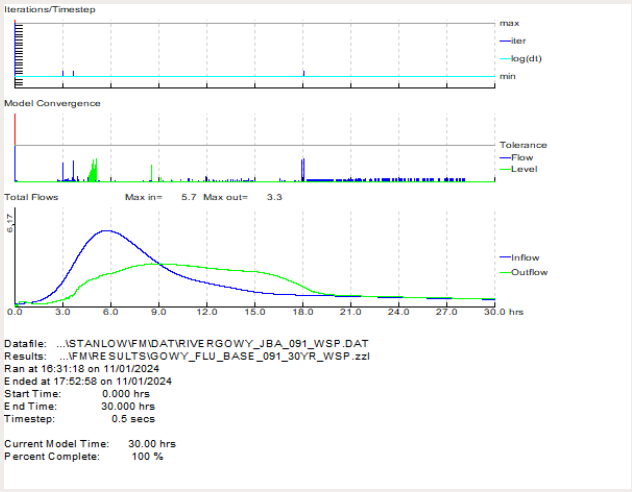
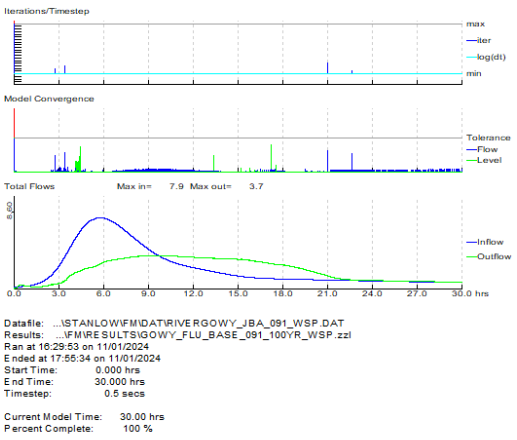
Item	Comments
Design Events (AEP %)	3.33 1 0.5 0.1 0.5 (HC – 67%) 0.5 (UE – 106%)
Scenario	Baseline fluvial
Run parameters	Model run parameters have been left unchanged from the original Stanlow model runs, with the exception of: - Dflood: 5 This was increased from the default value of 3 to improve model stability.
Flood Modeller network	RiverGowy_JBA_086_WSP.DAT
Flood Modeller event data	GB_ReFH_Q0030_WSP.IED GB_ReFH_Q0100_WSP.IED GB_ReFH_Q0200_WSP.IED GB_ReFH_Q0200CC67_WSP.IED GB_ReFH_Q0200CC106_WSP.IED GB_ReFH_Q1000_WSP.IED
Flood Modeller event files	GB_FLU_Base_086_0030yr_WSP.ief GB_FLU_Base_086_0100yr_WSP.ief GB_FLU_Base_086_0200yr_WSP.ief GB_FLU_Base_086_0200yrHC_WSP.ief GB_FLU_Base_086_0200yrUE_WSP.ief GB_FLU_Base_086_1000yr_WSP.ief
TUFLOW control files	Gowy_FLU_Base_086_0030yr_WSP.tcf Gowy_FLU_Base_086_0100yr_WSP.tcf Gowy_FLU_Base_086_0200yr_WSP.tcf Gowy_FLU_Base_086_0200yrHC_WSP.tcf Gowy_FLU_Base_086_0200yrUE_WSP.tcf Gowy_FLU_Base_086_1000yr_WSP.tcf

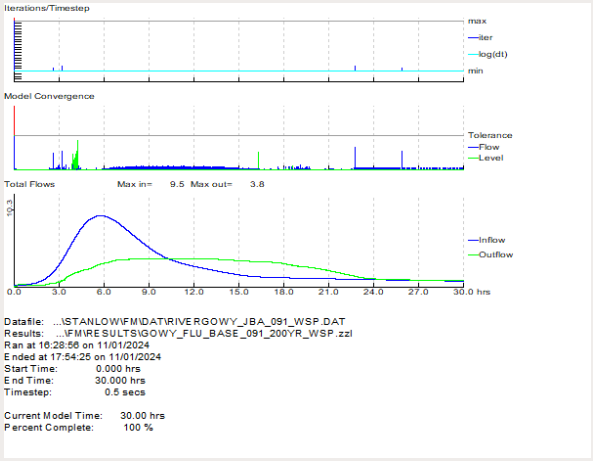
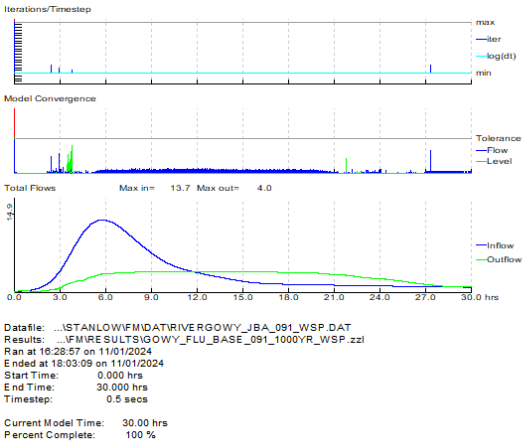
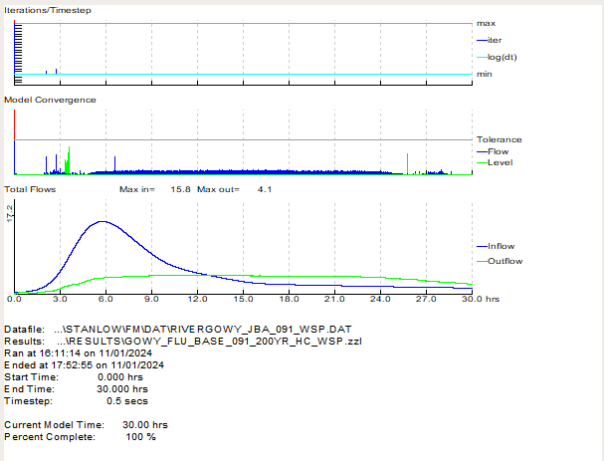
Item	Comments
Timestep	Floodmodeller: 0.5s TUFLOW: 1s

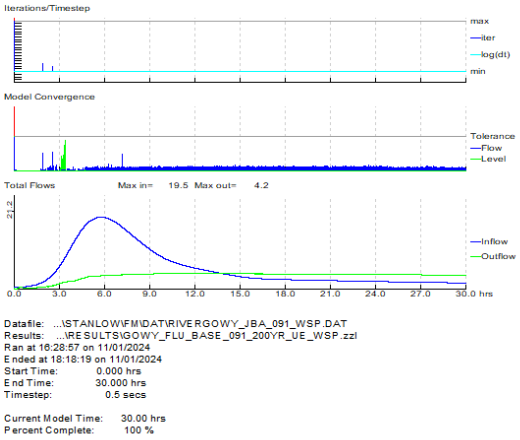
4.2 Run Performance

4.2.1 Run performance indicators for the design runs are provided in Table 4.2. In all cases the model is indicated as being stable and healthy.

Table 4.2 Model run performance

AEP (%)	TUFLOW Final Cumulative Mass Error (%)		Flood Modeller performance
	Whole simulation	Flow In + Out >5%	
3.33	-0.09%	-0.10%	
1	0.22%	0.34%	

AEP (%)	TUFLOW Final Cumulative Mass Error (%)		Flood Modeller performance
	Whole simulation	Flow In + Out >5%	
0.5	0.18%	0.24%	
0.1	0.10%	0.12%	
0.5 (HC – 67%)	0.08%	0.09%	

AEP (%)	TUFLOW Final Cumulative Mass Error (%)		Flood Modeller performance
	Whole simulation	Flow In + Out >5%	
0.5 (UE - 106%)	0.06%	0.07%	

4.3 Observations

4.3.1 The model results indicate that the M56 embankment and associated culvert serves as a significant hydraulic control though the modelled reach. Substantial out-of-bank flooding and attenuation upstream of the M56 embankment is anticipated in all modelled flood events. Adjacent to the Stanlow HAGI site, the A5117 road culvert also serves as a hydraulic control to floodwater, with overtopping anticipated in events of 3.33% AEP and greater. Floodwater is anticipated to flow north across the road, and into the industrial refinery site resulting in flooding of low-lying land west of the proposed HAGI location. However, the HAGI site remains outside of the flood extent for all modelled events.

4.3.2 As expected, the bund around the Little Meadow Caravan Park site excludes flood water from the site for all but the most extreme event. In the 0.5% AEP event with the upper end climate change allowance applied, some water ingresses via the dip in the bund in the north-eastern corner.

4.3.3 It is noted that there is some glasswalling at the upstream and downstream extents of the model in extreme events. At the upstream extent of the model, there is minor glasswalling of floodwater against the edges of the 2D model domain in events of 0.1% AEP and greater given the wide, low lying nature of the floodplain. Downstream of the Stanlow Oil Refinery, floodwater overtops the open channel section in events of 0.1% AEP and greater and glasswalls against the model extent to the north.

4.3.4 However, both instances are unlikely to have any impact to modelled flood levels at the Stanlow HAGI site, given the distances away from the area of interest and hydraulic controls along the reach (namely the M56 embankment and culvert, and the Stanlow Oil Refinery culvert). As shown via sensitivity testing of the

downstream boundary condition detailed further in Section 4.5 below, modelled results at the HAGI site are insensitive to the downstream boundary.

4.4 Calibration and Validation

4.4.1 No additional calibration analysis has been carried out as part of this study.

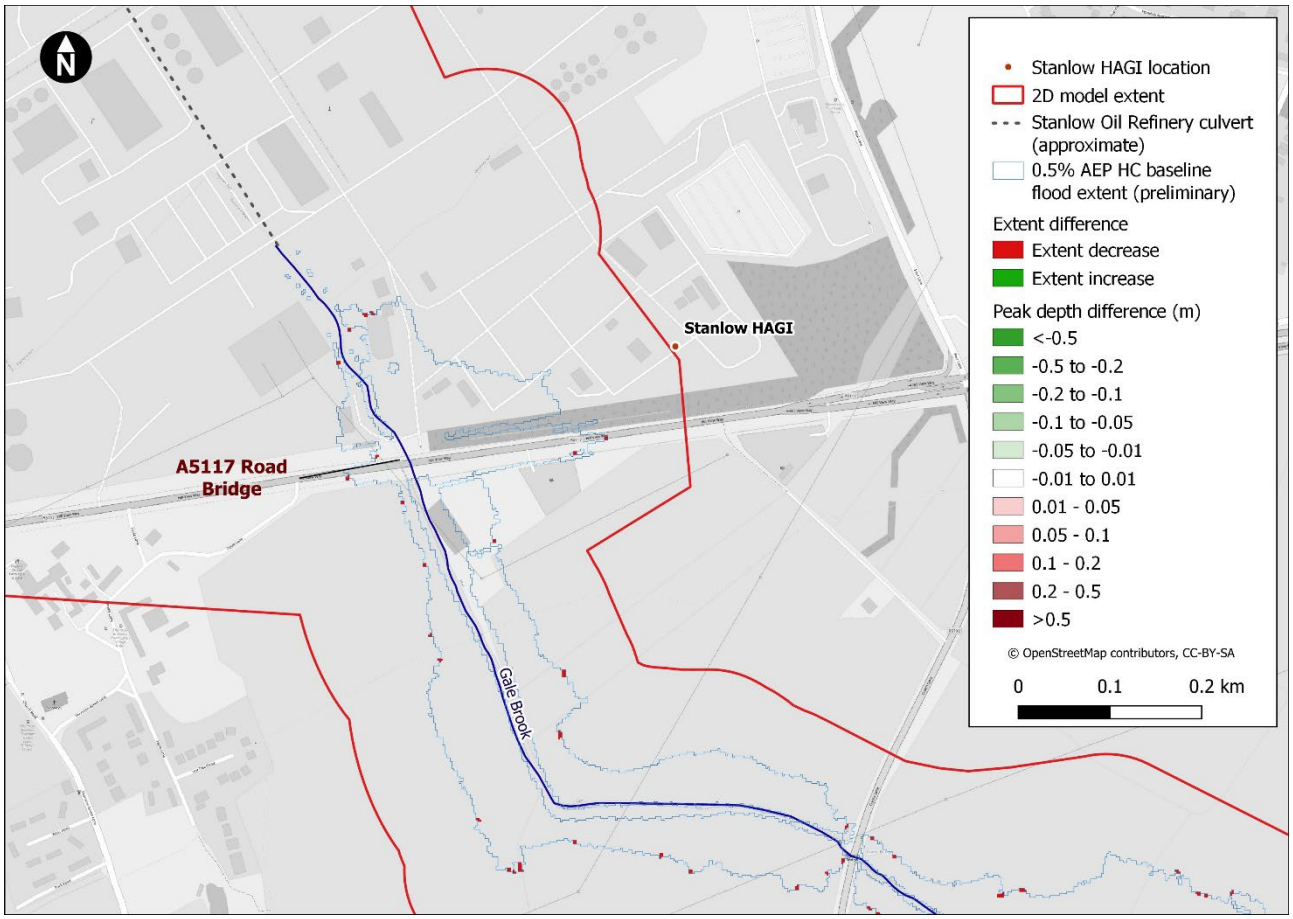
4.5 Sensitivity analysis

Tidal downstream boundary

4.5.1 Design fluvial runs have been run with a mean high-water spring (MHWS) tidal boundary, in accordance with the original 2019 Stanlow model. A joint probability assessment has not been undertaken for the purposes of this study, given the dominance of fluvial flood risk to the Stanlow HAGI. Furthermore, the Jacobs 2019 model report stated various combinations of tidal and fluvial events were tested, and that flooding from combined sources was not significantly different from individual fluvial or tidal contributions.

4.5.2 To provide confidence in this statement, a sensitivity test has been run through the updated model to assess the impact of the tidal downstream boundary to flood risk at the HAGI site. The fluvial design 0.5% AEP plus 67% climate change (Higher Central) event has been run with a 1% AEP tidal downstream boundary (rather than the baseline MHWS), to assess the sensitivity of model results adjacent to the HAGI site. The impact of this has been assessed using depth and extent difference comparisons relative to the fluvial only baseline, as shown in Figure 4.1. Peak flood depths adjacent to the HAGI site show negligible difference (+/- 0.01m, hence no depth difference is mapped) and peak flood extents are also comparable with only minor differences anticipated. It can be concluded therefore that the key flood risk to the HAGI site is associated with fluvial flood risk from the Gale Brook, and flooding mechanisms are insensitive to the tidal downstream boundary. Further joint probability assessment is therefore not deemed necessary.

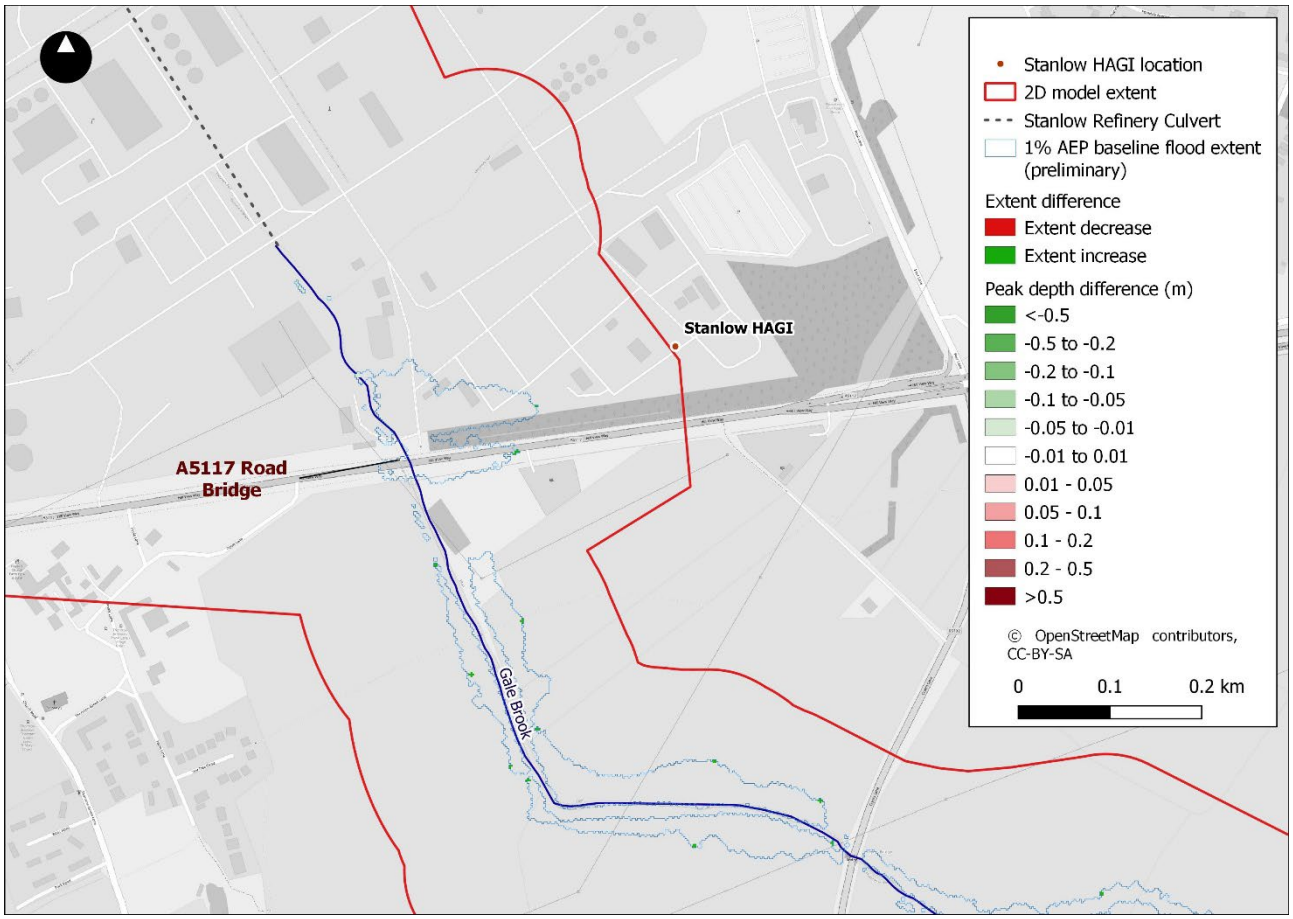
Graphic 4.1 Tidal boundary sensitivity test – peak depth difference



A5117 open-channel

4.5.3 As described in Table 3.3 and Table 3.4, the short (5m) open channel section of the watercourse immediately downstream of the A5117 road bridge has been removed from the model due to model instabilities in high magnitude events. In order to provide confidence that this update (and reduced model detail) has limited impact to the local model results, the 1% AEP event has been run with the open channel section included to be assessed relative to the updated baseline results (with open channel section represented as a culvert). The impact of this model update has been assessed using a depth and extent difference map, shown in Figure 4.2. Assessment of higher magnitude events is not possible given that the associated higher flows cause the model to fail.

Graphic 4.2 A5117 open-channel sensitivity test – 1% AEP peak depth and extent difference



4.5.4 Peak flood extents and depths are comparable in both model scenarios, with flood depth differences primarily between +/- 0.01m. The overarching control to local flooding mechanisms is the A5117 culvert capacity, resulting in overtopping of the road bridge on the right bank. Flows travel north along an access road into the Stanlow Oil Refinery and re-join the channel downstream of the unnamed access road bridge. Therefore, the replacement of the short open-channel section with an extended culvert has negligible consequence to the validity of the model outputs and is deemed to provide a suitable representation for this assessment.

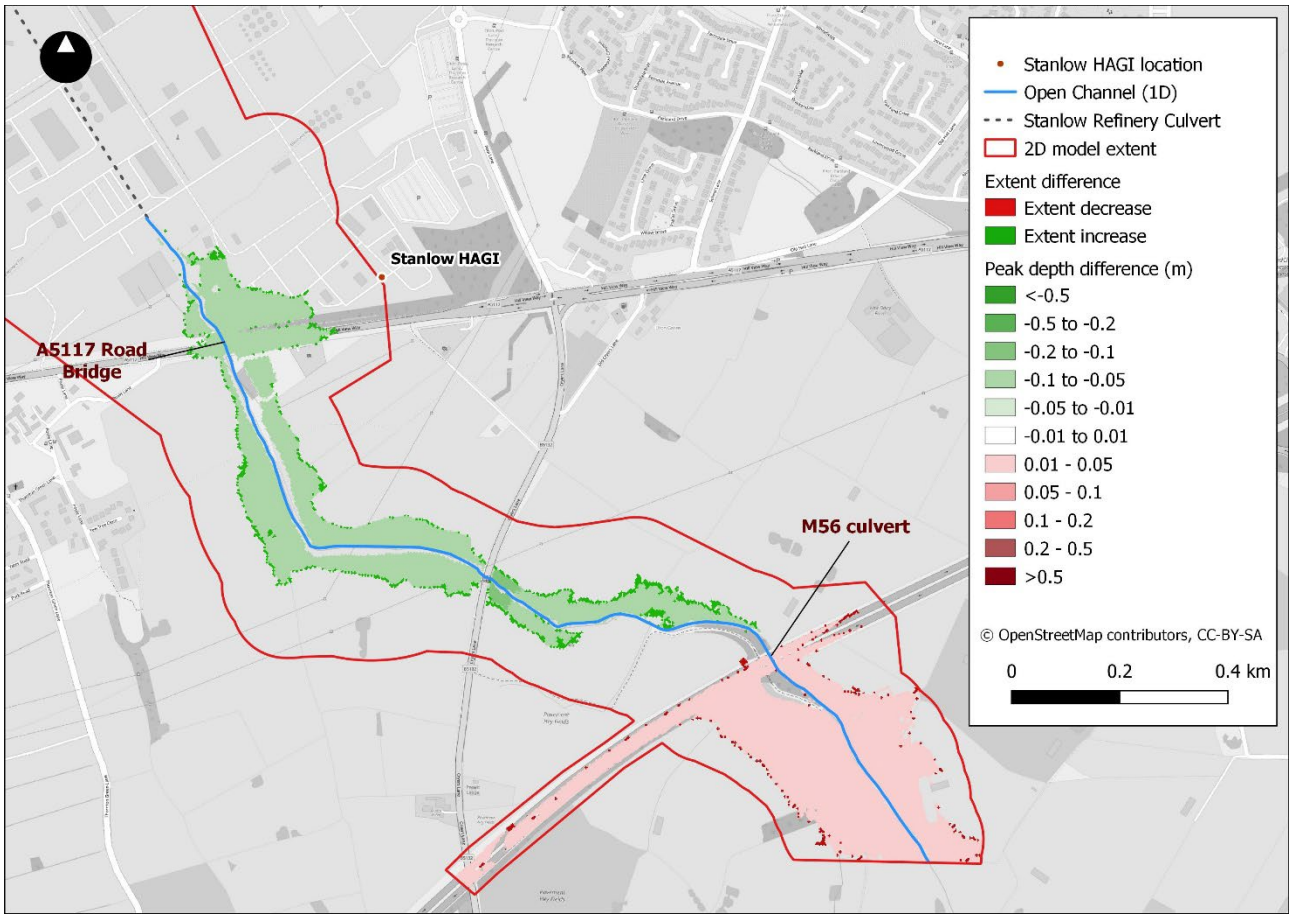
M56 culvert

4.5.5 The M56 culvert dimensions have been estimated based on site photographs (see Photos 2 and 3, Annex B), since the structure could not be surveyed due to safety concerns and proximity to the M56 motorway. The structure acts as a significant hydraulic control in attenuating downstream flow, resulting in widespread flooding upstream. Given the uncertainty with regard to model representation, a sensitivity test has been run to assess the sensitivity of model results to a change in the culvert dimensions.

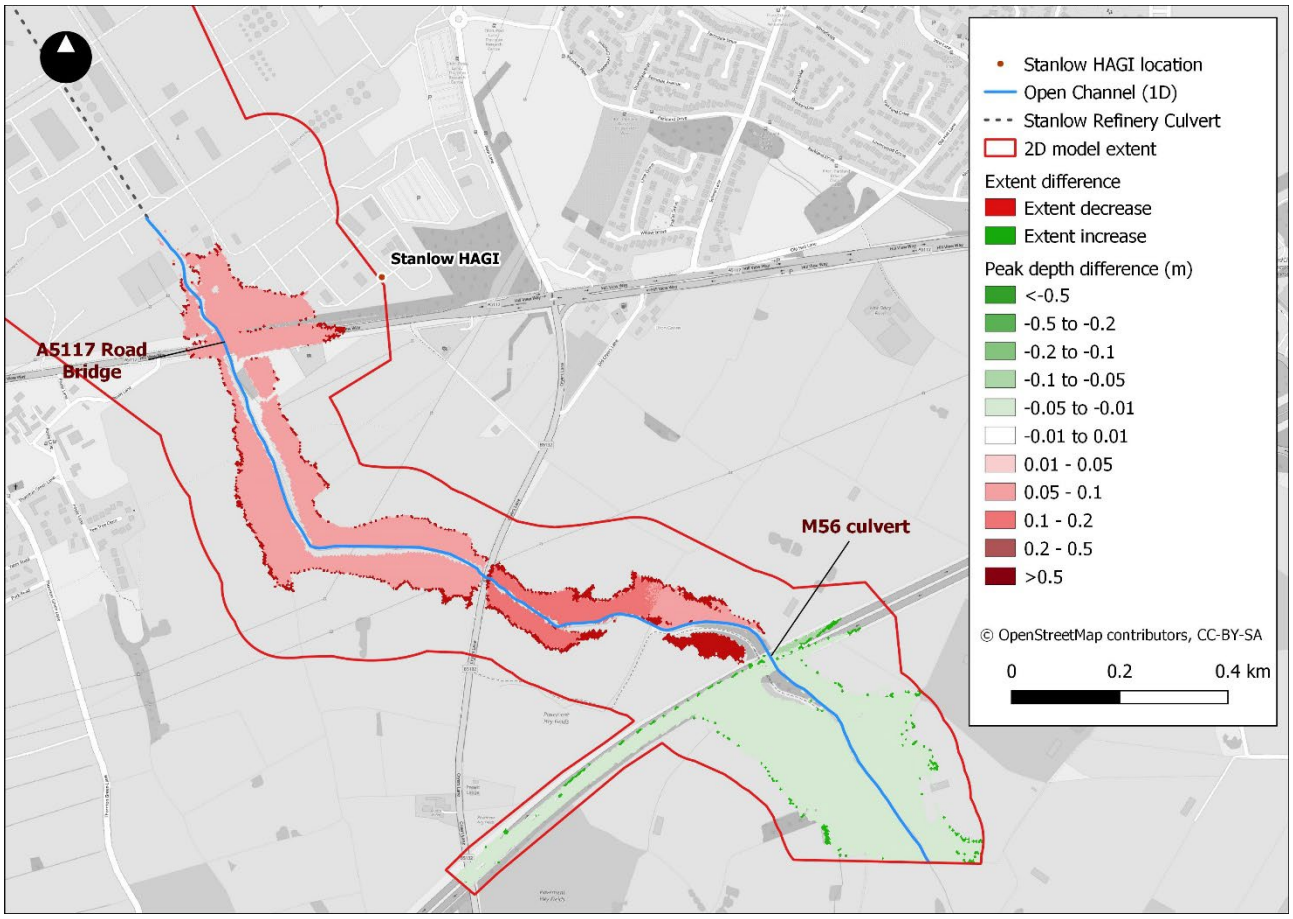
4.5.6 The model has been run using the design 0.5% AEP plus 67% climate change event considering a 0.8m x 0.8m box culvert and a 1.2m x 1.2m box culvert, representing (approximately) a 40% reduction/increase in flow area, respectively. The impact of each scenario has been assessed using a 2D depth and extent

difference map, relative to the updated baseline results. The impact of the reduced culvert capacity is shown in Figure 4.3, and the impact of the increased culvert capacity in Figure 4.4.

Graphic 4.3 M56 culvert reduced capacity – 0.5% AEP HC peak depth and extent difference



Graphic 4.4 M56 culvert increased capacity – 0.5% AEP HC peak depth and extent difference



4.5.7 As anticipated, the reduced culvert capacity results in increased attenuation of floodwater upstream, resulting in an increased peak flood depth of 0.01-0.02m upstream. As a result, peak flood depths downstream of the M56 are reduced by 0.08-0.09m resulting in some minor reductions in peak flood extent.

4.5.8 Conversely, the increased culvert capacity results in reduced flood depths of 0.02-0.03m upstream of the M56 and an increase of 0.09-0.11m downstream. Peak flood extents immediately downstream of the M56 are notably greater than the baseline, though at the HAGI location the flood extent increases are minor.

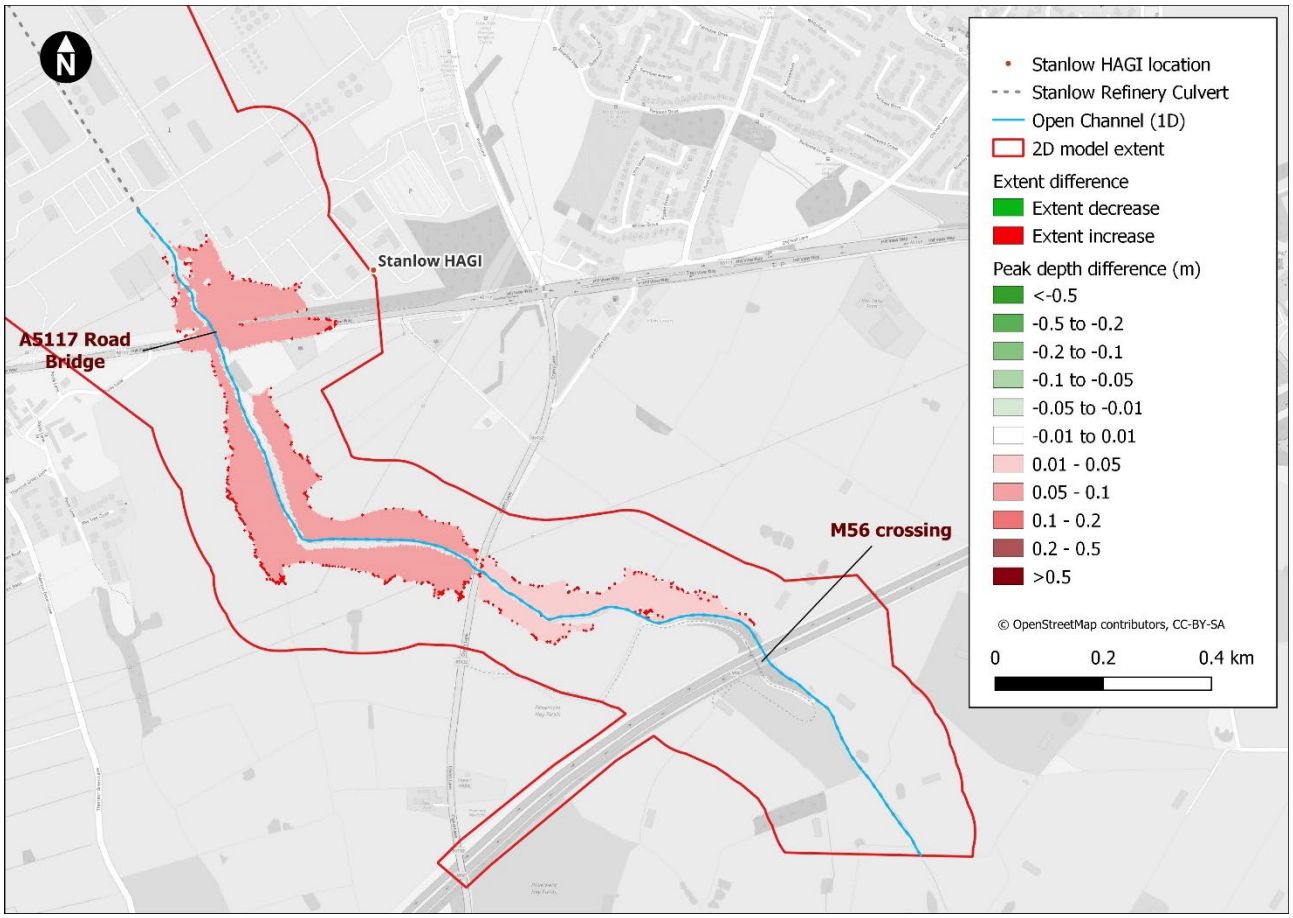
4.5.9 The sensitivity testing undertaken here demonstrates that the M56 culvert acts as a significant control to flooding mechanisms on the Gale Brook watercourse upstream and downstream of the culvert structure. However, in either scenario the Stanlow HAGI location remains outside of the design flood extent.

Stanlow refinery culvert blockage

4.5.10 The baseline model representation assumes that the Stanlow Refinery culvert remains free of blockages throughout the simulation. The structure acts as a key hydraulic control to flooding mechanisms immediately upstream of the structure and extending beyond the Stanlow HAGI site, upstream of the A5117 Road Bridge. Therefore, a blockage scenario has been simulated to assess the sensitivity of modelled flood levels adjacent to the HAGI site to a blockage in the

refinery culvert. A 50% blockage has been assumed, through addition of a trash screen at the upstream culvert inlet unit. The model has been run using the design 0.5% AEP plus 67% climate change event and assessed relative to the baseline results. A depth and extent difference comparison is shown in Figure 4.5.

Graphic 4.5 Stanlow Refinery culvert blockage - 0.5% AEP HC peak depth and extent difference



4.5.11 The blockage scenario results in peak 2D flood depth increases of up to 0.06m immediately upstream of the structure, with more minor impacts extending up to the M56 embankment. The blockage scenario results in minor increases in peak flood extent throughout this reach, however, the Stanlow HAGI site remains outside of the flood extent.

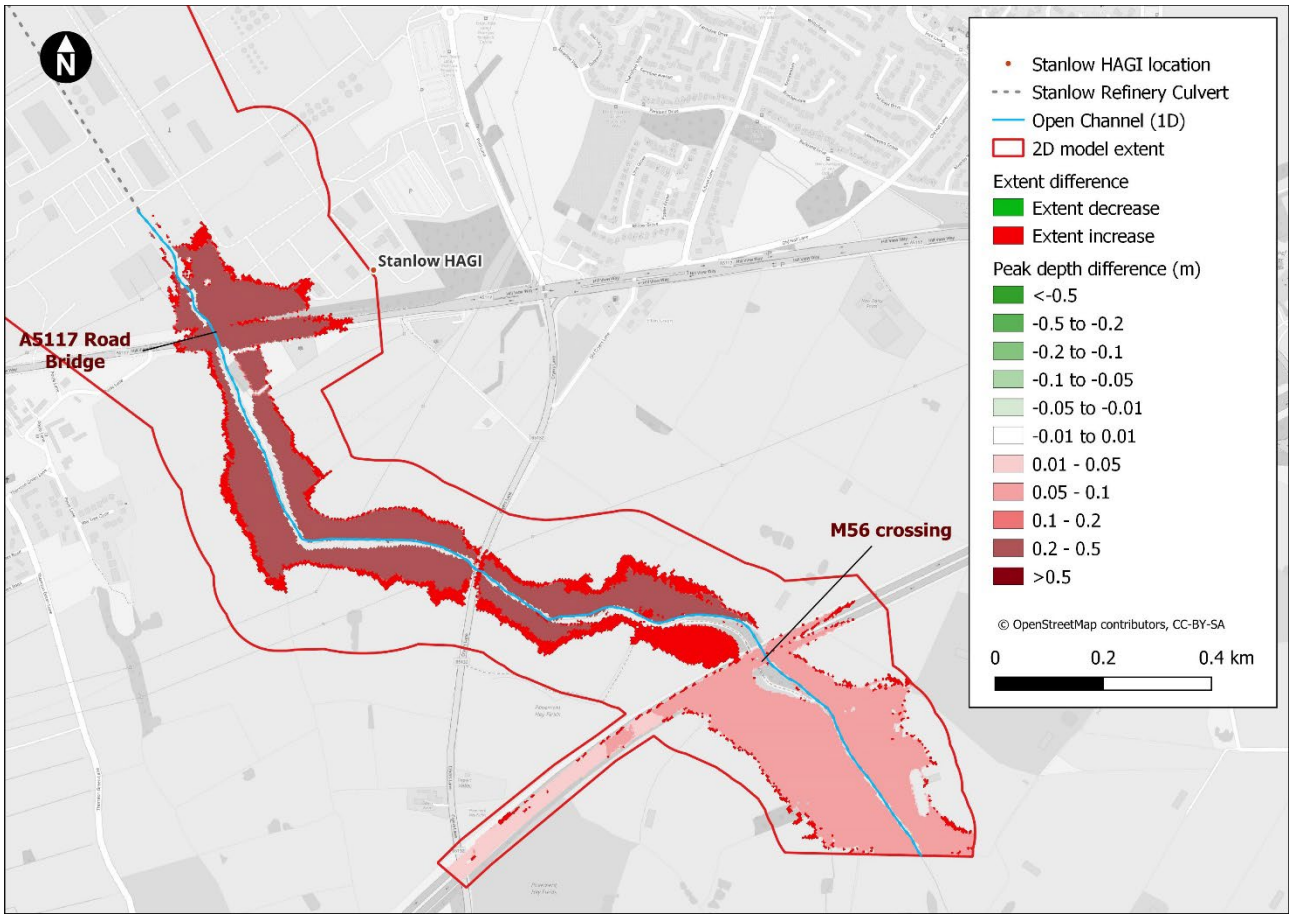
Inflow

4.5.12 The magnitude of inflows is typically the biggest uncertainty in flood modelling. Increasing the inflow peak of the design event by 20% is a common sensitivity test that helps bound the upper end of this uncertainty. Instead of a test with the peak of the 0.5 AEP + 67% (HC) design event increased by 20%, the 0.5% AEP + 106% (UE) was used as a proxy, as its peak value is 23% higher than that of the design event.

4.5.13 A depth and extent difference comparison is shown in Figure 4.6. Impact to modelled flood depths and extent is relatively limited upstream of the M56

embankment and culvert. However, peak flood depths downstream of the M56 embankment are typically 0.20-0.25m greater in the 0.5% AEP + 106% (UE) event. The associated peak flood extent shows relatively unform increases throughout this reach, however, the Stanlow HAGI site remains outside of the flood extent.

Graphic 4.6 Inflow sensitivity - 0.5% AEP UE peak depth and extent difference



Roughness

4.5.14 The model's sensitivity to changes in channel and floodplain roughness has not been formally tested. Typically, hydraulic models show some variations to peak flood flows and levels in response to changing roughness parameters to plausible minimum or maximum values. But as the expected variations in flood peaks would not put the HAGI site at risk, no separate roughness sensitivity test was performed.

5. Conclusions and recommendations

5.1 Conclusions

5.1.1 The existing EA-owned Stanlow model has been updated to provide an improved representation of flood risk along the Gale Brook to support the HyNet flood risk assessment. The existing model representation of the Gale Brook (which provides the overarching flood risk to the Stanlow HAGI site) was deemed to be insufficient for the purposes of this site-specific assessment.

5.1.2 The model extent has been extended upstream of the M56 (the previous upstream model extent) to model the attenuation impacts associated with the embankment and culvert. The new model reach has been represented based on channel survey data commissioned as part of this project and collected in 2022. Elsewhere throughout the Gale Brook modelled reach, the 2022 survey data has been used to verify the validity of the existing 1D channel representation and update the model where deemed necessary. The modelled reach has been updated to incorporate a full 1D/2D representation of the floodplain (based on the latest EA LiDAR data) to provide an accurate representation of flooding mechanisms through the reach. A conservative approach has been taken for the purpose of this study to assume that the floodplain bund noted upstream of the A5117 is complete, preventing throughflow and floodplain storage (unless the bund is overtopped).

5.1.3 The model files and networks representing the wider River Gowy, Thornton Brook, Mill Brook and the N38C interceptor channel have been excluded from this model update for efficiency. This is justified on the basis that flood risk adjacent to the Stanlow HAGI site is insensitive to downstream conditions in the River Gowy and dominated by fluvial flood risk associated with the Gale Brook only, as demonstrated through sensitivity testing.

5.1.4 New design hydrological inflows have been developed as part of this assessment, in accordance with the revised model extent and on the basis that the previous ReFH1 inflows did not represent current best practice. New hydrological inflows are over 100% greater than those previously modelled through the Gale Brook run in the Stanlow model, which considered a critical storm duration for the wider River Gowy which was the focus of the previous study.

5.1.5 The updated model developed in this study has been run with for the revised 3.33%, 1%, 0.5%, 0.1% and 0.5% AEP plus climate change (Higher Central and Upper End) events. In all cases, the Stanlow HAGI location remains outside of the flood extent and therefore no floodplain compensation is considered necessary. Resultant model performance for the updated model runs is within the acceptable bounds.

5.2 Recommendations

5.2.1 The following recommendations can be made in order to improve model representation and accuracy of the results:

- As noted in the original Jacobs 2019 report, it is recommended that survey of the Gale Brook culverted section should be undertaken in order to confirm the internal dimensions of the structures throughout its reach.
- For the purposes of this assessment, a conservative approach has been taken with regards to the floodplain bund noted upstream of the A5117 to assume that the bund is a complete barrier to flow, with the exception of the slight drop in the north-western corner of the bund. It is recommended that any future studies should consider liaison with the local planning authority to understand the nature and purpose of the bund, and consider new survey (and/or LiDAR) data to inform on the model representation.
- It is recommended that the M56 culvert is surveyed as part of any future studies, if it can be managed safely. Culvert dimensions and the upstream, invert level have been estimated as part of this assessment. Sensitivity testing of the culvert dimensions reveals that model results throughout the Gale Brook are particularly sensitive to the capacity of the culvert.

Appendix A
FEH Calculation Record

Flood estimation report: Stanlow HAGI, Hynet.

Introduction

This calculation record is based on a template provided in the Environment Agency's flood estimation guidelines (version 9, 2023). It provides a record of the hydrological context, the calculations and decisions made during flood estimation and the results for the Gale Brook, Stanlow, Cheshire. This assessment was carried out to provide the inflow boundaries for a hydraulic model to evaluate flood risk for the proposed Stanlow Hydrogen Above Ground Infrastructure (HAGI), which is part of the HyNet North West Hydrogen Pipeline project.

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Approval

Revision stage	Analyst / Reviewer name & qualifications	Amendments	Date
Method statement preparation	N/A		
Method statement sign-off	N/A		
Initial calculations preparation	Jack Park		January 2023
Initial calculations sign-off	Stephen Anderton		January 2023
Calculations - Revision 1 preparation	Jack Park		January 2023
Calculations - Revision 1 sign-off	Stephen Anderton		January 2023
Calculations - Revision 2 preparation			
Calculations - Revision 2 sign-off			

Abbreviations

AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
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
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 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 SUMMARY OF ASSESSMENT

1.1 Summary

Catchment location	Gale Brook, Stanlow
Purpose of study and scope	This assessment has been carried out to produce updated peak flows and hydrographs for the Gale Brook for input to a hydraulic model, to provide an updated baseline assessment of flood risk to the Stanlow HAGI which is situated on the right bank of the watercourse. An existing Environment Agency (EA) owned hydraulic model has been used as a basis of the assessment of flood risk to the HAGI site, which was developed by Jacobs in 2019[1]. However, for the purposes of the Stanlow HAGI flood risk assessment, the model has been updated to incorporate full 1D/2D floodplain links for the Gale Brook and also extended upstream. Existing hydrological inputs to the Gale Brook section of the model therefore need to be updated.
Key catchment features	Small, largely rural upstream of the Stanlow refinery, though extensively urbanised in the lower reaches. The watercourse is culverted beneath the Stanlow oil refinery, for a length of approximately 1.3km.
Flooding mechanisms	Fluvial/ordinary watercourse
Gauged / ungauged	Ungauged
Final choice of method	ReFH2
Key limitations / uncertainties in results	The main limitations in this study are the lack of gauged data to aid hydrological or hydraulic model calibration and the uncertainty with regards to the sub-surface drainage catchment and outlet point(s) across the Stanlow refinery, for which no data has been made available for this study. Peak flow estimates have been carried out using ReFH2 based solely on the catchment descriptors, and the ReFH2 hydrograph shape is assumed to be representative of the catchment response

1.2 Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence interval. Results tables in this document contain both return period and AEP titles.

The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

[1] Jacobs (2019). Stanlow and Tranmere Flood Risk Management Scheme. Stanlow and Tranmere Hydraulic Modelling Report. Environment Agency.

2 METHOD STATEMENT

2.1 Requirements for flood estimates

Overview	This study has been undertaken to provide updated hydrographs for a range of flood events (100% AEP to 0.1% AEP) for input into an updated EA owned hydraulic model of the Gale Brook (and River Gowy). Existing hydrological inputs to the River Gowy and other watercourses represented in the model remain unchanged from the original model. However, given the modelled section of the Gale Brook has been updated and extended upstream (to provide an improved representation of flood risk to the Stanlow HAGI site), updated hydrological inputs are required as input to the model. In addition, the pre-existing Gale Brook hydrographs had been parameterised with a critical duration of 45.5-hours consistent with the River Gowy, given that this was the main focus of the original modelling study. Existing hydrographs for the Gale Brook therefore do not represent the most conservative estimate of flood risk for that watercourse, and therefore for the Stanlow HAGI site. 'Higher Central' and 'Upper End' climate change allowances of 67% and 106%, respectively, have been applied in accordance with the EA climate change guidance[2] for Nationally Significant Infrastructure Projects. Flow estimates have been generated for three flow estimation points (FEPs), following a similar approach to the existing modelling study. These are at the upstream extent of the updated model (345850, 373850), at the start of the culverted section beneath the Stanlow refinery (375040, 344400), and at the confluence to the River Gowy (376200, 343060). The latter two flow estimates are applied as lateral inflows, whilst the former is applied as a lumped inflow at the upstream extent of the model. This document summarises the analysis undertaken.
Project scope	A simple ReFH2 methodology has been employed, given the size of the catchment and scope of the project. The catchment is ungauged, and the small size of the catchment and subcatchments (<5km^2) limits the applicability of the FEH statistical method.

[2] https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances (Accessed January 2023)

2.2 The catchment

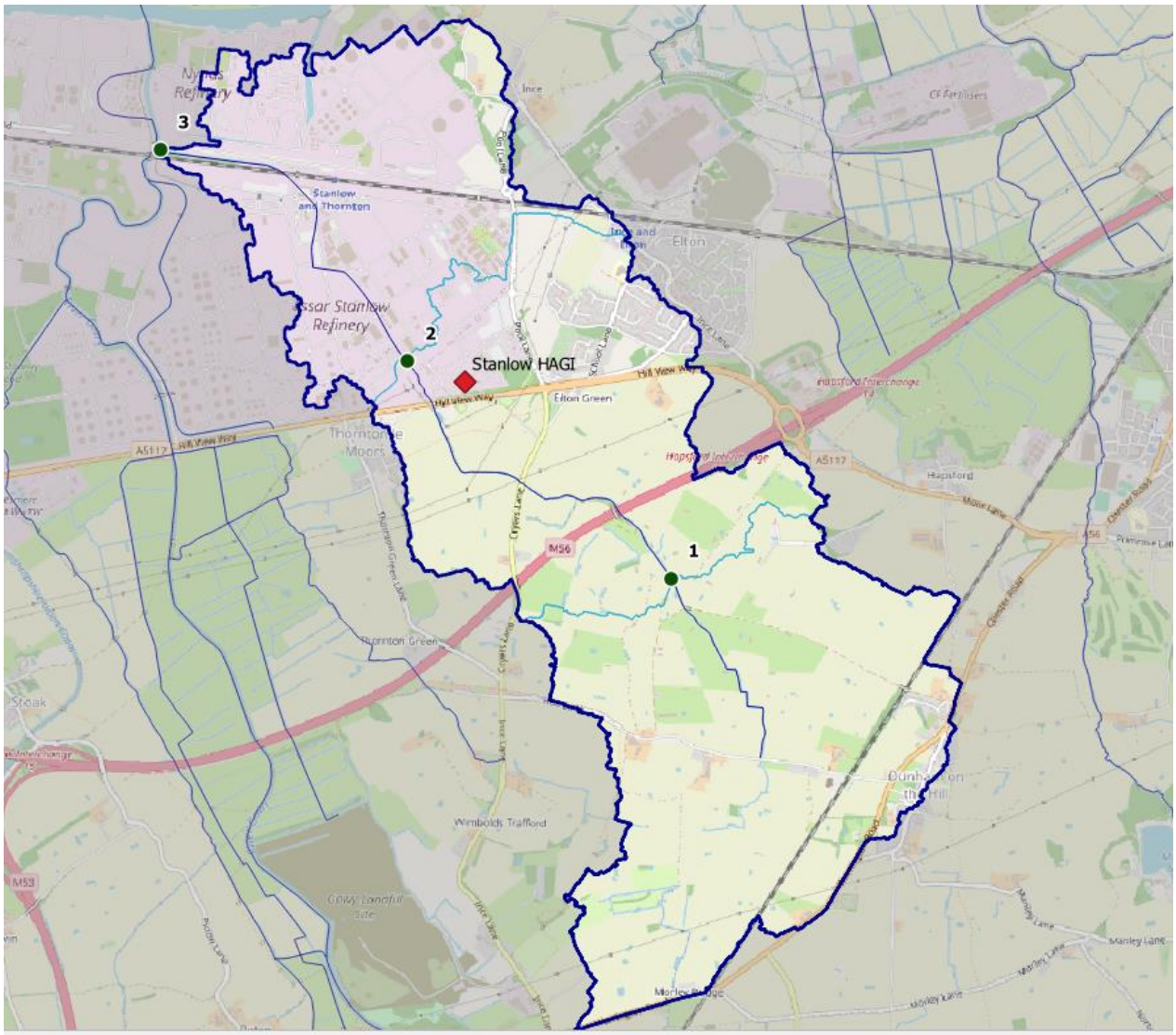


Figure showing full Gale Brook catchment (dark blue), sub-catchments (light blue) and associated flood estimation points (green). Stanlow HAGI location shown on the right bank of the watercourse, adjacent to the Stanlow refinery (red).

Description	The total Gale Brook catchment covers an area of 9.5km^2, as defined by the EA LiDAR data. The delineated sub-catchments at FEPs 1-3 each represent an area of approximately 3km^2. The catchment can be broadly split in terms of land-use at the A5117 crossing, upstream (south) of which the catchment is predominantly rural, whilst downstream (north) is significantly urbanised associated with the industrial areas of the Stanlow refinery. Topography varies from approximately 43mAOD in the southeast of the catchment to approximately 3mAOD at the outlet to the River Gowy. Soil data from SoilScapes indicates that the total Gale Brook catchment is predominantly underlain by slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils. The lower portion of the catchment has some underlying freely draining slightly acid sandy soils across the Stanlow refinery and bands of loamy and sandy soils with naturally high groundwater and a peaty surface to the east and west. This is reflected in the BFIHOST19 values of 0.361 taken at FEP1, representative of the slowly permeable soil that is dominant in the upper reaches, and a value of 0.603 taken at the Stanlow refinery which reflects the greater incidence of freely draining soil. However, given the high degree of urbanisation in this portion of the catchment, hydrological response will be dominated by impermeable areas and piped drainage systems rather than natural soils.

	BGS mapping indicates that the catchment is predominantly underlain by Chester Formation (Sandstone), with some Kinnerton Sandstone Formation and Wilmslow Sandstone Formation in the lower reaches across the Stanlow refinery. Superficial deposits of Till, Devensian – Diamicton underlay the majority of the catchment with some tidal flat deposts in the west of the catchment coincident with the River Gowy.

2.3 Source of flood peak data

Source	N/A – ungauged catchment

2.4 Gauging stations (flow or level)

Water-course	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
N/A – ungauged catchment						

2.5 Data available at each flow gauging station in Table 2.4

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
N/A – ungauged catchment						

2.6 Rating equations

Station name	Type of rating	Rating review needed?	Comments and link to any rating reviews
N/A – ungauged catchment			

2.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Check flow gaugings	N/A			
Historical flood data				
Flow or river level data for events				
Rainfall data for events				
Potential evaporation data				
Results from previous studies				
Other data or information				

2.8 Hydrological understanding of catchment

Plots of flow data and interpretation	N/A
Plots of flood peak data and interpretation	

Conceptual model	The total catchment is small (~10km^2), and predominantly low-lying. Upstream (south) of the A5117 crossing, the catchment is predominantly rural and relatively impermeable given the underlying soils. Downstream of the crossing (north), the catchment is heavily urbanised and markedly more permeable given the freely draining soils and underlying sandstone.
Unusual catchment features	

2.9 Initial choice of approach

Is FEH appropriate?	Yes, the catchment and associated sub-catchments re relatively small and ungauged, therefore ReFH2 is a suitable approach.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	As stated in the Environment Agency's Flood Estimation Guidelines : *"There are two principal techniques for flood estimation available:* • *the FEH statistical method;* • *a design flood method using a rainfall-runoff model, the Revitalised Flood Hydrograph (ReFH) model"* The FEH statistical method produces peak flow estimates, using observed AMAX data from 'donor gauges' in hydrologically similar catchments. However with small catchments, this method is less robust, as most donor gauges are from large catchments and therefore not hydrologically similar.

Software to be used (with version numbers)	FEH Web Service[3] / ReFH2.3

3 LOCATIONS WHERE FLOOD ESTIMATES REQUIRED

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

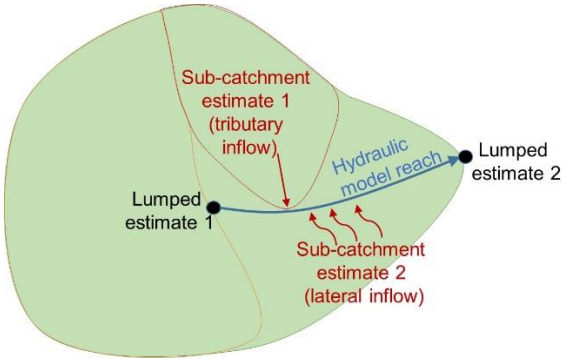
3.1 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
FEP1	L	Gale Brook	Lumped inflow taken from downstream model extent, and applied at the upstream extent.	345850	373850	1.96	3.81
FEP2	S	Gale Brook (sub-catchment)	Lateral inflow representing the intervening lateral catchment between FEP1 (upstream inflow) and Stanlow refinery culvert. Flows will be distributed along the length of the open channel section, as per the original EA model.	375040	344400	-	3.15
FEP3	S	Gale Brook (sub-catchment)	Lateral inflow representing the intervening lateral catchment between FEP2 (start of Stanlow refinery culvert) and outfall to the River Gowy. Flows will be applied as a 2D inflow to the open-channel (2D only) section of the Gale Brook downstream of the Stanlow refinery culvert, as per the original EA model.	376200	343060	-	2.51

Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required.

Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced.

The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.



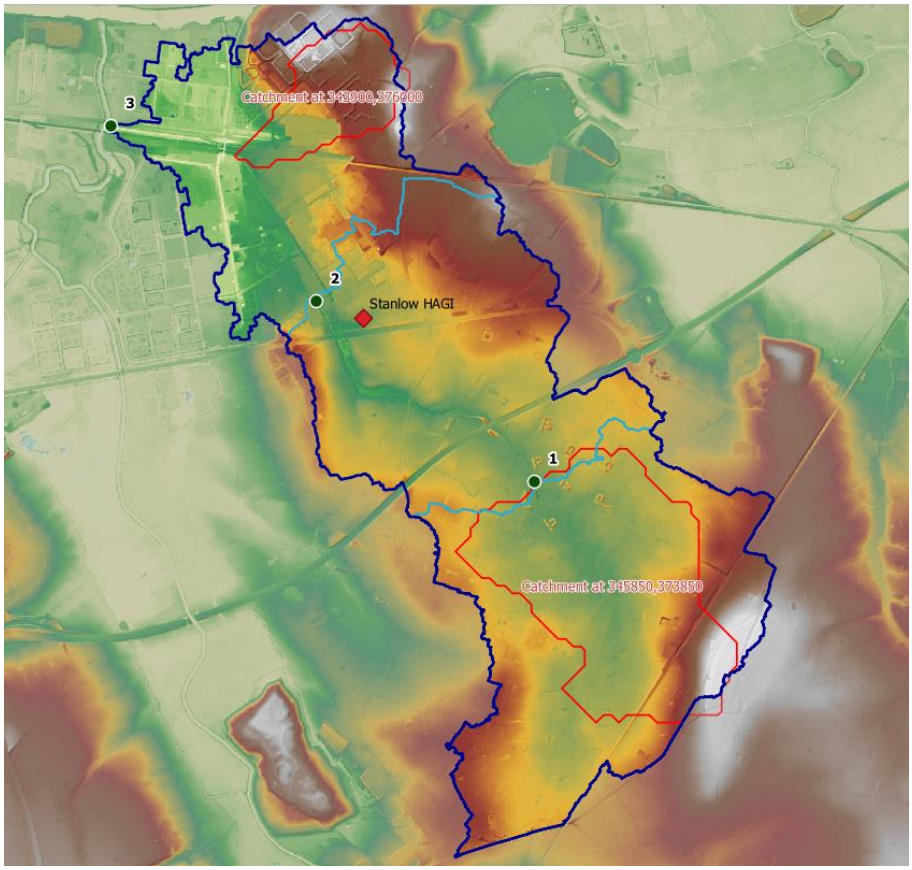
[3] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

3.2 Important catchment descriptors at each subject site (incorporating any changes made in red)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
FEP1	1	0.37	0.361	1.87	16.4	704	0.01	0.3453
FEP2[1]	1	0.37	0.361	0.70	16.4	704	0.09	0.3453
FEP3	1	0.37	0.603	0.87	17.82	722	0.50	0.1283

[1]Proxy values of FARL, PROPWET, BFIHOST, SAAR, DPSBAR, and FPEXT for FEP2 have been estimated based on the FEP1 descriptors

3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	Catchment descriptors have been purchased at FEP1 for use in flood estimates for the upstream FEP1 catchment and to also act as a proxy catchment for FEP2, given the proximity and similarity in terms of soils, area, and landuse. A second, downstream set of catchment descriptors have been taken from a minor tributary of the Gale Brook across the Stanlow refinery, to act as a proxy for catchment descriptors for the FEP3 intervening catchment. This FEH catchment is more representative in terms of soils and landuse. The FEH catchment for FEP1 (red) was checked against the catchment delineated from the latest available EA LiDAR data. The FEH catchment was found to significantly underestimate the catchment area by approximately 50%. Therefore the catchment area has been revised to 3.81km^2. The intervening catchments for FEPs 2 and 3 have been defined based on the EA LiDAR also, at 3.15 and 2.51km^2, respectively.  Figure showing the original FEH catchments in red, revised total catchment (dark blue) and subcatchments (light blue), and flood estimation points (green) displayed against the underlying LiDAR.

Record how other catchment descriptors were checked and describe any changes.	URBEXT was checked visually using satellite imagery and OS mapping. The URBEXT value for FEP1 was deemed to be representative of the upstream catchment which was almost entirely rural. Given that the catchment descriptors for FEP2 and FEP3 have been derived from proxy catchments, the URBEXT values have been manually calculated. The 'urban' and 'suburban' mapping within the FEH webservice (and upon which URBEXT2000 is based) was found to poorly represent the Stanlow refinery industrial area in particular, therefore, revised 'urban' and 'suburban' areas have been digitised based on the OS 1:50,000 mapping. Revised URBEXT2000 values have been derived based on the equation URBEXT2000 = 0.629 URBAN (as defined in the Flood Estimation Guidelines), where URBAN is the fraction of the urban class and half the fraction of the suburban class, assuming that half a grid square defined as suburban is covered by vegetation[4]. Soil descriptor values (BFIHOST and SPRHOST) were checked against soilscapes data. The FEH descriptors of the upper (FEP1 and FEP2) and lower portions (FEP3) of the catchment represent the soil types well, and the revised catchment areas are unlikely to impact the values. Therefore, these descriptor values remain unchanged. The FARL values were checked visually against OS Mapping for surface water features and satellite imagery. The Lakes and Waterways mapping within the FEH web service erroneously identifies a surface water body adjacent to the Stanlow refinery, reflected in the original FARL value of 0.91 for the downstream FEH catchment. There is no evidence of any surface waterbodies within OS mapping or satellite imagery within this intervening catchment, and therefore the FARL value has been updated to 1. Whilst there are numerous ponds identified within the upstream catchments for FEP1 and FEP2, these are predominantly very small and 'offline' (limited upstream catchments). Therefore, the associated degree of attenuation is anticipated to be minor, and the original descriptor value of 1 has been retained for these catchments to assume a reasonable conservative case. DPLBAR has been adjusted for the FEP1 catchment given that this is based on the catchment area. The standard equation for DPLBAR, given in the FEH Volume 5, uses a power term of 0.548 based on research for the UK as a whole. The power term has also been applied to the original FEH catchment area to derive an estimated DPLBAR value, and this has been used as a ratio against the the revised catchment area estimated DPLBAR value to scale the final updated value to ensure it is commensurate with the original value. The EA flood estimation guidelines state that it is unwise to rely on this regression equation for revising DPLBAR values for intervening areas, since it is designed for real catchments. Therefore, it is not strictly applicable for FEP2 or FEP3 in this case, though it is noted that this approach was used in the previous Jacobs study. The DPLBAR for FEP2 has been estimated using the above regression equation and based on a typical sub-catchment ($0.63km^2$) within the wider intervening area. The DPLBAR for FEP3 remains unchanged from the original FEH descriptor value, since this catchment is representative of a typical sub-catchment within this wider intervening area. This approach results in more conservative flood estimates than if the DPLBAR values were calculated over the entire catchment area, with peak flows typically 20-30% greater as a result of the reduced flowpath length. The topographic slope has been sampled for the delineated sub-catchments and FEH catchments, to act as an indicator for DPSBAR. The mean slope underlying the FEP1 and FEP2 upstream catchments is within 10% of the original FEH catchment, and therefore the original descriptor value is deemed to be representative for both of these two catchments. However, the mean underlying slope value for the FEP 3 catchment is approximately 25% less than that of the original FEH catchment boundary. Therefore, the

	DPSBAR value has been updated by applying the ratio between the mean underlying slope values for each catchment.
Source of URBEXT	URBEXT2000
Method for updating of URBEXT	N/A – original URBEXT value deemed to be representative for FEP 1 and delineated based on up-to-date OS mapping for FEP 2 and FEP 3.

[4] https://assets.publishing.service.gov.uk/media/602bb59de90e070556671419/URBEXT2000_-_A_new_FEH_catchment_descriptor_-_SID5_summary.pdf (Accessed January 2023)

4 REVITALISED FLOOD HYDROGRAPH 2 (REFH2) METHOD

3.4 Application of ReFH2 method

What is the purpose of applying this method?	The ReFH2 method has been applied to provide peak flows and hydrographs for the catchment.

3.5 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp} % runoff for impermeable surfaces	BL (hours)	BR
FEP1	L	3.503	2.63	279.645	0.7	30.201	1.251
FEP2	S	3.306	2.47	279.645	0.7	29.541	1.251
FEP3	S	3.515	2.64	524.34	0.7	41.691	2.534

Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)

3.6 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
FEP1	Urban	Winter	05:30
FEP2	Urban	Winter	05:30
FEP3	Urban	Winter	05:30

3.7 Flood estimates from the ReFH2 method

Site code		Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	30	100	200	100 (+67%)	100 (+106%)	200 (+67%)	200 (+106%)	1000
		Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	3.33	1	0.5	1 (+67%)	1 (+106%)	0.5 (67%)	0.5 (106%)	0.1
FEP1	1.23	1.67	2.03	2.73	3.80	4.55	6.34	7.82	7.59	9.37	6.58
FEP2	1.53	2.09	2.55	3.44	4.80	5.75	8.02	9.89	9.61	11.85	8.31
FEP3	1.18	1.61	1.96	2.63	3.68	4.46	6.15	7.58	7.45	9.19	6.55

5 DISCUSSION AND SUMMARY OF RESULTS

3.8 Comparison of results from different methods

Location	Comparison to existing Jacobs (2019) results – peak flow (m^3/s)			
	Return period 2 years / 50% AEP		Return period 100 years / 1% AEP	
	ReFH1 - Jacobs	ReFH2 - WSP	ReFH1 - Jacobs	ReFH2 - WSP
Gowy confluence	2.5	3.9	6.5	12.28
Commentary	Combined flows from the three sub-catchments used in this assessment (which use a default critical duration of 5.5-hours from FEP1) have been compared against the pre-existing ReFH1 flows derived at the River Gowy confluence as part of the previous Jacobs (2019) study. Although the final hydrographs for the Gale Brook used in the previous model used a critical duration representative of the River Gowy, results were also reported for a critical duration of 3.9 hours for the Gale Book (assumed to be the critical duration). Combined peak flows output from ReFH2 as part of this study are over 80% greater than those output as part of the previous study. Whilst some of these differences may be explained by the updated software to ReFH2, the key factors for this are due to the catchment areas and DPLBAR values for the intervening catchments. The previous Jacobs (2019) study concluded that the original FEH catchment of 6.66km^2 was deemed to be representative of the Gale Brook, based on catchment analysis using EA LiDAR data. However, updated catchment analysis undertaken as part of this study has clearly demonstrated that the FEH catchment underestimates the true catchment size as seen in Section 3.3, giving a catchment size of 9.5km^2 (43% greater than that used as part of the previous study). In addition, sensitivity testing on the DPLBAR values has suggested that peak flows are up to 30% greater when a smaller DPLBAR is used, representative of a sub-catchment within the intervening areas.			

3.9 Final choice of method

Choice of method and reasons	A simple ReFH2 methodology has been employed, given the size of the catchment and sub-catchments. The catchment is ungauged, and the small size of the subcatchments (<5km^2) limits the applicability of the FEH statistical method.
How will the flows be applied to a hydraulic model?	The inflows will be applied adopting a similar method as used as part of the Jacobs (2019) study. Flows from FEP1 will be input at the revised upstream extent of the model. Flows from FEP2 will be distributed along the open channel reach between FEP1 and the start of the Stanlow refinery culvert, and flows from FEP3 will be applied at the downstream extent of the Stanlow refinery culvert into the TUFLOW open-channel reach.

3.10 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	It is assumed the ReFH2 recommended storm duration for FEP1 is representative of the study site and sub-catchments. This was chosen as a consistent storm

	duration for all the sub-catchments given this is the largest sub-catchment and assumed to have the greatest influence over flows at the Stanlow HAGI site downstream. It is assumed that a uniform storm duration with a winter profile is appropriate for all FEPs. The FEH guidelines recommend a summer profile to be used in heavily urbanised catchments such as FEP3, though for consistency across the catchments in this case, a winter storm profile has been applied given that the upstream catchment from the site of interest is predominantly rural. The intervening catchment between FEPs 2 and 3 is heavily urbanised, and as such the true extent of the sub-surface catchment is uncertain given a lack of data with regards to the sub-surface drainage. In the absence of any gauge data, it is assumed that the ReFH2 hydrograph shape is representative of the catchment response.
Discuss any particular limitations	The main limitations in this study are the lack of gauged data to aid hydrological or hydraulic model calibration, and uncertainty with regards to the catchment size for the intervening catchment between FEP2 and 3. . Flood peaks have been derived using the ReFH2 method, based on catchment descriptors only. However this is a standard best practice tool, recommended by the EA for estimating peak flows in ungauged catchments in the UK.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	It is inherently difficult to quantify uncertainty in design flows estimated from the ReFH rainfall-runoff model. Wallingford HydroSolutions (2019c) evaluates the performance of ReFH2 by comparing its results with those from an enhanced single-site analysis at gauging stations. The factorial standard errors from ReFH2 are comparable to those observed for the FEH pooled statistical method when the catchment is treated as ungauged.
Comment on the suitability of the results for future studies	The design flow estimates have been derived for the purposes of this hydrological assessment in order to inform the flood risk to the proposed Stanlow HAGI associated with the Gale Brook. If peak flow estimates are required for different purposes it is recommended that, at a minimum, a review of results is carried out.
Give any other comments on the study	As in most ungauged catchments, it is recommended that temporary flow gauges be installed to better inform the design peak flow estimates. There are no other specific suggestions relevant to this study.

3.11 Checks

Are the results consistent, for example at confluences?	N/A
What do the results imply regarding the return periods /	N/A – ungauged.

frequency of floods during the period of record?	
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	N/A
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The 0.1% / 1% AEP event ratio varies between is 1.73 and 1.78. These are within the typical range (1.3 – 1.8)
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	Consistent with results from previous Jacobs (2019) study, taking into account the differing catchments used and approach to DPLBAR. Results of this assessment are preferred for reasons explained above, and tailored for the purposes of this assessment.
Are the results compatible with the longer-term flood history?	N/A
Describe any other checks on the results	N/A

3.12 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	30	100	200	100 (67%)	100 (106%)	200 (67%)	200 (106%)	1000
	Flood peak (m^3/s) for the following AEP (%) events										
	50	20	10	3.33	1	0.5	1 (+67%)	1 (+106%)	0.5 (67%)	0.5 (106%)	0.1
FEP1	1.23	1.67	2.03	2.73	3.80	4.55	6.34	7.82	7.59	9.37	6.58
FEP2	1.53	2.09	2.55	3.44	4.80	5.75	8.02	9.89	9.61	11.85	8.31
FEP3	1.18	1.61	1.96	2.63	3.68	4.46	6.15	7.58	7.45	9.19	6.55

Appendix B
Site photographs

Photo 1 Farm crossing (upstream of M56), represented by node GAL_3780 (WSP, 2022)

Photo 2 M56 crossing upstream face, represented by node GAL_3666cu (Warner Surveys, 2022)

Photo 3 M56 crossing downstream face, represented by node GAL_3666cd (Warner Surveys, 2022)

Photo 4 View upstream from A5117 road bridge, showing floodplain bund on the right (east) bank (WSP, 2022)

Photo 5 Short open channel section, immediately downstream of A5117 (WSP, 2022)

Photo 6 View downstream from Gale Brook flapped outfall to River Gowy at low tide conditions (WSP, 2022)

Photo 7 View of Gale Brook flapped outfall from downstream at high tide (WSP, 2022)

HyNet North West Hydrogen Pipeline

Delivering clean growth

Cadent

Your Gas Network

HyNet North West Hydrogen Pipeline

Delivering clean growth

Annex 7F4
River Mersey Model User Report (To be provided for Final ES)

HyNet North West Hydrogen Pipeline

Delivering clean growth

Annex 7F5 Puddinglake Brook Hydraulic Model Report

1. Introduction

1.1 Context

1.1.1 This hydraulic model report documents the modelling undertaken for the assessment of fluvial flood risk to the Hydrogen Storage Facility Hydrogen Above Ground Infrastructure (HAGI). It should be read in conjunction with the HyNet North West Hydrogen Pipeline Project Flood Risk Assessment (FRA).

1.1.2 The proposed Hydrogen Storage Facility HAGI is located at the eastern end of Line 2 (Eastern Corridor; Central Hub to Gas Storage) of the Hynet Hydrogen Pipeline, at Whatcroft, north of Middlewich, Cheshire.

1.2 Background

1.2.1 The HyNet North West Hydrogen Pipeline Project includes the construction and operation of approximately 125km of new underground pipelines to distribute hydrogen to a variety of industries across Merseyside, Greater Manchester, and Cheshire. Hydrogen produced at Stanlow will be distributed through the network of underground pipelines, with Hydrogen Above Ground Installations (HAGIs) connecting the major corridor pipelines with customers via smaller spur pipelines.

1.2.2 The Hydrogen Storage Facility HAGI, which will connect the main distribution network with an underground hydrogen storage facility, was identified as one of four HAGIs potentially at risk of fluvial flooding during the production of the Preliminary Environmental Information Report (PEIR) given the site's location approximately 30m to the north of the right bank of the Puddinglake Brook, adjacent to Flood Zones 2 and 3. These Flood Zones are associated with the Puddinglake Brook, an Environment Agency (EA) Main River, within the Weaver catchment.

1.2.3 The Environment Agency (EA) were not able to provide a Flood Model for this location.

1.3 Objectives

1.3.1 The objectives of this hydraulic modelling study are to:

- outline the fluvial flood risk from the Puddinglake Brook to the Hydrogen Storage Facility HAGI for the 0.5% AEP present day design event for the HAGI construction period;
- evaluate future fluvial flood risk accounting for the impact from climate change for the 0.5% design event for the HAGI operational period, utilising the latest guidance on climate change allowances on flows; and
- assess the sensitivity of the site to fluvial flooding for the 'Upper End' climate change allowance.

1.4 Scope and Method

1.4.1 The methodology steps of this modelling study are outlined below:

- Flood estimation producing peak flows and hydrographs for the Puddinglake Brook, as detailed in **Appendix A**;
- Manning's calculations assessing the conveyance capacity of the Puddinglake Brook channel to the south of the HAGI for the 1% AEP + 106% climate change event, as detailed in **Appendix B;**
- Construction of a simple 1D model for the reach of the Puddinglake Brook, as detailed in the main body of this report, with:
  - Model cross-sections informed by sampling LiDAR data;
  - The A530 bridge represented by a standard model structure unit;
  - Survey photos of the watercourses used to determine in-channel Manning's n roughness values. The values will be chosen based on Chow (1959[1]).
  - A single ReFH inflow point, at the upstream boundary informed by the Flood Estimation exercise (**Appendix A**). Sensitivity tests for roughness and blockage of the A530 road bridge.
- Running AEP events:
  - 50%;
  - 20%;
  - 10%;
  - 3.33%;
  - 1%;
  - 0.5%;
  - 0.5% + 2100 Higher Central climate change allowance;
  - 0.5% + 2100 Upper End climate change allowance; and
  - 0.1%
- Processing results files for the reach and analyse flood levels in relation to the HAGI.

1.5 Justification of Approach

1.5.1 The initial Manning's calculations exercise (**Appendix B**) indicated that the capacity of the Puddinglake Brook channel would be insufficient to contain flows at and above the 1% AEP event, with flow spilling onto the lower elevation left (south) floodplain. The floodplain on the right bank, where the HAGI is located, is a

[1] Chow, V.T., 1959, Open-channel hydraulics: New York, McGraw-Hill, 680 p.

consistent higher elevation than the floodplain on the left bank, which is agricultural land.

1.5.2 It is hypothesised that the main mechanism of flood risk to the HAGI arises from the constriction to the channel of the Puddinglake Brook caused by the A530 road bridge 260m downstream of the HAGI. This bridge is comprised of three culverts beneath the road. The combined capacity of the culverts was estimated to be 1.16 m^3/s, significantly lower than the peak flow of 6.24m^3/s during the 1% AEP + 106% CC event and also lower than the peak flow of 3.03m^3/s discharge during the 1% AEP present day event. Consequently, the culverts' capacity was deemed insufficient to convey the flows of either of these events and it can be expected that the structure would provide an obstacle to flows, leading to water backing up upstream of the A530 and increasing water levels close to the HAGI. However, as Manning's calculations only consider in-channel capacity, it is not possible to derive accurate estimations of water levels during these events, and there is no consideration of the effect of flow overtopping the A530 bridge, or the capacity of the lower elevation left floodplain.

1.5.3 A hydraulic model allows a more detailed assessment of whether the HAGI site is likely to experience flooding from the Puddinglake Brook during the design events.

1.5.4 A simple 1D hydraulic model, implemented in FloodModeller software, was deemed a suitable approach for several reasons:

- The objective of this modelling study is to more accurately define the fluvial flood risk to the Hydrogen Storage Facility HAGI from a single source, the Puddinglake Brook, than was possible with the Manning's calculation approach.
- At this stage of the project, the aim is to identify and quantify potential flood risk to the HAGI itself. If it is found that the HAGI is likely to interact with flood waters, the model could be refined to explore the risk this might pose to downstream receptors. Such a refinement may include collecting and using topographical survey of the Puddinglake Brook channel.
- Only a single river reach is involved.
- The elevations of the floodplain on each side of the river are of a consistent elevation and thus a 1D-2D approach is not required.
- Processing times for 1D are much reduced in comparison with a 2D or linked 1D-2D model.

1.6 Report structure

1.6.1 The structure of this report is as follows:

- Section 2 – provides an overview of the current understanding of the flood risk and flood response at the HAGI and along the Puddinglake Brook and the input data available for the purposes of this study and a detailed review of the existing EA model to determine its suitability for this assessment.
- Section 3 – details the model approach, reach and implementation.

- Section 4 – Hydrological assessment undertaken.
- Section 5 – Hydraulic Model Build.
- Section 6 – Model Runs and performance, provides an overview of the model runs undertaken, run parameters, run performance and sensitivity analysis carried out.
- Section 7 – details the results of the modelling runs.
- Section 8 – provides conclusions and recommendations from this modelling study.
- Appendix A: Flood estimation (hydrology) report.
- Appendix B: Manning's assessment.
- Appendix C: Channel photographs.

2. Qualitative Description of Flood Response

2.1 Modelled Watercourse and Catchment

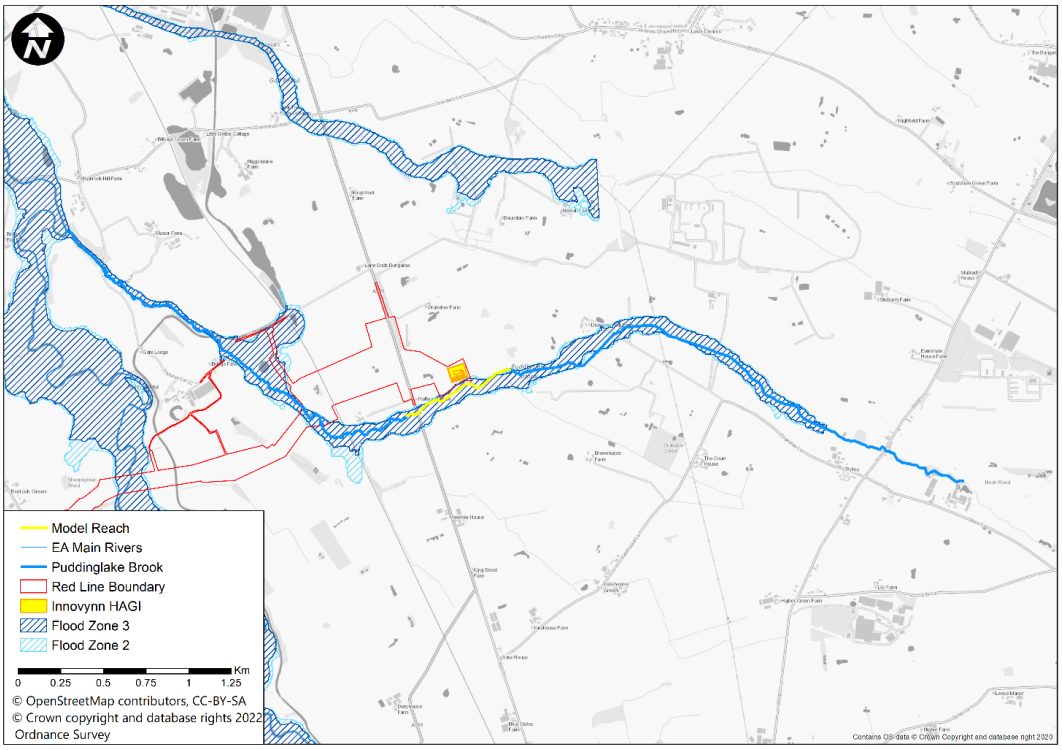
2.1.1 The Puddinglake Brook is 6.5km in length, and flows in a north-westerly direction through agricultural land, under roads and a railway, before it joins the River Dane (**Graphic 2.1)**.

2.1.2 The Puddinglake Brook initially flows North-West passing Byley village, before bending west past the southern boundary of the HAGI. It continues west under the A530, then flows past Brook House Farm, under a freight railway line, minor roads and the Trent and Mersey Canal, before joining the River Dane at Shipbrookhill (NGR SJ 67410 70845).

2.1.3 The HAGI will be located at the eastern end of Line 2 of the Hynet Hydrogen pipeline centred on NGR SJ 69700 70013 at Whatcroft, north of Middlewich, Cheshire.

2.1.4 The southern border of the proposed Hydrogen Storage Facility HAGI site, which is approximately 35m north of the Puddinglake Brook, is mapped as at risk of fluvial flooding. Flood Zones 2 and 3 at this location are associated with the Puddinglake Brook, an EA Main River.

Graphic 2.1 – Hydrogen Storage Facility model study area



2.2 Hydrological description

2.2.1 Upstream of the HAGI, the Puddinglake Brook drains a small catchment of approximately 5.0km^2. The catchment is largely rural (Urban extent 2.8%), and the dominant land cover is agricultural. The topography varies from 58mAOD to 28mAOD. The catchment is predominantly underlain by slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils with impeded drainage in the west and central areas of the catchment, and by naturally wet very acid sandy and loamy soils in the easternmost part of the catchment[2]. British Geological Survey [BGS] mapping[3] indicates that the entire catchment is underlain by Sidmouth Mudstone Formation (mudstone) with glaciofluvial sheet deposits of sand and gravel in the east of the catchment and Devensian till (Diamicton) present in the central and western parts of the catchment.

2.2.2 The catchment is ungauged. There are no EA or NRFA gauges on the Puddinglake Brook. The closest river gauge is located approximately 4.25km downstream of the HAGI, on the River Dane at Rudheath[4].

2.2.3 Additionally, there are no rainfall gauges within the Puddinglake Brook catchment, the closest being located in the centre of Northwich, over 6km north-east of the HAGI.

2.3 Flood mechanisms

2.3.1 The main mechanism of flooding to the HAGI site is fluvial flooding from the Puddinglake Brook. The HAGI is located approximately 30m to the north of the right bank top of the watercourse. The Flood Map for planning (FMfP)[5] shows that the southern border of the proposed Hydrogen Storage Facility HAGI site is located within both Flood Zones 2 and 3.

2.3.2 The Historic Flood map[6] shows no historic flood extents within the catchment of the Puddinglake Brook, with the closest extent of flooding approximately 10km downstream of the HAGI, on the River Dane and River Weaver in Northwich town centre.

[2] https://www.landis.org.uk/soilscapes/ accessed 08/06/2023
[3] https://geologyviewer.bgs.ac.uk/?_ga=2.197288751.1067203856.1686231024-2117698456.1680797711 accessed 08/06/2023
[4] https://nrfa.ceh.ac.uk/data/station/info/68003 accessed 08/06/2023
[5] https://flood-map-for-planning.service.gov.uk/ accessed 05/01/2023
[6] https://environment.data.gov.uk/DefraDataDownload/?Mode=OGCPreview&mapService=https%3A%2F%2Fenvironment.data.gov.uk%2Fspatialdata%2Fhistoric-flood-map%2Fwms accessed 14/06/2023

2.3.3 The Preliminary Flood Risk Assessment[7] (PFRA) and Strategic Flood Risk Assessment[8] (SFRA) reports produced by Cheshire West and Chester Council (CWCC) report no historic flood incidents from both fluvial and surface water sources within the Puddinglake Brook catchment.

2.3.4 The Surface water flood risk map[9] shows that surface water flow paths are limited to the same path as the watercourse with no flow pathways across the HAGI site.

2.3.5 The minimum elevation of the HAGI site (derived from EA 1m resolution Lidar) is 31.38mAOD, whereas the in-channel elevations in the adjacent brook as it flows past the southern side of the HAGI ranges between 29.2 - 31mAOD.

2.3.6 The Manning's calculation exercise concluded the main mechanism of flood risk to the HAGI arises from the obstruction caused by the A530 road bridge 260m downstream of the site. The combined capacity of the culverts was estimated to be 1.16 m^3/s, insufficient to convey the estimated 3.03m^3/s discharge during the 1% AEP event, creating a backwater upstream of the A530 and increasing water levels close to the HAGI.

2.4 Existing understanding of flood risk

2.4.1 Table 2.1 lists all relevant information and where it originates from.

Table 2.1 Available sources of information

Data	Description	Source
LiDAR	LiDAR (unfiltered & filtered), resolution, date Environment Agency Lidar, National Lidar Programme (NLP) (2021) Layers: DTM_SJ6565_P_10639_20210409_20210416 DTM_SJ6570_P_10639_20210409_20210416 DTM_SJ7065_P_10639_20210409_20210416 DTM_SJ7070_P_10639_20210409_20210416	Download Source: https://environment.data.gov.uk/DefraDataDownload/?Mode=survey from: https://www.data.gov.uk/dataset/f0db0249-f17b-4036-9e65-309148c97ce4/national-lidar-programme

7 https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/698398/PFRA_Cheshire_West_and_Chester_Council_2017.pdf accessed 14/06/2023

8 https://www.cheshirewestandchester.gov.uk/documents/parking-roads-and-travel/highways/flood-risk-assessment-final-report.pdf Accessed 14/06/2023

9 https://check-long-term-flood-risk.service.gov.uk/map accessed 08/06/2023

Data	Description	Source
	Merge of the 4 layers created to create a single layer for catchment delineation 1m horizontal resolution Flight date	
Other Topographic Data	No other topographic data is used Topographic survey data was not available at the time of model build.	N/A
Ordnance survey MasterMap	OS Local and OS open Raster used for basemapping purposes MasterMap data has also been used to inform the model Manning's roughness	Environment Agency
Site visit information and photographs	A site walkover was undertaken by two hydrologists on 09/11/2022. Photographs were taken and notes recorded. These were later used to derive channel roughness, estimate structure dimensions and materials.	Stakeholder (WSP) walkover – 11/09/2022
FEH catchment descriptors	Catchment descriptors to feed into the RefH2 analysis taken at the proposed discharge location	FEH web service https;//fehweb.ceh.ac.uk/
Gauging stations	Catchment is ungauged, and the small size of the catchment (~5km2) limits the applicability of the FEH statistical method	N/A

Analysis of historic flood risk

2.4.2 As detailed previously, the only available data detailing historical flood data for this catchment is the historic flood map which shows flooding in Northwich, far downstream.

2.4.3 The EA Flood map for planning is the only dataset which predicts flood risk in this location.

2.5 Availability of existing hydraulic models

2.5.1 The Environment Agency do not hold a flood model for this location.

3. Model approach and justification

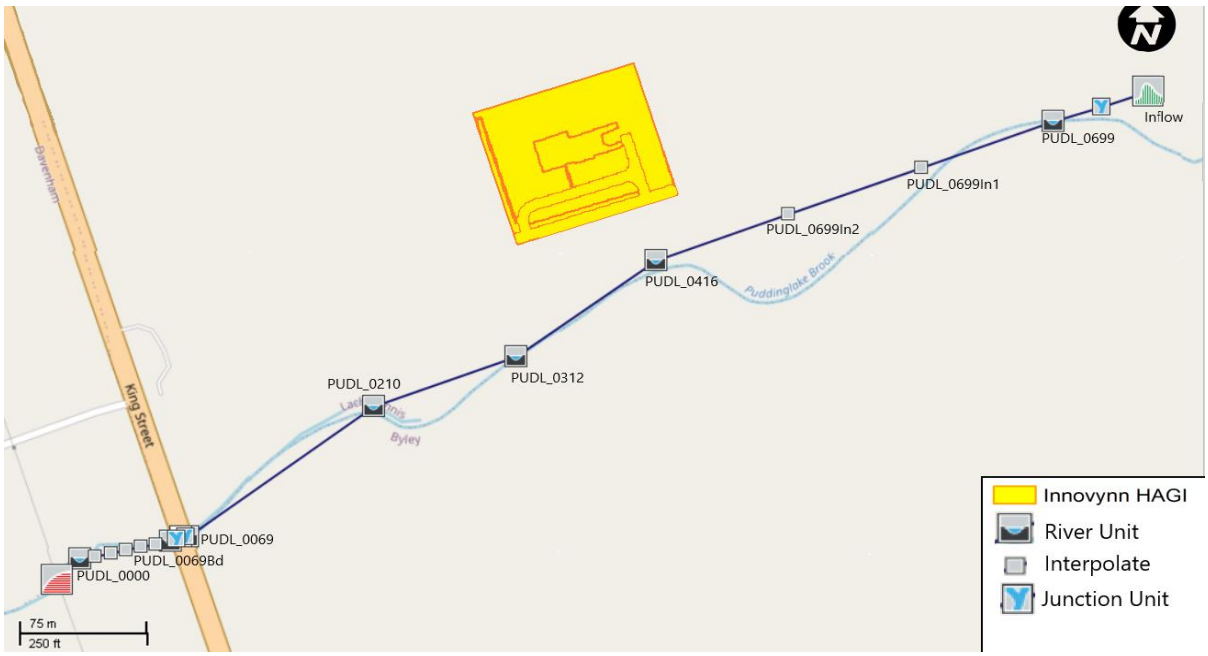
3.1.1 There was no Environment Agency model made available of the Puddinglake Brook, thus a new model construction was required.

3.1.2 The 1D model was constructed to determine water levels within the river channel and on the floodplain, relative to the elevation of the area where the HAGI is proposed. Additionally, the 1D model incorporates bespoke hydrology for the catchment and simulates the impact of the road bridge on flow hydraulics.

3.1.3 The 1D model covers approximately 700m of the Puddinglake Brook, adjacent to the HAGI. This includes approximately 620m of channel upstream of the upstream of the A530 road bridge, and 50m downstream. The model reach includes the A530 road bridge.

3.1.4 The Puddinglake Brook in this area sits in a well-defined channel – the lack of expansive areas of low-lying floodplain lends this watercourse to a 1D only representation. This approach is also commensurate with the level of detail in assessment necessary to determine the water levels associated with the expected river flows calculated in the hydrology assessment.

Graphic 3.1 Hydrogen Storage Facility Flood Modeller Model Schematic



4. Hydrological assessment

4.1.1 Full details of how the hydrology was determined are provided in Flood Estimation Report in **Appendix A**. A brief summary is provided here.

4.1.2 The catchment is small (~5km^2) and ungauged. As stated in the Environment Agency's Flood Estimation Guidelines[10]:

4.1.3 "There are two principal techniques for flood estimation available:

- The FEH statistical method; and
- A design flood method using a rainfall-runoff model, the revitalised flood hydrograph (ReFH) model."

4.1.4 A hybrid approach using Statistical Method flows scaled to the ReFH2 hydrograph was adopted to produce design hydrographs for the hydraulic model as recommended in the Environment Agency's FEH Guidelines. The FEH Statistical method for estimating flood flows is favoured as it is based on a much larger dataset of flood events and has been more directly calibrated to reproduce flood frequency on UK catchments giving it a greater confidence in deriving the index flood (QMED).

4.1.5 The FEH Statistical method is not recommended for use beyond the 200 year return period. The ratio between the ReFH2 100yr and 1000yr flows was applied to the 100yr FEH Statistical flow to estimate the 1000yr flow.

4.1.6 Flood peaks for the required return periods are shown in **Table 4.1.**

Table 4.1 Peak flows derived from ReFH2

Return Period (years)	Event (AEP)	Climate Change allowance	Peak flow (m^3/s)
2	50%	-	1.17
5	20%	-	1.77
10	10%	-	2.21
30	3.33%	-	2.97
100	1%	-	3.91

Return Period (years)	Event (AEP)	Climate Change allowance	Peak flow (m^3/s)
200	0.5%	-	4.52
200 (CC 67%)	0.5%	67%	7.55
200 (CC 106%)	0.5%	106%	9.32
1000	0.1%	-	6.39

5. Hydraulic model build

5.1.1 The model was developed using Flood Modeller Version 6.1 and run using Flood Modeller Version 5.1 – an industry standard software package, which is widely used for flood modelling.

5.2 Model boundaries

5.2.1 There is a single flow-time boundary at the upstream end of the modelled reach of the Puddinglake Brook.

5.2.2 The downstream end of the model is set up as a normal depth boundary, with a slope value of 0.0021, which corresponds to the slope of the channel immediately downstream of the modelled reach, informed by LIDAR data.

5.2.3 A range of unsteady inflow hydrographs was applied to the upstream boundary, as summarised in **Table 4.1**. This includes the 0.5% AEP present day event that is used as the 'design event' for the purposes of this assessment.

Cross sections

5.2.4 Cross sections were derived from LiDAR data and supplemented with interpolates where required. The cross sections used in the Manning's assessment were incorporated into the model.

5.2.5 **Graphic 3.1** shows the location of the cross sections.

5.2.6 **Table 5.1** lists all cross sections using a node identifier (which includes the width of each section) and linking it to the section reference.

Table 5.1 Model River and Auto-Interpolate Units

Node	Width (m)	Distance to next (m)	Informed by
PUDL_0699	106	94.33	Derived from LiDAR
PUDL_0699In1 (Interpolate)	-	94.33	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0699In2 (Interpolate)	-	94.33	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0416	115	104	Derived from LiDAR

Node	Width (m)	Distance to next (m)	Informed by
PUDL_0312	115	102	Derived from LiDAR
PUDL_0210	114	150	Derived from LiDAR
PUDL_0069	137	0	Derived from LiDAR
PUDL_0051	165	10	Derived from LiDAR
PUDL_0051In1(Interpolate)	-	10	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0051In2 (Interpolate)	-	10	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0051In3 (Interpolate)	-	10	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0051In4 (Interpolate)	-	10	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0051In5 (Interpolate)	-	10	Generated using Flood Modeller Auto-Interpolate tool
PUDL_0000	118	0	Derived from LiDAR

Floodplain representation

5.2.7 Cross-sections have been derived from EA LiDAR. The cross-section width is sufficient to avoid glass walling.

Key structures

5.2.8 As the Puddinglake Brook flows west past the proposed HAGI location it flows under the A530, approximately 260m downstream of the HAGI. This road bridge consists of three culvert barrels of different dimensions, beneath the road (see

photograph in **Table 5.2**). This bridge is the only known structure within the modelled reach of the Puddinglake Brook.

5.2.9 The two circular culverts have an estimated diameter of 0.6m and the arched culvert has an estimated width of 0.75m. These dimensions were used to assess the conveyance of the culverts against the 1% AEP + 106% CC event (6.24m3/s).

5.2.10 In the hydraulic model all three culvert barrels are represented using a single orifice unit (**Table 5.2**). As orifice units have zero length, the culvert length was added to the chainages of the upstream and downstream open channel reaches. The orifice unit was chosen for reasons of simplicity and model stability, commensurate with the level of detail for this simple 1D model. Sensitivity testing (replacing the orifice with three parallel conduits: two circular, one sprungarch culvert) showed that the peak water levels adjacent to the site are within 1mm of the peak water level when using the orifice unit (see **Paragraph 5.2.10**).

Table 5.2 Key Structures

Structure	**A530 road bridge culverts**
OS Grid Ref	SJ 69468 69780 (upstream) & SJ 69456 69774 (downstream)
Survey reference	No topographic survey data was available – viewed during site visit to establish the interior material, roughness of the inlet and condition.
Included in Model	Yes Model Labels PUDL_064, PUDL_064Bu, PUDL_064Su, PUDL_056
Representation	Single orifice unit with bore area set to the combined cross-sectional area of the three culvert openings. • Throat invert level: 28.426 mAOD • Throat soffit level: 28.950 mAOD • Bore area: 0.936 m^2 • Rectangular aperture shape • Default coefficients • Spill unit for road bridge deck, informed by LIDAR.
Assumptions	The dimensions of the bridge openings were estimated by eye during the site visit and are approximate. The elevations for the invert and soffit levels were estimated from LIDAR and the site visit. They are approximate. The spill unit for the road deck is based on LiDAR values extracted along the crest of the road centre.
Photograph	View of the downstream face of the culverts:

Roughness

5.2.11 The roughness of the bed, bank faces and bank tops were informed by photographs from the site visit of the channel and floodplain. **Table 5.3** shows the roughness values of the model cross-sections.

Table 5.3 Manning's values used in the model, based on Chow (1959)

Chow (1959) classification	Roughness (Manning's n)	Sections	Photographs
3.a.2 floodplains, pasture, high grass *** The left and right floodplains were consistently vegetated with a monoculture of pasture grass. This grass had grown quite long during the survey, but sheep are present in these fields in aerial imagery. A conservative**	0.035	Bank tops of PUDL_0699 PUDL_0416 PUDL_0312 PUDL_0210 PUDL_0069 PUDL_0051 PUDL_0000	

Chow (1959) classification	Roughness (Manning's n)	Sections	Photographs
category of 'high grass' was chosen.			
1.g sluggish reaches, weedy, deep pools ***Limited deep pools observed in the channel, so the average of the minimum and normal value has been chosen.**	0.06	Beds of PUDL_0699 PUDL_0416 PUDL_0312 PUDL_0210 PUDL_0069 PUDL_0051 PUDL_0000	
3.c.1 floodplains, brush, scattered brush, heavy weeds *** The scattered brush also includes mature trees, present all year round so the maximum value was chosen.**	0.07	Right banks of PUDL_0699 PUDL_0416 PUDL_0312 PUDL_0210 PUDL_0069 PUDL_0051 PUDL_0000	

Sensitivity analysis

5.2.12 The objective of sensitivity testing is to test the validity of underlying assumptions in the model, and ensure the model is behaving as expected. Sensitivity testing provides a means of assessing how sensitive the predicted flood depths are to changes to model parameters and assumptions. It is expected that if a parameter is changed then the resultant flood depth will change. The aim of the sensitivity testing is to highlight the potential degree of change associated with adjusted parameters so as to provide confidence with the selected value and bound the uncertainty of the model results.

Roughness sensitivity test

5.2.13 Sensitivity testing was conducted to assess the degree of change resultant of the roughness in the model. Manning's n was adjusted to minimum and maximum values respectively for each of the roughness parameters, guided by Chow (1959) and detailed in **Table 5.4**.

5.2.14 Varying the Manning's n to the minimum and maximum values, changes the maximum water level between -0.033m and 0.050m. This effect is within what can be expected in a roughness sensitivity test.

Table 5.4 Sensitivity test runs – Roughness (Manning's n)

Roughness category (guided by Chow, 1959)	Minimum	Baseline	Maximum
3.a.2 floodplains, pasture, high grass	0.030	0.035	0.050
1.g sluggish reaches weedy, deep pools	0.050	0.060	0.070
3.c.1 floodplains, brush, scattered brush, heavy weeds	0.050	0.070	0.080

Downstream boundary sensitivity test

5.2.15 The downstream normal depth boundary from the baseline model was replaced by a head-time boundary. This head-time boundary was informed by taking the modelled design water levels and increasing them by 0.10m. This approach follows the UK Government Guidance on River modelling[11]. The change in the downstream boundary has no effect on peak water levels near the site. This shows that the downstream boundary is set far enough from the site to influence model results.

Culvert size sensitivity test

5.2.16 The size of the orifice representing the A530 road bridge was decreased by 20%, from 0.936m^2 to 0.749m^2. This change does not affect peak water levels near the site.

Blockage sensitivity test

5.2.17 The size of the orifice representing the A530 road bridge was decreased by 80% (from 0.936m^2 to 0.187m^2), simulating a partial blockage of the culverts for the duration of the design event. This change has only a negligible effect on water levels near the site.

[11] Department for Environment, Food & Rural Affairs and Environment Agency. River modelling: technical standards and assessment. August 2021. https://www.gov.uk/government/publications/river-modelling-technical-standards-and-assessment [Accessed 04/01/2024 at 10:00]

Inflows sensitivity test

5.2.18 The magnitude of inflows is typically the biggest uncertainty in flood modelling. Increasing the inflow peak of the design event by 20% is a common sensitivity test that helps bound the upper end of this uncertainty. Instead of a test with the peak of the 0.5 AEP + 67% design event increased by 20%, the 0.5% AEP + 106% was used as a proxy, as its peak value is 23% higher than that of the design event. The 23% increase in peak inflows leads to an increase in peak water level of 0.043m.

Type of conduit unit sensitivity test

5.2.19 The A530 road bridge conduits are represented using an orifice unit in the baseline model. In this sensitivity test, the orifice unit was replaced with three parallel conduit units (two circular, one sprungarch culvert). The test showed that the peak water levels adjacent to the site are within 1mm of the peak water level when using the orifice unit.

Summary of Model Sensitivity

5.2.20 The impact of each sensitivity test has been assessed by comparing flood levels, relative to the baseline results. The impact of each test and the difference from the baseline are displayed in **Table 5.5.**

5.2.21 The ensemble of sensitivity tests helps quantify uncertainty in the model results. It has been shown that results from the Puddinglake Brook model should be seen as sitting within a band of -0.033 and +0.050 m uncertainty.

Table 5.5 Effect of sensitivity tests on peak water levels at node PUDL_0416 (section closest to the HAGI site)

	High roughness	Low roughness	Downstream boundary	Blockage	Inflows	Culvert Size	Conduit units instead of orifice
Event used and its baseline peak water level (mAOD)	0.5% + 106% 31.161	0.5% + 106% 31.161	0.5% + 67% 30.775	0.5% + 67% 30.775	0.5% + 67% 30.775	0.5% + 67% 30.775	0.5% + 106% 30.862
Peak water level (mAOD)	31.211	31.128	30.775	30.774	30.818	30.775	30.863
Difference to baseline (m)	+0.050	-0.033	0.000	-0.001	+0.043	0.000	+0.001

5.3 Assumptions

5.3.1 Assumptions made in the modelling process are detailed throughout the document, but in summary they are:

- It has been assumed that all topography can be approximated from EA 1m resolution LiDAR data;
- It has been assumed that the visual estimates of the culvert dimensions taken during the site walkover are sufficiently accurate for the purpose of establishing if the Hydrogen Storage Facility HAGI site is outside the area of fluvial flood risk associated with the Puddinglake Brook;

5.3.2 It has been assumed that the length of the culverts under the A road is immaterial and that the culverts can be represented using an orifice unit. This assumption has been verified through sensitivity testing.

6. Model proving

6.1 Run performance and parameters

Run performance indicators and other parameters for the two key runs are provided in **Table 6.1.** The model simulations were smooth and did not generate any errors or warning. There was no need to deviate from default parameters. The mass balances are stable.

Table 6.1 Model Proving

Event	0.5% + 67% CC	0.5% + 106% CC
Model Run parameters and settings	Default	Default
Model Network and version	Flood Modeller v5.1 Puddinglake_S1_011	Flood Modeller v5.1 Puddinglake_S1_011
Model Files	PUDL_FLU_Base_011_U_200CC67yr.ief	PUDL_FLU_Base_011_U_200CC106yr.ief
Timestep	20 seconds	20 seconds
Model stability	No non-convergences	No non-convergences
Model Warnings	No errors or warnings	No errors or warnings
Mass balance error	3.15% (of peak system volume); 0.13% (of boundary inflow volume)	2.78% (of peak system volume); 0.10% (of boundary inflow volume)
Glass walling	No glass walling	No glass walling

7. Model results

7.1.1 Peak water levels are shown for all modelled events in Table 7.1. The current minimum elevation of the HAGI site (derived from Sensat[12] detailed LiDAR derived DSM) is 31.21mAOD, whereas the elevation in the adjacent brook as it flows past the southern side of the HAGI (at Model Unit 416) ranges between 29.2 - 31mAOD. The proposed cut and fill break even height of the HAGI platform is 32.19mAOD.

7.1.2 The design water level at this model unit is 30.8mAOD (0.5% AEP + 67% CC) and therefore approximately 0.4m below elevation of the HAGI Limits of Deviation.

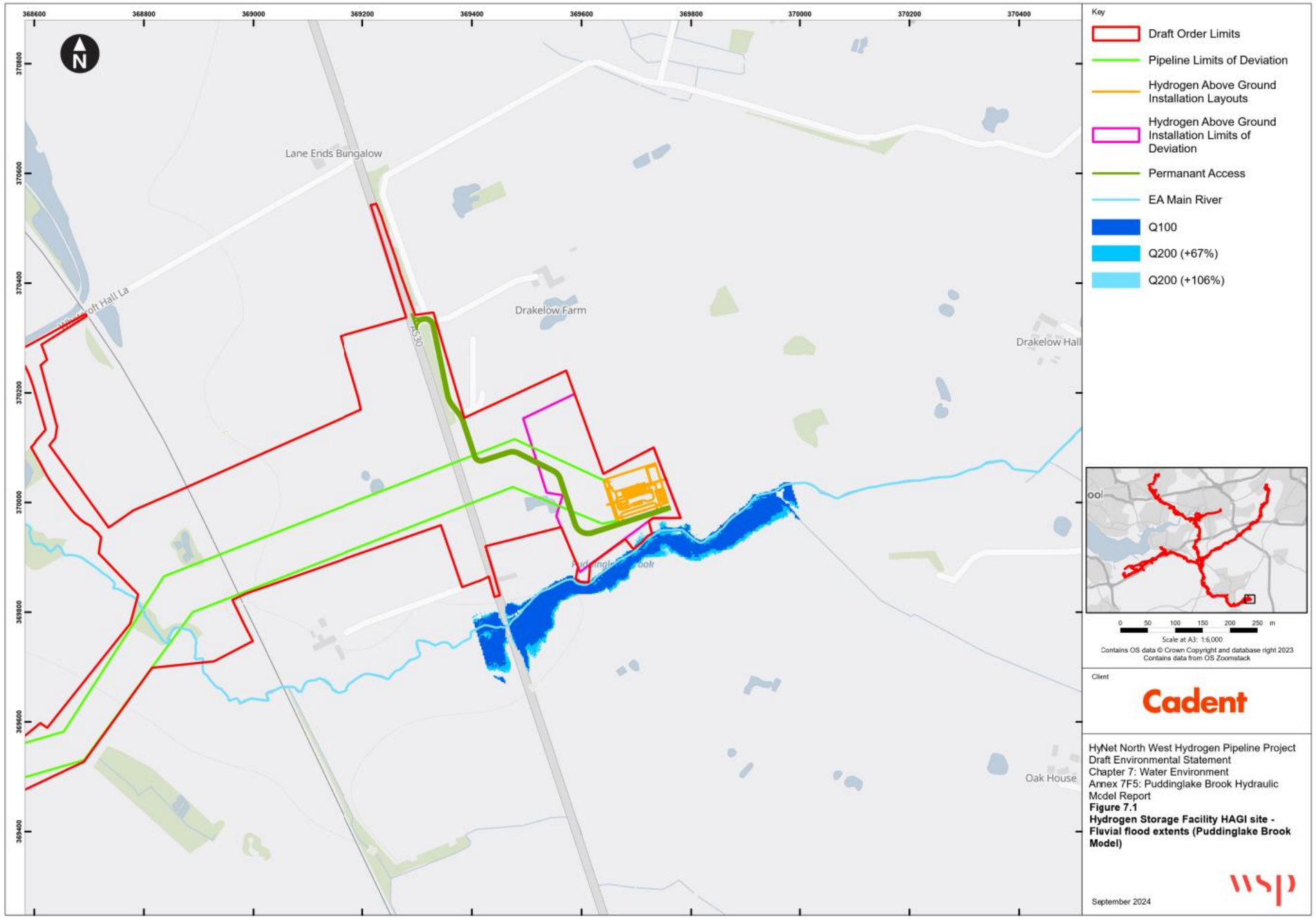
7.1.3 The highest modelled water level at this model unit is found during the 0.5% + 106% CC AEP event, and is 30.9mAOD, approximately 0.3m below the minimum elevation of the HAGI Limits of Deviation.

7.1.4 A 1D flood Map (**Figure 7.1**) has been produced. It is displayed below and shows that the flood extent is limited to within the Brook and on the left floodplain, rather than the right floodplain where the HAGI is proposed.

[12] Sensat survey: Photogrammetric drone survey commissioned by Cadent for the project that covers the entire Red Line Boundary (i.e. including the HAGI sites), DSM with horizontal resolution of 0.1m and vertical accuracy of 0.042m.

Table 7.1 Full results showing peak water levels for all cross sections during all simulated events. PUDL_0416 is the cross section nearest the Hydrogen Storage Facility HAGI site.

	Annual Exceedance Probability								
River Cross section	**50%**	**20%**	**10%**	**3.33 %**	**1%**	**0.5%**	**0.5% + 67%**	**0.5% + 106%**	**0.1%**
PUDL_0699	31.6	31.6	31.7	31.7	31.7	31.8	31.8	31.9	31.8
PUDL_0699ln1	31.2	31.2	31.3	31.3	31.4	31.4	31.5	31.5	31.4
PUDL_0699ln2	30.8	30.9	30.9	31.0	31.0	31.1	31.2	31.2	31.1
PUDL_0416	30.5	30.5	30.6	30.6	30.7	30.7	30.8	30.9	30.8
PUDL_0312	30.1	30.2	30.2	30.3	30.4	30.4	30.5	30.5	30.5
PUDL_0210	29.7	29.9	30.0	30.2	30.2	30.2	30.3	30.4	30.3
PUDL_0069	29.4	29.7	29.9	30.1	30.1	30.2	30.2	30.3	30.2
PUDL_0051	29.3	29.4	29.5	29.5	29.6	29.6	29.7	29.7	29.6
PUDL_0051ln1	29.3	29.4	29.5	29.5	29.5	29.6	29.6	29.7	29.6
PUDL_0051ln2	29.3	29.4	29.4	29.5	29.5	29.6	29.6	29.7	29.6
PUDL_0051ln3	29.2	29.4	29.4	29.5	29.5	29.5	29.6	29.6	29.6
PUDL_0051ln4	29.2	29.4	29.4	29.5	29.5	29.5	29.6	29.6	29.6
PUDL_0051ln5	29.2	29.3	29.4	29.4	29.5	29.5	29.6	29.6	29.5
PUDL_0000	29.2	29.3	29.4	29.4	29.5	29.5	29.5	29.6	29.5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369600
369400
N
Lane Ends Bungalow
Drakelow Farm
Drakelow Hall
Oak House
Key
Draft Order Limits
Pipeline Limits of Deviation
Hydrogen Above Ground Installation Layouts
Hydrogen Above Ground Installation Limits of Deviation
Permanant Access
EA Main River
Q100
Q200 (+67%)
Q200 (+106%)
0 50 100 150 200 250 m
Scale at A3: 1:6,000
Contains OS data © Crown Copyright and database right 2023
Contains data from OS Zoomstack
Client
Cadent
HyNet North West Hydrogen Pipeline Project
Draft Environmental Statement
Chapter 7: Water Environment
Annex 7F5: Puddinglake Brook Hydraulic Model Report
Figure 7.1
Hydrogen Storage Facility HAGI site - Fluvial flood extents (Puddinglake Brook Model)
September 2024

8. Conclusions

8.1.1 A new model has been constructed using Flood Modeller software to assess flood risk from the Puddinglake Brook to the proposed Hydrogen Storage Facility HAGI Limits of Deviation south of Northwich.

8.1.2 The model reach spans approximately 700m and includes the Puddinglake Brook, the left and right floodplains and a representation of the A5300 triple culverted road bridge. The model reach is based on 1m resolution EA Lidar Data, and roughness values and structure representations are based on notes and photographs collected during a field survey.

8.1.3 New design hydrological inflows have been developed as part of this assessment, and utilise a hybrid approach using Statistical Method flows scaled to the ReFH2 hydrograph.

8.1.4 The model has been run with seven AEP present day events ranging from 2% to 0.1% AEP. In addition, climate change uplifts of +67% and +106% AEP (Higher Central and Upper End) were applied for the 0.5% AEP events. In all cases, the Hydrogen Storage Facility HAGI Limits of Deviation remains outside of the flood extent. Resultant model performance for the model runs is within the acceptable bounds.

Appendix A
Flood estimation (hydrology) report

Client Name

INNOVYN HAGI, HYNET

Flood Estimation Report

PROJECT REF
JUNE 2024

CONFIDENTIAL

Client Name

INNOVYN HAGI, HYNET

Flood Estimation Report

TYPE OF DOCUMENT (VERSION) CONFIDENTIAL

PROJECT NO. PROJECT NUMBER

OUR REF. NO. PROJECT REF

DATE: JUNE 2024

WSP

70 Chancery Lane
London
WC2A 1AF

WSP.com

QUALITY CONTROL

Issue/revision	First issue	Revision 1	Revision 2	Revision 3
Remarks			FEH statistical flows added in response to EA comment.	
Date	February 2023	March 2023	June 2024	
Prepared by	James English	Jack Park	Mairi Conway	
Signature				
Checked by	Jack Park	Stephen Anderton	Laura Davey	
Signature				
Authorised by			Tim Jolley	
Signature				
Project number				
Report number				
File reference				

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INTRODUCTION

This calculation record is based on a template provided in the Environment Agency's flood estimation guidelines (version 9, 2023). It provides a record of the hydrological context, the calculations and decisions made during flood estimation and the results for the Puddinglake Brook, Cheshire West and Chester. This assessment was carried out to provide the inflow boundaries for a hydraulic model to evaluate flood risk for the proposed Innovyn Hydrogen Above Ground Infrastructure (HAGI), which is part of the HyNet North West Hydrogen Pipeline project.

ABBREVIATIONS

AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
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
IF	Impervious Fraction
IRF	Impervious Runoff Factor
LF	Low flow statistics (flow duration curve)
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 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	Windows Frequency Analysis Package – used for FEH statistical method

1 SUMMARY OF ASSESSMENT

1.1 SUMMARY

Catchment location	Puddinglake Brook, Puddinglake, Cheshire West and Chester
Purpose of study and complexity	This assessment has been carried out to produce peak flows and hydrographs for the Puddinglake Brook for input to a hydraulic model, to provide an updated baseline assessment of flood risk to the Innovyn HAGI which is situated on the right bank of the watercourse
Key catchment features	Small, largely rural catchment.
Flooding mechanisms	Fluvial/ordinary watercourse
Gauged / ungauged	Ungauged
Final choice of method	FEH Statistical Method for peak flows and ReFH2 for hydrograph shape
Key limitations / uncertainties in results	The main limitations in this study are the lack of gauged data to aid hydrological or hydraulic model calibration. The catchment is also small and so the response is likely to be uncertain.

1.2 NOTE ON FLOOD FREQUENCIES

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

The results presented in this document are quoted in terms of return period. The table below provides a quick conversion between return periods and annual exceedance probabilities.

AEP (%)	50	20	10	5	4	3.33	0	1.33	1	0.5	0.2	0.1
Return Period (yrs)	2	5	10	20	25	30	50	75	100	200	500	1000

2 METHOD STATEMENT

2.1 REQUIREMENTS FOR FLOOD ESTIMATES

Overview	This study has been undertaken to provide peak flow values and hydrographs for a range of flood events (100% AEP to 0.1% AEP) to input into a simple hydraulic model to assess the channel capacity of, and flood risk from the Puddlinglake Brook, Cheshire West and Chester. The catchment lies in the Weaver Gowy Management Catchment, as such the 'Higher Central' and 'Upper End' climate change allowances of 67% and 106%, respectively have been applied in accordance with the Environment Agency climate change guidance[1] for Nationally Significant Infrastructure Projects. Flow estimates have been generated at a single flow estimation point at the A530 crossing of Puddinglake Brook (369500, 369800), immediately downstream of the HAGI location. This document summarises the analysis undertaken.
Project Scope	The catchment is small (~5km^2) and ungauged. A simple ReFH2 methodology and FEH Statistical method have been employed for comparison of peak flows and the implications for the proposed Innovyn above ground infrastructure location.

[1] https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances (Accessed May 2024)

2.2 THE CATCHMENT

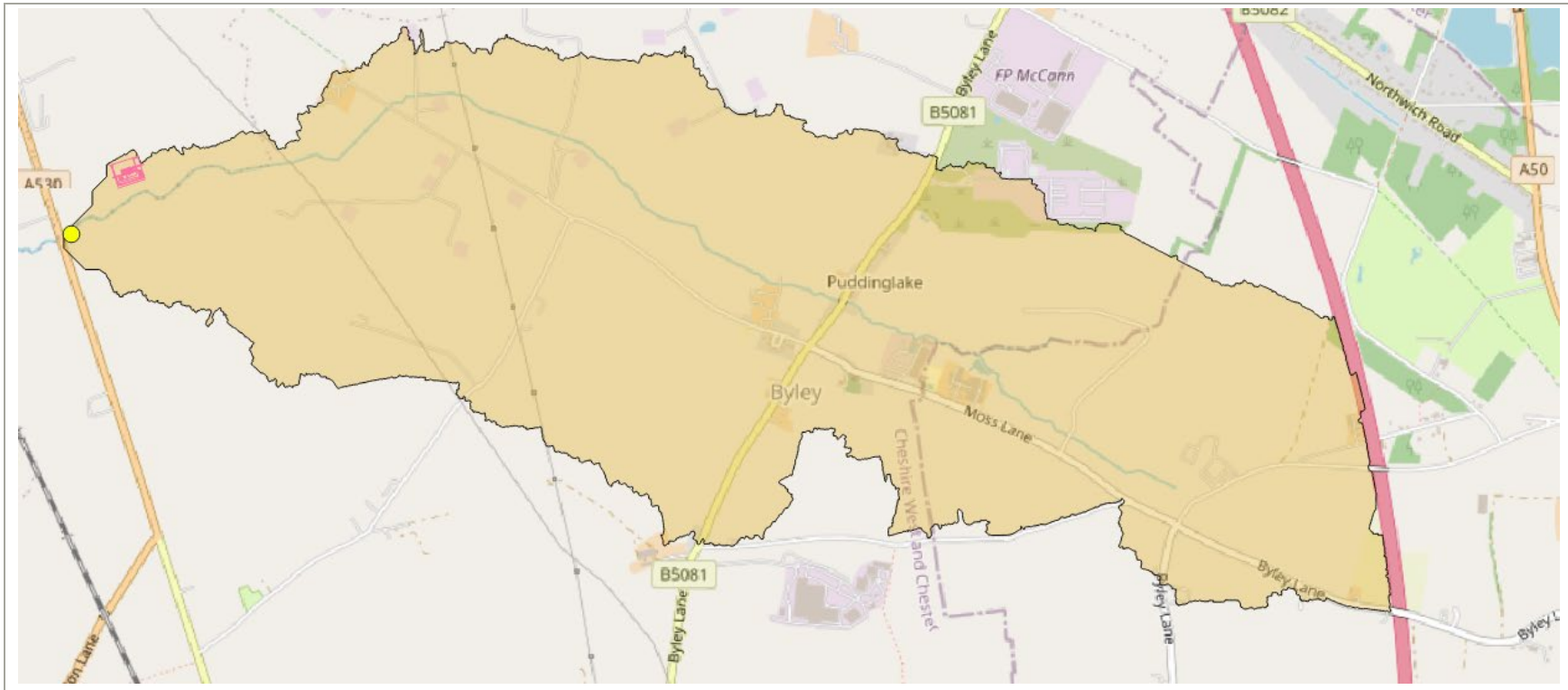


Figure 2-1 - Catchment showing flood estimation point (yellow) and location of the HAGI (pink).

Description	The catchment covers an area of ~5km^2, as defined by the EA LiDAR data. The catchment is largely rural, and the dominant land cover is agricultural. Topography varies from approximately 58mAOD in the east of the catchment to approximately 28mAOD in the west of the catchment. Soil data from SoilScapes indicates that the catchment is predominantly underlain by slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils with impeded drainage in the west and central areas of the catchment, and by naturally wet very acid sandy and loamy soils in the easternmost part of the catchment. This is reflected in a medium BFIHOST values of 0.473. BGS mapping indicates that the entire catchment is underlain by Sidmouth Mudstone Formation (mudstone) with glaciofluvial sheet deposits of sand and gravel in the east of the catchment and Devensian till (diamicton) present in the central and western parts of the catchment.

2.3 HYDROMETRIC DATA

Source of flood peak data	N/A – ungauged catchment

Gauging stations (flow and level)

Watercourse	Station Name	Gauging Authority Number	NRFA Number	Catchment area (km^2)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
N/A – ungauged catchment						

Data available at each flow gauging station

Station Name	Start and end of NRFA flood peak record	Updated for this study?	Suitable for QMED?	Suitable for Pooling?	Data quality check needed?	Station and flow data quality summary
N/A – ungauged catchment						
Updates or revisions to flood peak data				N/A		
Data quality checks carried out						

Rating equations

Station Name	Type of rating	Rating review needed?	Comments and link to any rating reviews
N/A – ungauged catchment			

Rating reviews

Station Name	Rating review
N/A – ungauged catchment	

Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Check flow gaugings (if planned to review ratings)	-	-	-	-
Historical flood data	-	-	-	-
Flow or river level data for events	-	-	-	-
Rainfall data for events	-	-	-	-
Potential evaporation data	-	-	-	-
Results from previous studies	-	-	-	-
Other data or information	-	-	-	-

Conclusions of hydrometric data review

Station Name	Rating suitability	Suitability for flood estimation calculations	Non-stationary analysis requirements
N/A – ungauged catchment			

2.4 HYDROLOGICAL UNDERSTANDING OF CATCHMENT

Plots of flood peak data and interpretation	N/A
Plots of flow data and interpretation	N/A
Plots of stage data and interpretation	N/A

Conceptual model	The catchment is small (~5km^2), and predominantly flat. The BFIHOST value of 0.473 indicate the catchment is relatively impermeable and likely to be moderately responsive, though possibly receives some shallow groundwater contributions from superficial deposits.
Unusual catchment features	

2.5 INITIAL CHOICE OF APPROACH

Are FEH methods appropriate?	Yes, the catchment is relatively small (~5km^2) and is ungauged.
Initial choice of method(s) and reasons	Standard FEH approaches – FEH Statistical method and ReFH2 will be used to allow for a comparison of both methods.
How will hydrograph shapes be derived if needed?	ReFH2 hydrograph shapes.
Will the catchment be split into sub-catchments? If so, how?	N/A
Software to be used (with version numbers)	FEH Web Service[2] / WINFAP 5[3] / ReFH2.3

[2] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.
[3] WINFAP 5 © Wallingford HydroSolutions Limited 2012.

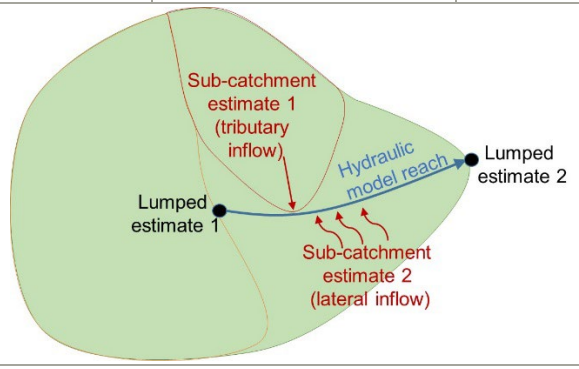
3 LOCATIONS WHERE FLOOD ESTIMATES REQUIRED

3.1 SUMMARY OF SUBJECT SITES

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

Site code	Type of estimate L: Lumped catchment S: Sub-catchment	Watercourse	Site name/ Description of site	Easting	Northing	AREA on FEH Webservice (km^2)	Revised AREA if altered (km^2)
PUDL_01	L	Puddinglake Brook	Lumped inflow taken from downstream model extent and applied at the upstream extent.	369500	369800	6.98	5.04

Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required.
Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced.
The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.

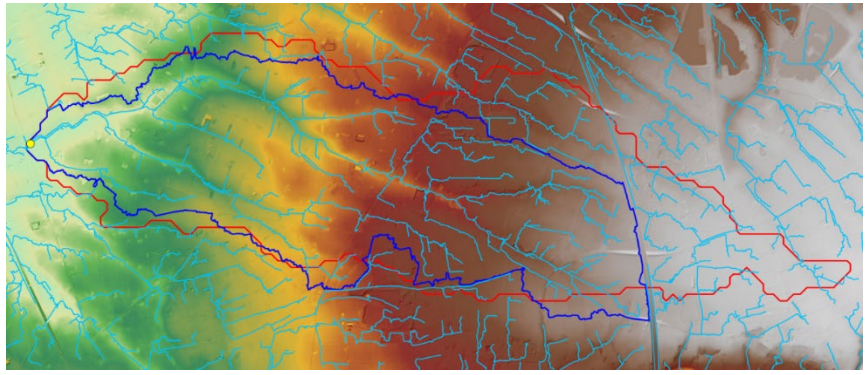


3.2 CATCHMENT DESCRIPTORS

Final catchment descriptors at each subject site (any changes made in red)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT1990*	URBEXT2000*	FPEXT
PUDL_01	0.981	0.37	0.473	2.86	9	764	0.0043	0.0233	0.3046

*Updated to 2024

Catchment boundary checks and revisions	The FEH catchment (red) has been checked against the catchment delineated from the 2021 EA LiDAR data. The FEH catchment erroneously extends upstream (east) of the M6 and includes a portion of a neighbouring catchment to the north as shown in Figure 3-1 below. Drainage lines delineated from the LiDAR data suggest that runoff generated upstream of the M6 would predominantly drain to the south and northwest, and any runoff intercepted by the M6 would be diverted into neighbouring catchments to the northwest and south. The catchment area has therefore been revised to 5.04km^2.  **Figure 3-1 - The original FEH catchment is shown in red, revised catchment in blue, drainage lines (light blue) and the flood estimation point (yellow) are also displayed.**

URBEXT source and method for updating	URBEXT2000 - URBEXT2000 was updated to 2024 using the updated CPRE formula from DEFRA R&D Technical Report FD1919/TR. URBEXT was checked visually using satellite imagery and OS mapping and deemed to be representative of the catchment. There is no discernible difference between urbanisation levels in the original FEH catchment and revised catchment, hence the URBEXT value has not been changed beyond updating to 2024.
BFIHOST source, checks and updates	Soil descriptor values (BFIHOST and SPRHOST) were checked against soilscapes data. The FEH descriptors represent the soil types well, and the revised catchment is unlikely to impact the values significantly, given that both catchments are almost entirely underlain by slowly permeable and naturally wet soils. Therefore, these descriptor values remain unchanged.
Checks and revisions to other catchment descriptors	The FARL value was checked against OS Mapping for surface water features and satellite imagery. There are a number of small ponds throughout the catchment, and hence the value of 0.981 is deemed to be representative. The revised catchment boundary makes no discernible impact to the prevalence of reservoirs and surface waterbodies within the catchment, and hence the original descriptor value remains unchanged. DPLBAR has been adjusted given that this is based on the catchment area. The standard equation for DPLBAR, given in the FEH Volume 5, uses a power term of 0.548 based on research for the UK as a whole. This has been used to update DPLBAR to 2.86. The topographic slope has been sampled for the revised Puddinglake Brook catchment and FEH catchment, as an indicator for DPSBAR. The mean slope within each catchment is similar and within 5% of each other, and hence the DPSBAR value remains unchanged.

4 STATIONARY STATISTICAL METHODS

4.1 METHOD OVERVIEW

What is the purpose of applying this method?	Lumped estimate at key locations for comparison with peak flow estimates from ReFH2.

What methods will be used to estimate QMED and growth curves?

Site code	Methods used for QMED	Methods used for growth curves
PUDL_01	DT	Pooled - ungauged

4.2 ESTIMATING QMED

QMED at ungauged subject sites

Site code	Method (CD/DT/BCW)	Initial QMED (rural) from CDs (m^3/s)	Donors used (NRFA numbers)	Donor distances from subject centroid	Individual donor weights	Combined and weighted donor adjustment factor	Urban adjustment factor	Final QMED (m^3/s)
PUDL_01	DT	1.33	68007	7.59	0.409	0.86	1.03	1.17 (urban)
			68011	13.33	0.353			
			68001	20.11	0.308			
			68020	21.57	0.299			
			68006	22.11	0.295			
			68018	22.58	0.293			

Method used for urban adjustment for subject and donor sites		WINFAP v5
Parameters used for WINFAP v5 urban adjustment (if applicable)		
Impervious fraction for built-up areas, IF	**Percentage runoff for impervious surfaces, PR_{imp}**	**Method for calculating fractional urban cover, URBAN**
0.4	70%	From updated URBEXT2000
Notes Methods: CD - Catchment descriptors alone; DT - catchment descriptors with donor transfer; BCW - catchment descriptors with bankfull channel width.		

Search for donor sites for QMED

Comment on potential donor sites	The closest sites up to a distance of 32km (based on centroid distance) were identified using WINFAP FEH v5. The parameters were set to only include sites suitable for QMED and sites with an URBEXT2000 <0.03. An additional check for other donor sites including sites not suitable for QMED was also undertaken. According to EA guidance for small catchments, “On average there is no advantage in choosing a small donor catchment or using catchment characteristics to select donors”. Therefore, the closest 6 catchments were chosen as the most suitable donors to give more certainty to the QMED estimate. It should be noted that four of the six closest catchments have centroids >20km from the subject centroid. EA guidance state that the effect of increased distance from the site of interest is relatively small when the centroid is over 10km apart and as greater weighting has been given to the closer sites it is deemed preferable to use multiple donors rather than a single donor to minimise the error of a single donor being un-representative.

Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	Deurbanised QMED from flow data (A)	Deurbanised QMED from catchment descriptors (B)	Adjustment ratio (A/B)
68007	AM	No	20.26	19.58	1.03
68011	AM	No	5.97	11.09	0.54
68001	AM	No	45.27	54.58	0.83
68020	AM	No	15.06	17.84	0.84
68006	AM	No	51.95	52.26	0.99
68018	AM	No	40.84	52.36	0.78

4.3 ESTIMATING GROWTH CURVES

Derivation of growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Any urban adjustment or non-flood years adjustment	Parameters for distribution (location, scale and shape after adjustments)	Growth factor for 100-year return period
PUDL_01	P - ungauged	PUDL_01_pooling	GEV – best fitting Z value according to WINFAP FEH v5.	N/A	Location: 0.847 Scale: 0.408 Shape: -0.119	3.35
Notes:						

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Any urban adjustment or non-flood years adjustment	Parameters for distribution (location, scale and shape after adjustments)	Growth factor for 100-year return period
Methods: SS - Single Site; P - Pooled; ESS - Enhanced Single Site; H - Historical. Pooled and ESS growth curves were derived using the procedures from Science Report SC050050 (2008). Urban adjustments are carried out using the method of Kjeldsen (2010).						

Flood frequency curve plots

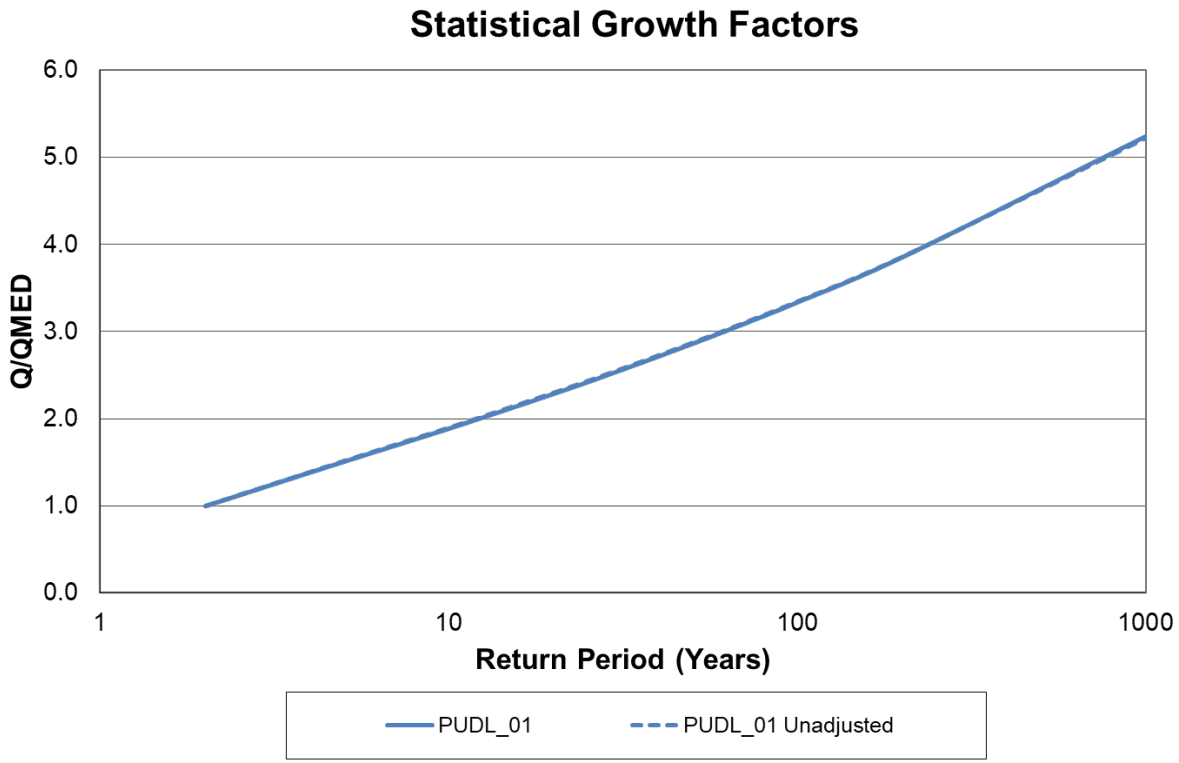


Figure 4-1 - Rural and urban growth curves for PUDL_01

Derivation of pooling groups

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	URBEXT2000 threshold applied to pooling group selection?	L-moments deurbanised (including subject site for ESS)	Small catchment pooling procedure applied?
PUDL_01_pooling	PUDL_01	No	0.03	L-CV: 0.281 L-Skew: 0.249	Yes

Methods: Unless otherwise stated, pooling groups were derived using the procedures from Science Report SC050050 (2008). The small catchment pooling procedure is given in the report on Phase 2 of project SC090031 (2021) and implemented in WINFAP v5.

Pooling group composition

Name of group	Changes made to default pooling group, with reasons	Weighted average L-moments
PUDL_01	No changes were made to the default pooling group. 49005 (Bolingey Stream @ Bolingey Cocks Bridge) was investigated due to discordancy however EA guidance states that discordancy alone is not a reason to reject a station from a pooling group. Additionally, as the station does not provide a long record of data it will not significantly impact results, consequently the station was retained. Moreover, a review of the group was desirable due to its heterogeneous nature, as such the catchment descriptors of the other sites within the group were checked. This check revealed a large range in BFIHOST values across the sites with four of the sties having BFIHOST values greater than 0.8. In such cases the other catchment descriptors were deemed okay so the sites retained as a high BFIHOST alone does not justify rejection. Non-flood year adjustment was not applied to the records of the catchments with high BFIHOST values as EA guidance notes 'the adjustment procedure applies only to the GL distribution' and in this case a GEV distribution was applied. Consequently, the pooling group is deemed acceptable with the inclusion of the sites discussed and no non-flood year adjustment applied. The full pooling group is available in Appendix 7.1.	L-CV: 0.278 (urban) L-Skew: 0.253 (urban)

4.4 FINAL CHOICE OF QMED AND GROWTH CURVES

Method choice and reasons

Site code	Final choice of QMED and reasons	Final choice of flood growth curve method and reasons
PUDL_01	DT – ungauged catchment, no flood peak data available at subject site	Pooling – ungauged catchment, no flood peak data available at subject site

Final flood estimates from the stationary Statistical methods

Site code	Flood peak (m^3/s) for the following return periods (in years)											
	2	5	10	20	30	50	75	100	100 (CC 67%)	100 (CC 106%)	200	1000
PUDL_01	1.17	1.77	2.21	2.68	2.97	3.35	3.67	3.91	6.53	8.05	4.52	6.39

5 REVITALISED FLOOD HYDROGRAPH 2 (REFH2) METHOD

5.1 APPLICATION OF THE REFH2 METHOD

What is the purpose of applying this method?	The ReFH2 method has been applied to provide peak flows and hydrographs for the catchment.
Rural and urban catchment sub divisions	Largely rural catchment
Version of ReFH2 applied	ReFH2.3-FEH22

5.2 MODEL PARAMETERS

Summary of model parameters

Site code	Method	Tp_{rural} (hours)	Cmax (mm)	BL (hours)	Area modelled as urban (km^2)	IF	IRF	DS
PUDL_01	L	5.405	374.074	39.031	0.18	0.4	0.7	0.5
Brief description of any flood event analysis carried out (further details should be given in the annex)								

Methods: OPT: Optimisation from event analysis, BR: Baseflow recession fitting, LAG: TP from lag analysis, CD: Catchment descriptors, DT: Data transfer, CAL: model calibration.

5.3 MODEL INPUTS FOR DESIGN EVENTS

Design events for Lumped Catchments

Site code	Rainfall DDF Model	Urban or rural	Highly permeable?	Season of design event	Storm duration (hours)	Initial Soil Moisture Cini	Initial Baseflow BF0
PUDL_01	FEH22	Urban	No	Winter	9:00	99.05	0.14

Results of storm duration testing.	The default storm duration has been applied.

5.4 FINAL CHOICE OF REFH2 FLOW ESTIMATES

Method choice and reasons

Site code	Final choice of design inputs and model parameters
PUDL_01	Default urban winter parameters.

Final flood estimates from the ReFH2 method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	30	50	75	100	100 (CC 67%)	100 (CC 106%)	1000
PUDL_01	0.97	1.3	1.59	2.19	2.53	2.82	3.03	5.06	6.24	4.95

6 DISCUSSION AND SUMMARY OF RESULTS

6.1 COMPARISON OF RESULTS FROM DIFFERENT METHODS

Site code	2 year			100 year		
	Statistical peak flow (m^3/s)	ReFH2 peak flow (m^3/s)	Ratio	Statistical peak flow (m^3/s)	ReFH2 peak flow (m^3/s)	Ratio
PUDL_01	1.17	0.97	0.83	3.91	3.03	0.77

6.2 FINAL CHOICE OF METHOD

Choice of method and reasons	The FEH Statistical method for estimating flood flows is favoured as it is based on a much larger dataset of flood events and has been more directly calibrated to reproduce flood frequency on UK catchments giving it a greater confidence in deriving the index flood (QMED). A hybrid approach using Statistical Method flows scaled to the ReFH2 hydrograph was adopted to produce design hydrographs for the hydraulic model as recommended in the Environment Agency's FEH Guidelines.
How will the 1000 year / 0.1% AEP flows be estimated?	The FEH Statistical method is not recommended for use beyond the 200 year return period. The ratio between the ReFH2 100yr and 1000yr flows was applied to the 100yr FEH Statistical flow to estimate the 1000yr flow.
How will the flows be applied to a hydraulic model?	The inflows will be applied as a single lumped inflow at the upstream extent of the model. This is deemed appropriate given the relatively short model reach and lack of confluences within the area of interest.

6.3 FINAL RESULTS

Site code	Flood peak (m^3/s) for the following return periods (in years)											
	2	5	10	20	30	50	75	100	100 (CC 67%)	100 (CC 106%)	200	1000
PUDL_01	1.17	1.77	2.21	2.68	2.97	3.35	3.67	3.91	6.53	8.05	4.52	6.39

Climate change allowances

Type of allowance	Higher Central (67%) and Upper End (106%) climate change allowances for peak flow river data.
Source of data	Environment Agency climate change guidance (https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances (Accessed May 2024))
River basin	Weaver Gowy Management Catchment
Epoch(s)	2080s
Percentiles	
Application	Applied to the 1% AEP.

6.4 CHECKS

Growth factor checks

Site code	100 year growth factor	1000/100 year ratio
PUDL_01	3.34	1.63
Discussion on growth factors	The 100 year growth factor falls within the typical range of 2.1-4.0. The 0.1%/1% AEP event ratio falls within the typical range of 1.3-1.8.	

Specific discharge

Site code	Specific discharge (l/s/ha) for the following return periods (in years)											
	2	**5**	**10**	**20**	**30**	**50**	**75**	**100**	**100 (CC 67%)**	**100 (CC 106%)**	**200**	**1000**
PUDL_01	2.32	3.51	4.39	5.31	5.89	6.64	7.28	7.75	12.94	15.97	8.95	12.67
Discussion on specific runoffs	The 1% AEP is 7.75 l/s/ha which falls within the typical range of 2-10 l/s/ha.											

Spatial consistency	N/A – single inflow points.
Return periods for notable historic floods	N/A – no historic flood records
Compatibility with longer-term flood history	N/A – no data was identified to support any such assessment
Comparison with previous studies	N/A - No previous studies
Checks on hydraulic model results	N/A

6.5 ASSUMPTIONS, LIMITATIONS AND UNCERTAINTY

List the main assumptions made (specific to this study)	The main assumptions in this study are that: • The pooling group suitably represents the Puddinglake Brook catchment. • The ReFH2 hydrograph shape is representative of the catchment response
Discuss any particular limitations	The main limitations in this study are the lack of gauged data to aid hydrological or hydraulic model calibration. The catchment is also small and so the response is likely to be uncertain.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	. The confidence limits given in the EA Flood Estimation Guidelines assume that the AMAX flows follow a generalised logistic distribution, therefore they cannot be applied to the flow estimates in this study as a GEV distribution has been fitted. Using donors to calculate QMED has reduced the uncertainty in the calculations leading to a reduction from 1.39 to 1.36 in the Factorial Standard Error (FSE).

Site code	**Upper and lower 95% confidence bounds for the flood peak (m3/s) for the following return periods (in years)**							
	2		**20**		**100**		**1000**	
	Lower	**Upper**	**Lower**	**Upper**	**Lower**	**Upper**	**Lower**	**Upper**
PUDL_01	N/A – GEV distribution fitted.							

Suitability of the results for future studies	The design flow estimates have been derived for the purposes of this hydrological assessment in order to inform the flood risk associated with the Puddinglake Brook. If peak flow estimates are required for different purposes, it is recommended that, at minimum, a review of results is carried out.

Recommendations for future work	As in most ungauged catchments, it is recommended that temporary flow gauges be installed to better inform the design peak flow estimates. There are no other specific suggestions relevant to this study.

7 APPENDIX

7.1 POOLING GROUP COMPOSITION

WINFAP v5 Final pooling group PUDL_01

Station	Distance	Years of Data	QMED AM	L-CV	L-SKEW	Discordancy
27073 (Brompton Beck @ Snainton Ings)	0.406	42	0.816	0.213	0.02	1.116
27051 (Crimple @ Burn Bridge)	0.5	50	4.641	0.218	0.133	0.406
25019 (Leven @ Easby)	0.899	44	5.384	0.341	0.367	0.258
26016 (Gypsey Race @ Kirby Grindalythe)	0.906	25	0.101	0.309	0.249	0.148
27010 (Hodge Beck @ Bransdale Weir)	1.274	41	9.42	0.224	0.293	0.345
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	1.281	12	4.924	0.267	0.268	3.457
45816 (Haddeo @ Upton)	1.339	29	3.248	0.29	0.432	0.648
44008 (South Winterbourne @ Winterbourne Steepleton)	1.361	31	0.544	0.414	0.268	0.99
76011 (Coal Burn @ Coalburn)	1.367	45	1.84	0.171	0.292	0.947
26014 (Water Forlornes @ Driffield)	1.481	24	0.431	0.319	0.185	0.337
36010 (Bumpstead Brook @ Broad Green)	1.538	55	7.59	0.354	0.109	1.519
7011 (Black Burn @ Pluscarden Abbey)	1.571	10	4.752	0.494	0.554	2.361
41020 (Bevern Stream @ Clappers Bridge)	1.601	53	13.66	0.202	0.172	0.68
28033 (Dove @ Hollinsclough)	1.661	47	4.15	0.231	0.381	0.788
Total		508				
Weighted Means				0.277	0.253	
H2 Value	3.55					
Goodness of Fit	**Generalised Logistic**		**Generalised Extreme Value**		**Kappa 3**	
	1.1972		-0.3637		0.6641	

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Appendix B
Manning's Assessment

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Technical note:
Innovyn HAGI Hydraulic Assessment

1. Context

The Innovyn HAGI was identified as one of four HAGIs at risk of fluvial flooding during the production of the Preliminary Environmental Information Report given the site's location adjacent to Puddinglake Brook, close to the extent of Flood Zone 2. ReFH2 calculations were undertaken to generate a series of peak flow events. Manning's calculations were then carried out to assess the capacity of the channel for the 1 in 100 year + 106% climate change event.

2. ReFH2 peak flows

ReFH2 calculations were carried out for a range of events (100% AEP to 0.1% AEP) to provide estimates for peak flows in order to assess the capacity of the channel. 'Higher Central' and 'Upper End' climate change allowances of 67% and 106%, respectively, have been applied in accordance with the Environment Agency climate change guidance for Nationally Significant Infrastructure Projects. The results are summarised in **Table 2.1**.

Table 2.1 ReFH2 peak flow values for a range of annual exceedance probabilities

Return Period / Annual Exceedance Probability (%)	Flow (m^3/s)
1 year / 100%	0.88
2 year / 50%	0.97
5 year / 20%	1.30
10 / 10%	1.59
30 / 3.3%	2.19
50 / 2%	2.53
75 / 1.33%	2.82
100 / 1%	3.03
100 + 67% CC	5.06
100 + 106% CC	6.24
200 / 0.5%	3.57

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Return Period / Annual Exceedance Probability (%)	Flow (m^3/s)
1000 / 0.1%	4.95

3. Manning's calculations

Manning's calculations were undertaken to assess the capacity of the channel adjacent to the Innovyn site. Environment Agency LiDAR data was used to derive cross sectional data for the channel. Thirteen cross sections were taken along the river reach of Puddinglake Brook, upstream of the A530 road bridge. The capacity of the channel at each cross section was assessed using the lower of the two bank levels as the bankfull level. **Figure 3.1** shows the location of the cross sections used in the Manning's calculations and highlights cross section 6, which has the lowest conveyance of all calculated sections. The flood estimation point is also displayed, which indicates the downstream most point in the catchment for which peak flows were derived using the upstream catchment. The flow direction is from east to west.

Figure 3.1 Location of cross sections, flood estimation point and Innovyn HAGI

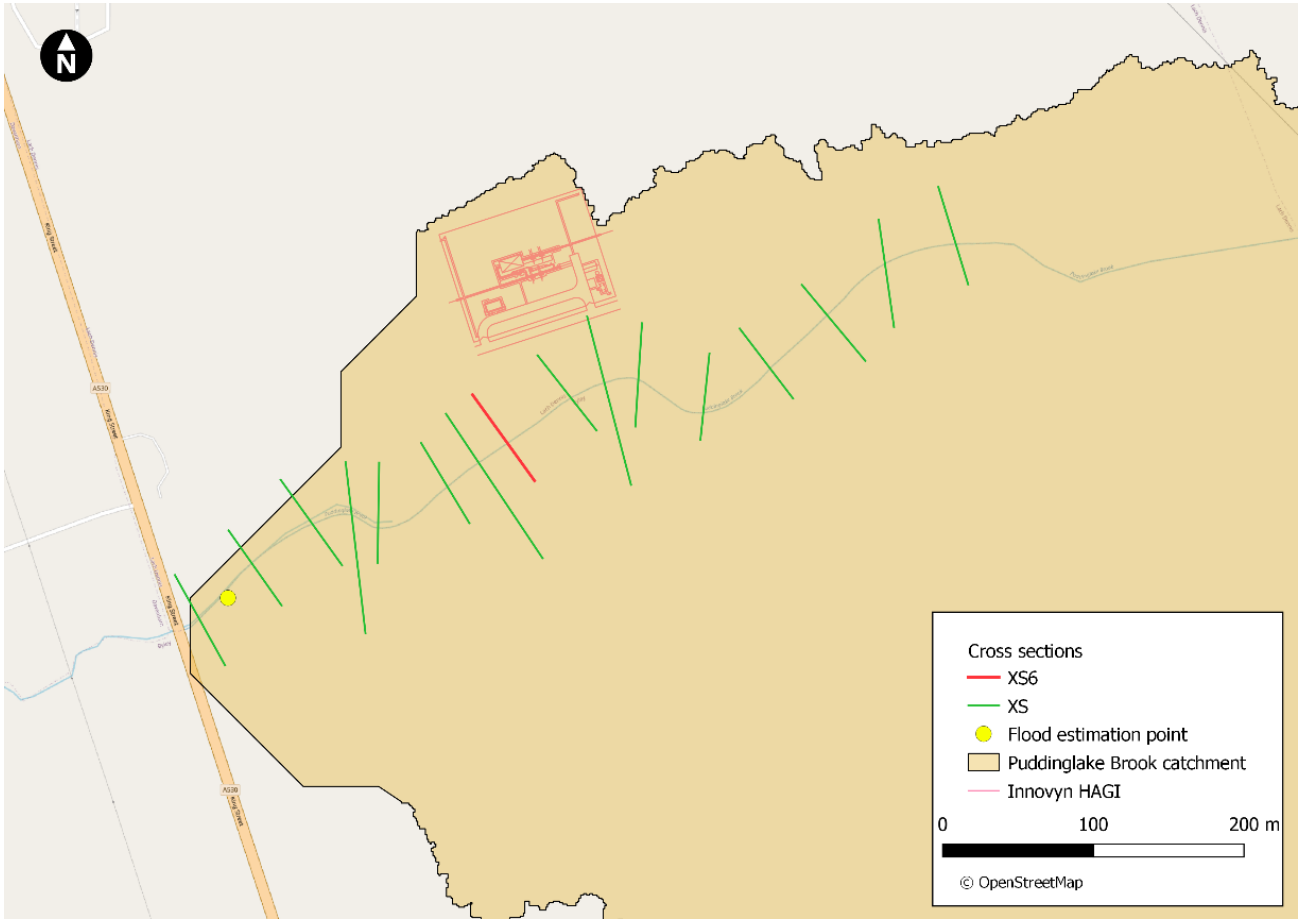

Table 3.1 Manning's calculations for Puddinglake Brook channel

Cross section ID	Conveyance (m^3/s)
XS1	3.80
XS2	5.78
XS3	4.53
XS4	5.75
XS5	3.85
XS6	2.43
XS7	3.71
XS8	4.68
XS9	4.79
XS10	4.53
XS11	4.56
XS12	4.21
XS13	2.35

A Manning's roughness coefficient of 0.03 was applied in line with observations made during site walkover that the channel bed is silt covering gravels and pebbles with some vegetation.

The most constricted section, XS6, is located immediately downstream of the Innovyn HAGI site, and is therefore most critical for causing flooding to the site. At 2.43 m^3/s the bankfull conveyance of the channel at XS6 is not sufficient to contain the 1% AEP event of 3.03m^3/s, causing water to flow on to the floodplain. The elevation of the right bank, where the HAGI is located, is consistently higher than the left bank. Therefore, when the stream level exceeds the bankfull level, the left bank floodplain would flood before the water level reached a point to exceed the right bank elevation.

The A530 road bridge at the downstream end of the reach presents a barrier to high flows. This watercourse crossing consists of three culverts beneath the road (see Figure 3.2). The two circular culverts have an estimated diameter of 0.6m and the arched culvert has an estimated width of 0.75m. These dimensions were used to assess the conveyance of the culverts against the 1% AEP + 106% CC event (6.24m^3/s).

For the two circular culverts, a version of the Manning's equation for pressure flow for circular pipes flowing full was used. The third culvert is an arched culvert, the closest approximation being a semi-circular shape. As there is no ideal equation to estimate flow through an irregular culvert, the capacity of the arched culvert was estimated using a conservative approach with the open channel capacity Manning's equation. The combined conveyance of the culverts was calculated to be 1.16m^3/s.

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Figure 3.2 View of downstream side of the A530 road bridge

The combined capacity of the culverts was estimated to be 1.16 m^3/s, which is less than the 1% AEP + 106% CC event of 6.24m^3/s and also less than the 1% AEP event without a climate change allowance of 3.03m^3/s. Consequently, the culverts' capacity is not enough to convey the flows of either of these events and it can be expected that the watercourse crossing would provide an obstacle to flow in such an event and water would back up upstream of the A530 towards the HAGI site. However, this Manning's assessment does not take into account the capacity of the floodplain on the left bank nor the possibility of water spilling over the road bridge deck away from the HAGI site. A hydraulic model will provide a more detailed assessment of whether the HAGI site is likely to experience flooding from the Puddinglake Brook during the design events including the climate change allowances.

Issued by

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Approved by

Angela Dhaliwal

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Document revisions

No.	Details	Date
1	Internal Tech Note	February 2023

Appendix C
Channel survey photographs

Photograph showing the Brook at the upstream limit (PUDL_699) viewed from the left bank

Photograph of the Brook flowing under the structure at the A530 road bridge (PUDL_0064) viewed from upstream on the left ban

Facing across the channel from the left bank, between the A530 Road bridge and HAGI (PUDL_0210)

Approximately 10m upstream of the A530 Road Bridge culvert (model River Unit PUDL_0069), facing downstream.

Facing across the channel at the proposed HAGI site, from the left bank (model River Unit PUDL_0416)

Facing downstream from the left bank at the upstream end of the model reach (model River Unit PUDL_0699)

Facing into the channel from the left bank at the upstream end of the model reach (model River Unit PUDL_0699)

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Annex 7F6
Outline Drainage Strategy

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Appendix 7F: Annex 7F6

Outline Drainage Strategy

October 2024

Revision 1
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Glossary

Term	Description
AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
BGS	British Geological Survey
BVIs	Block Valve Installations
CBSCB	Cheshire Brine Subsidence Compensation Board
CEMP	Construction Environmental Management Plan
CIRIA	Construction industry research and information association
CW&C	Cheshire West & Chester
DCO	Development Consent Order.
DCO	Development Consent Order
DEFRA	Department for Environment, Food and Rural Affairs
DoT	Department of Transport
EA	Environment Agency
EET	Essar Energy Transition Hydrogen
ES	Environmental Statement
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
HAGI	Hydrogen Above Ground Installation
HDD	Horizontal Directional Drilling
HPP	Hydrogen Production Plant
LLFA	Lead Local Flood Authority
LoD	Limits of Deviation
LPA	Local Planning Authority
NPPF	National Planning Policy Framework

Term	Description
SuDS	Sustainable Urban Drainage System
TSS	Total Suspended Solids
UU	United Utilities

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1 Introduction

1.1.1 This Outline Drainage Strategy forms **Appendix F** of the Flood Risk Assessment (FRA) which is **Appendix 7F** of **Chapter 7: Water Environment**, of the Draft Environmental Statement (ES) for the HyNet North West Hydrogen Pipeline Project ('the Project').

1.1.2 This document should also be read in conjunction with the full description of the Project provided in **Chapter 2: The Project** and **Chapter 12: Ground Conditions** in relation to expect ground conditions with respect to infiltration prospects.

1.1.3 The HyNet North West Hydrogen Pipeline project includes the construction and operation of approximately 100km of new pipeline to distribute hydrogen to industry in the North West of England. A number of Hydrogen Above Ground Installations (HAGIs), including Block Valve Installations (BVIs) are required to control the flow and pressure of hydrogen at key points along the proposed pipeline. **Figure 7F6 1.1** illustrates the extent of the Project.

1.1.4 The Project connects the Hydrogen Production Plant at the Essar Stanlow site south of Liverpool, as the source of hydrogen, for onward distribution to the Network. The pipeline transports hydrogen to the Central Hub at the centre of the Network, which then serves as the connection and onward distribution point to the North, East and South corridors, which would extend to St. Helens, Partington and Northwich respectively. Branching off from each main corridor would be pipeline spurs connecting hydrogen customers to the main pipeline. Infrastructure has also been included which supports the potential blending of hydrogen with methane in the existing gas network, should government support this activity.

1.1.5 The presently anticipated maximum extent of land in which the Project may take place, which is assessed in the Draft ES, is defined by the 'Draft Order Limits', land within which is hereafter referred to as 'land within the Draft Order Limits'.

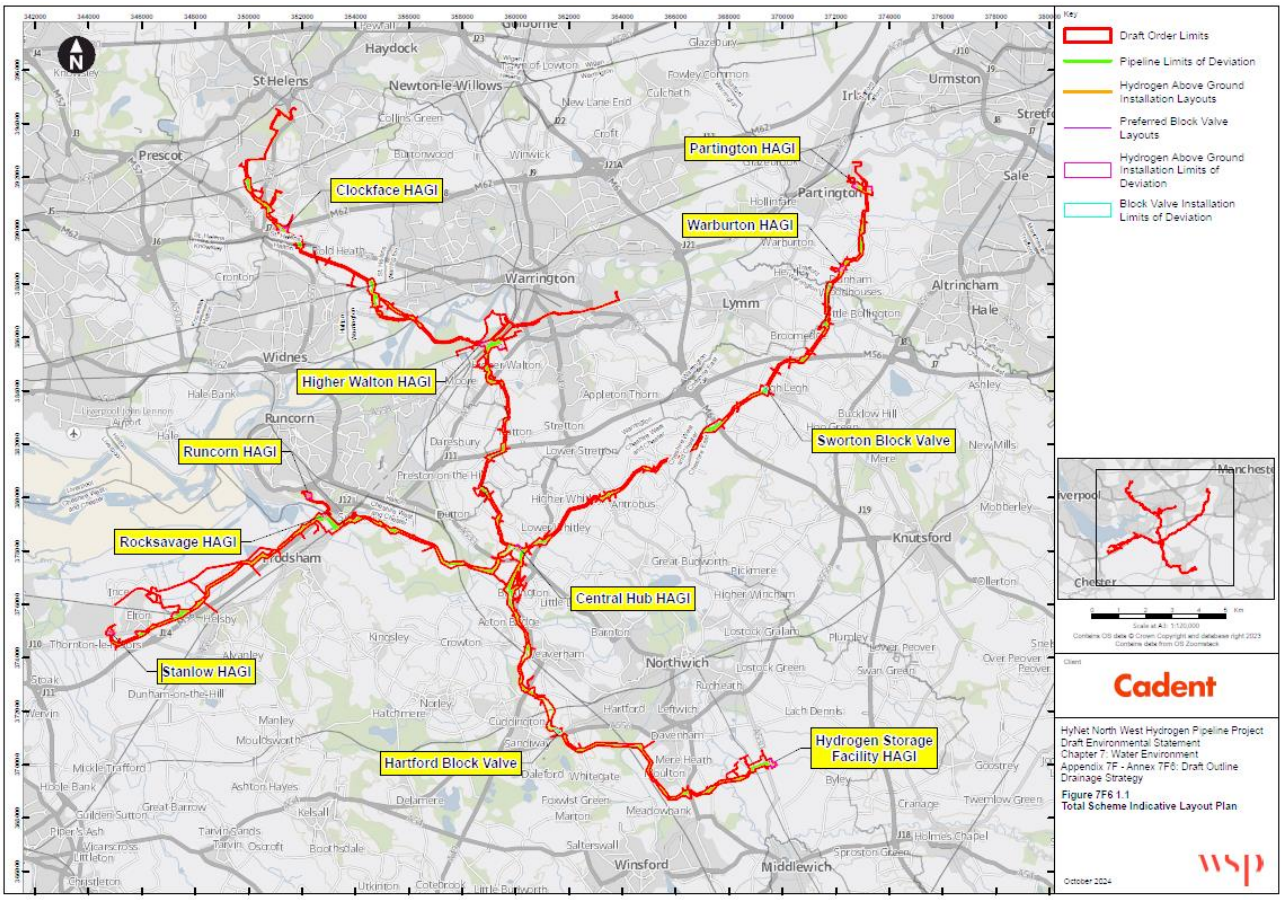
1.1.6 A proportionate degree of flexibility has been incorporated into the project design so that unforeseen issues that may be encountered following consent can be dealt with. For example, previously unidentified poor ground conditions may require a pipeline section to be re-routed slightly for geotechnical reasons. Therefore, to allow for this, new infrastructure would be constructed within specified Limits of Deviation (LoD) which identify a maximum distance or measurement of variation within which the works must be constructed.

1.1.7 The site-specific drainage designs presented in this document are preliminary at this stage based on the information presently available. We therefore expect changes to be applied throughout the developing design process before final approved construction design documentation would be available.

1.1.8 These designs therefore demonstrate the feasibility of the drainage systems presented but do not account for all the detail required for final approval and construction, as such a conservative approach has been adopted and detailed design by an elected skilled contractor will seek to refine the design within the parameters set.

1.1.9 Refinement and approval of final designs will occur post DCO consent and included in the DCO requirements will be the approval of the relevant final plans by the relevant planning authorities including the EA and LLFAs.

Figure 7F6 1.1 Total Extents of the Hynet North West Hydrogen Pipeline Project, including HAGI and BV site locations



1.1.10 The purpose of this document is to set out the Outline Drainage Strategy to be applied to all components of the Project which includes the permanent above ground installations including all Hydrogen Above Ground Installations (HAGI) and Block Valve Installation (BVI) sites. These sites are considered by way of site specific preliminary quantitative drainage designs.

1.1.11 In addition, the report also considers the drainage strategies to be applied to the temporary construction areas including:

- Temporary areas used as construction compounds for the HAGIs and BVIs.
- Temporary areas used for the storage of equipment (e.g. material storage areas).
- Temporary areas subject to soil stripping and excavation for pipeline construction, including:
  - open cut crossings of minor watercourses;
  - trenchless crossings for major watercourses; and

- linear infrastructure, potentially requiring dewatering.

1.1.12 These areas are the subject of clear drainage principles as described further in Section 15, which will be taken forward at detailed design.

1.2 Design Principles and Criteria

1.2.1 Generally, the following design criteria will apply to all permanent and temporary sites.

- The drainage design will be carried out with reference to the National Planning Policy Framework (NPPF)[1], Building Regulations Part H[2], CIRIA C753 SuDS Manual[3], DEFRA/Environment Agency guidance and the relevant LLFA's where appropriate.
- The drainage systems will be designed for 1% Annual Exceedance Probability (AEP) event with a climate change allowance that satisfies all the various LLFAs requirements. Further clarification of the climate change upliftment to be used is given in section 2.1.14
- The drainage designs will show that runoff from the site during the 1% AEP event will not increase flood risk to the surrounding environment (discharge to natural water course) as required by the NPPF. Furthermore, exceedance flows over and above this event (eg 0.5% AEP) will not increase flood risk to the site infrastructure by ensuring exceedance flow routes channel the runoff away from the infrastructure.

1.2.2 The topography of the sites has been approximated from available Lidar Contour data from DEFRA data service platform while the physical topoFigureal surveys of the sites are being undertaken. The outline drainage strategies are therefore based on this data.

1.2.3 At this stage the exact configuration of land drainage in the adjacent agricultural fields is not known, however reinstatement of these will be included in construction of the pipeline and components whether permanent or temporary. Refer to Section 15 for further detail of how these will be treated.

1.2.4 In the hydrological calculations for estimating attenuation volumes (in the MicroDrainage software) the synthetic rainfall storms were generated using FEH 2013 data in line with current best practice as FEH contains a more recent and more extensive dataset.

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[1] Ministry of Housing, Communities and Local Government. National Planning Policy Framework. 2023. (Online) Available from: https://www.gov.uk/guidance/national-planning-policy-framework (Accessed 02/08/2024)

[2] HM Government. The Building Regulations 2010. Drainage and Waste Disposal. 2015. (Online) Available from: https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/442889/BR_PDF_AD_H_2015.pdf (Accessed 02/08/2024)

[3] Woods Ballard, B.; Wilson, S.; Udale-Clarke, H.; Illman, S.; Scott, T.; Ashley, R.; Kellagher, R. The SuDS Manual (C753F), CIRIA.

1.2.5 In the calculations a conservative volumetric runoff coefficient (Cv) value has been used in accordance with industry best practice as it is assumed all rainfall will be deposited on the site directly.

1.2.6 In the current revision of the project all HAGI and BVI sites are greenfield sites except for Stanlow which is to be constructed on the existing Stanlow Refinery site in Ince and Partington HAGI Option 1 which is proposed to be constructed adjacent to the existing Cadent Gas Above Ground Installation site in Partington. Both sites are brownfield sites.

1.2.7 Greenfield runoff rates were calculated using the Wallingford website using FEH 2013 rainfall data and Qbar was chosen as the representative design runoff rate to be used. It is assumed for the greenfield sites that the surface water runoff is restricted to these calculated greenfield runoff rates.

1.2.8 In the absence of site specific infiltration testing results, a conservative assumption has been made that no infiltration is possible. This will be revisited during detailed design post-grant of the DCO. If site testing can demonstrate viable infiltration potential the relevant drainage strategies will require amending accordingly.

1.2.9 Information obtained to inform the Strategy includes the Halite Risk Assessment (2024). This provides information relating to possible subsidence due to historical brine related activities such as rock salt mining and brine pumping where the route of the proposed hydrogen distribution pipeline passes through the Cheshire Brine Subsidence Compensation District. This district affects approximately 45km of the pipeline route from Partington in the northeast through to the Hydrogen Storage Facility on the end of the southern leg.

1.2.10 The LLFA's from the six Local Planning Authorities (LPA's) covering the extent of the Project will be statutory consultees in the DCO process and therefore their requirements regarding the drainage designs and implementation of SuDS will need to be satisfied.

1.2.11 A summary of SuDS guidance from each LLFA is summarised in **Table 7F6 1.1** below.

Table 7F6 1.1 SuDS Guidance for Local Councils

Local Council	SuDS Guidance
Cheshire East Council	All developments in Cheshire East are required to implement SuDS in accordance with the SuDS's supplementary planning document for Cheshire East[4]. Development proposals must include a site- specific drainage strategy in line with SuDS guidelines[5]. It should be ensured that there is no increase in runoff rates for greenfield sites. For previously developed sites, the Non-Statutory Technical Standards for Sustainable Drainage Systems (2015)[6] must be followed to demonstrate a reduction in run-off rates.
Cheshire West and Chester Council	The possible runoff destinations in order of preference: • To ground; • To surface water body; • To surface water sewer; • To combined sewer. *"In line with policy ENV 1 of the Local Plan (Part One), residential development of* *10 dwellings or more and non-residential development of over 1,000 square* *metres of floorspace or with a site area over 1 hectare, will be required to incorporate SuDS scoping and design* *SuDS must be included within the early stages of the site design to incorporate appropriate SuDS within the development and will be required to satisfy technical standards and design requirements in accordance with the D5 SuDS Design Handbook.* *For brownfield sites, where current infrastructure may be in staying in place, then runoff should attempt to mimic that of greenfield rates with betterment of at least 30% as a minimum, unless it can be demonstrated that this is unachievable or hydraulically impractical to do so."*

[4] Sustainable (Urban) Drainage Guide Supplementary Planning Document (2024) Cheshire East Council https://moderngov.cheshireeast.gov.uk/ecminutes/documents/s114559/Sustainable%20Urban%20Drainage%20Supplementary%20Planning%20Document.pdf (Accessed 23/07/2024)

[5] Sustainable Urban Drainage Systems (2024) Cheshire East Council https://www.cheshireeast.gov.uk/planning/flooding/floods-and-flood-risk/surface-water-management.aspx (Accessed 23/07/2024)

[6] Non-Statutory Technical Standards for Sustainable Drainage Systems (2015) DEFRA https://assets.publishing.service.gov.uk/media/5a815646ed915d74e6231b43/sustainable-drainage-technical-standards.pdf (Accessed 23/07/2024)

Local Council	SuDS Guidance
Halton Council	Halton Borough Council initial assessments of the soil type and geology indicates that there is a low suitability for infiltration-type SuDS. Attenuation techniques which reduce runoff volumes and allow for water reuse may be more applicable[7].
St Helens Council	There is a comprehensive SuDS Design and Technical Guide (>100 pages), plus a SuDS assessment checklist available. Individual SuDS, within public and/ or private areas, need to follow the Management Train that reinforces the natural pattern of drainage. There is a hierarchy of techniques: • *"Prevention- use of good site design/housekeeping measures on individual sites to prevent runoff/pollution;* • *Source control – control of runoff at, or very near, its source;* • *Site control – management of water from several sub-catchments;* • *Regional control – management of runoff from several sites."*[8] For small developments, prevention and source controls must be considered. Site or regional controls e.g. detention ponds may not be feasible. The reduction of runoff and the treatment of runoff at source should follow the guidance document.
Trafford Council	Much of the Trafford Borough area is within a nationally recognised Critical Drainage Area[9]. Any new drainage systems for surface water management on development sites must be designed to 'greenfield' standards. This also applies if the land has been previously developed. *"For greenfield development, the runoff volume from the development to any sewer* *or surface water body in the 1 in 100 year, 6 hour rainfall event should never exceed the greenfield runoff volume for the same event."* "*For developments which have been previously developed, the runoff volume from the development to any sewer or surface water body in the 1 in 100 year, 6 hour rainfall event must be constrained to a value as close as is reasonably practicable to the greenfield runoff volume for the same*

[7] Local Flood Risk Management Strategy (2015) Halton Borough Council https://moderngov.halton.gov.uk/documents/s36570/Halton%20Draft%20LSD%20v13.pdf (Accessed 24/07/2024)

[8] Sustainable Urban Drainage Systems (2020) St Helens Council https://www.sthelens.gov.uk/media/5280/SuDS-Guidance-Document/pdf/sth_suds_2020_adoption_version_1.pdf?m=1672324166017 (Accessed 23/07/2024)

[9] Drainage and SuDS, Trafford Council https://trafforddesigncode.uk/landscape-and-nature/drainage-and-suds/ (Accessed 24/07/2024)

Local Council	SuDS Guidance
	event, but should never exceed the runoff volume from the development site prior to redevelopment for that event. *Where it is not reasonably practicable to constrain the volume of runoff to any drain, sewer or surface water body in accordance with the above, the runoff volume must be discharged at a rate that does not adversely affect flood risk (usually Qbar).”* The drainage hierarchy should be followed as reasonably practicable. In order of priority: 1. into the ground (infiltration); 2. to a surface waterbody; 3. to a surface water sewer; 4. to a combined sewer. Evidence and justification must be submitted to demonstrate the appropriate level of drainage has been taken. Further information is included in the SuDS Pro-forma.[10]
Warrington Council	In the event of low rainfall, the first 5mm of rainfall should not be discharged to a sewer or surface water body. Where this is not achievable (for low permeability soils), the developer must demonstrate that the SuDS Management Train has been adhered to and infiltration has been encouraged. For high rainfall events, there are two approaches which must be used to manage the surface runoff discharge unless the volume and peak flow rate do not apply. Approach 1: Peak flow and volume runoff restriction. *“The peak flow rates for the:* *A. 1 in 1 year rainfall event, and;* *B. 1 in 100 year + Climate Change rainfall event.* *Must not be greater than the equivalent greenfield runoff rates for these events. The critical duration rainfall event must be used to calculate the required storage volume for the 1 in 100 year + Climate Change rainfall event. The volume of runoff must not be greater than the greenfield runoff volume from the site for the 1 in 100 year + climate change, 6 hour rainfall event.”* Approach 2: Peak flow rate Restriction “*The critical duration rainfall event must be used to calculate the required storage volume for the 1 in 100 year + Climate Change rainfall event. The flow rate discharged:* *For the 1 in 1 year event, must not be greater than either* • *The greenfield runoff rate from the site for the 1 in 1 year event, or;* • *2 litres per second per hectare (l/s/ha);*

[10] North West SuDS Pro-Forma (2020) North West Regional Flood & Coast Committee https://www.trafford.gov.uk/planning/planning-for-householders/docs/Validation-Checklist-NW-SuDS-Pro-forma-Guidance.pdf (Accessed 24/07/2024)

Local Council	SuDS Guidance
	And for the 1 in 100 year + Climate Change event must not be greater than either: • *The greenfield mean annual flood for the site, or;* • *2 litres per second per hectare (l/s/ha).* *Further detail is included in the SuDS guidance document*."[11]

1.2.12 The preliminary designs used in the following sections for the preliminary drainage systems for the HAGIs and BVIs comply with the LLFA requirements shown above.

[11] Sustainable Urban Drainage Systems (SuDS) Design and Technical Guidance (2017) Warrington Borough Council https://www.warrington.gov.uk/sites/default/files/2019-10/suds_guidance_approved_document_0.pdf (Accessed 24/07/2024)

2 Proposed Drainage Strategy - General Principles

2.1 Permanent Above Ground Infrastructure

2.1.1 The following specifications generally apply to all the permanent above ground components (HAGIs and BVIs). Any site-specific variations are considered in the following site-specific sections. Best practise guidelines and standards are drawn from a variety of sectors and areas within the industry. These will be specifically referenced where appropriate.

2.1.2 Detailed construction specifications for construction methods and materials to be used will be included on the construction drawings which will be produced during the detailed design stage of the project. These will be produced post DCO consent.

2.1.3 The typical general layout for all HAGIs and BVIs includes an impermeable access road around the perimeter of the works with a layer consisting of imported DoT Type 3 stone (as specified by Ministry of Transport, Specification for Highway Works, clause 805[12]) covering the site within the bounds of the access road.

2.1.4 The nature of the proposed site works for the HAGIs and BVIs provides an opportunity to utilise the natural voids in the Type 3 porous material as in-situ attenuation storage. The site layout plans for each permanent site included in Appendix A clearly illustrates this general arrangement.

2.1.5 At each site the depth of Type 3 stone will be specified to ensure sufficient storage is provided in each case.

2.1.6 Apart from the above ground pipework of varying sizes, the site development comprises pressure reducing units, metering and pigging facilities, instrumentation and control kiosks, electrical equipment and access roads. This will generally be a typical arrangement across all the HAGI sites.

2.1.7 Each of the HAGI or BVI sites are all substantially located in Flood Zone 1 areas and consequently none of the permanent sites require extra flood mitigation measures for flood risk associated with local watercourses. For more detail of the flood risk of each HAGI or BVI site see section 6.3 of the **FRA (Appendix 7F of Chapter 7: Water Environment**).

[12] Ministry for Transport (2021) Manual Of Contract Documents For Highway Works Volume 1 Specification For Highway Works (online) https://www.standardsforhighways.co.uk/tses/attachments/c7f58300-b868-43ae-819a-7b600e764000?inline=true (Accessed 16th September 2024)

2.1.8 Across each site the stone layer will be set slightly lower than the adjacent access roads and equipment foundation plinths and a filter drain system will collect runoff water from the porous stone layer in these areas. The flow will be discharged into the nearby watercourse via a flow control valve at a greenfield rate in accordance with national[13] and local policies. The only brownfield sites in the project are Stanlow HPP and Partington 1 sites. Details of the proposed drainage from these sites are considered in sections 3 and 10 of the report respectively.

2.1.9 To mobilise this storage, in areas with high groundwater levels (not viable to use infiltration) it will be necessary to wrap the entire type 3 stone layer in an impermeable membrane. This will prevent ingress of groundwater into the stone layer and preserve this storage volume for storage of surface water running off from the site during the storm event. The attenuated water within the stone layer will drain via a filter drain installed beneath the stone layer and connected to the outfall.

2.1.10 If infiltration proves a viable method of surface water disposal then the membrane around the stone layer attenuating the surface water runoff will be a permeable type, to allow infiltration to occur directly from the stone layer into the subsurface material.

2.1.11 For the purposes of this exercise the greenfield runoff rates have been calculated at each HAGI and BVI site using the HR Wallingford Greenfield Runoff Rate Estimation tool on their website. The rainfall model used in each case was the FEH 2013 data set.

2.1.12 Most of the sites are relatively small in area and therefore generate small greenfield runoff flows. When considering the practicality of the operation of a flow control valve to restrict the flow to these very low flows, in terms of the blockage potential caused by a small orifice size, we have assumed a minimum orifice diameter of 75mm. This equates to an approximate flow of 3 l/s flowing through the control valve at an assumed design head of 1m. Therefore, the practical minimum greenfield flowrate of 3 l/s has been used in the hydraulic calculations.

2.1.13 **Section 7.16** of the Draft ES **Chapter 7: Water Environment** details the future baseline for the Project. In terms of climate change the recommendation for peak rainfall intensity allowances is shown in Table 7.13 and is consistent with Environment Agency guidelines. This table is reproduced in **Table 7F6 2.1** below:

[13] Department for Environment, Food & Rural Affairs (2015) Sustainable drainage systems: non-statutory technical standards (online) Available at: https://www.gov.uk/government/publications/sustainable-drainage-systems-non-statutory-technical-standards (accessed 17th September 2024)

Table 7F6 2.1 Environment Agency peak rainfall intensity allowances (% increase relative to 1981 to 2000 baseline)

Allowance category	Total potential change anticipated for the '2050s' (to 2060)	Total potential change anticipated for the '2070s' (2061 to 2125)
Peak rainfall – Lower Mersey Management Catchment – 3.33% AEP*		
Upper	35%	40%
Central	20%	30%
Peak rainfall – Lower Mersey Management Catchment – 1% AEP		
Upper	40%	45%
Central	25%	30%
Peak rainfall – Upper Mersey Management Catchment – 3.33% AEP		
Upper	35%	40%
Central	20%	30%
Peak rainfall – Upper Mersey Management Catchment – 1% AEP		
Upper	40%	45%
Central	25%	30%
Peak rainfall – Weaver Gowy Management Catchment – 3.3% AEP		
Upper	35%	40%
Central	20%	25%
Peak rainfall – Weaver Gowy Management Catchment – 1% AEP		
Upper	40%	45%
Central	25%	30%

*AEP – Annual Exceedance Probability

2.1.14 EA guidelines also state that a development with a lifetime of between the years 2061 and 2100 (such as this Project) should use the central allowance for the 2070s epoch. However, some LLFA's require a climate change uplift higher than that reflected in the EA guidelines therefore to consider a single criterion across all LLFA's, and in view of the significant nature of this project, it is proposed that the higher upper end allowances are used when considering the effects of climate change on the peak rainfall intensities.

2.1.15 It can be seen from **Table 7F6 2.1** above that the recommended rainfall intensity climate change allowances for the 1% AEP (1:100 yr) storm event is thus 45% for all three management catchments. This is therefore the climate change upliftment which has been used in sizing the attenuation storages across all the HAGI's and BVI's.

2.1.16 It is noted that drainage from any area that may contain contamination by oils or similar will need to pass through an oil/water interceptor before discharge to the environment. At the time of writing this report we are not aware of any specific components planned for the sites that may discharge this effluent type.

2.1.17 Any components that may require separate bunding to isolate high pollution risk areas will be assessed during detailed design stage.

2.2 SuDS Based Surface Water Quality Control Measures

2.2.1 SuDS provide natural variability in their ability to remove contamination from surface water runoff which drains across a development site, therefore the management of water quality is founded on a risk-based approach. The SuDS Manual CIRIA C753 (Ref. 3) suggests a risk-based approach based on land use type and specific contaminants.

2.2.2 The SuDS Manual CIRIA C753, Table 26.1 suggests a Simple Index Approach (SIA) for low-risk developments, which follows a three-step process, namely:

- Allocate suitable pollution hazard indices for the proposed land use;
- Select SuDS with a total pollution mitigation index that equals or exceeds the pollution hazard index; and
- Where the discharge is protected surface waters or groundwater, consider the need for a more precautionary approach.

2.2.3 To successfully deliver adequate treatment, the proposed SuDS components should have a total pollution mitigation index that equals or exceeds the pollution hazard index.

- **Total SuDS mitigation index ≥ Pollution hazard index:**
  - Where the mitigation index of an individual component is insufficient, two components (or more) in series will be required, where:
- **Total SuDS mitigation index = mitigation index1 + 0.5 (mitigation index2):**
  - Where - mitigation Indexn = mitigation index for component n

2.2.4 A factor of 0.50 is used to account for the reduced performance of secondary or tertiary components associated with already reduced inflow concentration.

2.2.5 **Table 7F6 2.2** below shows the pollution hazard indices for different land use classifications. The land use category in row 3 with Total suspended solids (TSS) (0.5), Metals (0.4) and Hydrocarbons (0.4) have been used to represent usage of the access roads in the HAGIs.

Table 7F6 2.2 – Pollution Hazard Indices for Different Land Use Classifications.(Source: The SuDS Manual, CIRIA C753)

Land use	Pollution hazard level	Total suspended solids (TSS)	Metals	Hydro-carbons
Residential roofs	Very low	0.2	0.2	0.05
Other roofs (typically commercial/ industrial roofs)	Low	0.3	0.2 (up to 0.8 where there is potential for metals to leach from the roof)	0.05
Individual property driveways, residential car parks, low traffic roads (eg cul de sacs, homezones and general access roads) and non-residential car parking with infrequent change (eg schools, offices) ie < 300 traffic movements/day	Low	0.5	0.4	0.4
Commercial yard and delivery areas, non-residential car parking with frequent change (eg hospitals, retail), all roads except low traffic roads and trunk roads/motorways[1]	Medium	0.7	0.6	0.7
Sites with heavy pollution (eg haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways[1]	High	0.8[2]	0.8[2]	0.9[2]

2.2.6 In England and Wales, where the principal destination of the runoff is to surface water, but small amounts of infiltration may occur from unlined components, then the groundwater indices (**Table 7F6 2.3** below) should be used for the discharge to groundwater, and the surface water indices (**Table 7F6 2.4** below) should be used for the main surface water discharge.

Table 7F6 2.3 – Indicative SuDS Mitigation Indices for Discharges to Groundwater. (Source: The SuDS Manual, CIRIA C753)

Characteristics of the material overlying the proposed infiltration surface, through which the runoff percolates[1]	TSS	Metals	Hydrocarbons
A layer of dense vegetation underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.6[4]	0.5	0.6
A soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.4[4]	0.3	0.3
Infiltration trench (where a suitable depth of filtration material is included that provides treatment, ie graded gravel with sufficient smaller particles but not single size coarse aggregate such as 20 mm gravel) underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.4[4]	0.4	0.4
Constructed permeable pavement (where a suitable filtration layer is included that provides treatment, and including a geotextile at the base separating the foundation from the subgrade) underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.7	0.6	0.7
Bioretention underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.8[4]	0.8	0.8
Proprietary treatment systems[5, 6]	These must demonstrate that they can address each of the contaminant types to acceptable levels for inflow concentrations relevant to the contributing drainage area.		

Table 7F6 2.4 – Indicative SuDS Mitigation Indices for Discharges to Surface Water (Source: The SuDS Manual, CIRIA C753)

	Mitigation indices[1]		
Type of SuDS component	TSS	Metals	Hydrocarbons
Filter strip	0.4	0.4	0.5
Filter drain	0.4[2]	0.4	0.4
Swale	0.5	0.6	0.6
Bioretention system	0.8	0.8	0.8
Permeable pavement	0.7	0.6	0.7
Detention basin	0.5	0.5	0.6
Pond[4]	0.7[3]	0.7	0.5
Wetland	0.8[3]	0.8	0.8
Proprietary treatment systems[5,6]	These must demonstrate that they can address each of the contaminant types to acceptable levels for frequent events up to approximately the 1 in 1 year return period event, for inflow concentrations relevant to the contributing drainage area.		

2.2.7 The tables below show summaries of the proposed mitigation indices against the pollution hazard indices to be applied for a typical HAGI or BVI site draining to surface water **Table 7F6 2.5**) and to groundwater (**Table 7F6 2.6**) as follows:

Table 7F6 2.5 – Summary of Proposed Hazard and Mitigation Indices for Discharges to Surface Water.

Type of SuDS Component	TSS	Metals	Hydrocarbons
Proposed Pollution Hazard Indices			
Low Traffic general access roads	0.5	0.4	0.4
Proposed SuDS Mitigation Indices			
Porous Stone Blanket Layer (This component has been assumed to be equivalent to Permeable Paving)	0.7	0.6	0.7
Filter Drain	0.4	0.4	0.4
Total Mitigation Indices	1.1	1.0	1.1

Table 7F6 2.6 – Summary of Proposed Hazard and Mitigation Indices for Discharges to Groundwater

Type of SuDS Component	TSS	Metals	Hydrocarbons
Proposed Pollution Hazard Indices			
Low Traffic general access roads	0.5	0.4	0.4
Proposed SuDS Mitigation Indices			
Porous Stone Blanket Layer (This component has been assumed to be equivalent to Permeable Paving)	0.7	0.6	0.7

2.2.8 As can be seen from the above, the total SuDS mitigation indices for the typical site draining to surface water or groundwater is greater than the pollution hazard index, therefore meeting the requirements of the SIA as specified in SuDS CIRIA C753 (Ref. 3).

2.2.9 This method of surface water management allows for the water to be treated as close to the source as possible prior to being conveyed elsewhere and is in line with SuDS management good practice.

2.2.10 The proposed SuDS components to be incorporated in the project will be incorporated in the drainage designs which will require final approval by the LLFAs by way of fulfilment of requirements of the DCO to be discharged post consent.

2.3 Drainage of Temporary Construction Compounds and Pipe Laydown Areas

2.3.1 All the temporary construction areas and pipe laydown areas will require temporary drainage and associated attenuation for the duration of construction.

2.3.2 Within the construction compound areas there will be a variety of uses such as cabins, vehicle parking and storage. It should be noted that areas with particular pollution risks such as chemical and fuel storage areas will need to be separately bunded, drained and discharged via a closed system.

2.3.3 The allowable discharge rate from these areas will also be based on the calculated Qbar Greenfield runoff rate. This applies to all greenfield sites which includes all sites except Stanlow and Partington site 1 which are brownfield sites. See the site specific sections for specific runoff calculations for these brownfield temporary construction sites.

2.3.4 It is proposed these temporary construction areas will be surfaced with Type 1 MOT granular material. A 50% runoff coefficient for surface water runoff from these areas will be applied in the hydraulic drainage calculations.

2.3.5 The expected construction period for the entire project is approximately 2 years and therefore it has been agreed that no climate change uplift for these temporary areas will be applied. See para 2.6.3 of the FRA report.

2.3.6 It is proposed that the climate change uplift of those compounds that may be constructed later, up to 2031, will be reassessed during detailed design stage (closer to start of construction) to ensure the upliftment is aligned with the current recommendations at that time. However, if the construction period is over a short period (2 years) no upliftment will apply during the construction period.

2.3.7 The principle of the drainage for these areas will be to grade the surfaces towards strategically positioned filter drain trenches, which will collect the surface water and discharge to a watercourse via suitable attenuation storage and a flow control device.

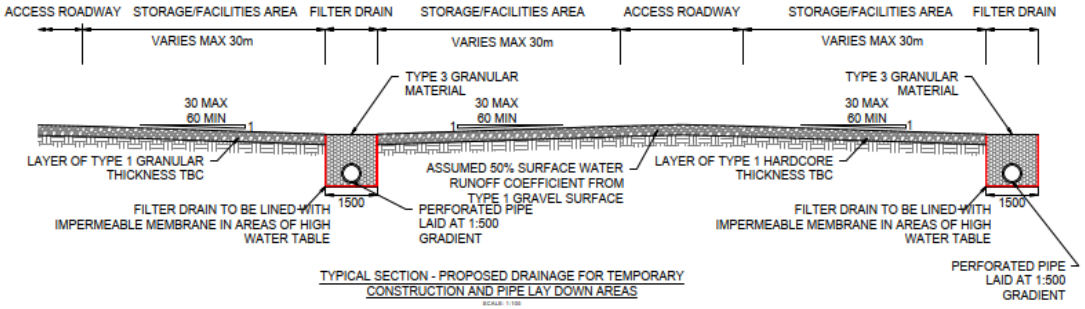
2.3.8 During the detailed design stage (post DCO), detailed design information will be produced for the drainage of these temporary areas including specification of methods and materials to be used to deliver the drainage for these areas. These will comprise “design packs” to be approved by the statutory authorities before construction of these areas begins.

2.3.9 Where possible the temporary drainage will discharge via the same outfall identified for the permanent drainage of the HAGI. This will economise on the number of outlets required across the Project. Instances where this is possible will be highlighted on a case by case basis in the following site specific sections.

2.3.10 Appendix D includes a typical detail drawing which shows a typical section through the proposed drainage as described above. The filter drains are shown positioned in the valleys of the Site and access roadways should generally be located on the crown of the graded area with storage and facilities zone between these.

2.3.11 An excerpt from the drawing showing the typical section through temporary area is shown in **Figure 7F6 2.1** below:

Figure 7F6 2.1 Section Through Temporary Construction Area



2.3.12 This arrangement should help minimise ponding of surface water on the temporary platform areas which will facilitate better, more durable accessways and storage areas.

2.3.13 Attenuation storage will be provided by temporary basins or swales positioned immediately adjacent to the temporary area.

2.3.14 Using filter trenches to collect the surface water from the gravel hardstanding area affords a degree of improvement to the water quality of the runoff and will also remove any silt from the outflowing surface water from the area.

2.3.15 Any additional dewatering that may be required during construction will need the appropriate pollution control measures to remove silt prior to discharging to the environment.

2.3.16 Spatial planning of the components occupying the construction compound area such as the cabins etc should be located away from the main overland flow routes, which should discharge unobstructed towards the filter drains.

2.3.17 The use of a swale around the boundary of the areas will provide significant in system water quality benefits in accordance with best SuDS practice, as well as providing additional attenuation storage.

2.4 Foul Drainage

Operational Sites

2.4.1 Most of the sites are remote from adopted drainage assets and so the foul drainage system will need to be designed and provided as appropriate to the individual site specific constraints.

2.4.2 It is assumed that the occupancy of these sites will be intermittent and hold a peak of five personnel at any one time.

2.4.3 The likely variation in flow is problematic, and while a package treatment option would provide the highest level of treatment, the flow regime may not be suitable for these systems.

2.4.4 At this stage it is assumed that a sealed cesspit solution may be utilised. To be compliant with Building Regulations Part H (H2 - 1.61) the minimum volume would be 38.4m^3. However, an alternative solution may be to supply temporary welfare facilities just for the duration of any work operations to be carried out on site.

2.4.5 With the progression of the more detailed design stages, more detailed information on the expected usage of sanitation facilities will become available, enabling identification and design of the appropriate solutions to accommodate the expected foul flow regime.

Temporary Construction Sites

2.4.6 Welfare facilities will be provided at the main construction compounds along the route where construction staff spend extended periods through the day.

2.4.7 The type of ablution facilities will most probably consist of sealed chemical based units, probably serviced by an external contractor.

2.4.8 These will require ongoing monitoring and servicing throughout the construction contract period.

2.4.9 A complete delivery, operation and maintenance plan for these facilities will be compiled and proposed by the construction contractor post DCO, as part of the detailed construction plans for delivery of the asset.

2.5 Existing Utilities

2.5.1 In each of the following site specific sections mention of existing utilities in the area of each site is made. This information has been drawn from statutory utilities search results which have been supplied by the utility providers and captured in the project GIS database.

2.5.2 No further on site investigations have been done to ascertain utility service layouts therefore the positions of these utilities as shown on the drawings are approximate at this stage and will require confirmation and more detailed location information in conjunction with the utility supplier companies during the detailed design stages of each site. This will be secured as part of the DCO requirements.

3 Stanlow Hydrogen Production Plant (HPP) HAGI Site

3.1.1 This section of the report sets out the outline drainage strategy for the proposed Stanlow HPP HAGI site, which is located at the western end of the western corridor of the hydrogen distribution network. This location can be seen in **Figure 7F6 1.1**

3.2 Existing Site

3.2.1 The proposed HAGI site is situated on the southern extremity of the existing Stanlow Refinery in Ellesmere Port industrial area. The proposed hydrogen production plant (which does not form part of this project) will be constructed in this industrial area adjacent to the proposed HAGI. The surrounding area to the north of the site is therefore all brownfield industrial area with the A5117 (NCR 5) main road bordering the southern boundary of the site. The area to the south of the A5117 is agricultural land.

3.2.2 In discussions with the existing site operators (EET) it was confirmed that there is an extensive drainage system in operation on this existing brownfield site.

3.2.3 The approximate site grid reference is 344752E 374967N.

3.2.4 The existing topography of the site has been approximated from Lidar Contour data and this shows that the southwestern corner is at approximately 8mAOD elevation sloping to approximately 12mAOD in the northeastern corner. This is a fall of approximately 4m in 345m which equates to an average slope of approximately 1:86 across the site.

3.2.5 Ground investigations are in progress however the British Geological Survey website indicates that the site is underlain by the Chester Formation, which consists of sandstone, pebbles and gravel. Sedimentary bedrock formed during the Triassic period. Superficial deposits of Diamicton glacial till exist across the HAGI site. The dominant soils in the area are described as slowly permeable, seasonally waterlogged reddish fine loamy and clayey soils.

3.2.6 This site is presently an industrial site and therefore confirmation of any contamination as a result of these activities will need to be determined with future onsite intrusive testing. Draft ES **Chapter 12: Ground Conditions** details the ground condition investigations that have been undertaken to date.

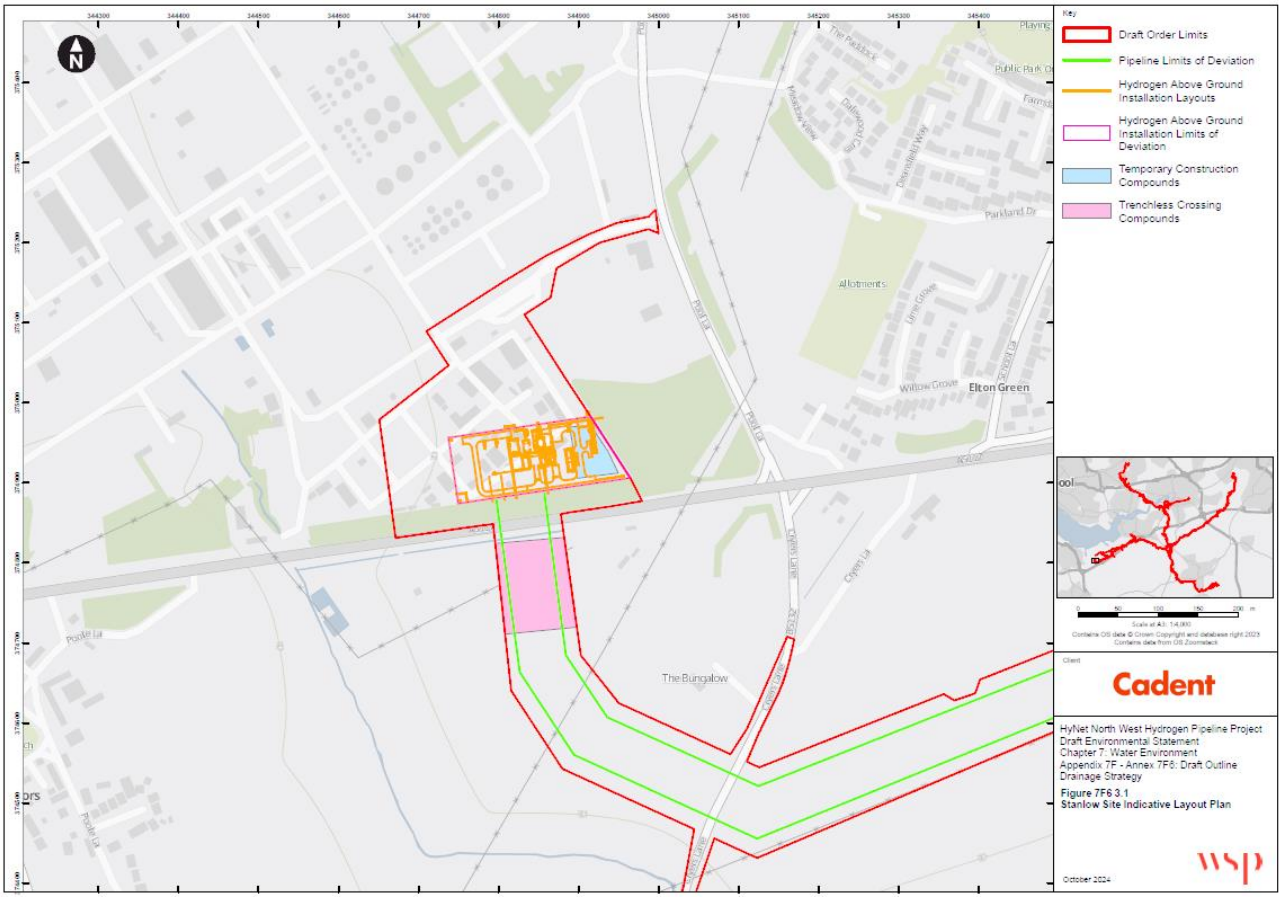
3.2.7 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 2m below ground level. This indicates that the poor permeability of the soils and probability of high-water table would probably mean that infiltration, as a surface water disposal method would not be viable.

3.3 Proposed Permanent Development

3.3.1 The proposed site is shown in **Figure 7F6 3.1** below. The Limit of Deviation (LoD) for the actual hydrogen pipeline is represented by the green lines and the red line represents the Draft Order Limits. The light blue area indicated is the proposed temporary construction compound area.

3.3.2 The proposed HAGI covers a total area of approximately 16,900m^2 (1.69 hectares).

Figure 7F6 3.1 Stanlow HPP HAGI Site Layout Plan



3.3.3 The Stanlow HPP HAGI Indicative Drainage Layout Plan is included in Appendix A1. As the site is brownfield and has access to an existing drainage network it is assumed that the proposed HAGI drainage will discharge to the existing network.

3.3.4 In discussion with the existing site operator, it was confirmed that the proposed drainage will connect into the existing network and an approximate position was proposed along the northern access road. The exact location of the connection point is to be determined prior to detailed design.

3.4 Allowable Site Discharge

3.4.1 The existing 'brownfield' runoff rate has been estimated using the BS EN 752 calculation method with an assumed rainfall intensity of 50mm/hr.

3.4.2 The impermeable area of the original site was measured at approximately 1.69 ha. The runoff from this area would therefore be approximately 2.781 x 1.69 ha x 50mm/h = 235l/s**.**

3.4.3 The Cheshire West and Chester (CW&C) LLFA require that new development on a brownfield site should discharge surface water runoff at a rate which is at least 30% less than the current brownfield runoff rate.

3.4.4 In accordance with this requirement the discharge rate of surface water from the new proposed site would therefore need to be less than 235l/s x 70% = 164.5 l/s.

3.4.5 It is proposed that a gravity discharge from the site will be restricted to a maximum of 164.5l/s and connected into the existing drainage network at a point to be determined in the northern access road adjacent to the site boundary as agreed with the current site operator (EET Hydrogen).

3.4.6 An unrestricted flow through a 225mm diameter pipe from the HAGI site northwestern corner to the connection point into the existing drainage network would yield a flow rate of approximately 58 l/s. This is in accordance with the above flow restriction requirement and therefore shows that a flow control valve is not required to restrict this flow. The normal flow through a 225mm diameter pipe will be less than the maximum allowed.

3.5 Provision of Attenuation Storage

3.5.1 It is proposed that the site platform will have an approximate surface level of 10.82mAOD and will contain a granular type 3 stone blanket layer across the site. The stone blanket is approximately 7645m^2 in area and will have a minimum depth of approximately 300mm which will yield a total potential available attenuation storage volume of approximately 690m^3 assuming a void ratio of 30% in the granular material.

3.5.2 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) show that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed restricted flowrate of 58l/s, to prevent any flooding from the site, the volume of attenuation storage required is 630m^3. These calculations are included in Appendix C1.

3.5.3 In order to mobilise this storage, it will be necessary to place an impermeable membrane at the base of the granular material. This will prevent any groundwater ingress into the granular layer. The attenuated water will be collected via a filter drain positioned along the edge of the stone blanket as shown in the indicative Drainage Layout drawing in Appendix A1.

3.6 Temporary Construction Compound Drainage

3.6.1 The site layout plan identifies a 0.26ha temporary construction compound demarcated on the east of the HAGI site. This area is however included in the total measured area of the HAGI, which will also require temporary drainage during construction. Therefore, the total catchment area of the construction compound and HAGI site during construction will also be 1.69ha.

3.6.2 The present surfacing of this area is hardstanding. This should be suitable to support the temporary construction compound activities to be carried out on the area. Phasing of demolition of the hardstanding will be dictated by the detailed construction programme once finalised.

Allowable Site Discharge

3.6.3 As referred to above, the allowable discharge from the total site (permanent HAGI and temporary construction camp) is estimated at 164.5 l/s (30% betterment on the present brownfield runoff rate).

3.6.4 Based on the preliminary hydraulic calculations for the temporary drainage a full-bore flow through a 225mm diameter pipe would discharge at a rate of 99.5 l/s. This is acceptable as it is less than the allowable 164.5 l/s and represents a 58% reduction from the present brownfield runoff rate.

3.6.5 It is proposed that the outfall from this temporary system will be to the same proposed drainage outlet for the permanent drainage as shown in the indicative drainage layout in Appendix A1.

Attenuation Provision

3.6.6 Based on the discharge flowrate noted above and the catchment area of the HAGI plus temporary compound, the volume of attenuation required to prevent flooding during the 1 in 100-year (1%AEP) event is estimated to be 185m^3.

3.6.7 It is proposed that this storage volume is provided in a temporary open basin created after demolition of the hardstanding covering the area of the permanent HAGI. The basin should be lined with an impermeable membrane to prevent ingress of groundwater into the subsurface layers and possible transportation of any potential pollution hazard in the existing site. The approximate dimensions of the pond are shown on the indicative drainage layout in Appendix A1.

3.7 Existing Utilities in the Area

3.7.1 Due to the Stanlow site being located on an existing industrial zone there will be extensive existing utilities on the site, which will need to be made safe and removed from the proposed HAGI site by a separate demolition contractor. No hazards from potential utilities are assumed on the HAGI site at this time.

3.7.2 There are a multitude of utilities running in the A5117 highway corridor which all cross the hydrogen pipeline LoD which will need to be documented and avoided by the trenchless crossing operation in this area.

3.7.3 The utilities zone in the A5117 corridor is shown on the layout plan in Appendix A1.

4 Rocksavage HAGI Site

4.1.1 This section of the report sets out the outline drainage strategy for the proposed Rocksavage HAGI site, which is located midway along the western corridor between the Central Hub HAGI and Stanlow HPP HAGI. This location can be seen in **Figure 7F6 1.1**

4.2 Existing Site

4.2.1 The existing site is a greenfield site situated in existing wooded land adjacent to the M56 between the River Weaver to the southwest and the Weaver Navigation canal to the northeast. The Frodsham Electrical Substation is located due west of the site on the opposite side of the M56 motorway and the old East Clifton Marsh historical industrial landfill site is located approximately 150m to the southeast of the site.

4.2.2 The approximate site grid reference is 352827E 379240N.

4.2.3 The topography of the site has been approximated from Lidar Contour data. This shows an elevated area of approximately 17.5m AOD on the northern edge of the permanent HAGI site sloping down to a level of approximately 13.5mAOD in the southern corner and 15.5m AOD in the western corner. It is proposed that the final level of the HAGI may be approximately 14.5mAOD, however this will be finalised during the detailed design stage.

4.2.4 There are two existing outfalls shown on the GIS discharging from this area westwards into the River Weaver.

4.2.5 There is also the Halton Borough Council lagoon shown approximately 100m to the southwest of the site.

4.2.6 In view of the above historical industrial nature of the site it is proposed that a detailed contamination investigation is undertaken at detailed design stage to ascertain the full impact of contamination levels in the area.

4.2.7 Based on information from Draft ES **Chapter 12: Ground Conditions** superficial alluvium deposits of clay, silt and sand (Tidal Flat Deposits) exist in the HAGI area. These deposits are expected to consist of weathered marl comprising firm red brown gravelly clay to a depth of c. 3.0m bgl and firm to soft sandy silty clay, silty gravelly sand and loose sandy gravel with bands of peaty clay to a depth of 17.1m bgl.

4.2.8 The bedrock underlying the area consists of mudstone (Bollin Mudstone) comprising of fractured sandstones, mudstones and siltstones with occasional calcite and possible nodules to a proven depth of at least 29m bgl.

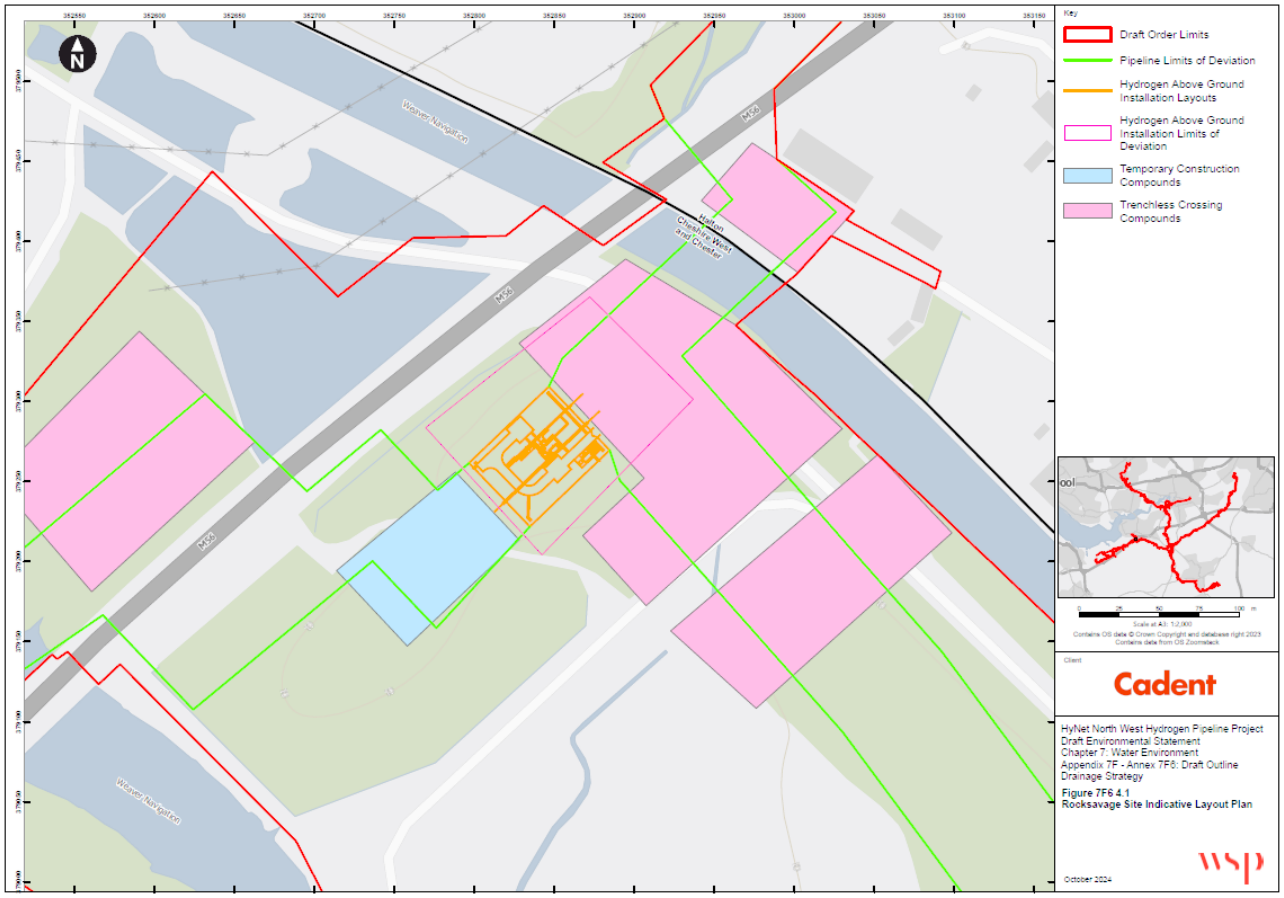
4.2.9 Groundwater levels were recorded at 5.4m bgl at the HAGI site.

4.3 Proposed Permanent Development

4.3.1 The proposed site layout is shown in **Figure 7F6 4.1** below. The limit of deviation (LoD) is shown in green and indicates the location of the pipeline route. The pink areas shown are construction compound sites for the trenchless crossing operations under the adjacent canal and river.

4.3.2 The proposed HAGI site covers a total area of approximately 3,950m^2 (0.4 hectares).

Figure 7F6 4.1 Rocksavage HAGI Site Plan



4.3.3 The Rocksavage HAGI Indicative Drainage Layout Plan is included in Appendix A2.

4.3.4 In the absence of confirmed infiltration test results for the site the initial proposal is to discharge surface water to the River Weaver approximately 260m southwest of the site.

4.3.5 This section of the River Weaver is an Open Inland River. Consent for discharge into the River will be required from the LLFA.

4.4 Attenuation Provision

4.4.1 The overall area of the development site is 4,000m^2. However, some of this area will be occupied by impermeable areas such as access roads and foundation plinths. The approximate impermeable area on the site is 2,330m^2. An estimated 1,670m^2 will be available as in-situ attenuation storage on the site.

4.4.2 It is proposed that the site platform will have an approximate surface level of 14.5mAOD and will contain a granular blanket across the site. The granular blanket will have a minimum depth of approximately 500mm that will yield a total potential available attenuation storage volume of approximately 250m^3 assuming a void ratio of 30% in the granular material.

4.4.3 A gravity outfall will discharge via a new headwall to the brook which is located approximately 130m to the southeast of the site. The brook is known as Flood Brook and discharges southwards into the River Weaver. This will require the consent of the Lead Local Flood Authority and will be subject to an ordinary watercourse discharge consent.

4.4.4 As the flow through the granular sub-base will receive a high level of percolation, it is proposed that flow rates from the site are controlled via a flow control valve in a chamber upstream of the connection to the offsite outfall pipe.

4.4.5 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) show that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed greenfield runoff rate, and in order to prevent any flooding from the site, the volume of attenuation storage required is 240m^3. These calculations are included in Appendix C2.

4.4.6 To mobilise this storage, it will be necessary to place an impermeable membrane at the base of the granular material. This will prevent water percolating vertically through to the underlying soil and horizontally through the embankments. The membrane will also prevent any high-level ground water seeping into the granular layer. The attenuated water will be collected via a perforated collection pipe measuring a minimum of 10m in length and 150mm in diameter on the southern side of the platform.

4.4.7 An Indicative Drainage Layout plan is included in Appendix A2.

4.5 Temporary Construction Compound Drainage

4.5.1 The site layout plan identifies a 0.5ha temporary construction compound to the south of the HAGI site.

4.5.2 In addition, there are three proposed trenchless crossing construction compound areas lying to the east and west of the site. These are 1.20ha, 2.00ha and 1.20ha in size respectively.

4.5.3 The western trenchless crossing site will drain west into the River Weaver via one outfall with the HAGI and temporary HAGI construction site compound drainage draining southward to Flood Brook. The eastern sites will all drain eastwards into the Weaver Navigation Canal.

4.5.4 All the above temporary areas (including the HAGI site itself) will require temporary drainage and associated attenuation for the duration of construction.

Allowable site discharge

4.5.5 The total site area of the HAGI plus HAGI construction compound is 0.9ha which yields an equivalent greenfield runoff rate (Qbar) of 1.71/s (based on a unit greenfield runoff rate of 1.9l/s/ha). This flowrate is less than the minimum 3 l/s, therefore a restricted discharge rate of 3 l/s will be used for sizing the required attenuation storage volume.

4.5.6 The total site area of the western trenchless construction compound is 1.2ha which yields an equivalent greenfield runoff rate (Qbar) of 2.28l/s (based on a unit greenfield runoff rate of 1.9l/s/ha). This flowrate is less than the minimum 3 l/s, therefore a restricted discharge rate of 3 l/s will be used for sizing the required attenuation storage volume.

4.5.7 Appendix B shows the unit Greenfield Runoff Rate calculation sheet.

4.5.8 For the above two sites it is proposed that a gravity discharge from the sites will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar or a minimum of 3 l/s.

4.5.9 Similarly, the greenfield runoff rate for the easterly draining areas is estimated at 6.1l/s.

4.5.10 The indicative temporary drainage layout plan is included in Appendix A2.

Attenuation Provision

4.5.11 For the 1 in 100-year (1%AEP) rainfall event assuming the discharge rates as above the attenuation storage volume required for the HAGI and temporary HAGI construction compound is 220m^3 and that of the western trenchless construction compound is 320m^3.

4.5.12 The attenuation storage volume for the two easterly draining trenchless construction compounds which will discharge at a single point into the Weaver Navigation Canal is estimated at 900m^3.

4.5.13 These storage volumes will be provided by detention basins as shown on the indicative layout plan in Appendix A2.

4.6 Existing Utilities in the Area

4.6.1 There is an ESSAR fuel pipeline traversing under the eastern section of the HAGI site as shown on the indicative drainage layout in Appendix A2. On the same alignment as this pipe there appear to be a number of abandoned pipelines.

4.6.2 To the southeast of the site is a number of United Utilities 2 No. 72 inch and 2No. 60-inch diameter approx. 60m from the HAGI site and below the proposed trenchless crossing compound.

4.6.3 There are a number of utility services shown within the road boundary to the east of the site including underground Openreach telecommunication cables and high voltage power cables.

4.6.4 To the north west of the site on the other side of the M56 motorway there are two 11kV overhead power lines. These are however not within the construction area of the HAGI site.

4.6.5 The above utilities are shown on the layout plan in Appendix A2.

5 Runcorn HAGI Site

5.1.1 This section of the report sets out the outline drainage strategy for the proposed Runcorn HAGI site, which is located on a northern spur, midway along the western leg of the network. This location can be seen in **Figure 7F6 1.1**.

5.2 Existing Site

5.2.1 The existing site is greenfield and is located immediately adjacent to the east end of the Integen Rocksavage power station. An existing above ground gas pipe installation is immediately south of the proposed HAGI site. Open agricultural land extends to the north of the site.

5.2.2 The approximate site grid reference is 352248E 380070N.

5.2.3 The topography of the site has been approximated from Lidar Contour data and this shows that the site has a steep gradient sloping from the northern boundary at a level of approximately 45 mAOD down to the southern boundary at a level of approximately 30 mAOD.

5.2.4 The HAGI site level of approximately 32 mAOD is proposed, this will result in substantial earthworks in order to form the HAGI platform. An appreciable cut embankment will be created around the northern half of the site with a surface water cut-off berm positioned along the top of the embankment to divert overland runoff flow around the HAGI site.

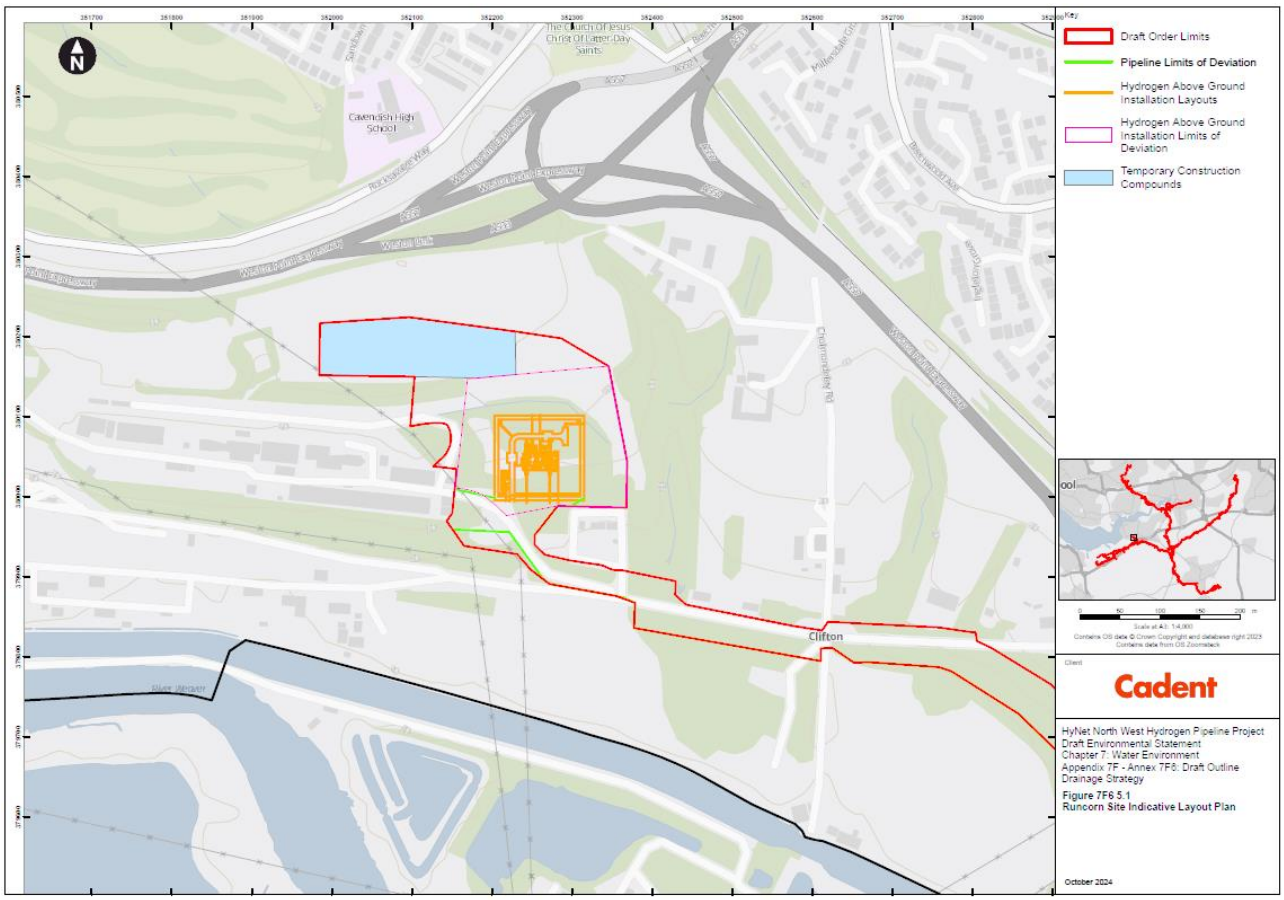
5.2.5 According to the British Geological Survey website data the site is underlain by the Bollin Mudstone Member - Mudstone with superficial deposits of Tidal Flat Deposits - clay, silt and sand. These superficial deposits may have relatively good permeability.

5.2.6 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 1.2m to 5m below ground level.

5.3 Proposed Permanent Development

5.3.1 The proposed site is shown in **Figure 7F6 5.1** below. The proposed HAGI site covers a total area of approximately 12,000m^2 (1.20 hectares).

Figure 7F6 5.1 Runcorn HAGI Site Layout Plan



5.4 Allowable Site Discharge

5.4.1 The total site area of 1.20ha yields an equivalent greenfield runoff rate (Qbar) of 4.36 l/s using the FEH method (3.80 l/s/ha). Appendix B3 shows the Greenfield Runoff Rate calculation sheet.

5.4.2 The Runcorn HAGI Indicative Drainage Layout Plan is included in Appendix A3. Should infiltration not be viable on this site, it is proposed that the drainage discharge is connected into the existing above ground gas pipe installation drainage system. Further investigation will be required to determine the viability of connecting into this drainage system. It is being considered whether this will be included in the DCO requirements.

5.4.3 It is proposed that gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar. If discharge under gravity is not possible then a pumped solution pumping at the agreed restricted rate will be required.

5.5 Attenuation Provision

5.5.1 The overall area of the development site is 12,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 5,940m^2, therefore an estimated 6,060m2 will be available as in-situ attenuation storage on the site.

5.5.2 It is proposed that the site platform will have an approximate surface level of 32.0mAOD and will contain a granular blanket across the site. The granular blanket will have a minimum depth of approximately 500mm that will yield a total potential available attenuation storage volume of around 900m^3 assuming a void ratio of 30% in the granular material.

5.5.3 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) shows that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed greenfield runoff rate, in order to prevent any flooding from the site, the volume of attenuation storage required is 870m^3. These calculations are included in Appendix C3.

5.5.4 The preliminary design indicates that with the available area of stone blanket on this site, in order to provide this volume of storage, approximately 500mm depth of type 3 stone will be required.

5.5.5 In order to mobilise this storage, it will be necessary to place an impermeable membrane at the base of the granular material. This will prevent water percolating vertically through to the underlying soil and horizontally through the embankments. The membrane will also prevent any high-level ground water seeping into the granular layer. The attenuated water will drain from the stone blanket area via a filter drain installed along the eastern side of the platform and connected to the outfall.

5.5.6 The preliminary hydraulic calculations for the attenuation storage volume sizing is included in Appendix C3 and an Indicative Drainage Layout is included in Appendix A3.

5.6 Temporary Construction Compound Drainage

5.6.1 The site layout plan in **Figure 7F6 5.1** shows the HAGI temporary construction compound to the north of the HAGI site approximately 1.6ha in area. This compound will require temporary drainage and associated attenuation for the duration of construction.

5.6.2 The total areas requiring drainage include the temporary construction compound and the permanent HAGI site while it is under construction. This area in total is approximately 1.60 + 1.20 = 2.80ha.

Allowable Site Discharge

5.6.3 Based on the calculated Qbar rate of 3.14 l/s/ha noted in Section 5.4, the allowed discharge rates from the temporary construction site will be 5 l/s from the northern site and 3.8 l/s from the HAGI site itself.

5.6.4 It is therefore proposed that the discharge rates for the temporary construction compound will be 5l/s from the northern construction compound area and 3.8l/s from the HAGI site during construction.

5.6.5 The temporary construction compound will discharge restricted flows into the proposed attenuation for the HAGI. The combined flows will then be discharged at a combined rate of 8.8l/s.

5.6.6 Appendix A3 shows an indicative drainage layout plan for these temporary works.

Attenuation Provision

5.6.7 Based on the allowable discharges noted above and the total areas of the compounds, the volume of attenuation storage required for the northern temporary compound is estimated to be 400m^3 for the 1 in 100-year (1%AEP) event, and the volume of attenuation storage required for the HAGI compound is estimated to be 300m^3. These details will be finalised in the next more detailed stage of design. It is being considered whether this will form a DCO requirement.

5.6.8 The approximate area of attenuation storage ponds, based on a depth of 1.7m is shown on the indicative temporary drainage layout in Appendix A3.

5.7 Existing Utilities in the Area

5.7.1 There is a United Utilities 225mm diameter foul water sewer encroaching across the proposed HAGI site. This existing sewer will require diversion/relocating in consultation with United Utilities. The recorded present alignment of this sewer is shown on the Drainage Layout Plan in Appendix A3.

5.7.2 An 11kV overhead powerline also spans east-west across the northern half of the site. A 132kV overhead electrical line runs immediately past the southwest corner of the site in a northwest-southeast direction.

5.7.3 There is a Sabic UK Petrochemicals Northwest pipeline running past the northern boundary of the site.

5.7.4 There is also a National Grid Gas Transmission high pressure main to the south of the proposed HAGI site which connects into the existing above ground installation immediately south of the site.

5.7.5 These utilities are shown on the layout plan in Appendix A3.

6 Central Hub HAGI Site

6.1.1 This section of the report sets out the Outline Drainage Strategy for the proposed Central Hub HAGI site, which is located in the centre of the distribution network as shown in **Figure 7F6 6.1**.

6.2 Existing Site

6.2.1 The existing site is greenfield surrounded by existing agricultural land. There is a small hardstanding immediately adjacent to the western corner of the HAGI site.

6.2.2 The approximate site grid reference is 360106E 378096N.

6.2.3 The topography of the site has been approximated from Lidar Contour data and this shows that the northern corner is at approximately 49mAOD falling to approximately 45mAOD in the lower southern corner. This is a fall of approximately 4m in 167m which equates to an average slope of approximately 1:42 across the site.

6.2.4 Based on the information from the British Geological Survey website, superficial deposits of Diamicton Glacial Till exist across the Central Hub area. Sand and gravel (Glaciofluvial sheet deposits) are located predominantly in the southwest of the HAGI area. However superficial deposits were absent at the Central HAGI site. The bedrock underlying the area consists of mudstone (Bollin Mudstone Member) across the entire area.

6.2.5 Records from the National Soil Map shows the dominant soils profile in this area is slowly permeable seasonally waterlogged reddish fine loamy over clayey soils. These soils are considered to have poor permeability with respect to infiltration.

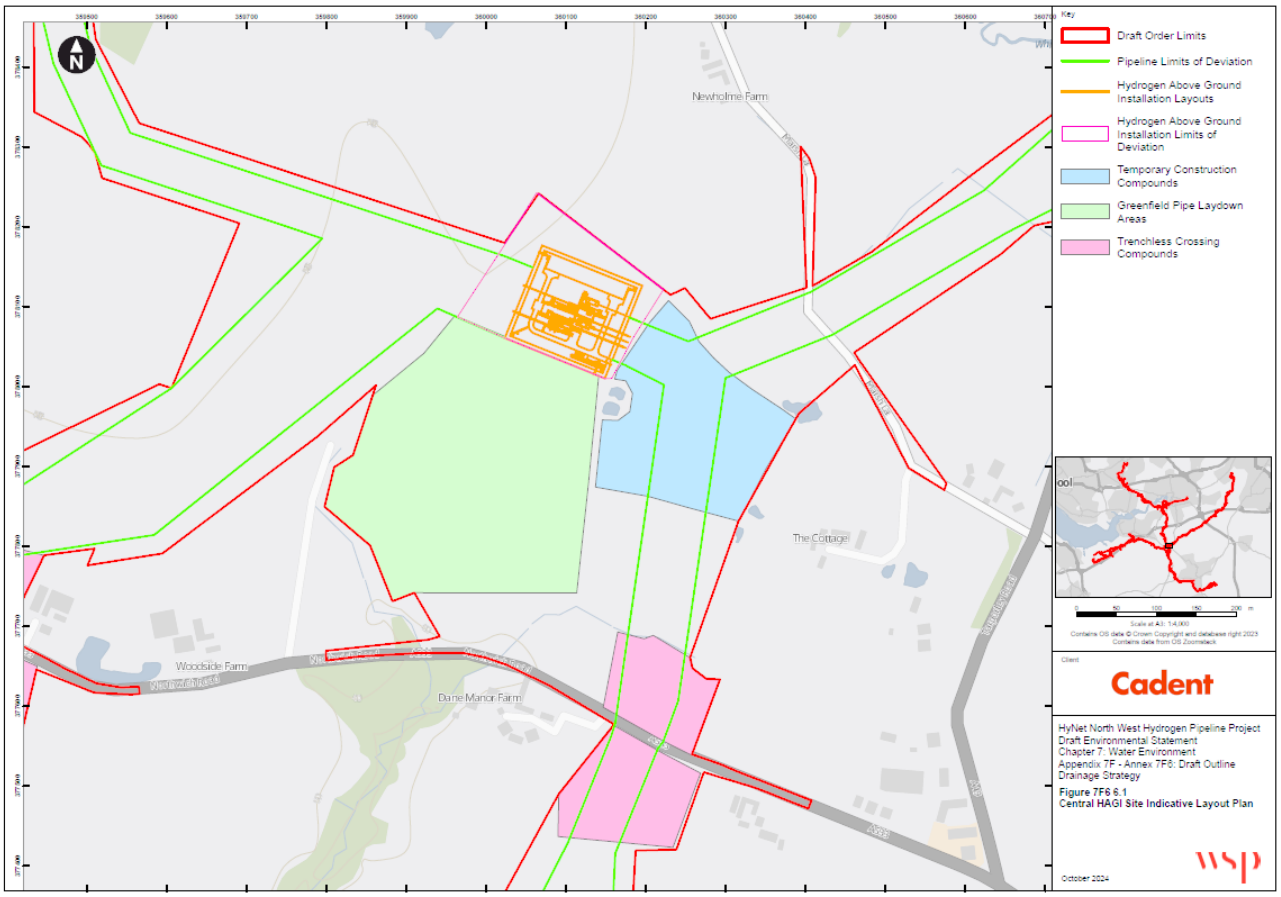
6.2.6 Based on interpretation of exploratory hole data from historical ground investigations in the surrounding areas it is anticipated that groundwater levels may be encountered between 5 and 8m bgl.

6.3 Proposed Permanent Development

6.3.1 The proposed site development is shown in **Figure 7F6 6.1** below. The blue area indicates the proposed temporary construction compound area for the HAGI and the green area indicates a proposed pipe laydown area.

6.3.2 The proposed Hydrogen Above Ground Installation (HAGI) site covers a total area of approximately 16,000m^2 (1.60Ha).

Figure 7F6 6.1 Central Hub HAGI Site Layout Plan



6.3.3 The Central Hub HAGI Indicative Drainage Layout Plan for the permanent drainage of the site is included in Appendix A4. This also shows the location of a land drainage ditch approximately 300m to the east of the site. If infiltration is proven not to be viable on this site, drainage will probably be discharged to this watercourse.

6.3.4 An ordinary watercourse consent will need to be agreed with the LLFA for this outfall discharge connection.

6.4 Allowable Site Discharge

6.4.1 The total site area of 1.6ha yields an equivalent greenfield runoff rate (Qbar) of 7.80l/s. The unit greenfield runoff rate for this area is calculated at 4.85l/s/ha.

6.4.2 The Greenfield Runoff Rate as calculated using the HR Wallingford greenfield runoff tool is shown in Appendix B4.

6.4.3 Surface water runoff from the proposed development will be restricted to this equivalent greenfield runoff rate.

6.4.4 It is proposed that a gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar. If discharge under gravity is not possible then a pumped solution pumping at the agreed restricted rate will be required. This will be finalised at the next, more detailed stage of

design. It is being considered whether this will be included in the DCO requirements.

6.5 Attenuation Provision

6.5.1 The overall area of the development site is 16,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 7,075m^2. An estimated 8,925m^2 will be available as in-situ attenuation storage on the site.

6.5.2 It is proposed that the site platform will have an approximate surface level of 47.36mAOD. The Type 3 stone blanket will have a minimum depth of around 450mm that will yield a total potential available attenuation storage volume of approximately 1200m3 assuming a void ratio of 30% in the granular material.

6.5.3 A positive outfall will discharge via a new headwall to the unnamed drainage ditch to the east of the site. This will require an ordinary watercourse discharge consent from the Lead Local Flood Authority (LLFA).

6.5.4 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) show that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed greenfield runoff rate, to prevent any flooding from the site, the volume of attenuation storage required is 1,124m^3. These calculations are included in Appendix C4.

6.5.5 To mobilise this storage, it will be necessary to place an impermeable membrane at the base of the granular material. This will prevent water percolating vertically through to the underlying soil and horizontally through the embankments. The membrane will also prevent any high-level ground water seeping into the granular layer. The attenuated water will drain from the stone blanket area via a filter drain installed along the northern side of the platform and connected to the outfall.

6.5.6 The preliminary Indicative Drainage Layout is included in Appendix A4.

6.6 Temporary Construction Compound Drainage

6.6.1 The site layout plan shows a 3.9ha temporary construction compound to the east of the HAGI site and a proposed pipe laydown area immediately south of the site. The area of this laydown area is proposed as 9.07ha.

6.6.2 All the above temporary areas will require temporary drainage and associated attenuation for the duration of construction.

6.6.3 For the purposes of this exercise, it was assumed that:

- The HAGI site plus the northern half of the HAGI construction compound site drained eastwards to the drainage ditch to the east of the site (total area of 3.45ha).
- The southern half of the HAGI construction compound drained southwards and discharged to an ordinary watercourse to the south (area of 1.78ha).
- The temporary pipe laydown area drained southwards to the ordinary watercourse in this area (area of 9.07ha).

Allowable Site Discharge

6.6.4 Based on the calculated Qbar rate noted in the previous section, the allowable discharge rates for all the above areas will be as follows:

- 16.1l/s to be discharged from the combined HAGI permanent and half of the construction site to be discharged to the drainage ditch to the east of the sites.
- 8.3l/s to be discharged from the southern half of the HAGI construction compound to be drained to the watercourse in the south.
- 42.4/s to be discharged from the temporary pipe laydown area to be drained to the watercourse in the south.

6.6.5 The above proposed outfalls are shown in Appendix A4. These outlets will all be new discharge headwalls to the ordinary watercourses installed in accordance with the LLFA consent requirements.

Attenuation Provision

6.6.6 Based on the allowable discharge rates noted above and the corresponding areas of the compounds draining to the three separate basins, the individual attenuation volumes required for the temporary compounds are estimated as follows:

- 820m^3 storage for the combined HAGI permanent and half the construction site.
- 425m^3 storage for half the HAGI construction compound draining southwards.
- 2,150m^3 storage for the pipe laydown area draining southwards.

6.6.7 It is noted that as these compounds are temporary, they will be removed after construction. It is being considered whether the rehabilitation of the site after removal will be the subject of a DCO requirement.

6.6.8 The approximate locations and sizes of detention basins are based on a 1.7m deep basin as shown on the indicative drainage layout in Appendix A4.

6.7 Existing Utilities in the Area

6.7.1 There is a SABIC UK Petrochemicals underground pipeline running across the northern corner of the proposed HAGI site from west to east approximately 10m from the actual HAGI structure.

6.7.2 There are several high voltage 11kV overhead electrical lines traversing the agricultural field to the south, crossing over the proposed temporary construction compound and pipe laydown area.

6.7.3 These existing utilities are shown on the layout plan in Appendix A4.

7 Higher Walton HAGI Sites

7.1.1 This section of the report sets out the Outline Drainage Strategy for two alternative proposed sites for the Higher Walton HAGI site which is located approximately mid-way along the northern corridor of the distribution network. This is approximately 3 km south west of Warrington. **Figure 7F6 1.1** shows the full extents of the Hynet North West Hydrogen Pipeline Project with Higher Walton HAGI site labelled.

7.1.2 The two sites to be considered in this section are Site 1 – HWH2 and Site 2 – HWH8 as shown on **Figure 7F6 7.1** below.

7.1.3 The final decision on which site will eventually be developed is still to be made hence assessment of both sites will be included in this study.

7.2 Site 1 (HWH2)

7.2.1 This site is situated immediately adjacent to the Manchester Ship Canal opposite the Acton Grange railway viaduct. Port Warrington is immediately west of the HAGI site with a local wildlife site and agricultural land to the north.

7.2.2 It appears this site has previously had some ground works performed on it however historically there is no record of previous building development. For the purpose of this study, it is anticipated that the LLFA will treat this site as a greenfield site as there is no evidence of previous drainage systems on the site.

7.2.3 The approximate site grid reference is 358861E 385746N.

7.2.4 The topography of the site has been approximated from Lidar Contour data and this shows that the levels across the site range from approximately 12mAOD in the northwest corner down to approximately 10.75mAOD in the northeast corner. This equates to an average slope of approximately 1:92 across the site west to east.

7.2.5 The British Geological Survey website indicates that the site is underlain by the Helsby Sandstone Formation - Sandstone, pebbly (gravelly), with superficial deposits of Alluvium - Clay, silt and sand. These deposits are considered to have relatively good permeability.

7.2.6 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 4m below ground level. The adjacent canal water level is at a level of approximately 4.5mAOD according to the Lidar topoFigureal information.

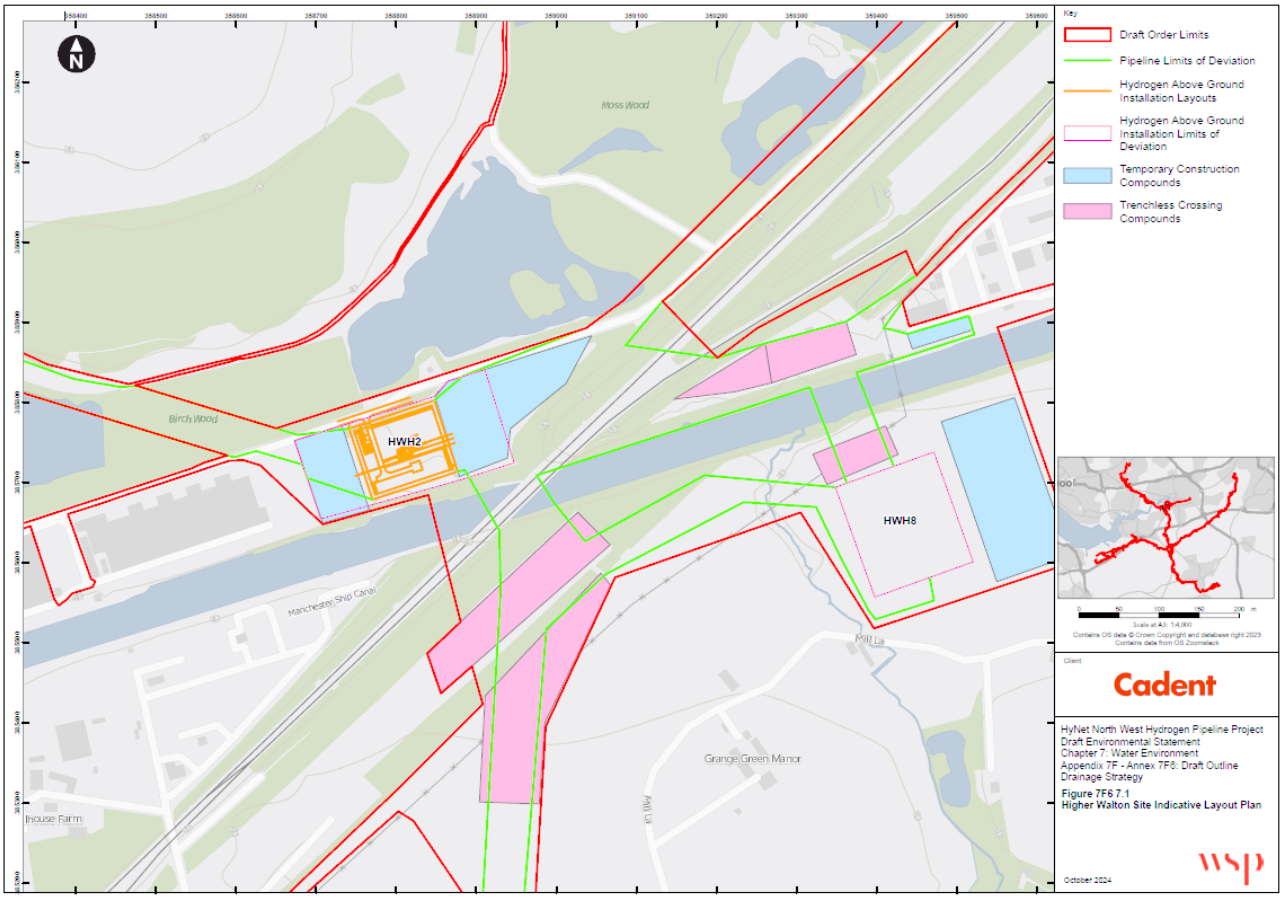
7.3 Proposed Permanent Development

7.3.1 The proposed site development plans for both sites are shown in **Figure 7F6 7.1** below. The pink areas show the proposed trenchless crossings construction compounds for the pipeline crossing the canal.

7.3.2 The alternative site location HWH8 will be assessed in the next section of this report. Until a decision has been made on which site will eventually be developed both are assessed.

7.3.3 The proposed HAGI site 1 (HWH2 outlined in orange) covers a total area of approximately 11,000m2 (1.10 hectares).

Figure 7F6 7.1 Higher Walton HAGI Sites (HWH2 and HWH8) Layout Plan



7.3.4 The Higher Walton HAGI Indicative Drainage Layout Plan for the permanent site drainage of HWH2 is included in Appendix A5. If infiltration proves not to be viable, the surface water drainage can potentially be discharged to the adjacent Manchester Ship Canal.

7.3.5 Consent for this discharge will be required from the owners of the Canal (Peel Holdings).

7.4 Allowable Site Discharge

7.4.1 The total site area of 1.1ha yields an equivalent greenfield runoff rate (Qbar) of 2.62l/s using the unit greenfield runoff rate of 2.38l/s/ha as calculated using the HR Wallingford tool using the FEH 2013 data set. The result of this calculation is shown in Appendix B5. This greenfield runoff rate has been uplifted to the practical minimum flow of 3 l/s. The layout of site 2 (HWH8) is assumed to be identical to site 1 (HWH2) and the unit greenfield runoff rates are assumed not to vary

between the two sites therefore the calculation shown in Appendix B5 will also apply to site 2(HWH8).

7.4.2 Surface water runoff from the proposed development will be restricted to this equivalent greenfield runoff rate.

7.4.3 It is proposed that a gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar.

7.5 Attenuation Provision

7.5.1 The approximate overall area of the development site is a maximum of 11,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 3,550m^2. An estimated 7,450m^2 will be available as in-situ attenuation storage on the site.

7.5.2 It is proposed that the HAGI site platform will have an approximate surface level of 11.75mAOD. The type 3 stone blanket will have a minimum depth of approximately 400mm that will yield a total potential available attenuation storage volume of around 900m^3 assuming a void ratio of 30% in the granular material.

7.5.3 A positive outfall will discharge via a new headwall to the Manchester Shipping Canal immediately south of the site. This will require the consent of the Canal owners who are Peel Holdings to be pursued post DCO consent.

7.5.4 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) show that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed greenfield runoff rate, to prevent any flooding from the site, the total minimum volume of attenuation storage required is 835m^3. These calculations are included in Appendix C5. As the layouts of both sites are assumed to be identical and the greenfield runoff rate is the same these calculations showing the attenuation storage will apply to both sites.

7.5.5 To mobilise this storage, it will be recommended to place an impermeable membrane at the base of the granular material. This will prevent water percolating vertically through to the underlying soil and horizontally through the embankments. The membrane will also prevent any possible high-level ground water seeping into the granular layer. The attenuated water will drain from the stone blanket area via a filter drain installed through the stone blanket and connected to the outfall.

7.5.6 The preliminary Indicative Drainage Layout is included in Appendix A5.

7.6 Temporary Construction Compound Drainage

7.6.1 As can be seen on the site layout plan, two areas for the HAGI construction site compounds have been proposed, a 0.60ha compound immediately west of the site and a 1.5ha site immediately east of the HAGI site. It is proposed that the eastern site will also accommodate the trenchless crossing construction compound for the canal crossing just south of this area.

7.6.2 Both of the above temporary areas and including the HAGI site itself will require temporary drainage and associated attenuation for the duration of construction of the HAGI and canal crossing.

Allowable Site Discharge

7.6.3 Based on the same unit greenfield flowrate (2.38l/s/ha Qbar) mentioned in the previous section, the allowable discharge rates from the temporary construction areas will be as follows:

- Western construction compound plus the HAGI site itself is a total area of 1.7ha which translates to an equivalent allowable discharge rate of 4.0l/s.
- Eastern construction compound at 1.5ha would translate to an allowable greenfield runoff rate of 3.6l/s. Due to the position and elevation of this proposed detention pond it is likely that this discharge will require a pumped solution to discharge at the required outfall position into the canal.

7.6.4 As shown in the preliminary drainage layout plan in Appendix A5, it is proposed that the discharges from all the temporary areas combine to discharge at the proposed permanent outfall position. The total discharge flowrate will be 7.6/s.

Attenuation Provision

7.6.5 Based on the allowable discharge rates noted above and the total areas of the temporary construction compounds, the volumes of attenuation storage required for each compound is estimated as follows:

- For the eastern compound and detention basin (known as Pond A on the drawing in Appendix A5) the required storage is estimated at 410m^3.
- For the combined western compound and HAGI construction site attenuation will be stored in detention Pond B and will need to provide 470m^3.

7.6.6 The approximate locations and sizes of detention basins based on a 1.7m water depth basin are shown on the indicative drainage layout in Appendix A5.

7.7 Existing Utilities in the Area

7.7.1 There's is a BT overhead telecommunications line along the northern boundary of the HAGI site and through the centre of the eastern temporary site. Another underground BT cable runs across along the southern boundary of the HAGI and the eastern temporary site.

7.7.2 A 2inch diameter United Utilities underground distribution water main lies along the southern boundary of the sites between the site and the Manchester shipping canal. An unknown utility also encroaches slightly onto the south side footprint of the eastern temporary compound.

7.7.3 An underground petrochemical SABIC pipeline also runs along the southern boundary of the HAGI and east compound sites. In addition, another SABIC pipeline crosses the canal running parallel to the railway line. Along this same

alignment is also a Shell NOP pipeline. These do not encroach onto the HAGI or temporary compound sites.

7.7.4 This is shown on the layout plan in Appendix A5.

7.8 Site 2 (HWH8)

7.8.1 This site is situated immediately adjacent to the south bank of the Manchester Ship Canal opposite the Perstorp/Solvay Industrial site just east of Acton Grange railway viaduct. Agricultural land extends to the south of the site.

7.8.2 This is presently a greenfield site.

7.8.3 The approximate site grid reference is 359400E 385645N.

7.8.4 The topography of the site has been approximated from Lidar Contour data and this shows that the levels across the site range from approximately 10.7mAOD in the northwest corner up to approximately 13.3mAOD in the southeast corner. This equates to an average slope of approximately 1:58 across the site southeast to northwest.

7.8.5 The British Geological Survey website indicates that the site is underlain by the Helsby Sandstone Formation - Sandstone, pebbly (gravelly), with superficial deposits of Alluvium - Clay, silt and sand and some Glaciofluvial sheet deposits. These deposits are considered to have relatively good permeability.

7.8.6 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 4m below ground level. The adjacent canal water level is at a level of approximately 4.5mAOD according to the Lidar topoFigureal information.

7.9 Proposed Permanent Development

7.9.1 The proposed site development plans for both sites are shown in **Figure 7F6 7.1** above. The pink areas show the proposed trenchless crossings construction compounds for the pipeline crossing the Canal.

7.9.2 The proposed HAGI site 1 (HWH8, eastern site) covers a total area of approximately 11,000m2 (1.10 hectares).

7.9.3 The Higher Walton HAGI Indicative Drainage Layout Plan for the permanent site drainage of HWH8 is included in Appendix A5. If infiltration proves not to be viable, the surface water drainage can be potentially discharged to the adjacent unknown water course to the west which drains into the Canal. An ordinary watercourse consent will most likely be required from the LLFA for this discharge.

7.9.4 Consent for this discharge may also be required from the owners of the Canal (Peel Holdings). It is proposed that these consenting issues are finalised post DCO consent.

7.10 Allowable Site Discharge

7.10.1 The total site area of 1.1ha yields an equivalent greenfield runoff rate (Qbar) of 2.62l/s using the unit greenfield runoff rate of 2.38l/s/ha as calculated using the HR Wallingford tool using the FEH 2013 data set. The result of this calculation is shown in Appendix B5. This greenfield runoff rate has been uplifted to the practical minimum flow of 3 l/s. (Same calculation as for site 1(HWH2)).

7.10.2 Surface water runoff from the proposed development will be restricted to this equivalent greenfield runoff rate.

7.10.3 It is proposed that a gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar.

7.11 Attenuation Provision

7.11.1 The approximate overall area of the development site is a maximum of 11,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 3,550m^2. An estimated 7,450m^2 will be available as in-situ attenuation storage on the site.

7.11.2 It is proposed that the HAGI site platform will have an approximate surface level of 12.5mAOD. The type 3 stone blanket will have a minimum depth of approximately 400mm that will yield a total potential available attenuation storage volume of approximately 900m^3 assuming a void ratio of 30% in the granular material.

7.11.3 A positive outfall will discharge via a new headwall to the watercourse to the west of the site. This will require the consent from the LLFA to be pursued post DCO consent.

7.11.4 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) show that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed greenfield runoff rate, to prevent any flooding from the site, the total minimum volume of attenuation storage required is 900m^3. These calculations are included in Appendix C5. (Same calculation as for site 1 (HWH2)).

7.11.5 The preliminary Indicative Drainage Layout for this site 2 (HWH8) is included in Appendix A5.

7.12 Temporary Construction Compound Drainage

7.12.1 As can be seen on the site layout plan for the temporary construction compound in Appendix A5, a 2.0ha area for the HAGI construction site compound has been proposed east of the HAGI site. It is proposed that drainage from the adjacent trenchless crossing construction compound for the canal crossing will also be accommodated. The approximate area of this compound is 0.4ha.

7.12.2 Both of the above temporary areas and including the HAGI site itself will require temporary drainage and associated attenuation for the duration of construction of the HAGI and canal crossing, total area of 3.5ha.

Allowable Site Discharge

7.12.3 Based on the same unit greenfield flowrate (2.38l/s/ha Qbar) mentioned in the previous section, the allowable discharge rate from the temporary construction areas is estimated at 8.3l/s.

7.12.4 As shown in the preliminary drainage layout plan in Appendix A5, it is proposed that the discharges from all the temporary areas combine to discharge at the proposed permanent outfall position. The total discharge flowrate will be 7.6/s.

Attenuation Provision

7.12.5 Based on the allowable discharge rates noted above and the total areas of the temporary construction compounds, the volume of attenuation storage required for all the compounds is estimated at 960m^3.

7.12.6 The approximate location and size of the detention basin based on a 1.7m water depth basin is shown on the indicative drainage layout in Appendix A5.

7.13 Existing Utilities in the Area

7.13.1 According to the GIS there were no existing utilities recorded in the immediate area of the site.

8 Clock Face HAGI Site

8.1.1 This section of the report sets out the Outline Drainage Strategy for the proposed Clock Face HAGI which is located on the northern corridor of the distribution network. This can be seen on **Figure 7F6 1.1** which shows the extents of the complete Hynet North West Hydrogen Pipeline Project.

8.2 Existing Site

8.2.1 The existing site is situated on existing agricultural land between the M62 motorway and B5419. The site is approximately 5km due south of St Helens. The surrounding area is extensive agricultural land.

8.2.2 The approximate site grid reference is 351331E 390013N.

8.2.3 The topography of the site has been approximated from Lidar Contour data and this shows that the land slopes gently from the highest point on the southern corner at an elevation approximately 57.5mAOD down to the lowest point midway along the northern boundary at an approximate elevation of 56mAOD. This equates to an average slope of approximately 1:103 across the site, sloping from south to north.

8.2.4 The British Geological Survey website indicates that the site is underlain by the Kinnerton Sandstone Formation in the southeast and mudstone, sandstone and conglomerate (Etruria Formation) in the northwest. Superficial deposits consist of Till, Devensian - Diamicton. These deposits are considered to be soils of relatively poor permeability.

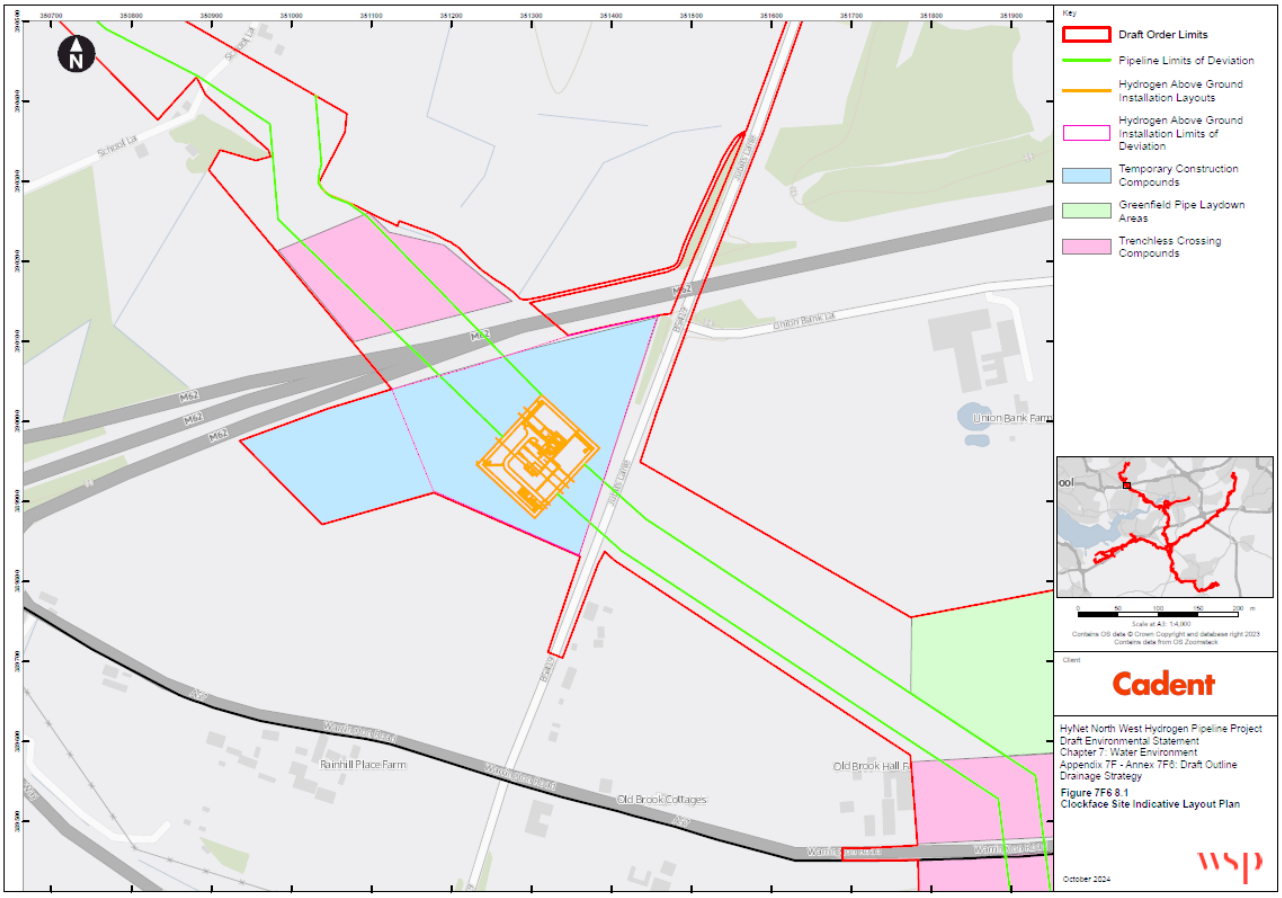
8.2.5 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 3.6m to 7m below ground level.

8.3 Proposed Permanent Development

8.3.1 The proposed site is shown in **Figure 7F6 8.1** below. The blue area represents the proposed temporary construction compound area for the HAGI.

8.3.2 The proposed HAGI covers a total area of approximately 12,000m^2 (1.20 hectares).

Figure 7F6 8.1 Clock Face HAGI Site Layout Plan



8.3.3 The Clock Face HAGI Indicative Drainage Layout Plan for the drainage of the permanent site is included in Appendix A6. The soils in this area are anticipated to have relatively poor permeability, therefore if infiltration is not a viable method of surface water disposal on this site, the surface water will require draining either to a water course or sewer.

8.3.4 Information for the site shows that there are no water courses that are accessible from the site, however there is an agricultural drainage ditch draining from the north side of the M62 motorway. There is also evidence of a culvert headwall or similar on the southern side of the M62, opposite the drainage ditch on the north side. Therefore, the proposal is to discharge to the drainage ditch via the existing culvert headwall on the south side of the motorway. More investigation and details are required to finalise this discharge position in the next detailed design stage. It is being considered whether this will be included in a DCO requirement.

8.4 Allowable Site Discharge

8.4.1 The total site area of 1.20ha yields an equivalent greenfield runoff rate (Qbar) of 9.14l/s. The unit greenfield runoff rate for this area is calculated at 7.62l/s/ha

8.4.2 Appendix B6 shows the Greenfield Runoff Rate calculation sheet.

8.4.3 Surface water runoff from the proposed development will be restricted to this equivalent greenfield runoff rate.

8.4.4 It is proposed that gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar. If discharge under gravity is not possible then a pumped solution, pumping at the agreed restricted rate will be required. This will be finalised at the next, more detailed stage of design. It is being considered whether this will be included in the DCO requirements.

8.5 Attenuation Provision

8.5.1 The overall area of the development site is 12,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 6,780m^2. An estimated 5,220m^2 will be available as in-situ attenuation storage on the site.

8.5.2 It is proposed that the site platform will have an approximate surface level of approximately 56.5mAOD. The type 3 stone blanket will have a minimum depth of approximately 450mm that will yield a total potential available attenuation storage volume of approximately 700m^3 assuming a void ratio of 30% in the stone material.

8.5.3 A positive outfall will discharge at the restricted greenfield runoff rate via a new headwall to the highways drainage running along the southern edge of the M62 motorway. This discharge connection will require the agreement of Highways England and the Lead Local Flood Authority.

8.5.4 The preliminary hydraulic model calculations (using MicroDrainage Source Control module) show that for the 1:100 year plus climate change rainfall event and restricting the discharge from the site to the agreed greenfield runoff rate, to prevent any flooding from the site, the volume of attenuation storage required is 700m^3. These calculations are included in Appendix C6.

8.5.5 The preliminary Indicative Drainage Layout is included in Appendix A6.

8.6 Temporary Construction Compound Drainage

8.6.1 In addition to the HAGI site itself, which will require temporary drainage during construction, there is a 7.8ha temporary construction compound surrounding the HAGI site.

8.6.2 All the above areas will require temporary drainage and associated attenuation for the duration of construction. The total area of both sites then will be approximately 9.0ha.

Allowable Site Discharge

8.6.3 Based on the calculated Qbar unit greenfield runoff rate noted in the previous section, the allowable discharge rate for all of the above areas will be 68.6/s.

8.6.4 There is a high risk that the capacity of the existing culvert under the M62 may not be sufficient to accommodate this proposed discharge rate. More temporary attenuation storage may need to be provided to reduce the discharge rate and

balance it with the culvert capacity. More investigation into the culvert capacity will be required in detailed design stage. It is being considered whether this will be secured within a DCO requirement.

8.6.5 It is proposed that the outfall from this system will be to the same discharge point to be used for the permanent scheme and is shown in Appendix A6.

Attenuation Provision

8.6.6 Based on the allowable discharge noted above and the total area of the temporary construction compounds, the volume of attenuation required for the temporary construction compound and HAGI site during construction is estimated to be 1,600m^3 to retain the 1 in 100-year (1%AEP) event without flooding occurring from the site.

8.6.7 It is noted that as the compound is a temporary feature that will be removed after construction.

8.6.8 The approximate size of basin is based on a 1.7m deep pond/swale and is shown on the indicative drainage layout in Appendix A6.

8.7 Existing Utilities in the Area

8.7.1 There are 4 No. United Utilities underground main water pipelines crossing the western section of the site from the south to the northwest of the HAGI. The pipe sizes range from 1000mm diameter, 42 inches and 72 inches.

8.7.2 There is a possible surface water culvert draining under the M62 motorway – size and capacity unknown.

8.7.3 These utilities are shown on the layout plan in Appendix A6.

9 Warburton HAGI Site

9.1.1 This section of the report sets out the outline drainage strategy for the proposed Warburton HAGI site which is located approximately 2.5km south east of Warburton village, on the eastern corridor of the distribution network as shown in **Figure 7F6 1.1**.

9.2 Existing Site

9.2.1 The existing site is situated on existing agricultural land immediately adjacent to an existing gas above ground installation on the west side of the HAGI with some residential buildings immediately east. The remainder of the surrounding area is agricultural lands.

9.2.2 The approximate site grid reference is 372238E 388586N.

9.2.3 The topography of the site has been approximated from Lidar Contour data and this shows that the southwestern corner is at approximately 25mAOD elevation falling to approximately 23.5mAOD in the eastern corner. This is a fall of approximately 1.5m in 120m which equates to an average slope of approximately 1:80 across the site.

9.2.4 Ground investigations are in progress however the British Geological Survey website indicates that the site is underlain Wilmslow Sandstone Formation - Sandstone. Superficial deposits of Shirdley Hill Sand Formation formed from windblown sand. These deposits are associated with deep well drained sandy soils with anticipated good permeability.

9.2.5 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 2.4m below ground level.

9.2.6 According to the Halite Risk Assessment 2024 (Wardell Armstrong) this HAGI site falls within the Cheshire Brine Compensation District. Although this HAGI site is within the compensation district, it is not in a “wet rockhead” area where the possibility of natural subsidence at the surface is greatest, through dissolution of the subterranean salt rock.

9.2.7 The Cheshire Brine Subsidence Compensation Board (CBSCB) will need to be consulted with regard to the surface water drainage proposals for the HAGI due to the sensitive nature associated with infiltration in this area. The CBSCB will be a statutory consultee in the DCO process.

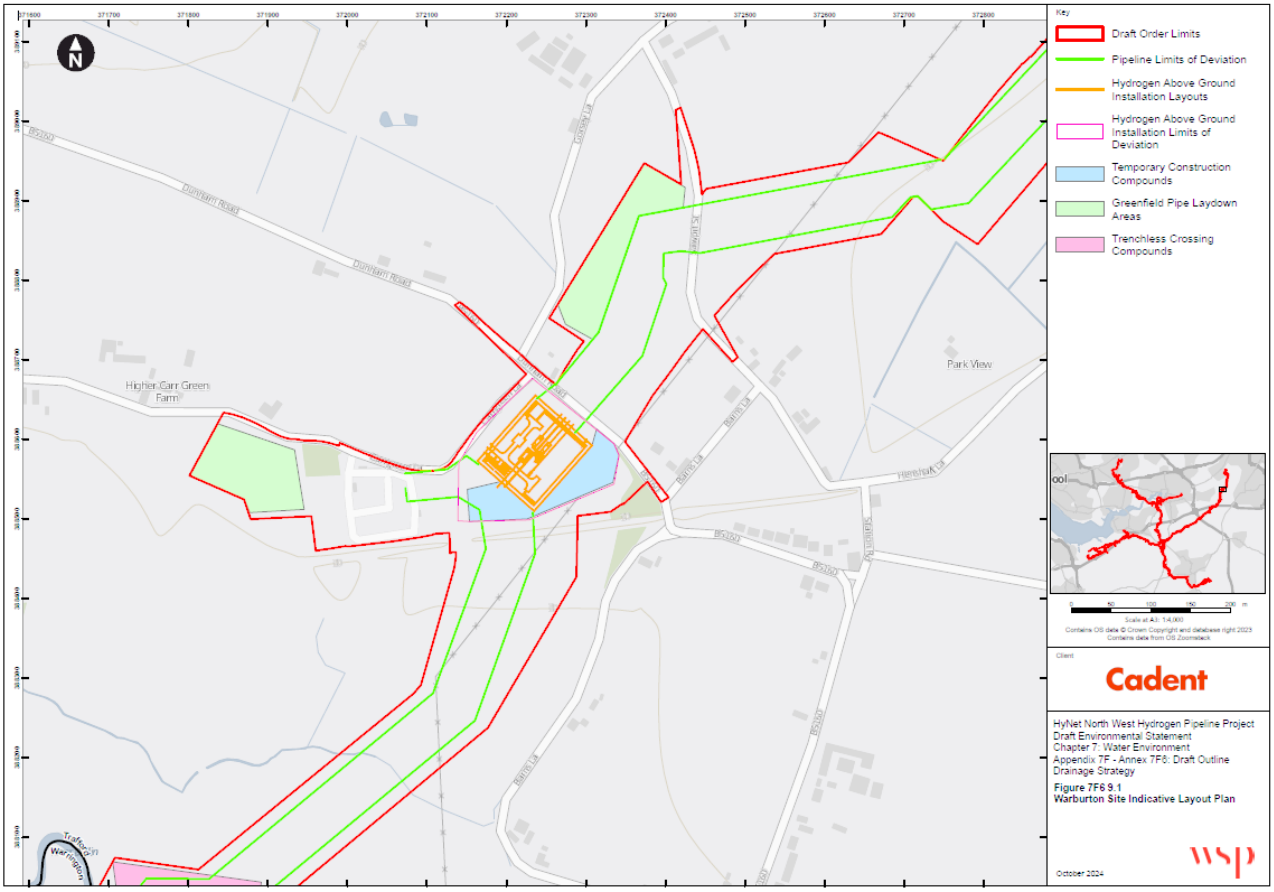
9.2.8 Should infiltration be shown to be a viable method of surface water disposal it will need to be considered in close consultation with the CBSCB.

9.3 Proposed Permanent Development

9.3.1 The proposed site development is shown in **Figure 7F6 9.1** below. The green areas indicate proposed greenfield pipe laydown areas to be used in the construction process. The blue area is the temporary construction compound area.

9.3.2 The proposed permanent HAGI site covers a total area of approximately 10,000m^2 (1.00Ha).

Figure 7F6 9.1 Warburton HAGI Site Layout Plan



9.3.3 The Warburton HAGI Indicative Drainage Layout Plan for the permanent drainage is shown in Appendix A7. This also shows the location of a watercourse approximately 450m to the south of the site. Preliminary indications are that infiltration may be a viable method of surface water disposal in this area, however, if infiltration is not viable, drainage will probably be discharged to this watercourse.

9.3.4 An ordinary watercourse consent will need to be agreed with the LLFA should this outfall discharge connection be required.

9.4 Allowable Site Discharge

9.4.1 The total site area of 1.00ha yields an equivalent greenfield runoff rate (Qbar) of 2.38l/s. This low total flow rate has been uplifted to the minimum practical flowrate of 3l/s to be used for the attenuation storage calculations for this HAGI.

9.4.2 Appendix B7 shows the Greenfield Runoff Rate calculation sheet using the HR Wallingford greenfield runoff tool with FEH 2013 rainfall data.

9.4.3 Surface water runoff from the proposed development will be restricted to the equivalent greenfield runoff rate.

9.4.4 It is proposed that a gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar. If discharge under gravity is not possible then a pumped solution pumping at the agreed restricted rate will be required. It is being considered whether this will be secured as a DCO requirement.

9.5 Attenuation Provision

9.5.1 The total area of the development site is 10,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 4,420m^2. An estimated 5,580m^2 will therefore be available as in-situ attenuation storage on the site.

9.5.2 It is proposed that the site platform will have an approximate surface level of 24.5mAOD.

9.5.3 A positive outfall will discharge via a new headwall to the River Bollin approximately 800m to the south of the site. This will probably require a FRAP from the EA as this is classified as a main river.

9.5.4 The preliminary hydraulic calculations indicate that using a vortex flow control valve to restrict the discharge to the required greenfield flow (3l/s) will require an attenuation storage volume of approximately 755m^3 , which translates to an equivalent 450mm depth of type 3 stone blanket to avoid flooding occurring during the critical 1 in 100-year + climate change event. These calculations are included in Appendix C7.

9.5.5 The preliminary Indicative Drainage Layout for the permanent drainage is included in Appendix A7.

9.6 Temporary Construction Compound Drainage

9.6.1 The site layout plan identifies a 0.72ha temporary construction compound adjacent to the south of the HAGI site. The total area then to be drained during the construction stage will be 1.72ha.

9.6.2 All the above temporary areas will require temporary drainage and associated attenuation for the duration of construction.

9.6.3 It is proposed that all the areas will drain to a temporary attenuation basin along the southern boundary of the construction sites, which will drain at an agreed restricted flow to the River Bollin as above.

Allowable Site Discharge

9.6.4 Based on the calculated Qbar unit rate of 2.38l/s/ha noted in the previous section, the allowable discharge rate for all of the above areas will be 4.1l/s. It is proposed that the outfall from this system will also be to the proposed outlet for the permanent drainage to the watercourse south of the site as shown in Appendix A7.

Attenuation Provision

9.6.5 Based on the allowable discharge noted above and the total area of the compounds, the volume of attenuation required for the compound is estimated to be 470m^3 for the 1 in 100-year (1%AEP) event. It is noted that as the compound is a temporary feature that will be removed after construction of the HAGI and it has been agreed in principal that these will be assessed using present day flood risk estimates only and therefore no allowance for climate change has been considered.

9.6.6 The approximate area based on a 1.7m deep pond/swale is shown on the indicative drainage layout in Appendix A7.

9.7 Existing Utilities in the Area

9.7.1 There is an overhead national grid electric power line running parallel with the southeast boundary of the HAGI. The overhead line is approximately 30m from the HAGI boundary but crosses the temporary construction compound area. There is also an SP Energy Networks underground low voltage cable in Dunham Road, which is approximately 20m north of the northeastern HAGI boundary.

9.7.2 There is a BT underground cable which runs into the eastern and southern area of the HAGI site. This branches off from a number of other BT cables in Dunham Road as well as in Carrgreen Lane running parallel to the northwestern HAGI boundary approximately 20m to the west.

9.7.3 There is also a National Grid high pressure gas transmission line connected to the above ground installation to the west of the site. This gas line runs in a north/south direction approximately 65m west of the western corner of the HAGI site.

9.7.4 There is a 400mm diameter United Utilities watermain running east/west in the Trans Pennine Trail route approximately 25m south of the southern corner of the site. There is also a 110mm diameter UU distribution watermain in Carrgreen Lane and Dunham Road.

9.7.5 All the above utilities are shown on the layout plan in Appendix A7.

10 Partington HAGI Site

10.1.1 This section of the report sets out the Outline Drainage Strategy for the proposed Partington HAGI sites, which is located at the northern end of the eastern corridor of the distribution network as shown in **Figure 7F6 1.1**. At Stage three statutory consultation, two alternative site locations are proposed, (see **Figure 7F6 10.1** below). This Outline Drainage Strategy will consider both alternatives, site 1 on the existing derelict Cadent Gas Installation brownfield site and site 2 approximately 550m due east of alternative site 1.

10.1.2 The alternate sites will be considered in the following two separate sections.

10.2 Site 1

10.2.1 The site is situated on the eastern portion of the existing Cadent Gas Installation site with derelict industrial land surrounding the new proposed site on all sides except the southern boundary which is adjacent to the previous railway link which is now abandoned and has naturally revegetated with thick growth forming a green corridor along this boundary.

10.2.2 For the purposes of this study the site is treated as a brownfield (predeveloped) site.

10.2.3 The approximate grid reference of Site 1 is 372737E 391668N.

10.2.4 The topography of the site has been approximated from Lidar Contour data and this shows that the site is very flat at an approximate elevation of 20.0mAOD. The location of the HAGI is positioned over the previously demolished gas holder which was on this site.

10.2.5 The British Geological Survey website indicates that the site is underlain by gravelly sandstone (Wilmslow Sandstone Formation). Superficial deposits in the area of the HAGI site are River Terrace deposits, sand and gravel. The area has also contained a layer of made ground from the previous development/s. These deposits are considered to have low permeability; therefore, infiltration would probably not be viable.

10.2.6 According to the Halite Risk Assessment this HAGI site falls within the Cheshire Brine Compensation District. Although this HAGI site is within the compensation district, it is not in a “wet rockhead” area where the possibility of natural subsidence at the surface, through dissolution of the subterranean salt rock, is greatest.

10.2.7 The Cheshire Brine Subsidence Compensation Board (CBSCB) will need to be consulted with regard to the drainage proposals for the HAGI. The CBSCB will be a statutory consultee in the DCO process.

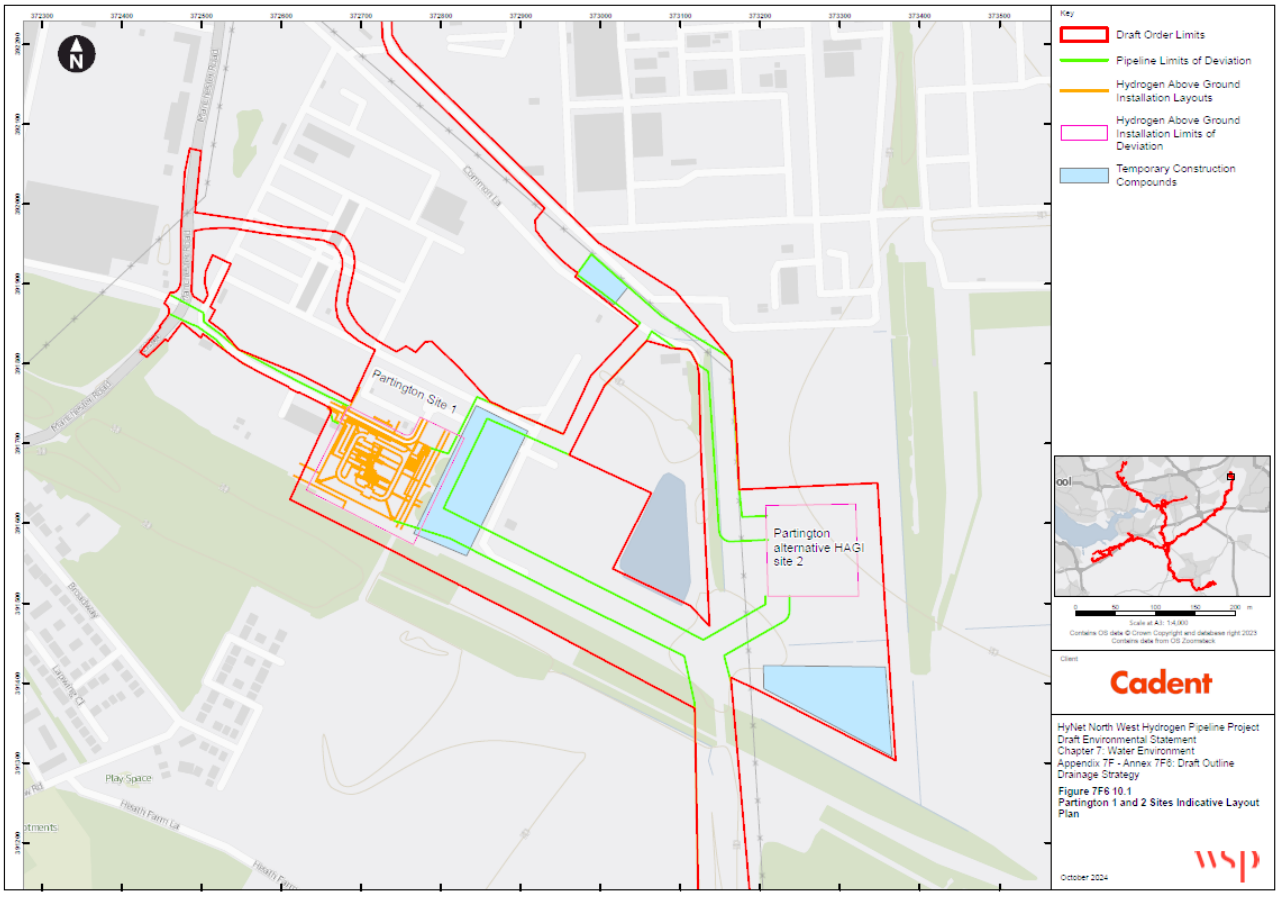
10.2.8 Should infiltration be shown to be a viable method of surface water disposal it will need to be considered in close consultation with the CBSCB.

10.3 Proposed Permanent Development

10.3.1 The proposed Sites (1 and 2) are shown in **Figure 7F6 10.1** below**.** The blue areas adjacent to the sites are to accommodate the temporary construction compounds for each site.

10.3.2 The proposed HAGI site covers a total area of approximately 11,000m^2 (1.1 hectares).

Figure 7F6 10.1 Partington HAGI Sites Plan



10.3.3 The Partington HAGI Indicative Drainage Layout Plan for the permanent drainage for site 1 is included in Appendix A8. As shown on this plan it is proposed to drain the new proposed site into the existing drainage infrastructure on the brownfield site.

10.3.4 It is proposed at this stage that the details of the exact connection point to the existing infrastructure will be finalised during the next stage of detailed design to be done post DCO consent.

10.4 Allowable Site Discharge

10.4.1 As this site is a brownfield site, normally the LLFA would allow a restricted flow from the site equal to the previous brownfield runoff less an agreed percentage betterment (usually ranging from 30% to 50%).

10.4.2 The Trafford LLFA requirement for discharge from a brownfield site is stated as; "*Previously Developed Site: For developments which were previously developed, the peak runoff rate from the development to any drain, sewer or surface water body for the 1 in 1 year rainfall event and the 1 in 100 year rainfall event must be as close as reasonably practicable to the greenfield runoff rate from the development for the same rainfall event, but should never exceed the rate of discharge from the development prior to redevelopment for that event.*" [10]

10.4.3 Using the above criteria, the unit greenfield runoff rate for the 1% AEP event for this area is calculated at 9.4l/s/ha. The total HAGI site area of 1.1ha yields an equivalent greenfield runoff rate of 10.4l/s. This will be used as the brownfield runoff rate according to Trafford LLFA requirements. This calculation is shown in Appendix B8.

10.4.4 Surface water runoff from the Project will be restricted to this equivalent "brownfield" runoff rate.

10.4.5 It is proposed that gravity discharge from the site will be restricted via a flow control valve to this flow rate. If discharge under gravity is not possible then a pumped solution, pumping at the agreed restricted rate will be required. This will be finalised at the next, more detailed stage of design. It is being considered whether this will be included as a DCO requirement.

10.5 Attenuation Provision

10.5.1 The overall area of the development site is 11,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 7,845m^2, therefore an estimated 3,155m^2 will be available as in-situ attenuation storage on the site.

10.5.2 It is proposed that the site platform will have an approximate surface level of 19.7mAOD. The type 3 stone blanket will have a minimum thickness of approximately 700mm that will yield a total potential available attenuation storage volume of approximately 660m^3 assuming a void ratio of 30% in the granular material.

10.5.3 It is proposed that a positive outfall will discharge via a new connection pipe to the existing site drainage to the north. This will probably require an internal approval from Cadent.

10.5.4 The preliminary hydraulic model calculations show that restricting the discharge rate from the site to the agreed greenfield runoff rate, an attenuation storage volume of approximately 660m^3 will be required to prevent flooding occurring during the 1 in 100-year + climate change event. These calculations are contained in Appendix C8.

10.5.5 The Indicative Drainage Layout for the permanent installation is shown in Appendix A8.

10.6 Temporary Construction Compound Drainage

10.6.1 As can be seen on the site layout plan, in addition to the HAGI site there is a 1.1ha temporary construction compound to the east of the permanent HAGI site.

10.6.2 Therefore, a total temporary area of 2.2ha will require temporary drainage and associated attenuation for the duration of construction.

Allowable Site Discharge

10.6.3 Based on the unit greenfield 1% AEP runoff rate of 9.4l/s/ha noted in the above section, the total allowable discharge rate for all of the above areas will be 22.6l/s. It is proposed that the outfall from this system will be to the newly installed drainage connection for the permanent drainage as shown in Appendix A8.

Attenuation Provision

10.6.4 Based on the allowable discharge noted above and the total area of the construction compound and HAGI site, the total volume of attenuation storage required to prevent flooding during the critical 1 in 100-year (1%AEP) event is estimated to be 470m^3.

10.6.5 It is noted that as the compound is a temporary feature it will be removed after construction.

10.6.6 The approximate location and size of the detention basin based on a 1.7m deep basin is shown on the indicative drainage layout in Appendix A8.

10.7 Existing Utilities in the Area

10.7.1 There is a National Grid high pressure underground gas line which runs between the permanent HAGI site and the temporary construction compound.

10.7.2 There is a Zayo Group underground telecom cable encroaching across the northern and eastern sections of the HAGI site, and an overhead BT telecom line encroaching across the western side of the site.

10.7.3 There is an abandoned underground high voltage Electricity North West cable skirting the northern side of the site and encroaching across the eastern half of the site.

10.7.4 These existing utilities are shown on the layout plan in Appendix A8.

10.8 Site 2

10.8.1 The Site is situated in an agricultural field approximately 550m due east of site 1 as shown in **Figure 7F6 10.1**. To the north lies the Partington industrial area with agricultural land stretching to the east and south of the site.

10.8.2 The approximate grid reference of Site 2 is 373264E 391568N.

10.8.3 The topography of the Site has been approximated from Lidar Contour data and this shows that the site is very flat at an approximate elevation of 20.0mAOD.

10.8.4 The British Geological Survey website indicates that the site is underlain by gravelly sandstone (Wilmslow Sandstone Formation). Superficial deposits in the area of the HAGI site are Peat. These deposits are considered to have very low permeability; therefore, infiltration would probably not be viable.

10.8.5 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 2m below ground level.

10.8.6 According to the Halite Risk Assessment this HAGI site falls within the Cheshire Brine Compensation District. Although this HAGI site is within the compensation district, it is not in a “wet rockhead” area where the possibility of natural subsidence at the surface, through dissolution of the subterranean salt rock, is greatest.

10.8.7 The Cheshire Brine Subsidence Compensation Board (CBSCB) will need to be consulted with regard to the drainage proposals for the HAGI. The CBSCB will be a statutory consultee in the DCO process.

10.8.8 Should infiltration be shown to be a viable method of surface water disposal it will need to be considered in close consultation with the CBSCB.

10.9 Proposed Permanent Development

10.9.1 The proposed Site 2 is shown in **Figure 7F6 10.1** above**.** The blue area to the south of the site is to accommodate the temporary construction compound.

10.9.2 The proposed HAGI site covers a total area of approximately 11,000m^2 (1.1 hectares).

10.9.3 The Partington HAGI Site 2 Indicative Drainage Layout Plan for the permanent drainage proposed for this site is included in Appendix A8. The Main Rivers Map shows a drainage ditch on the western boundary of the field in which the HAGI is positioned, therefore if infiltration is not viable on this site drainage will be discharged to this drain.

10.10 Allowable Site Discharge

10.10.1 The total HAGI site area of 1.1ha yields an equivalent greenfield runoff rate (Qbar) of 4.2l/s. The unit greenfield runoff rate for this area is calculated at 3.80l/s/ha. This calculation is shown in Appendix B8.

10.10.2 Surface water runoff from the Project will be restricted to this equivalent greenfield runoff rate.

10.10.3 It is proposed that gravity discharge from the site will be restricted via a flow control valve to this flow rate. If discharge under gravity is not possible then a pumped solution pumping at the agreed restricted rate will be required. This will be finalised at the next, more detailed stage of design. It is being considered whether this will be included as a DCO requirement.

10.11 Attenuation Provision

10.11.1 The overall area of the development site is 11,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 7,840m^2, therefore an estimated 3,160m^2 will be available as in-situ attenuation storage on the site.

10.11.2 It is proposed that the site platform will have an approximate surface level of 20.5mAOD. The type 3 stone blanket will have a minimum thickness of approximately 850mm that will yield a total potential available attenuation storage volume of approximately 810m^3 assuming a void ratio of 30% in the granular material.

10.11.3 A positive outfall will discharge via a new headwall to the drainage ditch to the west of the site. This will require the approval of the LLFA and will be subject to an ordinary watercourse discharge consent.

10.11.4 The preliminary hydraulic model calculations show that restricting the discharge rate from the site to the agreed greenfield runoff rate, an attenuation storage volume of approximately 810m^3 will be required to prevent flooding occurring during the 1 in 100-year + climate change event. These calculations are contained in Appendix C8.

10.11.5 The Indicative Drainage Layout for the permanent installation is shown in Appendix A8.

10.12 Temporary Construction Compound Drainage

10.12.1 As can be seen on the site layout plan in addition to the HAGI site there is a 1.1ha temporary construction compound to the south of the HAGI site.

10.12.2 Therefore, a total temporary area of 2.2ha will require temporary drainage and associated attenuation for the duration of construction.

Allowable Site Discharge

10.12.3 Based on the calculated Qbar rate of 3.8l/s/ha noted in the above section, the total allowable discharge rate for all of the above areas will be 8.4l/s. It is proposed that the outfall from this system will be to the newly installed drainage outlet for the permanent drainage as shown in Appendix A8.

Attenuation Provision

10.12.4 Based on the allowable discharge noted above and the total area of the construction compound and HAGI site, the total volume of attenuation storage required to prevent flooding during the critical 1 in 100-year (1% AEP) event is estimated to be 550m^3.

10.12.5 It is noted that as the compound is a temporary feature it will be removed after construction.

10.12.6 The approximate location and size of the detention basin based on a 1.7m deep basin is shown on the indicative drainage layout in Appendix A8.

10.13 Existing Utilities in the Area

10.13.1 There is an ESSAR petro-chemical pipeline running along the western boundary of the agricultural field approximately 50m from the west side of the HAGI site.

10.13.2 Also running along the western boundary of the field is an extra high voltage 132kV overhead electric power line. Providers are SP Energy Networks and Electricity North West.

10.13.3 These existing utilities are shown on the layout plan in Appendix A8.

11 Hydrogen Storage Facility HAGI Site

11.1.1 This section of the report sets out the outline drainage strategy for the proposed Hydrogen Storage Facility (HSF) HAGI site, which is located at the end of the South Corridor as shown in **Figure 7F6 1.1.**

11.2 Existing Site

11.2.1 The existing site is situated on a greenfield site which is surrounded by existing agricultural land. The site is located on the northern bank of the adjacent Puddinglake Brook, which is classified as an EA main river.

11.2.2 The approximate site grid reference is 369714E 370014N.

11.2.3 A topography survey has not been undertaken; however Lidar data of the site shows the lower south side of the site at an elevation of 31.5mAOD rising to the northern side at an approximate elevation of 32.5mAOD. This results in an average slope across the site of approximately 1:108 from north to south.

11.2.4 The British Geological Survey website indicates that the site is underlain by Sidmouth Mudstone Formation - Mudstone. Halite is also present in the mudstones. The superficial geology consists of deposits of Diamicton Glacial Till - Clay, silt and sand (Tidal Flat Deposits). These soils are seasonally wet deep red clay – considered to have relatively low permeability.

11.2.5 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 2m to 2.9m below ground level.

11.2.6 According to the Halite Risk Assessment this HAGI site falls within the Cheshire Brine Compensation District. In addition, this site falls just east of the “wet rockhead” area where the possibility of natural subsidence at the surface is greatest, through dissolution of the subterranean salt rock.

11.2.7 For this reason, we recommend that infiltration as a surface water disposal method should be avoided at this site and that discharge to the nearby Puddinglake Brook is preferred.

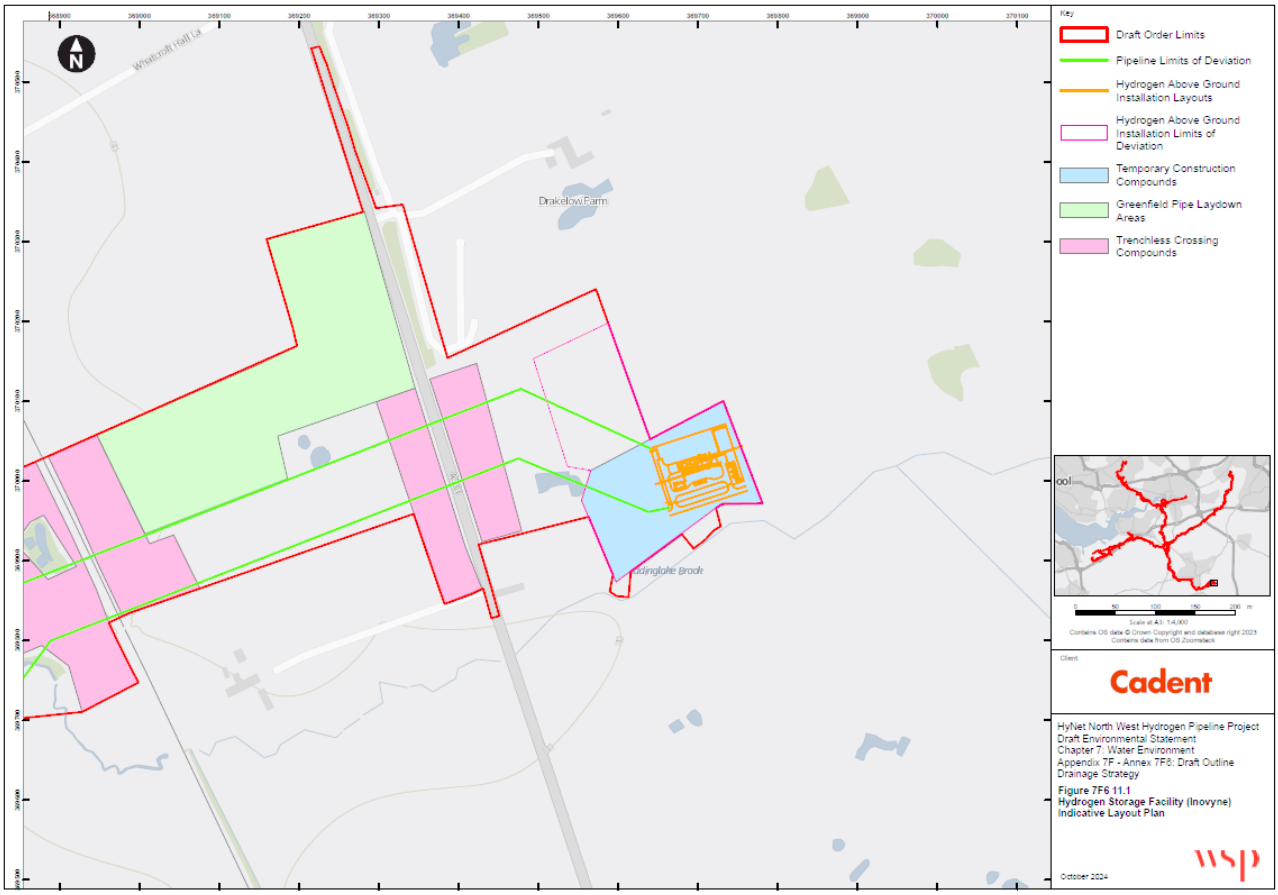
11.2.8 The Cheshire Brine Subsidence Compensation Board (CBSCB) will need to be consulted with regard to the drainage proposals for the HAGI. The CBSCB will be a statutory consultee in the DCO process.

11.3 Proposed Permanent Development

11.3.1 The proposed site is shown in **Figure 7F6 11.1** below. Two nearby trenchless crossing construction compounds and a greenfield pipe laydown area are also proposed. Only the drainage of the permanent HAGI site is considered here.

11.3.2 The proposed HAGI covers a total area of approximately 8,800m^2 (0.88 hectares).

Figure 7F6 11.1 Hydrogen Storage Facility HAGI Site Layout Plan



11.3.3 The Hydrogen Storage HAGI Indicative Drainage Layout Plan is included in Appendix A9. This also shows the location of the Puddinglake Brook approximately 25m to the south of the site. Due to the halite classification of this site, infiltration is to be avoided and therefore drainage will be discharged to this watercourse.

11.3.4 The Puddinglake Brook is an EA main river and therefore a Flood Risk Activity Permit (FRAP) may be required for this discharge outfall.

11.4 Allowable Site Discharge

11.4.1 The total site area of 0.88ha yields an equivalent greenfield runoff rate (Qbar) of 5.6l/s using the HR Wallingford Greenfield Runoff Tool with FEH (2013) rainfall data. The unit greenfield runoff rate is calculated at 6.35l/s/ha.

11.4.2 Appendix B9 shows the Greenfield Runoff Rate calculation sheet.

11.4.3 Surface water runoff from the proposed development will be restricted to the equivalent greenfield runoff rate.

11.4.4 It is proposed that a gravity discharge from the site will be restricted via a flow control valve to be equal to the greenfield runoff rate Qbar. The riverbank is shown at a level of approximately 30mAOB in this area therefore a gravity connection should be feasible.

11.5 Attenuation Provision

11.5.1 The overall area of the development site is 8,800m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 4,660m^2. An estimated 4,140m^2 will be available as in-situ attenuation storage on the site.

11.5.2 It is proposed that the site platform will have an approximate surface level of 32.18mAOD. The type 3 stone blanket will have a minimum depth of approximately 550mm that will yield a total potential available attenuation storage volume of approximately 690m^3 assuming a void ratio of 30% in the granular material.

11.5.3 A positive gravity outfall will discharge via a new headwall to the Puddinglake Brook immediately south of the site. This outfall will require a FRAP from the EA.

11.5.4 The preliminary hydraulic calculations indicate that for the 1 in 100-year + 45% climate change event and restricting the discharge to greenfield runoff rate, to prevent flooding during this event, an attenuation storage volume of 620m^3 will be required.

11.5.5 The preliminary hydraulic calculations for the attenuation volume sizing as summarised above is included in Appendix C9 and an Indicative Drainage Layout Plan is included in Appendix A9.

11.6 Temporary Construction Compound Drainage

11.6.1 A 2.6ha temporary construction compound is to enable the construction of the HAGI.

11.6.2 There is also a proposed pipe laydown area of approximately 6.86ha and two trenchless crossing compounds totalling 8.07ha within the vicinity of this HAGI site.

11.6.3 The above temporary areas will require temporary drainage and associated attenuation for the duration of construction.

11.6.4 For the purposes of this exercise the following was assumed:

- The HAGI site plus construction compound will drain to pond A. Total catchment area of 2.6ha.
- The eastern crossing compounds plus the eastern portion of the pipe laydown compound totalling an area of 7.1ha will drain to pond B.
- The western portion of the pipe laydown area plus the eastern section of the western crossing compound totalling an area of 1.8ha will drain to pond C.
- Western section of the western crossing compound totalling 4.0ha will drain to pond D.

Allowable Site Discharge

11.6.5 Based on the calculated Qbar rate of 6.35l/s/ha noted in the above section, the allowable discharge rates for the above areas will be as follows:

- Areas draining to basin A, 16.5l/s plus flow received from pond B = 61.4l/s discharging to the Brook at the permanent drainage outlet location.
- Areas draining to basin B, 45.1l/s discharging to pond A.
- Areas draining to basin C, 11.8l/s discharging to pond D.
- Areas draining to basin D, 25.4l/s plus 11.8l/s from basin C = 37.2l/s discharging to the Brook at a temporary location.

Attenuation Provision

11.6.6 Based on the allowable discharge rates noted above and the total areas of the compound areas, the attenuation volumes of the individual basins are estimated as follows:

- Attenuation volume of basin A is 600 m^3.
- Attenuation volume of basin B is 1620 m^3.
- Attenuation volume of basin C is 410 m^3.
- Attenuation volume of basin D is 920 m^3.

11.6.7 It is noted that as the compounds are temporary, they will be removed after construction.

11.6.8 The approximate locations and sizes of detention basins as shown in Appendix A9 are based on a 1.7m basin.

11.7 Existing Utilities in the Area

11.7.1 There is an existing National Grid high pressure gas pipeline approximately 180m to the west of the HAGI site. This pipeline runs to the existing gas above ground installation in this area.

11.7.2 There is also an 11kV overhead high voltage SP Energy Networks electrical power line running north/south approximately 140m to the west of the site.

11.7.3 There are BT telecom cables and a 6 inch diameter UU watermain running in King Street (A530) road reserve approximately 250m west of the site.

11.7.4 All the above utilities are shown on the layout plan in Appendix A9.

12 Sworton Block Valve Installation Site

12.1.1 This section of the report sets out the outline drainage strategy for the proposed Sworton BVI site which is located approximately halfway along the eastern corridor of the pipeline distribution network as shown in **Figure 7F6 1.1.**

12.2 Existing Site

12.2.1 The site is situated on existing agricultural land adjacent to the north side of the A50. The surrounding area is agricultural with the village of High Legh approximately 600m to the east of the site.

12.2.2 The approximate site grid reference is 369382E 384055N.

12.2.3 The topography of the site has been approximated from Lidar Contour data and this shows that the BVI site is situated on the highest ground in the field at an approximate elevation of 73mAOD. The site slopes away very gradually downhill towards the north, south and west from the site.

12.2.4 Ground investigations are in progress however the British Geological Survey website indicates that the site is underlain by the Bollin Mudstone Member - Mudstone. There is an absence of superficial deposits around this block valve site area. The soils in this area are seasonally wet deep red loam to clay and are considered to have relatively low permeability.

12.2.5 Information from the BGS website shows no available borehole logs in the area with information for the area ground water levels.

12.2.6 According to the Halite Risk Assessment this BVI site falls within the Cheshire Brine Compensation District. Although this site is within the compensation district, it is not in a “wet rockhead” area where the possibility of natural subsidence at the surface is greatest, through dissolution of the subterranean salt rock.

12.2.7 The Cheshire Brine Subsidence Compensation Board (CBSCB) will need to be consulted with regard to the drainage proposals for the BV. The CBSCB will be statutory consultees in the DCO process.

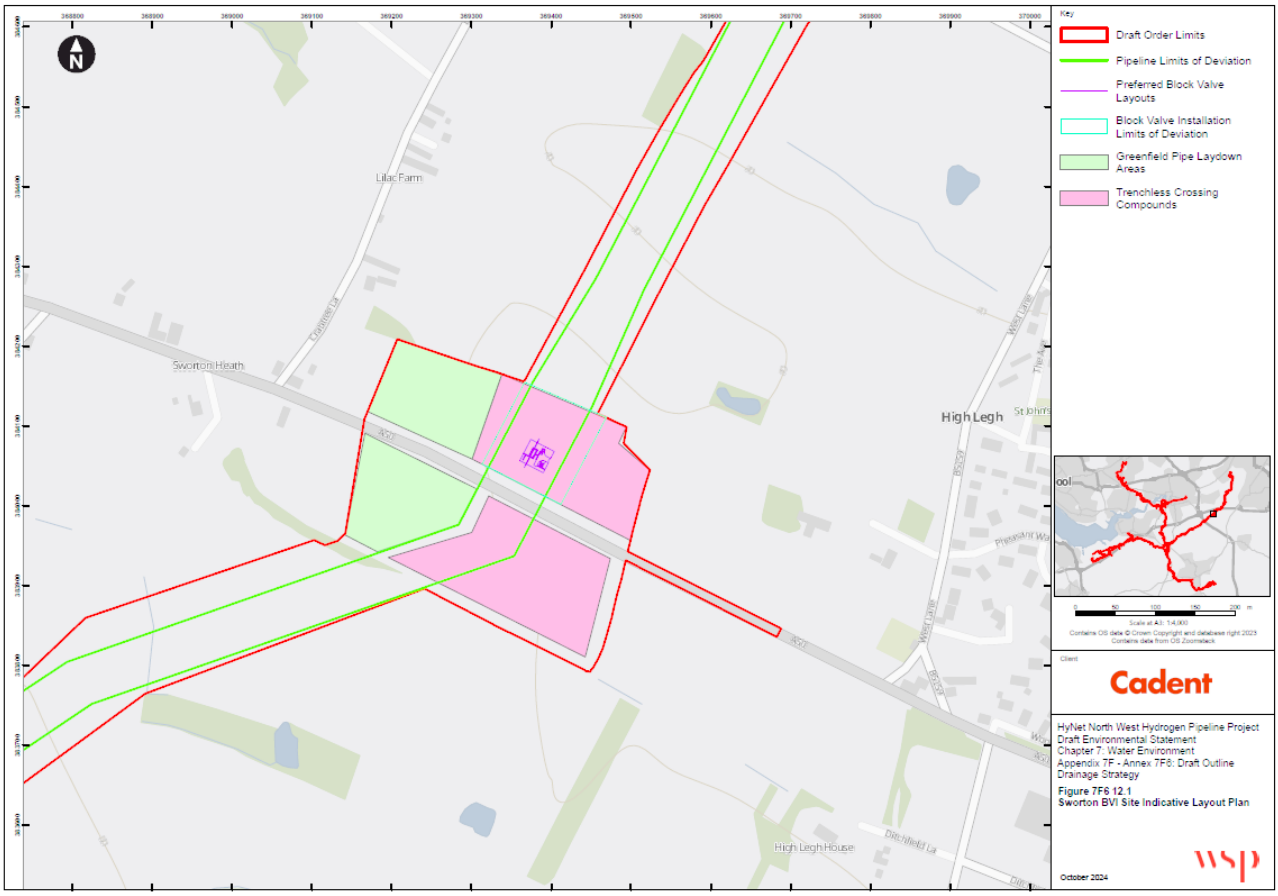
12.2.8 Should infiltration be shown to be a viable method of surface water disposal it will need to be considered in close consultation with the CBSCB.

12.3 Proposed Permanent Development

12.3.1 The proposed site is shown in **Figure 7F6 12.1** below. Note that the proposed construction compound area for the BVI site is the same area as the site itself, however the site is located within a trenchless crossing construction site (pink area) for the road crossing immediately south of the site.

12.3.2 The proposed BVI site covers a total area of approximately 1,000m^2 (0.1Ha).

Figure 7F6 12.1 Sworton Block Valve Installation Site Layout Plan



12.3.3 The Sworton BVI Indicative Drainage Layout Plan is included in Appendix A10. If infiltration is not viable and there are no accessible water courses to discharge to in the area, then there is a 750mm diameter United Utilities surface water sewer approximately 660m north of the site. Drainage from this site will probably need to be discharged to this sewer.

12.4 Allowable Site Discharge

12.4.1 The total site area of 0.10ha yields an equivalent greenfield runoff rate (Qbar) of 0.64l/s (using the calculated unit greenfield runoff rate of 6.43l/s/ha). As explained in section 2, this very low runoff rate will be uplifted to the minimum practical rate of 3l/s for the attenuation storage calculations for this BV.

12.4.2 Appendix B10 shows the Greenfield Runoff Rate calculation sheet using the HR Wallingford greenfield runoff tool with FEH 2013 rainfall data.

12.4.3 Surface water runoff from the proposed development will be restricted to this equivalent greenfield runoff rate.

12.4.4 It is proposed that the restricted discharge from the BV site is gravitated to connect into the 700mm diameter UU surface water sewer approximately 660m north of the site where the UU sewer crosses the Hydrogen pipeline route. The ground level at the proposed point of connection, according to Lidar data, is approximately 68mAOD.

12.4.5 An S106 connection will need to be agreed with UU during finalisation of the detailed drainage design.

12.4.6 If discharge under gravity is not possible then a pumped solution pumping at the agreed restricted rate will be required. It is proposed that this will be finalised during detailed design stage to be done post DCO consent.

12.5 Attenuation Provision

12.5.1 The overall area of the development site is 1,000m^2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths. The approximate impermeable area on the site is 200m^2. An estimated 800m2 will therefore be available as in-situ attenuation storage on the site.

12.5.2 It is proposed that the site platform will have an approximate surface level of 72.85mAOD.

12.5.3 The preliminary hydraulic calculations indicate that using a vortex flow control valve to restrict the discharge to the required greenfield flow (3l/s) will require an attenuation storage volume of approximately 50m^3, which translates to an equivalent 200mm depth of type 3 stone blanket to avoid flooding occurring during the critical 1 in 100-year + 45% climate change event. These hydraulic calculations are shown in Appendix C10.

12.5.4 The Indicative Drainage Layout for the permanent drainage from this BV is shown in Appendix A10.

12.6 Temporary Construction Compound Drainage

12.6.1 In addition to the BVI site itself, there is a proposed 2.5ha trenchless crossing and also a 1.4ha proposed pipe storage area lying to the west of the site. These are all shown to drain into attenuation basin A.

12.6.2 Across the main road the site the layout plan shows a 1.8ha proposed pipe storage area and a 2.25ha trenchless crossing compound draining to proposed attenuation basin B.

12.6.3 All the above temporary areas will require temporary drainage and associated attenuation storage for the duration of construction.

Allowable Site Discharge

12.6.4 Based on the calculated Qbar rate of 6.43l/s/ha noted in the previous section, the allowable discharge rates for all of the above areas will be:

- 25.1l/s from basin A, and
- 26.0l/s from basin B.

12.6.5 The combined discharge rate therefore during the construction stage will be approximately 51.1l/s discharging to the UU surface water sewer.

12.6.6 It is proposed that the outfall from these basins will discharge to the same UU surface water sewer described for the permanent scheme. This temporary discharge outlet will also need agreement with UU to utilise their existing sewer.

12.6.7 Should a gravity outfall not be feasible then a pumped solution for this outfall will be required. It is being considered whether this will be secured as a DCO requirement.

Attenuation Provision

12.6.8 Based on the allowable discharge rates noted above and the total areas of the compounds, preliminary hydraulic calculations show that the volume of attenuation storage required for these areas, to prevent flooding during 1 in 100-year (1%AEP) event, are as follows:

- 820m^3 for those areas draining into Basin A, and
- 850m^3 for those areas draining to basin B.

12.6.9 It is noted that these compounds and basins are temporary and will therefore be removed after construction.

12.6.10 The approximate locations and basin sizes, based on a 1.7m deep basin are shown on the indicative drainage layout in Appendix A10.

12.7 Existing Utilities in the Area

12.7.1 There is an overhead 11kV electric power line approximately 60m to the north of the BV site, travelling across the site and along the northern boundary of the temporary and trenchless crossing compounds.

12.7.2 Within the A50 reserve there are telecommunication overhead lines belonging to BT and underground cables belonging to Virgin media and Vodafone.

12.7.3 A 6 inch United Utilities water main also runs in the A50 main road reserve.

12.7.4 All the above utilities are shown on the layout plan in Appendix A10.

13 Hartford Block Valve Installation Site

13.1.1 This section of the report sets out the outline drainage strategy for the proposed Hartford Block Valve Installation, which is located approximately halfway along the southern corridor of the pipeline distribution network as shown in **Figure 7F6 1.1.**

13.2 Existing Site

13.2.1 The existing site is situated on existing agricultural land adjacent to the A556. The surrounding area is agricultural with the Sandiway golf course on the opposite side of the A556. There are scattered residential sites to the east of the site. Cemex Forrest Hill Quarry is situated approximately 600m west of the BVI site.

13.2.2 The approximate site grid reference is 361662E 371181N.

13.2.3 The topography of the site has been approximated from Lidar Contour data and this shows that the western corner is at approximately 39mAOD elevation rising to approximately 45m in the east corner. This is a fall of approximately 6m in 43m which equates to an average slope of approximately 1:7 across the site.

13.2.4 To create a level platform for the BVI, substantial earthworks will be required. See comment on this in the next section.

13.2.5 The British Geological Survey website indicates that the site is underlain by the Bollin Mudstone Member - Mudstone. Superficial deposits of Glaciofluvial Deposits, Devensian - Sand and gravel. The soils in this area are deep sandy well drained, and coarse loamy soils and are anticipated to have a relatively good permeability.

13.2.6 According to the Halite Risk Assessment this BVI site falls within the Cheshire Brine Compensation District. Although this HAGI site is within the compensation district, it is not in a “wet rockhead” area where the possibility of natural subsidence at the surface is greatest, through dissolution of the subterranean salt rock.

13.2.7 The Cheshire Brine Subsidence Compensation Board (CBSCB) will need to be consulted with regard to the proposals for the BV. The CBSCB will be statutory consultees in the DCO process.

13.2.8 Should infiltration be shown to be a viable method of surface water disposal it will need to be considered in close consultation with the CBSCB.

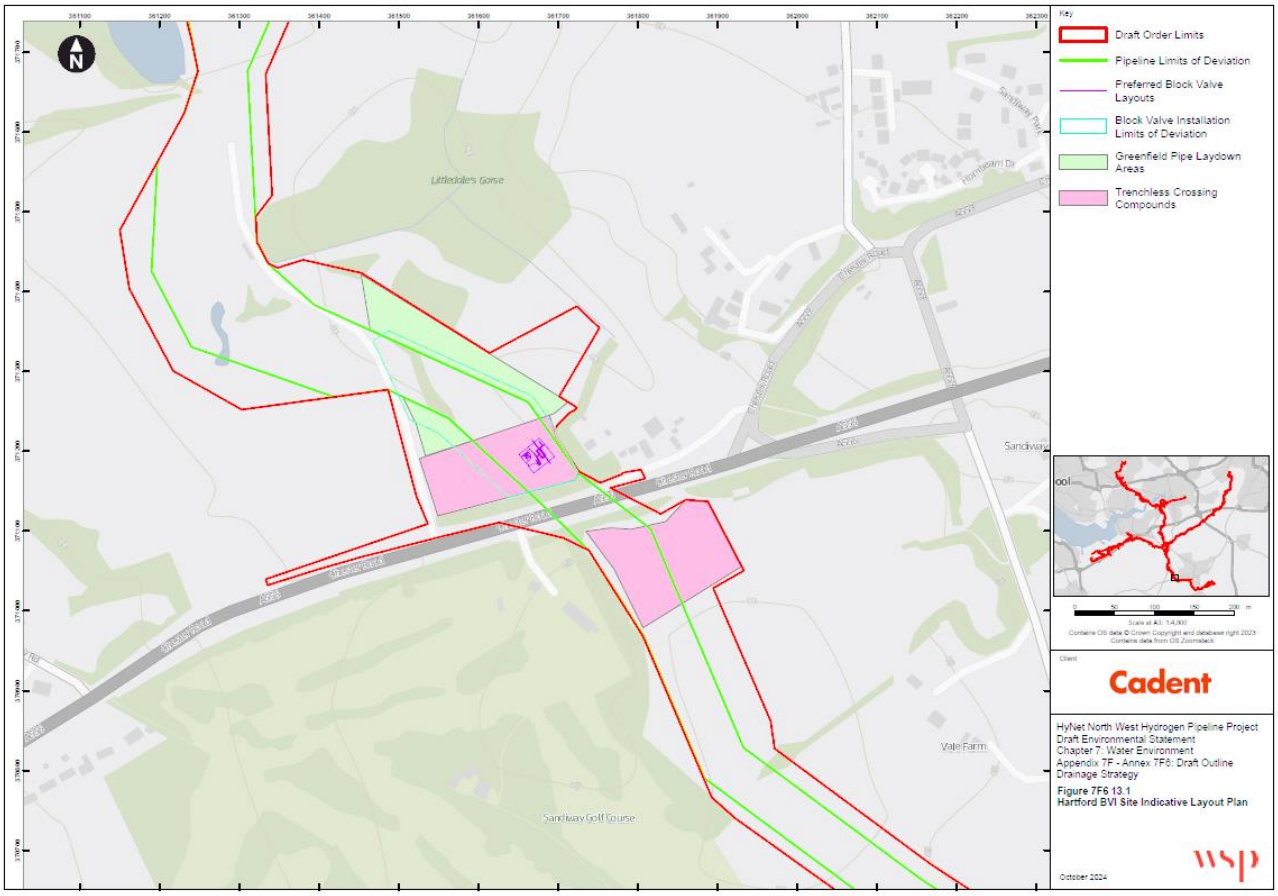
13.2.9 Information from the BGS website borehole logs within proximity to the site indicate that groundwater may be in the order of 17.4m to 22m below ground level.

13.3 Proposed Permanent Development

13.3.1 The proposed site is shown in **Figure 7F6 13.1** below. As can be seen the BVI site is located within a proposed trenchless crossing temporary construction compound (shown in pink) with two pipe laydown areas proposed to the north of the site.

13.3.2 The proposed Block Valve Installation (BVI) covers a total area of approximately 1,000m^2 (0.10Ha).

Figure 7F6 13.1 Hartford Block Valve Site Plan



13.3.3 The Hartford Option A Block Valve HAGI Indicative Drainage Layout Plan is included in Appendix A11. This also shows the location of an agricultural drainage ditch approximately 190m to the north of the site. If infiltration is not viable on this site, drainage will probably be discharged to this watercourse.

13.3.4 It is proposed that the BV site will have a platform level of approximately 41.38 mAOD. This will result in a cut embankment around the eastern corner of the site of an approximate height of 6m (nb. the DCO will allow for the micrositing of the HAGI which could allow significant earthworks to be avoided). A surface water cutoff berm will be required along the top edge of the embankment to divert surface water runoff around the site.

13.3.5 Discharge connection to the drainage ditch will need to be consented by the LLFA.

13.4 Allowable Site Discharge

13.4.1 The total site area of 0.10ha yields an equivalent greenfield runoff rate (Qbar) of 0.14l/s using the HR Wallingford Greenfield Runoff Tool with FEH 2013 rainfall data (unit greenfield runoff rate of 1.44l/s/ha). Due to the very low discharge

flowrate calculated, the flowrate will be uplifted to the practical minimum value of 3l/s for the attenuation storage calculations for this BV.

13.4.2 Appendix B11 shows the Greenfield Runoff Rate calculation sheet.

13.4.3 Surface water runoff from the proposed development will be restricted to this equivalent greenfield runoff rate.

13.4.4 It is proposed that the restricted discharge from the site is gravitated to the drainage ditch north of the site. If discharge under gravity is not possible then a pumped solution at the agreed restricted rate will be required. This will be finalised post DCO.

13.5 Attenuation Provision

13.5.1 The overall area of the development site is 1,000m2 however, some of this area will be occupied by impermeable area such as access roads and foundation plinths this area is estimated at 300m^2. Therefore, an estimated 700m^2 will be available as in-situ attenuation storage on the site.

13.5.2 The preliminary hydraulic calculations indicate that using a vortex flow control valve to restrict the discharge to the required greenfield flow (3l/s) will require an attenuation storage volume of approximately 45m^3 which translates to an equivalent 200mm depth of type 3 stone blanket to avoid flooding occurring during the critical 1 in 100-year + 45% climate change event. These hydraulic calculations are shown in Appendix C11.

13.5.3 The proposed Indicative Drainage Layout for the permanent drainage from this BV is included in Appendix A11.

13.6 Temporary Construction Compound Drainage

13.6.1 In addition to the BVI site, there is a 1.4ha trenchless crossing compound proposed together with two pipe laydown areas lying to the north of the site also totalling 1.4ha in area. These are all shown to drain into the attenuation basin along the northern boundary of the temporary construction site.

13.6.2 All the above temporary areas will require temporary drainage and associated attenuation for the duration of construction.

Allowable Site Discharge

13.6.3 Based on the calculated Qbar unit runoff rate of 1.44l/s/ha (uplifted to the practical minimum of 2l/s/ha as recommended in the HR Wallingford tool). The proposed greenfield runoff rate from the total area of 2.8ha will therefore be 5.6 l/s.

13.6.4 It is proposed that the outfall from this system will be to the same outfall as for the permanent drainage to the drainage ditch in the north. From preliminary observations it appears unlikely that this outfall will gravitate it will therefore require a pumped solution for the duration of the construction period. However, this will be finalised post DCO.

Attenuation Provision

13.6.5 Based on the allowable discharge flowrate noted above and the total area of the compounds and storage areas, the volume of attenuation required for the compound is estimated to be 810m^3 to prevent surface water flooding during the 1 in 100-year (1%AEP) event.

13.6.6 It is noted that as these compounds and attenuation basin are temporary, they will be removed after construction.

13.6.7 The approximate location and size of basin based on a 1.7m water depth is shown on the indicative drainage layout in Appendix A11.

13.7 Existing Utilities in the Area

13.7.1 There are no utilities shown below the footprint of the proposed BV site.

13.7.2 To the south of the site, within the road reserve of the A556 (Chester Road), there are overhead BT telecommunications lines and underground BT cables as well as several abandoned high and low voltage electrical power cables and high/ low voltage power lines.

13.7.3 There are live 11kV electrical underground cables in the middle and southern road reserves of the motorway, these are however well away from the BV site and should not impact on the construction work for the BV.

13.7.4 These utilities are shown on the layout plan in Appendix A11.

14 Greenfield Pipe Laydown Areas Along the Pipeline Route

14.1.1 In addition to the above HAGI and BVI sites there are 11 temporary greenfield pipe laydown sites identified along the pipeline route that are not associated with the individual HAGI or BVI sites and are of substantial footprint that will require a degree of temporary drainage infrastructure to control the surface water runoff from these areas.

14.1.2 The runoff from these areas have been assessed assuming the following overarching principles:

- The areas are assumed to be surfaced with an MOT type 1 gravel material and as previously assumed a 50% surface water runoff coefficient will apply.
- The runoff from these areas will be restricted to the greenfield runoff rates using similar unit rates to those calculated for the closest HAGI site.
- Attenuation volumes have been calculated in a similar manner to those of other temporary areas assuming no flooding occurs during the 1 in 100-year (1%AEP) event with no upliftment for climate change applied.
- Attenuation will generally be provided by means of an earth basin located on the site.
- In the absence of confirmed infiltration test results, it has initially been assumed that infiltration will not apply.

14.1.3 **Table 7F6 14.1** below shows a list of these sites with pertinent information for each.

Table 7F6 14.1 Summary of Greenfield Pipe Laydown Areas

Site Number	Eastings	Northings	Catchment Area (ha)	Unit Discharge rate (l/s/ha)	Discharge Rate (l/s)	Attenuation Volume (m^3)
1	354056	379264	2.7	1.9	5.1	810
2	366454	368805	1.9	1.9	3.6	600
3	345900	374926	2.4	4.4	10.6	555
4A	358773	380312	3.9	4.9	19.1	910
4B	358554	380234	3.6	4.9	17.6	840
5A	359510	386771	1.3	2.4	3.1	355

Site Number	Eastings	Northings	Catchment Area (ha)	Unit Discharge rate (l/s/ha)	Discharge Rate (l/s)	Attenuation Volume (m^3)
5B	359538	386543	1.2	2.4	Uplifted to 3	325
6	371495	386437	2.9	2.4	7.0	785
7	350806	390770	2.7	7.6	20.5	480
8	351881	389708	3.7	7.6	28.1	655
9A	354559	387821	2.8	2.4	6.7	765
9B	354789	387597	1.4	2.4	3.4	380
9C	354829	387530	2.6	2.4	6.2	710
9D	354768	387299	6.6	2.4	15.8	1800
10	360088	375590	2.3	4.9	11.3	540
11	360647	372806	2.5	4.9	11.5	595

14.1.4 Appendices E and F include illustrative drainage layout plans and hydraulic calculations respectively for all the above pipe laydown sites.

14.1.5 Outfall locations will be finalised post DCO.

15 Other Drainage Strategy Principles for Temporary Construction Areas

15.1.1 In addition to the temporary construction areas related to HAGIs, BVIs and greenfield pipe laydown areas as covered in the previous sections there are other temporary areas which will require a strategy to guide the construction activities.

15.1.2 Presently there is a single brownfield pipe laydown area proposed on the existing hardstanding yard of Gibbs Surfacing, off Burtonhead Road in St Helens. It is proposed that the drainage of this area will utilise the existing drainage system and continue to operate as existing during the construction period while pipes are stored on this site.

15.1.3 The following principles apply in general to construction along the length of the pipeline route.

15.2 Temporary Areas Subject to Soil Stripping and Excavation for Pipeline Construction

15.2.1 In addition to management of surface runoff from the pipeline construction swathe, there will also be areas where dewatering will be required from excavations for pipeline construction, HDD/micro tunnelling launch and reception pits for the crossing of major watercourses and linear infrastructure such as roads and railways.

15.2.2 Discharge from dewatering operations could include attenuation and infiltration into neighbouring ground, piping to a ditch or watercourse and/or tankering away and disposal at a remote site.

15.2.3 If dewatering entails pumping to a nearby watercourse, then the rate of discharge will be determined by the estimated volume of water to be dewatered, the available pumping equipment and the construction operations in the area of dewatering.

15.2.4 Discharge to the watercourse will occur via an appropriate sediment trap to ensure no silt is transported downstream into the watercourse. Application of embedded mitigation measure C8 (detailed in table 7.16 of Draft ES **Chapter 7: Water Environment**) will ensure the reduction of silt being transported to the existing water courses.

15.2.5 At this stage there is insufficient information available to accurately predict the amount of dewatering required at each relevant area of excavation or crossing and therefore a quantitative assessment of this is not possible and it will be provided post consent and prior to construction. It is being considered whether this will be secured via a DCO requirement.

15.2.6 Areas of effective infiltration along the pipeline route will be used as infiltration basins available as discharge points for the dewatering operations.

15.2.7 Areas of shallow groundwater along the pipeline route where groundwater flow has the potential to interact with the Hynet pipeline subsurface infrastructure have been identified in the baseline conditions within Draft ES **Chapter 7: Water Environment**. Works within the Draft Order Limits, such as pipeline installation have the potential to create barriers to groundwater flow or act as areas for preferential pathways. These areas are listed and screened in Table 7B.8 of Appendix 7B of the Draft ES **Chapter 7: Water Environment**.

15.2.8 Embedded environmental mitigation measures C5 and D5 (detailed in table 7.16 of Draft ES **Chapter 7: Water Environment** will apply in these instances.

15.2.9 Full details of these arrangements will set out in the Outline CEMP pre-submission and finalised post DCO consent.

15.3 Crossing Field Drainage Ditches or Small Watercourses Using Dry Open Cut Method

15.3.1 Most minor watercourse/ditch crossings may be carried out using a dry open cut trench methodology. In dry open cut methods water flow is maintained by damming and over pumping or using temporary “flume” pipes installed in the bed of the watercourse. Details of these methods are outlined below.

15.3.2 The site is first prepared by stripping the topsoil from the banks and areas adjacent to the river at the crossing point and storing it separately within the working area, taking care to exercise appropriate erosion prevention measures to avoid silt wash into the watercourse. If practicable the bed material and a selection of vegetation is stored, for replacement after the pipeline has been laid.

15.3.3 For dry open cut watercourses / ditch crossings a suitably sized flume pipe will be installed over the point of the proposed crossing ensuring that it extends on each side of the main pipeline trench line crossing point for a suitable distance. The flume pipe will then be bedded and packed or surrounded with soil filled sandbags to create a seal or dam across the watercourse, so that the flow is diverted through the flume pipes.

15.3.4 Excavation of the pipeline trench then proceeds beneath the flume pipe. The excavated material will be stored within the working width separately from the bank material.

15.3.5 If damming and over pumping method is adopted then soil filled sandbags can be used to create a seal or dam across the watercourse. However, a pump will be set up to take the flow from upstream to downstream of the crossing point. The discharge hose will be directed through a filtering medium to limit silt carry over or bed disturbance before the pumped water is returned to the watercourse.

15.3.6 After installation and backfilling of the hydrogen pipeline, the banks of the watercourse/drainage ditch will be reformed to their original profile in accordance with the EA / LLFA and the landowners’ requirements.

15.3.7 The flume pipe and packing or bags are removed once the bed materials and bank profile is reinstated.

15.3.8 The only exception to this might be locations where the water environment is particularly sensitive or of high value and where more detailed, site specific consideration may be required. These sites are identified in **Appendix 7B: Detailed Water Environment Baseline Information** to Draft ES **Chapter 7: Water Environment**. Additional site specific mitigation measures may be identified for these locations as appropriate for inclusion within the Outline CEMP.

15.3.9 In addition to the scrutiny afforded via the DCO application process, all watercourse crossings would be subject to the appropriate permit (either a Flood Risk Activities Permit from the Environment Agency for Main Rivers, or a Land Drainage Consent from the LLFA for Ordinary Watercourses). These permit applications would contain site specific detail for each crossing and would normally be applied for by the contractor post DCO consent.

15.4 Treatment of Agricultural Field Drains

15.4.1 To minimise impacts on surrounding agricultural land particular care will be taken to ensure that the existing land drainage regime is not compromised as a result of construction. Existing land drainage systems will be maintained and diverted as necessary during construction and reinstated on completion.

15.4.2 Field drains are often unrecorded but are an asset of high importance to landowners. It is therefore important to make every effort to, in consultation with landowners, confirm locations of field drainage systems, mark the existing positions of these for reinstatement after installation of the pipeline has been completed in the particular area.

15.4.3 It is the preference that the land drains are discontinued during pipeline installation to be reinstated on completion.

15.4.4 Unidentified field drains broken during the trenching operation will be physically marked for reconnection and reinstatement after completion of the pipeline laying operation in that area.

15.4.5 Reinstatement of the agricultural land drainage systems will be a requirement of the DCO.

15.4.6 The Outline CEMP for construction should therefore establish the overall approach to the diversion and reinstatement of agricultural land drainage.

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Delivering clean growth

Appendix A
Indicative Drainage Layout Plans

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 10.820 mAoD

7645m² OF POROUS STONE BLANKET TO PROVIDE 690m³ OF ATTENUATION AT 300mm DEPTH

DISCHARGE TO AGREED CONNECTION POINT ON EXISTING SITE DRAINAGE SYSTEM

FULL BORE FLOW THROUGH 0.225M PIPE TO DISCHARGE AT 58l/s

PROPOSED FILTER TRENCH WITHIN STONED AREA

A5117 HIGHWAY

11.00
10.50
10.00
9.50
9.00
8.50
8.00

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — Scale 1:500 @ A1 — 30 m

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HyNet Drainage Strategy Report

Appendix A1
Indicative Permanent HAGI Drainage Layout - Stanlow HPP

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FULL BORE THROUGH
0.225m PIPE TO DISCHARGE
AT 99.5l/s

TEMPORARY
DISCHARGE FROM
HAGI SITE DURING
CONSTRUCTION

CATCHMENT AREA = 1.69 Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO
STORE 185m³

DISCHARGE TO AGREED
CONNECTION POINT ON
EXISTING SITE DRAINAGE
SYSTEM

CONNECTION TO
PROPOSED
PERMANENT
DRAINAGE

A5117 HIGHWAY

10.50
10.00
9.50
9.00
8.50
8.00

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
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0 m — Scale 1:500 @ A1 — 30 m

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HyNet Drainage Strategy Report

Appendix XX
Indicative Temporary HAGI Drainage
Layout - Stanlow HPP

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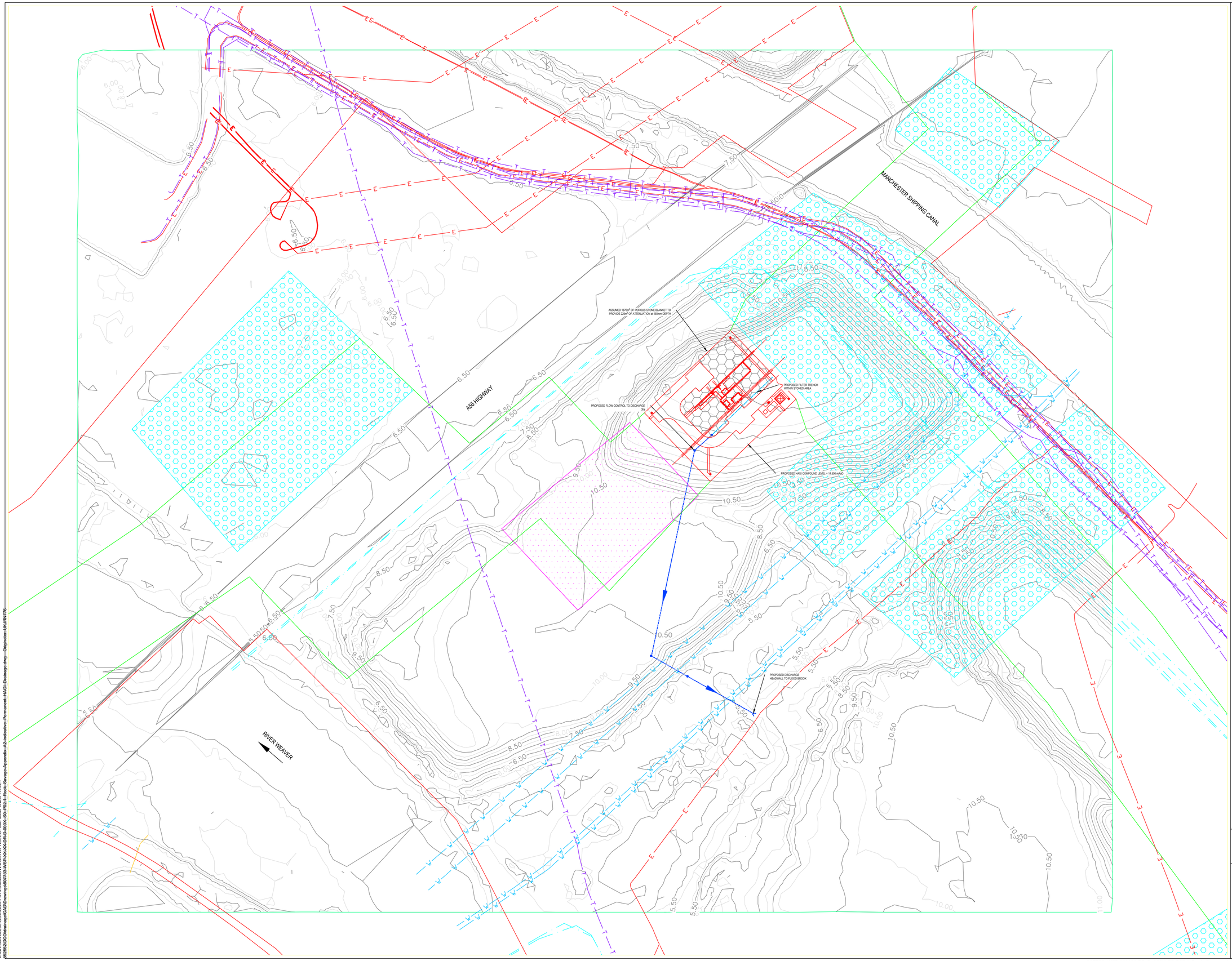

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m
Scale 1:1000 @ A1

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HyNet Drainage Strategy Report

Appendix A2
Indicative Permanent HAGI Drainage Layout - Rock Savage

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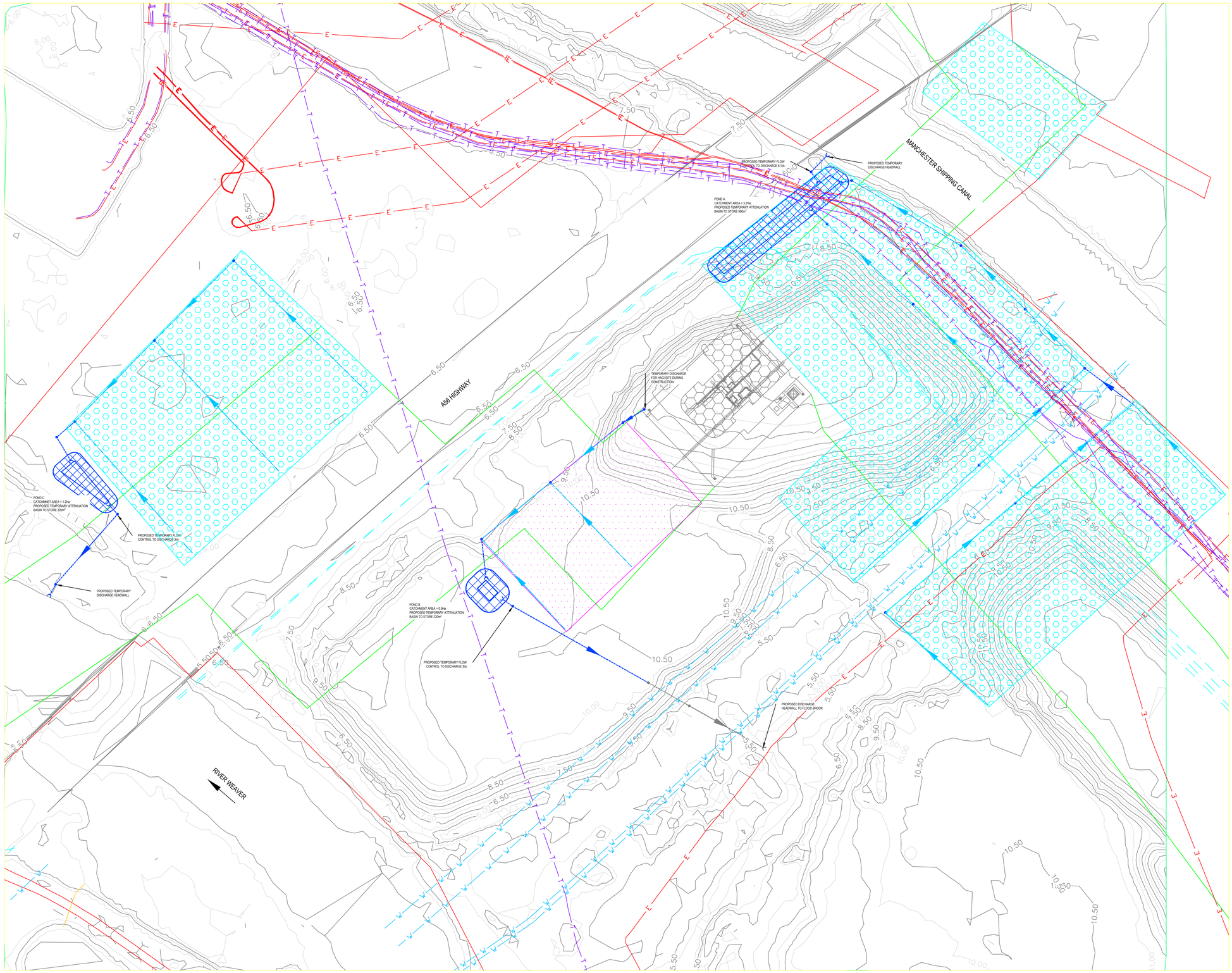

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
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- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1000 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix XX
IndicativeTemporary HAGI Drainage Layout - Rock Savage

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SABIC UK
PETROCHEMICALS
NORTHWEST PIPELINE

INDICATIVE POSITION OF
SURFACE WATER CUT OFF
BERM TO DIVERT OVERLAND
FLOW AROUND THE HAGI SITE

6060m² OF POROUS STONE
BLANKET TO PROVIDE 420m³ OF
ATTENUATION AT 300mm DEPTH

PROPOSED HAGI
COMPOUND ESTIMATED
APPROXIMATE
LEVEL =32.000 mAoD

PROPOSED FILTER TRENCH
WITHIN STONED AREA

EXISTING FOUL
SEWER TO BE
RELOCATED

CATCHPIT

PROPOSED FLOW
CONTROL TO
DISCHARGE 3.8l/s

PROPOSED DISCHARGE TO CONNECT
INTO EXISTING CADENT AGI SYSTEM.
LOCATION TO BE DETERMINED.

45.00
40.00
35.00
30.00

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 25 m
Scale 1:500 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A3
Indicative Permanent HAGI Drainage
Layout - Runcorn

August 2024

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POND A
CATCHMENT AREA = 1.6 Ha
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 400m³

PROPOSED FLOW CONTROL TO DISCHARGE 5l/s

SABIC UK PETROCHEMICALS NORTHWEST PIPELINE

EXISTING FOUL SEWER TO BE RELOCATED

PROPOSED TEMPORARY DISCHARGE FROM HAGI SITE DURING CONSTRUCTION

POND B
CATCHMENT AREA = 1.2 Ha
PROPOSED TEMPORARY STORAGE SWALE TO COLLECT RUNOFF FROM HAGI CONSTRUCTION SITE AND TO STORE 300m³

CONNECTION TO PROPOSED PERMANENT DRAINAGE

PROPOSED FLOW CONTROL TO DISCHARGE 8.8l/s

PROPOSED DISCHARGE TO CONNECT INTO EXISTING CADENT AGI SYSTEM. LOCATION TO BE DETERMINED.

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permanent Surface Water Drain
- E Electric Lines
- W Water Lines
- T Telecom Lines
- SS Sewer Lines
- Pipelines
- GAS Gas Lines

0 m — 40 m
Scale 1:750 @ A3

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HyNet Drainage Strategy Report

Appendix AX
Indicative Temporary HAGI Drainage Layout - Runcorn

August 2024

wsp

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SABIC UK PETROCHEMICALS
NORTH WEST PIPELINE

ASSUMED 8925m² OF POROUS STONE
BLANKET TO PROVIDE 1200m³ OF
ATTENUATION AT 450mm DEPTH

PROPOSED FLOW
CONTROL TO
DISCHARGE 7.8l/s

PROPOSED HAGI
COMPOUND ESTIMATED
APPROXIMATE LEVEL =
47.360 mAoD

PROPOSED FILTER
TRENCH WITHIN
STONED AREA

PROPOSED HEADWALL
TO DRAINAGE DITCH

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 50 m
Scale 1:1,250 @ A1

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HyNet Drainage Strategy Report

Appendix A4
Indicative Permanent HAGI Drainage Layout - Central Hub

August 2024

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SABIC UK PETROCHEMICALS
NORTH WEST PIPELINE

TEMPORARY DISCHARGE FOR
HAGI SITE DURING
CONSTRUCTION

LINK TO PROPOSED PERMANENT
DISCHARGE PIPE

PROPOSED TEMPORARY FLOW
CONTROL TO DISCHARGE 16.1l/s

PROPOSED HEADWALL
TO DRAINAGE DITCH

POND A
CATCHMENT AREA = 3.45 Ha
PROPOSED TEMPORARY ATTENUATION
BASIN TO STORE 820m^3

POND B
CATCHMENT AREA = 1.78 Ha
PROPOSED TEMPORARY ATTENUATION
BASIN TO STORE 425m^3

PROPOSED TEMPORARY FLOW
CONTROL TO DISCHARGE 8.3l/s

DISCHARGE TO ASSUMED WATERCOURSE

POND C
CATCHMENT AREA = 9.07 Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO STORE
2150m^3

DISCHARGE TO ASSUMED WATERCOURSE

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permenant Surface Water Drain
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m
Scale 1:1000 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A4
Indicative Temporary HAGI Drainage
Layout - Central Hub

August 2024

wsp

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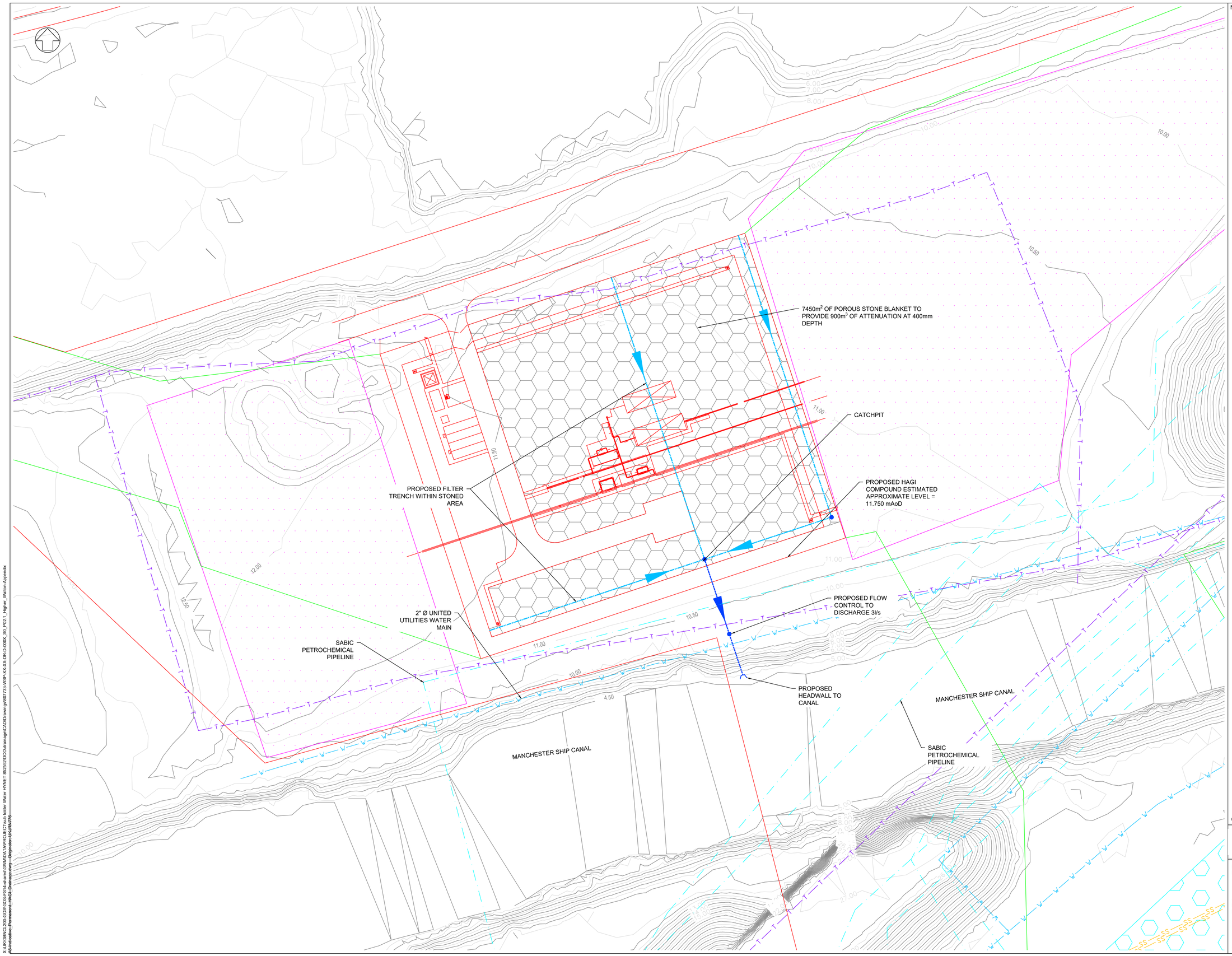

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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HyNet Drainage Strategy Report

Appendix A5
Indicative Permanent HAGI Drainage Layout - Higher Walton 2

August 2024

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POND A
CATCHMENT AREA = 1.5Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO STORE 410m^3

PROPOSED
TEMPORARY
PACKAGE PUMP TO
DISCHARGE AT 3.6l/s

SABIC
PETROCHEMICAL
PIPELINE

TEMPORARY
DISCHARGE FOR
HAGI SITE DURING
CONSTRUCTION

LINK TO PROPOSED
PERMANENT DISCHARGE

PROPOSED HEADWALL TO
CANAL TO BE THE
PERMANENT DISCHARGE
POSITION FOR THE
OPERATIONAL SCHEME.
TO TEMPORARILY
DISCHARGE AT 7.6l/s

2" Ø UNITED
UTILITIES WATER
MAIN

PROPOSED FLOW CONTROL
TO DISCHARGE 4/s

MANCHESTER SHIP CANAL

POND B
CATCHMENT AREA = 1.7Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO STORE
470m^3

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permenant Surface Water Drain
- Proposed Temporary Surface Water Rising Main
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m
Scale 1:500 @ A1

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HyNet Drainage Strategy Report

Appendix A5
Indicative Temporary HAGI Drainage Layout - Higher Walton 2

August 2024

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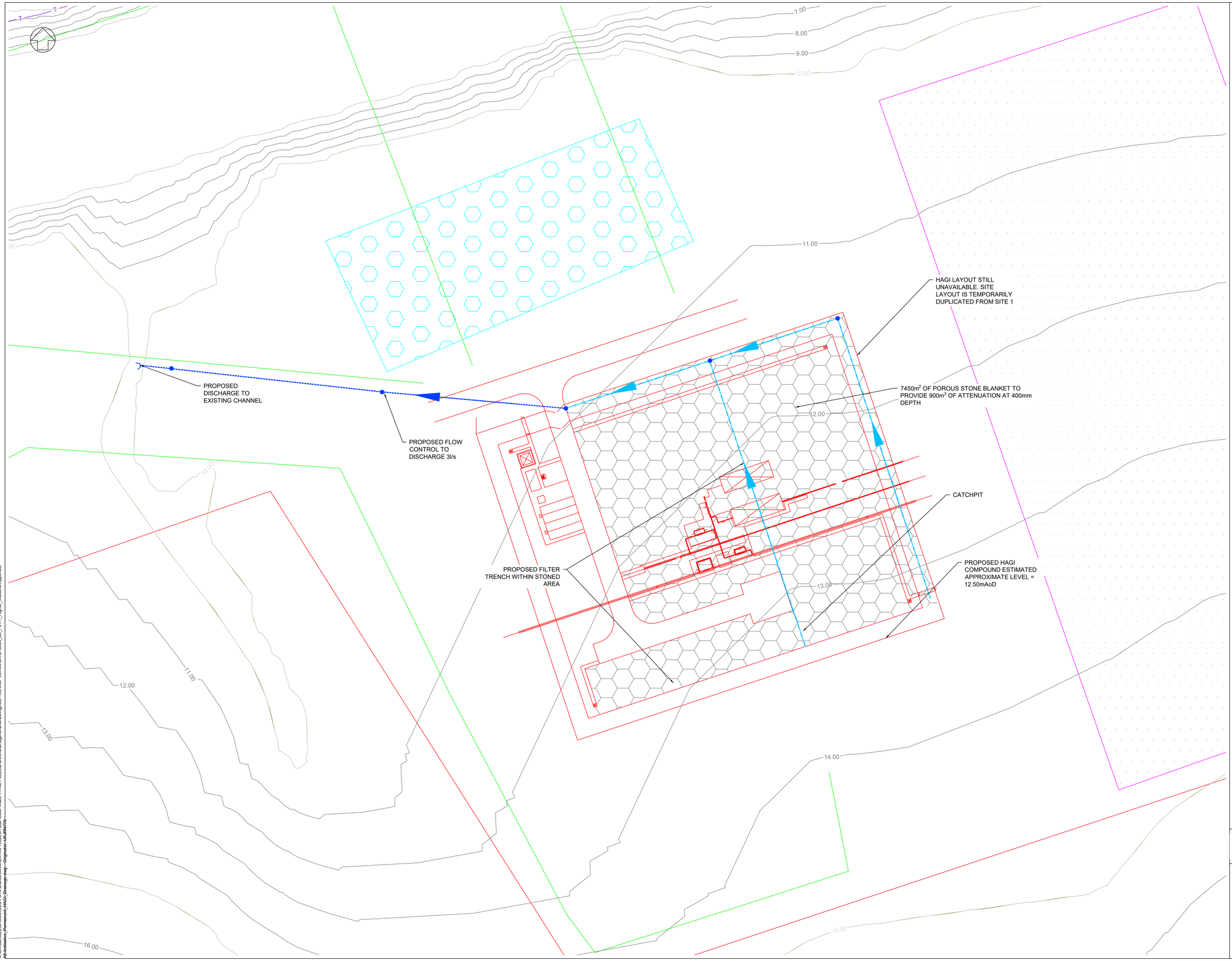

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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HyNet Drainage Strategy Report

Appendix A5
Indicative Permanent HAGI Drainage Layout - Higher Walton 8

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September 2024

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PROPOSED FLOW CONTROL TO DISCHARGE 8.3l/s

LINK TO PROPOSED PERMANENT DISCHARGE PIPE

CATCHMENT AREA = 3.5 Ha
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 960m^3

PROPOSED DISCHARGE TO EXISTING CHANNEL

TEMPORARY DISCHARGE FROM HAGI SITE DURING CONSTRUCTION

HAGI LAYOUT STILL UNAVAILABLE. SITE LAYOUT IS TEMPORARILY DUPLICATED FROM SITE 1

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permenant Surface Water Drain
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 50 m

Scale 1:500 @ A1

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HyNet Drainage Strategy Report

Appendix A5
Indicative Temporary HAGI Drainage Layout - Higher Walton 8

September 2024

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M62 HIGHWAY

DISCHARGE TO EXISTING CULVERT HEADWALL TO DRAIN UNDER MOTORWAY TO LAND DRAIN ON NORTHERN SIDE OF MOTORWAY

PROPOSED FLOW CONTROL TO DISCHARGE 9.1l/s

CATCHPIT

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 56.50 mAoD

5220m^2 OF POROUS STONE BLANKET TO PROVIDE 700m^3 OF ATTENUATION AT 450mm DEPTH

PROPOSED FILTER TRENCH WITHIN STONED AREA

B5419 HIGHWAY

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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HyNet Drainage Strategy Report

Appendix A6
Indicative Permanent HAGI Drainage - Clockface

August 2024

wsp

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DISCHARGE TO EXISTING CULVERT HEADWALL TO DRAIN UNDER MOTORWAY TO LAND DRAIN ON NORTHERN SIDE OF MOTORWAY

M62 HIGHWAY

PROPOSED TEMPORARY FLOW CONTROL TO DISCHARGE 68.6l/s

CATCHMENT AREA = 9 Ha. PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 1600m³

TEMPORARY DISCHARGE FOR HAGI SITE DURING CONSTRUCTION

B5419 HIGHWAY

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permanent Surface Water Drain
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1000 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix XX
Indicative Temporary HAGI Drainage - Clockface

August 2024

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CARRGREEN LANE

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 24.500 mAoD

DUNHAM ROAD

5580m² OF POROUS STONE BLANKET TO PROVIDE 755m³ OF ATTENUATION AT 450mm DEPTH

PROPOSED FILTER TRENCH WITHIN STONED AREA

CATCHPIT

PROPOSED FLOW CONTROL TO DISCHARGE 3l/s

TRANS PENNINE TRAIL

TRANS PENNINE TRAIL

REFER TO PLAN

PROPOSED OUTFALL HEADWALL DISCHARGE TO RIVER BOLLIN

REFER TO INSET

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 50 m

Scale 1:1000 @ A1

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Appendix A7
Indicative Permanent HAGI Drainage Layout - Warburton

August 2024

wsp

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CARRGREEN LANE

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 24.500 mAoD

PROPOSED FILTER TRENCH WITHIN STONED AREA

DUNHAM ROAD

TEMPORARY DISCHARGE FOR HAGI SITE DURING CONSTRUCTION

CATCHMENT AREA = 1.72Ha
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 470m³

PROPOSED TEMPORARY FLOW CONTROL TO DISCHARGE AT 4.1l/s

TEMPORARY NETWORK TO TIE INTO PROPOSED PERMANENT DRAINAGE

TRANS PENNINE TRAIL

TRANS PENNINE TRAIL

REFER TO PLAN

REFER TO INSET

PROPOSED OUTFALL HEADWALL DISCHARGE TO RIVER BOLLIN

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Permanent Surface Water Drain
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 50 m
Scale 1:1000 @ A1

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Appendix A7
Indicative Temporary HAGI Drainage Layout - Warburton

August 2024

wsp

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PROPOSED DISCHARGE TO EXISTING SITE

PROPOSED FLOW CONTROL TO DISCHARGE 10.4l/s

3155 m² OF POROUS STONE BLANKET TO PROVIDE 660m³ OF ATTENUATION at 700mm DEPTH

PROPOSED HAGI COMPOUND LEVEL = 19.700 mAoD

CATCHPIT

PROPOSED FILTER TRENCH WITHIN STONED AREA

20.00

25.00

20.00

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A8
Indicative Permanent HAGI Drainage Layout - Partington 1

September 2024

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Originator: LK-LRN776

PROPOSED DISCHARGE TO EXISTING SITE

PROPOSED FLOW CONTROL TO DISCHARGE 22.6l/s

CATCHMENT AREA = 2.2Ha. PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 470m³

TEMPORARY DISCHARGE FROM HAGI SITE DURING CONSTRUCTION

20.00

20.00

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 30 m

Scale 1:500 @ A1

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HyNet Drainage Strategy Report

Appendix XX
Indicative Temporary HAGI Drainage Layout - Partington 1

wsp

September 2024

807733-WSP-XX-XX-DR-D-000X_S0_P01.1_Partington1-Appendix_XX-Indicative_Temporary_HAGI_Drainage

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Plotted on: 04/09/2024 09:47:16

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 20.50mAoD

HAGI LAYOUT STILL UNAVAILABLE. SITE LAYOUT IS TEMPORARILY DUPLICATED FROM SITE 1

3160m^2 OF POROUS STONE BLANKET TO PROVIDE 810m^3 OF ATTENUATION at 850mm DEPTH

PROPOSED FILTER TRENCH WITHIN STONED AREA

PROPOSED DISCHARGE TO DRAINAGE DITCH

CATCHPIT

PROPOSED FLOW CONTROL TO DISCHARGE 4.2l/s

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A8
Indicative Permanent HAGI Drainage Layout - Partington 2

wsp

September 2024

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Engineer: UK-IRN776

HAGI LAYOUT STILL UNAVAILABLE. SITE LAYOUT IS TEMPORARILY DUPLICATED FROM SITE 1

TEMPORARY DISCHARGE FROM HAGI SITE DURING CONSTRUCTION

CATCHPIT

PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 550m³

PROPOSED FLOW CONTROL TO DISCHARGE 8.4l/s

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 45 m

Scale 1:750 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix XX
Indicative Temporary HAGI Drainage Layout - Partington 2

August 2024

wsp

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Partington, UK, RN176

4140m² OF POROUS STONE BLANKET TO PROVIDE 690m³ OF ATTENUATION AT 550mm DEPTH

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 32.180 mAoD

CATCHPIT

PROPOSED FILTER TRENCH WITHIN STONED AREA

PROPOSED FLOW CONTROL TO DISCHARGE 5.6l/s

PUDDINGLAKE BROOK

PROPOSED HEADWALL TO DITCH

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 30 m

Scale 1:500 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A9
Indicative Permanent HAGI Drainage - Hydrogen Storage

August 2024

wsp

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POND C
CATCHMENT AREA = 1.8 Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO
STORE 410m³

PROPOSED TEMPORARY PACKAGE PUMP
TO DISCHARGE 11.8l/s

POND D
CATCHMENT AREA = 4Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO
STORE 920m³

PROPOSED FLOW CONTROL
TO DISCHARGE 37.2l/s

PROPOSED TEMPORARY
DISCHARGE POINT

POND B
CATCHMENT AREA = 7.1Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO
STORE 1620m³

PROPOSED FLOW
CONTROL TO
DISCHARGE 45.1l/s

POND A
CATCHMENT AREA = 2.6Ha
PROPOSED TEMPORARY
ATTENUATION BASIN TO
STORE 600m³

PROPOSED FLOW CONTROL
TO DISCHARGE 61.4l/s

CONNECTION TO PROPOSED
PERMANENT DRAINAGE OUTFALL

PUDDINGLAKE BROOK

A530

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- **Pipe Dump locations**
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permenant Surface Water Drain
- Proposed Temporary Surface Water Rising Main
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 100 m
Scale 1:2,000 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix AX
Indicative Temporary HAGI Drainage - Hydrogen Storage

August 2024

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REFER TO INSET

DRAINAGE TO CONNECT TO EXISTING SEWER SHOULD INFILTRATION PROVE NON-VIABLE. TO BE OVERSIZED TO ACCOMODATE TEMPORARY DISCHARGE RATE.

PROPOSED FLOW CONTROL TO DISCHARGE 3.0l/s

PROPOSED FILTER TRENCH WITHIN STONED AREA

800m² OF POROUS STONE BLANKET TO PROVIDE 50m³ OF ATTENUATION AT 200mm DEPTH

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 72.85 mAoD

A50 HIGHWAY

69.00

70.00

71.00

72.00

REFER TO PLAN

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m 30 m

Scale 1:500 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A10
Indicative Permanent Drainage Layout - Swarton Block Valve

August 2024

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2655\GOS\Drainage\CAD\Drawings\807733-WSP-XX-XX-DR-D-000X_S0_P02.1_EBV3-Appendix_A10-Indicative_Permanent_HAGI_Drainage.dwg — Originator: UK-IRN776

POND A
CATCHMENT AREA = 3.9Ha
PROPOSED TEMPORARY
STORAGE BASIN TO STORE
820m^3

PROPOSED TEMPORARY PACKAGE PUMP
MAY BE REQUIRED SHOULD INFILTRATION
BE PROVEN NON-VIABLE TO DISCHARGE
25.1l/s

PROPOSED CONNECTION
TO PERMANENT
DRAINAGE

DRAINAGE TO CONNECT
TO EXISTING SEWER
SHOULD INFILTRATION
PROVE NON-VIABLE

PROPOSED TEMPORARY PACKAGE PUMP
MAY BE REQUIRED SHOULD INFILTRATION
BE PROVEN NON-VIABLE TO DISCHARGE
26l/s

POND B
CATCHMENT AREA = 4.05Ha
PROPOSED TEMPORARY
STORAGE BASIN TO STORE
850m^3

A50 HIGHWAY

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permanant Surface Water Drain
- Proposed Temporary Surface Water Rising Main
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 40 m
Scale 1:750 @ A3

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Appendix AX
Indicative Temporary Drainage Layout - Swarton Block Valve

August 2024

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PROPOSED HEADWALL TO WATERCOURSE

PROPOSED HAGI COMPOUND ESTIMATED APPROXIMATE LEVEL = 41.380 mAoD

PROPOSED FILTER TRENCH WITHIN STONED AREA

PROPOSED FLOW CONTROL TO DISCHARGE 3.0l/s

700m² OF POROUS STONE BLANKET TO PROVIDE 45m³ OF ATTENUATION AT 200mm DEPTH

A556 HIGHWAY

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.
2. 5M OUTSIDE OF SECURITY FENCE SHOULD BE CLEARED AND MAINTAINED FREE OF ALL UNNECESSARY VEGETATION.
3. MINIMUM PIPELINE COVER = 1200MM.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 45 m

Scale 1:750 @ A1

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HyNet Drainage Strategy Report

Appendix A11
Indicative Permanent HAGI Drainage Layout - Hartford Block Valve

August 2024

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2024\05\06\Drawings\807733-WSP-XX-XX-DR-D-0000X_S0_P02.1_SBV-Appendix_A11_Indicative_Permanent_HAGI_Drainage.dwg Originator: UKJRN776

PROPOSED TEMPORARY PACKAGE PUMP
MAY BE REQUIRED SHOULD INFILTRATION BE
PROVEN NON-VIABLE TO DISCHARGE 5.6l/s

CATCHMENT AREA - 2.8Ha.
PROPOSED TEMPORARY TEMPORARY
ATTENUATION BASIN TO STORE 810m^3

PROPOSED HEADWALL TO WATERCOURSE

CONNECTION TO PROPOSED
PERMANENT DRAINAGE

A556 HIGHWAY

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- Proposed Porous Blanket (TYPE - 3 Granular Material)
- Proposed Permanent Surface Water Drain
- Proposed Temporary Surface Water Rising Main
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m
Scale 1:500 @ A1

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HyNet North West Hydrogen Pipeline Project
HyNet Drainage Strategy Report

Appendix A11
Indicative Temporary HAGI Drainage
Layout - Hartford Block Valve

August 2024

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HyNet North West Hydrogen Pipeline

Delivering clean growth

Appendix B
Greenfield Runoff Calculations

Brownfield Runoff Rate

The existing ‘brownfield’ runoff rate has been estimated using the BS EN 752 calculation method and assuming a rainfall intensity of 50mm/hr.

The impermeable area of the original site was measured at a total area of 1.9 ha. The runoff from this area would be approximately 2.781 x 1.9 ha x 50mm/h = 264l/s - this would approximate the brown field runoff rate from the existing site.

Proposed discharge rate

To offer a 30% degree of betterment on the existing situation, a surface water discharge rate of 70% of the brownfield rate i.e., 264 x 0.7 = 185 l/s, is proposed as the restricted flow from the site to the public system.

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Rocksavage
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	3499645947
Date:	Aug 01 2024 14:56

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.704
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	0762
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		1.9
1 in 1 year (l/s):		1.62
1 in 30 years (l/s):		3.71
1 in 100 year (l/s):		4.72
1 in 200 years (l/s):		5.41

This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this tool are estimates of greenfield runoff rates. The use of these results is the responsibility of the users of this tool. No liability will be accepted by HR Wallingford, the Environment Agency, CEH, Hydrosolutions or any other organisation for the use of this data in the design or operational characteristics of any drainage scheme.

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Runcorn
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	3412945253
Date:	Aug 01 2024 14:32

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.589
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	776
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		3.14
1 in 1 year (l/s):		2.67
1 in 30 years (l/s):		6.13
1 in 100 year (l/s):		7.8
1 in 200 years (l/s):		8.93

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Central Hub
Site location:	

Site Details

Latitude:	
Longitude:	
Reference:	2146381471
Date:	Aug 01 2024 14:12

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.469
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	0790
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		4.85
1 in 1 year (l/s):		4.12
1 in 30 years (l/s):		9.46
1 in 100 year (l/s):		12.03
1 in 200 years (l/s):		13.78

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Higher Walton
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	911793276
Date:	Aug 01 2024 14:21

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.69
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	816
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		2.38
1 in 1 year (l/s):		2.02
1 in 30 years (l/s):		4.64
1 in 100 year (l/s):		5.9
1 in 200 years (l/s):		6.75

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Clock Face
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	3736553680
Date:	Aug 01 2024 14:17

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.371
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	862
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		7.62
1 in 1 year (l/s):		6.48
1 in 30 years (l/s):		14.86
1 in 100 year (l/s):		18.9
1 in 200 years (l/s):		21.65

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Warburton
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	1227958261
Date:	Aug 01 2024 14:34

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.69
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	816
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		2.38
1 in 1 year (l/s):		2.02
1 in 30 years (l/s):		4.64
1 in 100 year (l/s):		5.9
1 in 200 years (l/s):		6.75

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Partington
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	1841025085
Date:	Aug 01 2024 14:27

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.578
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	827
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		3.8
1 in 1 year (l/s):		3.23
1 in 30 years (l/s):		7.4
1 in 100 year (l/s):		9.42
1 in 200 years (l/s):		10.78

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	Innovyn HAGI
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	1590392877
Date:	Aug 01 2024 14:24

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.328
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	765
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		6.35
1 in 1 year (l/s):		5.39
1 in 30 years (l/s):		12.38
1 in 100 year (l/s):		15.74
1 in 200 years (l/s):		18.03

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	EBV
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	4246534825
Date:	Aug 01 2024 14:19

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.409
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	832
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

Default Edited

Q_{BAR} (l/s):		6.43
1 in 1 year (l/s):		5.47
1 in 30 years (l/s):		12.54
1 in 100 year (l/s):		15.95
1 in 200 years (l/s):		18.27

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Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Arun Millar
Site name:	SBV
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	
Longitude:	
Reference:	2863272125
Date:	Aug 01 2024 14:33

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha): 1

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.793
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.11

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	804
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.85
Growth curve factor 30 years:	-	1.95
Growth curve factor 100 years:	-	2.48
Growth curve factor 200 years:	-	2.84

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):		1.44
1 in 1 year (l/s):		1.22
1 in 30 years (l/s):		2.8
1 in 100 year (l/s):		3.56
1 in 200 years (l/s):		4.08

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HyNet North West Hydrogen Pipeline

Delivering clean growth

Appendix C
Attenuation Storage Calculations

Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 91 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.862	0.162	0.0	58.4	58.4	370.9	Flood Risk
30 min Summer	99.907	0.207	0.0	58.4	58.4	471.9	Flood Risk
60 min Summer	99.934	0.234	0.0	58.4	58.4	533.7	Flood Risk
120 min Summer	99.926	0.226	0.0	58.4	58.4	517.2	Flood Risk
180 min Summer	99.915	0.215	0.0	58.4	58.4	490.7	Flood Risk
240 min Summer	99.902	0.202	0.0	58.4	58.4	461.9	Flood Risk
360 min Summer	99.875	0.175	0.0	58.4	58.4	399.4	Flood Risk
480 min Summer	99.847	0.147	0.0	58.4	58.4	335.8	Flood Risk
600 min Summer	99.821	0.121	0.0	58.4	58.4	275.9	Flood Risk
720 min Summer	99.796	0.096	0.0	58.4	58.4	220.4	Flood Risk
960 min Summer	99.756	0.056	0.0	58.4	58.4	129.0	Flood Risk
1440 min Summer	99.710	0.010	0.0	58.4	58.4	21.8	Flood Risk
2160 min Summer	99.700	0.000	0.0	48.0	48.0	0.0	O K
2880 min Summer	99.700	0.000	0.0	37.7	37.7	0.0	O K
4320 min Summer	99.700	0.000	0.0	27.0	27.0	0.0	O K
5760 min Summer	99.700	0.000	0.0	21.5	21.5	0.0	O K
15 min Winter	99.887	0.187	0.0	58.4	58.4	427.5	Flood Risk
30 min Winter	99.939	0.239	0.0	58.4	58.4	545.1	Flood Risk
60 min Winter	99.974	0.274	0.0	58.4	58.4	625.9	Flood Risk
120 min Winter	99.970	0.270	0.0	58.4	58.4	616.4	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	144.884	0.0	419.4	18
30 min Summer	95.120	0.0	562.0	32
60 min Summer	59.740	0.0	713.8	60
120 min Summer	35.786	0.0	868.8	98
180 min Summer	26.380	0.0	963.1	130
240 min Summer	21.170	0.0	1031.9	164
360 min Summer	15.418	0.0	1129.4	232
480 min Summer	12.251	0.0	1194.2	298
600 min Summer	10.223	0.0	1248.2	362
720 min Summer	8.803	0.0	1288.2	422
960 min Summer	6.934	0.0	1353.1	540
1440 min Summer	4.929	0.0	1436.6	752
2160 min Summer	3.501	0.0	1525.0	0
2880 min Summer	2.753	0.0	1591.5	0
4320 min Summer	1.978	0.0	1698.4	0
5760 min Summer	1.578	0.0	1790.5	0
15 min Winter	144.884	0.0	476.2	18
30 min Winter	95.120	0.0	634.0	32
60 min Winter	59.740	0.0	806.9	60
120 min Winter	35.786	0.0	978.8	112

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.953	0.253	0.0	58.4	58.4	578.1	Flood Risk
240 min Winter	99.934	0.234	0.0	58.4	58.4	535.5	Flood Risk
360 min Winter	99.892	0.192	0.0	58.4	58.4	438.7	Flood Risk
480 min Winter	99.849	0.149	0.0	58.4	58.4	341.1	Flood Risk
600 min Winter	99.810	0.110	0.0	58.4	58.4	250.8	Flood Risk
720 min Winter	99.775	0.075	0.0	58.4	58.4	170.5	Flood Risk
960 min Winter	99.721	0.021	0.0	58.4	58.4	48.9	Flood Risk
1440 min Winter	99.700	0.000	0.0	48.9	48.9	0.0	O K
2160 min Winter	99.700	0.000	0.0	34.6	34.6	0.0	O K
2880 min Winter	99.700	0.000	0.0	27.2	27.2	0.0	O K
4320 min Winter	99.700	0.000	0.0	19.5	19.5	0.0	O K
5760 min Winter	99.700	0.000	0.0	15.5	15.5	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	26.380	0.0	1078.6	140
240 min Winter	21.170	0.0	1158.5	178
360 min Winter	15.418	0.0	1266.3	252
480 min Winter	12.251	0.0	1342.7	322
600 min Winter	10.223	0.0	1400.9	386
720 min Winter	8.803	0.0	1448.1	444
960 min Winter	6.934	0.0	1520.7	548
1440 min Winter	4.929	0.0	1618.3	0
2160 min Winter	3.501	0.0	1716.7	0
2880 min Winter	2.753	0.0	1792.5	0
4320 min Winter	1.978	0.0	1915.1	0
5760 min Winter	1.578	0.0	2021.3	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 344827 374941 SJ 44827 74941
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.690

Time (mins) From:	To:	Area (ha)
0	4	1.690

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	68.0
Membrane Percolation (mm/hr)	1000	Length (m)	112.0
Max Percolation (l/s)	2115.6	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.700	Cap Volume Depth (m)	0.300

Pipe Outflow Control

Diameter (m)	0.225	Entry Loss Coefficient	0.500
Slope (1:X)	200.0	Coefficient of Contraction	0.600
Length (m)	100.000	Upstream Invert Level (m)	98.850
Roughness k (mm)	0.600		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.689	0.689	99.5	183.4	O K
30 min Summer	99.617	0.617	99.5	160.6	O K
60 min Summer	99.508	0.508	99.5	128.0	O K
120 min Summer	99.326	0.326	99.5	76.7	O K
180 min Summer	99.170	0.170	99.5	38.0	O K
240 min Summer	99.065	0.065	99.5	13.8	O K
360 min Summer	99.000	0.000	91.3	0.0	O K
480 min Summer	99.000	0.000	72.3	0.0	O K
600 min Summer	99.000	0.000	60.4	0.0	O K
720 min Summer	99.000	0.000	52.1	0.0	O K
960 min Summer	99.000	0.000	41.8	0.0	O K
1440 min Summer	99.000	0.000	30.7	0.0	O K
2160 min Summer	99.000	0.000	22.5	0.0	O K
2880 min Summer	99.000	0.000	18.1	0.0	O K
4320 min Summer	99.000	0.000	13.2	0.0	O K
5760 min Summer	99.000	0.000	10.5	0.0	O K
7200 min Summer	99.000	0.000	8.8	0.0	O K
8640 min Summer	99.000	0.000	7.6	0.0	O K
10080 min Summer	99.000	0.000	6.8	0.0	O K
15 min Winter	99.692	0.692	99.5	184.7	O K
30 min Winter	99.611	0.611	99.5	158.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	130.184	0.0	274.5	16
30 min Summer	74.254	0.0	313.1	25
60 min Summer	42.353	0.0	356.8	42
120 min Summer	24.157	0.0	408.1	74
180 min Summer	17.395	0.0	440.6	104
240 min Summer	13.779	0.0	465.6	168
360 min Summer	9.922	0.0	503.0	0
480 min Summer	7.859	0.0	531.3	0
600 min Summer	6.560	0.0	554.3	0
720 min Summer	5.659	0.0	573.8	0
960 min Summer	4.544	0.0	614.3	0
1440 min Summer	3.335	0.0	676.3	0
2160 min Summer	2.447	0.0	744.5	0
2880 min Summer	1.965	0.0	797.0	0
4320 min Summer	1.429	0.0	869.6	0
5760 min Summer	1.140	0.0	925.1	0
7200 min Summer	0.957	0.0	970.6	0
8640 min Summer	0.830	0.0	1009.4	0
10080 min Summer	0.735	0.0	1043.4	0
15 min Winter	130.184	0.0	274.5	16
30 min Winter	74.254	0.0	313.1	26

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.462	0.462	99.5	113.7	O K
120 min Winter	99.178	0.178	99.5	40.0	O K
180 min Winter	99.006	0.006	99.5	1.2	O K
240 min Winter	99.000	0.000	81.8	0.0	O K
360 min Winter	99.000	0.000	58.9	0.0	O K
480 min Winter	99.000	0.000	46.7	0.0	O K
600 min Winter	99.000	0.000	38.9	0.0	O K
720 min Winter	99.000	0.000	33.6	0.0	O K
960 min Winter	99.000	0.000	27.0	0.0	O K
1440 min Winter	99.000	0.000	19.8	0.0	O K
2160 min Winter	99.000	0.000	14.5	0.0	O K
2880 min Winter	99.000	0.000	11.7	0.0	O K
4320 min Winter	99.000	0.000	8.5	0.0	O K
5760 min Winter	99.000	0.000	6.8	0.0	O K
7200 min Winter	99.000	0.000	5.7	0.0	O K
8640 min Winter	99.000	0.000	4.9	0.0	O K
10080 min Winter	99.000	0.000	4.4	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	42.353	0.0	357.7	44
120 min Winter	24.157	0.0	407.8	76
180 min Winter	17.395	0.0	440.8	170
240 min Winter	13.779	0.0	465.7	0
360 min Winter	9.922	0.0	503.0	0
480 min Winter	7.859	0.0	531.3	0
600 min Winter	6.560	0.0	554.3	0
720 min Winter	5.659	0.0	573.8	0
960 min Winter	4.544	0.0	614.3	0
1440 min Winter	3.335	0.0	676.3	0
2160 min Winter	2.447	0.0	744.5	0
2880 min Winter	1.965	0.0	797.0	0
4320 min Winter	1.429	0.0	869.6	0
5760 min Winter	1.140	0.0	925.1	0
7200 min Winter	0.957	0.0	970.6	0
8640 min Winter	0.830	0.0	1009.4	0
10080 min Winter	0.735	0.0	1043.4	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 345850 373850 SJ 45850 73850
C (1km)	-0.025
D1 (1km)	0.305
D2 (1km)	0.352
D3 (1km)	0.330
E (1km)	0.290
F (1km)	2.412
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.690

Time (mins) From:	To:	Area (ha)
0	4	1.690

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 99.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	209.0	1.000	391.0

Pipe Outflow Control

Diameter (m)	0.225	Entry Loss Coefficient	0.500
Slope (1:X)	10.0	Coefficient of Contraction	0.600
Length (m)	10.000	Upstream Invert Level (m)	99.000
Roughness k (mm)	0.600		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 672 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.696	0.196	0.0	3.0	3.0	97.3	O K
30 min Summer	99.758	0.258	0.0	3.0	3.0	128.5	Flood Risk
60 min Summer	99.817	0.317	0.0	3.0	3.0	157.8	Flood Risk
120 min Summer	99.867	0.367	0.0	3.0	3.0	182.5	Flood Risk
180 min Summer	99.892	0.392	0.0	3.0	3.0	194.9	Flood Risk
240 min Summer	99.905	0.405	0.0	3.0	3.0	201.3	Flood Risk
360 min Summer	99.912	0.412	0.0	3.0	3.0	204.9	Flood Risk
480 min Summer	99.905	0.405	0.0	3.0	3.0	201.7	Flood Risk
600 min Summer	99.892	0.392	0.0	3.0	3.0	195.2	Flood Risk
720 min Summer	99.880	0.380	0.0	3.0	3.0	189.0	Flood Risk
960 min Summer	99.858	0.358	0.0	3.0	3.0	178.3	Flood Risk
1440 min Summer	99.824	0.324	0.0	3.0	3.0	161.1	Flood Risk
2160 min Summer	99.777	0.277	0.0	3.0	3.0	138.0	Flood Risk
2880 min Summer	99.736	0.236	0.0	3.0	3.0	117.2	Flood Risk
4320 min Summer	99.664	0.164	0.0	3.0	3.0	81.7	O K
5760 min Summer	99.608	0.108	0.0	3.0	3.0	53.5	O K
15 min Winter	99.722	0.222	0.0	3.0	3.0	110.3	Flood Risk
30 min Winter	99.793	0.293	0.0	3.0	3.0	145.6	Flood Risk
60 min Winter	99.860	0.360	0.0	3.0	3.0	179.1	Flood Risk
120 min Winter	99.918	0.418	0.0	3.0	3.0	208.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	144.362	0.0	99.7	19
30 min Summer	94.598	0.0	133.2	34
60 min Summer	58.740	0.0	167.4	64
120 min Summer	35.126	0.0	201.7	122
180 min Summer	25.907	0.0	223.8	182
240 min Summer	20.815	0.0	240.3	242
360 min Summer	15.198	0.0	263.7	362
480 min Summer	12.115	0.0	280.6	480
600 min Summer	10.140	0.0	293.5	550
720 min Summer	8.757	0.0	304.1	606
960 min Summer	6.933	0.0	320.8	724
1440 min Summer	4.986	0.0	345.5	984
2160 min Summer	3.584	0.0	371.0	1388
2880 min Summer	2.840	0.0	390.7	1788
4320 min Summer	2.056	0.0	420.8	2552
5760 min Summer	1.642	0.0	444.2	3288
15 min Winter	144.362	0.0	112.6	19
30 min Winter	94.598	0.0	150.3	33
60 min Winter	58.740	0.0	188.6	62
120 min Winter	35.126	0.0	227.0	120

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Summary of Results for 100 year Return Period (+45%)

	Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180	min Winter	99.949	0.449	0.0	3.0	3.0	223.3	Flood Risk
240	min Winter	99.966	0.466	0.0	3.0	3.0	231.9	Flood Risk
360	min Winter	99.980	0.480	0.0	3.0	3.0	238.8	Flood Risk
480	min Winter	99.979	0.479	0.0	3.0	3.0	238.4	Flood Risk
600	min Winter	99.970	0.470	0.0	3.0	3.0	234.0	Flood Risk
720	min Winter	99.956	0.456	0.0	3.0	3.0	227.1	Flood Risk
960	min Winter	99.924	0.424	0.0	3.0	3.0	211.1	Flood Risk
1440	min Winter	99.878	0.378	0.0	3.0	3.0	188.0	Flood Risk
2160	min Winter	99.809	0.309	0.0	3.0	3.0	153.6	Flood Risk
2880	min Winter	99.745	0.245	0.0	3.0	3.0	121.7	Flood Risk
4320	min Winter	99.637	0.137	0.0	3.0	3.0	68.0	O K
5760	min Winter	99.557	0.057	0.0	3.0	3.0	28.5	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180	min Winter	25.907	0.0	251.8	180
240	min Winter	20.815	0.0	270.3	238
360	min Winter	15.198	0.0	296.4	352
480	min Winter	12.115	0.0	315.5	466
600	min Winter	10.140	0.0	330.0	574
720	min Winter	8.757	0.0	342.0	680
960	min Winter	6.933	0.0	361.0	772
1440	min Winter	4.986	0.0	388.7	1070
2160	min Winter	3.584	0.0	417.6	1516
2880	min Winter	2.840	0.0	439.3	1932
4320	min Winter	2.056	0.0	473.7	2728
5760	min Winter	1.642	0.0	501.1	3352

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352854 379250 SJ 52854 79250
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 0.400

Time (mins) From:	To:	Area (ha)
0	4	0.400

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	31.0
Membrane Percolation (mm/hr)	1000	Length (m)	53.5
Max Percolation (l/s)	460.7	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.500	Cap Volume Depth (m)	0.500

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0082-3000-1000-3000
Design Head (m)	1.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	82
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.0	Kick-Flo®	0.623	2.4
Flush-Flo™	0.297	3.0	Mean Flow over Head Range	-	2.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	1.200	3.3	3.000	5.0	7.000	7.4
0.200	2.9	1.400	3.5	3.500	5.4	7.500	7.7
0.300	3.0	1.600	3.7	4.000	5.7	8.000	7.9
0.400	2.9	1.800	3.9	4.500	6.0	8.500	8.2
0.500	2.8	2.000	4.1	5.000	6.3	9.000	8.4
0.600	2.5	2.200	4.3	5.500	6.6	9.500	8.6
0.800	2.7	2.400	4.5	6.000	6.9		
1.000	3.0	2.600	4.7	6.500	7.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.899	0.899	6.1	391.5	O K
30 min Summer	99.108	1.108	6.1	509.7	O K
60 min Summer	99.295	1.295	6.1	625.1	O K
120 min Summer	99.454	1.454	6.1	730.3	O K
180 min Summer	99.541	1.541	6.1	790.7	O K
240 min Summer	99.595	1.595	6.1	829.8	O K
360 min Summer	99.655	1.655	6.1	873.8	O K
480 min Summer	99.681	1.681	6.1	893.3	O K
600 min Summer	99.689	1.689	6.1	899.1	O K
720 min Summer	99.685	1.685	6.1	896.0	O K
960 min Summer	99.654	1.654	6.1	873.0	O K
1440 min Summer	99.568	1.568	6.1	810.2	O K
2160 min Summer	99.475	1.475	6.1	745.1	O K
2880 min Summer	99.402	1.402	6.1	695.4	O K
4320 min Summer	99.271	1.271	6.1	609.6	O K
5760 min Summer	99.148	1.148	6.1	533.7	O K
15 min Winter	98.899	0.899	6.1	391.5	O K
30 min Winter	99.108	1.108	6.1	509.6	O K
60 min Winter	99.295	1.295	6.1	625.1	O K
120 min Winter	99.455	1.455	6.1	731.0	O K
180 min Winter	99.542	1.542	6.1	792.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.560	0.0	385.6	19
30 min Summer	65.240	0.0	421.0	34
60 min Summer	40.510	0.0	643.9	64
120 min Summer	24.225	0.0	766.8	124
180 min Summer	17.867	0.0	838.7	184
240 min Summer	14.355	0.0	868.6	242
360 min Summer	10.482	0.0	882.3	362
480 min Summer	8.355	0.0	889.1	482
600 min Summer	6.993	0.0	895.2	602
720 min Summer	6.039	0.0	901.3	722
960 min Summer	4.781	0.0	913.6	960
1440 min Summer	3.438	0.0	930.4	1240
2160 min Summer	2.471	0.0	1420.9	1600
2880 min Summer	1.959	0.0	1500.2	1992
4320 min Summer	1.418	0.0	1599.7	2812
5760 min Summer	1.133	0.0	1739.0	3632
15 min Winter	99.560	0.0	385.6	19
30 min Winter	65.240	0.0	420.9	33
60 min Winter	40.510	0.0	643.9	64
120 min Winter	24.225	0.0	766.8	122
180 min Winter	17.867	0.0	838.5	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.598	1.598	6.1	831.8	O K
360 min Winter	99.659	1.659	6.1	876.6	O K
480 min Winter	99.687	1.687	6.1	897.4	O K
600 min Winter	99.696	1.696	6.1	904.7	O K
720 min Winter	99.695	1.695	6.1	903.6	O K
960 min Winter	99.671	1.671	6.1	885.5	O K
1440 min Winter	99.585	1.585	6.1	822.4	O K
2160 min Winter	99.468	1.468	6.1	740.1	O K
2880 min Winter	99.368	1.368	6.1	672.5	O K
4320 min Winter	99.167	1.167	6.1	544.9	O K
5760 min Winter	98.968	0.968	6.1	429.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.355	0.0	868.0	240
360 min Winter	10.482	0.0	880.8	356
480 min Winter	8.355	0.0	887.0	472
600 min Winter	6.993	0.0	892.3	588
720 min Winter	6.039	0.0	897.3	702
960 min Winter	4.781	0.0	907.2	926
1440 min Winter	3.438	0.0	928.0	1344
2160 min Winter	2.471	0.0	1420.9	1668
2880 min Winter	1.959	0.0	1500.5	2108
4320 min Winter	1.418	0.0	1616.9	2988
5760 min Winter	1.133	0.0	1739.1	3856

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352854 379250 SJ 52854 79250
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.200

Time (mins) From:	To:	Area (ha)
0	4	3.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	339.9	2.000	845.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0101-6100-2000-6100
Design Head (m)	2.000
Design Flow (l/s)	6.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	101
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	6.1	Kick-Flo®	0.908	4.2
Flush-Flo™	0.444	5.3	Mean Flow over Head Range	-	4.9

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.4	1.200	4.8	3.000	7.4	7.000	11.0
0.200	4.8	1.400	5.2	3.500	7.9	7.500	11.4
0.300	5.2	1.600	5.5	4.000	8.4	8.000	11.7
0.400	5.3	1.800	5.8	4.500	8.9	8.500	12.1
0.500	5.3	2.000	6.1	5.000	9.4	9.000	12.4
0.600	5.2	2.200	6.4	5.500	9.8	9.500	12.7
0.800	4.8	2.400	6.6	6.000	10.2		
1.000	4.4	2.600	6.9	6.500	10.6		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.138	1.138	3.0	108.7	O K
30 min Summer	99.324	1.324	3.0	140.8	O K
60 min Summer	99.478	1.478	3.0	171.0	O K
120 min Summer	99.592	1.592	3.0	195.9	O K
180 min Summer	99.646	1.646	3.0	208.4	O K
240 min Summer	99.674	1.674	3.0	214.9	O K
360 min Summer	99.687	1.687	3.0	218.1	O K
480 min Summer	99.672	1.672	3.0	214.4	O K
600 min Summer	99.641	1.641	3.0	207.2	O K
720 min Summer	99.612	1.612	3.0	200.5	O K
960 min Summer	99.563	1.563	3.0	189.4	O K
1440 min Summer	99.483	1.483	3.0	172.0	O K
2160 min Summer	99.368	1.368	3.0	149.0	O K
2880 min Summer	99.255	1.255	3.0	128.3	O K
4320 min Summer	99.032	1.032	3.0	92.4	O K
5760 min Summer	98.813	0.813	3.0	63.4	O K
15 min Winter	99.138	1.138	3.0	108.7	O K
30 min Winter	99.325	1.325	3.0	140.9	O K
60 min Winter	99.478	1.478	3.0	171.1	O K
120 min Winter	99.594	1.594	3.0	196.4	O K
180 min Winter	99.650	1.650	3.0	209.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.560	0.0	111.9	19
30 min Summer	65.240	0.0	146.6	34
60 min Summer	40.510	0.0	182.3	64
120 min Summer	24.225	0.0	217.9	122
180 min Summer	17.867	0.0	241.1	182
240 min Summer	14.355	0.0	258.3	242
360 min Summer	10.482	0.0	282.9	362
480 min Summer	8.355	0.0	300.7	480
600 min Summer	6.993	0.0	314.6	574
720 min Summer	6.039	0.0	326.0	622
960 min Summer	4.781	0.0	344.1	742
1440 min Summer	3.438	0.0	371.1	996
2160 min Summer	2.471	0.0	400.4	1408
2880 min Summer	1.959	0.0	423.0	1816
4320 min Summer	1.418	0.0	459.3	2596
5760 min Summer	1.133	0.0	489.3	3456
15 min Winter	99.560	0.0	111.9	19
30 min Winter	65.240	0.0	146.6	33
60 min Winter	40.510	0.0	182.2	62
120 min Winter	24.225	0.0	217.9	120
180 min Winter	17.867	0.0	241.2	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.678	1.678	3.0	216.0	O K
360 min Winter	99.695	1.695	3.0	220.0	O K
480 min Winter	99.685	1.685	3.0	217.5	O K
600 min Winter	99.660	1.660	3.0	211.5	O K
720 min Winter	99.626	1.626	3.0	203.6	O K
960 min Winter	99.562	1.562	3.0	189.2	O K
1440 min Winter	99.456	1.456	3.0	166.5	O K
2160 min Winter	99.282	1.282	3.0	133.1	O K
2880 min Winter	99.099	1.099	3.0	102.5	O K
4320 min Winter	98.714	0.714	3.0	52.2	O K
5760 min Winter	98.312	0.312	3.0	17.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.355	0.0	258.3	238
360 min Winter	10.482	0.0	282.9	352
480 min Winter	8.355	0.0	300.7	464
600 min Winter	6.993	0.0	314.6	572
720 min Winter	6.039	0.0	326.0	674
960 min Winter	4.781	0.0	344.1	760
1440 min Winter	3.438	0.0	371.1	1066
2160 min Winter	2.471	0.0	400.4	1496
2880 min Winter	1.959	0.0	423.0	1904
4320 min Winter	1.418	0.0	459.3	2640
5760 min Winter	1.133	0.0	489.3	3288

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352854 379250 SJ 52854 79250
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 0.900

Time (mins) From:	To:	Area (ha)
0	4	0.900

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	43.4	2.000	296.6

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0070-3000-2000-3000
Design Head (m)	2.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	70
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.0	Kick-Flo®	0.630	1.8
Flush-Flo™	0.310	2.2	Mean Flow over Head Range	-	2.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.8	1.200	2.4	3.000	3.6	7.000	5.4
0.200	2.1	1.400	2.5	3.500	3.9	7.500	5.6
0.300	2.2	1.600	2.7	4.000	4.1	8.000	5.7
0.400	2.2	1.800	2.9	4.500	4.4	8.500	5.9
0.500	2.1	2.000	3.0	5.000	4.6	9.000	6.1
0.600	1.9	2.200	3.1	5.500	4.8	9.500	6.2
0.800	2.0	2.400	3.3	6.000	5.0		
1.000	2.2	2.600	3.4	6.500	5.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.051	1.051	3.0	146.0	O K
30 min Summer	99.245	1.245	3.0	189.7	O K
60 min Summer	99.410	1.410	3.0	231.7	O K
120 min Summer	99.542	1.542	3.0	268.6	O K
180 min Summer	99.609	1.609	3.0	288.7	O K
240 min Summer	99.649	1.649	3.0	300.9	O K
360 min Summer	99.685	1.685	3.0	312.3	O K
480 min Summer	99.692	1.692	3.0	314.5	O K
600 min Summer	99.683	1.683	3.0	311.5	O K
720 min Summer	99.663	1.663	3.0	305.3	O K
960 min Summer	99.611	1.611	3.0	289.3	O K
1440 min Summer	99.536	1.536	3.0	266.9	O K
2160 min Summer	99.445	1.445	3.0	241.2	O K
2880 min Summer	99.361	1.361	3.0	218.8	O K
4320 min Summer	99.200	1.200	3.0	179.1	O K
5760 min Summer	99.044	1.044	3.0	144.7	O K
15 min Winter	99.051	1.051	3.0	146.0	O K
30 min Winter	99.245	1.245	3.0	189.7	O K
60 min Winter	99.410	1.410	3.0	231.7	O K
120 min Winter	99.543	1.543	3.0	269.0	O K
180 min Winter	99.612	1.612	3.0	289.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.560	0.0	148.5	19
30 min Summer	65.240	0.0	183.6	34
60 min Summer	40.510	0.0	242.9	64
120 min Summer	24.225	0.0	290.4	124
180 min Summer	17.867	0.0	321.1	182
240 min Summer	14.355	0.0	344.0	242
360 min Summer	10.482	0.0	376.5	362
480 min Summer	8.355	0.0	399.5	482
600 min Summer	6.993	0.0	415.7	600
720 min Summer	6.039	0.0	423.6	720
960 min Summer	4.781	0.0	429.9	858
1440 min Summer	3.438	0.0	429.8	1084
2160 min Summer	2.471	0.0	533.7	1476
2880 min Summer	1.959	0.0	563.9	1876
4320 min Summer	1.418	0.0	612.2	2684
5760 min Summer	1.133	0.0	652.4	3464
15 min Winter	99.560	0.0	148.5	19
30 min Winter	65.240	0.0	183.5	33
60 min Winter	40.510	0.0	242.8	62
120 min Winter	24.225	0.0	290.4	122
180 min Winter	17.867	0.0	321.2	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.652	1.652	3.0	301.9	O K
360 min Winter	99.690	1.690	3.0	313.9	O K
480 min Winter	99.700	1.700	3.0	317.0	O K
600 min Winter	99.694	1.694	3.0	315.1	O K
720 min Winter	99.678	1.678	3.0	310.1	O K
960 min Winter	99.630	1.630	3.0	295.0	O K
1440 min Winter	99.533	1.533	3.0	266.0	O K
2160 min Winter	99.411	1.411	3.0	231.9	O K
2880 min Winter	99.283	1.283	3.0	199.0	O K
4320 min Winter	99.020	1.020	3.0	139.8	O K
5760 min Winter	98.753	0.753	3.0	90.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.355	0.0	344.0	238
360 min Winter	10.482	0.0	376.5	354
480 min Winter	8.355	0.0	399.4	470
600 min Winter	6.993	0.0	415.1	584
720 min Winter	6.039	0.0	421.6	694
960 min Winter	4.781	0.0	427.9	906
1440 min Winter	3.438	0.0	430.5	1126
2160 min Winter	2.471	0.0	533.7	1580
2880 min Winter	1.959	0.0	563.9	2020
4320 min Winter	1.418	0.0	612.3	2856
5760 min Winter	1.133	0.0	652.3	3584

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352854 379250 SJ 52854 79250
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.200

Time (mins) From:	To:	Area (ha)
0	4	1.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	79.0	2.000	381.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0070-3000-2000-3000
Design Head (m)	2.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	70
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.0	Kick-Flo®	0.630	1.8
Flush-Flo™	0.310	2.2	Mean Flow over Head Range	-	2.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.8	1.200	2.4	3.000	3.6	7.000	5.4
0.200	2.1	1.400	2.5	3.500	3.9	7.500	5.6
0.300	2.2	1.600	2.7	4.000	4.1	8.000	5.7
0.400	2.2	1.800	2.9	4.500	4.4	8.500	5.9
0.500	2.1	2.000	3.0	5.000	4.6	9.000	6.1
0.600	1.9	2.200	3.1	5.500	4.8	9.500	6.2
0.800	2.0	2.400	3.3	6.000	5.0		
1.000	2.2	2.600	3.4	6.500	5.2		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 1913 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.658	0.158	0.0	3.8	3.8	287.3	O K
30 min Summer	99.712	0.212	0.0	3.8	3.8	384.2	Flood Risk
60 min Summer	99.764	0.264	0.0	3.8	3.8	479.7	Flood Risk
120 min Summer	99.813	0.313	0.0	3.8	3.8	568.0	Flood Risk
180 min Summer	99.842	0.342	0.0	3.8	3.8	621.0	Flood Risk
240 min Summer	99.862	0.362	0.0	3.8	3.8	656.9	Flood Risk
360 min Summer	99.887	0.387	0.0	3.8	3.8	701.7	Flood Risk
480 min Summer	99.900	0.400	0.0	3.8	3.8	726.4	Flood Risk
600 min Summer	99.908	0.408	0.0	3.8	3.8	740.5	Flood Risk
720 min Summer	99.912	0.412	0.0	3.8	3.8	747.7	Flood Risk
960 min Summer	99.912	0.412	0.0	3.8	3.8	748.0	Flood Risk
1440 min Summer	99.898	0.398	0.0	3.8	3.8	722.0	Flood Risk
2160 min Summer	99.869	0.369	0.0	3.8	3.8	670.2	Flood Risk
2880 min Summer	99.849	0.349	0.0	3.8	3.8	632.8	Flood Risk
4320 min Summer	99.817	0.317	0.0	3.8	3.8	574.3	Flood Risk
5760 min Summer	99.789	0.289	0.0	3.8	3.8	523.6	Flood Risk
15 min Winter	99.680	0.180	0.0	3.8	3.8	325.8	O K
30 min Winter	99.739	0.239	0.0	3.8	3.8	434.5	Flood Risk
60 min Winter	99.799	0.299	0.0	3.8	3.8	542.2	Flood Risk
120 min Winter	99.854	0.354	0.0	3.8	3.8	643.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	142.680	0.0	287.7	19
30 min Summer	93.525	0.0	277.2	34
60 min Summer	58.087	0.0	491.0	64
120 min Summer	34.677	0.0	571.4	124
180 min Summer	25.564	0.0	570.2	184
240 min Summer	20.536	0.0	571.2	244
360 min Summer	15.000	0.0	575.3	362
480 min Summer	11.964	0.0	581.0	482
600 min Summer	10.022	0.0	585.7	602
720 min Summer	8.664	0.0	589.1	722
960 min Summer	6.870	0.0	593.5	962
1440 min Summer	4.954	0.0	598.1	1430
2160 min Summer	3.567	0.0	1097.8	1752
2880 min Summer	2.830	0.0	1134.3	2108
4320 min Summer	2.050	0.0	1056.9	2900
5760 min Summer	1.639	0.0	1312.6	3752
15 min Winter	142.680	0.0	282.6	19
30 min Winter	93.525	0.0	276.0	34
60 min Winter	58.087	0.0	553.6	64
120 min Winter	34.677	0.0	570.7	122

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.888	0.388	0.0	3.8	3.8	704.2	Flood Risk
240 min Winter	99.911	0.411	0.0	3.8	3.8	746.3	Flood Risk
360 min Winter	99.941	0.441	0.0	3.8	3.8	799.7	Flood Risk
480 min Winter	99.958	0.458	0.0	3.8	3.8	830.9	Flood Risk
600 min Winter	99.969	0.469	0.0	3.8	3.8	850.7	Flood Risk
720 min Winter	99.975	0.475	0.0	3.8	3.8	862.4	Flood Risk
960 min Winter	99.980	0.480	0.0	3.8	3.8	870.6	Flood Risk
1440 min Winter	99.973	0.473	0.0	3.8	3.8	858.2	Flood Risk
2160 min Winter	99.942	0.442	0.0	3.8	3.8	802.6	Flood Risk
2880 min Winter	99.913	0.413	0.0	3.8	3.8	748.5	Flood Risk
4320 min Winter	99.867	0.367	0.0	3.8	3.8	666.2	Flood Risk
5760 min Winter	99.824	0.324	0.0	3.8	3.8	588.1	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	25.564	0.0	573.8	182
240 min Winter	20.536	0.0	578.9	240
360 min Winter	15.000	0.0	589.1	358
480 min Winter	11.964	0.0	596.0	476
600 min Winter	10.022	0.0	600.2	592
720 min Winter	8.664	0.0	603.3	708
960 min Winter	6.870	0.0	606.5	936
1440 min Winter	4.954	0.0	607.5	1386
2160 min Winter	3.567	0.0	1175.4	2012
2880 min Winter	2.830	0.0	1159.2	2276
4320 min Winter	2.050	0.0	1112.8	3160
5760 min Winter	1.639	0.0	1482.7	4096

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352287 380052 SJ 52287 80052
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.200

Time (mins) From:	To:	Area (ha)
0	4	1.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	72.0
Membrane Percolation (mm/hr)	1000	Length (m)	84.0
Max Percolation (l/s)	1680.0	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.500	Cap Volume Depth (m)	0.500

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0092-3800-1000-3800
Design Head (m)	1.000
Design Flow (l/s)	3.8
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	92
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.8	Kick-Flo®	0.632	3.1
Flush-Flo™	0.298	3.8	Mean Flow over Head Range	-	3.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.9	1.200	4.1	3.000	6.3	7.000	9.5
0.200	3.7	1.400	4.4	3.500	6.8	7.500	9.8
0.300	3.8	1.600	4.7	4.000	7.3	8.000	10.1
0.400	3.7	1.800	5.0	4.500	7.7	8.500	10.4
0.500	3.6	2.000	5.2	5.000	8.1	9.000	10.7
0.600	3.3	2.200	5.5	5.500	8.4	9.500	10.9
0.800	3.4	2.400	5.7	6.000	8.8		
1.000	3.8	2.600	5.9	6.500	9.1		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.065	1.065	5.0	191.3	O K
30 min Summer	99.269	1.269	5.0	248.0	O K
60 min Summer	99.441	1.441	5.0	301.6	O K
120 min Summer	99.570	1.570	5.0	345.7	O K
180 min Summer	99.633	1.633	5.0	368.4	O K
240 min Summer	99.666	1.666	5.0	380.7	O K
360 min Summer	99.686	1.686	5.0	388.3	O K
480 min Summer	99.675	1.675	5.0	384.1	O K
600 min Summer	99.646	1.646	5.0	373.2	O K
720 min Summer	99.614	1.614	5.0	361.5	O K
960 min Summer	99.562	1.562	5.0	342.9	O K
1440 min Summer	99.478	1.478	5.0	314.0	O K
2160 min Summer	99.360	1.360	5.0	275.8	O K
2880 min Summer	99.246	1.246	5.0	241.3	O K
4320 min Summer	99.022	1.022	5.0	180.3	O K
5760 min Summer	98.808	0.808	5.0	129.8	O K
15 min Winter	99.065	1.065	5.0	191.3	O K
30 min Winter	99.269	1.269	5.0	248.1	O K
60 min Winter	99.441	1.441	5.0	301.8	O K
120 min Winter	99.572	1.572	5.0	346.6	O K
180 min Winter	99.637	1.637	5.0	369.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	98.400	0.0	196.2	19
30 min Summer	64.500	0.0	257.0	34
60 min Summer	40.060	0.0	320.3	64
120 min Summer	23.915	0.0	382.2	122
180 min Summer	17.630	0.0	422.7	182
240 min Summer	14.163	0.0	452.9	242
360 min Summer	10.345	0.0	496.2	362
480 min Summer	8.251	0.0	527.6	480
600 min Summer	6.912	0.0	552.5	596
720 min Summer	5.975	0.0	573.1	644
960 min Summer	4.738	0.0	605.8	760
1440 min Summer	3.417	0.0	654.4	1038
2160 min Summer	2.460	0.0	708.5	1408
2880 min Summer	1.951	0.0	749.2	1820
4320 min Summer	1.414	0.0	814.2	2596
5760 min Summer	1.131	0.0	868.2	3352
15 min Winter	98.400	0.0	196.2	19
30 min Winter	64.500	0.0	257.0	33
60 min Winter	40.060	0.0	320.2	62
120 min Winter	23.915	0.0	382.2	120
180 min Winter	17.630	0.0	422.8	180

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Summary of Results for 100 year Return Period

	Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240	min Winter	99.671	1.671	5.0	382.5	O K
360	min Winter	99.695	1.695	5.0	391.4	O K
480	min Winter	99.688	1.688	5.0	389.0	O K
600	min Winter	99.666	1.666	5.0	380.5	O K
720	min Winter	99.633	1.633	5.0	368.5	O K
960	min Winter	99.562	1.562	5.0	342.9	O K
1440	min Winter	99.456	1.456	5.0	306.5	O K
2160	min Winter	99.280	1.280	5.0	251.3	O K
2880	min Winter	99.097	1.097	5.0	199.7	O K
4320	min Winter	98.725	0.725	5.0	112.2	O K
5760	min Winter	98.364	0.364	5.0	47.7	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240	min Winter	14.163	0.0	452.9	238
360	min Winter	10.345	0.0	496.2	352
480	min Winter	8.251	0.0	527.6	466
600	min Winter	6.912	0.0	552.5	574
720	min Winter	5.975	0.0	573.0	680
960	min Winter	4.738	0.0	605.8	772
1440	min Winter	3.417	0.0	654.6	1070
2160	min Winter	2.460	0.0	708.5	1516
2880	min Winter	1.951	0.0	749.2	1932
4320	min Winter	1.414	0.0	814.3	2748
5760	min Winter	1.131	0.0	868.2	3352

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352287 380052 SJ 52287 80052
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.600

Time (mins) From:	To:	Area (ha)
0	4	1.600

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	109.6	2.000	445.4

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0092-5000-2000-5000
Design Head (m)	2.000
Design Flow (l/s)	5.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	92
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	5.0	Kick-Flo®	0.816	3.3
Flush-Flo™	0.398	4.1	Mean Flow over Head Range	-	3.9

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.9	1.200	3.9	3.000	6.0	7.000	9.0
0.200	3.8	1.400	4.2	3.500	6.5	7.500	9.3
0.300	4.1	1.600	4.5	4.000	6.9	8.000	9.6
0.400	4.1	1.800	4.8	4.500	7.3	8.500	9.9
0.500	4.1	2.000	5.0	5.000	7.7	9.000	10.2
0.600	4.0	2.200	5.2	5.500	8.0	9.500	10.4
0.800	3.4	2.400	5.4	6.000	8.4		
1.000	3.6	2.600	5.6	6.500	8.7		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.093	1.093	8.8	141.4	O K
30 min Summer	99.290	1.290	8.8	184.0	O K
60 min Summer	99.454	1.454	8.8	224.2	O K
120 min Summer	99.578	1.578	8.8	257.6	O K
180 min Summer	99.638	1.638	8.8	274.5	O K
240 min Summer	99.669	1.669	8.8	283.7	O K
360 min Summer	99.688	1.688	8.8	289.3	O K
480 min Summer	99.667	1.667	8.8	283.1	O K
600 min Summer	99.630	1.630	8.8	272.4	O K
720 min Summer	99.594	1.594	8.8	262.0	O K
960 min Summer	99.527	1.527	8.8	243.3	O K
1440 min Summer	99.400	1.400	8.8	210.4	O K
2160 min Summer	99.196	1.196	8.8	163.0	O K
2880 min Summer	99.018	1.018	8.8	126.9	O K
4320 min Summer	98.090	0.090	8.8	6.7	O K
5760 min Summer	98.000	0.000	7.4	0.0	O K
15 min Winter	99.093	1.093	8.8	141.5	O K
30 min Winter	99.292	1.292	8.8	184.3	O K
60 min Winter	99.457	1.457	8.8	225.0	O K
120 min Winter	99.583	1.583	8.8	258.8	O K
180 min Winter	99.644	1.644	8.8	276.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	98.400	0.0	507.7	19
30 min Summer	64.500	0.0	553.7	34
60 min Summer	40.060	0.0	736.1	64
120 min Summer	23.915	0.0	782.8	122
180 min Summer	17.630	0.0	813.1	182
240 min Summer	14.163	0.0	835.7	242
360 min Summer	10.345	0.0	868.3	362
480 min Summer	8.251	0.0	891.9	480
600 min Summer	6.912	0.0	910.6	536
720 min Summer	5.975	0.0	926.0	588
960 min Summer	4.738	0.0	950.7	704
1440 min Summer	3.417	0.0	987.8	956
2160 min Summer	2.460	0.0	1027.6	1344
2880 min Summer	1.951	0.0	1058.2	1760
4320 min Summer	1.414	0.0	1107.0	2396
5760 min Summer	1.131	0.0	1147.4	0
15 min Winter	98.400	0.0	507.8	19
30 min Winter	64.500	0.0	553.6	33
60 min Winter	40.060	0.0	736.2	62
120 min Winter	23.915	0.0	782.8	122
180 min Winter	17.630	0.0	813.2	180

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Summary of Results for 100 year Return Period

	Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240	min Winter	99.677	1.677	8.8	286.0	O K
360	min Winter	99.699	1.699	8.8	292.6	O K
480	min Winter	99.682	1.682	8.8	287.7	O K
600	min Winter	99.649	1.649	8.8	277.7	O K
720	min Winter	99.604	1.604	8.8	264.9	O K
960	min Winter	99.522	1.522	8.8	242.2	O K
1440	min Winter	99.352	1.352	8.8	198.6	O K
2160	min Winter	99.041	1.041	8.8	131.2	O K
2880	min Winter	98.762	0.762	8.8	83.0	O K
4320	min Winter	98.014	0.014	8.8	1.0	O K
5760	min Winter	98.000	0.000	6.1	0.0	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240	min Winter	14.163	0.0	835.7	238
360	min Winter	10.345	0.0	868.3	352
480	min Winter	8.251	0.0	891.9	462
600	min Winter	6.912	0.0	910.6	566
720	min Winter	5.975	0.0	926.0	638
960	min Winter	4.738	0.0	950.7	734
1440	min Winter	3.417	0.0	987.8	1024
2160	min Winter	2.460	0.0	1027.6	1408
2880	min Winter	1.951	0.0	1058.2	1848
4320	min Winter	1.414	0.0	1107.0	2476
5760	min Winter	1.131	0.0	1147.4	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 352287 380052 SJ 52287 80052
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.200

Time (mins) From:	To:	Area (ha)
0	4	1.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	69.7	2.000	360.3

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0123-8800-2000-8800
Design Head (m)	2.000
Design Flow (l/s)	8.8
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	123
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	8.8	Kick-Flo®	1.092	6.6
Flush-Flo™	0.536	8.3	Mean Flow over Head Range	-	7.4

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.4	1.200	6.9	3.000	10.6	7.000	15.9
0.200	7.1	1.400	7.4	3.500	11.4	7.500	16.5
0.300	7.9	1.600	7.9	4.000	12.2	8.000	17.0
0.400	8.2	1.800	8.4	4.500	12.9	8.500	17.5
0.500	8.3	2.000	8.8	5.000	13.6	9.000	18.0
0.600	8.3	2.200	9.2	5.500	14.2	9.500	18.4
0.800	8.0	2.400	9.6	6.000	14.8		
1.000	7.3	2.600	9.9	6.500	15.4		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 1223 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.693	0.143	0.0	7.8	7.8	400.7	O K
30 min Summer	99.740	0.190	0.0	7.8	7.8	531.5	Flood Risk
60 min Summer	99.786	0.236	0.0	7.8	7.8	659.3	Flood Risk
120 min Summer	99.831	0.281	0.0	7.8	7.8	785.7	Flood Risk
180 min Summer	99.856	0.306	0.0	7.8	7.8	855.6	Flood Risk
240 min Summer	99.872	0.322	0.0	7.8	7.8	900.6	Flood Risk
360 min Summer	99.889	0.339	0.0	7.8	7.8	948.6	Flood Risk
480 min Summer	99.896	0.346	0.0	7.8	7.8	966.0	Flood Risk
600 min Summer	99.896	0.346	0.0	7.8	7.8	967.1	Flood Risk
720 min Summer	99.893	0.343	0.0	7.8	7.8	958.3	Flood Risk
960 min Summer	99.880	0.330	0.0	7.8	7.8	922.1	Flood Risk
1440 min Summer	99.853	0.303	0.0	7.8	7.8	847.1	Flood Risk
2160 min Summer	99.824	0.274	0.0	7.8	7.8	764.9	Flood Risk
2880 min Summer	99.800	0.250	0.0	7.8	7.8	697.2	Flood Risk
4320 min Summer	99.758	0.208	0.0	7.8	7.8	582.0	Flood Risk
5760 min Summer	99.723	0.173	0.0	7.8	7.8	482.9	Flood Risk
15 min Winter	99.713	0.163	0.0	7.8	7.8	454.8	Flood Risk
30 min Winter	99.766	0.216	0.0	7.8	7.8	602.5	Flood Risk
60 min Winter	99.817	0.267	0.0	7.8	7.8	747.2	Flood Risk
120 min Winter	99.869	0.319	0.0	7.8	7.8	891.4	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	151.322	0.0	405.7	19
30 min Summer	98.513	0.0	542.3	34
60 min Summer	60.987	0.0	682.8	64
120 min Summer	36.823	0.0	833.3	124
180 min Summer	27.226	0.0	928.6	182
240 min Summer	21.895	0.0	998.1	242
360 min Summer	15.996	0.0	1096.4	362
480 min Summer	12.731	0.0	1164.8	482
600 min Summer	10.637	0.0	1198.0	602
720 min Summer	9.172	0.0	1196.8	720
960 min Summer	7.244	0.0	1194.5	934
1440 min Summer	5.181	0.0	1176.3	1140
2160 min Summer	3.701	0.0	1509.3	1516
2880 min Summer	2.920	0.0	1578.9	1932
4320 min Summer	2.104	0.0	1686.0	2724
5760 min Summer	1.677	0.0	1773.4	3520
15 min Winter	151.322	0.0	460.1	19
30 min Winter	98.513	0.0	585.6	33
60 min Winter	60.987	0.0	770.5	64
120 min Winter	36.823	0.0	939.2	122

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.899	0.349	0.0	7.8	7.8	974.4	Flood Risk
240 min Winter	99.918	0.368	0.0	7.8	7.8	1027.6	Flood Risk
360 min Winter	99.939	0.389	0.0	7.8	7.8	1088.5	Flood Risk
480 min Winter	99.949	0.399	0.0	7.8	7.8	1115.5	Flood Risk
600 min Winter	99.952	0.402	0.0	7.8	7.8	1124.3	Flood Risk
720 min Winter	99.952	0.402	0.0	7.8	7.8	1122.1	Flood Risk
960 min Winter	99.943	0.393	0.0	7.8	7.8	1097.7	Flood Risk
1440 min Winter	99.911	0.361	0.0	7.8	7.8	1010.1	Flood Risk
2160 min Winter	99.872	0.322	0.0	7.8	7.8	899.4	Flood Risk
2880 min Winter	99.837	0.287	0.0	7.8	7.8	803.1	Flood Risk
4320 min Winter	99.774	0.224	0.0	7.8	7.8	627.0	Flood Risk
5760 min Winter	99.720	0.170	0.0	7.8	7.8	473.9	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	27.226	0.0	1046.5	180
240 min Winter	21.895	0.0	1124.5	240
360 min Winter	15.996	0.0	1204.9	356
480 min Winter	12.731	0.0	1204.8	472
600 min Winter	10.637	0.0	1205.5	588
720 min Winter	9.172	0.0	1206.3	700
960 min Winter	7.244	0.0	1207.5	924
1440 min Winter	5.181	0.0	1209.3	1328
2160 min Winter	3.701	0.0	1701.6	1644
2880 min Winter	2.920	0.0	1779.5	2104
4320 min Winter	2.104	0.0	1904.9	2944
5760 min Winter	1.677	0.0	2005.8	3752

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.600

Time (mins) From:	To:	Area (ha)
0	4	1.600

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	90.0
Membrane Percolation (mm/hr)	1000	Length (m)	103.5
Max Percolation (l/s)	2587.5	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.550	Cap Volume Depth (m)	0.450

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0130-7800-1000-7800
Design Head (m)	1.000
Design Flow (l/s)	7.8
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	130
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	7.8	Kick-Flo®	0.664	6.4
Flush-Flo™	0.302	7.8	Mean Flow over Head Range	-	6.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.7	1.200	8.5	3.000	13.1	7.000	19.6
0.200	7.6	1.400	9.1	3.500	14.1	7.500	20.3
0.300	7.8	1.600	9.7	4.000	15.0	8.000	20.9
0.400	7.7	1.800	10.3	4.500	15.9	8.500	21.6
0.500	7.5	2.000	10.8	5.000	16.7	9.000	22.2
0.600	7.1	2.200	11.3	5.500	17.5	9.500	22.7
0.800	7.0	2.400	11.8	6.000	18.2		
1.000	7.8	2.600	12.2	6.500	18.9		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.081	1.081	16.1	432.2	O K
30 min Summer	99.295	1.295	16.1	553.5	O K
60 min Summer	99.473	1.473	16.1	664.8	O K
120 min Summer	99.612	1.612	16.1	758.0	O K
180 min Summer	99.666	1.666	16.1	796.4	O K
240 min Summer	99.684	1.684	16.1	809.3	O K
360 min Summer	99.663	1.663	16.1	794.3	O K
480 min Summer	99.612	1.612	16.1	758.0	O K
600 min Summer	99.568	1.568	16.1	727.9	O K
720 min Summer	99.528	1.528	16.1	700.8	O K
960 min Summer	99.454	1.454	16.1	652.3	O K
1440 min Summer	99.313	1.313	16.1	564.3	O K
2160 min Summer	99.106	1.106	16.1	445.7	O K
2880 min Summer	98.907	0.907	16.1	343.4	O K
4320 min Summer	98.551	0.551	16.1	185.3	O K
5760 min Summer	98.272	0.272	16.1	83.1	O K
15 min Winter	99.080	1.080	16.1	431.9	O K
30 min Winter	99.297	1.297	16.1	554.6	O K
60 min Winter	99.475	1.475	16.1	666.1	O K
120 min Winter	99.616	1.616	16.1	761.2	O K
180 min Winter	99.673	1.673	16.1	801.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.360	0.0	448.3	19
30 min Summer	67.940	0.0	583.6	33
60 min Summer	42.060	0.0	724.7	64
120 min Summer	25.395	0.0	875.0	122
180 min Summer	18.777	0.0	970.8	182
240 min Summer	15.100	0.0	1040.9	242
360 min Summer	11.032	0.0	1140.7	360
480 min Summer	8.780	0.0	1210.6	422
600 min Summer	7.336	0.0	1264.3	482
720 min Summer	6.326	0.0	1308.3	542
960 min Summer	4.996	0.0	1377.5	674
1440 min Summer	3.573	0.0	1477.7	940
2160 min Summer	2.552	0.0	1584.5	1344
2880 min Summer	2.014	0.0	1667.0	1728
4320 min Summer	1.451	0.0	1800.8	2424
5760 min Summer	1.157	0.0	1915.0	3120
15 min Winter	104.360	0.0	447.9	18
30 min Winter	67.940	0.0	583.9	33
60 min Winter	42.060	0.0	724.7	62
120 min Winter	25.395	0.0	875.2	120
180 min Winter	18.777	0.0	970.7	178

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.694	1.694	16.1	816.3	O K
360 min Winter	99.682	1.682	16.1	807.5	O K
480 min Winter	99.632	1.632	16.1	772.0	O K
600 min Winter	99.569	1.569	16.1	728.8	O K
720 min Winter	99.523	1.523	16.1	697.9	O K
960 min Winter	99.427	1.427	16.1	634.9	O K
1440 min Winter	99.220	1.220	16.1	509.4	O K
2160 min Winter	98.898	0.898	16.1	338.8	O K
2880 min Winter	98.584	0.584	16.1	198.8	O K
4320 min Winter	98.078	0.078	16.1	22.1	O K
5760 min Winter	98.000	0.000	14.0	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	15.100	0.0	1040.9	236
360 min Winter	11.032	0.0	1140.7	348
480 min Winter	8.780	0.0	1210.5	454
600 min Winter	7.336	0.0	1264.3	494
720 min Winter	6.326	0.0	1308.3	564
960 min Winter	4.996	0.0	1377.5	714
1440 min Winter	3.573	0.0	1477.8	1010
2160 min Winter	2.552	0.0	1584.5	1408
2880 min Winter	2.014	0.0	1667.0	1784
4320 min Winter	1.451	0.0	1800.9	2380
5760 min Winter	1.157	0.0	1915.3	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.450

Time (mins) From:	To:	Area (ha)
0	4	3.450

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	277.9	2.000	822.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0166-1610-2000-1610
Design Head (m)	2.000
Design Flow (l/s)	16.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	166
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	16.1	Kick-Flo®	1.226	12.8
Flush-Flo™	0.585	16.1	Mean Flow over Head Range	-	14.1

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.9	1.200	13.1	3.000	19.5	7.000	29.3
0.200	13.4	1.400	13.6	3.500	21.0	7.500	30.3
0.300	14.9	1.600	14.5	4.000	22.4	8.000	31.2
0.400	15.7	1.800	15.3	4.500	23.7	8.500	32.2
0.500	16.0	2.000	16.1	5.000	24.9	9.000	33.1
0.600	16.1	2.200	16.8	5.500	26.1	9.500	33.9
0.800	15.8	2.400	17.5	6.000	27.2		
1.000	15.0	2.600	18.2	6.500	28.3		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.143	1.143	8.3	223.0	O K
30 min Summer	99.339	1.339	8.3	285.6	O K
60 min Summer	99.500	1.500	8.3	343.0	O K
120 min Summer	99.623	1.623	8.3	391.1	O K
180 min Summer	99.671	1.671	8.3	410.9	O K
240 min Summer	99.687	1.687	8.3	417.6	O K
360 min Summer	99.668	1.668	8.3	409.9	O K
480 min Summer	99.623	1.623	8.3	391.1	O K
600 min Summer	99.584	1.584	8.3	375.6	O K
720 min Summer	99.548	1.548	8.3	361.6	O K
960 min Summer	99.483	1.483	8.3	336.6	O K
1440 min Summer	99.356	1.356	8.3	291.2	O K
2160 min Summer	99.166	1.166	8.3	230.0	O K
2880 min Summer	98.980	0.980	8.3	177.3	O K
4320 min Summer	98.629	0.629	8.3	95.7	O K
5760 min Summer	98.330	0.330	8.3	43.0	O K
15 min Winter	99.142	1.142	8.3	222.8	O K
30 min Winter	99.341	1.341	8.3	286.1	O K
60 min Winter	99.502	1.502	8.3	343.7	O K
120 min Winter	99.627	1.627	8.3	392.7	O K
180 min Winter	99.677	1.677	8.3	413.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.360	0.0	231.9	19
30 min Summer	67.940	0.0	301.8	33
60 min Summer	42.060	0.0	374.2	64
120 min Summer	25.395	0.0	451.8	122
180 min Summer	18.777	0.0	501.2	182
240 min Summer	15.100	0.0	537.4	242
360 min Summer	11.032	0.0	588.9	360
480 min Summer	8.780	0.0	624.9	422
600 min Summer	7.336	0.0	652.7	482
720 min Summer	6.326	0.0	675.4	542
960 min Summer	4.996	0.0	711.1	674
1440 min Summer	3.573	0.0	762.9	940
2160 min Summer	2.552	0.0	817.7	1344
2880 min Summer	2.014	0.0	860.3	1732
4320 min Summer	1.451	0.0	929.5	2424
5760 min Summer	1.157	0.0	988.1	3120
15 min Winter	104.360	0.0	231.7	19
30 min Winter	67.940	0.0	301.9	33
60 min Winter	42.060	0.0	374.2	62
120 min Winter	25.395	0.0	451.9	120
180 min Winter	18.777	0.0	501.2	178

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Summary of Results for 100 year Return Period

	Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240	min Winter	99.695	1.695	8.3	421.2	O K
360	min Winter	99.685	1.685	8.3	416.7	O K
480	min Winter	99.641	1.641	8.3	398.4	O K
600	min Winter	99.585	1.585	8.3	376.1	O K
720	min Winter	99.545	1.545	8.3	360.2	O K
960	min Winter	99.458	1.458	8.3	327.7	O K
1440	min Winter	99.271	1.271	8.3	262.9	O K
2160	min Winter	98.971	0.971	8.3	174.9	O K
2880	min Winter	98.664	0.664	8.3	102.8	O K
4320	min Winter	98.100	0.100	8.3	11.5	O K
5760	min Winter	98.000	0.000	7.2	0.0	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240	min Winter	15.100	0.0	537.4	236
360	min Winter	11.032	0.0	588.9	348
480	min Winter	8.780	0.0	624.9	454
600	min Winter	7.336	0.0	652.7	494
720	min Winter	6.326	0.0	675.4	564
960	min Winter	4.996	0.0	711.1	714
1440	min Winter	3.573	0.0	763.0	1010
2160	min Winter	2.552	0.0	817.7	1408
2880	min Winter	2.014	0.0	860.3	1788
4320	min Winter	1.451	0.0	929.5	2380
5760	min Winter	1.157	0.0	988.2	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.780

Time (mins) From:	To:	Area (ha)
0	4	1.780

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	108.1	2.000	501.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0119-8300-2000-8300
Design Head (m)	2.000
Design Flow (l/s)	8.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	119
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	8.3	Kick-Flo®	1.065	6.2
Flush-Flo™	0.520	7.8	Mean Flow over Head Range	-	7.0

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.2	1.200	6.5	3.000	10.0	7.000	15.0
0.200	6.7	1.400	7.0	3.500	10.8	7.500	15.5
0.300	7.4	1.600	7.5	4.000	11.5	8.000	16.0
0.400	7.7	1.800	7.9	4.500	12.2	8.500	16.5
0.500	7.8	2.000	8.3	5.000	12.8	9.000	16.9
0.600	7.8	2.200	8.7	5.500	13.4	9.500	17.4
0.800	7.5	2.400	9.0	6.000	14.0		
1.000	6.7	2.600	9.4	6.500	14.5		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.018	1.018	42.4	1136.1	O K
30 min Summer	99.249	1.249	42.4	1454.9	O K
60 min Summer	99.447	1.447	42.4	1747.1	O K
120 min Summer	99.604	1.604	42.4	1991.8	O K
180 min Summer	99.666	1.666	42.4	2092.4	O K
240 min Summer	99.687	1.687	42.4	2125.6	O K
360 min Summer	99.662	1.662	42.4	2085.4	O K
480 min Summer	99.603	1.603	42.4	1989.9	O K
600 min Summer	99.553	1.553	42.4	1910.7	O K
720 min Summer	99.507	1.507	42.4	1839.5	O K
960 min Summer	99.424	1.424	42.4	1711.9	O K
1440 min Summer	99.266	1.266	42.4	1480.1	O K
2160 min Summer	99.041	1.041	42.4	1167.5	O K
2880 min Summer	98.833	0.833	42.4	898.3	O K
4320 min Summer	98.479	0.479	42.4	483.1	O K
5760 min Summer	98.224	0.224	42.4	214.8	O K
15 min Winter	99.017	1.017	42.4	1135.2	O K
30 min Winter	99.251	1.251	42.4	1457.8	O K
60 min Winter	99.450	1.450	42.4	1751.1	O K
120 min Winter	99.609	1.609	42.4	2000.2	O K
180 min Winter	99.674	1.674	42.4	2105.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.360	0.0	1169.1	19
30 min Summer	67.940	0.0	1523.2	33
60 min Summer	42.060	0.0	1900.6	64
120 min Summer	25.395	0.0	2295.4	122
180 min Summer	18.777	0.0	2546.8	182
240 min Summer	15.100	0.0	2731.0	242
360 min Summer	11.032	0.0	2993.0	360
480 min Summer	8.780	0.0	3176.4	422
600 min Summer	7.336	0.0	3317.4	482
720 min Summer	6.326	0.0	3432.6	542
960 min Summer	4.996	0.0	3613.9	674
1440 min Summer	3.573	0.0	3876.1	940
2160 min Summer	2.552	0.0	4162.7	1340
2880 min Summer	2.014	0.0	4379.0	1728
4320 min Summer	1.451	0.0	4728.0	2424
5760 min Summer	1.157	0.0	5032.7	3120
15 min Winter	104.360	0.0	1168.0	18
30 min Winter	67.940	0.0	1524.0	33
60 min Winter	42.060	0.0	1900.6	62
120 min Winter	25.395	0.0	2295.8	120
180 min Winter	18.777	0.0	2546.7	178

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.698	1.698	42.4	2144.2	O K
360 min Winter	99.684	1.684	42.4	2120.3	O K
480 min Winter	99.625	1.625	42.4	2026.2	O K
600 min Winter	99.554	1.554	42.4	1912.9	O K
720 min Winter	99.502	1.502	42.4	1831.7	O K
960 min Winter	99.393	1.393	42.4	1665.5	O K
1440 min Winter	99.164	1.164	42.4	1334.8	O K
2160 min Winter	98.822	0.822	42.4	885.5	O K
2880 min Winter	98.510	0.510	42.4	517.6	O K
4320 min Winter	98.059	0.059	42.4	54.8	O K
5760 min Winter	98.000	0.000	36.8	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	15.100	0.0	2731.0	236
360 min Winter	11.032	0.0	2993.0	348
480 min Winter	8.780	0.0	3176.3	454
600 min Winter	7.336	0.0	3317.4	494
720 min Winter	6.326	0.0	3432.6	564
960 min Winter	4.996	0.0	3614.0	714
1440 min Winter	3.573	0.0	3876.7	1010
2160 min Winter	2.552	0.0	4162.8	1408
2880 min Winter	2.014	0.0	4379.1	1784
4320 min Winter	1.451	0.0	4728.7	2340
5760 min Winter	1.157	0.0	5035.2	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 9.070

Time (mins) From:	To:	Area (ha)
0	4	9.070

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	919.0	2.000	1790.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0261-4240-2000-4240
Design Head (m)	2.000
Design Flow (l/s)	42.4
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	261
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	2100

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	42.4	Kick-Flo®	1.309	34.6
Flush-Flo™	0.602	42.4	Mean Flow over Head Range	-	36.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	8.4	1.200	37.7	3.000	51.6	7.000	77.7
0.200	26.9	1.400	35.7	3.500	55.5	7.500	80.4
0.300	39.2	1.600	38.1	4.000	59.2	8.000	83.0
0.400	41.2	1.800	40.3	4.500	62.7	8.500	85.4
0.500	42.2	2.000	42.4	5.000	66.0	9.000	87.9
0.600	42.4	2.200	44.4	5.500	69.1	9.500	90.2
0.800	41.8	2.400	46.3	6.000	72.1		
1.000	40.5	2.600	48.1	6.500	75.0		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 2340 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.716	0.116	0.0	3.0	3.0	261.2	Flood Risk
30 min Summer	99.757	0.157	0.0	3.0	3.0	351.9	Flood Risk
60 min Summer	99.798	0.198	0.0	3.0	3.0	443.4	Flood Risk
120 min Summer	99.837	0.237	0.0	3.0	3.0	532.1	Flood Risk
180 min Summer	99.861	0.261	0.0	3.0	3.0	585.1	Flood Risk
240 min Summer	99.877	0.277	0.0	3.0	3.0	621.3	Flood Risk
360 min Summer	99.897	0.297	0.0	3.0	3.0	667.2	Flood Risk
480 min Summer	99.909	0.309	0.0	3.0	3.0	692.8	Flood Risk
600 min Summer	99.915	0.315	0.0	3.0	3.0	707.9	Flood Risk
720 min Summer	99.919	0.319	0.0	3.0	3.0	716.1	Flood Risk
960 min Summer	99.921	0.321	0.0	3.0	3.0	720.7	Flood Risk
1440 min Summer	99.913	0.313	0.0	3.0	3.0	702.6	Flood Risk
2160 min Summer	99.891	0.291	0.0	3.0	3.0	653.6	Flood Risk
2880 min Summer	99.874	0.274	0.0	3.0	3.0	615.3	Flood Risk
4320 min Summer	99.850	0.250	0.0	3.0	3.0	561.0	Flood Risk
5760 min Summer	99.831	0.231	0.0	3.0	3.0	517.3	Flood Risk
15 min Winter	99.733	0.133	0.0	3.0	3.0	297.4	Flood Risk
30 min Winter	99.778	0.178	0.0	3.0	3.0	399.2	Flood Risk
60 min Winter	99.824	0.224	0.0	3.0	3.0	502.3	Flood Risk
120 min Winter	99.869	0.269	0.0	3.0	3.0	602.8	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	146.102	0.0	215.3	19
30 min Summer	95.613	0.0	220.8	34
60 min Summer	59.508	0.0	440.7	64
120 min Summer	35.735	0.0	450.2	124
180 min Summer	26.385	0.0	456.6	184
240 min Summer	21.203	0.0	460.9	244
360 min Summer	15.486	0.0	466.2	362
480 min Summer	12.320	0.0	469.1	482
600 min Summer	10.291	0.0	470.7	602
720 min Summer	8.872	0.0	471.7	722
960 min Summer	7.009	0.0	472.4	962
1440 min Summer	5.009	0.0	470.8	1440
2160 min Summer	3.581	0.0	913.2	1860
2880 min Summer	2.828	0.0	901.6	2220
4320 min Summer	2.039	0.0	853.4	2984
5760 min Summer	1.627	0.0	1160.2	3808
15 min Winter	146.102	0.0	217.5	19
30 min Winter	95.613	0.0	223.7	34
60 min Winter	59.508	0.0	446.5	64
120 min Winter	35.735	0.0	458.2	122

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.896	0.296	0.0	3.0	3.0	663.2	Flood Risk
240 min Winter	99.914	0.314	0.0	3.0	3.0	705.3	Flood Risk
360 min Winter	99.938	0.338	0.0	3.0	3.0	759.5	Flood Risk
480 min Winter	99.952	0.352	0.0	3.0	3.0	790.9	Flood Risk
600 min Winter	99.961	0.361	0.0	3.0	3.0	810.0	Flood Risk
720 min Winter	99.966	0.366	0.0	3.0	3.0	822.3	Flood Risk
960 min Winter	99.971	0.371	0.0	3.0	3.0	833.4	Flood Risk
1440 min Winter	99.968	0.368	0.0	3.0	3.0	825.4	Flood Risk
2160 min Winter	99.949	0.349	0.0	3.0	3.0	783.7	Flood Risk
2880 min Winter	99.925	0.325	0.0	3.0	3.0	729.2	Flood Risk
4320 min Winter	99.893	0.293	0.0	3.0	3.0	657.3	Flood Risk
5760 min Winter	99.864	0.264	0.0	3.0	3.0	592.1	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	26.385	0.0	465.4	182
240 min Winter	21.203	0.0	469.8	240
360 min Winter	15.486	0.0	475.5	358
480 min Winter	12.320	0.0	478.4	476
600 min Winter	10.291	0.0	480.2	594
720 min Winter	8.872	0.0	480.8	710
960 min Winter	7.009	0.0	480.9	942
1440 min Winter	5.009	0.0	477.7	1398
2160 min Winter	3.581	0.0	933.7	2052
2880 min Winter	2.828	0.0	930.4	2396
4320 min Winter	2.039	0.0	892.7	3244
5760 min Winter	1.627	0.0	1315.5	4152

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.100

Time (mins) From:	To:	Area (ha)
0	4	1.100

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	88.0
Membrane Percolation (mm/hr)	1000	Length (m)	85.0
Max Percolation (l/s)	2077.8	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.600	Cap Volume Depth (m)	0.400

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0082-3000-1000-3000
Design Head (m)	1.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	82
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.0	Kick-Flo®	0.623	2.4
Flush-Flo™	0.297	3.0	Mean Flow over Head Range	-	2.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	1.200	3.3	3.000	5.0	7.000	7.4
0.200	2.9	1.400	3.5	3.500	5.4	7.500	7.7
0.300	3.0	1.600	3.7	4.000	5.7	8.000	7.9
0.400	2.9	1.800	3.9	4.500	6.0	8.500	8.2
0.500	2.8	2.000	4.1	5.000	6.3	9.000	8.4
0.600	2.5	2.200	4.3	5.500	6.6	9.500	8.6
0.800	2.7	2.400	4.5	6.000	6.9		
1.000	3.0	2.600	4.7	6.500	7.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.007	1.007	3.6	184.9	O K
30 min Summer	99.206	1.206	3.6	240.0	O K
60 min Summer	99.379	1.379	3.6	294.2	O K
120 min Summer	99.522	1.522	3.6	343.1	O K
180 min Summer	99.596	1.596	3.6	369.8	O K
240 min Summer	99.640	1.640	3.6	386.3	O K
360 min Summer	99.682	1.682	3.6	402.5	O K
480 min Summer	99.690	1.690	3.6	405.9	O K
600 min Summer	99.682	1.682	3.6	402.5	O K
720 min Summer	99.663	1.663	3.6	395.1	O K
960 min Summer	99.608	1.608	3.6	374.5	O K
1440 min Summer	99.521	1.521	3.6	342.7	O K
2160 min Summer	99.422	1.422	3.6	308.3	O K
2880 min Summer	99.334	1.334	3.6	279.5	O K
4320 min Summer	99.169	1.169	3.6	229.4	O K
5760 min Summer	99.015	1.015	3.6	186.8	O K
15 min Winter	99.007	1.007	3.6	184.9	O K
30 min Winter	99.206	1.206	3.6	240.1	O K
60 min Winter	99.379	1.379	3.6	294.1	O K
120 min Winter	99.524	1.524	3.6	343.7	O K
180 min Winter	99.598	1.598	3.6	370.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	187.2	19
30 min Summer	65.940	0.0	224.4	34
60 min Summer	41.040	0.0	307.4	64
120 min Summer	24.645	0.0	369.0	124
180 min Summer	18.197	0.0	408.4	182
240 min Summer	14.623	0.0	437.3	242
360 min Summer	10.680	0.0	477.7	362
480 min Summer	8.496	0.0	500.6	482
600 min Summer	7.097	0.0	508.9	602
720 min Summer	6.118	0.0	514.1	720
960 min Summer	4.834	0.0	521.4	894
1440 min Summer	3.455	0.0	520.6	1112
2160 min Summer	2.469	0.0	666.5	1496
2880 min Summer	1.950	0.0	701.9	1904
4320 min Summer	1.406	0.0	758.7	2724
5760 min Summer	1.122	0.0	807.6	3464
15 min Winter	100.760	0.0	187.2	19
30 min Winter	65.940	0.0	224.3	33
60 min Winter	41.040	0.0	307.2	62
120 min Winter	24.645	0.0	369.0	122
180 min Winter	18.197	0.0	408.5	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.643	1.643	3.6	387.4	O K
360 min Winter	99.686	1.686	3.6	404.3	O K
480 min Winter	99.697	1.697	3.6	408.6	O K
600 min Winter	99.692	1.692	3.6	406.6	O K
720 min Winter	99.677	1.677	3.6	400.6	O K
960 min Winter	99.629	1.629	3.6	382.4	O K
1440 min Winter	99.520	1.520	3.6	342.2	O K
2160 min Winter	99.392	1.392	3.6	298.2	O K
2880 min Winter	99.262	1.262	3.6	256.9	O K
4320 min Winter	99.000	1.000	3.6	183.0	O K
5760 min Winter	98.741	0.741	3.6	121.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.623	0.0	437.3	238
360 min Winter	10.680	0.0	477.6	356
480 min Winter	8.496	0.0	499.8	470
600 min Winter	7.097	0.0	507.2	584
720 min Winter	6.118	0.0	511.5	696
960 min Winter	4.834	0.0	518.0	914
1440 min Winter	3.455	0.0	521.5	1140
2160 min Winter	2.469	0.0	666.5	1584
2880 min Winter	1.950	0.0	701.9	2020
4320 min Winter	1.406	0.0	758.9	2856
5760 min Winter	1.122	0.0	807.6	3640

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.500

Time (mins) From:	To:	Area (ha)
0	4	1.500

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	116.3	2.000	458.7

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0077-3600-2000-3600
Design Head (m)	2.000
Design Flow (l/s)	3.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	77
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.6	Kick-Flo®	0.691	2.2
Flush-Flo™	0.339	2.7	Mean Flow over Head Range	-	2.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.2	1.200	2.8	3.000	4.3	7.000	6.5
0.200	2.6	1.400	3.1	3.500	4.7	7.500	6.7
0.300	2.7	1.600	3.2	4.000	5.0	8.000	6.9
0.400	2.7	1.800	3.4	4.500	5.3	8.500	7.1
0.500	2.7	2.000	3.6	5.000	5.5	9.000	7.3
0.600	2.5	2.200	3.8	5.500	5.8	9.500	7.5
0.800	2.4	2.400	3.9	6.000	6.0		
1.000	2.6	2.600	4.1	6.500	6.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.991	0.991	4.0	209.7	O K
30 min Summer	99.192	1.192	4.0	272.2	O K
60 min Summer	99.369	1.369	4.0	333.7	O K
120 min Summer	99.515	1.515	4.0	389.5	O K
180 min Summer	99.591	1.591	4.0	420.1	O K
240 min Summer	99.636	1.636	4.0	439.0	O K
360 min Summer	99.681	1.681	4.0	457.9	O K
480 min Summer	99.691	1.691	4.0	462.4	O K
600 min Summer	99.684	1.684	4.0	459.2	O K
720 min Summer	99.665	1.665	4.0	451.4	O K
960 min Summer	99.611	1.611	4.0	428.5	O K
1440 min Summer	99.522	1.522	4.0	392.3	O K
2160 min Summer	99.422	1.422	4.0	353.6	O K
2880 min Summer	99.334	1.334	4.0	321.4	O K
4320 min Summer	99.172	1.172	4.0	265.6	O K
5760 min Summer	99.018	1.018	4.0	217.7	O K
15 min Winter	98.991	0.991	4.0	209.7	O K
30 min Winter	99.192	1.192	4.0	272.3	O K
60 min Winter	99.368	1.368	4.0	333.7	O K
120 min Winter	99.517	1.517	4.0	390.1	O K
180 min Winter	99.593	1.593	4.0	421.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	211.8	19
30 min Summer	65.940	0.0	252.9	34
60 min Summer	41.040	0.0	348.2	64
120 min Summer	24.645	0.0	418.0	124
180 min Summer	18.197	0.0	462.5	182
240 min Summer	14.623	0.0	495.1	242
360 min Summer	10.680	0.0	539.9	362
480 min Summer	8.496	0.0	561.6	482
600 min Summer	7.097	0.0	569.3	602
720 min Summer	6.118	0.0	574.5	720
960 min Summer	4.834	0.0	582.5	914
1440 min Summer	3.455	0.0	581.1	1124
2160 min Summer	2.469	0.0	755.3	1496
2880 min Summer	1.950	0.0	795.3	1904
4320 min Summer	1.406	0.0	859.7	2724
5760 min Summer	1.122	0.0	915.3	3520
15 min Winter	100.760	0.0	211.8	19
30 min Winter	65.940	0.0	252.9	33
60 min Winter	41.040	0.0	348.0	62
120 min Winter	24.645	0.0	418.0	122
180 min Winter	18.197	0.0	462.6	180

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Summary of Results for 100 year Return Period

	Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240	min Winter	99.639	1.639	4.0	440.3	O K
360	min Winter	99.685	1.685	4.0	460.0	O K
480	min Winter	99.698	1.698	4.0	465.4	O K
600	min Winter	99.694	1.694	4.0	463.6	O K
720	min Winter	99.679	1.679	4.0	457.3	O K
960	min Winter	99.633	1.633	4.0	437.7	O K
1440	min Winter	99.522	1.522	4.0	392.0	O K
2160	min Winter	99.394	1.394	4.0	343.2	O K
2880	min Winter	99.265	1.265	4.0	297.1	O K
4320	min Winter	99.007	1.007	4.0	214.4	O K
5760	min Winter	98.754	0.754	4.0	145.3	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240	min Winter	14.623	0.0	495.1	238
360	min Winter	10.680	0.0	539.8	356
480	min Winter	8.496	0.0	560.4	470
600	min Winter	7.097	0.0	567.3	584
720	min Winter	6.118	0.0	571.6	696
960	min Winter	4.834	0.0	578.2	914
1440	min Winter	3.455	0.0	582.4	1142
2160	min Winter	2.469	0.0	755.3	1600
2880	min Winter	1.950	0.0	795.4	2048
4320	min Winter	1.406	0.0	859.9	2856
5760	min Winter	1.122	0.0	915.3	3640

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.700

Time (mins) From:	To:	Area (ha)
0	4	1.700

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	140.1	2.000	504.9

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0082-4000-2000-4000
Design Head (m)	2.000
Design Flow (l/s)	4.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	82
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	4.0	Kick-Flo®	0.729	2.5
Flush-Flo™	0.356	3.1	Mean Flow over Head Range	-	3.1

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	1.200	3.2	3.000	4.8	7.000	7.2
0.200	3.0	1.400	3.4	3.500	5.2	7.500	7.4
0.300	3.1	1.600	3.6	4.000	5.5	8.000	7.7
0.400	3.1	1.800	3.8	4.500	5.8	8.500	7.9
0.500	3.1	2.000	4.0	5.000	6.1	9.000	8.1
0.600	2.9	2.200	4.2	5.500	6.4	9.500	8.3
0.800	2.6	2.400	4.3	6.000	6.7		
1.000	2.9	2.600	4.5	6.500	6.9		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.924	0.924	8.3	431.6	O K
30 min Summer	99.137	1.137	8.3	560.3	O K
60 min Summer	99.329	1.329	8.3	686.9	O K
120 min Summer	99.491	1.491	8.3	801.4	O K
180 min Summer	99.575	1.575	8.3	864.1	O K
240 min Summer	99.626	1.626	8.3	902.8	O K
360 min Summer	99.675	1.675	8.3	941.3	O K
480 min Summer	99.686	1.686	8.3	950.0	O K
600 min Summer	99.677	1.677	8.3	942.8	O K
720 min Summer	99.656	1.656	8.3	926.3	O K
960 min Summer	99.594	1.594	8.3	878.8	O K
1440 min Summer	99.495	1.495	8.3	804.3	O K
2160 min Summer	99.383	1.383	8.3	724.5	O K
2880 min Summer	99.286	1.286	8.3	657.8	O K
4320 min Summer	99.108	1.108	8.3	542.2	O K
5760 min Summer	98.944	0.944	8.3	443.0	O K
15 min Winter	98.924	0.924	8.3	431.6	O K
30 min Winter	99.137	1.137	8.3	560.4	O K
60 min Winter	99.329	1.329	8.3	686.7	O K
120 min Winter	99.492	1.492	8.3	802.6	O K
180 min Winter	99.578	1.578	8.3	866.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	434.5	19
30 min Summer	65.940	0.0	562.0	34
60 min Summer	41.040	0.0	715.5	64
120 min Summer	24.645	0.0	858.9	124
180 min Summer	18.197	0.0	950.4	182
240 min Summer	14.623	0.0	1017.7	242
360 min Summer	10.680	0.0	1113.1	362
480 min Summer	8.496	0.0	1177.6	482
600 min Summer	7.097	0.0	1224.5	602
720 min Summer	6.118	0.0	1258.8	720
960 min Summer	4.834	0.0	1291.4	896
1440 min Summer	3.455	0.0	1269.3	1114
2160 min Summer	2.469	0.0	1554.1	1496
2880 min Summer	1.950	0.0	1636.4	1904
4320 min Summer	1.406	0.0	1768.5	2724
5760 min Summer	1.122	0.0	1884.0	3464
15 min Winter	100.760	0.0	434.5	19
30 min Winter	65.940	0.0	561.9	33
60 min Winter	41.040	0.0	715.2	62
120 min Winter	24.645	0.0	858.9	122
180 min Winter	18.197	0.0	950.7	180

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Summary of Results for 100 year Return Period

	Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240	min Winter	99.629	1.629	8.3	905.4	O K
360	min Winter	99.681	1.681	8.3	945.6	O K
480	min Winter	99.694	1.694	8.3	956.3	O K
600	min Winter	99.689	1.689	8.3	952.1	O K
720	min Winter	99.672	1.672	8.3	938.8	O K
960	min Winter	99.619	1.619	8.3	897.5	O K
1440	min Winter	99.494	1.494	8.3	803.6	O K
2160	min Winter	99.351	1.351	8.3	702.2	O K
2880	min Winter	99.209	1.209	8.3	606.7	O K
4320	min Winter	98.931	0.931	8.3	435.6	O K
5760	min Winter	98.669	0.669	8.3	292.8	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240	min Winter	14.623	0.0	1017.8	238
360	min Winter	10.680	0.0	1113.0	356
480	min Winter	8.496	0.0	1177.3	470
600	min Winter	7.097	0.0	1223.8	584
720	min Winter	6.118	0.0	1256.4	696
960	min Winter	4.834	0.0	1282.7	914
1440	min Winter	3.455	0.0	1273.2	1142
2160	min Winter	2.469	0.0	1554.1	1600
2880	min Winter	1.950	0.0	1636.4	2048
4320	min Winter	1.406	0.0	1769.2	2856
5760	min Winter	1.122	0.0	1884.0	3640

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.500

Time (mins) From:	To:	Area (ha)
0	4	3.500

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	365.2	2.000	884.7

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0119-8300-2000-8300
Design Head (m)	2.000
Design Flow (l/s)	8.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	119
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	8.3
Flush-Flo™	0.520	7.8
Kick-Flo®	1.065	6.2
Mean Flow over Head Range	-	7.0

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.2	1.200	6.5	3.000	10.0	7.000	15.0
0.200	6.7	1.400	7.0	3.500	10.8	7.500	15.5
0.300	7.4	1.600	7.5	4.000	11.5	8.000	16.0
0.400	7.7	1.800	7.9	4.500	12.2	8.500	16.5
0.500	7.8	2.000	8.3	5.000	12.8	9.000	16.9
0.600	7.8	2.200	8.7	5.500	13.4	9.500	17.4
0.800	7.5	2.400	9.0	6.000	14.0		
1.000	6.7	2.600	9.4	6.500	14.5		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 610 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.725	0.175	0.0	9.1	9.1	274.8	Flood Risk
30 min Summer	99.784	0.234	0.0	9.1	9.1	366.1	Flood Risk
60 min Summer	99.839	0.289	0.0	9.1	9.1	452.5	Flood Risk
120 min Summer	99.879	0.329	0.0	9.1	9.1	514.7	Flood Risk
180 min Summer	99.897	0.347	0.0	9.1	9.1	544.2	Flood Risk
240 min Summer	99.907	0.357	0.0	9.1	9.1	558.3	Flood Risk
360 min Summer	99.909	0.359	0.0	9.1	9.1	561.5	Flood Risk
480 min Summer	99.899	0.349	0.0	9.1	9.1	546.8	Flood Risk
600 min Summer	99.887	0.337	0.0	9.1	9.1	527.2	Flood Risk
720 min Summer	99.876	0.326	0.0	9.1	9.1	510.3	Flood Risk
960 min Summer	99.858	0.308	0.0	9.1	9.1	481.7	Flood Risk
1440 min Summer	99.828	0.278	0.0	9.1	9.1	435.4	Flood Risk
2160 min Summer	99.788	0.238	0.0	9.1	9.1	372.6	Flood Risk
2880 min Summer	99.752	0.202	0.0	9.1	9.1	315.9	Flood Risk
4320 min Summer	99.691	0.141	0.0	9.1	9.1	220.4	O K
5760 min Summer	99.643	0.093	0.0	9.1	9.1	145.9	O K
15 min Winter	99.749	0.199	0.0	9.1	9.1	311.9	Flood Risk
30 min Winter	99.815	0.265	0.0	9.1	9.1	415.3	Flood Risk
60 min Winter	99.878	0.328	0.0	9.1	9.1	514.1	Flood Risk
120 min Winter	99.924	0.374	0.0	9.1	9.1	586.3	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	137.344	0.0	282.1	19
30 min Summer	90.567	0.0	380.6	34
60 min Summer	56.594	0.0	481.4	64
120 min Summer	33.350	0.0	571.9	122
180 min Summer	24.452	0.0	631.8	182
240 min Summer	19.582	0.0	675.3	242
360 min Summer	14.239	0.0	738.2	360
480 min Summer	11.326	0.0	783.7	478
600 min Summer	9.473	0.0	819.6	520
720 min Summer	8.183	0.0	848.9	580
960 min Summer	6.491	0.0	897.7	704
1440 min Summer	4.695	0.0	972.1	970
2160 min Summer	3.403	0.0	1052.9	1384
2880 min Summer	2.718	0.0	1116.0	1764
4320 min Summer	1.996	0.0	1220.3	2552
5760 min Summer	1.615	0.0	1306.2	3240
15 min Winter	137.344	0.0	319.0	19
30 min Winter	90.567	0.0	429.1	33
60 min Winter	56.594	0.0	543.4	62
120 min Winter	33.350	0.0	643.6	120

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.948	0.398	0.0	9.1	9.1	623.9	Flood Risk
240 min Winter	99.961	0.411	0.0	9.1	9.1	644.4	Flood Risk
360 min Winter	99.970	0.420	0.0	9.1	9.1	657.2	Flood Risk
480 min Winter	99.965	0.415	0.0	9.1	9.1	650.1	Flood Risk
600 min Winter	99.954	0.404	0.0	9.1	9.1	633.2	Flood Risk
720 min Winter	99.940	0.390	0.0	9.1	9.1	610.4	Flood Risk
960 min Winter	99.914	0.364	0.0	9.1	9.1	569.6	Flood Risk
1440 min Winter	99.872	0.322	0.0	9.1	9.1	504.5	Flood Risk
2160 min Winter	99.811	0.261	0.0	9.1	9.1	408.5	Flood Risk
2880 min Winter	99.755	0.205	0.0	9.1	9.1	320.4	Flood Risk
4320 min Winter	99.662	0.112	0.0	9.1	9.1	174.7	O K
5760 min Winter	99.594	0.044	0.0	9.1	9.1	69.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	24.452	0.0	710.1	180
240 min Winter	19.582	0.0	759.9	238
360 min Winter	14.239	0.0	829.9	352
480 min Winter	11.326	0.0	880.9	464
600 min Winter	9.473	0.0	921.5	572
720 min Winter	8.183	0.0	955.6	672
960 min Winter	6.491	0.0	1010.1	754
1440 min Winter	4.695	0.0	1093.7	1056
2160 min Winter	3.403	0.0	1184.6	1496
2880 min Winter	2.718	0.0	1257.4	1904
4320 min Winter	1.996	0.0	1375.1	2680
5760 min Winter	1.615	0.0	1473.3	3344

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 351306 389946 SJ 51306 89946
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.200

Time (mins) From:	To:	Area (ha)
0	4	1.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	58.0
Membrane Percolation (mm/hr)	1000	Length (m)	90.0
Max Percolation (l/s)	1450.0	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.550	Cap Volume Depth (m)	0.450

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0140-9100-1000-9100
Design Head (m)	1.000
Design Flow (l/s)	9.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	140
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	9.1	Kick-Flo®	0.671	7.6
Flush-Flo™	0.304	9.1	Mean Flow over Head Range	-	7.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.0	1.200	9.9	3.000	15.3	7.000	23.0
0.200	8.8	1.400	10.7	3.500	16.5	7.500	23.7
0.300	9.1	1.600	11.4	4.000	17.6	8.000	24.5
0.400	9.0	1.800	12.0	4.500	18.6	8.500	25.2
0.500	8.8	2.000	12.6	5.000	19.5	9.000	25.9
0.600	8.3	2.200	13.2	5.500	20.4	9.500	26.6
0.800	8.2	2.400	13.8	6.000	21.3		
1.000	9.1	2.600	14.3	6.500	22.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.179	1.179	68.6	989.7	O K
30 min Summer	99.433	1.433	68.6	1270.2	O K
60 min Summer	99.625	1.625	68.6	1500.0	O K
120 min Summer	99.683	1.683	68.6	1572.1	O K
180 min Summer	99.655	1.655	68.6	1536.3	O K
240 min Summer	99.608	1.608	68.6	1478.7	O K
360 min Summer	99.529	1.529	68.6	1382.7	O K
480 min Summer	99.456	1.456	68.6	1297.2	O K
600 min Summer	99.383	1.383	68.6	1213.5	O K
720 min Summer	99.311	1.311	68.6	1132.4	O K
960 min Summer	99.166	1.166	68.6	976.3	O K
1440 min Summer	98.889	0.889	68.6	700.9	O K
2160 min Summer	98.522	0.522	68.6	379.5	O K
2880 min Summer	98.246	0.246	68.6	167.5	O K
4320 min Summer	98.000	0.000	67.4	0.0	O K
5760 min Summer	98.000	0.000	54.6	0.0	O K
15 min Winter	99.180	1.180	68.6	991.4	O K
30 min Winter	99.435	1.435	68.6	1273.2	O K
60 min Winter	99.632	1.632	68.6	1507.6	O K
120 min Winter	99.697	1.697	68.6	1588.6	O K
180 min Winter	99.678	1.678	68.6	1565.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	94.720	0.0	1059.3	18
30 min Summer	62.460	0.0	1399.6	33
60 min Summer	39.030	0.0	1753.1	62
120 min Summer	23.000	0.0	2067.3	122
180 min Summer	16.863	0.0	2273.8	180
240 min Summer	13.505	0.0	2428.1	206
360 min Summer	9.820	0.0	2648.5	268
480 min Summer	7.811	0.0	2809.0	334
600 min Summer	6.533	0.0	2936.7	404
720 min Summer	5.643	0.0	3044.1	470
960 min Summer	4.477	0.0	3220.0	606
1440 min Summer	3.238	0.0	3492.6	864
2160 min Summer	2.347	0.0	3800.2	1212
2880 min Summer	1.874	0.0	4046.0	1560
4320 min Summer	1.377	0.0	4460.0	0
5760 min Summer	1.114	0.0	4811.9	0
15 min Winter	94.720	0.0	1060.2	18
30 min Winter	62.460	0.0	1398.8	32
60 min Winter	39.030	0.0	1753.7	62
120 min Winter	23.000	0.0	2067.0	118
180 min Winter	16.863	0.0	2273.8	174

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.624	1.624	68.6	1498.1	O K
360 min Winter	99.520	1.520	68.6	1372.6	O K
480 min Winter	99.419	1.419	68.6	1254.1	O K
600 min Winter	99.310	1.310	68.6	1131.7	O K
720 min Winter	99.198	1.198	68.6	1010.3	O K
960 min Winter	98.971	0.971	68.6	779.8	O K
1440 min Winter	98.541	0.541	68.6	394.9	O K
2160 min Winter	98.059	0.059	68.6	38.8	O K
2880 min Winter	98.000	0.000	59.3	0.0	O K
4320 min Winter	98.000	0.000	43.5	0.0	O K
5760 min Winter	98.000	0.000	35.2	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	13.505	0.0	2428.0	226
360 min Winter	9.820	0.0	2648.5	280
480 min Winter	7.811	0.0	2809.0	356
600 min Winter	6.533	0.0	2936.7	430
720 min Winter	5.643	0.0	3044.3	500
960 min Winter	4.477	0.0	3220.0	638
1440 min Winter	3.238	0.0	3492.7	882
2160 min Winter	2.347	0.0	3800.4	1172
2880 min Winter	1.874	0.0	4048.2	0
4320 min Winter	1.377	0.0	4460.0	0
5760 min Winter	1.114	0.0	4811.8	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 351306 389946 SJ 51306 89946
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 9.000

Time (mins) From:	To:	Area (ha)
0	4	9.000

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	643.9	2.000	1396.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0324-6860-2000-6860
Design Head (m)	2.000
Design Flow (l/s)	68.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	324
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	375
Suggested Manhole Diameter (mm)	Site Specific Design (Contact Hydro International)

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	68.6	Kick-Flo®	1.368	57.1
Flush-Flo™	0.627	68.6	Mean Flow over Head Range	-	58.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	9.7	1.200	63.1	3.000	83.4	7.000	126.0
0.200	33.1	1.400	57.7	3.500	89.9	7.500	130.4
0.300	58.6	1.600	61.5	4.000	95.9	8.000	134.5
0.400	66.3	1.800	65.1	4.500	101.6	8.500	138.6
0.500	68.0	2.000	68.6	5.000	107.0	9.000	142.5
0.600	68.5	2.200	71.8	5.500	112.0	9.500	146.3
0.800	67.9	2.400	74.9	6.000	116.9		
1.000	66.3	2.600	77.8	6.500	121.6		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 2078 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.704	0.154	0.0	3.0	3.0	255.2	Flood Risk
30 min Summer	99.756	0.206	0.0	3.0	3.0	342.0	Flood Risk
60 min Summer	99.809	0.259	0.0	3.0	3.0	430.2	Flood Risk
120 min Summer	99.854	0.304	0.0	3.0	3.0	504.5	Flood Risk
180 min Summer	99.879	0.329	0.0	3.0	3.0	545.5	Flood Risk
240 min Summer	99.895	0.345	0.0	3.0	3.0	572.4	Flood Risk
360 min Summer	99.915	0.365	0.0	3.0	3.0	605.0	Flood Risk
480 min Summer	99.926	0.376	0.0	3.0	3.0	623.1	Flood Risk
600 min Summer	99.932	0.382	0.0	3.0	3.0	633.4	Flood Risk
720 min Summer	99.935	0.385	0.0	3.0	3.0	639.3	Flood Risk
960 min Summer	99.937	0.387	0.0	3.0	3.0	641.7	Flood Risk
1440 min Summer	99.928	0.378	0.0	3.0	3.0	626.5	Flood Risk
2160 min Summer	99.904	0.354	0.0	3.0	3.0	587.5	Flood Risk
2880 min Summer	99.887	0.337	0.0	3.0	3.0	558.8	Flood Risk
4320 min Summer	99.861	0.311	0.0	3.0	3.0	515.2	Flood Risk
5760 min Summer	99.837	0.287	0.0	3.0	3.0	476.6	Flood Risk
15 min Winter	99.724	0.174	0.0	3.0	3.0	289.3	Flood Risk
30 min Winter	99.783	0.233	0.0	3.0	3.0	386.8	Flood Risk
60 min Winter	99.843	0.293	0.0	3.0	3.0	486.3	Flood Risk
120 min Winter	99.894	0.344	0.0	3.0	3.0	570.7	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	152.250	0.0	216.0	19
30 min Summer	99.934	0.0	218.5	34
60 min Summer	62.408	0.0	439.0	64
120 min Summer	36.830	0.0	447.8	124
180 min Summer	26.840	0.0	453.1	184
240 min Summer	21.373	0.0	457.9	244
360 min Summer	15.433	0.0	464.0	362
480 min Summer	12.224	0.0	467.6	482
600 min Summer	10.198	0.0	470.3	602
720 min Summer	8.794	0.0	471.9	722
960 min Summer	6.969	0.0	474.3	962
1440 min Summer	5.031	0.0	475.8	1440
2160 min Summer	3.638	0.0	915.9	1800
2880 min Summer	2.896	0.0	900.0	2164
4320 min Summer	2.109	0.0	855.7	2944
5760 min Summer	1.691	0.0	1122.4	3808
15 min Winter	152.250	0.0	215.9	19
30 min Winter	99.934	0.0	222.3	34
60 min Winter	62.408	0.0	446.4	64
120 min Winter	36.830	0.0	456.1	122

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.922	0.372	0.0	3.0	3.0	617.9	Flood Risk
240 min Winter	99.942	0.392	0.0	3.0	3.0	649.6	Flood Risk
360 min Winter	99.965	0.415	0.0	3.0	3.0	688.9	Flood Risk
480 min Winter	99.979	0.429	0.0	3.0	3.0	711.8	Flood Risk
600 min Winter	99.988	0.438	0.0	3.0	3.0	726.3	Flood Risk
720 min Winter	99.993	0.443	0.0	3.0	3.0	735.7	Flood Risk
960 min Winter	99.999	0.449	0.0	3.0	3.0	744.6	Flood Risk
1440 min Winter	99.996	0.446	0.0	3.0	3.0	740.6	Flood Risk
2160 min Winter	99.975	0.425	0.0	3.0	3.0	705.5	Flood Risk
2880 min Winter	99.949	0.399	0.0	3.0	3.0	661.7	Flood Risk
4320 min Winter	99.913	0.363	0.0	3.0	3.0	601.6	Flood Risk
5760 min Winter	99.877	0.327	0.0	3.0	3.0	542.7	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	26.840	0.0	463.8	182
240 min Winter	21.373	0.0	468.7	240
360 min Winter	15.433	0.0	474.8	358
480 min Winter	12.224	0.0	478.6	476
600 min Winter	10.198	0.0	481.0	592
720 min Winter	8.794	0.0	482.4	708
960 min Winter	6.969	0.0	483.7	940
1440 min Winter	5.032	0.0	482.9	1388
2160 min Winter	3.638	0.0	932.3	2032
2880 min Winter	2.896	0.0	931.1	2308
4320 min Winter	2.109	0.0	898.0	3204
5760 min Winter	1.691	0.0	1269.4	4152

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 372232 388607 SJ 72232 88607
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.000

Time (mins) From:	To:	Area (ha)
0	4	1.000

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000
Membrane Percolation (mm/hr)	1000
Max Percolation (l/s)	1536.1
Safety Factor	2.0
Porosity	0.30
Invert Level (m)	99.550
Width (m)	70.0
Length (m)	79.0
Slope (1:X)	0.0
Depression Storage (mm)	5
Evaporation (mm/day)	3
Cap Volume Depth (m)	0.450

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0082-3000-1000-3000
Design Head (m)	1.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	82
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.0
Flush-Flo™	0.297	3.0
Kick-Flo®	0.623	2.4
Mean Flow over Head Range	-	2.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	0.800	2.7	2.000	4.1	4.000	5.7
0.200	2.9	1.000	3.0	2.200	4.3	4.500	6.0
0.300	3.0	1.200	3.3	2.400	4.5	5.000	6.3
0.400	2.9	1.400	3.5	2.600	4.7	5.500	6.6
0.500	2.8	1.600	3.7	3.000	5.0	6.000	6.9
0.600	2.5	1.800	3.9	3.500	5.4	6.500	7.2

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Hydro-Brake® Optimum Outflow Control

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
7.000	7.4	8.000	7.9	9.000	8.4		
7.500	7.7	8.500	8.2	9.500	8.6		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.030	1.030	4.1	221.2	O K
30 min Summer	99.239	1.239	4.1	288.1	O K
60 min Summer	99.425	1.425	4.1	354.7	O K
120 min Summer	99.558	1.558	4.1	406.6	O K
180 min Summer	99.621	1.621	4.1	432.4	O K
240 min Summer	99.656	1.656	4.1	447.4	O K
360 min Summer	99.686	1.686	4.1	460.2	O K
480 min Summer	99.689	1.689	4.1	461.6	O K
600 min Summer	99.679	1.679	4.1	457.0	O K
720 min Summer	99.659	1.659	4.1	448.7	O K
960 min Summer	99.608	1.608	4.1	427.1	O K
1440 min Summer	99.534	1.534	4.1	396.8	O K
2160 min Summer	99.449	1.449	4.1	363.9	O K
2880 min Summer	99.373	1.373	4.1	335.2	O K
4320 min Summer	99.225	1.225	4.1	283.5	O K
5760 min Summer	99.081	1.081	4.1	236.7	O K
15 min Winter	99.030	1.030	4.1	221.2	O K
30 min Winter	99.239	1.239	4.1	288.1	O K
60 min Winter	99.425	1.425	4.1	354.5	O K
120 min Winter	99.560	1.560	4.1	407.2	O K
180 min Winter	99.624	1.624	4.1	433.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	105.000	0.0	223.2	19
30 min Summer	68.920	0.0	261.4	34
60 min Summer	43.040	0.0	369.5	64
120 min Summer	25.400	0.0	435.9	124
180 min Summer	18.510	0.0	476.0	182
240 min Summer	14.740	0.0	505.2	242
360 min Summer	10.643	0.0	545.7	362
480 min Summer	8.430	0.0	571.8	482
600 min Summer	7.033	0.0	583.6	602
720 min Summer	6.065	0.0	589.8	720
960 min Summer	4.806	0.0	597.8	896
1440 min Summer	3.470	0.0	596.3	1112
2160 min Summer	2.509	0.0	776.5	1496
2880 min Summer	1.997	0.0	824.1	1904
4320 min Summer	1.455	0.0	899.6	2724
5760 min Summer	1.166	0.0	962.7	3520
15 min Winter	105.000	0.0	223.2	19
30 min Winter	68.920	0.0	261.4	33
60 min Winter	43.040	0.0	369.3	62
120 min Winter	25.400	0.0	435.9	122
180 min Winter	18.510	0.0	476.2	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.659	1.659	4.1	448.7	O K
360 min Winter	99.691	1.691	4.1	462.3	O K
480 min Winter	99.697	1.697	4.1	464.8	O K
600 min Winter	99.689	1.689	4.1	461.6	O K
720 min Winter	99.674	1.674	4.1	455.0	O K
960 min Winter	99.630	1.630	4.1	436.1	O K
1440 min Winter	99.533	1.533	4.1	396.4	O K
2160 min Winter	99.421	1.421	4.1	353.2	O K
2880 min Winter	99.305	1.305	4.1	310.8	O K
4320 min Winter	99.064	1.064	4.1	231.5	O K
5760 min Winter	98.820	0.820	4.1	162.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.740	0.0	505.2	238
360 min Winter	10.643	0.0	545.7	356
480 min Winter	8.430	0.0	571.2	470
600 min Winter	7.033	0.0	581.7	584
720 min Winter	6.065	0.0	586.8	696
960 min Winter	4.806	0.0	593.8	914
1440 min Winter	3.470	0.0	597.4	1142
2160 min Winter	2.509	0.0	776.5	1600
2880 min Winter	1.998	0.0	824.2	2048
4320 min Winter	1.455	0.0	899.9	2896
5760 min Winter	1.166	0.0	962.7	3688

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 372232 388607 SJ 72232 88607
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.720

Time (mins) From:	To:	Area (ha)
0	4	1.720

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	140.0	2.000	504.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0083-4100-2000-4100
Design Head (m)	2.000
Design Flow (l/s)	4.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	83
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	4.1	Kick-Flo®	0.738	2.6
Flush-Flo™	0.362	3.2	Mean Flow over Head Range	-	3.2

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	1.200	3.2	3.000	5.0	7.000	7.4
0.200	3.0	1.400	3.5	3.500	5.3	7.500	7.6
0.300	3.2	1.600	3.7	4.000	5.7	8.000	7.9
0.400	3.2	1.800	3.9	4.500	6.0	8.500	8.1
0.500	3.2	2.000	4.1	5.000	6.3	9.000	8.3
0.600	3.0	2.200	4.3	5.500	6.6	9.500	8.5
0.800	2.7	2.400	4.5	6.000	6.9		
1.000	3.0	2.600	4.6	6.500	7.1		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 1615 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.467	0.317	0.0	4.2	4.2	300.8	O K
30 min Summer	99.571	0.421	0.0	4.2	4.2	399.2	O K
60 min Summer	99.675	0.525	0.0	4.2	4.2	497.6	O K
120 min Summer	99.759	0.609	0.0	4.2	4.2	577.6	Flood Risk
180 min Summer	99.803	0.653	0.0	4.2	4.2	619.7	Flood Risk
240 min Summer	99.832	0.682	0.0	4.2	4.2	646.7	Flood Risk
360 min Summer	99.862	0.712	0.0	4.2	4.2	675.8	Flood Risk
480 min Summer	99.877	0.727	0.0	4.2	4.2	689.6	Flood Risk
600 min Summer	99.883	0.733	0.0	4.2	4.2	695.0	Flood Risk
720 min Summer	99.883	0.733	0.0	4.2	4.2	695.0	Flood Risk
960 min Summer	99.873	0.723	0.0	4.2	4.2	686.0	Flood Risk
1440 min Summer	99.830	0.680	0.0	4.2	4.2	645.2	Flood Risk
2160 min Summer	99.781	0.631	0.0	4.2	4.2	598.2	Flood Risk
2880 min Summer	99.745	0.595	0.0	4.2	4.2	564.2	Flood Risk
4320 min Summer	99.685	0.535	0.0	4.2	4.2	507.4	O K
5760 min Summer	99.632	0.482	0.0	4.2	4.2	457.0	O K
15 min Winter	99.508	0.358	0.0	4.2	4.2	339.2	O K
30 min Winter	99.624	0.474	0.0	4.2	4.2	449.9	O K
60 min Winter	99.741	0.591	0.0	4.2	4.2	561.0	Flood Risk
120 min Winter	99.838	0.688	0.0	4.2	4.2	652.7	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	155.440	0.0	304.2	19
30 min Summer	102.457	0.0	351.6	34
60 min Summer	64.018	0.0	511.6	64
120 min Summer	37.736	0.0	605.4	124
180 min Summer	27.468	0.0	662.2	184
240 min Summer	21.852	0.0	671.0	242
360 min Summer	15.752	0.0	668.3	362
480 min Summer	12.466	0.0	666.0	482
600 min Summer	10.395	0.0	664.3	602
720 min Summer	8.961	0.0	662.9	722
960 min Summer	7.100	0.0	661.5	960
1440 min Summer	5.112	0.0	659.5	1312
2160 min Summer	3.687	0.0	1064.7	1664
2880 min Summer	2.932	0.0	1125.6	2048
4320 min Summer	2.135	0.0	1191.2	2856
5760 min Summer	1.713	0.0	1303.7	3688
15 min Winter	155.440	0.0	342.6	19
30 min Winter	102.457	0.0	344.5	34
60 min Winter	64.018	0.0	575.1	64
120 min Winter	37.736	0.0	670.8	122

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.890	0.740	0.0	4.2	4.2	702.4	Flood Risk
240 min Winter	99.924	0.774	0.0	4.2	4.2	734.3	Flood Risk
360 min Winter	99.963	0.813	0.0	4.2	4.2	771.2	Flood Risk
480 min Winter	99.984	0.834	0.0	4.2	4.2	790.9	Flood Risk
600 min Winter	99.995	0.845	0.0	4.2	4.2	801.7	Flood Risk
720 min Winter	100.000	0.850	0.0	4.2	4.2	806.2	Flood Risk
960 min Winter	99.999	0.849	0.0	4.2	4.2	805.4	Flood Risk
1440 min Winter	99.969	0.819	0.0	4.2	4.2	776.8	Flood Risk
2160 min Winter	99.897	0.747	0.0	4.2	4.2	708.8	Flood Risk
2880 min Winter	99.850	0.700	0.0	4.2	4.2	664.4	Flood Risk
4320 min Winter	99.762	0.612	0.0	4.2	4.2	580.3	Flood Risk
5760 min Winter	99.678	0.528	0.0	4.2	4.2	500.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	27.468	0.0	668.5	180
240 min Winter	21.852	0.0	666.3	240
360 min Winter	15.752	0.0	663.4	358
480 min Winter	12.466	0.0	661.7	474
600 min Winter	10.395	0.0	660.5	590
720 min Winter	8.961	0.0	659.8	706
960 min Winter	7.100	0.0	659.3	934
1440 min Winter	5.112	0.0	662.3	1372
2160 min Winter	3.687	0.0	1195.9	1776
2880 min Winter	2.932	0.0	1265.8	2196
4320 min Winter	2.135	0.0	1230.2	3116
5760 min Winter	1.713	0.0	1465.1	3984

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 372706 391679 SJ 72706 91679
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 1.100

Time (mins) From:	To:	Area (ha)
0	4	1.100

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	51.0
Membrane Percolation (mm/hr)	1000	Length (m)	62.0
Max Percolation (l/s)	878.3	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.150	Cap Volume Depth (m)	0.850

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0097-4200-1000-4200
Design Head (m)	1.000
Design Flow (l/s)	4.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	97
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	4.2	Kick-Flo®	0.635	3.4
Flush-Flo™	0.299	4.2	Mean Flow over Head Range	-	3.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.2	1.200	4.6	3.000	7.0	7.000	10.4
0.200	4.1	1.400	4.9	3.500	7.5	7.500	10.8
0.300	4.2	1.600	5.2	4.000	8.0	8.000	11.1
0.400	4.1	1.800	5.5	4.500	8.5	8.500	11.5
0.500	4.0	2.000	5.8	5.000	8.9	9.000	11.8
0.600	3.6	2.200	6.0	5.500	9.3	9.500	12.1
0.800	3.8	2.400	6.3	6.000	9.7		
1.000	4.2	2.600	6.5	6.500	10.1		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.256	1.256	22.6	296.8	O K
30 min Summer	99.481	1.481	22.6	379.5	O K
60 min Summer	99.643	1.643	22.6	445.6	O K
120 min Summer	99.678	1.678	22.6	460.6	O K
180 min Summer	99.630	1.630	22.6	440.0	O K
240 min Summer	99.580	1.580	22.6	419.4	O K
360 min Summer	99.496	1.496	22.6	385.3	O K
480 min Summer	99.418	1.418	22.6	355.1	O K
600 min Summer	99.340	1.340	22.6	326.5	O K
720 min Summer	99.264	1.264	22.6	299.4	O K
960 min Summer	99.113	1.113	22.6	249.3	O K
1440 min Summer	98.814	0.814	22.6	162.5	O K
2160 min Summer	98.413	0.413	22.6	70.1	O K
2880 min Summer	98.123	0.123	22.6	18.4	O K
4320 min Summer	98.000	0.000	19.2	0.0	O K
5760 min Summer	98.000	0.000	15.4	0.0	O K
15 min Winter	99.258	1.258	22.6	297.4	O K
30 min Winter	99.484	1.484	22.6	380.7	O K
60 min Winter	99.648	1.648	22.6	448.0	O K
120 min Winter	99.692	1.692	22.6	466.7	O K
180 min Winter	99.651	1.651	22.6	449.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	107.200	0.0	321.0	18
30 min Summer	70.660	0.0	423.6	33
60 min Summer	44.150	0.0	529.6	62
120 min Summer	26.025	0.0	624.1	120
180 min Summer	18.943	0.0	681.8	170
240 min Summer	15.070	0.0	723.2	196
360 min Summer	10.863	0.0	782.0	258
480 min Summer	8.598	0.0	825.2	326
600 min Summer	7.169	0.0	860.1	394
720 min Summer	6.180	0.0	889.8	462
960 min Summer	4.897	0.0	940.0	590
1440 min Summer	3.526	0.0	1015.2	840
2160 min Summer	2.543	0.0	1098.5	1188
2880 min Summer	2.022	0.0	1164.7	1504
4320 min Summer	1.472	0.0	1272.1	0
5760 min Summer	1.182	0.0	1361.3	0
15 min Winter	107.200	0.0	321.3	18
30 min Winter	70.660	0.0	423.4	32
60 min Winter	44.150	0.0	529.7	62
120 min Winter	26.025	0.0	624.5	118
180 min Winter	18.943	0.0	681.8	172

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.582	1.582	22.6	420.2	O K
360 min Winter	99.478	1.478	22.6	378.5	O K
480 min Winter	99.366	1.366	22.6	336.0	O K
600 min Winter	99.249	1.249	22.6	294.1	O K
720 min Winter	99.128	1.128	22.6	254.1	O K
960 min Winter	98.885	0.885	22.6	181.5	O K
1440 min Winter	98.400	0.400	22.6	67.5	O K
2160 min Winter	98.000	0.000	21.4	0.0	O K
2880 min Winter	98.000	0.000	17.0	0.0	O K
4320 min Winter	98.000	0.000	12.4	0.0	O K
5760 min Winter	98.000	0.000	10.0	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	15.070	0.0	723.2	202
360 min Winter	10.863	0.0	782.0	274
480 min Winter	8.598	0.0	825.2	348
600 min Winter	7.169	0.0	860.1	422
720 min Winter	6.180	0.0	889.8	500
960 min Winter	4.897	0.0	940.0	626
1440 min Winter	3.526	0.0	1015.2	852
2160 min Winter	2.543	0.0	1098.6	0
2880 min Winter	2.022	0.0	1164.8	0
4320 min Winter	1.472	0.0	1272.1	0
5760 min Winter	1.182	0.0	1361.3	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 372706 391679 SJ 72706 91679
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.400

Time (mins) From:	To:	Area (ha)
0	4	2.400

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	141.8	2.000	508.2

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0195-2260-2000-2260
Design Head (m)	2.000
Design Flow (l/s)	22.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	195
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	22.6
Flush-Flo™	0.579	22.6
Kick-Flo®	1.240	18.0
Mean Flow over Head Range	-	19.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	6.8	1.200	18.8	3.000	27.4	7.000	41.2
0.200	18.2	1.400	19.1	3.500	29.5	7.500	42.6
0.300	21.0	1.600	20.3	4.000	31.5	8.000	44.0
0.400	22.0	1.800	21.5	4.500	33.3	8.500	45.3
0.500	22.5	2.000	22.6	5.000	35.1	9.000	46.6
0.600	22.6	2.200	23.6	5.500	36.7	9.500	47.8
0.800	22.2	2.400	24.6	6.000	38.3		
1.000	21.2	2.600	25.6	6.500	39.8		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.086	1.086	8.4	285.5	O K
30 min Summer	99.311	1.311	8.4	371.8	O K
60 min Summer	99.502	1.502	8.4	453.9	O K
120 min Summer	99.624	1.624	8.4	510.6	O K
180 min Summer	99.671	1.671	8.4	533.4	O K
240 min Summer	99.688	1.688	8.4	541.5	O K
360 min Summer	99.675	1.675	8.4	535.3	O K
480 min Summer	99.634	1.634	8.4	515.0	O K
600 min Summer	99.587	1.587	8.4	492.9	O K
720 min Summer	99.550	1.550	8.4	475.8	O K
960 min Summer	99.490	1.490	8.4	448.6	O K
1440 min Summer	99.387	1.387	8.4	403.5	O K
2160 min Summer	99.240	1.240	8.4	343.5	O K
2880 min Summer	99.097	1.097	8.4	289.4	O K
4320 min Summer	98.825	0.825	8.4	197.8	O K
5760 min Summer	98.579	0.579	8.4	126.7	O K
15 min Winter	99.086	1.086	8.4	285.5	O K
30 min Winter	99.312	1.312	8.4	372.1	O K
60 min Winter	99.503	1.503	8.4	454.3	O K
120 min Winter	99.628	1.628	8.4	512.2	O K
180 min Winter	99.676	1.676	8.4	535.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	107.200	0.0	293.8	19
30 min Summer	70.660	0.0	387.2	34
60 min Summer	44.150	0.0	485.2	64
120 min Summer	26.025	0.0	571.8	122
180 min Summer	18.943	0.0	624.6	182
240 min Summer	15.070	0.0	662.5	242
360 min Summer	10.863	0.0	716.2	360
480 min Summer	8.598	0.0	755.9	480
600 min Summer	7.169	0.0	787.9	526
720 min Summer	6.180	0.0	815.0	586
960 min Summer	4.897	0.0	861.0	706
1440 min Summer	3.526	0.0	929.6	980
2160 min Summer	2.543	0.0	1006.8	1384
2880 min Summer	2.022	0.0	1067.5	1788
4320 min Summer	1.472	0.0	1165.6	2552
5760 min Summer	1.182	0.0	1247.8	3240
15 min Winter	107.200	0.0	293.8	19
30 min Winter	70.660	0.0	387.2	33
60 min Winter	44.150	0.0	485.2	62
120 min Winter	26.025	0.0	571.9	120
180 min Winter	18.943	0.0	624.5	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.694	1.694	8.4	544.7	O K
360 min Winter	99.687	1.687	8.4	541.3	O K
480 min Winter	99.653	1.653	8.4	524.6	O K
600 min Winter	99.606	1.606	8.4	502.0	O K
720 min Winter	99.553	1.553	8.4	477.0	O K
960 min Winter	99.483	1.483	8.4	445.3	O K
1440 min Winter	99.338	1.338	8.4	382.9	O K
2160 min Winter	99.109	1.109	8.4	293.8	O K
2880 min Winter	98.878	0.878	8.4	214.6	O K
4320 min Winter	98.434	0.434	8.4	90.0	O K
5760 min Winter	98.078	0.078	8.4	14.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	15.070	0.0	662.5	236
360 min Winter	10.863	0.0	716.4	350
480 min Winter	8.598	0.0	755.9	462
600 min Winter	7.169	0.0	787.9	566
720 min Winter	6.180	0.0	815.0	644
960 min Winter	4.897	0.0	861.0	742
1440 min Winter	3.526	0.0	929.7	1040
2160 min Winter	2.543	0.0	1006.8	1472
2880 min Winter	2.022	0.0	1067.5	1876
4320 min Winter	1.472	0.0	1165.6	2596
5760 min Winter	1.182	0.0	1247.8	3120

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 372706 391679 SJ 72706 91679
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.200

Time (mins) From:	To:	Area (ha)
0	4	2.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	175.2	2.000	569.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0120-8400-2000-8400
Design Head (m)	2.000
Design Flow (l/s)	8.4
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	120
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	8.4	Kick-Flo®	1.066	6.2
Flush-Flo™	0.523	7.9	Mean Flow over Head Range	-	7.1

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.2	1.200	6.6	3.000	10.2	7.000	15.2
0.200	6.8	1.400	7.1	3.500	10.9	7.500	15.7
0.300	7.5	1.600	7.6	4.000	11.6	8.000	16.2
0.400	7.8	1.800	8.0	4.500	12.3	8.500	16.7
0.500	7.9	2.000	8.4	5.000	12.9	9.000	17.1
0.600	7.8	2.200	8.8	5.500	13.5	9.500	17.6
0.800	7.6	2.400	9.1	6.000	14.1		
1.000	6.7	2.600	9.5	6.500	14.7		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 928 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.637	0.187	0.0	5.6	5.6	229.2	O K
30 min Summer	99.698	0.248	0.0	5.6	5.6	303.9	O K
60 min Summer	99.759	0.309	0.0	5.6	5.6	378.4	Flood Risk
120 min Summer	99.816	0.366	0.0	5.6	5.6	448.6	Flood Risk
180 min Summer	99.847	0.397	0.0	5.6	5.6	485.6	Flood Risk
240 min Summer	99.864	0.414	0.0	5.6	5.6	506.5	Flood Risk
360 min Summer	99.880	0.430	0.0	5.6	5.6	526.2	Flood Risk
480 min Summer	99.881	0.431	0.0	5.6	5.6	528.2	Flood Risk
600 min Summer	99.876	0.426	0.0	5.6	5.6	521.1	Flood Risk
720 min Summer	99.865	0.415	0.0	5.6	5.6	508.0	Flood Risk
960 min Summer	99.841	0.391	0.0	5.6	5.6	478.9	Flood Risk
1440 min Summer	99.804	0.354	0.0	5.6	5.6	433.4	Flood Risk
2160 min Summer	99.759	0.309	0.0	5.6	5.6	378.4	Flood Risk
2880 min Summer	99.720	0.270	0.0	5.6	5.6	331.0	Flood Risk
4320 min Summer	99.654	0.204	0.0	5.6	5.6	250.2	O K
5760 min Summer	99.601	0.151	0.0	5.6	5.6	184.3	O K
7200 min Summer	99.559	0.109	0.0	5.6	5.6	133.2	O K
8640 min Summer	99.526	0.076	0.0	5.6	5.6	92.5	O K
10080 min Summer	99.500	0.050	0.0	5.6	5.6	61.5	O K
15 min Winter	99.662	0.212	0.0	5.6	5.6	259.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	154.292	0.0	233.3	19
30 min Summer	101.171	0.0	312.4	34
60 min Summer	63.304	0.0	396.1	64
120 min Summer	38.385	0.0	484.6	124
180 min Summer	28.390	0.0	539.7	182
240 min Summer	22.822	0.0	579.1	242
360 min Summer	16.665	0.0	635.7	362
480 min Summer	13.266	0.0	675.2	482
600 min Summer	11.085	0.0	705.5	600
720 min Summer	9.555	0.0	729.7	714
960 min Summer	7.536	0.0	766.7	810
1440 min Summer	5.365	0.0	816.4	1042
2160 min Summer	3.806	0.0	864.4	1452
2880 min Summer	2.987	0.0	901.1	1848
4320 min Summer	2.134	0.0	955.9	2640
5760 min Summer	1.692	0.0	1001.0	3400
7200 min Summer	1.424	0.0	1045.3	4112
8640 min Summer	1.243	0.0	1087.1	4760
10080 min Summer	1.111	0.0	1124.7	5536
15 min Winter	154.292	0.0	263.9	19

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
30 min Winter	99.731	0.281	0.0	5.6	5.6	344.1	Flood Risk
60 min Winter	99.800	0.350	0.0	5.6	5.6	428.7	Flood Risk
120 min Winter	99.867	0.417	0.0	5.6	5.6	509.9	Flood Risk
180 min Winter	99.902	0.452	0.0	5.6	5.6	553.8	Flood Risk
240 min Winter	99.924	0.474	0.0	5.6	5.6	579.8	Flood Risk
360 min Winter	99.946	0.496	0.0	5.6	5.6	607.1	Flood Risk
480 min Winter	99.953	0.503	0.0	5.6	5.6	615.3	Flood Risk
600 min Winter	99.951	0.501	0.0	5.6	5.6	613.2	Flood Risk
720 min Winter	99.944	0.494	0.0	5.6	5.6	604.7	Flood Risk
960 min Winter	99.920	0.470	0.0	5.6	5.6	575.7	Flood Risk
1440 min Winter	99.868	0.418	0.0	5.6	5.6	512.3	Flood Risk
2160 min Winter	99.807	0.357	0.0	5.6	5.6	436.5	Flood Risk
2880 min Winter	99.749	0.299	0.0	5.6	5.6	366.2	Flood Risk
4320 min Winter	99.650	0.200	0.0	5.6	5.6	244.3	O K
5760 min Winter	99.570	0.120	0.0	5.6	5.6	147.0	O K
7200 min Winter	99.511	0.061	0.0	5.6	5.6	74.5	O K
8640 min Winter	99.470	0.020	0.0	5.6	5.6	24.5	O K
10080 min Winter	99.450	0.000	0.0	5.6	5.6	0.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
30 min Winter	101.171	0.0	352.6	33
60 min Winter	63.304	0.0	446.3	62
120 min Winter	38.385	0.0	545.3	122
180 min Winter	28.390	0.0	607.2	180
240 min Winter	22.822	0.0	651.8	238
360 min Winter	16.665	0.0	715.3	354
480 min Winter	13.266	0.0	759.5	470
600 min Winter	11.085	0.0	793.2	584
720 min Winter	9.555	0.0	820.2	694
960 min Winter	7.536	0.0	862.3	906
1440 min Winter	5.365	0.0	853.9	1126
2160 min Winter	3.806	0.0	973.9	1580
2880 min Winter	2.987	0.0	1013.9	2020
4320 min Winter	2.134	0.0	1077.6	2812
5760 min Winter	1.692	0.0	1130.3	3576
7200 min Winter	1.424	0.0	1180.9	4256
8640 min Winter	1.243	0.0	1228.6	4760
10080 min Winter	1.111	0.0	1274.0	5144

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369669 370032 SJ 69669 70032
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+45

Time Area Diagram

Total Area (ha) 0.880

Time (mins) From:	To:	Area (ha)
0	4	0.880

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	53.0
Membrane Percolation (mm/hr)	1000	Length (m)	77.0
Max Percolation (l/s)	1133.6	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.450	Cap Volume Depth (m)	0.550

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0111-5600-1000-5600
Design Head (m)	1.000
Design Flow (l/s)	5.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	111
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	5.6	Kick-Flo®	0.641	4.6
Flush-Flo™	0.294	5.6	Mean Flow over Head Range	-	4.9

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.9	1.200	6.1	3.000	9.4	7.000	14.0
0.200	5.4	1.400	6.6	3.500	10.1	7.500	14.5
0.300	5.6	1.600	7.0	4.000	10.7	8.000	15.0
0.400	5.5	1.800	7.4	4.500	11.4	8.500	15.4
0.500	5.3	2.000	7.7	5.000	11.9	9.000	15.8
0.600	4.9	2.200	8.1	5.500	12.5	9.500	16.2
0.800	5.0	2.400	8.4	6.000	13.0		
1.000	5.6	2.600	8.8	6.500	13.5		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.101	1.101	61.4	319.7	O K
30 min Summer	99.328	1.328	61.4	415.1	O K
60 min Summer	99.515	1.515	61.4	501.9	O K
120 min Summer	99.645	1.645	61.4	567.4	O K
180 min Summer	99.679	1.679	61.4	585.4	O K
240 min Summer	99.649	1.649	61.4	569.6	O K
360 min Summer	99.554	1.554	61.4	521.1	O K
480 min Summer	99.447	1.447	61.4	469.3	O K
600 min Summer	99.334	1.334	61.4	417.5	O K
720 min Summer	99.281	1.281	61.4	394.4	O K
960 min Summer	99.262	1.262	61.4	385.9	O K
1440 min Summer	98.860	0.860	61.4	230.2	O K
2160 min Summer	98.000	0.000	51.3	0.0	O K
2880 min Summer	98.000	0.000	48.6	0.0	O K
4320 min Summer	98.000	0.000	47.2	0.0	O K
5760 min Summer	98.000	0.000	46.6	0.0	O K
7200 min Summer	98.000	0.000	46.3	0.0	O K
8640 min Summer	98.000	0.000	46.2	0.0	O K
10080 min Summer	98.000	0.000	46.0	0.0	O K
15 min Winter	99.103	1.103	61.4	320.6	O K
30 min Winter	99.332	1.332	61.4	416.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	106.800	0.0	2418.2	19
30 min Summer	70.000	0.0	2526.4	33
60 min Summer	43.800	0.0	2653.2	64
120 min Summer	26.550	0.0	2774.3	122
180 min Summer	19.633	0.0	2850.1	180
240 min Summer	15.775	0.0	2904.5	230
360 min Summer	11.500	0.0	2981.4	272
480 min Summer	9.145	0.0	3035.5	332
600 min Summer	7.634	0.0	3076.8	410
720 min Summer	6.575	0.0	3110.1	492
960 min Summer	5.176	0.0	3161.1	636
1440 min Summer	3.675	0.0	3231.0	780
2160 min Summer	2.599	0.0	3302.4	0
2880 min Summer	2.035	0.0	3356.1	0
4320 min Summer	1.450	0.0	3443.2	0
5760 min Summer	1.148	0.0	3520.1	0
7200 min Summer	0.966	0.0	3594.2	0
8640 min Summer	0.844	0.0	3666.7	0
10080 min Summer	0.756	0.0	3738.8	0
15 min Winter	106.800	0.0	2418.3	18
30 min Winter	70.000	0.0	2526.2	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.523	1.523	61.4	506.2	O K
120 min Winter	99.656	1.656	61.4	573.4	O K
180 min Winter	99.694	1.694	61.4	593.4	O K
240 min Winter	99.669	1.669	61.4	579.8	O K
360 min Winter	99.560	1.560	61.4	524.4	O K
480 min Winter	99.428	1.428	61.4	460.5	O K
600 min Winter	99.302	1.302	61.4	403.5	O K
720 min Winter	99.218	1.218	61.4	367.3	O K
960 min Winter	99.134	1.134	61.4	332.8	O K
1440 min Winter	98.610	0.610	61.4	149.8	O K
2160 min Winter	98.000	0.000	54.3	0.0	O K
2880 min Winter	98.000	0.000	49.0	0.0	O K
4320 min Winter	98.000	0.000	47.4	0.0	O K
5760 min Winter	98.000	0.000	46.9	0.0	O K
7200 min Winter	98.000	0.000	46.6	0.0	O K
8640 min Winter	98.000	0.000	46.3	0.0	O K
10080 min Winter	98.000	0.000	46.1	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.800	0.0	2653.8	62
120 min Winter	26.550	0.0	2774.7	120
180 min Winter	19.633	0.0	2849.9	178
240 min Winter	15.775	0.0	2904.6	230
360 min Winter	11.500	0.0	2981.4	282
480 min Winter	9.145	0.0	3035.5	352
600 min Winter	7.634	0.0	3076.8	442
720 min Winter	6.575	0.0	3110.1	528
960 min Winter	5.176	0.0	3161.0	690
1440 min Winter	3.675	0.0	3231.0	764
2160 min Winter	2.599	0.0	3302.4	0
2880 min Winter	2.035	0.0	3356.1	0
4320 min Winter	1.450	0.0	3443.2	0
5760 min Winter	1.148	0.0	3520.1	0
7200 min Winter	0.966	0.0	3594.2	0
8640 min Winter	0.844	0.0	3666.7	0
10080 min Winter	0.756	0.0	3738.8	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369500 369800 SJ 69500 69800
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.600

Time (mins) From:	To:	Area (ha)
0	4	2.600

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	196.8	2.000	608.2

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0309-6160-2000-6160
Design Head (m)	2.000
Design Flow (l/s)	61.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	309
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	375
Suggested Manhole Diameter (mm)	Site Specific Design (Contact Hydro International)

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	61.4	Kick-Flo®	1.360	51.0
Flush-Flo™	0.625	61.4	Mean Flow over Head Range	-	52.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	9.4	1.200	56.4	3.000	74.8	7.000	112.9
0.200	31.7	1.400	51.7	3.500	80.6	7.500	116.8
0.300	54.6	1.600	55.2	4.000	86.0	8.000	120.5
0.400	59.4	1.800	58.4	4.500	91.0	8.500	124.1
0.500	60.9	2.000	61.4	5.000	95.8	9.000	127.6
0.600	61.4	2.200	64.3	5.500	100.4	9.500	131.0
0.800	60.8	2.400	67.1	6.000	104.7		
1.000	59.3	2.600	69.8	6.500	108.9		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.020	1.020	45.1	896.6	O K
30 min Summer	99.273	1.273	45.1	1152.6	O K
60 min Summer	99.491	1.491	45.1	1384.2	O K
120 min Summer	99.646	1.646	45.1	1555.8	O K
180 min Summer	99.686	1.686	45.1	1601.3	O K
240 min Summer	99.676	1.676	45.1	1589.9	O K
360 min Summer	99.600	1.600	45.1	1505.1	O K
480 min Summer	99.534	1.534	45.1	1431.3	O K
600 min Summer	99.472	1.472	45.1	1364.1	O K
720 min Summer	99.412	1.412	45.1	1299.7	O K
960 min Summer	99.292	1.292	45.1	1172.5	O K
1440 min Summer	99.058	1.058	45.1	933.9	O K
2160 min Summer	98.737	0.737	45.1	626.2	O K
2880 min Summer	98.470	0.470	45.1	386.9	O K
4320 min Summer	98.124	0.124	45.1	97.8	O K
5760 min Summer	98.000	0.000	44.4	0.0	O K
7200 min Summer	98.000	0.000	37.3	0.0	O K
8640 min Summer	98.000	0.000	32.6	0.0	O K
10080 min Summer	98.000	0.000	29.2	0.0	O K
15 min Winter	99.022	1.022	45.1	898.1	O K
30 min Winter	99.276	1.276	45.1	1155.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	106.800	0.0	937.5	19
30 min Summer	70.000	0.0	1232.1	33
60 min Summer	43.800	0.0	1548.9	62
120 min Summer	26.550	0.0	1880.4	122
180 min Summer	19.633	0.0	2086.1	182
240 min Summer	15.775	0.0	2234.7	240
360 min Summer	11.500	0.0	2444.3	312
480 min Summer	9.145	0.0	2591.8	372
600 min Summer	7.634	0.0	2704.5	446
720 min Summer	6.575	0.0	2795.1	500
960 min Summer	5.176	0.0	2933.9	636
1440 min Summer	3.675	0.0	3123.9	898
2160 min Summer	2.599	0.0	3318.8	1280
2880 min Summer	2.035	0.0	3464.8	1640
4320 min Summer	1.450	0.0	3699.6	2292
5760 min Summer	1.148	0.0	3912.1	0
7200 min Summer	0.966	0.0	4114.4	0
8640 min Summer	0.844	0.0	4312.4	0
10080 min Summer	0.756	0.0	4509.3	0
15 min Winter	106.800	0.0	938.3	18
30 min Winter	70.000	0.0	1232.4	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.496	1.496	45.1	1390.0	O K
120 min Winter	99.654	1.654	45.1	1565.4	O K
180 min Winter	99.699	1.699	45.1	1616.7	O K
240 min Winter	99.697	1.697	45.1	1613.7	O K
360 min Winter	99.621	1.621	45.1	1528.0	O K
480 min Winter	99.532	1.532	45.1	1429.3	O K
600 min Winter	99.455	1.455	45.1	1346.0	O K
720 min Winter	99.374	1.374	45.1	1258.4	O K
960 min Winter	99.201	1.201	45.1	1078.5	O K
1440 min Winter	98.857	0.857	45.1	738.8	O K
2160 min Winter	98.402	0.402	45.1	327.8	O K
2880 min Winter	98.084	0.084	45.1	66.0	O K
4320 min Winter	98.000	0.000	36.2	0.0	O K
5760 min Winter	98.000	0.000	28.6	0.0	O K
7200 min Winter	98.000	0.000	24.1	0.0	O K
8640 min Winter	98.000	0.000	21.0	0.0	O K
10080 min Winter	98.000	0.000	18.9	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.800	0.0	1550.5	62
120 min Winter	26.550	0.0	1880.4	120
180 min Winter	19.633	0.0	2085.7	176
240 min Winter	15.775	0.0	2234.9	234
360 min Winter	11.500	0.0	2444.3	338
480 min Winter	9.145	0.0	2591.8	382
600 min Winter	7.634	0.0	2704.5	458
720 min Winter	6.575	0.0	2795.1	534
960 min Winter	5.176	0.0	2934.1	676
1440 min Winter	3.675	0.0	3124.1	954
2160 min Winter	2.599	0.0	3318.9	1320
2880 min Winter	2.035	0.0	3465.0	1588
4320 min Winter	1.450	0.0	3706.1	0
5760 min Winter	1.148	0.0	3912.1	0
7200 min Winter	0.966	0.0	4114.4	0
8640 min Winter	0.844	0.0	4312.4	0
10080 min Winter	0.756	0.0	4509.3	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369500 369800 SJ 69500 69800
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 7.100

Time (mins) From:	To:	Area (ha)
0	4	7.100

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	776.8	2.000	1208.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0269-4510-2000-4510
Design Head (m)	2.000
Design Flow (l/s)	45.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	269
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	2100

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	45.1	Kick-Flo®	1.312	36.8
Flush-Flo™	0.599	45.1	Mean Flow over Head Range	-	38.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	8.5	1.200	40.1	3.000	54.8	7.000	82.7
0.200	27.6	1.400	38.0	3.500	59.0	7.500	85.5
0.300	41.7	1.600	40.5	4.000	63.0	8.000	88.2
0.400	43.8	1.800	42.8	4.500	66.7	8.500	90.9
0.500	44.8	2.000	45.1	5.000	70.2	9.000	93.4
0.600	45.1	2.200	47.2	5.500	73.5	9.500	95.9
0.800	44.5	2.400	49.2	6.000	76.7		
1.000	43.1	2.600	51.1	6.500	79.7		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.095	1.095	11.8	226.9	O K
30 min Summer	99.328	1.328	11.8	291.5	O K
60 min Summer	99.521	1.521	11.8	349.6	O K
120 min Summer	99.652	1.652	11.8	391.7	O K
180 min Summer	99.683	1.683	11.8	402.0	O K
240 min Summer	99.670	1.670	11.8	397.8	O K
360 min Summer	99.604	1.604	11.8	375.9	O K
480 min Summer	99.545	1.545	11.8	357.2	O K
600 min Summer	99.490	1.490	11.8	340.1	O K
720 min Summer	99.436	1.436	11.8	323.4	O K
960 min Summer	99.325	1.325	11.8	290.5	O K
1440 min Summer	99.103	1.103	11.8	229.0	O K
2160 min Summer	98.784	0.784	11.8	150.1	O K
2880 min Summer	98.504	0.504	11.8	89.8	O K
4320 min Summer	98.119	0.119	11.8	19.1	O K
5760 min Summer	98.000	0.000	11.2	0.0	O K
7200 min Summer	98.000	0.000	9.5	0.0	O K
8640 min Summer	98.000	0.000	8.3	0.0	O K
10080 min Summer	98.000	0.000	7.4	0.0	O K
15 min Winter	99.097	1.097	11.8	227.3	O K
30 min Winter	99.331	1.331	11.8	292.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	106.800	0.0	239.5	19
30 min Summer	70.000	0.0	314.3	33
60 min Summer	43.800	0.0	393.6	62
120 min Summer	26.550	0.0	477.7	122
180 min Summer	19.633	0.0	529.8	182
240 min Summer	15.775	0.0	567.6	240
360 min Summer	11.500	0.0	620.7	308
480 min Summer	9.145	0.0	658.2	368
600 min Summer	7.634	0.0	686.8	430
720 min Summer	6.575	0.0	709.8	498
960 min Summer	5.176	0.0	745.1	628
1440 min Summer	3.675	0.0	793.4	896
2160 min Summer	2.599	0.0	842.0	1276
2880 min Summer	2.035	0.0	879.1	1616
4320 min Summer	1.450	0.0	939.2	2292
5760 min Summer	1.148	0.0	991.8	0
7200 min Summer	0.966	0.0	1043.1	0
8640 min Summer	0.844	0.0	1093.3	0
10080 min Summer	0.756	0.0	1143.2	0
15 min Winter	106.800	0.0	239.7	18
30 min Winter	70.000	0.0	314.4	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.526	1.526	11.8	351.1	O K
120 min Winter	99.660	1.660	11.8	394.3	O K
180 min Winter	99.695	1.695	11.8	406.1	O K
240 min Winter	99.690	1.690	11.8	404.2	O K
360 min Winter	99.618	1.618	11.8	380.5	O K
480 min Winter	99.542	1.542	11.8	356.2	O K
600 min Winter	99.472	1.472	11.8	334.4	O K
720 min Winter	99.397	1.397	11.8	311.5	O K
960 min Winter	99.235	1.235	11.8	264.8	O K
1440 min Winter	98.898	0.898	11.8	177.1	O K
2160 min Winter	98.420	0.420	11.8	73.2	O K
2880 min Winter	98.065	0.065	11.8	10.3	O K
4320 min Winter	98.000	0.000	9.2	0.0	O K
5760 min Winter	98.000	0.000	7.3	0.0	O K
7200 min Winter	98.000	0.000	6.1	0.0	O K
8640 min Winter	98.000	0.000	5.3	0.0	O K
10080 min Winter	98.000	0.000	4.8	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.800	0.0	394.0	62
120 min Winter	26.550	0.0	477.7	120
180 min Winter	19.633	0.0	529.8	176
240 min Winter	15.775	0.0	567.6	232
360 min Winter	11.500	0.0	620.7	336
480 min Winter	9.145	0.0	658.2	378
600 min Winter	7.634	0.0	686.8	454
720 min Winter	6.575	0.0	709.8	530
960 min Winter	5.176	0.0	745.1	676
1440 min Winter	3.675	0.0	793.4	952
2160 min Winter	2.599	0.0	842.0	1300
2880 min Winter	2.035	0.0	879.2	1588
4320 min Winter	1.450	0.0	939.6	0
5760 min Winter	1.148	0.0	991.8	0
7200 min Winter	0.966	0.0	1043.1	0
8640 min Winter	0.844	0.0	1093.3	0
10080 min Winter	0.756	0.0	1143.2	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369500 369800 SJ 69500 69800
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.800

Time (mins) From:	To:	Area (ha)
0	4	1.800

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	155.4	2.000	374.5

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0142-1180-2000-1180
Design Head (m)	2.000
Design Flow (l/s)	11.8
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	142
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	11.8	Kick-Flo®	1.213	9.3
Flush-Flo™	0.589	11.8	Mean Flow over Head Range	-	10.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.1	1.200	9.5	3.000	14.3	7.000	21.4
0.200	9.8	1.400	10.0	3.500	15.4	7.500	22.2
0.300	10.9	1.600	10.6	4.000	16.4	8.000	22.9
0.400	11.5	1.800	11.2	4.500	17.3	8.500	23.5
0.500	11.7	2.000	11.8	5.000	18.2	9.000	24.2
0.600	11.8	2.200	12.3	5.500	19.1	9.500	24.8
0.800	11.6	2.400	12.9	6.000	19.9		
1.000	10.9	2.600	13.4	6.500	20.7		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.092	1.092	37.2	502.7	O K
30 min Summer	99.324	1.324	37.2	647.0	O K
60 min Summer	99.517	1.517	37.2	778.0	O K
120 min Summer	99.652	1.652	37.2	875.4	O K
180 min Summer	99.686	1.686	37.2	900.7	O K
240 min Summer	99.673	1.673	37.2	891.4	O K
360 min Summer	99.602	1.602	37.2	838.4	O K
480 min Summer	99.535	1.535	37.2	790.2	O K
600 min Summer	99.468	1.468	37.2	743.8	O K
720 min Summer	99.410	1.410	37.2	703.6	O K
960 min Summer	99.311	1.311	37.2	638.3	O K
1440 min Summer	99.034	1.034	37.2	468.8	O K
2160 min Summer	98.365	0.365	37.2	138.1	O K
2880 min Summer	98.103	0.103	37.2	36.3	O K
4320 min Summer	98.000	0.000	31.6	0.0	O K
5760 min Summer	98.000	0.000	25.0	0.0	O K
7200 min Summer	98.000	0.000	21.0	0.0	O K
8640 min Summer	98.000	0.000	18.4	0.0	O K
10080 min Summer	98.000	0.000	16.5	0.0	O K
15 min Winter	99.093	1.093	37.2	503.5	O K
30 min Winter	99.326	1.326	37.2	648.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	106.800	0.0	1058.8	19
30 min Summer	70.000	0.0	1225.2	33
60 min Summer	43.800	0.0	1404.3	62
120 min Summer	26.550	0.0	1590.6	122
180 min Summer	19.633	0.0	1706.4	182
240 min Summer	15.775	0.0	1790.5	240
360 min Summer	11.500	0.0	1908.5	302
480 min Summer	9.145	0.0	1991.8	362
600 min Summer	7.634	0.0	2055.3	424
720 min Summer	6.575	0.0	2106.5	494
960 min Summer	5.176	0.0	2184.9	636
1440 min Summer	3.675	0.0	2292.3	854
2160 min Summer	2.599	0.0	2400.7	1192
2880 min Summer	2.035	0.0	2483.2	1504
4320 min Summer	1.450	0.0	2617.8	0
5760 min Summer	1.148	0.0	2734.3	0
7200 min Summer	0.966	0.0	2848.3	0
8640 min Summer	0.844	0.0	2959.8	0
10080 min Summer	0.756	0.0	3070.7	0
15 min Winter	106.800	0.0	1059.2	18
30 min Winter	70.000	0.0	1224.9	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.523	1.523	37.2	782.0	O K
120 min Winter	99.660	1.660	37.2	881.1	O K
180 min Winter	99.699	1.699	37.2	911.0	O K
240 min Winter	99.693	1.693	37.2	905.9	O K
360 min Winter	99.615	1.615	37.2	848.3	O K
480 min Winter	99.531	1.531	37.2	787.6	O K
600 min Winter	99.449	1.449	37.2	730.5	O K
720 min Winter	99.369	1.369	37.2	676.7	O K
960 min Winter	99.220	1.220	37.2	580.7	O K
1440 min Winter	98.781	0.781	37.2	331.7	O K
2160 min Winter	98.000	0.000	36.6	0.0	O K
2880 min Winter	98.000	0.000	28.6	0.0	O K
4320 min Winter	98.000	0.000	20.4	0.0	O K
5760 min Winter	98.000	0.000	16.1	0.0	O K
7200 min Winter	98.000	0.000	14.1	0.0	O K
8640 min Winter	98.000	0.000	13.7	0.0	O K
10080 min Winter	98.000	0.000	13.4	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.800	0.0	1404.6	62
120 min Winter	26.550	0.0	1590.0	120
180 min Winter	19.633	0.0	1706.4	176
240 min Winter	15.775	0.0	1790.5	232
360 min Winter	11.500	0.0	1908.5	334
480 min Winter	9.145	0.0	1991.8	392
600 min Winter	7.634	0.0	2055.3	452
720 min Winter	6.575	0.0	2106.5	530
960 min Winter	5.176	0.0	2184.9	676
1440 min Winter	3.675	0.0	2292.3	878
2160 min Winter	2.599	0.0	2401.2	0
2880 min Winter	2.035	0.0	2483.9	0
4320 min Winter	1.450	0.0	2617.8	0
5760 min Winter	1.148	0.0	2734.3	0
7200 min Winter	0.966	0.0	2848.3	0
8640 min Winter	0.844	0.0	2959.8	0
10080 min Winter	0.756	0.0	3070.7	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369500 369800 SJ 69500 69800
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 4.000

Time (mins) From:	To:	Area (ha)
0	4	4.000

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	341.8	2.000	848.2

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0246-3720-2000-3720
Design Head (m)	2.000
Design Flow (l/s)	37.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	246
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	2100

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	37.2	Kick-Flo®	1.290	30.1
Flush-Flo™	0.596	37.0	Mean Flow over Head Range	-	32.1

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	8.0	1.200	32.5	3.000	45.2	7.000	68.1
0.200	25.1	1.400	31.3	3.500	48.7	7.500	70.4
0.300	34.3	1.600	33.4	4.000	51.9	8.000	72.7
0.400	36.0	1.800	35.3	4.500	55.0	8.500	74.8
0.500	36.8	2.000	37.2	5.000	57.8	9.000	77.0
0.600	37.0	2.200	38.9	5.500	60.6	9.500	79.0
0.800	36.5	2.400	40.6	6.000	63.2		
1.000	35.3	2.600	42.2	6.500	65.7		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 112 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.892	0.092	0.0	3.0	3.0	21.9	Flood Risk
30 min Summer	99.920	0.120	0.0	3.0	3.0	28.5	Flood Risk
60 min Summer	99.939	0.139	0.0	3.0	3.0	33.0	Flood Risk
120 min Summer	99.940	0.140	0.0	3.0	3.0	33.4	Flood Risk
180 min Summer	99.934	0.134	0.0	3.0	3.0	31.8	Flood Risk
240 min Summer	99.927	0.127	0.0	3.0	3.0	30.2	Flood Risk
360 min Summer	99.913	0.113	0.0	3.0	3.0	26.8	Flood Risk
480 min Summer	99.898	0.098	0.0	3.0	3.0	23.4	Flood Risk
600 min Summer	99.884	0.084	0.0	3.0	3.0	20.1	Flood Risk
720 min Summer	99.871	0.071	0.0	3.0	3.0	17.0	Flood Risk
960 min Summer	99.849	0.049	0.0	3.0	3.0	11.5	Flood Risk
1440 min Summer	99.817	0.017	0.0	3.0	3.0	4.1	Flood Risk
2160 min Summer	99.800	0.000	0.0	2.9	2.9	0.0	Flood Risk
2880 min Summer	99.800	0.000	0.0	2.3	2.3	0.0	Flood Risk
4320 min Summer	99.800	0.000	0.0	1.7	1.7	0.0	Flood Risk
5760 min Summer	99.800	0.000	0.0	1.3	1.3	0.0	Flood Risk
15 min Winter	99.906	0.106	0.0	3.0	3.0	25.1	Flood Risk
30 min Winter	99.937	0.137	0.0	3.0	3.0	32.7	Flood Risk
60 min Winter	99.962	0.162	0.0	3.0	3.0	38.5	Flood Risk
120 min Winter	99.967	0.167	0.0	3.0	3.0	39.7	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	149.756	0.0	24.0	18
30 min Summer	97.904	0.0	32.5	32
60 min Summer	61.002	0.0	41.4	62
120 min Summer	36.417	0.0	50.2	106
180 min Summer	26.743	0.0	55.8	136
240 min Summer	21.409	0.0	59.9	170
360 min Summer	15.561	0.0	65.4	238
480 min Summer	12.356	0.0	69.3	304
600 min Summer	10.314	0.0	72.3	370
720 min Summer	8.891	0.0	74.9	434
960 min Summer	7.028	0.0	78.7	558
1440 min Summer	5.045	0.0	84.3	792
2160 min Summer	3.630	0.0	90.5	0
2880 min Summer	2.882	0.0	95.0	0
4320 min Summer	2.094	0.0	102.0	0
5760 min Summer	1.678	0.0	107.4	0
15 min Winter	149.756	0.0	27.3	18
30 min Winter	97.904	0.0	36.9	32
60 min Winter	61.002	0.0	46.9	60
120 min Winter	36.417	0.0	57.0	116

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.959	0.159	0.0	3.0	3.0	37.7	Flood Risk
240 min Winter	99.949	0.149	0.0	3.0	3.0	35.5	Flood Risk
360 min Winter	99.928	0.128	0.0	3.0	3.0	30.4	Flood Risk
480 min Winter	99.906	0.106	0.0	3.0	3.0	25.1	Flood Risk
600 min Winter	99.884	0.084	0.0	3.0	3.0	20.0	Flood Risk
720 min Winter	99.865	0.065	0.0	3.0	3.0	15.4	Flood Risk
960 min Winter	99.832	0.032	0.0	3.0	3.0	7.6	Flood Risk
1440 min Winter	99.800	0.000	0.0	2.9	2.9	0.0	Flood Risk
2160 min Winter	99.800	0.000	0.0	2.1	2.1	0.0	Flood Risk
2880 min Winter	99.800	0.000	0.0	1.7	1.7	0.0	Flood Risk
4320 min Winter	99.800	0.000	0.0	1.2	1.2	0.0	Flood Risk
5760 min Winter	99.800	0.000	0.0	1.0	1.0	0.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	26.743	0.0	62.9	146
240 min Winter	21.409	0.0	67.6	184
360 min Winter	15.561	0.0	73.6	260
480 min Winter	12.356	0.0	78.0	330
600 min Winter	10.314	0.0	81.4	398
720 min Winter	8.891	0.0	84.3	462
960 min Winter	7.028	0.0	88.9	578
1440 min Winter	5.045	0.0	95.4	0
2160 min Winter	3.630	0.0	102.2	0
2880 min Winter	2.882	0.0	107.5	0
4320 min Winter	2.094	0.0	115.6	0
5760 min Winter	1.678	0.0	122.0	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369383 384061 SJ 69383 84061
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 0.100

Time (mins) From:	To:	Area (ha)
0	4	0.100

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	24.0
Membrane Percolation (mm/hr)	1000	Length (m)	33.0
Max Percolation (l/s)	220.0	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.800	Cap Volume Depth (m)	0.200

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0082-3000-1000-3000
Design Head (m)	1.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	82
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.0	Kick-Flo®	0.623	2.4
Flush-Flo™	0.297	3.0	Mean Flow over Head Range	-	2.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	1.200	3.3	3.000	5.0	7.000	7.4
0.200	2.9	1.400	3.5	3.500	5.4	7.500	7.7
0.300	3.0	1.600	3.7	4.000	5.7	8.000	7.9
0.400	2.9	1.800	3.9	4.500	6.0	8.500	8.2
0.500	2.8	2.000	4.1	5.000	6.3	9.000	8.4
0.600	2.5	2.200	4.3	5.500	6.6	9.500	8.6
0.800	2.7	2.400	4.5	6.000	6.9		
1.000	3.0	2.600	4.7	6.500	7.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.145	1.145	25.0	475.2	O K
30 min Summer	99.378	1.378	25.0	608.5	O K
60 min Summer	99.564	1.564	25.0	725.9	O K
120 min Summer	99.671	1.671	25.0	797.0	O K
180 min Summer	99.686	1.686	25.0	807.1	O K
240 min Summer	99.663	1.663	25.0	791.4	O K
360 min Summer	99.591	1.591	25.0	743.4	O K
480 min Summer	99.530	1.530	25.0	703.9	O K
600 min Summer	99.475	1.475	25.0	668.8	O K
720 min Summer	99.421	1.421	25.0	635.1	O K
960 min Summer	99.314	1.314	25.0	570.7	O K
1440 min Summer	99.105	1.105	25.0	453.2	O K
2160 min Summer	98.806	0.806	25.0	304.3	O K
2880 min Summer	98.540	0.540	25.0	188.9	O K
4320 min Summer	98.154	0.154	25.0	48.0	O K
5760 min Summer	98.000	0.000	24.6	0.0	O K
15 min Winter	99.147	1.147	25.0	476.0	O K
30 min Winter	99.380	1.380	25.0	609.9	O K
60 min Winter	99.569	1.569	25.0	729.0	O K
120 min Winter	99.679	1.679	25.0	802.4	O K
180 min Winter	99.699	1.699	25.0	816.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	103.280	0.0	501.3	19
30 min Summer	67.520	0.0	656.4	33
60 min Summer	42.070	0.0	818.9	62
120 min Summer	25.115	0.0	978.7	122
180 min Summer	18.443	0.0	1077.7	182
240 min Summer	14.765	0.0	1150.8	240
360 min Summer	10.732	0.0	1254.7	300
480 min Summer	8.521	0.0	1328.4	360
600 min Summer	7.113	0.0	1386.1	424
720 min Summer	6.132	0.0	1433.9	492
960 min Summer	4.847	0.0	1511.2	626
1440 min Summer	3.480	0.0	1627.3	894
2160 min Summer	2.503	0.0	1756.9	1276
2880 min Summer	1.987	0.0	1859.5	1616
4320 min Summer	1.444	0.0	2026.0	2292
5760 min Summer	1.157	0.0	2166.5	0
15 min Winter	103.280	0.0	501.7	18
30 min Winter	67.520	0.0	656.6	33
60 min Winter	42.070	0.0	819.6	62
120 min Winter	25.115	0.0	978.7	120
180 min Winter	18.443	0.0	1078.1	176

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Summary of Results for 100 year Return Period

	Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240	min Winter	99.683	1.683	25.0	805.5	O K
360	min Winter	99.598	1.598	25.0	748.3	O K
480	min Winter	99.525	1.525	25.0	700.2	O K
600	min Winter	99.451	1.451	25.0	653.8	O K
720	min Winter	99.374	1.374	25.0	606.4	O K
960	min Winter	99.213	1.213	25.0	512.6	O K
1440	min Winter	98.886	0.886	25.0	342.1	O K
2160	min Winter	98.422	0.422	25.0	142.5	O K
2880	min Winter	98.069	0.069	25.0	21.1	O K
4320	min Winter	98.000	0.000	19.8	0.0	O K
5760	min Winter	98.000	0.000	15.9	0.0	O K

	Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240	min Winter	14.765	0.0	1150.9	232
360	min Winter	10.732	0.0	1254.7	332
480	min Winter	8.521	0.0	1328.4	374
600	min Winter	7.113	0.0	1386.1	448
720	min Winter	6.132	0.0	1433.9	526
960	min Winter	4.847	0.0	1511.2	674
1440	min Winter	3.480	0.0	1627.3	940
2160	min Winter	2.503	0.0	1756.9	1300
2880	min Winter	1.987	0.0	1859.5	1560
4320	min Winter	1.444	0.0	2027.2	0
5760	min Winter	1.157	0.0	2166.5	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369383 384061 SJ 69383 84061
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.900

Time (mins) From:	To:	Area (ha)
0	4	3.900

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	297.5	2.000	777.4

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0205-2510-2000-2510
Design Head (m)	2.000
Design Flow (l/s)	25.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	205
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	25.0	Kick-Flo®	1.249	20.0
Flush-Flo™	0.583	24.9	Mean Flow over Head Range	-	21.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	7.0	1.200	21.0	3.000	30.4	7.000	45.7
0.200	19.5	1.400	21.1	3.500	32.7	7.500	47.2
0.300	23.2	1.600	22.5	4.000	34.9	8.000	48.7
0.400	24.3	1.800	23.8	4.500	36.9	8.500	50.2
0.500	24.8	2.000	25.0	5.000	38.8	9.000	51.6
0.600	24.9	2.200	26.2	5.500	40.7	9.500	53.0
0.800	24.5	2.400	27.3	6.000	42.4		
1.000	23.5	2.600	28.4	6.500	44.1		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.141	1.141	26.0	493.4	O K
30 min Summer	99.374	1.374	26.0	631.9	O K
60 min Summer	99.561	1.561	26.0	753.7	O K
120 min Summer	99.668	1.668	26.0	827.5	O K
180 min Summer	99.683	1.683	26.0	838.0	O K
240 min Summer	99.660	1.660	26.0	821.5	O K
360 min Summer	99.588	1.588	26.0	771.7	O K
480 min Summer	99.527	1.527	26.0	730.7	O K
600 min Summer	99.472	1.472	26.0	694.2	O K
720 min Summer	99.417	1.417	26.0	659.2	O K
960 min Summer	99.310	1.310	26.0	592.2	O K
1440 min Summer	99.099	1.099	26.0	470.2	O K
2160 min Summer	98.800	0.800	26.0	315.4	O K
2880 min Summer	98.535	0.535	26.0	195.7	O K
4320 min Summer	98.151	0.151	26.0	49.5	O K
5760 min Summer	98.000	0.000	25.5	0.0	O K
15 min Winter	99.143	1.143	26.0	494.3	O K
30 min Winter	99.376	1.376	26.0	633.3	O K
60 min Winter	99.566	1.566	26.0	757.0	O K
120 min Winter	99.676	1.676	26.0	833.1	O K
180 min Winter	99.697	1.697	26.0	847.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	103.280	0.0	520.5	19
30 min Summer	67.520	0.0	681.6	33
60 min Summer	42.070	0.0	850.4	62
120 min Summer	25.115	0.0	1016.3	122
180 min Summer	18.443	0.0	1119.1	182
240 min Summer	14.765	0.0	1195.0	240
360 min Summer	10.732	0.0	1303.0	300
480 min Summer	8.521	0.0	1379.5	360
600 min Summer	7.113	0.0	1439.4	440
720 min Summer	6.132	0.0	1488.9	492
960 min Summer	4.847	0.0	1569.3	626
1440 min Summer	3.480	0.0	1689.8	894
2160 min Summer	2.503	0.0	1824.4	1276
2880 min Summer	1.987	0.0	1931.0	1616
4320 min Summer	1.444	0.0	2103.9	2300
5760 min Summer	1.157	0.0	2249.8	0
15 min Winter	103.280	0.0	520.9	18
30 min Winter	67.520	0.0	681.7	33
60 min Winter	42.070	0.0	851.1	62
120 min Winter	25.115	0.0	1016.3	120
180 min Winter	18.443	0.0	1119.5	176

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.681	1.681	26.0	836.2	O K
360 min Winter	99.595	1.595	26.0	776.7	O K
480 min Winter	99.521	1.521	26.0	726.8	O K
600 min Winter	99.448	1.448	26.0	678.6	O K
720 min Winter	99.370	1.370	26.0	629.3	O K
960 min Winter	99.208	1.208	26.0	531.9	O K
1440 min Winter	98.880	0.880	26.0	354.7	O K
2160 min Winter	98.417	0.417	26.0	147.4	O K
2880 min Winter	98.068	0.068	26.0	21.6	O K
4320 min Winter	98.000	0.000	20.5	0.0	O K
5760 min Winter	98.000	0.000	16.5	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.765	0.0	1195.1	232
360 min Winter	10.732	0.0	1303.0	332
480 min Winter	8.521	0.0	1379.5	374
600 min Winter	7.113	0.0	1439.4	448
720 min Winter	6.132	0.0	1489.0	524
960 min Winter	4.847	0.0	1569.3	674
1440 min Winter	3.480	0.0	1689.8	940
2160 min Winter	2.503	0.0	1824.4	1300
2880 min Winter	1.987	0.0	1931.0	1560
4320 min Winter	1.444	0.0	2105.2	0
5760 min Winter	1.157	0.0	2249.8	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369383 384061 SJ 69383 84061
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 4.050

Time (mins) From:	To:	Area (ha)
0	4	4.050

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	312.8	2.000	802.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0208-2600-2000-2600
Design Head (m)	2.000
Design Flow (l/s)	26.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	208
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	26.0	Kick-Flo®	1.247	20.7
Flush-Flo™	0.577	25.9	Mean Flow over Head Range	-	22.5

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	7.1	1.200	21.8	3.000	31.6	7.000	47.5
0.200	20.0	1.400	21.9	3.500	34.0	7.500	49.1
0.300	24.1	1.600	23.4	4.000	36.3	8.000	50.7
0.400	25.3	1.800	24.7	4.500	38.4	8.500	52.2
0.500	25.9	2.000	26.0	5.000	40.4	9.000	53.7
0.600	25.9	2.200	27.2	5.500	42.3	9.500	55.1
0.800	25.5	2.400	28.4	6.000	44.1		
1.000	24.4	2.600	29.5	6.500	45.8		

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Summary of Results for 100 year Return Period (+45%)

Half Drain Time : 112 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	99.908	0.108	0.0	3.0	3.0	22.3	Flood Risk
30 min Summer	99.939	0.139	0.0	3.0	3.0	28.8	Flood Risk
60 min Summer	99.961	0.161	0.0	3.0	3.0	33.3	Flood Risk
120 min Summer	99.962	0.162	0.0	3.0	3.0	33.5	Flood Risk
180 min Summer	99.954	0.154	0.0	3.0	3.0	31.8	Flood Risk
240 min Summer	99.946	0.146	0.0	3.0	3.0	30.2	Flood Risk
360 min Summer	99.930	0.130	0.0	3.0	3.0	26.8	Flood Risk
480 min Summer	99.913	0.113	0.0	3.0	3.0	23.4	Flood Risk
600 min Summer	99.897	0.097	0.0	3.0	3.0	20.1	Flood Risk
720 min Summer	99.882	0.082	0.0	3.0	3.0	17.0	Flood Risk
960 min Summer	99.856	0.056	0.0	3.0	3.0	11.6	Flood Risk
1440 min Summer	99.820	0.020	0.0	3.0	3.0	4.1	Flood Risk
2160 min Summer	99.800	0.000	0.0	2.9	2.9	0.0	Flood Risk
2880 min Summer	99.800	0.000	0.0	2.3	2.3	0.0	Flood Risk
4320 min Summer	99.800	0.000	0.0	1.7	1.7	0.0	Flood Risk
5760 min Summer	99.800	0.000	0.0	1.3	1.3	0.0	Flood Risk
15 min Winter	99.924	0.124	0.0	3.0	3.0	25.7	Flood Risk
30 min Winter	99.961	0.161	0.0	3.0	3.0	33.2	Flood Risk
60 min Winter	99.988	0.188	0.0	3.0	3.0	38.9	Flood Risk
120 min Winter	99.992	0.192	0.0	3.0	3.0	39.8	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	149.756	0.0	24.4	18
30 min Summer	97.904	0.0	33.0	32
60 min Summer	61.002	0.0	42.2	62
120 min Summer	36.417	0.0	50.9	106
180 min Summer	26.743	0.0	56.5	136
240 min Summer	21.409	0.0	60.3	170
360 min Summer	15.561	0.0	66.1	238
480 min Summer	12.356	0.0	70.0	304
600 min Summer	10.314	0.0	73.0	370
720 min Summer	8.891	0.0	75.3	434
960 min Summer	7.028	0.0	79.4	558
1440 min Summer	5.045	0.0	85.3	780
2160 min Summer	3.630	0.0	91.5	0
2880 min Summer	2.882	0.0	96.2	0
4320 min Summer	2.094	0.0	103.4	0
5760 min Summer	1.678	0.0	109.1	0
15 min Winter	149.756	0.0	27.9	18
30 min Winter	97.904	0.0	37.4	32
60 min Winter	61.002	0.0	47.6	60
120 min Winter	36.417	0.0	57.3	116

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Summary of Results for 100 year Return Period (+45%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	99.982	0.182	0.0	3.0	3.0	37.7	Flood Risk
240 min Winter	99.971	0.171	0.0	3.0	3.0	35.5	Flood Risk
360 min Winter	99.947	0.147	0.0	3.0	3.0	30.4	Flood Risk
480 min Winter	99.921	0.121	0.0	3.0	3.0	25.1	Flood Risk
600 min Winter	99.897	0.097	0.0	3.0	3.0	20.1	Flood Risk
720 min Winter	99.875	0.075	0.0	3.0	3.0	15.4	Flood Risk
960 min Winter	99.837	0.037	0.0	3.0	3.0	7.7	Flood Risk
1440 min Winter	99.800	0.000	0.0	3.0	3.0	0.0	Flood Risk
2160 min Winter	99.800	0.000	0.0	2.1	2.1	0.0	Flood Risk
2880 min Winter	99.800	0.000	0.0	1.7	1.7	0.0	Flood Risk
4320 min Winter	99.800	0.000	0.0	1.2	1.2	0.0	Flood Risk
5760 min Winter	99.800	0.000	0.0	1.0	1.0	0.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	26.743	0.0	63.7	146
240 min Winter	21.409	0.0	68.1	184
360 min Winter	15.561	0.0	74.4	258
480 min Winter	12.356	0.0	78.7	330
600 min Winter	10.314	0.0	82.1	398
720 min Winter	8.891	0.0	85.0	462
960 min Winter	7.028	0.0	89.5	578
1440 min Winter	5.045	0.0	96.2	0
2160 min Winter	3.630	0.0	103.2	0
2880 min Winter	2.882	0.0	108.6	0
4320 min Winter	2.094	0.0	117.0	0
5760 min Winter	1.678	0.0	123.7	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369383 384061 SJ 69383 84061
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+45

Time Area Diagram

Total Area (ha) 0.100

Time (mins) From:	To:	Area (ha)
0	4	0.100

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	23.0
Membrane Percolation (mm/hr)	1000	Length (m)	30.0
Max Percolation (l/s)	191.7	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.800	Cap Volume Depth (m)	0.200

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0082-3000-1000-3000
Design Head (m)	1.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	82
Invert Level (m)	99.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	3.0	Kick-Flo®	0.623	2.4
Flush-Flo™	0.297	3.0	Mean Flow over Head Range	-	2.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.4	1.200	3.3	3.000	5.0	7.000	7.4
0.200	2.9	1.400	3.5	3.500	5.4	7.500	7.7
0.300	3.0	1.600	3.7	4.000	5.7	8.000	7.9
0.400	2.9	1.800	3.9	4.500	6.0	8.500	8.2
0.500	2.8	2.000	4.1	5.000	6.3	9.000	8.4
0.600	2.5	2.200	4.3	5.500	6.6	9.500	8.6
0.800	2.7	2.400	4.5	6.000	6.9		
1.000	3.0	2.600	4.7	6.500	7.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.930	0.930	5.6	355.3	O K
30 min Summer	99.139	1.139	5.6	461.4	O K
60 min Summer	99.330	1.330	5.6	567.9	O K
120 min Summer	99.487	1.487	5.6	662.0	O K
180 min Summer	99.568	1.568	5.6	713.2	O K
240 min Summer	99.617	1.617	5.6	745.2	O K
360 min Summer	99.669	1.669	5.6	779.9	O K
480 min Summer	99.688	1.688	5.6	792.8	O K
600 min Summer	99.690	1.690	5.6	794.1	O K
720 min Summer	99.682	1.682	5.6	788.3	O K
960 min Summer	99.645	1.645	5.6	763.6	O K
1440 min Summer	99.556	1.556	5.6	705.6	O K
2160 min Summer	99.466	1.466	5.6	648.8	O K
2880 min Summer	99.394	1.394	5.6	605.5	O K
4320 min Summer	99.265	1.265	5.6	530.7	O K
5760 min Summer	99.145	1.145	5.6	464.6	O K
15 min Winter	98.930	0.930	5.6	355.3	O K
30 min Winter	99.139	1.139	5.6	461.4	O K
60 min Winter	99.330	1.330	5.6	567.9	O K
120 min Winter	99.488	1.488	5.6	662.7	O K
180 min Winter	99.570	1.570	5.6	714.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	103.280	0.0	350.1	19
30 min Summer	67.520	0.0	376.3	34
60 min Summer	42.070	0.0	585.7	64
120 min Summer	25.115	0.0	696.7	124
180 min Summer	18.443	0.0	760.1	184
240 min Summer	14.765	0.0	788.5	242
360 min Summer	10.732	0.0	802.4	362
480 min Summer	8.521	0.0	809.6	482
600 min Summer	7.113	0.0	815.7	602
720 min Summer	6.132	0.0	821.7	722
960 min Summer	4.847	0.0	833.5	960
1440 min Summer	3.480	0.0	846.8	1214
2160 min Summer	2.503	0.0	1259.8	1580
2880 min Summer	1.987	0.0	1332.8	1964
4320 min Summer	1.444	0.0	1438.3	2772
5760 min Summer	1.157	0.0	1555.0	3584
15 min Winter	103.280	0.0	350.1	19
30 min Winter	67.520	0.0	376.2	33
60 min Winter	42.070	0.0	585.7	64
120 min Winter	25.115	0.0	696.7	122
180 min Winter	18.443	0.0	760.0	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.620	1.620	5.6	747.1	O K
360 min Winter	99.673	1.673	5.6	782.5	O K
480 min Winter	99.694	1.694	5.6	796.6	O K
600 min Winter	99.698	1.698	5.6	799.4	O K
720 min Winter	99.692	1.692	5.6	795.4	O K
960 min Winter	99.663	1.663	5.6	775.5	O K
1440 min Winter	99.568	1.568	5.6	713.6	O K
2160 min Winter	99.456	1.456	5.6	642.7	O K
2880 min Winter	99.355	1.355	5.6	582.6	O K
4320 min Winter	99.154	1.154	5.6	469.6	O K
5760 min Winter	98.956	0.956	5.6	367.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.765	0.0	787.9	240
360 min Winter	10.732	0.0	801.1	356
480 min Winter	8.521	0.0	807.6	472
600 min Winter	7.113	0.0	813.0	588
720 min Winter	6.132	0.0	818.0	702
960 min Winter	4.847	0.0	827.5	924
1440 min Winter	3.480	0.0	846.6	1340
2160 min Winter	2.503	0.0	1259.9	1648
2880 min Winter	1.987	0.0	1332.9	2108
4320 min Winter	1.444	0.0	1446.1	2984
5760 min Winter	1.157	0.0	1555.0	3808

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369383 384061 SJ 69383 84061
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.800

Time (mins) From:	To:	Area (ha)
0	4	2.800

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	289.8	2.000	765.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0097-5600-2000-5600
Design Head (m)	2.000
Design Flow (l/s)	5.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	97
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	5.6	Kick-Flo®	0.868	3.8
Flush-Flo™	0.425	4.8	Mean Flow over Head Range	-	4.5

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.2	1.200	4.4	3.000	6.8	7.000	10.1
0.200	4.3	1.400	4.7	3.500	7.3	7.500	10.4
0.300	4.7	1.600	5.0	4.000	7.7	8.000	10.8
0.400	4.8	1.800	5.3	4.500	8.2	8.500	11.1
0.500	4.7	2.000	5.6	5.000	8.6	9.000	11.4
0.600	4.6	2.200	5.8	5.500	9.0	9.500	11.7
0.800	4.2	2.400	6.1	6.000	9.4		
1.000	4.1	2.600	6.3	6.500	9.7		

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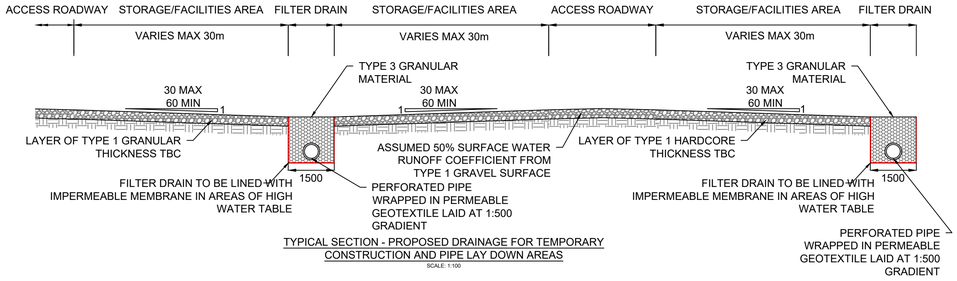
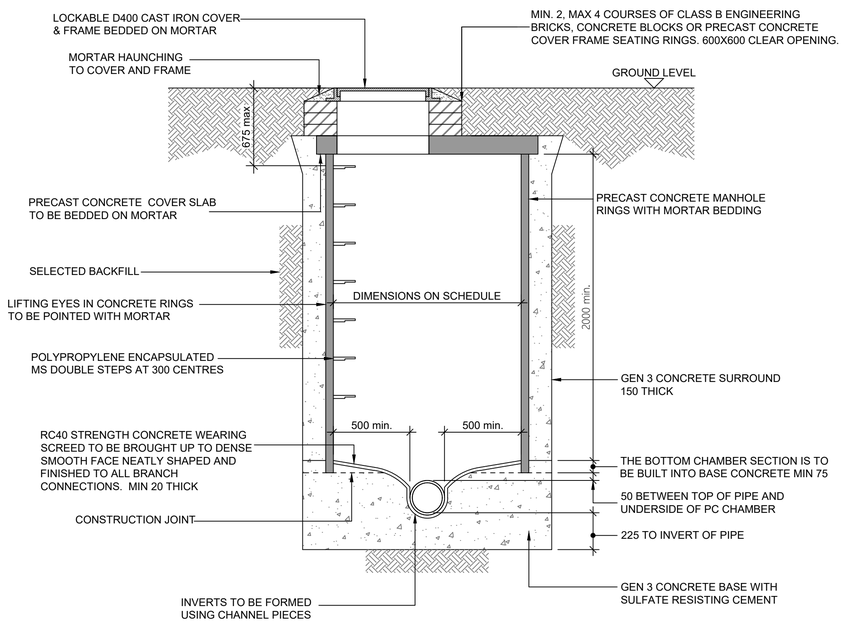
Appendix D
Temporary Construction Area Details



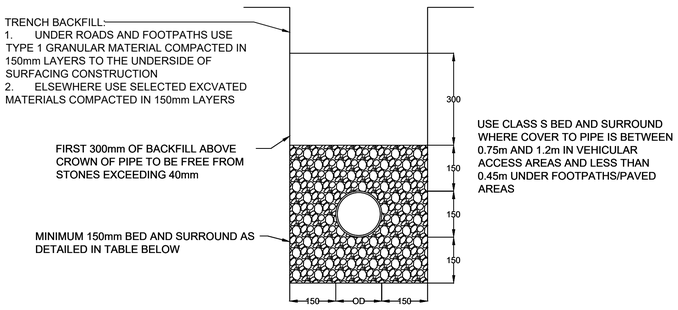
TYPICAL SECTION - PROPOSED DRAINAGE FOR TEMPORARY CONSTRUCTION AND PIPE LAY DOWN AREAS

SCALE: 1:100



CONCRETE MANHOLE - SECTION

SCALE: 1:20

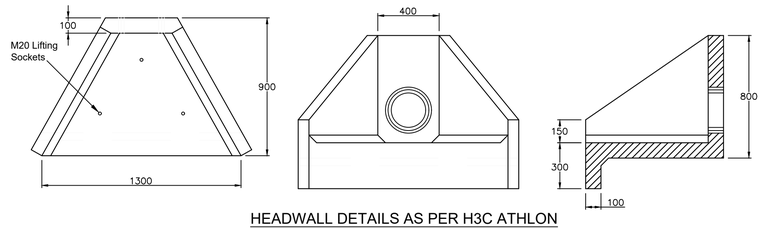


PIPE DIAMETER:	BEDDING MATERIAL:
100 DIA	10mm SINGLE SIZE AGGREGATE
101 - 150 DIA	10mm OR 14mm SINGLE SIZE AGGREGATE
151 - 500 DIA	10mm, 14mm OR 20mm SINGLE SIZE AGGREGATE

NOTE:
THE ABOVE MATERIALS ARE FOR GUIDANCE OTHER BEDDING MATERIALS CAN BE USED IN ACCORDANCE WITH THE PIPE MANUFACTURERS RECOMMENDATIONS

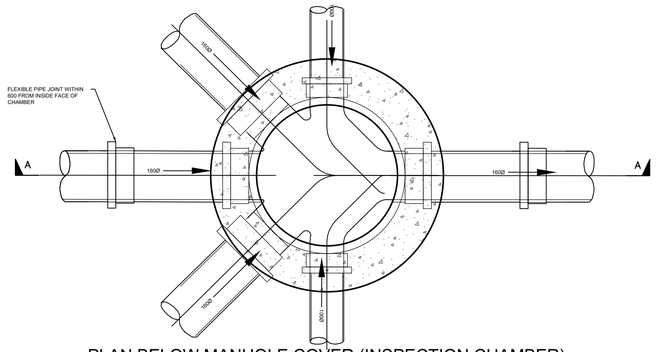
CLASS S GRANULAR BEDDING AND SURROUND FOR FLEXIBLE PIPES

SCALE 1:10



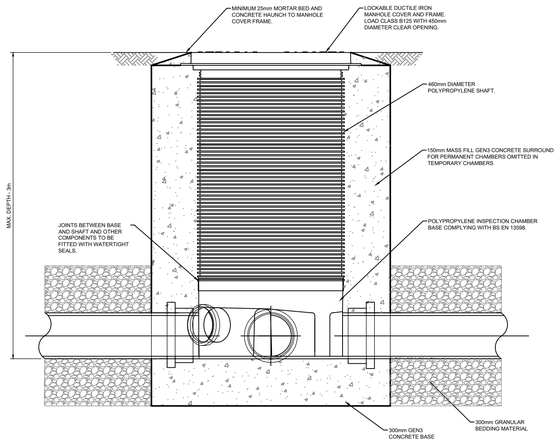
HEADWALL DETAILS AS PER H3C ATHLON

SCALE: 1:20



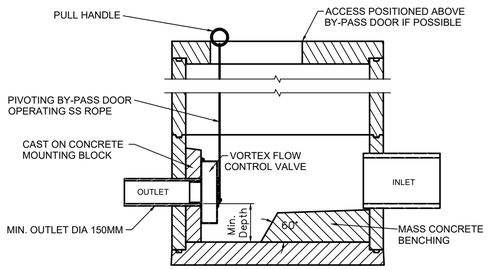
PLAN BELOW MANHOLE COVER (INSPECTION CHAMBER)

SCALE: 1:10



POLYPROPYLENE INSPECTION CHAMBER DETAIL

SCALE: 1:10



FLOW CONTROL MANHOLE (HYDROBRAKE)

SCALE: 1:20

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE STATED.
2. GROUND CONDITIONS ARE CURRENTLY UNKNOWN. GEOTECHNICAL STUDIES REQUIRED TO DETERMINE RECEIVING WATERCOURSE BANK STABILITY AND CONFIRM APPROPRIATE HEADWALL SELECTION AND PLACEMENT.
3. EXISTING GROUND LEVEL SURVEY DATA IS BASED ON LIDAR DATA. DETAILED TOPOGRAPHIC SURVEYS ARE REQUIRED TO ACCURATELY DETERMINE EXISTING GROUND LEVELS.
4. BASIN DETAILS INCLUDING OUTLET FROM BASIN AND FLOW CONTROL DEVICE TO BE CONFIRMED FOLLOWING DETAILED DESIGN.
5. EROSION PROTECTION WILL BE SPECIFIED FOLLOWING TOPOGRAPHIC SURVEY.
6. DRAIN FLOW DIRECTIONS ASSUMED BASED ON LIDAR AND OS MAP DATA.THIS WILL BE CONFIRMED DURING DETAILED DESIGN STAGE.
7. THE WATERCOURSE BANK WILL NEED TO BE REPROFILED TO SUIT THE HEADWALL PLACEMENT.

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HyNet Drainage Strategy Report

Appendix X.X
TYPICAL DRAINAGE DETAILS FOR TEMPORARY CONSTRUCTION AND PIPE LAY DOWN AREAS

August 2024

wsp

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HyNet North West Hydrogen Pipeline

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Appendix E
Indicative Pipe Laydown Site Plans

PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 810m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 5.1l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1000 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 1

August 2024

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\807733-WSP-XX-XX-DR-D-000X_S0_P02.1_Additional_Area_1_-Appendix_AX-Indicative_Temporary_HAGI_Drainage.dwg -- Originator: UKJRN776

PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 600m^3

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 3.6l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 2

August 2024

wsp

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PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 555m^3

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 10.6l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 3

August 2024

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POND B
CATCHMENT AREA = 3.6Ha.
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 840m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 17.6l/s. DISCHARGE POINT TO BE CONFIRMED

POND A
CATCHMENT AREA = 3.9Ha.
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 910m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 19.1l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m
Scale 1:1000 @ A1

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HyNet Drainage Strategy Report

Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 4

August 2024

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SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 3.1l/s. DISCHARGE POINT TO BE CONFIRMED

POND A
CATCHMENT AREA = 1.3Ha
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 355m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 3l/s. DISCHARGE POINT TO BE CONFIRMED

POND B
CATCHMENT AREA = 1.2Ha
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 325m³

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m
Scale 1:1000 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 5

August 2024

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NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1000 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 6

August 2024

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PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 480m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 20.5l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 7

August 2024

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PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 655m^3

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 28.1l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 30 m

Scale 1:500 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 8

August 2024

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EXISTING DITCH

PROPOSED TEMPORARY FLOW CONTROL TO DISCHARGE AT 3.4l/s

PROPOSED TEMPORARY FLOW CONTROL TO DISCHARGE AT 6.1l/s

POND C
CATCHMENT AREA = 2.6Ha.
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 710m³

POND B
CATCHMENT AREA = 1.4Ha.
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 380m³

POND D
CATCHMENT AREA = 6.6Ha
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 1800m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 19.8l/s. DISCHARGE POINT TO BE CONFIRMED

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 6.7l/s. DISCHARGE POINT TO BE CONFIRMED

POND A
CATCHMENT AREA = 2.80Ha.
PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 765m³

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1250 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 9

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NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1000 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 10

August 2024

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PROPOSED TEMPORARY ATTENUATION BASIN TO STORE 595m³

SHOULD INFILTRATION BE PROVEN NON-VIABLE, SITE TO DISCHARGE AT 11.5l/s. DISCHARGE POINT TO BE CONFIRMED

NOTES:

1. ALL DIMENSIONS IN MILLIMETRES UNLESS STATED OTHERWISE.

Key

- DCO Boundary
- Temporary Construction Compounds
- Pipe Dump locations
- Trenchless Crossing Compounds
- Limits of Deviation - Pipelines
- Proposed Temporary Surface Water Drain
- Proposed Headwall Discharge Point
- Proposed Filter Trench
- E — Electric Lines
- W — Water Lines
- T — Telecom Lines
- SS — Sewer Lines
- Pipelines
- GAS — Gas Lines

0 m — 50 m

Scale 1:1000 @ A1

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Appendix XX
Indicative Temporary Pipe Laydown Area Drainage Layout - Additional Area 11

August 2024

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HyNet North West Hydrogen Pipeline

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Appendix F
Pipe Laydown Areas - Attenuation Storage Calculations

Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.899	0.899	5.1	346.5	O K
30 min Summer	99.100	1.100	5.1	448.4	O K
60 min Summer	99.281	1.281	5.1	548.5	O K
120 min Summer	99.448	1.448	5.1	648.0	O K
180 min Summer	99.537	1.537	5.1	704.5	O K
240 min Summer	99.593	1.593	5.1	740.9	O K
360 min Summer	99.656	1.656	5.1	782.7	O K
480 min Summer	99.682	1.682	5.1	800.7	O K
600 min Summer	99.690	1.690	5.1	806.3	O K
720 min Summer	99.687	1.687	5.1	804.1	O K
960 min Summer	99.659	1.659	5.1	785.1	O K
1440 min Summer	99.570	1.570	5.1	725.7	O K
2160 min Summer	99.471	1.471	5.1	662.4	O K
2880 min Summer	99.395	1.395	5.1	615.6	O K
4320 min Summer	99.264	1.264	5.1	538.8	O K
5760 min Summer	99.146	1.146	5.1	473.1	O K
7200 min Summer	99.037	1.037	5.1	415.5	O K
8640 min Summer	98.936	0.936	5.1	364.3	O K
10080 min Summer	98.839	0.839	5.1	317.9	O K
15 min Winter	98.899	0.899	5.1	346.5	O K
30 min Winter	99.100	1.100	5.1	448.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.350	0.0	331.5	19
30 min Summer	67.948	0.0	332.9	34
60 min Summer	42.061	0.0	563.4	64
120 min Summer	25.395	0.0	671.1	124
180 min Summer	18.778	0.0	702.2	184
240 min Summer	15.101	0.0	709.8	242
360 min Summer	11.032	0.0	719.9	362
480 min Summer	8.780	0.0	728.9	482
600 min Summer	7.336	0.0	737.6	602
720 min Summer	6.326	0.0	746.2	722
960 min Summer	4.996	0.0	763.7	960
1440 min Summer	3.573	0.0	787.1	1258
2160 min Summer	2.552	0.0	1237.8	1624
2880 min Summer	2.014	0.0	1300.6	2020
4320 min Summer	1.451	0.0	1333.5	2812
5760 min Summer	1.157	0.0	1498.3	3640
7200 min Summer	0.975	0.0	1579.6	4400
8640 min Summer	0.852	0.0	1655.3	5192
10080 min Summer	0.762	0.0	1726.6	5952
15 min Winter	104.350	0.0	331.5	19
30 min Winter	67.948	0.0	332.8	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.281	1.281	5.1	548.5	O K
120 min Winter	99.449	1.449	5.1	648.6	O K
180 min Winter	99.539	1.539	5.1	705.5	O K
240 min Winter	99.596	1.596	5.1	742.6	O K
360 min Winter	99.659	1.659	5.1	784.9	O K
480 min Winter	99.687	1.687	5.1	804.0	O K
600 min Winter	99.697	1.697	5.1	810.8	O K
720 min Winter	99.696	1.696	5.1	810.1	O K
960 min Winter	99.674	1.674	5.1	795.0	O K
1440 min Winter	99.591	1.591	5.1	739.3	O K
2160 min Winter	99.466	1.466	5.1	659.5	O K
2880 min Winter	99.367	1.367	5.1	598.8	O K
4320 min Winter	99.171	1.171	5.1	486.7	O K
5760 min Winter	98.981	0.981	5.1	387.0	O K
7200 min Winter	98.801	0.801	5.1	300.5	O K
8640 min Winter	98.631	0.631	5.1	225.4	O K
10080 min Winter	98.472	0.472	5.1	160.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	42.061	0.0	563.4	64
120 min Winter	25.395	0.0	671.1	122
180 min Winter	18.778	0.0	701.7	180
240 min Winter	15.101	0.0	709.2	240
360 min Winter	11.032	0.0	718.8	356
480 min Winter	8.780	0.0	727.3	474
600 min Winter	7.336	0.0	735.5	590
720 min Winter	6.326	0.0	743.5	702
960 min Winter	4.996	0.0	759.6	926
1440 min Winter	3.573	0.0	782.7	1356
2160 min Winter	2.552	0.0	1237.8	1688
2880 min Winter	2.014	0.0	1301.0	2136
4320 min Winter	1.451	0.0	1366.0	3028
5760 min Winter	1.157	0.0	1498.3	3864
7200 min Winter	0.975	0.0	1579.7	4616
8640 min Winter	0.852	0.0	1655.4	5368
10080 min Winter	0.762	0.0	1726.8	6056

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.700

Time (mins) From:	To:	Area (ha)
0	4	2.700

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	295.6	2.000	774.3

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0093-5100-2000-5100
Design Head (m)	2.000
Design Flow (l/s)	5.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	93
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	5.1	Kick-Flo®	0.825	3.4
Flush-Flo™	0.405	4.2	Mean Flow over Head Range	-	4.0

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.9	1.200	4.0	3.000	6.2	7.000	9.2
0.200	3.9	1.400	4.3	3.500	6.6	7.500	9.5
0.300	4.2	1.600	4.6	4.000	7.1	8.000	9.8
0.400	4.2	1.800	4.9	4.500	7.5	8.500	10.1
0.500	4.2	2.000	5.1	5.000	7.8	9.000	10.4
0.600	4.1	2.200	5.3	5.500	8.2	9.500	10.6
0.800	3.5	2.400	5.6	6.000	8.6		
1.000	3.7	2.600	5.8	6.500	8.9		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.885	0.885	3.6	248.7	O K
30 min Summer	99.087	1.087	3.6	324.2	O K
60 min Summer	99.274	1.274	3.6	401.1	O K
120 min Summer	99.443	1.443	3.6	476.4	O K
180 min Summer	99.532	1.532	3.6	518.5	O K
240 min Summer	99.587	1.587	3.6	545.5	O K
360 min Summer	99.649	1.649	3.6	576.8	O K
480 min Summer	99.677	1.677	3.6	591.3	O K
600 min Summer	99.687	1.687	3.6	596.4	O K
720 min Summer	99.686	1.686	3.6	595.6	O K
960 min Summer	99.661	1.661	3.6	582.6	O K
1440 min Summer	99.569	1.569	3.6	536.8	O K
2160 min Summer	99.462	1.462	3.6	485.4	O K
2880 min Summer	99.381	1.381	3.6	448.1	O K
4320 min Summer	99.243	1.243	3.6	387.9	O K
5760 min Summer	99.122	1.122	3.6	337.9	O K
7200 min Summer	99.014	1.014	3.6	295.9	O K
8640 min Summer	98.914	0.914	3.6	259.1	O K
10080 min Summer	98.820	0.820	3.6	225.9	O K
15 min Winter	98.885	0.885	3.6	248.7	O K
30 min Winter	99.087	1.087	3.6	324.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	106.408	0.0	219.0	19
30 min Summer	69.773	0.0	217.0	34
60 min Summer	43.658	0.0	410.6	64
120 min Summer	26.472	0.0	466.2	124
180 min Summer	19.579	0.0	475.6	184
240 min Summer	15.739	0.0	482.7	242
360 min Summer	11.493	0.0	496.2	362
480 min Summer	9.149	0.0	509.5	482
600 min Summer	7.645	0.0	520.6	602
720 min Summer	6.590	0.0	529.3	722
960 min Summer	5.197	0.0	542.7	960
1440 min Summer	3.700	0.0	559.2	1312
2160 min Summer	2.625	0.0	895.6	1644
2880 min Summer	2.060	0.0	935.1	2020
4320 min Summer	1.472	0.0	922.0	2812
5760 min Summer	1.167	0.0	1063.8	3640
7200 min Summer	0.982	0.0	1119.3	4400
8640 min Summer	0.857	0.0	1171.8	5192
10080 min Summer	0.766	0.0	1221.5	5952
15 min Winter	106.408	0.0	219.0	19
30 min Winter	69.773	0.0	217.0	34

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.274	1.274	3.6	401.1	O K
120 min Winter	99.444	1.444	3.6	476.8	O K
180 min Winter	99.533	1.533	3.6	519.2	O K
240 min Winter	99.589	1.589	3.6	546.7	O K
360 min Winter	99.652	1.652	3.6	578.4	O K
480 min Winter	99.682	1.682	3.6	593.5	O K
600 min Winter	99.693	1.693	3.6	599.5	O K
720 min Winter	99.694	1.694	3.6	599.6	O K
960 min Winter	99.674	1.674	3.6	589.4	O K
1440 min Winter	99.591	1.591	3.6	547.7	O K
2160 min Winter	99.459	1.459	3.6	484.0	O K
2880 min Winter	99.356	1.356	3.6	437.0	O K
4320 min Winter	99.156	1.156	3.6	351.8	O K
5760 min Winter	98.964	0.964	3.6	277.4	O K
7200 min Winter	98.787	0.787	3.6	214.8	O K
8640 min Winter	98.621	0.621	3.6	161.0	O K
10080 min Winter	98.466	0.466	3.6	115.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.658	0.0	410.6	64
120 min Winter	26.472	0.0	466.1	122
180 min Winter	19.579	0.0	475.2	180
240 min Winter	15.739	0.0	482.4	240
360 min Winter	11.493	0.0	495.5	358
480 min Winter	9.149	0.0	508.6	474
600 min Winter	7.645	0.0	519.4	590
720 min Winter	6.590	0.0	527.7	706
960 min Winter	5.197	0.0	539.8	932
1440 min Winter	3.700	0.0	555.0	1358
2160 min Winter	2.625	0.0	895.7	1688
2880 min Winter	2.060	0.0	935.6	2136
4320 min Winter	1.472	0.0	945.3	3028
5760 min Winter	1.167	0.0	1063.8	3864
7200 min Winter	0.982	0.0	1119.3	4616
8640 min Winter	0.857	0.0	1171.8	5368
10080 min Winter	0.766	0.0	1221.6	6056

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 369669 370032 SJ 69669 70032
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.900

Time (mins) From:	To:	Area (ha)
0	4	1.900

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	212.5	2.000	587.5

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0077-3600-2000-3600
Design Head (m)	2.000
Design Flow (l/s)	3.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	77
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.6	Kick-Flo®	0.691	2.2
Flush-Flo™	0.339	2.7	Mean Flow over Head Range	-	2.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.2	1.200	2.8	3.000	4.3	7.000	6.5
0.200	2.6	1.400	3.1	3.500	4.7	7.500	6.7
0.300	2.7	1.600	3.2	4.000	5.0	8.000	6.9
0.400	2.7	1.800	3.4	4.500	5.3	8.500	7.1
0.500	2.7	2.000	3.6	5.000	5.5	9.000	7.3
0.600	2.5	2.200	3.8	5.500	5.8	9.500	7.5
0.800	2.4	2.400	3.9	6.000	6.0		
1.000	2.6	2.600	4.1	6.500	6.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.081	1.081	10.6	288.0	O K
30 min Summer	99.297	1.297	10.6	372.1	O K
60 min Summer	99.488	1.488	10.6	454.3	O K
120 min Summer	99.616	1.616	10.6	514.4	O K
180 min Summer	99.667	1.667	10.6	539.4	O K
240 min Summer	99.684	1.684	10.6	547.5	O K
360 min Summer	99.662	1.662	10.6	536.5	O K
480 min Summer	99.609	1.609	10.6	511.1	O K
600 min Summer	99.565	1.565	10.6	489.9	O K
720 min Summer	99.524	1.524	10.6	470.8	O K
960 min Summer	99.449	1.449	10.6	436.9	O K
1440 min Summer	99.303	1.303	10.6	374.5	O K
2160 min Summer	99.093	1.093	10.6	292.7	O K
2880 min Summer	98.894	0.894	10.6	223.5	O K
4320 min Summer	98.540	0.540	10.6	118.7	O K
5760 min Summer	98.264	0.264	10.6	52.4	O K
7200 min Summer	98.087	0.087	10.6	16.0	O K
8640 min Summer	98.000	0.000	10.6	0.0	O K
10080 min Summer	98.000	0.000	9.6	0.0	O K
15 min Winter	99.080	1.080	10.6	287.8	O K
30 min Winter	99.299	1.299	10.6	372.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.936	0.0	299.0	19
30 min Summer	65.592	0.0	392.4	33
60 min Summer	41.196	0.0	494.0	64
120 min Summer	24.678	0.0	591.7	122
180 min Summer	18.192	0.0	654.5	182
240 min Summer	14.600	0.0	700.4	242
360 min Summer	10.633	0.0	765.1	360
480 min Summer	8.449	0.0	810.6	464
600 min Summer	7.050	0.0	845.5	484
720 min Summer	6.071	0.0	873.7	544
960 min Summer	4.782	0.0	917.5	674
1440 min Summer	3.399	0.0	978.2	940
2160 min Summer	2.414	0.0	1042.7	1344
2880 min Summer	1.899	0.0	1093.5	1728
4320 min Summer	1.364	0.0	1178.0	2424
5760 min Summer	1.088	0.0	1253.5	3120
7200 min Summer	0.922	0.0	1328.1	3752
8640 min Summer	0.811	0.0	1401.8	7608
10080 min Summer	0.732	0.0	1475.0	0
15 min Winter	99.936	0.0	298.7	18
30 min Winter	65.592	0.0	392.6	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.490	1.490	10.6	455.2	O K
120 min Winter	99.621	1.621	10.6	516.5	O K
180 min Winter	99.674	1.674	10.6	542.5	O K
240 min Winter	99.693	1.693	10.6	552.0	O K
360 min Winter	99.679	1.679	10.6	545.1	O K
480 min Winter	99.630	1.630	10.6	521.0	O K
600 min Winter	99.567	1.567	10.6	490.7	O K
720 min Winter	99.521	1.521	10.6	469.3	O K
960 min Winter	99.424	1.424	10.6	425.8	O K
1440 min Winter	99.214	1.214	10.6	338.6	O K
2160 min Winter	98.891	0.891	10.6	222.2	O K
2880 min Winter	98.576	0.576	10.6	128.4	O K
4320 min Winter	98.069	0.069	10.6	12.6	O K
5760 min Winter	98.000	0.000	9.2	0.0	O K
7200 min Winter	98.000	0.000	7.8	0.0	O K
8640 min Winter	98.000	0.000	6.8	0.0	O K
10080 min Winter	98.000	0.000	6.2	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	41.196	0.0	494.0	62
120 min Winter	24.678	0.0	591.8	120
180 min Winter	18.192	0.0	654.4	178
240 min Winter	14.600	0.0	700.4	236
360 min Winter	10.633	0.0	765.1	348
480 min Winter	8.449	0.0	810.6	454
600 min Winter	7.050	0.0	845.5	500
720 min Winter	6.071	0.0	873.7	564
960 min Winter	4.782	0.0	917.5	714
1440 min Winter	3.399	0.0	978.2	1010
2160 min Winter	2.414	0.0	1042.7	1408
2880 min Winter	1.899	0.0	1093.5	1784
4320 min Winter	1.364	0.0	1178.0	2340
5760 min Winter	1.088	0.0	1253.6	0
7200 min Winter	0.922	0.0	1328.3	0
8640 min Winter	0.811	0.0	1401.8	0
10080 min Winter	0.732	0.0	1475.0	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 344827 374941 SJ 44827 74941
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.400

Time (mins) From:	To:	Area (ha)
0	4	2.400

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	178.7	2.000	576.2

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0135-1060-2000-1060
Design Head (m)	2.000
Design Flow (l/s)	10.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	135
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	10.6
Flush-Flo™	0.593	10.5
Kick-Flo®	1.206	8.4
Mean Flow over Head Range	-	9.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.9	1.200	8.4	3.000	12.8	7.000	19.2
0.200	8.8	1.400	9.0	3.500	13.8	7.500	19.9
0.300	9.8	1.600	9.5	4.000	14.7	8.000	20.5
0.400	10.3	1.800	10.1	4.500	15.6	8.500	21.1
0.500	10.5	2.000	10.6	5.000	16.4	9.000	21.7
0.600	10.5	2.200	11.1	5.500	17.1	9.500	22.3
0.800	10.3	2.400	11.6	6.000	17.9		
1.000	9.8	2.600	12.0	6.500	18.6		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.083	1.083	19.1	487.5	O K
30 min Summer	99.300	1.300	19.1	624.0	O K
60 min Summer	99.479	1.479	19.1	747.1	O K
120 min Summer	99.618	1.618	19.1	850.2	O K
180 min Summer	99.670	1.670	19.1	890.5	O K
240 min Summer	99.685	1.685	19.1	901.7	O K
360 min Summer	99.655	1.655	19.1	878.3	O K
480 min Summer	99.602	1.602	19.1	837.6	O K
600 min Summer	99.556	1.556	19.1	803.5	O K
720 min Summer	99.514	1.514	19.1	772.8	O K
960 min Summer	99.436	1.436	19.1	717.0	O K
1440 min Summer	99.285	1.285	19.1	614.1	O K
2160 min Summer	99.063	1.063	19.1	476.0	O K
2880 min Summer	98.853	0.853	19.1	358.3	O K
4320 min Summer	98.483	0.483	19.1	180.8	O K
5760 min Summer	98.207	0.207	19.1	70.7	O K
7200 min Summer	98.043	0.043	19.1	14.0	O K
8640 min Summer	98.000	0.000	18.1	0.0	O K
10080 min Summer	98.000	0.000	16.2	0.0	O K
15 min Winter	99.082	1.082	19.1	487.2	O K
30 min Winter	99.302	1.302	19.1	625.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.350	0.0	506.6	19
30 min Summer	67.948	0.0	659.6	33
60 min Summer	42.061	0.0	818.1	64
120 min Summer	25.395	0.0	989.1	122
180 min Summer	18.778	0.0	1097.4	182
240 min Summer	15.101	0.0	1176.7	242
360 min Summer	11.032	0.0	1289.5	360
480 min Summer	8.780	0.0	1368.3	412
600 min Summer	7.336	0.0	1429.1	472
720 min Summer	6.326	0.0	1478.7	534
960 min Summer	4.996	0.0	1557.0	664
1440 min Summer	3.573	0.0	1670.4	938
2160 min Summer	2.552	0.0	1791.1	1324
2880 min Summer	2.014	0.0	1884.3	1704
4320 min Summer	1.451	0.0	2035.6	2420
5760 min Summer	1.157	0.0	2164.8	3064
7200 min Summer	0.975	0.0	2282.2	4696
8640 min Summer	0.852	0.0	2392.3	0
10080 min Summer	0.762	0.0	2496.4	0
15 min Winter	104.350	0.0	506.1	18
30 min Winter	67.948	0.0	660.0	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.483	1.483	19.1	749.8	O K
120 min Winter	99.623	1.623	19.1	854.1	O K
180 min Winter	99.678	1.678	19.1	896.4	O K
240 min Winter	99.695	1.695	19.1	910.2	O K
360 min Winter	99.676	1.676	19.1	894.7	O K
480 min Winter	99.617	1.617	19.1	849.6	O K
600 min Winter	99.556	1.556	19.1	803.5	O K
720 min Winter	99.507	1.507	19.1	767.3	O K
960 min Winter	99.401	1.401	19.1	692.5	O K
1440 min Winter	99.177	1.177	19.1	545.1	O K
2160 min Winter	98.832	0.832	19.1	347.2	O K
2880 min Winter	98.503	0.503	19.1	189.3	O K
4320 min Winter	98.023	0.023	19.1	7.4	O K
5760 min Winter	98.000	0.000	15.8	0.0	O K
7200 min Winter	98.000	0.000	13.4	0.0	O K
8640 min Winter	98.000	0.000	11.7	0.0	O K
10080 min Winter	98.000	0.000	10.4	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	42.061	0.0	819.2	62
120 min Winter	25.395	0.0	989.3	120
180 min Winter	18.778	0.0	1097.4	178
240 min Winter	15.101	0.0	1176.6	236
360 min Winter	11.032	0.0	1289.5	346
480 min Winter	8.780	0.0	1368.3	450
600 min Winter	7.336	0.0	1429.1	482
720 min Winter	6.326	0.0	1478.7	556
960 min Winter	4.996	0.0	1557.0	706
1440 min Winter	3.573	0.0	1670.5	996
2160 min Winter	2.552	0.0	1791.1	1404
2880 min Winter	2.014	0.0	1884.3	1756
4320 min Winter	1.451	0.0	2035.8	2356
5760 min Winter	1.157	0.0	2165.0	0
7200 min Winter	0.975	0.0	2282.6	0
8640 min Winter	0.852	0.0	2392.3	0
10080 min Winter	0.762	0.0	2496.4	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.900

Time (mins) From:	To:	Area (ha)
0	4	3.900

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	319.5	2.000	895.4

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0180-1910-2000-1910
Design Head (m)	2.000
Design Flow (l/s)	19.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	180
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	19.1
Flush-Flo™	0.584	19.1
Kick-Flo®	1.231	15.2
Mean Flow over Head Range	-	16.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	6.3	1.200	15.7	3.000	23.2	7.000	34.8
0.200	15.9	1.400	16.1	3.500	24.9	7.500	36.0
0.300	17.7	1.600	17.2	4.000	26.6	8.000	37.1
0.400	18.6	1.800	18.2	4.500	28.1	8.500	38.3
0.500	19.0	2.000	19.1	5.000	29.6	9.000	39.3
0.600	19.1	2.200	20.0	5.500	31.0	9.500	40.4
0.800	18.7	2.400	20.8	6.000	32.3		
1.000	17.8	2.600	21.6	6.500	33.6		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.088	1.088	17.6	450.0	O K
30 min Summer	99.304	1.304	17.6	576.1	O K
60 min Summer	99.480	1.480	17.6	689.7	O K
120 min Summer	99.618	1.618	17.6	785.0	O K
180 min Summer	99.670	1.670	17.6	822.3	O K
240 min Summer	99.684	1.684	17.6	832.7	O K
360 min Summer	99.655	1.655	17.6	811.3	O K
480 min Summer	99.602	1.602	17.6	773.7	O K
600 min Summer	99.558	1.558	17.6	742.2	O K
720 min Summer	99.516	1.516	17.6	713.9	O K
960 min Summer	99.439	1.439	17.6	662.4	O K
1440 min Summer	99.290	1.290	17.6	567.6	O K
2160 min Summer	99.070	1.070	17.6	440.2	O K
2880 min Summer	98.861	0.861	17.6	331.6	O K
4320 min Summer	98.492	0.492	17.6	167.7	O K
5760 min Summer	98.212	0.212	17.6	65.8	O K
7200 min Summer	98.045	0.045	17.6	13.1	O K
8640 min Summer	98.000	0.000	16.7	0.0	O K
10080 min Summer	98.000	0.000	14.9	0.0	O K
15 min Winter	99.088	1.088	17.6	449.8	O K
30 min Winter	99.306	1.306	17.6	577.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.350	0.0	467.8	19
30 min Summer	67.948	0.0	609.1	33
60 min Summer	42.061	0.0	755.3	64
120 min Summer	25.395	0.0	913.1	122
180 min Summer	18.778	0.0	1013.1	182
240 min Summer	15.101	0.0	1086.3	242
360 min Summer	11.032	0.0	1190.4	360
480 min Summer	8.780	0.0	1263.2	412
600 min Summer	7.336	0.0	1319.3	476
720 min Summer	6.326	0.0	1365.1	534
960 min Summer	4.996	0.0	1437.4	664
1440 min Summer	3.573	0.0	1542.0	938
2160 min Summer	2.552	0.0	1653.4	1324
2880 min Summer	2.014	0.0	1739.4	1704
4320 min Summer	1.451	0.0	1879.2	2420
5760 min Summer	1.157	0.0	1998.3	3096
7200 min Summer	0.975	0.0	2106.7	4480
8640 min Summer	0.852	0.0	2208.2	0
10080 min Summer	0.762	0.0	2304.4	0
15 min Winter	104.350	0.0	467.4	18
30 min Winter	67.948	0.0	609.4	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.484	1.484	17.6	692.2	O K
120 min Winter	99.623	1.623	17.6	788.5	O K
180 min Winter	99.677	1.677	17.6	827.7	O K
240 min Winter	99.695	1.695	17.6	840.6	O K
360 min Winter	99.676	1.676	17.6	826.4	O K
480 min Winter	99.618	1.618	17.6	784.9	O K
600 min Winter	99.558	1.558	17.6	742.2	O K
720 min Winter	99.509	1.509	17.6	708.9	O K
960 min Winter	99.405	1.405	17.6	640.0	O K
1440 min Winter	99.184	1.184	17.6	504.1	O K
2160 min Winter	98.841	0.841	17.6	321.5	O K
2880 min Winter	98.512	0.512	17.6	175.8	O K
4320 min Winter	98.025	0.025	17.6	7.2	O K
5760 min Winter	98.000	0.000	14.6	0.0	O K
7200 min Winter	98.000	0.000	12.3	0.0	O K
8640 min Winter	98.000	0.000	10.8	0.0	O K
10080 min Winter	98.000	0.000	9.6	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	42.061	0.0	756.3	62
120 min Winter	25.395	0.0	913.3	120
180 min Winter	18.778	0.0	1013.1	178
240 min Winter	15.101	0.0	1086.2	236
360 min Winter	11.032	0.0	1190.4	346
480 min Winter	8.780	0.0	1263.2	450
600 min Winter	7.336	0.0	1319.3	484
720 min Winter	6.326	0.0	1365.1	556
960 min Winter	4.996	0.0	1437.4	706
1440 min Winter	3.573	0.0	1542.1	996
2160 min Winter	2.552	0.0	1653.4	1404
2880 min Winter	2.014	0.0	1739.4	1756
4320 min Winter	1.451	0.0	1879.3	3008
5760 min Winter	1.157	0.0	1998.5	0
7200 min Winter	0.975	0.0	2107.0	0
8640 min Winter	0.852	0.0	2208.2	0
10080 min Winter	0.762	0.0	2304.4	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.600

Time (mins) From:	To:	Area (ha)
0	4	3.600

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	288.0	2.000	842.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0173-1760-2000-1760
Design Head (m)	2.000
Design Flow (l/s)	17.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	173
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	17.6
Flush-Flo™	0.578	17.6
Kick-Flo®	1.223	13.9
Mean Flow over Head Range	-	15.4

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	6.1	1.200	14.3	3.000	21.4	7.000	32.1
0.200	14.7	1.400	14.9	3.500	23.0	7.500	33.2
0.300	16.4	1.600	15.8	4.000	24.5	8.000	34.2
0.400	17.2	1.800	16.7	4.500	25.9	8.500	35.2
0.500	17.5	2.000	17.6	5.000	27.3	9.000	36.2
0.600	17.6	2.200	18.4	5.500	28.6	9.500	37.2
0.800	17.3	2.400	19.2	6.000	29.8		
1.000	16.4	2.600	19.9	6.500	31.0		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.052	1.052	3.1	160.3	O K
30 min Summer	99.240	1.240	3.1	208.1	O K
60 min Summer	99.403	1.403	3.1	255.0	O K
120 min Summer	99.536	1.536	3.1	297.6	O K
180 min Summer	99.604	1.604	3.1	320.8	O K
240 min Summer	99.645	1.645	3.1	335.1	O K
360 min Summer	99.684	1.684	3.1	349.3	O K
480 min Summer	99.692	1.692	3.1	352.4	O K
600 min Summer	99.685	1.685	3.1	349.6	O K
720 min Summer	99.668	1.668	3.1	343.4	O K
960 min Summer	99.618	1.618	3.1	325.6	O K
1440 min Summer	99.538	1.538	3.1	298.1	O K
2160 min Summer	99.446	1.446	3.1	268.3	O K
2880 min Summer	99.364	1.364	3.1	243.5	O K
4320 min Summer	99.211	1.211	3.1	200.4	O K
5760 min Summer	99.065	1.065	3.1	163.5	O K
7200 min Summer	98.928	0.928	3.1	132.5	O K
8640 min Summer	98.795	0.795	3.1	105.6	O K
10080 min Summer	98.671	0.671	3.1	83.2	O K
15 min Winter	99.052	1.052	3.1	160.3	O K
30 min Winter	99.240	1.240	3.1	208.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.750	0.0	162.6	19
30 min Summer	65.938	0.0	190.5	34
60 min Summer	41.037	0.0	266.5	64
120 min Summer	24.646	0.0	320.0	124
180 min Summer	18.196	0.0	354.1	182
240 min Summer	14.623	0.0	379.3	242
360 min Summer	10.680	0.0	414.3	362
480 min Summer	8.496	0.0	430.5	482
600 min Summer	7.097	0.0	436.3	602
720 min Summer	6.119	0.0	440.9	720
960 min Summer	4.834	0.0	448.2	896
1440 min Summer	3.455	0.0	451.8	1112
2160 min Summer	2.469	0.0	577.7	1496
2880 min Summer	1.950	0.0	608.4	1904
4320 min Summer	1.406	0.0	657.8	2724
5760 min Summer	1.122	0.0	700.0	3464
7200 min Summer	0.947	0.0	738.6	4256
8640 min Summer	0.828	0.0	774.8	5016
10080 min Summer	0.741	0.0	809.1	5656
15 min Winter	100.750	0.0	162.6	19
30 min Winter	65.938	0.0	190.4	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.403	1.403	3.1	255.0	O K
120 min Winter	99.537	1.537	3.1	298.0	O K
180 min Winter	99.606	1.606	3.1	321.6	O K
240 min Winter	99.647	1.647	3.1	336.1	O K
360 min Winter	99.688	1.688	3.1	350.9	O K
480 min Winter	99.699	1.699	3.1	354.7	O K
600 min Winter	99.694	1.694	3.1	353.1	O K
720 min Winter	99.681	1.681	3.1	348.1	O K
960 min Winter	99.637	1.637	3.1	332.5	O K
1440 min Winter	99.537	1.537	3.1	297.7	O K
2160 min Winter	99.419	1.419	3.1	259.8	O K
2880 min Winter	99.298	1.298	3.1	224.2	O K
4320 min Winter	99.053	1.053	3.1	160.5	O K
5760 min Winter	98.804	0.804	3.1	107.4	O K
7200 min Winter	98.559	0.559	3.1	65.0	O K
8640 min Winter	98.320	0.320	3.1	32.3	O K
10080 min Winter	98.112	0.112	3.1	9.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	41.037	0.0	266.4	62
120 min Winter	24.646	0.0	320.0	122
180 min Winter	18.196	0.0	354.3	180
240 min Winter	14.623	0.0	379.4	238
360 min Winter	10.680	0.0	414.2	356
480 min Winter	8.496	0.0	429.5	470
600 min Winter	7.097	0.0	434.8	584
720 min Winter	6.119	0.0	438.8	696
960 min Winter	4.834	0.0	445.4	914
1440 min Winter	3.455	0.0	452.5	1142
2160 min Winter	2.469	0.0	577.7	1600
2880 min Winter	1.950	0.0	608.4	2048
4320 min Winter	1.406	0.0	657.9	2856
5760 min Winter	1.122	0.0	700.0	3640
7200 min Winter	0.947	0.0	738.7	4328
8640 min Winter	0.828	0.0	774.8	4936
10080 min Winter	0.741	0.0	809.1	5552

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.300

Time (mins) From:	To:	Area (ha)
0	4	1.300

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	82.1	2.000	442.9

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0071-3100-2000-3100
Design Head (m)	2.000
Design Flow (l/s)	3.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	71
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.1
Flush-Flo™	0.313	2.3
Kick-Flo®	0.640	1.8
Mean Flow over Head Range	-	2.4

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.9	1.200	2.4	3.000	3.7	7.000	5.6
0.200	2.2	1.400	2.6	3.500	4.0	7.500	5.7
0.300	2.3	1.600	2.8	4.000	4.3	8.000	5.9
0.400	2.3	1.800	2.9	4.500	4.5	8.500	6.1
0.500	2.2	2.000	3.1	5.000	4.7	9.000	6.3
0.600	2.0	2.200	3.2	5.500	5.0	9.500	6.4
0.800	2.0	2.400	3.4	6.000	5.2		
1.000	2.3	2.600	3.5	6.500	5.4		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.063	1.063	3.0	147.8	O K
30 min Summer	99.248	1.248	3.0	191.8	O K
60 min Summer	99.408	1.408	3.0	234.9	O K
120 min Summer	99.538	1.538	3.0	273.6	O K
180 min Summer	99.603	1.603	3.0	294.6	O K
240 min Summer	99.642	1.642	3.0	307.3	O K
360 min Summer	99.677	1.677	3.0	319.4	O K
480 min Summer	99.683	1.683	3.0	321.3	O K
600 min Summer	99.672	1.672	3.0	317.8	O K
720 min Summer	99.653	1.653	3.0	311.0	O K
960 min Summer	99.601	1.601	3.0	294.0	O K
1440 min Summer	99.522	1.522	3.0	268.8	O K
2160 min Summer	99.429	1.429	3.0	240.9	O K
2880 min Summer	99.345	1.345	3.0	217.2	O K
4320 min Summer	99.185	1.185	3.0	176.2	O K
5760 min Summer	99.033	1.033	3.0	141.3	O K
7200 min Summer	98.887	0.887	3.0	111.8	O K
8640 min Summer	98.749	0.749	3.0	87.2	O K
10080 min Summer	98.619	0.619	3.0	66.7	O K
15 min Winter	99.063	1.063	3.0	147.8	O K
30 min Winter	99.248	1.248	3.0	191.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.750	0.0	150.4	19
30 min Summer	65.938	0.0	184.0	34
60 min Summer	41.037	0.0	246.1	64
120 min Summer	24.646	0.0	295.5	124
180 min Summer	18.196	0.0	327.1	182
240 min Summer	14.623	0.0	350.5	242
360 min Summer	10.680	0.0	383.6	362
480 min Summer	8.496	0.0	405.9	482
600 min Summer	7.097	0.0	419.1	600
720 min Summer	6.119	0.0	424.6	720
960 min Summer	4.834	0.0	430.6	858
1440 min Summer	3.455	0.0	430.7	1084
2160 min Summer	2.469	0.0	533.3	1476
2880 min Summer	1.950	0.0	561.6	1876
4320 min Summer	1.406	0.0	607.3	2684
5760 min Summer	1.122	0.0	646.2	3464
7200 min Summer	0.947	0.0	681.8	4184
8640 min Summer	0.828	0.0	715.2	4936
10080 min Summer	0.741	0.0	746.9	5648
15 min Winter	100.750	0.0	150.4	19
30 min Winter	65.938	0.0	183.9	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.408	1.408	3.0	234.9	O K
120 min Winter	99.539	1.539	3.0	274.1	O K
180 min Winter	99.606	1.606	3.0	295.4	O K
240 min Winter	99.645	1.645	3.0	308.3	O K
360 min Winter	99.682	1.682	3.0	321.0	O K
480 min Winter	99.690	1.690	3.0	323.7	O K
600 min Winter	99.683	1.683	3.0	321.3	O K
720 min Winter	99.667	1.667	3.0	315.8	O K
960 min Winter	99.619	1.619	3.0	299.9	O K
1440 min Winter	99.519	1.519	3.0	268.0	O K
2160 min Winter	99.396	1.396	3.0	231.5	O K
2880 min Winter	99.270	1.270	3.0	197.3	O K
4320 min Winter	99.011	1.011	3.0	136.7	O K
5760 min Winter	98.748	0.748	3.0	86.9	O K
7200 min Winter	98.486	0.486	3.0	48.3	O K
8640 min Winter	98.236	0.236	3.0	19.9	O K
10080 min Winter	98.039	0.039	3.0	2.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	41.037	0.0	246.0	62
120 min Winter	24.646	0.0	295.5	122
180 min Winter	18.196	0.0	327.2	180
240 min Winter	14.623	0.0	350.5	238
360 min Winter	10.680	0.0	383.6	356
480 min Winter	8.496	0.0	405.8	470
600 min Winter	7.097	0.0	417.9	584
720 min Winter	6.119	0.0	422.5	694
960 min Winter	4.834	0.0	428.5	906
1440 min Winter	3.455	0.0	431.4	1126
2160 min Winter	2.469	0.0	533.3	1580
2880 min Winter	1.950	0.0	561.6	2020
4320 min Winter	1.406	0.0	607.3	2852
5760 min Winter	1.122	0.0	646.1	3648
7200 min Winter	0.947	0.0	681.9	4256
8640 min Winter	0.828	0.0	715.2	4848
10080 min Winter	0.741	0.0	746.9	5352

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.200

Time (mins) From:	To:	Area (ha)
0	4	1.200

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	71.7	2.000	418.3

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0070-3000-2000-3000
Design Head (m)	2.000
Design Flow (l/s)	3.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	70
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.0	Kick-Flo®	0.630	1.8
Flush-Flo™	0.310	2.2	Mean Flow over Head Range	-	2.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.8	1.200	2.4	3.000	3.6	7.000	5.4
0.200	2.1	1.400	2.5	3.500	3.9	7.500	5.6
0.300	2.2	1.600	2.7	4.000	4.1	8.000	5.7
0.400	2.2	1.800	2.9	4.500	4.4	8.500	5.9
0.500	2.1	2.000	3.0	5.000	4.6	9.000	6.1
0.600	1.9	2.200	3.1	5.500	4.8	9.500	6.2
0.800	2.0	2.400	3.3	6.000	5.0		
1.000	2.2	2.600	3.4	6.500	5.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.983	0.983	7.0	372.9	O K
30 min Summer	99.204	1.204	7.0	485.6	O K
60 min Summer	99.403	1.403	7.0	597.6	O K
120 min Summer	99.547	1.547	7.0	685.0	O K
180 min Summer	99.615	1.615	7.0	728.3	O K
240 min Summer	99.653	1.653	7.0	753.1	O K
360 min Summer	99.685	1.685	7.0	774.2	O K
480 min Summer	99.688	1.688	7.0	775.9	O K
600 min Summer	99.675	1.675	7.0	767.5	O K
720 min Summer	99.653	1.653	7.0	753.0	O K
960 min Summer	99.596	1.596	7.0	716.1	O K
1440 min Summer	99.515	1.515	7.0	665.1	O K
2160 min Summer	99.423	1.423	7.0	609.4	O K
2880 min Summer	99.339	1.339	7.0	560.6	O K
4320 min Summer	99.179	1.179	7.0	472.2	O K
5760 min Summer	99.024	1.024	7.0	392.8	O K
7200 min Summer	98.877	0.877	7.0	322.7	O K
8640 min Summer	98.738	0.738	7.0	260.8	O K
10080 min Summer	98.608	0.608	7.0	206.8	O K
15 min Winter	98.983	0.983	7.0	372.9	O K
30 min Winter	99.204	1.204	7.0	485.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	105.017	0.0	375.8	19
30 min Summer	68.922	0.0	481.4	34
60 min Summer	43.037	0.0	622.1	64
120 min Summer	25.400	0.0	734.0	124
180 min Summer	18.512	0.0	801.7	182
240 min Summer	14.741	0.0	850.9	242
360 min Summer	10.644	0.0	920.5	362
480 min Summer	8.430	0.0	970.1	482
600 min Summer	7.033	0.0	1008.2	602
720 min Summer	6.065	0.0	1037.6	720
960 min Summer	4.806	0.0	1065.9	878
1440 min Summer	3.470	0.0	1049.2	1112
2160 min Summer	2.509	0.0	1308.8	1496
2880 min Summer	1.998	0.0	1389.0	1904
4320 min Summer	1.455	0.0	1516.0	2724
5760 min Summer	1.166	0.0	1622.9	3680
7200 min Summer	0.986	0.0	1714.8	4256
8640 min Summer	0.861	0.0	1797.7	5016
10080 min Summer	0.770	0.0	1874.0	5752
15 min Winter	105.017	0.0	375.8	19
30 min Winter	68.922	0.0	481.4	33

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.402	1.402	7.0	597.4	O K
120 min Winter	99.548	1.548	7.0	686.0	O K
180 min Winter	99.618	1.618	7.0	730.2	O K
240 min Winter	99.656	1.656	7.0	755.4	O K
360 min Winter	99.690	1.690	7.0	777.8	O K
480 min Winter	99.696	1.696	7.0	781.4	O K
600 min Winter	99.687	1.687	7.0	775.5	O K
720 min Winter	99.669	1.669	7.0	763.7	O K
960 min Winter	99.619	1.619	7.0	731.0	O K
1440 min Winter	99.513	1.513	7.0	664.1	O K
2160 min Winter	99.390	1.390	7.0	590.4	O K
2880 min Winter	99.263	1.263	7.0	518.0	O K
4320 min Winter	99.004	1.004	7.0	382.9	O K
5760 min Winter	98.750	0.750	7.0	265.7	O K
7200 min Winter	98.510	0.510	7.0	168.2	O K
8640 min Winter	98.293	0.293	7.0	90.6	O K
10080 min Winter	98.116	0.116	7.0	33.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.037	0.0	621.8	62
120 min Winter	25.400	0.0	734.0	122
180 min Winter	18.512	0.0	802.0	180
240 min Winter	14.741	0.0	851.0	238
360 min Winter	10.644	0.0	920.4	356
480 min Winter	8.430	0.0	969.8	470
600 min Winter	7.033	0.0	1007.7	584
720 min Winter	6.065	0.0	1035.8	696
960 min Winter	4.806	0.0	1059.4	914
1440 min Winter	3.470	0.0	1051.7	1140
2160 min Winter	2.509	0.0	1308.8	1600
2880 min Winter	1.998	0.0	1389.1	2048
4320 min Winter	1.455	0.0	1516.5	2896
5760 min Winter	1.166	0.0	1622.9	3640
7200 min Winter	0.986	0.0	1714.8	4400
8640 min Winter	0.861	0.0	1797.8	5016
10080 min Winter	0.770	0.0	1874.2	5552

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 372232 388607 SJ 72232 88607
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.900

Time (mins) From:	To:	Area (ha)
0	4	2.900

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	282.2	2.000	752.6

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0109-7000-2000-7000
Design Head (m)	2.000
Design Flow (l/s)	7.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	109
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	7.0
Flush-Flo™	0.475	6.3
Kick-Flo®	0.973	5.0
Mean Flow over Head Range	-	5.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.7	1.200	5.5	3.000	8.5	7.000	12.7
0.200	5.6	1.400	5.9	3.500	9.1	7.500	13.1
0.300	6.1	1.600	6.3	4.000	9.7	8.000	13.5
0.400	6.3	1.800	6.7	4.500	10.3	8.500	13.9
0.500	6.3	2.000	7.0	5.000	10.8	9.000	14.3
0.600	6.2	2.200	7.3	5.500	11.3	9.500	14.6
0.800	5.9	2.400	7.6	6.000	11.8		
1.000	5.1	2.600	7.9	6.500	12.2		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.241	1.241	20.5	297.1	O K
30 min Summer	99.468	1.468	20.5	381.3	O K
60 min Summer	99.636	1.636	20.5	450.4	O K
120 min Summer	99.686	1.686	20.5	472.4	O K
180 min Summer	99.663	1.663	20.5	462.1	O K
240 min Summer	99.623	1.623	20.5	444.7	O K
360 min Summer	99.554	1.554	20.5	416.0	O K
480 min Summer	99.491	1.491	20.5	390.3	O K
600 min Summer	99.428	1.428	20.5	365.4	O K
720 min Summer	99.364	1.364	20.5	341.2	O K
960 min Summer	99.234	1.234	20.5	294.5	O K
1440 min Summer	98.978	0.978	20.5	212.2	O K
2160 min Summer	98.614	0.614	20.5	115.6	O K
2880 min Summer	98.312	0.312	20.5	51.8	O K
4320 min Summer	98.000	0.000	20.2	0.0	O K
5760 min Summer	98.000	0.000	16.4	0.0	O K
7200 min Summer	98.000	0.000	14.0	0.0	O K
8640 min Summer	98.000	0.000	12.3	0.0	O K
10080 min Summer	98.000	0.000	11.1	0.0	O K
15 min Winter	99.243	1.243	20.5	297.6	O K
30 min Winter	99.471	1.471	20.5	382.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	94.738	0.0	319.1	18
30 min Summer	62.468	0.0	421.3	33
60 min Summer	39.034	0.0	526.6	62
120 min Summer	22.999	0.0	620.8	122
180 min Summer	16.864	0.0	682.9	180
240 min Summer	13.505	0.0	729.1	208
360 min Summer	9.820	0.0	795.3	274
480 min Summer	7.811	0.0	843.4	334
600 min Summer	6.533	0.0	881.8	404
720 min Summer	5.643	0.0	914.0	470
960 min Summer	4.477	0.0	966.8	606
1440 min Summer	3.238	0.0	1048.7	866
2160 min Summer	2.347	0.0	1140.4	1216
2880 min Summer	1.874	0.0	1214.3	1560
4320 min Summer	1.376	0.0	1337.9	0
5760 min Summer	1.114	0.0	1443.6	0
7200 min Summer	0.951	0.0	1540.3	0
8640 min Summer	0.839	0.0	1630.8	0
10080 min Summer	0.757	0.0	1717.0	0
15 min Winter	94.738	0.0	319.4	18
30 min Winter	62.468	0.0	421.0	32

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	99.641	1.641	20.5	452.7	O K
120 min Winter	99.697	1.697	20.5	477.3	O K
180 min Winter	99.683	1.683	20.5	470.9	O K
240 min Winter	99.637	1.637	20.5	450.8	O K
360 min Winter	99.547	1.547	20.5	413.1	O K
480 min Winter	99.459	1.459	20.5	377.7	O K
600 min Winter	99.364	1.364	20.5	341.2	O K
720 min Winter	99.264	1.264	20.5	304.9	O K
960 min Winter	99.056	1.056	20.5	236.0	O K
1440 min Winter	98.635	0.635	20.5	120.6	O K
2160 min Winter	98.085	0.085	20.5	12.8	O K
2880 min Winter	98.000	0.000	17.8	0.0	O K
4320 min Winter	98.000	0.000	13.1	0.0	O K
5760 min Winter	98.000	0.000	10.6	0.0	O K
7200 min Winter	98.000	0.000	9.0	0.0	O K
8640 min Winter	98.000	0.000	8.0	0.0	O K
10080 min Winter	98.000	0.000	7.2	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	39.034	0.0	526.8	62
120 min Winter	22.999	0.0	620.7	118
180 min Winter	16.864	0.0	682.9	174
240 min Winter	13.505	0.0	729.1	226
360 min Winter	9.820	0.0	795.2	280
480 min Winter	7.811	0.0	843.4	356
600 min Winter	6.533	0.0	881.8	440
720 min Winter	5.643	0.0	914.0	502
960 min Winter	4.477	0.0	966.8	638
1440 min Winter	3.238	0.0	1048.8	884
2160 min Winter	2.347	0.0	1140.5	1284
2880 min Winter	1.874	0.0	1214.4	0
4320 min Winter	1.376	0.0	1337.9	0
5760 min Winter	1.114	0.0	1443.6	0
7200 min Winter	0.951	0.0	1540.3	0
8640 min Winter	0.839	0.0	1630.8	0
10080 min Winter	0.757	0.0	1717.0	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 351306 389946 SJ 51306 89946
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.700

Time (mins) From:	To:	Area (ha)
0	4	2.700

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	145.3	2.000	514.7

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0186-2050-2000-2050
Design Head (m)	2.000
Design Flow (l/s)	20.5
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	186
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	20.5
Flush-Flo™	0.582	20.4
Kick-Flo®	1.230	16.2
Mean Flow over Head Range	-	17.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	6.5	1.200	16.8	3.000	24.8	7.000	37.3
0.200	16.9	1.400	17.3	3.500	26.7	7.500	38.6
0.300	18.9	1.600	18.4	4.000	28.5	8.000	39.8
0.400	19.9	1.800	19.5	4.500	30.2	8.500	41.0
0.500	20.3	2.000	20.5	5.000	31.7	9.000	42.1
0.600	20.3	2.200	21.4	5.500	33.2	9.500	43.3
0.800	20.0	2.400	22.3	6.000	34.6		
1.000	19.0	2.600	23.2	6.500	36.0		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.220	1.220	28.1	407.0	O K
30 min Summer	99.457	1.457	28.1	522.4	O K
60 min Summer	99.634	1.634	28.1	617.1	O K
120 min Summer	99.687	1.687	28.1	647.1	O K
180 min Summer	99.662	1.662	28.1	632.8	O K
240 min Summer	99.620	1.620	28.1	609.0	O K
360 min Summer	99.547	1.547	28.1	569.6	O K
480 min Summer	99.481	1.481	28.1	534.5	O K
600 min Summer	99.414	1.414	28.1	500.2	O K
720 min Summer	99.347	1.347	28.1	467.0	O K
960 min Summer	99.211	1.211	28.1	403.0	O K
1440 min Summer	98.947	0.947	28.1	290.1	O K
2160 min Summer	98.580	0.580	28.1	157.7	O K
2880 min Summer	98.286	0.286	28.1	70.4	O K
4320 min Summer	98.000	0.000	27.7	0.0	O K
5760 min Summer	98.000	0.000	22.4	0.0	O K
15 min Winter	99.222	1.222	28.1	407.7	O K
30 min Winter	99.460	1.460	28.1	523.6	O K
60 min Winter	99.640	1.640	28.1	620.2	O K
120 min Winter	99.699	1.699	28.1	653.9	O K
180 min Winter	99.683	1.683	28.1	644.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	94.720	0.0	436.9	18
30 min Summer	62.460	0.0	576.9	33
60 min Summer	39.030	0.0	721.4	62
120 min Summer	23.000	0.0	850.6	122
180 min Summer	16.863	0.0	935.5	180
240 min Summer	13.505	0.0	999.0	208
360 min Summer	9.820	0.0	1089.6	268
480 min Summer	7.811	0.0	1155.6	334
600 min Summer	6.533	0.0	1208.2	404
720 min Summer	5.643	0.0	1252.4	470
960 min Summer	4.477	0.0	1324.6	606
1440 min Summer	3.238	0.0	1437.0	866
2160 min Summer	2.347	0.0	1562.8	1216
2880 min Summer	1.874	0.0	1663.9	1584
4320 min Summer	1.377	0.0	1833.5	0
5760 min Summer	1.114	0.0	1978.2	0
15 min Winter	94.720	0.0	437.3	18
30 min Winter	62.460	0.0	576.6	32
60 min Winter	39.030	0.0	721.7	62
120 min Winter	23.000	0.0	850.5	118
180 min Winter	16.863	0.0	935.5	174

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.635	1.635	28.1	617.3	O K
360 min Winter	99.540	1.540	28.1	565.6	O K
480 min Winter	99.447	1.447	28.1	517.1	O K
600 min Winter	99.347	1.347	28.1	466.9	O K
720 min Winter	99.242	1.242	28.1	417.2	O K
960 min Winter	99.027	1.027	28.1	322.6	O K
1440 min Winter	98.601	0.601	28.1	164.5	O K
2160 min Winter	98.075	0.075	28.1	17.1	O K
2880 min Winter	98.000	0.000	24.4	0.0	O K
4320 min Winter	98.000	0.000	17.9	0.0	O K
5760 min Winter	98.000	0.000	14.5	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	13.505	0.0	999.0	226
360 min Winter	9.820	0.0	1089.6	280
480 min Winter	7.811	0.0	1155.6	356
600 min Winter	6.533	0.0	1208.1	430
720 min Winter	5.643	0.0	1252.4	502
960 min Winter	4.477	0.0	1324.7	638
1440 min Winter	3.238	0.0	1437.0	884
2160 min Winter	2.347	0.0	1562.9	1176
2880 min Winter	1.874	0.0	1664.3	0
4320 min Winter	1.377	0.0	1833.5	0
5760 min Winter	1.114	0.0	1978.2	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 351306 389946 SJ 51306 89946
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 3.700

Time (mins) From:	To:	Area (ha)
0	4	3.700

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	222.7	2.000	653.3

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0216-2810-2000-2810
Design Head (m)	2.000
Design Flow (l/s)	28.1
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	216
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	28.1
Flush-Flo™	0.586	27.9
Kick-Flo®	1.256	22.5
Mean Flow over Head Range	-	24.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	7.3	1.200	23.8	3.000	34.1	7.000	51.3
0.200	21.1	1.400	23.7	3.500	36.7	7.500	53.1
0.300	25.9	1.600	25.2	4.000	39.2	8.000	54.8
0.400	27.2	1.800	26.7	4.500	41.5	8.500	56.4
0.500	27.8	2.000	28.1	5.000	43.6	9.000	58.0
0.600	27.9	2.200	29.4	5.500	45.7	9.500	59.5
0.800	27.5	2.400	30.6	6.000	47.6		
1.000	26.4	2.600	31.8	6.500	49.5		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.947	0.947	6.7	345.2	O K
30 min Summer	99.158	1.158	6.7	448.1	O K
60 min Summer	99.346	1.346	6.7	549.2	O K
120 min Summer	99.503	1.503	6.7	640.6	O K
180 min Summer	99.585	1.585	6.7	690.6	O K
240 min Summer	99.634	1.634	6.7	721.3	O K
360 min Summer	99.681	1.681	6.7	751.7	O K
480 min Summer	99.691	1.691	6.7	758.1	O K
600 min Summer	99.682	1.682	6.7	752.0	O K
720 min Summer	99.661	1.661	6.7	738.3	O K
960 min Summer	99.600	1.600	6.7	699.9	O K
1440 min Summer	99.503	1.503	6.7	640.5	O K
2160 min Summer	99.394	1.394	6.7	576.4	O K
2880 min Summer	99.298	1.298	6.7	522.7	O K
4320 min Summer	99.121	1.121	6.7	429.4	O K
5760 min Summer	98.958	0.958	6.7	350.0	O K
15 min Winter	98.947	0.947	6.7	345.2	O K
30 min Winter	99.158	1.158	6.7	448.2	O K
60 min Winter	99.346	1.346	6.7	549.1	O K
120 min Winter	99.505	1.505	6.7	641.6	O K
180 min Winter	99.588	1.588	6.7	692.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	348.3	19
30 min Summer	65.940	0.0	447.9	34
60 min Summer	41.040	0.0	572.8	64
120 min Summer	24.645	0.0	687.7	124
180 min Summer	18.197	0.0	761.0	182
240 min Summer	14.623	0.0	814.9	242
360 min Summer	10.680	0.0	891.2	362
480 min Summer	8.496	0.0	942.4	482
600 min Summer	7.097	0.0	978.5	602
720 min Summer	6.118	0.0	1001.6	720
960 min Summer	4.834	0.0	1018.3	896
1440 min Summer	3.455	0.0	1000.5	1112
2160 min Summer	2.469	0.0	1243.6	1496
2880 min Summer	1.950	0.0	1309.4	1904
4320 min Summer	1.406	0.0	1415.3	2724
5760 min Summer	1.122	0.0	1507.4	3464
15 min Winter	100.760	0.0	348.3	19
30 min Winter	65.940	0.0	447.8	33
60 min Winter	41.040	0.0	572.6	62
120 min Winter	24.645	0.0	687.7	122
180 min Winter	18.197	0.0	761.2	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.637	1.637	6.7	723.5	O K
360 min Winter	99.687	1.687	6.7	755.1	O K
480 min Winter	99.699	1.699	6.7	763.3	O K
600 min Winter	99.693	1.693	6.7	759.5	O K
720 min Winter	99.676	1.676	6.7	748.5	O K
960 min Winter	99.624	1.624	6.7	714.7	O K
1440 min Winter	99.502	1.502	6.7	639.6	O K
2160 min Winter	99.361	1.361	6.7	557.8	O K
2880 min Winter	99.220	1.220	6.7	480.7	O K
4320 min Winter	98.943	0.943	6.7	343.1	O K
5760 min Winter	98.679	0.679	6.7	228.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.623	0.0	814.9	238
360 min Winter	10.680	0.0	891.2	356
480 min Winter	8.496	0.0	942.1	470
600 min Winter	7.097	0.0	977.6	584
720 min Winter	6.118	0.0	998.2	696
960 min Winter	4.834	0.0	1011.3	914
1440 min Winter	3.455	0.0	1003.1	1140
2160 min Winter	2.469	0.0	1243.6	1600
2880 min Winter	1.950	0.0	1309.5	2020
4320 min Winter	1.406	0.0	1415.7	2856
5760 min Winter	1.122	0.0	1507.4	3640

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.800

Time (mins) From:	To:	Area (ha)
0	4	2.800

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	272.8	2.000	737.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0107-6700-2000-6700
Design Head (m)	2.000
Design Flow (l/s)	6.7
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	106
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	6.7
Flush-Flo™	0.466	6.0
Kick-Flo®	0.953	4.7
Mean Flow over Head Range	-	5.5

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.6	1.200	5.3	3.000	8.1	7.000	12.1
0.200	5.3	1.400	5.7	3.500	8.7	7.500	12.5
0.300	5.8	1.600	6.0	4.000	9.3	8.000	12.9
0.400	5.9	1.800	6.4	4.500	9.8	8.500	13.3
0.500	6.0	2.000	6.7	5.000	10.3	9.000	13.7
0.600	5.9	2.200	7.0	5.500	10.8	9.500	14.0
0.800	5.5	2.400	7.3	6.000	11.3		
1.000	4.9	2.600	7.6	6.500	11.7		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.017	1.017	3.4	172.6	O K
30 min Summer	99.214	1.214	3.4	224.0	O K
60 min Summer	99.386	1.386	3.4	274.5	O K
120 min Summer	99.527	1.527	3.4	320.0	O K
180 min Summer	99.599	1.599	3.4	344.8	O K
240 min Summer	99.642	1.642	3.4	360.0	O K
360 min Summer	99.682	1.682	3.4	374.9	O K
480 min Summer	99.690	1.690	3.4	377.8	O K
600 min Summer	99.681	1.681	3.4	374.4	O K
720 min Summer	99.662	1.662	3.4	367.2	O K
960 min Summer	99.607	1.607	3.4	347.7	O K
1440 min Summer	99.521	1.521	3.4	318.1	O K
2160 min Summer	99.422	1.422	3.4	286.0	O K
2880 min Summer	99.334	1.334	3.4	258.9	O K
4320 min Summer	99.170	1.170	3.4	211.8	O K
5760 min Summer	99.014	1.014	3.4	171.9	O K
15 min Winter	99.017	1.017	3.4	172.6	O K
30 min Winter	99.214	1.214	3.4	224.0	O K
60 min Winter	99.385	1.385	3.4	274.4	O K
120 min Winter	99.528	1.528	3.4	320.5	O K
180 min Winter	99.601	1.601	3.4	345.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	174.9	19
30 min Summer	65.940	0.0	210.3	34
60 min Summer	41.040	0.0	286.9	64
120 min Summer	24.645	0.0	344.5	124
180 min Summer	18.197	0.0	381.3	182
240 min Summer	14.623	0.0	408.3	242
360 min Summer	10.680	0.0	446.4	362
480 min Summer	8.496	0.0	469.4	482
600 min Summer	7.097	0.0	478.3	602
720 min Summer	6.118	0.0	483.5	720
960 min Summer	4.834	0.0	490.4	878
1440 min Summer	3.455	0.0	489.9	1110
2160 min Summer	2.469	0.0	622.1	1492
2880 min Summer	1.950	0.0	655.1	1904
4320 min Summer	1.406	0.0	708.2	2684
5760 min Summer	1.122	0.0	753.8	3464
15 min Winter	100.760	0.0	174.9	19
30 min Winter	65.940	0.0	210.2	33
60 min Winter	41.040	0.0	286.8	62
120 min Winter	24.645	0.0	344.5	122
180 min Winter	18.197	0.0	381.4	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.645	1.645	3.4	361.1	O K
360 min Winter	99.687	1.687	3.4	376.6	O K
480 min Winter	99.697	1.697	3.4	380.4	O K
600 min Winter	99.692	1.692	3.4	378.2	O K
720 min Winter	99.676	1.676	3.4	372.4	O K
960 min Winter	99.628	1.628	3.4	355.0	O K
1440 min Winter	99.519	1.519	3.4	317.6	O K
2160 min Winter	99.391	1.391	3.4	276.2	O K
2880 min Winter	99.261	1.261	3.4	237.3	O K
4320 min Winter	98.997	0.997	3.4	167.8	O K
5760 min Winter	98.736	0.736	3.4	110.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.623	0.0	408.3	238
360 min Winter	10.680	0.0	446.3	356
480 min Winter	8.496	0.0	468.8	470
600 min Winter	7.097	0.0	476.7	584
720 min Winter	6.118	0.0	481.1	696
960 min Winter	4.834	0.0	487.4	914
1440 min Winter	3.455	0.0	490.6	1140
2160 min Winter	2.469	0.0	622.1	1584
2880 min Winter	1.950	0.0	655.1	2020
4320 min Winter	1.406	0.0	708.3	2856
5760 min Winter	1.122	0.0	753.8	3632

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.400

Time (mins) From:	To:	Area (ha)
0	4	1.400

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	104.6	2.000	435.3

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0075-3400-2000-3400
Design Head (m)	2.000
Design Flow (l/s)	3.4
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	75
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	3.4
Flush-Flo™	0.330	2.6
Kick-Flo®	0.671	2.1
Mean Flow over Head Range	-	2.6

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	2.1	1.200	2.7	3.000	4.1	7.000	6.1
0.200	2.4	1.400	2.9	3.500	4.4	7.500	6.3
0.300	2.6	1.600	3.1	4.000	4.7	8.000	6.5
0.400	2.5	1.800	3.2	4.500	5.0	8.500	6.7
0.500	2.5	2.000	3.4	5.000	5.2	9.000	6.9
0.600	2.3	2.200	3.5	5.500	5.4	9.500	7.0
0.800	2.2	2.400	3.7	6.000	5.7		
1.000	2.5	2.600	3.8	6.500	5.9		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.952	0.952	6.2	320.6	O K
30 min Summer	99.161	1.161	6.2	416.2	O K
60 min Summer	99.348	1.348	6.2	510.1	O K
120 min Summer	99.504	1.504	6.2	595.0	O K
180 min Summer	99.585	1.585	6.2	641.5	O K
240 min Summer	99.633	1.633	6.2	670.1	O K
360 min Summer	99.680	1.680	6.2	698.5	O K
480 min Summer	99.690	1.690	6.2	704.7	O K
600 min Summer	99.681	1.681	6.2	699.1	O K
720 min Summer	99.661	1.661	6.2	686.6	O K
960 min Summer	99.601	1.601	6.2	651.0	O K
1440 min Summer	99.505	1.505	6.2	595.8	O K
2160 min Summer	99.397	1.397	6.2	536.4	O K
2880 min Summer	99.303	1.303	6.2	486.7	O K
4320 min Summer	99.128	1.128	6.2	400.4	O K
5760 min Summer	98.966	0.966	6.2	326.6	O K
15 min Winter	98.952	0.952	6.2	320.6	O K
30 min Winter	99.161	1.161	6.2	416.2	O K
60 min Winter	99.348	1.348	6.2	510.0	O K
120 min Winter	99.505	1.505	6.2	596.0	O K
180 min Winter	99.588	1.588	6.2	643.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	323.5	19
30 min Summer	65.940	0.0	413.6	34
60 min Summer	41.040	0.0	532.0	64
120 min Summer	24.645	0.0	638.7	124
180 min Summer	18.197	0.0	706.7	182
240 min Summer	14.623	0.0	756.7	242
360 min Summer	10.680	0.0	827.4	362
480 min Summer	8.496	0.0	874.1	482
600 min Summer	7.097	0.0	905.1	602
720 min Summer	6.118	0.0	921.3	720
960 min Summer	4.834	0.0	933.8	896
1440 min Summer	3.455	0.0	918.5	1112
2160 min Summer	2.469	0.0	1154.8	1496
2880 min Summer	1.950	0.0	1216.0	1904
4320 min Summer	1.406	0.0	1314.2	2724
5760 min Summer	1.122	0.0	1399.7	3464
15 min Winter	100.760	0.0	323.5	19
30 min Winter	65.940	0.0	413.6	33
60 min Winter	41.040	0.0	531.8	62
120 min Winter	24.645	0.0	638.7	122
180 min Winter	18.197	0.0	706.9	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.636	1.636	6.2	672.1	O K
360 min Winter	99.685	1.685	6.2	701.7	O K
480 min Winter	99.698	1.698	6.2	709.4	O K
600 min Winter	99.692	1.692	6.2	706.1	O K
720 min Winter	99.676	1.676	6.2	695.9	O K
960 min Winter	99.624	1.624	6.2	664.8	O K
1440 min Winter	99.504	1.504	6.2	595.1	O K
2160 min Winter	99.366	1.366	6.2	519.4	O K
2880 min Winter	99.227	1.227	6.2	448.1	O K
4320 min Winter	98.952	0.952	6.2	320.6	O K
5760 min Winter	98.690	0.690	6.2	214.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.623	0.0	756.8	238
360 min Winter	10.680	0.0	827.3	356
480 min Winter	8.496	0.0	873.7	470
600 min Winter	7.097	0.0	903.8	584
720 min Winter	6.118	0.0	917.3	696
960 min Winter	4.834	0.0	927.0	914
1440 min Winter	3.455	0.0	920.9	1140
2160 min Winter	2.469	0.0	1154.9	1600
2880 min Winter	1.950	0.0	1216.0	2048
4320 min Winter	1.406	0.0	1314.6	2856
5760 min Winter	1.122	0.0	1399.7	3640

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.600

Time (mins) From:	To:	Area (ha)
0	4	2.600

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	248.3	2.000	696.6

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0102-6200-2000-6200
Design Head (m)	2.000
Design Flow (l/s)	6.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	102
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	6.2
Flush-Flo™	0.448	5.4
Kick-Flo®	0.912	4.3
Mean Flow over Head Range	-	5.0

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.4	1.200	4.9	3.000	7.5	7.000	11.2
0.200	4.9	1.400	5.2	3.500	8.1	7.500	11.6
0.300	5.2	1.600	5.6	4.000	8.6	8.000	11.9
0.400	5.4	1.800	5.9	4.500	9.1	8.500	12.3
0.500	5.4	2.000	6.2	5.000	9.5	9.000	12.6
0.600	5.3	2.200	6.5	5.500	10.0	9.500	13.0
0.800	4.9	2.400	6.7	6.000	10.4		
1.000	4.5	2.600	7.0	6.500	10.8		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	98.886	0.886	15.8	813.7	O K
30 min Summer	99.106	1.106	15.8	1056.3	O K
60 min Summer	99.309	1.309	15.8	1294.6	O K
120 min Summer	99.481	1.481	15.8	1510.0	O K
180 min Summer	99.572	1.572	15.8	1627.6	O K
240 min Summer	99.626	1.626	15.8	1700.1	O K
360 min Summer	99.679	1.679	15.8	1771.5	O K
480 min Summer	99.691	1.691	15.8	1786.7	O K
600 min Summer	99.680	1.680	15.8	1772.1	O K
720 min Summer	99.656	1.656	15.8	1739.8	O K
960 min Summer	99.588	1.588	15.8	1649.1	O K
1440 min Summer	99.480	1.480	15.8	1509.1	O K
2160 min Summer	99.360	1.360	15.8	1358.1	O K
2880 min Summer	99.256	1.256	15.8	1231.4	O K
4320 min Summer	99.067	1.067	15.8	1011.5	O K
5760 min Summer	98.896	0.896	15.8	824.1	O K
15 min Winter	98.886	0.886	15.8	813.7	O K
30 min Winter	99.106	1.106	15.8	1056.4	O K
60 min Winter	99.308	1.308	15.8	1294.3	O K
120 min Winter	99.483	1.483	15.8	1512.3	O K
180 min Winter	99.575	1.575	15.8	1631.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.760	0.0	811.5	19
30 min Summer	65.940	0.0	1055.1	34
60 min Summer	41.040	0.0	1345.2	64
120 min Summer	24.645	0.0	1614.9	124
180 min Summer	18.197	0.0	1786.7	182
240 min Summer	14.623	0.0	1913.3	242
360 min Summer	10.680	0.0	2093.0	362
480 min Summer	8.496	0.0	2215.7	482
600 min Summer	7.097	0.0	2308.0	602
720 min Summer	6.118	0.0	2381.3	720
960 min Summer	4.834	0.0	2488.4	896
1440 min Summer	3.455	0.0	2508.5	1112
2160 min Summer	2.469	0.0	2927.9	1496
2880 min Summer	1.950	0.0	3082.6	1904
4320 min Summer	1.406	0.0	3331.3	2724
5760 min Summer	1.122	0.0	3551.6	3464
15 min Winter	100.760	0.0	811.5	19
30 min Winter	65.940	0.0	1054.9	33
60 min Winter	41.040	0.0	1344.6	62
120 min Winter	24.645	0.0	1614.9	122
180 min Winter	18.197	0.0	1787.3	180

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.630	1.630	15.8	1705.1	O K
360 min Winter	99.685	1.685	15.8	1779.6	O K
480 min Winter	99.699	1.699	15.8	1798.8	O K
600 min Winter	99.693	1.693	15.8	1789.9	O K
720 min Winter	99.674	1.674	15.8	1763.8	O K
960 min Winter	99.614	1.614	15.8	1684.0	O K
1440 min Winter	99.479	1.479	15.8	1507.1	O K
2160 min Winter	99.324	1.324	15.8	1314.0	O K
2880 min Winter	99.172	1.172	15.8	1132.3	O K
4320 min Winter	98.881	0.881	15.8	807.8	O K
5760 min Winter	98.615	0.615	15.8	537.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.623	0.0	1913.4	238
360 min Winter	10.680	0.0	2092.8	356
480 min Winter	8.496	0.0	2215.3	470
600 min Winter	7.097	0.0	2307.4	584
720 min Winter	6.118	0.0	2379.6	696
960 min Winter	4.834	0.0	2482.8	914
1440 min Winter	3.455	0.0	2516.0	1140
2160 min Winter	2.469	0.0	2928.0	1584
2880 min Winter	1.950	0.0	3083.0	2020
4320 min Winter	1.406	0.0	3332.2	2856
5760 min Winter	1.122	0.0	3551.6	3632

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 358800 385744 SJ 58800 85744
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 6.600

Time (mins) From:	To:	Area (ha)
0	4	6.600

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	779.0	2.000	1485.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0164-1580-2000-1580
Design Head (m)	2.000
Design Flow (l/s)	15.8
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	164
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	15.8
Flush-Flo™	0.590	15.8
Kick-Flo®	1.228	12.5
Mean Flow over Head Range	-	13.8

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.9	1.200	13.0	3.000	19.2	7.000	28.8
0.200	13.2	1.400	13.3	3.500	20.6	7.500	29.7
0.300	14.6	1.600	14.2	4.000	22.0	8.000	30.7
0.400	15.4	1.800	15.0	4.500	23.3	8.500	31.6
0.500	15.7	2.000	15.8	5.000	24.5	9.000	32.5
0.600	15.8	2.200	16.5	5.500	25.6	9.500	33.3
0.800	15.5	2.400	17.2	6.000	26.7		
1.000	14.8	2.600	17.9	6.500	27.8		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.105	1.105	11.3	287.5	O K
30 min Summer	99.316	1.316	11.3	367.9	O K
60 min Summer	99.487	1.487	11.3	440.4	O K
120 min Summer	99.621	1.621	11.3	501.2	O K
180 min Summer	99.670	1.670	11.3	524.8	O K
240 min Summer	99.683	1.683	11.3	531.3	O K
360 min Summer	99.654	1.654	11.3	517.3	O K
480 min Summer	99.604	1.604	11.3	493.3	O K
600 min Summer	99.560	1.560	11.3	473.2	O K
720 min Summer	99.521	1.521	11.3	455.1	O K
960 min Summer	99.446	1.446	11.3	422.2	O K
1440 min Summer	99.300	1.300	11.3	361.4	O K
2160 min Summer	99.084	1.084	11.3	279.8	O K
2880 min Summer	98.877	0.877	11.3	210.4	O K
4320 min Summer	98.505	0.505	11.3	105.6	O K
5760 min Summer	98.219	0.219	11.3	41.0	O K
15 min Winter	99.105	1.105	11.3	287.3	O K
30 min Winter	99.318	1.318	11.3	368.6	O K
60 min Winter	99.491	1.491	11.3	442.0	O K
120 min Winter	99.625	1.625	11.3	503.5	O K
180 min Winter	99.677	1.677	11.3	528.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.360	0.0	299.3	19
30 min Summer	67.940	0.0	389.6	33
60 min Summer	42.060	0.0	482.7	64
120 min Summer	25.395	0.0	583.6	122
180 min Summer	18.777	0.0	647.4	182
240 min Summer	15.100	0.0	694.2	242
360 min Summer	11.032	0.0	760.8	360
480 min Summer	8.780	0.0	807.3	412
600 min Summer	7.336	0.0	843.2	472
720 min Summer	6.326	0.0	872.5	534
960 min Summer	4.996	0.0	918.7	664
1440 min Summer	3.573	0.0	985.6	938
2160 min Summer	2.552	0.0	1056.5	1324
2880 min Summer	2.014	0.0	1111.5	1704
4320 min Summer	1.451	0.0	1200.8	2420
5760 min Summer	1.157	0.0	1276.8	3064
15 min Winter	104.360	0.0	299.1	18
30 min Winter	67.940	0.0	389.8	33
60 min Winter	42.060	0.0	483.4	62
120 min Winter	25.395	0.0	583.7	120
180 min Winter	18.777	0.0	647.4	178

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.694	1.694	11.3	536.3	O K
360 min Winter	99.675	1.675	11.3	527.0	O K
480 min Winter	99.619	1.619	11.3	500.3	O K
600 min Winter	99.560	1.560	11.3	473.2	O K
720 min Winter	99.513	1.513	11.3	451.8	O K
960 min Winter	99.412	1.412	11.3	407.6	O K
1440 min Winter	99.195	1.195	11.3	320.5	O K
2160 min Winter	98.855	0.855	11.3	203.5	O K
2880 min Winter	98.524	0.524	11.3	110.5	O K
4320 min Winter	98.022	0.022	11.3	3.8	O K
5760 min Winter	98.000	0.000	9.3	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	15.100	0.0	694.1	236
360 min Winter	11.032	0.0	760.8	346
480 min Winter	8.780	0.0	807.3	450
600 min Winter	7.336	0.0	843.2	482
720 min Winter	6.326	0.0	872.5	556
960 min Winter	4.996	0.0	918.7	706
1440 min Winter	3.573	0.0	985.6	996
2160 min Winter	2.552	0.0	1056.5	1404
2880 min Winter	2.014	0.0	1111.5	1756
4320 min Winter	1.451	0.0	1200.9	2728
5760 min Winter	1.157	0.0	1276.8	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.300

Time (mins) From:	To:	Area (ha)
0	4	2.300

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	171.6	2.000	563.3

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0139-1130-2000-1130
Design Head (m)	2.000
Design Flow (l/s)	11.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	139
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	11.3
Flush-Flo™	0.593	11.3
Kick-Flo®	1.214	8.9
Mean Flow over Head Range	-	9.9

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.0	1.200	9.1	3.000	13.7	7.000	20.5
0.200	9.4	1.400	9.5	3.500	14.7	7.500	21.2
0.300	10.4	1.600	10.2	4.000	15.7	8.000	21.9
0.400	11.0	1.800	10.7	4.500	16.6	8.500	22.5
0.500	11.2	2.000	11.3	5.000	17.5	9.000	23.1
0.600	11.3	2.200	11.8	5.500	18.3	9.500	23.8
0.800	11.1	2.400	12.3	6.000	19.1		
1.000	10.5	2.600	12.8	6.500	19.8		

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	99.085	1.085	11.5	313.4	O K
30 min Summer	99.297	1.297	11.5	401.4	O K
60 min Summer	99.474	1.474	11.5	482.3	O K
120 min Summer	99.612	1.612	11.5	550.5	O K
180 min Summer	99.667	1.667	11.5	578.9	O K
240 min Summer	99.686	1.686	11.5	588.8	O K
360 min Summer	99.667	1.667	11.5	579.1	O K
480 min Summer	99.617	1.617	11.5	552.9	O K
600 min Summer	99.573	1.573	11.5	531.0	O K
720 min Summer	99.534	1.534	11.5	511.4	O K
960 min Summer	99.462	1.462	11.5	476.5	O K
1440 min Summer	99.324	1.324	11.5	413.2	O K
2160 min Summer	99.122	1.122	11.5	328.0	O K
2880 min Summer	98.928	0.928	11.5	254.5	O K
4320 min Summer	98.574	0.574	11.5	139.4	O K
5760 min Summer	98.295	0.295	11.5	64.7	O K
15 min Winter	99.084	1.084	11.5	313.1	O K
30 min Winter	99.299	1.299	11.5	402.2	O K
60 min Winter	99.476	1.476	11.5	483.4	O K
120 min Winter	99.616	1.616	11.5	552.7	O K
180 min Winter	99.673	1.673	11.5	582.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	104.360	0.0	325.2	19
30 min Summer	67.940	0.0	423.2	33
60 min Summer	42.060	0.0	525.3	64
120 min Summer	25.395	0.0	634.2	122
180 min Summer	18.777	0.0	703.6	182
240 min Summer	15.100	0.0	754.5	242
360 min Summer	11.032	0.0	826.8	360
480 min Summer	8.780	0.0	877.4	430
600 min Summer	7.336	0.0	916.4	484
720 min Summer	6.326	0.0	948.2	560
960 min Summer	4.996	0.0	998.4	674
1440 min Summer	3.573	0.0	1071.1	940
2160 min Summer	2.552	0.0	1148.3	1364
2880 min Summer	2.014	0.0	1208.0	1732
4320 min Summer	1.451	0.0	1305.1	2424
5760 min Summer	1.157	0.0	1387.7	3120
15 min Winter	104.360	0.0	324.9	18
30 min Winter	67.940	0.0	423.5	33
60 min Winter	42.060	0.0	525.3	62
120 min Winter	25.395	0.0	634.4	120
180 min Winter	18.777	0.0	703.6	178

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Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	99.695	1.695	11.5	593.8	O K
360 min Winter	99.685	1.685	11.5	588.4	O K
480 min Winter	99.637	1.637	11.5	563.6	O K
600 min Winter	99.575	1.575	11.5	531.9	O K
720 min Winter	99.531	1.531	11.5	509.7	O K
960 min Winter	99.437	1.437	11.5	464.6	O K
1440 min Winter	99.236	1.236	11.5	374.8	O K
2160 min Winter	98.921	0.921	11.5	251.9	O K
2880 min Winter	98.613	0.613	11.5	150.9	O K
4320 min Winter	98.099	0.099	11.5	20.3	O K
5760 min Winter	98.000	0.000	10.2	0.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	15.100	0.0	754.5	236
360 min Winter	11.032	0.0	826.8	348
480 min Winter	8.780	0.0	877.4	454
600 min Winter	7.336	0.0	916.4	500
720 min Winter	6.326	0.0	948.2	564
960 min Winter	4.996	0.0	998.4	714
1440 min Winter	3.573	0.0	1071.1	1010
2160 min Winter	2.552	0.0	1148.3	1408
2880 min Winter	2.014	0.0	1208.1	1788
4320 min Winter	1.451	0.0	1305.2	2380
5760 min Winter	1.157	0.0	1387.9	0

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Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 360158 378110 SJ 60158 78110
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.500
Cv (Winter)	0.500
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+0

Time Area Diagram

Total Area (ha) 2.500

Time (mins) From:	To:	Area (ha)
0	4	2.500

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Model Details

Storage is Offline Dividing Weir Level (m) 100.000
Cover Level (m) 100.000

Tank or Pond Structure

Invert Level (m) 98.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	196.8	2.000	608.2

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0140-1150-2000-1150
Design Head (m)	2.000
Design Flow (l/s)	11.5
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	140
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	11.5
Flush-Flo™	0.581	11.5
Kick-Flo®	1.202	9.0
Mean Flow over Head Range	-	10.1

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.1	1.200	9.1	3.000	13.9	7.000	20.9
0.200	9.6	1.400	9.7	3.500	15.0	7.500	21.6
0.300	10.7	1.600	10.3	4.000	16.0	8.000	22.3
0.400	11.2	1.800	10.9	4.500	16.9	8.500	22.9
0.500	11.4	2.000	11.5	5.000	17.8	9.000	23.5
0.600	11.5	2.200	12.0	5.500	18.6	9.500	24.2
0.800	11.2	2.400	12.5	6.000	19.4		
1.000	10.6	2.600	13.0	6.500	20.1		

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