A1 in Northumberland: Morpeth to Ellingham

Scheme Number: TR010041

6.7 Environmental Statement – Appendix 10.1 Flood Risk Assessment

Part A

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)
Planning Inspectorate Scheme Reference	TR010041
Application Document Reference	TR010041/APP/6.7
Author:	A1 in Northumberland: Morpeth to Ellingham Project Team, Highways England

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Rev 0	June 2020	Application Issue

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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: Morpeth to Ellingham (the Scheme), Part A: Morpeth to Felton (hereafter referred to as Part A). Part A would include approximately 6.6 km of online widening of the existing carriageway and approximately 6 km of new offline highway.

Review of the Environment Agency's Flood Map for Planning (Rivers and Sea) indicates that the majority of Part A's alignment is located in the low-risk Flood Zone 1. However, Part A does include sections located in the medium risk Flood Zone 2 and the high-risk Flood Zone 3.

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

Part A alignment crosses ten watercourses and associated tributaries (listed from south to north): Cotting Burn; Shieldhill Burn; Floodgate Burn; River Lyne; Fenrother Burn; Earsdon Burn; Longdike Burn; Unnamed tributary of Thirston Burn; River Coquet; and Bradley Brook. The assessment has taken all of these watercourse crossings into account.

The development of the proposals for each watercourse crossed by Part A has been dictated by the baseline flood risk situation and whether the design is an extension of an existing culvert, replacement of an existing culvert or the construction of a new structure where an open channel is currently present.

Detailed 1D hydraulic modelling has been undertaken for the Cotting Burn, River Lyne, Fenrother Burn, Earsdon Burn and Longdike 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.

Detailed hydraulic analysis of the River Coquet has not been undertaken. The proposals comprise a new River Coquet bridge adjacent to the existing one, that requires the construction of two new piers: one on the north bank of the river above the expected 100 year flood level and one on the south bank of the river within the floodplain. A simple analysis was undertaken as the proposed southern pier would be aligned with the existing pier. It shows that the new southern pier within the floodplain would have no impact on receptors sitting approximately 8 m above the 1 in 1000 year water level. To further manage flood risk resulting from the works, it has been agreed that the temporary works for this structure would use a kingpost solution to maintain the levels of the bridge deck as it is pushed across the river rather than a temporary pier. This would remove the need for any temporary works within the channel not associated with the single pier described above.

Discussions with the Environment Agency have confirmed that the recommendations described are acceptable.

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 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 a Flood Risk Assessment (FRA) to support the Environmental Statement (ES) (**Application Document Reference: TR010041/APP/6.2**) and Development Consent Order (DCO) application for the A1 in Northumberland: Morpeth to Ellingham Scheme (the Scheme) Part A: Morpeth to Felton (Part A). The assessment has been conducted in accordance with the National Planning Policy Framework (NPPF) (**Ref 10.1.1**) and Planning Practice Guidance (PPG) (**Ref 10.1.2**), the National Policy Statement for National Networks (NPS NN) (**Ref 10.1.3**), the Design Manual for Roads and Bridges (DMRB) Volume 11, Section 3, Part 10 (HD 45/09) (**Ref 10.1.4**), 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. Review of the Environment Agency's Flood Map for Planning (Rivers and Sea) (**Ref 10.1.5**) indicates that the majority of Part A's alignment is located in the low-risk Flood Zone 1 where the risk of flooding from fluvial sources is less than 1 in 1000 (0.1 %) in any year. However, Part A does include sections located in the medium risk Flood Zone 2 and the high-risk Flood Zone 3. The identified fluvial flood risk (Flood Zones 2 and 3) is associated with the following watercourses: The River Coquet, Longdike Burn (and the Poxtondean Burn that discharges into the Longdike Burn), Earsdon Burn, the River Lyne and Floodgate Burn.

1.1.3. The Environment Agency's standing advice on flood risk (**Ref 10.1.6**) states that a FRA would be required to support the DCO application for Part A and our assessment includes the following:

a. Confirmation of the sources of flooding which may affect Part A.
b. A quantitative assessment of the risk of flooding to Part A and to adjacent sites as a result of Part A.
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 SITE DETAILS

1.2.1. Part A is located within the County of Northumberland and forms part of the Applicant's strategic road network. Part A is located between Warrener's House Interchange at Morpeth and the dual carriageway at Felton and is approximately 12.6 km in length. It would include approximately 6.6 km of online widening of the existing carriageway and approximately 6 km of new offline highway. A more detailed description of Part A is found in **Chapter 2: The Scheme, Volume 1** of this ES (**Application Document Reference: TR010041/APP/6.1**). The approximate location of Part A is shown in **Figure 1** below.

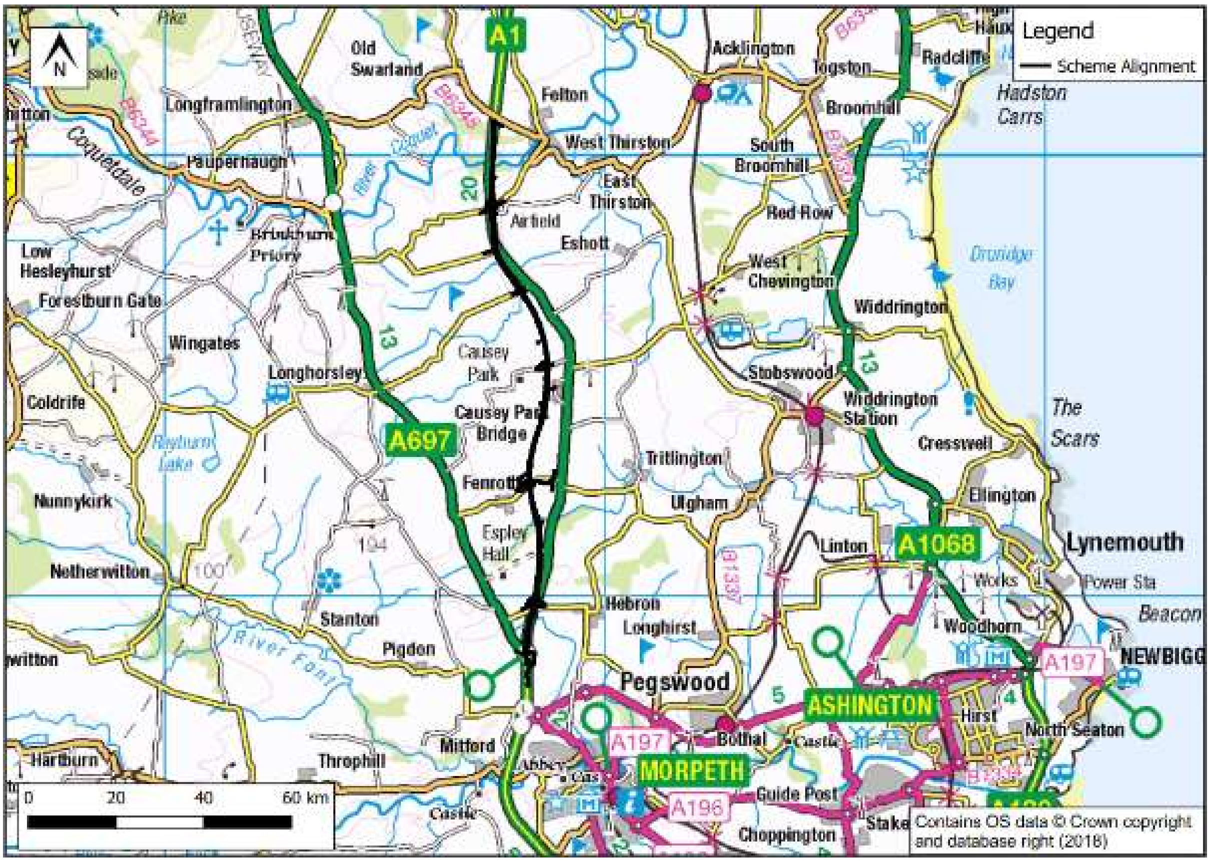

Figure 1 – Part A: Location Plan

1.3 CONSULTATION

1.3.1. Consultation has been undertaken with the following authorities:

a. Meeting held with the Environment Agency and NCC in January 2018 to discuss stakeholder requirements and review the available flood information and agree (in principle) methodology, appropriate mitigation and management options during the construction and operation phases.

b. Meeting held with the Environment Agency and NCC in September 2018 to discuss the results of the hydraulic modelling undertaken and review the methodology, Part A proposals and proposed mitigation and discuss and address specific areas of concern.

c. Meeting held with the Environment Agency and NCC in November 2018 to discuss the Part A proposals for the new River Coquet bridge crossing.

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 visits completed for Part A on 7 and 8 June 2018 and to the River Coquet on 5 December 2018.
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 10.1.5**) and Long Term Flood Risk Map (**Ref 10.1.7**)) and groundwater data (Environment agency groundwater data is hosted on The Multi-Agency Geographic Information for the Countryside (MAGIC) online map (**Ref 10.1.8**)) (accessed July 2018), information provided by the Environment Agency on historical flooding during consultation.
c. Obtained LiDAR and topographic survey data.
d. Review of the **Ground Investigation Report** undertaken (dated September 2018) (refer to **Appendix 11.2, Volume 7** of this ES (**Application Document Reference: TR010041/APP/6.7**)).
e. Consultation with the Environment Agency and NCC to confirm potential flood risk to Part A and agree principles for the mitigation of potential flood risk to Part A and third-party land arising from Part A (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 A may affect fluvial flood risk, informed by the development of five 1D Flood Modeller hydraulic models and six Culvert Master models.
g. Development of mitigation measures, as necessary, to reduce flood risk to Part A 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 scheme generated surface water runoff from Part A.

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 NPPF (**Ref 10.1.1**) 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 NPPF (**Ref 10.1.1**)).

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 10.1.1**) 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 10.1.2**) to the NPPF (**Ref 10.1.1**).

2.2.6. The Sequential Test as defined in NPPF (**Ref 10.1.1**) 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	✓	✓	✓	✓	✓
Zone 2	✓	✓	Exception test required	✓	✓
Zone 3a	Exception test required	✓	×	Exception test required	✓
Zone 3b	Exception test required	✓	×	×	×

✓ Development considered acceptable

× Development considered unacceptable

2.2.8. The Scheme is classed as Essential Infrastructure under the NPPF (**Ref 10.1.1**). Essential Infrastructure within Flood Zone 3 requires the Sequential Test and Exception Test to be passed before it is considered to be acceptable.

2.2.9. Part A is required to improve the A1 between Morpeth and Felton. Improvements are required to enhance resilience and improve journey times and safety along the route. Part A is therefore deemed to pass the Sequential Test in this instance.

2.2.10. In terms of the Exception Test, this FRA demonstrates that Part A would remain safe throughout its design life and that flood risk would not be increased elsewhere.

2.3 POTENTIAL SOURCES OF FLOODING

2.3.1. In accordance with NPPF (**Ref 10.1.1**), the following sources of flooding have been considered in this assessment:

a. Fluvial flood risk from nearby watercourses.
b. Surface water flooding from within Part A's boundary 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 about how a changing climate affects 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 10.1.9**) were published by the Environment Agency in February 2016 (and updated in February 2017), which superseded the previous recommendations that were included within the NPPF PPG (**Ref 10.1.2**). The impacts of climate change are expected to increase over time and the Environment Agency guidance (**Ref 10.1.9**) 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 10.1.9**) 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 10.1.9**) divides England into eleven river basin districts. Part A 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 10.1.9**) 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 10.1.9**) 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 the Scheme 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 A and taking into account the development's vulnerability are as follows:

a. 25 % increase in peak river flow for the assessment of risk to Part A, 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 1000 year peak flow (whichever is greatest) for the residual risk assessments to understand risks to Part A 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. The development of the proposals for each watercourse crossed by Part A is dictated by the baseline flood risk situation, both upstream and downstream, and the form of the proposed solution such as extension of an existing culvert, replacement of an existing culvert or the construction of a new structure where an open channel is currently present. 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 A 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 (online Section)	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 1000 year flood events (baseline). – Assess implications of climate change with the 100 year + 25 % climate change event as set out in **Section 2.4**. – 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**, or 1000 year event, whichever is higher and a scenario representing partial blockage of the structure.

Proposed Solution	Hydraulic Analysis Approach
Replacement of existing culvert (online Section)	The size of replacement culverts would be informed by hydraulic analysis of the culvert to meet DMRB (**Ref 10.1.10**) requirements wherever possible whilst preventing an increase in downstream flows resulting from the removal of the downstream structure. Consideration would also be given to 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 1000 year flood events (baseline). – Assess implications of climate change with the 100 year + 25 % climate change event as set out in **Section 2.4**. – Design proposed replacement culvert with consideration of mammal passage, fish passage, flood risk impacts and design constraints as described below.
Construction of a new culvert where open channel is currently present (offline Section)	New culverts would be informed by hydraulic analysis of the culvert to meet DMRB (**Ref 10.1.10**) requirements and consider fish or mammal passage where recommended: Assess performance of existing channel and local structures using 2 year, 10 year, 100 year and 1000 year flood events (baseline). Assess implications of climate change with the 100 year + 25 % climate change event as set out in **Section 2.4**. Design proposed new culvert with consideration of mammal passage, fish passage, flood risk impacts and design constraints as described below.

2.5.13. The hydraulic analysis of each culvert was agreed with the Environment Agency and NCC at a meeting in January 2018 and was undertaken using one of the following methods:

a. A short 1D hydraulic model incorporating the local channel and other structures using Flood Modeller Pro. This approach was used for Cotting Burn, the River Lyne, Fenrother Burn, Earsdon Burn and Longdike Burn.
b. A hydraulic assessment of the structures using Culvert Master. This approach was used for all remaining watercourses not listed above.

2.5.14. 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).

2.5.15. 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

2.5.16. A detailed assessment of fluvial flood risk has been completed for the largest watercourses (excluding the River Coquet) crossed by Part A using five hydraulic models to provide an improved understanding of the fluvial flood risk in the vicinity of Part A and a basis for assessing the impact of Part A on third parties.

2.5.17. For this assessment, five 1D Flood Modeller Pro hydraulic models were created based on the topographic survey undertaken in April 2018 for the Cotting Burn, River Lyne, Fenrother Burn, Earsdon Burn and Longdike Burn.

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

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

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

2.5.20. 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 10.1.11**) and enables the assessment of culverts for both pipe and open channel flow scenarios.

2.5.21. **Section 5** of this FRA sets out the methodology for the assessment of these local drains and discusses any resulting changes. Detailed technical information is provided in **Appendix B: Culvert Master Analysis**.

FLOOD RISK FROM THE RIVER COQUET

2.5.22. Following consultation with the Environment Agency it was agreed that detailed hydraulic modelling of the River Coquet would not be required, as the proposed southern pier would be aligned with the existing pier. To assess the impact of Part A on the River Coquet a simple assessment has been undertaken. The simple assessment comprised a manning's calculation using desktop based information and informed by previous assessments. Mannings is a coefficient which represents the roughness or friction applied to the flow by the watercourse channel.

2.5.23. The simple assessment provided an approximate peak water level where Part A crosses the River Coquet.

LEGISLATIVE FRAMEWORK AND GUIDANCE

2.5.24. 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.

2.5.25. The applicable legislative framework is summarised below.

EUROPEAN LEGISLATION

Water Framework Directive (2000/60/EC)

2.5.26. The overall objective of the Water Framework Directive (WFD) (**Ref 10.1.12**) 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.

2.5.27. The WFD (**Ref 10.1.12**) 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)

2.5.28. This Groundwater Directive (**Ref 10.1.13**) 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 10.1.12**), specifically the assessment of chemical status of groundwater and objectives to achieve 'good' status.

Floods Directive (2007/60/EC)

2.5.29. The key objective of the Floods Directive (**Ref 10.1.14**) 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

2.5.30. 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 10.1.15**). 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 to ensure that local flood risk is not increased.

2.5.31. The Land Drainage Act (**Ref 10.1.15**) specifies that the following works will 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.

2.5.32. The Land Drainage Act (**Ref 10.1.15**) also sets out the maintenance responsibilities riparian owners have 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

2.5.33. The Flood and Water Management Act 2010 (**Ref 10.1.16**) extended the role of the LLFA (NCC) set out in the Flood Risk Regulations (**Ref 10.1.17**) 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.

2.5.34. 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 regarding Sustainable Drainage Systems (SUDS).

2.5.35. Schedule 3 of the Flood and Water Management Act (**Ref 10.1.16**) introduces National Standards for SUDS against which proposed drainage systems should comply. These are discussed below.

Environmental Permitting (England and Wales) Regulations 2010

2.5.36. The Environmental Permitting (England and Wales) Regulations 2010 (**Ref 10.1.18**) replaced the Water Resources Act 1991 (**Ref 10.1.19**) as the key legislation for water pollution in the UK. Under the Environmental Permitting Regulations (**Ref 10.1.18**), 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.

2.5.37. 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 10.1.18**).

NATIONAL POLICY

National Policy Statement for National Networks 2014

2.5.38. The National Policy Statement for National Networks (NPS NN) (**Ref 10.1.3**) 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 A should be supported by a 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

2.5.39. The National Planning Policy Framework (NPPF) (**Ref 10.1.1**) 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 10.1.2**) has been published alongside the NPPF (**Ref 10.1.1**) in 2014 to set out how certain policies, including those relating to flood risk, should be implemented.

2.5.40. The NPPF (**Ref 10.1.1**) and relevant PPG (**Ref 10.1.2**) 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.

2.5.41. The PPG (**Ref 10.1.2**) sets out the requirement to consider 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

2.5.42. The Non-Statutory Technical Standards for SUDS (**Ref 10.1.20**), 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 10.1.1**) and PPG (**Ref 10.1.2**). 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

2.5.43. NCC are currently in the process of updating their Local Plan (**Ref 10.1.21**), the consolidated planning policy framework (**Ref 10.1.22**) details the saved polices that are currently used for planning applications. Part A is located within the former district areas of Castle Morpeth and Alnwick. The relevant saved policies are detailed below.

Castle Morpeth District Local Plan 1991 – 2006

2.5.44. There is one saved policy from the Castle Morpeth District Local Plan (**Ref 10.1.23**) that applies to this FRA for Part A.

2.5.45. Policy RE5 (Surface water runoff and flood defences) sets out to ensure that new development does not increase local flood risk through the application of the Sequential Test. It also states that appropriate mitigation measures should be in place to minimise the risk of flooding.

Alnwick District Wide Local Plan 1997

2.5.46. There is one saved policy from the Alnwick District Wide Local Plan Policy (**Ref 10.1.24**) that is applicable to this FRA for Part A. Policy CD33 sets out to ensure that new development is not located in areas of known flood risk and will not increase local flood risk elsewhere as a result of the development.

Northumberland Draft Local Plan 2019

2.5.47. The Northumberland Draft Local Plan (**Ref 10.1.21**) provides guidance for new development with the Council's administrative area. It is currently intended that the plan will 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 A:

2.5.48. 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.

2.5.49. 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

2.5.50. Northumberland's Local Flood Risk Management Strategy (LFRMS) (**Ref 10.1.25**) provides information and technical guidance on how flood risk would be managed within Northumberland. The LFRMS (**Ref 10.1.25**) 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 A. 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. Land surrounding Part A generally consists of woodland and agricultural land, with the Eshott Airfield located approximately 2 km to the south of Felton. The most notable urban areas surrounding Part A are the town of Morpeth to the south and the village of Felton to the north.

3.1.2. A detailed description of the surrounding areas to each watercourse is provided in more detail below within **Section 3.2**.

3.2 EXISTING SURFACE WATER FEATURES

3.2.1. Part A alignment crosses ten watercourses and associated tributaries that would be impacted by Part A. These are listed below from south to north:

a. Cotting Burn
b. Shieldhill Burn
c. Floodgate Burn
d. River Lyne
e. Fenrother Burn
f. Earsdon Burn
g. Longdike Burn
h. Unnamed tributary of Thirston Burn
i. River Coquet
j. Bradley Brook

COTTING BURN

3.2.2. Cotting Burn flows in a west to east direction underneath the existing A1 alignment at the junction with the A697 and it is classified as an ordinary watercourse under the jurisdiction of NCC as LLFA.

3.2.3. The source of the Cotting Burn is just upstream of the existing A1 alignment. Its catchment is relatively flat with an approximate upstream catchment area of 0.75 km². The catchment is entirely rural with no flood risk receptors upstream of the A1 crossing.

3.2.4. Cotting Burn flows through five existing culverts within close proximity of Part A as identified and numbered in **Figure 2** below.

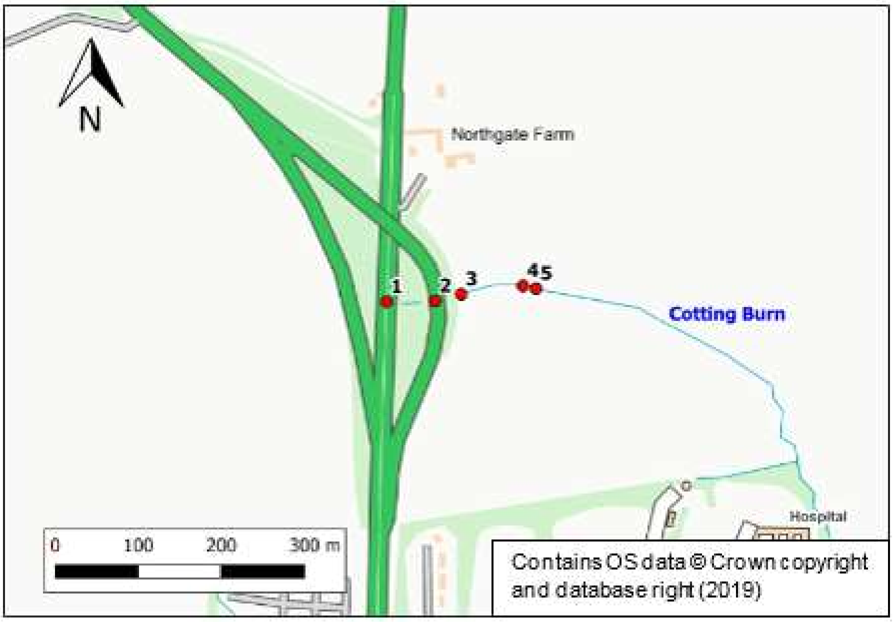

Figure 2 – Cotting Burn Existing Culverts

3.2.5. Cotting Burn flows through a culvert beneath the A1, identified as number one within **Figure 2**. **Figure 3** shows the outlet of this circular concrete culvert, with a diameter of approximately 900 mm. The culvert is approximately 28 m in length.

3.2.6. Cotting Burn then flows through a culvert beneath the eastern slip road which joins with the A697, identified as number two within **Figure 2**. **Figure 4** shows the outlet of this circular 900 mm diameter concrete culvert, which is approximately 41 m in length.

3.2.7. **Figure 5** shows the downstream face of the culvert beneath the farm access track just downstream of the crossing underneath the A697 slip road, identified as number three within **Figure 2**. The culvert is a circular concrete pipe with a diameter of approximately 350 mm and approximately 7 m in length. There is approximately 80 m of open channel before Cotting Burn flows through another circular concrete 350 mm pipe underneath a farm access track, as shown in **Figure 6**. This culvert is identified as number four within **Figure 2**.

3.2.8. Cotting Burn then flows beneath an access track immediately downstream of culvert number four through a circular 450 mm diameter and 15 m long culvert, identified as number five within **Figure 2**. **Figure 7** shows the top of the access track. During the site visit it was noted that the culvert appeared to be blocked and in a poor condition.

Figure 3 – Outlet of Cotting Burn Culvert (1)

Figure 4 – Outlet of Cotting Burn Culvert (2)

Figure 5 – Outlet of Cotting Burn Culvert (3)

Figure 6 – Inlet of Cotting Burn Culvert (4)

Figure 7 – Cotting Burn Crossing (Upstream)

3.2.9. A short distance downstream of Part A is Northgate Hospital and the watercourse passes close to buildings in this area on the right bank.

3.2.10. The Environment Agency have also confirmed there are major flooding issues downstream and any proposals should not increase flood risk downstream.

3.2.11. The Cotting Burn eventually discharges into the River Wansbeck approximately 3 km to the south-east of Part A.

SHIELDHILL BURN

3.2.12. Shieldhill Burn flows in a west to east direction and flows beneath the existing A1 alignment approximately 1 km to the north of the A697 junction.

3.2.13. Shieldhill Burn is classified as an ordinary watercourse and under the jurisdiction of NCC as LLFA.

3.2.14. The source of Shieldhill Burn is approximately 0.8 km to the west of the existing A1 alignment, adjacent to the A697. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 0.94 km².

3.2.15. Shieldhill Burn flows beneath the A1 through an arch culvert approximately 1.2 km to the west of the village of Hebron. The culvert is approximately 1.3 m wide and 30 m long. **Figures 8** and **9** below show the upstream and downstream ends of the culvert. Upstream of the existing crossing of the A1 the Shieldhill Burn enters a 300 mm diameter below ground pipe that conveys the watercourse to the existing culvert. During the site walkover a 300 mm pipe was observed at the outlet of the culvert. The watercourse enters this pipe immediately downstream of the culvert and is conveyed below ground for approximately 210 m at which point the watercourse returns to an open channel. Review of satellite imagery and flood mapping indicates that when the capacity of the pipe is exceeded the watercourse flows overland along what is assumed to be the natural alignment to re-join the open channel downstream.

3.2.16. The Shieldhill Burn discharges into Cotting Burn approximately 2.5 km downstream from the existing A1 crossing.

Figure 8 – Shieldhill Burn culvert, Upstream

Figure 9 –Shieldhill Burn culvert, Downstream

FLOODGATE BURN

3.2.17. Floodgate Burn flows in a south-west to north-east direction beneath the existing A1 and it is classified as an ordinary watercourse and under the jurisdiction of NCC as LLFA.

3.2.18. The source of Floodgate Burn is approximately 1.5 km to the south-west of the existing A1 alignment within the Spruce Plantation. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 2 km².

3.2.19. The Floodgate Burn flows beneath the A1 through an arch culvert (approximately 1.9 m wide and 1 m high) which is 26 m in length, as shown in **Figure 10**. Approximately 50 m downstream of this culvert the watercourse flows beneath a farm access track through a circular 900 mm diameter culvert, which is 7 m in length. This is shown in **Figure 11**.

3.2.20. Approximately 1.3 km downstream of the A1, the Floodgate Burn discharges into the River Lyne.

Figure 10 – Floodgate Burn Culvert Beneath the A1, Downstream

Figure 11 – Floodgate Burn Culvert Underneath Farm Access Track, Downstream

RIVER LYNE

3.2.21. The River Lyne flows in a west to east direction beneath the existing A1 alignment through a culvert at Priest's Bridge. The River Lyne has a number of significant tributaries including Floodgate Burn and Fenrother Burn and it is classified as an ordinary watercourse under the jurisdiction of NCC as LLFA.

3.2.22. Gorfen Letch and Heronsclose Burn merge before becoming the River Lyne. The source of the watercourse is approximately 1.2 km to the west of the A1, near to Gorfenletch Wood.

3.2.23. The catchment of the River Lyne is gently sloping from the west to the east and it has an upstream catchment area of 8.27 km². The catchment is entirely rural with no flood risk receptors within 2 km upstream of the A1 crossing.

3.2.24. The River Lyne flows beneath the existing A1 alignment through a concrete culvert. As shown in **Figures 12** and **13**, the inlet of the culvert is circular and the outlet is an arch structure. The culvert is 34 m in length and approximately 2 m wide and 2.6 m high.

3.2.25. Priest's Bridge House is located immediately downstream of the A1 crossing on the northern bank. The village of Tritlington is approximately 5 km downstream of the A1 crossing where the River Lyne is classified as a main river and under the jurisdiction of the Environment Agency.

Figure 12 – River Lyne Culvert, Upstream

Figure 13 – River Lyne Culvert, Downstream

FENROTHER BURN

3.2.26. Fenrother Burn flows beneath Fenrother Lane just to the west of the A1 in a predominantly north to south direction. The Fenrother Burn is classified as an ordinary watercourse and under the jurisdiction of NCC as LLFA.

3.2.27. The source of Fenrother Burn is just to the south of Longhorsley Moor and the catchment of the watercourse is gently sloping towards to east with an approximate upstream catchment area of 3 km².

3.2.28. Fenrother Burn flows beneath Fenrother Lane through a stone circular culvert, as pictured in **Figure 14** below. The culvert has an approximate diameter of 500 mm and is approximately 120 m in length.

3.2.29. Fenrother Burn then discharges into the River Lyne approximately 1 km downstream from the Fenrother Lane watercourse crossing.

Figure 14 - Fenrother Burn Culvert, Upstream

EARSDON BURN

3.2.30. Earsdon Burn and its tributaries flow in a predominantly west to east direction, beneath the existing A1 alignment at Causey Park Bridge and beneath the local side road to the west.

3.2.31. Earsdon Burn is classified as an ordinary watercourse and under the jurisdiction of NCC as LLFA.

3.2.32. The source of Earsdon Burn is approximately 2.5 km to the south-west of the existing A1 alignment, to the south of the village Fieldhead. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 4.2 km².

3.2.33. Earsdon Burn flows through two culverts and one bridge beneath the adjacent unnamed roads and the existing A1 alignment, as identified in **Figure 15** below.

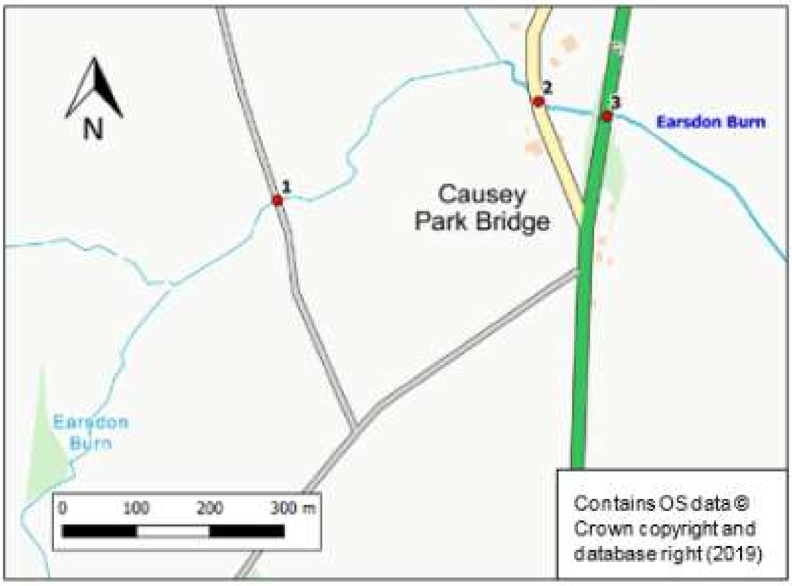

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Figure 15 – Earsdon Burn Existing Structures

3.2.34. Earsdon Burn flows through a culvert beneath an unnamed road to the west of the A1, identified as number one within **Figure 15**. **Figure 16** below shows the inlet of the triple circular parallel concrete culverts. The diameter of each culvert (from left to right) are approximately 450 mm, 650 mm and 650 mm respectively. The culverts are approximately 10 m in length.

3.2.35. **Figure 17** shows the bridge crossing over Earsdon Burn beneath the unnamed road to the west of the existing A1, identified as number two within **Figure 15**. The bridge crossing is approximately 5.8 m wide and 29 m in length. The walls of the bridge are made of concrete.

3.2.36. Earsdon Burn flows beneath the existing A1 alignment through a 3 m wide culvert, as identified as number three within **Figure 15**. **Figure 18** shows the inlet of the culvert. The culvert is approximately 32 m in length.

Figure 16 – Earsdon Burn Culvert (1)

Figure 17 – Earsdon Burn Crossing (2)

Figure 18 – Earsdon Burn Crossing (3)

3.2.37. Earsdon Burn eventually discharges into the River Lyne approximately 4.2 km downstream of the existing A1.

LONGDIKE BURN

3.2.38. Longdike Burn flows in a predominantly south-west to north-east direction, flowing beneath the existing A1 alignment just downstream of where the Bywell Letch discharges into it.

3.2.39. Longdike Burn is classified as a main river and under the jurisdiction of the Environment Agency.

3.2.40. The source of Longdike Burn is approximately 5.9 km to the south-west of the existing A1 alignment just to the west of Longhorsley Moor. The catchment of the watercourse is gently sloping towards the north-east with an approximate upstream catchment area of 23.4 km². There are no flood risk receptors upstream of Part A.

3.2.41. Approximately 0.5 km upstream of the existing A1, Longdike Burn flows beneath East Road to the south of Burgham Park Golf Club through a concrete arch culvert. **Figure 19** below shows the outlet of the culvert; as shown in the photograph there are wooden baffles along the base of the culvert to facilitate fish passage. The culvert is approximately 3.4 m wide, 4.8 m high and 30 m long.

3.2.42. **Figure 20** below shows Longdike Burn flowing beneath the existing A1 alignment through Bockenfield Bridge. The concrete arch bridge is approximately 6.6 m wide, 2.4 m high and approximately 30.6 m long.

Figure 19 – Longdike Burn Culvert, Outlet

Figure 20 – Bockenfield Bridge

3.2.43. Immediately downstream of the A1 there are a number of holiday cabins located on both banks of the watercourse.

3.2.44. Approximately 2.7 km downstream of the existing A1 Longdike Burn discharges into Thirston Burn.

UNNAMED TRIBUTARY OF THIRSTON BURN

3.2.45. The unnamed tributary of the Thirston Burn flows in a west to east direction and beneath the existing A1 alignment approximately 0.7 km south of the River Coquet bridge.

3.2.46. The Thirston Burn and its tributaries are classified as ordinary watercourses and under the jurisdiction of NCC as LLFA.

3.2.47. The source of the unnamed tributary of Thirston Burn is approximately 0.5 km to the west of the existing A1 alignment. The catchment of the watercourse is relatively flat with an approximate upstream catchment area of 0.7 km².

3.2.48. **Figure 21** below shows the precast concrete circular culvert that conveys the watercourse beneath the A1 with a diameter of approximately 1.2 m and 24.3 m in length. The base of the culvert has been reinforced with concrete and a cover slab.

Figure 21 – Unnamed Tributary of Thirston Burn culvert

3.2.49. Approximately 2.1 km downstream of the A1 watercourse crossing, the unnamed tributary of Thirston Burn discharges into the Thirston Burn.

RIVER COQUET

3.2.50. The River Coquet flows beneath the existing A1 in a predominantly south-west to north-east direction.

3.2.51. The River Coquet is classified as a main river and under the jurisdiction of the Environment Agency.

3.2.52. The source of the River Coquet is approximately 40 km to the north-west of the existing A1 alignment just south of Coquet Head within Northumberland National Park. The catchment of the watercourse is significantly larger than the other catchments with an approximate upstream catchment area of 486 km².

3.2.53. The River Coquet flows beneath the existing A1 within Dukes Bank Wood. **Figure 22** below shows a photograph of the existing bridge looking downstream. The as built drawing for the existing bridge is in **Figure 10.2: River Coquet As-Built Drawing**, **Volume 5** of this ES (**Application Document Reference: TR010041/APP/6.5**).

Figure 22 – River Coquet Bridge, Looking Downstream

3.2.54. The River Coquet eventually discharges into the North Sea approximately 17 km downstream of the existing A1 bridge at Amble.

BRADLEY BROOK

3.2.55. Bradley Brook flows in a west to east direction and flows beneath the existing A1 alignment through a culvert within Park Wood.

3.2.56. Bradley Brook is classified as an ordinary watercourse and under the jurisdiction of NCC as LLFA.

3.2.57. The source of Bradley Brook is approximately 0.3 km to the west of the existing A1 alignment within Park Wood. The catchment of the watercourse is relatively small with an upstream catchment area of less than 0.5 km².

3.2.58. Bradley Brook flows beneath the existing A1 alignment through a precast concrete circular culvert as shown in **Figures 23** and **24**. The culvert has a diameter of approximately 1.2 m at the inlet and is 125 m in length. The base of the culvert has been reinforced with concrete and a cover slab. At the outlet of the culvert the diameter is reduced to 900 mm for approximately 20 m. It is assumed that the culvert was previously extended to enable construction of an above ground attenuation area. There is a smaller circular pipe just above the main culvert as shown in **Figure 23**, which has an approximate diameter of 300 mm.

Figure 23 – Bradley Brook Culvert, Outlet

Figure 24 – Bradley Brook Culvert, Inlet

3.2.59. Bradley Brook discharges into Back Burn approximately 0.9 km downstream of the existing culvert.

3.3 GEOLOGY AND HYDROGEOLOGY

3.3.1. Review of the British Geological Survey (BGS) 1:50,000 data (**Ref 10.1.26**) indicates that the majority of Part A is underlain by bedrock geology of the Stainmore Formation comprising mudstone, siltstone and sandstone. There is also a seam of the Northern England Late Carboniferous Tholeiitic Dyke-Swarm (Quartz-microgabbro) comprising igneous bedrock located to the north of Causey Park. There is also a small deposit of Corbridge Limestone located along the River Coquet to the north of Part A.

3.3.2. Review of BGS 1:50,000 data (**Ref 10.1.26**) indicates that superficial deposits within the Study Area are mostly glacial till with an area of glacial sands and gravels located to the north of Part A scheme area surrounding the River Coquet. There are also alluvium deposits consisting clay, silt, sand and gravel associated with the Longdike Burn and the Earsdon Burn.

3.3.3. Review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref 10.1.8**) indicates that the majority of the bedrock geology is classified as a Secondary A Aquifer, 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. A very small seam of a Secondary B Aquifer is located to the north of Causey Park Bridge, described as predominantly lower permeability layers which may store and yield amounts of groundwater due to localised features such as fissures, thin permeable horizons and weathering. This is the seam of the Northern England Late Carboniferous Tholeiitic Dyke-Swarm as described in **paragraph 3.3.1** above.

3.3.4. Review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref 10.1.8**) indicates that the majority of the superficial deposits are classified as a Secondary (Undifferentiated) Aquifer. A small area, associated with the sand and gravels along the River Coquet and Longdike Burn are classified as a Secondary A Aquifer.

3.3.5. Review of the Environment Agency Groundwater data available on MAGIC online mapping (**Ref 10.1.8**) indicates that the southern section of the Study Area, just to the north of Morpeth, is located within a total catchment (Zone 3) groundwater Source Protection Zone (SPZ). Total catchment (Zone 3) is defined as the area around a source within which all groundwater recharge is presumed to be discharged at the source. SPZs are typically used to protect abstractions for public water supply. The quality of surface water runoff discharged to ground within designated SPZs is of key importance.

3.3.6. A review of the Cranfield University Soilscapes (**Ref 10.1.27**) mapping indicates that soils within the Study Area are slowly permeable loamy and clayey soils.

3.3.7. The ground investigation work undertaken in 2018 (refer to **Appendix 11.2: Ground Investigation Report, Volume 7** of this ES (**Application Document Reference: TR010041/APP/6.7**)) to improve understanding of baseline conditions included groundwater monitoring of sixteen locations where groundwater strikes had previously been recorded during investigation work. The groundwater monitoring indicated that groundwater levels are relatively stable between 0.5 and 1 m below ground level (bgl). The glacial deposits along Part A within the Study Area recorded groundwater levels between 1.5 and 2.5 m bgl. All of the groundwater monitoring results therefore indicate that that groundwater levels are relatively high across the 1 km Study Area. This is of note in locations near to watercourses and areas at high risk of surface water flooding as discussed in **Section 4.3**.

3.3.8. Sections of Part A to the north and to the east are located within the Coal Authority's (CA) reporting area. The online CA's screening tool (**Ref 10.1.28**) indicates that Part A is not located within a constraint area with regards to groundwater.

4 EXISTING FLOOD RISK

4.1 HISTORIC FLOOD RECORDS

4.1.1. Consultation with NCC has highlighted a few issues regarding fluvial flooding from ordinary watercourses including:

a. Flooding issues in Morpeth relating to the Cotting Burn.
b. Flooding issues in Felton relating to the Bradley Brook, Back Burn and other watercourses.
c. Performance of attenuation features associated with the existing alignment of the A1 near Felton.

4.1.2. The NCC Level 1 Strategic Flood Risk Assessment (**Ref 10.1.29**) indicates significant flooding within the North East Northumberland river catchments from fluvial and pluvial sources since 1744. Several significant flood events are attributed to the River Coquet and impacted settlements and roads within 0.5 km.

4.1.3. The HADDMS (Highways Agency Drainage Data Management System) online database (**Ref 10.1.30**) indicates that the Morpeth to Felton section of the existing A1 has eight documented historical surface water flood events of which two are detailed as high severity events resulting in the total closure of the carriageway. The two flood events were associated with blocked highway gullies.

4.1.4. Historic flood incidents were identified during the November 2016 public consultation:

a. An existing outfall from the A1 surface water drainage system is understood to discharge into the Back Burn via a settlement pond but without any attenuation. Anecdotal evidence suggests that the discharge has contributed to flooding at nearby properties.
b. Another historic flooding issue was highlighted to have occurred approximately 500 m to the west of the existing A1. Anecdotal reports suggest that this flooding event occurred along the length of the field north from Fenrother Lane and was associated with the unnamed tributary of Fenrother Burn.

4.1.5. The 2018 public consultation identified further anecdotal flooding information. A natural spring is located just to the south of the A697 slip road which has previously caused flooding to the centre of the A1.

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 10.1.5**) indicates that the majority of Part A's alignment is located in the low-risk Flood Zone 1. However, Part A does include sections located in the medium risk Flood Zone 2, and the high-risk Flood Zone 3, as shown in **Figures 25** and **26**. The identified fluvial flood risk is associated with the following watercourses: The River Lyne; Earsdon Burn; Longdike Burn (and the Poxtondean Burn that discharges into the Longdike Burn); and River Coquet.

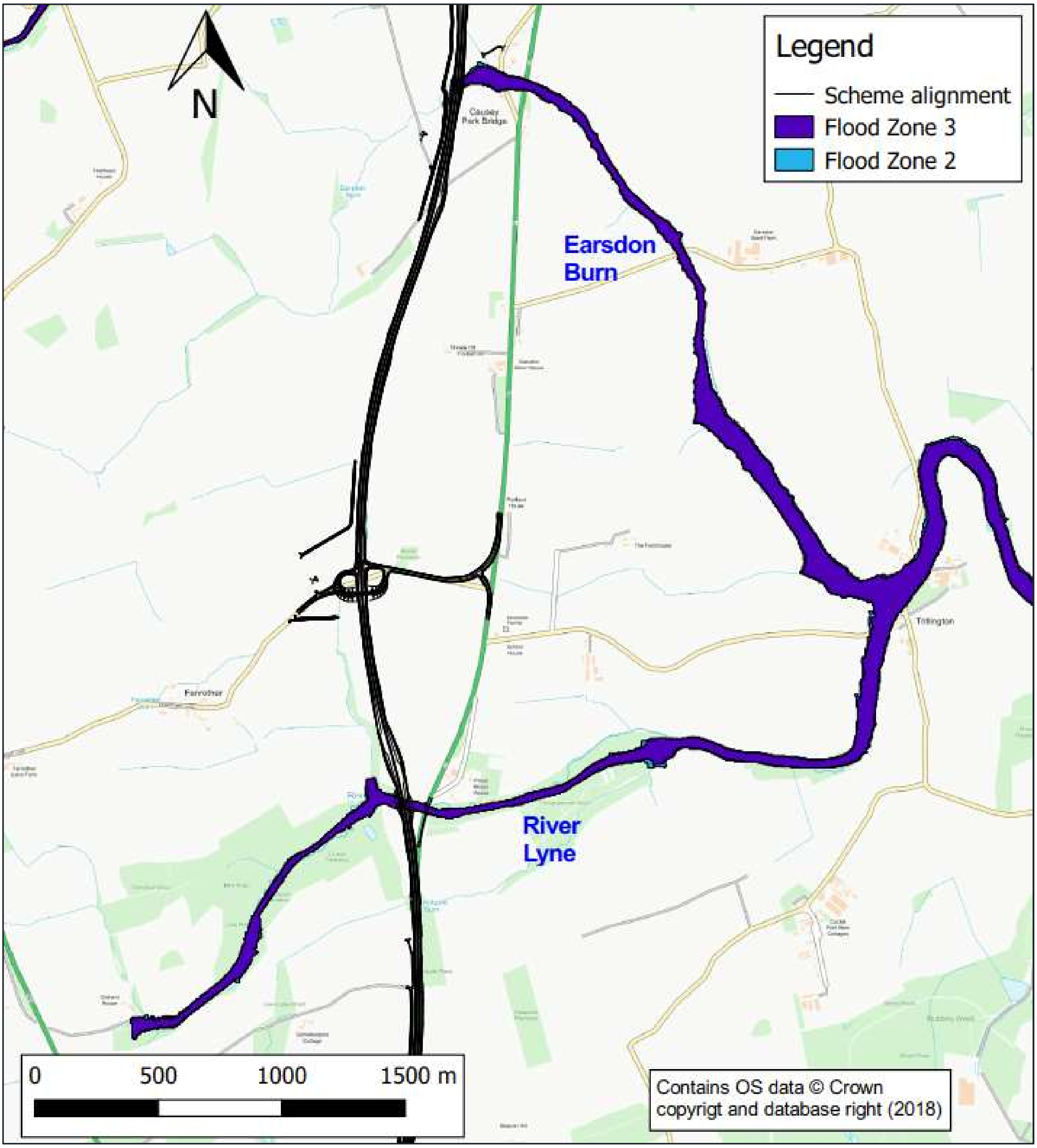

Figure 25 – Extract from Environment Agency Flood Map for Planning (September 2018) for the River Lyne and Earsdon

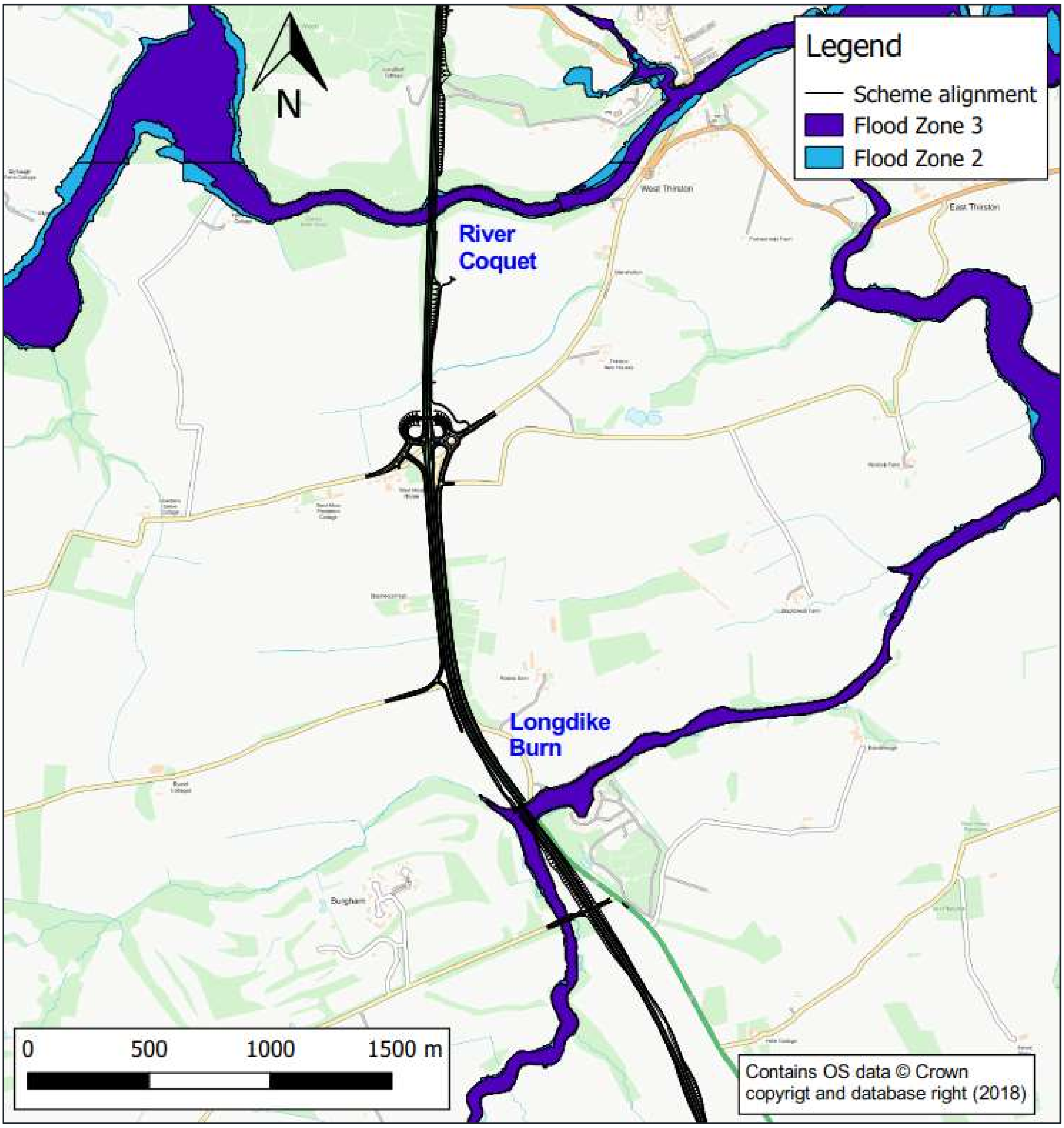

Figure 26 – Extract from Environment Agency Flood Map for Planning (September 2018) for Longdike Burn and River Coquet

DETAILED HYDRAULIC MODELLING

4.2.2. Detailed 1D hydraulic modelling has been undertaken for the Cotting Burn, River Lyne, Fenrother Burn, Earsdon Burn and Longdike Burn. These watercourses either have large drainage catchments with large tributaries, and hence inflows, close to Part A or a number of structures in the vicinity of the A1 that require detailed modelling to quantify the cumulative effect of these. The exception to this is the River Coquet which has not been assessed as part of this FRA and further details are provided in **Section 5.3**.

4.2.3. Full details of the hydraulic modelling work undertaken are provided in **Appendix A: Hydraulic Modelling Analysis**. **Figure 27** to **31** provide details of the flood risk extents in the existing condition for the 100 year and 1000 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.

4.2.4. The quality of the maps produced is dependent on the availability of local ground level data. LiDAR data was available for Earsdon Burn and Longdike Burn, the ground level data for the remaining watercourses has been developed from spot level data which did not pick up the channel to the same degree of accuracy.

4.2.5. Full details of the existing A1 structures and their hydraulic capacity are discussed in **Section 5: Post Development Flood Risk** to provide a comparison to the Part A proposals.

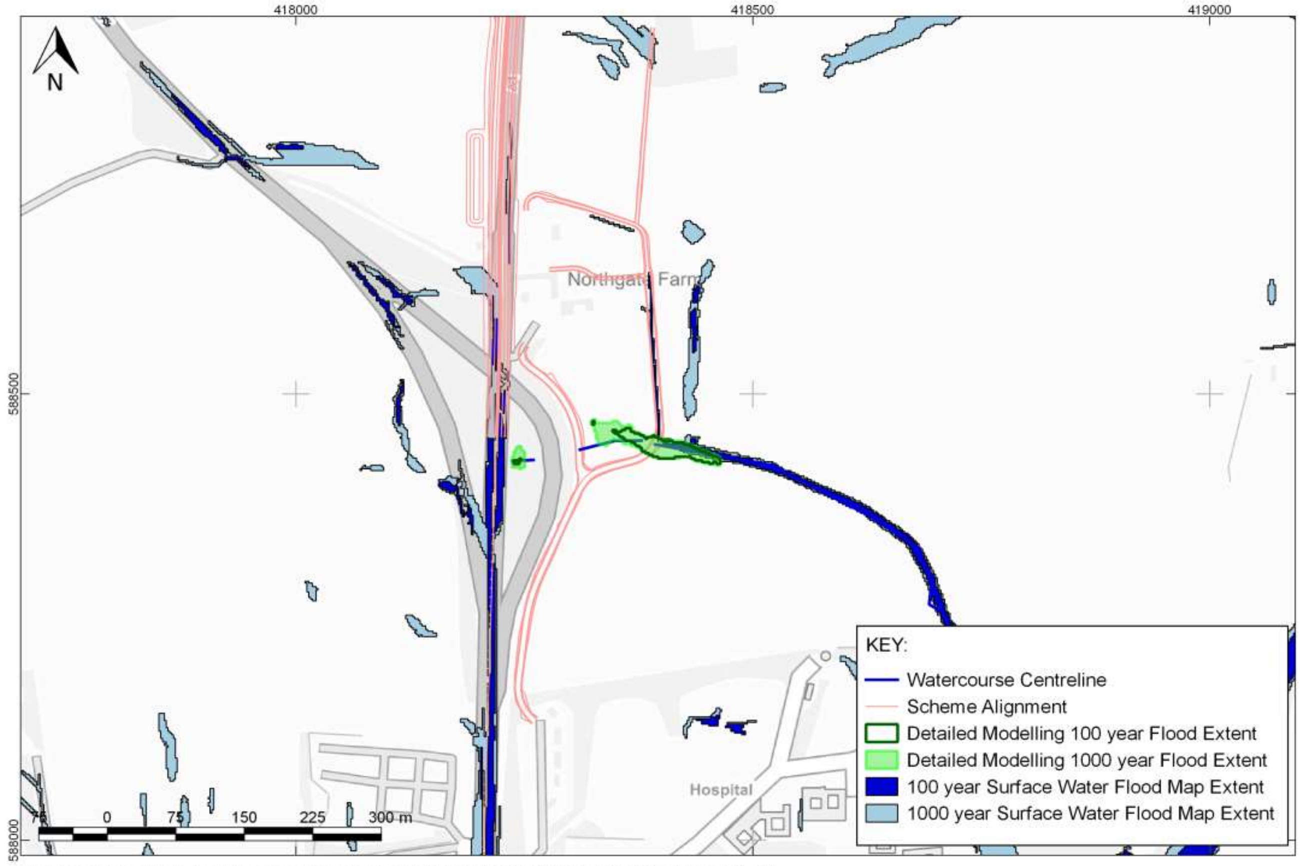

Figure 27 - Existing Flood Risk Extents for Cotting Burn

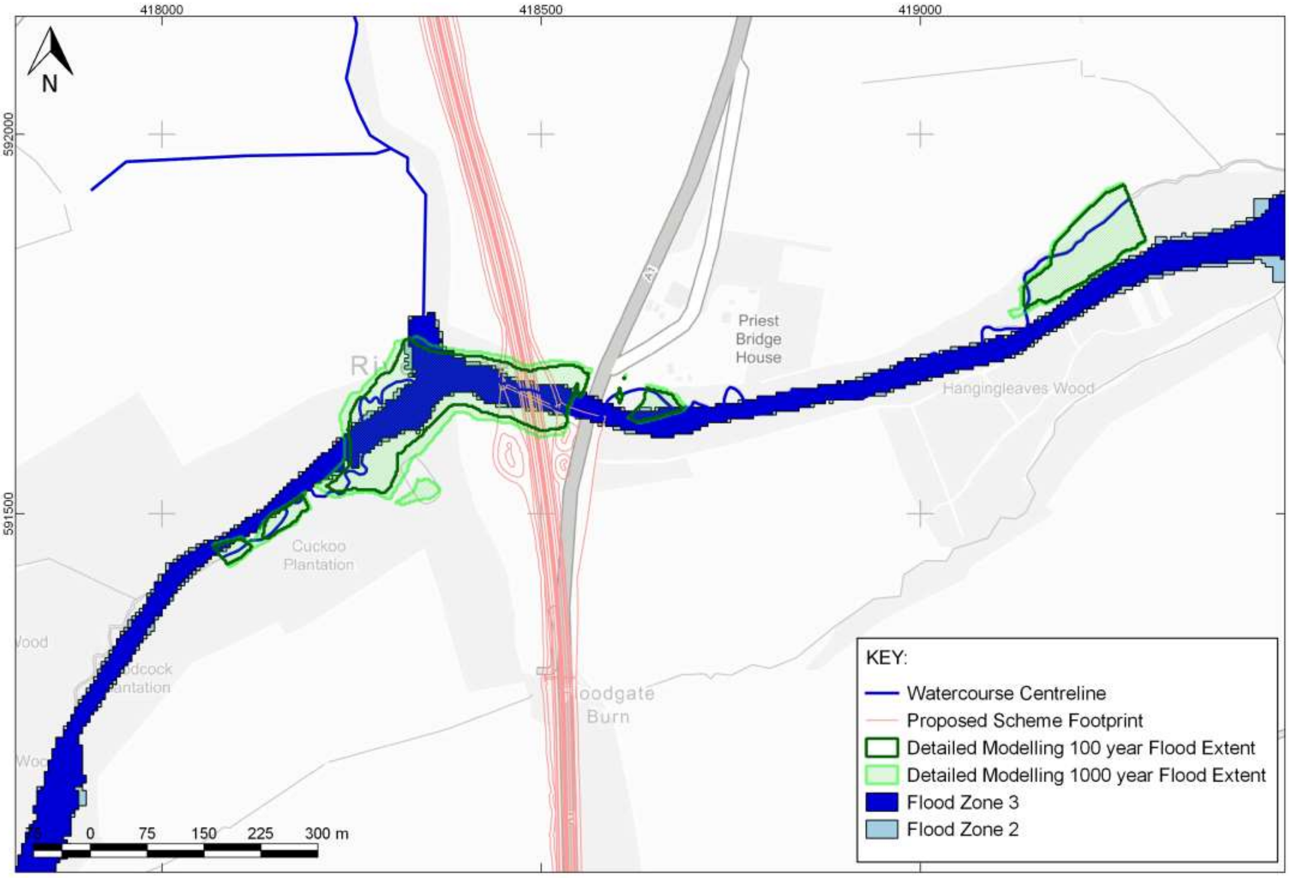

Figure 28 - Existing Flood Risk Extents for River Lyne

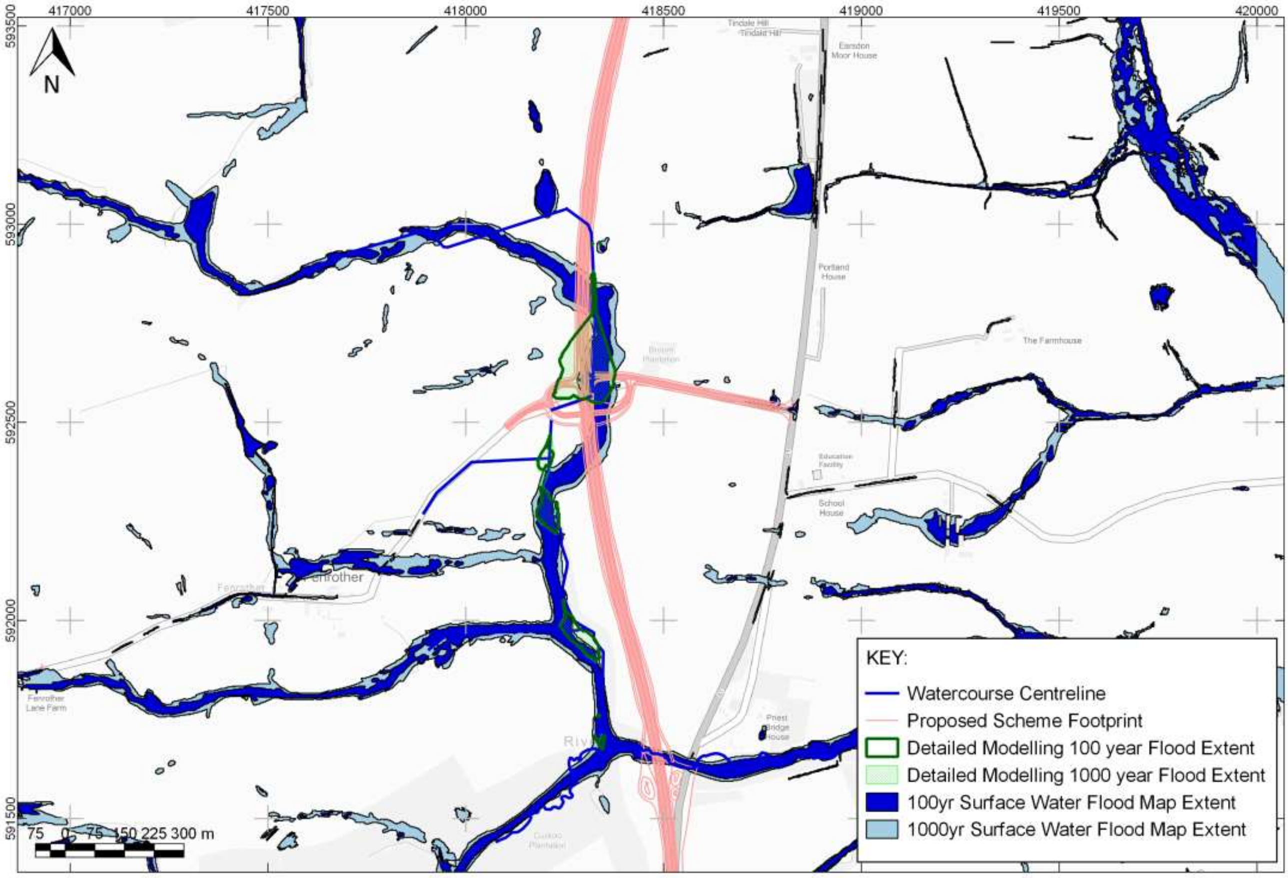

Figure 29 - Existing Flood Risk Extents for Fenrother Burn

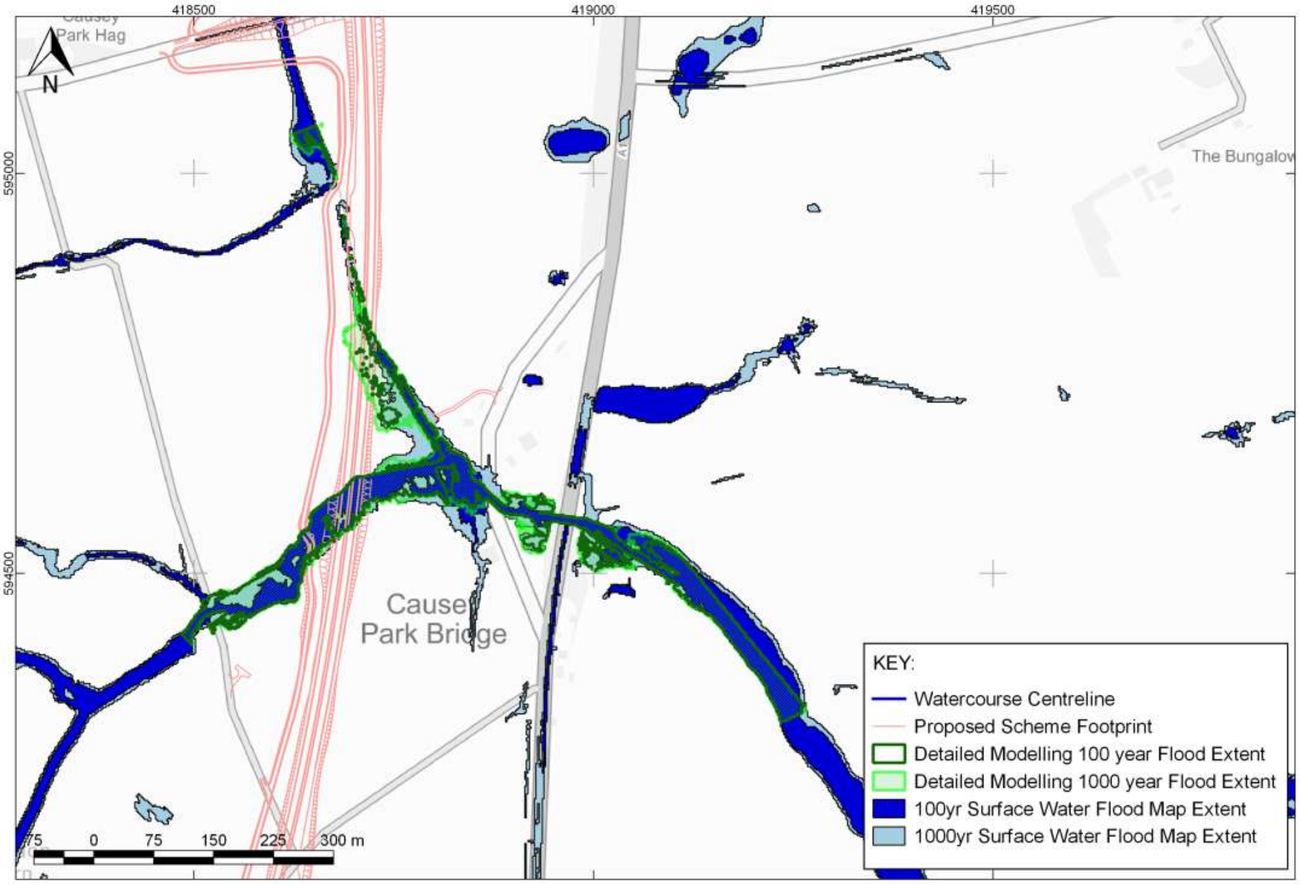

Figure 30 - Existing Flood Risk Extents for Earsdon Burn

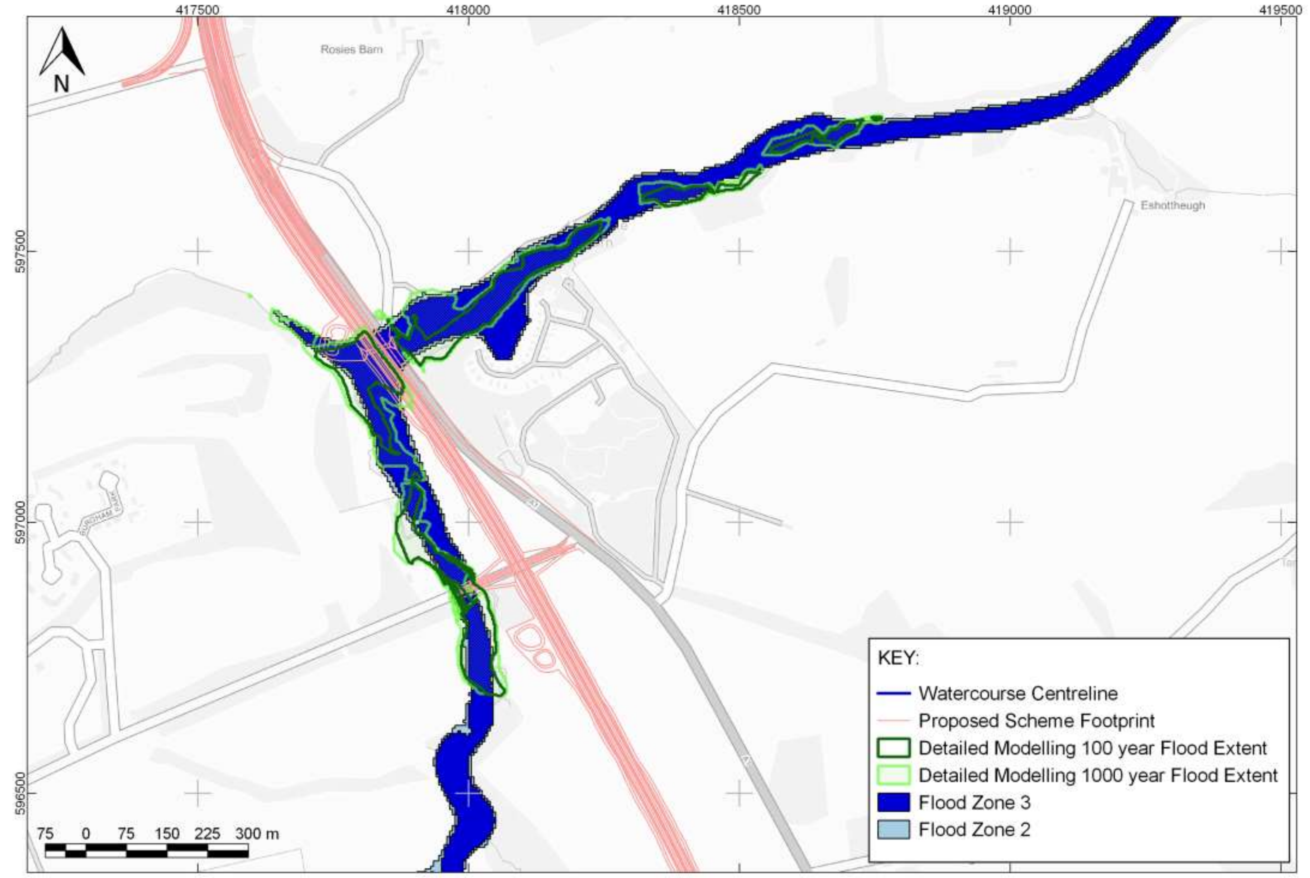

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Figure 31 - Existing Flood Risk Extents for Longdike Burn

4.3 OTHER SOURCES OF FLOOD RISK

TIDAL FLOODING

4.3.1. Part A is not at risk of tidal flooding as the tidal limits for the River Coquet and River Wansbeck are downstream of the Study Area. The tidal limit for the River Coquet is on the outskirts of Warkworth which is located approximately 9 km to the east of Part A. The tidal limit for the River Wansbeck is at Sheepwash which is located approximately 8 km to the east of Part A. The lowest elevation along Part A alignment is at the River Coquet which is located in a deep valley at approximately 35 m AOD, but the majority of Part A is between 80 to 150 m AOD.

SURFACE WATER FLOODING

4.3.2. A review of the Environment Agency's Flood Risk from Surface Water map (**Ref 10.1.7**) (shown in **Figure 32**) indicates that sections of Part A 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 A and details are provided in **Section 5** of this document.

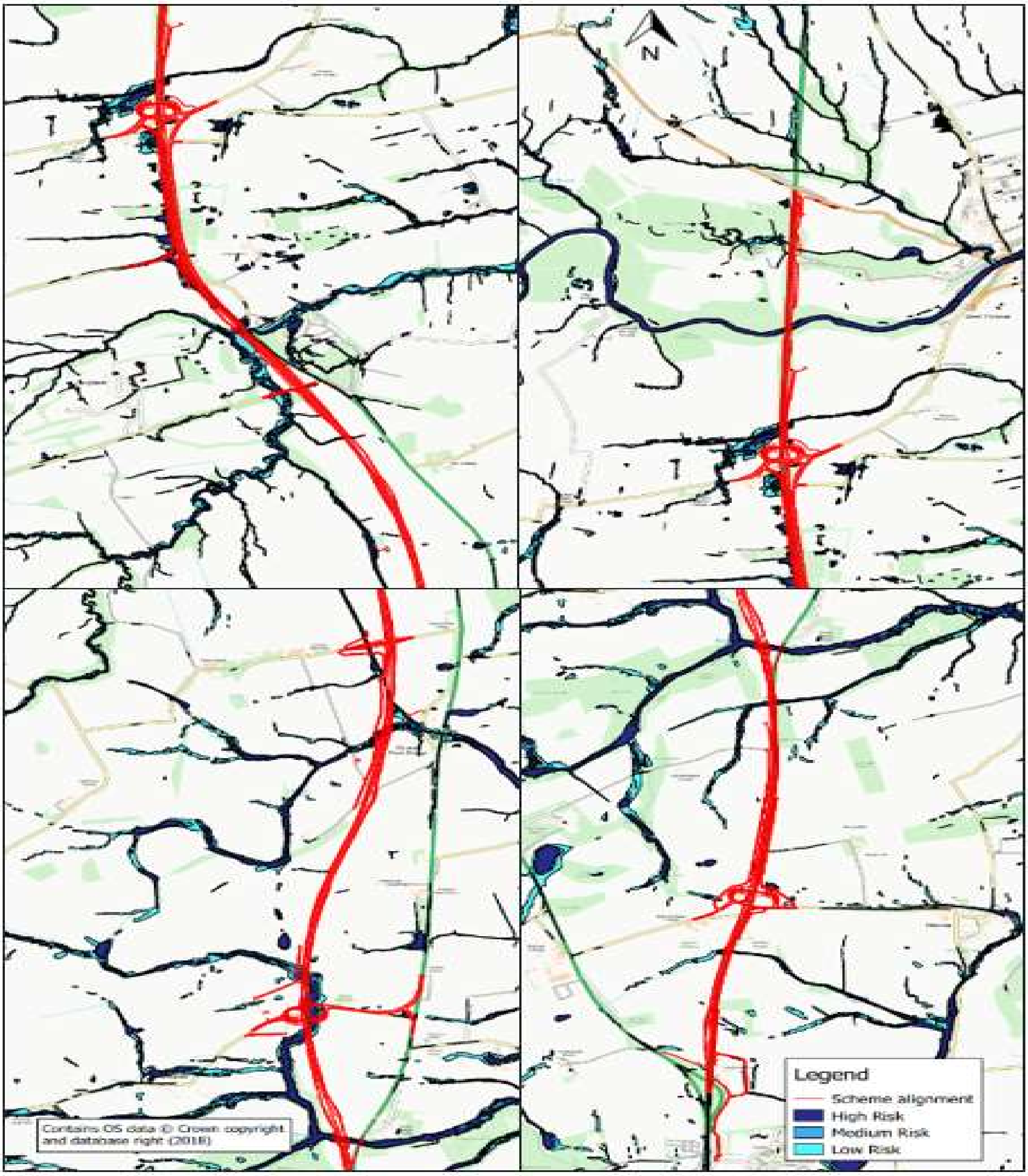

Figure 32 – Extract from Environment Agency Surface Water Flood Risk Map (September 2018)

GROUNDWATER

4.3.4. 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 A, groundwater levels are unlikely to fluctuate significantly and as a result, groundwater flooding is unlikely to occur.

4.3.5. The smaller areas of sand and gravel, associated with Earsdon Burn and Longdike Burn, which have a higher permeability may experience groundwater flooding. However, this should not affect Part A as any groundwater emergence would overflow to the adjacent watercourses.

ARTIFICIAL SOURCES

4.3.6. A review of the Environment Agency's Flood Risk from Reservoirs map (**Ref 10.1.7**) indicates that the River Coquet is located at the downstream extent of the area identified to be at risk of flooding from the potential failure of Rayburn Lake located approximately 9.3 km to the south-west of where the existing A1 crosses the River Coquet.

4.3.7. As Part A is located a significant distance from the reservoir, and the likelihood of reservoir failure is considered to be very small, the risk to Part A is not deemed to be significant.

OTHER SOURCES OF FLOOD RISK

4.3.8. 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 A includes the following works from south to north as set out in **Figure 33** below (the numbers in brackets relate to the approximate location of the works):

a. The replacement of the three existing circular culverts along Cotting Burn, downstream of the existing A1 and slip road with two new box culverts (1).
b. The replacement of the existing culvert along Shieldhill Burn with a new circular culvert (2).
c. The replacement of the existing arch culvert along Floodgate Burn with a new circular culvert (3).
d. The construction of a new culvert where Part A crosses the River Lyne (4).
e. The removal of the existing culvert along the tributary of Fenrother Burn, and the construction of two new culverts where Fenrother Burn crosses Fenrother Lane. The Fenrother Burn would be diverted along the west side of Part A between the two new culverts (5).
f. Construction of two new box culverts where Part A crosses the Earsdon Burn, the first situated beneath the new A1 alignment and the second beneath a new access road that runs along the western side of the A1 (6).
g. The diversion and channel realignment of an unnamed watercourse to a new confluence with the Earsdon Burn. This would include a new circular culvert beneath a new access road track upstream of the realignment and culverting of the downstream half of the diversion via the construction of a new circular culvert adjacent to the main A1 alignment (7).
h. Modification of the headwall of the existing culvert along Longdike Burn (8).
i. The extension of the existing culvert at Longdike Burn (and the Poxtondean Burn that discharges into the Longdike Burn) (9).
j. Construction of a new circular culvert where Part A crosses a surface water flow path south of Felmoor Park (10).
k. Replacement of the culvert that drains agricultural land to the west of Eshott Airfield (11).
l. Extension of the existing culvert on an unnamed watercourse which drains to the Thirston Burn (12).
m. New bridge crossing the River Coquet to the immediate east of the existing bridge (13).
n. Extension of the existing culvert on Bradley Brook (14).
o. 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.

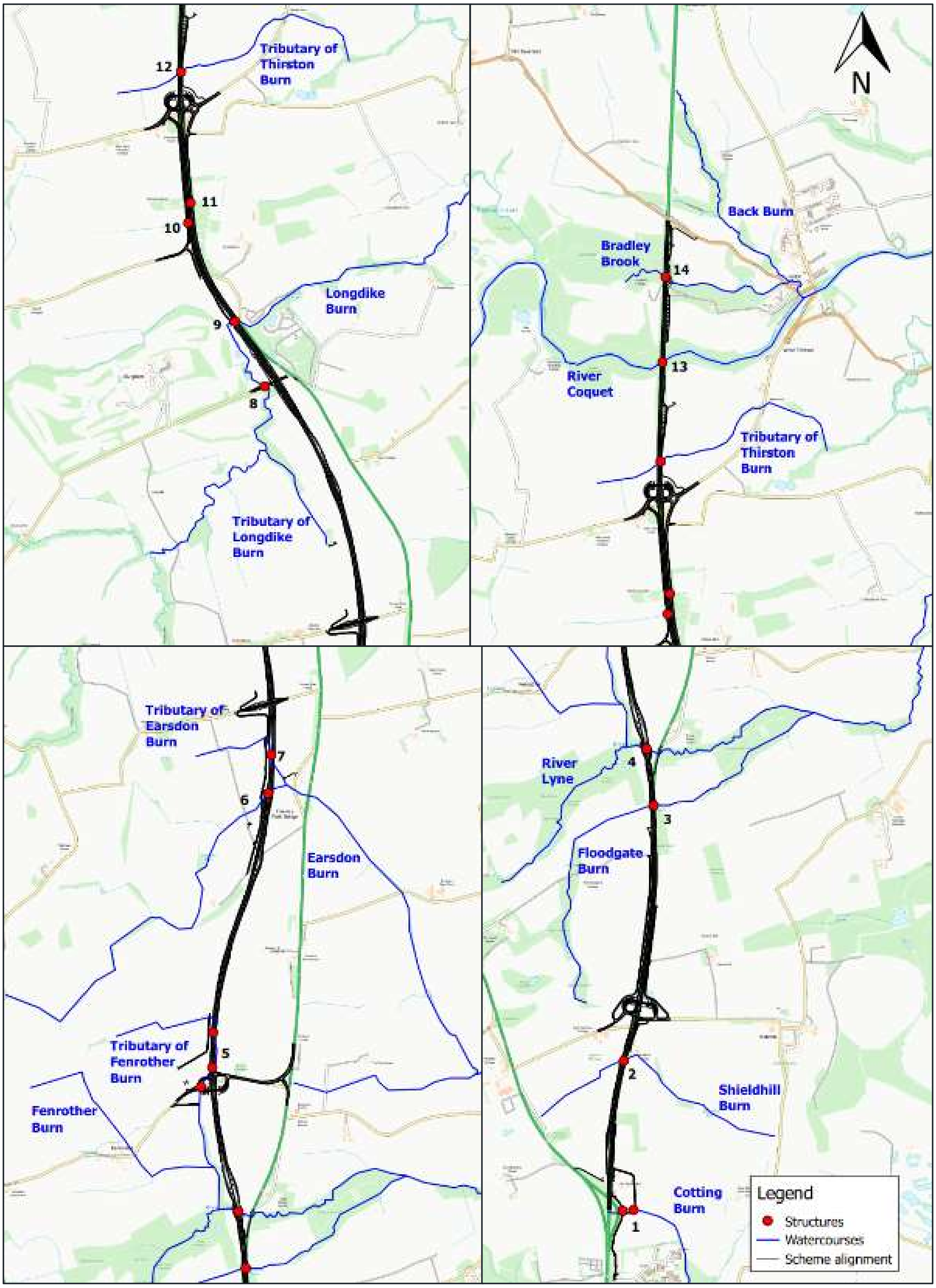

Figure 33 – Part A Extent and Proposed Works with Regards to Flooding

5.1.2. 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 5.4** to **5.8** with details of the hydrology and hydraulic modelling for each included in **Appendix A: Hydraulic Modelling Analysis**.

5.2.2. The remaining watercourses, drainage ditches and identified surface water flow paths crossing Part A are summarised in **Section 5.9** with the details of culvert modelling, including the culvert hydrology analysis, included in **Appendix B: Culvert Master Analysis**.

5.2.3. As detailed in the baseline section (**Section 4.2**) parts of Part A are identified as being located in the high-risk Flood Zone 3. The identified fluvial flood risk is associated with the following watercourses: The River Coquet, Longdike Burn (and the Poxtondean Burn that discharges into the Longdike Burn), Earsdon Burn, the River Lyne and Floodgate Burn. The fluvial floodplains associated with these watercourses are largely contained within the watercourse channels. As a result, any loss of floodplain has been accounted for within the hydraulic modelling and design of the watercourse crossings as set out below.

5.2.4. It should be noted that design of the culverts 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 RIVER COQUET

5.3.1. Detailed hydraulic analysis of the River Coquet has not been undertaken given the limited effect of Part A on flows within the River Coquet, as discussed with the Environment Agency. The proposals comprise a new River Coquet bridge adjacent to the existing one, that requires the construction of two new piers that are in the same alignment as the existing piers: one on the north bank of the river above the expected 100 year flood level and one on the south bank of the river below the expected 100 year flood level. **Figure 34** provides an overview of the existing and proposed piers.

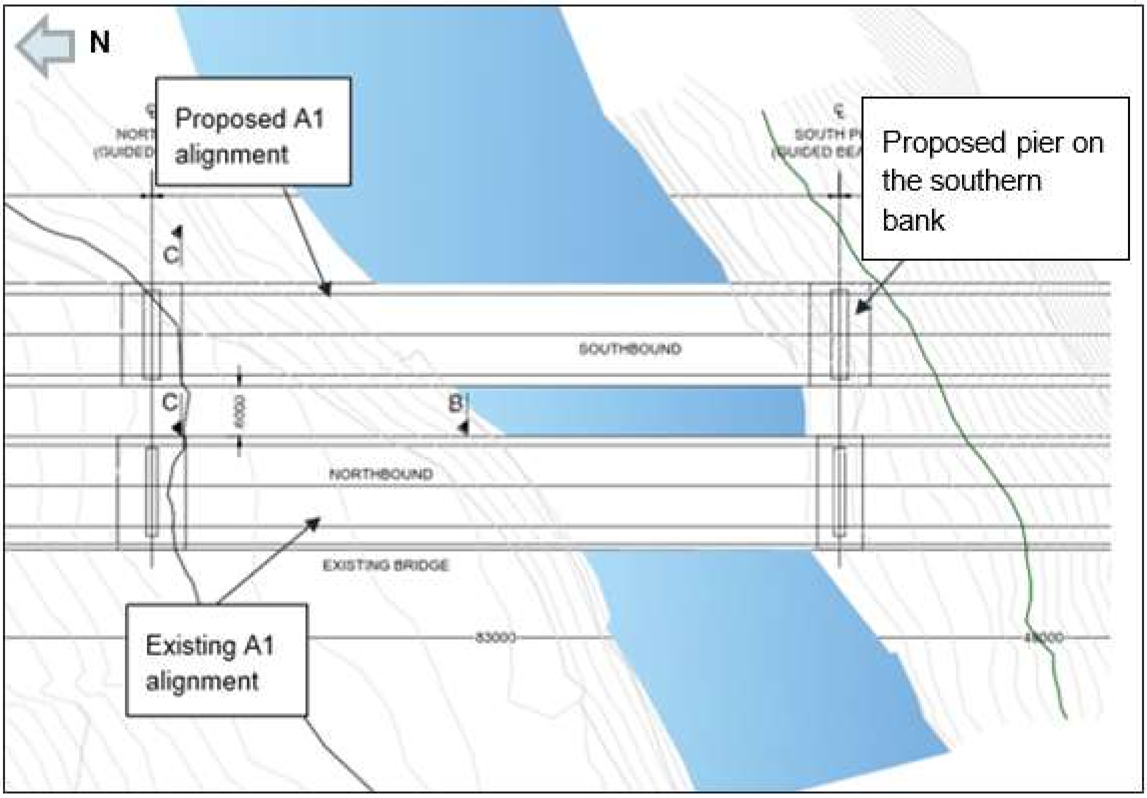

Figure 34 - Overview of Proposals on the River Coquet

5.3.2. The River Coquet at this location is within a deep valley, the top of which on either side is approximately 30 m above the bed level. This represents a challenge for the collection of survey data and as such to understand the need for detailed modelling of this structure a preliminary review of potential risk was undertaken.

5.3.3. The channel itself, given its steep sided topography, is picked out well in the LiDAR data. The bed of the channel beneath the A1 has a gradient in the region of 1 in 150. Approximately 700 m downstream of the A1 is a weir with a 2 m drop and it is evident this would act as the downstream control on the water levels in the reach upstream. It can be concluded the backwater effect from the downstream boundary would be limited. A preliminary assessment of peak water levels in the channel was completed using a flow of 525 m^3/s, which is the highest recorded flow on the river. When compared to the hydrology developed for the Felton Flood Mapping and Gravel Assessment (**Ref 10.1.31**) completed by the Environment Agency in 2009 this flow is greater than the 1000 year event. All flow up to the 1000 year event is contained within the steep sided channel valley.

5.3.4. A manning's calculation using the above data and a cross section taken from the topographic data indicates a peak water level in a 1 in 1000 year event of 36.7 m AOD.

5.3.5. The proposed pier on the south bank would be located along the same alignment as the existing pier with the base at a minimum elevation of 34 m AOD at its upstream end extending onto higher ground at its downstream face. Consideration has been given to the area of the proposed pier in comparison to the cross-sectional area of the channel at this location. Removing this section from the cross section of the channel and again completing a manning's calculation indicates a revised peak water level of 36.8 m AOD; 0.1 m higher than the peak water level estimated above. This is a localised effect that is not considered to extend a notable distance upstream or downstream of the proposed pier.

5.3.6. The nearest flood risk receptors are Shothaugh Farm High Cottage and Otter House located approximately 800 m upstream of the River Coquet bridge at an estimated elevation of 44.4 m AOD. The analysis presented in **paragraphs 5.3.4** and **5.3.5** indicates that the potential for localised changes in water level at the bridge of approximately 0.1 m in the peak event would have no impact on identified receptors.

5.3.7. To manage any risk resulting from the construction of the bridge deck it has been agreed that the temporary works for this structure would use a kingpost solution to maintain the levels of the bridge deck as it is pushed across the river rather than a temporary pier. This would remove the need for any temporary works within the channel for all aspects apart from the single pier (discussed above) that is located on the south bank of the river below the expected 100 year flood level. Discussions with the Environment Agency have confirmed that the recommendations described are acceptable and are included in **Appendix 4.2: Environmental Consultation, Volume 1** of this ES (**Application Document Reference: TR010041/APP/6.1**).

5.4 COTTING BURN

OVERVIEW OF PART A REQUIREMENTS

5.4.1. Cotting Burn is a rural watercourse with an upstream catchment of 0.75 km^2. There are no flood risk receptors upstream of the A1 but a short distance downstream is the Northgate Hospital where the watercourse passes close to buildings on the right bank. The Environment Agency have also confirmed there are major flooding issues downstream and any proposals should not increase flood risk downstream.

5.4.2. An overview of the proposals in relation to Cotting Burn is provided in **Figure 35**. This is crossing number one in the list of works associated with Part A in **Section 5.1**.

5.4.3. The existing culverts beneath the A1 and slip road would not be affected as part of Part A (shown as culverts 1 and 2 in **Figure 35**). The three existing circular culverts labelled as 3, 4 and 5 in **Figure 35** would be replaced with two new box culverts (West Cotting Burn Culvert (1.4) and East Cotting Burn Culvert (1.5)) to provide new access roads for Northgate Farm, Warrener's House and Capri Lodge.

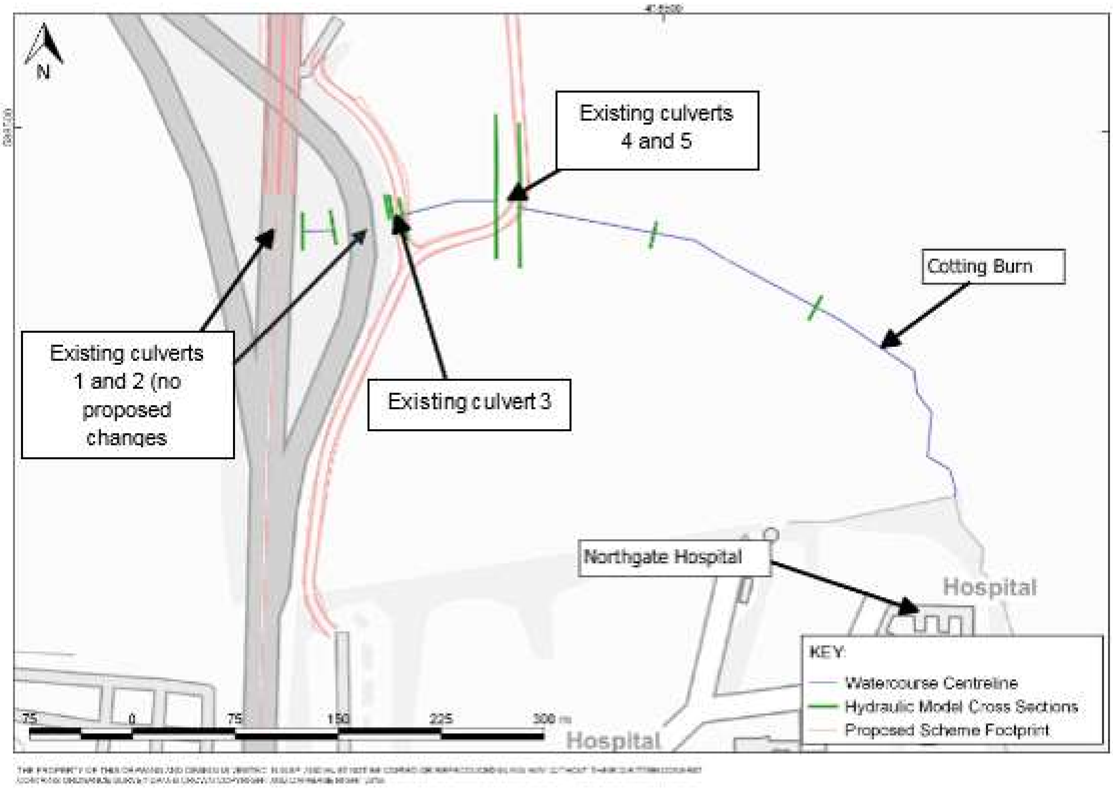

Figure 35 – Overview of Proposals in Relation to Cotting Burn

PART A PROPOSALS

5.4.4. The iterative process described in **Table 2-7** to develop a design for the culverts which satisfies both the flood risk and environmental requirements has resulted in proposals for the new culverts as set out in **Table 5-1**. The proposed culverts are substantially larger than the existing structures with the height being constrained by the elevation of the access tracks. **Table 5-1** also details the dimensions of the existing structures for comparison.

Table 5-1 - Existing and Proposed Dimensions of Cotting Burn Structures

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing culvert 1	28	Circular	0.3	-
Existing culvert 2	41.7	Circular	0.9	-
Existing culvert 3	7	Circular	0.35	-
Existing culvert 4	4	Circular	0.35	-
Existing culvert 5	15.6	Circular	0.45	-

Structure	Length (m)	Shape	Width (m)	Height (m)
Proposed West Cotting Burn Culvert (1.4)	12.8	Box	2.7	1.25
Proposed East Cotting Burn Culvert (1.5)	12.8	Box	3.0	1.20

DESIGN OUTCOMES

5.4.5. **Table 5-2** provides details of the freeboard associated with each structure for a range of flood events. As Cotting Burn is a small ordinary watercourse a design freeboard of 300 mm is preferred in the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref 10.1.10**). The 1000 year event is larger than the 100 year + 50 % climate change event and so has been used to assess resilience and risk in an extreme event. Blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 67 %.

Table 5-2 - Design Freeboard for Cotting Burn Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage
West Cotting Burn Culvert (1.4)	0.31	0.59	0.2	0.07	0.00	0.43	0.05	-0.07	-
East Cotting Burn Culvert (1.5)	0.00	0.58	0.26	0.16	0.13	0.59	0.25	0.19	-

5.4.6. **Table 5-2** shows that a freeboard of 300 mm is not achieved in the 100 year + 25 % climate change event for either structure. Divergence from the preferred standard in this instance is considered acceptable on the basis that these are farm access tracks only and the design provides significant betterment compared to the size of the existing structures. Traffic flow along the tracks would be highly infrequent and any overtopping flows return to the channel immediately downstream.

5.4.7. There is very limited headroom between the culvert soffits and the carriageway crest level for these structures. However, neither structure overtopped in the hydraulic assessment of either the 1000 year event or the blockage scenario.

PREDICTED FLOOD RISK IMPACTS

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

5.4.9. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. The Environment Agency have raised concerns regarding downstream flood risk. It is noted that during this assessment it was not possible to collect survey of the upstream face of the existing A1 culvert. Design flows have therefore been included in the model downstream of this structure and the potential attenuating effects of the A1 are not included in the assessment. The flows presented below are therefore conservative.

5.4.10. **Figure 36** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part A. **Figure 37** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 250 m downstream of the proposed culvert East.

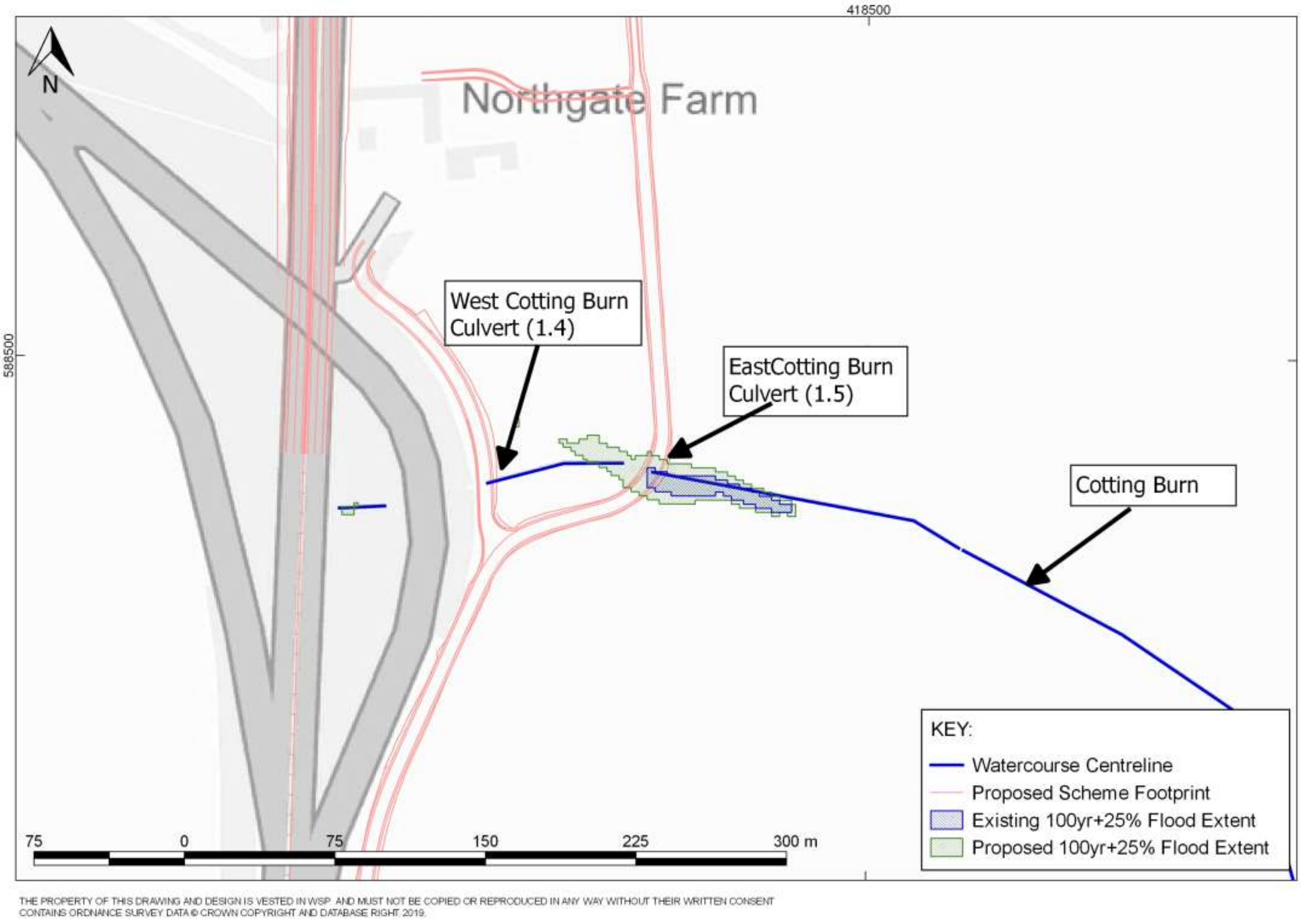

Figure 36 – Flood Extents in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

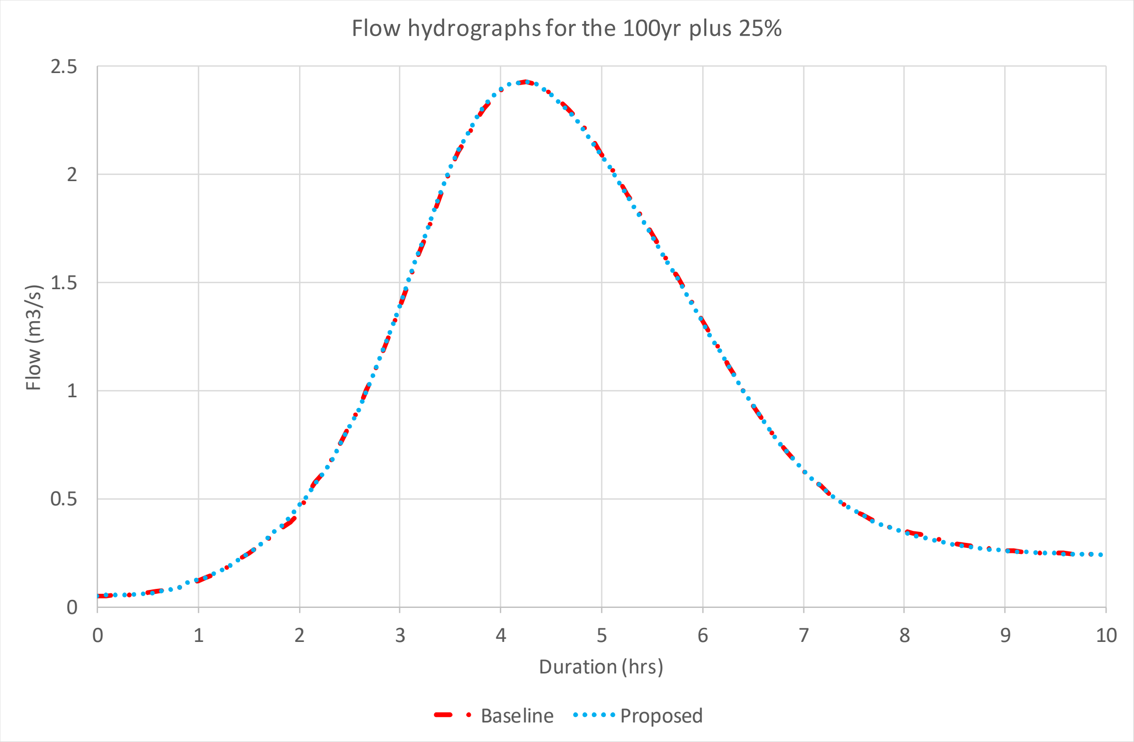

Figure 37 – Pass Forward Flows in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

5.4.11. **Figure 36** demonstrates that the proposals would reduce water levels in the vicinity of the A1 as the new culverts are significantly larger than existing. **Figure 37** confirms that there would be no change to the downstream flows resulting from Part A for all flows up to the 100 year + 25 % climate change event. This is because in the existing situation, the small dimensions of the existing culverts allow flows to back up and overtop the existing access tracks, as a result providing no attenuating function. Therefore, increasing the size of these structures does not result in an increase in downstream flows.

5.5 RIVER LYNE

OVERVIEW OF PART A REQUIREMENTS

5.5.1. The River Lyne drains a rural catchment approximately 8 km^2 in size to the downstream side of the A1, incorporating the Fenrother Burn tributary (refer to **Section 5.6**). The catchment is entirely rural with no flood risk receptors within 2 km upstream of the A1. Priest's Bridge House is located immediately downstream of the A1 on the northern bank. The village of Tritlington is approximately 5 km downstream; there were no concerns raised about flood risk to this village in discussions with the Environment Agency.

5.5.2. An overview of Part A in relation to the River Lyne is provided in **Figure 38**.

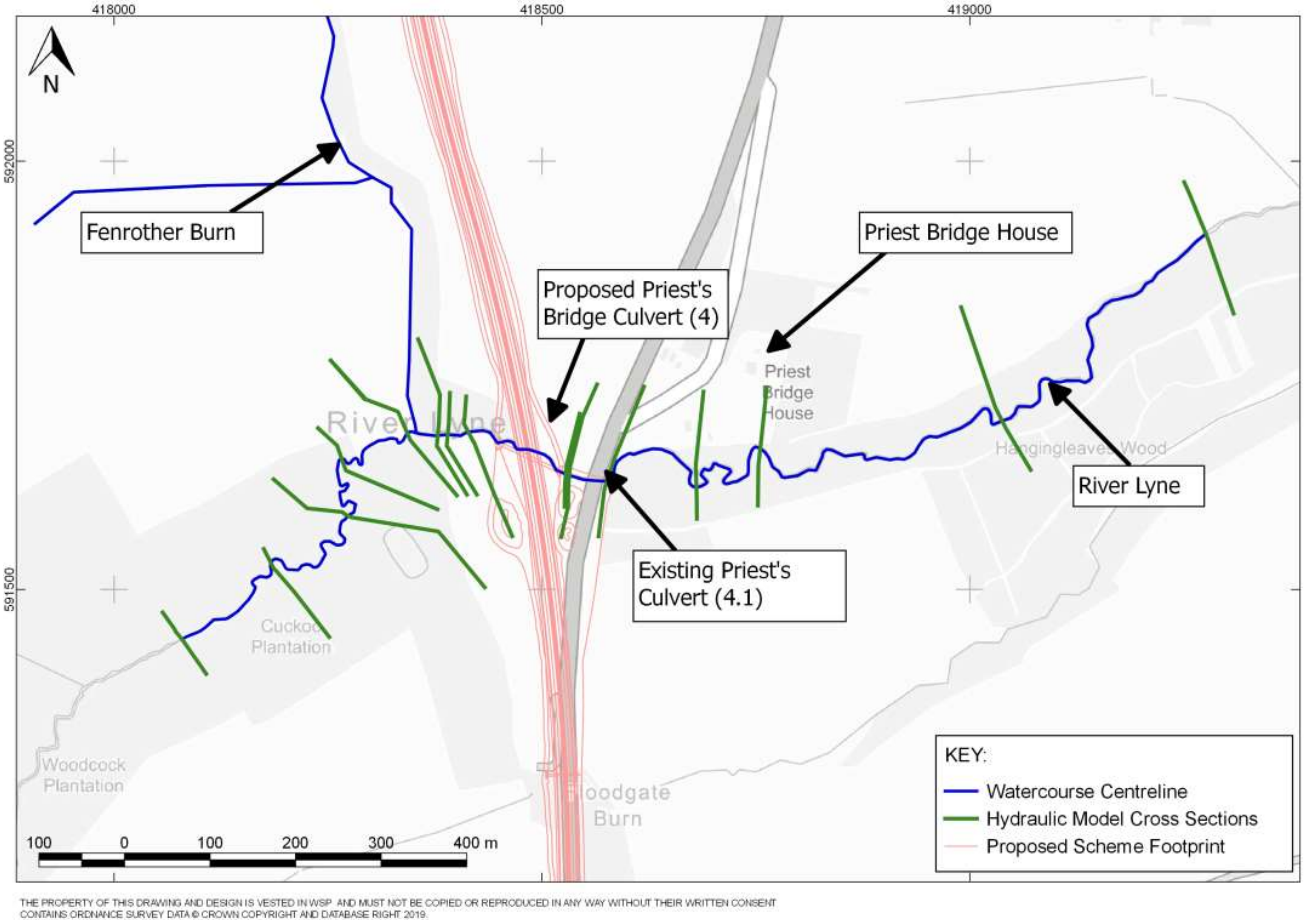

Figure 38 – Overview of Proposals in Relation to the River Lyne

5.5.3. Part A diverges from the existing A1 alignment in the vicinity of the River Lyne, taking a route to the west. Part A would require the construction of one new structure (Priest’s Bridge Culvert (4)) on the west side of the existing A1 beneath this new alignment. The structure would tie into the existing alignment of the River Lyne.

5.5.4. The existing culvert beneath the A1 (Priest’s Bridge (4.1)) is situated 45 m downstream of the proposed culvert. There would be no changes to the existing structure beneath the current alignment.

PART A PROPOSALS

5.5.5. 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 new culvert as set out in **Table 5-3** below. **Table 5-3** also details the dimensions of the existing A1 culvert located downstream of the proposed culvert for comparison. The proposed culvert is substantially larger than the existing structure.

Table 5-3 - Existing and Proposed Dimensions of River Lyne Structure

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing Priest's Bridge (4.1)	34	Ovoid	1.95	2.66
Proposed Priest's Bridge Culvert (4)	53	Box	4.0	3.75

DESIGN OUTCOMES

5.5.6. **Table 5-4** provides details of the freeboard associated with each structure for a range of flood events. The River Lyne is classified as a main river downstream from the village of Tritlington but is classified as an ordinary watercourse in the location of the A1. As such a design freeboard of 300 mm is preferred in the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref 10.1.10**). The 1000 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 %.

Table 5-4 - Design Freeboard for River Lyne Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25% cc	1000 year	100 year + 25 % cc with blockage
Existing Culvert	4.37	0.56	-1.05	-1.83	-	1.25	0.34	0.11	-
Proposed Priest's Bridge Culvert (4)	2.57	2.18	0.56	-0.24	0.39	2.19	0.56	-0.22	-

5.5.7. **Table 5-4** shows that a freeboard of 300 mm is achieved in the 100year + 25 % climate change event. There is significant freeboard to the carriageway crest level and Part A is not overtopped in either the extreme 1000 year event or allowing for 30 % blockage of the structure.

PREDICTED FLOOD RISK IMPACTS

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

5.5.9. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. **Figure 39** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part A. **Figure 40** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 1 km downstream of the existing A1.

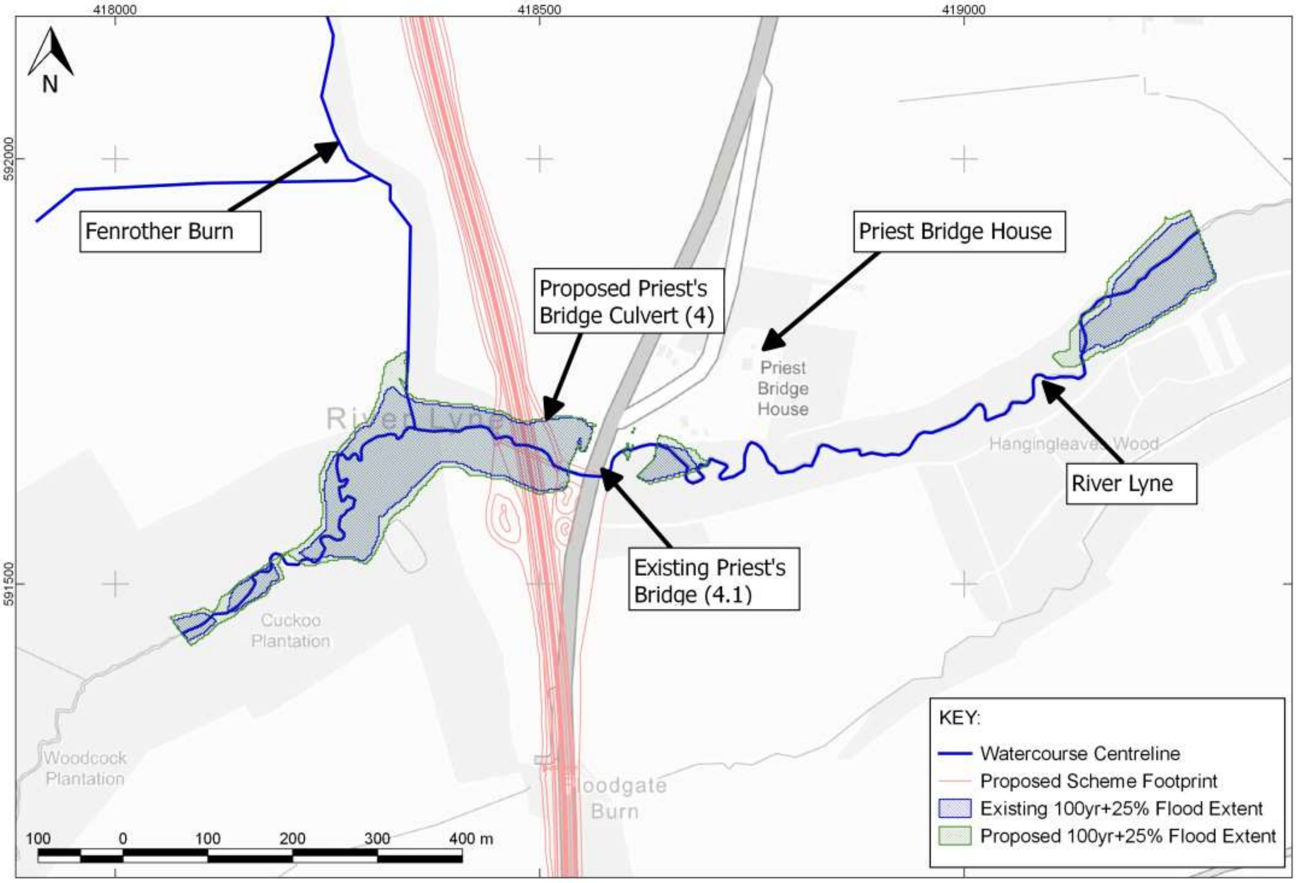

Figure 39 – Flood Extents in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

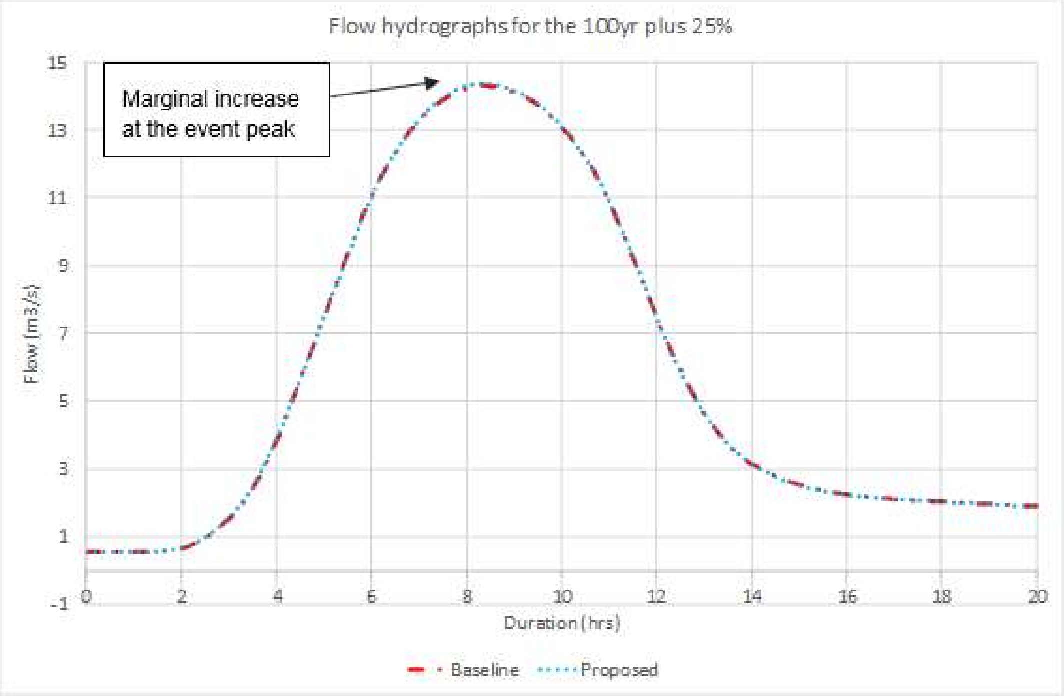

Figure 40 – Pass Forward Flows in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

5.5.10. **Figure 39** demonstrates that the proposals would result in a marginal increase in water levels upstream of the A1 for a distance of approxmiately 500 m. There is no discernable increase in water levels downstream (less than 0.01 m) and this is reflected in **Figure 40**, which shows an increase in downstream flows of 0.06 m^3/s at the peak of the event. This results from the marginally more efficient conveyance represented in the model for the proposed culvert when compared to the existing channel.

5.5.11. The effect of this increase in flows is offset by the reduced flows coming from the Fenrother Burn tributary following Part A, discussed in **Section 5.6**. The River Lyne is a larger catchment than Fenrother Burn and has a slower flood response to the confluence with the Fenrother Burn as a result. To quantify the impact of Part A the River Lyne and Fenrother Burn models have been combined and the pass forward flows extracted, as shown in **Figure 41**. The effect is an overall reduction in flows resulting from Part A.

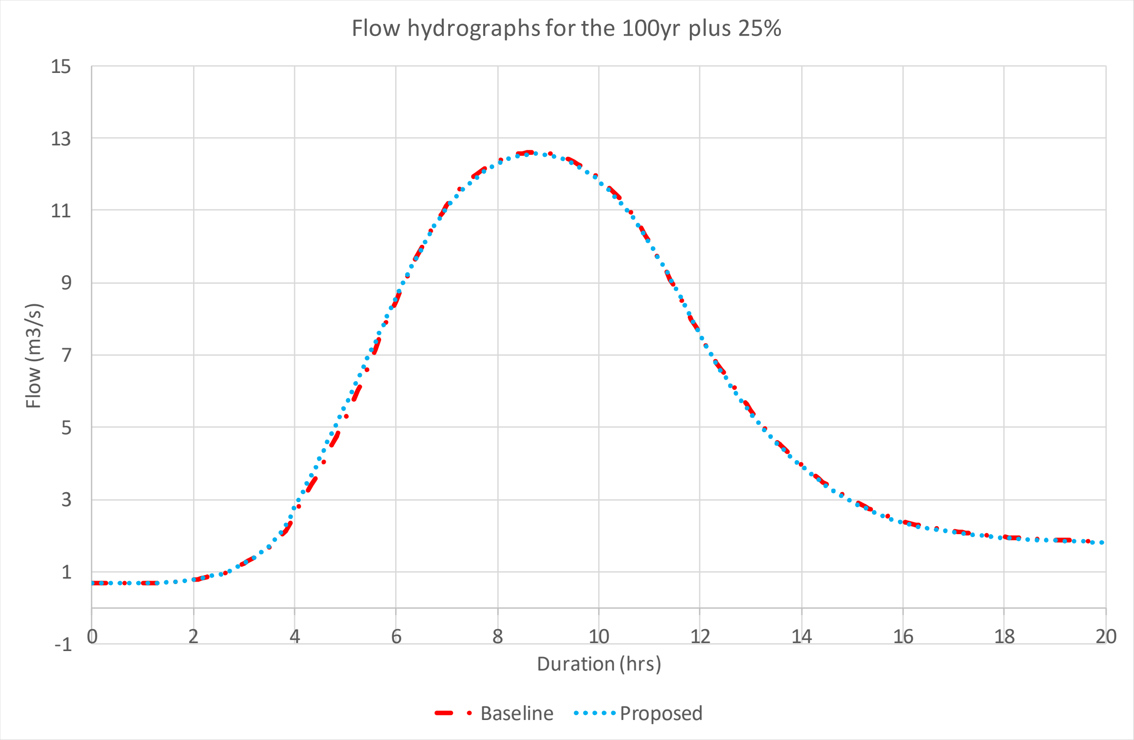

Figure 41 – Pass Forward Flows in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

5.6 FENROTHER BURN

OVERVIEW OF PART A REQUIREMENTS

5.6.1. Fenrother Burn is a tributary of the River Lyne (discussed above in **Section 5.5**) and drains a rural catchment approximately 3 km^2 to its confluence with the River Lyne. Most of this catchment is drained via two tributaries from the west. The catchment of the watercourse that flows adjacent to and across the Order Limits of Part A is 0.5 km^2 only. The catchment is entirely rural with no flood risk receptors upstream of the A1 crossing. The downstream receptors are as described for the River Lyne watercourse.

5.6.2. An overview of Part A in relation to the Fenrother Burn is provided in **Figure 42**.

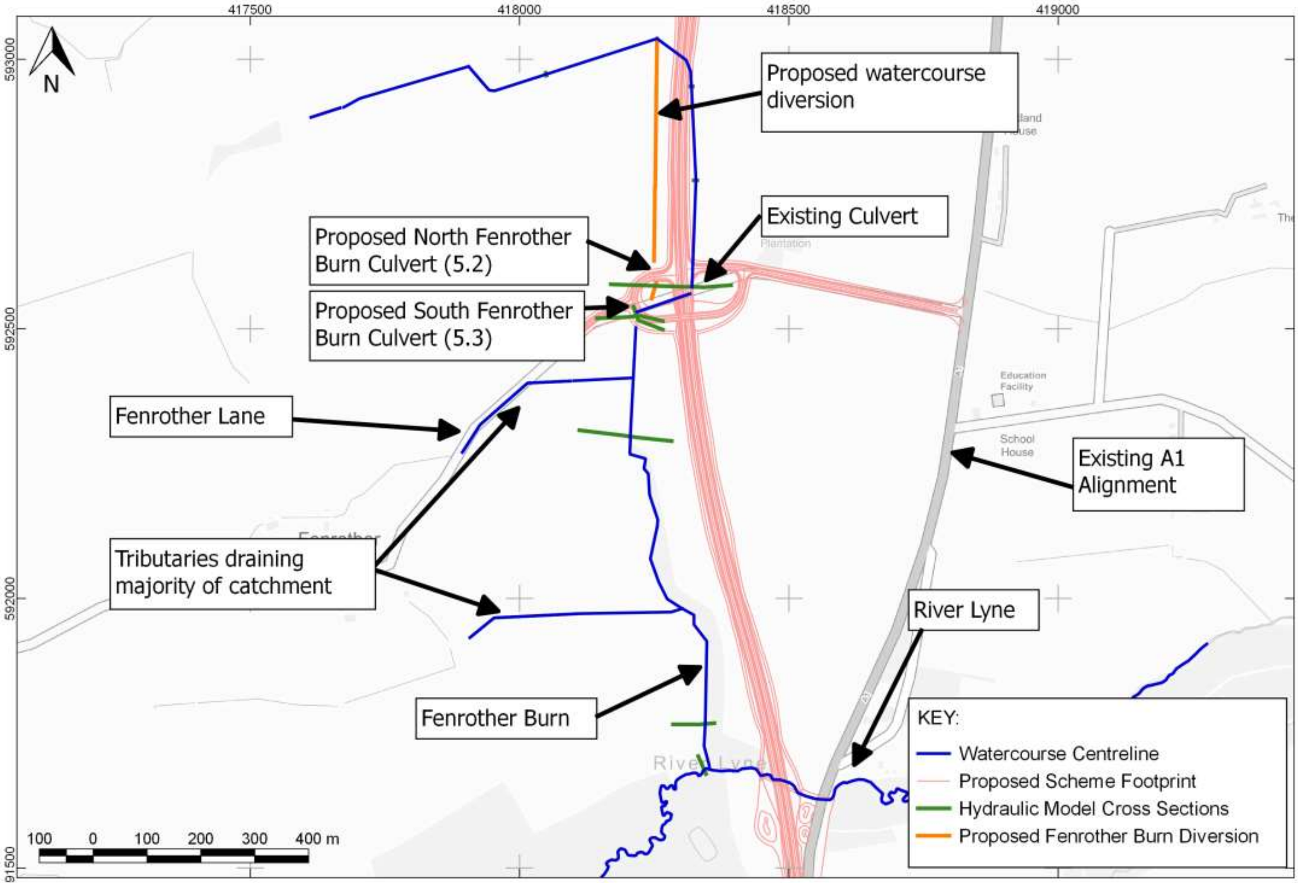

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Figure 42 – Overview of Proposals in Relation to the Fenrother Burn

5.6.3. An approximate 550 m stretch of the Fenrother Burn is proposed to be diverted to the west of Part A upstream of Fenrother Lane. The existing crossing of Fenrother Burn beneath Fenrother Lane is to the east of Part A and this would be replaced with two new culverts beneath the proposed junction. This would result in a reduction in channel length of approximately 100 m.

PART A PROPOSALS

5.6.4. 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 new culverts as set out in **Table 5-5**. **Table 5-5** also details the dimensions of the existing structure located to the east of Part A for comparison. The proposed culverts are substantially larger than the existing structure.

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

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing A1 Culvert	119.5	Circular	1.0 upstream 0.5 downstream	-
Proposed North Fenrother Burn Culvert (5.2)	33.1	Twin Box	1.5 (each culvert)	1.25 (each culvert)
Proposed South Fenrother Burn Culvert (5.3)	52.7	Box	3.0	1.75

DESIGN OUTCOMES

5.6.5. **Table 5-6** provides details of the freeboard associated with each structure for a range of flood events. Fenrother Burn is an ordinary watercourse. As such a design freeboard of 300 mm is preferred in the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref 10.1.10**). The 1000 year event is larger than the 100 year + 50 % climate change event so 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 67 % for culvert A and by 30 % for culvert B reflecting the different sizes of these structures and hence the likelihood of blockage.

Table 5-6 - Design Freeboard for Fenrother Burn Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25% cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25% cc	1000 year	100 year + 25 % cc with blockage
Existing A1 Culvert	0.45	-0.74	-0.92	-0.96	-	-0.21	- 0.69	-0.80	-
Proposed North Fenrother Burn Culvert (5.2)	0.37	0.55	-0.10	-0.02	-0.03	0.56	- 0.12	-0.01	-
Proposed South Fenrother Burn Culvert (5.3)	3.97	1.06	0.63	0.52	0.60	0.93	0.51	0.42	-

5.6.6. **Table 5-6** shows that a freeboard of 300 mm is broadly achieved in the 100 year+ 25 % climate change event for culvert B, however the 300 mm freeboard is not achieved for culvert A. The height of culvert A is constrained by the design height of the road in this location and as such has required divergence from the preferred design requirements. Both culverts are, however, significantly larger and provide improved freeboard to the existing culvert.

5.6.7. The crest level of the carriageway is 0.62 m above the soffit of culvert A and 3.97 m above the soffit of culvert B. The results suggest Part A would not be overtopped in either the extreme 1000 year event or allowing for blockage of the structures.

PREDICTED FLOOD RISK IMPACTS

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

5.6.9. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. **Figure 43** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part A. **Figure 44** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 900 m downstream of Part A.

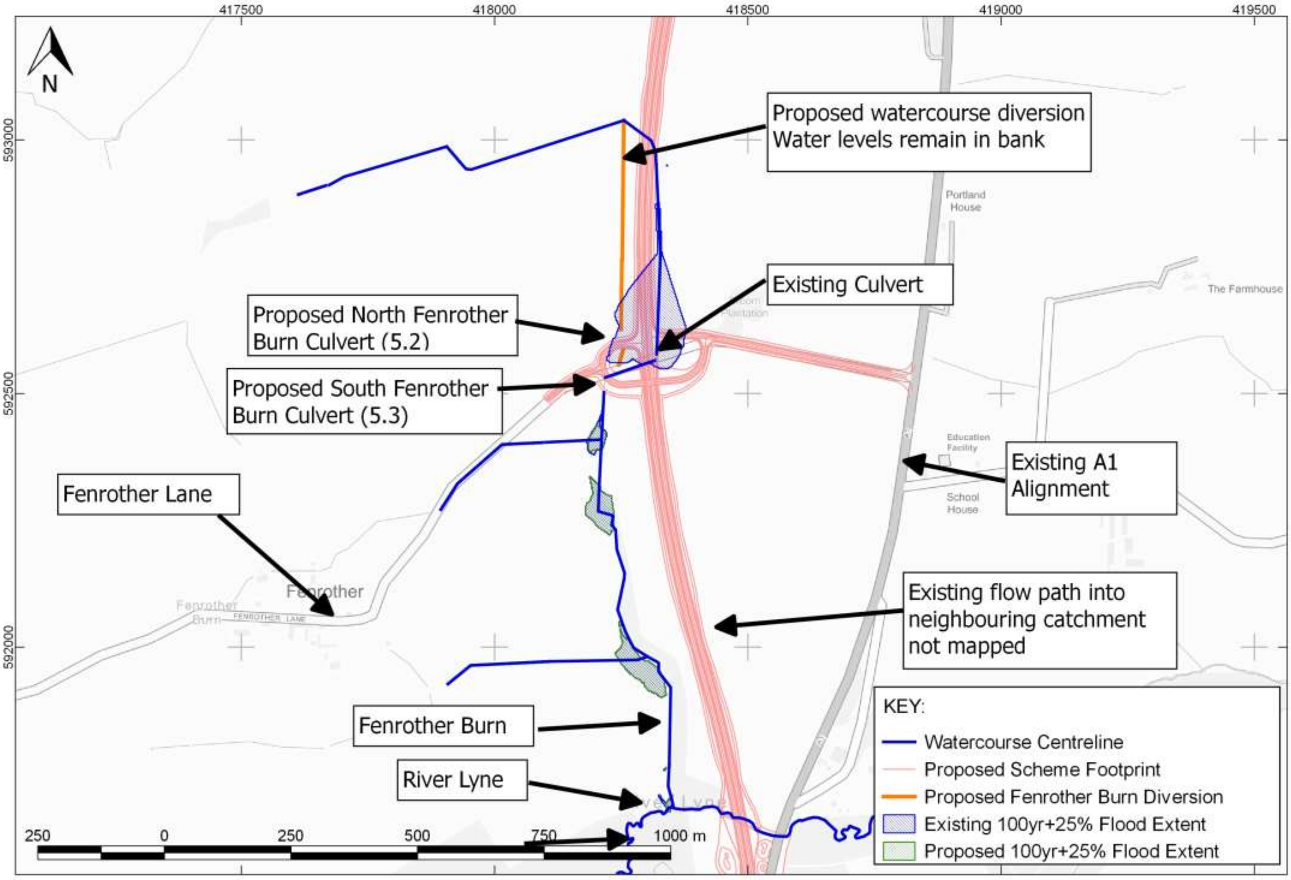

Figure 43 – Flood Extents in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

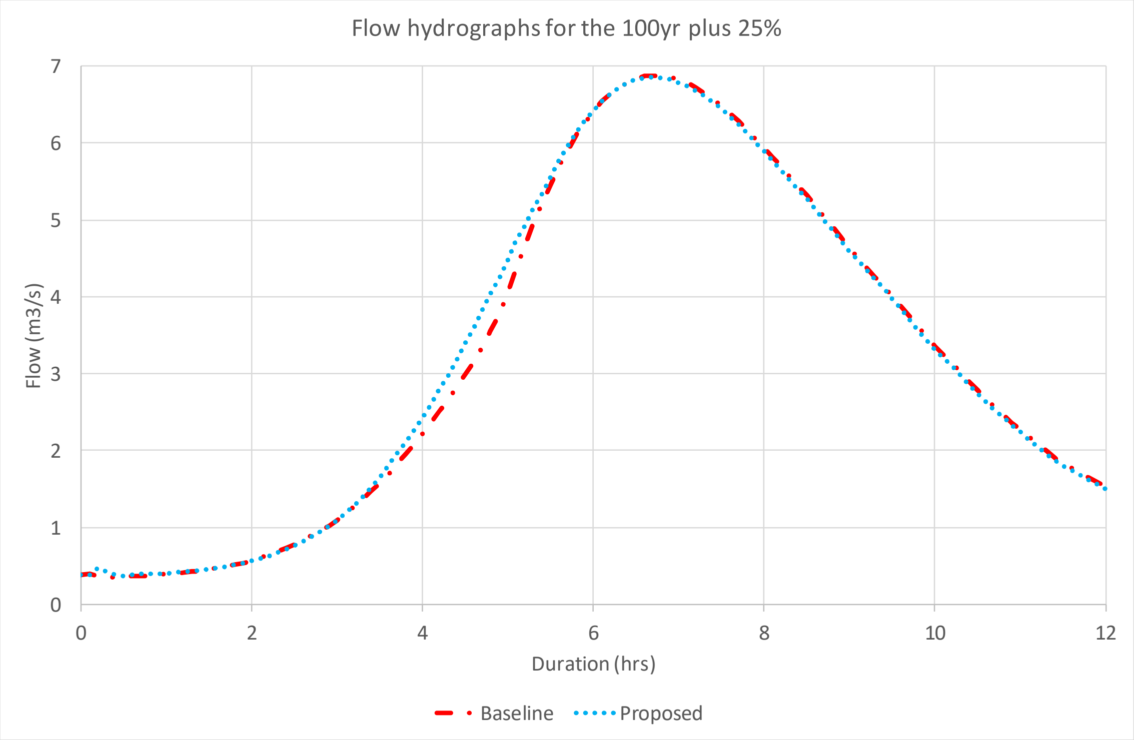

Figure 44 – Pass Forward Flows in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

5.6.10. **Figure 43** demonstrates that Part A would result in a decrease in water levels upstream of the junction as the increased capacity of the channel and culverts significantly reduces out of bank flooding. There is no discernable change in water levels downstream and this is reflected in **Figure 44**, which shows a marginal decrease in downstream flows at the peak of the event.

5.6.11. **Figure 44** also shows an increase in flows on the rising limb of the event. This is a result of the reduced length of the channel and the improved conveyance capacity of the culverts in the Scheme design of Part A. The overall effect of the improved conveyance through this reach is reduced peak flows as the flood response of the larger catchment draining from the west is slower than the catchment draining to the junction and the proposals increase this response difference. The overall effect of Part A in this area, when combining the proposals on Fenrother Burn and the River Lyne, is discussed in **Section 5.5**.

5.7 EARSDON BURN

OVERVIEW OF PART A REQUIREMENTS

5.7.1. Part A is located approximately 250 m to the west of the existing A1 alignment. The Earsdon Burn catchment to the A1 crossing is approximately 4 km^2 and is rural. There are several properties located at Causey Park Bridge, served by a small diversion off the A1 that sits between the existing alignment of the A1 and Part A.

5.7.2. An overview of Part A in relation to Earsdon Burn is provided in **Figure 45**.

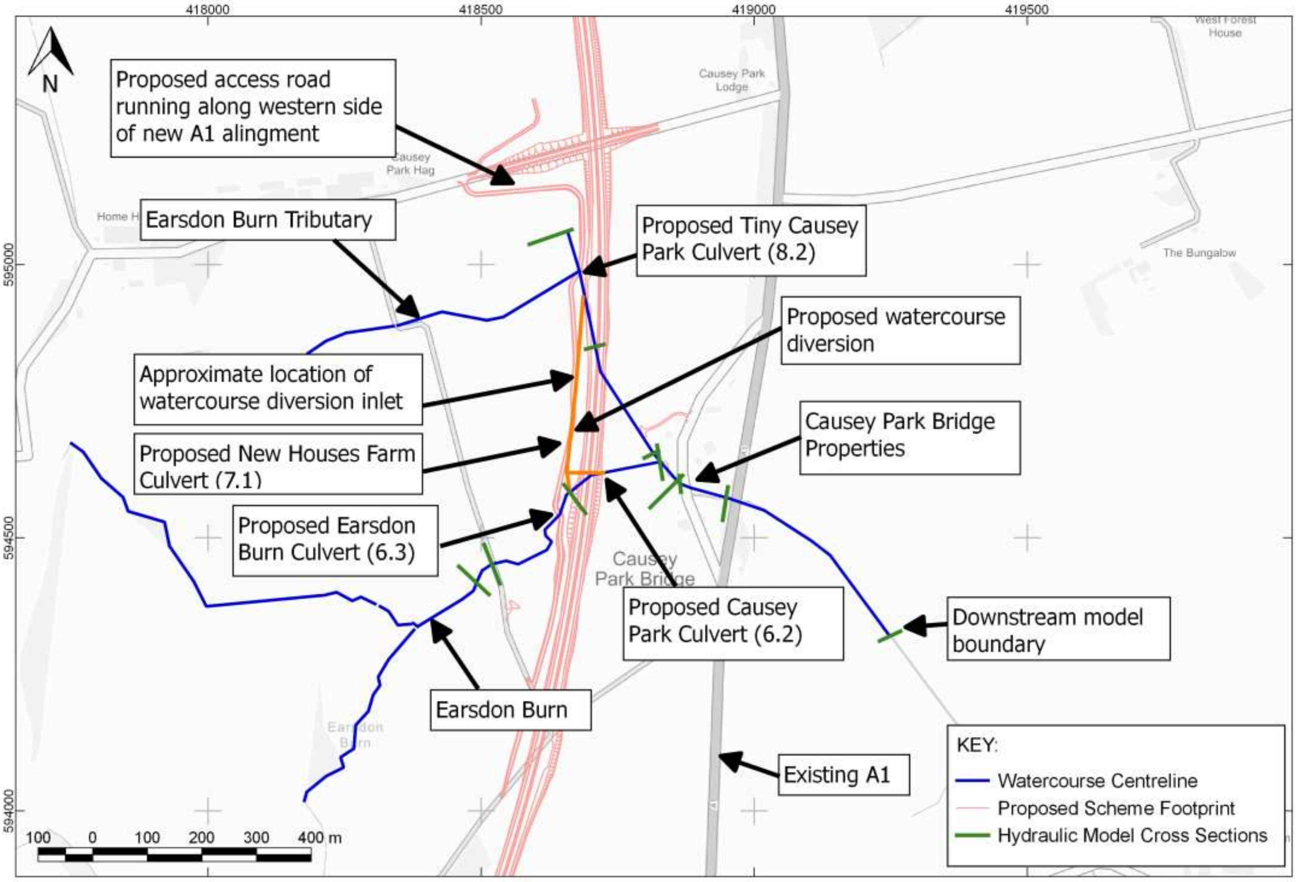

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Figure 45 – Overview of Proposals in Relation to Earsdon Burn

5.7.3. Part A cuts diagonally across both an unnamed tributary of Earsdon Burn and Earsdon Burn itself and consists of both the new A1 alignment and an access road running along the west of the A1. To reduce the number of proposed culverts, the unnamed tributary is to be diverted along the western side of Part A to a new confluence with Earsdon Burn so that a single culvert (New Houses Farm Culvert (7.1)) perpendicular to the proposed A1 alignment can be constructed. The culvert crossing would tie in with the existing alignment of Earsdon Burn at its downstream face.

5.7.4. The new channel for the tributary would be 320 m in length and result in an overall increase in channel length to the existing confluence with Earsdon Burn of 150 m. Similarly, the Earsdon Burn would be diverted northwards for 30 m to the new confluence. The diversion would result in an increase in channel length on Earsdon Burn of 30 m.

5.7.5. The diversion of the unnamed tributary would cut through an area of raised ground for approximately 140 m upstream of the new confluence. The ground levels in this location are such that an open channel here would need to be of the order of 30 m wide if constructed with 1 in 3 side slopes. This is considered to be excessive design for a channel with a QMED (2 year flood event) of 0.35 m^3/s. The watercourse would therefore be culverted for this length.

5.7.6. There would be two culverts constructed beneath the access road. One would be on the unnamed tributary upstream of the diversion (Tiny Cause Park Culvert (8.2)) and one on the Earsdon Burn again upstream of the diversion of the watercourse (Earsdon Burn Culvert (6.3)). A culvert would convey Earsdon Burn downstream of the confluence with the unnamed tributary underneath the new A1 alignment (Causey park Culvert (6.2)).

PART A PROPOSALS

5.7.7. 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 new culverts as set out in **Table 5-7**. **Table 5-7** also details the dimensions of the existing A1 culvert for comparison.

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

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing A1 Culvert	32	Box	3.0	1.86
Proposed Causey Park Culvert (6.2)	36.2	Box	3.0	2.1
Proposed Earsdon Burn Culvert (6.3)	11	Box	3.0	2.1
Proposed New Houses Farm Culvert (7.1)	148	Circular	1.6	1.6
Proposed Little Causey Park Culvert (7.2)	9	Circular	1.6	1.6

DESIGN OUTCOMES

5.7.8. **Table 5-8** provides details of the freeboard associated with each structure for a range of flood events. Earsdon Burn is an ordinary watercourse in the vicinity of the A1 and as such a design freeboard of 300 mm is preferred for the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref 10.1.10**). The 1000 year event is larger than the 100 year + 50 % climate change event so the 1000 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 Earsdon Burn culverts and by 67 % for the tributary culverts reflecting the different sizes of these structures and hence the likelihood of blockage.

Table 5-8 - Design Freeboard for Earsdon Burn Structures

Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage
Existing A1 Culvert	4.04	1.11	0.24	-0.06	-	1.08	0.36	0.13	-
Proposed Causey Park Culvert (6.2)	1.95	1.23	0.51	0.21	0.24	1.21	0.57	0.43	-
Proposed Earsdon Burn Culvert (6.3)	0.60	1.21	0.57	0.30	0.30	1.27	0.75	0.5	-
Proposed New Houses Farm Culvert (7.1)	-	0.85	0.11	-0.24	-0.11	0.82	0.10	-0.20	-
Proposed Little Causey Park Culvert (7.2)	0.6	1.1	0.47	0.14	0.18	1.1	0.46	0.13	-

5.7.9. **Table 5-8** also shows that a freeboard of 300 mm is achieved in the 100 year + 25 % climate change for the Earsdon Burn culverts and the access road culvert on the unnamed tributary but is not achieved for the longer unnamed tributary realignment culvert. Similarly, the unnamed tributary realignment culvert is the only culvert shown to overtop in the blockage assessment.

5.7.10. The crest level of the Part A carriageway, which runs parallel to the unnamed tributary, is approximately 2.0 m above the soffit of the proposed tributary realignment culvert indicating

Part A would not be overtopped in either the extreme 1000 year event or allowing for blockage of this structure.

PREDICTED FLOOD RISK IMPACTS

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

5.7.12. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of Part A, there are however several properties immediately downstream. **Figure 46** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part A. **Figure 47** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 350 m downstream of the existing A1 alignment.

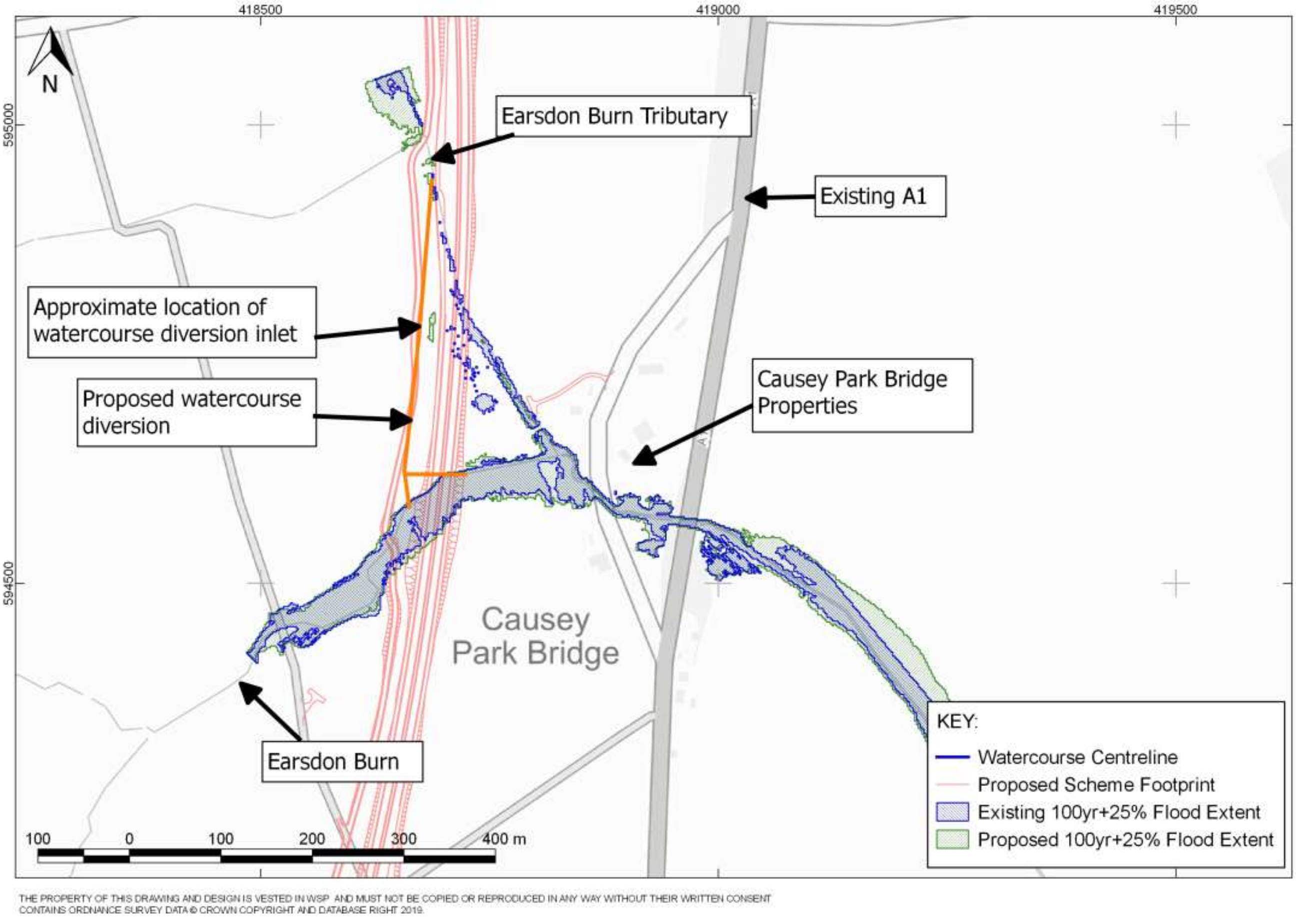

Figure 46 – Flood Extents in the Existing and Proposed Design for the 100 year + 25 % Climate Change Event

5.7.13. **Figure 46** shows an increase in flood risk upstream of Part A as a result of relocation of the tributary confluence further upstream and the presence of the new culvert. The same factors result in a marginal decrease in flows and flood risk downstream, as the longer watercourse length attenuates flows.

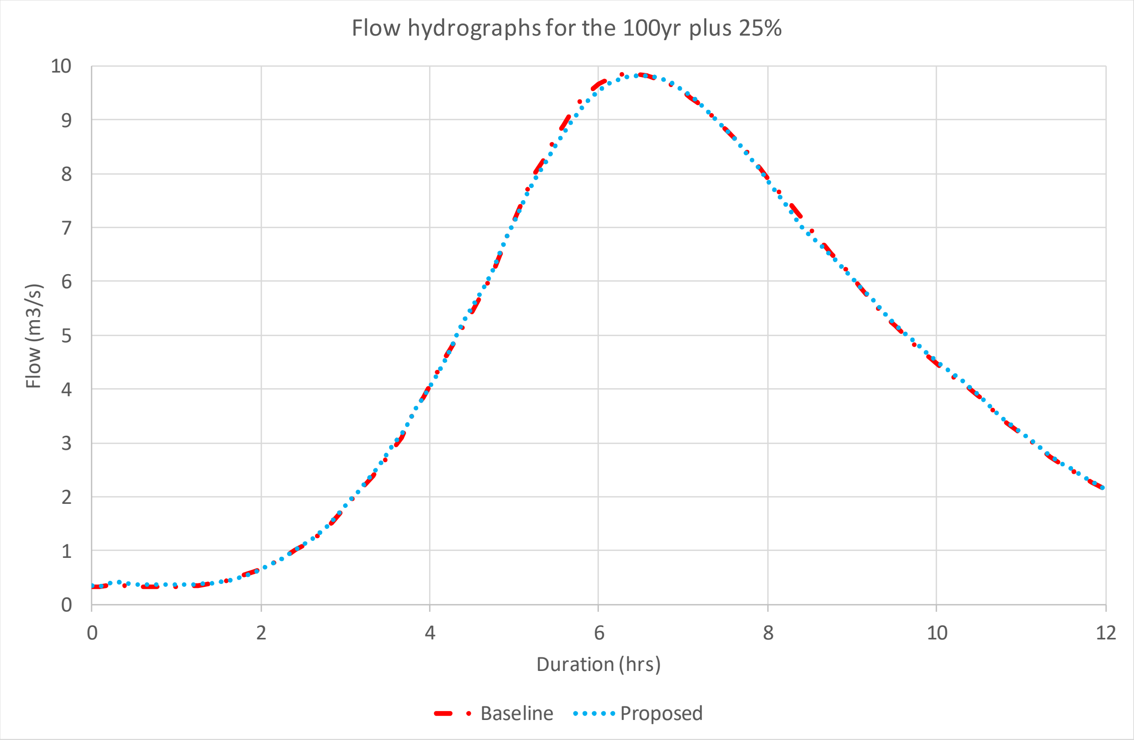

Figure 47 – Pass Forward Flows in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

5.8 LONGDIKE BURN

OVERVIEW OF PART A REQUIREMENTS

5.8.1. Longdike Burn drains a predominantly rural catchment of approximately 23 km^2 to Part A. There are no flood risk receptors upstream of Part A but immediately downstream are a number of holiday cabins on the left and right bank of the watercourse.

5.8.2. An overview of Part A in relation to the Longdike Burn is provided in **Figure 48**.

5.8.3. The proposed works at Longdike Burn consist of an extension of the existing Bockenfield Culvert (12) westwards on the inlet side by 34.4 m with the remainder of the dimensions kept constant.

5.8.4. Whilst Part A shows some changes to East Road, this consists of the construction of a wingwall on the downstream face and raising the highway only and there are no proposals to change the culvert (Burgham Culvert (10.1)) in this location.

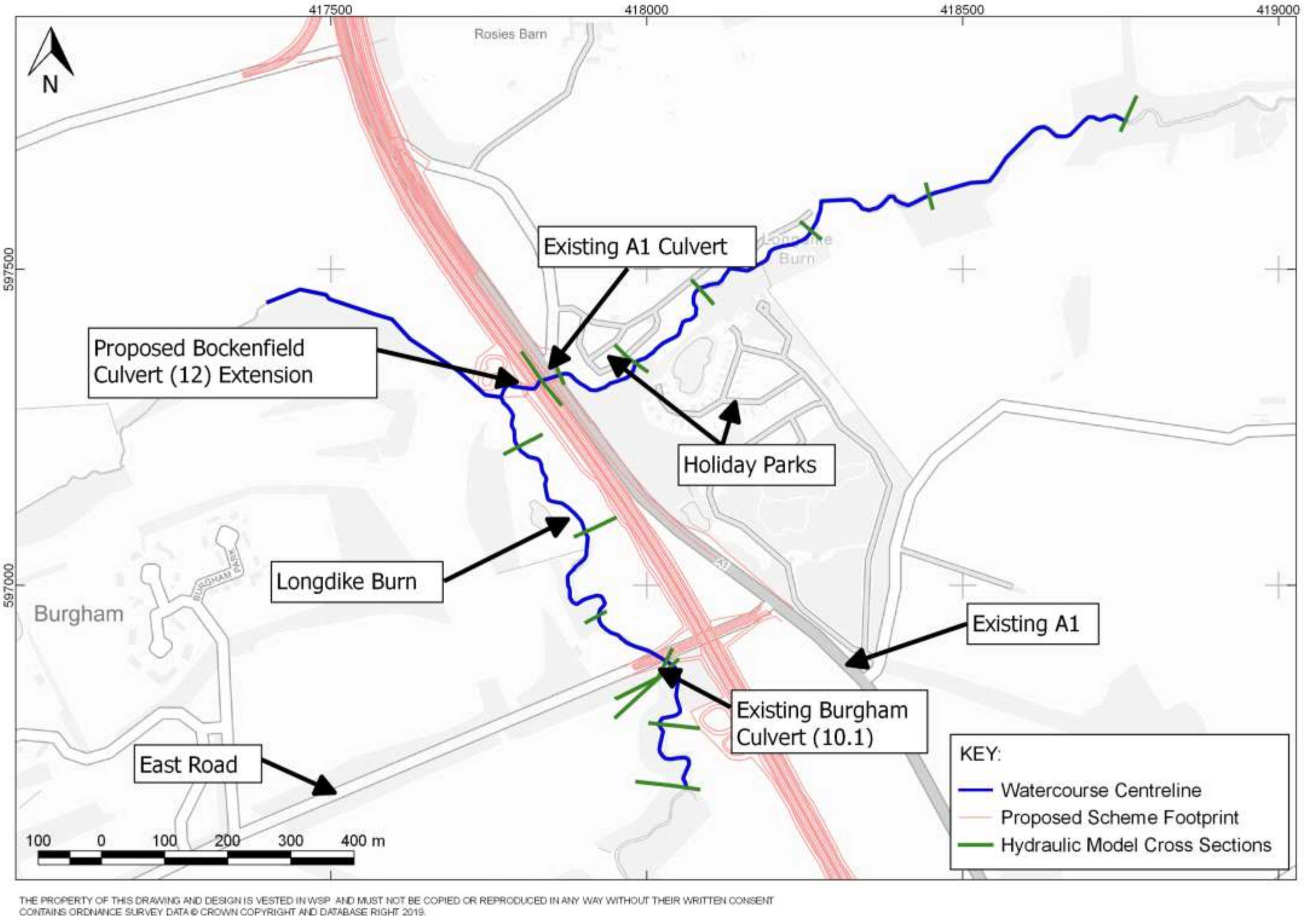

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Figure 48 – Overview of Proposals in Relation to Longdike Burn

PART A PROPOSALS

5.8.5. 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 new culvert as set out in **Table 5-9. Table 5-9** also details the dimensions of the existing A1 and East Road Culverts for comparison.

Table 5-9 - Existing and Proposed Dimensions of Longdike Burn Structures

Structure	Length (m)	Shape	Width (m)	Height (m)
Existing Burgham Culvert (10.1)	30	Ovoid	3.44	4.82
Existing Bockenfield Culvert (12)	30	Sprung Arch	6.1	2.41 (1.44 Arch)

Structure	Length (m)	Shape	Width (m)	Height (m)
Extended Bockenfield Culvert (12) for Part A	64.4 (total length)	Sprung Arch	6.1	2.49 (1.44 Arch)

DESIGN OUTCOMES

5.8.6. **Table 5-10** provides details of the freeboard associated with each structure for a range of flood events. Longdike Burn is classified as a main river and as such a design freeboard of 600 mm is preferred in the 100 year + 25 % climate change event in accordance with DMRB (HD 107/04) (**Ref 10.1.10**). The 1000 year event is larger than the 100 year + 50 % climate change event so 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 % reflecting the size of the proposed structures and hence the likelihood of blockage.

Table 5-10 - Design Freeboard for Longdike Burn Structures

Structure	Carriageway Freeboard above Inlet Soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage
Existing Burgham Culvert (10.1)	0.7	1.97	0.38	0.17	-	2.19	0.94	0.69	-
Existing Bockenfield Culvert (12)	3.53	0.88	-0.57	-1.42	-	1.02	0.05	-0.22	-
Extended Bockenfield Culvert (12) for Part A	3.53	0.84	-0.78	-1.76	-1.80	1.02	0.06	-0.21	-

5.8.7. **Table 5-10** shows that maintaining and extending the existing culvert would result in limited changes to the existing freeboard of the structure in the 100 year + 25 % climate change event.

5.8.8. The crest level of the carriageway is more than 2.5 m above the soffit of the proposed culvert indicating Part A would not be overtopped in either the extreme 1000 year event or allowing for blockage of the structure.

PREDICTED FLOOD RISK IMPACTS

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

5.8.10. As detailed above the upstream catchment is rural and there are no receptors of concern upstream of the A1. **Figure 49** presents the mapped flood risk extents for the 100 year + 25 % climate change event in the existing situation and following the construction of Part A. **Figure 50** compares the pass forward flows associated with the same event and scenarios at the downstream limit of the hydraulic model, located approximately 900 m downstream of Part A.

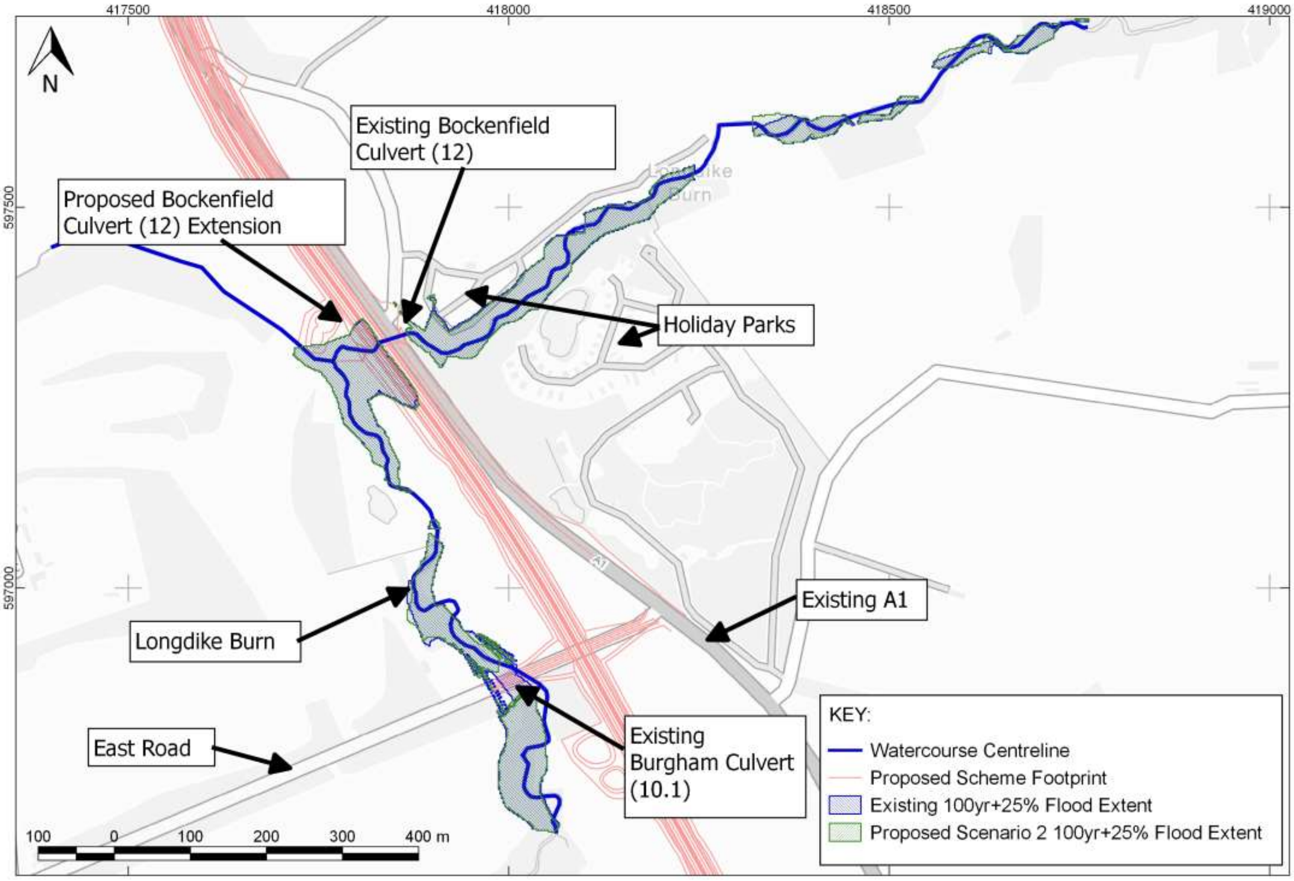

Figure 49 – Flood Extents in the Existing and Proposed Design for the 100 year + 25 % Climate Change Event

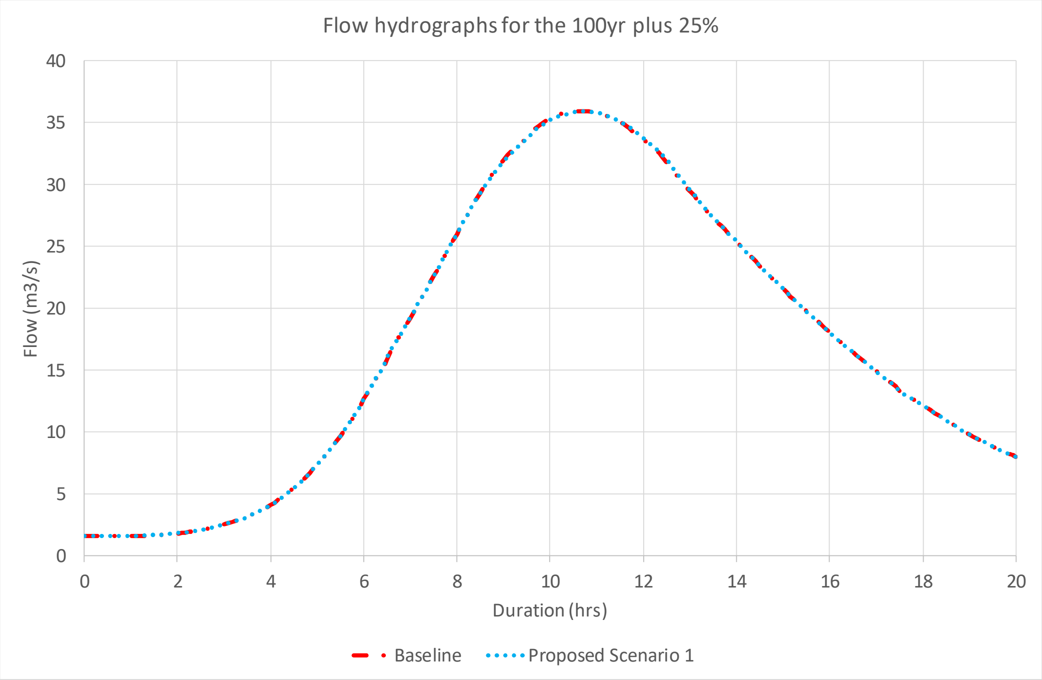

Figure 50 – Pass Forward Flows in the Existing and Proposed Scenarios for the 100 year + 25 % Climate Change Event

5.8.11. **Figure 49** demonstrates that the design would result in a marginal increase in water levels upstream of the A1; this increase is not observed at Burgham Culvert (10.1). As would be expected downstream flows are marginally lower.

5.9 CULVERT CROSSINGS

5.9.1. Part A crosses several minor watercourses and surface water flow paths which are not discussed in the Sections above. These are summarised in the table below, which includes information on whether the culverts are existing or new.

5.9.2. Details on the culvert modelling, including the culvert hydrology analysis, can be found in **Appendix B: Culvert Master Analysis**.

Table 5-11 - Summary of the Minor Watercourses, Drainage Ditches and Identified Surface Water Flow Paths crossed by Part A

Structure Name or Description	Scheme Description (Part A)	Structure Dimensions	Freeboard in the 100 year +25 % cc event	Freeboard in the 1000 year event	Comments
Shieldhill Culvert (1A) Existing	Part A is widening the existing A1 in this location and the proposal would replace the existing culvert	Shape: Arch Width: 1.31 m Height: 1.01 m Length: 30.39 m Inlet Soffit to Crest: 2.75 m	Inlet: 0.07 m Outlet: 0.71 m	Inlet: -0.11 m Outlet: 0.68 m	Existing structure is sufficiently large to convey the 100 year + 25 % climate change design flow and is assumed to provide no attenuation benefits downstream. The highway crest is not overtopped in the 1000 year event.
Shieldhill Culvert (1A) Proposed		Shape: Circular Diameter: 1.2 m Length: 43.4 m Inlet Soffit to Crest: 1.96 m	Inlet: 0.48 m Outlet: 0.79 m	Inlet: 0.15 m Outlet: 0.73 m	Proposed structure achieves 300 mm freeboard in the 100 year + 25 % climate change event. No change to downstream flows resulting from the proposal. The highway crest is not overtopped in the 1000 year event.
Paradise Culvert (3) Existing	Part A is widening the existing A1 in this location and the proposal would replace the existing culvert	Shape: Arch Width: 1.96 m Height: 1.07 m Length: 26.66 m Inlet Soffit to Crest: 2.61 m	Inlet: -1.11 m Outlet: 0.57 m	Inlet: -2.41 m Outlet: -1.66 m	Existing structure is surcharging in the 100 year + 25 % climate change event. A hydraulic assessment of this structure indicates it is currently attenuating peak flows in the 100 year + 25 % climate change event from 4.9 m^3/s to 4.2 m^3/s. The highway crest is not overtopped in the 1000 year event.
Paradise Culvert (3) Proposed		Shape: Circular Diameter: 1.8 m Length: 32.7 m Inlet Soffit to Crest: 1.84 m	Inlet: 0.1 m Outlet: 0.22 m	Inlet: -1.44 m Outlet: -1.15 m	Proposed structure does not achieve 300 mm freeboard in the 100 year + 25 % climate change event but would convey the peak flows. The peak downstream flows in the 100 year + 25 % climate change event are 4.4 m^3/s. This increase in flows is considered acceptable on the basis that the change in flood risk is minimised. The increase in flows is small in relation to the total flows (<5 %) and the nearest receptors are 3 km downstream. A further consideration in this conclusion is that the proposed pipe size is the minimum that would allow mammal passage to be incorporated. A reduction in pipe size could remove the increase in flows at the loss of mammal passage provision. The highway crest is not overtopped in the 1000 year event.
South Longdike Culvert (9.1) Proposed	Construction of a new culvert where open channel is currently	Shape: Circular Diameter: 1.2 m	Inlet: 0.88 m Outlet: 0.80 m	Inlet: 0.78 m Outlet: 0.80 m	Proposed structure achieves 300 mm freeboard in the 100 year + 25 % climate change event. No change to downstream flows resulting from the proposal. The highway crest is not overtopped in the 1000 year event.

Structure Name or Description	Scheme Description (Part A)	Structure Dimensions	Freeboard in the 100 year +25 % cc event	Freeboard in the 1000 year event	Comments
	present (offline section)	Length: 39 m Inlet Soffit to Crest: 1.37 m			
Blackwood Hall (13.1) Existing	Part A is widening the existing A1 in this location and the proposal would replace the existing culvert	Shape: Circular Diameter: 0.3 m Length: Unknown Inlet Soffit to Crest: Unknown	Assumed submerged	Assumed submerged	Unable to locate inlet or outlet but pipe observed to be 300 mm from manhole. Existing structure assumed to be submerged.
Blackwood Hall (13.1) Proposed		Shape: Circular x 3 Diameter: 0.45 m Length: 61.59 m Inlet Soffit to Crest: 1.59 m	Inlet: -0.79 m Outlet: 0.11 m	Inlet: -1.51 m Outlet: 0.06 m	Proposal includes new channel along the east side of Part A discharging to an unnamed watercourse north of Eshott Airfield upstream. The proposed structure does not achieve 300 mm freeboard in the 100 year + 25 % climate change event. The proposed approach has been adopted in this instance to minimise the risk of overtopping of the highway and additional flood risk from increasing pass forward flows. In attempting to achieve this balance, freeboard requirements have not been achieved. Water levels are close to the highway crest in the 1000 year event.
Glenshotton Culvert (14)Existing	Part A is widening the existing A1 in this location and the proposal would extend the existing culvert	Shape: Circular Diameter: 1.35 m Length: 24.3 m Inlet Soffit to Crest: 0.35 m	Inlet: -0.31 m Outlet: 0.71 m	Inlet: -0.77 m Outlet: 0.47 m	Existing structure would surcharge in the 100 year + 25 % climate change event. Predicted headwater is within reasonable bounds and does not exceed the carriageway crest level. It is expected this structure would convey the 100 year + 25 % climate change design flow. Provides no attenuation benefits downstream. The highway crest is likely to be overtopped in the 1000 year event.
Glenshotton Culvert (14) Proposed		Shape: Circular Diameter: 1.32 m Total Length: 47.6 m consisting of 24.3 m existing and 23.3 m new Inlet Soffit to Crest: 1.16 m	Inlet: -0.47 m Outlet: 0.63 m	Inlet: -1.09 m Outlet: 0.39 m	Proposed structure would surcharge in the 100 year + 25 % climate change event. Predicted headwater is greater than the existing structure and pass forward flows would be reduced compared to the existing. The nearest receptors are 4 m above the channel and so the increase in upstream water levels would have no impact. Water levels are close to the highway crest in the 1000 year event.
Parkwood Culvert (16) Existing	Part A is widening the existing A1 in this	Shape: Circular Diameter: 0.9 m	Inlet: -0.6 m	Inlet: -1.24 m Outlet: 0.07 m	Existing structure would surcharge in the 100 year + 25 % climate change event. Predicted headwater is within reasonable bounds and does not exceed the

Structure Name or Description	Scheme Description (Part A)	Structure Dimensions	Freeboard in the 100 year +25 % cc event	Freeboard in the 1000 year event	Comments
	location and the proposal would extend the existing culvert	Length: 125 m Inlet Soffit to Crest: 13.1 m	Outlet: 0.50 m		carriageway crest level. It is expected this structure would convey the 100 year + 25 % climate change design flow. Provides no attenuation benefits downstream. The highway crest is not overtopped in the 1000 year event.
Parkwood Culvert (16) Proposed		Shape: Circular Diameter: 0.9 m Total Length: 145 m consisting of 125 m existing and 20 m new Inlet Soffit to Crest: 13.3 m	Inlet: -0.6 m Outlet: 0.51 m	Inlet: -1.3 m Outlet: 0.07 m	Proposed structure would surcharge in the 100 year + 25 % climate change event. Predicted headwater is equivalent to the existing structure and flood response would be consistent with the existing. The highway crest is not overtopped in the 1000 year event.

5.10 INCREASE IN SURFACE WATER RUNOFF RATE AND VOLUME

5.10.1. A detailed description of the surface water drainage strategy is provided in **Appendix 10.5: Drainage Strategy Report, Volume 7** of this ES (**Application Document Reference: TR010041/APP/6.7**). The surface water drainage system has been designed according to DMRB (HD 107/04) (**Ref 10.1.10**) and taking into account the low points in elevation along Part A.

5.10.2. The surface water drainage strategy has been designed using a 20 % climate change allowance as agreed through consultation with the LLFA. Sensitivity testing for the 40 % climate change allowance was also undertaken to understand which is detailed in **Appendix 10.5: Drainage Strategy Report, Volume 7** of this ES (**Application Document Reference: TR010041/APP/6.7**).

5.10.3. The surface water drainage strategy is summarised below:

a. Runoff from Part A (online and offline sections) would be discharged into the existing watercourses via storage swales/detention basins/tanks, where required.
b. Drainage discharge from highways remaining part of the local road network is kept separate from discharge associated with Part A, as agreed with NCC. This strategy includes separate detention basins or SUDS features where appropriate. However, controlled runoff from both trunk and non-trunk detention basins/features would discharge to a common outfall to minimise the overall Order Limits of Part A.
c. Maintenance of trunk and local drainage assets including the offline section of the A1 would be subject to a 'Memorandum of Understanding' between the Applicant and NCC.
d. Roads/tracks which are not to be incorporated as access roads to the online section of Part A, are assumed to be abandoned/truncated, and would continue to drain as existing. All existing watercourses crossing the proposed route, to which these roads/tracks may drain, would be maintained using culverts or other means.
e. Locations of detention features have been agreed with NCC, and Environment Agency.
f. Allowable runoff rates are restricted to the existing greenfield runoff values for the equivalent storm event.
g. 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).
h. Attenuation controls would be provided for the 1 in 1, 30 and 100 year events plus climate change.
i. Where detention basins, tanks or storage swales are used for attenuation these are located outside of Environment Agency Flood Zone 2 and 3 areas.
j. Detention basins would be lined, therefore, there is no impacts to groundwater ingress that might increase flood risk. Upheave would be considered during the detailed design stage.
k. Online controls would be provided to restrict discharges to allowable values.

l. Any new local access tracks, bridleways and private means of access (PMAs) are designed to drain to local land drains and watercourses.
m. Runoff from the running lanes and hardstrips would follow the road camber to both channels, and the central reservation where there is a crossfall.
n. Runoff to the central reservation would be to concrete V-channels.
o. 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.
p. 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.
q. As there is a requirement 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. The exception to this are detention basins DB15 and DB15a where there is a requirement to keep them as dry as possible.

5.11 RESIDUAL FLOOD RISK

5.11.1. The residual flood risk associated with Part A watercourse crossings, culverts and identified surface water flow paths (detailed above in **Sections 5.2** to **5.9**) has been investigated through the following:

a. The residual risks associated with an increase in flow has been assessed using the 1000 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).

5.11.2. During a 1000 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. Review of the Environment Agency's Flood Map for Planning (Rivers and Sea) (**Ref 10.1.5**) indicates that the majority of Part A's alignment is located in the low-risk Flood Zone 1. However, Part A does include sections located in the medium risk Flood Zone 2 and the high-risk Flood Zone 3.

6.1.2. Part A crosses ten watercourses and associated tributaries (listed from south to north): Cotting Burn; Shieldhill Burn; Floodgate Burn; River Lyne; Fenrother Burn; Earsdon Burn; Longdike Burn; Unnamed tributary of Thirston Burn; River Coquet; and Bradley Brook.

6.1.3. Detailed 1D hydraulic modelling has been undertaken for the Cotting Burn, River Lyne, Fenrother Burn, Earsdon Burn and Longdike 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. The identified fluvial floodplains associated with: The River Coquet, Longdike Burn (and the Poxtondean Burn that discharges into the Longdike Burn), Earsdon Burn, the River Lyne and Floodgate Burn are largely contained within the watercourse channels. As a result, any loss of floodplain has been accounted for within the hydraulic modelling and design of the watercourse crossings.

6.1.4. Detailed hydraulic analysis of the River Coquet has not been undertaken given the limited effect of Part A on flows within the River Coquet. Furthermore, the temporary works for this structure would use a kingpost solution to maintain the levels of the bridge deck as it is pushed across the river rather than a temporary pier in the river channel. This would remove the need for any temporary works within the channel for all aspects apart from the single pier that is located on the south bank of the river below the expected 100 year flood level.

6.1.5. A review of the Environment Agency's Flood Risk from Surface Water map (**Ref 10.1.7**) indicates that sections of Part A are at high, medium and low risk of flooding from surface water sources. Existing surface water flow paths have been incorporated into Part A. The surface water drainage system has been designed according to DMRB (HD 107/04) (**Ref 10.1.10**) and taking into account the low points in elevation along Part A.

6.1.6. Due to the relatively low permeability of the bedrock and superficial deposits underlying Part A, groundwater levels are unlikely to fluctuate significantly. Therefore, groundwater flooding is unlikely to occur. A small area of sand and gravel, associated with Earsdon Burn and Longdike Burn, which have a higher permeability, may experience groundwater flooding. However, this should not affect Part A.

6.1.7. 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.

7 REFERENCES

Ref 10.1.1: Ministry of Housing, Communities & Local Government (2019) *National Planning Policy Framework.* February 2019. London: Her Majesty's Stationary Office (HMSO).

Ref 10.1.2: Ministry of Housing, Communities & Local Government (2014) *Planning Practice Guidance: Flood Risk and Coastal Change*. [Available online] https://www.gov.uk/guidance/flood-risk-and-coastal-change [Accessed July 2018].

Ref 10.1.3: Department for Transport (2014) *National Policy Statement for National Networks*. London: HMSO.

Ref 10.1.4: Highways Agency (2009) *Design Manual for Roads and Bridges (DMRB) Volume 11, Section 3, Part 10 (HD 45/09)*.

Ref 10.1.5: Environment Agency (2019) *Flood Map for Planning*. [Available online] https://flood-map-for-planning.service.gov.uk/ [Accessed July 2018].

Ref 10.1.6: Environment Agency (2019) *Preparing a Flood Risk Assessment: Standing Advice*. [Available online] https://www.gov.uk/guidance/flood-risk-assessment-standing-advice [Accessed July 2018].

Ref 10.1.7: Environment Agency (2019) *Long Term Flood Risk Map*. [Available online] https://flood-warning-information.service.gov.uk/long-term-flood-risk/map [Accessed July 2018].

Ref 10.1.8: *Multi-Agency Geographic Information for the Countryside (MAGIC)*. [Available online] https://magic.defra.gov.uk/MagicMap.aspx [Accessed July 2018].

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Appendix A

HYDRAULIC MODELLING ANALYSIS

Project	A1 - Northumberland
Job Number	70044136
Location	Morpeth, Northumberland, England (418223, 588448)
Watercourse(s)	Cotting Burn

1. Objectives/Areas of interest

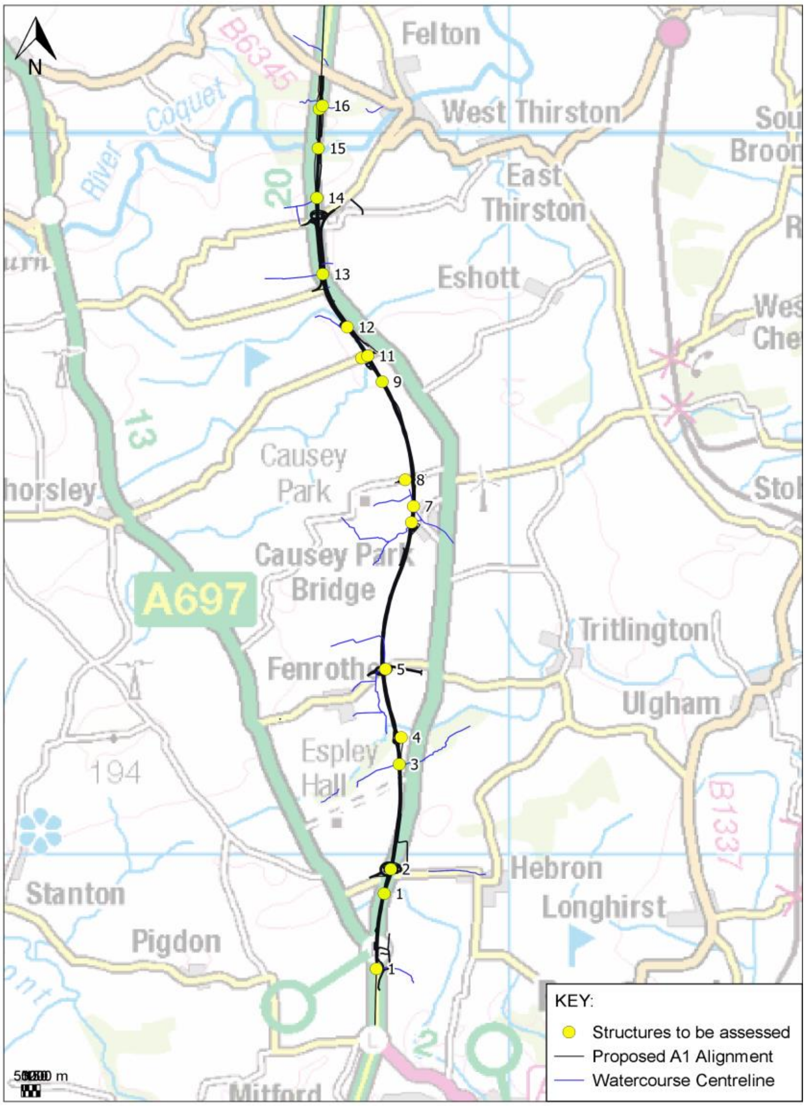


Figure 1: Location of structures in overall Scheme

The Scheme includes approximately 6.6 km of online widening and approximately 6 km of new offline highway. The existing carriageway would be widened on its current line up to Priest's Bridge, from where the proposed offline section of the Scheme would move west off the current road and pass west of Tindale Hill and Causey Park Bridge. Just north of Burgham Park, it would re-join the line of the existing carriageway and widening would continue along the existing road northwards, until it meets the existing dual carriageway north of Felton.

Figure 1 above shows the location of various structures along the Scheme. There are 16 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.

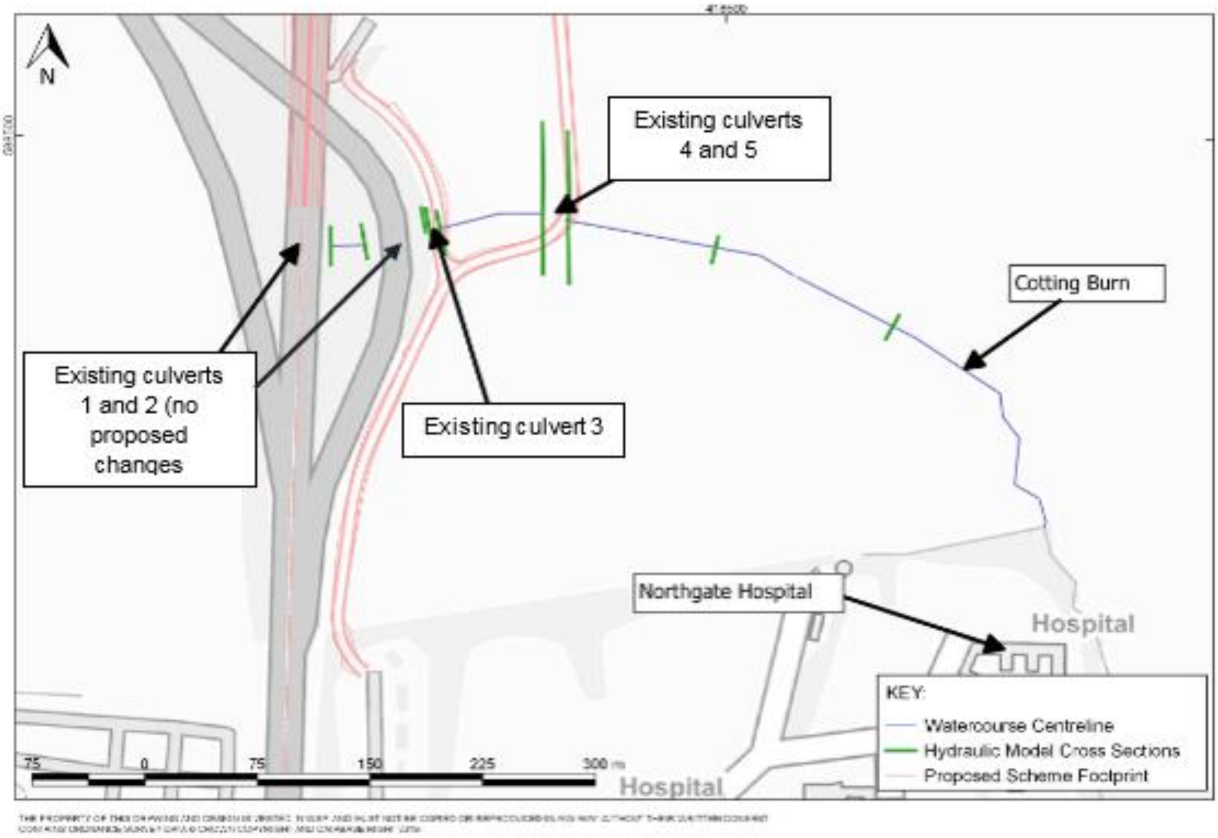


Figure 2: Location of structures on Cotting Burn

This report relates to the proposed works on the Cotting Burn watercourse. There are three existing structures within the reach that will be affected by the Scheme. All three are culverts located to the east of A1.

The surveyors were unable to locate the inlet to the A1 culvert on the west side of the A1 or the watercourse upstream of this point. For this reason the assessment has focussed on the Cotting Burn downstream of the A1. There are no proposed works upstream of the A1 and as such it is considered the assessment can progress with the data available. It is recognised this approach will not include the attenuating effects of the A1, however this is mitigated to some extent by the presence of a similar sized culvert immediately downstream beneath the slip road. 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
M2F XS01.xlsx	Topographic Survey	Detailed topographic survey of area around the Cotting 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
Spot level grid -	Source	Topographic survey of proposed A1 corridor between Morpeth and Alnwick collected by Jabobs. Final issue January 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP by using recent topographic survey. The model extent is approximately 0.4km in length. The watercourse is Cotting Burn.

There are three existing structures within the reach. These structures were modelled in 1D domain only 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. LIDAR data is not available for this model. Local ground levels are available from survey collect by Jacobs was available and this data covers the entire extent of the model.

To confirm the agreement between the river section survey and the Jacobs topo 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.

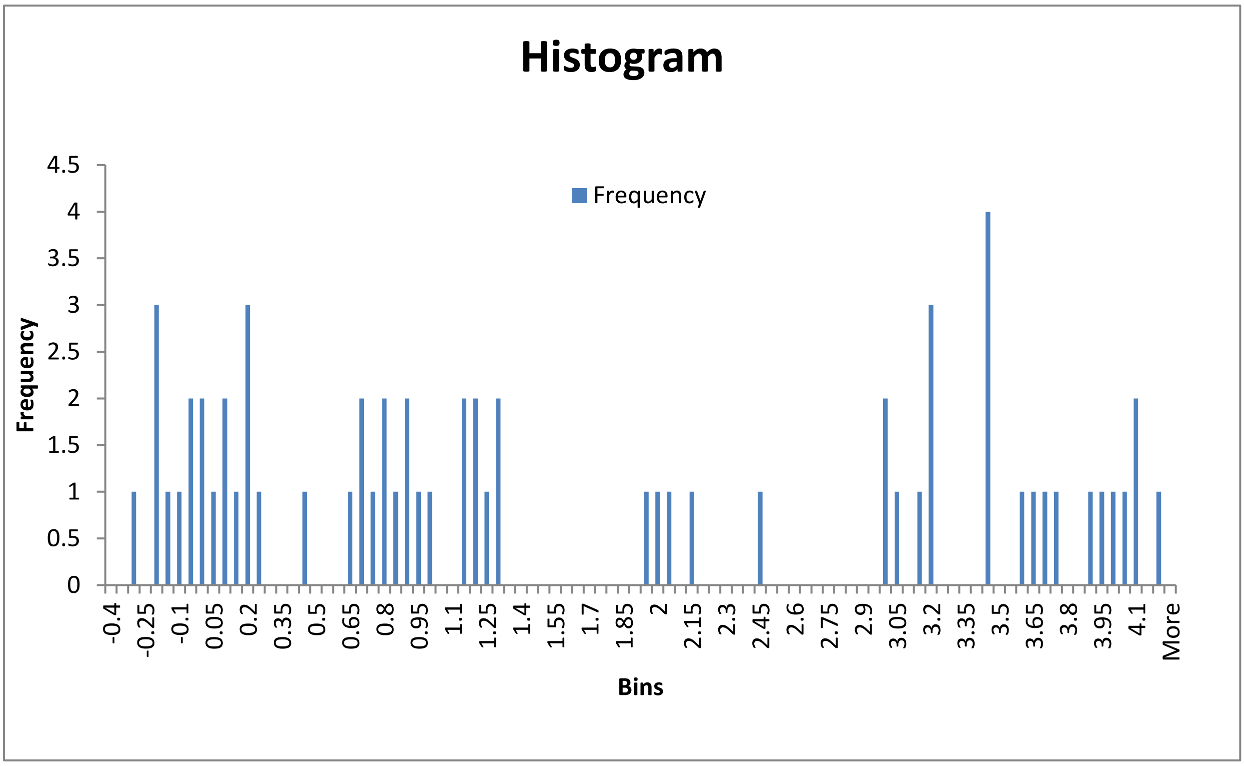


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift in the Jacobs topo of 0.25m was applied. It is however noted that there is a wide variation between the data sets. The low flows in this particular watercourse means that the locations where sections are required to be extended are limited.

5. Model Setup

Model Method	1D
Software	FMP (v4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using Jacobs topo

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 3 locations for the Cotting Burn model. These are on Cotting Burn upstream of the A1 (CB01) and at the downstream of the model. Two flows have been derived for the downstream limit of the model one covering the contributing catchment downstream of the A1 (CB02) and the second covering the whole catchment (CB03). 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%**
CB_01	0.38	0.53	0.63	0.78	0.91	1.00	1.06	1.25	1.82	1.33
CB_02	0.32	0.44	0.53	0.65	0.76	0.83	0.89	1.04	1.51	1.11
CB_03	0.63	0.86	1.03	1.28	1.49	1.63	1.74	2.04	2.97	2.18

Flow hydrographs for the inflows have been developed using the ReFH methodology and nd a design storm duration derived from sensitivity testing of the critical duration in ReFH using catchment descriptors for the CB_03 catchment. These have been applied at the upstream limit of the model (CB_01) and approximately 130m upstream of the downstream boundary which is downstream of the proposed works (CB_02). Hydrographs have been scaled to reflect the peaks detailed in the table above. The values in CB_03 confirm that the two contributing catchments broadly reflect the calculated flows for the downstream catchment.

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

Two new culverts are proposed as part of the works. These are both located to the east of existing A1 culvert and are access roads only for Northgate Farm (418494, 588427). The western culvert will replace Culvert 3 in Figure 2 and the eastern culvert will replace Culverts 4 and 5.

Cross section data for the upstream and downstream faces of these structures was 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 and downstream faces.

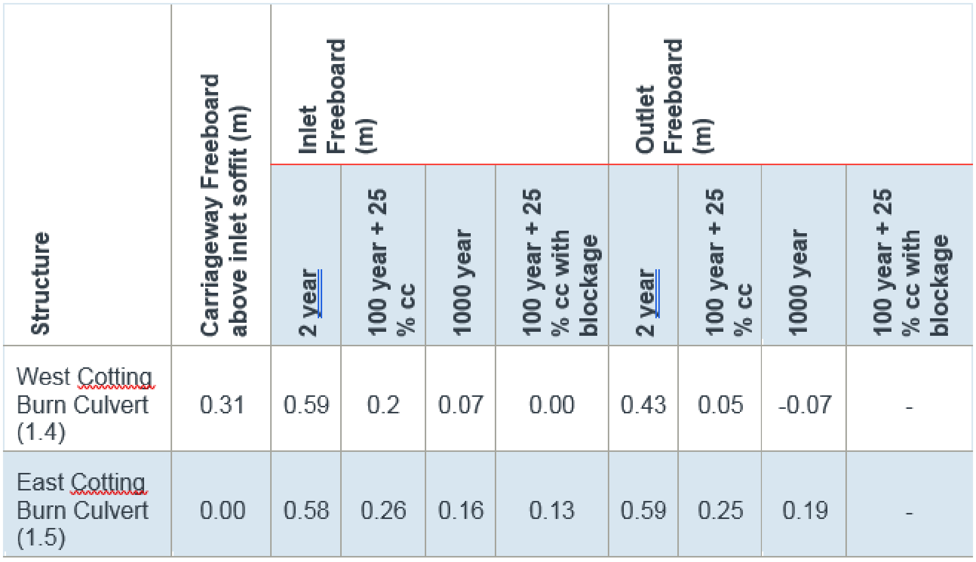
The culverts beneath the new A1 alignment will be box culverts. The location of the inlets of the two culverts have been assumed to remain unchanged, the outfall will be extended downstream in both cases. The western culvert will be 1m in height and 2.7m wide and the eastern culvert will be 1m in height and 3m wide. 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

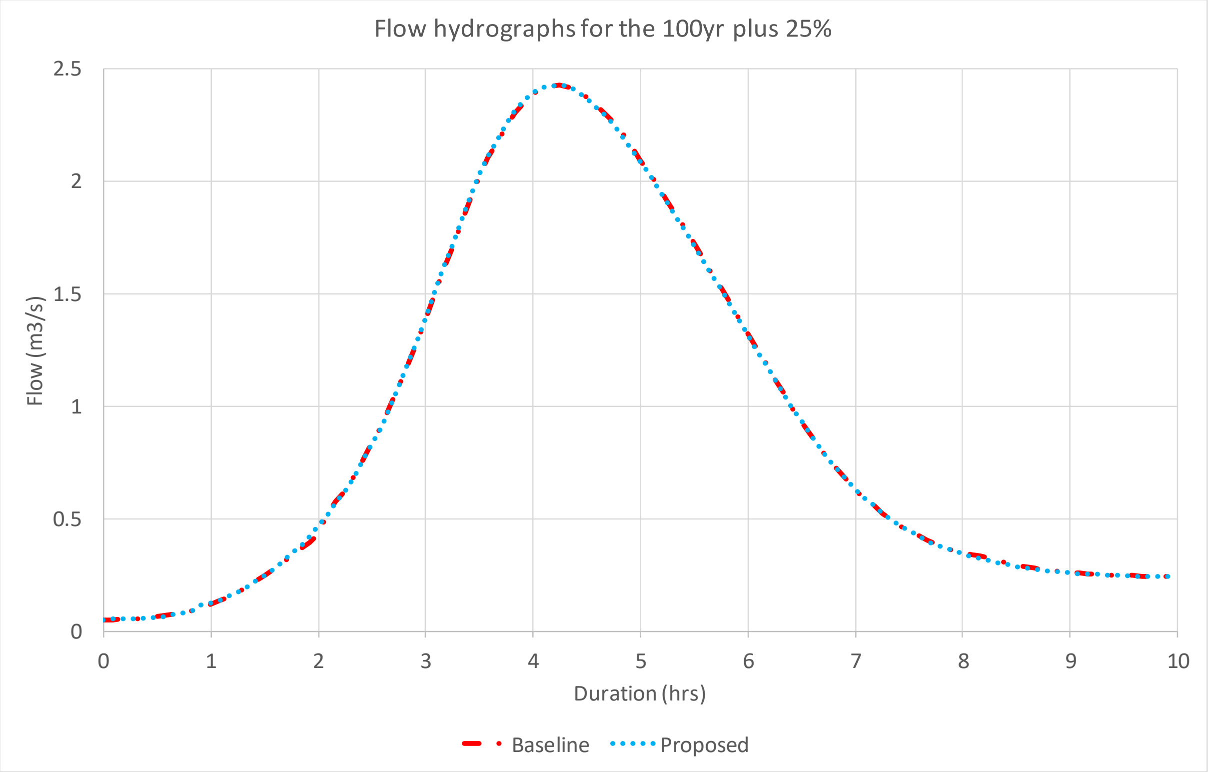
11. Model Results

The following table provides details of the freeboard associated with each structure for a range of flood events. As Cotting Burn is a small ordinary watercourse a design freeboard of 300mm is preferred in the 100yr+25% climate change event in accordance with DMRB. The 1000yr event is larger than the 100yr+50% climate change event and so has been used to assess resilience and risk in an extreme event. Blockage has been assessed by assuming the inlet capacity of the culvert structures is reduced by 67%.



Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage
West Cotting Burn Culvert (1.4)	0.31	0.59	0.2	0.07	0.00	0.43	0.05	-0.07	-
East Cotting Burn Culvert (1.5)	0.00	0.58	0.26	0.16	0.13	0.59	0.25	0.19	-

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. This demonstrates that the proposed design will not have impact on the downstream flows. The larger new structures are providing minimal additional attenuation having prevented the overtopping of the structures that currently occurs.


Flow hydrographs for the 100yr plus 25%
Flow (m3/s)
0
0.5
1
1.5
2
2.5
Duration (hrs)
0
1
2
3
4
5
6
7
8
9
10
Baseline
Proposed

Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Data source	Dimensions	Modelling Approach
1 (XS01_05)	Culvert on A697 crossing the Cotting Burn.	Cross sectional data provided by the surveyor.	The culvert length is 41.7m. The culvert consists of single pipe of diameter 0.9m. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as 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 (XS01_04a)	Culvert crossing the Cotting Burn on East of A1.	Cross sectional data provided by the surveyor.	The culvert length is 6.9m. The culvert consists of single pipe of diameter 0.35m. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as 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.
3 (XS01_03)	Culvert crossing the Cotting Burn on East of A1.	Cross sectional data provided by the surveyor.	The culvert length is 15.6m. The culvert consists of single pipe of diameter 0.45m. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as 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)

West (XS01_04a)	Proposed culvert on the new A1 alignment crossing the Cotting Burn.	Local cross section dimensions derived from the spot level grid and topo survey data provided by the surveyor.	The proposed structure spans 12.8m with 1m height and 2.7m wide, allowing for 0.25m silt in the structure bed. The culvert dimensions were set to maximise freeboard. The preferred freeboard of 300mm in the 100yr+25% event was not achieved, further discussion is provided in the main body of the FRA.	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.

East (XS01_03)	Proposed culvert on the new A1 alignment crossing the Cotting Burn.	Local cross section dimensions derived from the spot level grid and topo survey data provided by the surveyor.	The proposed structure spans 12.8m with 1m height and 2.7m wide, allowing for 0.25m silt in the structure bed. The culvert dimensions were set to maximise freeboard. The preferred freeboard of 300mm in the 100yr+25% event was not achieved, further discussion is provided in the main body of the FRA.	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.

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
XS01_06 to XS01_03 Tortuosity: Low			
Left Bank	0.06	Grass & Trees	
Channel	0.04	Gravel upto 100mm	
Right Bank	0.06	Grass & Trees	

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS01_04b Tortuosity: Low			
Left Bank	0.06	Grass & Trees	
Channel	0.02	Concrete	
Right Bank	0.06	Grass & Trees	

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS01_02 to XS01_01 Tortuosity: Low			
Left Bank	0.08	Trees & Brambles	
Channel	0.04	Silt/Gravel	
Right Bank	0.08	Trees & Brambles	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result File
Baseline Scenario						
CB	Baseline	2yr	v4.4	A1_Northumberland_Cotting Burn_v06_2yr Sensitivity.ief	A1_Northumberland_Cotting Burn_v06.DAT	A1_NORTHUMBERLAND_COTTING BURN_V06_2YR SENSITIVITY.zzd
CB	Baseline	100yr + 25%	v4.4	A1_Northumberland_Cotting Burn_v06_100yr+25% Sensitivity.ief	A1_Northumberland_Cotting Burn_v06.DAT	A1_NORTHUMBERLAND_COTTING BURN_V06_100YR+25% SENSITIVITY.zzd
CB	Baseline	1000yr	v4.4	A1_Northumberland_Cotting Burn_v06_1000yr Sensitivity.ief	A1_Northumberland_Cotting Burn_v06.DAT	A1_NORTHUMBERLAND_COTTING BURN_V06_1000YR SENSITIVITY.zzd
Design Scenario 1						
CB	Proposed	2yr	v4.4	A1_Northumberland_Cotting Burn_v06_Design_v02_2yr_Sensitivity.ief	A1_Northumberland_Cotting Burn_v06_Design_v02.DAT	A1_NORTHUMBERLAND_COTTING BURN_V06_DESIGN_V02_2YR_SENSITIVITY.zzd
CB	Proposed	100yr + 25%	v4.4	A1_Northumberland_Cotting Burn_v06_Design_v02_100yr+25%_Sensitivity.ief	A1_Northumberland_Cotting Burn_v06_Design_v02.DAT	A1_NORTHUMBERLAND_COTTING BURN_V06_DESIGN_V02_100YR+25%_SENSITIVITY.zzd
CB	Proposed	1000yr	v4.4	A1_Northumberland_Cotting Burn_v06_Design_v02_1000yr_Sensitivity.ief	A1_Northumberland_Cotting Burn_v06_Design_v02.DAT	A1_NORTHUMBERLAND_COTTING BURN_V06_DESIGN_V02_1000YR_SENSITIVITY.zzd
CB	West Blockage	100yr + 25%	v4.4	Cotting Burn_v06_Design_v02_100yr+25%_Sensitivity_US_Blockage.ief	A1_Cotting Burn_v06_Design_Blockage_v02_US_Structure.dat	COTTING BURN_V06_DESIGN_V02_100YR+25%_SENSITIVITY_US_BLOCKAGE.zzd
CB	East Blockage	100yr + 25%	v4.4	Cotting Burn_v06_Design_v02_100yr+25%_Sensitivity_DS_Blockage.ief	A1_Cotting Burn_v06_Design_Blockage_v02_DS_Structure.dat	COTTING BURN_V06_DESIGN_V02_100YR+25%_SENSITIVITY_DS_BLOCKAGE.zzd

Appendix D. Model Schematics

Baseline Model (BSC)

Project	A1 - Northumberland
Job Number	70044136
Location	Priest's Bridge, Northumberland, England (418542 591629)
Watercourse(s)	River Lyne

1. Objectives/Areas of interest

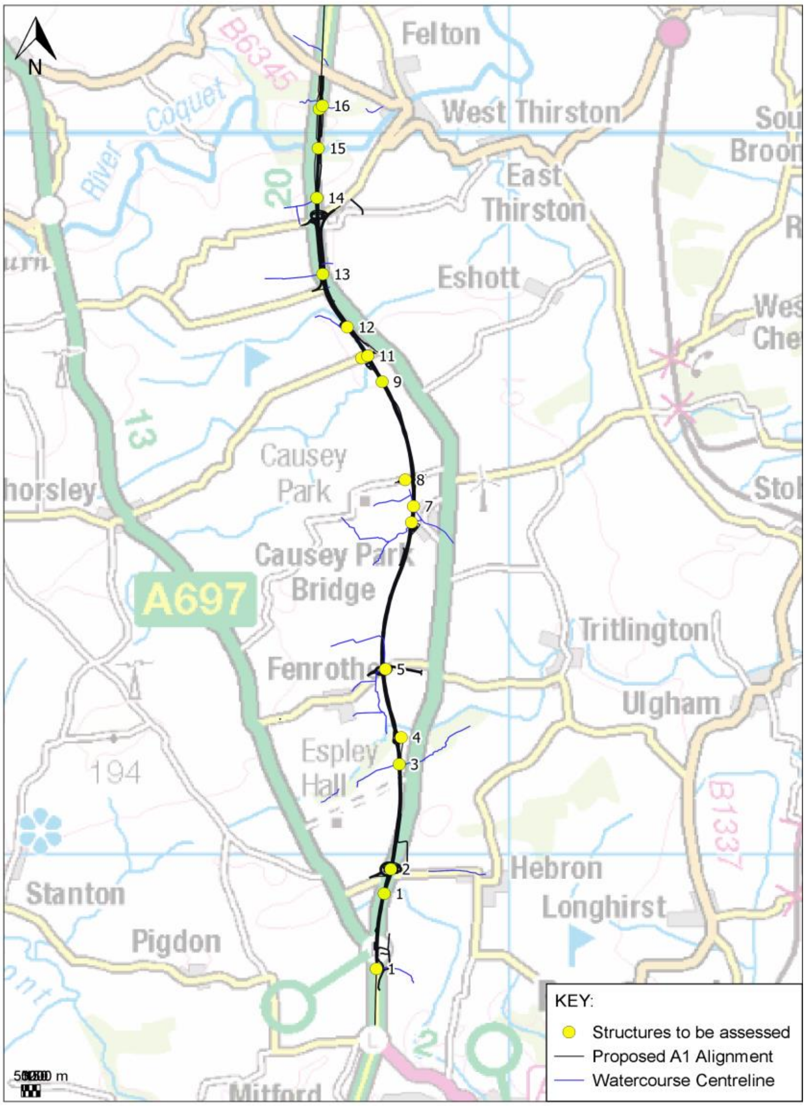


Figure 1: Location of structures in overall Scheme

The Scheme includes approximately 6.6 km of online widening and approximately 6 km of new offline highway. The existing carriageway would be widened on its current line up to Priest's Bridge, from where the proposed offline section of the Scheme would move west off the current road and pass west of Tindale Hill and Causey Park Bridge. Just north of Burgham Park, it would re-join the line of the existing carriageway and widening would continue along the existing road northwards, until it meets the existing dual carriageway north of Felton.

Figure 1 above shows the location of various structures along the Scheme. There are 16 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.

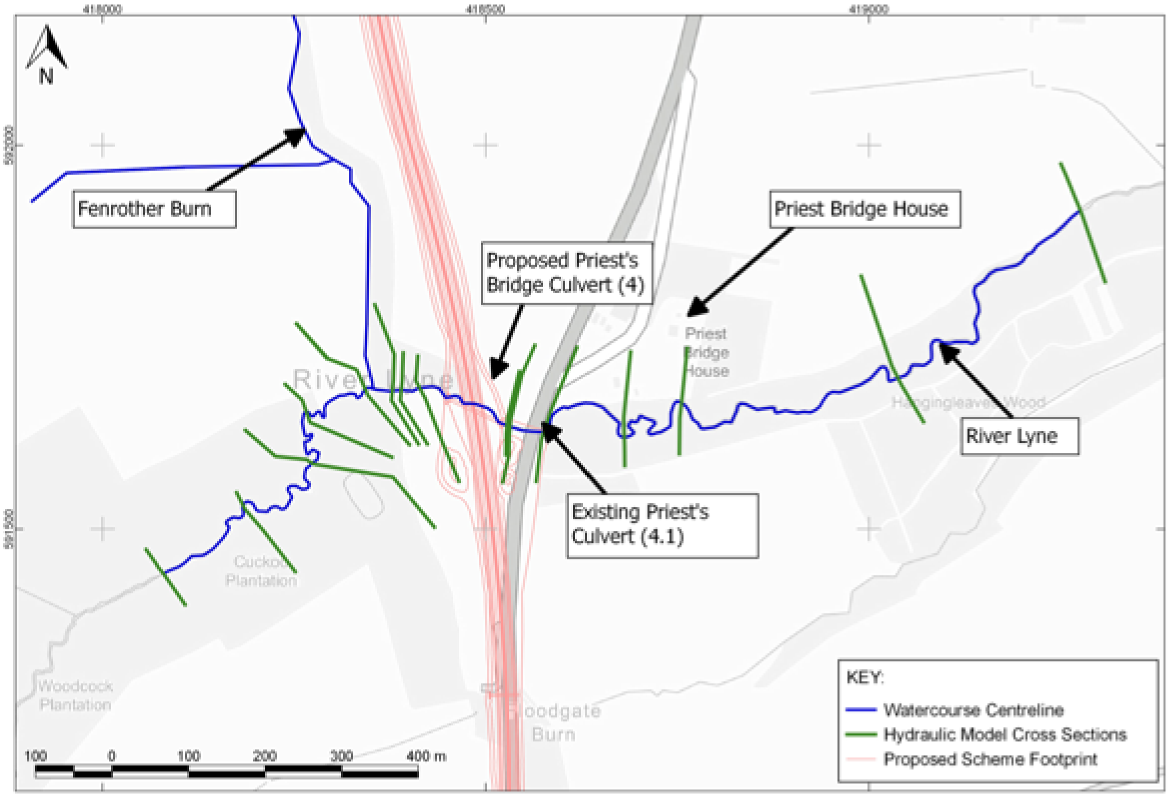


Figure 2: Location of structures on River Lyne

This report relates to the proposed works on the River Lyne. There are two existing structures within the reach. These are an existing culvert located at upstream of the proposed A1 alignment and approximately 36m downstream of the confluence with Fenrother Burn, and the existing A1 bridge. Further details on how each of these structures were modelled is presented within Appendix A of this

2. Model Input Data

Title	Type	Notes
M2F XS04.xlsx	Topographic Survey	Detailed topographic survey of area around the Fenrother 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
LIDAR -	Resolution: Date :	1m - Downstream last three cross sections are covered LIDAR - Data downloaded from survey open data at data.gov.uk , Date flown 2015
Spot level grid -	Source	Topographic survey of proposed A1 corridor between Morpeth and Alnwick collected by Jabobs. Final issue January 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 1.8km in length. The watercourse is the River Lyne.

There are two existing structures within the reach. These structures were modelled in 1D domain only 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. LIDAR data covers only the last three cross sections of the model. Local ground levels are available from survey collect by Jacobs was available and this data covers the entire extent of the model.

To confirm the agreement between the river section survey and the Jacobs topo 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.

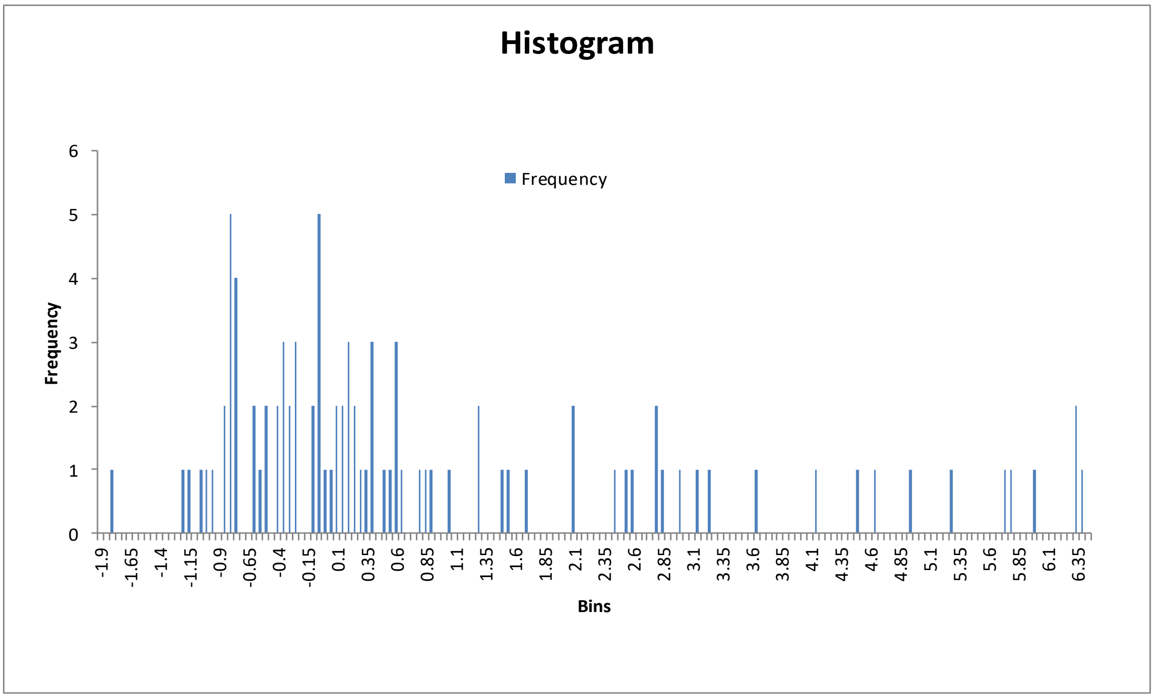


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift in the Jacobs topo of 0.25m was applied. It is however noted that there is a wide variation in the data resulting in poor agreement in some cross sections, particularly towards the upstream limits of the model.

5. Model Setup

Model Method	1D
Software	FMP (v4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using available spot level 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 River Lyne model. These are on upstream of the A1 in the vicinity of the proposed new road (RL01) and at the downstream limit of the model (RL02). 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%**
RL_01	5.00	6.72	8.00	9.80	11.35	12.39	13.19	15.38	22.13	16.49
RL_02	4.72	6.34	7.53	9.22	10.68	11.65	12.41	14.45	20.78	15.51

It can be seen from the design flows that the flows are RL02 are lower than the upstream flows. This is a result of the additional catchment between the two flow points simply including a band around the watercourse. This has the effect of increasing the mean drainage path length for the whole catchment without a significant increase in area. For this reason the flows at RL_01 have been used in the model only and are applied at the upstream limit of the model.

Flow hydrographs for the inflows have been developed using the ReFH methodology and a design storm duration derived from sensitivity testing of the critical duration in ReFH.

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, one new culvert is proposed on the west of existing A1 road crossing the River Lyne. The proposed culvert is near Priest Bridge (418542, 591629). Cross section data for the upstream and downstream faces of this structure was 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 and downstream faces.

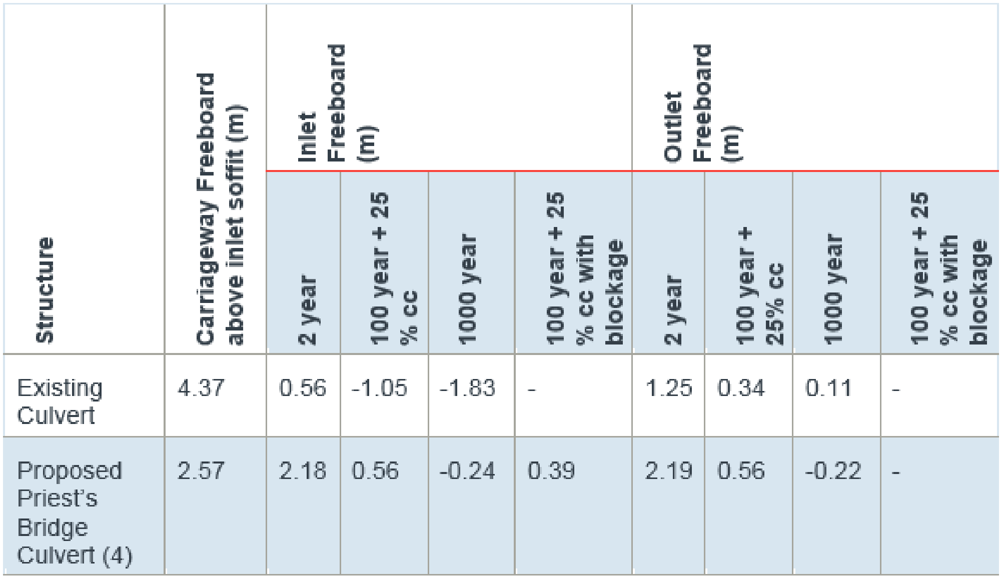
The culvert beneath the new A1 alignment will be a box culvert. The width of this culvert is currently assumed to be 4m, although this is significantly wider than the existing channel. Initial iterations modelled the culvert soffit artificially high so that a free water surface could be determined and from this the design soffit level inclusive of freeboard requirements for otter passage. Following the iterative design process a final culvert height of 3.75m has been preferred. 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

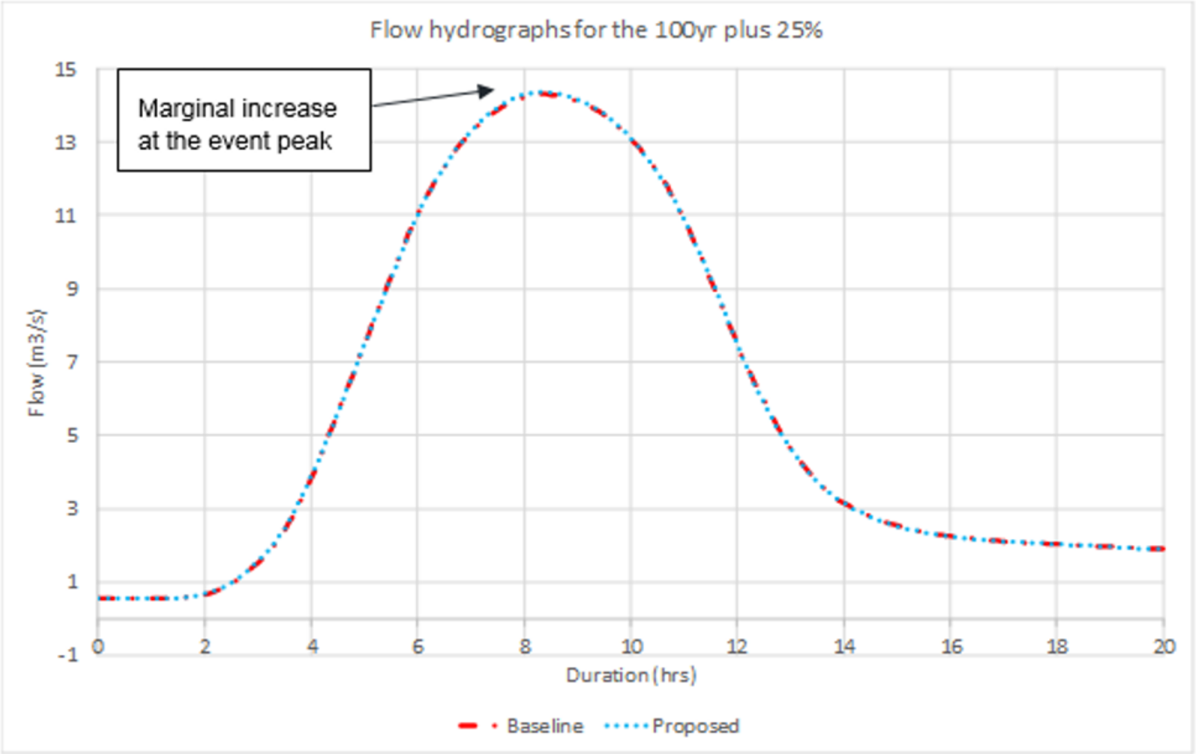
11. Model Results

The following table provides details of the freeboard associated with each structure for a range of flood events. The River Lyne is classified as a main river downstream from the village of Tritlington but is classified as an ordinary watercourse in the location of the A1. As such a design freeboard of 300mm is preferred in the 100yr+25% climate change event in accordance with DMRB. The 1000yr event is larger than the 100yr+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%.

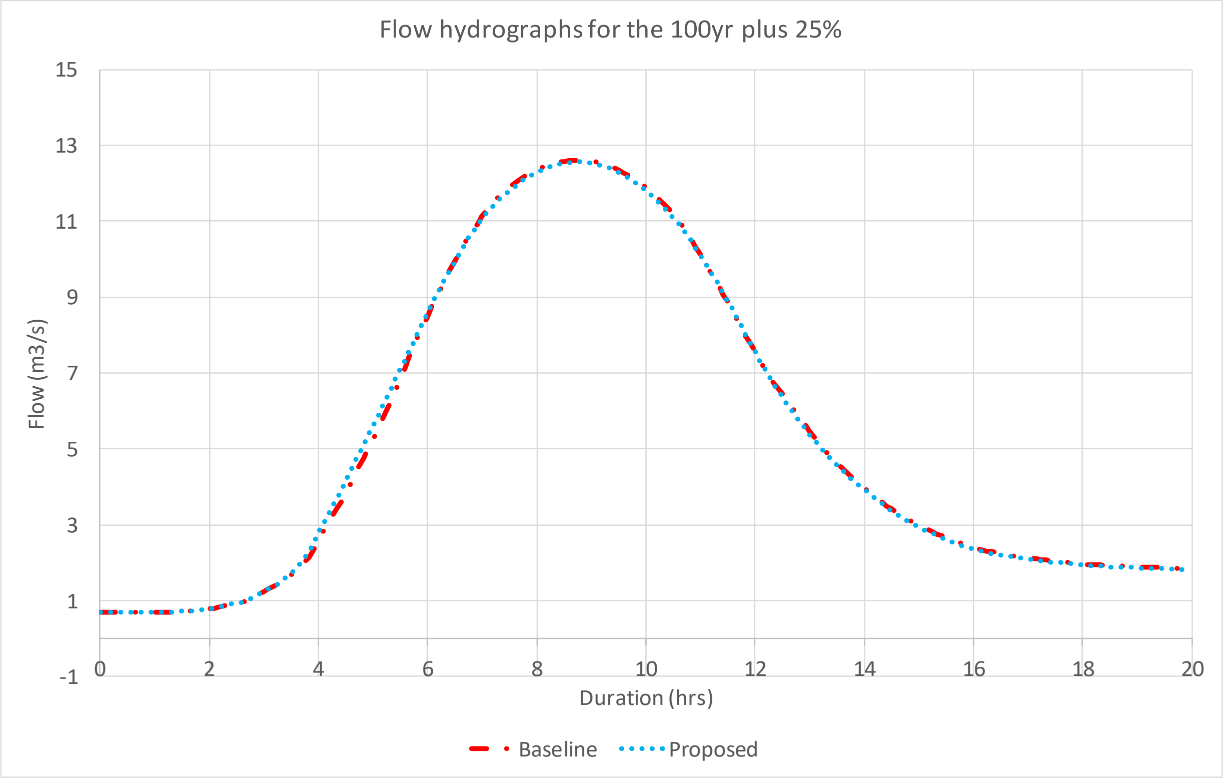


Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25% cc	1000 year	100 year + 25 % cc with blockage
Existing Culvert	4.37	0.56	-1.05	-1.83	-	1.25	0.34	0.11	-
Proposed Priest's Bridge Culvert (4)	2.57	2.18	0.56	-0.24	0.39	2.19	0.56	-0.22	-

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. This demonstrates that the proposed design will marginally increase downstream flows. This is attributed to the current oversizing of the structure and the more efficient transportation of flows through the culvert compared to the existing culvert.



The effect of this increase in flows is offset by the reduced flows coming from the Fenrother Burn tributary following the Scheme, full details of this model are provided in the Fenrother Burn Modelling Report. The River Lyne is a larger catchment than Fenrother Burn and has a slower flood response to the confluence with the Fenrother Burn as a result. To quantify the impact of the overall Scheme the River Lyne and Fenrother Burn models have been combined and the pass forward flows extracted, as shown below. The effect is an overall reduction in flows resulting from the Scheme.



Appendix A Structures

Baseline Model (BSC)

Ref.	Description	Data source	Dimensions	Modelling Approach
1 (XS04_07 A)	Culvert crossing the River Lyne on the west of A1.	Cross sectional data provided by the surveyor.	The culvert length is 5m. The culvert consists of double circular conduits of diameters 0.82m and 0.75m. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as double 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 (XS04_06)	Culvert crossing the River Lyne on A1.	Cross sectional data provided by the surveyor.	The culvert length is 34m. The culvert consist of single ovoid conduit. These dimensions have been taken directly from the data provided by surveyor.	The culvert has been modelled as symmetrical 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 (XS04_07)	Proposed Priest Bridge culvert on the new A1 alignment crossing the River Lyne.	Local channel dimensions derived from the spot level grid and topo survey data provided by the surveyor.	The proposed structure spans 53m with 4.0m width and 3.5m height allowing for 0.25m silt in the bed.. The width and height of the culvert was set so as to get 600mm freeboard over the maximum water level for 100 year flood event with 25% increase for climate change.	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.

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
XS04_12 to XS04_08 Tortuosity: Low			
Left Bank	0.08	Grass,Shrubs,Trees	
Channel	0.04	Gravel upto 100mm	
Right Bank	0.08	Grass,Shrubs,Trees	

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS04_07A to XS04_07 Tortuosity: Low			
Left Bank	0.06	Grass	
Channel	0.04	Gravel upto 100mm	
Right Bank	0.06	Grass	

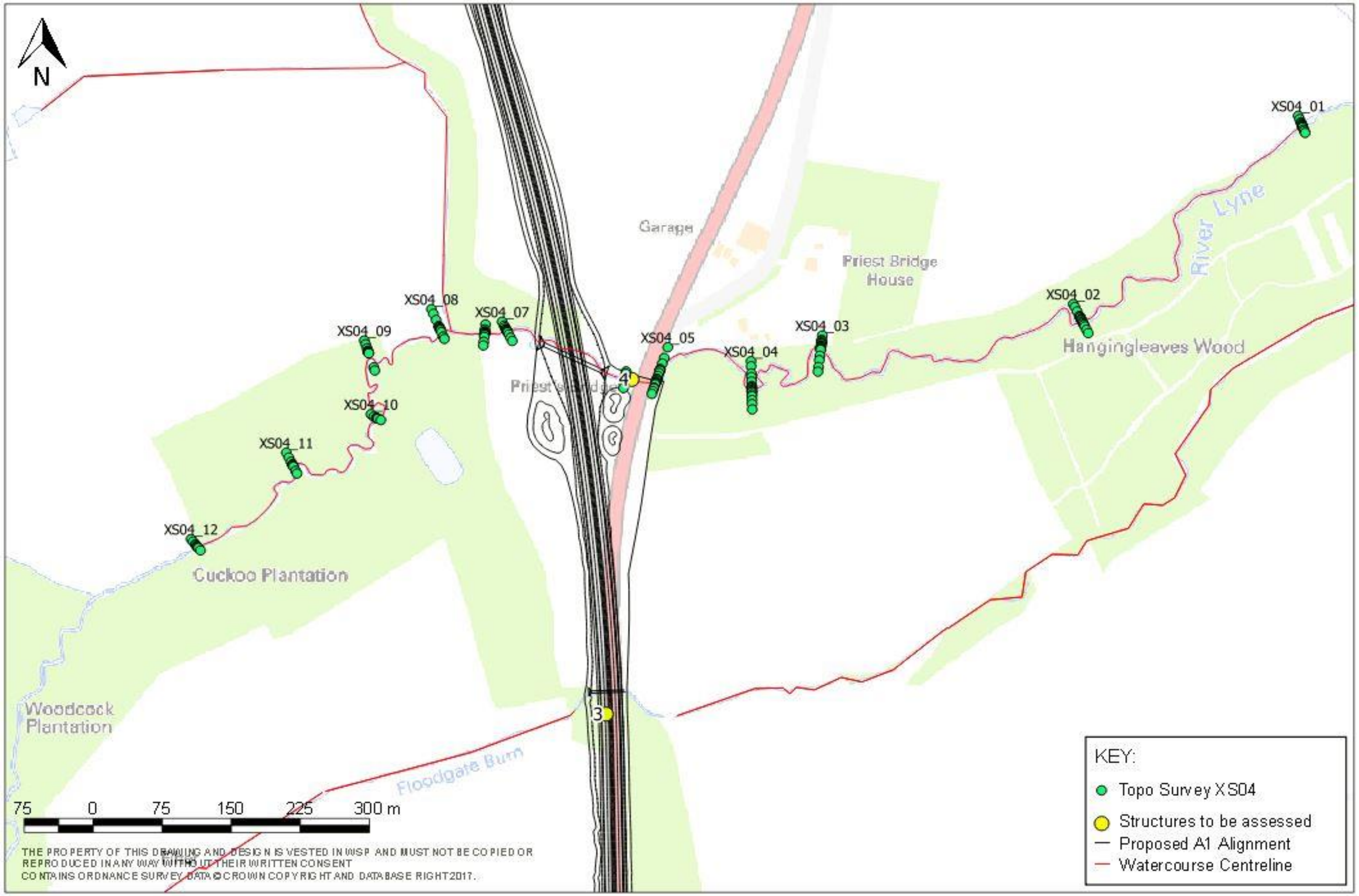
Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS04_06 to XS04_01 Tortuosity: Low			
Left Bank	0.08	Grass,Shrubs,Trees	
Channel	0.04	Silt/Stones	
Right Bank	0.08	Grass,Shrubs,Trees	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result File
Baseline Scenario						
RL	Baseline	2yr	v4.4	A1_Northumberland_River Lyne_v05_2yr_Sensitivity.ief	A1_Northumberland_River Lyne_v05.DAT	A1_NORTHUMBERLAND_RIVER LYNE_V05_2YR_SENSITIVITY
RL	Baseline	100yr + 25%	v4.4	A1_Northumberland_River Lyne_v05_100yr+25% CC_Sensitivity.ief	A1_Northumberland_River Lyne_v05.dat	A1_NORTHUMBERLAND_RIVER LYNE_V05_100YR+25% CC_SENSITIVITY.zzd
RL	Baseline	1000yr	v4.4	A1_Northumberland_River Lyne_v05_1000yr_Sensitivity.ief	A1_Northumberland_River Lyne_v05.dat	A1_NORTHUMBERLAND_RIVER LYNE_V05_1000YR_SENSITIVITY.zzd
RL_FB	Baseline	100yr + 25%	v4.4	A1_RL_FB_v01_100yr+25% CC_Sensitivity.ief	A1_Northumberland_River Lyne_Fenrother_v01.dat	A1_RL_FB_V01_100YR+25% CC_SENSITIVITY.zzd
Design Scenario						
RL	Proposed	2yr	v4.4	A1_Northumberland_River Lyne_v05_Design_v02_2yr_Sensitivity.ief	A1_Northumberland_River Lyne_v05_Design_v02.DAT	A1_NORTHUMBERLAND_RIVER LYNE_V05_DESIGN_V02_2YR_SENSITIVITY
RL	Proposed	100yr + 25%	v4.4	A1_Northumberland_River Lyne_v05_Design_v02_100yr+25%_Sensivity.ief	A1_Northumberland_River Lyne_v05_Design_v02.dat	A1_NORTHUMBERLAND_RIVER LYNE_V05_DESIGN_V02_100YR+25%_SENSITIVITY
RL	Proposed	1000yr	v4.4	A1_Northumberland_River Lyne_v05_Design_v02_1000yr_Sensitivity.ief	A1_Northumberland_River Lyne_v05_Design_v02.dat	A1_NORTHUMBERLAND_RIVER LYNE_V05_DESIGN_V02_1000YR_SENSITIVITY
RL	Blockage	100yr + 25%	v4.4	A1_River Lyne_v05_Design_v02_100yr+25%_Sensivity_Blockage.ief	A1_Northumberland_River Lyne_v05_Design_Blockage_v02.DAT	A1_RIVER LYNE_V05_DESIGN_V02_100YR+25%_SENSIVITY_BLOCKAGE
RL_FB	Proposed	100yr + 25%	v4.4	A1_RL_FB_v01_Design_v02_100yr+25%_Sensivity.ief	A1_Northumberland_River Lyne_Fenrother_v01_Design_v02.dat	A1_RL_FB_V01_DESIGN_V02_100YR+25%_SENSIVITY

Appendix D. Model Schematics

Baseline Model (BSC)



Project	A1 - Northumberland
Job Number	70044136
Location	Fenrother, Northumberland, England (418215 592530)
Watercourse(s)	Fenrother Burn

1. Objectives/Areas of interest

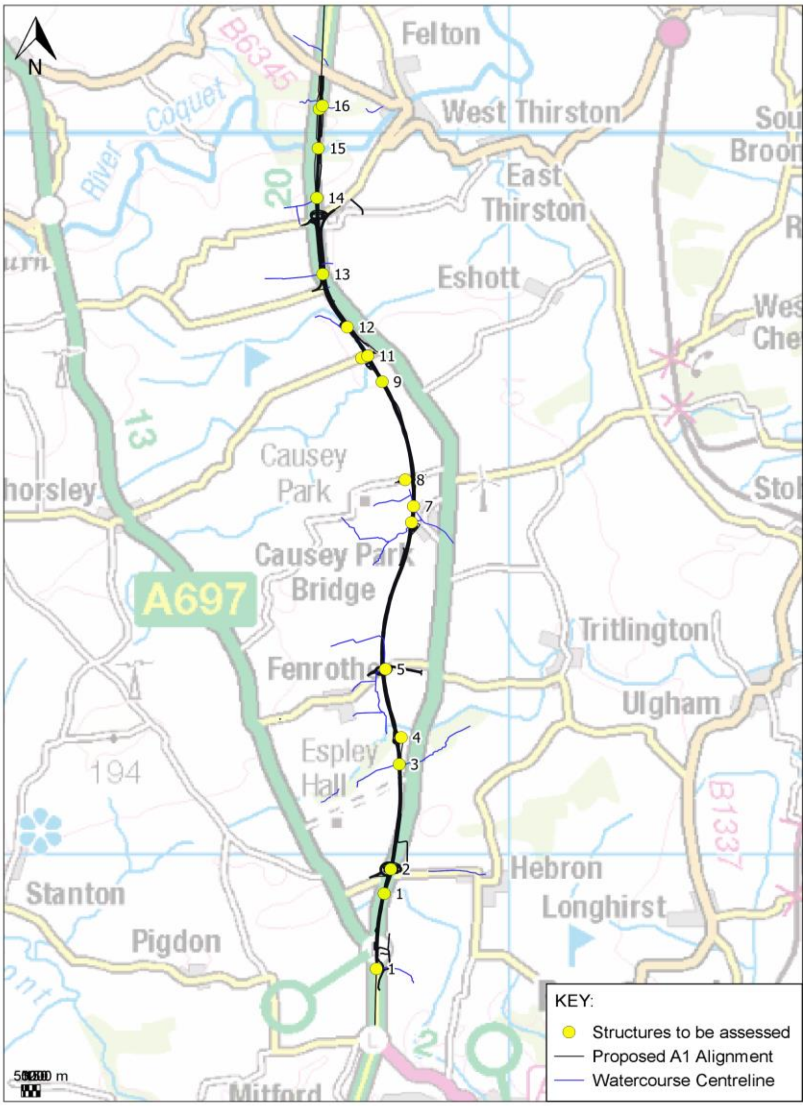


Figure 1: Location of structures in overall Scheme

The Scheme includes approximately 6.6 km of online widening and approximately 6 km of new offline highway. The existing carriageway would be widened on its current line up to Priest's Bridge, from where the proposed offline section of the Scheme would move west off the current road and pass west of Tindale Hill and Causey Park Bridge. Just north of Burgham Park, it would re-join the line of the existing carriageway and widening would continue along the existing road northwards, until it meets the existing dual carriageway north of Felton.

Figure 1 above shows the location of various structures along the Scheme. There are 16 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.

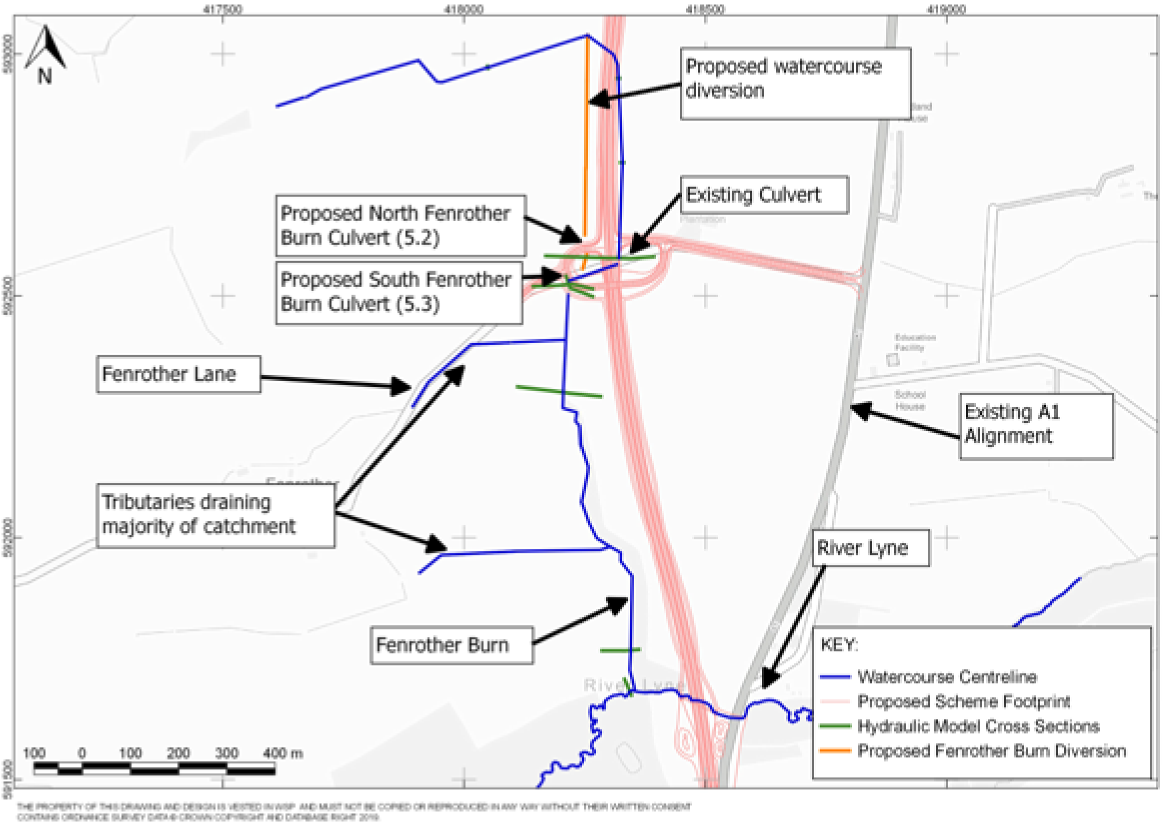


Figure 2: Location of structures on Fenrother Burn

This report relates to the proposed works on the Fenrother Burn watercourse. There is one existing structure within the reach. This is the existing culvert on Fenrother Lane crossing the Fenrother Burn approximately 915m upstream of the confluence with River Lyne. The length of the culvert is approximately 120m. Further details on how this structure was modelled is presented within Appendix A of this report.

2. Model Input Data

Title	Type	Notes
M2F XS05.xlsx	Topographic Survey	Detailed topographic survey of area around the Fenrother 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
Spot level grid -	Source	Topographic survey of proposed A1 corridor between Morpeth and Alnwick collected by Jabobs. Final issue January 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 1.75km in length. The watercourse is Fenrother Burn.

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

Model cross sections have been extended into the floodplain to accomodate the design flows. LIDAR data is not available for this model. Local ground levels are available from survey collect by Jacobs was available and this data covers the entire extent of the model.

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

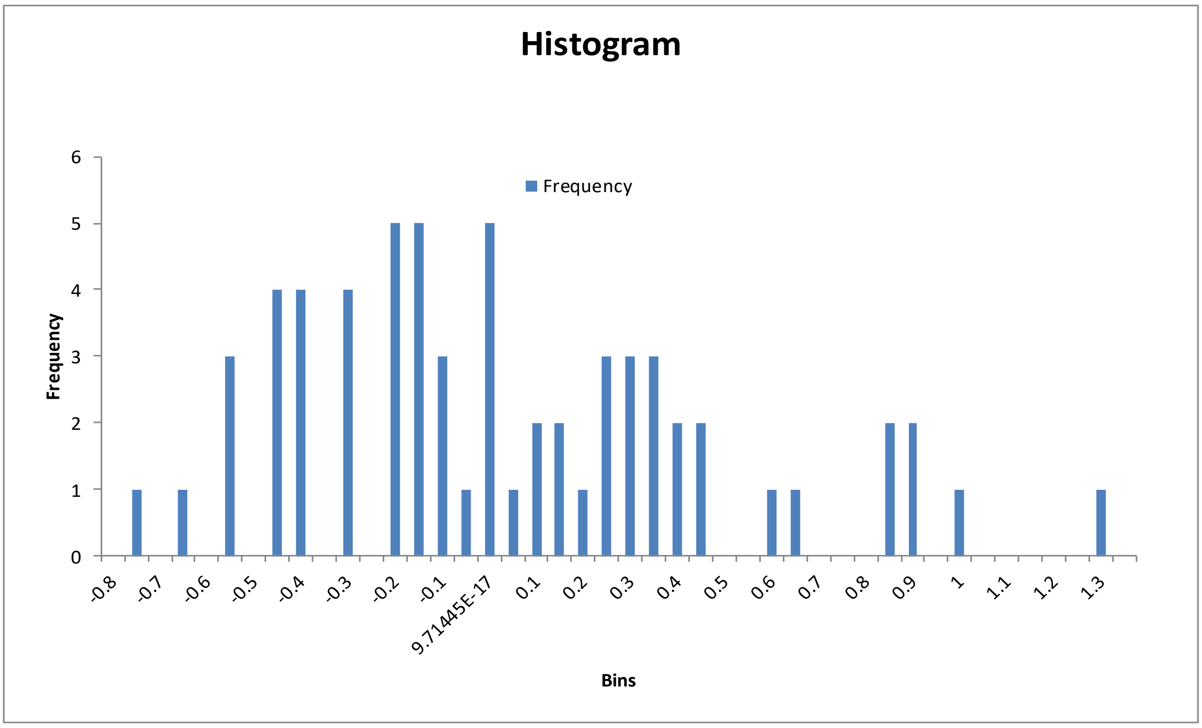


Figure 3: Survey Comparison

The plot shows a wide variability in the levels between the two datasets. On the basis of the data above a shift in the Jacobs topo of 0.25m was applied. It is however noted that there is a wide variation in the data resulting in poor agreement in some cross sections, particularly towards the upstream limits of the model.

5. Model Setup

Model Method	1D
Software	FMP (v4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using available spot level 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 Fenrother Burn model. These are at the upstream limit of the Fenrother Burn model (FB01), and for downstream limit of the model incorprating an unnamed triburaty that discharges into the watercourse downstream of the Fenrother Lane culvert (FB02). 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%**
FB_01	0.39	0.53	0.63	0.77	0.90	0.98	1.04	1.21	1.75	1.30
FB_02	2.06	2.79	3.32	4.07	4.71	5.14	5.47	6.38	9.17	6.84

Flow hydrographs for the inflows have been developed using the ReFH methodology and a and a design storm duration derived from sensitivity testing of the critical duration in ReFH using catchment descriptors for the FB_02 catchment. These have been applied at the upstream limit of the model and and the location of the unnamed tributary and scaled to reflect the peaks detailed in the table above.
The inflow for the tributary has been derived by subtracting the hydrograph for the upstream of the catchment from the hydrograph for the downstream of the catchment and scaled to the difference in peak.
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, a stretch of about 550m of Fenrother Burn is to be diverted along the A1 upstream of Fenrother Lane (418212, 592536). Five trapezoidal cross sections were proposed with side slope of 1:3. The elevation data was extracted from the spot level grid.

Two new culverts are proposed near the crossing of Fenrother Burn watercourse with the Fenrother Lane. The base data for this structure was obtained from the spot level grid. Further details on how this structure was modelled is presented within Appendix A of this report.

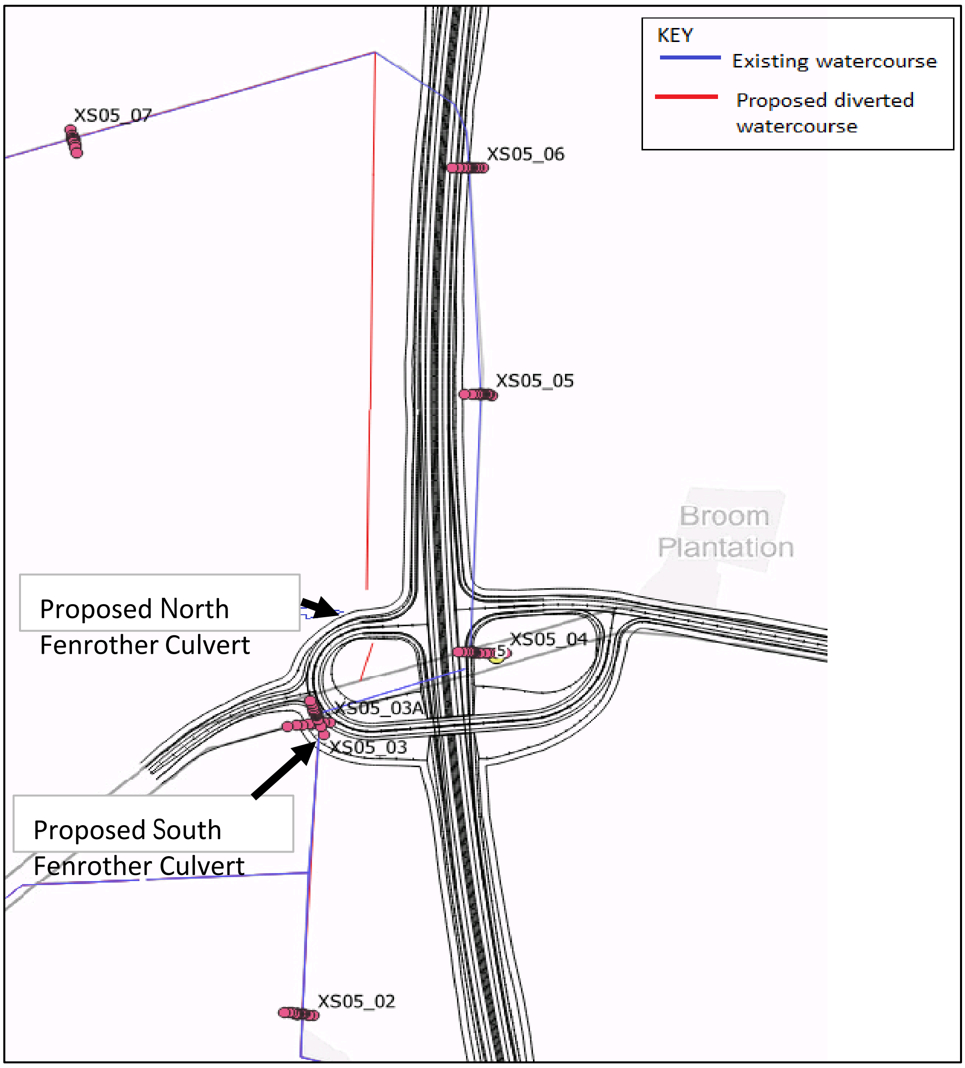


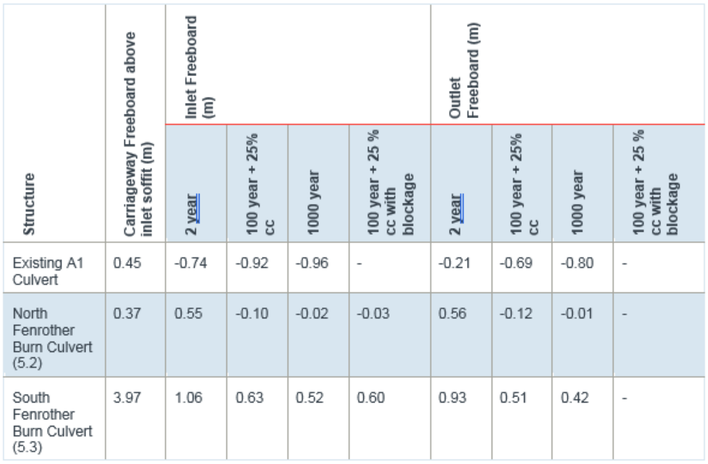
Figure 4: Proposed aligments for Fenrother Burn

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

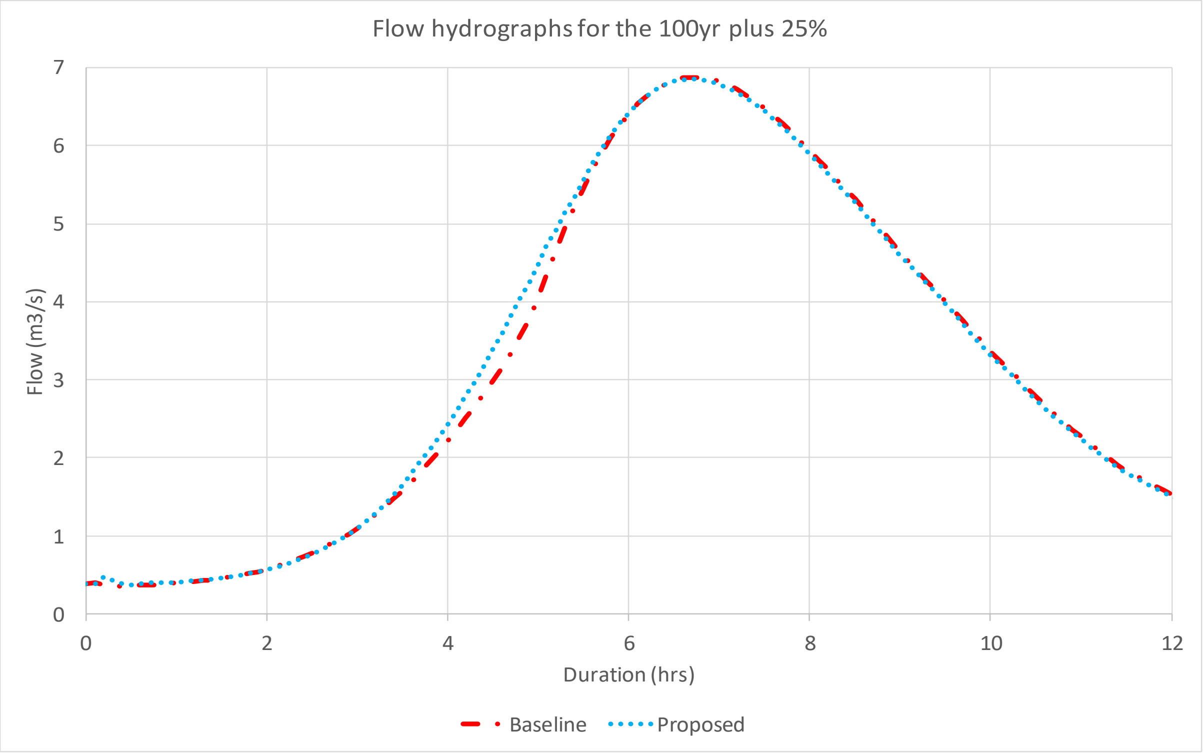
11. Model Results

The following table provides details of the freeboard associated with each structure for a range of flood events. Fenrother Burn is an ordinary watercourse. As such a design freeboard of 300mm is preferred in the 100yr+25% climate change event in accordance with DMRB. The 1000yr event is larger than the 100yr+50% climate change event so 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 67% for culvert A and by 30% for culvert B reflecting the different sizes of these structures and hence the likelihood of blockage.



Structure	Carriageway Freeboard above inlet soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25% cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25% cc	1000 year	100 year + 25 % cc with blockage
Existing A1 Culvert	0.45	-0.74	-0.92	-0.96	-	-0.21	-0.69	-0.80	-
North Fenrother Burn Culvert (5.2)	0.37	0.55	-0.10	-0.02	-0.03	0.56	-0.12	-0.01	-
South Fenrother Burn Culvert (5.3)	3.97	1.06	0.63	0.52	0.60	0.93	0.51	0.42	-

To understand the implications of flood risk dowsntream of the culvert, a flow hydrograph has been compared between the existing and proposed models at the downstream boundary of the model. This demonstrates that the proposed design will marginally reduce downstream flows, despite the reduction in channel length, largely due to the assumption that the new design will prevent overtopping of Fenrother Lane and so slightly increase attenuation.



Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Data source	Dimensions	Modelling Approach
1 (XS05_04)	Culvert on Fenrother Burn crossing the Fenrother Lane on the west of A1.	Cross sectional data provided by the surveyor.	The culvert length is 119.5m. The culvert consists of single circular conduit with inlet diameter of 1.0m, and outfall of 0.5m. 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 (XS05_04 Prop)	Proposed bridge on the Fenrother Burn crossing new A1 alignment.	Local channel dimensions derived from spot level grid	The proposed structure is a twin box culvert 33m in length. Each culvert is 1.5m wide and 1m high allowing for 0.25m silt in the bed.	The bridge has been modelled as a double rectangular culvert within the channel. A spill section was created using the cross-section data from the upstream section in order to provide information around potential overtopping of the structure.
1 (XS05_03 PropU)	Proposed bridge on the Fenrother Burn crossing new A1 alignment.	Local channel dimensions derived from spot level grid	The proposed structure is 53m in length. The structure is 3.0m wide and 1.5m high allowing for 0.25m silt in the bed.	The bridge has been modelled as a single rectangular culvert within the channel. A spill section was created using the cross-section data from the upstream section 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
XS05_07 to XS05_06 Tortuosity: High			
Left Bank	0.06	Grass	
Channel	0.05	Mud/Dry Bed	
Right Bank	0.06	Grass	

Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS05_05 Tortuosity: Low			
Left Bank	0.06	Grass	
Channel	0.04	Dry Bed	
Right Bank	0.06	Grass	

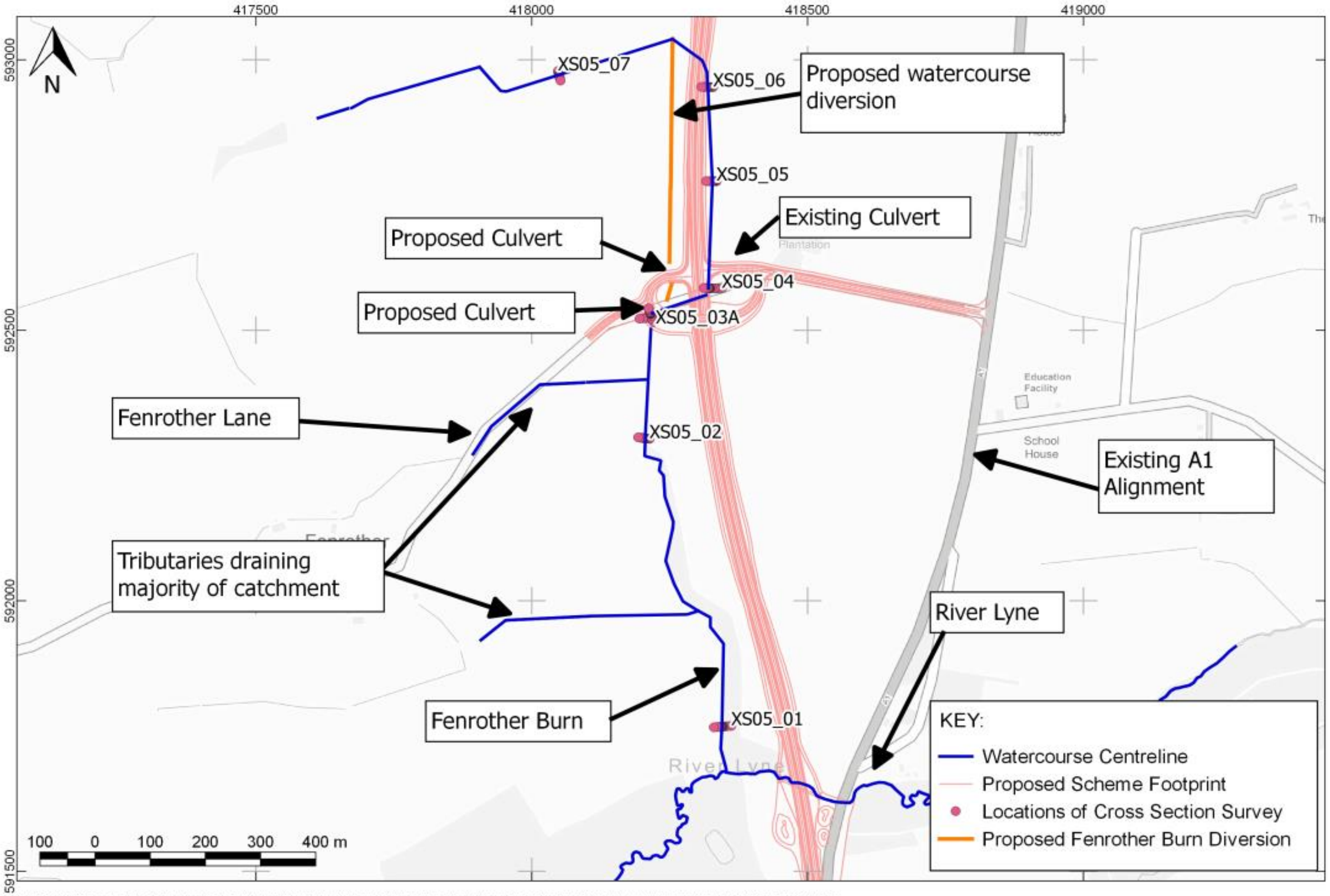
Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS05_04 to XS05_01 Tortuosity: Low			
Left Bank	0.08	Grass/Hedge	
Channel	0.04	Gravels/Rocks	
Right Bank	0.08	Grass/Hedge	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result File
Baseline Scenario						
FB	Baseline	2yr	v4.4	A1_Northumberland_Fenrother Burn_v06_2yr_Sensitivity.ief	A1_Northumberland_Fenrother Burn_v06.dat	A1_NORTHUMBERLAND_FENROTHER BURN_V06_2YR_SENSITIVITY.zzd
FB	Baseline	100yr + 25%	v4.4	A1_Northumberland_Fenrother Burn_v06_100yr+25% CC_Sensitivity.ief	A1_Northumberland_Fenrother Burn_v06.dat	A1_NORTHUMBERLAND_FENROTHER BURN_V06_100YR+25% CC_SENSITIVITY.zzd
FB	Baseline	1000yr	v4.4	A1_Northumberland_Fenrother Burn_v06_1000yr_Sensitivity.ief	A1_Northumberland_Fenrother Burn_v06.dat	A1_NORTHUMBERLAND_FENROTHER BURN_V06_1000YR_SENSITIVITY.zzd
Design Scenario						
FB	Proposed	2yr	v4.4	A1_Northumberland_Fenrother Burn_v06_Design_v04_2yr_Sensitivity.ief	A1_Northumberland_Fenrother Burn_v06_Design_v04.dat	A1_NORTHUMBERLAND_FENROTHER BURN_V05_DESIGN_V04_2YR
FB	Proposed	100yr + 25%	v4.4	A1_Northumberland_Fenrother Burn_v06_Design_v04_100yr+25%_Sensitivity.ief	A1_Northumberland_Fenrother Burn_v06_Design_v04.dat	A1_NORTHUMBERLAND_FENROTHER BURN_V05_DESIGN_V04_10YR
FB	Proposed	1000yr	v4.4	A1_Northumberland_Fenrother Burn_v06_Design_v04_1000yr_Sensitivity.ief	A1_Northumberland_Fenrother Burn_v06_Design_v04.dat	A1_NORTHUMBERLAND_FENROTHER BURN_V05_DESIGN_V04_100YR+25% CC
FB	Blockage	100yr + 25%	v4.4	A1_Fenrother Burn_v06_Design_v04_100yr+25%_Sensitivity_US_Blocakge.ief	A1_Fenrother Burn_v06_Design_v04_US_Blockage.dat	A1_FENROTHER BURN_V06_DESIGN_V04_100YR+25%_SENSITIVITY_US_BLOCAKGE.zzd
FB	Blockage	100yr + 25%	v4.4	A1_Fenrother Burn_v06_Design_v04_100yr+25%_Sensitivity_DS_Blockage.ief	A1_Fenrother Burn_v06_Design_v04_DS_Blockage.dat	A1_FENROTHER BURN_V06_DESIGN_V04_100YR+25%_SENSITIVITY_DS_BLOCKAGE.zzd

Appendix D. Model Schematics

Baseline Model (BSC)



Project	A1 - Northumberland
Job Number	70044136
Location	Causey Park, Northumberland, England (418655 594596)
Watercourse(s)	Earsdon Burn

1. Objectives/Areas of interest

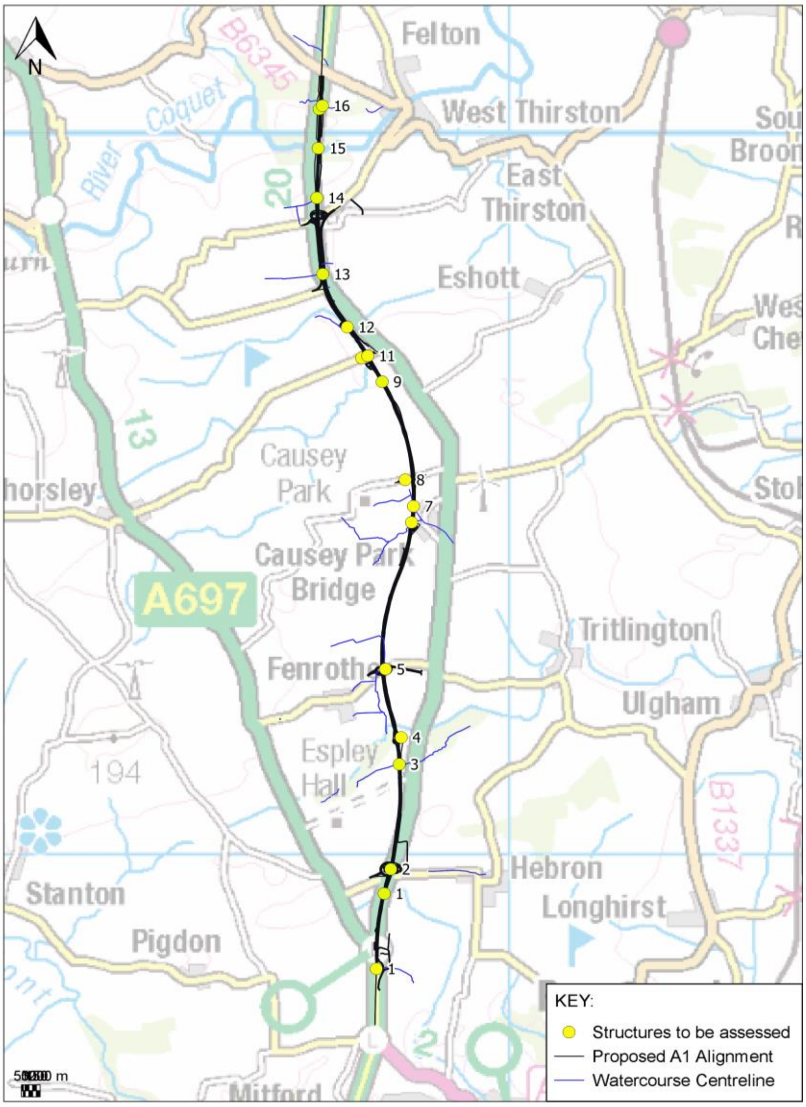


Figure 1: Location of structures in overall Scheme

The Scheme includes approximately 6.6 km of online widening and approximately 6 km of new offline highway. The existing carriageway would be widened on its current line up to Priest's Bridge, from where the proposed offline section of the Scheme would move west off the current road and pass west of Tindale Hill and Causey Park Bridge. Just north of Burgham Park, it would re-join the line of the existing carriageway and widening would continue along the existing road northwards, until it meets the existing dual carriageway north of Felton.

Figure 1 above shows the location of various structures along the Scheme. There are 16 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.

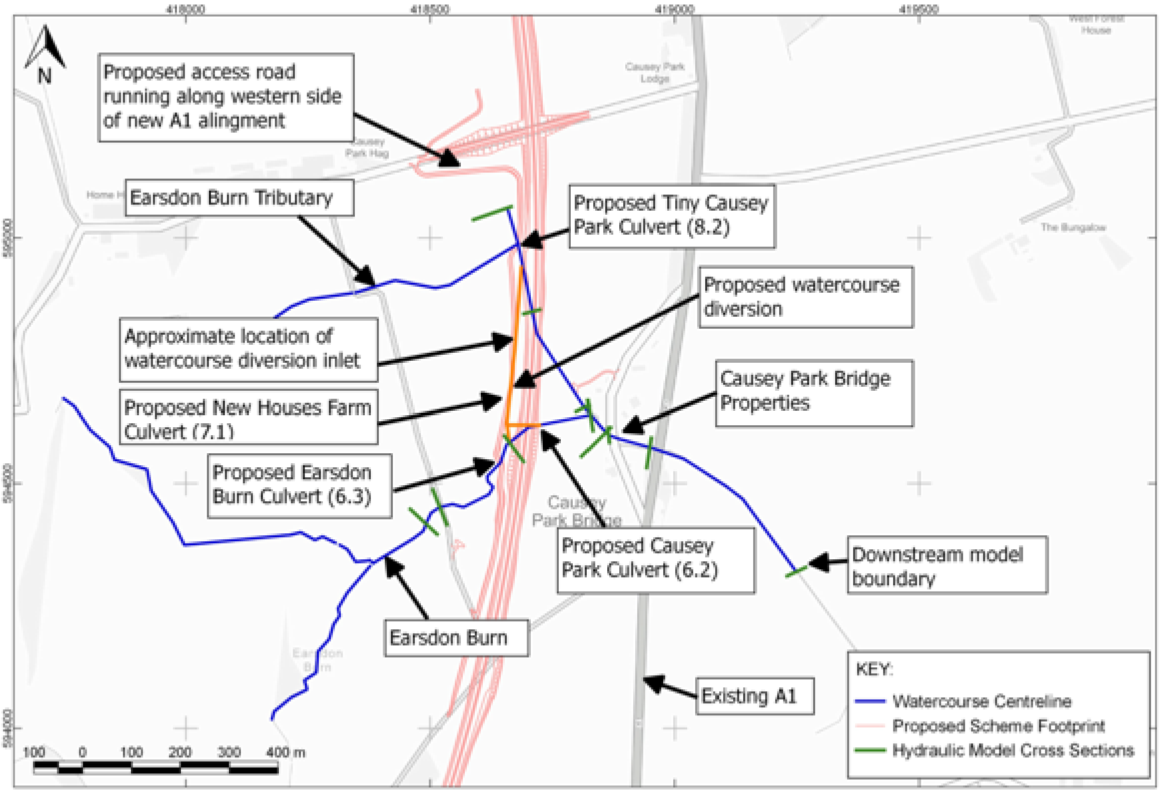


Figure 2: Location of structures on Earsdon Burn

This report relates to the proposed works on and around the Earsdon Burn watercourse. There are 4 existing structures along two watercourses. These include a small road bridge on Earsdon Burn upstream of the Scheme, a bridge near the Oak Inn in Causey Park, the A1 culvert itself and a footbridge at the confluence of the two watercourses.

It was originally assumed the unnamed watercourse that joins Earsdon Burn continued further north, however the findings from the survey show that the east west road at the the top of Figure 2 is located at the top of the local topography and there is no culvert beneath it. Any channels present drain away from the road on either side, as such this structure is excluded from any further assessment. 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
XRFEF - TOPO 6 & 7.dwg M2F XS06.xlsx XS07.xlsx	Topographic Survey	Detailed topographic survey of area around the Causey Park 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
LIDAR -	Resolution:	1m - Downstream last three cross sections are covered
	Date :	LIDAR - Data downloaded from survey open data at data.gov.uk , Date flown 2015

Spot level grid -	Source	Topographic survey of proposed A1 corridor between Morpeth and Alnwick collected by Jabobs. Final issue January 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 1.5km in length. The main watercourse is Earsdon Burn and one unnamed watercourse which flows into the Earsdon Burn from the north.

There are 4 existing structures within the two reaches. These structures were modelled in 1D domain only 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 levels are available from 1m LIDAR tiles. These cover the entire extent of the model.

To confirm the agreement between the river section survey and the LIDAR 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.

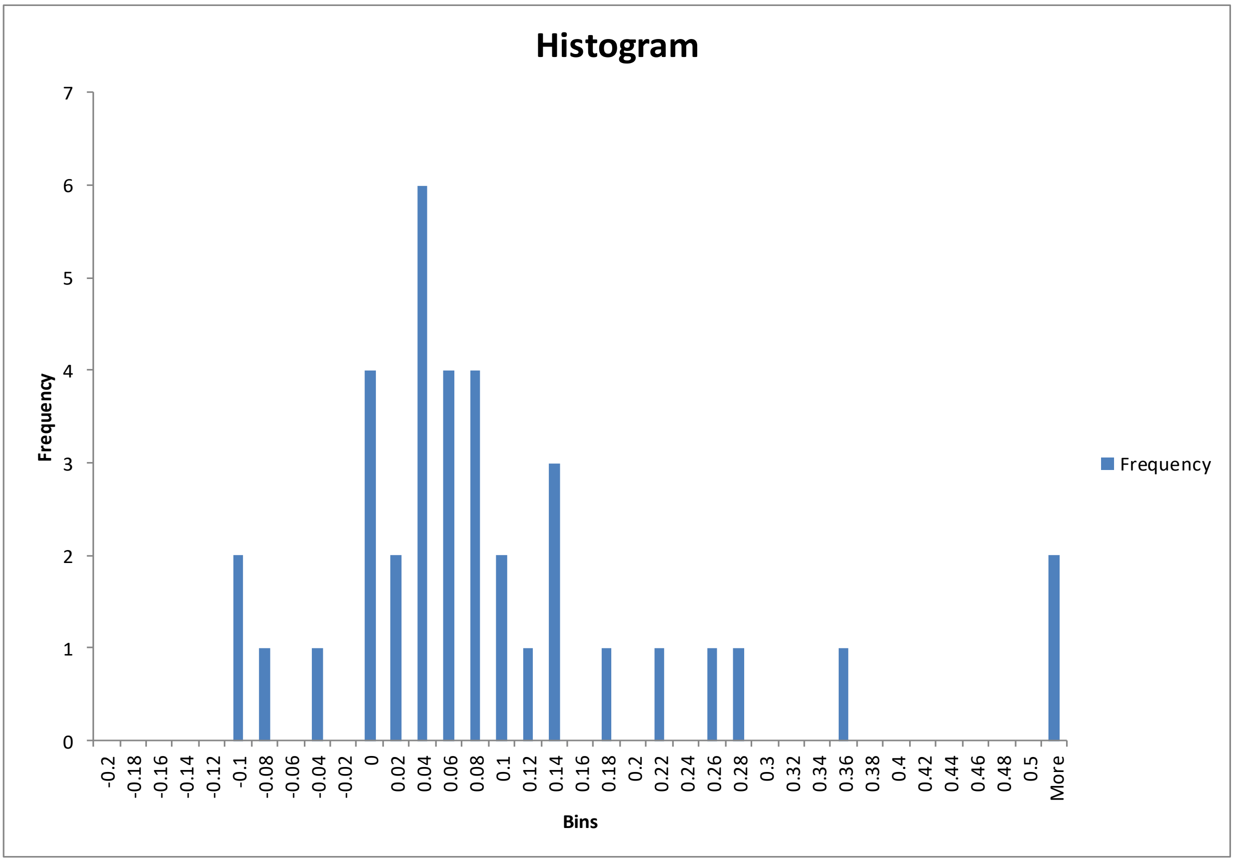


Figure 3: Survey Comparison

The plot shows a consistent shift between the LIDAR and survey data of approximately 0.05m. On this basis the LIDAR data has been adjusted down by 0.05m globally before incorporation into the model.

5. Model Setup

Model Method	1D
Software	FMP (v4.4)
Channel	1D sections modelled using FMP.
Floodplain	Extended cross sections using LIDAR
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 3 locations for the Earsdon Burn model. These are on Earsdon Burn upstream of the proposed new alignment for the A1 (EB01), on the unnamed tributary at the confluence with Earsdon Burn (EB02) and at the downstream limit of the model located downstream of the existing A1. 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%**
EB_01	2.12	2.90	3.46	4.26	4.94	5.38	5.73	6.67	9.62	7.16
EB_02	0.37	0.50	0.60	0.74	0.86	0.94	1.00	1.17	1.69	1.25
EB_03	2.87	3.91	4.65	5.72	6.63	7.23	7.69	8.97	12.96	9.61

Flow hydrographs for the inflows have been developed using the ReFH methodology and a design storm duration derived from sensitivity testing of the critical duration in ReFH using catchment descriptors for the EB03 catchment. These have been applied at the upstream limits of the two watercourses and scaled to reflect the peaks detailed in the table above. A third inflow has been derived reflecting difference between the peak flows at EB_03 and the two upstream inflows. This has been applied in the model downstream of proposed A1 alignment such that, in the absence of hydraulic controls along the watercourses, the downstream flow in the model would reflect EB_03 in the table above.

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. As the length of the reach being small, single reach roughness value assessed. 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 A1 road alignment cuts diagonally across both the unnamed watercourse and Earsdon Burn. To reduce the number of culverts beneath the new road, the unnamed watercourse is to be diverted along the western side of the new A1 alignment to a new confluence with Earsdon Burn. Similarly the Earsdon Burn watercourse will be diverted northwards for a short distance so that the new culvert to be constructed will be perpendicular to the A1. The culvert crossing will tie in with the existing alignment of Earsdon Burn at its downstream face. Figure 2 shows the new alignments for the watercourses and the proposed new culvert.

The diversion of the unnamed tributary to the south will cut through an area of raised ground for approximately 120m upstream of the new confluence. The ground levels in this location are such that an open channel here is not viable. As such the watercourse will be culverted for this length. There is an existing gas main the runs beneath the proposed road alignment from southwest to northeast in this area. The tributary culvert will be located so that the inlet is immediately downstream of this existing gas main.

The culvert beneath the new A1 alignment will be a box culvert. The proposed width of this culvert is representative of the downstream channel width. Initial iterations modelled the culvert soffit artificially high so that a free water surface could be determined and from this the design soffit level inclusive of freeboard requirements for flood risk and for otter passage determined. Following the iterative design process a final culvert height of 2.1m has been preferred. The same height has been used for the upstream culvert beneath the access road.

The culverts on the unnamed tributary will be pipe culverts, these will also require a freeboard of 300mm and the size of these structures has been determined iteratively to provide this freeboard level.

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

Upstream of the tributary culvert the channel has been modelled as a trapezoidal channel with side slope of 1:2. The elevation data for bank levels for these new sections was extracted from the LIDAR.

10. Model Runs

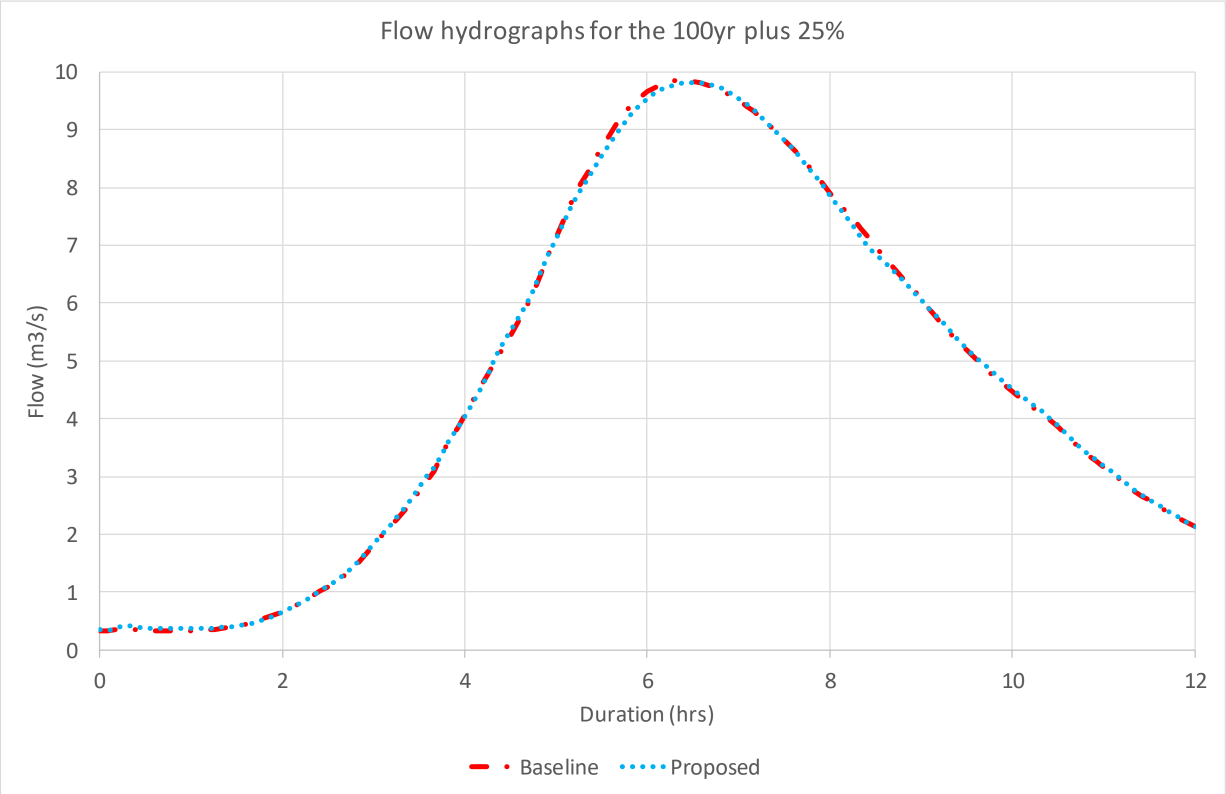
Model Scenario	Return Periods / Events
Baseline (BSC)	2yr, 100yr+25%, 1000yr
Proposed Scenario (PRO)	2yr, 100yr+25%, 1000yr, 100yr+25% with blockage

11. Model Results

The following provides details of the freeboard associated with each structure for a range of flood events. Earsdon 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 DMRB. The 1000yr event is larger than the 100yr+50% 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 Earsdon Burn culverts and by 67% for the tributary culverts reflecting the different sizes of these structures and hence the likelihood of blockage.

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 A1	4.04	1.11	0.24	-0.06	-	1.08	0.36	0.13	-
Proposed A1	1.95	1.23	0.51	0.21	0.24	1.21	0.57	0.43	-
Proposed EB Access Road	0.60	1.21	0.57	0.30	0.30	1.27	0.75	0.5	-
Proposed Trib Relignment	-	0.85	0.11	-0.24	-0.11	0.82	0.10	-0.20	-
Proposed Trib Access Road	0.6	1.1	0.47	0.14	0.18	1.1	0.46	0.13	-

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. This demonstrates that the proposed design will marginally reduce downstream flows. This is as would be expected with the construction of a new structure in the channel that slightly attenuates peak flows.



Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Data source	Dimensions	Modelling Approach
1 (XS06_06)	Bridge crossing the unnamed road on the west of A1 in Earsdon Burn.	Cross sectional data provided by the surveyor.	This bridge spans 10m. The bridge consists of three circular conduits with diameter 0.45m, 0.65, and 0.65m respectively. These dimensions have been taken directly from the data provided by surveyor.	The bridge has been modelled as a triple circular parallel culverts 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 (XS06_03)	Bridge crossing the unnamed road on the west of A1 in Earsdon Burn.	Cross sectional data provided by the surveyor.	This bridge spans 29m and is 5.8m wide. These dimensions have been taken directly from the data provided by surveyor.	The bridge 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.
3 (XS06_02)	Bridge on the Earsdon Burn crossing A1	Cross sectional data provided by the surveyor.	This bridge spans 32m and is 3.0m wide. These dimensions have been taken directly from the data provided by surveyor.	The bridge 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.
4 (XS07_01)	Footbridge in the field on the west of A1 in Earsdon Burn.	Cross sectional data provided by the surveyor.	This footbridge spans 2m. The bridge consists of single circular conduit of 0.75m diameter These dimensions have been taken directly from the data provided by surveyor.	The bridge 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 (XS06_06)	Proposed bridge on the Earsdon Burn crossing A1 alignment	Local channel dimensions derived from topo survey data provided by the surveyor.	The proposed structure is 45m in length. The structure is 3.0m wide and 2.1m height allowing for 0.15m silt in the bed.	The bridge 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 (XS06_05)	Proposed bridge on the Earsdon Burn crossing access road.	Local channel dimensions derived from topo survey data provided by the surveyor.	The proposed structure is 13m in length. The structure is 3.0m wide and 2.1m height allowing for 0.15m silt in the bed.	The bridge 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.
3 (XS07_Prop1)	Proposed culvert on the unnamed watercourse	Local channel dimensions derived from topo survey data provided by the surveyor and LIDAR data.	The proposed structure is 147m in length. The structure is a 1.6m diameter culvert.	The culvert has been modelled as a single circular culvert within the channel. The inlet assumes a headwall with square edge.
4 (XS07_2.5u)	Proposed access road culvert on the unnamed watercourse	Local channel dimensions derived from topo survey data provided by the surveyor and LIDAR data.	The proposed structure is 9m in length. The structure is a 1.6m diameter culvert.	The culvert has been modelled as a single circular culvert within the channel. The inlet assumes a headwall with square edge.

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
XS06_06 to XS06_03 Tortuosity: Low			
Left Bank	0.08	Grass/heavy scrub/Brambles	
Channel	0.04	Rocks/Silt/Gravel	
Right Bank	0.08	Grass/heavy scrub/Brambles	

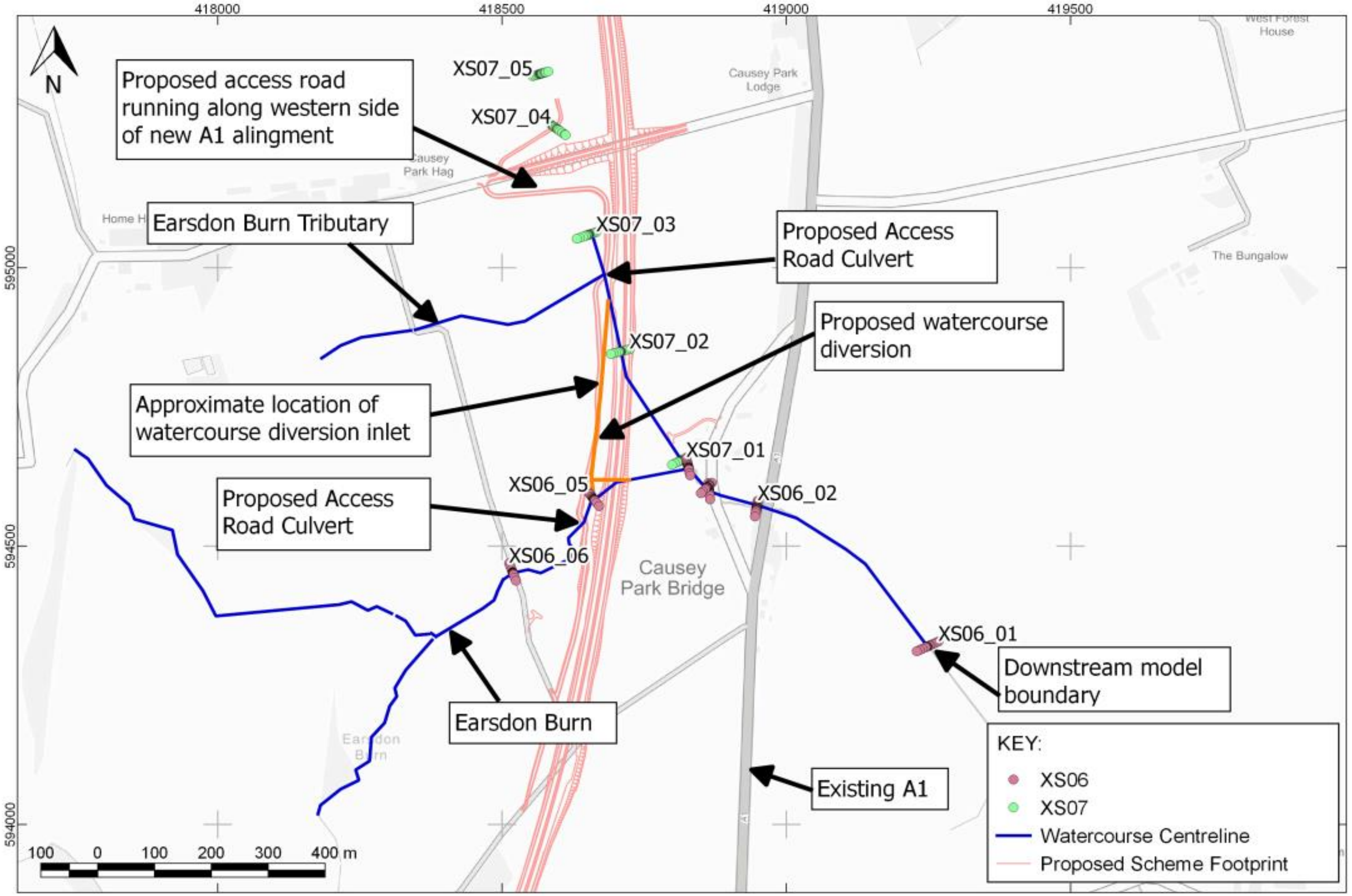
Reach	Manning's Roughness	Description of typical reach cover	Typical photo
XS07_03 to XS07_01 Tortuosity: Low			
Left Bank	0.08	Grass	
Channel	0.04	Silt/Gravels	
Right Bank	0.08	Grass/heavy scrub/trees	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result File
Baseline Scenario						
RL	Baseline	2yr	v4.4	A1_Northumberland_v08_2yr_Sensitivity.ief	A1_Northumberland_v08.DAT	A1_NORTHUMBERLAND_V08_2YR_SENSITIVITY.zzd
RL	Baseline	100yr + 25%	v4.4	A1_Northumberland_v08_100yr+25%_Sensitivity.ief	A1_Northumberland_v08.DAT	A1_NORTHUMBERLAND_V08_100YR+25%_SENSITIVITY.zzd
RL	Baseline	1000yr	v4.4	A1_Northumberland_v08_1000yr_Sensitivity.ief	A1_Northumberland_v08.DAT	A1_NORTHUMBERLAND_V08_1000YR_SENSITIVITY.zzd
Design Scenario						
RL	Proposed	2yr	v4.4	A1_Northumberland_v09_Design_v07_2yr_Sensitivity.ief	A1_Northumberland_v09_Design_v07.dat	A1_NORTHUMBERLAND_V09_DESIGN_V07_2YR_SENSITIVITY.zzd
RL	Proposed	100yr + 25%	v4.4	A1_Northumberland_v09_Design_v07_100yr+25%_Sensitivity.ief	A1_Northumberland_v09_Design_v07.dat	A1_NORTHUMBERLAND_V09_DESIGN_V07_100YR+25%_SENSITIVITY.zzd
RL	Proposed	1000yr	v4.4	A1_Northumberland_v9_Design_v07_1000yr_Sensitivity.ief	A1_Northumberland_v09_Design_v07.dat	A1_NORTHUMBERLAND_V9_DESIGN_V07_1000YR_SENSITIVITY.zzd
RL	Blockage	100yr + 25%	v4.4	A1_Earsdon_v09_Design_v07_100yr+25%_Sensitivity_A1_Blockage.ief	A1_Earsdon_v09_Design_v07_A1_Blockage.dat	A1_EARSDON_V09_DESIGN_V07_100YR+25%_SENSITIVITY_A1_BLOCKAGE.zzd
RL	Blockage	100yr + 25%	v4.4	A1_Earsdon_v09_Design_v07_100yr+25%_Sensitivity_EB_Access_Blockage.ief	A1_Earsdon_v09_Design_v07_EB_Access_Blockage.dat	A1_EARSDON_V09_DESIGN_V07_100YR+25%_SENSITIVITY_EB_ACCESS_BLOCKAGE.zzd
RL	Blockage	100yr + 25%	v4.4	A1_Earsdon_v09_Design_v07_100yr+25%_Sensitivity_Trib_Blockage.ief	A1_Earsdon_v09_Design_v07_Trib_Blockage.dat	A1_EARSDON_V09_DESIGN_V07_100YR+25%_SENSITIVITY_TRIB_BLOCKAGE.zzd
RL	Blockage	100yr + 25%	v4.4	A1_Earsdon_v09_Design_v07_100yr+25%_Sensitivity_Trib_Access_Blockage.ief	A1_Earsdon_v09_Design_v07_Trib_Access_Blockage.dat	A1_EARSDON_V09_DESIGN_V07_100YR+25%_SENSITIVITY_TRIB_ACCESS_BLOCKAGE.zzd

Appendix D. Model Schematics

Baseline Model (BSC)



Project	A1 - Northumberland
Job Number	70044136
Location	Bockenfield Bridge, Northumberland, England (417859, 597341)
Watercourse(s)	Longdike Burn

1. Objectives/Areas of interest

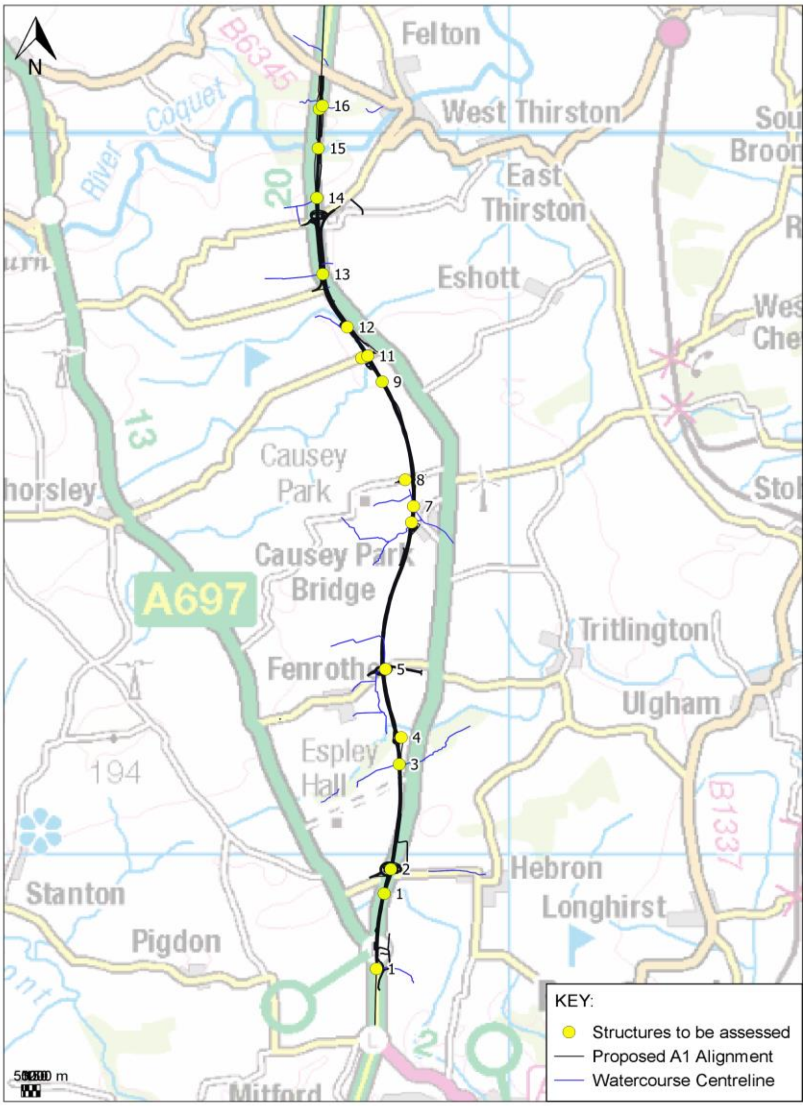


Figure 1: Location of structures in overall Scheme

The Scheme includes approximately 6.6 km of online widening and approximately 6 km of new offline highway. The existing carriageway would be widened on its current line up to Priest's Bridge, from where the proposed offline section of the Scheme would move west off the current road and pass west of Tindale Hill and Causey Park Bridge. Just north of Burgham Park, it would re-join the line of the existing carriageway and widening would continue along the existing road northwards, until it meets the existing dual carriageway north of Felton.

Figure 1 above shows the location of various structures along the Scheme. There are 16 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.

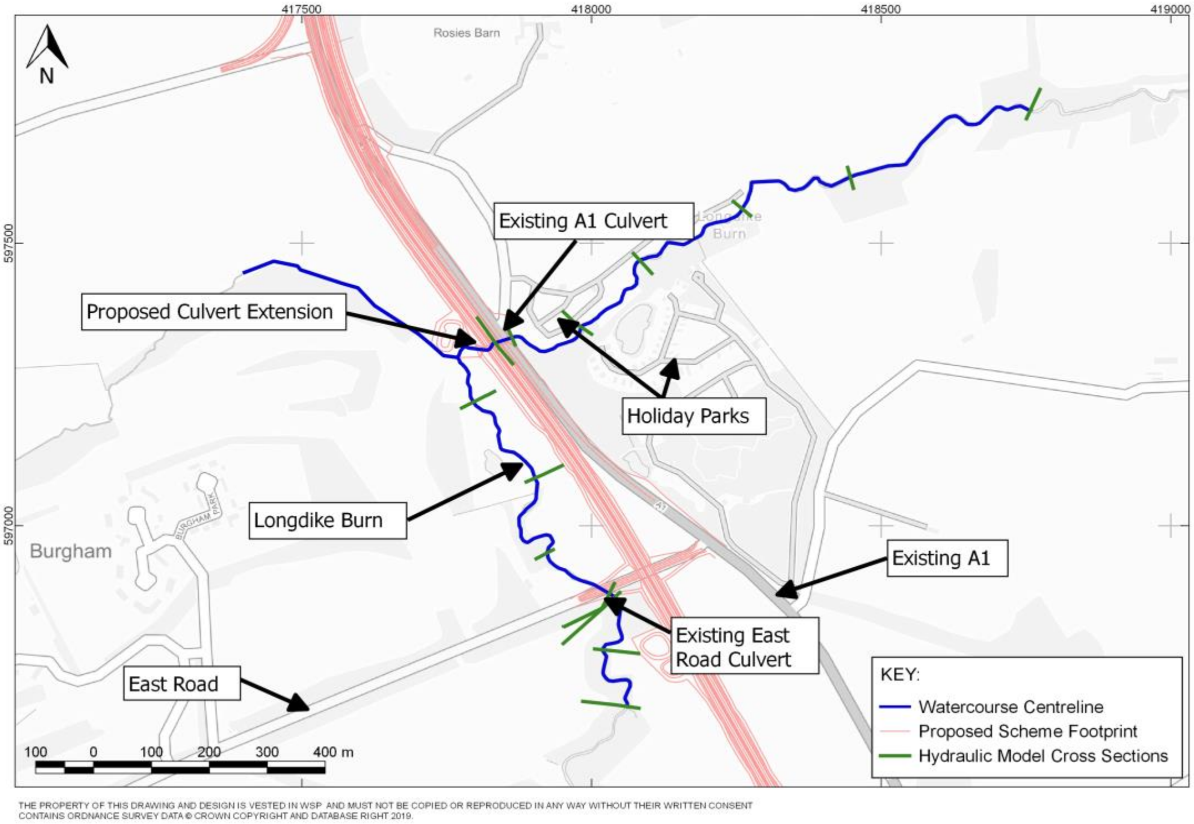


Figure 2: Location of structures on Longdike Burn

This report relates to the proposed works on the Longdike Burn watercourse. There are two existing structures within the reach. These include an existing culvert crossing beneath East Road which links the A1 to Burgham Park, and the existing A1 bridge near Bockenfield Holiday Park. 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
M2F XS10.xlsx M2F XS12.xlsx	Topographic Survey	Detailed topographic survey of area around the Longdike 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
LIDAR -	Resolution: Date :	1m - Downstream last three cross sections are covered LIDAR - Data downloaded from survey open data at data.gov.uk , Date flown 2015
Spot level grid -	Source	Topographic survey of proposed A1 corridor between Morpeth and Alnwick collected by Jabobs. Final issue January 2018

4. Baseline Model Development

The 1D hydraulic model has been developed in FMP using the topographic survey. The model extent is approximately 2.2km in length. The watercourse is Longdike Burn.

There are two existing structures within the reach. These structures were modelled in 1D domain only 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 accomodate the design flows. Local ground levels are available from 1m LIDAR tiles. These cover the entire extent of the model. In order to provide more detail in the area of interest further topographic survey of the area around the Bockenfield Holiday Park area has been provided. This survey includes channel, bank, bed, flood plain, existing bridge deck, existing bridge soffit, existing bridge parapet.

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

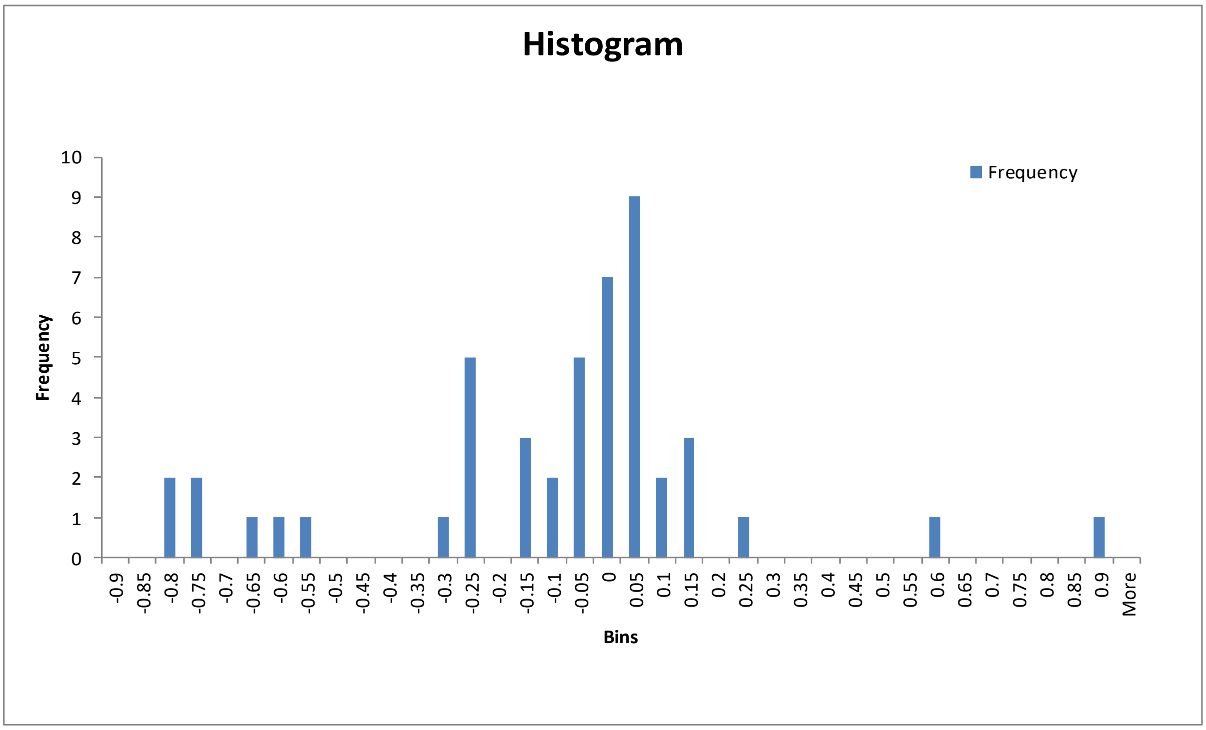


Figure 3: Survey Comparison

The plot shows a consistent shift between the LIDAR and survey data of approximatley 0.05m. On this basis the LIDAR data has been adjusted down by 0.05m globally before incorporation into the model.

5. Model Setup

Model Method	1D
Software	FMP (v4.4)
Channel	1D sections modelled using FMP.

Floodplain	Extended cross sections using LIDAR
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 4 locations for the Longdike Burn model. These are on Longdike Burn at the upstream of the model (LD01), on Longdike Burn upstream of an unnamed tributary from the north approximately 100m upstream of the A1 (LD_02), on the unnamed tributary immediately upstream of the confluence with Longdike Burn (LD_03), and at the downstream limit of the model downstream of the A1 (LD_04). 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%**
LD_01	9.64	12.77	15.05	18.31	21.17	23.06	24.53	28.56	40.75	30.66
LD_02	9.81	12.98	15.29	18.63	21.55	23.47	24.95	29.08	41.49	31.19
LD_03	1.78	2.37	2.81	3.42	3.96	4.31	4.59	5.35	7.67	5.74
LD_04	11.36	15.04	17.73	21.55	24.90	27.14	28.87	33.60	47.96	36.09

Flow hydrographs for the inflows have been developed using the ReFH methodology and a design storm duration derived from sensitivity testing of the critical duration in ReFH using catchment descriptors for the LD04 catchment. A comparison of the design hydrographs between all inflows demonstrates that the design flows at LD_04 agree closely with the peak flows from the combined hydrographs of LD_02 and LD_03. It can be inferred that the contributing catchment downstream of the A1 is minimal. Given the short length of the model it is considered reasonable to apply the design flows from LD_04 as a single inflow to the upstream limit of 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 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 works in this location are a widening of the existing A1 culvert only. The existing structure is 30m in length and is a sprung arch culvert 6.6m wide and 2.4m high. The width of the proposed A1 will result in an increased length of culvert of 34.5m.

10. Model Runs

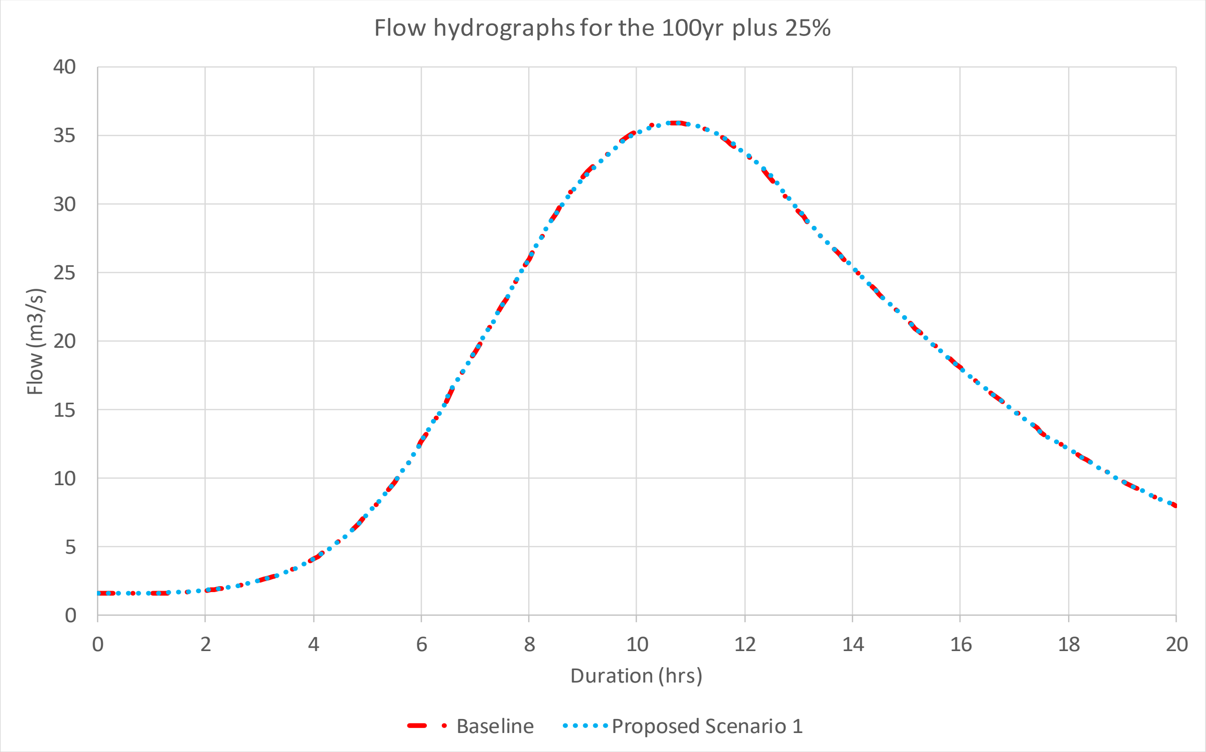
Model Scenario	**Return Periods / Events**
Baseline (BSC)	2yr, 100yr+25%, 1000yr
Proposed Scenario (PRO)	2yr, 100yr+25%, 1000yr, 100yr+25% with blockage

11. Model Results

The following table provides details of the freeboard associated with each structure for a range of flood events. Longdike Burn is classified as a main river and as such a design freeboard of 600mm is preferred in the 100yr+25% climate change event in accordance with DMRB. The 1000yr event is larger than the 100yr+50% climate change event so 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% reflecting the size of the proposed structures and hence the likelihood of blockage.

Structure	Carriageway Freeboard above Inlet Soffit (m)	Inlet Freeboard (m)				Outlet Freeboard (m)			
		2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage	2 year	100 year + 25 % cc	1000 year	100 year + 25 % cc with blockage
Existing Burgham Culvert (10.1)	0.7	1.97	0.38	0.17	-	2.19	0.94	0.69	-
Existing Bockenfield Culvert (12)	3.53	0.88	-0.57	-1.42	-	1.02	0.05	-0.22	-
Extended Bockenfield Culvert (12)	3.53	0.84	-0.78	-1.76	-1.80	1.02	0.06	-0.21	-

To understand the implications of flood risk dowsntream of the culvert, a flow hydrograph has been plotted at the downstream boundary of the model. This highlights the change in flows between the existing and proposed designs. The results show a reduction in downstream flows as a result of the proposals.


Flow hydrographs for the 100yr plus 25%
Flow (m3/s)
40
35
30
25
20
15
10
5
0
0
2
4
6
8
10
12
14
16
18
20
Duration (hrs)
Baseline
Proposed Scenario 1

Appendix A. Structures

Baseline Model (BSC)

Ref.	Description	Data source	Dimensions	Modelling Approach
1 (XS10_04)	Bridge on unnamed road crossing the Longdike Burn on the west of A1 near Burgham Park.	Cross sectional data provided by the surveyor.	The structure length is 30m. The bridge consists of single arch conduit. The dimensions have been taken directly from the data provided by the surveyor.	The bridge has been modelled as symmetrical 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_07)	Bockenfied bridge on A1 crossing the Longdike Burn near Bockenfield Holiday Park.	Cross sectional data provided by the surveyor.	The structure length is 30.6m. The bridge consists of single arch conduit. The dimensions have been taken directly from the data provided by the surveyor.	The bridge has been modelled as single arch 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)

2 extension (XS12_07)	Extension of existing Bockenfield bridge is proposed.	Cross sectional data provided by the surveyor.	The proposed extension changes the length of the bridge from 30m to 64m. The other dimensions of the bridge remains the same.	The bridge has been extended on the inlet side. The length of the culvert changed accordingly and other dimensions kept constant.

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**
XS10_06 to XS10_03 Tortuosity: Low			
Left Bank	0.08	Grass & Trees	
Channel	0.04	Stones and rocks	
Right Bank	0.08	Grass	

Reach	**Manning's Roughness**	**Description of typical reach cover**	**Typical photo**
XS10_02 to XS10_01 Tortuosity: Low			
Left Bank	0.06	Grass	
Channel	0.04	Rocks<100mm	
Right Bank	0.06	Trees & Grass	

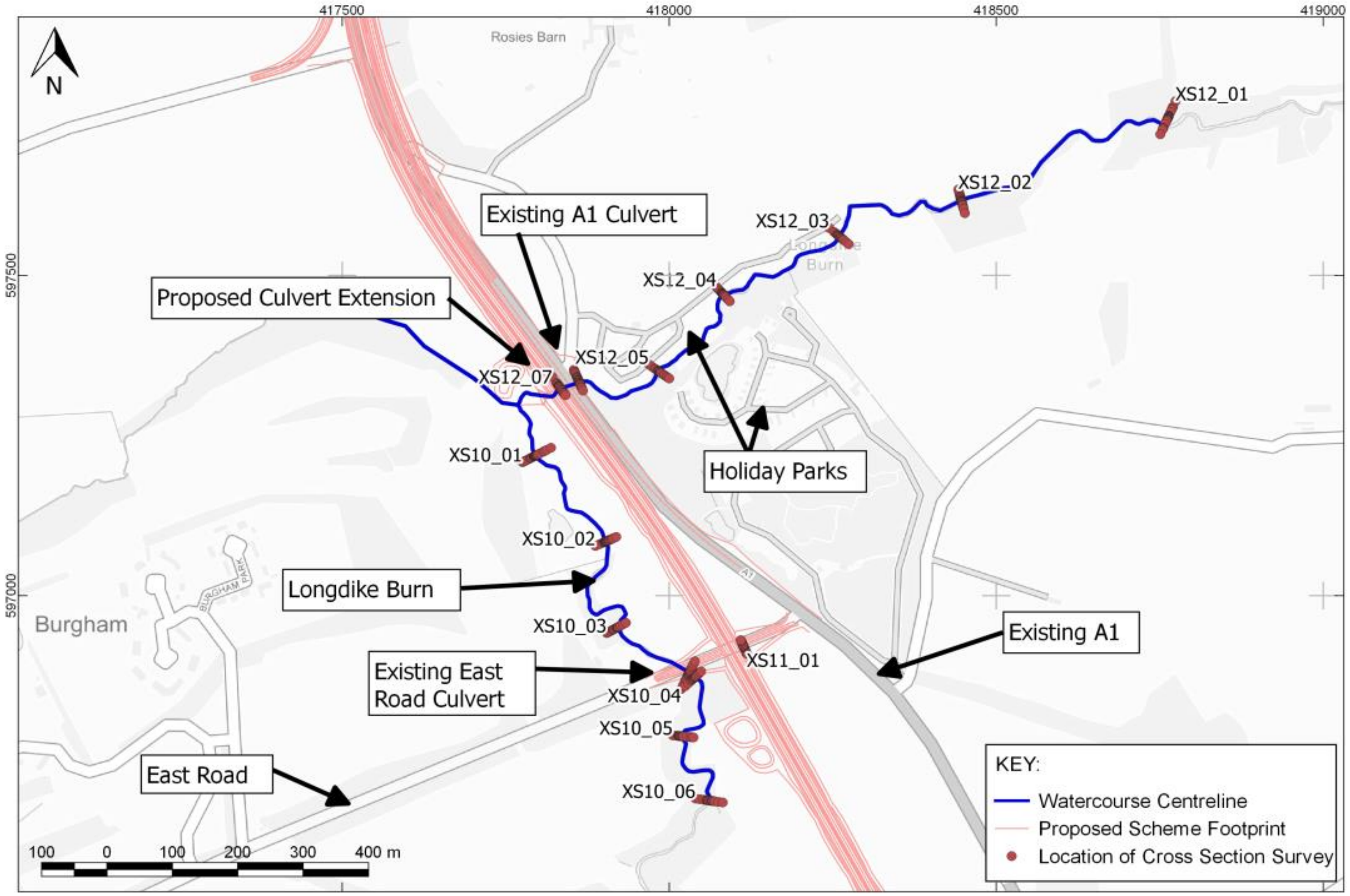
Reach	**Manning's Roughness**	**Description of typical reach cover**	**Typical photo**
XS12_07 to XS12_01 Tortuosity: Low			
Left Bank	0.08	Grass & Brambles	
Channel	0.04	Gravel & Rocks	
Right Bank	0.08	Grass & Trees	

Appendix C. Simulation Run List

Model Ref.	Scenarios (~S)	Flood Event (~E)	FMP build Number	ISIS Event file (.IEF)	ISIS file (.DAT)	Result File
Baseline Scenario						
LB	Baseline	2yr	v4.4	A1_Northumberland_Longdike Burn_v05_2yr_Sensitivty.ief	A1_Northumberland_Longdike Burn_v05.dat	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_2YR_SENSITIVTY.zzd
LB	Baseline	100yr + 25%	v4.4	A1_Northumberland_Longdike Burn_v05_100yr+25%_Sensitivty.ief	A1_Northumberland_Longdike Burn_v05.dat	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_100YR+25%_SENSITIVTY.zzd
LB	Baseline	1000yr	v4.4	A1_Northumberland_Longdike Burn_v05_1000yr_Sensitivty.ief	A1_Northumberland_Longdike Burn_v05.dat	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_1000YR_SENSITIVTY.zzd
Design Scenario						
LB	Proposed	2yr	v4.4	A1_Northumberland_Longdike Burn_v05_Design_v02_2yr_Sensitivity.ief	A1_Northumberland_Longdike Burn_v05_Design_v01.DAT	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_DESIGN_V01_2YR_SENSITIVITY.zzd
LB	Proposed	100yr + 25%	v4.4	A1_Northumberland_Longdike Burn_v05_Design_v02_100yr+25%_Sensitivity.ief	A1_Northumberland_Longdike Burn_v05_Design_v01.DAT	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_DESIGN_V01_100YR+25%_SENSITIVITY.zzd
LB	Proposed	1000yr	v4.4	A1_Northumberland_Longdike Burn_v05_Design_v02_1000yr_Sensitivity.ief	A1_Northumberland_Longdike Burn_v05_Design_v01.DAT	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_DESIGN_V01_1000YR_SENSITIVITY.zzd
LB	Blockage	100yr + 25%	v4.4	A1_Northumberland_Longdike Burn_v05_Design_Blockage_v01.ief	A1_Northumberland_Longdike Burn_v05_Design_Blockage_v01.DAT	A1_NORTHUMBERLAND_LONGDIKE BURN_V05_DESIGN_BLOCKAGE_V01.zzd

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:	[redacted]	Stephanie Haberfield MSc BSc (Hons)	
Calculations checked by:	[redacted]	Sam Willis MSc BSc CSci CEnv C.WEM	
Calculations approved by:	[redacted]	Sam Willis MSc BSc CSci CEnv C.WEM	

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

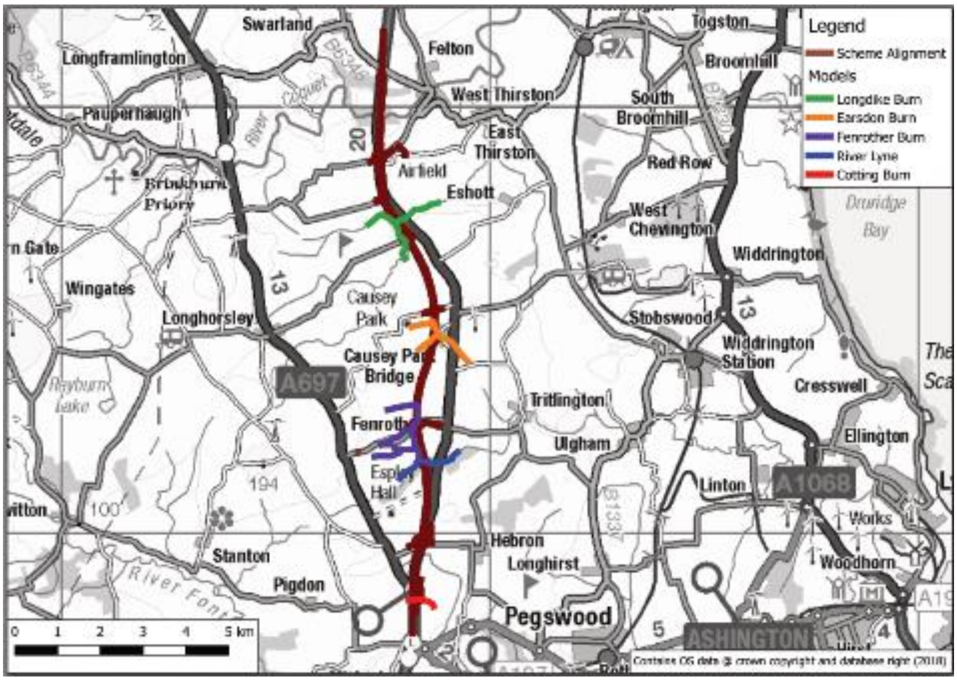
- Level 1 – Hydrologist with minimum approved experience in flood estimation
- Level 2 – Senior Hydrologist
- Level 3 – Senior Hydrologist with extensive experience of flood estimation

ABBREVIATIONS

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

1 Method statement

1.1 Overview of requirements for flood estimates

Item	Comments
Give an overview which includes: - Purpose of study - Approx. no. of flood estimates required - Peak flows or hydrographs? - Range of return periods and locations - Approx. time available	Highways England is proposing to provide additional capacity along the A1 with the A1 in Northumberland: Morpeth to Felton Scheme, hereafter referred to as ‘the Scheme’. The improvement works comprise approximately 6.6km online widening and approximately 6km of new offline highway. The existing carriageway will be widened on its current alignment up to Priest’s Bridge, from where the proposed offline section of the Scheme will move west of the current road and pass west of Tindale Hill and Causey Park Bridge. Just north of Burgham Park, it will re-join the line of the existing carriageway and widening will continue along the existing road northwards, until it meets the existing dual carriageway north of Felton. The Scheme alignment crosses or is located near to approximately 31 watercourses within 0.5km. Hydraulic modelling is required for five 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 five watercourses being assessed are: - Cotting Burn - River Lyne - Fenrother Burn - Earsdon Burn - Longdike Burn The other watercourses that the Scheme crosses have been assessed separately as part of the standalone Flood Risk Assessment (FRA). **Figure 1 Model Locations**  The objective of the study is to provide peak flow estimates and hydrographs for each watercourse and its tributaries. Peak flow estimates are required at 15 locations along the Scheme alignment as shown in Table 1 below. This

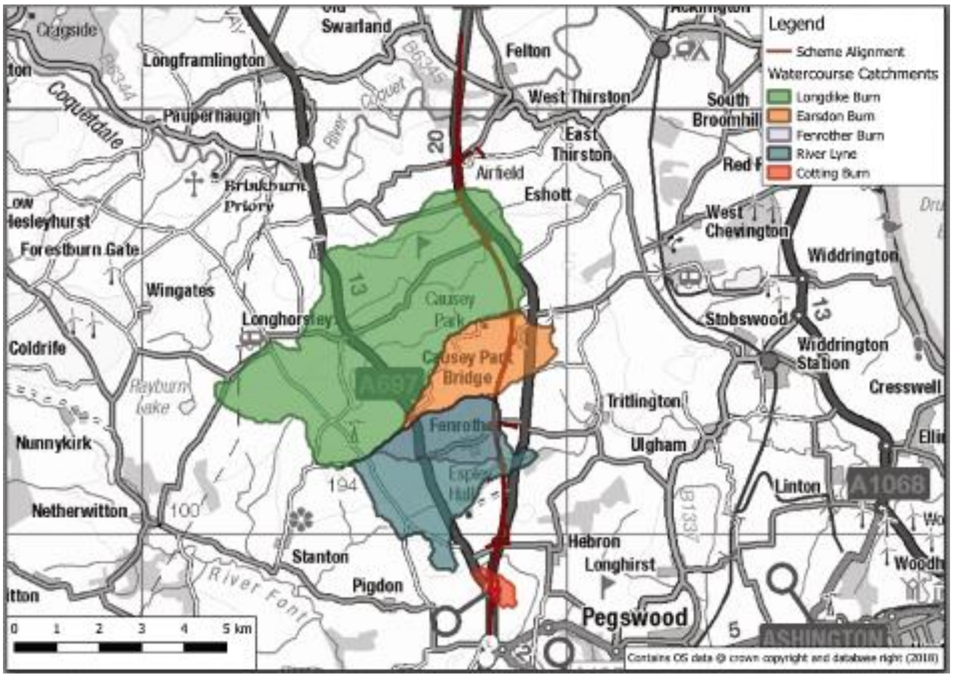
calculation record presents the estimates for all of these locations. The peak flow estimates for the River Lyne and Fenrother Burn have been grouped together as the Fenrother Burn discharges into the River Lyne, although there will be two hydraulic models.

Table 1 Flow Nodes

Flow Node	Watercourse
CB_01	Cotting Burn
CB_02	Cotting Burn
CB_03	Cotting Burn
RL_01	River Lyne
RL_02	River Lyne
FB_01	Fenrother Burn
FB_02	Fenrother Burn
EB_01	Earsdon Burn
EB_02	Earsdon Burn
EB_03	Earsdon Burn
LD_01	Longdike Burn
LD_02	Longdike Burn
LD_03	Longdike Burn
LD_04	Longdike Burn
LD_I	Longdike Burn

The following return period events were assessed: 2 year, 5 year, 10 year, 25 year, 50 year, 75 year, 100 year, 200 year, 500 year, 1000 year and 100 year plus 25% climate change allowance.

1.2 Overview of catchment

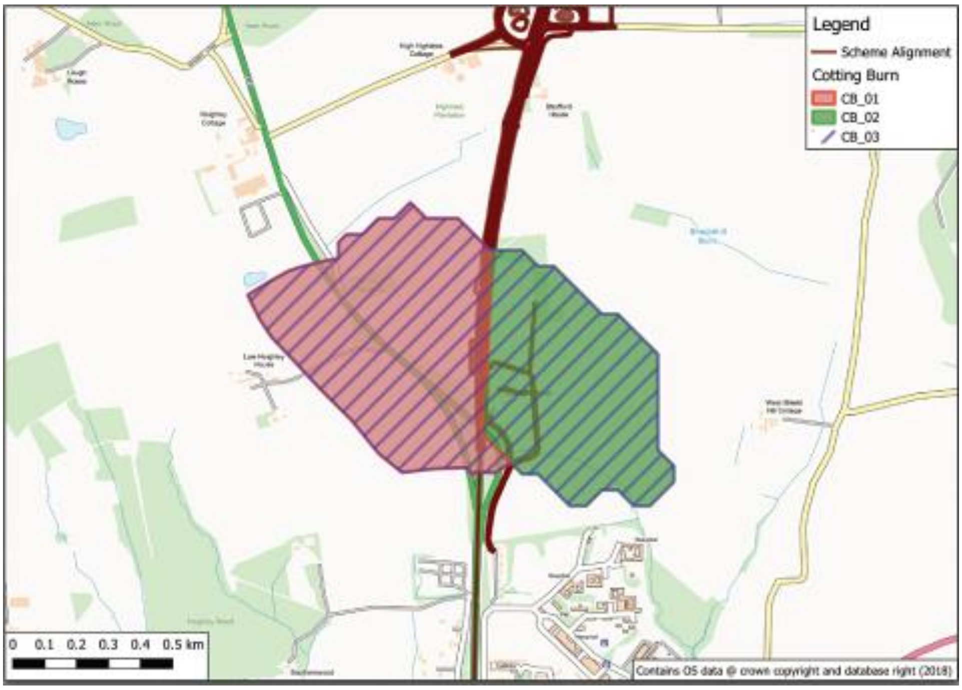
Item	Comments
Brief description of catchment, or reference to section in accompanying report	Figure 2 below shows the overall catchments for each of the watercourses that have been assessed. **Figure 2 Catchments** 

Cotting Burn

The total catchment area is approximately 0.75km². The catchment is predominately a rural catchment consisting of agricultural land, the existing A1 and the A697. It slopes from approximately 100m AOD in the north-west to approximately 80m AOD in the south-east. 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. Soil mapping indicates the catchment is underlain by slowly permeable slightly acid but base-rich loamy and clayey soil which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 3 shows the individual adjusted FEH catchments for Cotting Burn. These have been changed from the original FEH catchment to reflect the impact of the A1 alignment on the hydraulics of the catchment.

Figure 3 Cotting Burn Catchments



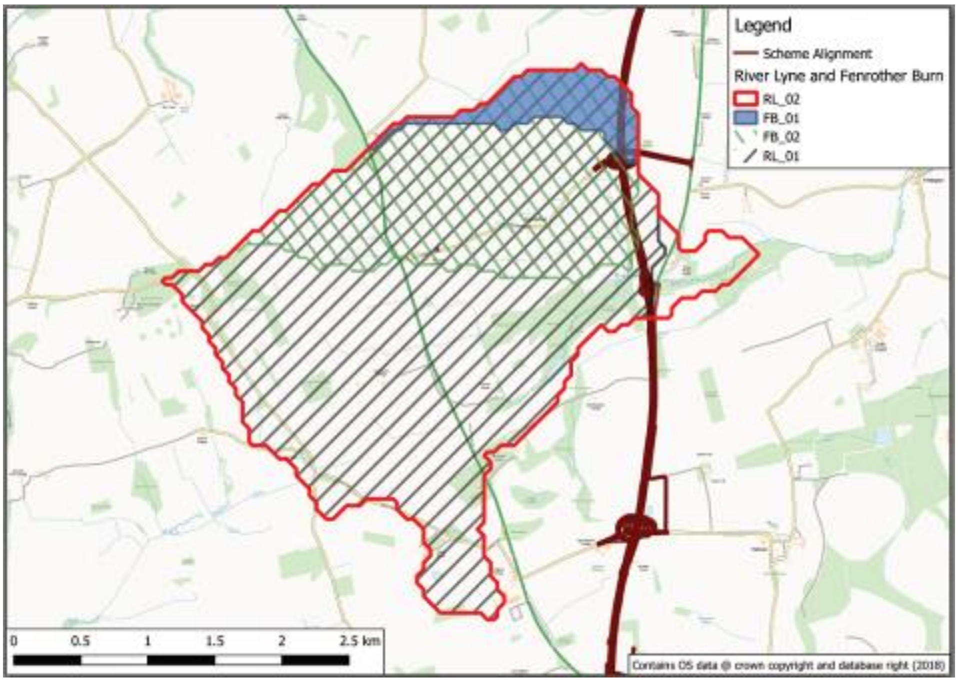
Contains OS data © crown copyright and database right (2018)

River Lyne and Fenrother Burn

The total catchment area is approximately 8.27km². The catchment is predominatly a rural catchment consisting of agricultural land. It slopes from approximately 180m AOD in the east to approximately 80m AOD in the west. There is one online lake located to the west of the existing A1 alignment upstream of the Woodcock Plantation. There is also a small pond located just to the west of Priest's Bridge.

The BGS hydrogeology information on the FEH web service shows the catchment geology to have moderate permeability. Soil mapping indicates the catchment is underlain by slowly permeable slightly acid but base-rich loamy and clayey soil which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 4 shows the individual FEH catchments for Fenrother Burn and the River Lyne.

Figure 4 Fenrother Burn and River Lyne Catchments

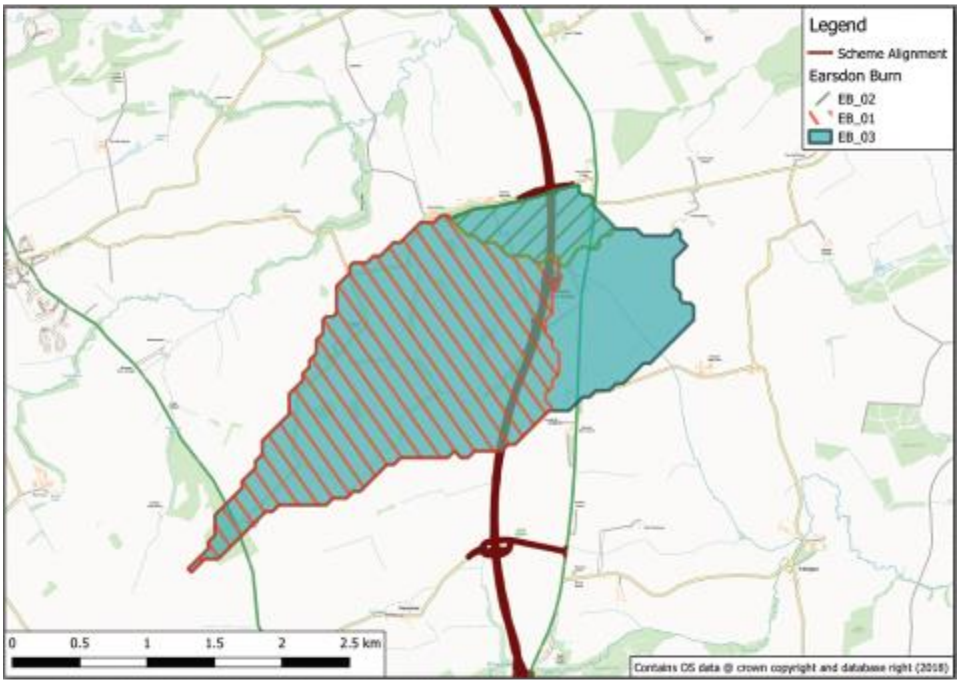


Earsdon Burn

The total catchment area is approximately 4.72km². The catchment is predominately a rural catchment consisting of agricultural land. It slopes from approximately 140m AOD in the west to approximately 75m AOD in the east. No lakes, reservoirs or artificial features have been identified from OS mapping. There is a small pond located to the south of Causey Park and another located to the south-east of Fieldhead.

The BGS hydrogeology information on the FEH web service shows the catchment geology to have moderate permeability. Soil mapping indicates the catchment is underlain by slowly permeable slightly acid but base-rich loamy and clayey soil which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 5 shows the individual adjusted FEH catchments for Earsdon Burn. The FEH catchments have been adjusted where the road cuts across the north of the catchment. This is explained in more detail in Section 2.3.

Figure 5 Earsdon Burn Catchments

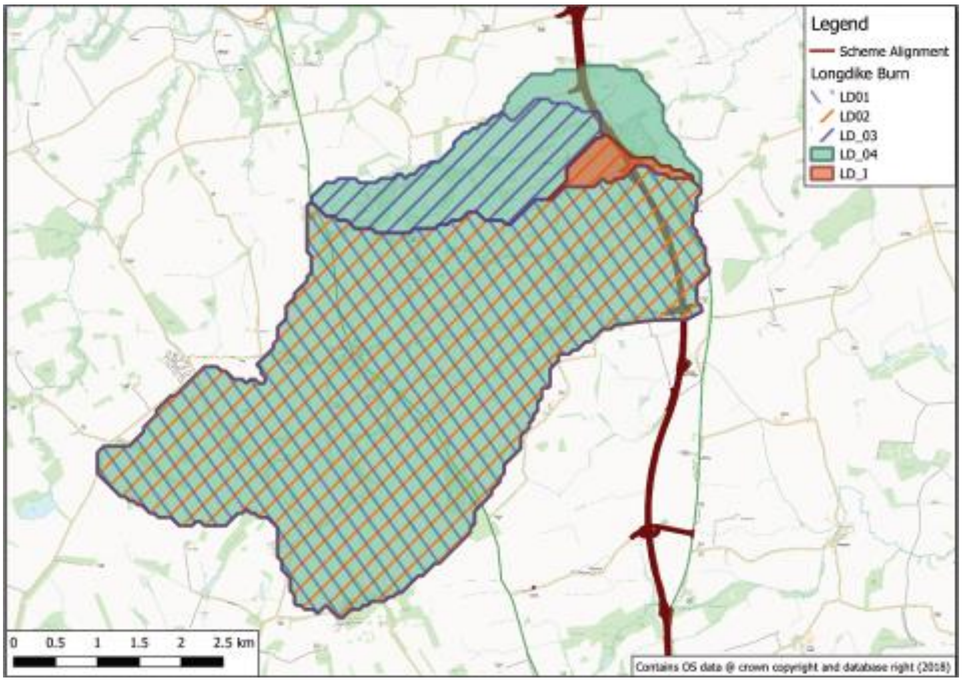


Longdike Burn

The total catchment area is approximately 23.03km². The catchment is predominately a rural catchment consisting of agricultural land and the small village of Longhorsley. It slopes from approximately 152m AOD in the west to approximately 60m AOD in the east. No lakes, reservoirs or artificial features have been identified from OS mapping. There are a number of small online ponds within the catchment located along Linden Burn and Bywell Letch which discharge into the Longdike Burn.

The BGS hydrogeology information on the FEH web service shows the catchment geology to have moderate permeability. Soil mapping indicates the catchment is underlain by slowly permeable slightly acid but base-rich loamy and clayey soil which corresponds with the BFIHOST and SPRHOST values obtained from the FEH. Figure 6 shows the individual FEH catchments for Longdike Burn.

Figure 6 Longdike Burn Catchments



All of the catchment areas have been assessed to ensure that they are appropriate and reflect the local topography. Some of the catchments marginally cross the A1, where this is the case local LiDAR data was used to check the suitability. As the areas of the catchments are not considered to be critical to the final outputs from the hydrualic modelling and culvert assessments, and considering that the calculated peak flows will be more conservative, no further changes to the catchment areas have been made.

1.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – Version 6, March 2018

1.4 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. The proposed works between Morpeth and Felton sit between the River Wansbeck to the South and the River Coquet to the north. These two watercourses have catchments in excess of 300km^2 in the region of the proposed works and drain the majority of the hills to the west. In comparison the catchments under investigation in this study drain the flat agricultural land between these watercourses that drains to the coastline, less than 10km to the east.

There are no gauging stations within the subject site catchment or on the downstream watercourse. The three nearest gauges to the site are located on either the River Wansbeck or the River Coquet and as such drain catchments significantly different to the subject site. These 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							

1.5 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						

1.6 Rating equations

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

1.7 Other data available and how it has been obtained

Type of data	**Data relevant to this study?**	**Data available?**	**Source of data and licence reference if from EA**	**Date obtained**	**Details**
Check flow gaugings (if planned to review ratings)					
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)					

1.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	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 five 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. Cotting Burn The Cotting Burn flows from west to east and flows underneath the existing A1 alignment and the A697 slip road. The Cotting Burn discharges into the River Wansbeck in Morpeth. The Scheme proposes to widen the existing A1 and the construction of local access roads to properties located immediately to the east of the A1, crossing the Cotting Burn. Flow estimates are required for the Cotting 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. River Lyne and Fenrother Burn The Fenrother Burn and its tributaries flow north-west to south-east and flow beneath Fenrother Lane. The Fenrother Burn discharges into the River Lyne just upstream of the existing A1 alignment. The River Lyne flows in a west to east direction and passes underneath the existing A1 alignment. The proposed offline section of the Scheme is located to the west of the existing A1. The new offline section will cross both the Fenrother Burn and River Lyne. It is also proposed to permanently realign the Fenrother Burn in order to minimise the length of culverting required. Flow estimates are required for the Fenrother Burn, its main tributary to the north, and for the River Lyne so that the implications of the new crossings can be assessed.

	Earsdon Burn The Earsdon Burn flows from west to east and passes beneath the existing A1 and a local side loop road. A small tributary joins the Earsdon Burn immediately upstream of the two existing roads. The proposed alignment of the new road is to the west of the existing roads and crosses both the Earsdon Burn and its small tributary. Flow estimates are required for the Earsdon Burn and the tributary separately so that the implications of the new road on both watercourses can be assessed. A final flow estimate is required downstream of the existing A1 to understand contributing flows downstream of the A1 and to resolve the flow contributions from both upstream watercourses. Longdike Burn The Longdike Burn flows south-west to north-east and flows underneath the existing A1 alignment through Bockenfield Bridge. Bywell Letch joins the Longdike Burn immediately upstream of the A1 crossing. The Longdike Burn also flows underneath an unnamed road upstream of where the watercourses join. The Scheme proposals are to widen the existing alignment of the A1, extending the existing culvert and to provide an overbridge where the Bywell Letch joins the Longdike Burn and an overbridge where the upstream culvert is located to go across the offline section of the A1. It is only proposed to improve the wingwall at this location. Flow estimates are required for the two Longdike Burn existing culverts and Bywell Letch to assess the implications of the culvert extension and wingwall improvements. A final flow estimate is required downstream of the Scheme to understand contributing flows downstream of the A1 and to resolve the flow contributions from both upstream watercourses.
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

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[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.

2.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH Web Service (km^2)	Revised AREA if altered
CB_01	Cotting Burn	Cotting Burn (upstream of existing A1 alignment)	417993	588752	-	0.42
CB_02	Cotting Burn	Cotting Burn (downstream of existing A1 alignment)	418517	588625	-	0.34
CB_03	Cotting Burn	Downstream extent of model (whole catchment)	418714	588284	0.51	0.75
FB_01	Fenrother Burn and tributaries	Fenrother Burn and tributaries	417816	592949	0.51	-
FB_02	Fenrother Burn	Fenrother Burn	418350	591750	3.04	-
RL_01	River Lyne	River Lyne	418500	591650	7.96	-
RL_02	River Lyne	Downstream extent of model (whole catchment)	419250	591900	8.27	-
EB_01	Earsdon Burn	Earsdon Burn	418750	594650	3.19	-
EB_02	Tributary of Earsdon Burn	Tributary of Earsdon Burn	418550	595350	0.52	0.41
EB_03	Earsdon Burn and tributary	Downstream extent of model (whole catchment)	419500	594000	4.2	4.58
LD_01	Longdike Burn	Longdike Burn	418050	596800	18.96	18.62
LD_02	Longdike Burn	Longdike Burn	417800	597200	18.29	19.03
LD_03	Bywell Letch	Bywell Letch	417750	597300	2.75	-
LD_04	Longdike Burn	Downstream extent of model (whole catchment)	418450	597600	23.37	23.03
LD_I	Longdike Burn	Incremental catchment between LD_01 and LD_02	417780	596945	-	0.41
Reasons for choosing above locations		DS model extents and location of main inflows to study watercourses.				

2.2 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	FPEXT
CB_01	1.00	0.45	0.312	0.62	32.40	718.00	39.70	0.0000	0.01
CB_02	1.00	0.45	0.312	0.55	32.40	718.00	39.70	0.0000	0.01
CB_03	1.00	0.45	0.312	0.85	32.40	718.00	39.70	0.0000	0.01
FB_01	1.00	0.45	0.312	0.94	21.50	732.00	39.70	0.0000	0.17
FB_02	1.00	0.45	0.312	1.89	34.60	740.00	39.70	0.0026	0.09
RL_01	0.98	0.45	0.312	2.54	36.90	740.00	39.70	0.0009	0.07

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
RL_02	0.98	0.45	0.312	3.36	36.80	740.00	39.70	0.0009	0.07
EB_01	1.00	0.45	0.312	1.83	31.60	731.00	39.70	0.0004	0.06
EB_02	1.00	0.45	0.312	0.71	29.20	718.00	39.70	0.0000	0.11
EB_03	1.00	0.45	0.312	2.34	34.20	727.00	39.70	0.0003	0.05
LD_01	1.00	0.45	0.313	4.97	41.10	742.00	39.63	0.0063	0.05
LD_02	1.00	0.45	0.312	5.02	41.00	741.00	39.63	0.0062	0.05
LD_03	1.00	0.45	0.313	2.07	35.60	722.00	39.70	0.0000	0.04
LD_04	1.00	0.45	0.313	5.58	39.40	737.00	39.63	0.0051	0.05
LD_I	1.00	0.45	0.313	0.61	36.46	695.59	39.630	0.0015	0.05

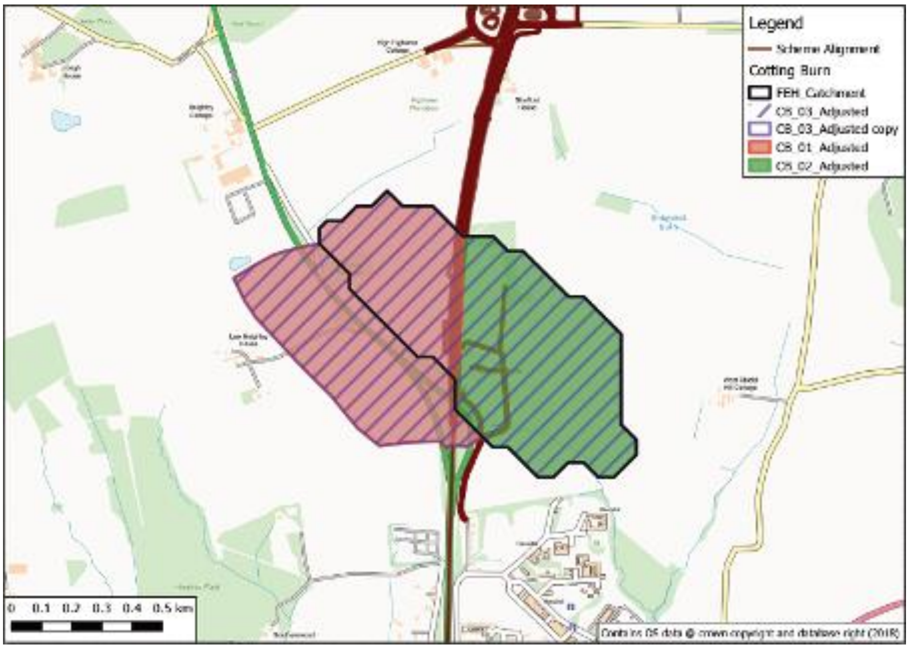
2.3 Checking catchment descriptors

<table>
<tr>
<td>Record how catchment boundary was checked and describe any changes (refer to maps if needed)</td>
<td>

The study site catchment boundaries for all of the hydraulic models were based on the closet 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.

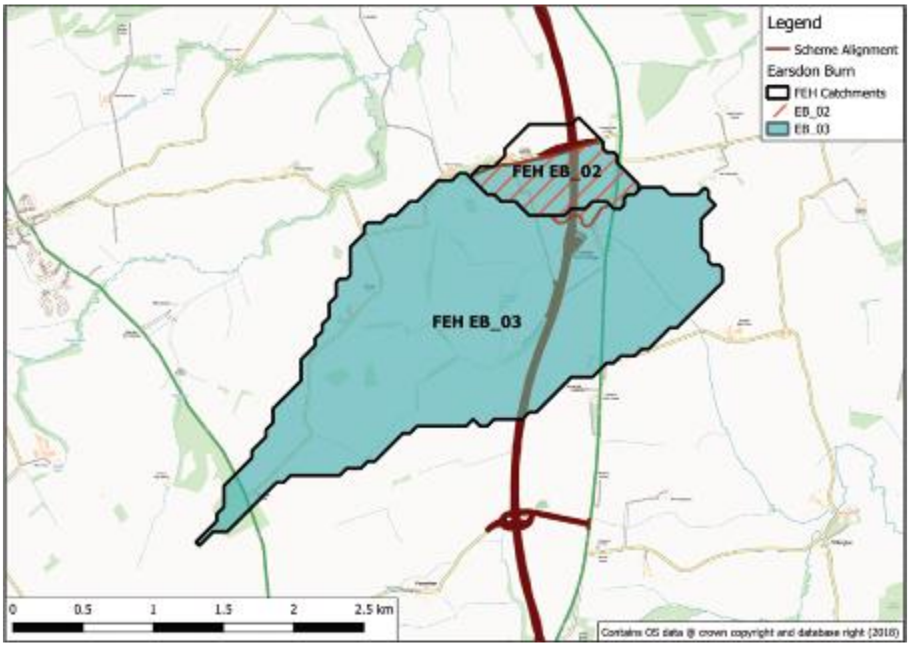
Figures 7 to 9 below show how the study catchment boundaries have been adjusted from the original FEH catchments. The Fenrother Burn and River Lyne catchments were not adjusted, the FEH catchments are shown in Figure 4 above in Section 1.2.

Figure 7 Cotting Burn Adjustments



The three Cotting Burn catchments were derived from and adjusted using one FEH catchment.

Figure 8 Earsdon Burn Adjustments



Topographic survey data collected for the project showed the tributary didn't extend upstream of the road to the north. As a result both EB_02 and EB_03 catchments were adjusted accordingly. EB_02 was also extended to the south to where the tributary discharges into the Earsdon Burn.

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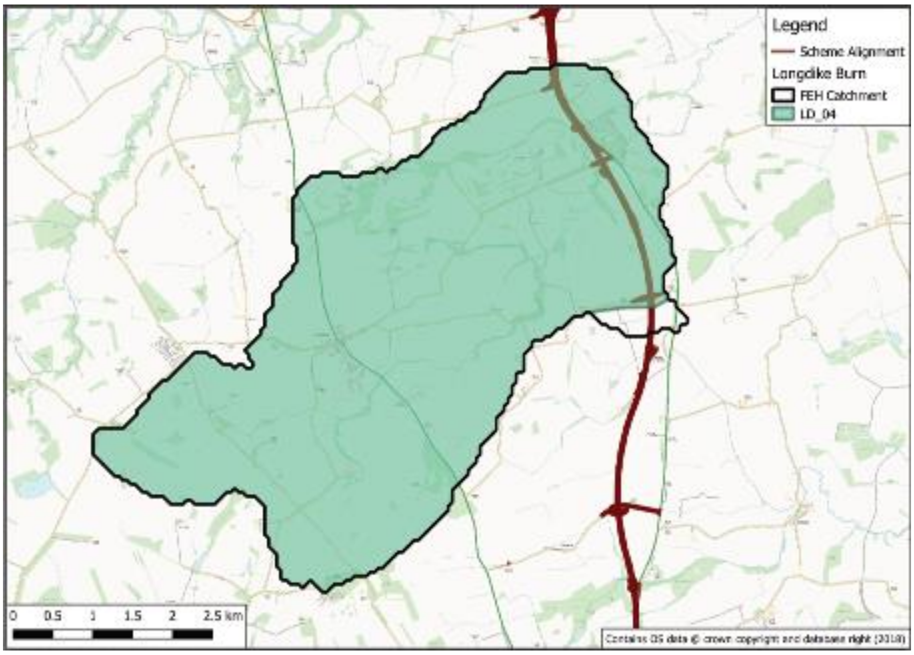
	Figure 9 Longdike Burn Adjustments  Three of the Londike Burn catchments were adjusted at the same location as shown in Figure 9 above. LD_01, LD_02 and LD_04 were all adjusted to the east of the catchments as this tributary was deemed to be draining to the Earsdon Burn catchment.
Record how other catchment 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: • BFIHOST and SPRHOST – Values adopted from the FEH catchments. FEH values were checked against soil mapping and appear to be reasonable. • FARL – One online pond was identified on OS mapping within the River Lyne catchment located upstream of Woodcock Plantation. No other online 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 completed using LiDAR data where available. The FEH values were deemed appropriate for the catchments. • DPLBAR – The FEH equation was used to calculate the DPLBAR for the adjusted catchments. FEH values were used for all of the other catchments.
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

3.1 Search for donor sites for QMED (if applicable)

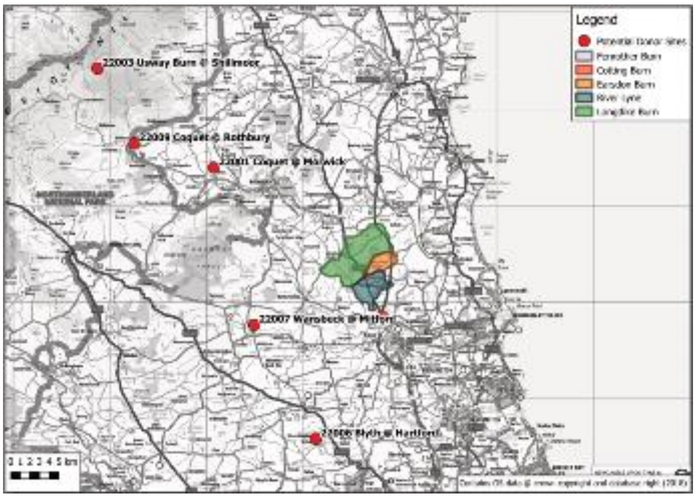
Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	Figure 10 below shows the location of each potential donor site that were considered for all the study watercourses. **Figure 10 Potential Donor Sites**  Table 2 below details the catchment descriptors for each of the potential donor sites.

Table 2 Potential Donor Sites

Station ID	Station Name	Area	BFIHOST	SPRHOST	FARL	URBEXT	PROPWET	SAAR (mm)	DPSBAR (m/km)	DPLBAR (km)
22007	Wansbeck @ Mitford	282.01	0.35	41.66	0.973	0.00	0.45	794	50.8	20.15
22006	Blyth @ Hartford	273.67	0.33	38.61	0.99	0.01	0.01	0.42	696.00	31.90
22001	Coquet @ Morwick	578.21	0.39	42.53	0.99	0.00	0.00	0.44	850.00	110.00
22009	Coquet @ Rothbury	345.99	0.40	45.50	0.99	0.00	0.00	0.45	905.00	140.70
22003	Usway Burn @ Shillmoor	21.88	0.30	56.92	1.00	0.00	0.00	0.45	1056.00	205.20

3.2 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)
22007	Rejected – reservoirs and abstractions affect flood flows through the catchment through drawdown and attenuation. Catchment area significantly larger than study sites.	AM	-	98.40	59.97	1.641

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)
22006	Rejected - Artificial influences including water bring diverted to reservoirs and public water supply abstraction. Surface water run-off increased by effluent returns. Catchment area significantly larger than study sites.	AM	-	52.55	45.83	1.137
22001	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	-	152.18	124.66	1.218
22009	Rejected - Recent gaugings suggested peak events may be underestimated. Catchment area significantly larger than study sites.	AM	-	133.00	91.93	1.444
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	14.701	1.100
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		

3.3 Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.2)	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	
CB_01	CD	0.22	N/A						0.22
CB_02	CD	0.18							0.18
CB_03	CD	0.35							0.35
FB_01	CD	0.27	N/A						0.27
FB_02	CD	1.26							1.26
RL_01	CD	2.64							2.64
RL_02	CD	2.74							2.74
EB_01	CD	1.27	N/A						1.27
EB_02	CD	0.21							0.21
EB_03	CD	1.71							1.71

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	
LD_01	CD	5.96	N/A						5.96
LD_02	CD	6.05							6.05
LD_03	CD	1.09							1.09
LD_04	CD	7.01							7.01
LD_I	CD	0.19							0.19

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.
Which version of the urban adjustment was used for QMED, and why?	WinFAP4 UAF values

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$^{\text{Error! Bookmark not defined.}}$ should be used. If the original FEH equation has been used, say so and give the reason why.

The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.

The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.2. 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.

3.4 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 (CB_03, RL_02, EB_03 and LD_04). 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. All of the sites marked not suitable for pooling were removed due to missing data, short records, lack of high flow gaugings and artificial influences on flows. 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)
CB_Pooling Group	CB_03	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.	L-CV – 0.218 L-skew – 0.262
FB_RL_Pooling Group	RL_02	No	All sites marked as unsuitable for pooling were removed based on a high level review. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data). Discordant site 49006 (Camel @ Camelford) was removed following high level review due to flat growth curve.	L-CV – 0.249 L-skew – 0.239
EB_Pooling Group	EB_03	No	All sites marked as unsuitable for pooling were removed based on a high level review. Two sites with a short record were also removed. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data). Discordant site 49006 (Camel @ Camelford) was removed following high level review due to flat growth curve.	L-CV – 0.234 L-skew – 0.261
LD_Pooling Group	LD_04	No	All sites marked as unsuitable for pooling were removed based on a high level review. Site 49005 (Bollingey Stream @ Bolingey Cocks Bridge) was removed as was a short record (6 years of data).	L-CV – 0.277 L-skew – 0.218

Notes

Pooling groups were derived using the revised procedures from Science Report SC050050 (2008).

The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.

3.5 Derivation of flood growth curves at subject sites

Site code	**Method** (SS, P, ESS, J)	**If P, ESS or J, name of pooling group (3.4)**	**Distribution used and reason for choice**	**Note any urban adjustment or permeable adjustment**	**Parameters of distribution** (location, scale and shape) after adjustments	**Growth factor for 100-year return period**
CB_03	P	CB_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.214 Shape – -0.262 Bound – 0.183	2.902
RL_02	P	FB_RL_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.250 Shape – -0.239 Bound – -0.047	3.092

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (3.4)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustments	Growth factor for 100-year return period
EB_03	P	EB_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.231 Shape – -0.261 Bound – 0.115	3.057
LD_04	P	LD_Pooling Group	GL – best fit	Winfap UAF	Location – 1.000 Scale – 0.284 Shape – -0.218 Bound – -0.300	3.243

Notes

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

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

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

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

3.6 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
CB_01	0.22	0.29	0.35	0.45	0.53	0.58	0.63	0.75	1.12
CB_02	0.18	0.25	0.30	0.37	0.44	0.49	0.52	0.62	0.93
CB_03	0.35	0.48	0.58	0.73	0.87	0.96	1.03	1.22	1.83
FB_01	0.27	0.38	0.46	0.59	0.70	0.77	0.83	0.98	1.45
FB_02	1.26	1.78	2.18	2.77	3.29	3.64	3.90	4.62	6.83
RL_01	2.64	3.73	4.55	5.79	6.89	7.61	8.17	9.68	14.29
RL_02	2.74	3.86	4.72	6.00	7.14	7.89	8.47	10.03	14.81
EB_01	1.27	1.77	2.15	2.73	3.26	3.62	3.89	4.64	7.00
EB_02	0.21	0.29	0.36	0.46	0.54	0.60	0.65	0.77	1.17
EB_03	1.71	2.37	2.88	3.67	4.38	4.86	5.22	6.23	9.40
LD_01	5.96	8.69	10.72	13.71	16.32	18.02	19.32	22.79	33.16
LD_02	6.05	8.82	10.88	13.91	16.56	18.29	19.61	23.14	33.66
LD_03	1.09	1.58	1.96	2.50	2.97	3.29	3.52	4.16	6.05
LD_04	7.01	10.2	12.6	16.1	19.2	21.2	22.7	26.8	39.0
LD_I	0.19	0.28	0.35	0.45	0.53	0.59	0.63	0.75	1.08

4 Revitalised flood hydrograph (ReFH) method 2.2

4.1 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
CB_01	CD	1.20	236.27	19.88	0.80
CB_02	CD	1.12	236.27	19.37	0.80
CB_03	CD	1.44	236.27	21.30	0.80
FB_01	CD	1.74	236.27	21.77	0.80
FB_02	CD	2.23	236.27	25.35	0.80
RL_01	CD	2.58	236.27	27.04	0.80
RL_02	CD	3.03	236.27	28.74	0.80
EB_01	CD	2.25	236.27	25.17	0.80
EB_02	CD	1.23	236.27	19.81	0.80
EB_03	CD	2.50	236.27	26.46	0.80
LD_01	CD	3.67	236.89	31.36	0.80
LD_02	CD	3.69	236.89	31.43	0.80
LD_03	CD	3.32	236.27	25.86	0.80
LD_04	CD	3.97	236.89	32.16	0.80
LD_I	CD	1.15	236.89	19.85	0.80

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

4.2 Design events for ReFH method

Site code	**Urban or rural**	**Season of design event (summer or winter)**	**Storm duration (hours)**	**Storm area for ARF** (if not catchment area)
CB_01	Urban	Winter	2.25	0.75
CB_02	Urban	Winter	2.25	0.75
CB_03	Urban	Winter	2.25	0.75
FB_01	Urban	Winter	5.5	8.27
FB_02	Urban	Winter	5.5	8.27
RL_01	Urban	Winter	5.5	8.27
RL_02	Urban	Winter	5.5	8.27
EB_01	Urban	Winter	4.5	4.58

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
EB_02	Urban	Winter	4.5	4.58
EB_03	Urban	Winter	4.5	4.58
LD_01	Urban	Winter	6.5	23.03
LD_02	Urban	Winter	6.5	23.03
LD_03	Urban	Winter	6.5	23.03
LD_04	Urban	Winter	6.5	23.03
LD_I	Urban	Winter	6.5	23.03
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 not likely to be changed. Storm duration, SCF and ARF based on the flow nodes CB_03, RL_02, EB_03 and LD_04, as they represent the whole catchment for each study site.	

In line with comments received from the Environment Agency's (EA) initial hydrology review, 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 ReFH 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)
CB_01	Urban	Winter	5.25	0.75
CB_02	Urban	Winter	5.25	0.75
CB_03	Urban	Winter	5.25	0.75
FB_01	Urban	Winter	7.1	8.27
FB_02	Urban	Winter	7.1	8.27
RL_01	Urban	Winter	7.1	8.27
RL_02	Urban	Winter	7.1	8.27
EB_01	Urban	Winter	6.1	4.58
EB_02	Urban	Winter	6.1	4.58
EB_03	Urban	Winter	6.1	4.58
LD_01	Urban	Winter	10.1	23.03
LD_02	Urban	Winter	10.1	23.03
LD_03	Urban	Winter	10.1	23.03
LD_04	Urban	Winter	10.1	23.03
LD_I	Urban	Winter	10.1	23.03

4.3 Flood estimates from the ReFH method

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
CB_01	0.33	0.48	0.59	0.74	0.87	0.96	1.02	1.20	1.75
CB_02	0.28	0.40	0.50	0.63	0.74	0.81	0.86	1.01	1.48
CB_03	0.51	0.75	0.92	1.16	1.36	1.49	1.59	1.86	2.72
FB_01	0.38	0.53	0.63	0.78	0.90	0.99	1.05	1.22	1.77

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	**5**	**10**	**25**	**50**	**75**	**100**	**200**	**1000**
FB_02	2.00	2.73	3.27	4.03	4.67	5.09	5.42	6.32	9.11
RL_01	4.78	6.52	7.80	9.60	11.14	12.16	12.94	15.07	21.72
RL_02	4.48	6.08	7.27	8.93	10.36	11.30	12.02	13.99	20.16
EB_01	1.95	2.71	3.26	4.03	4.68	5.10	5.42	6.31	9.11
EB_02	0.35	0.49	0.59	0.73	0.85	0.93	0.99	1.15	1.67
EB_03	2.62	3.62	4.35	5.37	6.23	6.79	7.22	8.40	12.09
LD_01	9.05	12.23	14.53	17.80	20.57	22.39	23.80	27.68	39.64
LD_02	9.20	12.44	14.78	18.11	20.93	22.78	24.21	28.16	40.32
LD_03	1.77	2.40	2.87	3.52	4.08	4.44	4.72	5.49	7.91
LD_04	10.58	14.28	16.96	20.74	23.99	26.12	27.76	32.23	46.15
LD_I	0.37	0.50	0.60	0.74	0.86	0.94	1.00	1.16	1.68

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**
CB_01	0.38	0.53	0.63	0.78	0.91	1.00	1.06	1.25	1.82
CB_02	0.32	0.44	0.53	0.65	0.76	0.83	0.89	1.04	1.51
CB_03	0.63	0.86	1.03	1.28	1.49	1.63	1.74	2.04	2.97
FB_01	0.39	0.53	0.63	0.77	0.90	0.98	1.04	1.21	1.75
FB_02	2.06	2.79	3.32	4.07	4.71	5.14	5.47	6.38	9.17
RL_01	5.00	6.72	8.00	9.80	11.35	12.39	13.19	15.38	22.13
RL_02	4.72	6.34	7.53	9.22	10.68	11.65	12.41	14.45	20.78
EB_01	2.12	2.90	3.46	4.26	4.94	5.38	5.73	6.67	9.62
EB_02	0.37	0.50	0.60	0.74	0.86	0.94	1.00	1.17	1.69
EB_03	2.87	3.91	4.65	5.72	6.63	7.23	7.69	8.97	12.93
LD_01	9.64	12.77	15.05	18.31	21.17	23.06	24.53	28.56	40.75
LD_02	9.81	12.98	15.29	18.63	21.55	23.47	24.95	29.08	41.49
LD_03	1.78	2.37	2.81	3.42	3.96	4.31	4.59	5.35	7.67
LD_04	11.36	15.04	17.73	21.55	24.90	27.14	28.87	33.60	47.96
LD_I	0.34	0.45	0.53	0.65	0.75	0.82	0.87	1.02	1.45

5 FEH rainfall-runoff method

5.1 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

5.2 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								

5.3 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

6.1 Comparison of results from different methods

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

Site code	**Ratio of peak flow to FEH Statistical peak**					
	Return period 2 years			**Return period 100 years**		
	ReFH	**Statistical**	**Ratio (ReFH / Statistical)**	**ReFH**	**Statistical**	**Ratio (ReFH / Statistical)**
CB_01	0.33	0.22	1.51	1.02	0.63	1.62
CB_02	0.28	0.18	1.53	0.86	0.52	1.64
CB_03	0.51	0.35	1.45	1.59	1.03	1.55
FB_01	0.38	0.27	1.43	1.05	0.83	1.27
FB_02	2.00	1.26	1.58	5.42	3.90	1.39
RL_01	4.78	2.64	1.81	12.94	8.17	1.58
RL_02	4.48	2.74	1.63	12.02	8.47	1.42
EB_01	1.95	1.27	1.53	5.42	3.89	1.39
EB_02	0.35	0.21	1.64	0.99	0.65	1.53
EB_03	2.62	1.71	1.53	7.22	5.22	1.38
LD_01	9.05	5.96	1.52	23.80	19.32	1.23
LD_02	9.20	6.05	1.52	24.21	19.61	1.23
LD_03	1.77	1.09	1.63	4.72	3.52	1.34
LD_04	10.58	7.01	1.51	27.76	22.73	1.22
LD_I	0.37	0.19	1.89	1.00	0.63	1.58

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The Statistical method peak flow estimates are low in comparison to the peak flow estimates produced by the ReFH 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 ReFH growth curves as shown in Figures 11 to 14 below indicates that the growth curves are relatively similar up to the up to the 100 year return period. Beyond this point there is a slight difference, with the Statistical growth curve slightly steeper than the ReFH growth curve. The main difference between the two methodologies are the QMED peak flow estimates.

Figure 11 Cotting Burn Growth Curves

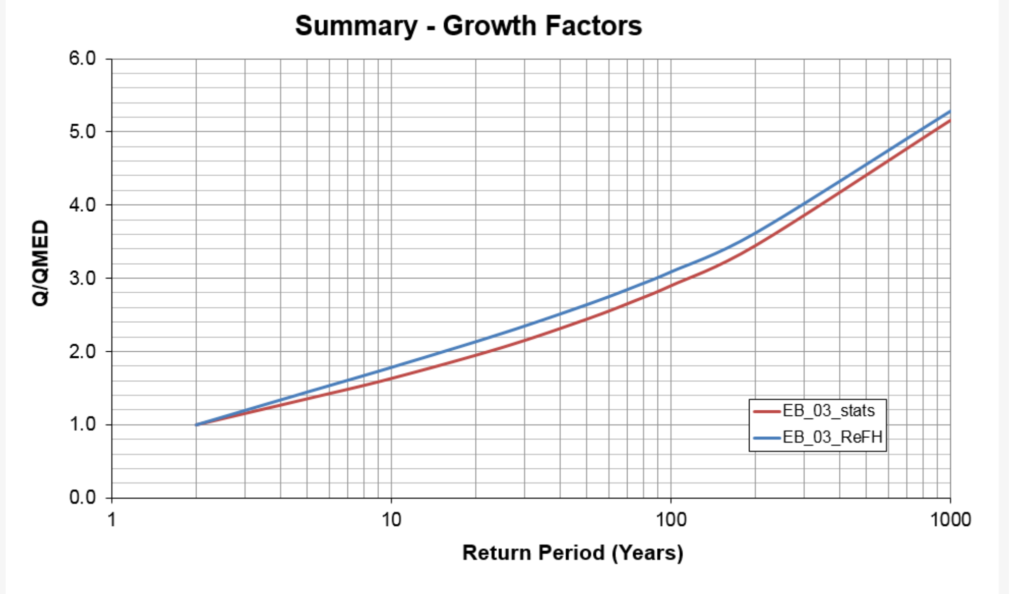


Figure 12 Fenrother Burn and River Lyne Growth Curves

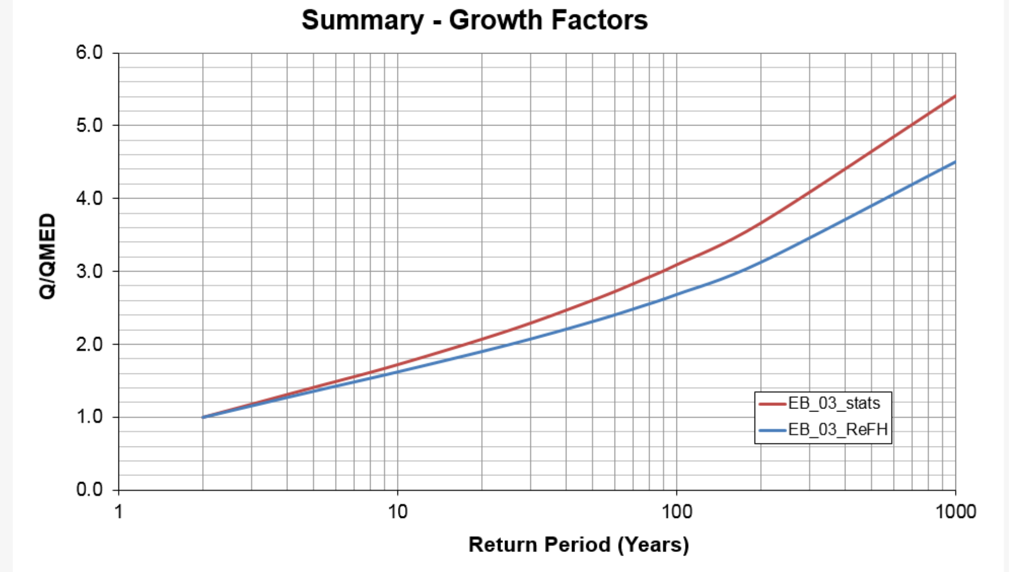


Figure 13 Earsdon Burn Growth Curves

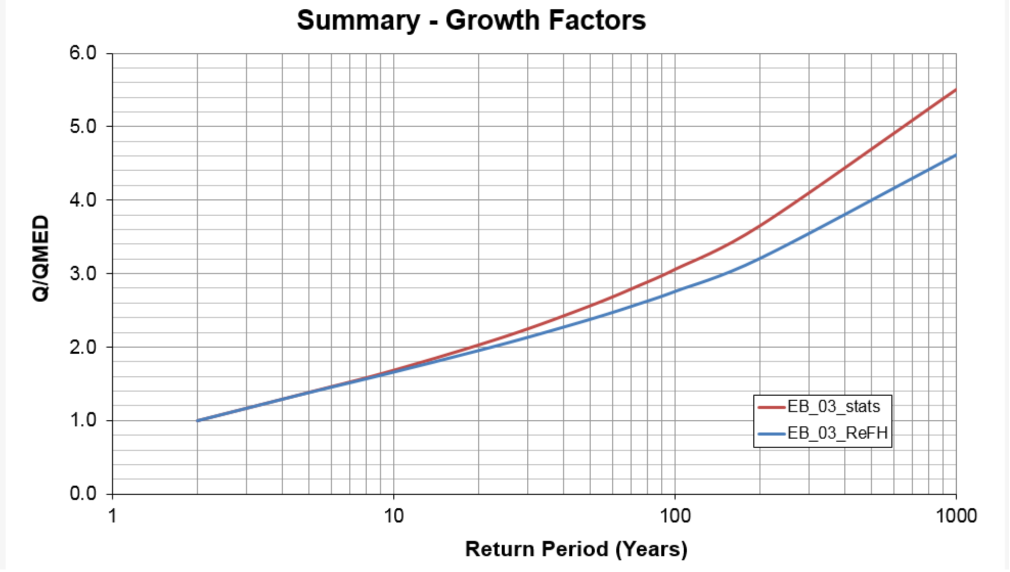
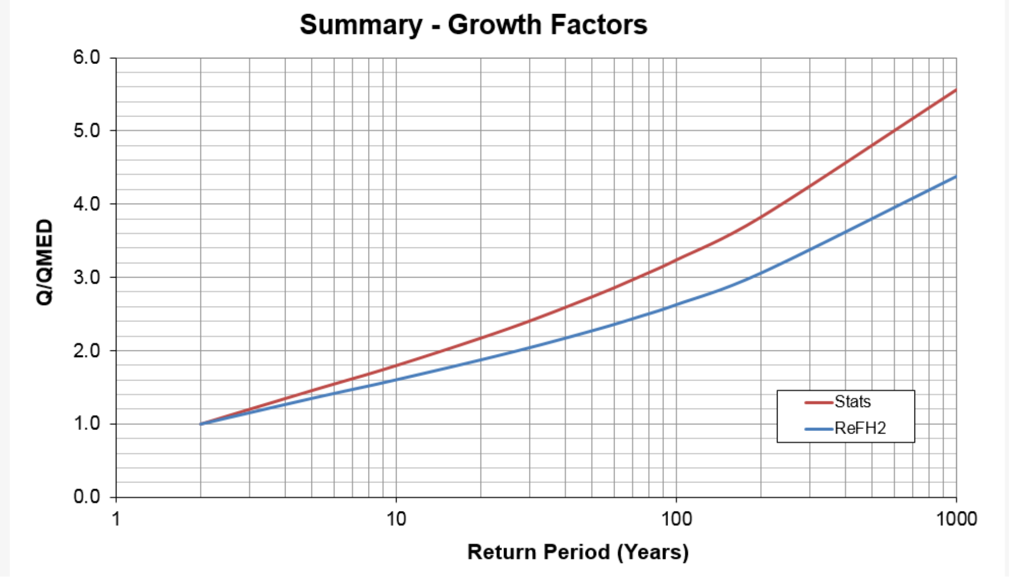


Figure 14 Longdike Burn Growth Curves



Local data collected as part of the topographic survey of each study site was also used to estimate QMED based on the Environment Agency's technical guidance[2] (2017). Table 3 below shows the QMED peak flow estimates calculated using FEH Local in comparison to the QMED peak flow estimates calculated using the Statistical method and ReFH method. The results show that the QMED values estimated using FEH Local are similar to the QMED values calculated by the ReFH2.2 method. The QMED calculated from the Statistical method is not being used as this assessment was high level and was undertaken to validate the preferred approach.

Table 3 Comparison of QMED

Watercourse	Flow Node	ReFH2.2 QMED (m^3/s)	Stats QMED (m^3/s)	Local Data QMED (m^3/s)
Cotting Burn	CB_03	0.51	0.35	0.50
Fenrother Burn and River Lyne	RL_02	4.44	2.74	3.95
Earsdon Burn	EB_03	2.62	1.71	2.46
Longdike Burn	LD_04	10.53	7.01	10.09

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 further away from the estimates calculated using FEH Local in comparison to the original ReFH2 QMED peak flow estimates.

Watercourse	Flow Node	Updated	Stats QMED	Local Data

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[2] Using local data to reduce uncertainty in flood frequency estimation (2017) Environment Agency

		ReFH2.2 QMED (m^3/s)	(m^3/s)	QMED (m^3/s)
Cotting Burn	CB_03	0.51	0.35	0.50
Fenrother Burn and River Lyne	RL_02	4.72	2.74	3.95
Earsdon Burn	EB_03	2.87	1.71	2.46
Longdike Burn	LD_04	11.36	7.01	10.09

Taking into consideration the difference in peak flow estimates produced between the Statistical method and ReFH method, the final peak flow estimates have been derived using the ReFH method. This is supported by the QMED calculations using FEH local data. The ReFH method has also been used for the higher return periods, based on the similarity between the growth curves produced from both methods. This potentially provides conservative flow estimates but is considered appropriate for flood risk assessment where a precautionary approach is advisable.

6.3 Assumptions, limitations and uncertainty

<table>
<tr><td>List the main assumptions made (specific to this study)</td><td>Standard FEH assumptions.</td></tr>
<tr><td>Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed</td><td>The estimated 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.</td></tr>
<tr><td>Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).</td><td>Detailed assessment outside of scope. Comparison of ReFH and Statistical method peak flow estimates provides some indication of the range of uncertainty. Environment Agency technical guidance[3] provides confidence intervals for design flows at ungauged rural sites. Table 5 below shows the confidence intervals for the QMED peak flow estimates for the downstream catchments for each watercourse based on no donor sites.

Table 5 Confidence Intervals
<table>
<tr><th rowspan="2">Watercourse</th><th colspan="2">68%</th><th colspan="2">95%</th></tr>
<tr><th>Lower</th><th>Upper</th><th>Lower</th><th>Upper</th></tr>
<tr><td>Cotting Burn</td><td>0.35</td><td>0.74</td><td>0.24</td><td>1.07</td></tr>
<tr><td>Fenrother Burn and River Lyne</td><td>3.06</td><td>6.44</td><td>2.11</td><td>9.24</td></tr>
<tr><td>Earsdon Burn</td><td>1.81</td><td>3.80</td><td>1.26</td><td>5.50</td></tr>
<tr><td>Longdike Burn</td><td>7.27</td><td>15.27</td><td>5.05</td><td>22.11</td></tr>
</table></td></tr>
<tr><td>Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.</td><td>N/A</td></tr>
<tr><td>Give any other comments on the study, for example suggestions for additional work.</td><td>Installation of a flow gauge (temporary or permanent) would help to verify and improve flow estimates for all of the study catchments.</td></tr>
</table>

---

[3] Using local data to reduce uncertainty in flood frequency estimation (2017) Environment Agency

6.4 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 5 100year Growth Factors
<table>
<tr><th>Watercourse</th><th>Flow Node</th><th>Growth Factor</th></tr>
<tr><td>Cotting Burn</td><td>CB_03</td><td>3.09</td></tr>
<tr><td>Fenrother Burn and River Lyne</td><td>RL_02</td><td>2.69</td></tr>
<tr><td>Earsdon Burn</td><td>EB_03</td><td>2.76</td></tr>
<tr><td>Longdike Burn</td><td>LD_04</td><td>2.62</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 6 Ratios
<table>
<tr><th>Watercourse</th><th>Ratio</th></tr>
<tr><td>Cotting Burn</td><td>1.71 – 1.72</td></tr>
<tr><td>Fenrother Burn and River Lyne</td><td>1.68</td></tr>
<tr><td>Earsdon Burn</td><td>1.67 – 1.69</td></tr>
<tr><td>Longdike Burn</td><td>1.66 – 1.68</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 7 shows the range of specific runoffs for the downstream flow nodes (CB_03, RL02, EB_03 and LD_04) for each study area.
Table 7 Specific Runoffs
<table>
<tr><th>Watercourse</th><th>2 Year (m3/s/km)</th><th>100 Year (m3/s/km)</th></tr>
<tr><td>Cotting Burn</td><td>0.69</td><td>2.12</td></tr>
<tr><td>Fenrother Burn and River Lyne</td><td>0.54</td><td>1.45</td></tr>
<tr><td>Earsdon Burn</td><td>0.57</td><td>1.58</td></tr>
<tr><td>Longdike Burn</td><td>0.46</td><td>1.21</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>

6.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
CB_01	0.33	0.48	0.59	0.74	0.87	0.96	1.02	1.20	1.75
CB_02	0.28	0.40	0.50	0.63	0.74	0.81	0.86	1.01	1.48

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	25	50	75	100	200	1000
CB_03	0.51	0.75	0.92	1.16	1.36	1.49	1.59	1.86	2.72
FB_01	0.38	0.53	0.63	0.78	0.90	0.99	1.05	1.22	1.77
FB_02	2.00	2.73	3.27	4.03	4.67	5.09	5.42	6.32	9.11
RL_01	4.78	6.52	7.80	9.60	11.14	12.16	12.94	15.07	21.72
RL_02	4.48	6.08	7.27	8.93	10.36	11.30	12.02	13.99	20.16
EB_01	1.95	2.71	3.26	4.03	4.68	5.10	5.42	6.31	9.11
EB_02	0.35	0.49	0.59	0.73	0.85	0.93	0.99	1.15	1.67
EB_03	2.62	3.62	4.35	5.37	6.23	6.79	7.22	8.40	12.09
LD_01	9.05	12.23	14.53	17.80	20.57	22.39	23.80	27.68	39.64
LD_02	9.20	12.44	14.78	18.11	20.93	22.78	24.21	28.16	40.32
LD_03	1.77	2.40	2.87	3.52	4.08	4.44	4.72	5.49	7.91
LD_04	10.58	14.28	16.96	20.74	23.99	26.12	27.76	32.23	46.15
LD_I	0.37	0.50	0.60	0.74	0.86	0.94	1.00	1.16	1.68

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.

7 Annex - supporting information

7.1 Pooling group compositions

Cotting Burn (CB_03)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	1.93	39	1.84	0.164	0.316	0.634
45816 (Haddeo @ Upton)	3.757	23	3.456	0.307	0.418	0.506
27051 (Crimple @ Burn Bridge)	3.891	44	4.539	0.223	0.156	0.186
28033 (Dove @ Hollinsclough)	4.018	37	4.2	0.237	0.418	0.595
91802 (Allt Leachdach @ Intake)	4.314	34	6.35	0.153	0.257	0.455
54022 (Severn @ Plynlimon Flume)	4.626	38	14.988	0.156	0.171	0.481
71003 (Croasdale Beck @ Croasdale Flume)	4.636	37	10.9	0.212	0.323	0.27
25011 (Langdon Beck @ Langdon)	4.704	28	15.878	0.238	0.318	0.48
25003 (Trout Beck @ Moor House)	4.735	43	15.164	0.17	0.288	0.378
25019 (Leven @ Easby)	4.742	38	5.333	0.338	0.391	1.929
47022 (Tory Brook @ Newnham Park)	4.762	23	7.123	0.262	0.115	1.722
206006 (Annalong @ Recorder)	4.958	48	15.33	0.189	0.052	2.435
27010 (Hodge Beck @ Bransdale Weir)	5.08	41	9.42	0.224	0.293	0.049
22003 (Usway Burn @ Shillmoor)	5.306	13	16.17	0.282	0.311	1.075
203046 (Rathmore Burn @ Rathmore Bridge)	5.379	34	10.788	0.146	0.136	0.793

River Lyne (RL_02)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
27051 (Crimple @ Burn Bridge)	0.741	44	4.539	0.223	0.156	0.162
25019 (Leven @ Easby)	1.067	38	5.333	0.338	0.391	0.541
45816 (Haddeo @ Upton)	1.208	23	3.456	0.307	0.418	0.638
28033 (Dove @ Hollinsclough)	1.369	37	4.2	0.237	0.418	1.229
27010 (Hodge Beck @ Bransdale Weir)	1.469	41	9.42	0.224	0.293	0.629
47022 (Tory Brook @ Newnham Park)	1.524	23	7.123	0.262	0.115	0.156
203046 (Rathmore Burn @ Rathmore Bridge)	1.551	34	10.788	0.146	0.136	1.062
25011 (Langdon Beck @ Langdon)	1.598	28	15.878	0.238	0.318	0.507
22003 (Usway Burn @ Shillmoor)	1.702	13	16.17	0.282	0.311	0.286
20002 (West Peffer Burn @ Luffness)	1.723	41	3.299	0.292	0.015	1.199
36010 (Bumpstead Brook @ Broad Green)	1.774	49	7.585	0.365	0.173	1.834
206006 (Annalong @ Recorder)	1.862	48	15.33	0.189	0.052	1.049
25003 (Trout Beck @ Moor House)	1.902	43	15.164	0.17	0.288	0.8
71003 (Croasdale Beck @ Croasdale Flume)	1.934	37	10.9	0.212	0.323	0.457

Earsdon Burn (EB_03)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
27051 (Crimple @ Burn Bridge)	0.914	44	4.539	0.223	0.156	0.21
45816 (Haddeo @ Upton)	1.175	23	3.456	0.307	0.418	0.479
28033 (Dove @ Hollinsclough)	1.455	37	4.2	0.237	0.418	0.839
25019 (Leven @ Easby)	1.675	38	5.333	0.338	0.391	0.457
76011 (Coal Burn @ Coalburn)	1.704	39	1.84	0.164	0.316	1.721
25011 (Langdon Beck @ Langdon)	1.972	28	15.878	0.238	0.318	0.426
47022 (Tory Brook @ Newnham Park)	1.982	23	7.123	0.262	0.115	0.238
27010 (Hodge Beck @ Bransdale Weir)	2.065	41	9.42	0.224	0.293	0.426
71003 (Croasdale Beck @ Croasdale Flume)	2.178	37	10.9	0.212	0.323	0.669
25003 (Trout Beck @ Moor House)	2.217	43	15.164	0.17	0.288	0.411
206006 (Annalong @ Recorder)	2.277	48	15.33	0.189	0.052	2.45
203046 (Rathmore Burn @ Rathmore Bridge)	2.305	34	10.788	0.146	0.136	0.882

22003 (Usway Burn @ Shillmoor)	2.307	13	16.17	0.282	0.311	1.171
91802 (Allt Leachdach @ Intake)	2.496	34	6.35	0.153	0.257	0.672
36010 (Bumpstead Brook @ Broad Green)	2.5	49	7.585	0.365	0.173	2.255

Longdike Burn (LD_04)

Station	Distance Value	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
36010 (Bumpstead Brook @ Broad Green)	0.502	47	7.5	0.375	0.186	1.493
26802 (Gypsey Race @ Kirby Grindalythe)	0.564	15	0.109	0.284	0.27	0.31
26803 (Water Forlornes @ Driffield)	0.607	15	0.437	0.288	0.146	1.083
203046 (Rathmore Burn @ Rathmore Bridge)	0.714	32	10.821	0.133	0.1	1.305
25019 (Leven @ Easby)	0.716	36	5.538	0.345	0.383	1.16
41020 (Bevern Stream @ Clappers Bridge)	0.759	45	13.66	0.21	0.189	0.721
44008 (South Winterbourne @ Winterbourne Steepleton)	0.74	35	0.448	0.414	0.336	1.424
27010 (Hodge Beck @ Bransdale Weir)	0.766	41	9.42	0.224	0.293	0.9
22003 (Usway Burn @ Shillmoor)	0.841	13	16.17	0.282	0.311	1.116
28058 (Henmore Brook @ Ashbourne)	0.848	12	9.006	0.155	-0.064	1.701
44013 (Piddle @ Little Puddle)	0.887	23	1.103	0.463	0.254	1.938
20002 (West Peffer Burn @ Luffness)	0.946	41	3.299	0.292	0.015	2.365
39033 (Winterbourne Stream @ Bagnor)	0.965	54	0.404	0.344	0.386	1.216
73015 (Keer @ High Keer Weir)	0.993	25	12.239	0.174	0.191	0.549
28041 (Hamps @ Waterhouses)	1.01	31	26.664	0.22	0.295	1.109
24007 (Browney @ Lanchester)	1.013	15	10.981	0.222	0.212	1.085
53017 (Boyd @ Bitton)	1.029	43	13.82	0.247	0.106	0.22

7.2 Additional supporting information

Appendix B

CULVERT MASTER ANALYSIS

[redacted]

MEMO

TO	James Hitching (Northumberland County Council)	**FROM**	Lee Bedford (WSP)
DATE	**22 March 2018**	**CONFIDENTIALITY**	[redacted]
SUBJECT	**A1 Improvement works – Hydrology for minor watercourse culvert assessments**		

OVERVIEW

This memo details the approach to the hydrological calculations to the support the assessment of the hydraulic capacity of a number of small culverts along the length of the proposed improvement works along the A1 in Northumberland (the Scheme). The objective of the work is to confirm the proposed culvert designs are appropriate to convey the design flows and that the implications of climate change are appropriately understood.

The culverts covered by this memo do not constitute all watercourse crossings along the length of the A1. The culverts are along the smaller watercourses and drains for which the hydraulic capacity calculations are a simple assessment of the structure and local channel only. The location of these culverts and estimated catchments are shown in Figure 1 below.

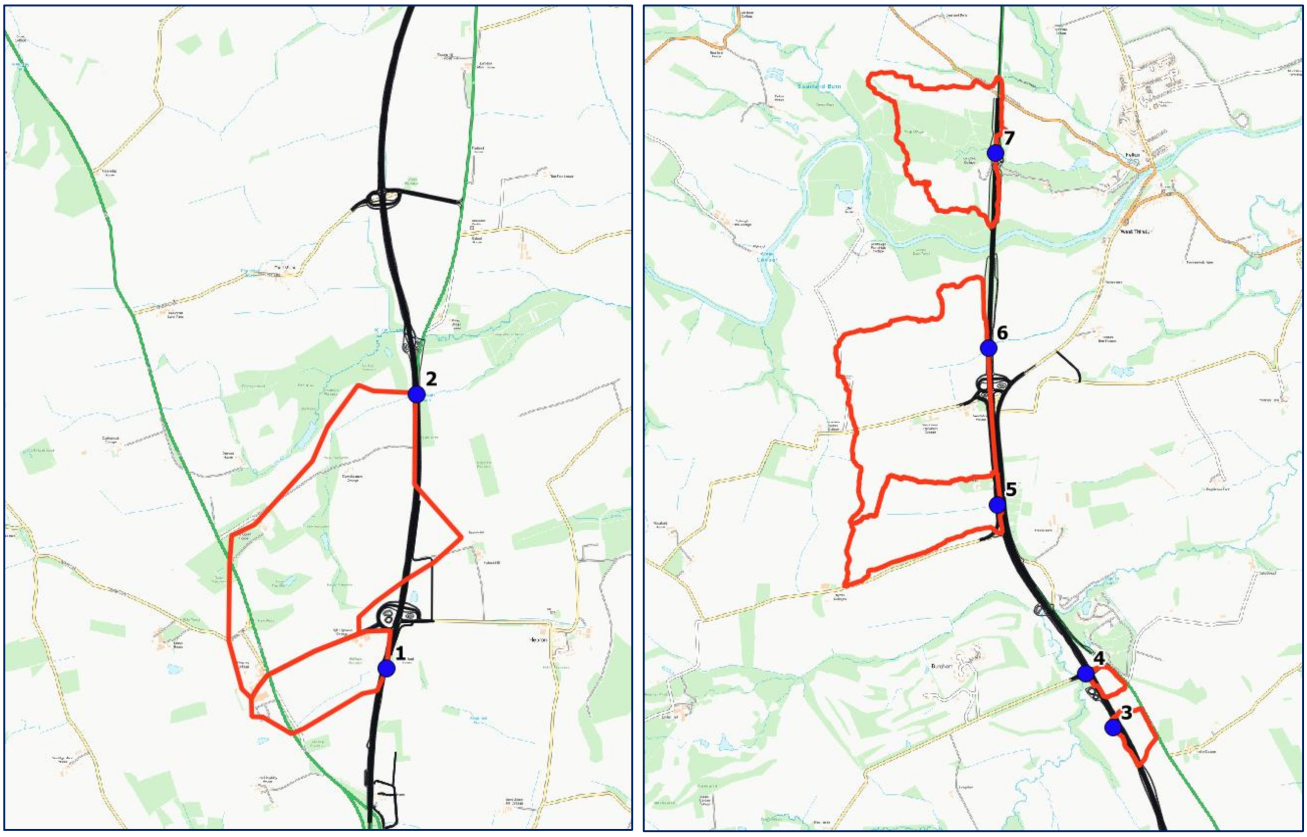


Figure 1 Overview of culvert locations and the estimated catchment areas

HYDROLOGY

The proposed approach is to use the ReFH2 method to determine design flows. This approach has been preferred over the statistical method given the scale of the catchments being assessed. The number of small catchments within the Hi-Flows dataset is limited and use of the statistical method at this scale can result in very similar pooling groups being derived for all sites.

Table 1 presents the estimated size of the catchments associated and the proposed ReFH2 approach, catchment or point scale, for each of the culverts. The catchment area for each of the culverts may change pending a site visit.

Table 1: Summary of ReFH2 Approach

Culvert Number	Catchment Area (km^2)	ReFH2 Method
1	0.35	Catchment
2	1.64	Catchment
3	0.06	Point
4	0.03	Point
5	0.44	Catchment
6	1.13	Catchment
7	0.56	Point

The ReFH2 hydrological model allows estimates of peak flow for catchment and plot scales and both approaches will be utilised. For the catchment scale method catchment descriptors will be extracted from the Flood Estimation Handbook web service. The small size of some of the catchments means that where FEH catchments are available, these are normally larger and contain the catchment of interest. In these instances the large catchments will be extracted and the catchment descriptors used and adjusted to reflect the smaller catchment area. The plot scale approach will be adopted when the size of the catchment is too small for appropriate FEH catchment descriptors to be derived. None of the catchments are urban therefore all catchments will be assessed within ReFH2 as rural.

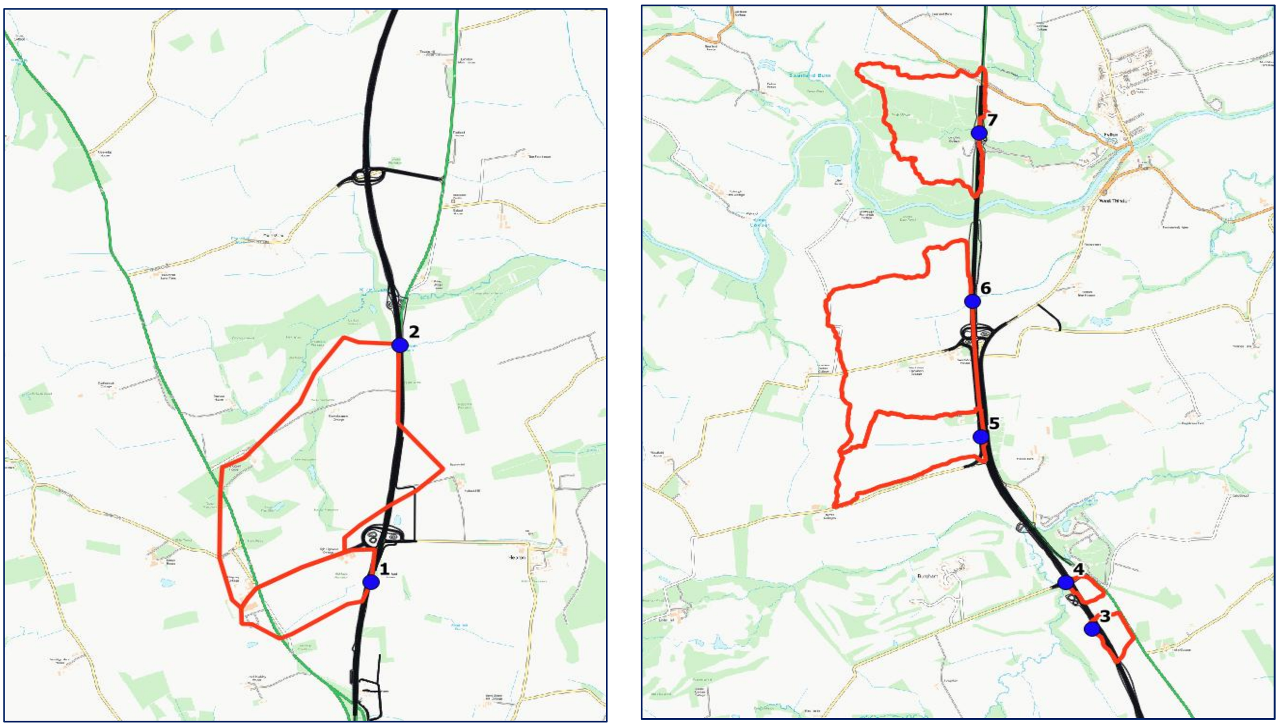
For the plot scale method (Culverts 3, 4 and 7), runoff has been calculated using the greenfield runoff method outlined in the ReFH2 technical guidance. For culverts with an area of less than 0.5km^2, the approach derives peak flows for a 0.5km^2 catchment and then scales according to area; for culvert 7, which has an area greater than 0.5km^2 its area has been used directly.

Flood hydrographs will not be used for the assessment. The objective of the assessment is to assess peak flow capacity and as such flood volume is not critical for this purpose. The climate change allowances for these culverts will be 25% based on discussions with the Environment Agency, recorded in minutes from a meeting on 9th January 2018.

Method:
Peak flows derived from ReFH2 software using the catchment or point descriptor data. Where point data is used the area has been updated to 0.5km2. The peak flows have then been scaled by the catchment area ratio based on the delineated culvert catchment area. BFIHOST parameters have been adjusted for culverts 14 and 16 based on soilscape data. This is considered conservative as it increases the estimated peak flows. All catchments are rural.

ReFH2 peak flows: \\uk.wspgroup.com\central data\Projects\700360xx\70036004 - A1 Northumberland\03 Environment\Water\10 Modelling\Hydrology\Culvert Hydrology\ReFH2

Culvert Locations and Catchment Areas



Catchment Descriptors (Aleration to the description values in brackets)

Flow Node ID	1A (1)	3 (2)	9 (3) point	11 (4) point	13 (5)	14 (6)	16 (7) point
Catchment X	418700	418500			417750	417450	
Catchment Y	589200	591300			597300	599100	
Centroid X	418107	417968			416204	416922	
Centroid Y	589391	590319			596827	598872	
AREA	0.503	1.623	0.500	0.500	2.748	0.640	0.500
ALTBAR	111	107			89	67	
ASPBAR	113	7			75	86	
ASPVAR	0.74	0.53			0.57	0.50	
BFIHOST	0.312	0.312	0.312	0.312	0.312	0.448 (0.312)	0.653 (0.312)
DPLBAR	0.81	1.48			2.07	0.78	
DPSBAR	30.3	29.1			35.6	14.5	
FARL	1.000	0.980			1.000	1.000	
FPEXT	0.08	0.05			0.04	0.17	
FPDBAR	0.816	0.471			0.379	0.773	
FPLOC	0.6	0.7			0.8	0.9	
LDP	1.7	2.8			4.4	1.7	
PROPWET	0.5	0.5	0.5	0.450	0.5	0.5	0.450
RMED-1H	9.2	9.1			9	8.9	
RMED-1D	34.2	34.5			34.7	34.8	
RMED-2D	43.30	43.10			44.30	44.10	
SAAR	724	731	708	707	722	701	701
SAAR4170	752	757			746	734	
SPRHOST	39.700	39.700			39.700	35.330	
URBCONC1990	-999999.000	-999999.000			-999999.000	-999999.000	
URBEXT1990	0.0000	0.0000			0.0000	0.0000	
URBLOC1990	-999999.000	-999999.000			-999999.000	-999999.000	
URBCONC2000	-999999.000	-999999.000			-999999.000	-999999.000	
URBEXT2000	0.0000	0.0008			0.0000	0.0000	
URBLOC2000	-999999.000	-999999.000			-999999.000	-999999.000	
C	-0.01900	-0.01900			-0.01893	-0.01900	
D1	0.41153	0.41466			0.41238	0.41822	
D2	0.37705	0.36889			0.39710	0.40388	
D3	0.267	0.274			0.260	0.266	
E	0.265	0.266			0.264	0.265	
F	2.339	2.339			2.333	2.321	
C(1 km)	-0.0190	-0.0190			-0.0190	-0.0190	
D1(1 km)	0.4130	0.4170			0.4080	0.4180	
D2(1 km)	0.3820	0.3650			0.3940	0.4040	
D3(1 km)	0.2630	0.2780			0.2670	0.2660	
E(1 km)	0.2650	0.2660			0.2650	0.2650	
F(1 km)	2.3320	2.3370			2.3330	2.3210	

Area adjustment ratio

Culvert Number	Descriptor Area	Catchment Area (km²)	ReFH2 Method	Ajustment for catchment ratio
1A (1)	0.5025	0.348	Catchment	**0.692**
3 (2)	1.6225	2.019	Catchment	**1.244**
9 (3) point	0.5	0.060	Point	**0.120**
11 (4) point	0.5	0.030	Point	**0.060**
13 (5)	2.7475	0.440	Catchment	**0.160**
14 (6)	0.64	1.130	Catchment	**1.766**
16 (7) point	0.5	0.560	Point	**1.120**

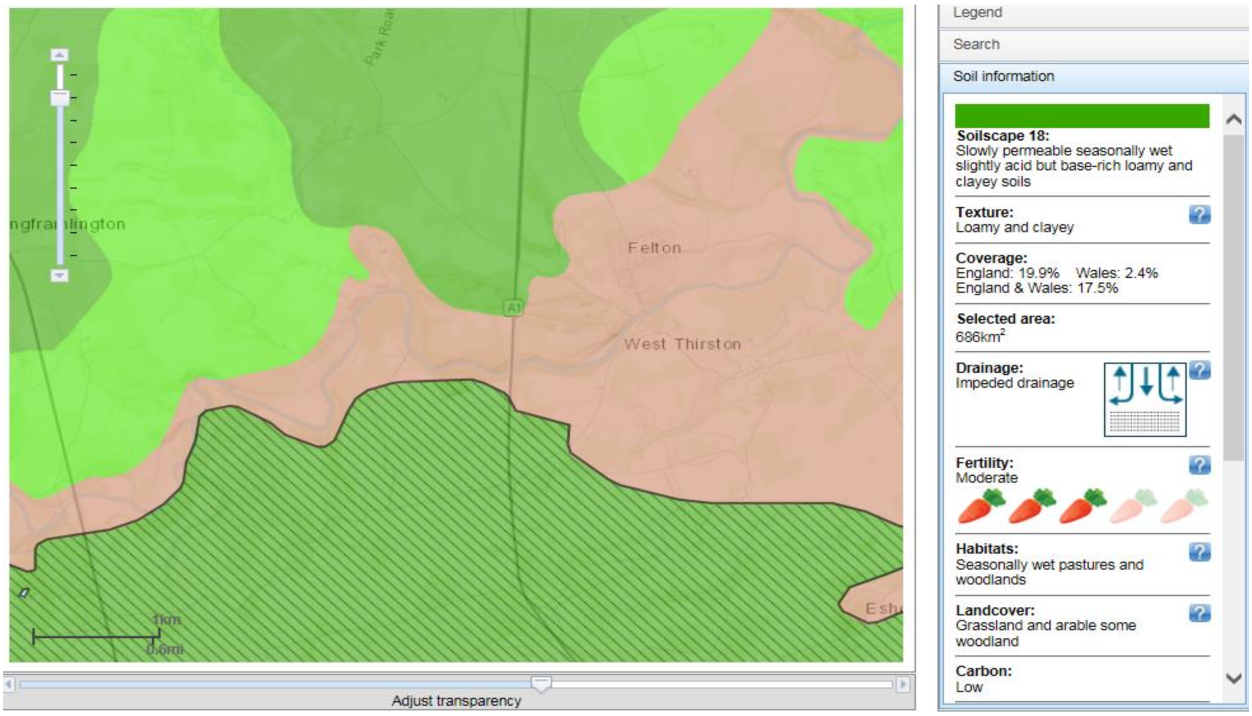
Peak Flows default FEH catchment area or 0.5km^2 for point (m^3/s)

RP	1A (1)	3 (2)	9 (3) point	11 (4) point	13 (5)	14 (6)	16 (7) point
2	0.350	1.007	0.346	0.346	1.663	0.419	0.340
5	0.508	1.411	0.509	0.509	2.312	0.602	0.504
10	0.626	1.707	0.629	0.628	2.784	0.734	0.624
30	0.825	2.215	0.831	0.830	3.588	0.957	0.825
50	0.928	2.483	0.934	0.933	4.000	1.072	0.927
75	1.017	2.711	1.021	1.021	4.359	1.171	1.013
100	1.085	2.884	1.087	1.086	4.635	1.245	1.079
200	1.269	3.364	1.262	1.261	5.379	1.445	1.251
1000	1.850	4.880	1.818	1.816	7.751	2.080	1.790

Final Peak Flows adjusted for area ratio (m^3/s)

RP	1A (1)	3 (2)	9 (3) point	11 (4) point	13 (5)	14 (6)	16 (7) point
2	0.425	1.163	0.422	0.413	1.808	0.482	0.419
5	0.583	1.578	0.581	0.575	2.457	0.664	0.580
10	0.700	1.884	0.697	0.692	2.940	0.799	0.697
30	0.902	2.409	0.896	0.894	3.755	1.027	0.896
50	1.010	2.688	0.999	0.997	4.183	1.145	0.999
75	1.104	2.935	1.089	1.088	4.556	1.247	1.089
100	1.177	3.127	1.159	1.157	4.843	1.362	1.157
200	1.377	3.659	1.350	1.347	5.638	1.542	1.345
1000	2.004	5.288	1.953	1.949	8.129	2.225	1.939

Soil Type Culvert 14



Soilscape 18:
Slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils

Texture:
Loamy and clayey

Coverage:
England: 19.9% Wales: 2.4%
England & Wales: 17.5%

Selected area:
686km^2

Drainage:
Impeded drainage

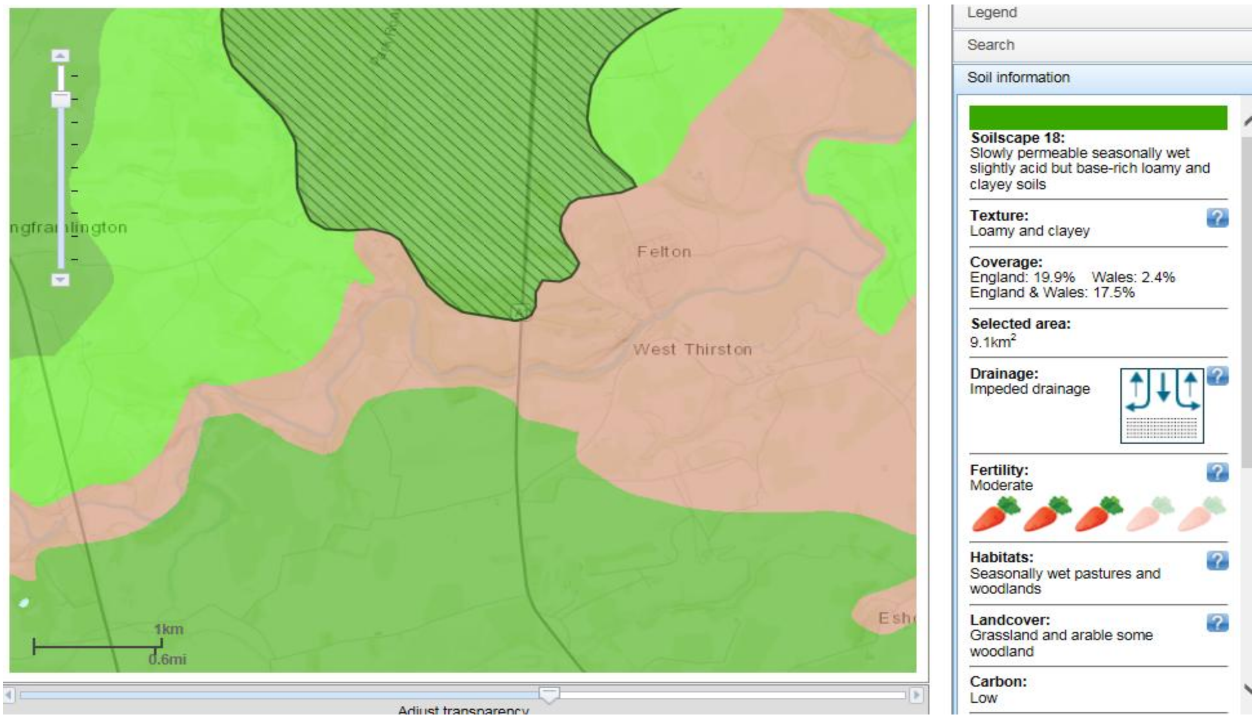
Fertility:
Moderate

Habitats:
Seasonally wet pastures and woodlands

Landcover:
Grassland and arable some woodland

Carbon:
Low

Soil Type Culvert 14



Soilscape 18:
Slowly permeable seasonally wet slightly acid but base-rich loamy and clayey soils

Texture:
Loamy and clayey

Coverage:
England: 19.9% Wales: 2.4%
England & Wales: 17.5%

Selected area:
9.1km^2

Drainage:
Impeded drainage

Fertility:
Moderate

Habitats:
Seasonally wet pastures and woodlands

Landcover:
Grassland and arable some woodland

Carbon:
Low

Culvert Designer/Analyzer Report
Shieldhill Culvert (1A) - Existing

Analysis Component			
Storm Event	Design	Discharge	1.0200 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	1.0200 m³/s	Check Discharge	1.0200 m³/s

Tailwater properties: Irregular Channel

Roughness Segments

Start Station	End Station	Mannings Coefficient
0+00	0+43	0.030

Natural Channel Points

Station (m)	Elevation (m)
0+00	104.45
0+07	104.25
0+13	104.08
0+21	104.03
0+25	104.11
0+29	104.21
0+37	104.54
0+43	104.82

Tailwater conditions for Design Storm.			
Discharge	1.0200 m³/s	Actual Depth	0.14 m
Velocity	0.67 m/s		

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-1a Actual Arch	1.0202 m³/s	105.29 m	2.15 m/s
Weir	Roadway	0.0000 m³/s	105.29 m	N/A
Total	----------------	1.0202 m³/s	105.29 m	N/A

Culvert Designer/Analyzer Report
Shieldhill Culvert (1A) - Existing

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	105.29	m	Discharge	1.0202	m³/s
Inlet Control HW Elev.	105.12	m	Tailwater Elevation	104.17	m
Outlet Control HW Elev.	105.29	m	Control Type	Outlet Control	
Headwater Depth/Height	0.98				

Grades

Upstream Invert	104.30	m	Downstream Invert	103.86	m
Length	30.39	m	Constructed Slope	0.014705	m/m

Hydraulic Profile

Profile	M2		Depth, Downstream	0.56	m
Slope Type	Mild		Normal Depth	0.86	m
Flow Regime	Subcritical		Critical Depth	0.56	m
Velocity Downstream	2.15	m/s	Critical Slope	0.048941	m/m

Section

Section Shape	Arch		Mannings Coefficient	0.032	
Section Material	Concrete		Span	1.31	m
Section Size	1a Actual		Rise	1.01	m
Number Sections	1				

Outlet Control Properties

Outlet Control HW Elev.	105.29	m	Upstream Velocity Head	0.10	m
Ke	0.50		Entrance Loss	0.05	m

Inlet Control Properties

Inlet Control HW Elev.	105.12	m	Flow Control	Unsubmerged	
Inlet Type	Square edge w/headwall (arch)		Area Full	1.2	m²
K	0.00980		HDS 5 Chart	0	
M	2.00000		HDS 5 Scale	0	
C	0.03980		Equation Form	1	
Y	0.67000				

Culvert Designer/Analyzer Report
Shieldhill Culvert (1A) - Existing

Component:Weir

Hydraulic Component(s): Roadway

Discharge	0.0000	m³/s	Allowable HW Elevation	105.29	m
Roadway Width	9.78	m	Overtopping Coefficient	1.60	SI
Low Point	105.56	m	Headwater Elevation	N/A	m
Discharge Coefficient (Cr)	2.90		Submergence Factor (Kt)	1.00	
Tailwater Elevation	104.17	m			

Sta (m)	Elev. (m)
0.00	105.96
0.16	106.00
1.24	106.25
2.35	106.50
3.26	106.75
4.03	107.00
4.91	107.25
5.82	107.50
8.16	107.75
11.03	108.00
11.58	107.98
12.54	108.00
16.66	108.68
20.40	107.95
21.36	107.90
23.61	108.00
24.31	107.75
24.99	107.50
25.52	107.25
26.57	106.75
27.09	106.50
27.61	106.25
28.64	106.00
29.40	105.75
30.87	105.56

Culvert Designer/Analyzer Report
Shieldhill Culvert (1A) - Proposed

Analysis Component

Storm Event	Design	Discharge	1.0200 m³/s

Peak Discharge Method: User-Specified

Design Discharge	1.0200 m³/s	Check Discharge	1.0200 m³/s

Tailwater Conditions: Constant Tailwater

Tailwater Elevation	104.26 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-1200mm reduced 150silt Circular	1.0200 m³/s	105.45 m	3.31 m/s
Weir	Roadway (Constant Elevation)	0.0000 m³/s	105.45 m	N/A
Total	----------------	1.0200 m³/s	105.45 m	N/A

Culvert Designer/Analyzer Report
Shieldhill Culvert (1A) - Proposed

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	105.45	m	Discharge	1.0200	m³/s
Inlet Control HW Elev.	105.36	m	Tailwater Elevation	104.26	m
Outlet Control HW Elev.	105.45	m	Control Type	Entrance Control	
Headwater Depth/Height	0.76				

Grades					
Upstream Invert	104.57	m	Downstream Invert	103.86	m
Length	48.54	m	Constructed Slope	0.014705	m/m

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.39	m
Slope Type	Steep		Normal Depth	0.38	m
Flow Regime	Supercritical		Critical Depth	0.55	m
Velocity Downstream	3.31	m/s	Critical Slope	0.003871	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	1.16	m
Section Size	1200mm reduced 150silt		Rise	1.16	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	105.45	m	Upstream Velocity Head	0.22	m
Ke	0.50		Entrance Loss	0.11	m

Inlet Control Properties					
Inlet Control HW Elev.	105.36	m	Flow Control	Unsubmerged	
Inlet Type	Square edge w/headwall		Area Full	1.0	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
Shieldhill Culvert (1A) - Proposed

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)

Discharge	0.0000	m³/s	Allowable HW Elevation	105.45	m
Roadway Width	23.96	m	Overtopping Coefficient	1.60	SI
Length	48.54	m	Crest Elevation	107.73	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	107.73
48.54	107.73

Culvert Designer/Analyzer Report
Paradise Culvert (3) - Existing

Analysis Component

Storm Event	Design	Discharge	4.8700 m³/s

Peak Discharge Method: User-Specified

Design Discharge	4.8700 m³/s	Check Discharge	4.8700 m³/s

Tailwater properties: Irregular Channel

Roughness Segments

Start Station	End Station	Mannings Coefficient
0+00	0+15	0.030
0+15	0+18	0.013
0+18	0+36	0.030

Natural Channel Points

Station (m)	Elevation (m)
0+00	85.72
0+06	85.49
0+12	85.25
0+14	84.90
0+15	83.89
0+16	83.69
0+18	83.65
0+18	83.69
0+19	83.54
0+19	83.73
0+20	84.36
0+21	84.60
0+23	84.69
0+27	84.45
0+31	84.59
0+36	84.41

Tailwater conditions for Design Storm.

Discharge	4.8700 m³/s	Actual Depth	0.65 m
Velocity	2.07 m/s		

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-3 existing Arch	4.8702 m³/s	86.13 m	3.32 m/s
Weir	Roadway (Constant Elevation)	0.0000 m³/s	86.13 m	N/A
Total	----------------	4.8702 m³/s	86.13 m	N/A

Culvert Designer/Analyzer Report
Paradise Culvert (3) - Existing

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	86.13	m	Discharge	4.8702	m³/s
Inlet Control HW Elev.	85.40	m	Tailwater Elevation	84.19	m
Outlet Control HW Elev.	86.13	m	Control Type	Outlet Control	
Headwater Depth/Height	2.17				

Grades					
Upstream Invert	83.80	m	Downstream Invert	83.66	m
Length	26.66	m	Constructed Slope	0.005225	m/m

Hydraulic Profile					
Profile	CompositeM2PressureProfile		Depth, Downstream	0.82	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.82	m
Velocity Downstream	3.32	m/s	Critical Slope	0.039668	m/m

Section					
Section Shape	Arch		Mannings Coefficient	0.032	
Section Material	Stone masonry		Span	1.96	m
Section Size	3 existing		Rise	1.07	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	86.13	m	Upstream Velocity Head	0.42	m
Ke	0.20		Entrance Loss	0.08	m

Inlet Control Properties					
Inlet Control HW Elev.	85.40	m	Flow Control	Submerged	
Inlet Type	Groove end projecting (arch)		Area Full	1.7	m²
K	0.00450		HDS 5 Chart	0	
M	2.00000		HDS 5 Scale	0	
C	0.03170		Equation Form	1	
Y	0.69000				

Culvert Designer/Analyzer Report
Paradise Culvert (3) - Existing

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.0000	m³/s	Allowable HW Elevation	86.13	m
Roadway Width	10.82	m	Overtopping Coefficient	1.60	SI
Length	26.31	m	Crest Elevation	87.48	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	87.48
26.31	87.48

Culvert Designer/Analyzer Report
Paradise Culvert (3) - Proposed

Analysis Component			
Storm Event	Design	Discharge	4.8700 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	4.8700 m³/s	Check Discharge	4.8700 m³/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	85.24 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-1800 mm Circular	4.8689 m³/s	85.66 m	2.02 m/s
Weir	Roadway (Constant Elevation)	0.0000 m³/s	85.66 m	N/A
Total	----------------	4.8689 m³/s	85.66 m	N/A

Culvert Designer/Analyzer Report
Paradise Culvert (3) - Proposed

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	85.66	m	Discharge	4.8689	m³/s
Inlet Control HW Elev.	85.49	m	Tailwater Elevation	85.24	m
Outlet Control HW Elev.	85.66	m	Control Type	Outlet Control	
Headwater Depth/Height	0.99				

Grades

Upstream Invert	83.84	m	Downstream Invert	83.66	m
Length	35.00	m	Constructed Slope	0.005225	m/m

Hydraulic Profile

Profile	M1		Depth, Downstream	1.58	m
Slope Type	Mild		Normal Depth	1.16	m
Flow Regime	Subcritical		Critical Depth	1.09	m
Velocity Downstream	2.02	m/s	Critical Slope	0.006389	m/m

Section

Section Shape	Circular		Mannings Coefficient	0.017	
Section Material	Concrete		Span	1.83	m
Section Size	1800 mm		Rise	1.83	m
Number Sections	1				

Outlet Control Properties

Outlet Control HW Elev.	85.66	m	Upstream Velocity Head	0.24	m
Ke	0.50		Entrance Loss	0.12	m

Inlet Control Properties

Inlet Control HW Elev.	85.49	m	Flow Control	Unsubmerged	
Inlet Type	Square edge w/headwall		Area Full	2.6	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
Paradise Culvert (3) - Proposed

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.0000	m³/s	Allowable HW Elevation	85.66	m
Roadway Width	21.30	m	Overtopping Coefficient	1.60	SI
Length	35.00	m	Crest Elevation	87.48	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	87.48
35.00	87.48

Culvert Designer/Analyzer Report
South Longdike Culvert (9.1) - Proposed

Analysis Component			
Storm Event	Design	Discharge	0.1740 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	0.1740 m³/s	Check Discharge	0.0000 m³/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	67.50 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-1200 mm Circular	0.1740 m³/s	67.53 m	0.52 m/s
Weir	Not Considered	N/A	N/A	N/A

Culvert Designer/Analyzer Report
South Longdike Culvert (9.1) - Proposed

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	67.53	m	Discharge	0.1740	m³/s
Inlet Control HW Elev.	67.50	m	Tailwater Elevation	67.50	m
Outlet Control HW Elev.	67.53	m	Control Type	Outlet Control	
Headwater Depth/Height	0.33				

Grades

Upstream Invert	67.14	m	Downstream Invert	67.10	m
Length	39.00	m	Constructed Slope	0.001026	m/m

Hydraulic Profile

Profile	M1		Depth, Downstream	0.40	m
Slope Type	Mild		Normal Depth	0.30	m
Flow Regime	Subcritical		Critical Depth	0.22	m
Velocity Downstream	0.52	m/s	Critical Slope	0.003692	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.	67.53	m	Upstream Velocity Head	0.02	m
Ke	0.50		Entrance Loss	0.01	m

Inlet Control Properties

Inlet Control HW Elev.	67.50	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
Blackwood Hall Culvert (13.1): - Proposed

Analysis Component			
Storm Event	Design	Discharge	0.0000 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	0.9700 m³/s	Check Discharge	0.9700 m³/s

Tailwater properties: Trapezoidal Channel

Tailwater conditions for Design Storm.			
Discharge	0.9700 m³/s	Bottom Elevation	60.92 m
Depth	0.35 m	Velocity	1.78 m/s

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	3-450 mm Circular	0.9701 m³/s	62.22 m	2.15 m/s
Weir	Roadway (Constant Elevation)	0.0000 m³/s	62.22 m	N/A
Total	----------------	0.9701 m³/s	62.22 m	N/A

Culvert Designer/Analyzer Report
Blackwood Hall Culvert (13.1) - Proposed

Component:Culvert-1

Culvert Summary					
Computed Headwater Eleva	62.22	m	Discharge	0.9701	m³/s
Inlet Control HW Elev.	61.69	m	Tailwater Elevation	61.27	m
Outlet Control HW Elev.	62.22	m	Control Type	Outlet Control	
Headwater Depth/Height	2.73				

Grades					
Upstream Invert	60.98	m	Downstream Invert	60.92	m
Length	53.63	m	Constructed Slope	0.000999	m/m

Hydraulic Profile					
Profile	CompositeM2PressureProfile		Depth, Downstream	0.39	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.39	m
Velocity Downstream	2.15	m/s	Critical Slope	0.010951	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	0.46	m
Section Size	450 mm		Rise	0.46	m
Number Sections	3				

Outlet Control Properties					
Outlet Control HW Elev.	62.22	m	Upstream Velocity Head	0.20	m
Ke	0.20		Entrance Loss	0.04	m

Inlet Control Properties					
Inlet Control HW Elev.	61.69	m	Flow Control	Submerged	
Inlet Type	Groove end w/headwall		Area Full	0.5	m²
K	0.00180		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	2	
C	0.02920		Equation Form	1	
Y	0.74000				

Culvert Designer/Analyzer Report
Blackwood Hall Culvert (13.1) - Proposed

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.0000	m³/s	Allowable HW Elevation	62.22	m
Roadway Width	46.61	m	Overtopping Coefficient	1.60	SI
Length	61.59	m	Crest Elevation	63.02	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	63.02
61.59	63.02

Culvert Designer/Analyzer Report
Glenshotton Culvert (14) - Existing

Analysis Component			
Storm Event	Design	Discharge	2.9400 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	2.9400 m³/s	Check Discharge	2.9400 m³/s

Tailwater properties: Irregular Channel

Roughness Segments

Start Station	End Station	Mannings Coefficient
0+00	0+05	0.030
0+05	0+06	0.040
0+06	0+15	0.030

Natural Channel Points

Station (m)	Elevation (m)
0+00	58.42
0+03	58.42
0+05	57.27
0+05	56.94
0+05	56.94
0+06	56.96
0+06	57.06
0+06	57.08
0+06	57.49
0+07	57.83
0+08	58.20
0+09	58.27
0+12	58.29
0+15	58.44

Tailwater conditions for Design Storm.			
Discharge	2.9400 m³/s	Actual Depth	1.19 m
Velocity	1.17 m/s		

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-1200 mm Circular	2.9369 m³/s	59.31 m	3.04 m/s
Weir	Roadway	0.0028 m³/s	59.31 m	N/A
Total	----------------	2.9397 m³/s	59.31 m	N/A

Culvert Designer/Analyzer Report
Glenshotton Culvert (14) - Existing

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	59.31	m	Discharge	2.9369	m³/s
Inlet Control HW Elev.	59.31	m	Tailwater Elevation	58.13	m
Outlet Control HW Elev.	59.30	m	Control Type	Inlet Control	
Headwater Depth/Height	1.35				

Grades

Upstream Invert	57.67	m	Downstream Invert	57.58	m
Length	24.32	m	Constructed Slope	0.003644	m/m

Hydraulic Profile

Profile	M2		Depth, Downstream	0.94	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.94	m
Velocity Downstream	3.04	m/s	Critical Slope	0.005891	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.	59.30	m	Upstream Velocity Head	0.37	m
Ke	0.50		Entrance Loss	0.19	m

Inlet Control Properties

Inlet Control HW Elev.	59.31	m	Flow Control	Submerged	
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

Glenshotton Culvert (14) - Existing

Component:Weir

Hydraulic Component(s): Roadway					
Discharge	0.0028	m³/s	Allowable HW Elevation	59.31	m
Roadway Width	10.52	m	Overtopping Coefficient	1.61	SI
Low Point	59.25	m	Headwater Elevation	59.31	m
Discharge Coefficient (Cr)	2.91		Submergence Factor (Kt)	1.00	
Tailwater Elevation	58.13	m			

Sta (m)	Elev. (m)
0.00	59.25
1.73	59.75
3.86	60.00
6.49	60.15
7.37	60.15
12.43	60.22
16.42	60.29
17.89	60.29
18.23	60.25
21.91	60.00
23.96	59.25

Culvert Designer/Analyzer Report
Glenshotton culvert (14) - proposed

Analysis Component			
Storm Event	Design	Discharge	2.7479 m³/s

Peak Discharge Method: User-Specified			
Design Discharge	2.7479 m³/s	Check Discharge	2.7479 m³/s

Tailwater properties: Irregular Channel

Roughness Segments

Start Station	End Station	Mannings Coefficient
0+00	0+05	0.030
0+05	0+06	0.040
0+06	0+15	0.030

Natural Channel Points

Station (m)	Elevation (m)
0+00	58.42
0+03	58.42
0+05	57.27
0+05	56.94
0+05	56.94
0+06	56.96
0+06	57.06
0+06	57.08
0+06	57.49
0+07	57.83
0+08	58.20
0+09	58.27
0+12	58.29
0+15	58.44

Tailwater conditions for Design Storm.			
Discharge	2.7479 m³/s	Actual Depth	1.16 m
Velocity	1.16 m/s		

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-200 mm Circular	2.7484 m³/s	59.25 m	2.92 m/s
Weir	Roadway	0.0000 m³/s	59.25 m	N/A
Total	----------------	2.7484 m³/s	59.25 m	N/A

Culvert Designer/Analyzer Report
Glenshotten Culvert (14) - proposed

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	59.25	m	Discharge	2.7484	m³/s
Inlet Control HW Elev.	59.22	m	Tailwater Elevation	58.10	m
Outlet Control HW Elev.	59.25	m	Control Type	Outlet Control	
Headwater Depth/Height	1.26				

Grades

Upstream Invert	57.70	m	Downstream Invert	57.53	m
Length	47.00	m	Constructed Slope	0.003617	m/m

Hydraulic Profile

Profile	M2		Depth, Downstream	0.91	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.91	m
Velocity Downstream	2.92	m/s	Critical Slope	0.005433	m/m

Section

Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	1.23	m
Section Size	14 A1 existing		Rise	1.23	m
Number Sections	1				

Outlet Control Properties

Outlet Control HW Elev.	59.25	m	Upstream Velocity Head	0.33	m
Ke	0.50		Entrance Loss	0.16	m

Inlet Control Properties

Inlet Control HW Elev.	59.22	m	Flow Control	Transition	
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
Glenshotton Culvert (14) - Proposed

Component:Weir

Hydraulic Component(s): Roadway

Discharge	0.0000	m³/s	Allowable HW Elevation	59.25	m
Roadway Width	10.52	m	Overtopping Coefficient	1.60	SI
Low Point	59.25	m	Headwater Elevation	N/A	m
Discharge Coefficient (Cr)	2.90		Submergence Factor (Kt)	1.00	
Tailwater Elevation	58.10	m			

Sta (m)	Elev. (m)
0.00	59.25
1.73	59.75
3.86	60.00
6.49	60.15
7.37	60.15
12.43	60.22
16.42	60.29
17.89	60.29
18.23	60.25
21.91	60.00
23.96	59.25

Culvert Designer/Analyzer Report
Parkwood Culvert (16) - Existing

Comments: Culvert assumed to follow a straight alignment between inlet and outlet invert levels.

Analysis Component			
Storm Event	Design	Discharge	1.6200 m^3/s

Peak Discharge Method: User-Specified			
Design Discharge	1.6200 m^3/s	Check Discharge	1.6200 m^3/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	38.13 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-site 13 900+150mm silt bed Circular	1.6199 m^3/s	40.79 m	3.07 m/s
Weir	Roadway (Constant Elevation)	0.0000 m^3/s	40.79 m	N/A
Total	----------------	1.6199 m^3/s	40.79 m	N/A

Culvert Designer/Analyzer Report
Parkwood Culvert (16) - Existing

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	40.79	m	Discharge	1.6199	m³/s
Inlet Control HW Elev.	40.79	m	Tailwater Elevation	38.13	m
Outlet Control HW Elev.	40.62	m	Control Type	Inlet Control	
Headwater Depth/Height	1.92				

Grades

Upstream Invert	39.15	m	Downstream Invert	37.93	m
Length	125.47	m	Constructed Slope	0.009755	m/m

Hydraulic Profile

Profile	M2		Depth, Downstream	0.75	m
Slope Type	Mild		Normal Depth	0.75	m
Flow Regime	Subcritical		Critical Depth	0.75	m
Velocity Downstream	3.07	m/s	Critical Slope	0.009825	m/m

Section

Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	0.85	m
Section Size	site 13 900+150mm silt bed		Rise	0.85	m
Number Sections	1				

Outlet Control Properties

Outlet Control HW Elev.	40.62	m	Upstream Velocity Head	0.47	m
Ke	0.50		Entrance Loss	0.24	m

Inlet Control Properties

Inlet Control HW Elev.	40.79	m	Flow Control	Submerged	
Inlet Type	Square edge w/headwall		Area Full	0.6	m²
K	0.00980		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	1	
C	0.03980		Equation Form	1	
Y	0.67000				

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)

Discharge	0.0000	m³/s	Allowable HW Elevation	40.79	m
Roadway Width	27.20	m	Overtopping Coefficient	1.60	SI
Length	144.00	m	Crest Elevation	53.37	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	53.37
144.00	53.37

Culvert Designer/Analyzer Report
Parkwood Culvert (16) - Proposed

Comments: Culvert assumed to follow a straight alignment between inlet and outlet invert levels.

Analysis Component			
Storm Event	Design	Discharge	1.6200 m^3/s

Peak Discharge Method: User-Specified			
Design Discharge	1.6200 m^3/s	Check Discharge	1.6200 m^3/s

Tailwater Conditions: Constant Tailwater	
Tailwater Elevation	38.13 m

Name	Description	Discharge	HW Elev.	Velocity
Culvert-1	1-site 13 900+150mm silt bed Circular	1.6199 m^3/s	40.79 m	3.07 m/s
Weir	Roadway (Constant Elevation)	0.0000 m^3/s	40.79 m	N/A
Total	----------------	1.6199 m^3/s	40.79 m	N/A

Culvert Designer/Analyzer Report
Parkwood Culvert (16) - Proposed

Component:Culvert-1

Culvert Summary

Computed Headwater Eleva	40.79	m	Discharge	1.6199	m³/s
Inlet Control HW Elev.	40.79	m	Tailwater Elevation	38.13	m
Outlet Control HW Elev.	40.62	m	Control Type	Inlet Control	
Headwater Depth/Height	1.92				

Grades

Upstream Invert	39.15	m	Downstream Invert	37.74	m
Length	144.49	m	Constructed Slope	0.009755	m/m

Hydraulic Profile

Profile	M2		Depth, Downstream	0.75	m
Slope Type	Mild		Normal Depth	0.75	m
Flow Regime	Subcritical		Critical Depth	0.75	m
Velocity Downstream	3.07	m/s	Critical Slope	0.009825	m/m

Section

Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	0.85	m
Section Size	site 13 900+150mm silt bed		Rise	0.85	m
Number Sections	1				

Outlet Control Properties

Outlet Control HW Elev.	40.62	m	Upstream Velocity Head	0.47	m
Ke	0.50		Entrance Loss	0.24	m

Inlet Control Properties

Inlet Control HW Elev.	40.79	m	Flow Control	Submerged	
Inlet Type	Square edge w/headwall		Area Full	0.6	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
Parkwood Culvert (16) - Existing

Component:Weir

Hydraulic Component(s): Roadway (Constant Elevation)					
Discharge	0.0000	m³/s	Allowable HW Elevation	40.79	m
Roadway Width	27.20	m	Overtopping Coefficient	1.60	SI
Length	144.00	m	Crest Elevation	53.37	m
Headwater Elevation	N/A	m	Discharge Coefficient (Cr)	2.90	
Submergence Factor (Kt)	1.00				

Sta (m)	Elev. (m)
0.00	53.37
144.00	53.37

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