A1 in Northumberland: Morpeth to Ellingham

Scheme Number: TR010041

6.8 Environmental Statement – Appendix 10.1 Flood Risk Assessment

Part B

APFP Regulation 5(2)(a)

Planning Act 2008

Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009

June 2020

Infrastructure Planning

Planning Act 2008

The Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009

The A1 in Northumberland: Morpeth to Ellingham
Development Consent Order 20[xx]

Environmental Statement - Appendix

Regulation Reference:	APFP Regulation 5(2)(a)
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EXECUTIVE SUMMARY

The Applicant has undertaken a Flood Risk Assessment (FRA) to support the Environmental Statement (ES) and Development Consent Order (DCO) application for the A1 in Northumberland: Alnwick to Ellingham (Part B). Part B would include approximately 8 km of online widening to the east of the existing carriageway.

A review of the Environment Agency Flood Map for Planning (Rivers and Sea) indicates that the alignment in the Part B Main Scheme Area is located in the low-risk Flood Zone 1. However, within the Order Limits of Part B there are two areas located within the medium risk Flood Zone 2, and the high-risk Flood Zone 3. There is one area located to the south within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Denwick Burn. The other area is located to the north within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Shipperton Burn.

The Part B alignment crosses the following watercourses and associated tributaries (listed from south to north): Denwick Burn and its tributaries; White House Burn; two tributaries of Kittycarter Burn; tributary of Embleton Burn; and Shipperton Burn.

The development of the proposals for each watercourse (and tributaries) crossed by Part B has been dictated by the baseline flood risk situation and whether the design is an extension of an existing culvert or the replacement of an existing culvert.

Detailed 1D hydraulic modelling has been undertaken for Denwick Burn and its tributaries, White House Burn, two tributaries of Kittycarter Burn and Shipperton Burn. A hydraulic assessment using Culvert Master has been undertaken for the other watercourses. The modelling shows that there would be no increase in fluvial flood risk to any upstream or downstream receptors.

A review of the Environment Agency's Flood Risk from Surface Water map indicates that sections of Part B are at high, medium and low risk of flooding from surface water sources. Flooding from surface water is typically associated with natural overland flow paths (including the watercourses discussed above) and local depressions in topography where surface water runoff can accumulate during or following heavy rainfall events. Known surface water flow paths have been incorporated into Part B.

The proposed drainage strategy restricts surface water runoff rates to the existing greenfield runoff values for the equivalent storm event, as follows:

a. Highway drainage would be designed to accommodate a 1 in 1 year design flow without surcharging and a 1 in 5 year design flow without surface flooding of the running carriageways (with a 20 % allowance for climate change).
b. Attenuation controls would be provided for the 1 in 1, 30 and 100 year plus 20 % allowance for climate change.

1. INTRODUCTION

1.1. SCHEME OVERVIEW

1.1.1. The Applicant has undertaken an FRA to support this ES and DCO application for the A1 in Northumberland: Alnwick to Ellingham (Part B). The assessment has been conducted in accordance with the National Planning Policy Framework (NPPF) (**Ref. 10.1**) and Planning Practice Guidance (PPG) (**Ref. 10.2**), the National Policy Statement for National Networks (NPS NN) (**Ref. 10.3**), the Design Manual for Roads and Bridges (DMRB) Volume 11, Section 3, Part 10 (HD 45/09) (**Ref. 10.4**), local planning policy, as well as other relevant standards as agreed through consultation with the Environment Agency and Northumberland County Council (NCC).

1.1.2. A review of the Environment Agency's Flood Map for Planning (Rivers and Sea) (**Ref. 10.5**) indicates that the alignment in the Part B Main Scheme Area is located in the low-risk Flood Zone 1. However, within the Order Limits of Part B there are two areas located within the medium risk Flood Zone 2, and the high-risk Flood Zone 3. There is one area located to the south within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Denwick Burn. The other area is located to the north within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Shipperton Burn.

1.1.3. The Environment Agency's standing advice on flood risk (**Ref. 10.6**) states that an FRA would be required to support the DCO application for Part B and our assessment includes the following:

a. Confirmation of the sources of flooding which may affect Part B.
b. A quantitative assessment of the risk of flooding to Part B and to adjacent sites as a result of Part B.
c. Identification of possible measures which could reduce flood risk to acceptable levels and a summary of residual risks.
d. A summary of the proposed surface water drainage strategy.

1.2. LOCATION

1.2.1. Part B is located within the County of Northumberland and forms part of the strategic road network (SRN). Part B is located along the A1 between Alnwick and Ellingham and is approximately 8 km in length. Further details of the location of the Scheme can be found on the **Location Plan** (**Application Document Reference: TR010041/APP/2.1**).

1.2.2. Part B comprises online improvements consisting of carriageway widening to the east of the existing alignment and a more detailed description of Part B is found in **Chapter 2: The Scheme, Volume 1** of this ES (**Application Document Reference: TR010041/APP/6.1**). The **General Arrangement Plans** (**Application Document Reference: TR010041/APP/2.4**) show Part B layout.

1.2.3. Part B also includes three construction compounds. The Charlton Mires Site Compound is located in the Part B Main Scheme Area to the east of the existing A1, in a field to the south of Charlton Mires; the Main Compound is located to the west of Thirston New Houses and is to be shared with the A1 in Northumberland: Morpeth to Felton (Part A); and the Lionheart Enterprise Park Compound is located in the Lionheart Enterprise Park just to the south of Alnwick The **Location Plan** (**Application Document Number: TR010041/APP/APP/62.1**) shows the location of the compounds.

1.3. CONSULTATION

1.3.1. Consultation has been undertaken with the following authorities:

a. Meeting held with the Environment Agency and NCC in November 2018 to discuss stakeholder requirements and review the available flood information and agree (in principle) the methodology, appropriate mitigation and management options during the construction and operation stages.
b. Two teleconferences held with NCC in May 2019 to discuss the results of the hydraulic modelling undertaken for the tributaries of the Kittycarter Burn and to review the Part B proposals and proposed mitigation.

1.3.2. The meeting minutes have been included in **Appendix 4.2: Environmental Consultation**, **Volume 1** of this ES (**Application Document Reference: TR010041/APP/6.1**).

2. ASSESSMENT METHODOLOGY

2.1. OVERVIEW

2.1.1. In brief the methodology used for this FRA comprises:

a. Site walkover completed for Part B (excluding compounds) on 13 and 14 February 2019.
b. Review of available relevant flood risk information to identify existing risks from all sources. The information reviewed includes: Environment Agency's online maps for flood risk (Flood Map for Planning (**Ref. 10.5**), Long Term Flood Risk Map (**Ref. 10.7**) and groundwater (Environment Agency groundwater data is hosted on The Multi-Agency Geographic Information for the Countryside (MAGIC) online map (**Ref. 10.8**)) (accessed January 2019), information provided by NCC on historical flooding during consultation.
c. Obtained LiDAR and topographic survey data.
d. Review of the Ground Investigation Report undertaken (dated April 2019) (**Appendix 11.3: Ground Investigation Report** of this ES).
e. Consultation with the Environment Agency and NCC to confirm potential flood risk to Part B and agree principles for the mitigation of potential flood risk to Part B and third-party land arising from Part B (refer to **Appendix 4.2: Environmental Consultation**, **Volume 1** of this ES (**Application Document Reference: TR010041/APP/6.1**).
f. A detailed assessment of how Part B may affect fluvial flood risk, informed by the development of five 1D Flood Modeller hydraulic models and three Culvert Master models.
g. Development of mitigation measures, as necessary, to reduce flood risk to Part B and third-party land to an acceptable level as informed by the 1D hydraulic models and Culvert Master models.
h. A summary of the strategy for the management of Part B generated surface water runoff.

2.2. DEFINITION OF FLOOD RISK

2.2.1. Flood risk is the product of the likelihood or chance of a flood occurring (flood frequency) and the consequence or impact of the flooding (flood consequence).

FLOOD FREQUENCY

2.2.2. Flood frequency is identified in terms of the return period and annual probability. For example, a 1 in 100 year flood event has a 1 % annual probability of occurring. **Table 2-1** provides a conversion between return periods and annual flood probabilities.

Table 2-1 Flood Probability Conversion Table

Return Period (Years)	2	5	10	20	50	100	200	1000
Annual Probability %	50	20	10	5	2	1	0.5	0.1

2.2.3. The Flood Risk and Coastal Change PPG (Paragraph 065 reference ID: 7-065-20140306) (**Ref. 10.2**) identifies Flood Zones in relation to flood frequency. The zones refer to the probability of river (fluvial) and sea (tidal) flooding, whilst ignoring the presence of defences. **Table 2-2** summarises the relationship between Flood Zone category and the identified flood probability (as defined in the PPG (**Ref. 10.2**)).

Table 2-2 Flood Zones

Flood Risk Area	Identification	Annual Probability of Fluvial Flooding	Annual Probability of Tidal Flooding
Zone 1	Low probability	< 0.1 %	< 0.1 %
Zone 2	Medium probability	1 % - 0.1 %	0.5 % - 0.1 %
Zone 3a	High probability	> 1 %	> 0.5 %
Zone 3b*	Functional Floodplain	> 5 %	> 5 %

*The definition of the functional floodplain should take account of local circumstances. The annual flood probability is stated as a starting point for consideration.

FLOOD CONSEQUENCES

2.2.4. The consequence of a flood event describes the potential damage, danger and disruption caused by flooding. This is dependent on the mechanism and characteristics of the flood event and the vulnerability of the affected land and land use.

2.2.5. The NPPF (**Ref. 10.1**) identifies five classifications of flood risk vulnerability and provides recommendations on the compatibility of each vulnerability classification with the Flood Zones, as shown in **Table 2-3**. Full details of the Flood Zones and flood risk vulnerability classifications can be found in the PPG (**Ref. 10.2**) to the NPPF (**Ref. 10.1**).

2.2.6. The Sequential Test as defined in NPPF (**Ref. 10.1**) ensures that a sequential approach is followed to steer new development to areas with the lowest probability of flooding.

2.2.7. The Exception Test is a method to demonstrate and help ensure that flood risk to people and property would be managed satisfactorily, while allowing necessary development to go ahead in situations where suitable sites at lower risk of flooding are not available. Essentially, the two parts to the Test require proposed development to show that it would provide wider sustainability benefits to the community that outweigh flood risk, and that it would be safe for its lifetime, without increasing flood risk elsewhere and where possible reduce flood risk overall.

Table 2-3 Flood Risk Vulnerability and Flood Zone Compatibility

Environment Agency Flood Zone	Essential Infrastructure	Water Compatible	Highly Vulnerable	More Vulnerable	Less Vulnerable
Zone 1	ü	ü	ü	ü	ü
Zone 2	ü	ü	Exception test required	ü	ü
Zone 3a	Exception test required	ü	û	Exception test required	ü
Zone 3b	Exception test required	ü	û	û	û
ü Development considered acceptable û Development considered unacceptable					

2.2.8. Part B is classed as 'Essential Infrastructure' under the NPPF (**Ref. 10.1**). Essential Infrastructure within Flood Zone 1 is acceptable in policy terms.

2.3. POTENTIAL SOURCES OF FLOODING

2.3.1. In accordance with NPPF (**Ref. 10.1**), the following sources of flooding have been considered in this assessment:

a. Fluvial flood risk from nearby watercourses
b. Surface water flooding from within the Order Limits of Part B and adjacent land
c. Tidal flood risk
d. Surcharging of sewers and other infrastructure
e. Groundwater flooding
f. Flood risk from other artificial sources such as canals and impounded reservoirs

2.4. POTENTIAL EFFECTS OF CLIMATE CHANGE

2.4.1. Scientific consensus is that the global climate is changing as a result of human activity. Whilst there remain uncertainties as to how changing climate affect areas already vulnerable to flooding, it is expected to increase risk significantly over time. For the UK, projections of future climate change indicate that more frequent short-duration high-intensity rainfall events and more frequent periods of long-duration rainfall could be expected.

2.4.2. Updated climate change recommendations (**Ref. 10.9**) were published by the Environment Agency in February 2016 (and updated in February 2017 and February 2019), which supersedes the previous recommendations that were included within the NPPF PPG (**Ref. 10.2**). The impacts of climate change are expected to increase over time and the Environment Agency guidance (**Ref. 10.9**) provides a range of estimates for increases in peak river flow, peak rainfall intensity and sea level rise over the next 100 years. This is reflected by larger allowances recommended for developments with a longer design life.

2.4.3. The precise extent of the impacts of climate change is unknown. This is reflected in the Environment Agency's guidance (**Ref. 10.9**) which provides 'Central', 'Higher Central' and 'Upper End' estimates that are based on the 50th, 70th and 90th percentile predictions for climate change.

2.4.4. The increases in peak fluvial flows are also expected to vary depending on geographical location. To account for this the Environment Agency climate change guidance (**Ref. 10.9**) divides England into eleven river basin districts. Part B is located within the Northumbria River Basin District. **Table 2-4** shows the Environment Agency's recommended climate change increase for peak river flow in this district.

Table 2-4 Recommended Peak River Flow Allowances for the Northumbria River Basin District

	Allowance Category	Total Potential Change Anticipated 2015 - 2039	Total Potential Change Anticipated 2040 - 2069	Total Potential Change Anticipated 2070 - 2115
Peak river flow allowances for Northumbria	Upper End	20 %	30 %	50 %
	Higher Central	15 %	20 %	25 %
	Central	10 %	15 %	20 %

2.4.5. **Table 2-5** summarises the Environment Agency's climate change guidance (**Ref. 10.9**) for increases to peak rainfall intensity throughout England. This information is typically applied to the assessment of surface water runoff but can also be applied to small watercourses that have a catchment of less than approximately 3 km^2 which respond much more quickly to intense rainfall events.

Table 2-5 Peak Rainfall Intensity Allowance in Small and Urban Catchments

	Allowance Category	Total Potential Change Anticipated 2017 - 2039	Total Potential Change Anticipated 2040 - 2069	Total Potential Change Anticipated 2070 - 2115
Peak rainfall intensity allowance	Upper End	10 %	20 %	40 %
	Central	5 %	10 %	20 %

2.4.6. **Table 2-6** summarises the Environment Agency's climate change guidance (**Ref. 10.9**) for recommended contingency allowances for net sea level rise noting that higher sea levels can also influence flood risk associated with tidally influenced watercourses.

Table 2-6 Recommended Sea Level Allowances for each Epoch in Millimetres (mm) Per Year with Cumulative Sea Level Rise for each Epoch in Brackets (use 1990 Baseline)

Location	1990 - 2025	2026 - 2055	2056 - 2085	2086 - 2115	Cumulative Rise 1990 – 2115 (m)
North East	3.5 mm (122.5 mm)	8 mm (240 mm)	11.5 mm (345 mm)	14.5 mm (435 mm)	1.14 m

2.4.7. The design life of Part B is taken as 100 years. In accordance with the guidance above and following discussions with the Environment Agency, the allowances to be used in the assessment and design of Part B and taking into account Part B's vulnerability are as follows:

a. 25 % increase in peak river flow for the assessment of risk to Part B, assessment of risk to third parties, design of the watercourse crossings and design of other required mitigation if required.
b. 50 % increase in peak river flow or the 1,000 year peak flow (whichever is greatest) for the residual risk assessments to understand risks to Part B and third parties in the event of a more extreme event or uncertainty in climate change predictions.

2.5. HYDRAULIC ASSESSMENT

HYDRAULIC DESIGN OF WATERCOURSE CROSSINGS

2.5.1. Hydraulic assessment has been undertaken for each watercourse crossed by Part B to confirm baseline flood risk situation, both upstream and downstream, and inform the proposed solution such as extension of an existing culvert or the replacement of an existing culvert. For example, in most cases the existing structures along the A1 constrain downstream flows. The proposals aim to maintain the downstream flood risk and the simplest way to do this is to retain the existing structure.

2.5.1. **Table 2-7** sets out the range of existing site conditions observed along the length of Part B and as such the preferred design solution reflecting these conditions. The site conditions and resulting approach are applicable for all watercourses assessed.

Table 2-7 Summary of Hydraulic Analysis Approach

Proposed Solution	Hydraulic Analysis Approach
Extension of existing culvert	In these instances, the hydraulic assessment would consider an increase in length of the existing structure typically maintaining the same structure dimensions. The impact of the proposed extension on flood risk would then be assessed: - Assess performance of existing culvert and local structures using 2-year, 10-year, 100 year and 1,000-year flood events (baseline). - Assess implications of climate change with the 100 year + 25 % climate change events as set out in **Section** 2.4 above. - Increase culvert length as required. - Assess performance of proposed culvert to ensure the pass forward flow for the 100 year + 25 % climate change event remains unchanged and elevated upstream water levels do not impact flood risk receptors. - Assess the residual flood risk with the 100 year + 50 % climate change event, as set out in Section 2.4 above, or 1,000 year event, whichever is higher and a scenario representing partial blockage of the structure.
Replacement of existing culvert	The size of replacement culverts would be informed by hydraulic analysis of the culvert to meet DMRB (**Ref. 10.10**) requirements wherever possible whilst preventing an increase in downstream flows resulting from the removal of the downstream structure. Consideration would also be given to

Proposed Solution	Hydraulic Analysis Approach
	improved fish and mammal passage where engineering and flood risk constraints allow: Assess performance of existing culvert and local structures using 2-year, 10-year, 100 year and 1,000-year flood events (baseline). Assess implications of climate change with the 100 year + 25 % climate change events as set out in Section 2.4 above. Design proposed replacement culvert with consideration of mammal passage, fish passage, flood risk impacts and design constraints as described below.

2.5.4. Once the initial hydraulic analysis was complete, the geometry of the structure was assessed for the following:

a. Physical constraints – including the depth of cover to the carriageway and local utility service locations.
b. Mammal passage – the incorporation of a route that remains accessible in flood conditions.
c. Fish passage – low flow channels, baffles or a natural bed.
d. Access requirements – culverts greater than 12 m should be 1.2 m diameter (subject to flood risk and physical constraints).

2.5.5. Consideration of the above required an iterative process in conjunction with various disciplines and the results are presented in the following sections.

FLOOD RISK FROM LARGER WATERCOURSES

2.5.6. A detailed assessment of fluvial flood risk has been completed for the largest watercourses and their tributaries crossed by Part B using five hydraulic models to provide an improved understanding of the fluvial flood risk in the vicinity of Part B and a basis for assessing the impact of Part B on third parties.

2.5.7. For this assessment, five 1D Flood Modeller Pro hydraulic models were created based on the topographic survey undertaken in November 2018 for Denwick Burn and its tributaries (two models), White House Burn, tributaries of Kittycarter Burn and Shipperton Burn.

2.5.8. Detailed technical information relating to the hydraulic modelling assessment is provided in **Appendix A: Hydraulic Modelling Analysis** of this FRA.

FLOOD RISK FROM OTHER WATERCOURSES, DRAINAGE DITCHES AND SURFACE WATER FLOW PATHS

2.5.9. For the other watercourses, drainage ditches and identified surface water flow paths crossed by Part B a simpler approach has been undertaken which reflects the lower risk associated with these structures.

2.5.10. Hydraulic analysis used Bentley Culvert Master software. The software is based upon U.S. Department of Transportation, Hydraulic Design Series Number 5 – Hydraulic Design of Highway Culverts, Third Edition FHWA-HIF-12-026 (**Ref. 10.11**) and enables the assessment of culverts for both pipe and open channel flow scenarios.

2.5.11. **Section 5** of this FRA sets out the methodology for the assessment of these other watercourses, drainage ditches and surface water flow paths and discusses any resulting changes. Detailed technical information is provided in **Appendix B: Culvert Master Analysis** of this FRA.

2.6. LEGISLATIVE FRAMEWORK AND GUIDANCE

2.6.1. The coordination of policies for the water environment is managed by the UK Government. Many flood risk and water quality requirements are set at European level, which are then transposed into UK law. The Environment Agency has a strategic overview regarding the management of all of sources of flooding and an operational responsibility for managing the risk of flooding from main rivers, reservoirs, estuaries and tidal sources. Lead Local Flood Authorities (LLFAs), in this case, NCC are responsible for managing the risk of flooding from local sources, including surface water, groundwater and ordinary watercourses.

2.6.2. The applicable legislative framework is summarised below.

EUROPEAN LEGISLATION

Water Framework Directive (2000/60/EC)

2.6.3. The overall objective of the Water Framework Directive (WFD) (**Ref. 10.12**) is to bring about the effective co-ordination of water environment policy and regulation across Europe. The main aims of the legislation are to ensure that all surface water and groundwater reaches ‘good’ status (in terms of ecological and chemical quality and water quantity, as appropriate), promote sustainable water use, reduce pollution and contribute to the mitigation of flood and droughts.

2.6.4. The WFD (**Ref. 10.12**) also contains provisions for controlling discharges of dangerous substances to surface waters and groundwater and includes a ‘List of Priority Substances’. Various substances are listed as either List I or List II substances, with List I substances considered the most harmful to human health and the aquatic environment. The purpose of the directive is to eliminate pollution from List I substances and reduce pollution from List II substances.

Groundwater Directive (2006/118/EC)

2.6.5. This Groundwater Directive (**Ref. 10.13**) aims to set groundwater quality standards and introduce measures to prevent or limit pollution of groundwater, including those listed with the 'List of Priority Substances'. The directive has been developed in response to the requirements of Article 17 of the WFD (**Ref. 10.12**), specifically the assessment of chemical status of groundwater and objectives to achieve 'good' status.

Floods Directive (2007/60/EC)

2.6.6. The key objective of the Floods Directive (**Ref. 10.14**) is to coordinate the assessment and management of flood risks within Member States. Specifically, it requires Member States to assess if all watercourses and coastlines are at risk of flooding, map the flood extent, flood assets and humans at risk in these areas, and take adequate and coordinated measures to reduce this risk.

NATIONAL LEGISLATION

Land Drainage Act 1991

2.6.7. Local Authorities and Internal Drainage Boards have additional duties and powers associated with the management of flood risk under the Land Drainage Act 1991 (**Ref. 10.15**). As Land Drainage Authorities, consent must be given for any permanent or temporary works that could affect the flow within an ordinary watercourse under their jurisdiction, in order to ensure that local flood risk is not increased.

2.6.8. The Land Drainage Act (**Ref. 10.15**) specifies that the following works would require formal consent from the appropriate authority:

a. Construction, raising or alteration of any mill dam, weir or other like obstructions to the flow of a watercourse.
b. Construction of a new culvert.
c. Any alterations to an existing culvert that would affect the flow of water within a watercourse.

2.6.9. The Land Drainage Act (**Ref. 10.15**) also sets out the maintenance responsibilities riparian owners have in order to reduce local flood risks. Riparian owners, who are land owners with a watercourse either running through their land or adjacent to, have the responsibility to ensure that the free flow of water is not impeded by any obstruction or build-up of material within the watercourse.

Flood and Water Management Act 2010

2.6.10. The Flood and Water Management Act 2010 (**Ref. 10.16**) extended the role of the LLFA (NCC) set out in the Flood Risk Regulations (2009) (**Ref. 10.17**) to take responsibility for leading the co-ordination of local flood risk management in their areas. In accordance with the Act the Environment Agency is responsible for the management of risks associated with main rivers, the sea and reservoirs. LLFAs are responsible for the management of risks associated with local sources of flooding such as ordinary watercourses, surface water and groundwater.

2.6.11. The Act is also guiding the role of the LLFA in the review and approval of surface water management systems. This has led to a recent change that requires the LLFA to review and comment on significant development in regard to Sustainable Drainage Systems (SUDS).

2.6.12. Schedule 3 of the Flood and Water Management Act (**Ref. 10.16**) introduces National Standards for SUDS against which proposed drainage systems should comply. These are discussed below.

Environmental Permitting (England and Wales) Regulations 2010

2.6.13. The Environmental Permitting (England and Wales) Regulations 2010 (**Ref. 10.18**) replaced the Water Resources Act 1991 (**Ref. 10.19**) as the key legislation for water pollution in the UK. Under the Environmental Permitting Regulations (**Ref. 10.18**), it is an offence to cause or knowingly permit a water discharge activity, including the discharge of polluting materials to freshwater, coastal waters, relevant territorial waters or groundwater, unless complying with an exemption or an environmental permit. An environmental permit is obtained from the Environment Agency.

2.6.14. With regards to the water environment any works in, under or near a main river requires permission from the Environment Agency to ensure no detrimental impacts on the watercourse. Previously, this was a Flood Defence Consent; however, in April 2016 consent for flood risk activities was included under the Environmental Permitting Regulations (**Ref. 10.18**).

NATIONAL POLICY

National Policy Statement for National Networks 2014

2.6.15. The NPS NN (**Ref. 10.3**) set out the policies for nationally significant infrastructure road projects in England. Flood risk is covered as a specific generic impact in paragraphs 5.90 to 5.115, which outline the following:

a. Part B should be supported by an FRA in accordance with NPPF.
b. Surface water discharge should be 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.
c. Opportunities can be taken to lower flood risk by improving flow routes, flood storage capacity and using SUDS.

National Planning Policy Framework 2019

2.6.16. NPPF (**Ref. 10.1**) sets out the Government's planning policies for England, providing a framework within which local councils can produce their own plans that better reflect the specific needs of their communities. PPG (**Ref. 10.2**) has been published alongside the NPPF (**Ref. 10.1**) in 2014 to set out how certain policies, including those relating to flood risk, should be implemented.

2.6.17. The NPPF (**Ref. 10.1**) and relevant PPG (**Ref. 10.2**) identify how new developments must take flood risks into account, including making an allowance for climate change impacts, and steer development to those areas at lowest risk.

2.6.18. The PPG (**Ref. 10.2**) sets out the requirement to consider Sustainable Drainage Systems (SUDS) within all new development where appropriate. It states that developments should aim to discharge surface run off as high up the following hierarchy of drainage options as reasonably practicable:

a. Into the ground (infiltration)
b. To a surface water body
c. To a surface water sewer, highway drain, or another drainage system
d. To a combined sewer

Non-Statutory Technical Standards for Sustainable Drainage Systems 2015

2.6.19. The Non-Statutory Technical Standard (**Ref. 10.20**) for SUDS, published by DEFRA in March 2015, set out the core technical standards for SUDS proposed within England. These standards should be used in accordance with the NPPF (**Ref. 10.1**) and PPG (**Ref. 10.2**). The standards include guidance on controlling flood risk within a development boundary and elsewhere, peak flow and runoff volume control, and the structural integrity of SUDS.

LOCAL POLICY

2.6.20. NCC is currently in the process of updating its Local Plan, the consolidated planning policy framework, which details the saved polices that are currently used in the determination of planning applications. Part B is located within the former district area of Alnwick. The relevant saved policies are detailed below.

Alnwick District Wide Local Plan 1997

2.6.21. There is one saved policy from the Alnwick District Wide Local Plan (**Ref. 10.21**) that is applicable to this FRA for Part B. Policy CD33 sets out to ensure that new development is not located in areas of known flood risk and would not increase local flood risk elsewhere as a result of the development.

Northumberland Draft Local Plan 2019

2.6.22. The Northumberland Draft Local Plan (**Ref. 10.22**) provides guidance for new development within the Council’s administrative area. It is currently intended that the plan would be adopted in March 2020. In order to achieve the vision, set out in the plan, a number of policies have been proposed. The following policies are considered relevant to the assessment of flood risk for Part B:

2.6.23. Policy WAT 3 (Flooding) sets out to ensure that development proposals minimise local flood risk to people, property and infrastructure from all sources of flooding through the following principles:

a. Locating development in areas not at risk of flooding, taking into account future climate change, and if applicable, using a sequential approach to locating development to areas at lowest risk of flooding.
b. Development proposals should be made resistant and resilient through appropriate mitigation measures.
c. Built development proposals should minimise and control surface water runoff using SUDS. The hierarchy for surface water should be the following:

   i. To a soakaway system, unless it can be demonstrated that this is not feasible due to poor infiltration due to the underlying ground conditions.
   ii. To a watercourse, unless there is no alternative or suitable receiving watercourse available.
   iii. To a surface water sewer; as a last resort once all other methods have been explored.

2.6.24. Policy WAT 4 (Sustainable Drainage Systems) sets out to ensure that SUDS are considered to minimise and control surface water runoff. The policy also sets out a requirement for the management and maintenance of SUDS to be taken into consideration for the lifetime of the development.

Northumberland Local Flood Risk Management Strategy 2015

2.6.25. Northumberland's Local Flood Risk Management Strategy (LFRMS) (**Ref. 10.23**) provides information and technical guidance on how flood risk would be managed within Northumberland. The LFRMS (**Ref. 10.23**) sets out five local objectives and details a number of measures and an action plan that would be implemented to achieve the objectives. Objective Two is considered relevant to the assessment of flood risk for Part B. The five local objectives are:

a. Improve knowledge and understanding of flood risk throughout Northumberland.
b. Promote sustainable development to reduce local flood risk with consideration to the anticipated impact of climate change.
c. Actively manage flood risk and drainage infrastructure to reduce likelihood of flooding throughout Northumberland.
d. Encourage communities to become more resilient to flooding by increasing public awareness and understanding their concerns.
e. Be better prepared for flood events and post flood recovery.

3. SITE DESCRIPTION

3.1. SITE DESCRIPTION

3.1.1. This section provides a description of the current baseline conditions with respect to the water environment and has been divided into the Part B Main Scheme Area including the Charlton Mires Site Compound, the Main Compound and the Lionheart Enterprise Park Compound.

3.2. THE PART B MAIN SCHEME AREA INCLUDING THE CHARLTON MIRES SITE COMPOUND

3.2.1. Land surrounding the Part B Main Scheme Area and the Charlton Mires Site Compound generally consists of woodland and agricultural land.

3.2.2. The most notable urban areas are the town of Alnwick to the south-west, the village of Denwick to the south and the village of North Charlton to the north.

EXISTING SURFACE WATER FEATURES

3.2.3. The Part B alignment crosses five watercourses and associated tributaries which are listed below from south to north:

a. Denwick Burn and its tributaries
b. White House Burn
c. Tributaries of Kittycarter Burn
d. Tributary of Embleton Burn
e. Shipperton Burn

DENWICK BURN

3.2.4. Denwick Burn and its tributaries flow in a north to south-east direction underneath the existing A1 alignment at four locations to the north of the village of Denwick. The watercourse is classified as an ordinary watercourse under the jurisdiction of NCC as LLFA.

3.2.5. The source of Denwick Burn is just to the west of the existing A1 alignment to the south of Heckley Fence. The catchment of the watercourse is gently sloping towards the watercourse from both the east and west. It has an approximate upstream catchment area of 3.8 km^2 and consists primarily of agricultural land with no flood risk receptors upstream of the A1 crossings of the watercourse or its tributaries.

3.2.6. Denwick Burn and its tributaries flow through a number of crossings underneath the A1, farm access tracks and a Public Right of Way (PRoW), as labelled in **Figure 3-1** below.

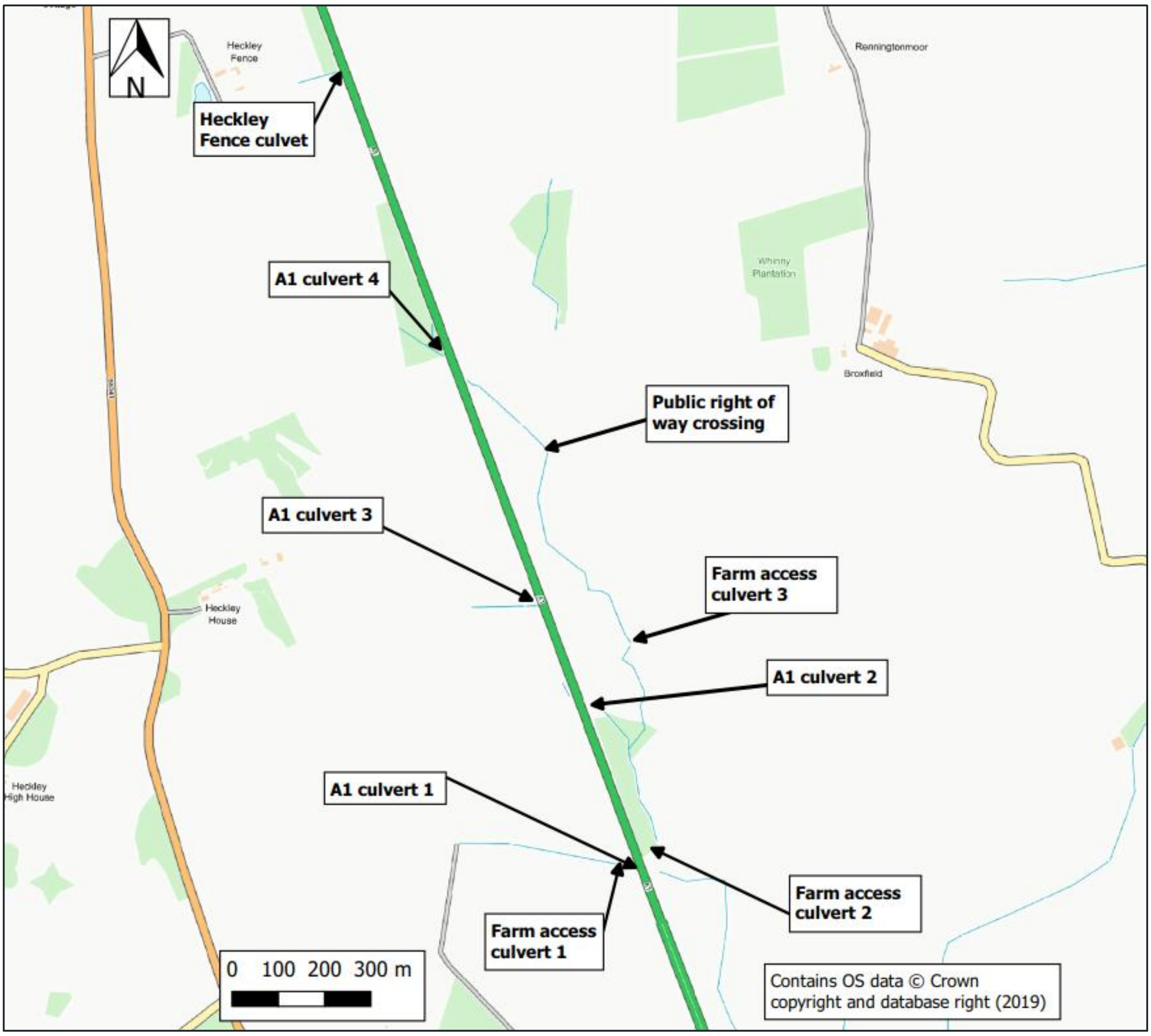

Figure 3-1 Denwick Burn and Tributaries Existing Structures

3.2.7. A detailed description of these crossings (from north to south) is provided below.

3.2.8. A small field ditch at Heckley Fence flows adjacent to the A1 and flows into a 36 m long circular culvert with a diameter of 300 mm as shown in **Figure 3-2** below. The culvert then discharges into another culvert which runs parallel to the A1 for approximately 580 m to the south and discharges into the Denwick Burn.

3.2.9. During the site walkover on 12 to 14 February 2019, a small inlet on the eastern side of the A1 was observed in line with the Heckley Fence culvert. It is assumed that this collects surface water runoff from fields to the east of the A1 and connects into the Heckley Fence culvert as no separate ditch or watercourse was observed during the walkover.

Figure 3-2 Heckley Fence Culvert Inlet

3.2.10. Denwick Burn flows underneath the A1 through culvert 4 as labelled in **Figure 3-1**. **Figures 3-3** and **3-4** show the inlet and outlet of the structure. The circular culvert has a diameter of approximately 1.2 m and is approximately 72 m in length.

Figure 3-3 Denwick Burn A1 Culvert 4 Inlet

Figure 3-4 Denwick Burn A1 Culvert 4 Outlet

3.2.11. Approximately 230 m downstream of culvert 4, Denwick Burn flows beneath a PRoW through a bridge as shown in **Figure 3-5** below. The watercourse crossing is approximately 750 mm in width and height. During the site walkover it was noted that downstream of the crossing the channel banks were concrete walls for approximately 20 m.

Figure 3-5 Public Right of Way Denwick Burn Crossing

3.2.12. Approximately 500 m downstream of the PRoW bridge Denwick Burn flows beneath a farm access track, labelled as farm access culvert 3 in **Figure 3-1**. **Figure 3-6** below shows the concrete inlet of the concrete circular pipe which is approximately 600 mm in diameter and 10 m in length.

Figure 3-6 Farm Access Denwick Burn Crossing Three

3.2.13. Another tributary of Denwick Burn flows beneath the A1 through a concrete circular pipe labelled as culvert three in **Figure 3-1**. The culvert has an approximate diameter of 600 mm and is approximately 21 m in length and is shown in **Figure 3-7** below. At the outlet of the culvert there is approximately 2 m of open channel before the watercourse enters another culvert. It is assumed that the watercourse discharges into Denwick Burn to the south-east of the A1, however during the site walkover the outlet of the downstream culvert was not identified.

Figure 3-7 Denwick Burn A1 culvert 3 Inlet

3.2.14. A tributary of Denwick Burn flows beneath the A1 labelled as A1 culvert two in **Figure 3-1**. The culvert, shown in **Figure 3-8** below, is circular with a diameter of approximately 300 mm and is approximately 86 m in length. The tributary discharges into Denwick Burn approximately 100 m downstream from the watercourse crossing.

Figure 3-8 Denwick Burn A1 Culvert 2 Inlet

3.2.15. Denwick Burn flows beneath a farm access track, labelled farm access culvert two in **Figure 3-1** and as shown in **Figures 3-9** and **3-10** below. **Figure 3-9** shows the inlet of the culvert which is located underneath a footbridge and **Figure 3-10** shows the outlet of the culvert. The circular concrete culvert is approximately 65 m on length and has a diameter of approximately 300 mm.

Figure 3-9 Farm Access Culvert 2 Inlet

Figure 3-10 Farm Access Culvert 2 Cutlet

3.2.16. The most southern tributary of Denwick Burn within the Part B Main Scheme Area flows beneath the A1 through a circular culvert labelled as culvert one in **Figure 3-1**. The inlet of the culvert is shown in **Figure 3-11** below. The culvert has a diameter of approximately 500 mm and is approximately 50 m in length. Immediately upstream of the A1 culvert the tributary flows beneath a farm access track as shown in **Figure 3-12** below. The crossing consists of twin 150 mm pipes and is approximately 20 m in length. The outlet of culvert one discharges into the Denwick Burn at the same location as the farm access culvert two.

Figure 3-11 Denwick Burn A1 Culvert 1 Inlet

Figure 3-12 Farm Access Culvert 1 Inlet

3.2.17. Denwick Burn discharges into the River Aln approximately 4.4 km downstream from Part B.

WHITE HOUSE BURN

3.2.18. White House Burn flows in an east to south-west direction beneath the existing A1 alignment to the west of Rock South Farm as shown in **Figure 3-13** below. White House Burn is classified as an ordinary watercourse under the jurisdiction of NCC as LLFA. The source of White House Burn is located approximately 1.3 km upstream of the A1 crossing within the Wisplaw Whin plantation. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 1.22 km^2.

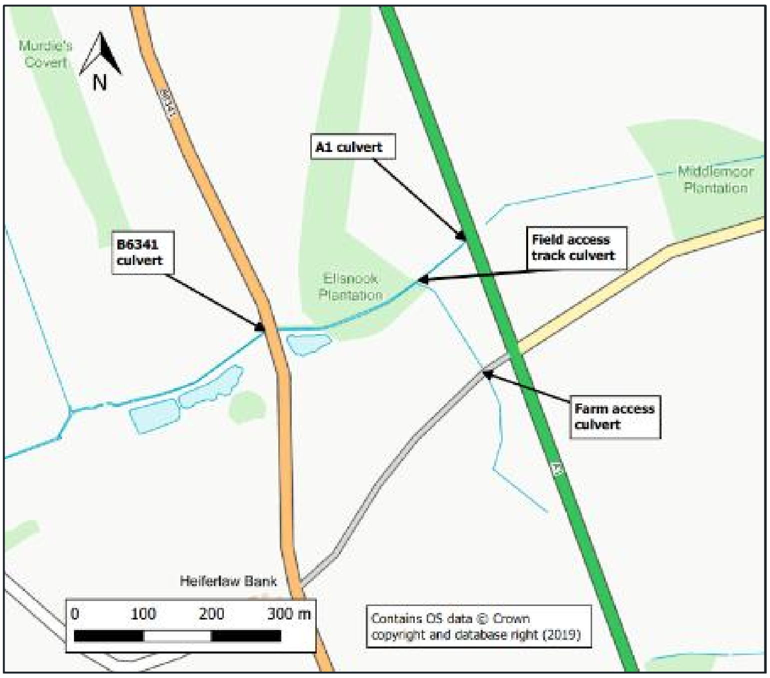

Figure 3-13 White House Burn Existing Structure

3.2.19. **Figure 3-14** below shows White House Burn flowing beneath the A1 through an oversized concrete box culvert which is thought to also be used as a passage underneath the road for animals between fields. To prevent animals from entering the watercourse which flows along the northern side of the culvert there is a fence running through the culvert as evident in the photograph. The culvert is approximately 3.25 m wide, 3.45 m high and approximately 21.9 m long.

3.2.20. White House Burn then flows through a concrete circular culvert underneath a field access track approximately 80 m downstream from the A1 watercourse crossing. **Figure 3-15** below shows the culvert underneath the field access track. The culvert has a diameter of approximately 1.5 m and is approximately 5.3 m in length.

Figure 3-14 White House Burn A1 Culvert (Outlet)

Figure 3-15 White House Burn Field Access Culvert (Outlet)

3.2.21. A small unnamed tributary of White House Burn flows in a south to north direction adjacent to the A1 and discharges into White House Burn immediately downstream of the field access culvert. Approximately 160 m upstream of where the tributary discharges into White House Burn the tributary flows underneath a farm access track through a culvert. A circular pipe discharges into a masonry box culvert as shown in **Figure 3-16** below. There are also a number of outfalls discharging into the culvert as can be seen in the photograph.

Figure 3-16 Farm Access Track Culvert along Tributary of White House Burn (Outlet)

3.2.22. Approximately 315 m downstream from the A1 culvert, White House Burn flows underneath the B6341 through a concrete box culvert, as shown in **Figure 3-17** below.

Figure 3-17 Culvert Underneath the B6341 (Inlet)

3.2.23. Approximately 4.3 km downstream from the B6341 culvert White House Burn discharges into the River Aln adjacent to the remains of Hulne Priory, located to the south-west of Part B.

TRIBUTARIES OF KITTYCARTER BURN

3.2.24. Two tributaries of Kittycarter Burn flow beneath the existing A1 alignment. **Figure 3-18** below identifies the two tributaries (southern and western) and locations of existing structures. The southern tributary flows in a south-west to north-east direction beneath the A1 and two adjacent side roads, and the western tributary flows in a west to east direction beneath the A1. Kittycarter Burn and its tributaries are classified as ordinary watercourses under the jurisdiction of NCC as LLFA.

3.2.25. The source of the unnamed southern tributary of Kittycarter Burn is just upstream of Part B within the South Charlton Bog. The source of the unnamed western tributary of Kittycarter Burn is approximately 1.7 km to the north-west of Part B adjacent to Victory Wood. The catchment for where the two tributaries meet is relatively flat with an approximate upstream catchment area of 3.98 km^2.

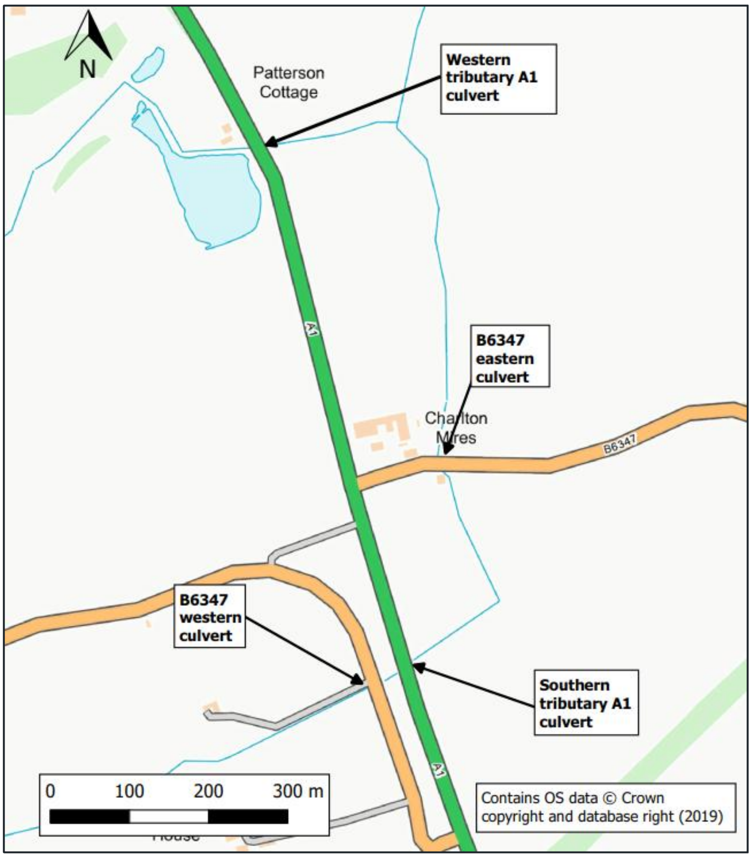

Figure 3-18 Tributaries of Kittycarter Burn Existing Structures

3.2.26. The unnamed southern tributary of Kittycarter Burn flows beneath the western section of the B6347 through a circular concrete culvert, as shown in **Figure 3-19** below. The culvert is approximately 21.2 m in length with a diameter of 0.45 m. Approximately 25 m downstream of this culvert the unnamed southern tributary flows beneath the A1 through another circular concrete culvert. **Figure 3-20** below shows the inlet of the culvert which has an approximate diameter of 0.6 m and is approximately 25.5 m in length. During the topographic survey undertaken in May and June 2018 it was noted that there was approximately 0.15 m deep silt deposit at the base of the culvert.

3.2.27. Approximately 315 m downstream of the A1 watercourse crossing the unnamed southern tributary of Kittycarter Burn flows beneath a small farm access track as shown in **Figure 3-21** below. The crossing is a circular concrete pipe with a diameter of approximately 0.6 m and approximately 3 m in length. Approximately 10 m downstream of the farm access track the unnamed southern tributary of Kittycarter Burn flows beneath the eastern section of the B6347 through a circular culvert. As shown in **Figure 3-22** below there is a brick headwall at the inlet. The culvert has an approximate diameter of 0.6 m and is approximately 15 m in length.

Figure 3-19 B6347 Western Culvert (Inlet)

Figure 3-20 Southern Tributary A1 Culvert (Inlet)

Figure 3-21 Small Access Track Culvert (Outlet)

Figure 3-22 B6347 Eastern Culvert (Inlet)

3.2.28. The unnamed western tributary of Kittycarter Burn flows beneath the A1 through a box culvert as shown in **Figure 3-23** below. There are wooden debris fences just upstream and downstream of the culvert as shown in **Figure 3-24** below and a fence running through the centre of the culvert as shown in **Figure 3-23**. It is considered likely that the fence is to facilitate animal passage between fields when required. The culvert has an approximate width of 21.4 m and height of 22.5 m and is approximately 20 m in length. In the adjacent field to the south-west of the culvert there is a pond as shown on the OS mapping. Consultation with the LLFA identified that the pond is ephemeral and floods when the water level exceeds the banks of the watercourse.

Figure 3-23 Western Tributary of Kittycarter Burn Culvert Underneath A1 (Inlet)

Figure 3-24 Debris Fence along Unnamed Western Tributary of Kittycarter Burn

3.2.29. Approximately 2 km downstream from Part B, the unnamed tributaries of Kittycarter Burn discharge into the Kittycarter Burn by the Kittycarter Plantation.

TRIBUTARY OF EMBLETON BURN

3.2.30. The unnamed tributary of Embleton Burn flows in a west to east direction beneath an access track approximately 0.95 km to the east of the existing main A1 alignment through a kiln plantation as shown in **Figure 3-25** below. Embleton Burn and its tributaries are classified as ordinary watercourses under the jurisdiction of NCC as LLFA.

3.2.31. The source of the unnamed tributary of Embleton Burn is just upstream of the access track watercourse crossing. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 0.58 km^2.

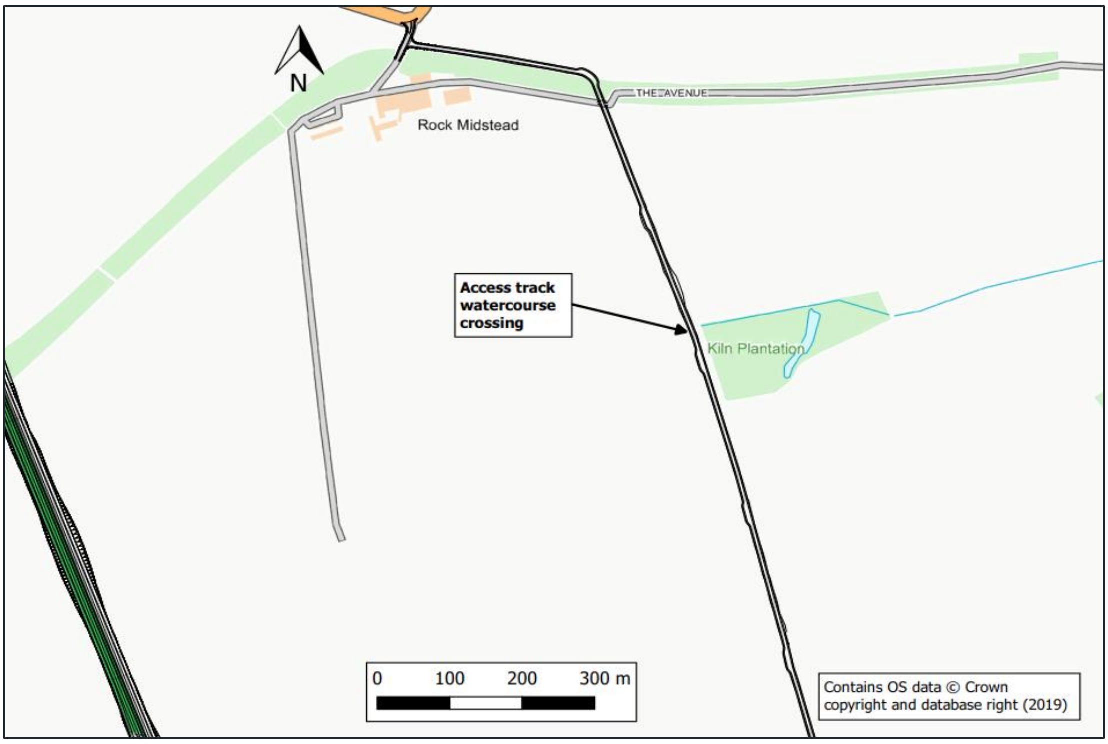

Figure 3-25 Tributary of Embleton Burn

3.2.32. **Figure 3-26** below shows the watercourse crossing that conveys the watercourse beneath an access track with a diameter of approximately 450 mm, height of approximately 310 mm and length of approximately 5.7 m. During the site walkover on 12 to 14 February 2019, it was observed the culvert was submerged. Upstream of the watercourse crossing the channel was heavily vegetated.

Figure 3-26 Tributary of Embleton Burn culvert (outlet)

3.2.33. Approximately 4.1 km downstream of the access track crossing, the unnamed tributary of Embleton Burn discharges into the Embleton Burn by Prickley Bridge.

SHIPPERTON BURN

3.2.34. Shipperton Burn flows in a west to east direction and flows beneath the existing A1 alignment through the Lodge Plantation, and then under Shipperton Bridge just downstream underneath a local private road as shown in **Figure 3-27** below. Shipperton Burn is classified as an ordinary watercourse under the jurisdiction of NCC as LLFA.

3.2.35. The source of Shipperton Burn is approximately 2.7 km to the north-west of the existing main A1 alignment, to the north of Middlemoor Wind Farm. The catchment of the watercourse is gently sloping from the north-west to the south-east with an approximate upstream catchment area of 3.09 km².

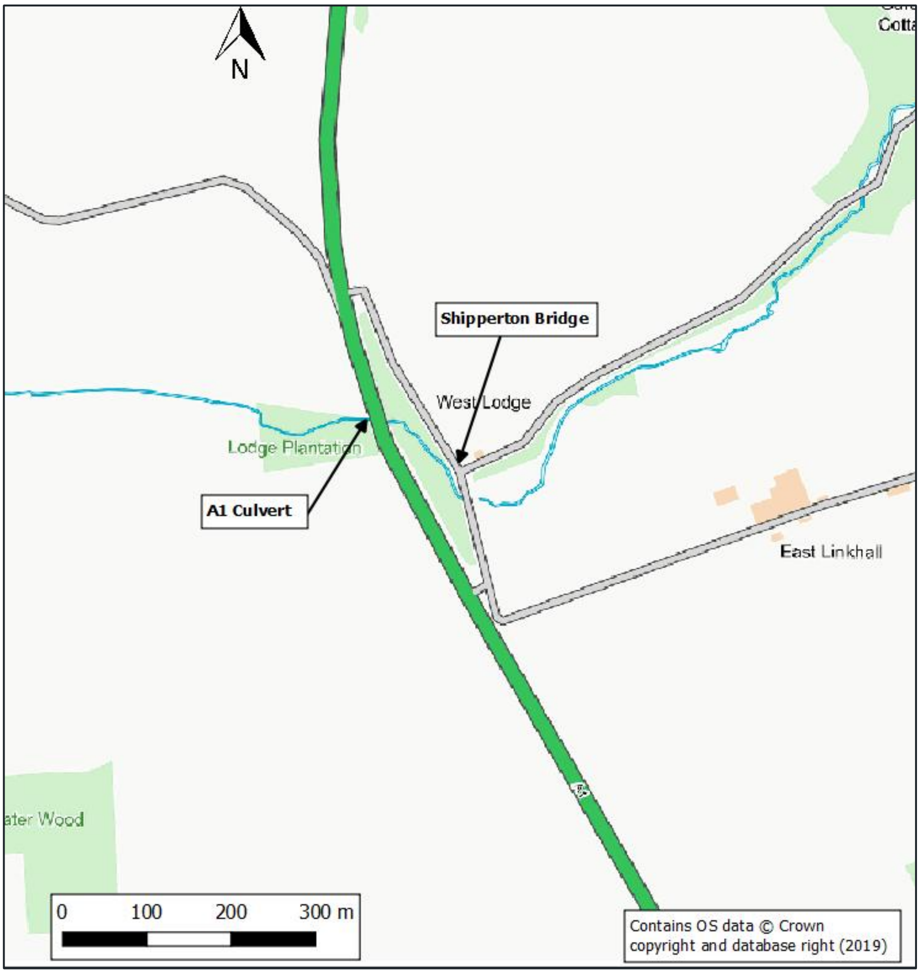

Figure 3-27 Shipperton Burn Existing Structures

3.2.36. Shipperton Burn flows beneath the A1 through a rectangular culvert (approximately 2.1 m wide and 1.2 m high) which is 18.3 m in length with the inlet and outlet shown in **Figures 3-28** and **3-29** below. Approximately 100 m downstream of this culvert the watercourse flows under Shipperton Bridge that serves as a local private road, as shown in **Figures 3-30** and **3-31** below. The bridge has a diameter of approximately 1.9 m, a height of approximately 1.1 m and length of approximately 21 m.

3.2.37. During the site walkover on 12 to 14 February 2019, immediately upstream of the existing A1 watercourse crossing, a metal gate was observed in the watercourse that was collecting debris. This is shown in **Figure 3-30**.

Figure 3-28 Shipperton Burn A1 Culvert (Inlet)

Figure 3-29 Shipperton Burn A1 Culvert (Outlet)

Figure 3-30 Shipperton Bridge (Inlet)

Figure 3-31 Shipperton Bridge (Outlet)

3.2.38. Shipperton Burn eventually discharges into Doxford Lake and becomes Mill Burn approximately 2.7 km downstream of the existing A1 crossing, to the north-east of Part B.

GEOLOGY AND HYDROGEOLOGY

3.2.39. A review of the British Geological Survey (BGS) 1:625,000 data (**Ref. 10.24**) indicates that the majority of the land located to the east of the Part B alignment is underlain by bedrock geology of the Yoredale Group comprising limestone and argillaceous rocks. Land located to the west of the Part B alignment is underlain by bedrock geology of the Yoredale Group and the Border Group consisting of limestone, sandstone and argillaceous rocks.

3.2.40. A review of BGS 1:625,000 data (**Ref. 10.24**) indicates that superficial deposits within the Part B Main Scheme Area are mostly glacial till with areas of glacial sands and gravels located to the north of South Charlton and to the south-west of Denwick. There is also a small peat deposit located to the south of South Charlton.

3.2.41. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the bedrock geology is classified as a Secondary A Aquifer. This is described as permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers.

3.2.42. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the majority of the superficial deposits are classified as a Secondary (Undifferentiated) Aquifer. The areas of glacial sands and gravels identified in **paragraph 3.2.40** above are classified as a Secondary A Aquifer. This is described as permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers.

3.2.43. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that there are no Source Protection Zones (SPZ) located within the Part B Main Scheme Area.

3.2.44. A review of the Cranfield University Soilscapes mapping (**Ref. 10.25**) indicates that the majority of the soils within the Part B Main Scheme Area are slowly permeable loamy and clayey soils. Freely draining slightly acid and loamy soils are located in the areas of glacial sands and gravels identified in **paragraph 3.2.40** above.

3.2.45. The ground investigation work undertaken in April 2019 (refer to **Appendix 11.3: Ground Investigation Report** of this ES) was completed to enhance understanding of baseline conditions. Groundwater was encountered in 21 trial pits and six boreholes during the ground investigations typically between depths of 1 m below ground level (bgl) and 3.5 m bgl. The groundwater is considered to be relatively shallow along the Part B alignment due to the presence of low permeability glacial materials overlying bedrock.

3.2.46. Sections of Part B to the north and south are located within the Coal Authority's (CA) reporting area. The online CA's screening tool (**Ref. 10.26**) indicates that Part B is not located within a constraint area with regards to groundwater.

3.3. MAIN COMPOUND

3.3.1. The Main Compound is located approximately 16.4 km to the south of the Part B Main Scheme Area and is to the south of Felton.

EXISTING SURFACE WATER FEATURES

3.3.2. A review of OS mapping indicates that the Main Compound, near West Thirston, is located in close proximity to one watercourse; an unnamed tributary of the Thirston Burn which flows along the northern boundary of the compound.

3.3.3. The unnamed tributary of the Thirston Burn flows in a west to east direction and beneath the A1 approximately 0.7 km south of the River Coquet bridge. The Thirston Burn and its tributaries are classified as ordinary watercourses and under the jurisdiction of NCC as LLFA.

3.3.4. The source of the unnamed tributary of Thirston Burn is approximately 0.5 km to the west of the Main Compound. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 0.7 km^2.

3.3.5. Approximately 2 km downstream of the Main Compound the unnamed tributary of Thirston Burn discharges into the Thirston Burn.

GEOLOGY AND HYDROGEOLOGY

3.3.6. A review of BGS 1:625,000 data (**Ref. 10.24**) indicates that the Main Compound is underlain by bedrock geology of the Yoredale Group comprising limestone, sandstone, siltstone and mudstone. A review of BGS 1:625,000 data (**Ref. 10.24**) indicates that the Main Compound is underlain by superficial deposits of glacial till and glaciofluvial deposits of sand and gravel.

3.3.7. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the bedrock geology is classified as a Secondary A Aquifer and the superficial deposits are classified as a Secondary (Undifferentiated) Aquifer.

3.3.8. Review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the Main Compound is not located within a SPZ.

3.3.9. A review of the Cranfield University Soilscapes mapping (**Ref. 10.25**) indicates that the Main Compound is underlain by freely draining slightly acid loamy soils.

3.4. LIONHEART ENTERPRISE PARK COMPOUND

3.4.1. The Lionheart Enterprise Park Compound is located approximately 4 km to the south of the Part B Main Scheme Area in the Lionheart Enterprise Park, just to the south of Alnwick.

EXISTING SURFACE WATER FEATURES

3.4.2. The Lionheart Enterprise Park Compound is located approximately 200 m to the north-west of Cawledge Burn.

3.4.3. Cawledge Burn flows in a west to east direction to the south of Alnwick. Cawledge Burn is classified as an ordinary watercourse and under the jurisdiction of NCC as LLFA.

3.4.4. The source of Cawledge Burn is approximately 6 km to the west of the Lionheart Enterprise Park Compound. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 15 km^2.

3.4.5. Approximately 2 km downstream of the Lionheart Enterprise Park Compound Cawledge Burn discharges into the River Aln.

GEOLOGY AND HYDROGEOLOGY

3.4.6. A review of BGS 1:625,000 data (**Ref. 10.24**) indicates that the Lionheart Enterprise Park Compound is underlain by bedrock geology of the Yoredale Group comprising limestone, sandstone, siltstone and mudstone. A review of BGS 1:625,000 data (**Ref. 10.24**) indicates that the Lionheart Enterprise Park Compound is underlain by superficial deposits of glacial till and glaciofluvial deposits of sand and gravel.

3.4.7. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the bedrock geology underlying the Lionheart Enterprise Park Compound is classified as a Secondary A Aquifer. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the superficial deposits are classified as a Secondary (Undifferentiated) Aquifer.

3.4.8. A review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref. 10.8**) indicates that the Lionheart Enterprise Park Compound is not located within a SPZ.

3.4.9. A review of the Cranfield University Soilscapes mapping (**Ref. 10.25**) indicates that the Lionheart Enterprise Park Compound is underlain by freely draining slightly acid loamy soils.

4. EXISTING FLOOD RISK

4.1. HISTORIC FLOOD RECORDS

4.1.1. Consultation with NCC has highlighted an existing flooding issue associated with fluvial flooding from the tributaries of Kittycarter Burn. It is believed that lack of maintenance along the watercourses has led to local flooding issues affecting isolated properties. It is also believed that the existing culvert underneath the A1 along the western tributary of Kittycarter Burn is not conveying flow through the structure efficiently due to the lack of maintenance and the fence that runs through the centre of the culvert. This was also highlighted in the responses collated during the 2019 statutory consultation.

4.1.2. The NCC Level 1 Strategic Flood Risk Assessment (**Ref. 10.27**) indicates significant flooding within the north-east Northumberland river catchments from fluvial and pluvial sources since 1744. A number of significant flood events are attributed to the River Aln which is located downstream of the Study Area.

4.1.3. The HADDMS (Highways Agency Drainage Data Management System) online database (**Ref. 10.28**) indicates that the Alnwick to Ellingham section of the existing A1 has two documented surface water flood events. These are not classified as severe flood events and have a severity index of less than one. The flooding was associated with blocked gullies. There are no flooding hotspots within the Study Area.

4.1.4. There is no historic flooding information associated with the three construction compounds.

4.2. FLUVIAL FLOOD RISK

INDICATIVE FLOOD MAPPING

4.2.1. A review of the Environment Agency Flood Map for Planning (Rivers and Sea) (**Ref. 10.5**) indicates that the alignment in the Part B Main Scheme Area is located in the low-risk Flood Zone 1. Extracts from the Environment Agency Flood Map for Planning (Rivers and Sea) (**Ref. 10.5**) are provided in **Figures 4-1** and **4-2** below.

4.2.2. However, within the Order Limits of Part B there are two areas located within the medium risk Flood Zone 2, and the high-risk Flood Zone 3. There is one area located to the south within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Denwick Burn. The other area is located to the north within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Shipperton Burn.

4.2.3. **Figure 10.1: Water Constraints Plan**, **Volume 6** of this ES (**Application Document Reference: TR10041/APP/6.6**) shows the construction compounds are located within Flood Zone 1.

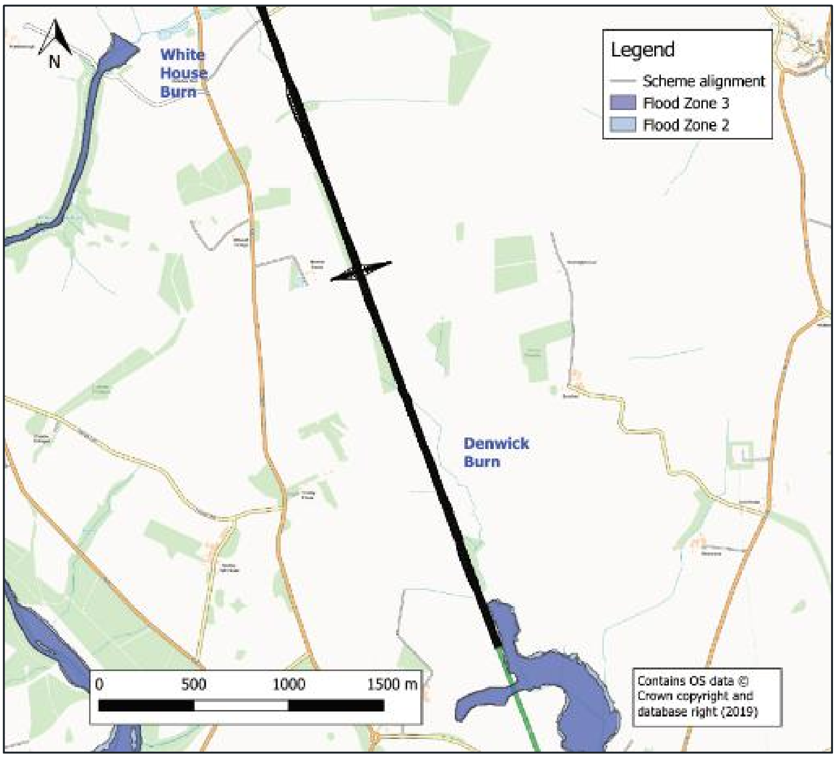

Figure 4-1 Extract from Environment Agency Flood Map for Planning (May 2019) for Denwick Burn and White House Burn

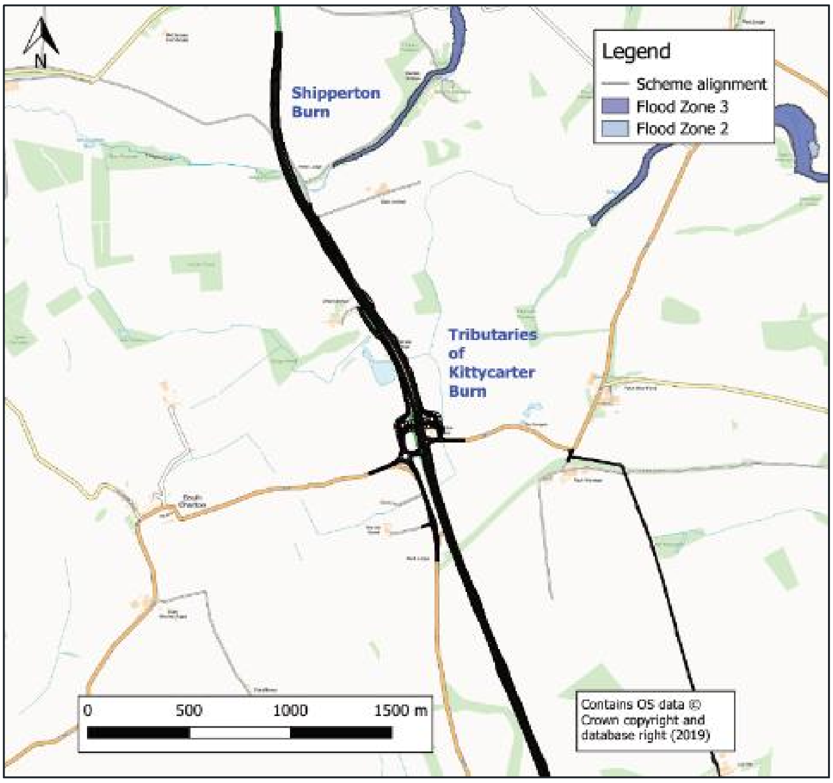

Figure 4-2 Extract from Environment Agency Flood Map for Planning (May 2019) for Shipperton Burn and Kittycarter Burn

DETAILED HYDRAULIC MODELLING

4.2.4. Detailed 1D hydraulic modelling has been undertaken for Denwick Burn, White House Burn, the tributaries of Kittycarter Burn and Shipperton Burn. These watercourses either have large drainage catchments with large tributaries (and hence inflows) close to Part B or a number of structures in the vicinity of the A1 that require detailed modelling to quantify the cumulative effect of these.

4.2.5. Full details of the hydraulic modelling work undertaken are provided in **Appendix A: Hydraulic Modelling Analysis** of this FRA. **Figures 4-3** to **4-6** provide details of the flood risk extents in the existing condition for the 100 year and 1,000 year flood events for the modelled extents. The derived baseline extents are compared to the national mapping available for each watercourse, i.e. the Flood Zones or the Surface Water Flood Extents, prioritised in that order. These maps have been included for information but highlight where the existing national mapping is coarse and does not reflect the actual alignment of the watercourse.

4.2.6. The quality of the maps produced is dependent on the availability of local ground level data. LiDAR data was not available for the Study Area. Instead ground levels are based on unfiltered 2 m photogrammetry data, which was available for all watercourses. The photogrammetry data has been compared to survey data and adjusted where necessary, however where dense vegetation is present the channel and floodplain may not be represented accurately.

4.2.7. Full details of the existing A1 structures and their hydraulic capacity are discussed in **Section 5** to provide a comparison to the Part B proposals.

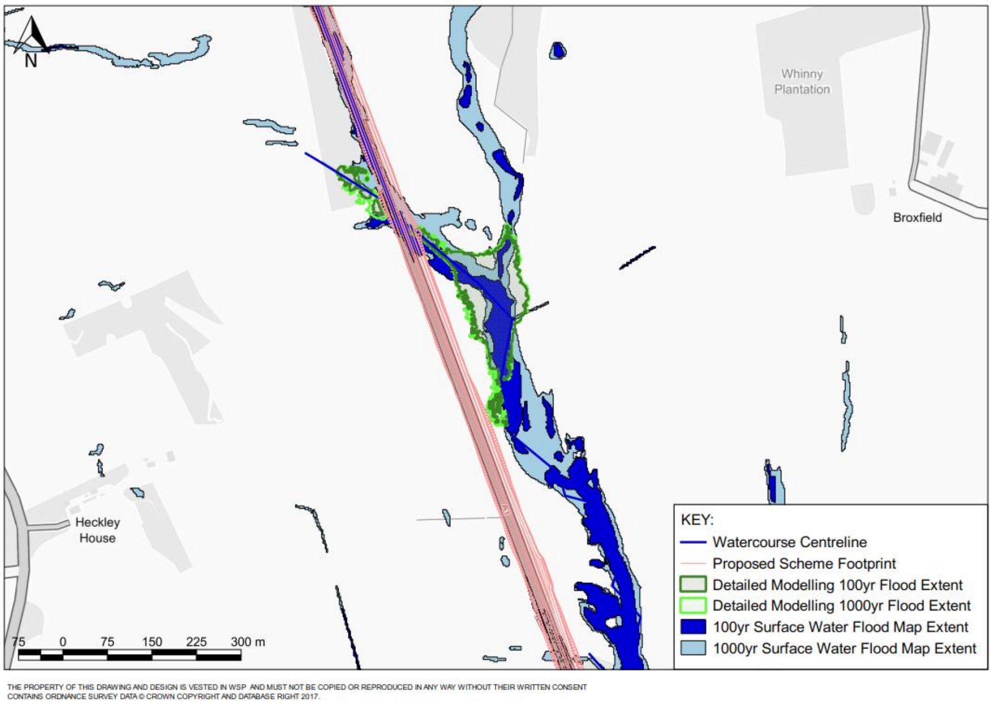

Figure 4-3 Existing Flood Risk Extents for Denwick Burn

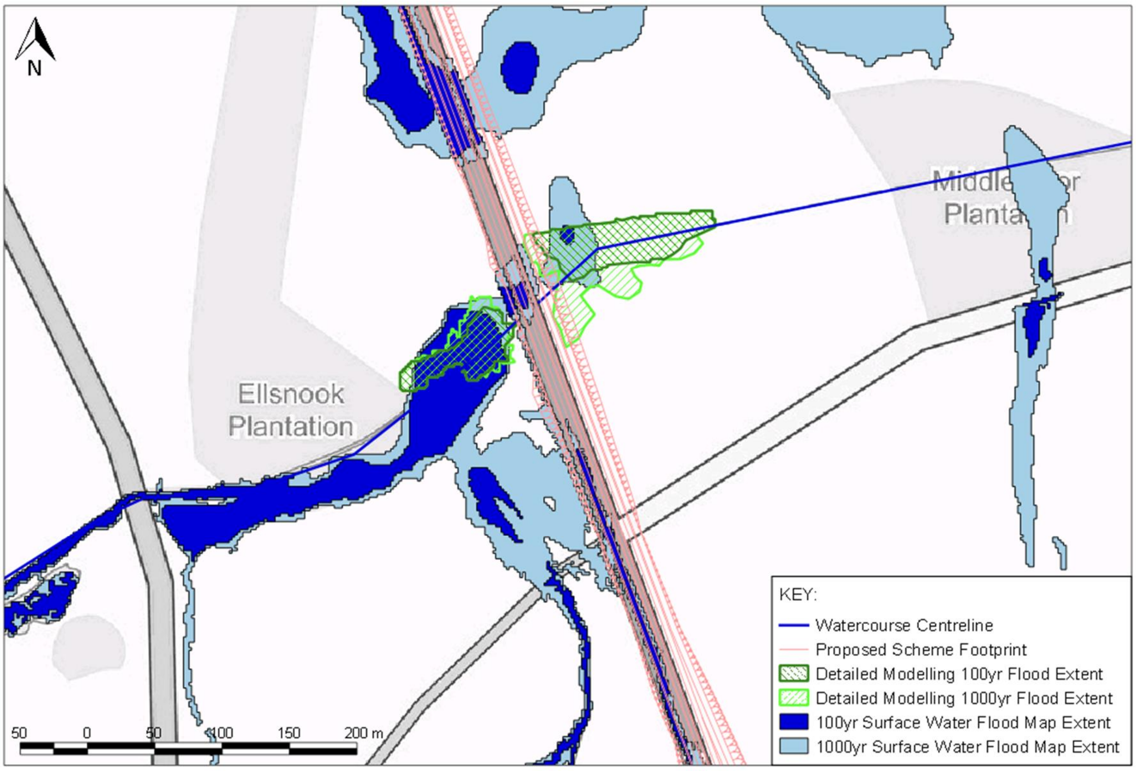

Figure 4-4 Existing Flood Risk Extents for White House Burn

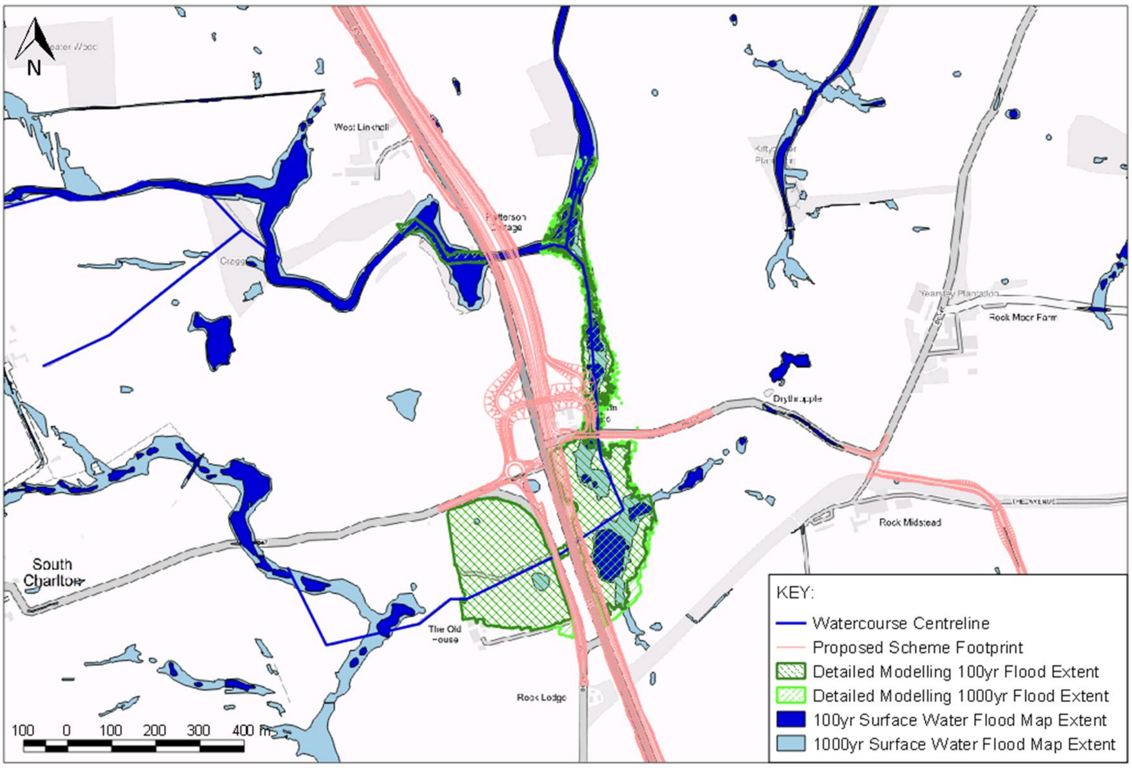

Figure 4-5 Existing Flood Risk Extents for the Tributaries of Kittycarter Burn

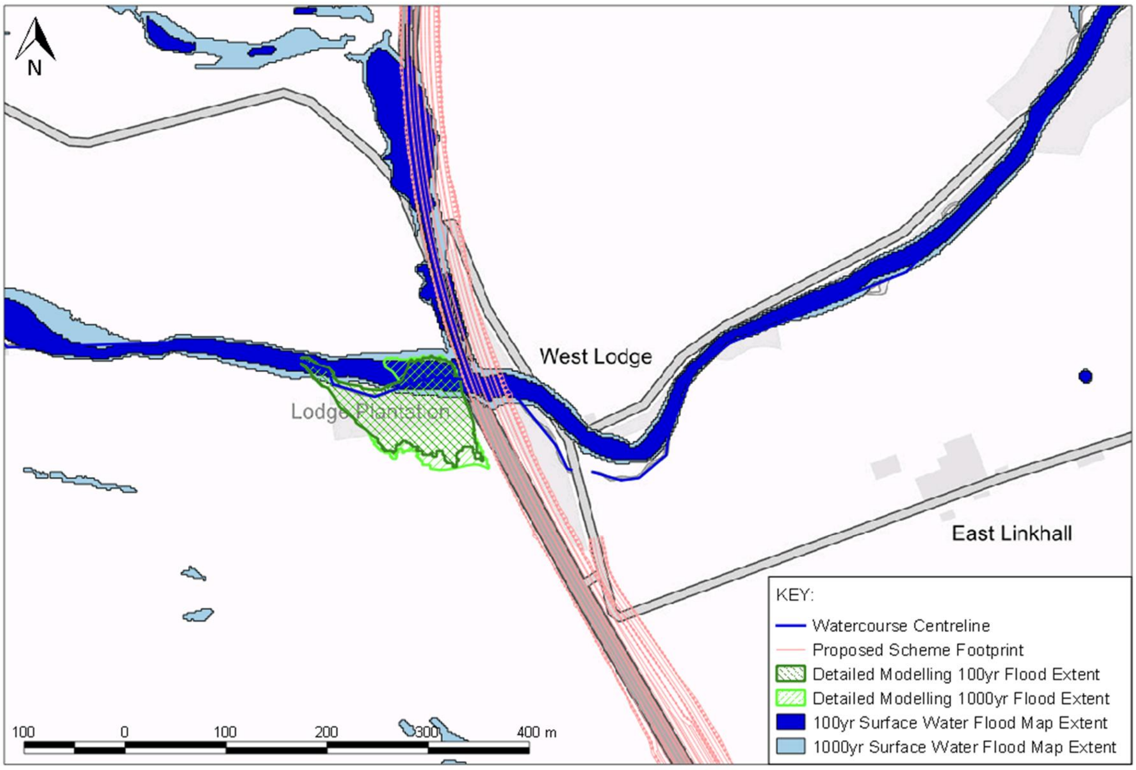

Figure 4-6 Existing Flood Risk Extents for Shipperton Burn

4.3. OTHER SOURCES OF FLOOD RISK

TIDAL FLOODING

4.3.1. Part B is not at risk of tidal flooding as the tidal limits for the River Aln, Embleton Burn and Long Nanny are downstream of the Study Area and are located at a lower elevation than the minimum Part B elevation (60.09 mAOD at the southern part of Part B). The tidal limit for the River Aln is at Lesbury is located approximately 5.7 km to the east of Part B and at an elevation of approximately 20 mAOD. The tidal limit for Long Nanny is located approximately 7.2 km to the north-east of Part B at Tughall Mill and at an elevation of approximately 5 mAOD. The tidal limit for Embleton Burn is located approximately 6.5 km to the east of Part B to the east of Embleton, and at an elevation of approximately 25 mAOD.

SURFACE WATER FLOODING

4.3.2. A review of the Environment Agency's Flood Risk from Surface Water map (**Ref. 10.7**) (shown in **Figure 4-7**) indicates that sections of Part B (excluding construction compounds) are at high, medium and low risk of flooding from surface water sources. Flooding from surface water is typically associated with natural overland flow paths (including the watercourses discussed above) and local depressions in topography where surface water runoff can accumulate during or following heavy rainfall events.

4.3.3. Known surface water flow paths have been incorporated into Part B and details are provided in **Section 5** of this FRA.

4.3.4. A review of the Environment Agency's Flood Risk from Surface Water map (**Ref. 10.7**) indicates that the construction compounds are at a low risk of flooding from surface water sources.

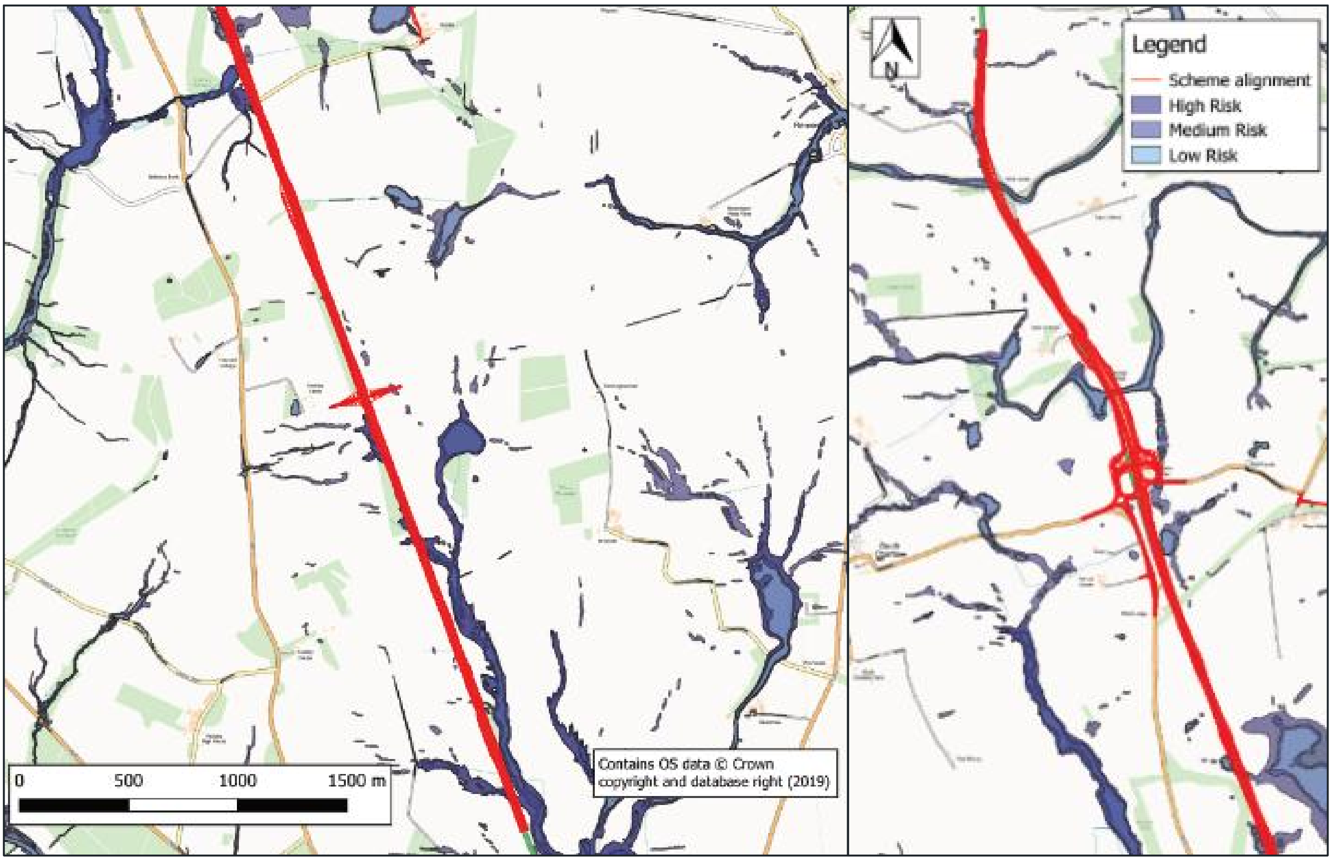

Figure 4-7 Extract from Environment Agency Surface Water Flood Risk Map (May 2019)

GROUNDWATER

4.3.5. Groundwater flooding occurs when water stored below ground reaches the surface. It is commonly associated with porous underlying geology, such as chalk, limestone and gravels. Based on the baseline geology and hydrogeology information previously described, it is assumed that groundwater is likely to be close to the surface and therefore has the potential to cause groundwater flooding. However, due to the relatively low permeability of the majority of the bedrock and the superficial deposits underlying Part B, groundwater levels are unlikely to fluctuate significantly and as a result groundwater flooding is unlikely to occur.

4.3.6. The areas of glacial sands and gravels located to the north of South Charlton and to the south-west of Denwick, associated with Denwick Burn and the tributaries of Kittycarter Burn, which have a higher permeability may experience groundwater flooding. However, this should not affect Part B as any groundwater emergence would overflow to the adjacent watercourses.

ARTIFICIAL SOURCES

4.3.7. A review of the Environment Agency's Flood Risk from Reservoirs map (**Ref. 10.7**) indicates that Part B is not at risk of flooding from potential failure of reservoirs located upstream of the Study Area.

4.3.8. A review of OS mapping indicates that there is an unnamed covered reservoir within 0.5 km of Part B. It is located approximately 0.1 km to the west of the existing A1 near Craggy Wood. Although the covered reservoir is not visible on satellite imagery, due to the spatial constraints around the site it is likely that the reservoir would be small in size. As a result, it is assumed that there is no risk associated with the potential failure of the reservoir to Part B.

OTHER SOURCES OF FLOOD RISK

4.3.9. No other sources of flood risk have been identified.

5. POST DEVELOPMENT FLOOD RISK

5.1. DESIGN MEASURES

5.1.1. With specific regards to flooding Part B includes the following works from south to north as set out in **Figure 5-1** below (the numbers in brackets relate to the approximate location of the works):

a. There are no proposed works to the existing culvert Denwick Burn at chainage 53470 as the existing culvert is of sufficient length (1).
b. There are no proposed works to the existing culvert Denwick Burn at chainage 53850 as the existing culvert is of sufficient length (2).
c. The extension of the existing culvert Denwick Burn at chainage 54080 (3).
d. The extension of the existing culvert Denwick Burn at chainage 54600 (4).
e. The replacement of the existing culvert at Heckley Fence at chainage 55300. The small drainage ditch upstream of the culvert would be realigned to discharge into the new culvert (5).
f. The extension of the existing culvert White House Burn at chainage 56920 (6).
g. The extension of the existing culvert Kittycarter Burn at chainage 58600 (7).
h. The removal of the existing culvert along the southern tributary of Kittycarter Burn and the construction of a new circular culvert (Proposed culvert 25.1) underneath the B6347 at chainage 58840 (8).
i. The diversion and channel realignment of the southern tributary of Kittycarter Burn to reduce the length of culvert required (9).
j. The extension of the existing Linkhall Culvert along the western tributary of Kittycarter Burn at chainage 59275 (10).
k. The extension of the existing culvert Shipperton Burn at chainage 60385 (11).
l. The demolition of the existing culvert along the unnamed tributary of Embleton Burn and the construction of a new circular culvert called Rock Culvert at chainage 58100 (12).
m. Installation of new drainage infrastructure to accommodate increased runoff rates and volumes from the increase in impermeable area and construction of runoff detention basins to manage surface water flow from the drainage network.

5.1.2. No permanent works are proposed to any watercourse in close proximity to the Main Compound or Lionheart Enterprise Park Compound and therefore no operational assessment has been undertaken.

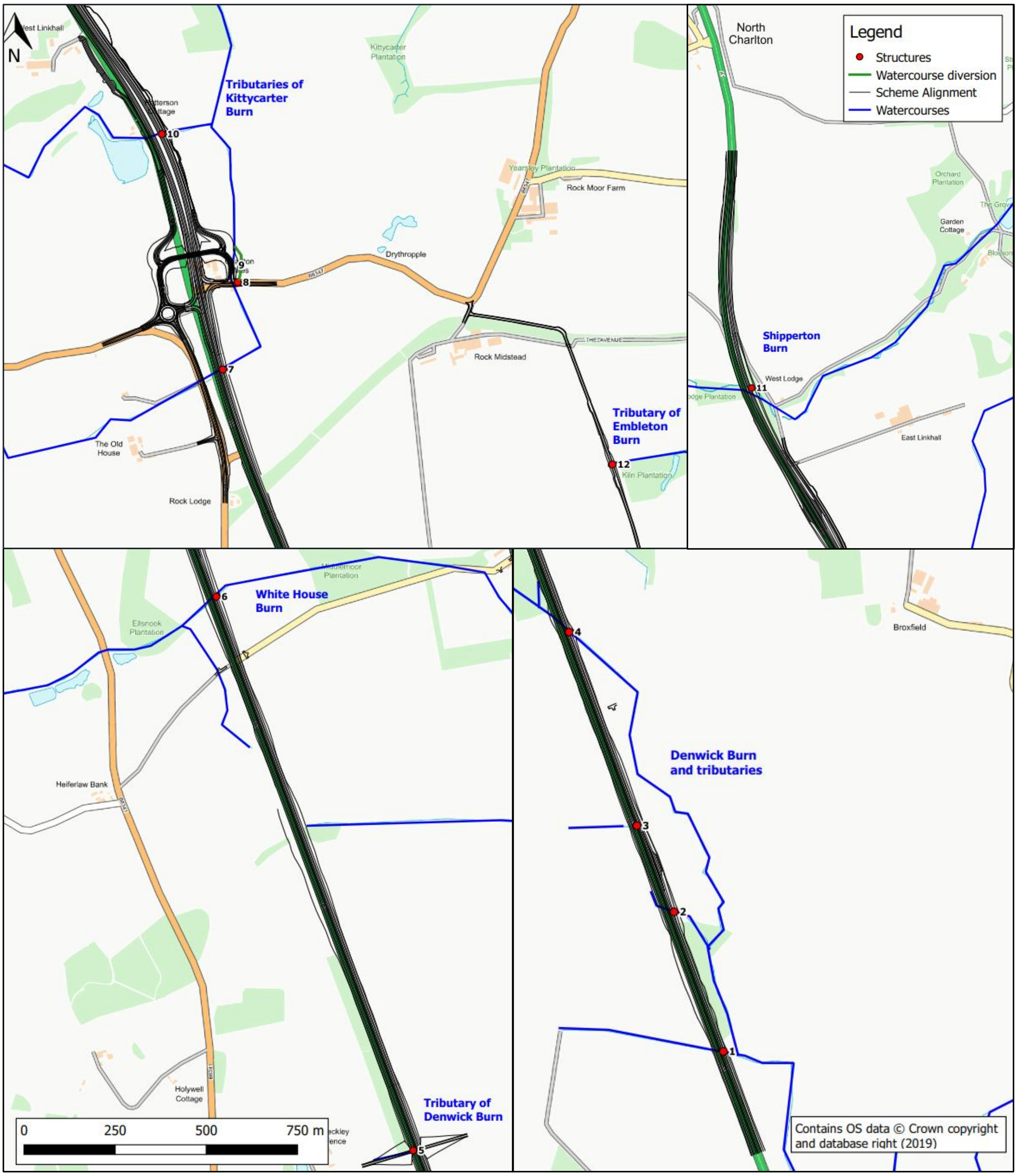

Figure 5-1 Part B Extent and Proposed Works with Regards to Flooding

5.1.3. A summary of the of the proposed works, assessment of flood risk and proposed mitigation for each of these aspects is provided below.

5.2. HYDRAULIC DESIGN OF WATERCOURSE CROSSINGS

5.2.1. A summary of the larger watercourses that were subject to 1D hydraulic modelling are provided in **Sections 5.3** to **5.7** with details of the hydrology and hydraulic modelling for each included in **Appendix A: Hydraulic Modelling Analysis** of this FRA.

5.2.2. The remaining watercourses, drainage ditches and identified surface water flow paths crossing Part B are summarised in **Section 5.8** with the details of culvert modelling included in **Appendix B: Culvert Master Analysis** of this FRA.

5.2.3. It should be noted that design of the culvert lengths has been refined since the completion of the hydraulic analysis. In all instances the lengths of the culverts have reduced in comparison to the modelled dimensions, and this is no more than a 20 % reduction in culvert length. These changes are not considered to materially affect the findings of the FRA. Further modelling would be undertaken at the detailed design stage once the design of these culverts has been finalised.

5.3. DENWICK BURN

5.3.1. Denwick Burn has been modelled across two hydraulic models, one for the southern extent and one for the northern extent.

DENWICK BURN (SOUTH)

Overview of Part B Requirements

5.3.2. Denwick Burn (South) is a rural watercourse with an upstream catchment of 3.8 km^2 to the most southern cross section in **Figure 5-2**. The catchment is entirely rural with no flood risk receptors upstream and downstream of the A1 crossing.

5.3.3. An overview of the proposals in relation to Denwick Burn (South) is provided in **Figure 5-2**.

5.3.4. The proposed works in this area consist of widening the A1 only.

5.3.5. Existing culverts one (proposed culvert 17.1) and two (proposed culvert 18.1) underneath the A1 (labelled in **Figure 5-2**) are already of a sufficient length to convey flows beneath the footprint of the proposed embankment. The existing three farm access culverts would all be retained as part of Part B with no works proposed. As such, there would be no change to the flood risk as a result of Part B. No further analysis has been undertaken for this part of Denwick Burn.

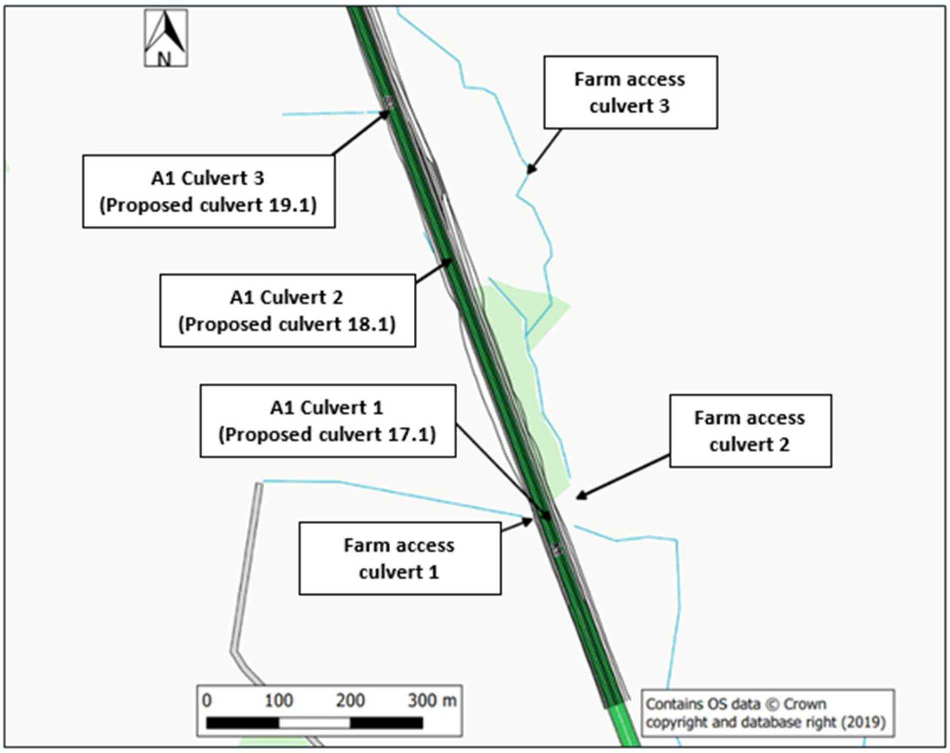

Figure 5-2 Overview of Proposals in Relation to Denwick Burn

DENWICK BURN (NORTH)

Overview of Part B Requirements

5.3.6. Denwick Burn (North) drains a rural catchment approximately 2.3 km^2 to the most southern cross section in **Figure 5-5**. The catchment is entirely rural with no flood risk receptors within the proposed A1 alignment.

5.3.7. An overview of Part B in relation to the Denwick Burn (North) is provided in **Figure 5-3**. The existing PRoW crossing would be retained as part of Part B with no works proposed.

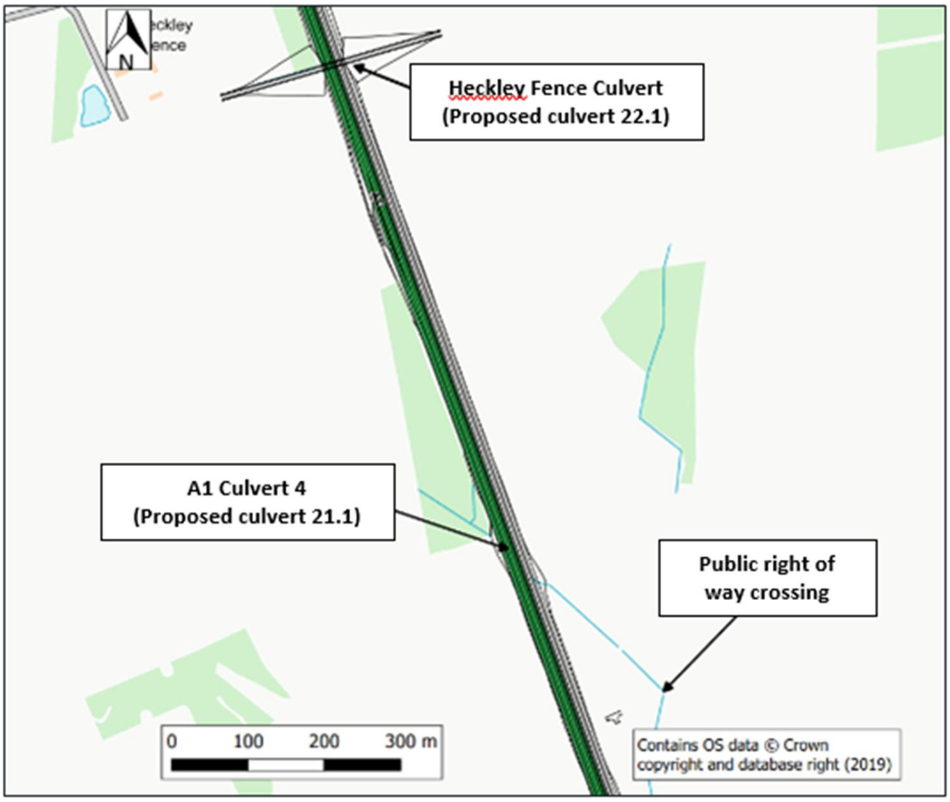

Figure 5-3 Overview of Proposals in Relation to the Denwick Burn

A1 Culvert 3 (Proposed Culvert 19.1)

5.3.8. The proposed work at Denwick Burn (North) consists of an extension of the existing A1 culvert three (proposed culvert 19.1) on the outlet side by 21.25 m. The extension would be a new 600 mm circular pipe, and tie into the existing culvert. This culvert was not included in the hydraulic model and was assessed using Culvert Master based on professional judgement and an understanding of the area. Refer to **Table 5-9** in **Section 5.7** of this FRA for the assessment.

A1 Culvert 4 (Proposed Culvert 21.1)

5.3.9. The existing culvert (A1 culvert 4) located at chainage 54600 underneath the A1 alignment would be extended by 38 m with a new precast concrete 1.2 m pipe, and the construction of a new headwall and wing wall at the culvert outlet. The length of the extended culvert would be 110.3 m. The culvert extension would be on the same alignment of the watercourse.

Heckley Fence Culvert (Proposed Culvert 22.1)

5.3.10. The new accommodation overbridge at Heckley Fence would replace the existing culvert arrangement with a realigned watercourse channel immediately to the north of the proposed earthworks for the overbridge. The new alignment would tie into the extended culvert. Pipe sizes and inlets would match the existing culvert and is assumed to be a 300 mm circular pipe that would be approximately 43.75 m in length. This culvert was not included in the hydraulic model and was assessed using Culvert Master based on professional judgement and an understanding of the area. Refer to **Table 5-9** in **Section 5.7** of this report for the assessment.

PART B PROPOSALS

5.3.11. The proposals for the culvert extension are set out in **Table 5-1** below. **Table 5-1** also details the dimensions of the existing A1 culvert four and PRoW crossing for comparison.

Table 5-1 Existing and Proposed Dimensions of Denwick Burn (North) Structures

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing A1 culvert 4	72.3	Circular	1.2	-
Proposed A1 culvert 4 (Proposed culvert 21.1)	110.3	Circular	1.2	-
Existing PRoW crossing	10	Arch	6.6	2.41 (1.44 Arch)

DESIGN OUTCOMES

5.3.12. **Table 5-2** provides details of the freeboard associated with each structure for a range of flood events. Denwick Burn is classified as an ordinary watercourse. As such a design freeboard within the watercourse structure of 300 mm is preferred in the 100 year + 25 % climate change events in accordance with DMRB (HD 107/04) (**Ref. 10.10**). The 1,000-year event is larger than the 100 year + 50 % climate change events so has been used to assess residual risk in an extreme event. Given the size of the proposed structures, blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 67 %. The negative values in **Table 5-2** below shows that the respective structure is surcharged while a positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

Table 5-2 Design freeboard for Denwick Burn (North) structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage
Existing A1 culvert 4	2.34	0.48	-0.58	-1.10	-	0.71	0.12	-0.07	-
Proposed A1 culvert 4 (Proposed culvert 21.1)	2.36	0.48	-0.58	-1.35	-1.66	0.65	0.17	-0.01	-
Existing PRoW crossing	0.13	-0.28	-0.78	-0.86	-	0.22	-0.14	-0.26	-

5.3.13. **Table 5-2** shows that a freeboard of 300 mm is not achieved in the 100 year + 25 % climate change event for either the existing or proposed scenarios. However, there is significant freeboard to the carriageway crest level (noting the carriageway is 2.36 m above the inlet soffit) and Part B is not overtopped for the PRoW crossing in either the extreme 1,000 year event or allowing for 67 % blockage of the structure.

PREDICTED FLOOD RISK IMPACTS

5.3.14. The effect of Part B on upstream water levels and pass forward flows has been reviewed to understand the wider implications of Part B on flood risk.

5.3.15. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. **Figure 5-4** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part B. **Figure 5-5** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 0.5 km downstream of the existing A1.

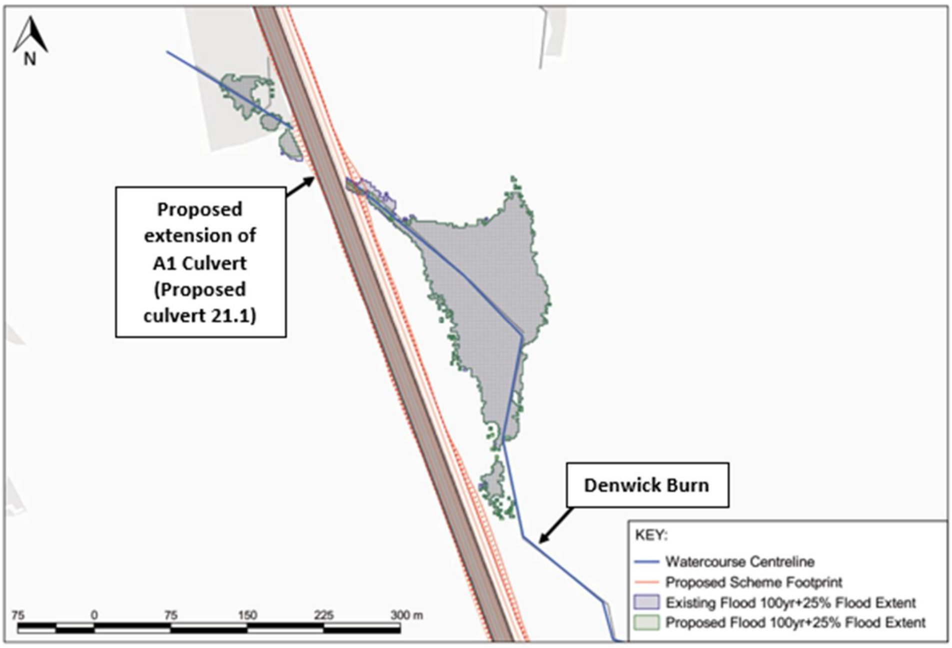

Figure 5-4 Flood Extents in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

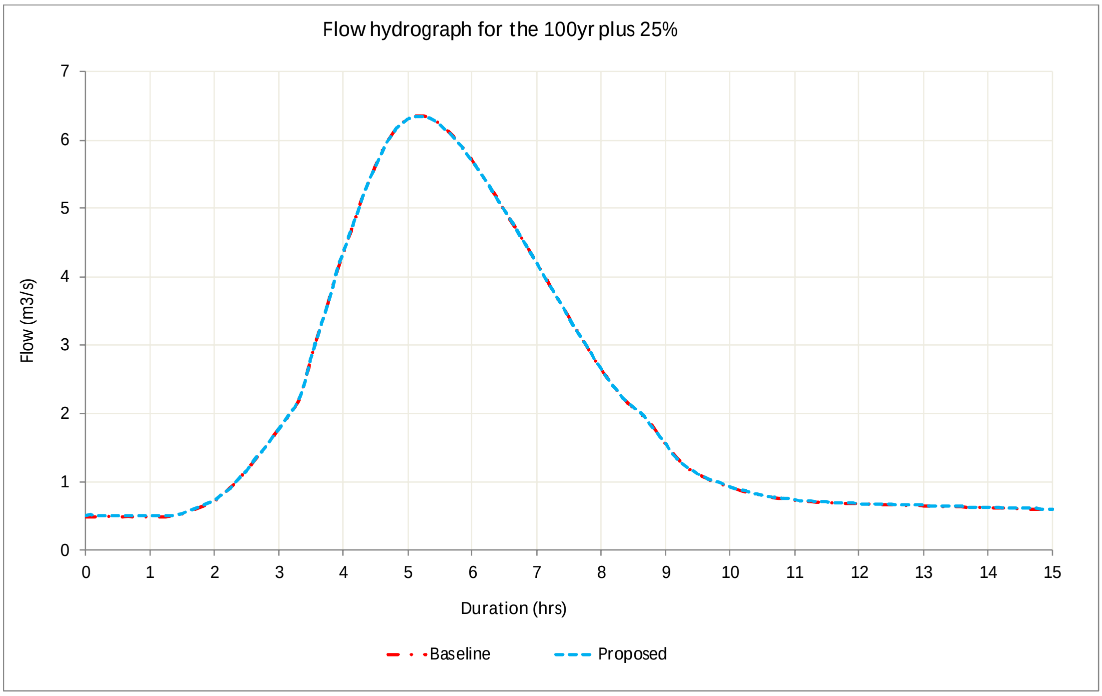

Figure 5-5 Pass Forward Flows in the Existing and Proposed Scenarios for the 100 Year + 25 % Climate Change Event

5.3.16. **Figure 5-4** demonstrates that Part B would have a negligible impact on water levels upstream of the A1 because the culvert is extended on the outlet side. There is some change in the extents shown downstream of the culvert but these are associated with localised mapping effects. **Figure 5-5** confirms there is also no downstream impact on flows associated with Part B.

5.4. WHITE HOUSE BURN

OVERVIEW OF PART B REQUIREMENTS

5.4.1. White House Burn drains a predominantly rural catchment of approximately 2.4 km^2 to the east of Part B. The catchment is entirely rural with no flood risk receptors upstream or downstream of the A1 crossing.

5.4.2. An overview of Part B in relation to White House Burn is provided in **Figure 5-6**.

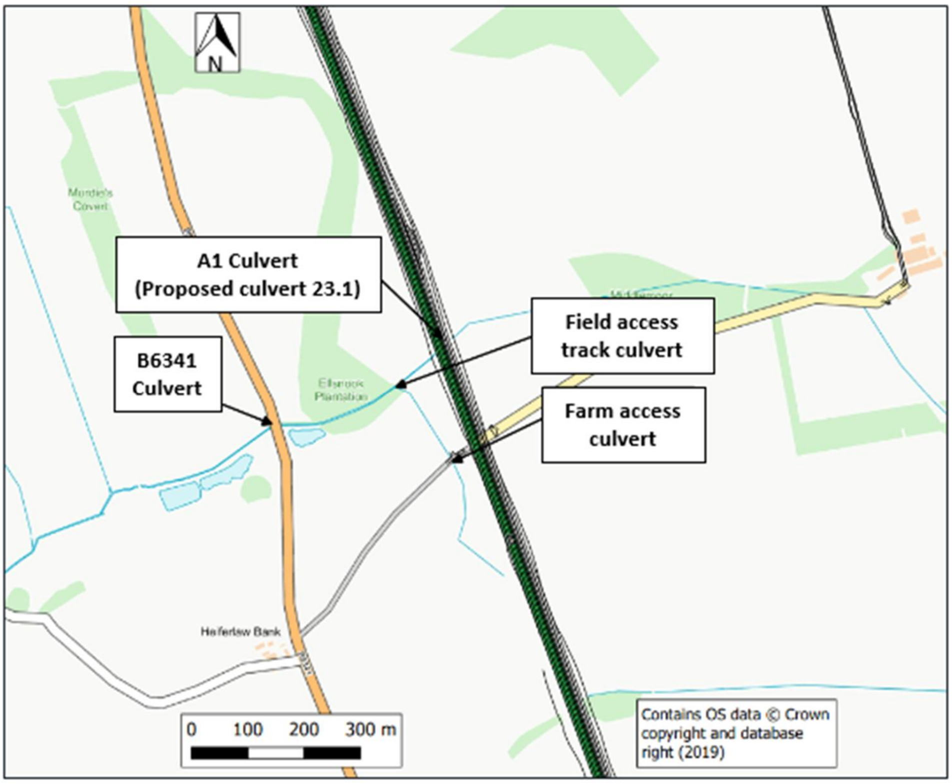

Figure 5-6 Overview of Proposals in Relation to the White House Burn

5.4.3. The proposed works at White House Burn consist of an extension of the existing A1 culvert eastwards on the inlet side by 15.6 m. The proposed extension would be a precast reinforced concrete box culvert with 3.23 m width and 3.44 m height to match the existing culvert dimensions.

5.4.4. The existing farm access culverts and culvert beneath the B6341 would all be retained as part of Part B with no works proposed.

PART B PROPOSALS

5.4.5. The proposals for the culvert extension is set out in **Table 5-3**. **Table 5-3** also details the dimensions of the existing field access track culvert for comparison.

Table 5-3 Existing and Proposed Dimensions of White House Burn structures

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing field access track culvert	6.0	Circular	1.5	-
Existing A1 culvert	21.7	Rectangular	3.23	3.44
Proposed A1 culvert (Proposed culvert 23.1)	37.3	Rectangular	3.23	3.44

DESIGN OUTCOMES

5.4.6. **Table 5-4** provides details of the freeboard associated with each structure for a range of flood events. White House Burn is an ordinary watercourse. As such a design freeboard of 300 mm within the watercourse structure is preferred in the 100 year + 25 % climate change event in accordance with the recommendations in the DMRB (HD 107/04) (**Ref. 10.10**). The 1,000-year event is larger than the 100 year + 25 % climate change event so has been used to assess risk in an extreme event. Given the size of the proposed extended structure, blockage has been assessed by assuming the inlet capacity of the culvert structure is reduced by 30 %. The negative values in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

Table 5-4 Design Freeboard for White House Burn Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage
Existing A1 culvert	1.81	2.64	2.13	1.98	-	2.43	1.97	1.85	-
Proposed A1 culvert (proposed culvert 23.1)	1.57	2.79	2.35	2.19	2.1	2.43	1.97	1.85	-

5.4.7. **Table 5-4** shows that a freeboard of 300 mm is achieved in the 100 year+ 25 % climate change event for the A1 culvert in the existing and proposed scenarios.

5.4.8. The crest level of the carriageway is 1.57 m above the soffit of the A1 culvert. The results suggest Part B would not be overtopped in either the extreme 1,000-year event or allowing for blockage of the structures.

PREDICTED FLOOD RISK IMPACTS

5.4.9. The effect of Part B on upstream water levels and pass forward flows has been reviewed to understand the wider implications of Part B on flood risk.

5.4.10. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. **Figure 5-7** Flood Extents in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Eventpresents the mapped flood risk extents for the 100 year + 25 % climate change events in the existing situation and following the construction of Part B. **Figure 5-8** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 200 m downstream of Part B.

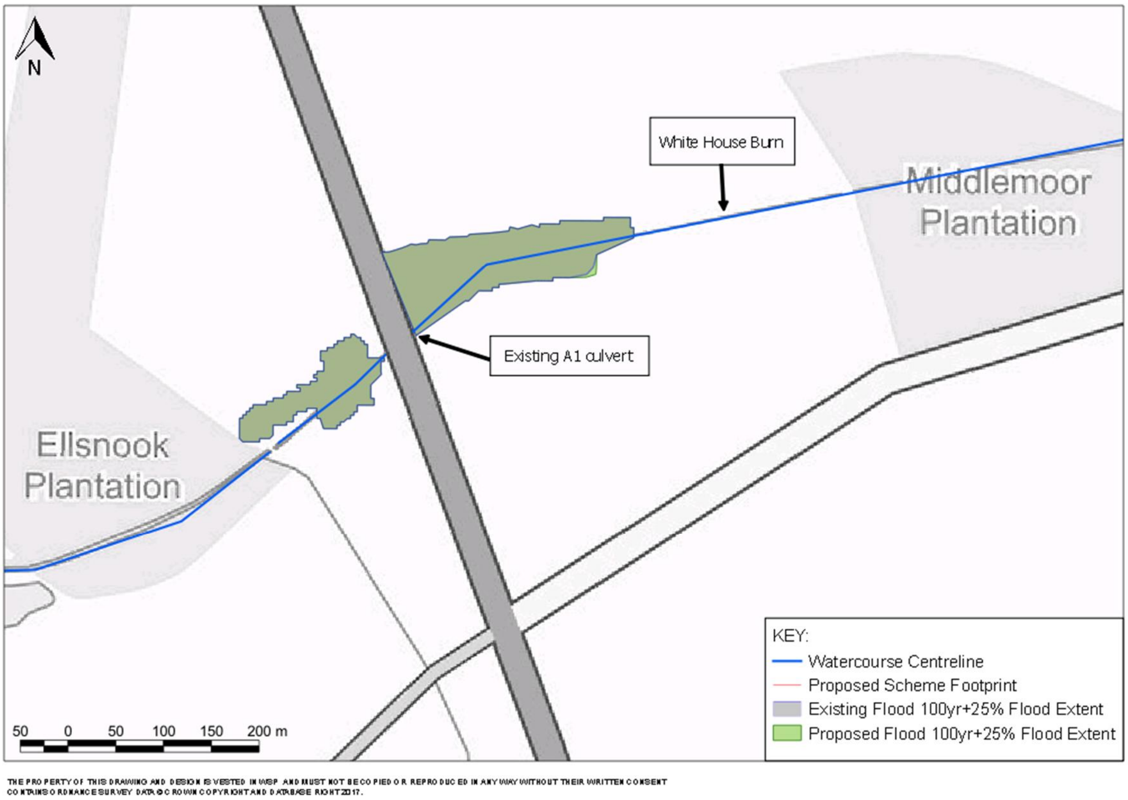

Figure 5-7 Flood Extents in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

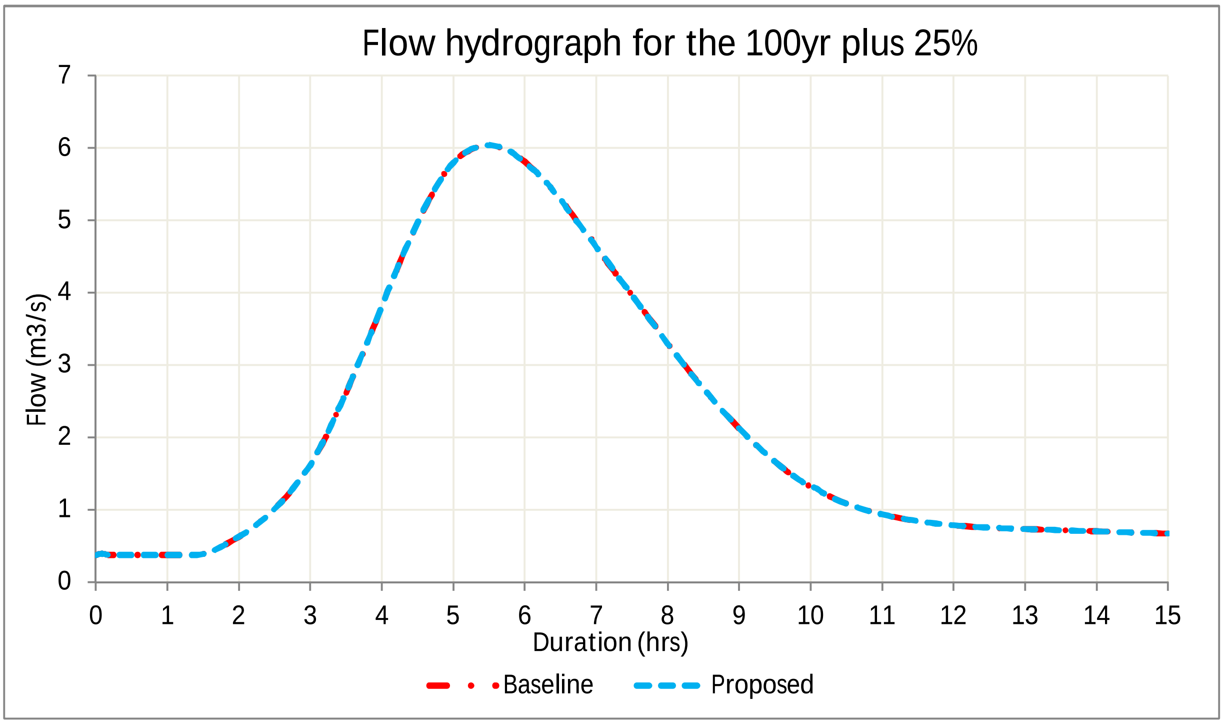

Figure 5-8 Pass Forward Flows in the Existing and Proposed Scenarios for the 100 Year + 25 % Climate Change Event

5.4.11. **Figure 5-7** demonstrates that Part B would result in a slight increase in water levels upstream of the A1 culvert; there are no receptors that would be affected by this increase. **Figure 5-8** confirms that there would be no change to the downstream flows resulting from Part B for all flows up to the 100 year + 25 % climate change event.

5.5. TRIBUTARIES OF KITTYCARTER BURN

OVERVIEW OF PART B REQUIREMENTS

5.5.1. Both of the two tributaries of Kittycarter Burn (southern and western) pass beneath the A1. The catchment for the southern tributary to the A1 crossing is approximately 1.1 km^2 and the catchment for the western tributary to the A1 crossing is approximately 2.0 km^2. Both catchments are rural with some individual properties located within the catchments. Of these properties the key receptor of concern is a property located on the left bank of the western tributary immediately upstream of the western tributary A1 culvert.

5.5.2. An overview of Part B in relation to the tributaries of Kittycarter Burn is provided in **Figure 5-9**. The existing B6347 western culvert would be retained as part of Part B with no works proposed.

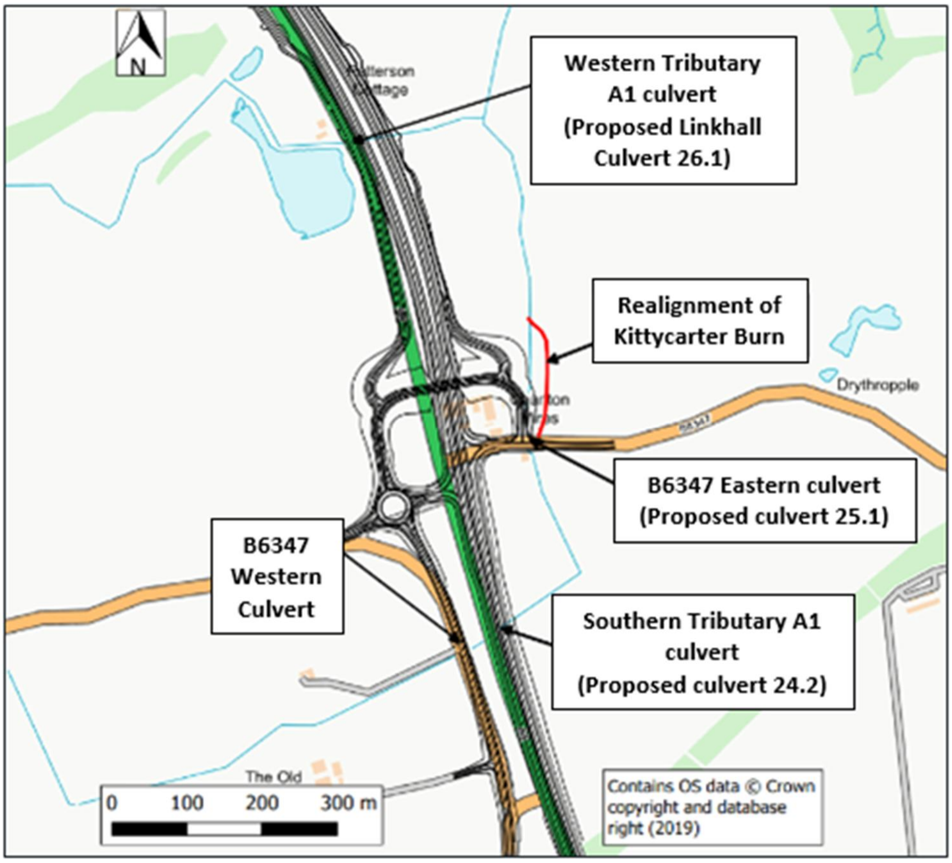

Figure 5-9 Overview of Proposals in Relation to the Tributaries of Kittycarter Burn

Southern tributary A1 culvert (Proposed Culvert 24.2)

5.5.3. The existing culvert located along the southern tributary of Kittycarter Burn at approximately chainage 58600 underneath the A1 alignment would be extended by 26.5 m. The extension would be a new precast concrete circular 600 mm pipe, and the construction of a new headwall and wing wall at the culvert outlet. The length of the extended culvert would be 50 m. The culvert extension would be on the same alignment of the watercourse.

B6347 Eastern Culvert (Proposed Culvert 25.1)

5.5.4. The existing culvert underneath the B6347 would be demolished and replaced with the same dimensions as the existing culvert but would move slightly to the east to tie into the realigned tributary of Kittycarter Burn. The new culvert would be located at chainage 58850, with a circular 600 mm culvert and would be 17 m in length.

Western Tributary A1 Culvert (Proposed Linkhall Culvert 26.1)

5.5.5. The existing western tributary A1 culvert is located at approximately chainage 59275. The culvert would need to be lengthened to accommodate the wider layout of Part B, including an access road to the west of the main carriageway and a slip road on the opposite side, to the east of the carriageway. The proposed new extension to the culvert would comprise a number of precast reinforced concrete box units, which would have an internal width of 1.88 m and height of 2.25 m. The extension of the culvert would have an approximate length of 50.8 m. The total length of the culvert including the length of the retained existing culvert would be 70.9 m. The culvert extension would be on the same alignment of the watercourse.

5.5.6. NCC have raised concerns with the existing A1 culvert on the western tributary causing flooding to a property in this location. The culvert itself is large, however there is a fence running though the centre of it believed to be used to allow passage of farm animals beneath it. The culvert may be prone to blockage and the downstream channel is heavily overgrown with sediment build up observed on the downstream face. **Figure 3-23** in **Section 3.2** of this FRA shows the fence running through the centre of the culvert.

PART B PROPOSALS

5.5.7. The iterative process described in **Table 2-7** to develop a design for the culverts that satisfies both the flood risk and environmental requirements has resulted in proposals for the culvert extensions as set out in **Table 5-5**. **Table 5-5** also details the dimensions of the existing culverts for comparison. The B6347 eastern culvert would be replaced with a culvert of the same dimensions.

Table 5-5 Existing and Proposed Dimensions of Kittycarter Burn Structures

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing southern tributary A1 culvert	25.5	Circular	0.6	-
Proposed southern tributary A1 culvert (Proposed culvert 24.2)	50	Circular	0.6	-
Existing western tributary A1 culvert	20.1	Rectangular	1.88	2.25
Proposed western tributary A1 culvert (Proposed Linkhall culvert 26.1)	70.9	Rectangular	1.88	2.25
Existing B6347 Eastern culvert	15.0	Circular	0.6	-
Proposed B6347 Eastern culvert (Proposed culvert 25.1)	15.0	Circular	0.6	-

DESIGN OUTCOMES

5.5.8. **Table 5-6** provides details of the freeboard associated with each structure for a range of flood events. The tributaries of Kittycarter Burn are ordinary watercourses and as such a design freeboard of 300 mm within the watercourse structure is preferred for the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref. 10.10**). The 1,000-year event is larger than the 100 year + 50 % climate change event so the 1,000 year event has been used to assess risk in an extreme event. Blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 30 % for the western tributary culverts and by 67 % for culvert on the southern tributary reflecting the different sizes of these structures and hence the likelihood of blockage. The negative values in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

5.5.9. **Table 5-6** shows that a freeboard of 300 mm is achieved for the proposed Linkhall culvert in all scenarios including the 1,000-year event and the blockage scenario. A freeboard of 300 mm is not achieved in the 100 year + 25 % climate change for the southern tributary A1 culvert or the B6347 eastern culvert either in the existing or proposed scenarios. The crest level of the carriageway for both the southern tributary A1 culvert and the B6347 eastern culvert (noting the carriageway is 2.81 m and 1.57 m respectively above the inlet soffit) is sufficiently high to prevent overtopping in both of the residual risk 1,000 year and blockage scenarios.

Table 5-6 Design Freeboard for Kittycarter Burn Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage
Existing southern tributary A1 culvert	2.61	-0.11	-0.43	-0.98	-	-0.05	-0.34	-0.57	-
Proposed southern tributary A1 culvert (Proposed culvert 24.2)	2.81	-0.11	-0.38	-0.98	-0.73	-0.01	-0.20	-0.27	-
Existing western tributary A1 culvert	1.85	1.51	0.81	0.67	-	1.46	0.81	0.69	-
Proposed western tributary A1 culvert (Proposed Linkhall culvert 26.1)	1.85	1.58	0.89	0.72	0.79	1.36	0.74	0.61	-
Existing B6347 Eastern culvert	0.74	-0.02	-0.81	-0.88	-	0.02	-0.27	-0.39	-

Proposed B6347 Eastern culvert (Proposed culvert 25.1)	1.57	0.18	-0.69	-0.78	-0.82	0.31	-0.08	-0.16	-

PREDICTED FLOOD RISK IMPACTS

5.5.10. The effect of Part B on upstream water levels and pass forward flows has been reviewed to understand the wider implications of Part B on flood risk.

5.5.11. The key receptor of concern upstream of Part B is a property on the left bank of the western tributary. Elsewhere, the upstream catchments are predominantly rural, and the only other receptor identified is a property located on the right bank at the upstream limit of the southern tributary. There are no receptors in the vicinity of Part B downstream. **Figure 5-10** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part B. **Figure 5-11** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 400 m downstream of the existing A1 alignment.

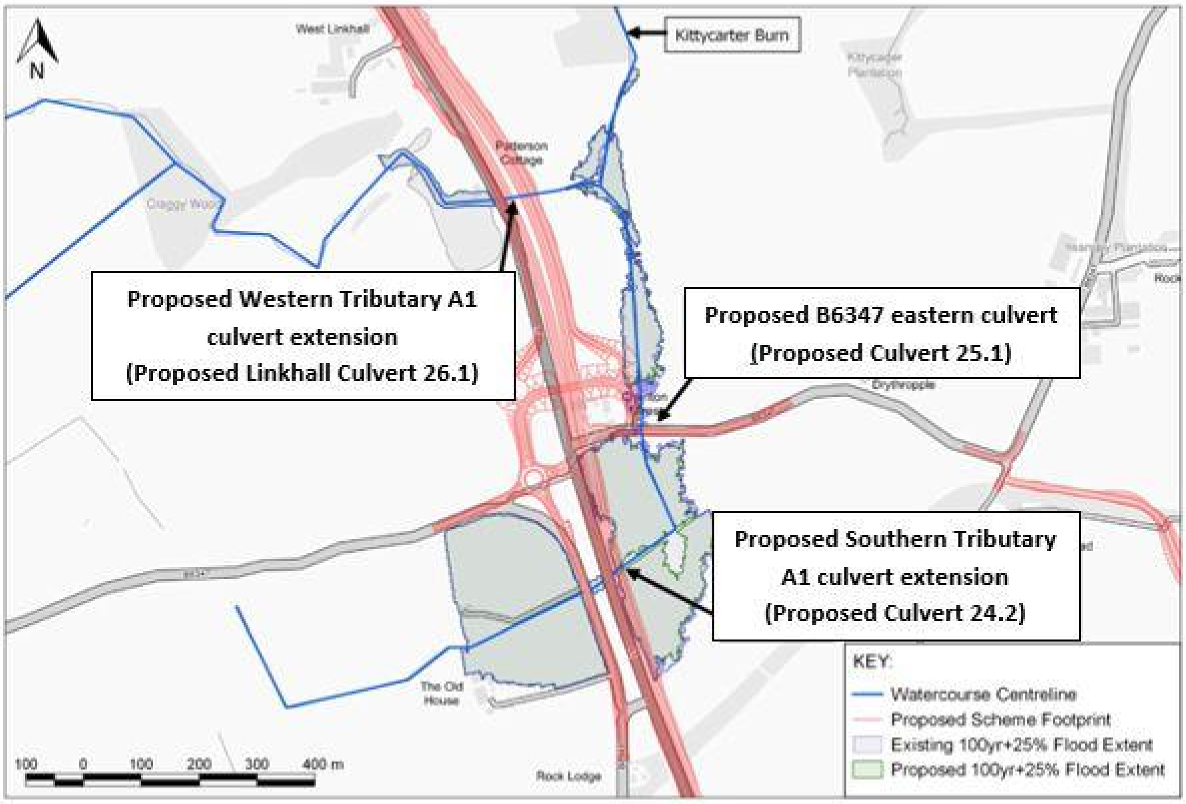

Figure 5-10 Flood Extents in the Existing and Proposed Design for the 100 Year + 25 % Climate Change Event

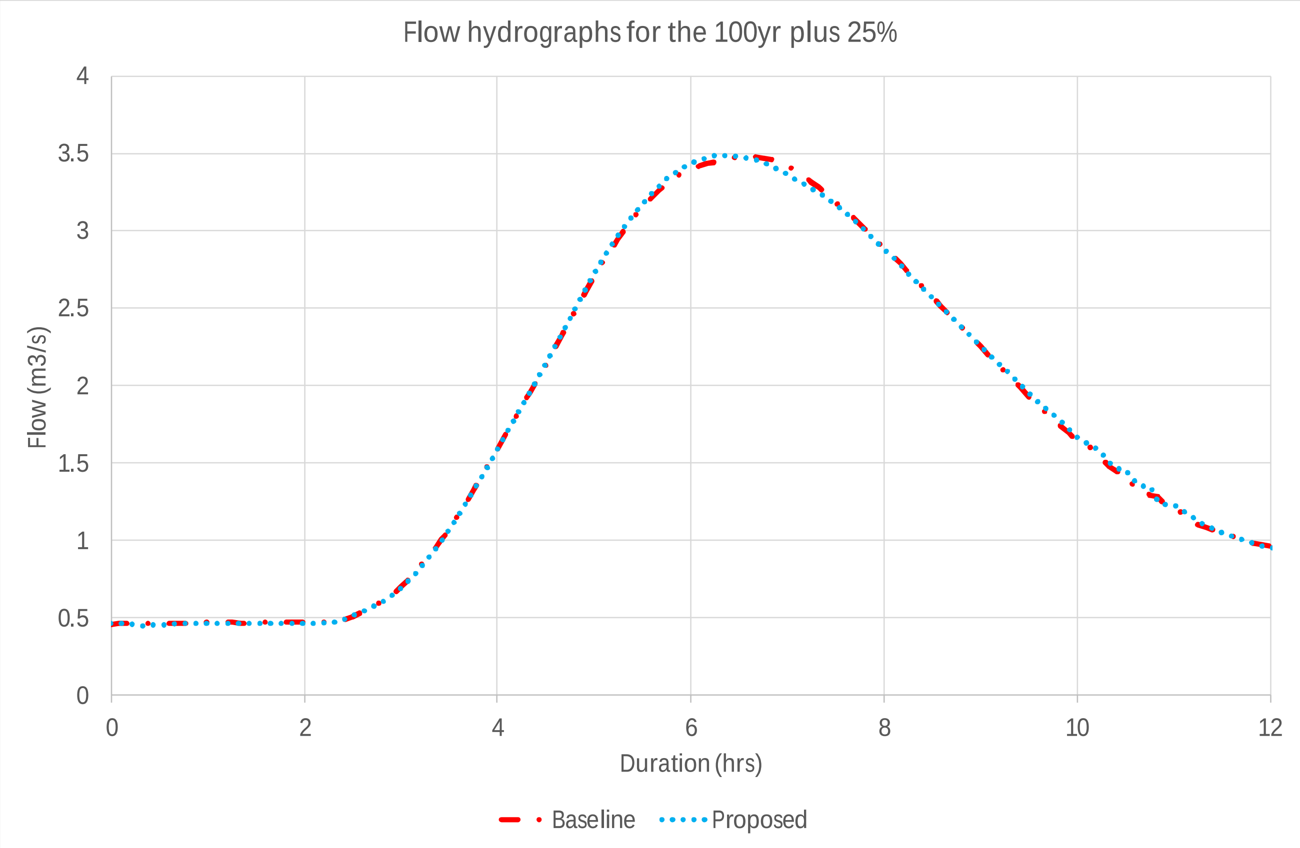

Figure 5-11 Pass Forward Flows in the Existing and Proposed Scenarios for the 100 Year + 25 % Climate Change Event

5.5.12. **Figure 5-10** demonstrates that Part B would result in a small decrease in water levels upstream of the eastern B6347 culvert as the increased capacity of the downstream channel reduces downstream water levels. There is however a reduction in flows associated with Part B as the raised crest level of the road prevents overtopping and so attenuates more flow.

5.5.13. There is also an increase in downstream flows from the western tributary. This is a result of the increased culvert length improving the flow of the channel and resulting in a marginal reduction in water levels upstream. In the baseline scenario flows overtop the right bank of the western tributray upstream of the A1. The reduction in water levels caused by Part B means a resulting increase in downstream flows as less flow overtops the right bank. It is the increase in flows on the western tributary that contribute to the overall increase in downstream flows shown in **Figure 5-11**.

5.5.14. As discussed previously, NCC has raised concerns associated with the existing fence located in the centre of the culvert that may increase the risk of flooding to the property on the left bank of the western tributary upstream of the A1. A review of the operation of this culvert confirms that it is of sufficient size to convey the design flood without a significant headloss across the structure. Whilst blockage of the culvert is predicted to increase flood levels by approximatly 100 mm, the retention of the fence in the extended culvert is not predicted to exacerbate the existing flood risk to the adjcent property. Removal of the fence is therefore not required to mitigate the effects of Part B. Similarly extension of the culvert on the downstream face would necessarily require clearance of sediment and vegetation to construct. This could be undertaken during detailed design.

5.5.15. Part B would lower the right bank level upstream of the A1 on the western tributary. The area on the right bank in this location is seasonally wet and some minor lowering would result in the reestablishment of the existing overtopping frequency and offset the increase in flows from the western tributary. Further lowering would increase the frequency of overtopping and would be sufficient to deliver a net benefit to downstream flows. In addition to the lowering of the right bank, any low spots identified along the left bank would be raised. These improvements are agreed in principle with the landowers and further detailed modelling.

5.6. SHIPPERTON BURN

OVERVIEW OF PART B REQUIREMENTS

5.6.1. Shipperton Burn drains a predominantly rural catchment of approximately 2.9 km^2 to Part B. There are no flood risk receptors upstream of Part B but immediately downstream is a farmhouse on the left bank of the watercourse.

5.6.2. An overview of Part B in relation to Shipperton Burn is provided in **Figure 5-12**. The existing Shipperton Bridge would be retained as part of Part B with no works proposed.

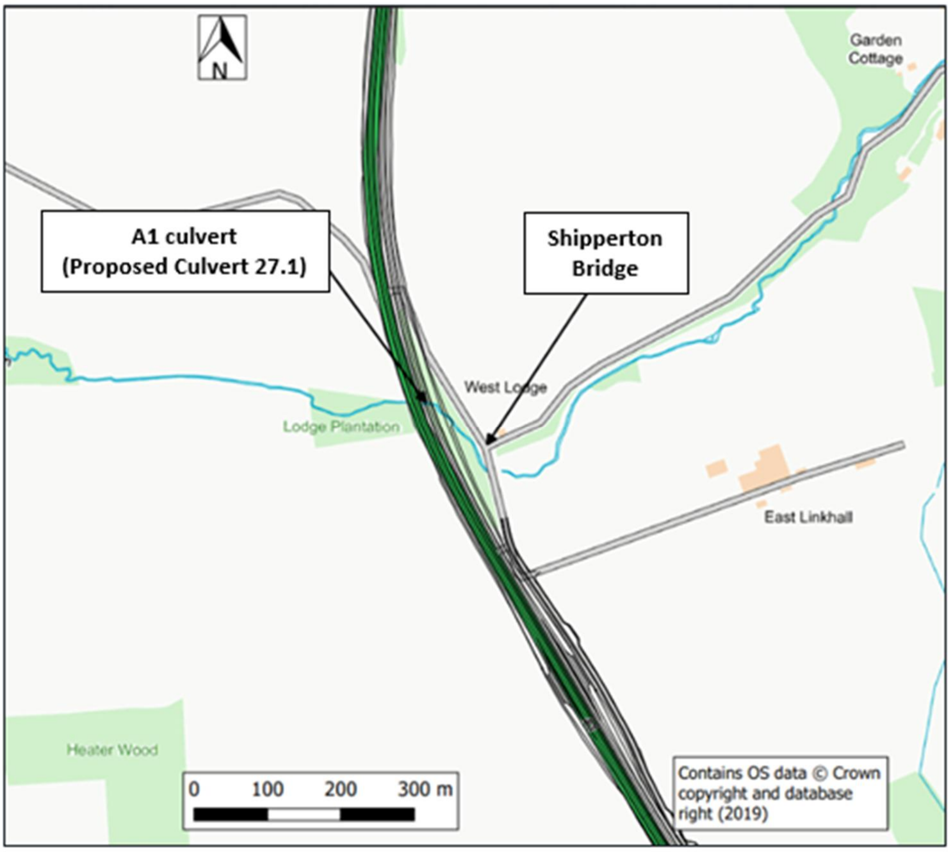

Figure 5-12 Overview of Proposals in Relation to Shipperton Burn

5.6.3. The proposed works at Shipperton Burn consist of an extension of the existing A1 culvert eastwards on the outlet side by 27.65 m. The proposed new extension would be a precast reinforced concrete box with internal 2 m width and 1.25 m height.

PART B PROPOSALS

5.6.4. The proposals for the extended culvert are set out in **Table 5-7**. **Table 5-7** also details the dimensions of the existing Shipperton Bridge and A1 Culvert for comparison.

Table 5-7 Existing and Proposed Dimensions of Shipperton Burn Structures

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing Shipperton Bridge	21	Rectangular	1.89	1.08
Existing A1 culvert	19.1	Rectangular	2.05	1.29
Proposed A1 culvert (Proposed culvert 27.1)	46.75	Rectangular	2	1.25

DESIGN OUTCOMES

5.6.5. **Table 5-8** provides details of the freeboard associated with each structure for a range of flood events. Shipperton Burn is an ordinary watercourse and as such a design freeboard of 300 mm within the watercourse structure is preferred in the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref. 10.10**). The 1,000-year event is larger than the 100 year + 50 % climate change event so has been used to assess risk in an extreme event. Given the size of the proposed structure, blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 30 %. The negative values in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

5.6.6. **Table 5-8** also shows that maintaining and extending the existing culvert would result in no change to the existing freeboard of the structure in the 100 year + 25 % climate change event; the structure is currently surcharging. The results confirm that Part B is not predicted to overtop in the residual risk scenarios associated with the 1,000-year event flows and the blockage scenario.

Table 5-8 Design Freeboard for Shipperton Burn Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1,000 year	100 year + 25 % cc with
Existing A1	1.23	0.72	-0.05	-0.43	-	0.86	0.35	0.17	-
Proposed A1 culvert (Proposed culvert 27.1)	1.23	0.72	-0.05	-0.43	-0.01	0.86	0.39	0.23	-

PREDICTED FLOOD RISK IMPACTS

5.6.7. The effect of Part B on upstream water levels and pass forward flows has been reviewed to understand the wider implications of Part B on flood risk.

5.6.8. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. **Figure 5-13** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part B. It compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 400 m downstream of Part B.

5.6.9. **Figure 5-13** demonstrates that there are no discernable changes in the water levels in the vicinity of the A1 culvert and the same is refelcted in **Figure 5-14**.

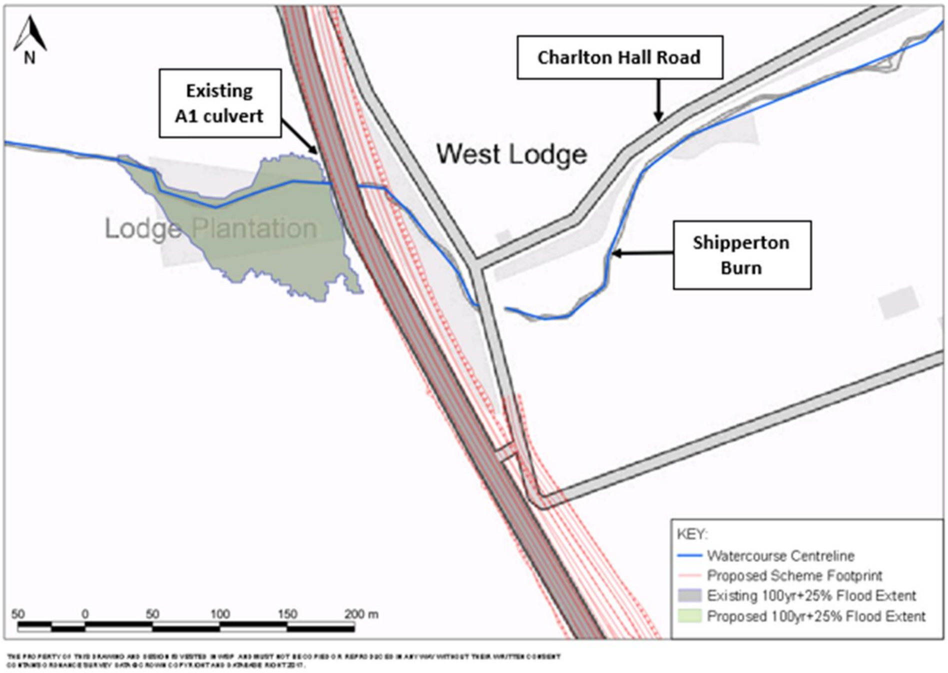

Figure 5-13 Flood Extents in the Existing and Proposed Design for the 100 Year + 25 % Climate Change Event

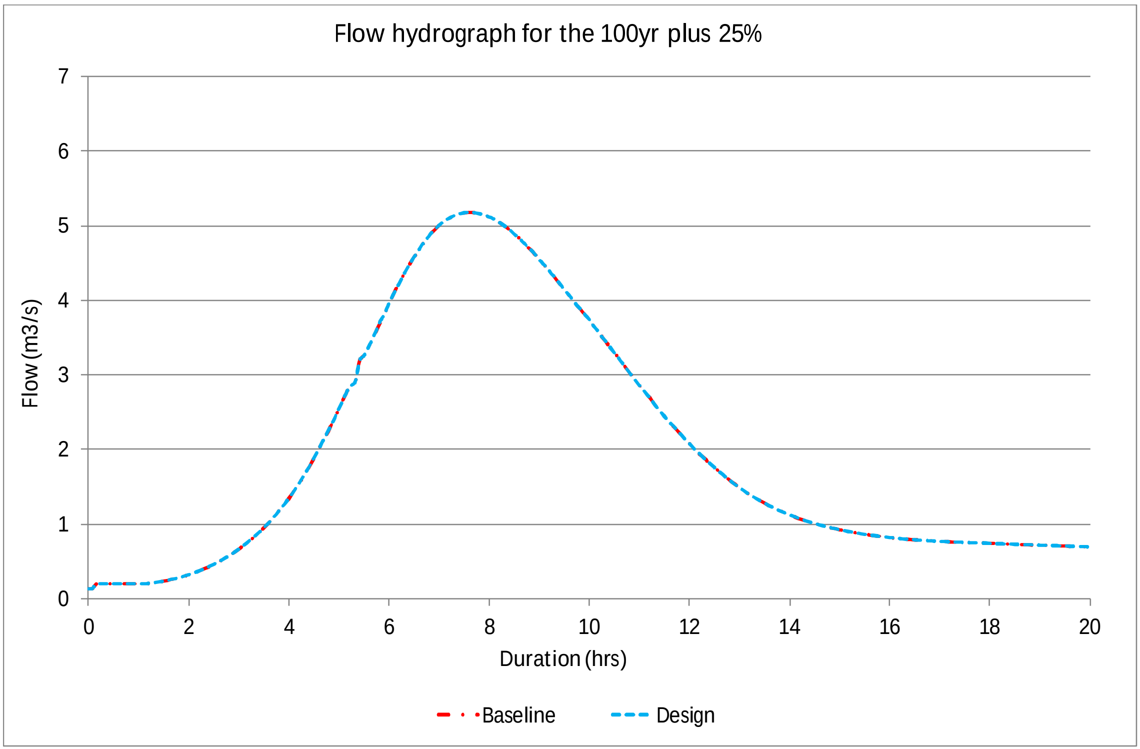

Figure 5-14 Pass Forward Flows in the Existing and Proposed Scenarios for the 100 Year + 25 % Climate Change Event

5.7. MINOR WATERCOURSES AND SURFACE WATER FLOW PATHS

5.7.1. Part B crosses several minor watercourses and surface water flow paths which are not discussed in the sections above. These are summarised in **Table 5-9** below, which includes information on whether the culverts are existing or new.

5.7.2. Details on the culvert modelling can be found in **Appendix B: Culvert Master Analysis** of this FRA.

Table 5-9 Summary of the Minor Watercourses, Drainage Ditches and Identified Surface Water Flow Paths Crossed by Part B

Structure Name or Description	Scheme Description	Structure Dimensions	Freeboard in the 100 year +25 % event	Freeboard in the 1,000 year event	Comments
Existing A1 culvert 3 along Denwick Burn	Part B would widen the existing A1 in this location and the proposal would extend the existing pipe to manhole due east of A1 mainline.	Shape: Circular Diameter: 0.6 m Length: 21.25 m Inlet Soffit to Crest: 0.924 m	Inlet: -0.88 m Outlet: 0.00 m	Inlet: -0.95 m Outlet: -0.02 m	Existing structure is submerged however the highway crest level is not overtopped during a 100 year + 25 % climate change design flow. The highway crest is overtopped in the 1,000-year event.
Proposed A1 culvert 3 (Proposed culvert 19.1) along Denwick Burn		Shape: Circular Diameter: 0.6 m Length:35.95 m Inlet Soffit to Crest: 0.954 m	Inlet: -0.59 m Outlet: 0.0 m	Inlet: -0.94 m Outlet: -0.07 m	Structure is submerged during both 100 year + 25 % climate change and 1,000-year design flows but does not overtop.
Existing Heckley Fence culvert	Part B would include an accommodation overbridge at Heckley Fence. As previously described, at this location there is a culvert beneath the A1 which collects surface water runoff from the east of the A1 and discharges into the Denwick Burn via a culvert which runs parallel to the A1 southwards. A small drainage ditch is located to the west of the A1 at Heckley Fence. Part B would ensure the hydraulic connectivity of the drainage ditch and the surface water runoff. The details regarding the design of the overbridge in relation to the culverts would be confirmed during detailed design with additional survey information on the existing culvert dimensions.	Shape: Circular Diameter: 0.3 m (assumed) Length: 36.62 m Inlet Soffit to Crest (assumed): 2.36 m	Inlet: -2.41 m Outlet: -0.06 m	Inlet: -2.43 m Outlet: -0.06 m	Structure overtops during both 100 year + 25 % climate change and 1,000-year design flows.
Proposed Heckley Fence culvert (Proposed culvert 22.1)		Shape: Circular Diameter: 0.3 m (assumed) Length: 44.62 m Inlet Soffit to Crest (assumed): 2.83 m	Inlet: - 2.87 m Outlet: 0.0 m	Inlet: - 2.88 m Outlet: 0.0 m	Structure overtops during both 100 year + 25 % climate change and 1,000-year design flows.
Existing access track watercourse crossing along the tributary of Embleton Burn	Part B would provide a permanent means of access to Middlemoor Cottage and its neighbouring properties. A new culvert would be constructed upstream of the existing access track watercourse crossing to accommodate the permanent access road. A weir would be placed upstream of culvert inlet with top level matching existing masonry wall parapet level, to be overtopped as per existing carriageway. Weir would have orifice approximating existing culvert cross sectional area for low flows. Main structure would be sized to take combined orifice and weir flow.	Shape: Rectangular Span: 0.45 m Rise: 0.316 m Length: 5.747 m Inlet Soffit to Crest: 1.767 m (top of masonry wall parapet).	Inlet: -2.047 m Outlet: -1.91 m	Inlet: -2.107 m Outlet: -1.91 m	Structure is in a poor state of maintenance and submerged at time of visit. Modelling shows that 100 year + 25 % climate change and 1,000-year design flows would overtop the parapet.
Proposed Rock culvert (Proposed culvert 28.1) along the		Shape: Circular Diameter: 1.2 m	Inlet: 0.31 m Outlet: 0.42 m	Inlet: 0.0 m Outlet: 0.0 m	Structure is sufficiently large to convey the 100 year + 25 % climate change with free flow and the 1,000-year design flow with a submerged inlet.

Structure Name or Description	Scheme Description	Structure Dimensions	Freeboard in the 100 year +25 % event	Freeboard in the 1,000 year event	Comments
tributary of Embleton Burn		Inlet Soffit to Crest: 1.16 m			Provision of upstream weir/orifice maintains existing upstream and downstream flood risk.

5.8. INCREASE IN SURFACE WATER RUNOFF RATE AND VOLUME

5.8.1. A detailed description of the surface water drainage strategy is provided in **Appendix 10.4: Drainage Strategy Report** of this ES and is summarised below:

a. Runoff from Part B is discharged into the existing watercourses via grassed detention basins where required.
b. Allowable runoff rates are restricted to the existing greenfield runoff values for the equivalent storm event.
c. Highway drainage is designed to accommodate a 1 in 1-year design flow without surcharging; and a 1 in 5-year flow without surface flooding of the running carriageways (with a 20 % allowance for climate change).
d. Attenuation controls would be provided for the 1 in 1, 30- and 100-year events plus climate change.
e. Where grassed detention basins are used for attenuation these are located outside of Flood Zone 2 and 3 areas and the modelled flood extents produced for this assessment.
f. Detention basins would be lined, therefore there is no impact to groundwater ingress that might increase flood risk.
g. Online controls would be provided to restrict discharges to allowable values.
h. It is assumed that any new local access tracks, bridleways and private means of access are drained to local land drains and watercourses.
i. Runoff from the running lanes and hardstrips would follow the road camber to both channels, and the central reservation where there is a crossfall.
j. Runoff to the central reservation would be to concrete V-channels.
k. Where the highway is to be within a cutting the runoff from the cutting would be to the single filter drain at either side of the highway, except in one location where a surface water channel is proposed.
l. Where the highway is to be within a cutting it is proposed that the field runoff would be taken by a cut-off ditch at the top of the cutting slope and would discharge through private ditches, etc. and would not contribute to the highway drainage network.
m. As there is a requirement (further to the HAWRAT assessment) to provide treatment prior to discharge to many of the watercourses, a permanent wet shallow area is required in the detention basins. The size and depth of this permanently wetted area is envisaged to be a small part of the overall basins, and this would be confirmed in the detailed design.

5.8.2. The grassed detention basins locations have been informed by the hydraulic modelling undertaken as part of this assessment. A sequential approach has been undertaken to ensure that the detention basins are not located in the 1 in 100 year + 25 % climate change allowance flood extent.

5.9. FLOOD RISK DURING CONSTRUCTION

5.9.1. The **Outline Construction Environmental Management Plan (Outline CEMP)** (**Application Document Reference: TR010041/APP/7.3**) would set out the measures for managing flood risks during construction. Measures would include:

a. Ensuring that flood conveyance routes are maintained during construction.
b. Moving any plant away from the banks of watercourses following heavy rainfall events.
c. Monitoring of Environment Agency's flood warnings.
d. Creating safe working areas for the storage of plant and materials if a flood warning is received during construction.

5.9.2. The proposed construction compound at Charlton Mires would be located adjacent to the southern tributary of Kittycarter Burn. Although this is an area located in Flood Zone 1, our modelling has identified it to be at fluvial flood risk. The compound is temporary and would be in place for approximately 17 months. The compound would be located on land above 86.8 mAOD (the 1 in 25 annual probability event flood level). No bunding is proposed around the compound to allow the compound to flood to reduce the volume of temporarily displaced floodplain. This risk is considered to be acceptable due to the temporary nature of the compound and no vulnerable flood risk receptors located in the surrounding area.

5.10. RESIDUAL FLOOD RISK

5.10.1. The residual flood risk associated with Part B watercourse crossings, culverts and identified surface water flow paths (detailed above in **Sections 5.3** to **5.7**) has been investigated through the following:

a. The residual risks associated with an increase in flow has been assessed using the 1,000-year event.
b. The residual risks associated with a decrease in structure capacity has been assessed using either 30 % or 60 % blockage (dependent on the size of the structure).

5.10.2. During a 1,000-year flood event, no watercourse crossing, culvert or surface water flow path overtops the highway crest. Blockage scenarios on the watercourse crossings show that the highway crest is not overtopped when the inlet capacity is reduced. Regular maintenance should ensure that residual flood risk from any watercourse crossing, culvert or surface water flow path is minimal and no further flood risk mitigation measures are considered necessary.

6. CONCLUSION

6.1.1. A review of the Environment Agency Flood Map for Planning (Rivers and Sea) indicates that the alignment in the Part B Main Scheme Area is located in the low-risk Flood Zone 1. However, within the Order Limits of Part B there are two areas located within the medium risk Flood Zone 2, and the high-risk Flood Zone 3. There is one area located to the south within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Denwick Burn. The other area is located to the north within the Part B Main Scheme Area and the identified fluvial flood risk is associated with Shipperton Burn.

6.1.2. Part B crosses five watercourses and associated tributaries (listed from south to north): Denwick Burn and its tributaries; White House Burn; two tributaries of Kittycarter Burn; tributary of Embleton Burn; and Shipperton Burn.

6.1.3. Detailed 1D hydraulic modelling has been undertaken for Denwick Burn and its tributaries, White House Burn, tributaries of Kittycarter Burn, and Shipperton Burn. Hydraulic assessment using Culvert Master has been undertaken for the other watercourses and surface water flow paths. The modelling shows that there would be no increase in fluvial flood risk to any upstream or downstream receptors or to Part B.

6.1.4. A review of the Environment Agency's Flood Risk from Surface Water map indicates that sections of Part B are at high, medium and low risk of flooding from surface water sources. Existing surface water flow paths have been incorporated into Part B.

6.1.5. Groundwater levels are estimated to be between 1 - 3.5 m bgl however flood risk from groundwater is considered to be low and the risk of high groundwater affecting drainage capacity has been considered.

6.1.6. The proposed drainage strategy restricts surface water runoff rates to the existing greenfield runoff values for the equivalent storm event. Highway drainage would be designed to accommodate a 1 in 1-year design flow without surcharging and a 1 in 5 year flow without surface flooding of the running carriageways (with a 20 % allowance for climate change). Attenuation controls would be provided for the 1 in 1, 30 and 100 year plus climate change scenarios.

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Appendix A

HYDRAULIC MODELLING ANALYSIS

Project	A1 - Northumberland
Job Number	70044137
Location	Heckley House, Alnwick, Northumberland, England (419287 616573)
Watercourse(s)	Denwick Burn
1. Objectives/Areas of interest	

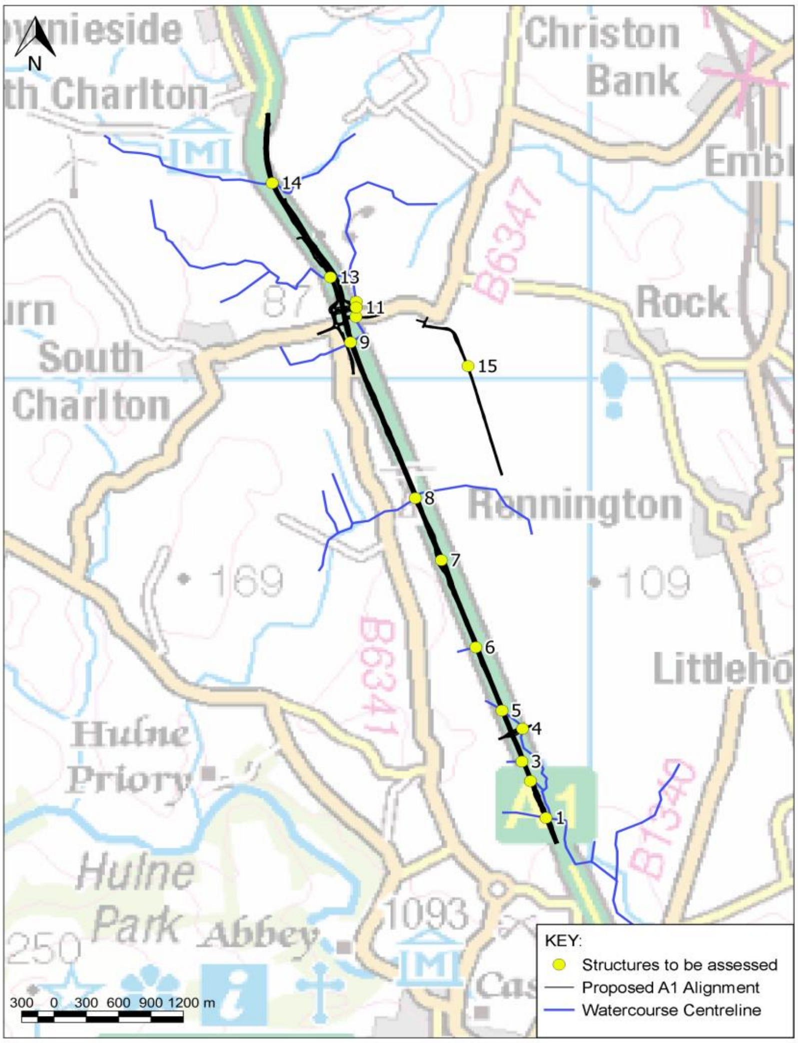


Figure 1: Location of structures in overall scheme

Highways England (HE) has identified the need to improve the existing A1 in Northumberland between Denwick and North Charlton. The Scheme is approximately 8 km in length and comprises online improvements consisting of carriageway widening.

Figure 1 above shows the location of various structures along the scheme. There are 15 culverts in total across the site which are to be assessed. The purpose of the assessment is to understand the impacts on flood risk and ecology as a result of the works.

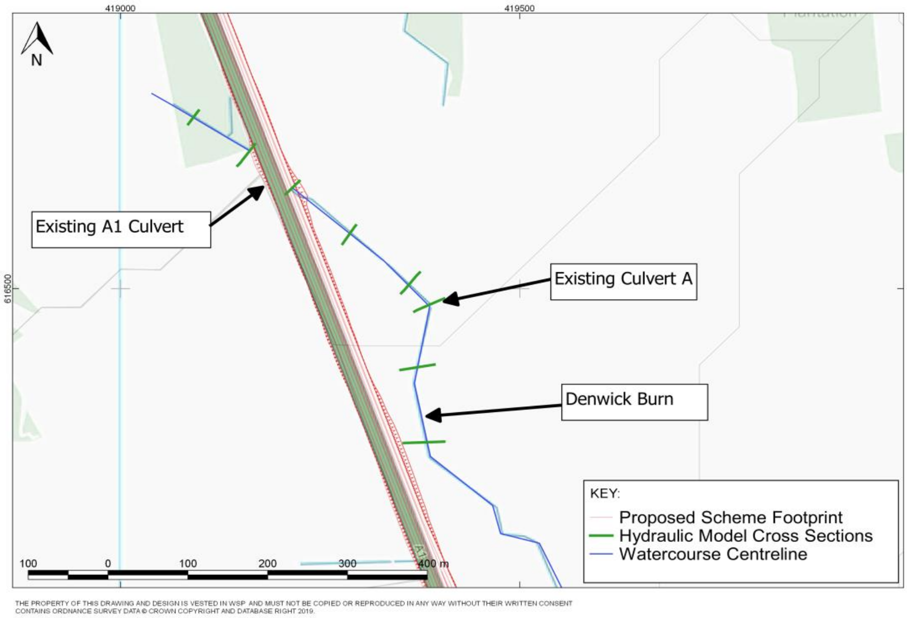


Figure 2: Location of structures on Denwick Burn

This report relates to the proposed works on the Denwick Burn. There are two existing structures within the reach. These are the existing A1 culvert and a second access bridge (labelled A) located east of the A1. Further details on how each of these structures were modelled is presented within Appendix A of this report.

2. Model Input Data

Title	Type	Notes
A2E XS19.xlsx	Topographic Survey	Detailed topographic survey of area around the Denwick Burn and existing structures. Surveyed information includes channel, bank, bed, flood plain, existing bridge deck, existing bridge soffit, existing bridge parapet. Data has been used to build 1D model.

3. GIS Data

OS Tiles -	Source:	OS Open Map Local downloaded from OS OpenData website
Ground Level Data -	Resolution: Date :	2m Digital Surface Model Data and 5m Digital Terrain Model Data Photogrammetric Digital Models from supplied https://apgb.blueskymapshop.com in November 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 580m in length. The watercourse is the Denwick Burn.

There are two existing structures within the reach. These structures were modelled in 1D domain using FMP structure nodes. The data for these structures was obtained from the topographic survey received from the surveyor. Further details on how each of these structures were modelled is presented within Appendix A of this report.

Model cross sections have been extended into the floodplain to accommodate the design flows. Local ground level data are available from 2m DSM tiles. These cover the entire extent of the model.

To confirm the agreement between the river section survey and the photogrammetry data, a comparison was done at out of bank sections. A histogram detailing the frequency curve of the difference between these two data sets is shown in Figure 3.

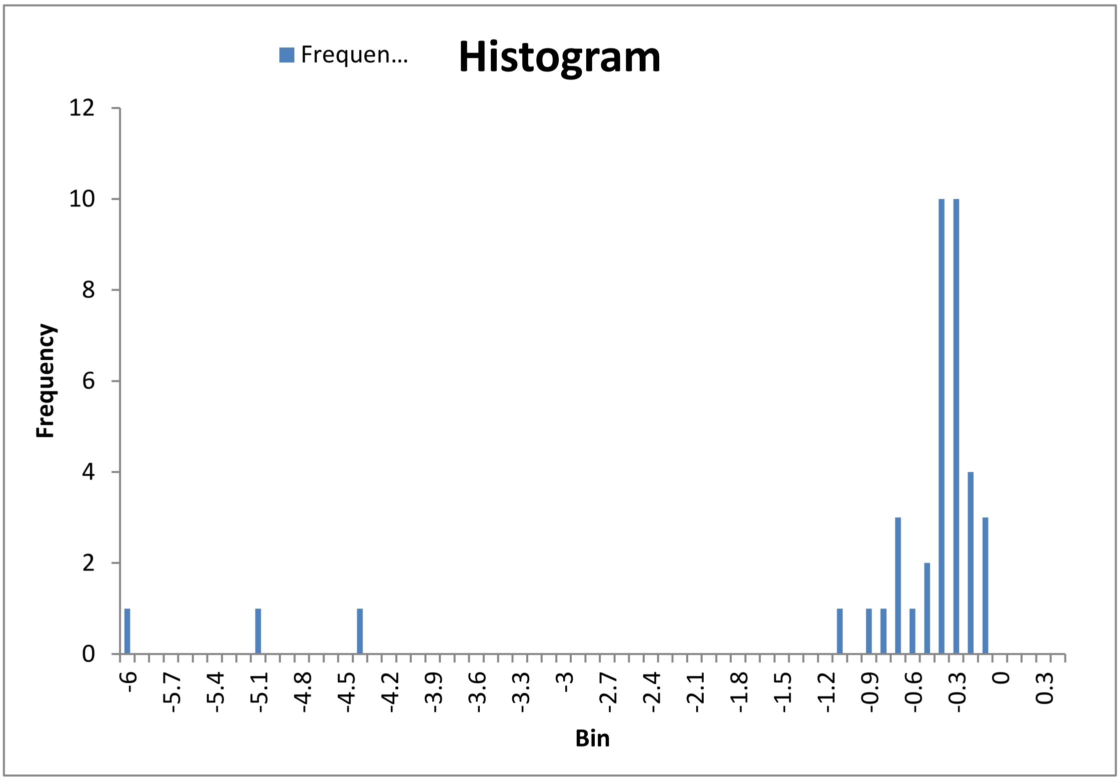


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift of 0.4m in the photogrammetry data was applied.

5. Model Setup

Model Method	1D
Software	FMP (4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using available photogrammetry data

Run Settings	Run parameters - Unsteady simulation, Single Precision, Cold Start with initial conditions for 1D domain.
Other comments	

6. Model inflows and Boundary Conditions

Peak flow estimates have been derived at 2 locations for the Denwick Burn model. These are on upstream of the A1 in the vicinity of the proposed new road (DB_01) and at the downstream limit of the model (DB_02). The design flow estimates have been developed using the ReFH2 methodology and are shown in the table below. Full details of the calculations and the justification for this approach are provided in the FEH calculation record.

Flow Node	Annual Probability Event									
	2	5	10	25	50	75	100	200	1000	100+25%
DB_01	0.87	1.19	1.43	1.78	2.07	2.26	2.41	2.81	4.07	3.01
DB_02	1.82	2.51	3.01	3.72	4.33	4.74	5.05	5.90	8.54	6.32

The flows from DB_01 have been applied at the upstream limit of the model and an additional inflow has been applied at cross section XS19_02. The additional inflow has been derived by subtracting the hydrograph of DB_01 from DB_02.

The downstream boundary of the 1D FMP model has been defined as a normal depth boundary using a bed slope.

7. Manning's 'n' Roughness Coefficients

The Manning's roughness coefficient values used in the river sections were derived from the information provided in the topo survey and the site photographs. The Manning's n values utilised have been listed within Appendix B of this report.

8. Model Calibration and Verification

No data was available with which to calibrate the model. The results have therefore been sensibility checked for model stability and appropriateness using engineering judgement only.

9. Proposed Model Development

Based on the proposed A1 road alignment, the proposals will extend the existing culvert beneath the A1 on the downstream side only. Cross section data for the upstream and downstream faces of this structure was derived from the topographic survey provided by the surveyor. It is not proposed to realign the watercourse and the culvert crossing will tie in with the existing alignment at the downstream face.

The existing culvert at XS19_04 beneath the A1 is free of sediment and has a small weir at the downstream that will prevent silt backing up into the culvert. This existing culvert has been extended on the outlet side by 29m using the same dimensions. Invert levels are extrapolated based on the existing culvert gradient. The resulting downstream channel gradient compares well with the gradient of XS19_02 and XS19_01.

10. Model Runs

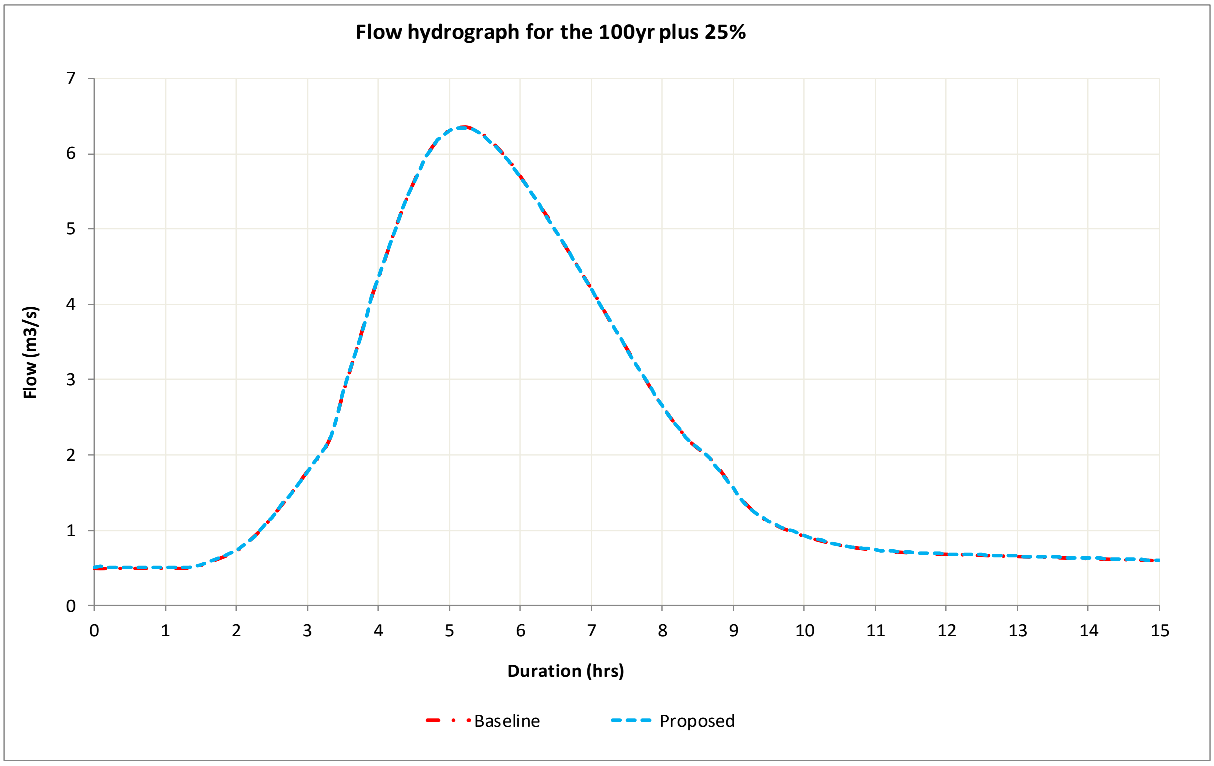
Model Scenario	Return Periods / Events
Baseline (BSC)	2yr, 100yr+25%, 1000yr
Proposed Scenario (PRO)	2yr, 100yr+25%, 1000yr, 100yr+25% with blockage

11. Model Results

The following table provides details of the freeboard associated with each structure for a range of flood events. Denwick Burn is an ordinary watercourse. As such a design freeboard of 300mm is preferred in the 100yr+25% climate change event in accordance with the recommendations in the DMRB. The 1000yr event is larger than the 100yr+50% climate change event so has been used to assess residual risk in an extreme event. Given the size of the proposed structures, blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 67%. The negative value in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

Structure	Carriageway Freeboard above Inlet Soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2yr	100yr +25% cc	1000 yr	100yr +25% cc with blockage	2yr	100 yr +25 % cc	1000 yr	100yr +25% cc with blockage
Existing A1 culvert	2.34	0.48	-0.58	-1.10	-	0.71	0.12	-0.07	-
Proposed A1 culvert	2.36	0.48	-0.58	-1.35	-1.66	0.65	0.17	-0.01	-
Existing culvert A	0.13	-0.28	-0.78	-0.86	-	0.22	-0.14	-0.26	-

To understand the implications of flood risk downstream of the culvert, a flow hydrograph has been compared between the existing and proposed models at the downstream boundary of the model. There is no downstream impact on flows associated with the proposals.


Flow hydrograph for the 100yr plus 25%
Flow (m3/s)
0
1
2
3
4
5
6
7
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Duration (hrs)
Baseline
Proposed

Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Photo	Dimensions	Modelling Approach
1 (XS19_04)	Culvert crossing the Denwick Burn on A1.		The culvert length is 72.3m. The culvert consists of a single circular conduit of diameter 1.18m. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a single circular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.
2 (XS12)	Bridge crossing Denwick Burn on the east of A1.	Not available	The bridge dimensions have been taken directly from the data provided by surveyor.	The bridge has been modelled using arch bridge unit within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.

Proposed Model (PRO)

1 (XS19_04)	Proposed culvert on the new A1 alignment crossing the Denwick Burn.		The proposed culvert length is 101.3m with 1.18m conduit diameter. Structure dimensions have been taken directly from the provided structure schedule.	The culvert has been modelled as a single circular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.

Appendix B. 1D Channel Roughness

The following table summarises the Manning's n values applied to the river channel

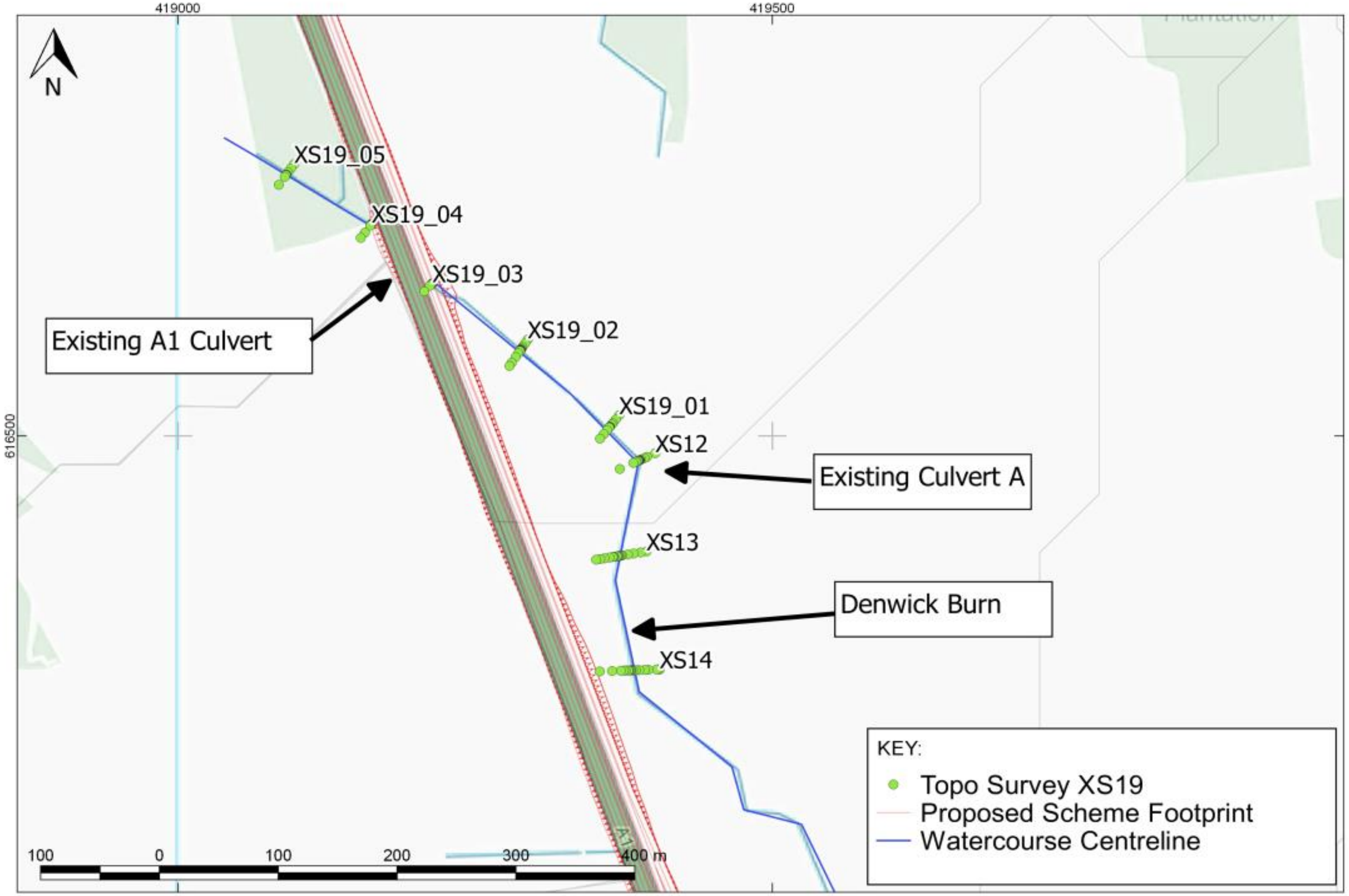
Reach	**Manning's Roughness**	**Description of typical reach cover**	**Typical photo**
XS19_05 to XS14 Tortuosity: Low			 
Left Bank	0.06	Grass/ Long Grass & Brambles	
Channel	0.04	Silt / Gravel	
Right Bank	0.06	Grass	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result Files
Baseline Scenario						
DB	Baseline	2yr	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_2yr.ief	A1_Northumberland_Denwick_Burn_XS19_v08.dat	A1_NORTHUMBERLAND_DENWICK_BURN_XS19_V08_2YR
DB	Baseline	100yr	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_100yr.ief	A1_Northumberland_Denwick_Burn_XS19_v08.dat	A1_NORTHUMBERLAND_DENWICK_BURN_XS19_V08_100YR
DB	Baseline	100yr+25%	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_100yr+25%CC.ief	A1_Northumberland_Denwick_Burn_XS19_v08.dat	A1_NORTHUMBERLAND_DENWICK_BURN_XS19_V08_100YR+25%CC
DB	Baseline	1000yr	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_1000yr.ief	A1_Northumberland_Denwick_Burn_XS19_v08.dat	A1_NORTHUMBERLAND_DENWICK_BURN_XS19_V08_1000YR
Design Scenario						
DB	Proposed	2yr	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04_2yr.ief	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04.dat	DENWICK_BURN_XS19_V08_DESIGN_V04_2YR
DB	Proposed	100yr	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04_100yr.ief	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04.dat	DENWICK_BURN_XS19_V08_DESIGN_V04_100YR
DB	Proposed	100yr+25%	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04_100yr+25%CC.ief	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04.dat	DENWICK_BURN_XS19_V08_DESIGN_V04_100YR+25%CC
DB	Proposed	1000yr	v4.4	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04_1000yr.ief	A1_Northumberland_Denwick_Burn_XS19_v08_Design_v04.dat	DENWICK_BURN_XS19_V08_DESIGN_V04_1000YR
DB	Blockage	100yr+25%	v4.4	Denwick_Burn_v08_Design_v04_Blockage_XS19_04.ief	Denwick_Burn_v08_Design_v04_Blockage_XS19_04.dat	DENWICK_BURN_V08_DESIGN_V04_BLOCKAGE_XS19_04

Appendix D. Model Schematics

Baseline Model (BSC)



Project	A1 - Northumberland
Job Number	70044137
Location	Alnwick, Northumberland, England (418392 618808)
Watercourse(s)	White House Burn

1. Objectives/Areas of interest

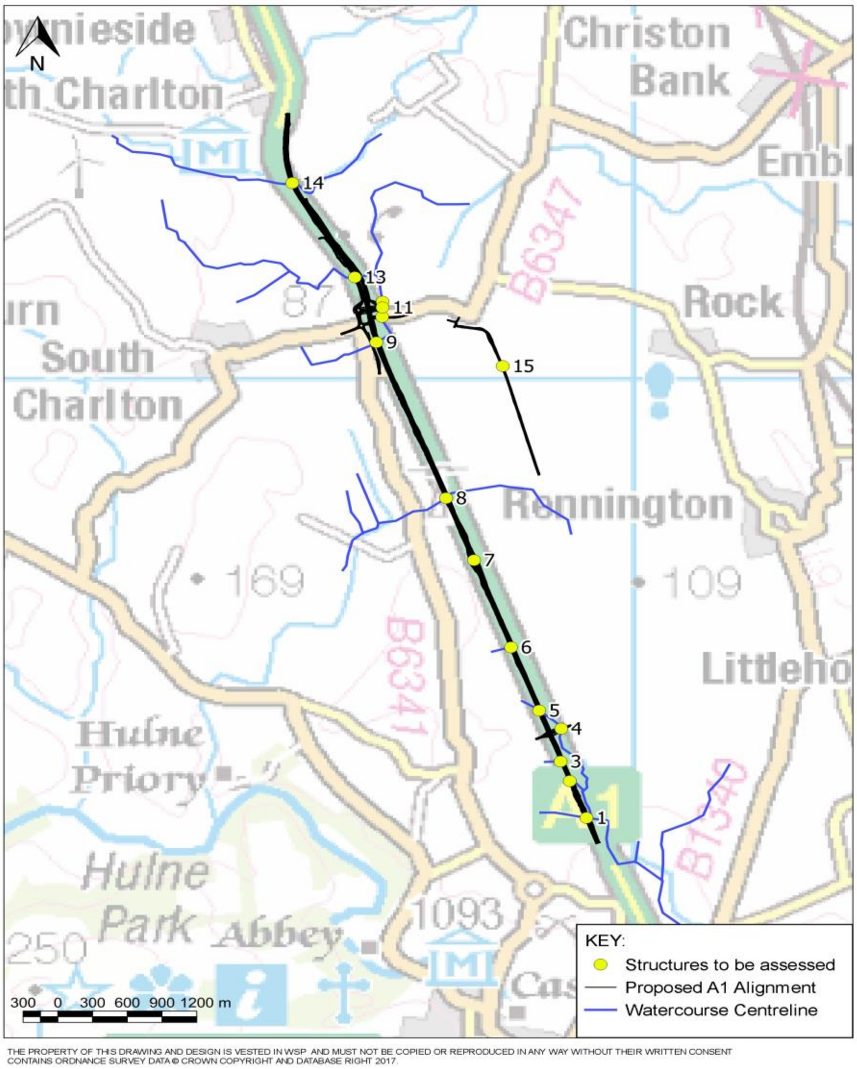


Figure 1: Location of structures in overall scheme

Highways England (HE) has identified the need to improve the existing A1 in Northumberland between Denwick and North Charlton. The Scheme is approximately 8 km in length and comprises online improvements consisting of carriageway widening.

Figure 1 above shows the location of various structures along the scheme. There are 15 culverts in total across the site which are to be assessed. The purpose of the assessment is to understand the impacts on flood risk and ecology as a result of the works.

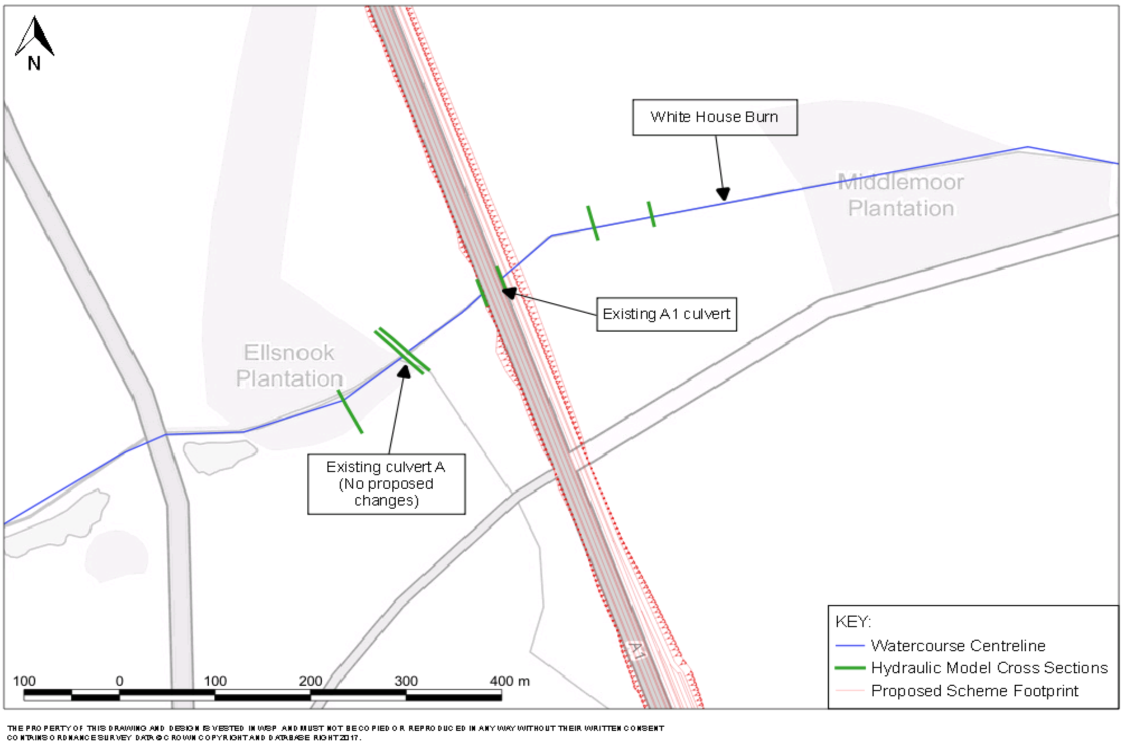


Figure 2: Location of structures on White House Burn

This report relates to the proposed works on the White House Burn. There are two existing structures within the reach. These are the existing A1 culvert and a small access bridge located approximately 80m downstream. Further details on how each of these structures were modelled is presented within Appendix A of this report.

2. Model Input Data

Title	Type	Notes
A2E XS20.xlsx	Topographic Survey	Detailed topographic survey of area around the White House Burn and existing structures. Surveyed information includes channel, bank, bed, flood plain, existing bridge deck, existing bridge soffit, existing bridge parapet. Data has been used to build 1D model.

3. GIS Data

OS Tiles -	Source:	OS Open Map Local downloaded from OS OpenData website
Ground Level Data -	Resolution: Date :	2m Digital Surface Model Data and 5m Digital Terrain Model Data Photogrammetric Digital Models from supplied https://apgb.blueskymapshop.com in November 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 325m in length. The watercourse is the White House burn.

There are two existing structures within the reach. These structures were modelled in 1D domain using FMP structure nodes. The data for these structures was obtained from the topographic survey received from the surveyor. Further details on how each of these structures were modelled is presented within Appendix A of this report.

Model cross sections have been extended into the floodplain to accommodate the design flows. Local ground level data are available from 2m DSM tiles. These cover the entire extent of the model.

To confirm the agreement between the river section survey and the photogrammetry data, a comparison was done at out of bank sections. A histogram detailing the frequency curve of the difference between these two data sets is shown in Figure 3.

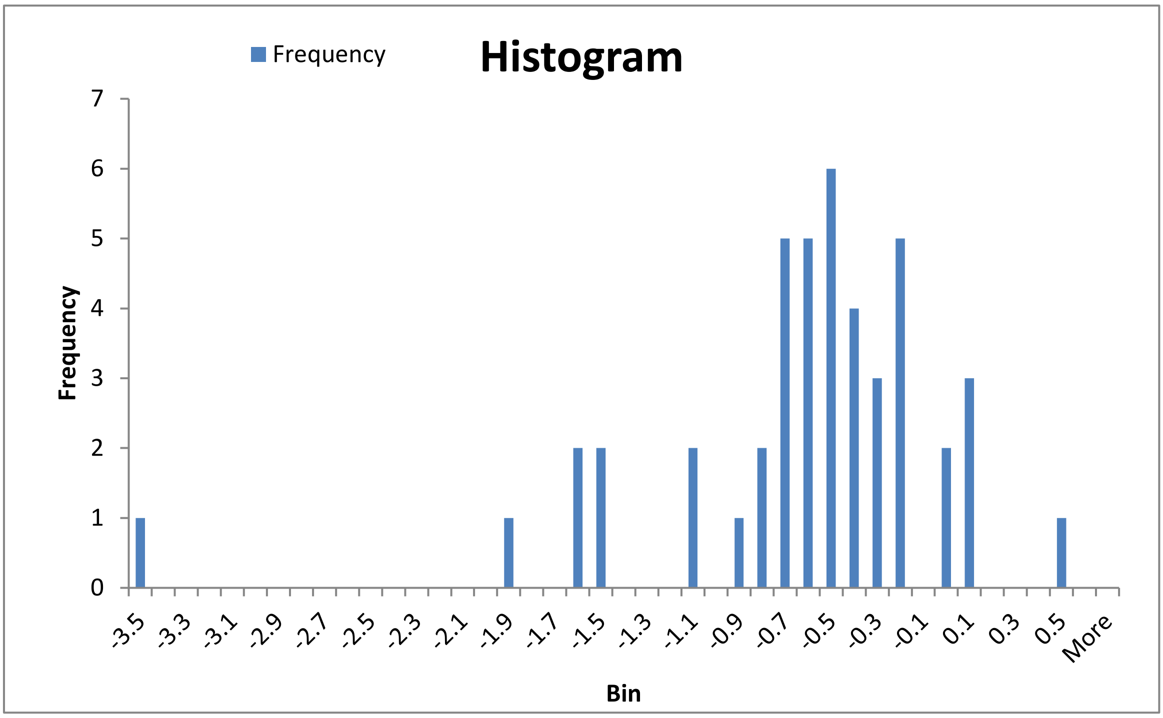


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift of 0.5m in the photogrammetry data was applied.

5. Model Setup

Model Method	1D
Software	FMP (4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using available photogrammetry data
Run Settings	Run parameters - Unsteady simulation, Single Precision, Cold Start with initial conditions for 1D domain.
Other comments	

6. Model inflows and Boundary Conditions

Peak flow estimates have been derived at 2 locations for the White House Burn model. These are at the upstream limit of the White House Burn model (WH_01), and for downstream limit of the model incorporating an unnamed tributary that discharges into the watercourse downstream of the access culvert at XS20_05, see model schematic for location. The design flow estimates have been developed using the ReFH2 methodology and are shown in the table below. Full details of the calculations and the justification for this approach are provided in the FEH calculation record.

Flow Node	Annual Probability Event									
	2	5	10	25	50	75	100	200	1000	100+25%
WH_01	1.43	1.98	2.38	2.95	3.44	3.77	4.01	4.69	6.80	5.02
WH_04	1.73	2.38	2.86	3.54	4.13	4.52	4.81	5.62	8.14	6.02

The flows from WH_01 have been applied at the upstream limit of the model. The inflow for the tributary has been derived by subtracting the hydrograph of WH_01 from WH_04.

The downstream boundary of the 1D FMP model has been defined as a normal depth boundary using a bed slope.

7. Manning's 'n' Roughness Coefficients

The Manning's roughness coefficient values used in the river sections were derived from the information provided in the topo survey and the site photographs. The Manning's n values utilised have been listed within Appendix B of this report.

8. Model Calibration and Verification

No data was available with which to calibrate the model. The results have therefore been sensibility checked for model stability and appropriateness using engineering judgement only.

9. Proposed Model Development

The proposed scheme will extend the existing A1 culvert on the upstream face. Cross section data for the upstream face of the proposed structure has been inferred from the topographic survey provided by the surveyor. It is not proposed to realign the watercourse and the culvert crossing will tie in with the existing alignment at the upstream face.

The existing culvert at XS20_03 beneath the A1 is extended upstream by 19.9m with same dimensions and culvert levels are extrapolated based on the existing gradient. This is a rectangular culvert with an elevated walkway runs through the culvert. This culvert with walkway is represented as a symmetrical culvert allowing half the walkway on either side of the culvert. The upstream opening overall is marginally smaller than the downstream but the walkway is more pronounced on the downstream face. Given the size of the structure it is considered the walkway will be a more critical constraint in terms of the assessments required than the soffit level so the downstream face has been used to represent the structure. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure. Further details on how the two new culverts have been modelled is presented within Appendix A of this report.

10. Model Runs

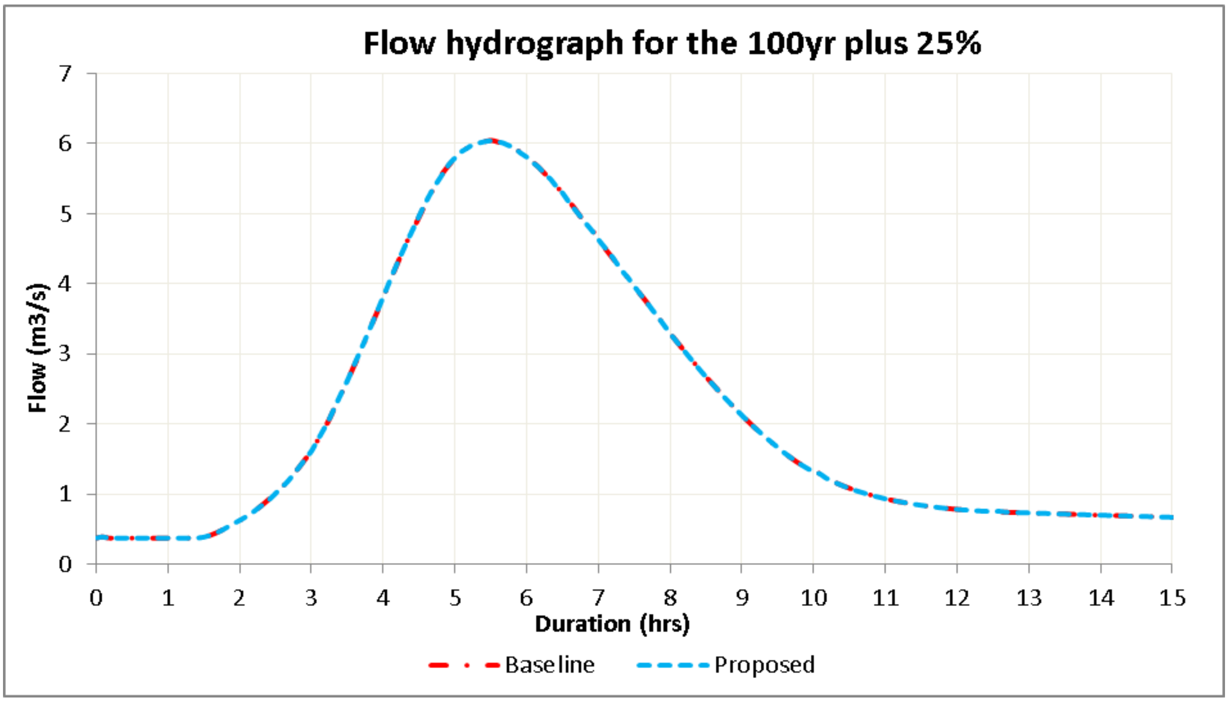
Model Scenario	Return Periods / Events
Baseline (BSC)	2yr, 100yr+25%, 1000yr
Proposed Scenario (PRO)	2yr, 100yr+25%, 1000yr, 100yr+25% with blockage

11. Model Results

The table below provides details of the freeboard associated with each structure for a range of flood events. White House Burn is an ordinary watercourse. As such a design freeboard of 300mm is preferred in the 100yr+25% climate change event in accordance with the recommendations in the DMRB. The 1000yr event is larger than the 100yr+25% climate change event so has been used to assess risk in an extreme event. Given the size of the proposed structure, blockage has been assessed by assuming the inlet capacity of the culvert structure is reduced by 30%. The negative values in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is

Structure	Carriageway Freeboard above Inlet Soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2yr	100 yr +25% cc	1000 yr	100yr +25% cc with blockage	2yr	100yr +25% cc	1000 yr	100yr +25% cc with blockage
Existing A1	1.81	2.64	2.13	1.98	-	2.43	1.97	1.85	-
Proposed A1	1.57	2.79	2.35	2.19	2.1	2.43	1.97	1.85	-

To understand the implications of flood risk downstream of the culvert, a flow hydrograph has been compared between the existing and proposed models at the downstream boundary of the model. Figure below confirms that there would be no change to the downstream flows resulting from the Scheme for all flows up to the 100 year + 25 % climate change event.


Flow hydrograph for the 100yr plus 25%
Flow (m3/s)
0
1
2
3
4
5
6
7
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Duration (hrs)
Baseline
Proposed

Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Photo	Dimensions	Modelling Approach
1 (XS20_03)	Culvert on A1 crossing the White House Burn.		The culvert length is 22.0m. This is a rectangular culvert with an elevated walkway runs through the culvert. The dimensions have been taken directly from the data provided by surveyor.	The culvert with walkway is represented as a symmetrical culvert allowing half the walkway on either side of the culvert. The upstream opening overall is marginally smaller than the downstream but the walkway is more pronounced on the downstream face. Given the size of the structure it is considered the walkway will be a more critical constraint in terms of the assessments required than the soffit level so the downstream face has been used to represent the structure. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.
2 (XS20_05)	Culvert crossing the White House Burn on west of A1.		The culvert length is 6.0m. The culvert consists of a single circular conduit of 1.5m diameter. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a single circular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.

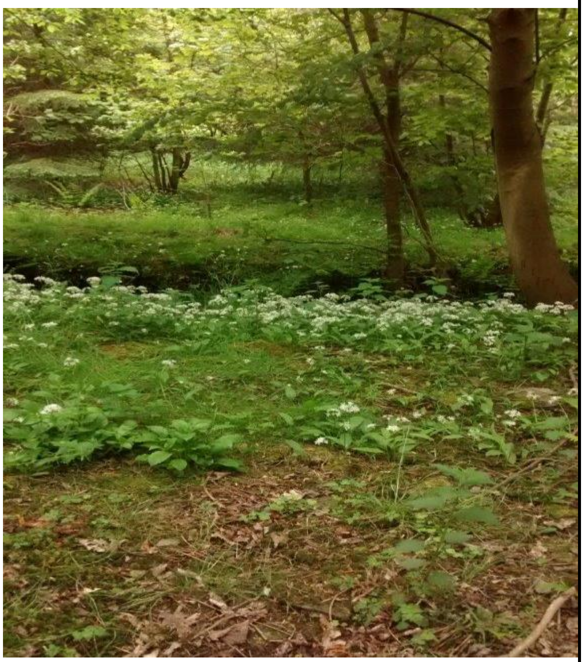
Proposed Model (PRO)

1 (XS20_03)	Proposed culvert on the new A1 alignment crossing the White House Burn.	Structure dimensions have been taken directly from the provided structure schedule.	The proposed structure length is 41.9m.	New structure is represented by replicating the existing structure with the walkway as only the length of the structure is changed.

Appendix B. 1D Channel Roughness

The following table summarises the Manning's n values applied to the river channel

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS20_01 to XS20_05 Tortuosity: Low			
Left Bank	0.05	Grass	
Channel	0.04	Silt / Gravel	
Right Bank	0.05	Grass	

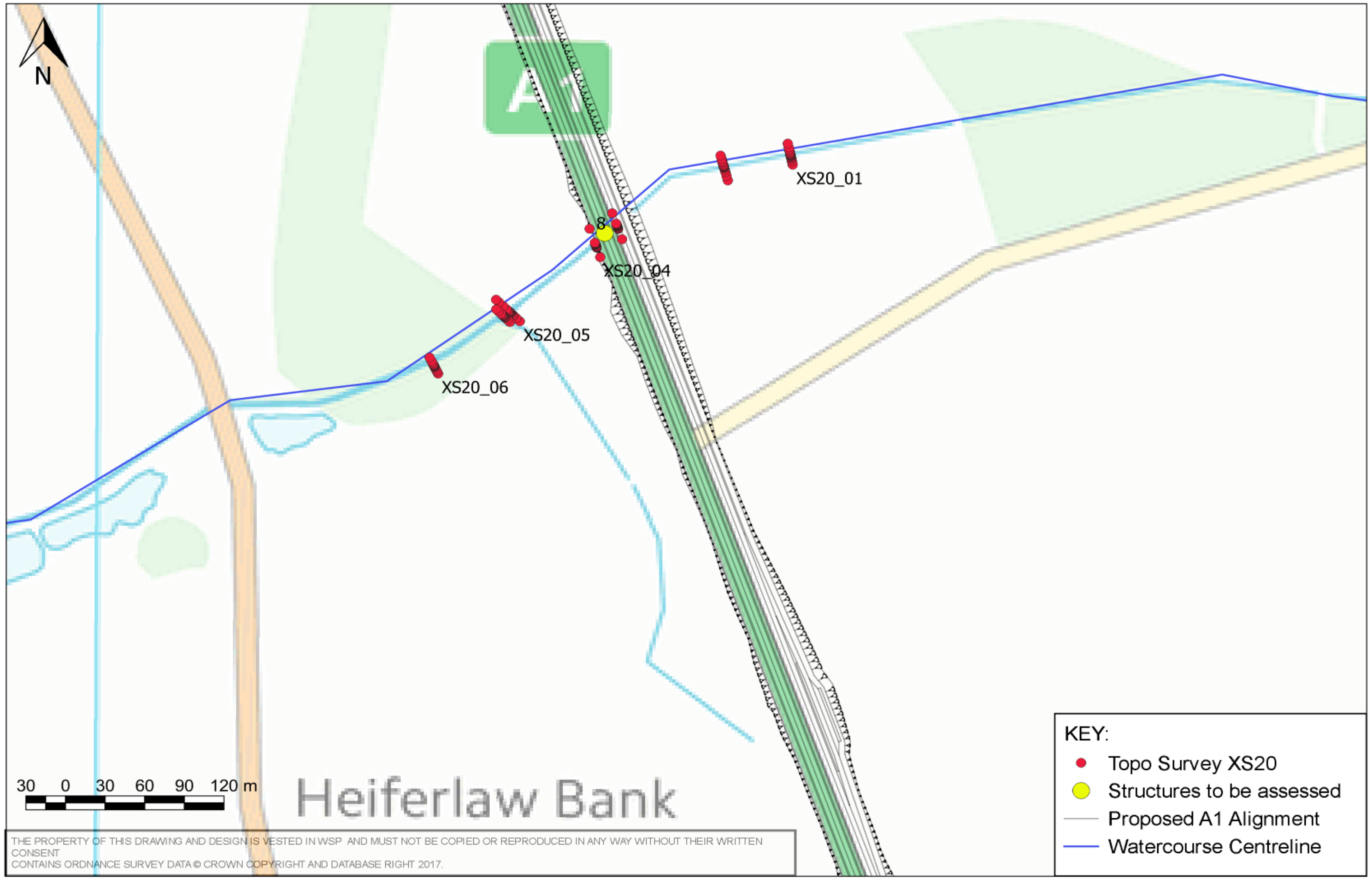
XS20_05A to XS20_06 Tortuosity: Low			
Left Bank	0.08	Trees	
Channel	0.035	Silt	
Right Bank	0.08	Trees	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result Files
Baseline Scenario						
WHB	Baseline	2yr	v4.4	WHB_Baseline_2yr_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07.dat	WHB_BASELINE_2YR_V01
WHB	Baseline	100yr	v4.4	WHB_Baseline_100yr_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07.dat	WHB_BASELINE_100YR_V01
WHB	Baseline	100yr+25%	v4.4	WHB_Baseline_100yr+25%_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07.dat	WHB_BASELINE_100YR+25%_V01
WHB	Baseline	1000yr	v4.4	WHB_Baseline_1000yr_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07.dat	WHB_BASELINE_1000YR_V01
Design Scenario						
WHB	Proposed	2yr	v4.4	WHB_Design_01_2yr_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07_Design_v01.dat	WHB_DESIGN_01_2YR_V01
WHB	Proposed	100yr	v4.4	WHB_Design_01_100yr_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07_Design_v01.dat	WHB_DESIGN_01_100YR_V01
WHB	Proposed	100yr+25%	v4.4	WHB_Design_01_100yr+25%_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07_Design_v01.dat	WHB_DESIGN_01_100YR+25%_V01
WHB	Proposed	1000yr	v4.4	WHB_Design_01_1000yr_v01.ief	A1_Northumberland_Whitehouse_Burn_XS20_v07_Design_v01.dat	WHB_DESIGN_01_1000YR_V01
WHB	Blockage	100yr+25%	v4.4	Whitehouse_Burn_v07_Design_v01_Blockage.ief	Whitehouse_Burn_v07_Design_v01_Blockage.dat	WHITEHOUSE_BURN_V07_DESIGN_V01_BLOCKAGE

Appendix D. Model Schematics

Baseline Model (BSC)



Project	A1 - Northumberland
Job Number	70044137
Location	Charlton Mires, Alnwick, Northumberland, England (417694 621042)
Watercourse(s)	Kittycarter Burn

1. Objectives/Areas of interest

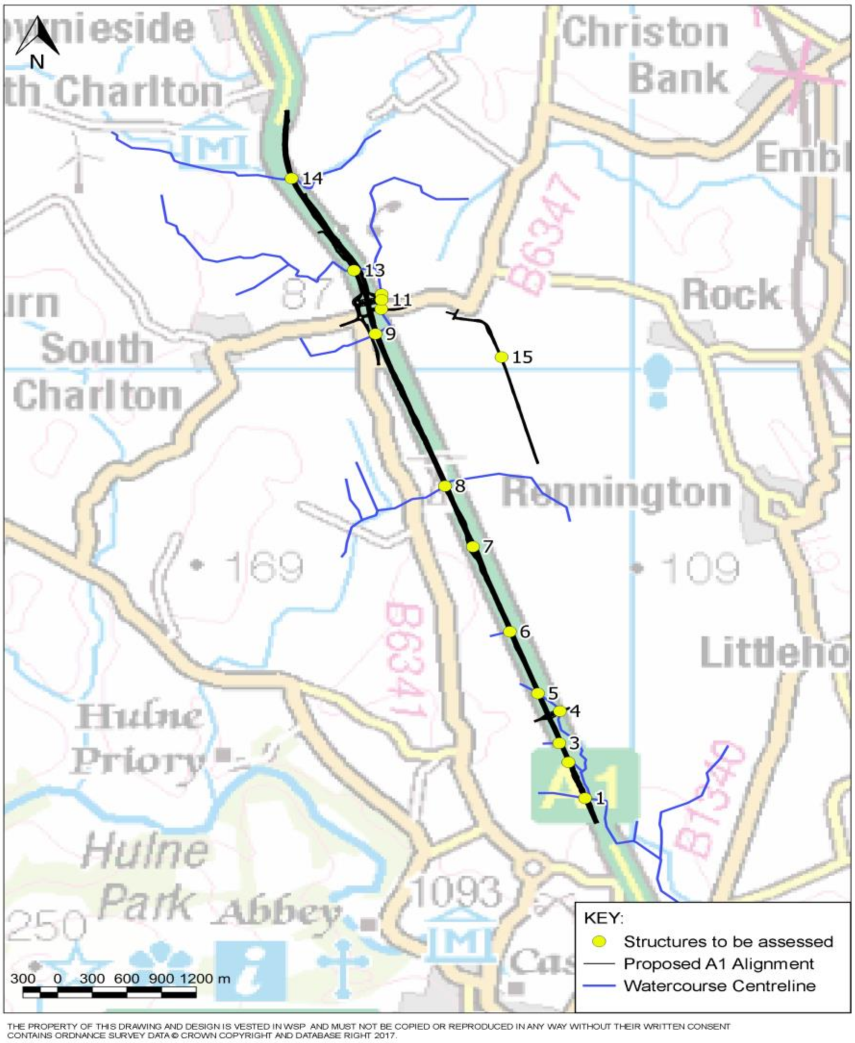


Figure 1: Location of structures in overall scheme

Highways England (HE) has identified the need to improve the existing A1 in Northumberland between Denwick and North Charlton. The Scheme is approximately 8 km in length and comprises online improvements consisting of carriageway widening.

Figure 1 above shows the location of various structures along the scheme. There are 15 culverts in total across the site which are to be assessed. The purpose of the assessment is to understand the impacts on flood risk and ecology as a result of the works.

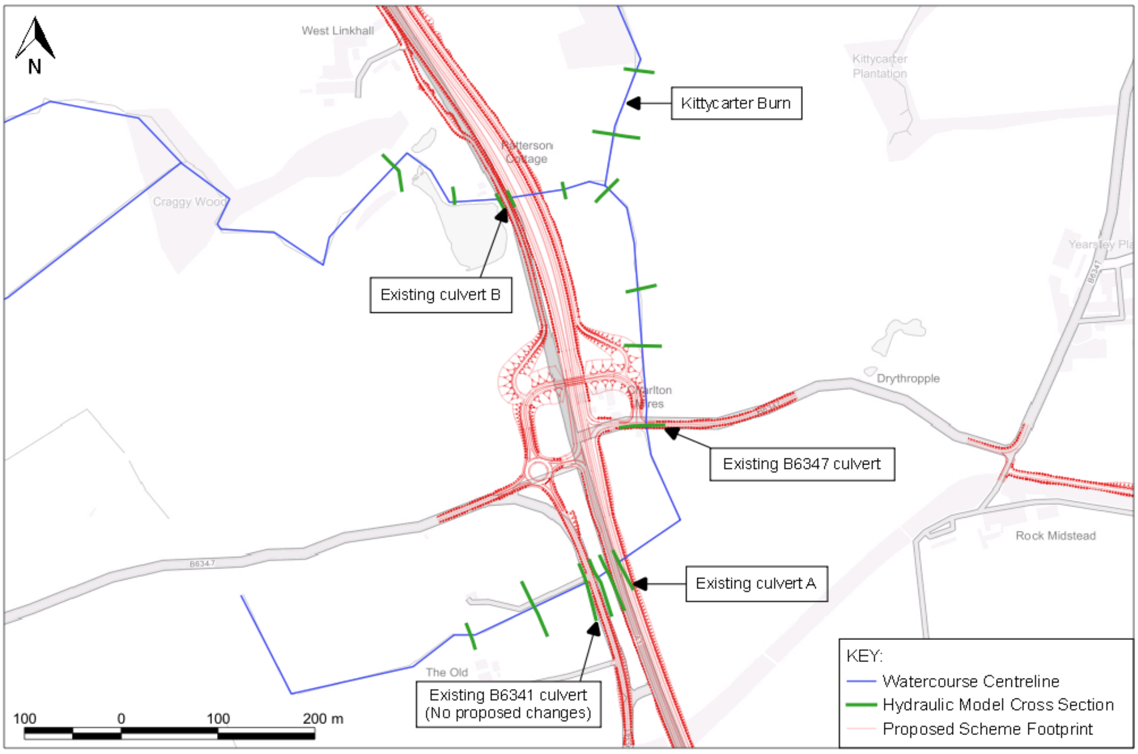


Figure 2: Location of structures on Kittycarter Burn

This report relates to the proposed works on the Kittycarter Burn. There are four existing structures within the reach. Three of these structures are on the southern tributary and include the A1 culvert, a B6341 culvert west of the A1 and a B6347 culvert east of the A1. The only structure on the western tributary is the A1 culvert. Further details on how each of these structures were modelled is presented within Appendix A of this report.

2. Model Input Data

Title	Type	Notes
A2E XS21.xlsx A2E XS22.xlsx	Topographic Survey	Detailed topographic survey of area around the Kittycarter Burn and existing structures. Surveyed information includes channel, bank, bed, flood plain, existing bridge deck, existing bridge soffit, existing bridge parapet. Data has been used to build 1D model.

3. GIS Data

OS Tiles -	Source:	OS Open Map Local downloaded from OS OpenData website
Ground Level Data -	Resolution: Date :	2m Digital Surface Model Data and 5m Digital Terrain Model Data Photogrammetric Digital Models from supplied https://apgb.blueskymapshop.com in November 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 1.7km in length. The watercourse is the Kittycarter Burn.

There are four existing structures within the reach. These structures were modelled in 1D domain using FMP structure nodes. The data for these structures was obtained from the topographic survey received from the surveyor. Further details on how each of these structures were modelled is presented within Appendix A of this report.

Model cross sections have been extended into the floodplain to accommodate the design flows. Local ground level data are available from 2m DSM tiles. These cover the entire extent of the model.

To confirm the agreement between the river section survey and the photogrammetry data, a comparison was done at out of bank sections. A histogram detailing the frequency curve of the difference between these two data sets is shown in Figure 3.

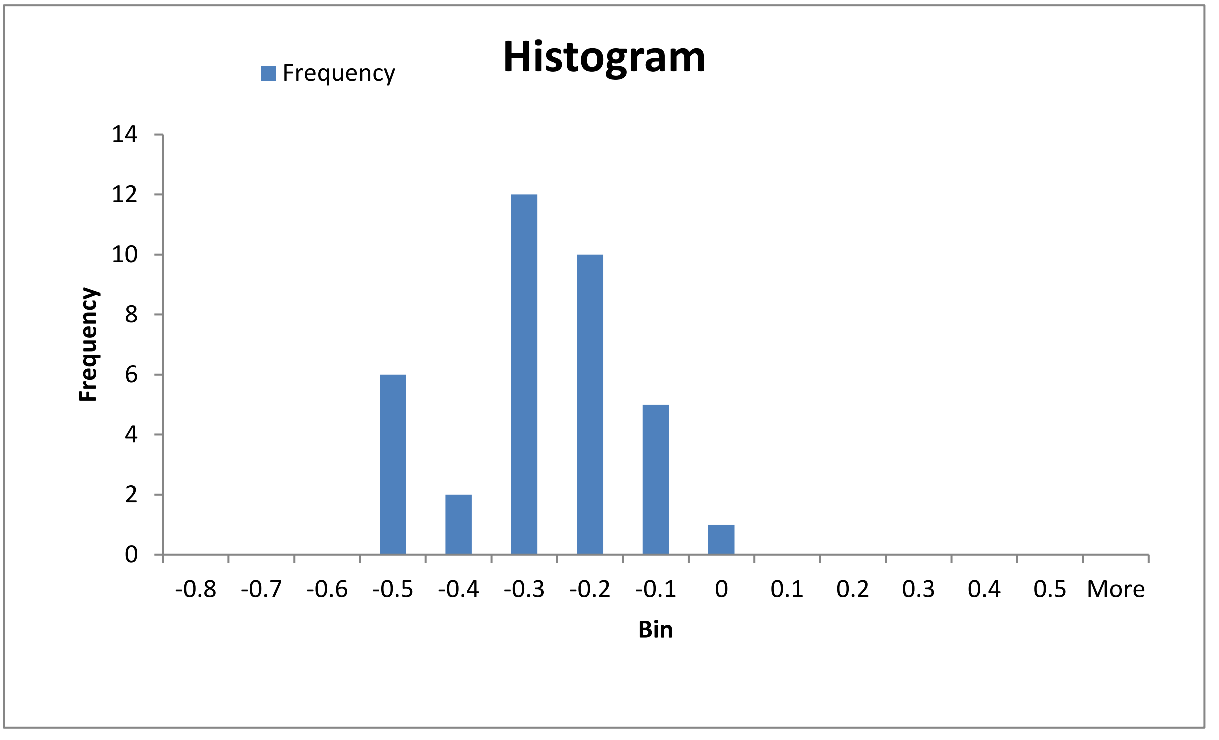


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift of 0.3m in the photogrammetry data was applied.

5. Model Setup

Model Method	1D
Software	FMP v4.4
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using available photogrammetry data
Run Settings	Run parameters - Unsteady simulation, Single Precision, Cold Start with initial conditions for 1D domain.
Other comments	

6. Model inflows and Boundary Conditions

Peak flow estimates have been derived at 5 locations for the Kittycarter Burn model. The design flow estimates have been developed using the ReFH2 methodology and are shown in the table below. Full details of the calculations and the justification for this approach are provided in the FEH calculation record.

Flow Node	Annual Probability Event								
	2	5	10	25	50	75	100	200	1000	100+25%
KB_01	0.39	0.54	0.65	0.81	0.95	1.04	1.11	1.31	1.92	1.39
KB_02	0.54	0.75	0.90	1.17	1.31	1.44	1.54	1.81	2.65	1.92
KB_03	0.58	0.80	0.96	1.19	1.40	1.54	1.64	1.93	2.83	2.05
KB_04	0.71	0.98	1.17	1.45	1.71	1.88	2.00	2.36	3.45	2.51
KB_05	1.35	1.86	2.23	2.78	3.26	3.58	3.82	4.50	6.59	4.78

The flows from KB_01 and KB_04 has been applied at the upstream limit of the two tributaries XS21 and XS22 respectively. Inflow KB_02 has been derived by subtracting the hydrograph of KB_01 from KB_02 and applied at the downstream of A1 culvert at cross section XS21_04. Additional inflow KB_03 has been derived by subtracting the hydrograph of KB_02 from KB_03 and applied at the cross section XS03. One more inflow KB_05 has been derived by subtracting the hydrographs KB_03 and KB_04 from KB_05 and applied at the cross section XS22_03.

The downstream boundary of the 1D FMP model has been defined as a normal depth boundary using a bed slope.

7. Manning's 'n' Roughness Coefficients

The Manning's roughness coefficient values used in the river sections were derived from the information provided in the topo survey and the site photographs. The Manning's n values utilised have been listed within Appendix C of this report.

8. Model Calibration and Verification

No data was available with which to calibrate the model. The results have therefore been sensibility checked for model stability and appropriateness using engineering judgement only.

The proposed scheme will extend the two existing A1 culverts on the downstream face. The B6347 culvert to the east of the A1 will be replaced. Cross section data for the upstream and downstream facesfor the two A1 structures were inferred from the topographic survey provided by the surveyor. For the B3647 culvert the channel is to be diverted around the junction; the channel in this instance was developed with appropriate side slopes.
The downstream channel of existing culvert at XS21_04 beneath the A1 drops away compared to the gradient of the culvert. Extrapolating the existing gradient of the culvert will result in increasing the downstream invert by 80mm. Hence it is assumed that 0.15m silt is at bottom and thus marginally changing the gradient of the downstream channel. The existing culvert is extended on the outlet side by 26.5m with same dimensions.

The existing culvert at XS22_05 beneath the A1 has a natural bed and hence it is assumed to be free of sediment. This existing culvert is extended on the outlet side by 47.5m with same dimensions. As downstream face of the structure is not surveyed, culvert levels are extrapolated based on the existing gradient.
The existing culvert at XS04 beneath the B6347 is assumed to be replaced by a circular culvert of 600mm diameter. The size of the culvert has is unchanged from previously to maintain a similar level of flood risk downstream. The downstream culvert and channel gradient have been derived from the calculated length to tie into XS03. The culvert length is assumed to remain consistent given it reflects the width of the road. The downstream channel is assumed to have a 1m wide bed and side slopes of 3:1 tying into the local ground level.

Further details on how the three new culverts have been modelled is presented within Appendix A of this report.

10. Model Runs

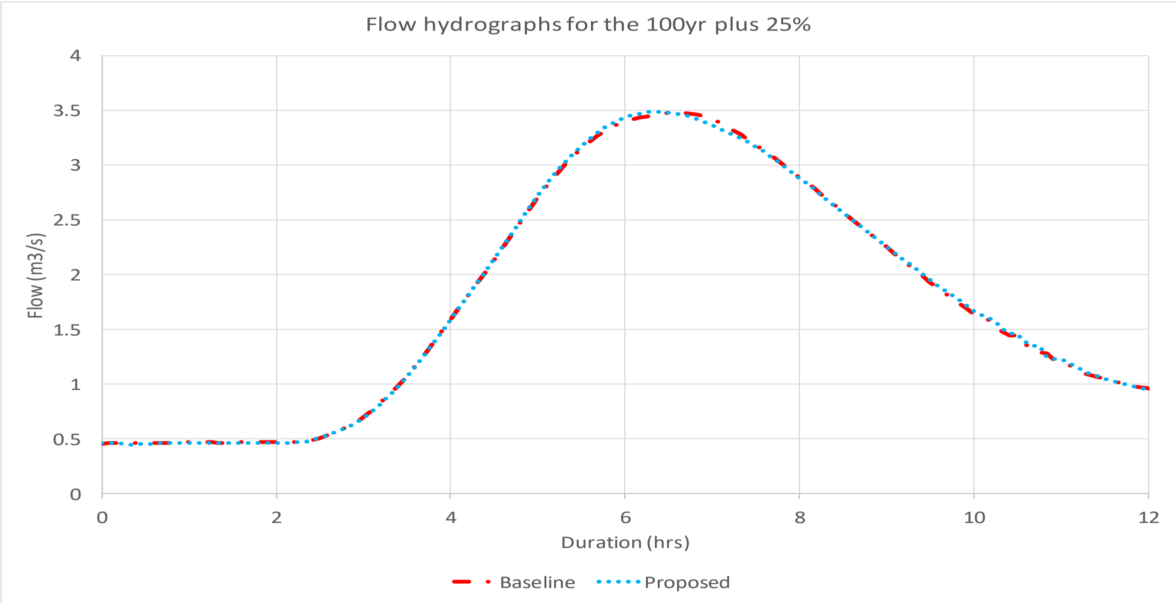
Model Scenario	**Return Periods / Events**
Baseline (BSC)	2yr, 100yr+25%, 1000yr
Proposed Scenario (PRO)	2yr, 100yr+25%, 1000yr, 100yr+25% with blockage

11. Model Results

The table below provides details of the freeboard associated with each structure for a range of flood events. Kittycarter Burn is an ordinary watercourse in the vicinity of the A1 and as such a design freeboard of 300mm is preferred for the 100yr+25% climate change event in accordance with the recommendations in the DMRB. The 1000yr event is larger than the 100yr+25% climate change event so the 1000yr event has been used to assess risk in an extreme event. Blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 30% for the culvert B and by 67% for culvert A and the B6347 culvert on the southern tributary reflecting the different sizes of these structures and hence the likelihood of blockage. The negative values in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

Structure	Carriageway Freeboard above Inlet Soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2yr	100yr +25% cc	1000 yr	100yr +25% cc with blockage	2yr	100yr +25% cc	1000 yr	100yr +25% cc with blockage
Existing culvert A	2.61	-0.11	-0.43	-0.98	-	-0.05	-0.34	-0.57	-
Proposed culvert A	2.81	-0.11	-0.38	-0.98	-0.73	-0.01	-0.20	-0.27	-
Existing culvert B	1.85	1.51	0.81	0.67	-	1.46	0.81	0.69	-
Proposed culvert B	1.85	1.58	0.89	0.72	0.79	1.36	0.74	0.61	-
Existing culvert B6347	0.74	-0.02	-0.81	-0.88	-	0.02	-0.27	-0.39	-
Proposed culvert B6347	1.57	0.18	-0.69	-0.78	-0.82	0.31	-0.08	-0.16	-

To understand the implications of flood risk downstream of the culvert, a flow hydrograph has been compared between the existing and proposed models at the downstream boundary of the model. The results show a minor increase in downstream flows as a result of the changes on the western tributary. This is a result of the increased culvert length improving the conductivity of the channel and resulting in a marginal reduction in water levels upstream. Currently flows overtop the right bank of the western tributray upstream of the A1. The reduction in water levels caused by the scheme means a resulting increase in downstream flows as less flow overtops the right bank. To mitigate against this it is recommended that the Scheme improvements should lower the right bank level upstream of the A1 on the western tributary. The area on the right bank in this location is seasonally wet and making use of this area more frequently will have flood risk benefits, if the landowner is amenable to the proposals. Some minor lowering will result in the reestablishment of the existing overtopping frequency and offset the increase in flows from the western tributary. Further lowering will increase the frequency of overtopping and will be sufficient to deliver a net benefit to downstream flows.


Flow hydrographs for the 100yr plus 25%
Flow (m3/s)
4
3.5
3
2.5
2
1.5
1
0.5
0
0
2
4
6
8
10
12
Duration (hrs)
Baseline
Proposed

Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Data source	Dimensions	Modelling Approach
1 (XS21_06)	Culvert on B6341 crossing the Kittycarter Burn.		The culvert length is 21.2m. This is a circular culvert with 0.45m diameter. The dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a single circular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.
2 (XS21_04)	Culvert on A1 crossing the Kittycarter Burn.		The culvert length is 25.5m. This is a circular culvert with 0.6m diameter and 0.15m silt. The dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a symmetrical culvert to represent the silt at the bottom. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.
3 (XS04)	Culvert on B6347 crossing the Kittycarter Burn.	Not available	The culvert length is 15.0m. This is a circular culvert with 0.6m diameter. The dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a single circular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.

4 (XS22_05)	Culvert on A1 crossing the Kittycarter Burn.		The culvert length is 20.5m. This is a rectangular culvert. The dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a rectangular culvert within the channel. There is a bend within the culvert and the losses are represented by a bend unit. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.

Proposed Model (PRO)

1 (XS21_04)	Proposed culvert on the southern tributary beneath the A1.		The proposed structure length is 52.0m. Structure dimensions have been taken directly from the provided structure schedule.	New structure is represented by replicating the existing structure as only the length of the structure is changed.
2 (XS04)	Proposed culvert on the southern tributary beneath the B6347.		The proposed structure is a circular conduit of 0.60m diameter and 15m length. Structure dimensions have been taken directly from the provided structure schedule.	The new culvert has been modelled as a circular culvert. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.
3 (XS22_05)	Proposed culvert on the western tributary beneath the A1.		The proposed structure length is 68.0m. Structure dimensions have been taken directly from the provided structure schedule.	New structure is represented by replicating the existing structure as only the length of the structure is changed.

Appendix B. 1D Channel Roughness

The following table summarises the Manning's n values applied to the river channel

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS21_08 to XS01 Tortuosity: Low			
Left Bank	0.06	Grass / Shrubs	
Channel	0.04	Dry Mud	
Right Bank	0.06	Grass	

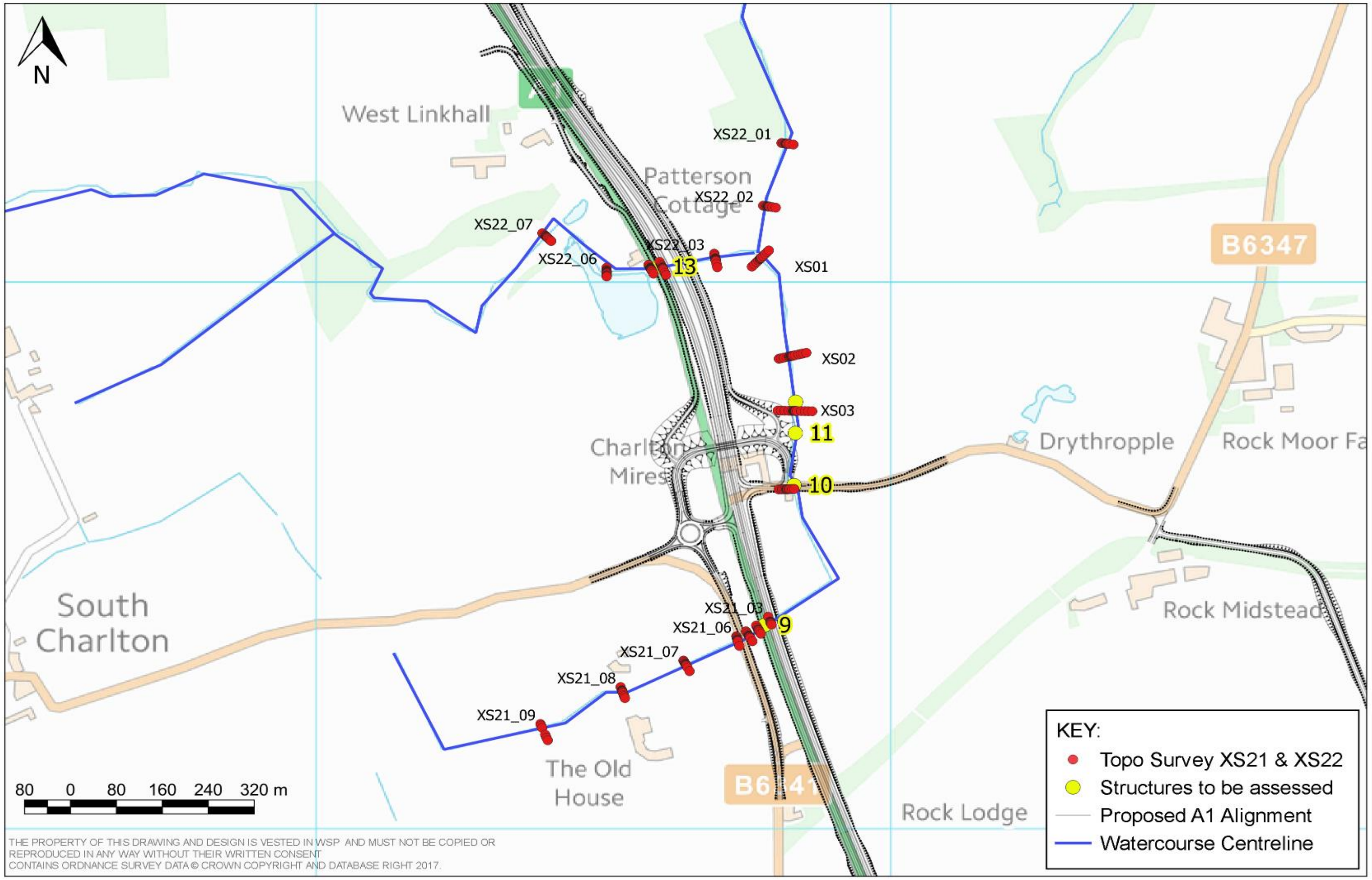
XS22_07 to XS22_01 Tortuosity: Low			
Left Bank	0.06	Grass	
Channel	0.04	Silt / Rocks	
Right Bank	0.06	Grass	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result Files
Baseline Scenario						
KB	Baseline	2yr	v4.4	KCB_Baseline_2yr_001.ief	KCB_XS21&XS22_v05.dat	KCB_BASELINE_2YR_001
KB	Baseline	100yr	v4.4	KCB_Baseline_100yr_001.ief	KCB_XS21&XS22_v05.dat	KCB_BASELINE_100YR_001
KB	Baseline	100yr+25%	v4.4	KCB_Baseline_100yr+25%_001.ief	KCB_XS21&XS22_v05.dat	KCB_BASELINE_100YR+25%_001
KB	Baseline	1000yr	v4.4	KCB_Baseline_1000yr_001.ief	KCB_XS21&XS22_v05.dat	KCB_BASELINE_1000YR_001
Design Scenario						
KB	Proposed	2yr	v4.4	KCB_Design_v04_02_2yr_001.ief	KCB_XS21&XS22_Design_02_v04.dat	KCB_DESIGN_V04_02_2YR_001
KB	Proposed	100yr	v4.4	KCB_Design_v04_02_100yr_001.ief	KCB_XS21&XS22_Design_02_v04.dat	KCB_DESIGN_V04_02_100YR_001
KB	Proposed	100yr+25%	v4.4	KCB_Design_v04_02_100yr+25%_001.ief	KCB_XS21&XS22_Design_02_v04.dat	KCB_DESIGN_V04_02_100YR+25%_001
KB	Proposed	1000yr	v4.4	KCB_Design_v04_02_1000yr_001.ief	KCB_XS21&XS22_Design_02_v04.dat	KCB_DESIGN_V04_02_1000YR_001
KB	Blockage	100yr+25%	v4.4	KCB_XS21&XS22_Design_02_v04_Blockage_XS04.ief	KCB_XS21&XS22_Design_02_v04_Blockage_XS04.dat	KCB_XS21&XS22_DESIGN_02_V04_BLOCKAGE_XS04
KB	Blockage	100yr+25%	v4.4	KCB_XS21&XS22_Design_01_v04_Blockage_XS21_04.ief	KCB_XS21&XS22_Design_01_v04_Blockage_XS21_04.dat	KCB_XS21&XS22_DESIGN_01_V04_BLOCKAGE_XS21_04
KB	Blockage	100yr+25%	v4.4	KCB_XS21&XS22_Design_01_v04_Blockage_XS22_05.ief	KCB_XS21&XS22_Design_01_v04_Blockage_XS22_05.dat	KCB_XS21&XS22_DESIGN_01_V04_BLOCKAGE_XS22_05

Appendix D. Model Schematics

Baseline Model (BSC)



Project	A1 - Northumberland
Job Number	70044137
Location	Chathill, Alnwick, Northumberland, England (417059 621977)
Watercourse(s)	Shipperton Burn
1. Objectives/Areas of interest	

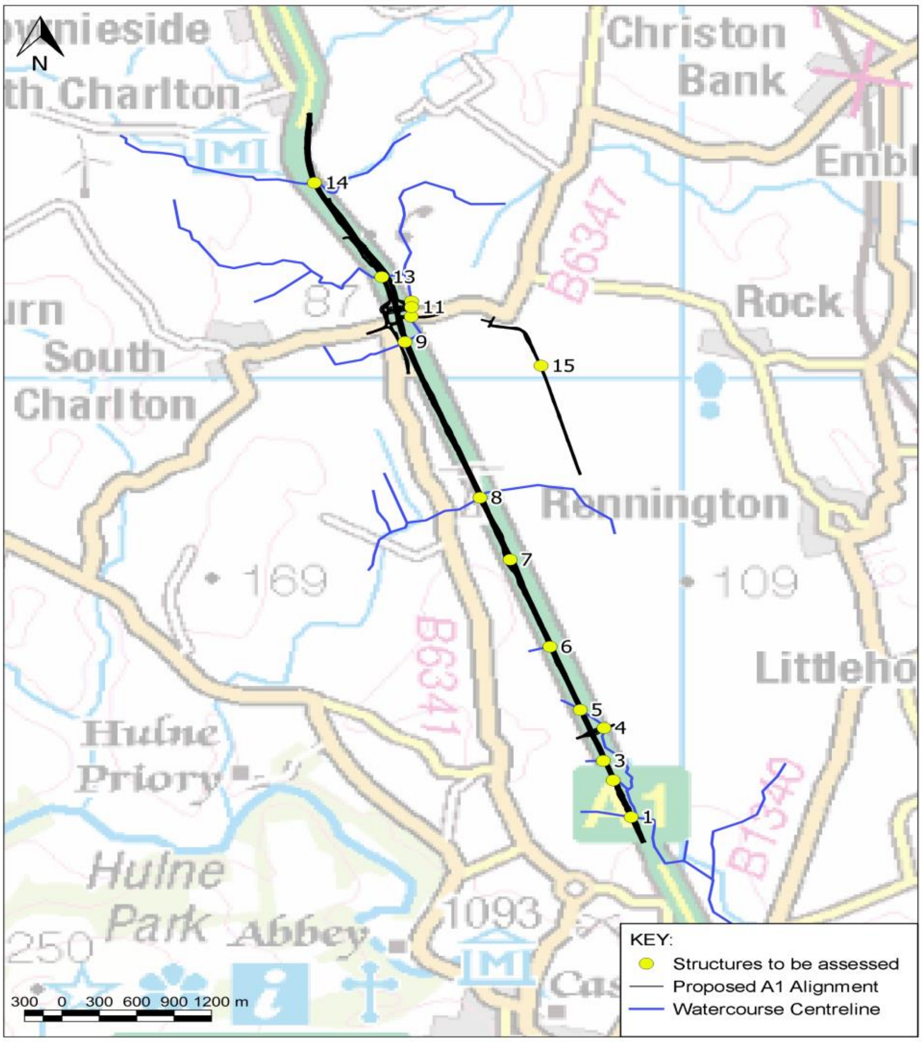


Figure 1: Location of structures in overall scheme

Highways England (HE) has identified the need to improve the existing A1 in Northumberland between Denwick and North Charlton. The Scheme is approximately 8 km in length and comprises online improvements consisting of carriageway widening.

Figure 1 above shows the location of various structures along the scheme. There are 15 culverts in total across the site which are to be assessed. The purpose of the assessment is to understand the impacts on flood risk and ecology as a result of the works.

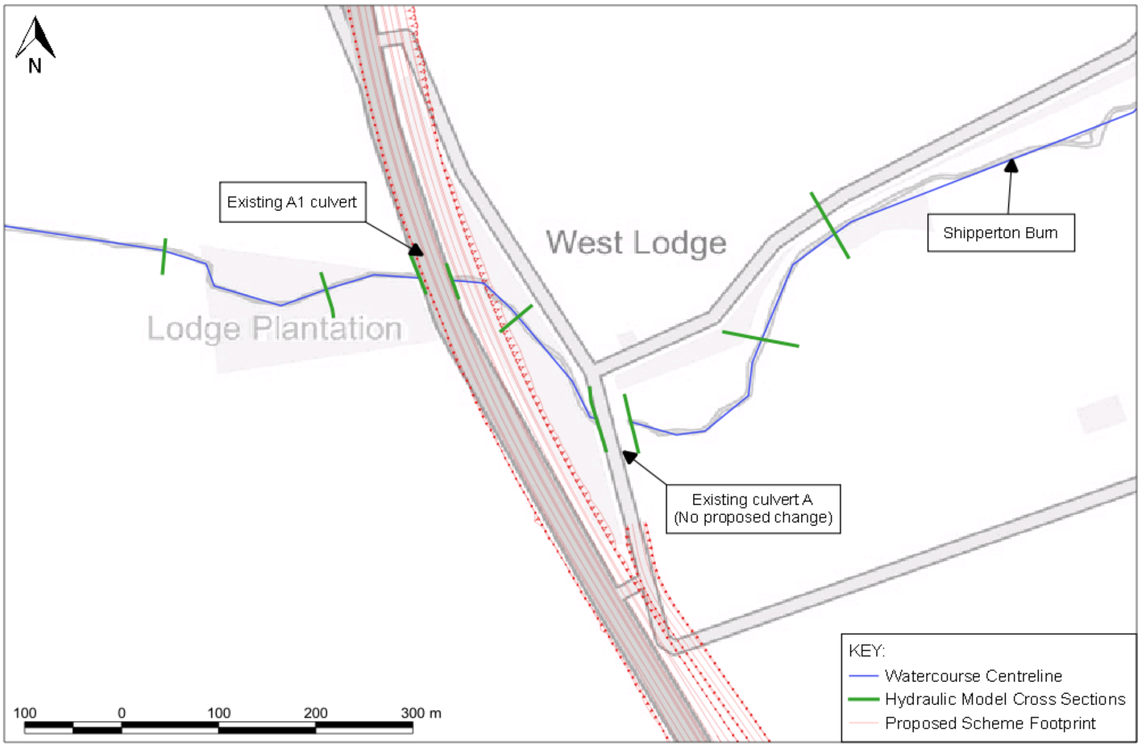


Figure 2: Location of structures on Shipperton Burn

This report relates to the proposed works on the Shipperton Burn. There are two existing structures within the reach. Cross section references are shown in the model schematic at the end of the model report. The structures are the A1 culvert and an access road bridge located at XS23_04 east of the A1. Further details on how each of these structures were modelled is presented within Appendix A of this report

2. Model Input Data

Title	Type	Notes
A2E XS23.xlsx	Topographic Survey	Detailed topographic survey of area around the Shipperton Burn and existing structures. Surveyed information includes channel, bank, bed, flood plain, existing bridge deck, existing bridge soffit, existing bridge parapet. Data has been used to build 1D model.

3. GIS Data

OS Tiles -	Source:	OS Open Map Local downloaded from OS OpenData website
Ground Level Data -	Resolution: Date :	2m Digital Surface Model Data and 5m Digital Terrain Model Data Photogrammetric Digital Models from supplied https://apgb.blueskymapshop.com in November 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 550m in length. The watercourse is the Shipperton Burn.

There are two existing structures within the reach. These structures were modelled in 1D domain using FMP structure nodes. The data for these structures was obtained from the topographic survey received from the surveyor. Further details on how each of these structures were modelled is presented within Appendix A of this report.

Model cross sections have been extended into the floodplain to accommodate the design flows. Local ground level data are available from 2m DSM tiles. These cover the entire extent of the model.

To confirm the agreement between the river section survey and the photogrammetry data, a comparison was done at out of bank sections. A histogram detailing the frequency curve of the difference between these two data sets is shown in Figure 3.

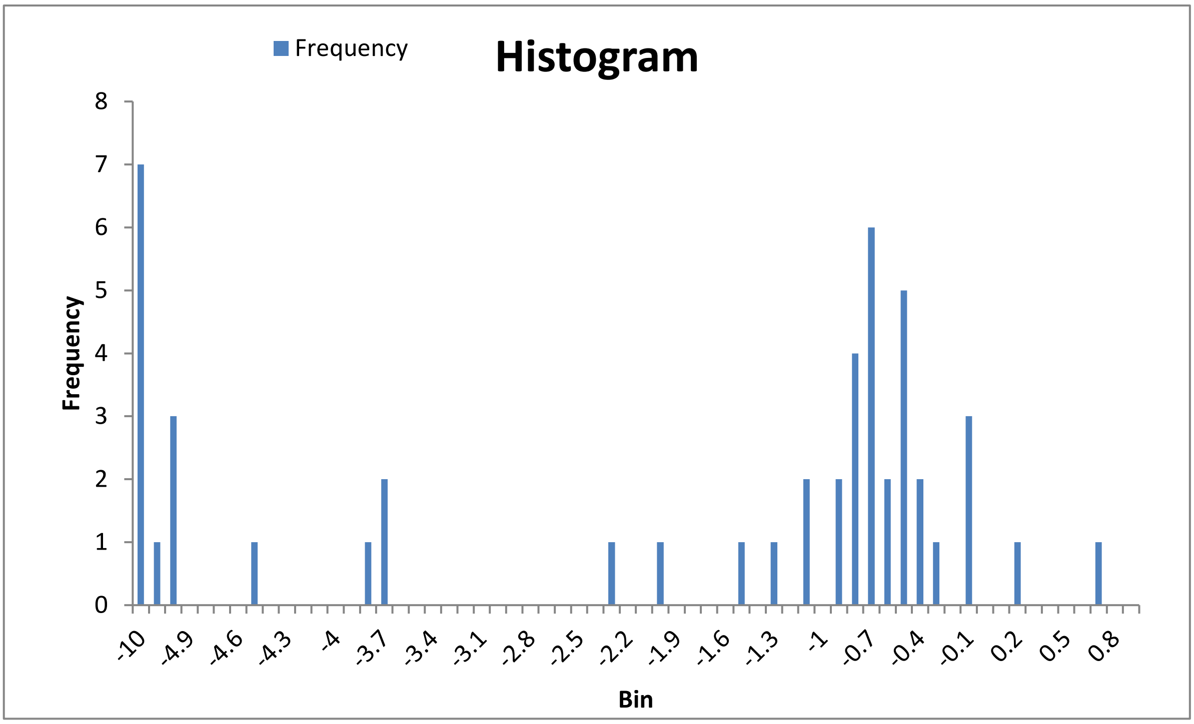


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift of 0.6m in the photogrammetry data was applied.

5. Model Setup

Model Method	1D
Software	FMP (4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using available photogrammetry data
Run Settings	Run parameters - Unsteady simulation, Single Precision, Cold Start with initial conditions for 1D domain.
Other comments	

6. Model inflows and Boundary Conditions

Peak flow estimates have been derived at 2 locations for the Shipperton Burn model. These are on upstream of the A1 in the vicinity of the proposed new road (SB_US) and at the downstream limit of the model (SB_DS). The design flow estimates have been developed using the ReFH2 methodology and are shown in the table below. Full details of the calculations and the justification for this approach are provided in the FEH calculation record.

Flow Node	Annual Probability Event									
	2	5	10	25	50	75	100	200	1000	100+25%
SB_US	1.54	2.07	2.46	3.04	3.55	3.89	4.15	4.88	7.05	5.19
SB_DS	1.53	2.05	2.44	3.01	3.52	3.85	4.11	4.83	6.98	5.14

The flows from SB_US have been applied at the upstream limit of the model only, no further inflows have been applied to the model.

The downstream boundary of the 1D FMP model has been defined as a normal depth boundary using a bed slope.

7. Manning's 'n' Roughness Coefficients

The Manning's roughness coefficient values used in the river sections were derived from the information provided in the topo survey and the site photographs. The Manning's n values utilised have been listed within Appendix C of this report.

8. Model Calibration and Verification

No data was available with which to calibrate the model. The results have therefore been sensibility checked for model stability and appropriateness using engineering judgement only.

The proposed scheme will extend the existing A1 culvert on the downstream face. Cross section data for the downstream face of this structures has been inferred from the topographic survey provided by the surveyor. It is not proposed to realign the watercourse and the culvert crossing will tie in with the existing alignment at the downstream face.

The existing culvert at XS23_07 beneath the A1 is extended on the outlet by 30.5m with same dimensions and culvert levels are extrapolated based on the existing gradient. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure. Further details on how the two new culverts have been modelled is presented within Appendix A of this report.

10. Model Runs

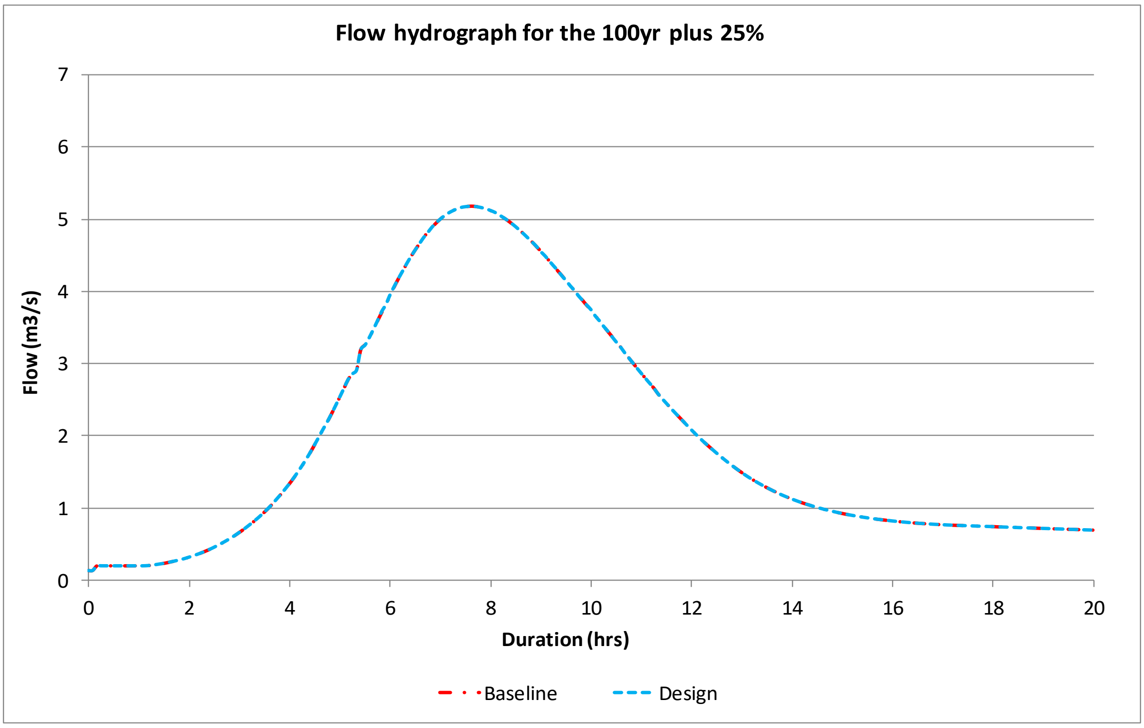
Model Scenario	Return Periods / Events
Baseline (BSC)	2yr, 100yr+25%, 1000yr
Proposed Scenario (PRO)	2yr, 100yr+25%, 1000yr, 100yr+25% with blockage

11. Model Results

The table below provides details of the freeboard associated with each structure for a range of flood events. Shipperton Burn is an ordinary watercourse in the vicinity of the A1 and as such a design freeboard of 300mm is preferred for the 100yr+25% climate change event in accordance with the recommendations in the DMRB. The 1000yr event is larger than the 100yr+25% climate change event so the 1000yr event has been used to assess risk in an extreme event. Given the size of the proposed structure, blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 30%. The negative values in the table shows that the respective structure is surcharged while positive value represents the available freeboard. A negative value in excess of the carriageway freeboard indicates the carriageway is overtopped.

Structure	Carriageway Freeboard above Inlet Soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2yr	100yr +25% cc	1000yr	100yr +25% cc with blockage	2yr	100yr +25% cc	1000 yr	100yr +25% cc with blockage
Existing A1	1.23	0.72	-0.05	-0.43	-	0.86	0.35	0.17	-
Scheme	1.23	0.72	-0.05	-0.43	-0.01	0.86	0.39	0.23	-

To understand the implications of flood risk downstream of the culvert, a flow hydrograph has been compared between the existing and proposed models at the downstream boundary of the model. Figure below demonstrates that there are no discernable changes in the water levels in the vicinity of A1 culvert


Flow hydrograph for the 100yr plus 25%
Flow (m3/s)
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
14
16
18
20
Duration (hrs)
Baseline
Design

Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Photo	Dimensions	Modelling Approach
1 (XS23_07)	Culvert on A1 crossing the Shipperton Burn.		The culvert length is 18.3m. This is a rectangular culvert. The dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a single rectangular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.
2 (XS23_04)	Culvert crossing the Shipperton Burn on east of A1.		The culvert length is 21.0m. This is a rectangular culvert. The dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as a single rectangular culvert within the channel. A spill section was created using the cross-section data from the surveyor in order to provide information around potential overtopping of the structure.

Proposed Model (PRO)

1 (XS23_07)	Proposed culvert on the new A1 alignment crossing the Shipperton Burn.		The proposed structure length is 48.8m. Structure dimensions have been taken directly from the provided structure schedule.	New structure is represented by replicating the existing structure as only the length of the structure is changed.

Appendix B. 1D Channel Roughness

The following table summarises the Manning's n values applied to the river channel

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS23_01 to XS23_03 Tortuosity: Low			
Left Bank	0.08	Trees	
Channel	0.035	Silt / Rocks	
Right Bank	0.05	Grass	

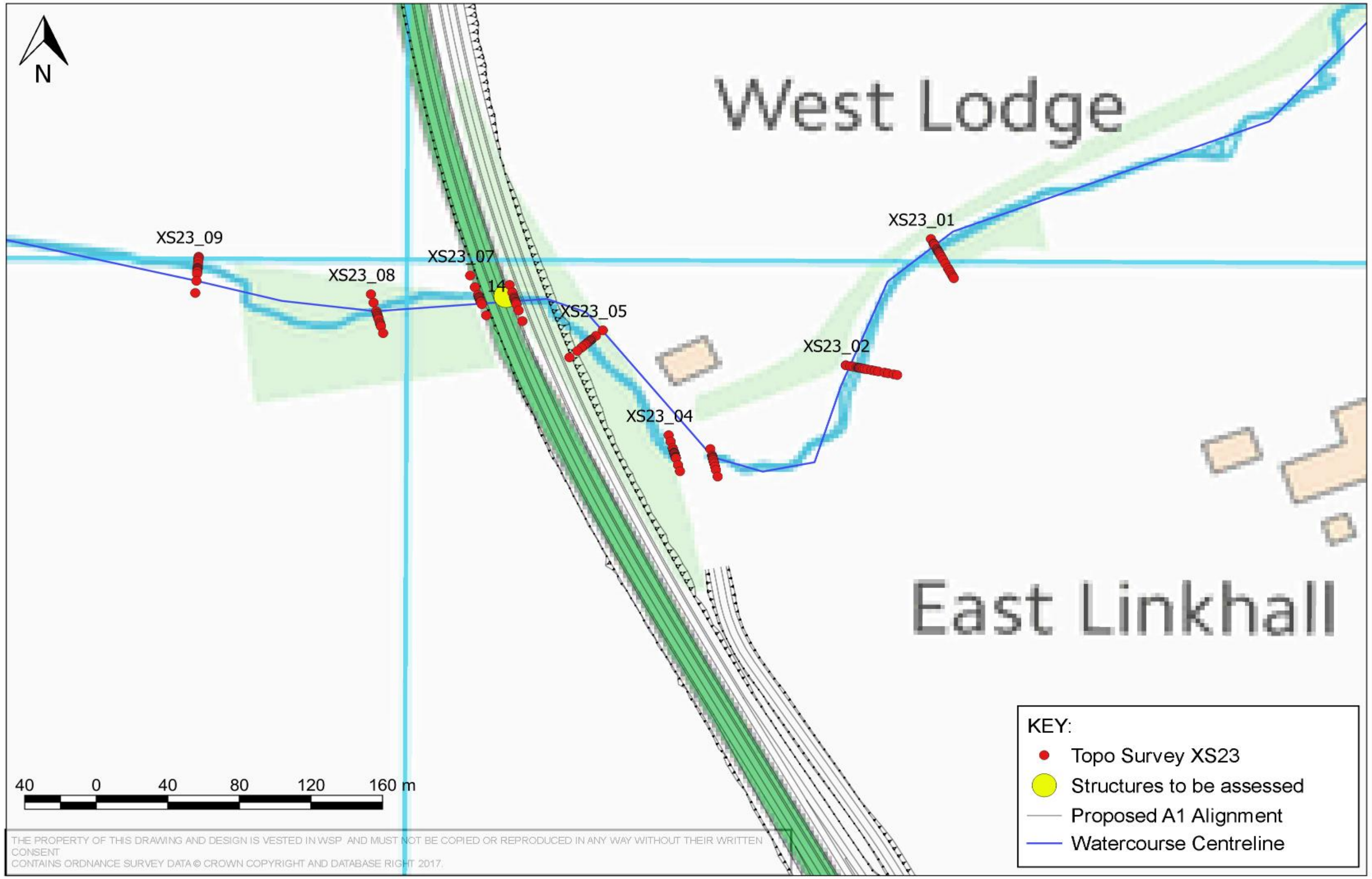
XS23_04 to XS23_09 Tortuosity: Low			
Left Bank	0.08	Grass / Trees	
Channel	0.04	Gravel / Rocks	
Right Bank	0.08	Grass / Trees	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result Files
Baseline Scenario						
SB	Baseline	2yr	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_2yr.ief	A1_Northumberland_Shipperton Burn_XS23_v05.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_2YR
SB	Baseline	100yr	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_100yr.ief	A1_Northumberland_Shipperton Burn_XS23_v05.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_100YR
SB	Baseline	100yr+25%	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_100yr+25%CC.ief	A1_Northumberland_Shipperton Burn_XS23_v05.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_100YR+25%CC
SB	Baseline	1000yr	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_1000yr.ief	A1_Northumberland_Shipperton Burn_XS23_v05.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_1000YR
Design Scenario						
	Proposed	2yr	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01_2yr.ief	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_DESIGN_V01_2YR
SB	Proposed	100yr	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01_100yr.ief	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_DESIGN_V01_100YR
SB	Proposed	100yr+25%	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01_100yr+25%CC.i ef	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_DESIGN_V01_100YR+25 %CC
SB	Proposed	1000yr	v4.4	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01_1000yr.ief	A1_Northumberland_Shipperton Burn_XS23_v05_Design_v01.dat	A1_NORTHUMBERLAND_SHIPPERTON BURN_XS23_V05_DESIGN_V01_1000YR
SB	Blockage	100yr+25%	v4.4	Shipperton Burn_v05_Design_v01_Blockage.ief	Shipperton Burn_v05_Design_v01_Blockage.dat	SHIPPERTON BURN_V05_DESIGN_V01_BLOCKAGE

Appendix D. Model Schematics

Baseline Model (BSC)



Flood estimation calculation record

Introduction

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

Contents

Approval

	Signature	Name and qualifications	For Environment Agency staff: Competence level (see below)
Calculations prepared by:		Lucy Rushmer BSc PhD MCIWEM CWEM CSci CGeog	
Calculations checked by:		Claire Storer MEng CEng MICE	
Calculations approved by:			

Environment Agency competence levels are covered in Section 2.1 of the flood estimation guidelines:

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

ABBREVIATIONS

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

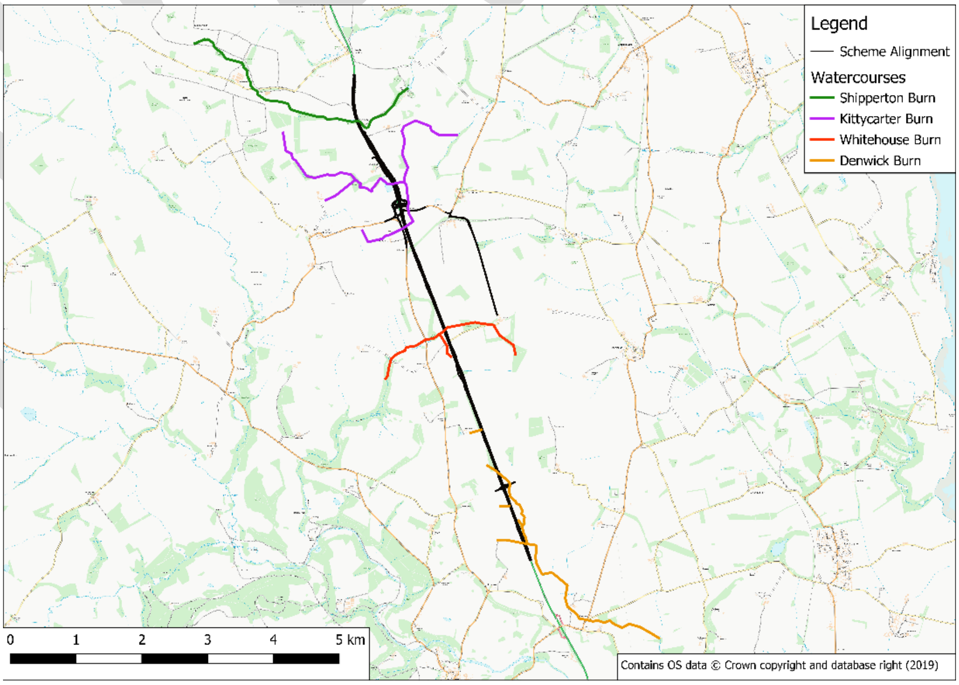
1 Method statement

Overview of requirements for flood estimates

<table>
<tr><th>Item</th><th>Comments</th></tr>
<tr><td>Give an overview which includes:
• Purpose of study
• Approx. no. of flood estimates required
• Peak flows or hydrographs?
• Range of return periods and locations
• Approx. time available</td><td>Highways England is proposing to provide additional capacity along the A1 in Northumberland: Alnwick to Ellingham Scheme, hereafter referred to as 'the Scheme'. The Scheme is located within the County of Northumberland and forms part of the strategic road network. The Scheme is located along the A1 between Denwick and North Charlton and is approximately 8 km in length. The Scheme comprises online improvements consisting of carriageway widening. The Scheme is designed to improve the capacity of the A1, reduce congestion within the surrounding road network and improve the connectivity within the area.

The Scheme alignment crosses five watercourses and their tributaries within 0.5km. Hydraulic modelling is required for four watercourses to inform the appropriate sizing and design of the proposed watercourse crossings, taking into account the potential effects of climate change. Figure 1 below shows the location of the five hydraulic models. The four watercourses being assessed are:
• Denwick Burn
• Kittycarter Burn
• Shipperton Burn
• White House Burn

The other watercourse (tributary of Embleton Burn) that the Scheme crosses has been assessed separately as part of the standalone Flood Risk Assessment (FRA).

Figure 1 Watercourse Locations


The objective of the study is to provide peak flow estimates and hydrographs for each watercourse and its tributaries. The location of each watercourse is shown in Figure 1 above. Peak flow estimates are required at 17 locations along the Scheme alignment as shown in Table 1 below. This calculation record presents the estimates for all of these locations.</td></tr>
</table>

Table 1 Flow Nodes

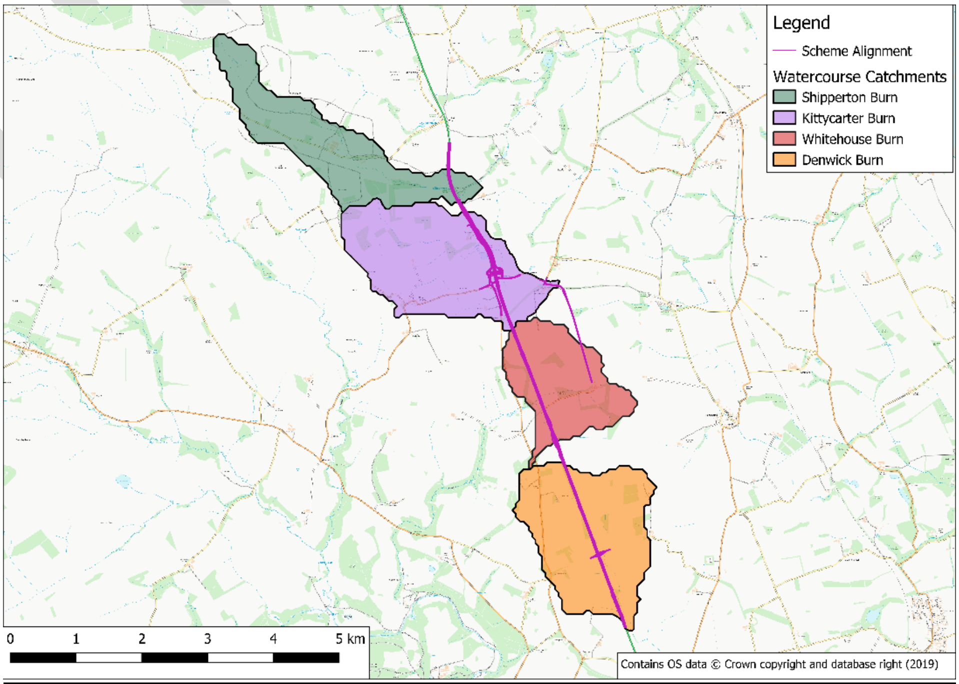
Flow Node	Watercourse
DB_01	Denwick Burn
DB_02	Denwick Burn
DB_03	Denwick Burn
DB_04	Denwick Burn
DB_05	Denwick Burn
DB_06	Denwick Burn
DB_07	Denwick Burn
KB_01	Kittycarter Burn
KB_02	Kittycarter Burn
KB_03	Kittycarter Burn
KB_04	Kittycarter Burn
KB_05	Kittycarter Burn
SB_US	Shipperton Burn
SB_DS	Shipperton Burn
WB_01	White House Burn
WB_02	White House Burn
WB_03	White House Burn
WB_04	White House Burn

The following return period events were assessed: 2 year, 5 year, 10 year, 25 year, 50 year, 75 year, 100 year, 100 year plus 25% climate change allowance, 200 year and 1000 year.

Overview of catchments

Item	Comments
Brief description of catchment, or reference to section in accompanying report	Figure 2 below shows the overall catchments for each of the watercourses that have been assessed.

Figure 2 Catchments

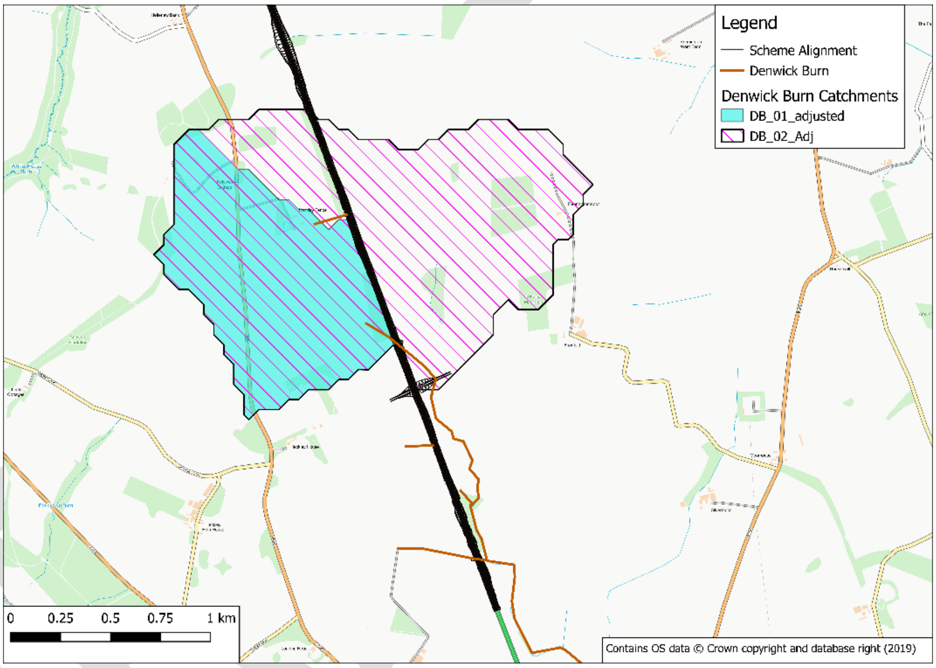


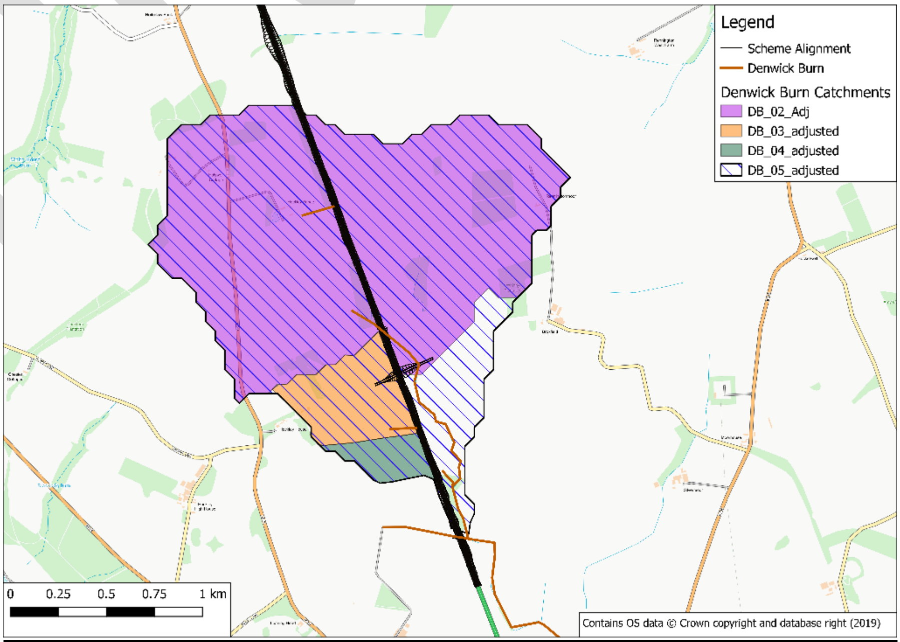
Denwick Burn

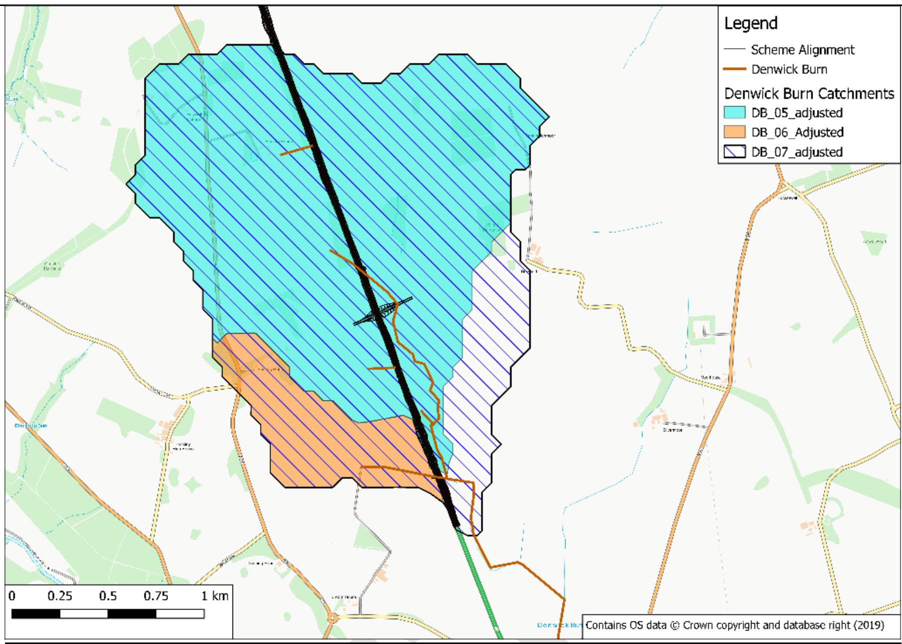
The total catchment area is approximately 3.76km². The catchment is predominately a rural catchment consisting of agricultural land, the existing A1 and the B6341. There is a small influence of flood attenuation due to reservoirs and lakes in the catchment, as a small lake has been identified in the upper part of the catchment from OS mapping.

The BGS bedrock geology information shows the catchment geology comprises limestone, sandstone, siltstone and mudstone which are moderately impermeable rocks. The BGS hydrogeology information on the FEH Web Service shows that the catchment is moderately permeable. Soil mapping indicates the catchment is underlain by slowly permeable seasonally wet acid loamy and clayey soils which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 3 shows the individual adjusted FEH catchments for Denwick Burn.

Figure 3 Denwick Burn Catchments





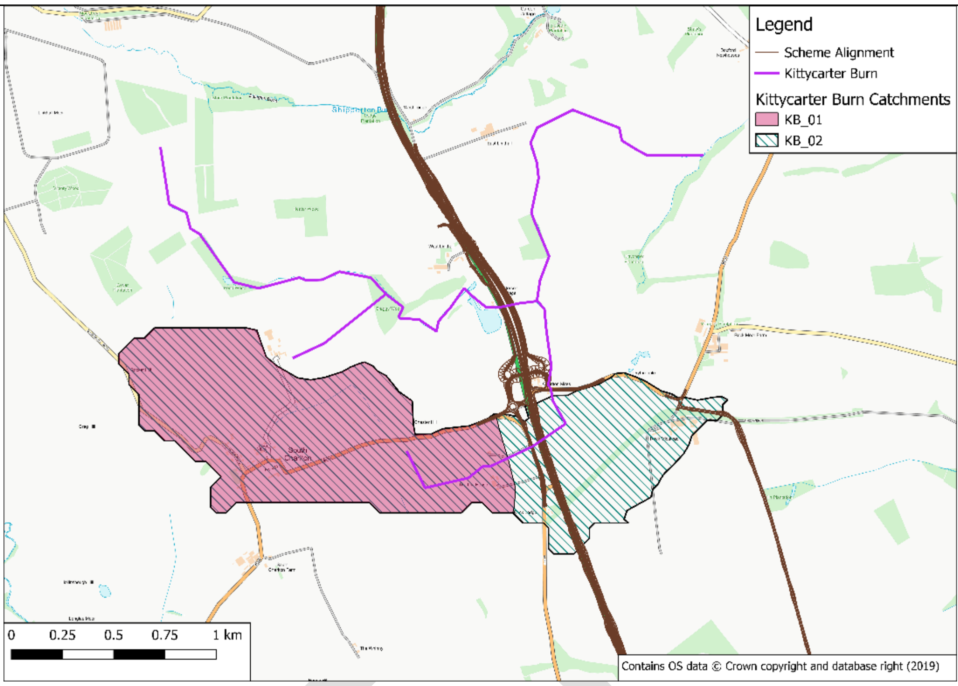


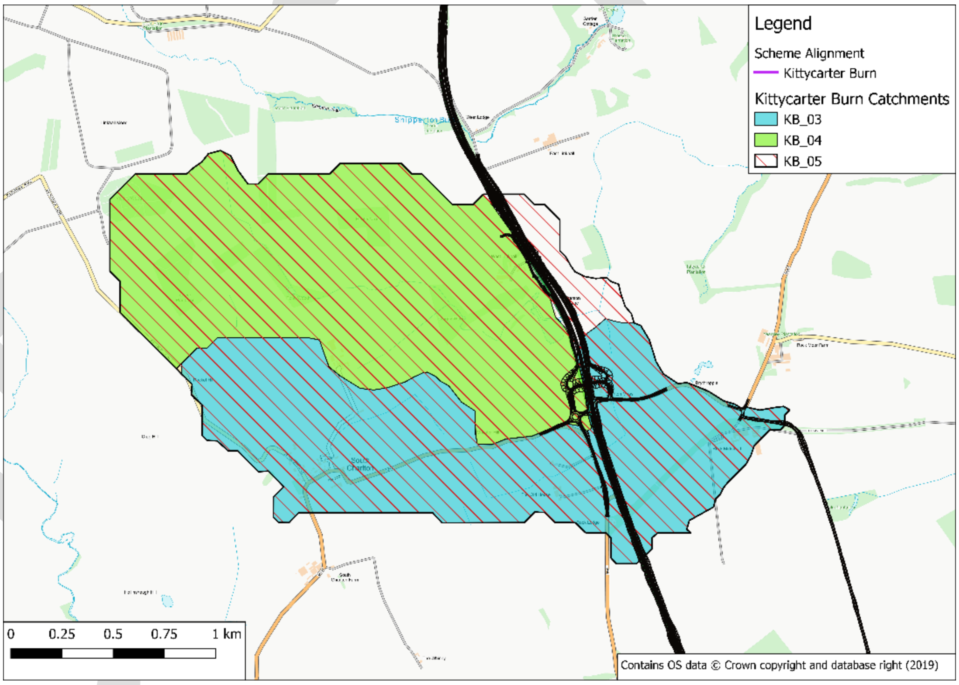
<u>Kittycarter Burn</u>

In the upper catchment of the Kittycarter Burn, there are two branches of the watercourse which both flow in a west to east alignment before joining together approximately 2km downstream. These two upper branches of Kittycarter Burn are the focus of this assessment. The northern catchment has a total catchment area of 1.6km². The southern catchment has a total catchment area of approximately 4.0km². The catchment is predominantly a rural catchment consisting of agricultural land and the B6341 road. There is a large lake located next to the A1 in the northern catchment, and a small pond further upstream in this catchmemt.

The BGS hydrogeology information on the FEH Web Service shows the catchments have moderate permeability. The BGS bedrock geology information shows the catchment geology comprises limestone, sandstone, siltstone and mudstone which are moderately permeable rocks; overlain by glaciofluvial sand and gravel superficial deposits. Soil mapping indicates the these upper catchments are underlain by freely draining, slightly acid loamy soils, which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 4 shows the individual sub-catchments for Kittycarter Burn.

Figure 4 Kittycarter Burn sub-catchments



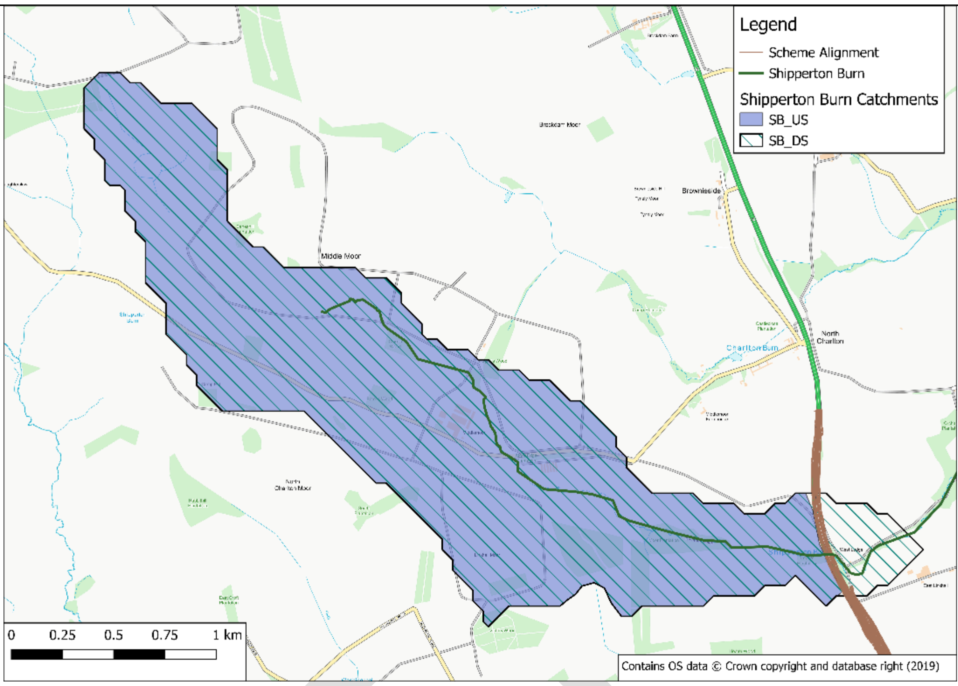


Shipperton Burn

The total catchment area is approximately 3.13km². The catchment is predominately a rural catchment consisting of agricultural land. No lakes, reservoirs or artificial features have been identified from OS mapping.

The BGS hydrogeology information on the FEH web service shows the catchment geology to have moderate to high permeability. The BGS bedrock geology information shows the catchment geology comprises a combination of limestone, sandstone, siltstone and mudstone overlain by till superficial deposits. Soil mapping indicates that the catchment is predomnatley underlain by slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 5 shows the individual adjusted FEH catchments for Shipperton Burn. The original FEH catchments have been adjusted where the road cuts across the catchment.

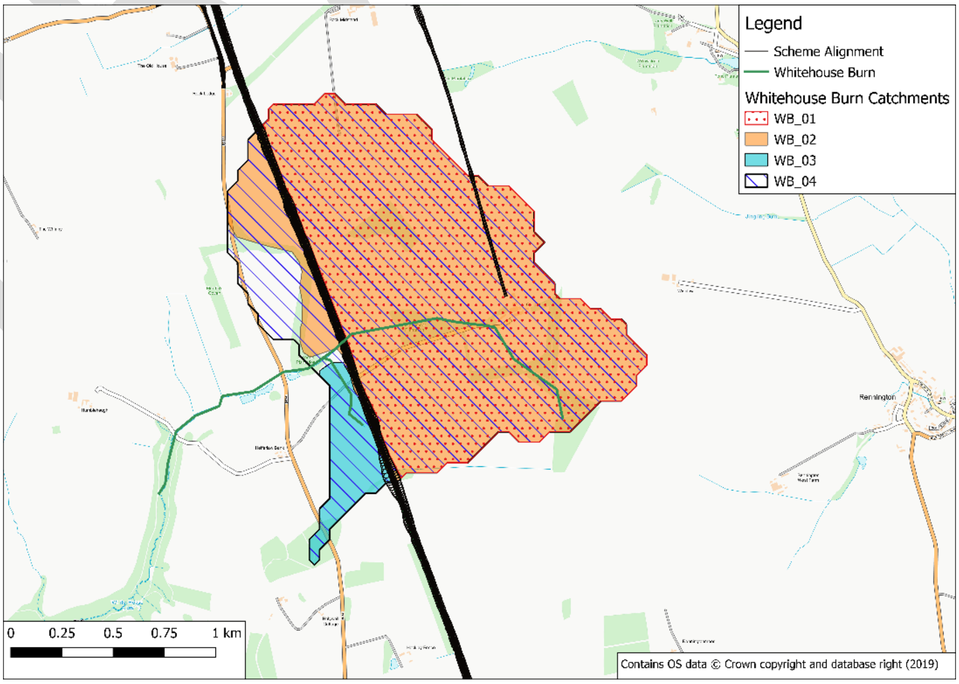
Figure 5 Shipperton Burn Catchments



<u>White House Burn</u>

The total catchment area is approximately 2.4km². The catchment is predominately a rural catchment consisting of agricultural land and the A1 road. No lakes, reservoirs or artificial features have been identified from OS mapping. The BGS hydrogeology information on the FEH web service shows the catchment geology to have moderate permeability. The BGS bedrock geology information shows the catchment geology comprises sandstone, mudstone and siltstone overlain by till superfical deposits. Soil mapping indicates the catchment is underlain by slowly permeable seasonally wet acid loamy and clayey soil which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 6 shows the individual FEH catchments for White House Burn.

Figure 6 White House Burn Catchments



All of the catchment boundaries have been assessed using local LiDAR data to ensure that they are appropriate and reflect the local topography.

Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – Version 7, October 2018

Gauging stations (flow or level)

An online search for potential gauging stations within the vicinity of the site and within the wider project area was undertaken using the FEH Web Service looking at all NRFA sites. White House Burn and Denwick Burn are tributaries of the River Aln. There is a gauging station at Hawkhill on the River Aln. However, the River Aln drains a significantly bigger catchment area than these subject sites.

There are no gauging stations in the same catchments or further downstream from Shipperton Burn or Kittycarter Burn. Shipperton Burn and Kittycarter Burn drain directly to the coastline, less than 10km to the east.

The four nearest gauges to the subject sites are located on either the River Aln, River Till or the River Coquet, and as such drain catchments significantly larger catchments than the subject sites. In comparison, the subject sites are significantly smaller and drain rural, agricultural land.

The four nearest gauges to the subject sites are discussed further as part of the donor site assessment in Section 3.

Water-course	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catch-ment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
N/A							

Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	**Other comments on station and flow data quality** – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Give link/reference to any further data quality checks carried out						

Rating equations

Station name	**Type of rating** e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	**Reasons** – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Give link/reference to any rating reviews carried out			

Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)					
Historic flood data – give link to historic review if carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies					
Other data or information (e.g. groundwater, tides)					

Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	FEH is considered to be appropriate; both the statistical and ReFH2.2 methods will be used as part of the study. There are no significant artificial influences.
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	All four of the hydraulic models require full hydrographs. A catchment wide storm scenario is considered appropriate for the flow estimation for each watercourse. Flooding from the watercourses is likely to be controlled by the capacity and hydraulic characteristics of the watercourse and structures located on the watercourse. Peak flows, rather than volume, are likely to be the main factor considered. Site specific considerations are noted below. <u>Denwick Burn</u> The Denwick Burn flows from north to south and flows underneath the existing A1 alignment and the B1340 road. The Denwick Burn is a tributary of the River Aln, and discharges into the River Aln in Hawkhill, just downstream of Alnwick. Flow estimates are required at 7 locations on the Denwick Burn upstream of the existing A1 alignment and downstream of the proposed local access roads to understand contributing flows downstream of the proposed crossings and to resolve the flow contributions from both upstream watercourses. <u>Kittycarter Burn</u> The Kittycarter Burn and its tributaries flow west to east and discharges into the sea at Embleton Bay. The River Lyne flows in a west to east direction and passes

	underneath the existing A1 alignment in two locations. Flow estimates are required for the Kittycarter Burn in 5 locations so that the implications of the new crossings can be assessed. <u>White House Burn</u> The White House Burn flows from east to west and passes beneath the existing A1 and the B6341 road. Flow estimates are required at 4 locations on White House Burn so that the implications of the new road can be assessed. <u>Shipperton Burn</u> The Shipperton Burn flows north-west to north-east and flows underneath the existing A1 alignment through Shipperton Bridge. Flow estimates are required for the Shipperton Burn existing culvert on the upstream side of the A1, and slightly further downstream from the A1 so that the implications of the new road can be assessed.
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	There are no unusual characteristics identified in any of the catchments.
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	Both the Statistical and ReFH2.2 methods were assessed in order to allow for a comparison of both methods. Hydrographs are required for all of the models which will be derived using the ReFH2.2 method and scaled if appropriate. A description of the conceptual models are provided above.
Software to be used (with version numbers)	WINFAP-FEH v4[1] / ReFH2.2 Design Flood Modelling Software

[1] WINFAP-FEH v4 © Wallingford HydroSolutions Limited and NERC (CEH) 2016.

2 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. To make a clear distinction between the different hydrological inputs for the hydraulic models, the tables below have been categorised by colour.

Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH Web Service (km^2)	Revised AREA if altered
DB_01	Denwick Burn	Upstream extent of the Denwick Burn model on the west side of the A1	419150	616650	0.98	0.94
DB_02	Denwick Burn	Denwick Burn	419350	616450	2.33	2.33
DB_03	Denwick Burn	Denwick Burn	419600	615650	2.95	0.25
DB_04	Tributary of Denwick Burn	Denwick Burn	419600	615650	2.95	0.09
DB_05	Tributary of Denwick Burn	Denwick Burn	419600	615650	2.95	2.91
DB_06	Tributary of Denwick Burn	Tributary of Denwick Burn joining from west of the A1	419600	615650	2.95	0.45
DB_07	Denwick Burn	Downstream extent of the Denwick Burn model on the east side of the A1	419750	615300	3.81	3.76
KC_01	Kittycarter Burn	Upstream extent of the southern fork of the Kittycarter Burn, west of the A1	417800	621300	-	1.09
KC_02	Kittycarter Burn	Southern fork of the Kittycarter Burn, south of B6347	417800	621300	-	1.64
KC_03	Kittycarter Burn	Downstream extent of southern fork of Kittycarter Burn	417800	621300	1.88	1.78
KC_04	Kittycarter Burn	Upstream extent of northern fork of Kittycarter Burn, west of the A1	418050	621100	-	2.01
KC_05	Kittycarter Burn	Downstream extent of the Kittycarter Burn, downstream of confluence of northern and southern forks.	418050	621100	2.23	3.98
WH_01	White House Burn	Upstream extent of the White House Burn model on the upstream of A1.	418400	618800	-	1.86
WH_02	White House Burn	White House Burn immediately downstream of A1.	418350	618800	-	2.11
WH_03	White House Burn	Unnamed tributary of White House Burn.	418300	618750	-	0.16

Site code	Watercourse	Site	Easting	Northing	AREA on FEH Web Service (km^2)	Revised AREA if altered
WH_04	White House Burn	White House Burn downstream model extent.	418250	618700	2.41	-
SB_US	Shipperton Burn upstream	Shipperton Burn on the upstream side of the A1	417450	622050	2.93	-
SB_DS	Shipperton Burn downstream	Downstream extent of the Shipperton Burn model located on the downstream side of the A1 just downstream of Shipperton Bridge	417450	622050	3.13	-
Reasons for choosing above locations	DS model extents and location of main inflows to study watercourses.					

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

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT 2000	FPEXT
DB_01	1.00	0.45	0.313	0.81	56.4	720	39.88	0.000	0.031
DB_02	1.00	0.45	0.317	1.12	42.0	718	41.02	0.000	0.041
DB_03	1.00	0.45	0.316	0.40	65.2	717	40.85	0.000	0.041
DB_04	1.00	0.45	0.316	0.28	48.0	717	40.85	0.000	0.041
DB_05	1.00	0.45	0.316	1.73	65.02	717	40.75	0.000	0.041
DB_06	1.00	0.45	0.315	0.74	47.6	715	40.63	0.000	0.041
DB_07	1.00	0.45	0.315	1.93	45.4	715	40.63	0.000	0.048
KC_01	1.000	0.45	0.515	1.05	33.02	726	32.24	0.000	0.088
KC_02	1.000	0.45	0.515	1.31	28.94	726	32.24	0.000	0.088
KC_03	1.000	0.45	0.496	1.37	28.77	721	33.24	0.000	0.088
KC_04	1.000	0.45	0.505	1.47	32.53	723	33.71	0.000	0.124
KC_05	1.000	0.45	0.313	1.42	30.17	720	33.02	0.000	0.124
WH_01	1.000	0.45	0.324	0.80	20.0	716	39.99	0.000	0.145
WH_02	1.000	0.45	0.324	1.51	20.0	716	39.99	0.000	0.145
WH_03	1.000	0.45	0.324	0.36	20.0	716	39.99	0.000	0.145
WH_04	1.000	0.45	0.324	1.02	20.0	716	39.99	0.000	0.145
SB_US	1.000	0.45	0.377	2.97	39.2	737	39.80	0.000	0.145
SB_DS	1.000	0.45	0.377	2.40	39.2	737	39.80	0.000	0.145

Checking Catchment Descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)

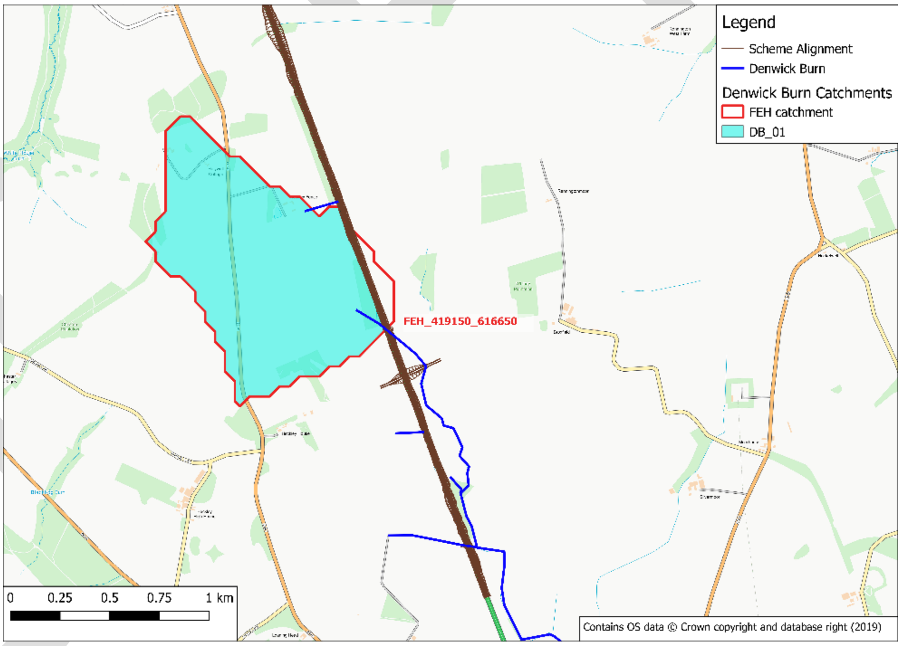
The study site catchment boundaries for all of the hydraulic models were based on the closest FEH catchments and adjusted using LiDAR data where available or OS data. The location of all of the flow nodes takes into consideration the different watercourses and tributaries within each study area contributing flow to the modelled watercourses and the location of the Scheme.

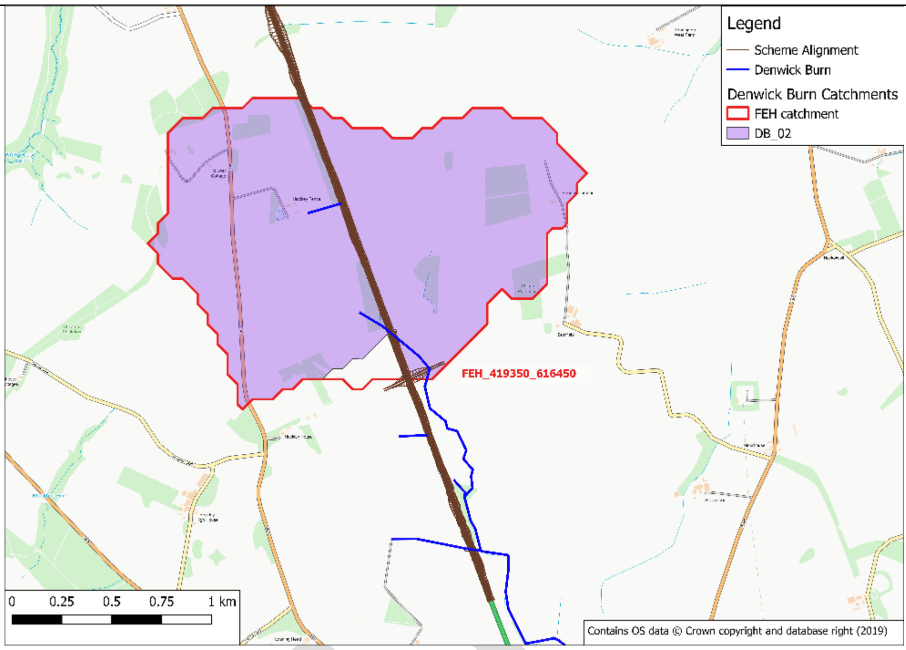
Figures 7 to 10 below show how the study catchment boundaries have been adjusted from the original FEH catchments.

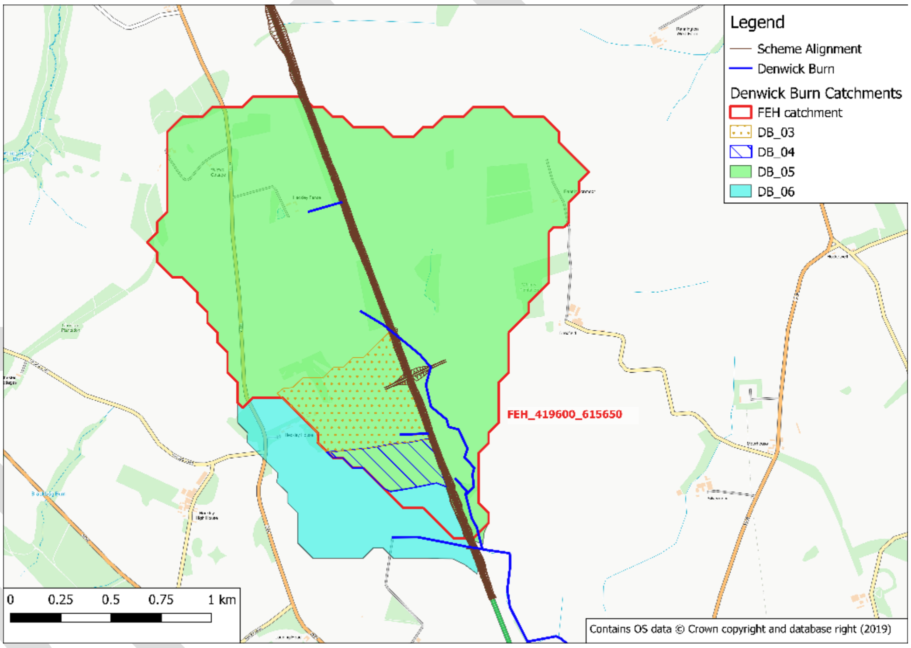
Figure 7 Denwick Burn Adjustments

Denwick Burn sub-catchments were based on four FEH catchments and adjusted as shown below.

Denwick Burn catchment	Original FEH catchment	Original FEH catchment area	Adjusted catchment area
BD_01	419150 616650	0.98	0.94
BD_02	419350 616450	2.33	2.27
BD_03	419600 615650	2.95	0.25
BD_04			0.09
BD_05			2.91
BD_06			0.45
BD_07	419750 615300	3.81	3.76




Legend
Scheme Alignment
Denwick Burn
Denwick Burn Catchments
FEH catchment
DB_02
FEH_419350_616450
0 0.25 0.5 0.75 1 km
Contains OS data © Crown copyright and database right (2019)


Legend
Scheme Alignment
Denwick Burn
Denwick Burn Catchments
FEH catchment
DB_03
DB_04
DB_05
DB_06
FEH_419600_615650
0 0.25 0.5 0.75 1 km
Contains OS data © Crown copyright and database right (2019)

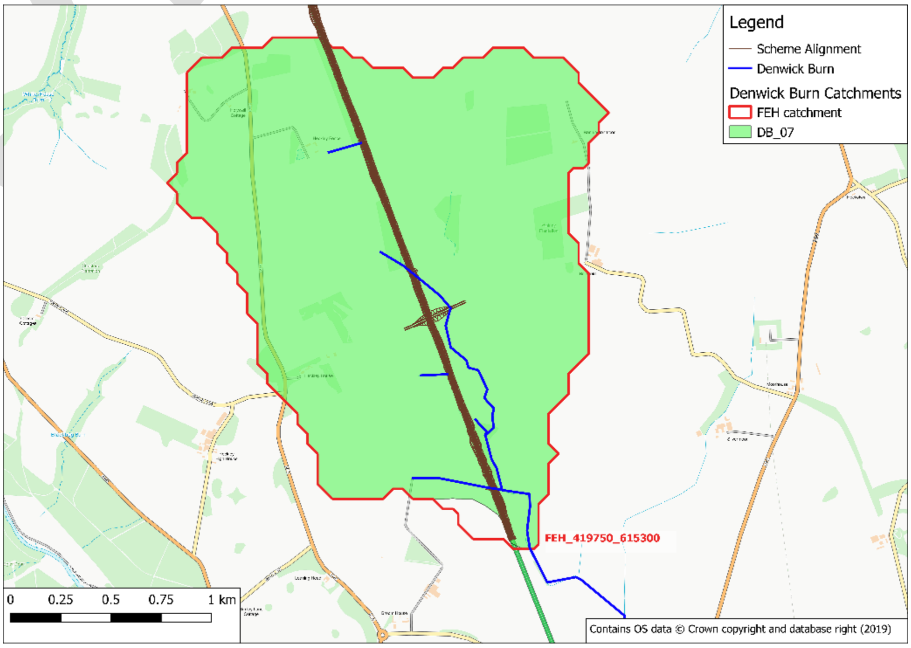

Legend
Scheme Alignment
Denwick Burn
Denwick Burn Catchments
FEH catchment
DB_07
FEH_419750_615300
0 0.25 0.5 0.75 1 km
Contains OS data © Crown copyright and database right (2019)

Figure 8a Kittycarter Burn Adjustments

KB_01, KB_02 and KB_03 were based on the northern Kittycarter Burn catchment. The area of all three catchments was adjusted from the original FEH catchment 417800 621300, as shown below.

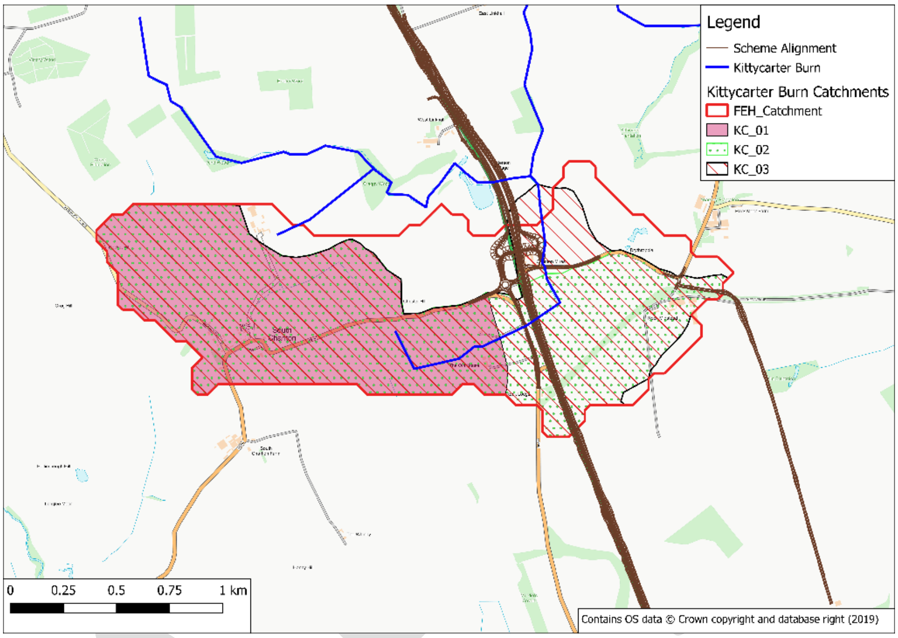


Figure 8b Kittycarter Burn Adjustments

KB_04 and KB_05 were based on the southern Kittycarter Burn catchment. The area of both catchments were adjusted from the original FEH catchment 417800 621100, as shown below.

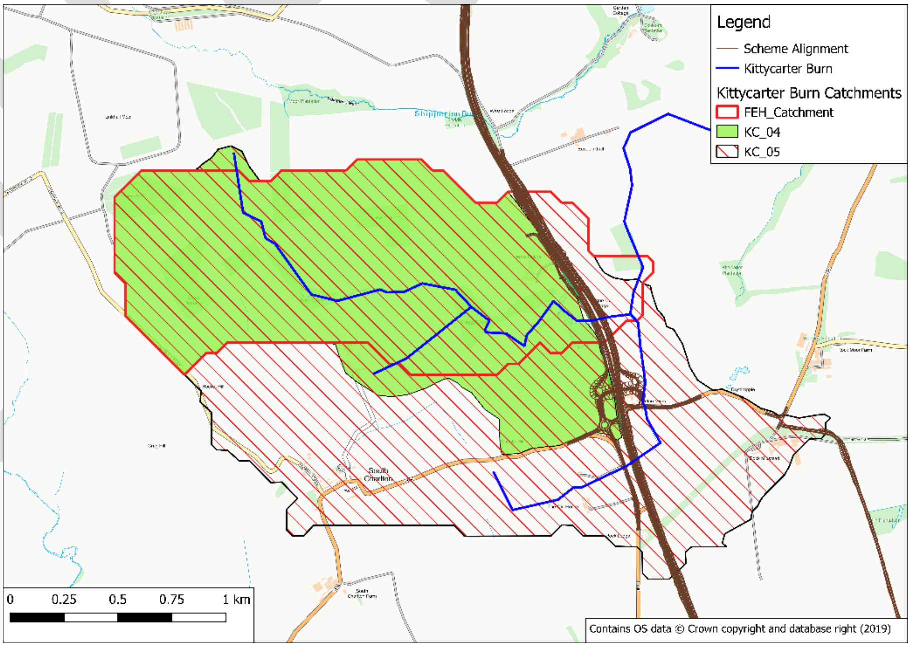


Figure 9 Shipperton Burn Adjustments

One FEH Shipperton Burn catchment was adjusted to reflect the water draining through the existing culvert on the western side of the existing A1. The Shipperton Burn upstream catchment (SB_US) drains to the western side of the A1, and Shipperton Burn_DS (SB_DS) is the downstream extent of the model on the east side of the A1.

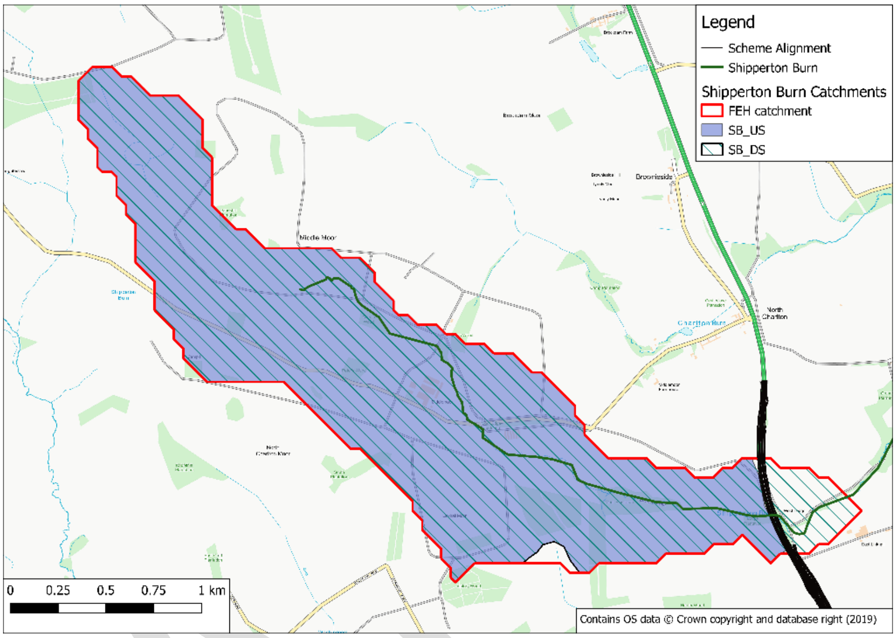
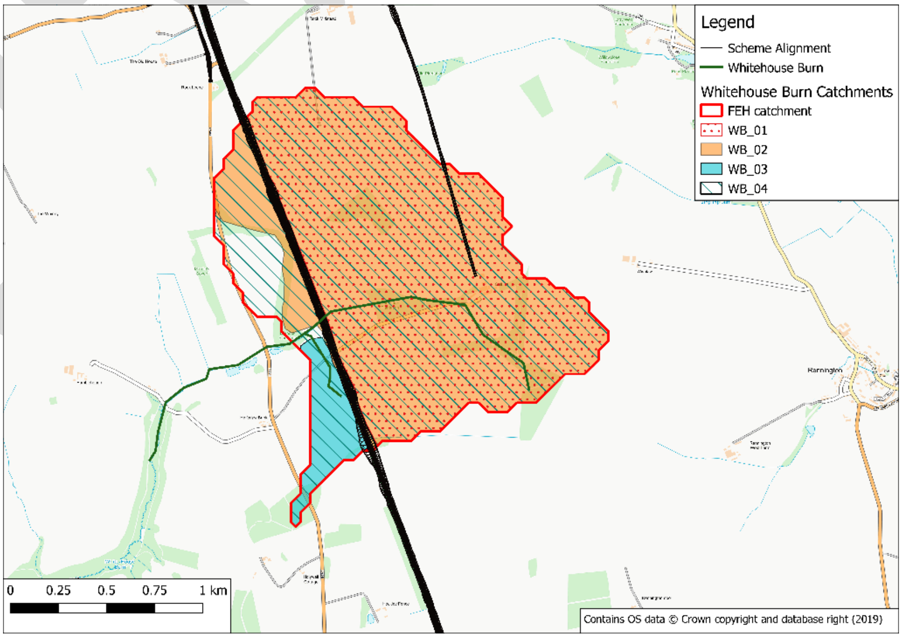


Figure 10 White House Burn Adjustments

Three catchments were adjusted based on one FEH catchment for the downstream extent of the model on White House Burn. The tributary catchment is not identified on the FEH online service, therefore catchment descriptors have been derived using the FEH catchment as a substitute.



Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	The following checks were undertaken for each of the catchments: • AREA – original FEH catchment areas have been checked, and adjusted if necessary, for all catchments based on LiDAR and culvert locations. The areas have been measured using QGIS. • BFIHOST and SPRHOST – Values adopted from the FEH catchments. FEH values were checked against soil mapping and appear to be reasonable. • FARL – No additional ponds or reservoirs have been identified. As a result, the FEH values were deemed to be representative for all catchments. • URBEXT2000 – The FEH URBEXT values were checked against 10k OS mapping and appear to be reasonable. The values were updated to 2018 using the FEH UEF formula. • PROPWET / SAAR – FEH values adopted. • DPSBAR – Manual check in GIS was completed using LiDAR data where available. For Denwick Burn only, DPSBAR has been changed for DB_03, DB_04 and DB_06 using QGIS to calculate the gradient and finding the average for the adjusted catchment. The FEH values were deemed appropriate for all other catchments. • DPLBAR – Due to the long shapes of catchments Denwick Burn 03, 04 and 06 (DB_03, DB_04 and DB_06), 5 measurements of drainage pathways were taken across these catchments in QGIS, and the average taken to estimate DPLBAR; – For all catchments where area had been adjusted from the original FEH catchment area, the FEH equation (equation 7.1) was used to calculate the DPLBAR; – The DPLBAR for KB_05 is the weighted average of DPLBAR for KB_03 and KB_04;
Source of URBEXT	FEH URBEXT2000 (updated to 2018) were used for all of the FEH catchments.
Method for updating of URBEXT	Updated to 2018 using the standard FEH UEF formula for URBEXT2000.

3 Statistical method

Search for donor sites for QMED (if applicable)

Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	Nearby gauging stations within the vicinity of the site and within the wider project area were considered as potential donor sites for all the study watercourses. The four nearest gauges to the subject sites are located on either the River Aln, River Till or River Coquet, and as such drain catchments significantly larger catchments than the subject sites. In comparison, the subject sites are significantly smaller and drain rural, agricultural land. Therefore, it has been concluded that no donor sites were appropriate to use for this study. Table 2 below details the catchment descriptors for each of the potential donor sites considered.

Table 2 Potential Donor Sites

Station ID	Station Name	Area	BFIHOST	SPRHOST	FARL	URBEXT	PROPWET	SAAR (mm)	DPSBAR (m/km)	DPLBAR (km)
22004	Aln @ Hawkhill	202.91	0.43	35.72	1.00	0.006	0.45	758	80.00	19.60
22001	Coquet @ Morwick	578.21	0.39	42.53	0.99	0.000	0.45	850	110.10	45.11
21031	21031 Till @ Etal	634.68	0.50	41.45	0.99	0.002	0.45	827	127.30	39.44
21806	Till @ Heaton Mill	655.54	0.52	41.40	0.99	0.002	0.45	822	125.20	44.51
22009	Coquet @ Rothbury	345.99	0.40	45.50	0.99	0.000	0.45	905	140.70	26.43
22003	Usway Burn @ Shillmoor	21.88	0.30	56.92	1.00	0.000	0.45	1056	205.20	9.28

Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjust-ment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)
22004	Rejected - Catchment area significantly larger than study sites.	AM	-	63.23	36.64	1.73
22001	Rejected - Catchment area significantly larger than study sites.	AM	-	152.44	124.66	1.22
21031	Rejected - There is potential for bypassing at this station during high flows and there are some abstractions in the area. Catchment area significantly larger than study sites.	AM	-	82.90	93.08	0.89

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjust-ment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)
22009	Rejected - Recent gaugings suggested peak events may be underestimated. Catchment area significantly larger than study sites.	AM	-	133.00	91.93	1.45
22003	Rejected - Discontinued in 1980 and weir plates removed. Recommissioned as a level-only station in 1995 for flood warning. Only has a marginal impact on QMED peak flow.	AM	-	16.17	14.70	1.32
Which version of the urban adjustment was used for QMED at donor sites, and why? Note: The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8).				UAF applied in WinFAP4		

Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.2)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, (A/B)[a]	If more than one donor		
							Weight	Weighted average adjustment factor	
DB_01	CD	0.43	N/A						0.43
DB_02	CD	0.89							0.89
DB_03	CD	0.14							0.14
DB_04	CD	0.06							0.06
DB_05	CD	1.10							1.10
DB_06	CD	0.22							0.22
DB_07	CD	1.37							1.37
KC_01	CD	0.30	N/A						0.30
KC_02	CD	0.42							0.42
KC_03	CD	0.45							0.45
KC_04	CD	0.53							0.53
KC_05	CD	0.92							0.92
WH_01	CD	0.75	N/A						0.75
WH_02	CD	0.83							0.83
WH_03	CD	0.09							0.09
WH_04	CD	0.93							0.93
SH_US	CD	1.11	N/A						1.11
SH_DS	CD	1.05							1.05
Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?						The QMED values are reasonably consistent with the increases in catchment area for all the study areas.			

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.2)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average adjustment factor	
Which version of the urban adjustment was used for QMED, and why?							No urban adjustment has been applied.		

Notes
Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.
When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added.
When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050 should be used. If the original FEH equation has been used, say so and give the reason why.
The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.
The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.2. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.
If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

Derivation of pooling groups

The composition of each pooling group is provided in the Annex.

A single pooling group was derived in WINFAP for the downstream catchments for each study site (DB_07, KC_05, WB_04 and SB_DS). A single pooling group was considered appropriate to be applied to all of the study nodes within each study area considering their size and hydrological similarities. The scope of the study allowed for a brief review of the pooling group. Sites marked not suitable for pooling were reviewed in more detail to see if they were reasonable. Sites that were marked as discordant were reviewed in more detail. Where no underlying catchment factors were identified that would cause the discordance, the station was viewed to be reasonable and remained in the pooling group. A number of sites at the top of each pooling group were reviewed in greater detail using the online NRFA data. Each pooling group achieved the 500 years of data required with the catchments included in the pooling group being relatively hydrologically similar to the subject catchments.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons Note also any sites that were investigated but retained in the group.	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
DB_Pooling Group	DB_07	No	All sites marked as unsuitable for pooling were removed based on a high level review. Discordant site 49006 (Camel @ Camelford) was removed following high level review due to flat growth curve. Discordant site 206006 (Annalong @ Recorder) was removed. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data).	L-CV – 0.241 L-skew – 0.268
KC_Pooling Group	KC_05	No	All sites marked as unsuitable for pooling were removed based on a high level review. Discordant site 49006 (Camel @ Camelford) was removed following high level review due to flat growth curve. Discordant site 206006 (Annalong @ Recorder) was removed. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data). Discordant site 20002 (West Peffer Burn @ Luffness) removed as high flow calibration not reliable and channel known to be dammed upstream. 44008 (South Winterbourne @ Winterbourne Steepleton) and 91802 (Allt Leachdach @ Intake) added to make 500+ years of data.	L-CV – 0.240 L-skew – 0.268
WH_Pooling Group	WH_04	No	All sites marked as unsuitable for pooling were removed based on a high level review. (Camel @ Camelford) was removed following high level review due to flat growth curve. Discordant site 206006 (Annalong @ Recorder) was removed. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data). Discordant site 20002 (West Peffer Burn @ Luffness) removed as high flow calibration not reliable and channel known to be dammed upstream.	L-CV – 0.219 L-skew – 0.227
SH_Pooling Group	SB_DS	No	All sites marked as unsuitable for pooling were removed based on a high level review. (Camel @ Camelford) was removed following high level review due to flat growth curve. Discordant site 206006 (Annalong @ Recorder) was removed. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data).	L-CV – 0.237 L-skew – 0.280

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

Derivation of flood growth curves at subject sites

Site code	**Method** (SS, P, ESS, J)	If P, ESS or J, name of pooling group (0)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	**Parameters of distribution** (location, scale and shape) after adjustments	Growth factor for 100-year return period
DB_07	P	DB_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.237 Shape – -0.268 Bound – 0.113	3.148
KC_05	P	KC_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.236 Shape – -0.268 Bound – 0.117	3.140
WH_04	P	WH_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.211 Shape – -0.260 Bound – 0.188	2.872
SB_DS	P	SB_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.232 Shape – -0.218 Bound – -0.170	3.175

Notes

Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis

A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.

Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010).

Growth curves were derived using the revised procedures from Science Report SC050050 (2008).

Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
DB_01	0.43	0.60	0.74	0.95	1.13	1.26	1.36	1.63	2.48
DB_02	0.89	1.25	1.53	1.95	2.34	2.61	2.81	3.36	5.13
DB_03	0.14	0.19	0.23	0.30	0.36	0.40	0.43	0.52	0.79
DB_04	0.06	0.08	0.10	0.13	0.15	0.17	0.18	0.22	0.33
DB_05	1.10	1.54	1.89	2.42	2.90	3.22	3.47	4.16	6.35
DB_06	0.22	0.31	0.38	0.49	0.59	0.65	0.71	0.85	1.29
DB_07	1.37	1.91	2.33	2.99	3.59	3.99	4.30	5.15	7.85
KC_01	0.30	0.42	0.51	0.65	0.78	0.87	0.94	1.12	1.71
KC_02	0.42	0.59	0.72	0.92	1.11	1.23	1.33	1.59	2.42
KC_03	0.45	0.63	0.78	0.99	1.19	1.32	1.43	1.71	2.60
KC_04	0.53	0.73	0.90	1.15	1.38	1.53	1.65	1.98	3.01
KC_05	0.92	1.29	1.57	2.01	2.41	2.68	2.89	3.46	5.28
WH_01	0.75	1.01	1.21	1.53	1.81	2.00	2.14	2.54	3.79
WH_02	0.83	1.12	1.35	1.69	2.01	2.22	2.38	2.82	4.22
WH_03	0.09	0.12	0.15	0.19	0.22	0.24	0.26	0.31	0.46

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
WH_04	0.93	1.26	1.51	1.90	2.25	2.48	2.66	3.16	4.72
SH_US	1.54	2.07	2.46	3.04	3.55	3.89	4.15	4.88	7.05
SH_DS	1.53	2.05	2.44	3.01	3.52	3.85	4.11	4.83	6.98

4 Revitalised flood hydrograph (ReFH) method 2.2

Parameters for ReFH2.2 model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible so it is not essential to enter them in the table.

Site code	**Method:** OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	**Tp (hours)** Time to peak	**C_{max} (mm)** Maximum storage capacity	**BL (hours)** Baseflow lag	**BR** Baseflow recharge
DB_01	CD	1.17	236.89	21.11	0.80
DB_02	CD	1.55	239.36	22.84	0.81
DB_03	CD	1.00	238.74	18.21	0.81
DB_04	CD	1.00	238.74	16.84	0.81
DB_05	CD	1.96	238.74	25.06	0.81
DB_06	CD	1.18	238.12	20.78	0.81
DB_07	CD	2.07	238.12	25.61	0.81
KC_01	CD	1.61	400.33	30.22	1.42
KC_02	CD	1.91	400.33	31.71	1.42
KC_03	CD	1.96	400.33	32.02	1.42
KC_04	CD	1.97	381.05	31.79	1.36
KC_05	CD	1.97	390.07	31.89	1.39
WH_01	CD	1.62	243.75	21.50	0.83
WH_02	CD	2.33	243.75	24.70	0.83
WH_03	CD	1.03	243.75	18.07	0.83
WH_04	CD	1.86	243.75	22.67	0.83
SB_US	CD	2.77	279.73	31.38	0.99
SB_DS	CD	2.45	279.73	29.95	0.99

Brief description of any flood event analysis carried out (further details should be given below or in a project report)	N/A

Design events for ReFH2 method (original)

Site code	**Urban or rural**	**Season of design event (summer or winter)**	**Storm duration (hours)**	**Storm area for ARF** (if not catchment area)
DB_01	Urban	Winter	2.25	-
DB_02	Urban	Winter	2.75	-
DB_03	Urban	Winter	3.25	-
DB_04	Urban	Winter	3.25	-
DB_05	Urban	Winter	3.25	-
DB_06	Urban	Winter	3.25	-
DB_07	Urban	Winter	3.5	-
KC_01	Urban	Winter	3.5	-
KC_02	Urban	Winter	3.5	-
KC_03	Urban	Winter	3.5	-
KC_04	Urban	Winter	3.5	-
KC_05	Urban	Winter	3.5	-
WH_01	Urban	Winter	3.25	-
WH_02	Urban	Winter	3.25	-

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
WH_03	Urban	Winter	3.25	-
WH_04	Urban	Winter	3.25	-
SB_US	Urban	Winter	4.5	-
SB_DS	Urban	Winter	4.5	-
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?			Storm durations are likely to be changed. Storm duration, SCF and ARF based on the flow nodes DB_07, KC_05, WH_04 and SB_DS, as they represent the whole catchment for each study site.	

New critical storm durations for each watercourse were calculated using a trial-and-error approach to find the largest peak flow. These updated critical storm durations have been used to create hydrographs. As a result, these values have been used in the sensitivity testing of the hydraulic models. The updated design events for the ReFH2 method are shown in the table below.

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
DB_01	Urban	Winter	-	-
DB_02	Urban	Winter	-	-
DB_03	Urban	Winter	-	-
DB_04	Urban	Winter	-	-
DB_05	Urban	Winter	-	-
DB_06	Urban	Winter	-	-
DB_07	Urban	Winter	6.1	0.706
KC_01	Urban	Winter	-	-
KC_02	Urban	Winter	-	-
KC_03	Urban	Winter	-	-
KC_04	Urban	Winter	-	-
KC_05	Urban	Winter	6.5	0.709
WH_01	Urban	Winter	-	-
WH_02	Urban	Winter	-	-
WH_03	Urban	Winter	-	-
WH_04	Urban	Winter	6.1	0.706
SB_US	Urban	Winter	-	-
SB_DS	Urban	Winter	8.5	0.974

Flood estimates from the ReFH2 method

The table below shows the flood estimates from the ReFH2 method based on the original storm durations as selected by ReFH2.

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
DB_01	0.73	1.08	1.33	1.69	1.99	2.18	2.32	2.72	3.94
DB_02	1.56	2.27	2.78	3.51	4.11	4.51	4.80	5.61	8.14
DB_03	0.12	0.17	0.20	0.26	0.30	0.33	0.35	0.41	0.59
DB_04	0.06	0.08	0.10	0.12	0.14	0.16	0.17	0.19	0.28
DB_05	1.76	2.51	3.05	3.82	4.47	4.89	5.20	6.06	8.79
DB_06	0.28	0.39	0.48	0.60	0.70	0.77	0.82	0.96	1.39
DB_07	2.23	3.16	3.83	4.79	5.59	6.11	6.51	7.58	10.97
KC_01	0.29	0.41	0.50	0.63	0.74	0.81	0.87	1.02	1.50
KC_02	0.44	0.62	0.75	0.95	1.11	1.22	1.30	1.52	2.24
KC_03	0.47	0.67	0.82	1.03	1.20	1.32	1.41	1.65	2.43
KC_04	0.56	0.79	0.97	1.21	1.42	1.56	1.67	1.96	2.88
KC_05	1.09	1.55	1.89	2.38	2.79	3.06	3.27	3.84	5.64

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
WH_01	1.14	1.64	2.00	2.51	2.94	3.22	3.43	4.00	5.80
WH_02	1.30	1.85	2.26	2.84	3.33	3.64	3.88	4.53	6.57
WH_03	0.10	0.14	0.18	0.22	0.26	0.28	0.30	0.35	0.51
WH_04	1.48	2.11	2.58	3.24	3.79	4.15	4.42	5.16	7.48
SB_US	1.27	1.76	2.13	2.64	3.09	3.38	3.61	4.23	6.17
SB_DS	1.34	1.86	2.25	2.79	3.26	3.57	3.82	4.47	6.52

The table below shows the flood estimates from the ReFH2 method based on the updated critical storm durations that has been used during the sensitivity testing of the hydraulic models.

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
DB_01	0.87	1.19	1.43	1.78	2.07	2.26	2.41	2.81	4.07
DB_02	1.82	2.51	3.01	3.72	4.33	4.74	5.05	5.90	8.54
DB_03	0.24	0.33	0.40	0.50	0.58	0.63	0.68	0.79	1.14
DB_04	0.09	0.12	0.15	0.18	0.21	0.23	0.24	0.29	0.41
DB_05	2.08	2.86	3.42	4.24	4.93	5.39	5.74	6.71	9.69
DB_06	0.41	0.56	0.68	0.84	0.98	1.07	1.14	1.33	1.93
DB_07	2.62	3.58	4.30	5.32	6.19	6.76	7.20	8.41	12.15
KC_01	0.39	0.54	0.65	0.81	0.95	1.04	1.11	1.31	1.92
KC_02	0.54	0.75	0.90	1.12	1.31	1.44	1.54	1.81	2.65
KC_03	0.58	0.80	0.96	1.19	1.40	1.54	1.64	1.93	2.83
KC_04	0.71	0.98	1.17	1.46	1.71	1.88	2.00	2.36	3.45
KC_05	1.35	1.86	2.24	2.78	3.26	3.58	3.82	4.50	6.59
WH_01	1.44	1.98	2.38	2.95	3.44	3.77	4.01	4.69	6.80
WH_02	1.34	1.84	2.20	2.73	3.18	3.48	3.71	4.33	6.27
WH_03	0.15	0.21	0.25	0.31	0.36	0.40	0.42	0.50	0.72
WH_04	1.73	2.38	2.86	3.54	4.13	4.52	4.81	5.62	8.14
SB_US	1.54	2.07	2.46	3.04	3.55	3.89	4.15	4.88	7.05
SB_DS	1.53	2.05	2.44	3.01	3.52	3.85	4.11	4.83	6.98

5 FEH rainfall-runoff method

Parameters for FEH rainfall-runoff model

Methods:
FEA : Flood event analysis
LAG : Catchment lag
DT : Catchment descriptors with data transfer from donor catchment
CD : Catchment descriptors alone
BFI : SPR derived from baseflow index calculated from flow data

Site code	Rural (R) or urban (U)	Tp(0): method	Tp(0): value (hours)	SPR: method	SPR: value (%)	BF: method	BF: value (m^3/s)	If DT, numbers of donor sites used (see Section 5.2) and reasons

Donor sites for FEH rainfall-runoff parameters

No.	Watercourse	Station	Tp(0) from data (A)	Tp(0) from CDs (B)	Adjustment ratio for Tp(0) (A/B)	SPR from data (C)	SPR from CDs (D)	Adjust-ment ratio for SPR (C/D)
1								
2								

Inputs to and outputs from FEH rainfall-runoff model

Site code	Storm duration (hours)	Storm area for ARF (if not catchment area)	Flood peaks (m^3/s) or volumes (m^3) for the following return periods (in years)							
			2							

Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?	

6 Discussion and summary of results

Comparison of results from different methods

This table compares peak flows from the ReFH2 method with those from the FEH Statistical method at all sites for two key return periods.

Site code	Ratio of ReFH2 peak flow to FEH Statistical peak flow					
	Return period 2 years			Return period 100 years		
	ReFH2	Statistical	Ratio (ReFH2 / Statistical)	ReFH2	Statistical	Ratio (ReFH2 / Statistical)
DB_01	0.87	0.43	2.02	2.41	1.36	1.77
DB_02	1.82	0.89	2.04	5.05	2.81	1.80
DB_03	0.24	0.14	1.71	0.68	0.43	1.58
DB_04	0.09	0.06	1.50	0.24	0.18	1.33
DB_05	2.08	1.10	1.89	5.74	3.47	1.65
DB_06	0.41	0.22	1.86	1.14	0.71	1.61
DB_07	2.62	1.37	1.91	7.20	4.30	1.67
KC_01	0.39	0.30	1.30	1.11	0.94	1.18
KC_02	0.54	0.42	1.29	1.54	1.33	1.16
KC_03	0.58	0.45	1.29	1.64	1.43	1.15
KC_04	0.71	0.53	1.34	2.00	1.65	1.21
KC_05	1.35	0.92	1.47	3.82	2.89	1.32
WH_01	1.44	0.75	1.92	4.01	2.14	1.87
WH_02	1.34	0.83	1.61	3.71	2.38	1.56
WH_03	0.15	0.09	1.67	0.42	0.26	1.62
WH_04	1.73	0.93	1.86	4.81	2.66	1.81
SB_US	1.54	1.05	1.47	4.15	3.34	1.24
SB_DS	1.53	1.10	1.39	4.11	3.51	1.17

Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The Statistical method peak flow estimates are low in comparison to the ReFH2 peak flow estimates for all of the study catchments. There is not a high level of certainty in either method due to the lack of suitable donor sites. Comparison of the Statistical and ReFH2 growth curves as shown in Figures 11 to 14 below indicates that the growth curves are relatively similar up to the 100 year return period. Beyond this point there is a slight difference, with the Statistical growth curve slightly steeper than the ReFH2 growth curve. The main difference between the two methodologies are the QMED peak flow estimates.

Figure 11 Denwick Burn Growth Curves

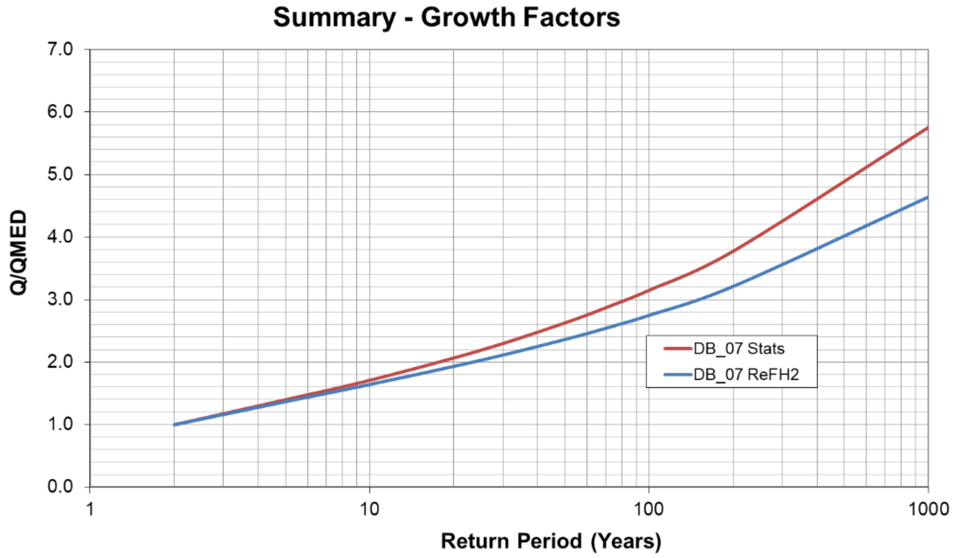


Figure 12 Kittycarter Burn Growth Curves

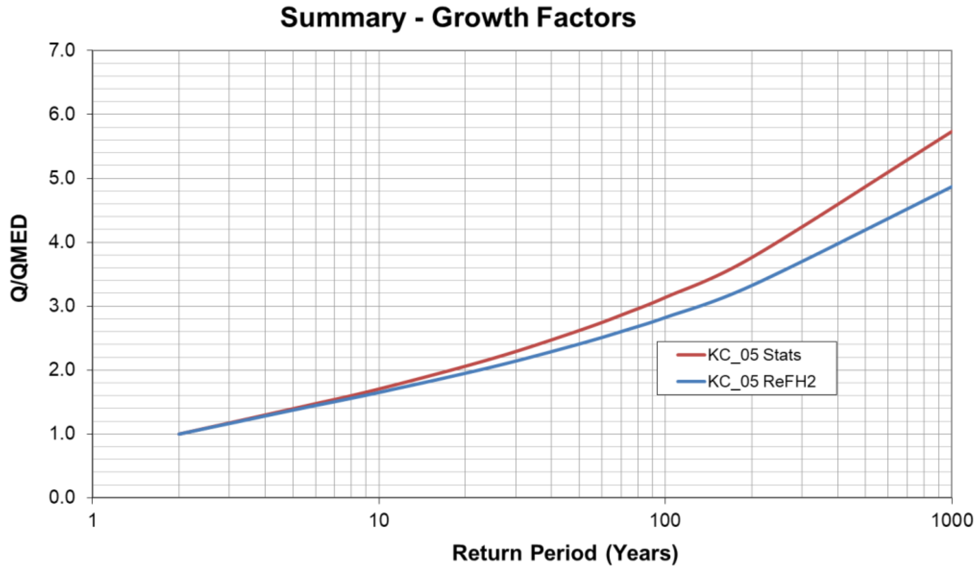


Figure 13 White House Burn Growth Curves

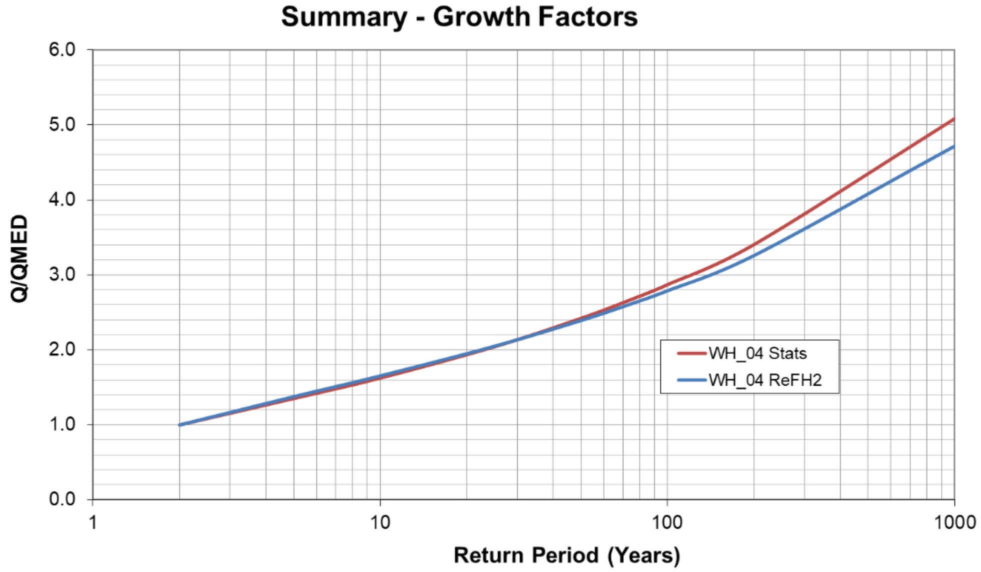
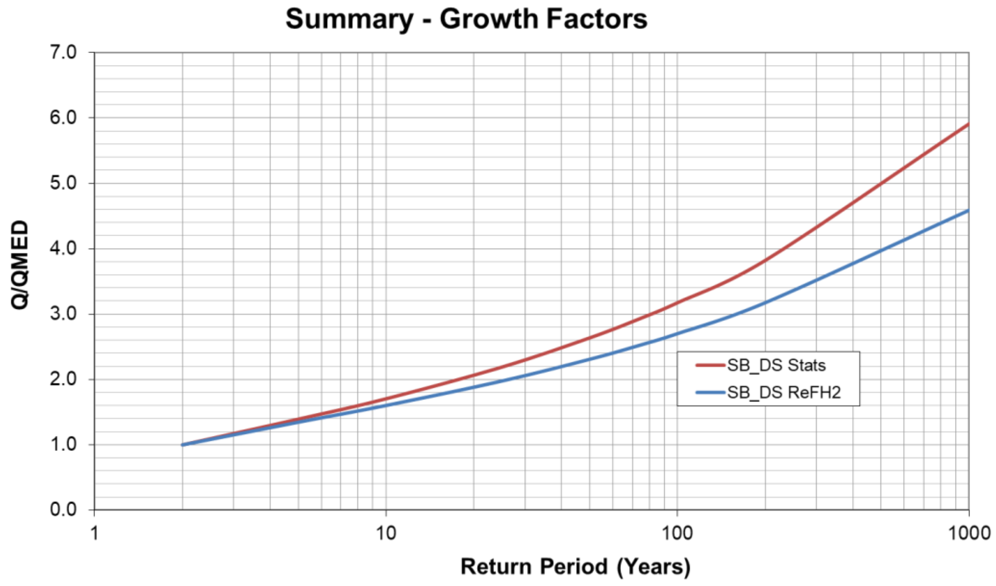


	Figure 14 Shipperton Burn Growth Curves



The ReFH2 QMED peak flow estimates using the updated critical storm durations for each watercourse catchment are shown in the table below. The updated estimates are higher than the estimates calculated using FEH Statistical method or the original ReFH2 QMED peak flow estimates.

Watercourse	Flow Node	Stats QMED (m^3/s)	Original ReFH2.2 QMED (m^3/s)	Updated ReFH2.2 QMED (m^3/s)
Denwick Burn	DB_07	1.37	2.23	2.62
Kittycarter Burn	KC_05	0.92	1.09	1.35
White House Burn	WH_04	0.93	1.48	1.73
Shipperton Burn	SB_DS	1.05	1.34	1.54

Taking into consideration the difference in peak flow estimates produced between the Statistical method and ReFH2 method, the final peak flow estimates have been derived using the ReFH2 method. The Statistical method is not based on donor transfer so there is less confidence in the QMED calculated using FEH CDs. The ReFH2 method potentially provides conservative flow estimates but is considered appropriate for flood risk assessment where a precautionary approach is advisable.

Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	Standard FEH assumptions.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	The estimated FEH Statistical peak flows produced are uncertain due to lack of any gauged data needed to calibrate and verify the methods. There were no suitable donor sites identified.
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH **3** 12.5 or the	Detailed assessment outside of scope.

factorial standard error from Science Report SC050050 (2008).	
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	N/A
Give any other comments on the study, for example suggestions for additional work.	Installation of a flow gauge (temporary or permanent) would help to verify and improve flow estimates for all of the study catchments.

Checks

<table>
<tr><td>Are the results consistent, for example at confluences?</td><td>The results are reasonably consistent, with the specific runoff rates for the tributaries across the different catchments reflecting the differences in catchment descriptors.
Considering the different response times, the peak flow estimates are considered reasonable however this will be checked with the hydraulic model and appropriate adjustments made if necessary.</td></tr>
<tr><td>What do the results imply regarding the return periods of floods during the period of record?</td><td>N/A</td></tr>
<tr><td>What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)</td><td>Table 6 100 year ReFH2 Growth Factors
<table>
<tr><th>Watercourse</th><th>Flow Node</th><th>Growth Factor</th></tr>
<tr><td>Denwick Burn</td><td>DB_07</td><td>2.75</td></tr>
<tr><td>Kittycarter Burn</td><td>KC_05</td><td>2.83</td></tr>
<tr><td>White House Burn</td><td>WH_04</td><td>2.87</td></tr>
<tr><td>Shipperton Burn</td><td>SB_DS</td><td>3.18</td></tr>
</table></td></tr>
<tr><td>If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?</td><td>Table 7 Ratios
<table>
<tr><th>Watercourse</th><th>Ratio</th></tr>
<tr><td>Denwick Burn</td><td>1.69</td></tr>
<tr><td>Kittycarter Burn</td><td>1.72 - 1.73</td></tr>
<tr><td>White House Burn</td><td>1.69 – 1.70</td></tr>
<tr><td>Shipperton Burn</td><td>1.70</td></tr>
</table></td></tr>
<tr><td>What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?</td><td>Table 8 shows the range of specific runoffs for the downstream flow nodes (DB_07, KC_05, WH_04, SB_DS) for each study area.

Table 8 Specific Runoffs
<table>
<tr><th>Watercourse</th><th>2 Year ($m^3/s/km$)</th><th>100 Year ($m^3/s/km$)</th></tr>
<tr><td>Denwick Burn</td><td>0.70</td><td>1.91</td></tr>
<tr><td>Kittycarter Burn</td><td>0.34</td><td>0.96</td></tr>
<tr><td>White House Burn</td><td>0.72</td><td>2.00</td></tr>
<tr><td>Shipperton Burn</td><td>0.52</td><td>1.42</td></tr>
</table></td></tr>
<tr><td>How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.</td><td>N/A</td></tr>
<tr><td>Are the results compatible with the longer-term flood history?</td><td>N/A</td></tr>
<tr><td>Describe any other checks on the results</td><td>N/A</td></tr>
</table>

Final results

The table below shows the final ReFH2 flows that were shown to take forward in this study.

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
DB_01	0.87	1.19	1.43	1.78	2.07	2.26	2.41	2.81	4.07
DB_02	1.82	2.51	3.01	3.72	4.33	4.74	5.05	5.90	8.54
DB_03	0.24	0.33	0.40	0.50	0.58	0.63	0.68	0.79	1.14
DB_04	0.09	0.12	0.15	0.18	0.21	0.23	0.24	0.29	0.41
DB_05	2.08	2.86	3.42	4.24	4.93	5.39	5.74	6.71	9.69
DB_06	0.41	0.56	0.68	0.84	0.98	1.07	1.14	1.33	1.93
DB_07	2.62	3.58	4.30	5.32	6.19	6.76	7.20	8.41	12.15
KC_01	0.39	0.54	0.65	0.81	0.95	1.04	1.11	1.31	1.92
KC_02	0.54	0.75	0.90	1.12	1.31	1.44	1.54	1.81	2.65
KC_03	0.58	0.80	0.96	1.19	1.40	1.54	1.64	1.93	2.83
KC_04	0.71	0.98	1.17	1.46	1.71	1.88	2.00	2.36	3.45
KC_05	1.35	1.86	2.24	2.78	3.26	3.58	3.82	4.50	6.59
WH_01	1.44	1.98	2.38	2.95	3.44	3.77	4.01	4.69	6.80
WH_02	1.34	1.84	2.20	2.73	3.18	3.48	3.71	4.33	6.27
WH_03	0.15	0.21	0.25	0.31	0.36	0.40	0.42	0.50	0.72
WH_04	1.73	2.38	2.86	3.54	4.13	4.52	4.81	5.62	8.14
SB_US	1.53	2.05	2.44	3.01	3.52	3.85	4.11	4.83	6.98
SB_DS	1.54	2.07	2.46	3.04	3.55	3.89	4.15	4.88	7.05

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	Hydrographs derived from the ReFH2.2 method are saved in the hydrographs spreadsheets.

Pooling group compositions

Denwick Burn (DB_07)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
27051 (Crimple @ Burn Bridge)	1.202	45	4.564	0.221	0.144	0.507
45816 (Haddeo @ Upton)	1.367	24	3.489	0.306	0.387	0.635
76011 (Coal Burn @ Coalburn)	1.454	40	1.840	0.166	0.310	1.304
28033 (Dove @ Hollinsclough)	1.659	38	4.225	0.234	0.405	0.760
25019 (Leven @ Easby)	1.989	39	5.677	0.340	0.377	0.920
26802 (Gypsey Race @ Kirby Grindalythe)	2.023	18	0.108	0.316	0.217	0.906
25011 (Langdon Beck @ Langdon)	2.227	28	15.878	0.238	0.318	2.937
47022 (Tory Brook @ Newnham Park)	2.242	24	6.651	0.265	0.138	0.944
27073 (Brompton Beck @ Snainton Ings)	2.370	36	0.816	0.203	0.060	1.433
27010 (Hodge Beck @ Bransdale Weir)	2.373	41	9.420	0.224	0.293	0.286
71003 (Croasdale Beck @ Croasdale Flume)	2.385	37	10.900	0.212	0.323	0.347
25003 (Trout Beck @ Moor House)	2.432	44	15.142	0.168	0.294	0.475
44008 (South Winterbourne @ Winterbourne Steepleton)	2.468	38	0.434	0.417	0.336	1.920
91802 (Allt Leachdach @ Intake)	2.602	34	6.350	0.153	0.257	0.766
203046 (Rathmore Burn @ Rathmore Bridge)	2.617	35	10.720	0.147	0.144	0.859

Kittycarter Burn (KC_05)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.582	40	1.840	0.166	0.310	1.304
27073 (Brompton Beck @ Snainton Ings)	1.608	36	0.816	0.203	0.060	1.433
27051 (Crimple @ Burn Bridge)	1.624	45	4.564	0.221	0.144	0.507
45816 (Haddeo @ Upton)	1.763	24	3.489	0.306	0.387	0.635
28033 (Dove @ Hollinsclough)	2.006	38	4.225	0.234	0.405	0.760
26802 (Gypsey Race @ Kirby Grindalythe)	2.196	18	0.108	0.316	0.217	0.906
25019 (Leven @ Easby)	2.213	39	5.677	0.340	0.377	0.920
47022 (Tory Brook @ Newnham Park)	2.427	24	6.651	0.265	0.138	0.944
25011 (Langdon Beck @ Langdon)	2.455	28	15.878	0.238	0.318	2.937
25003 (Trout Beck @ Moor House)	2.539	44	15.142	0.168	0.294	0.475
203046 (Rathmore Burn @ Rathmore Bridge)	2.585	35	10.720	0.147	0.144	0.859
71003 (Croasdale Beck @ Croasdale Flume)	2.588	37	10.900	0.212	0.323	0.347
27010 (Hodge Beck @ Bransdale Weir)	2.589	41	9.420	0.224	0.293	0.286
44008 (South Winterbourne @ Winterbourne Steepleton)	2.653	38	0.434	0.417	0.336	1.920

White House Burn (WH_04)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.267	40	1.840	0.166	0.310	0.482
27073 (Brompton Beck @ Snainton Ings)	1.976	36	0.816	0.203	0.060	1.450
27051 (Crimple @ Burn Bridge)	2.282	45	4.564	0.221	0.144	0.530
45816 (Haddeo @ Upton)	2.315	24	3.489	0.306	0.387	0.976
28033 (Dove @ Hollinsclough)	2.568	38	4.225	0.234	0.405	0.706
26802 (Gypsey Race @ Kirby Grindalythe)	2.931	18	0.108	0.316	0.217	1.256
25019 (Leven @ Easby)	2.938	39	5.677	0.340	0.377	1.699
47022 (Tory Brook @ Newnham Park)	3.069	24	6.651	0.265	0.138	0.999
25011 (Langdon Beck @ Langdon)	3.085	28	15.878	0.238	0.318	1.840
25003 (Trout Beck @ Moor House)	3.095	44	15.142	0.168	0.294	0.300
71003 (Croasdale Beck @ Croasdale Flume)	3.137	37	10.900	0.212	0.323	0.152

91802 (Allt Leachdach @ Intake)	3.223	34	6.350	0.153	0.257	0.797
203046 (Rathmore Burn @ Rathmore Bridge)	3.306	35	10.720	0.147	0.144	0.706
27010 (Hodge Beck @ Bransdale Weir)	3.307	41	9.420	0.224	0.293	0.092
92002 (Allt Coire Nan Con @ Polloch)	3.330	16	13.540	0.101	0.337	1.533
54022 (Severn @ Plynlimon Flume)	3.345	38	14.988	0.156	0.171	2.481

Shipperton Burn (SB_DS)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.212	40	1.840	0.166	0.310	0.817
27051 (Crimple @ Burn Bridge)	1.435	45	4.564	0.221	0.144	0.663
45816 (Haddeo @ Upton)	1.506	24	3.489	0.306	0.387	0.673
28033 (Dove @ Hollinsclough)	1.801	38	4.225	0.234	0.405	0.640
25019 (Leven @ Easby)	2.245	39	5.677	0.340	0.377	0.943
26802 (Gypsey Race @ Kirby Grindalythe)	2.289	18	0.108	0.316	0.217	0.928
25011 (Langdon Beck @ Langdon)	2.407	28	15.878	0.238	0.318	2.863
47022 (Tory Brook @ Newnham Park)	2.433	24	6.651	0.265	0.138	0.924
27073 (Brompton Beck @ Snainton Ings)	2.509	36	0.816	0.203	0.060	1.745
71003 (Croasdale Beck @ Croasdale Flume)	2.517	37	10.900	0.212	0.323	0.174
25003 (Trout Beck @ Moor House)	2.571	44	15.142	0.168	0.294	0.292
27010 (Hodge Beck @ Bransdale Weir)	2.616	41	9.420	0.224	0.293	0.175
91802 (Allt Leachdach @ Intake)	2.642	34	6.350	0.153	0.257	0.747
44008 (South Winterbourne @ Winterbourne Steepleton)	2.712	38	0.434	0.417	0.336	1.797
92002 (Allt Coire Nan Con @ Polloch)	2.754	16	13.540	0.101	0.337	1.618

Additional supporting information

Appendix B

CULVERT MASTER ANALYSIS

Culvert Designer/Analyzer Report
Culvert 6.1 Existing

Analysis Component			
Storm Event	Design	Discharge	0.6600 m^3/s

Peak Discharge Method: User-Specified			
Design Discharge	0.6600 m^3/s	Check Discharge	0.0000 m^3/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	94.03 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-300 mm Circular	0.2149 m^3/s	96.80 m	2.96 m/s
Weir	Roadway (Constant Elevation)	0.4469 m^3/s	96.80 m	N/A
Total	----------------	0.6618 m^3/s	96.80 m	N/A

Culvert Designer/Analyzer Report
Culvert 6.1 Existing

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	96.80	m	Discharge	0.2149	m³/s
Inlet Control HW Elev.	95.58	m	Tailwater Elevation	94.03	m
Outlet Control HW Elev.	96.80	m	Control Type	Outlet Control	
Headwater Depth/Height	8.39				

Grades					
Upstream Invert	94.24	m	Downstream Invert	94.17	m
Length	36.62	m	Constructed Slope	0.001912	m/m

Hydraulic Profile					
Profile	CompositeM2PressureProfile		Depth, Downstream	0.30	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.30	m
Velocity Downstream	2.96	m/s	Critical Slope	0.040816	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	0.30	m
Section Size	300 mm		Rise	0.30	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	96.80	m	Upstream Velocity Head	0.44	m
Ke	0.50		Entrance Loss	0.22	m

Inlet Control Properties					
Inlet Control HW Elev.	95.58	m	Flow Control	Submerged	
Inlet Type	Square edge w/headwall		Area Full	0.1	m²
K	0.00980		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	1	
C	0.03980		Equation Form	1	
Y	0.67000				

Culvert Designer/Analyzer Report
Culvert 6.1 Existing

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.4469	m^3/s	Allowable HW Elevation	96.80	m
Roadway Width	9.90	m	Overtopping Coefficient	1.62	SI
Length	36.62	m	Crest Elevation	96.76	m
Headwater Elevation	96.80	m	Discharge Coefficient (Cr)	2.94	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	96.76
36.62	96.76

Culvert Designer/Analyzer Report
Culvert 6.1 Proposed

Analysis Component			
Storm Event	Design	Discharge	0.6600 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	0.6600 m³/s	Check Discharge	0.0000 m³/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	94.03 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-300 mm Circular	0.2241 m³/s	97.26 m	3.08 m/s
Weir	Roadway (Constant Elevation)	0.4389 m³/s	97.26 m	N/A
Total	----------------	0.6629 m³/s	97.26 m	N/A

Culvert Designer/Analyzer Report
Culvert 6.1 Proposed

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	97.26	m	Discharge	0.2241	m³/s
Inlet Control HW Elev.	95.24	m	Tailwater Elevation	94.03	m
Outlet Control HW Elev.	97.26	m	Control Type	Outlet Control	
Headwater Depth/Height	9.91				

Grades					
Upstream Invert	94.24	m	Downstream Invert	94.18	m
Length	44.62	m	Constructed Slope	0.001345	m/m

Hydraulic Profile					
Profile	CompositeM2PressureProfile		Depth, Downstream	0.30	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.30	m
Velocity Downstream	3.08	m/s	Critical Slope	0.044691	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	0.30	m
Section Size	300 mm		Rise	0.30	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	97.26	m	Upstream Velocity Head	0.48	m
Ke	0.20		Entrance Loss	0.10	m

Inlet Control Properties					
Inlet Control HW Elev.	95.24	m	Flow Control	Submerged	
Inlet Type	Beveled ring, 33.7° bevels		Area Full	0.1	m²
K	0.00180		HDS 5 Chart	3	
M	2.50000		HDS 5 Scale	B	
C	0.02430		Equation Form	1	
Y	0.83000				

Culvert Designer/Analyzer Report
Culvert 6.1 Proposed

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.4389	m^3/s	Allowable HW Elevation	97.26	m
Roadway Width	22.28	m	Overtopping Coefficient	1.62	SI
Length	44.62	m	Crest Elevation	97.23	m
Headwater Elevation	97.26	m	Discharge Coefficient (Cr)	2.94	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	97.23
44.62	97.23

Culvert Designer/Analyzer Report
Culvert 15.1 Existing

Analysis Component			
Storm Event	Design	Discharge	1.5400 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	1.5400 m³/s	Check Discharge	0.0000 m³/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	91.57 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-Site 15 rectangle reduced for silt Box	0.1233 m³/s	91.85 m	1.65 m/s
Weir	Roadway (Constant Elevation)	1.4104 m³/s	91.85 m	N/A
Total	----------------	1.5337 m³/s	91.85 m	N/A

Culvert Designer/Analyzer Report
Culvert 15.1 Existing

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	91.85	m	Discharge	0.1233	m³/s
Inlet Control HW Elev.	91.57	m	Tailwater Elevation	91.57	m
Outlet Control HW Elev.	91.85	m	Control Type	Outlet Control	
Headwater Depth/Height	14.20				

Grades					
Upstream Invert	89.49	m	Downstream Invert	89.49	m
Length	5.75	m	Constructed Slope	0.000522	m/m

Hydraulic Profile					
Profile	PressureProfile		Depth, Downstream	2.08	m
Slope Type	N/A		Normal Depth	N/A	m
Flow Regime	N/A		Critical Depth	0.17	m
Velocity Downstream	1.65	m/s	Critical Slope	0.019334	m/m

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	0.45	m
Section Size	Site 15 rectangle reduced for silt		Rise	0.17	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	91.85	m	Upstream Velocity Head	0.14	m
Ke	0.20		Entrance Loss	0.03	m

Inlet Control Properties					
Inlet Control HW Elev.	91.57	m	Flow Control	Submerged	
Inlet Type	90° headwall w 45° bevels		Area Full	0.1	m²
K	0.49500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	2	
C	0.03140		Equation Form	2	
Y	0.82000				

Culvert Designer/Analyzer Report
Culvert 15.1 Existing

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	1.4104	m^3/s	Allowable HW Elevation	91.85	m
Roadway Width	5.75	m	Overtopping Coefficient	1.68	SI
Length	5.75	m	Crest Elevation	91.57	m
Headwater Elevation	91.85	m	Discharge Coefficient (Cr)	3.04	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	91.57
5.75	91.57

Culvert Designer/Analyzer Report
Culvert 15.1 Proposed

Analysis Component			
Storm Event	Design	Discharge	1.5400 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	1.5400 m³/s	Check Discharge	1.5400 m³/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	90.22 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-1200 mm Circular	1.5403 m³/s	90.68 m	1.95 m/s
Weir	Roadway (Constant Elevation)	0.0000 m³/s	90.68 m	N/A
Total	----------------	1.5403 m³/s	90.68 m	N/A

Culvert Designer/Analyzer Report
Culvert 15.1 Proposed

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	90.68	m	Discharge	1.5403	m³/s
Inlet Control HW Elev.	90.59	m	Tailwater Elevation	90.22	m
Outlet Control HW Elev.	90.68	m	Control Type	Entrance Control	
Headwater Depth/Height	0.89				

Grades					
Upstream Invert	89.59	m	Downstream Invert	89.44	m
Length	17.00	m	Constructed Slope	0.008824	m/m

Hydraulic Profile					
Profile	CompositeS1S2		Depth, Downstream	0.78	m
Slope Type	Steep		Normal Depth	0.54	m
Flow Regime	N/A		Critical Depth	0.68	m
Velocity Downstream	1.95	m/s	Critical Slope	0.004077	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	1.22	m
Section Size	1200 mm		Rise	1.22	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	90.68	m	Upstream Velocity Head	0.27	m
Ke	0.50		Entrance Loss	0.14	m

Inlet Control Properties					
Inlet Control HW Elev.	90.59	m	Flow Control	Unsubmerged	
Inlet Type	Square edge w/headwall		Area Full	1.2	m²
K	0.00980		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	1	
C	0.03980		Equation Form	1	
Y	0.67000				

Culvert Designer/Analyzer Report
Culvert 15.1 Proposed

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.0000	m^3/s	Allowable HW Elevation	90.68	m
Roadway Width	4.50	m	Overtopping Coefficient	1.60	SI
Length	17.00	m	Crest Elevation	91.95	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	91.95
17.00	91.95

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