A303 Amesbury to Berwick Down

TR010025

Volume 6

6.3 Environmental Statement Appendices

Appendix 11.5 (1) Level 3 Flood Risk Assessment

APFP Regulation 5(2)(a)

Planning Act 2008

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

May 2019

Infrastructure Planning

Planning Act 2008

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

A303 Amesbury to Berwick Down

Development Consent Order 20[**]

6.3 Environmental Statement Appendices

Appendix 11.5 Level 3 Flood Risk Assessment

Regulation Number:	Regulation 5(2)(a)
Planning Inspectorate Scheme Reference	TR010025
Application Document Reference	6.3 Appendix 11.5
Author:	A303 Amesbury to Berwick Down Project Team, Highways England

Version	Date	Status of Version
Rev 0	25.10.2018	First Issue
Rev 1	31.05.2019	Updated

Table of Contents

Figures

Tables

Executive Summary

This Flood Risk Assessment (FRA) has been prepared to assess the risk of flooding to and from the A303 Amesbury to Berwick Down scheme both during construction and operational phases. The study area of the FRA includes key features of the water environment within 1km of the proposed scheme boundary. Two main rivers in the study area are the River Till and River Avon, which are underlain by a Chalk Principal aquifer. The main sources of flood risk to the study area are fluvial, surface water (pluvial) and groundwater.

To better understand the fluvial flood risk posed to the study area and to assess the potential impacts to and from the proposed scheme, hydraulic modelling has been undertaken for the River Till and the River Avon for a range of return periods. Due to the proposed changes to the local topography to the area west of the River Till, at Parsonage Down, pluvial modelling was completed to assess the impact of the proposed scheme to the surface water flood risk in this area. A numerical groundwater model was constructed to assess the impact of the proposed scheme to groundwater, including the risk of groundwater flooding. In addition to hydraulic modelling assessments, the remainder of the study area was assessed as to its existing level of risk and potential sensitivity of change as a result of the proposed scheme.

A number of key scheme elements (permanent and temporary features) were identified as having the potential to influence flood risk within the study area. These features were assessed against the identified baseline flood risk to determine the potential impact to and from the proposed scheme. Only an assessment of flooding *from* the temporary works was completed, since any flood risk *to* the temporary works will be suitably managed by the appointed Contractor through their Construction Environment Management Plan (CEMP) derived from the Outline EMP.

Current Environment Agency fluvial flood maps for both the River Till and River Avon within the study area are based upon broadscale hydraulic modelling information. Therefore in order to create a refined representation of flood risk within the areas of interest, and facilitate a more robust assessment of the proposed scheme, within the Till and Avon catchments, site specific hydraulic models were created and hydrological assessments undertaken. The Environment Agency has been consulted throughout the hydraulic modelling process, including obtaining its agreement to the methodology for hydrological analysis and hydraulic modelling, and procuring its review and approval of modelling outputs. As such, in consultation with the Environment Agency, the hydraulic modelling outputs inform the Environment Agency Flood Zones for the purposes of this FRA.

The majority of the study area is within Flood Zone 1 (low probability), except where it traverses the two river channels, where areas of Flood Zone 2 and 3 are present. The baseline modelling flood extents for the River Till largely coincide with the corresponding Flood Zones produced by the Environment Agency. The baseline modelling flood extents for the River Avon are largely consistent with the Environment Agency Flood Zones through the study area, showing minor decreases in a number of locations Decreases compared to flood zones are generally observed within areas of undeveloped green space

both to the north and south of the A303 crossing, the most significant being just downstream of the Nine Mile River confluence at Bulford..

The majority of surface water flood risk in the study area, illustrated by the Environment Agency Flood Risk from Surface Water data, is categorised as ‘Low’; with some small ‘pockets’ of ‘Medium’ or ‘High’ flood risk. These are typically in valley bottoms and where surface water flow paths are impeded by artificial structures.

The pluvial baseline modelling flood extents are similar to that shown by the Environment Agency ‘Low’ surface water flood risk extent in the Parsonage Down area. The current Environment Agency Flood Risk from Surface Water flood map is based on broad-scale hydraulic modelling information. The Environment Agency and Wiltshire Council have been engaged through the development of the pluvial model and have agreed to the hydraulic modelling methodology which has been used to create a more detailed and site-specific assessment of the design surface water flood events for the catchment east of Parsonage Down Natural Nature Reserve (NNR).

The risk of groundwater flooding in the study area is considered to be High. The baseline groundwater model predicts that peak groundwater levels can be above the ground level and therefore groundwater flooding is likely to occur along the rivers and dry valleys, such as Stonehenge Bottom.

The risk of flooding from artificial sources and snowmelt is considered to be Negligible.

The assessment of flood risk to the permanent features of the proposed scheme has concluded that with design mitigation, the risk to the proposed scheme from fluvial, pluvial groundwater and sewer flooding would be Low.

The assessment of flood risk from the permanent features of the proposed scheme has concluded that with design mitigation, the risk to other receptors from fluvial, pluvial and groundwater flooding would be Low. Modelling undertaken shows that there would be no increase in flood risk to properties as a result of the proposed scheme, whilst flood risk to the B3083 is reduced. The permanent features would not alter sewer flood risk, therefore, the risk to receptors from sewer flooding as a result of the proposed scheme would be Negligible.

The assessment of flood risk from the temporary features of the proposed scheme has concluded that with design mitigation, the risk to other receptors from fluvial and pluvial flooding would be Low. The temporary features would not alter groundwater or sewer flood risk, therefore, the risk to receptors from groundwater or sewer flooding as a result of the proposed scheme would be Negligible.

1 Introduction

1.1 Commission

1.1.1 Highways England commissioned the production of a Flood Risk Assessment (FRA) to support the Development Consent Order (DCO) application for the A303 Amesbury to Berwick Down scheme (hereafter referred to as the 'proposed scheme').

1.1.2 The proposed scheme is approximately 13km in length and comprised of a new dual two-lane carriageway between Amesbury and Berwick Down, approximately 11.5km north of the town of Salisbury.

1.2 Scope, Assumptions and Limitations

1.2.1 The purpose of this FRA is to consider the flood risk implications of the permanent works, and key temporary construction works associated with the proposed scheme.

1.2.2 All sources of flood risk are considered, other than tidal flooding, which has been excluded on the grounds of elevation above predicted future tide levels and distance from coastal regions. This assessment therefore includes fluvial, surface water, groundwater, sewers and artificial sources[1].

1.2.3 The assessment of flood risk has been undertaken iteratively as the design has developed and the outcomes have informed the development of flood management and drainage mitigation to minimise the effect that the proposed scheme would have on flood risk, both to and from the proposed scheme.

1.2.4 Receptors considered in this assessment include the proposed scheme itself, and any people or buildings which are exposed to the flood source.

1.2.5 The assessment has included information provided by statutory consultees and stakeholders and has involved extensive liaison with these stakeholders to ensure all flood sources have been adequately considered and assessed.

1.2.6 Channel cross-section surveys and photogrammetry surveys of the River Avon and River Till were undertaken during November 2017 to April 2018 for the purpose of setting up hydraulic models of the watercourses.

1.2.7 Hydraulic modelling has been undertaken in key flood risk areas including fluvial modelling for the River Avon and River Till and surface water modelling at Parsonage Down due to the proposed changes in local topography. These have been completed for the baseline and proposed scenarios (temporary and permanent) for a range of return periods. A number of assumptions have been made within the hydraulic models and these are described in detail in the Fluvial Hydraulic Modelling Report, Annex 1 Part A and the Pluvial Hydraulic Modelling Report, Annex 1 Part B.

[1] Flood risk from reservoirs has been considered in this FRA due to the identification of a reservoir proposal on the River Till upstream of the proposed scheme

1.2.8 A numerical groundwater model has been constructed to assess the impact of the proposed scheme to and from groundwater, including the risk of groundwater flooding.

1.3 Study Area Location and Extent

1.3.1 The spatial scope of the FRA includes, as a minimum, key features of the water environment within 1km of the proposed scheme boundary (Figure 1.1).

1.3.2 Figure 1.1 encompasses the proposed areas to be used for construction and the potential zone of influence caused by temporary works or operational purposes associated with the proposed scheme.

1.3.3 The two main rivers in the study area are the River Avon and the River Till which are underlain by a Chalk Principal aquifer.

1.4 Document Revision History

1.4.1 The first version of this FRA was submitted with the DCO application as an Appendix to the Environmental Statement and given examination library reference [APP-283]. This documented results from initial hydraulic modelling and groundwater modelling that had been undertaken prior to the date of the application, i.e. 19 October 2018.

1.4.2 This updated version of the FRA (v2.0) incorporates confirmatory updates from additional fluvial and surface water (pluvial) hydraulic modelling that have been undertaken between January 2019 and May 2019. This additional modelling resulted from discussions with the Environment Agency and Wiltshire Council that have taken place since submission of the DCO application.

1.4.3 Key updates to this document are summarised as:

- Incorporation of confirmatory results from updated hydraulic modelling for the River Avon, with changes to hydrological inflows and changes in indicative areas assigned to highway drainage ponds.
- Incorporation of confirmatory results from updated surface water hydraulic modelling for the Parsonage Down catchment, including surface water hydrology and updated drainage solutions at Parsonage Down.

1.4.4 Details of updates to the hydraulic modelling assessments are contained within Annexes 1 and 2 of this document.

1.4.5 The conclusions of FRA Version 2.0 remain unchanged from the version submitted with the DCO application ([APP-283]). Importantly, additional hydraulic modelling undertaken confirms and shows that the proposed scheme does not increase flood risk to properties during construction or operation.

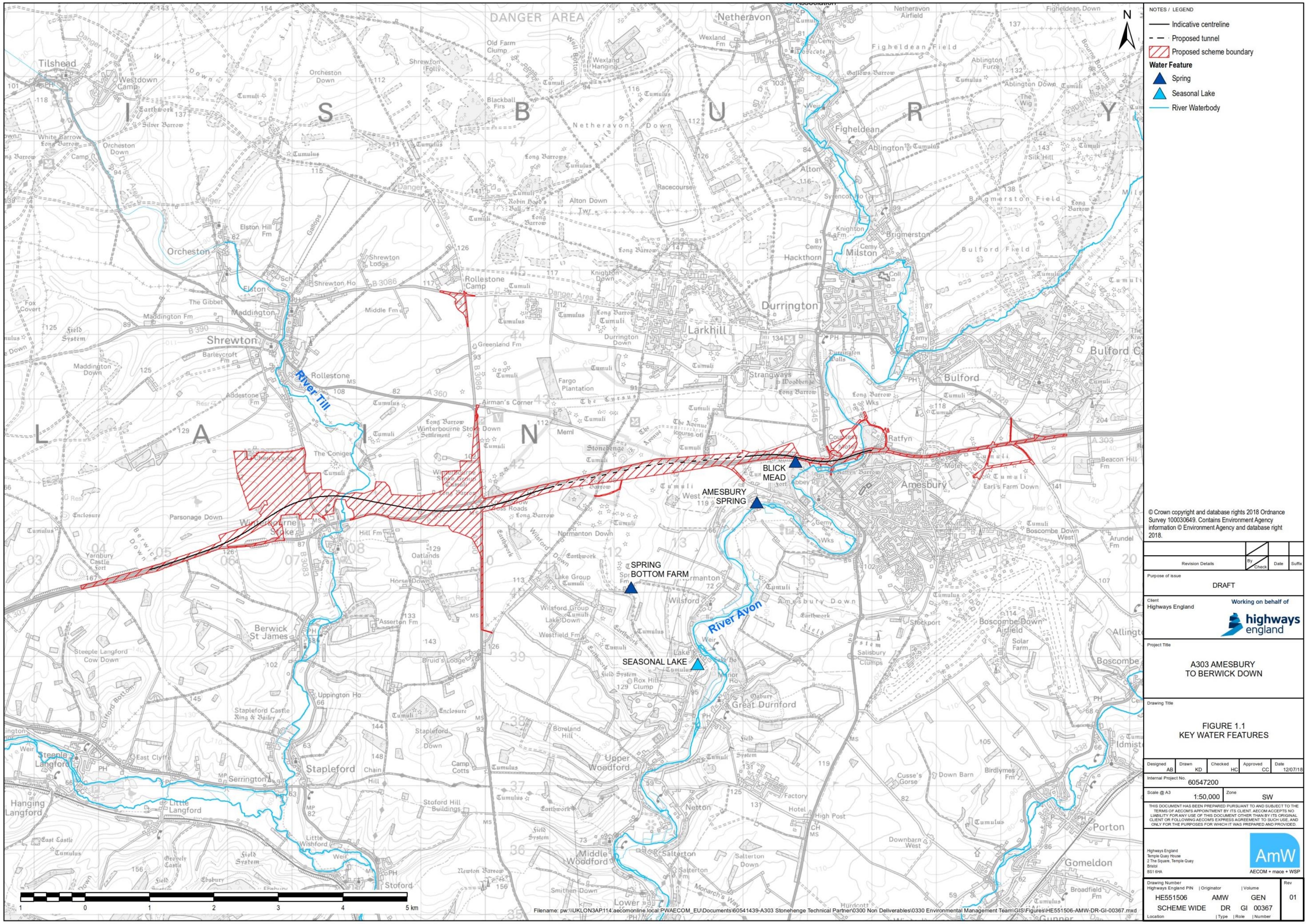


Figure 1.1: Extent of Study Area

2 Study Area Hydrological Context

2.1 Overview

2.1.1 This section provides an overview of the hydrology and hydrogeology of the study area.

2.2 Hydrogeology

2.2.1 The topography of the study area consists of low relief, gently sloping Chalk downland. Dry valleys are a feature typical of the Chalk landscape of southern Britain. Their characteristics derive from periglacial conditions in this area during the last ice age. The most prominent dry valley crossed by the route is that at Stonehenge Bottom, shown on Figure 2.1. This has an elevation of around 80m AOD at the current A303 road, an elevation of 70m AOD close to Spring Bottom Farm, and ends at Lake at an elevation of 62m AOD. Other dry valleys are crossed north of Winterbourne Stoke, east of Winterbourne Stoke, at Wilsford Down, west of Vespasian's Camp, and north of the Blick Mead archaeological site.

2.2.2 Figure 2.1 shows the superficial and bedrock geology within the study area, taken from British Geological Survey (BGS) mapping.

Bedrock geology

2.2.3 The bedrock underlying the study area comprises the White Chalk; an Upper Cretaceous succession of the Chalk group, including the Newhaven and Seaford Chalk Formations, with deposits of Phosphatic Chalk (Ref 2.1). The majority of the Chalk outcrop is the Seaford Chalk, with a north-east south-west trending outcrop of Newhaven Chalk present in the area between the Avenue and Normanton Down, and an outcrop on Coneybury Hill.

2.2.4 The Seaford Chalk Formation is described by the BGS as a 'firm white chalk with conspicuous semi-continuous nodular and tabular flint seams'. The Seaford Chalk Formation is up to approximately 60m thick in the study area. The Newhaven Chalk is described by the BGS as 'a soft to medium hard, smooth white chalk with numerous marl seams and flint bands', and is approximately 10m thick.

2.2.5 The Lewes Chalk is the oldest formation and comprises hard nodular chalks and hardgrounds interbedded with softer grainy chalks and marls, and widespread sheet flints. This unit outcrops at Berwick St James in the Till Valley around 2km south of Winterbourne Stoke (Ref 2.2).

2.2.6 The White Chalk bedrock (including the Seaford, Newhaven and Lewes Nodular Chalk Formations) in the study area is classified by the Environment Agency as a principal aquifer. As a principal aquifer the Chalk provides water supply on a strategic scale and significant river base flow, and forms an aquifer of regional importance.

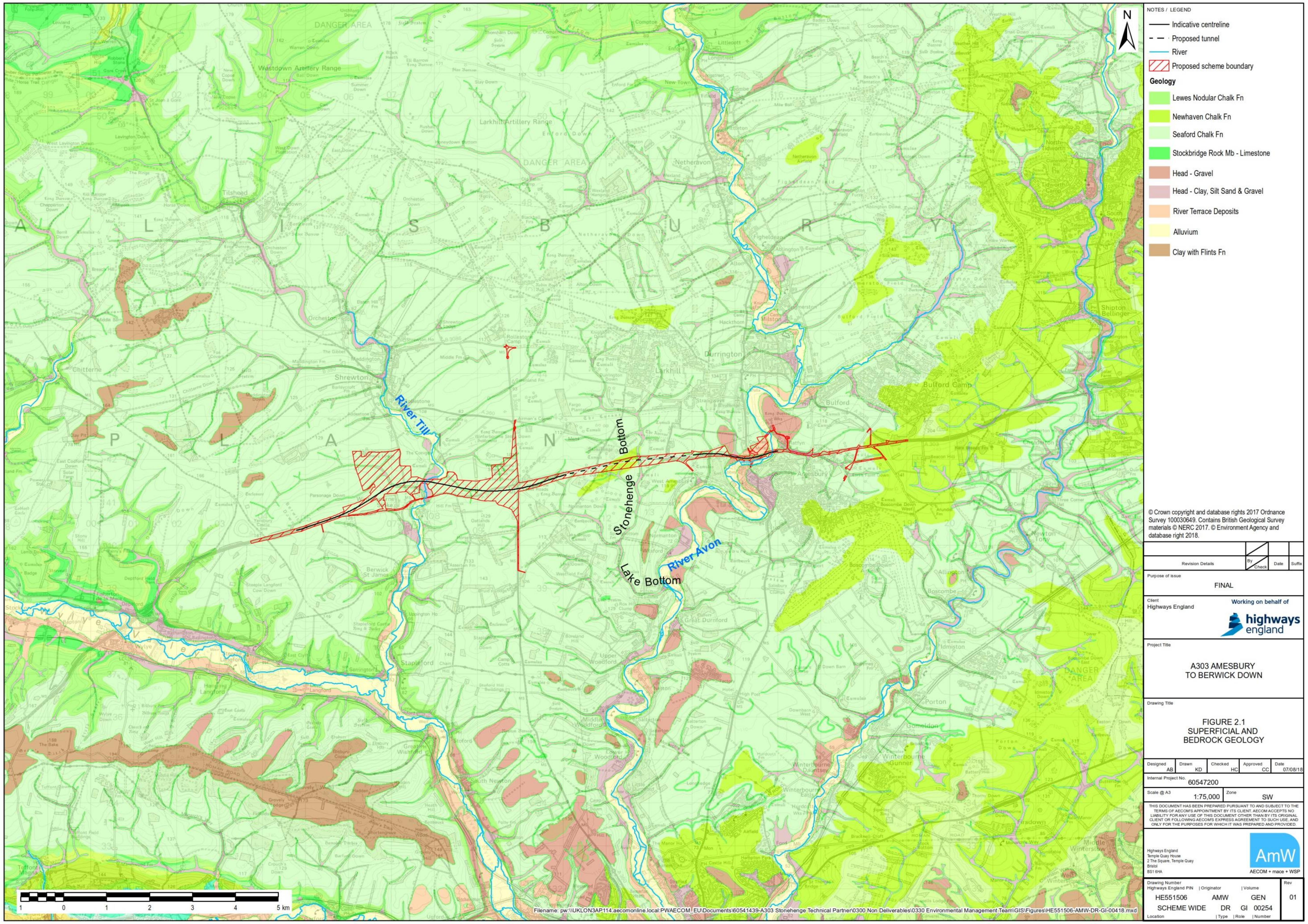


Figure 2.1: Superficial and Bedrock Geology

Superficial geology

2.2.7 The superficial deposits within the study area typically comprise alluvium, sands and gravels, localised river terrace deposits, and head deposits, which are largely remobilised weathered Chalk material deposited as a result of periglacial processes.

2.2.8 The dry valleys contain head deposits, comprising clay, silt, sand and gravel, overlying the Chalk. The active river valleys of the River Avon and River Till contain alluvial and terrace gravel deposits, as well as head deposits of gravel.

2.2.9 There are three types of superficial aquifers classified by the Environment Agency within the study area:

a) Alluvium, river terrace gravel deposits, and head deposits (where they consist of gravel) are classified as Secondary A aquifers. These are permeable layers with a moderate to high primary permeability and which are capable of supporting water supplies at a local rather than strategic scale, and in some cases form an important source of baseflow to rivers. The deposits provide groundwater that flows to the River Avon and River Till;

b) Clay and sand deposits located on interfluves towards the River Avon are classified as Secondary B aquifers, and;

c) Head deposits (comprising clay, silt, sand and gravel) located in dry valleys and the River Till and River Avon valleys are classified as Secondary (undifferentiated) aquifers. These aquifers are defined where it has not been possible to define an A or B category.

Groundwater level fluctuations

2.2.10 Monitoring data shows that groundwater levels in the Chalk aquifer respond rapidly to recharge events at the surface due to a low storage capacity, and significant changes in groundwater level can occur over short periods of time.

2.2.11 Groundwater levels in the Chalk are controlled by recharge from rainfall infiltration and by natural discharge to the River Avon and River Till, as well as groundwater abstractions. The seasonal fluctuations in the groundwater level tend to be less in the dry valleys (between 8m and 10m), than below topographic divides (about 15m) as the storage capacity is usually greater beneath dry valley systems, than in the interfluve areas.

2.2.12 Groundwater is known to rise to the surface in otherwise dry valleys during periods of high rainfall and in the River Till north of Berwick St James.

Groundwater flow

2.2.13 Groundwater flow in the Chalk aquifer in the study area is generally from north to south with flow at high groundwater levels converging towards the River Till in the west of the study area and towards the River Avon in the east of the study area. The groundwater discharges naturally as baseflow to the River Avon and River Till. The discharge to the River Avon is perennial via springs along the margins of overlying superficial deposits and upward flow via superficial deposits, whereas the River Till is a winterbourne (dry through periods of low groundwater levels) north of Berwick St James.

2.3 Hydrology

Rainfall

2.3.1 Rainfall data from the Meteorological Office for 1981 to 2000 show that the study area receives an annual average rainfall total of between 748mm (Boscombe Down) and 770mm (Larkhill).

2.3.2 The Environment Agency has provided daily rainfall data in the vicinity to the proposed scheme from two sites at Boscombe Down (3.2km southeast of the scheme), and Larkhill (2.6km north of the scheme). These locations are shown on Figure 2.2.

2.3.3 Table 2.1 provides the monthly average rainfall for these stations and the annual average, as well as those presented by the Meteorological Office for the 1981 to 2000 period. This shows that the highest rainfall generally occurs between October and January.

Table 2.1: Average rainfall (mm)

Average	Larkhill		Boscombe Down	
	Environment Agency data (1921-2017)	Met. Office (1981-2000)	Environment Agency data (2010 – 2017)	Met. Office (1981-2000)
January	82.8	80.3	102.2	74.5
February	57.1	53.9	60.2	52.0
March	52.2	58.5	46.8	57.2
April	46.5	51.4	45.6	51.4
May	55.5	52.8	50.6	54.4
June	51.0	50.3	55.6	51.0
July	55.0	51.4	62.6	48.9
August	54.6	53.7	68.1	51.5
September	60.0	62.7	46.5	59.4
October	80.2	85.6	72.9	82.6
November	83.8	83.7	77.0	84.0
December	80.2	86.0	73.0	81.7
Annual Average	**753.1**	**770.4**	**760.9**	**748.6**

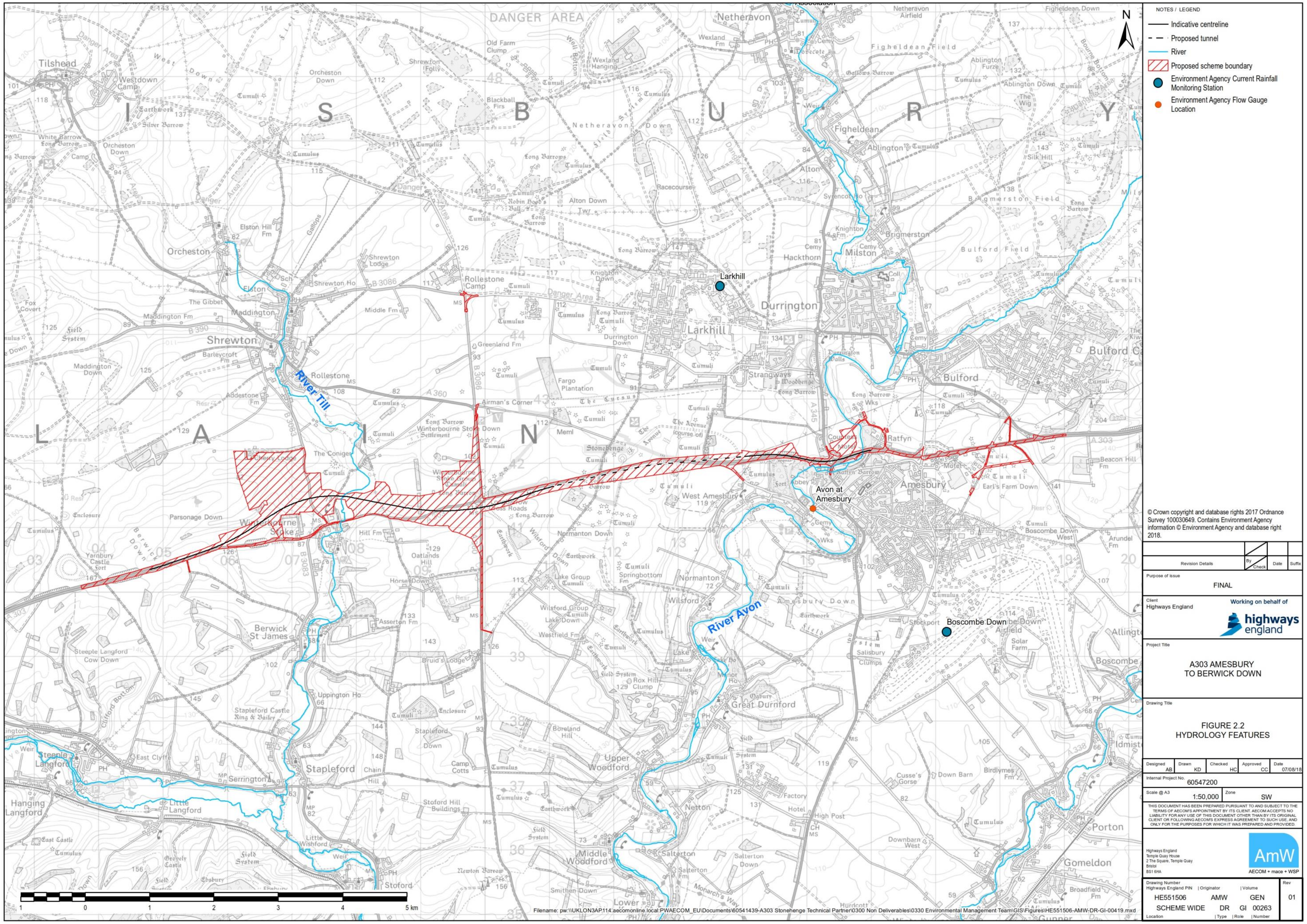


Figure 2.2: Hydrology features

River Avon

2.3.4 The River Avon, designated as a Main River, is a perennial, largely groundwater fed Chalk River. It flows in a southerly direction in the eastern part of the study area. The Environment Agency has a river level and flow gauge station located at Amesbury, shown on Figure 2.2. The flows recorded between 1965 and 2016 range between 1.13m^3/s (at the Q95 for low flows), and 25.75m^3/s (at the maximum gauged flow). There are also a number of small channels, ponds and ditches located within the River Avon floodplain.

River Till

2.3.5 The River Till, designated as a Main River, flows southwards in the west of the study area. The River Till is groundwater fed and in its upper reaches north of Berwick St James it flows as a winterbourne on an intermittent basis. The headwaters of the River Till are typically at Shrewton in winter; however, in wet years (e.g. 2014) the headwaters of the river can reach Tilshead. There are no flow monitoring locations on the River Till, with the nearest gauging station located at South Newton on the River Wylye.

Ordinary Watercourses

2.3.6 No ordinary watercourses are located within the study area. As such, no further reference is required for the purposes of this FRA.

Flood Defences

2.3.7 No flood defences are located within the study area. As such, no further reference is required for the purposes of this FRA.

3 Scheme Description

3.1 Proposed Scheme

3.1.1 The proposed scheme (Figure 3.1) would include the following key features:

a) A bypass to the north of Winterbourne Stoke with a viaduct over the River Till valley;

b) A new Longbarrow junction with the A360 to the west of and outside the World Heritage Site (WHS), with the A303 passing under the junction;

c) A section through the WHS with a twin-bore tunnel past Stonehenge at least 1.9 miles (approximately 3km) long and a maximum depth of 50m;

d) An upgraded junction with the A345 at Countess Roundabout to the north of Amesbury, with the A303 passing over the junction;

e) Grassland habitat creation that would allow extension of the Parsonage Down NNR;

f) The conversion of the existing A303 through the WHS into a route for walking, cycling and horse riding; and

g) New ‘green bridges’ at various points along the length of the scheme to connect existing habitats and allow the movement of wildlife, maintain existing agricultural access and provide crossings for existing and new bridleways and public footpaths.

This chapter provides a summary of the key scheme elements which have the potential to influence flood risk within the study area.

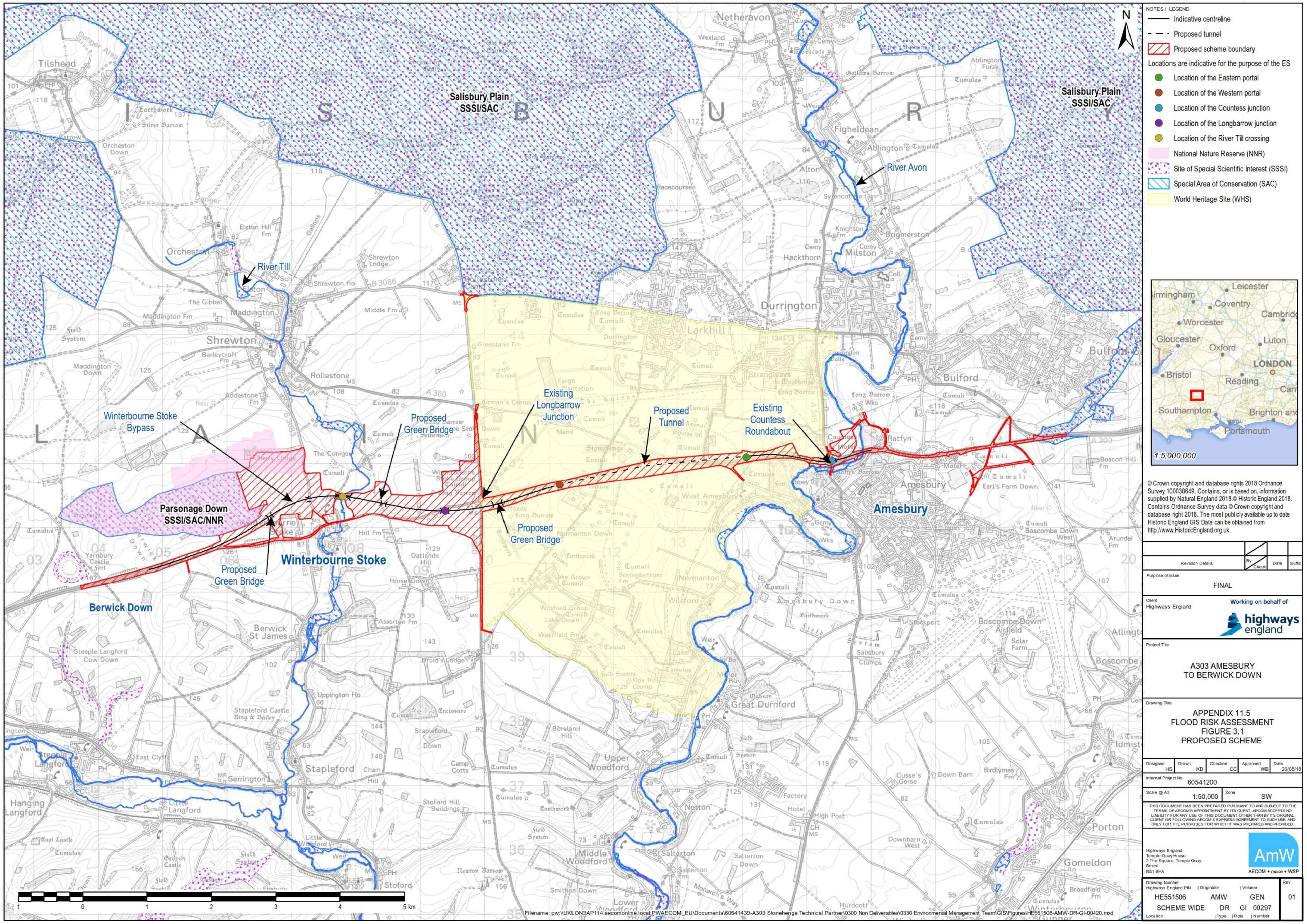


Figure 3.1: Proposed Scheme

3.2 Permanent Features

3.2.1 A number of permanent features of the proposed scheme have the potential to influence flood risk. These include:

a) River Till viaduct;
b) Longbarrow Junction upgrades;
c) Twin-bore tunnel, including portals;
d) Countess Roundabout flyover;
e) Embankments and cuttings;
f) Landscaping;
g) Provision of new utilities;
h) Road drainage; and
i) High Load route.

River Till viaduct

3.2.2 The proposed scheme would cross the River Till and its floodplain. The new viaduct, located north of Winterbourne Stoke, would be a 5-span structure, comprising of 2 decks approximately 7m apart to carry the new eastbound and westbound carriageways. Each deck would be supported by in-situ reinforced concrete abutments and four reinforced concrete piers (Figure 3.2). The piers and adjoining embankments have been designed and located to avoid the river channel and provide minimal obstruction of floodplain flows during both construction and operation, nevertheless, the introduction of piers into the floodplain has potential to interrupt flood flows and create a local backwater effect.

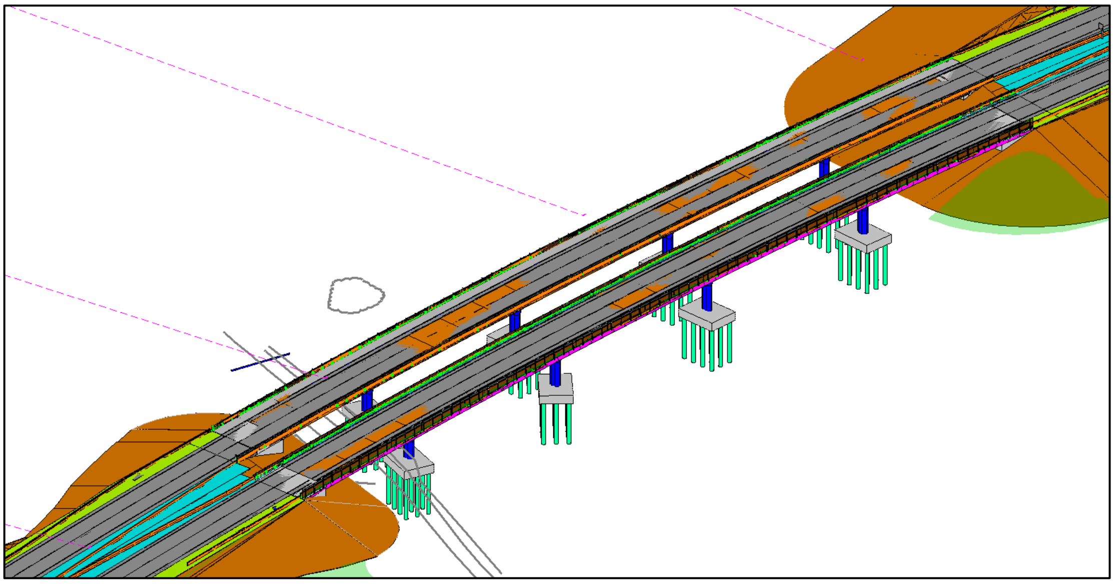


Figure 3.2: Proposed River Till Viaduct Configuration

Longbarrow Junction

3.2.3 A new junction with the A360 would be proposed approximately 600m west of the current Longbarrow roundabout. The new Longbarrow junction would comprise new slip road connections into two roundabouts linked by a green bridge over the new A303. The new junction would result in an increase in impermeable ground at this area which could potentially increase surface water flood risk.

Twin-bore tunnel (including portals)

3.2.4 The presence of part of the tunnel below the groundwater level in the Chalk has the potential to interfere with groundwater flow. This could lead to increased groundwater levels up hydraulic gradient of the tunnel, and decreased groundwater levels down hydraulic gradient.

3.2.5 Vertical retaining walls would be constructed along the approaches to both the western and eastern portals. The deepest two-thirds of the cutting would be formed with vertical retaining walls, with the top third formed with rolling grassed slopes to provide a softer finish for views towards the cutting.

3.2.6 At the location of the existing agricultural underpass (eastern portal) the proposed highway is at grade with the adjacent land. The land falls in a valley towards this point and a flood flow route has been identified. The catchment draining to this point has been estimated to be 85ha in plan area. A preliminary peak flow estimate using HA 106 methodology for a 1 in 100 year rainfall event has shown the peak flow rate at this location from this catchment to be approximately 110l/s. The calculation results are presented in Appendix 11.3 of the A303 Amesbury to Berwick Down Environmental Statement (Road Drainage Strategy).

3.2.7 The runoff will be intercepted by a ditch located at the highway boundary. The ditch will then outfall into a carrier pipe system to convey the flow westwards along the base of the highway cutting before discharging into the ditch which ultimately outfalls into the existing culvert to the west of Countess Roundabout.

Countess Roundabout flyover

3.2.8 The existing Countess Roundabout is an at-grade junction between the A303 and A345. It is located approximately 0.5km west of where the A303 crosses the River Avon. The proposal is to convert it to a grade separated junction by elevating the A303 over the roundabout on a flyover.

3.2.9 The flyover would be a multi-span viaduct across the existing roundabout (Figure 3.3). Two single 20.8m span bridges over the roundabout carriageways would be constructed with an earth embankment between the bridges on the roundabout island.

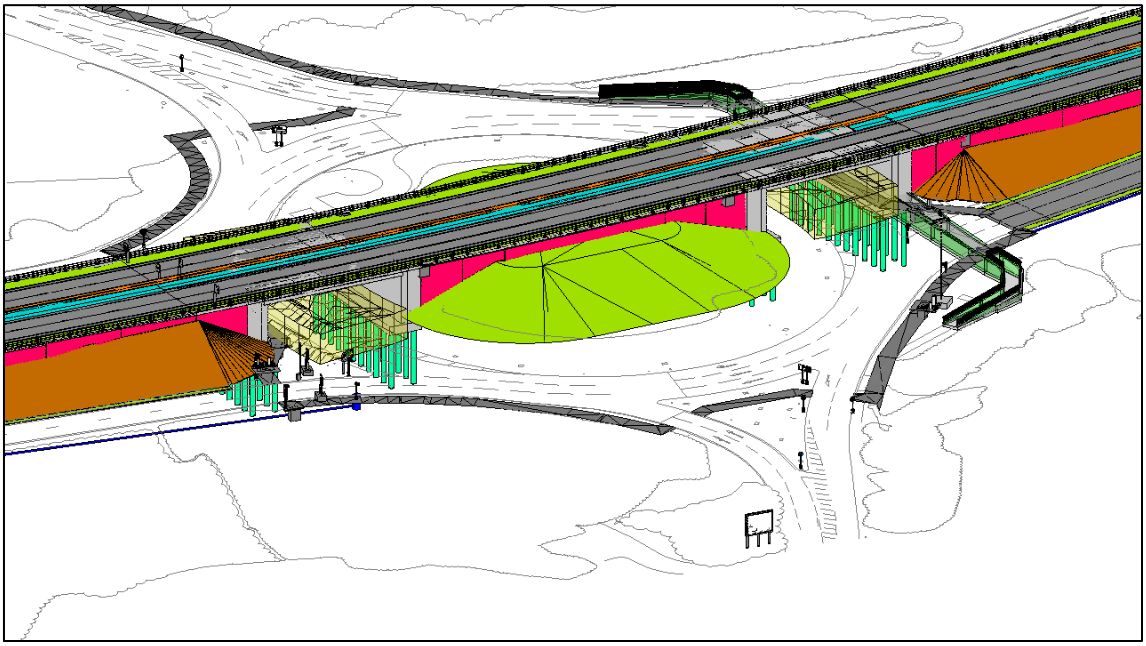


Figure 3.3: Proposed Countess Roundabout Flyover Configuration

3.2.10 The new flyover would be approximately 7m above the existing roundabout carriageway, with slip road connections from the roundabout (using the existing dual carriageways entries and exits) accommodating all movements to and from the A345.

3.2.11 There would be a need for minor topographical alterations to be located within the wider floodplain to provide the required space for construction. These topographical alterations include the embankments at the side of the A303, the structures to support the A303 flyover and the removal of the subway underneath the Countess Roundabout. The open viaducts of the flyover would help minimise impacts on overland flow; however, the introduction of embankments and the infill of the existing subway has the potential to alter flood flow pathways.

Embankments and cuttings

3.2.12 The proposed scheme would include the introduction of embankments or cuttings to integrate the new road alignment into the existing landscape. Adjustments to the land profile to facilitate the creation of embankments and cuttings have the potential to change the catchment characteristics, such as altering surface water overland flow paths.

Landscaping

3.2.13 A landscape design has been developed for the proposed scheme which consists of varying depths of fill and re-soiling along the route. These changes include landscaping associated with the implementation of embankments and cuttings, along with larger landscape areas for screening or habitat creation. Permanent topographic changes following deposition of tunnel excavated material and embankment creation may impact by altering flow paths.

3.2.14 The depth of re-soiling would vary along the proposed scheme. Verges and batters may require a depth of 150mm but some may be left bare allowing the natural chalk to remain exposed. Some landscape areas such as Parsonage Down and others would have varying depths of topsoil ranging from none to 300mm or 600mm where tree and shrub planting is envisaged. The landscape areas may also have areas of exposed chalk in some locations.

3.2.15 The landscaping planned at Parsonage Down will involve deposition of unstructured chalk, sourced from the tunnel arisings, upon the current ground surface. The chalk tunnel arisings will be deposited at varying depths between 0-10m, and within Parsonage Down some areas would have 100mm of topsoil rotovated into the chalk to provide a specific habitat.

Landscaping across Parsonage Down, described above, redirects the conveyance of overland surface water flow path. The formalising of this flowpath has also required an engineered solution for how surface water interacts between the catchment, A303 realignment and maintained B3083 highway.

Provision of new utilities

3.2.16 Construction of the proposed scheme is likely to require the diversion, relocation or protection of approximately 25 existing utility assets including water, wastewater, electricity, gas and telecommunications. These proposed routes or locations are described in detail within Chapter 2 of the Environmental Statement.

3.2.17 The electricity connection towards the eastern end of the route, where the route crosses the River Avon floodplain is in an area at low risk of surface water flooding and high risk of fluvial flooding (Flood Zone 3). The presence of underground structures (foundations or cables) could affect groundwater flows to the River Avon Special Area of Conservation (SAC).

Increased road surface

3.2.18 The introduction of new impermeable areas as part of the proposed scheme has the potential to increase the amount of surface water runoff.

High Load Route

3.2.19 The existing A303 on the approaches to the proposed scheme area is identified as a high load route for vehicles with a maximum height of 6.1m. A restriction on abnormal height vehicles in the new tunnel would mean that only normal height vehicles can use the new tunnel. The High Load Route would therefore be diverted from the Longbarrow Junction, north on the A360 and B3086, then east on The Packway and A3028, and south on Salisbury Road to Solstice Park. This route functions in both easterly and westerly directions.

3.2.20 Road widening at the crossroads near Rollestone Camp (red shaded area to the north of Figure 2.1) would take place in order to allow large vehicles to manoeuvre. All alterations will be at grade and so no alterations to the land level would occur. A minor increase in impermeable ground is expected due to the road widening at this section of the route.

3.3 Temporary Features

3.3.1 A number of temporary features of the proposed scheme have the potential to influence flood risk elsewhere (Figure 3.4 A, B, C and D). These include:

a) Temporary River Till crossing;

b) Site compound areas;

c) Stockpile areas; and

d) Haul routes.

Temporary River Till Crossing

3.3.2 A temporary crossing over the River Till would be constructed to provide access between the tunnel laydown area and Parsonage Down, avoiding use of the existing A303. This crossing would be inaccessible to the public at all times.

3.3.3 The crossing would provide early continuous access along the line of the new works, to permit the movement of excavated material from the eastern side of the River Till to the embankment fill and essential fill areas on the west whilst the permanent Till viaduct is being constructed. The construction of the temporary “Bailey”/Mabey Bridge would permit access to both sides of the River Till for construction personnel, site traffic and material transfer.

3.3.4 The temporary River Till crossing would have a 6m wide running track with 1m verges to provide a single operation lane. The centre of the temporary crossing would be positioned on the south side of the proposed Till viaduct, approximately 60m downstream of that location.

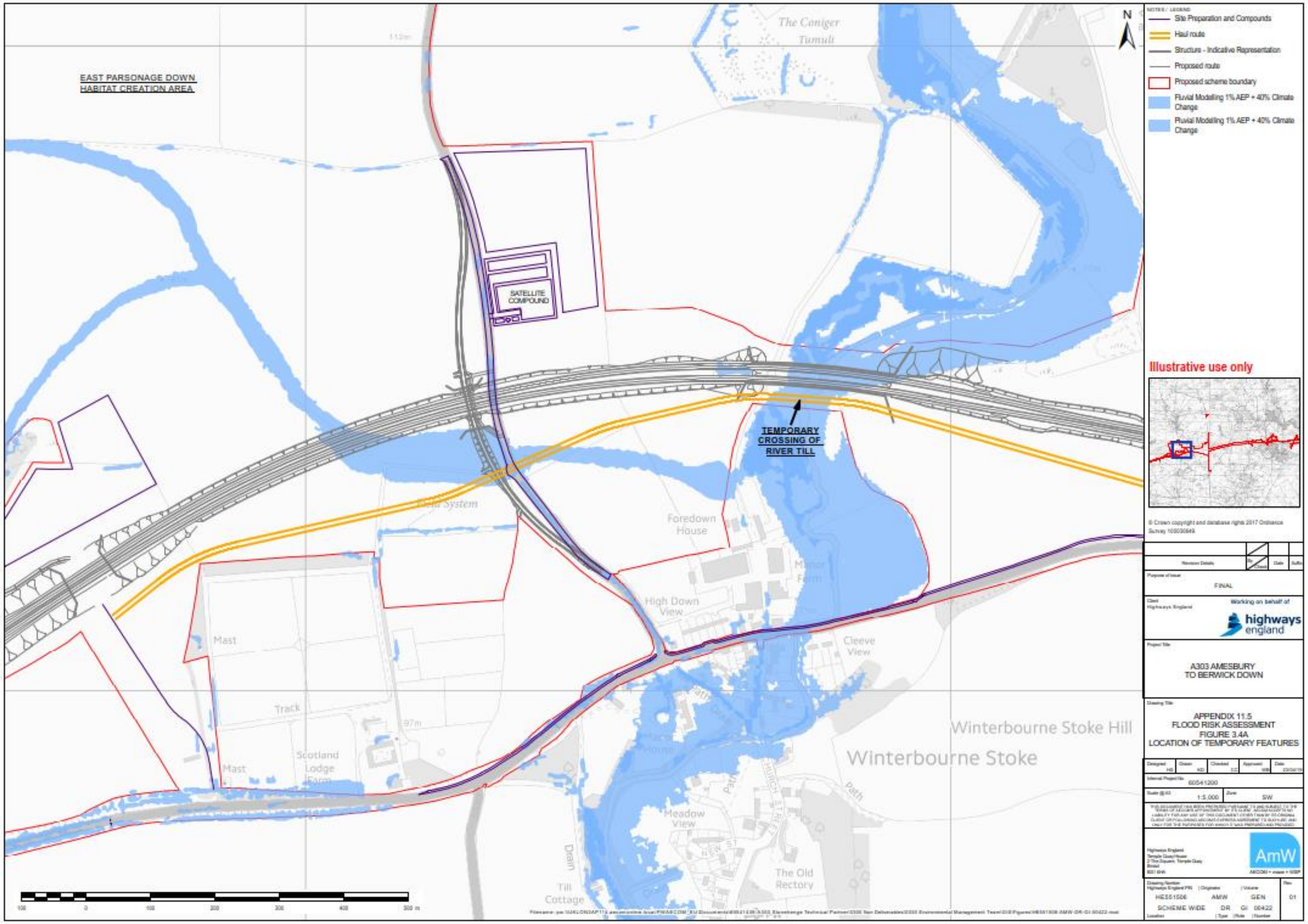


Figure 3.4A: Location of temporary features

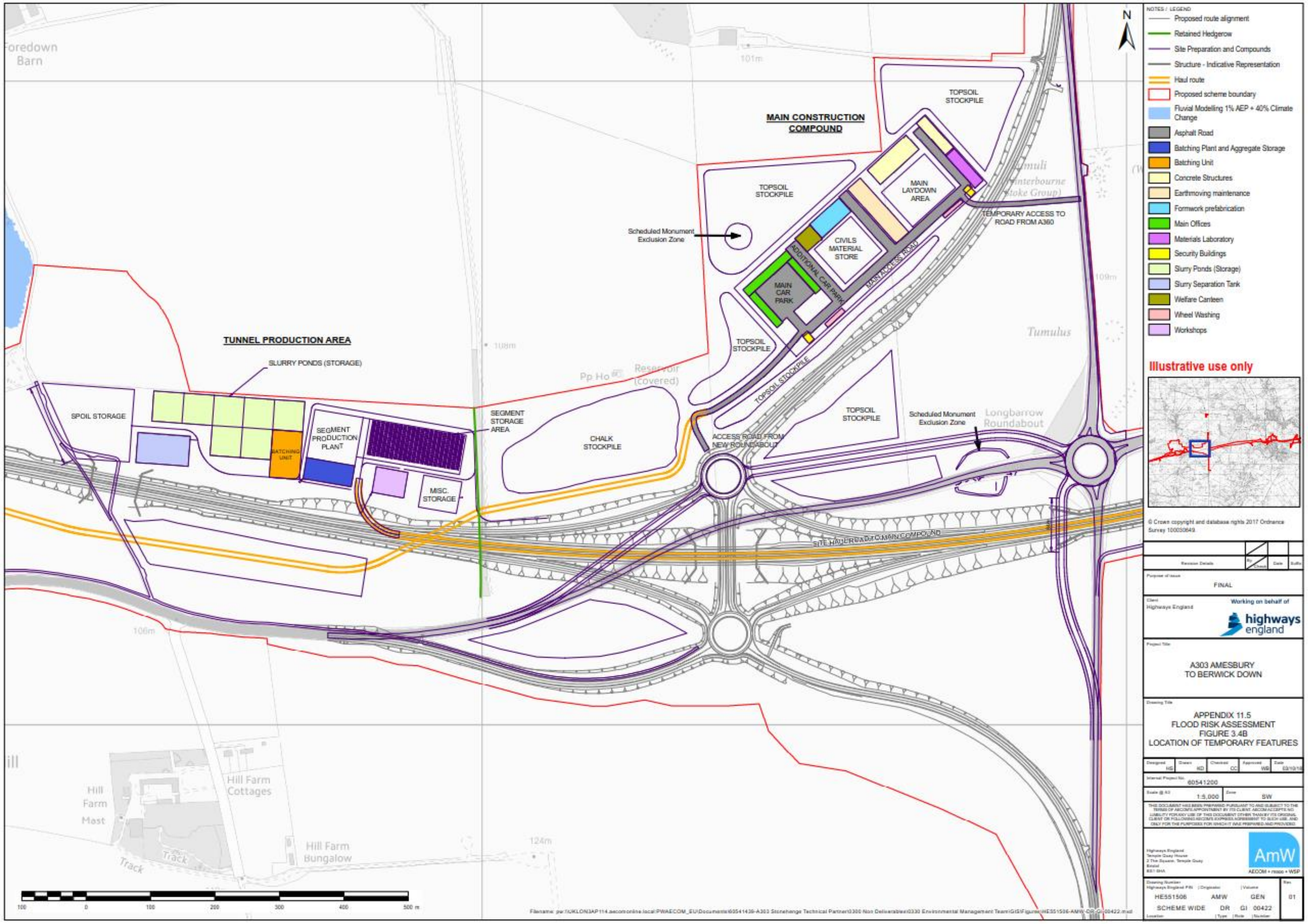


Figure 3.4B: Location of temporary features

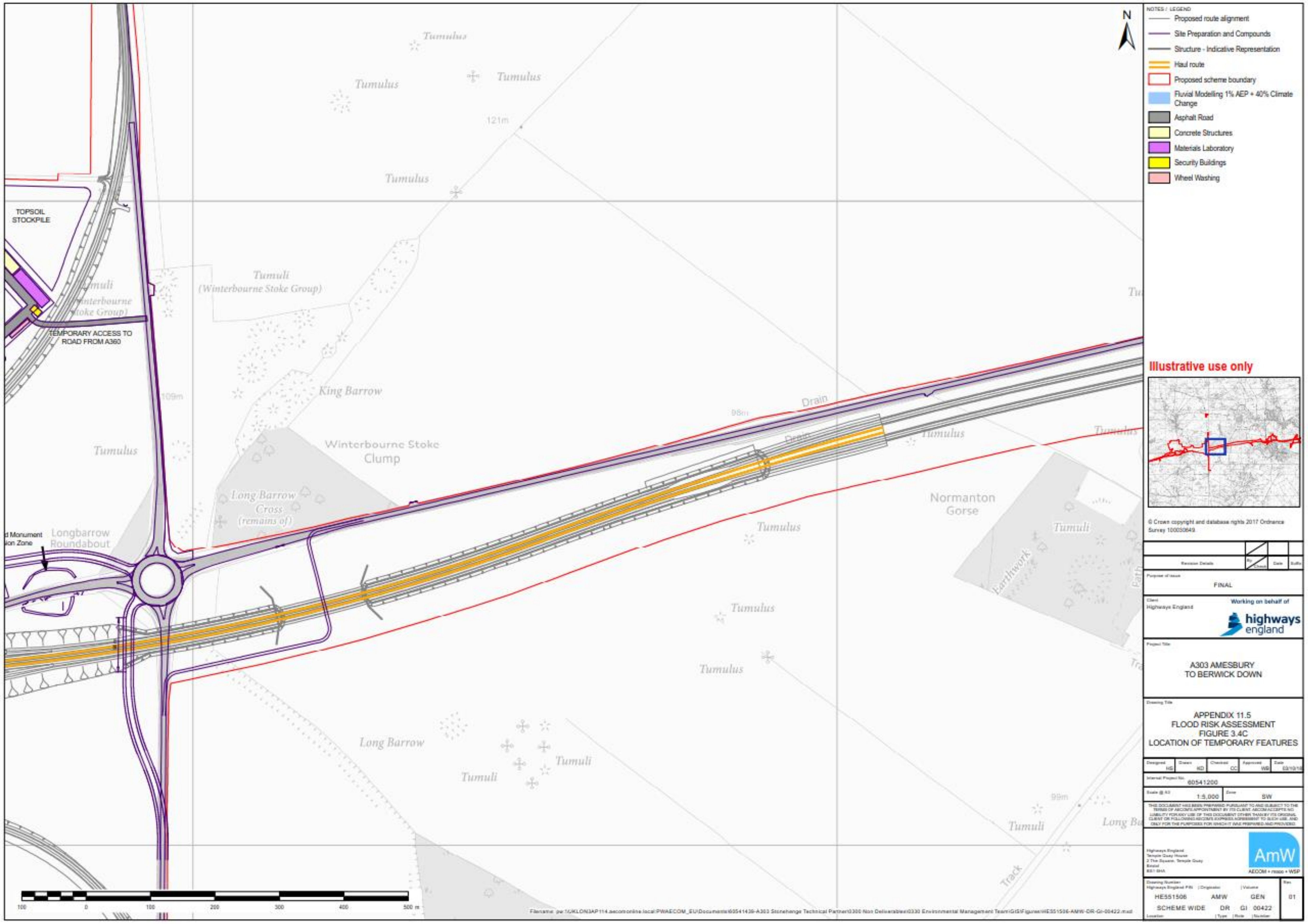


Figure 3.4C: Location of temporary features

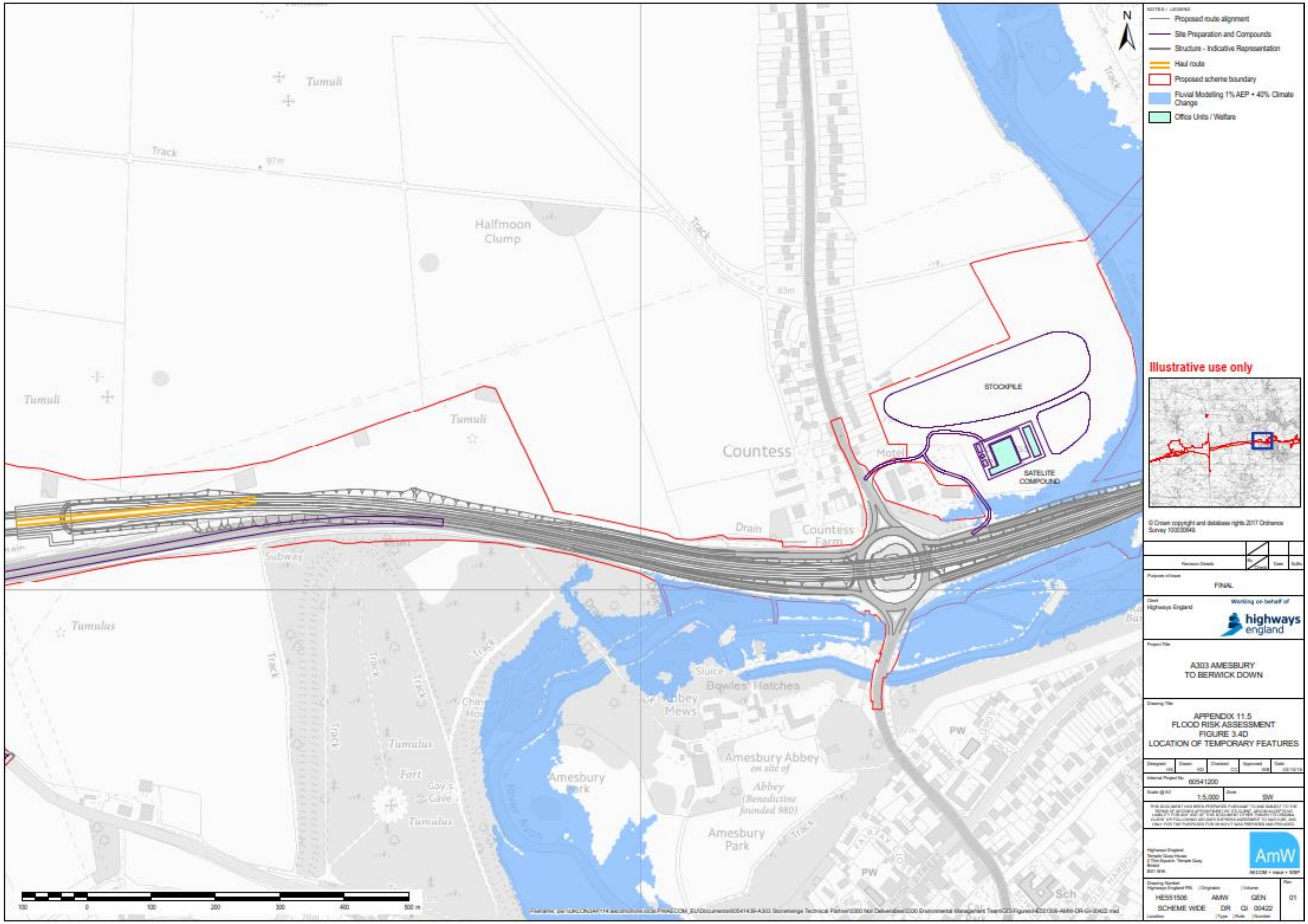


Figure 3.4D: Location of temporary features

3.3.5 Figure 3.5 shows the “Bailey”/Mabey Compact bridge type which would be constructed to span the River Till. The bridge would have a 35m span.

Figure 3.5: “Bailey”/ Mabey Compact Bridge

3.3.6 The temporary bridge crossing would be an open structure within the natural channel area, supported by embankments on the far west and east of the bridge launch areas as opposed to an embankment structure throughout the entirety of the floodplain. The structures would be removed when construction is complete.

3.4 Design Philosophy

3.4.1 Influencing the proposed scheme’s design has been a key consideration to maximise the opportunities for delivering mitigation of flood risk impacts by avoidance and reduction. The opportunities realised to provide embedded mitigation are described in further detail below.

Watercourse Crossings

3.4.2 A number of design influences were incorporated in relation to the watercourse crossings. These included:

a) Selecting a location for the crossing of the River Till that requires the minimum length to create a clear span;

b) Selecting a route that avoids any new crossing of the River Avon; and,

c) The River Till viaduct is proposed to be a five span structure with the location and orientation of the piers and foundations optimised to place them as far away from the River Till as possible and to minimise obstruction of overland directional flow within the floodplain.

Road drainage

3.4.3 The proposed scheme and its drainage measures (Appendix 11.3 of the A303 Amesbury to Berwick Down Environmental Statement) are designed to manage surface water runoff to minimise the risk of causing flooding elsewhere through the use of attenuation features to detain runoff from all events expected to occur with 1% annual exceedance probability (including climate change) or more frequently. The drainage measures comply with the principles of the non-statutory technical standards for Sustainable Drainage Systems (SuDS) (Ref 3.1) and the Design Manual for Roads and Bridges (DMRB) (Ref 3.2).

Other design considerations

3.4.4 A number of other design influences were incorporated into the proposed scheme to minimise the potential impact on flood risk. These included:

a) Siting of permanent facilities outside of the flood zones or surface water flow paths, such as, the operational facilities for the tunnel; and,

b) Avoiding the siting of embankments and cuttings within the known floodplains.

Design Standard

3.4.5 The proposed scheme has been designed to minimise the risk of it flooding by incorporating current design standards and future climate change allowances to improve its resilience. The standards are referred to in Section 3.4.3 and Section 4.

4 Policy Context and Consultation

4.1 National

National Policy Statement for National Networks (NPSNN)

4.1.1 NPSNN sets out the need for, and Government's policies to deliver, development of nationally significant infrastructure projects (NSIPs) on the national road and rail networks in England. NPSNN explains that essential transport infrastructure is permissible in areas of high flood risk, subject to the satisfaction of the National Planning Policy Framework (NPPF) Exception Test (Ref 4.1).

4.1.2 Paragraphs 5.92 and 5.93 of the NPSNN specify that applications for projects in Flood Zones 2 and 3, such as the proposed scheme, should be accompanied by a FRA. The FRA should identify and assess the risks of all forms of flooding to and from the project and demonstrate how these risks will be managed. These requirements are fulfilled in Section 7 (Flood Risk to the Proposed Scheme), Section 8 (Flood Risk from the Proposed Scheme – Temporary Works) and Section 9 (Flood Risk from the Proposed Scheme – Permanent Works) of this report.

4.1.3 Paragraphs 5.94 to 5.95 outline the key considerations in preparing a FRA, including taking into account the effects of climate change over the proposed scheme lifetime, consideration of arrangements for safe access and egress for those using the infrastructure, assessing residual flood risk, and providing evidence of satisfaction of the Sequential and Exception Tests. This FRA assesses the impacts of climate change with regard to fluvial, groundwater and surface water. The proposed scheme is discussed in the context of the Sequential and Exception Tests in Sections 4.1.16 and 4.1.20 respectively.

4.1.4 Paragraph 5.96 emphasises the importance of consultation with the Environment Agency, Lead Local Flood Authorities (LLFAs) and other organisations with a role in flood risk management. This FRA and supporting modelling studies have been informed by detailed and regular consultation with relevant parties. Flood risk data has been gathered and assessment methodologies and approaches to flood risk mitigation have been agreed.

4.1.5 Paragraph 5.97 relates to assessing local forms of flood risk (for example, groundwater and surface water) and points to local flood risk management strategies and surface water management plans as useful sources of information. All available information on local sources of flood risk has been reviewed to inform this FRA.

4.1.6 Other key considerations of the NPSNN include:

a) Managing flood risk through good design. 'This may include the use of sustainable drainage systems but could also include vegetation to help to slow runoff, hold back peak flows and make landscapes more able to absorb the impact of severe weather events' (Paragraph 5.110); and

b) Site layout and surface water drainage systems. Paragraphs 5.112 to 5.114 set out that these systems 'should be designed to cope with events that exceed the design capacity of the system, so that excess water can be safely stored on or conveyed from the site without adverse impacts. Arrangements 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, unless specific off-site arrangements are made and result in the same net effect. It may be necessary to provide surface water storage and infiltration to limit and reduce both the peak rate of discharge from the site and the total volume discharged from the site.'

4.1.7 The proposed road drainage strategy, which has been formed in consultation with the Environment Agency and Wiltshire Council, is described in Appendix 11.3 of the A303 Amesbury to Berwick Down Environmental Statement. It complies with the requirements of the NPSNN.

National Planning Policy Framework (NPPF) and Flood Risk

4.1.8 The NPPF and accompanying 'Planning Practice Guidance' (PPG) (Ref 4.1) set out the Government's planning policies for England and how these are expected to be applied.

4.1.9 The principal aim of the NPPF is to contribute to the achievement of sustainable development. This includes ensuring that flood risk is taken into account at all stages of the planning process, avoiding inappropriate development in areas at risk of flooding and directing development away from those areas where risks are highest. Where development is necessary, it should be safe, without increasing flood risk elsewhere.

4.1.10 New development should also be planned for in ways that avoid increased vulnerability to the range of impacts arising from climate change and that can help to reduce greenhouse gas emissions, such as through its location or design.

4.1.11 A site-specific FRA is required for:

a) Proposals of 1 hectare or greater in Flood Zone 1;

b) All proposals for new development in Flood Zones 2 and 3;

c) Proposals in an area within Flood Zone 1 which has critical drainage problems;

d) Land identified in a strategic flood risk assessment as being at increased flood risk in future; or,

e) Where proposed development or a change of use to a more vulnerable class may be subject to other sources of flooding.

4.1.12 The FRA should identify and assess the risks of all forms of flooding to and from the development and demonstrate how these flood risks will be managed so that the development remains safe throughout its lifetime, taking climate change into account.

4.1.13 Early adoption of and adherence to the principles set out in the NPPF can ensure that proposals take due account of the importance of flood risk and the need for appropriate mitigation, if required.

4.1.14 A sequential, risk-based approach to the location of development taking account of climate change should be undertaken. Residual risk should also be managed by:

a) Applying the Sequential Test and then, if necessary, the Exception Test;

b) Safeguarding land from development that is required, or likely to be required, for current or future flood management;

c) Using opportunities provided by new development to reduce the causes and impacts of flooding (where appropriate through the use of natural flood management techniques); and,

d) Where climate change is expected to increase flood risk so that some existing development may not be sustainable in the long-term, seeking opportunities to relocate the development to more sustainable locations.

4.1.15 The NPPF Sequential Test classifies proposed development into one of four Flood Zones, detailed in Table 4.1.

Table 4.1: Flood Zones (Ref 4.1)

Flood Zone	Annual Exceedance Probability of Flooding (%)	Corresponding Return Period (1 in x year)
1 – Low probability	Fluvial and Tidal <0.1%	>1,000
2 – Medium probability	Fluvial 0.1-1.0% Tidal 0.1-0.5%	1,000-100 1,000-200
3a – High probability	Fluvial >1.0% Tidal >0.5%	<100 <200
3b – Functional floodplain	5.0%	<20

4.1.16 The NPPF, and paragraph 5.105 of the NPSNN, give preference to locating new development in Flood Zone 1 and that the Sequential Test should be applied to demonstrate that there are no reasonably available sites in areas with a lower probability of flooding that would be appropriate to the type of development proposed.

4.1.17 As part of the option selection stage, an appraisal of over 60 different route options was undertaken to inform the selection of the route for the proposed scheme. The route appraisal and selection process involved multi-criteria assessment of the merits of each route against different environmental aspects including consideration of flood risk issues as part of the water environment / water quality and resources appraisal. The relative flood risk of each route, using the Environment Agency fluvial flood zones, was reported in the A303 Amesbury to Berwick Down Scheme Assessment Report (SAR)[2] and the Technical Appraisal Report (TAR)[3].

4.1.18 The SAR and TAR were subject to statutory and public consultation to communicate the wider sustainability benefits of the project beyond flood risk and informed the Secretary of State's decision on selection of the final route for the proposed scheme. The application of the Sequential Test was therefore undertaken through this process.

4.1.19 The NPPF provides guidance on the compatibility of each land use classification in relation to each of the Flood Zones as summarised in Table 4.2.

Table 4.2: Flood Risk vulnerability classification (Ref 4.1)

Flood Risk Vulnerability Classification		Essential Infrastructure	Highly Vulnerable	More Vulnerable	Less Vulnerable	Water Compatible
Flood Zone	1	✓	✓	✓	✓	✓
	2	✓	Exception Test Required	✓	✓	✓
	3a	Exception Test Required	×	Exception Test Required	✓	✓
	3b	Exception Test Required	×	×	×	✓

4.1.20 The Exception Test is a method used to demonstrate that flood risk to people and property will be managed satisfactorily, while allowing necessary development to go ahead in situations where suitable sites at lower risk of flooding are not available. The Exception Test should demonstrate that:

a) the development provides wider sustainability benefits to the community that outweigh flood risk; and,

b) the development will be safe for its lifetime taking into account the vulnerability of its users, without increasing flood risk elsewhere, and, where possible, reduce flood risk overall.

[2] Highways England, 2018. A303 Stonehenge. 2017 Consultation reports (Scheme Assessment Report).

[3] Highways England, 2018. A303 Stonehenge. 2017 Consultation reports (Technical Assessment Report).

4.1.21 According to Table 2[4] within the PPG, the proposed scheme can be classified as 'Essential Infrastructure' in relation to flood risk vulnerability. The definition of Essential Infrastructure is 'essential transport infrastructure (including mass evacuation routes) which has to cross the area at risk'.

4.1.22 Since the proposed scheme is partially located in Flood Zone 3a and 3b, an Exception Test is required. This FRA demonstrates how the proposed scheme meets the requirements of the Exception Test.

4.1.23 Any project that is classified as 'Essential Infrastructure' and proposed to be located in Flood Zone 3a or 3b should be designed and constructed to remain operational and safe for users in times of flood; and any scheme in Flood Zone 3b should result in no net loss of floodplain storage and not impede water flows. This FRA demonstrates how the proposed scheme meets these requirements.

4.1.24 Development should only be allowed in areas at risk of flooding where, in the light of this assessment (and Sequential and Exception Tests, as applicable) it can be demonstrated that:

a) Within the site, the most vulnerable development is located in areas of lowest flood risk, unless there are overriding reasons to prefer a different location;

b) The development is appropriately flood resistant and resilient;

c) It incorporates sustainable drainage systems, unless there is clear evidence that this would be inappropriate;

d) Any residual risk can be safely managed; and,

e) Safe access and egress routes are included where appropriate, as part of an agreed emergency plan.

4.1.25 As discussed in Sections 4.1.20 and 4.1.24, the Exception Test is only required for elements of proposed development (Essential Infrastructure) in Flood Zone 3. The appraisal of the scheme against revised Flood Zone 3 extents is provided below.

4.2 Local

4.2.1 Local policy has also been considered as part of the proposed scheme development. Wiltshire Core Strategy[5] was adopted in January 2015 and the flood risk policy (Core Policy 67) states that 'a*ll new development will include measures to reduce the rate of rainwater run-off and improve rainwater infiltration to soil and ground (sustainable urban drainage) unless site or environmental conditions make these measures unsuitable.'*

[4] Table 2 PPG : https://www.gov.uk/guidance/flood-risk-and-coastal-change#flood-zone-and-flood-risk-tables

[5] Wiltshire Core Strategy 2015. Available from: https://pages.wiltshire.gov.uk/adopted-local-plan-jan16-low-res.pdf. Last Accessed: 16/05/18.

Consultation

4.2.2 The Environment Agency and Wiltshire Council have been consulted throughout the development of the proposed scheme.

4.2.3 Discussion and agreement of approaches and methodologies has been undertaken with the Risk Management Authorities (Environment Agency and Wiltshire Council as the LLFA) to ensure that the assessment of flood risk within the study area is appropriate for the nature and scale of the proposed scheme.

4.2.4 The responses to the statutory consultation that was carried out between January and March 2018, along with separate discussions with stakeholders, have been considered to identify issues raised regarding road drainage and the water environment. Subsequent and ongoing discussions have been held with the Environment Agency, Wiltshire Council, Wessex Water and the Wiltshire South Operational Flood Working Group (OFWG) which includes community representatives.

4.2.5 The Statutory Consultees' responses to flood risk and the A303 Amesbury to Berwick Down Environment Impact Assessment Scoping Report (2017) echoed the issues made in the statutory consultation, outlined below:

a) Provision of an environmental permit for flood risk activities;

b) Uncertainties in groundwater data sampling to be taken into consideration, where duration of recorded data should be extended;

c) Impact of topography amendments proposed should demonstrate relationship between finished ground levels and all sources of flood risk;

d) Demonstration that the proposed scheme would not negatively impact on the floodwater environment, particularly the River Avon SAC and River Till SAC; and,

e) Completion of a Flood Risk Assessment (this document).

5 Flood Risk Assessment Methodology

5.1 Methodology approach

5.1.1 The approach to the FRA is based on the Source-Pathway-Receptor model.

5.1.2 The Source-Pathway-Receptor model firstly identifies the causes or 'sources' of flooding to and from a development. The identification is based on a review of local conditions and consideration of the effects of climate change using Environment Agency guidance. The nature and likely extent of flooding arising from any one source is considered, e.g. whether such flooding is likely to be localised or widespread.

5.1.3 The presence of a flood source does not always infer a risk. It is the exposure pathway or the 'flooding mechanism' that determines the risk to the receptor and the effective consequence of exposure. For example, sewer flooding does not necessarily increase the risk of flooding unless the sewer is local to the site and groundwater levels encourage surcharged water to accumulate.

5.1.4 The varying effect of flooding on the 'receptors' depends largely on the sensitivity of the target. Receptors include any people or buildings within the range of the flood source, which are connected to the sources of flooding by a pathway.

5.1.5 In order for there to be a flood risk, all elements of the model (a flood source, a pathway and a receptor) must be present. Furthermore, effective mitigation can be provided by removing one element of the model, for example by removing the pathway or receptor.

5.1.6 This FRA identifies and assesses the risks of all relevant forms of flooding to and from the permanent works associated with the proposed scheme, but only assesses the risk of flooding from the temporary works, since any risk to the temporary works will be suitably managed by the appointed Contractor through their Construction Environment Management Plan (CEMP) derived from the Outline CEMP.

5.2 Source-Pathway-Receptor

5.2.1 The potential flood sources which could be impacted from the temporary and permanent works of the proposed scheme are identified as:

a) River Avon;

b) River Till;

c) Surface water;

d) Groundwater;

e) Sewers; and

f) Artificial sources (such as reservoirs).

5.2.2 The pathways present or potentially created or modified by the proposed scheme are identified as:

a) Floodplain inundation due to the river levels exceeding the channel capacity;

b) Overland flow paths; and,

c) Flow of groundwater through the Chalk aquifer and superficial deposit aquifers.

5.2.3 The receptors of concern include any people or buildings within the range of the flood source, which are connected to it by a pathway.

5.3 Modelling

5.3.1 Hydraulic modelling was undertaken to support the development of the FRA to provide a more detailed understanding of the baseline flood risk within the study area. The outputs were used to augment existing Environment Agency flood risk mapping and to assess the potential impacts of flood risk to and from the proposed scheme.

5.3.2 The methodology for the fluvial flood risk hydraulic modelling and pluvial modelling (Ref 5.1) was agreed with the Environment Agency and Wiltshire Council. Further detail on the hydraulic modelling methodology is available in Annex 1 Part A and Annex 1 Part B. A detailed hydrology study undertaken to support the hydraulic modelling is available in Annex 2 Part A and Annex 2 Part B.

Fluvial

5.3.3 Approximately 13.5km of the River Till and 14.2km of the River Avon has been modelled using Flood Modeller Pro-TUFLOW as a 1 dimensional (1D) / 2 dimensional (2D) single domain model. The 2D element of the model has a maximum grid size of 5m. Cross sectional survey information has been collected and used to represent the channel geometry and structures within the 1 dimensional (1D) network.

5.3.4 Multiple flood event scenarios were modelled for the River Till and River Avon and were selected to enable a comparison with the existing Environment Agency flood risk mapping data. The modelled scenarios included:

a) 1% AEP event;

b) 0.1% AEP event; and

c) 1% AEP plus climate change event.

5.3.5 There is a significant gap in quantitative calibration and verification data within the River Till catchment, as the watercourse is entirely ungauged within the study area. As such, a quantitative assessment of the accuracy of the model outputs for this watercourse has not been possible. Instead, qualitative validation and liaison with stakeholders has been used to confirm that modelled outputs replicate as closely as possible to flood events experienced (Annex 1, Part A. Section 6.6).

5.3.6 The gauge record at Amesbury was utilised within the FEH hydrological analysis, along with gauges upstream at Upavon, to enhance confidence in peak flow estimates generated for the Avon (Annex 2, Part B) that were applied within the hydraulic model. Further details of qualitative model validation undertaken in order to increase confidence within the River Avon modelling are discussed in Annex 1, Part A Section 6.6.

5.3.7 The peak river flow climate change allowances adopted to consider the impacts on future fluvial flood risk are in accordance with the latest Environment Agency guidance (Ref 5.2). Given that the lifetime of the proposed scheme is expected to be greater than 100 years, the Higher Central estimate was applied to peak flows along with a sensitivity test using the Upper End estimate. For the South West River Basin District, the allowances are detailed in Table 5.1.

Table 5.1: Climate Change Peak River Flow Allowances (Ref 5.2)

River Basin District	Allowance Category	Total potential change anticipated for the '2020s' (2015 to 2039)	Total potential change anticipated for the '2050s' (2040 to 2069)	Total potential change anticipated for the '2080s' (2070 to 2115)
South West	Upper End	25%	***40%***	***85%***
	Higher Central	20%	30%	40%
	Central	10%	20%	30%
	Lower	5%	5%	10%

Surface Water (Pluvial)

5.3.8 The most significant changes in topography associated with the proposed scheme are from the western end through to the Western Portal. In particular, the layout of the new carriageway close to the River Till crossing is likely to intersect a significant surface water flow pathway close to Parsonage Down. Furthermore, the proposed scheme includes regrading of the land at Parsonage Down for landscaping and habitat creation. Accordingly, a pluvial model has been created for the Parsonage Down area, the coverage of which is identified within Figure 5.1 below.

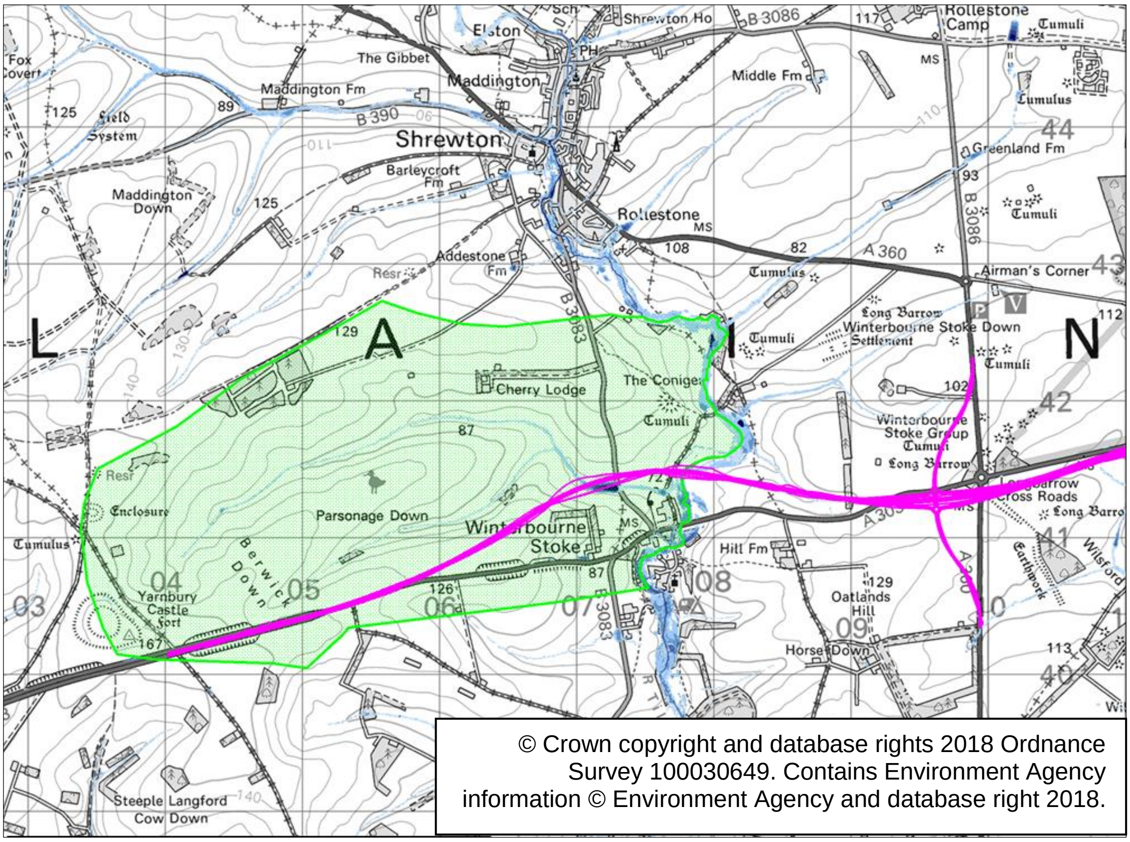


Figure 5.1: Parsonage Down Pluvial Model Extent (Green Outline)

5.3.9 Multiple rainfall event scenarios were modelled, including:

a) 1% AEP event; and

b) 1% AEP plus 40% climate change event.

5.3.10 The peak rainfall intensity climate change allowances adopted to consider the impacts on pluvial flood risk are in accordance with the latest Environment Agency guidance (Ref 5.2). Given that the lifetime of the proposed scheme is expected to be greater than 100 years, the Upper End estimate was applied to peak rainfall intensity (Table 5.2).

Table 5.2: Climate Change Peak Rainfall Intensity Allowances (Ref 5.2)

Applies across all of England	Total potential change anticipated for the ‘2020s’ (2015 to 2039)	Total potential change anticipated for the ‘2050s’ (2040 to 2069)	Total potential change anticipated for the ‘2080s’ (2070 to 2115)
Upper End	10%	20%	***40%***
Central	5%	10%	20%

Groundwater

5.3.11 The existing numerical Wessex Basin model (developed for the Environment Agency) was adapted for use in the study area to support the assessment of the environmental impacts of the proposed scheme to groundwater, including the risk of groundwater flooding which is of interest to the FRA. The model was adapted using MODFLOW and predictions were developed for peak, average and lowest flow/groundwater level conditions within the study area.

5.3.12 The peak flow/groundwater level conditions within the model were modified for climate change predictions by altering the recharge stress period. Two scenarios were modelled, with an increase of 20% and 40%. Further information explaining the modelling results can be found within Numerical Model Report (A303 Amesbury to Berwick Down Environmental Statement Appendix 11.4: Annex 1).

5.3.13 With reference to climate change representation, groundwater and surface water respond differently to rainfall so the rainfall events that are considered in a flood risk assessment can differ. The fluvial model uses a +40% uplift to the peak recorded flow, pluvial modelling uses a +40% uplift of the peak rainfall intensity, while groundwater modelling uses an increase in recharge. Unlike for fluvial and pluvial flood risk where a rainfall event can be specified, for groundwater modelling the proportion of any rainfall event that becomes recharge to the aquifer will vary with the antecedent conditions and the intensity of the rainfall event.

5.3.14 A +20% increase in recharge was applied to the groundwater model. This is equivalent to an average increase of 100mm across the model area over the 2013-14 water year, and therefore is greater than the pluvial model rainfall event. The 2013-2014 recharge event without uplift is estimated to be a 1 in 90 year event. With a +20% increase the event is in excess of a 1 in 2000 year event so by considering a 20% uplift a very extreme event is being simulated.

5.3.15 A further run using a +40 % increase in recharge was applied to the groundwater model. This did not significantly change the assessment of effects as a result of the Scheme. Details are provided in the final version of AS-018 ‘Stage 4 – Supplementary Groundwater Model Runs to Annex 1 Numerical Model Report’

6 Flood Risk Baseline

6.1 Overview

6.1.1 This section provides an overview of the baseline flood risk for the identified sources within the study area.

6.2 Fluvial Flood Risk

Flood Sources

6.2.1 It can be identified from the Environment Agency Flood Map for Planning that fluvial flood risk from the River Avon and River Till are present within the study area, as illustrated in Figure 6.1. The majority of the study area is within Flood Zone 1 (low probability), except where it traverses the two river channels.

Historical flooding

6.2.2 Both the River Till and River Avon catchments have a history of fluvial flooding. Records of historic fluvial flooding events in the study area have been collected from the Environment Agency and Wiltshire Council. Table 6.1 shows a summary of flood events recorded between 1841-present.

Table 6.1: Historic Fluvial Flood Events

Location/Community	Years
River Till	
Orcheston	1841, 1995, 1996, 1999, 2000, 2001, 2003, 2014
Shrewton	1841, 1915, 1960, 1990, 1993, 1995, 1998, 2000, 2001, 2002, 2013
Stapleford	2003
Tilshead	2000, 2001, 2003,
Winterbourne Stoke	1976, 1990, 1995, 1998, 2004
Maddington	1841
River Avon	
Enford	2000, 2001
Netheravon	2000, 2001
Bulford	2014

6.2.3 Data retrieved from the Environment Agency for the river level gauge at Amesbury on the River Avon (during the record period of 1965 to present) shows that the highest recorded water level was 68.05m AOD on January 3rd 2003. The second highest level recorded at this gauge was 68.02m AOD on January 5th 2014. The Environment Agency indicate that flooding is possible where the Amesbury gauge records a water level above 67.72m AOD, which suggests that flooding may have occurred in Amesbury on the two dates stated above.

6.2.4 Many communities were affected in Wiltshire during the winter of 2013–2014. Extreme rainfall events in combination with high groundwater levels during the winter of 2013–2014, meant that the fluvial levels in the River Till exceeded culvert outfall levels causing water to ‘back-up’ through the drainage network, leading to public highway and property flooding.

6.2.5 Highways England’s Drainage Data Management System (HEDDMS) contains information on seventeen events where flooding affected the current A303 between Winterbourne Stoke and Amesbury. These occurred in 2006, 2007, 2010, 2013, 2014 and 2015. Of these, 15 were rated with a severity of between 0 – 4 out of 10 and two rated as 5 out of 10. The severity is rated by Highways England using the following factors: impact on traffic, duration of impact, road classification and annual average daily traffic for one carriageway. Information on the sources of these flood events is not noted in HEDDMS but 11 are located within the floodplain of the River Avon and one within the floodplain of the River Till.

Baseline Hydraulic Modelling

River Till

6.2.6 The outputs of the baseline hydraulic modelling for the 1% AEP flood event, 0.1% AEP flood event and 1% AEP plus 40% climate change for the River Till are presented in Figures 6.2, 6.3 and 6.4, respectively.

6.2.7 In order to compare the Environment Agency Flood Zones against site specific hydraulic modelling the corresponding extents for the same flood event have been overlaid. These are presented for the 1% AEP flood event in Figure 6.5 and the 0.1% AEP flood event in Figure 6.6.

6.2.8 For the 0.1% AEP event, the compared flood extents seen in Figure 6.6 are similar; however, there are three main areas where the baseline modelled flood zone has shown a greater flood extent than the Environment Agency’s model. These are located within Shrewton near Elston and in Winterbourne Stoke downstream of the existing A303.

6.2.9 There are also areas where the flood extent is smaller in comparison, such as the area to the North of Winterbourne Stoke and East of Orcheston.

6.2.10 In terms of flood extent, the River Till 1% AEP (Figure 6.2) and 1% AEP plus climate change (40%) (Figure 6.4) events are very similar to the Environment Agency Flood Zone 2 (0.1% AEP event).

6.2.11 The current Environment Agency Flood Map for Planning Flood Zone outlines at this location are based on JFLOW modelling which uses less refined input data.

6.2.12 The River Till fluvial modelling results for this study have been reviewed by the Environment Agency and it has been agreed that the baseline scenario against which the proposed situation will be ‘measured’ will utilise site specific hydraulic modelling of the River Till as opposed to the Environment Agency Flood Map for Planning data.

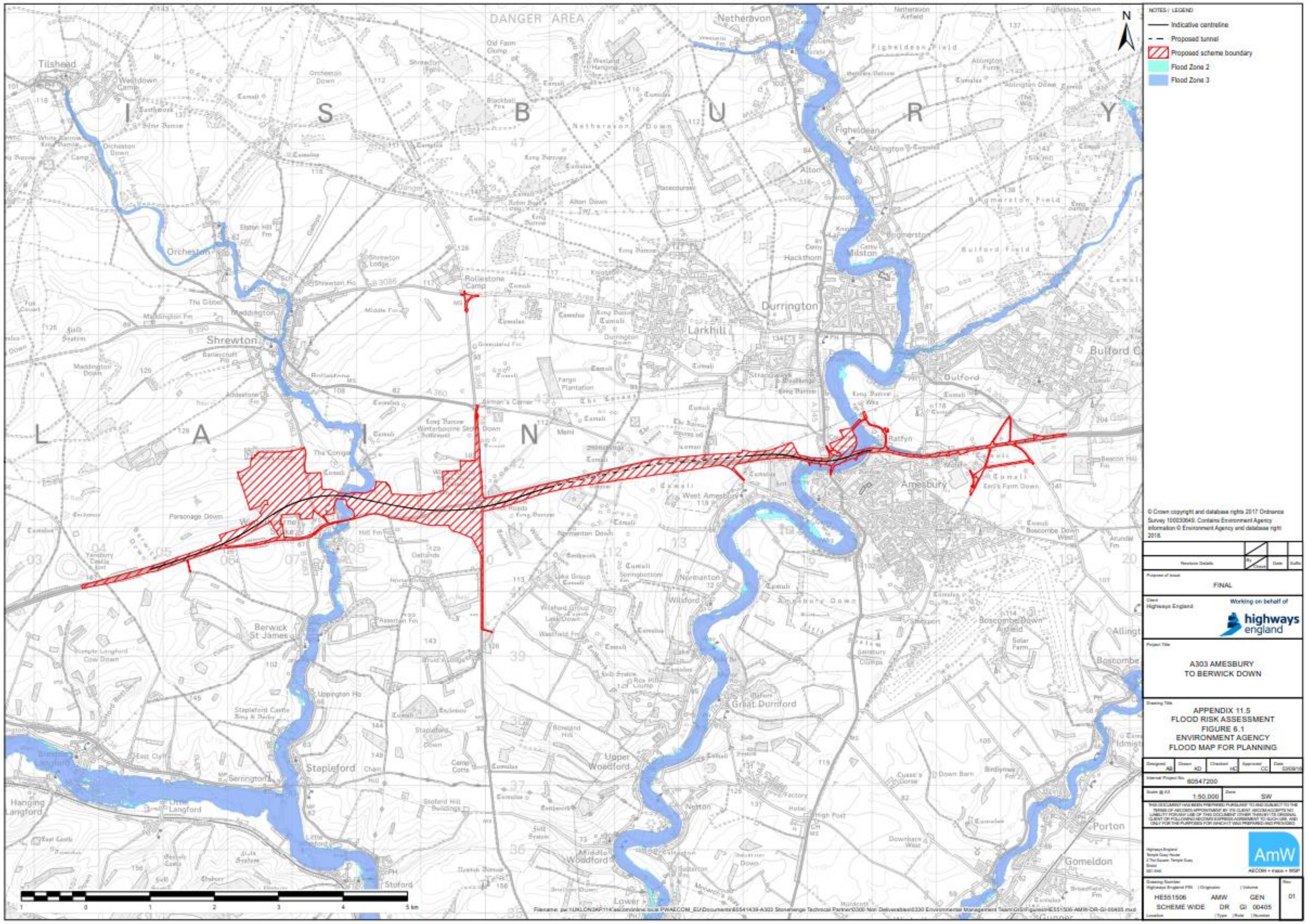


Figure 6.1: Flood Map for Planning (Source: Environment Agency)

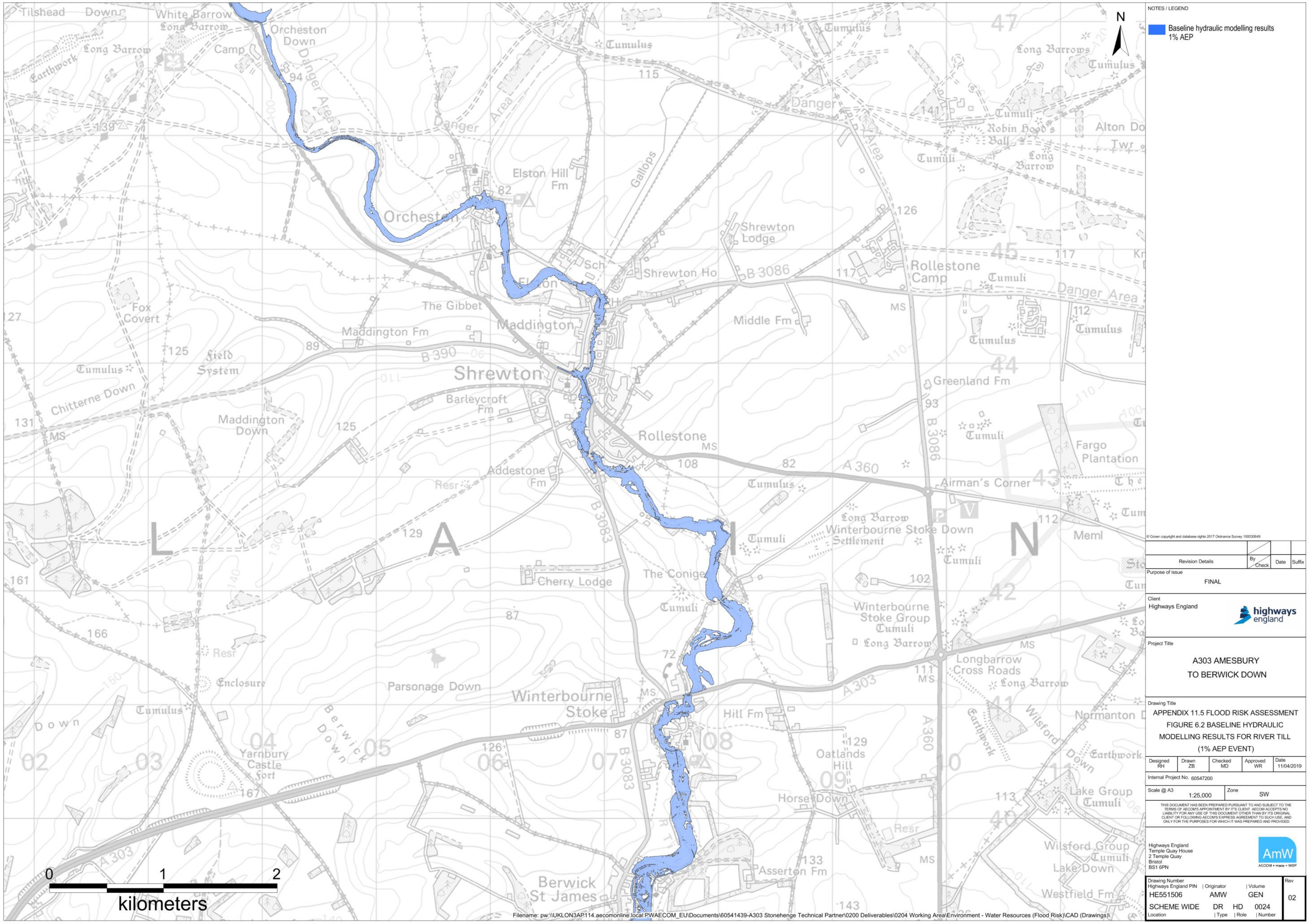


Figure 6.2: Baseline hydraulic modelling results for River Till (1% AEP event)

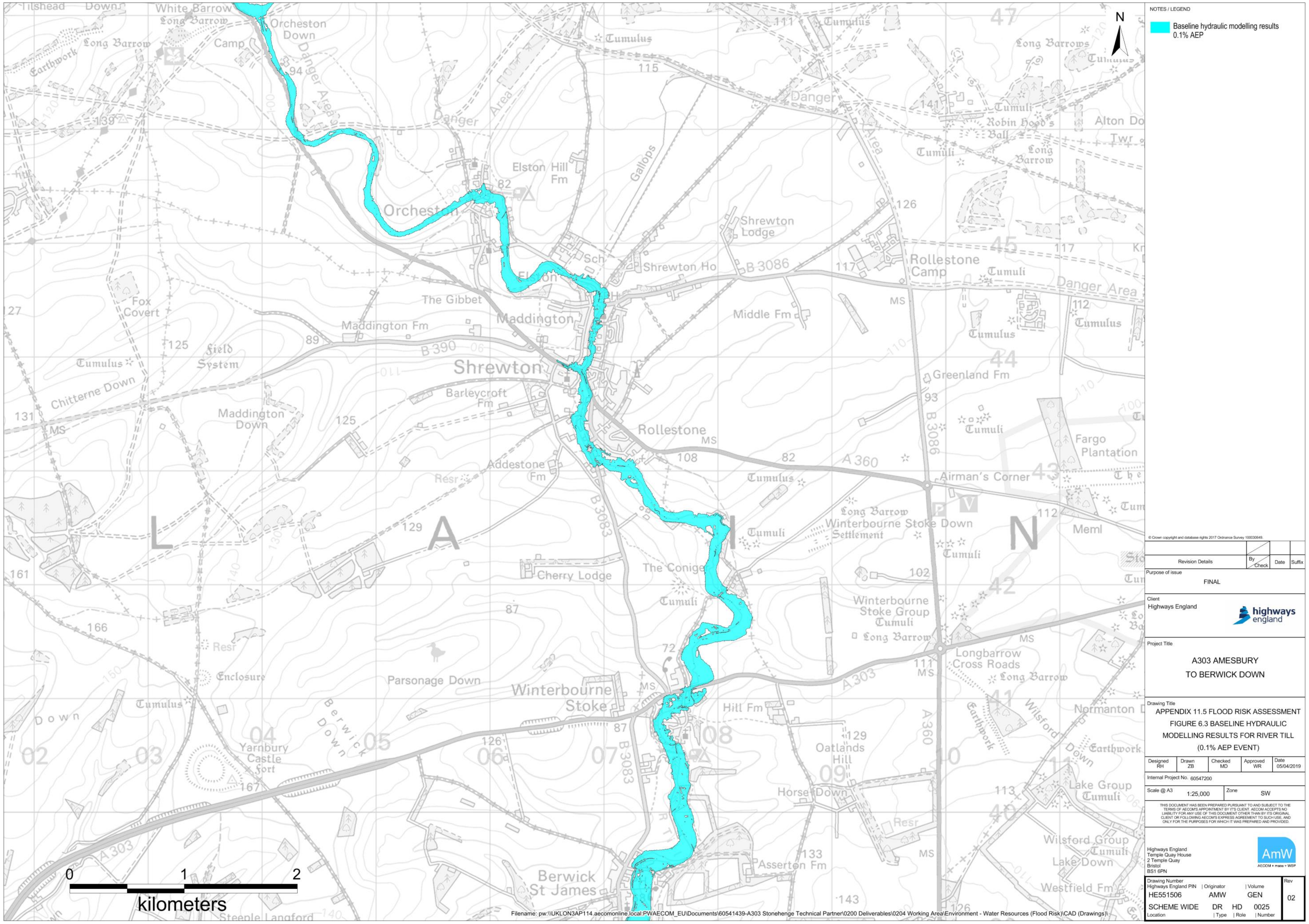


Figure 6.3: Baseline hydraulic modelling results for River Till (0.1% AEP event)

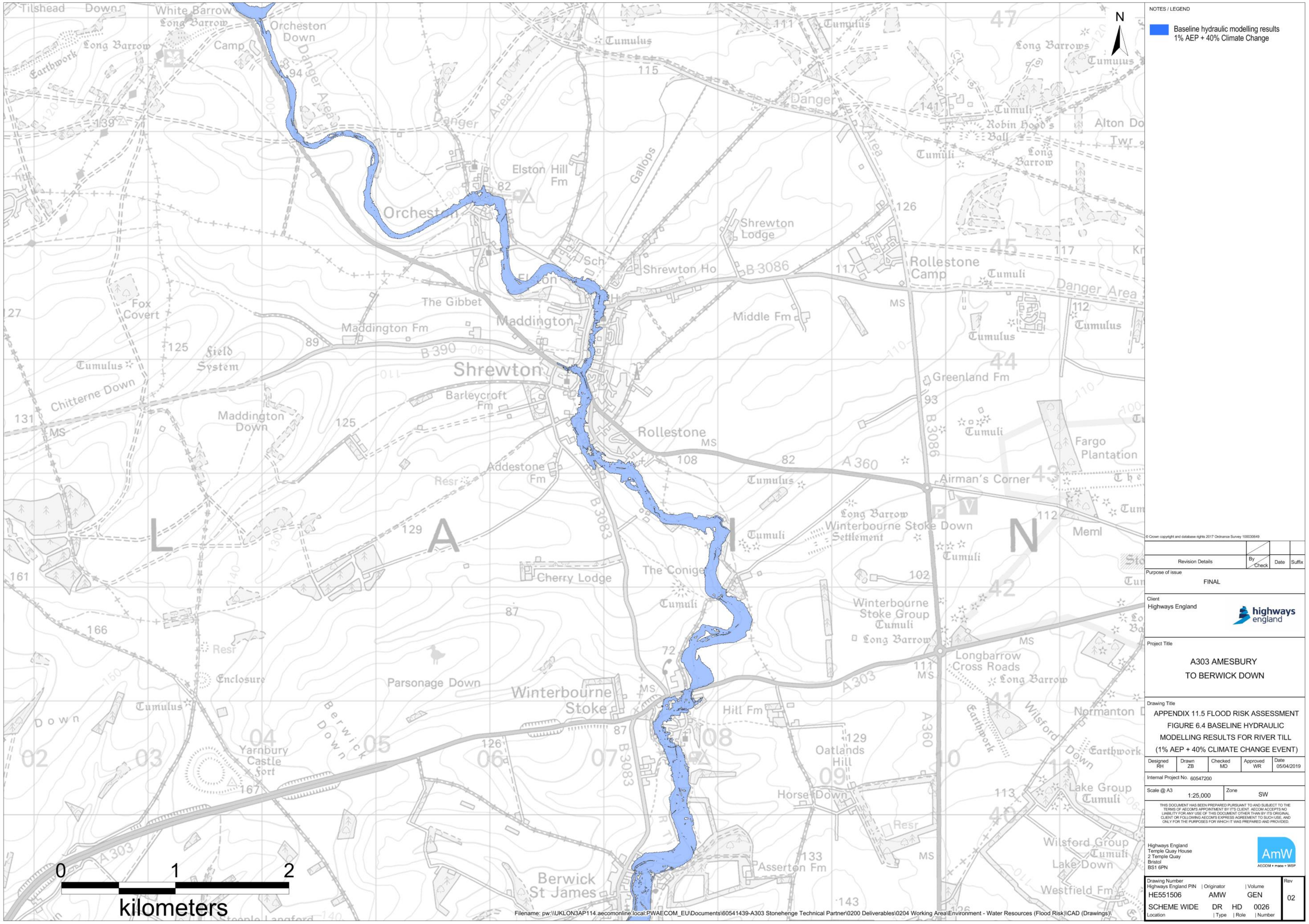


Figure 6.4: Baseline hydraulic modelling results for River Till (1% AEP + 40% climate change event)

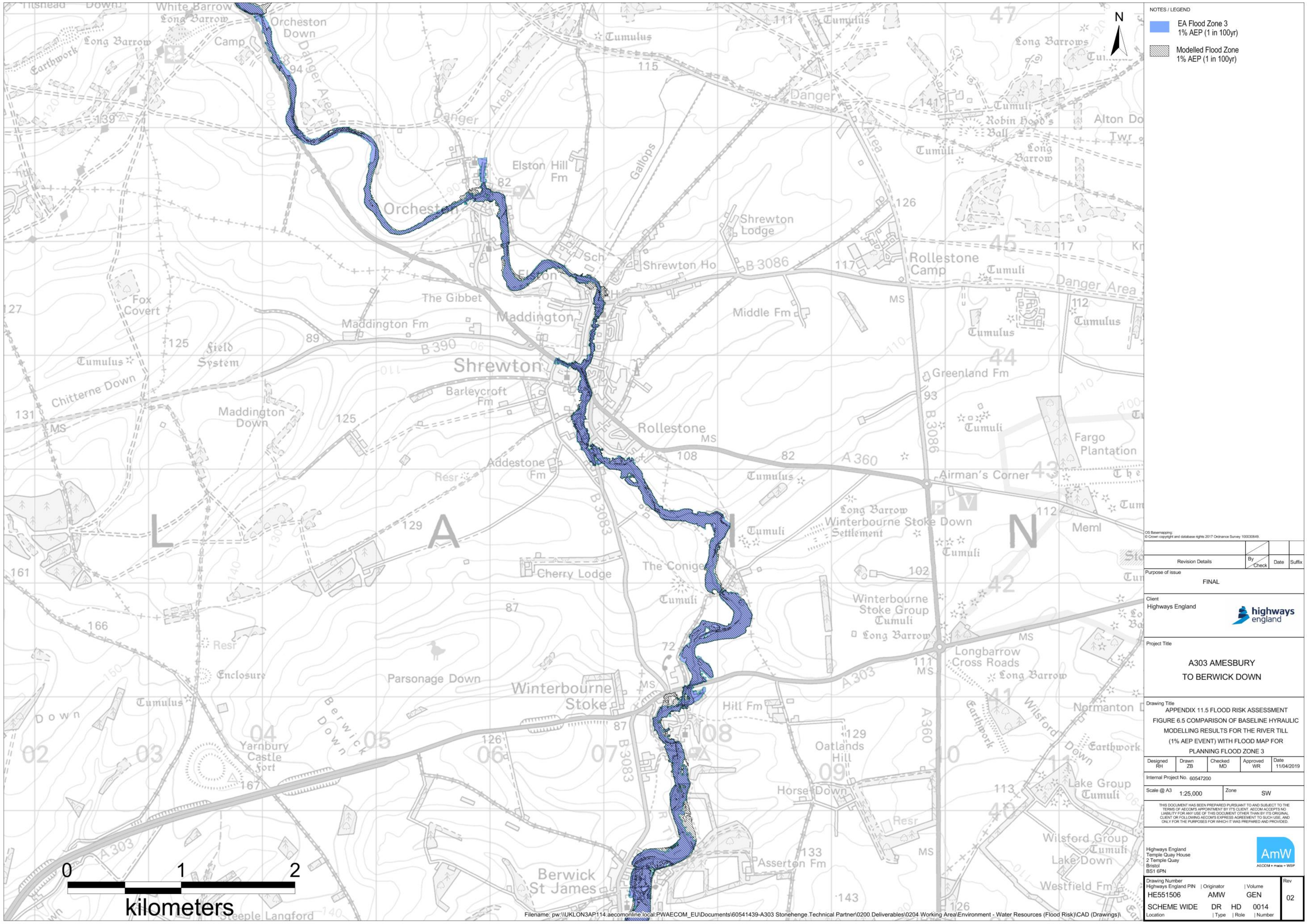


Figure 6.5: Comparison of baseline hydraulic modelling results for River Till (1% AEP event) with Flood Map for Planning (Flood Zone 3)

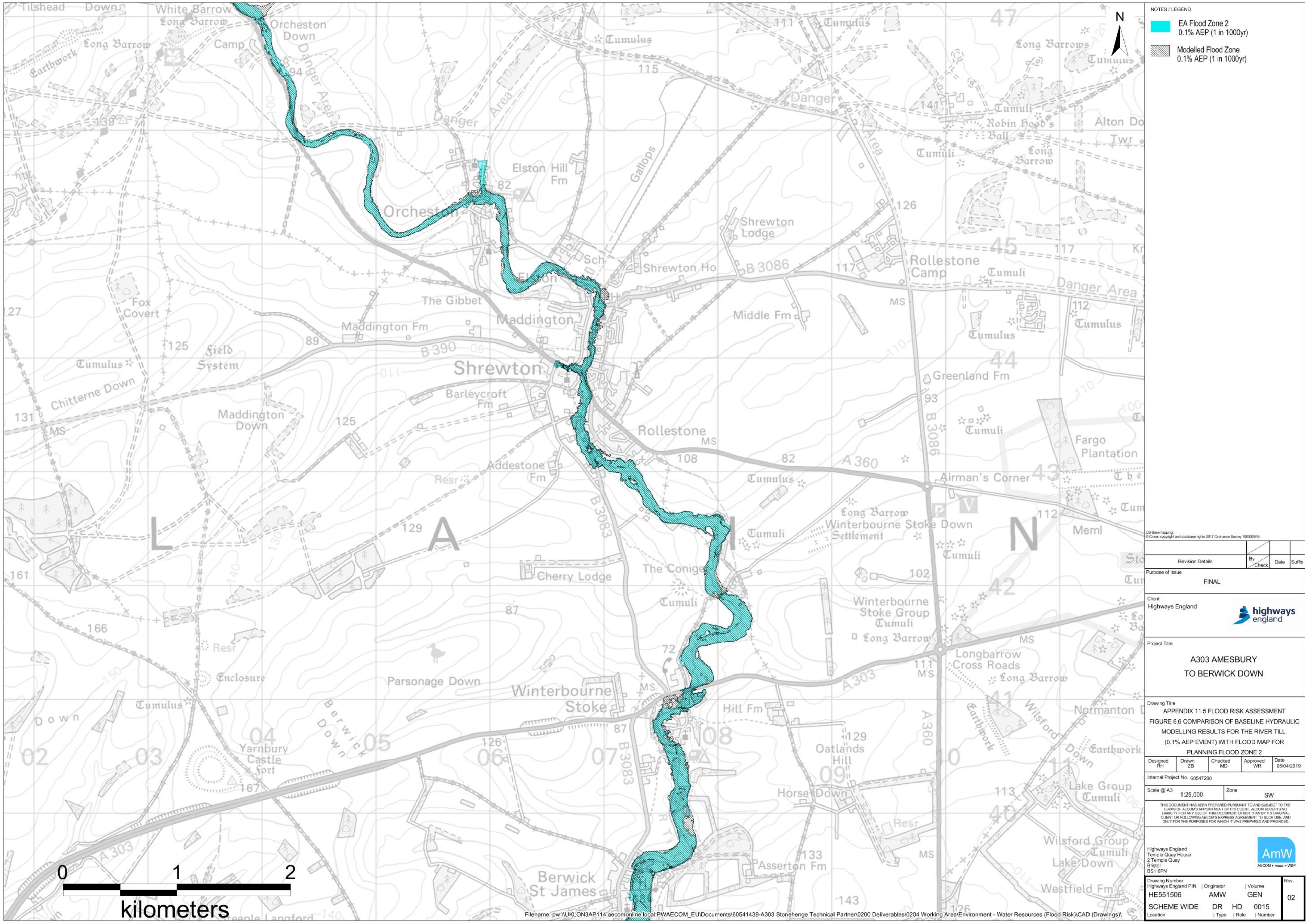


Figure 6.6: Comparison of baseline hydraulic modelling results for River Till (0.1% AEP event) with Flood Map for Planning (Flood Zone 2)

River Avon

6.2.13 The outputs of the baseline hydraulic modelling for the 1% AEP flood event, 0.1% AEP flood event and 1% AEP plus climate change (+40%) for the River Avon are presented in Figure 6.7, 6.8 and 6.9, respectively.

6.2.14 In order to compare the Environment Agency Flood Zones against site specific hydraulic modelling the corresponding extents for the same flood event have been overlaid. These are presented for the 1% AEP flood event in Figure 6.10 and the 0.1% AEP flood event in Figure 6.11.

6.2.15 For the 1% AEP event, the compared flood extents seen in Figure 6.10 show a slight decrease in fluvial modelling extents at a number of locations within the area of interest, particularly within Amesbury Park and west and southwest of Bulford, as well as East of Durrington. To the south of the existing A303, the model results suggest that there is a reduction in extents within the floodplain of the River Avon.

6.2.16 For the 0.1% AEP event, the compared flood extents seen in Figure 6.11, shows a slight decrease in fluvial modelling extents.

6.2.17 In terms of flood extent, the River Avon 1% AEP plus climate change (+40%) event (Figure 6.9) is very similar to the Environment Agency Flood Zone 2 (0.1% AEP event).

6.2.18 Site specific fluvial modelling baseline results have been reviewed by the Environment Agency for the River Avon. It has been agreed that the site specific hydraulic modelling results for the study area, which are locationa dn project specific, will be used to represent the baseline scenario in place of the Environment Agency Flood Map for Planning which uses more generalised strategic input data.

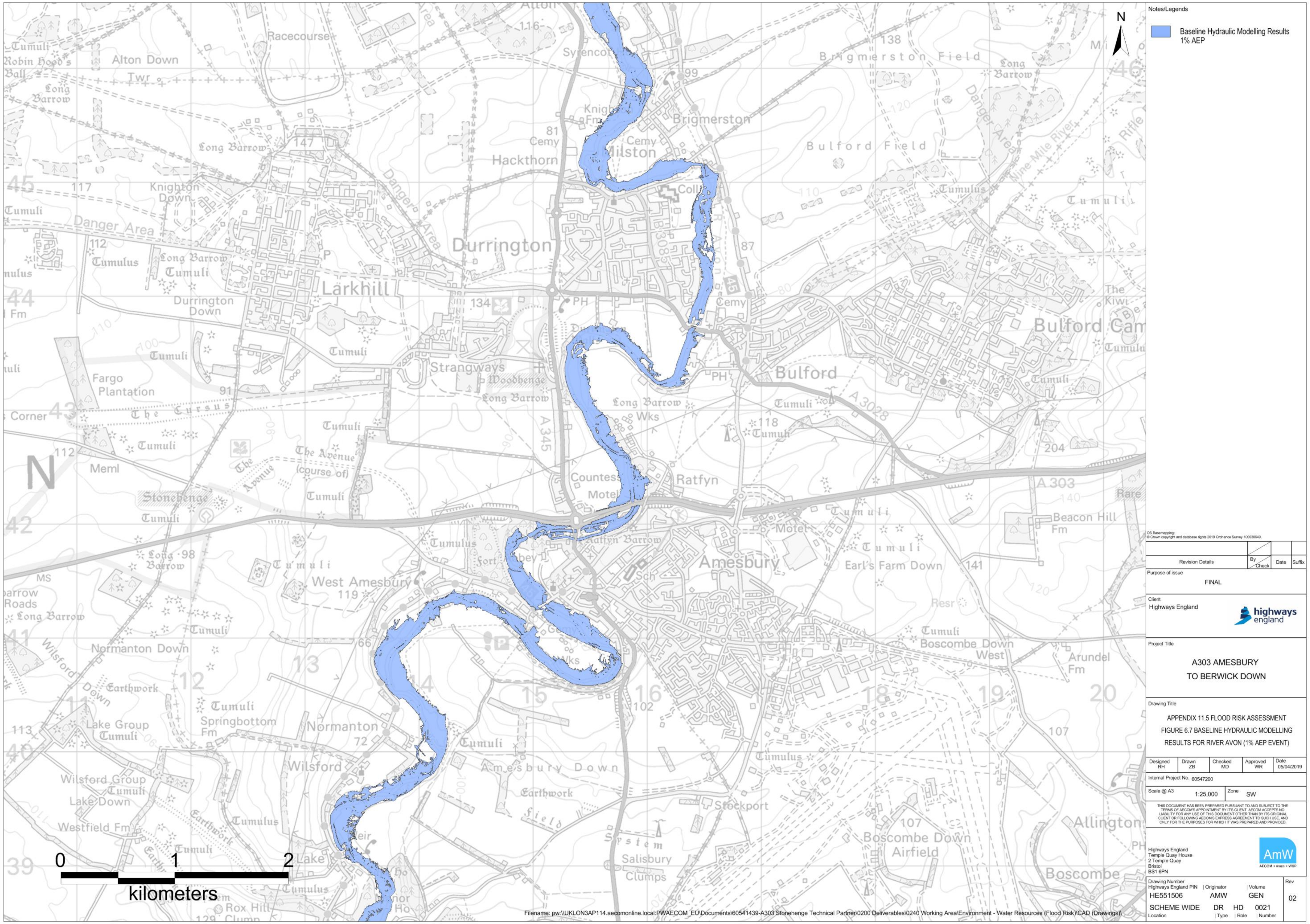


Figure 6.7: Baseline hydraulic modelling results for River Avon (1% AEP event)

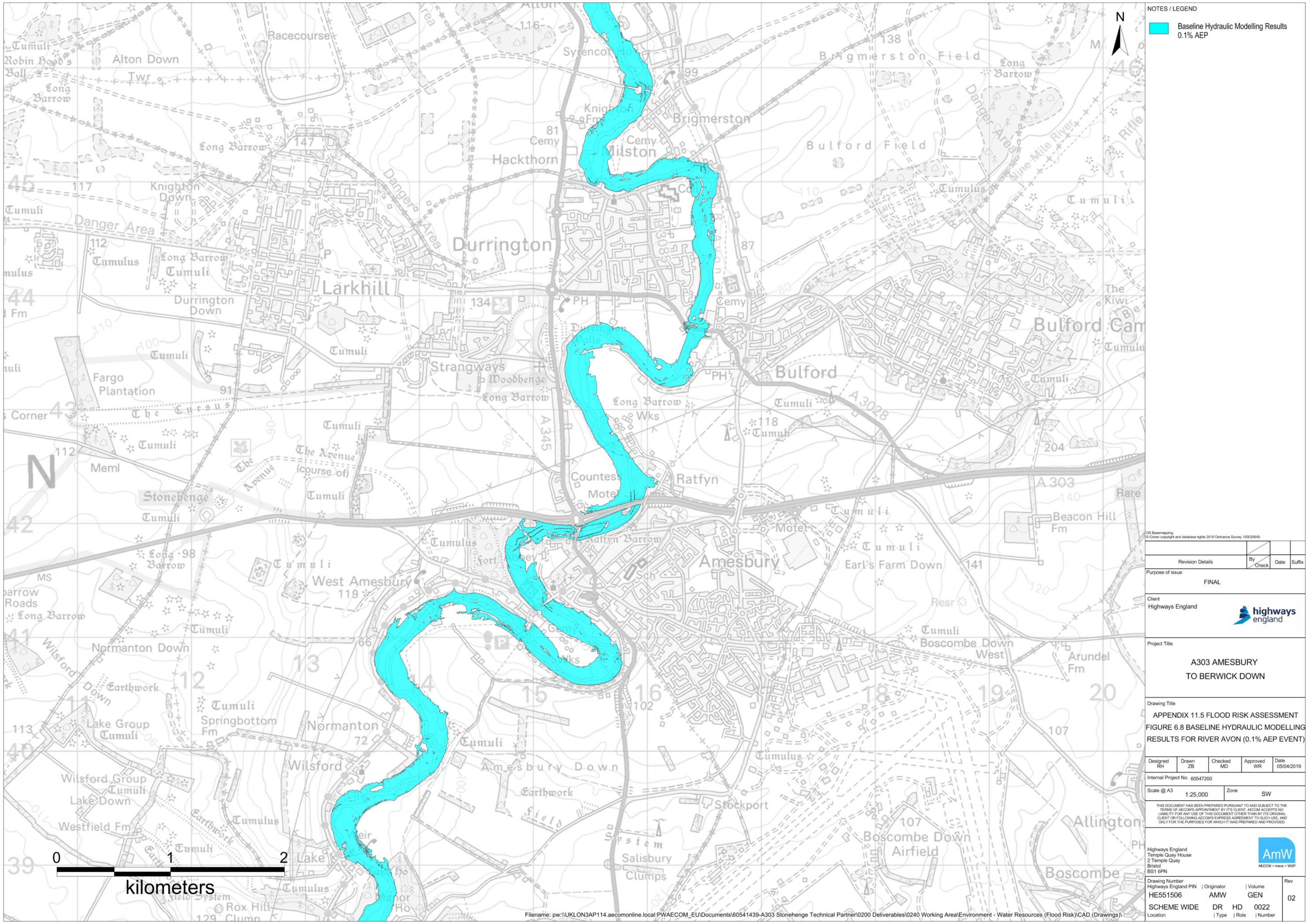


Figure 6.8: Baseline hydraulic modelling results for River Avon (0.1% AEP event)

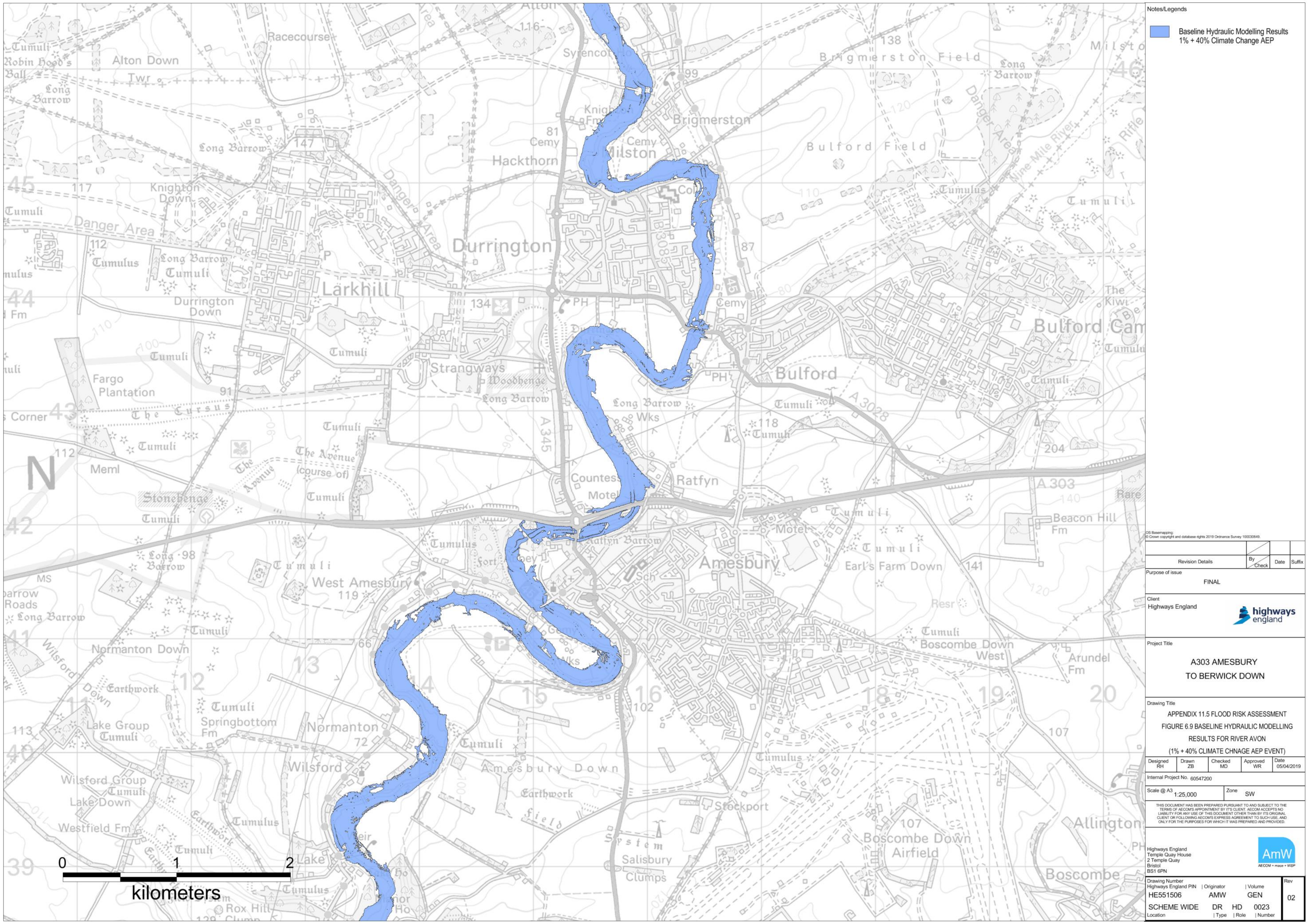


Figure 6.9: Baseline hydraulic modelling results for River Avon (1% AEP + 40% climate change event)

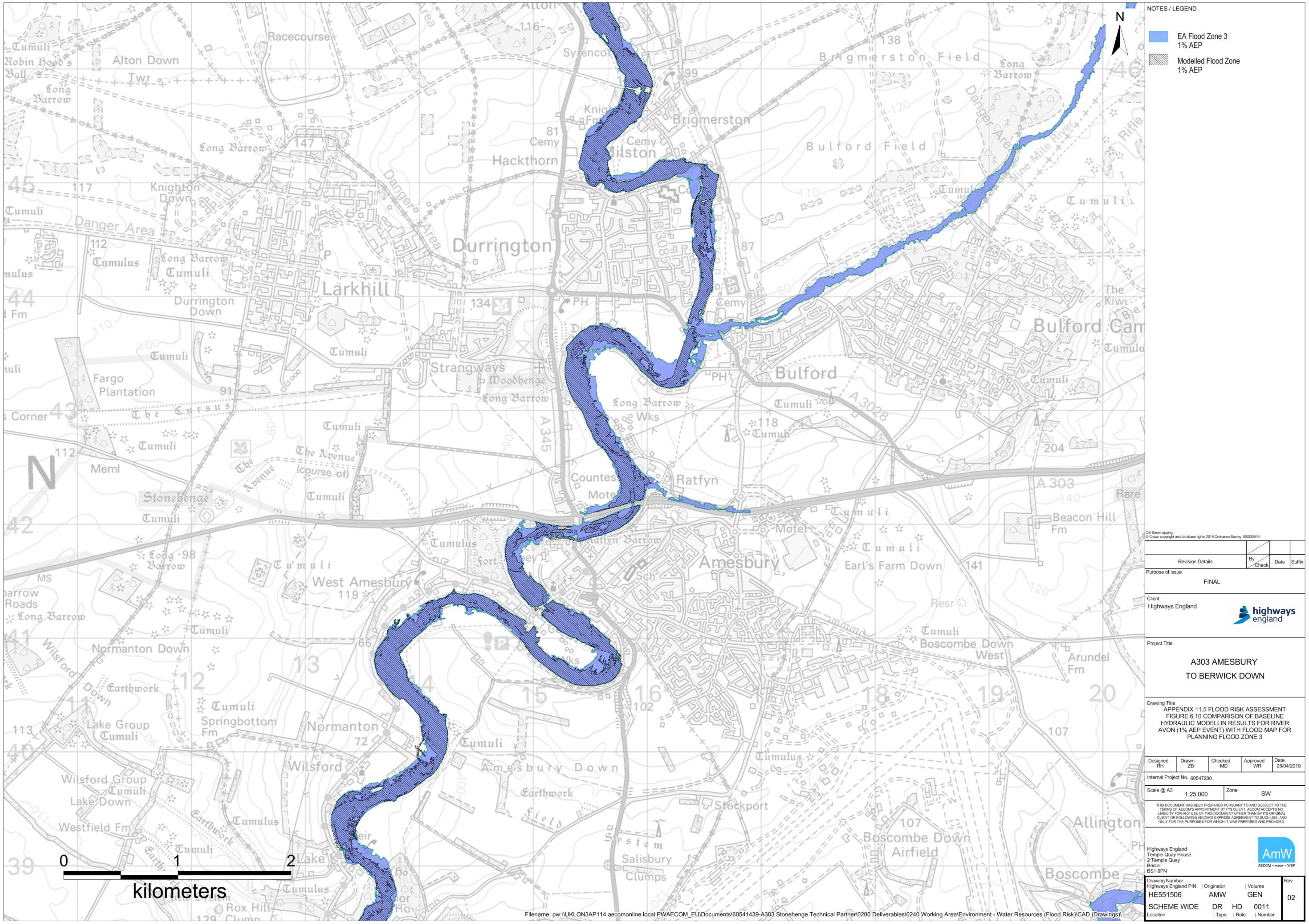


Figure 6.10: Comparison of baseline hydraulic modelling results for River Avon (1% AEP event) with Flood Map for Planning (Flood Zone 3)

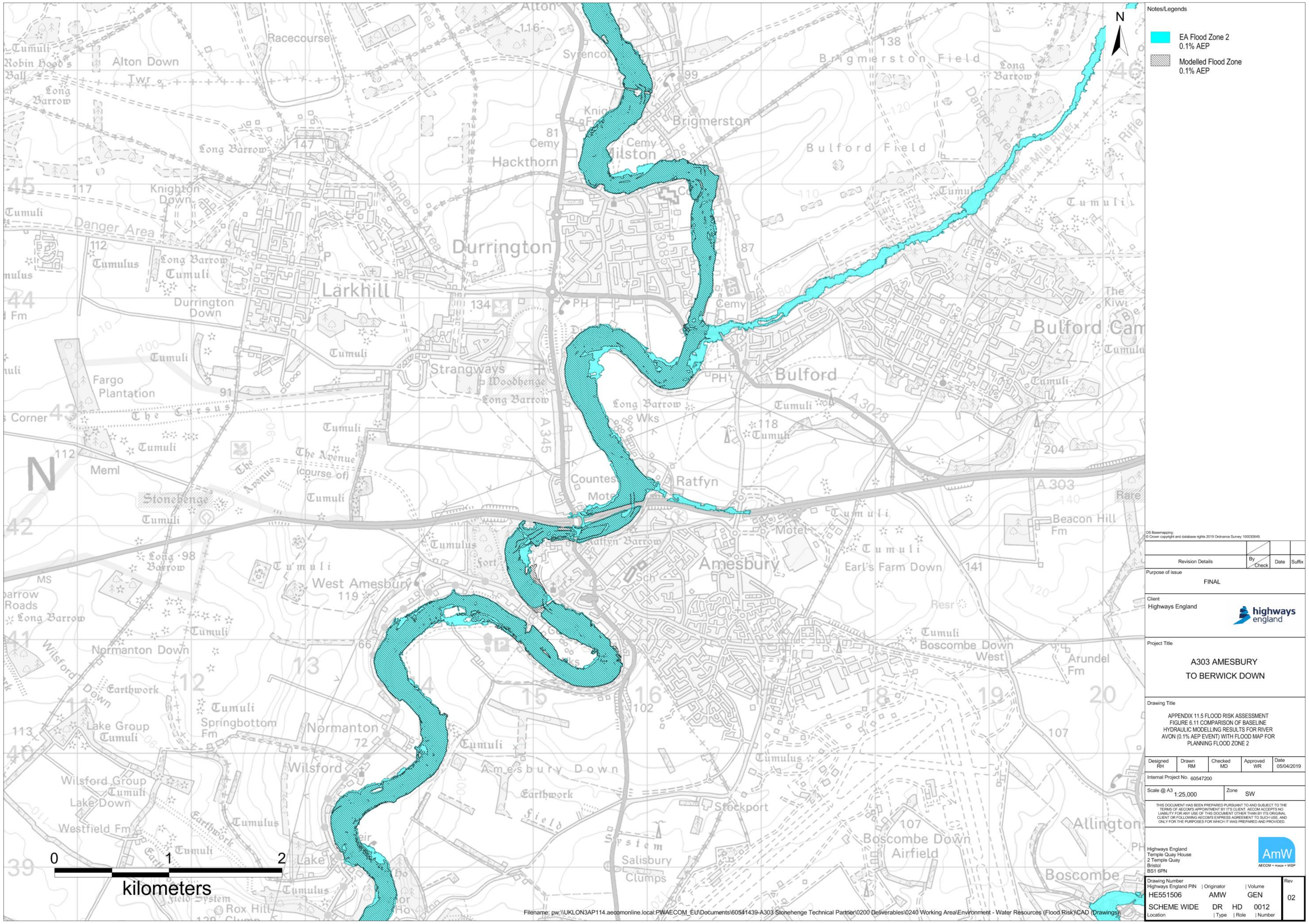


Figure 6.11: Comparison of baseline hydraulic modelling results for River Avon (0.1% AEP event) with Flood Map for Planning (Flood Zone 2)

6.3 Surface Water Flood Risk

Flood Sources

6.3.1 It can be identified from the Environment Agency Flood Risk from Surface Water (FRfSW) mapping that areas at risk from surface water flooding are present within the study area, as illustrated in Figure 6.12.

6.3.2 The majority of the surface water flood risk in the study area is categorised as 'Very Low' (less than 0.1% AEP) or 'Low' (between 0.1% and 1% AEP), with some relatively small areas at 'Medium' (between 1% and 3.3% AEP) or 'High' (greater than 3.3% AEP). The areas at Medium and High risk are typically in the dry valleys such as Stonehenge Bottom, or the River Till and River Avon valleys (in coincidence with fluvial floodplains) and where surface water flow paths are impeded by artificial structures such as existing road embankments and other man-made structures.

Historical flooding

6.3.3 Records of historic surface water flooding events in the study area have been collected from the Environment Agency and Wiltshire Council. A summary of these is provided in Table 6.2.

Table 6.2: Historic Surface Water Flood Events

Location/Community	Years
River Till	
Orcheston	1986, 1998, 2003, 2014
Salisbury Plain military camps	1912
Shrewton	1841, 1915, 1960, 1990, 1993, 1995, 1997, 2001, 2004
Tilshead	1986, 1992, 1993, 1995, 1999, 2000, 2001, 2003, 2014
Chitterne	1986, 1992, 2003
River Avon	
Durrington	1980, 2008
Amesbury	1999
Enford	2000, 2001, 2003
Great Durnford	1977
Wilsford-Cum-Lake	1995

6.3.4

6.3.5 Historic data has also identified rapid snow melt run-off over still frozen ground as a potential source of surface water flood risk in the study area. When these circumstances occur, the impermeable nature of the frozen ground results in the meltwater flowing overland, discharging throughout the River Avon and River Till catchments.

Baseline Pluvial Modelling

6.3.6 Due to proposed changes to the local topography to the area to the west of the River Till, at Parsonage Down, confirmatory pluvial modelling has been completed in this area. Please refer to Annex 1 Part B for the pluvial hydraulic modelling report.

6.3.7 The outputs of the Parsonage Down baseline pluvial modelling for the 1% AEP flood and 1% AEP plus climate change (+40%) events are presented in Figure 6.13 and 6.14, respectively.

6.3.8 In order to compare the Environment Agency FRfSW against site specific hydraulic modelling the corresponding extents for the same flood event have been overlaid. These are presented for the 0.1% AEP flood event in Figure 6.15 and shows that the flood extents for the 0.1% AEP results and the Environment Agency's 'Low' surface water risk outline are comparable.

6.3.9 The baseline site specific pluvial model for the Parsonage Down area for both the 1% AEP and 1% AEP plus climate change (+40%) rainfall events, have pluvial flood extent outlines which are comparable to the Environment Agency surface water flood maps, as shown by Figure 6.12, through the dry valley at Parsonage Down.

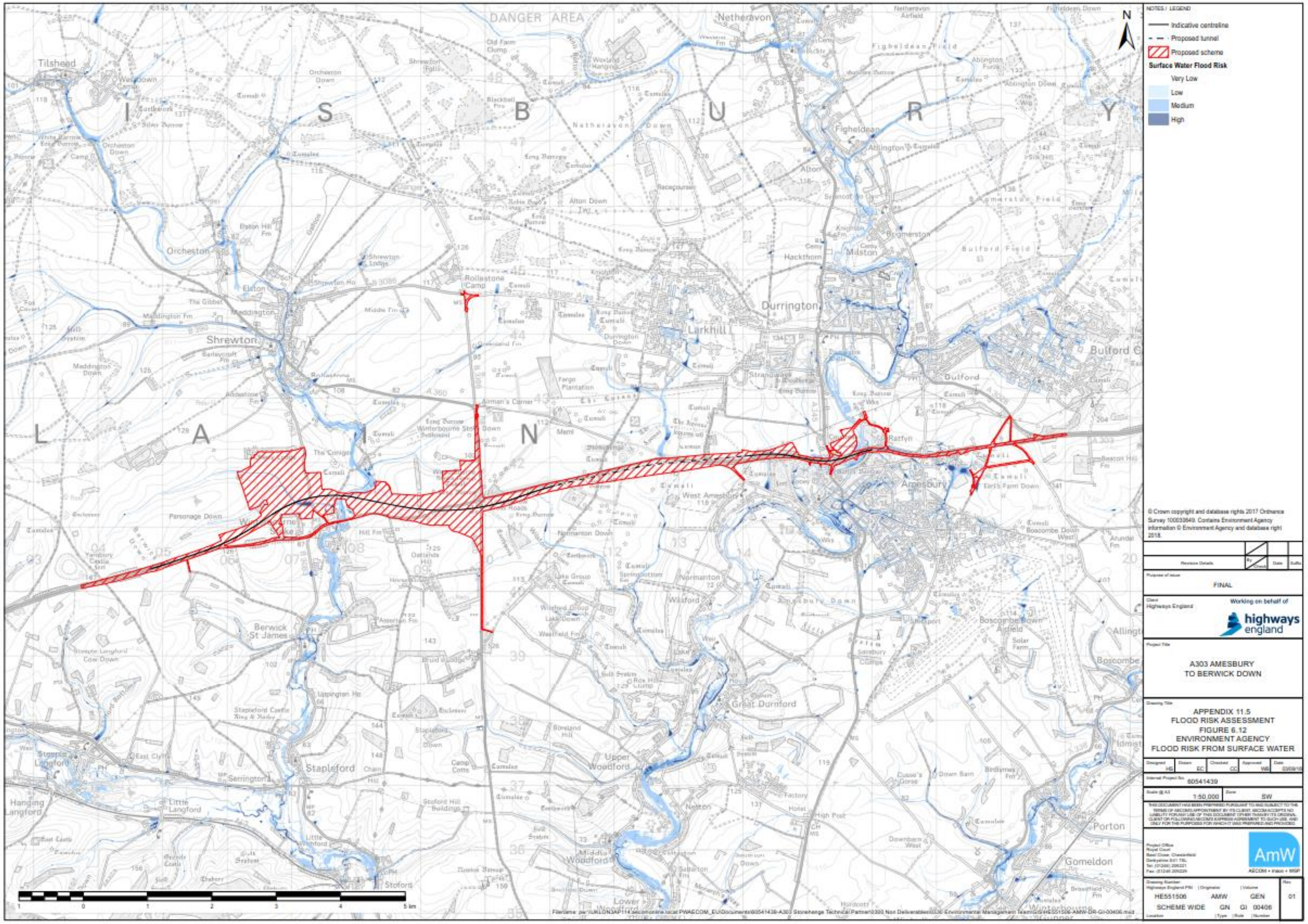


Figure 6.12: Flood Risk from Surface Water Mapping (Source: Environment Agency)

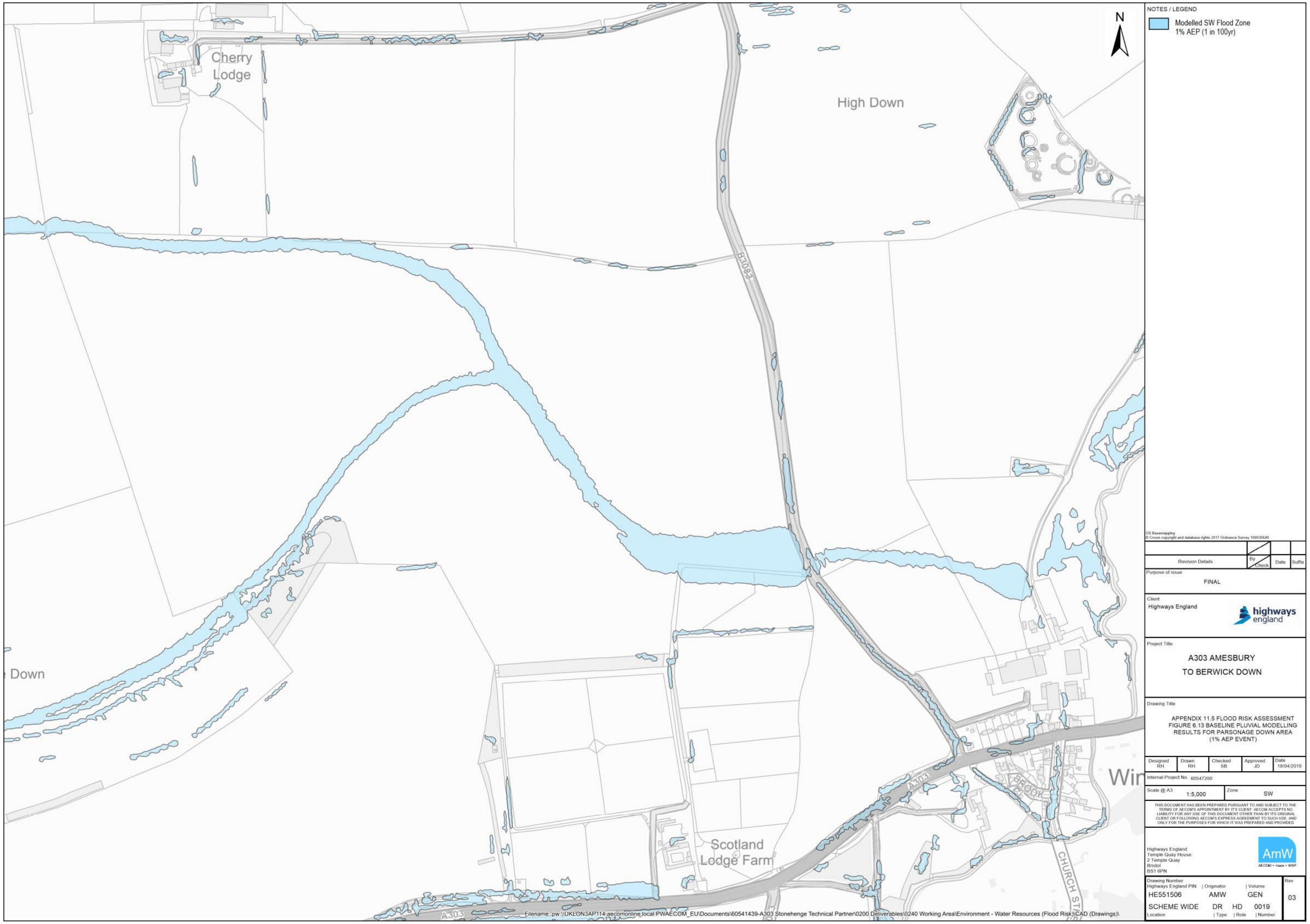


Figure 6.13: Baseline pluvial modelling results for Parsonage Down area (1% AEP event)

Figure 6.14: Baseline pluvial modelling results for Parsonage Down area (1% AEP + 40% climate change event)

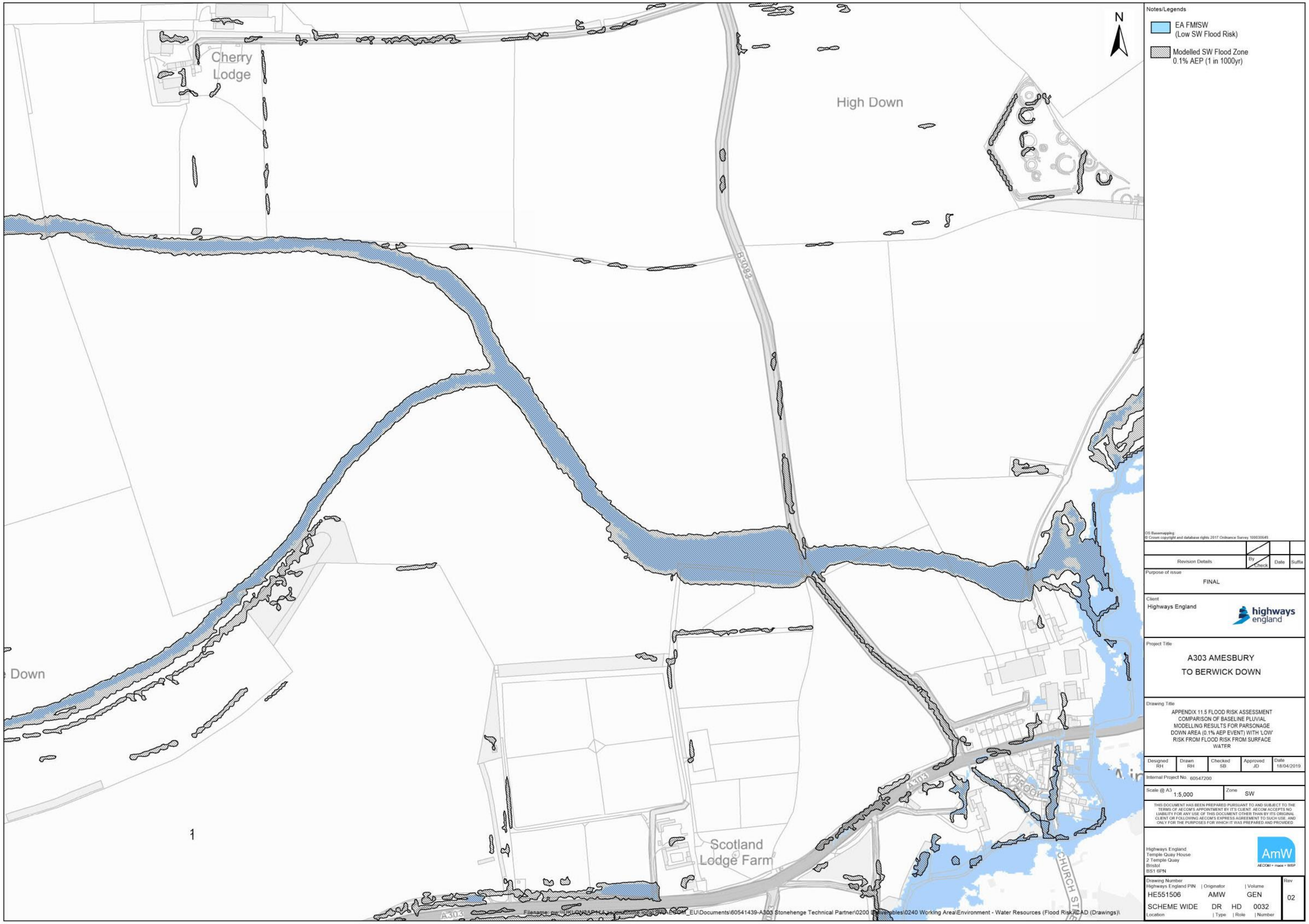


Figure 6.15: Comparison of baseline pluvial modelling results for Parsonage Down area (0.1% AEP event) with 'Low' risk from Flood Risk from Surface Water

6.4 Groundwater Flood Risk

Flood Sources

6.4.1 Groundwater flooding occurs when groundwater rises and emerges at ground level rather than by direct rainfall or surface water runoff. The Wessex Basin model predicts in a number of areas, along the rivers and in dry valleys such as Stonehenge Bottom, where peak groundwater levels can rise above the ground level and therefore groundwater flooding is likely to occur.

6.4.2 There are four groundwater flooding mechanisms that may exist in the study area:

a) *Water table elevation in the Chalk aquifer rising above the ground surface:* groundwater flooding during periods of elevated groundwater levels results in the water table rising above the ground surface, via springs and seepages: such that the flooded area is a representation of the groundwater table. This occurs in locations such as at Stonehenge Bottom, Spring Bottom Farm and Lake.

b) *Water table in the Chalk aquifer induced groundwater floods by increasing baseflow:* water table rises in the Chalk aquifer in the catchments of the River Avon and its tributaries can result in the flowing of ephemeral springs and streams, some of which rarely flow, resulting in greater river flows downstream.

c) *Superficial aquifers along the River Avon and its tributaries:* flooding may be associated with alluvium and the river terrace deposits where they are in hydraulic continuity with surface watercourses. Stream levels may rise following high rainfall events but still remain “in-bank”, and this can trigger a rise in groundwater levels in the adjacent superficial deposits. The properties at risk from this type of groundwater flooding are probably limited to those in the vicinity of the watercourses, with basements / cellars, which have been constructed within the superficial deposits.

d) *Superficial aquifers in various locations:* a second mechanism for groundwater flooding associated with superficial deposits occurs when they are not connected to surface watercourses. Perched groundwater tables can exist within these deposits (river terrace deposits and head (gravel deposits), developed through a combination of natural rainfall recharge and artificial recharge e.g. leaking water mains. The properties at risk from this type of groundwater flooding are probably limited to those with basements / cellars; and in close proximity to the course of the River Avon and its tributaries.

6.4.3 It is also important to consider the secondary impacts of higher groundwater levels on other types of flooding, for example high groundwater levels within the Chalk mean there is less floodwater storage and therefore there is a higher risk of pluvial and fluvial flooding. High groundwater levels can also flood storm sewers making them ineffective.

Historical flooding

6.4.4 Records of historic groundwater flooding events in the study area have been collected from the Environment Agency and Wiltshire Council. Table 6.3 shows a summary of the years and locations in which these flood events have been reported.

Table 6.3: Summary of Historical Reporting of Flooding

Location/Community	Years
River Till	
Berwick St James	2014
Orcheston	1990, 1992, 1993, 1995, 1996, 1998, 1999, 2000, 2003, 2013, 2014
Salisbury Plain military camps	1912
Shrewton	1841, 1915, 1960, 1990, 1993, 1995, 2000, 2001, 2003, 2014,
Stapleford	2003
Till Valley	1986, 1990, 1995, 2003
Tilshead	1951, 1977, 1992, 1993, 1995, 1999, 2000, 2001, 2003, 2014
Winterbourne Stoke	1990, 1995, 1998, 2004
River Avon	
Durrington	2008
Enford	1994, 1995, 2000, 2001, 2002, 2003, 2004, 2005, 2014
Haxton	2006
Netheravon	Prior to 2001 (Specific Year Unconfirmed)
Wilsford-Cum-Lake	2003
Woodford (Flooding also noted in Lower and Upper Woodford without a date)	2014

6.4.5 Information provided by Wessex Water states that groundwater flooding of their sewer network has occurred in Tilshead, Orcheston, Shrewton and Berwick St James. In these locations an Infiltration Reduction Plan, and an Operational Management Action Plan, are in place that are actioned when there is a risk of flooding.

Baseline Groundwater Modelling

6.4.6 The groundwater model developed for this study predicts in a number of areas, along the rivers and in dry valleys such as Stonehenge Bottom, that peak groundwater levels can be above the ground level and therefore groundwater flooding is likely to occur, as shown in Figure 6.16. The areas reporting likely historical groundwater flooding are consistent with the locations where the peak modelled groundwater levels are predicted to be above ground level. For more detailed information the reader is referred to A303 Amesbury to Berwick Down

Environmental Statement Appendix 11.4- Groundwater Numerical Modelling Report.

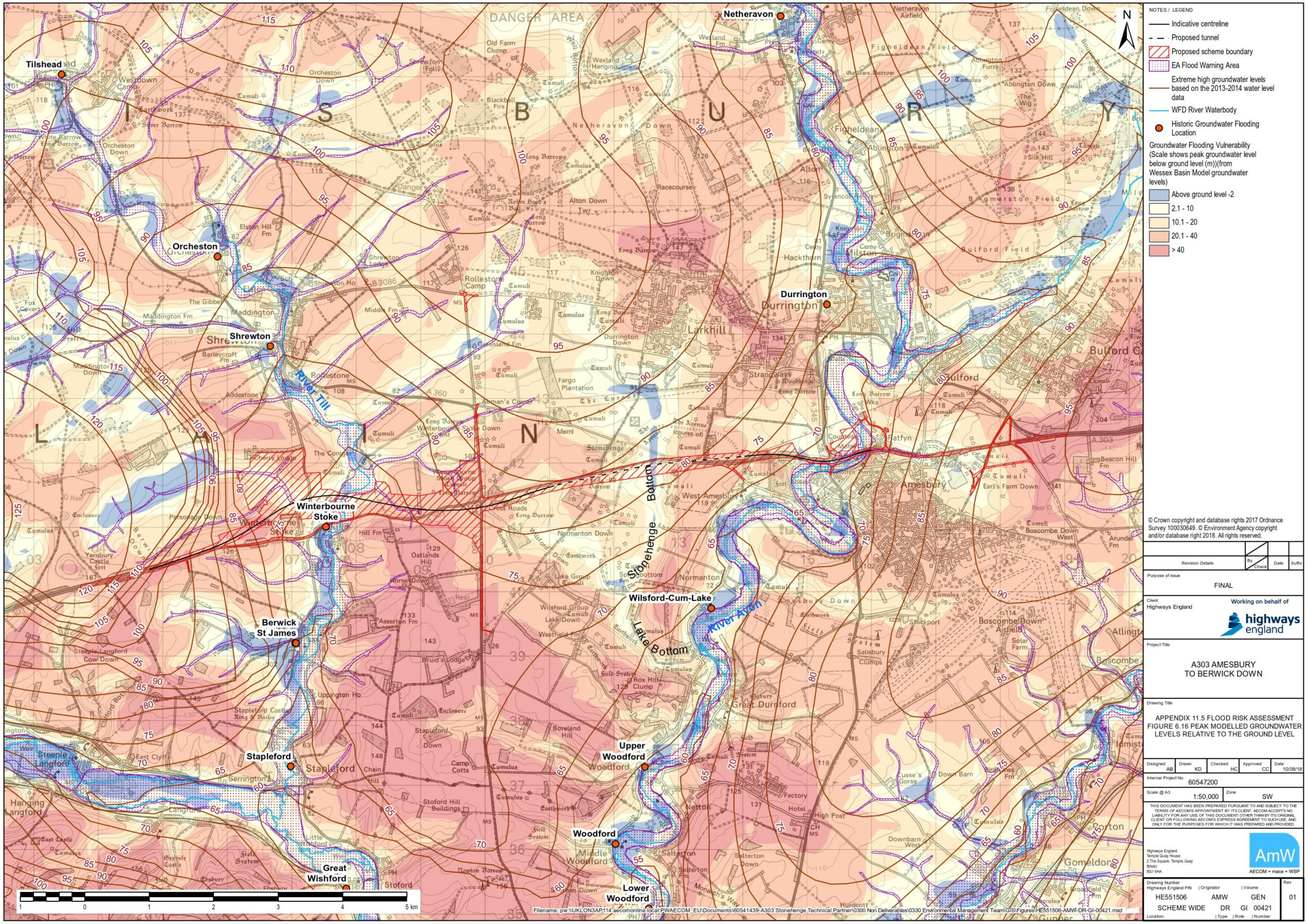


Figure 6.16: Peak modelled groundwater levels relative to the ground level

6.5 Sewer Flood Risk

Flood Sources

6.5.1 Sewer records from Wessex Water reveal two sewers at the Countess Roundabout end of the proposed scheme. One sewer runs north to south along Countess Road crossing the Countess Roundabout. The other is further east, along the Ratfyn access road situated north of the existing A303 and joins the Porton Road roundabout. These sewers are understood to be combined sewers.

Historical flooding

6.5.2 Wessex Water hold no records of historic sewer flooding from the public sewer network in the study area.

6.5.3 Extreme events in combination with high groundwater levels during the winter of 2013–2014, meant that the drainage network outfalls to the River Till were submerged by flood flows and the capacity of the network was exceeded causing public highway and property flooding.

6.6 Artificial Sources of Flood Risk

Flood Sources

6.6.1 Artificial sources include raised channels such as canals or storage features such as ponds and reservoirs. The Environment Agency's Risk of Flooding from Reservoirs Map indicates that there's no risk of flooding from these sources within the study area.

6.6.2 Wiltshire Council, as LLFA, have designed a 95,000m^3 storage area (Tilshead Reservoir) that attenuates flow from excess groundwater emerging from springs and overland runoff to the north of Tilshead, within the River Till catchment. The proposed raised bank and capacity of the flood storage area means that once constructed (started during 2018), this would eventually be designated as a reservoir under the Reservoirs Act 1975. The Tilshead and Orcheston Flood Attenuation Scheme Business Case report (Ref 6.1) describes the proposals in more detail.

6.6.3 The proposed Tilshead Reservoir has been designed as a flood alleviation scheme to reduce flood risk to properties in the villages of Tilshead and Orcheston and the A360 highway.

6.6.4 The main source of flooding in Tilshead village is surface water runoff from the Westdown Artillery Range, where there is a contributing catchment area of 23km^2, extending to the north and northeast of Tilshead. Any flood alleviation works constructed in Tilshead that involve the provision of flood attenuation areas would reduce the flows received by Orcheston, and therefore at the location of the proposed scheme, during a flood event.

6.6.5 Based on this information, the risk of flooding from artificial sources within the study area is considered to be negligible and is therefore not considered further within this assessment.

6.7 Snowmelt

6.7.1 Within the wider Hampshire Avon catchment there are a number of historic flood events that have been identified where snowmelt and frozen ground have contributed to flood events. Whilst flooding of this type is noted, these historic events are within the 'Little Ice Age' period circa 1300 – 1850 AD where climatic conditions do not reflect the current conditions of milder, wetter winters. This indicates that the flood record is not stationary and the use of earlier records should not be used to assess present day flooding in isolation. Furthermore, a review of the Met Office 'Days of Snow Lying' annual average for the period 1961 to 1990 against the period 1981 to 2010 indicates that there is a decrease in snow lying days. The proposed scheme area receives 5 to 10 days on average and this is likely to decrease with climate change based on Kay (Ref 6.2).

6.7.2 On the 16th January 1841, snow melt and rainfall on frozen ground caused extensive flooding within the River Till and River Avon catchments. The communities affected by this included Tilshead, Berwick St James, Winterbourne Stoke, Orcheston, Shrewton and Salisbury. The extent of flooding in these locations is unconfirmed other than flood depths of 2.1m-2.4m recorded at Shrewton.

6.7.3 It is acknowledged that there is a historical risk of flooding from this source relating to multi factorial antecedent conditions. Having considered the atmospheric trends the risk of flooding from the combination of snowfall, snowmelt, frozen ground and rainfall is considered to be low (<0.1% AEP) and is therefore not considered further within this assessment.

7 Flood Risk *to* the Proposed Scheme

7.1 Overview

7.1.1 This section assesses the risk of flooding *to* the proposed scheme from the identified sources within the study area.

7.1.2 The impact of the permanent scheme proposals on flood risk *to other receptors* is assessed in Section 8.

7.1.3 The potential temporary impacts of the scheme on flood risk are discussed in Section 9.

7.2 Fluvial Flood Risk

River Avon

7.2.1 There is no alteration to the existing crossing of the River Avon proposed as part of this scheme. However, the provision of utilities (buried services including cabling) to provide power to the eastern portal would cross the River Avon floodplain and therefore would be located partially within the 1% AEP flood extent.

7.2.2 While there are no changes proposed to the existing River Avon crossing of the A303, there are alterations to the Countess Roundabout. Hydraulic modelling shows that the 1% AEP + 40% climate change flood outline flows to the south of the roundabout. The A303 itself is elevated above the 1% plus climate change flood level and a very low risk is posed to the proposed scheme.

7.2.3 As discussed in Section 4.1.20 and 4.1.24, the Exception Test is only required for elements of proposed development (Essential Infrastructure) in Flood Zone 3. The appraisal of the proposed scheme has shown that the only element within Flood Zone 3 is the existing River Avon Bridge crossing, which is remaining as per its existing construction. Therefore, no Exception Test is specifically required. Within Annex 1 (Part A and Part B) of this report, it is also demonstrated that the proposed scenario does not have a detrimental impact on flooding.

Proposed Mitigation

7.2.4 The electricity connection towards the eastern end of the route crosses the River Avon floodplain and would therefore be located partially within the 1% AEP extent. This cable would be buried at an average depth of 1m.

7.2.5 With design mitigation, the risk to the proposed scheme from fluvial flooding from the River Avon would be Low.

River Till

7.2.6 The permanent works would include piers of the River Till viaduct located within the 1% AEP flood extent.

7.2.7 Hydraulic modelling has been used to assess the potential impact to the permanent works of flood risk. Reference to baseline and permanent works modelling demonstrates that the proposed scheme would be affected by fluvial flooding from the River Till. No detrimental impact is observed from the fluvial hydraulic modelling results and the road itself would be located suitably above any flood levels and therefore not considered to be at risk during the 1% AEP plus climate change scenario.

7.2.8 As discussed in Sections 4.1.20 and 4.1.24, the Exception Test is only required for elements of proposed development (Essential Infrastructure) in Flood Zone 3. The appraisal of the proposed scheme has shown that elements positioned within Flood Zone 3 include the River Till viaduct piers and slight encroachment of landscape profiling of embankment to the east of the River Till. The temporary works located within Flood Zone 3 is the River Till Haul Route. Within Annex 1 (Part A and Part B) of this report, it is demonstrated that under both proposed and temporary scenarios, neither have a detrimental impact on flooding to the satisfaction of the Exception Test.

Proposed Mitigation

7.2.9 The pier foundations have been designed to withstand fluvial flood flows interacting with the piers.

7.2.10 With design mitigation, the risk to the proposed scheme from fluvial flooding from the River Till would be Low.

7.3 Surface Water Flood Risk

7.3.1 The permanent scheme elements at risk from surface water flooding are described in more detail below.

Longbarrow Junction upgrades

7.3.2 The new Longbarrow junction will comprise new slip road connections into two roundabouts linked by a green bridge over the new A303. The new slip roads and new junction could potentially impact surface water flow paths and result in an increase in flood risk to the scheme.

Twin-bore tunnel, including portals

7.3.3 Vertical retaining walls will be constructed along the approaches to both the western and eastern portals. Alterations to local topography and increases in impermeable area, inclusive of tunnel maintenance buildings at the western portal entrance, could result in an increase in surface water flood risk posed to the permanent works at this location.

Countess Roundabout flyover to the eastern portal

7.3.4 Topographical alterations will be required to support the A303 flyover and infilling of the subway underneath the Countess Roundabout. Both alterations have the potential to alter surface water flow paths and potentially increase flood risk posed to the permanent works. The proposed drainage scheme has been designed to mitigate against any change in overland flows as a result of the existing agricultural underpass adjacent to the eastern portal being filled in.

Embankments and cuttings

7.3.5 Adjustments to the land profile to facilitate the creation of embankments and cuttings has the potential to change the catchment characteristics, such as altering surface water overland flow paths which could increase surface water flood risk to the permanent works.

7.3.6 Baseline pluvial modelling at Parsonage Down and interrogation of the Environment Agency's FMfSW mapping demonstrates that the proposed scheme is at risk from surface water flooding.

Proposed Mitigation

7.3.7 The road is designed to minimise the risk of it flooding by incorporating current design standards and future climate change allowances to improve its resilience through the use of sustainable drainage techniques.

7.3.8 The proposed scheme comprises three distinct drainage sections, the roads west of the tunnel, the tunnel and the roads east of the tunnel. Each of the three sections uses different sustainable drainage features to treat and attenuate the highway water runoff, inclusive of tunnel maintenance buildings at the western portal entrance, prior to discharge. Attenuation features have been designed to detain runoff from all events expected to occur with 1% annual probability or more frequently.

7.3.9 Further details on the drainage strategy for the proposed scheme are included in the A303 Amesbury to Berwick Down Environmental Statement Appendix 11.3.

7.3.10 With design mitigation, the risk to the proposed scheme from surface water flooding would be Low.

7.4 Groundwater Flood Risk

7.4.1 The permanent works include the construction of a twin bore tunnel.

7.4.2 Numerical modelling has been undertaken to assess the potential groundwater flood risk to the proposed scheme as described in the Groundwater Numerical Model Report (A303 Amesbury to Berwick Down Environmental Statement Appendix 11.4: Annex 1).

7.4.3 Results show that groundwater levels are predicted to rise in the order of 0.5-1.0m in the vicinity of the tunnel, reducing to less than 0.2m in the area of Larkhill as shown in Figure 8.4 and 8.5.

7.4.4 The results of climate change scenarios found no additional increase in groundwater flood risk as a result of the tunnel impeding groundwater flow.

7.4.5 The risk of groundwater flooding posed to the highway would be Low and no further mitigation measures are proposed.

7.5 Sewer Flood Risk

7.5.1 Historic records indicate a risk of surface water flooding in the vicinity of the countess roundabout when surface water drainage outfalls to the River Avon are submerged by flood flows which prevent discharge.

Proposed Mitigation

7.5.2 The proposed scheme is designed to minimise the risk of it flooding by incorporating current design standards and future climate change allowances to improve its resilience through the use of sustainable drainage techniques. Attenuation features have been designed to detain runoff from all events expected to occur with 1% annual probability or more frequently which will reduce the risk of flooding when the drainage network is unable to discharge due to high water levels. Further details on the drainage strategy for the proposed scheme are included in Appendix 11.3 of the A303 Amesbury to Berwick Down Environmental Statement.

7.5.3 With design mitigation, the risk to the proposed scheme from sewer flooding would be Low.

8 Flood Risk *from* the Proposed Scheme – Permanent Works

8.1 Overview

8.1.1 This section assesses the risk of flooding from the permanent works of proposed scheme to other receptors.

8.1.2 The impact of the temporary works associated with the proposed scheme on flood risk to other receptors is assessed in Section 9.

8.2 Fluvial Flood Risk

River Avon

8.2.1 Local topographical alterations will be required to support the A303 flyover and infilling of the subway underneath of the Countess Roundabout. Furthermore, linear highways drainage ponds located adjacent to the A303 carriageway also have the potential to alter fluvial flow paths from the River Avon. Based upon baseline modelling results, four of the highways drainage ponds at this location encroach upon the periphery of Flood Zone 3 (1% AEP floodplain), with the placement of the ponds dictated by space constraints within the red line boundary.

8.2.2 To determine the potential change in flood risk from the scheme elements described in 8.2.1, hydraulic modelling has been undertaken. Details of how the scheme elements have been represented are included in Fluvial Hydraulic Modelling Report (Annex 1 Part A). In summary, topographical alterations have been made to represent the changes in land profile relating to the A303 flyover within the model, whilst the crest level of drainage pond bunds have been raised to an elevation above the peak baseline flood level in the 0.1% AEP event to ensure that the ponds are not filled with fluvial flood water. Modelling results will be used to inform the drainage pond bund heights and configurations at the detailed design stage.

8.2.3 To the south of the existing A303, hydraulic modelling for the for the 1% AEP +40% CC design event shows one limited area, of approximately 0.003km^2, with increased flood depths of up to +0.025m compared to the baseline as a result of the proposed scheme, as illustrated in Figure 8.1. To the north of the existing A303, there is also a reduction in flood depths of up to -0.10m within the floodplain on both the left and right bank. It should be noted that there are no increases in flood extents within the proposed scenario.

8.2.4 Flood depth differences can be viewed within two highways drainage attenuation ponds to the east of Countess Roundabout in Figure 8.1, a decrease in flood depths of more than -0.20m is shown within the footprint of the drainage ponds. This decrease in depth occurs with respect to the baseline due to raising of drainage pond bund crest levels above peak flood levels to prevent flow of fluvial flood waters into the footprints of the drainage ponds.

8.2.5 Figure 8.1 demonstrates that displacement of fluvial floodwater by the highways drainage ponds leads to a small increase in the maximum flood depth on the floodplain of the River Avon within the area of interest around the A303 crossing. This increase occurs over an area of approximately 0.003km^2. Modelling results demonstrate that this increase is between 0.01m and 0.025m.Changes associated with the flyover and infilling of the subway are above maximum flood levels and hence present a very limited impact upon flood risk.

8.2.6 With informed flood mitigation through the design process, such as maintaining the existing A303/River Avon Bridge crossing dimensions, the risk to receptors, such as properties, from fluvial flooding from the River Avon as a result of the proposed scheme is Low.

8.2.7 As discussed in Section 4.1.20 and 4.1.24, the Exception Test is only required for elements of proposed development (Essential Infrastructure) in Flood Zone 3. The appraisal of the proposed scheme has shown that the only element within Flood Zone 3 is the existing River Avon Bridge crossing, which is remaining as per its existing construction. Therefore, no Exception Test is specifically required.

Third Party Flood Risk

8.2.8 As displayed in Figure 8.1, a large majority of the areas where flood depth has decreased are located along the banks of the River Avon and are not within the red line boundary for the scheme. There is an area immediately to the south of the existing A303 where there is a small increase in flood depths (shown on the Figure 8-1 as pink colouration), which is detailed within Section 8.2.5 above. The extent of the increase in flood depth is limited to a 0.003km^2 area of the existing River Avon floodplain, located outside of the red line boundary for the scheme. This therefore suggests that there is an isolated increase in flood depth upon third party land during a 1% AEP + 40% climate change event. It should be noted that this minor increase in flooding is confined to an area of undeveloped green space on the floodplain adjacent to the river channel and does not increase flood risk to receptors such as properties. A similar increase is observed for the 1% AEP event and results for this event are contained within the Fluvial Hydraulic Modelling Report (Annex 1 Part A).

8.2.9 Given that hydraulic modelling results indicate that there will be a small increase in flood depth upon third party land as a result of the proposed scheme within the 1% AEP + 40% climate change event, mitigation will be discussed with the Environment Agency at the detailed design stage where this is considered necessary.

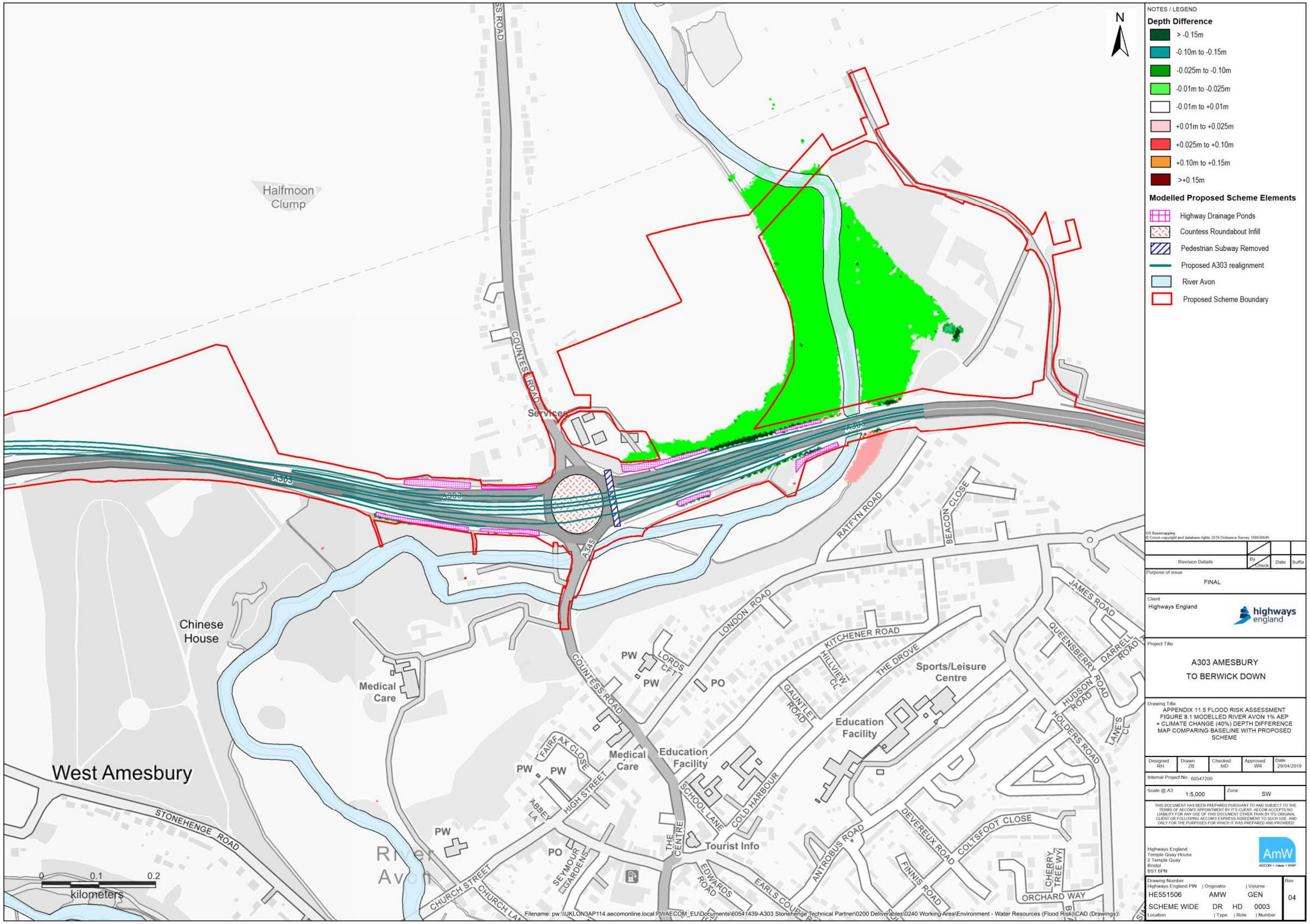


Figure 8.1: Modelled River Avon 1% AEP + Climate Change (+40%) Depth Difference Map comparing baseline with proposed scheme

River Till

8.2.10 The introduction of piers into the floodplain as part of the permanent works at the River Till has potential to interrupt flood flows and create a local backwater effect.

8.2.11 To determine the potential change in flood risk from these scheme elements, hydraulic modelling has been undertaken with the scheme included in the model. Details of how the scheme elements have been represented are included in the Fluvial Hydraulic Modelling Report (Annex 1 Part A).

8.2.12 The hydraulic modelling shows an area of relatively shallow flood depth difference (>-0.20m) overlaying the proposed A303 re-profiling and greater flood depth (+0.10m to +0.20m) towards the southern end of the embankment, near the river channel, as illustrated in Figure 8.2.

8.2.13 The upstream increase in flood risk towards the river channel is due to fluvial floodwater being more contained within the floodplain to the north of the proposed landscaped area and a corresponding decrease in flood risk within the landscaped area. The relatively shallow flood depth difference is due to the steep topographical alterations that will result in floodwaters not reaching this point.

8.2.14 This area is currently used for livestock grazing and the current use will continue once the proposed scheme is implemented. Since this area is already at risk of fluvial flooding the minor changes in flood depths will not increase the risk since there are no sensitive receptors that could be impacted by this change.

8.2.15 With informed flood mitigation through the design process, such as minimising embankment impact within existing flood zones, the risk to receptors from fluvial flooding from the River Till as a result of the proposed scheme would be Low.

8.2.16 As discussed in Sections 4.1.20 and 4.1.24, the Exception Test is only required for elements of proposed development (Essential Infrastructure) in Flood Zone 3. The appraisal of the proposed scheme has shown that elements positioned within Flood Zone 3 include the River Till viaduct piers and slight encroachment of landscape profiling of embankment to the east of the River Till. The temporary works located within Flood Zone 3 is the River Till Haul Route. Within Annex 1 of this report, it is demonstrated that under both proposed and temporary scenarios, neither have a detrimental impact on flooding to the satisfaction of the Exception Test.

Third Party Flood Risk

8.2.17 As displayed in Figure 8.2, all areas where depth has decreased are within the scheme red line boundary. The majority of depth increase is located outside the red line boundary, within agricultural land near the River Till. As discussed in Chapter 11 of the Environmental Statement, the impact magnitude of flooding on agricultural land is considered to be low. It should be noted that there are no impacts on the flood risk north of Foredown Barn or south of Winterbourne Stoke, with the only changes in flood depths throughout the modelled watercourse shown in Figure 8.2.

8.2.18 Given that detailed hydraulic modelling results indicate that there will be a small increase in flood depth upon a 0.02km^2 area of third party land as a result of the proposed scheme within the 1% AEP + 40% climate change event, mitigation will be discussed with the Environment Agency at the detailed design stage where this is considered necessary.

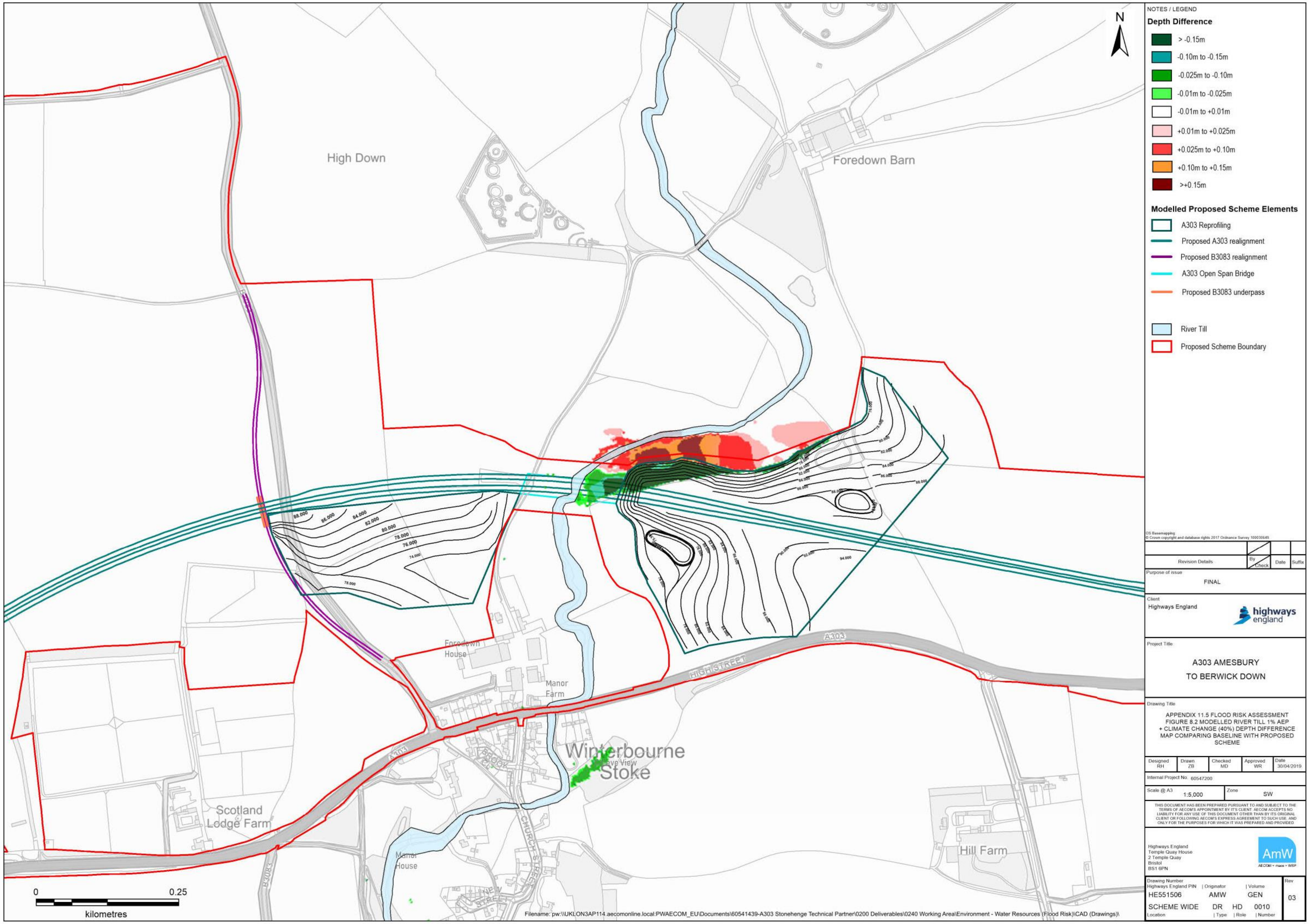


Figure 8.2: Modelled River Till 1% AEP plus Climate Change (+40% Depth Difference Map comparing baseline with proposed scheme

8.3 Surface Water Flood Risk

8.3.1 The permanent scheme elements which have the potential to alter surface water flooding are:

a) *Longbarrow Junction upgrades*: an increase in impermeable ground at this area which could potentially increase surface water flood risk;

b) *Embankments and cuttings:* adjustments to the land profile to facilitate the creation of embankments and cuttings has the potential to change the catchment characteristics, such as altering surface water overland flow paths;

c) *Landscaping:* permanent topographic changes following deposition of tunnel excavated material and embankment creation may alter surface water overland flow paths;

d) *Increased road surface:* introduction of new impermeable areas as part of the proposed scheme has the potential to increase the amount of surface water runoff;

e) *Tunnel Maintenance Buildings:* introduction of new impermeable building footprints as part of the proposed scheme has the potential to increase the amount of surface water runoff;

f) *High Load route:* a minor increase in impermeable ground is expected due to the road widening at this section of the route; and

g) *Tunnel Service Buildings:* a minor increase in impermeable ground is expected at these areas.

8.3.2 Any scheme elements which will result in an increase in impermeable area have design mitigation incorporated. The road is designed to minimise the risk of surface water flooding with attenuation features to detain runoff from all events expected to occur with 1% annual probability or more frequently. Further details on the drainage strategy for the proposed scheme are included in the A303 Amesbury to Berwick Down Environmental Statement Appendix 11.3.

8.3.3 To determine the potential change in flood risk from the landscaping at Parsonage Down, pluvial modelling has been undertaken for this area with the scheme included in the model. Details of how the scheme elements have been represented are included in the Pluvial Hydraulic Modelling Report (Annex 1 Part B).

8.3.4 The proposed mitigation is to implement a land drainage solution to enable the overland flow path to continue towards a culvert, with its inlet situated north of the realigned A303 and west of the proposed B3083 realignment, conveying the flow to the River Till.

8.3.5 The hydraulic modelling shows areas of flood depth differences to the existing surface water overland flow path at Parsonage Down, as illustrated in Figure 8.3. There are increases in depth along the 84.0m AOD contour line within the re-profiled area south of Cherry Lodge of up to +0.50m, with an isolated area within this contour line having an increased depth of up to +1.00m. Additionally, there is an increase in flood depths (+0.05m to +0.20m) between the 86.0m AOD and 87.5m AOD contour line, south of the area with the lowest ground level of 85.0m AOD.

8.3.6 South of Cherry Lodge, there is an area of decreased flood depth of up to -0.50m leading up to the proposed A303. South of the Proposed A303 and west of the B3083, there is an area of approximately 0.6km^2 where there is a decrease in flood depths of up to -1.00m, this is considered to be due to the proposed A303 north of this location cutting off the surface water flow path. In addition, there is a large area of both reduced and increased flood depth between the B3083 and the River Till (depth difference of -0.50m to +0.50m). These changes in flood depths between Parsonage Down and the River Till are due to the existing flow path being blocked by the realigned A303. As a result of decreased flood risk, the B3083 would experience reduced risk of flooding.

8.3.7 Examination of model results demonstrates that there is an increase in peak surface water flow onto the River Till floodplain from Parsonage Down in the proposed scenario, relative to the baseline. Within the baseline scenario peak flow onto the River Till floodplain from Parsonage Down is 0.97m^3/s, whilst in the proposed scenario peak flow is 1.14m^3/s, equating to an increase of approximately 0.17m^3/s. Based upon modelling results presented in Figure 8.3, the increase in the peak flow to the Till floodplain is not sufficient to lead to a significant change in depth on the Till floodplain It is likely that this is due to the additional flow being distributed to a shallow depth over a wider extent of the Till floodplain.

8.3.8 With design mitigation, the risk to receptors from surface water flooding as a result of the proposed scheme would be Low. Modelling results demonstrate that there is no increase in risk to properties.

Third Party Flood Risk

8.3.9 As displayed in Figure 8.3, the most significant areas with changes in flood depths occur within the red line boundary. There are no increases in flood depth within the proposed scenario shown outside the red line boundary, as a result of the scheme arrangement at Parsonage Down. It should be noted that increases in flood depth observed to the north of the new alignment of the A303 will be upon land retained by Highways England, whilst land to the south of the new alignment of the A303 will be returned to the owner.

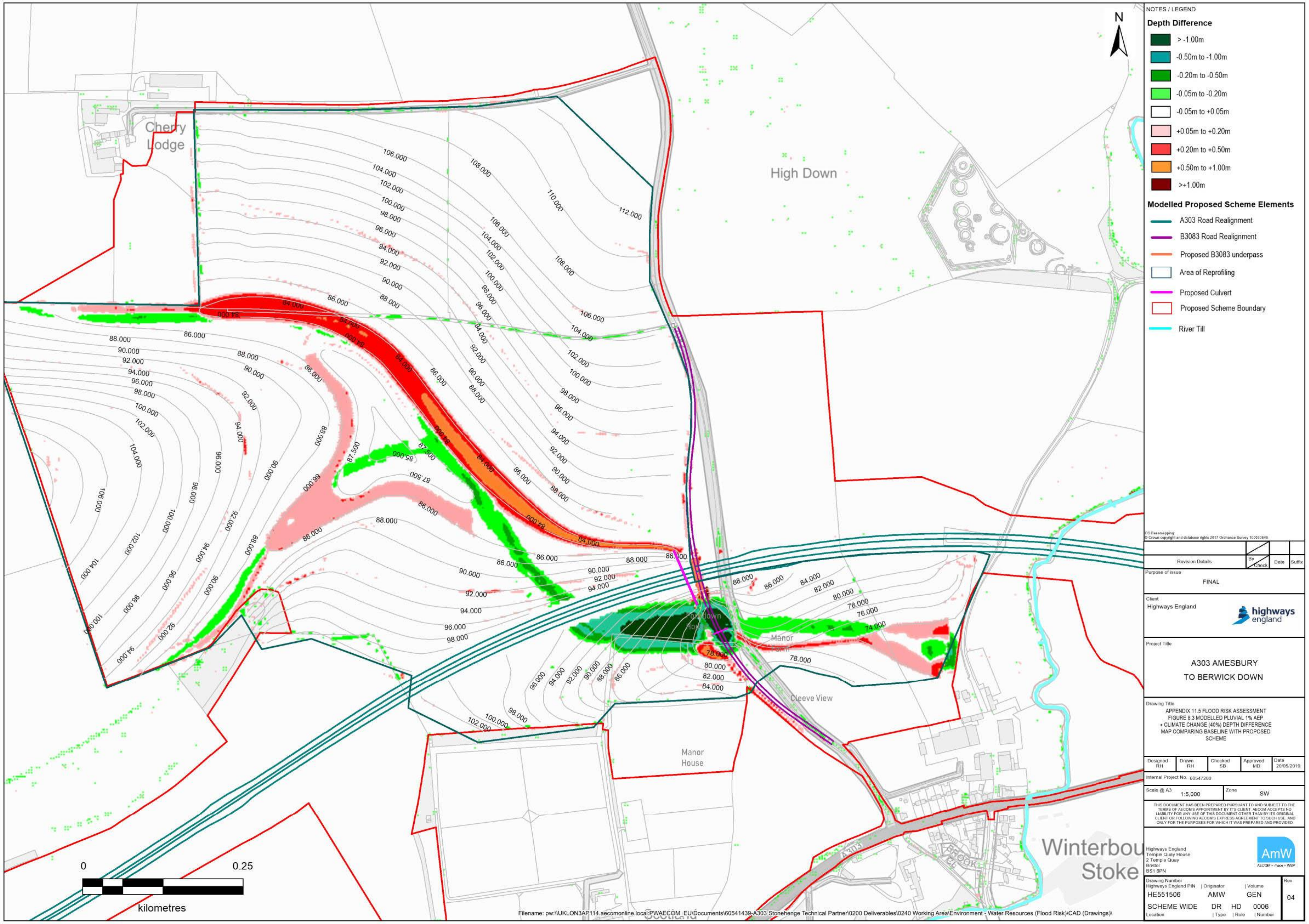


Figure 8.3: Modelled Pluvial 1% AEP plus Climate Change (+40%) Depth Difference Map comparing baseline with proposed scheme

8.4 Groundwater Flood Risk

8.4.1 To determine the potential change in groundwater flood risk posed by the scheme to local receptors, numerical modelling has been undertaken. Details of how the scheme elements have been represented are included in Groundwater Numerical Model Report (A303 Amesbury to Berwick Down Environmental Statement Appendix 11.4: Annex 1).

Increase to groundwater levels

8.4.2 Results show that groundwater levels are predicted to rise up hydraulic gradient (to the north) of the tunnel following the tunnel construction. These changes are in the order of 0.5m-1.0m in the vicinity of the tunnel, reducing to less than 0.2m to the north in the area of Larkhill. The water table is in excess of 10m deep in the vicinity of Larkhill, therefore the predicted rise does not result in an increased risk from groundwater flooding. The modelled depth to groundwater with the tunnel in place is shown in Figure 8.5 and the predicted increase in water table elevation with the tunnel in place is shown in Figure 8.4.

8.4.3 Modelling also indicates a rise in water table elevation in areas with a baseline shallow water which could therefore potentially lead to groundwater flooding. Areas where this occurs are limited to very small parts of rural Stonehenge Bottom valley, shown in Figure 8.6.

8.4.4 The results of climate change scenarios found no additional increase in groundwater flood risk as a result of the tunnel impeding groundwater flow.

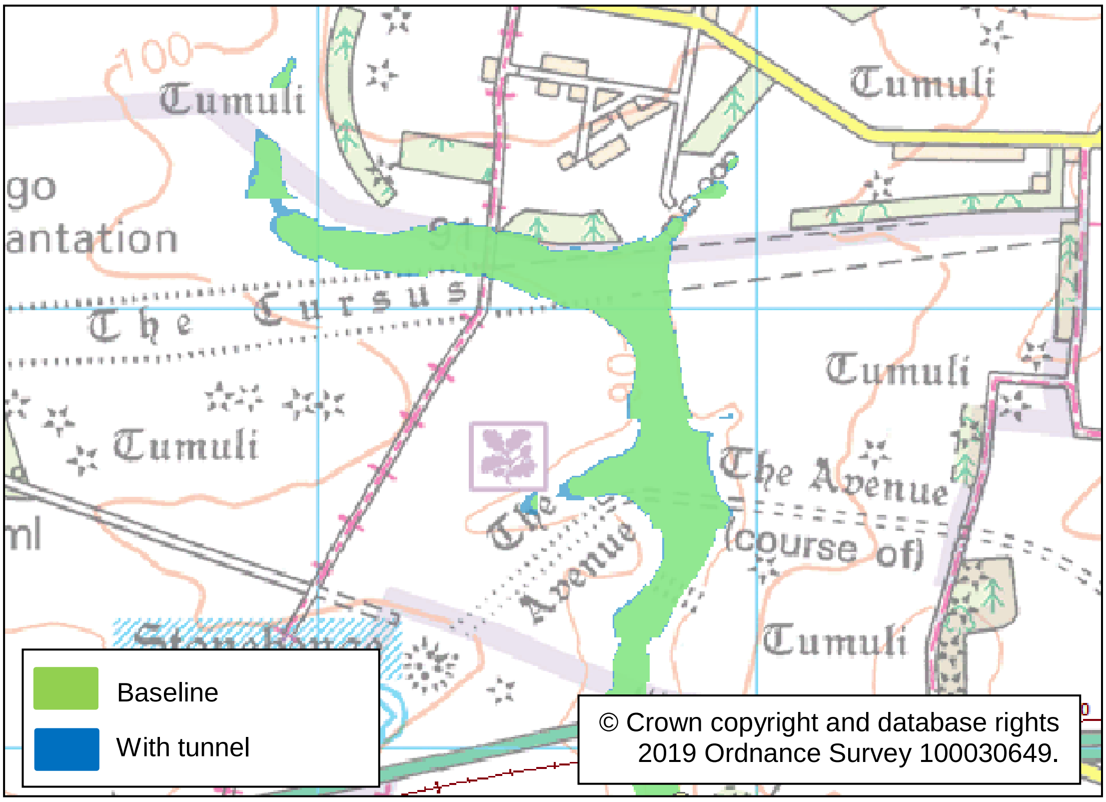


Figure 8.4: Groundwater level at depth shallower than 2m bgl

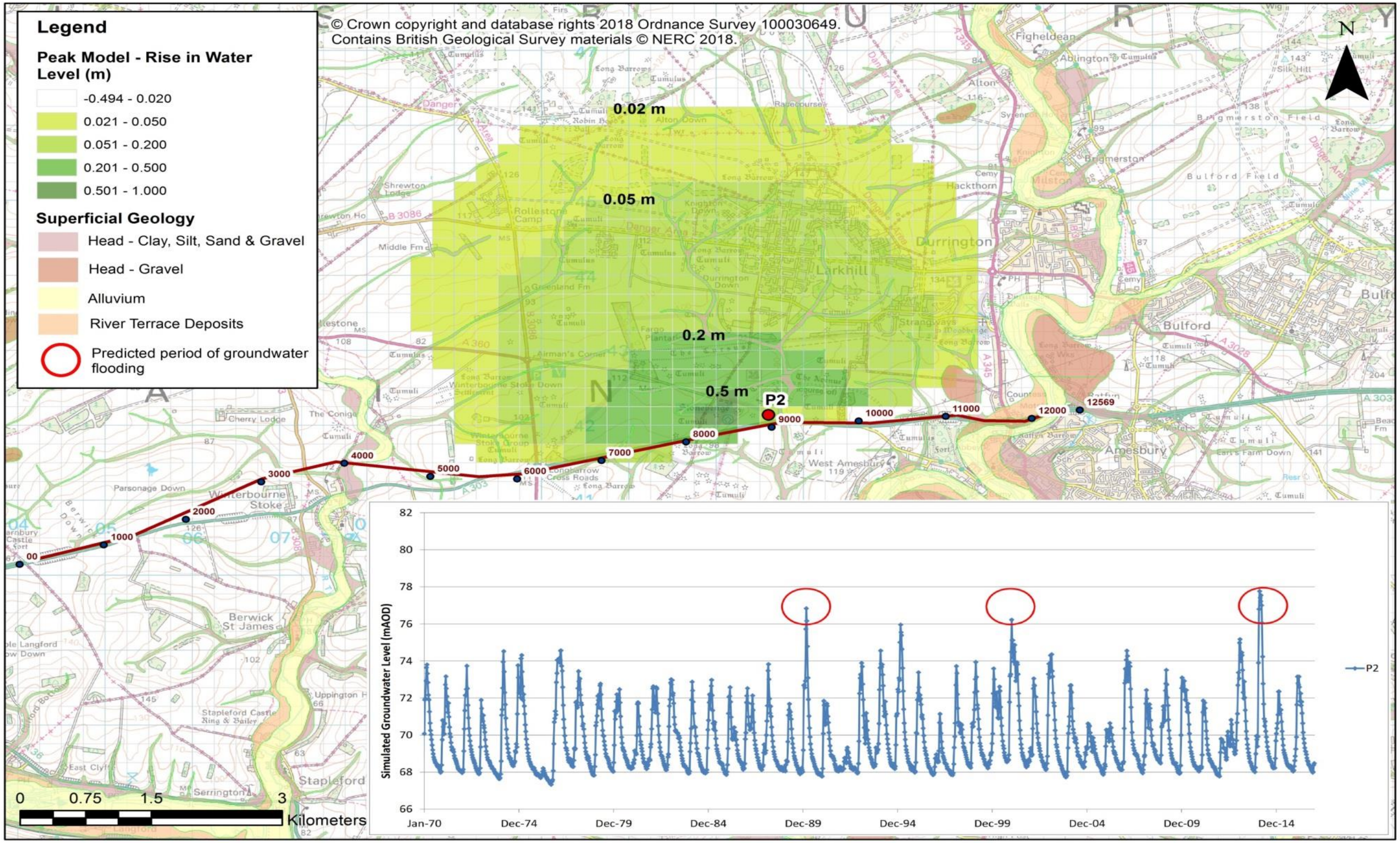


Figure 8.5: Modelled rise in groundwater level at peak (flood) groundwater condition

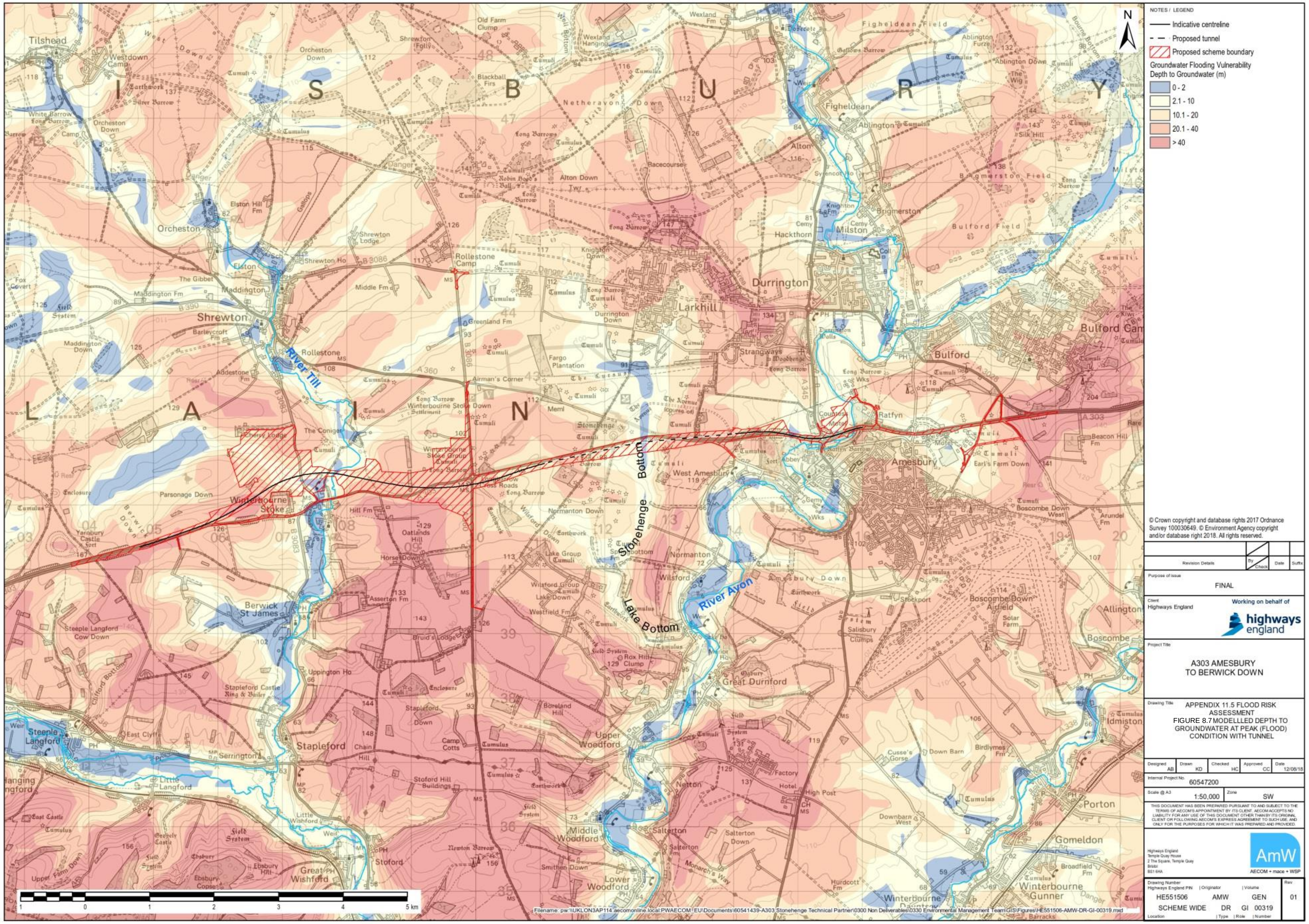


Figure 8.6: Modelled depth to groundwater at peak (flood) condition with tunnel

Increase to River Base flow

8.4.5 A rise in water table is not predicted in areas where groundwater discharges to the River Avon and River Till.

8.4.6 Flow changes in the River Avon average approximately 200m^3/d compared to flows in excess of 1,000,000m^3/d. In the River Avon flows are up to 78m^3/d higher from Durrington to Amesbury GS and up to 500m^3/d lower downstream of Amesbury GS to Little Durnford. These results equate to a maximum change of 0.05% of the flow which is not significant. River Avon accretion profile is given in Figure 8.7.

8.4.7 Flow changes in the River Till is up to 128m^3/d higher in the River Till during baseline periods from approximately 300,000m^3/d. Below the confluence with the River Wylye, flows are in excess of 1,000,000m^3/d with a predicted increase of up to 118m^3/d. Flows increase from the baseline between Tilshead and Shrewton with the highest difference at Winterbourne Stoke. These changes equate to a maximum change of 0.04% of the flow, which is again not significant. The River Till and River Wylye accretion profile is given in Figure 8.8.

8.4.8 The groundwater model therefore predicts no significant change in flow in any reach at peak flows in the River Avon or the River Till. The figures presented show little difference in the total flow scale, so a flow difference plot is also provided.

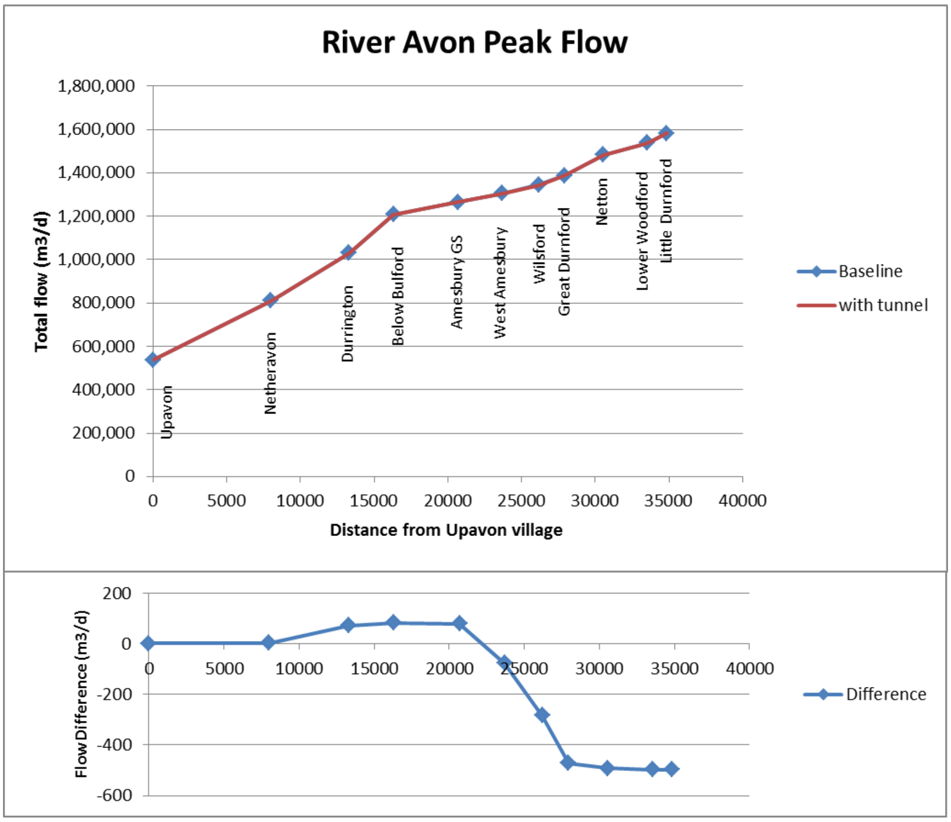


Figure 8.7: River Avon peak flow accretion profile

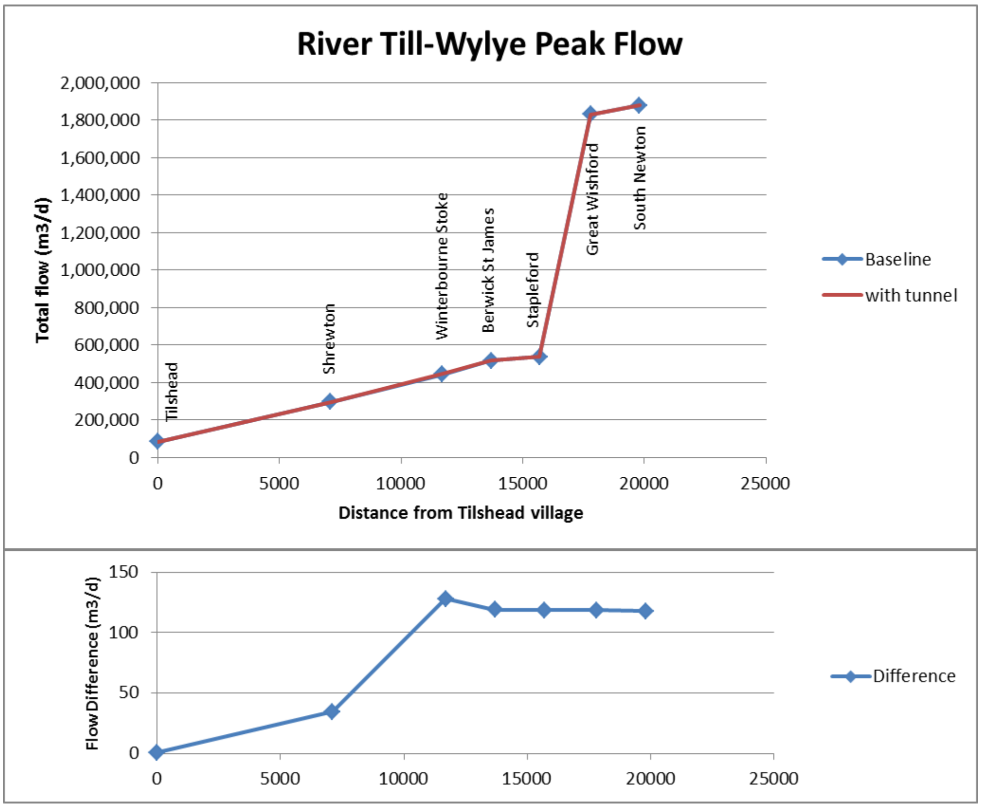


Figure 8.8: River Till and Wylye peak flow accretion profile

8.4.9 Modelled results show that the effects of the tunnel on high river flows would be Negligible.

8.4.10 With design mitigation, the risk to receptors from groundwater flooding as a result of the proposed scheme would be Low.

8.5 Sewer Flood Risk

8.5.1 The permanent scheme elements will not alter sewer flood risk, therefore, the risk to receptors from sewer flooding as a result of the proposed scheme would be Negligible.

9 Flood Risk from the Proposed Scheme – Temporary Works

9.1 Overview

9.1.1 This section assesses the risk of flooding from the temporary works of the proposed scheme to receptors.

9.1.2 The impact of the permanent works associated with the proposed scheme on flood risk to other receptors is assessed in Section 8.

9.2 Fluvial Flood Risk

9.2.1 This section assesses the risk of fluvial flood risk to other receptors as a result of the temporary works associated with the proposed scheme.

River Avon

9.2.2 The temporary scheme elements which have the potential to alter fluvial flooding from the River Avon are:

a) The stockpile area which is located northeast of Countess Roundabout (identified on Figure 3.4D) includes a chalk stockpile and a topsoil stockpile which surrounds the northern and eastern edges of a temporary compound facilities site. This stockpile area is located within an area at very low risk (less than 0.1% AEP) of surface water flooding and both Flood Zone 1 and Flood Zone 2 of the River Avon.

9.2.3 The hydraulic modelling for the River Avon shows that the stockpile area near Countess Roundabout is no longer identified within the 1% AEP + 40% climate change flood extent.

9.2.4 The risk to receptors from fluvial flooding from the River Avon as a result of the temporary works associated with the proposed scheme would be Low.

River Till

9.2.5 The temporary scheme elements which have the potential to alter fluvial flooding from the River Till are:

a) Temporary River Till crossing; and

b) Haul route.

9.2.6 The supporting embankments for the temporary River Till crossing/haul route are within the River Till floodplain within Flood Zone 3. The 1% AEP event fluvial modelling for the River Till (Figure 6.2) also shows that the supporting embankments are within the modelled flood extents for this event. Therefore, the temporary works have the potential to impact existing flood flow pathways and flood storage volume.

9.2.7 To determine the potential change in flood risk from these scheme elements, hydraulic modelling has been undertaken with these temporary works included in the model. Details of how the scheme elements have been represented are included in the Fluvial Hydraulic Modelling Report (Annex 1 Part A).

9.2.8 The hydraulic modelling shows a decrease in flood depths (between -0.01m to -0.10m) in an area to the south of the temporary Bailey Bridge and haul route, as illustrated in Figure 9.1. Since this area is already at risk of flooding the minor changes in flood depths will not increase the risk since there are no sensitive receptors that could be impacted by this change.

9.2.9 The risk to receptors from fluvial flooding from the River Till as a result of the temporary works associated with the proposed scheme would be Low.

Third Party Flood Risk

9.2.10. As displayed in Figure 9.1, the majority of flood depth decrease is shown outside of the red line boundary, where the land is owned by a third party. There is a reduction in flood depths between -0.01m and -0.10m within the River Till, to the south of the red line boundary. Additionally, there is an area on the left bank of the River Till, downstream of the temporary haul route, where there is a reduction in flood depths of between -0.01m and -0.10m. There are no areas where there is an increase in flood depths in this location and therefore flood risk to third parties is not expected to increase as a result of the proposed temporary works during a 1% AEP event.

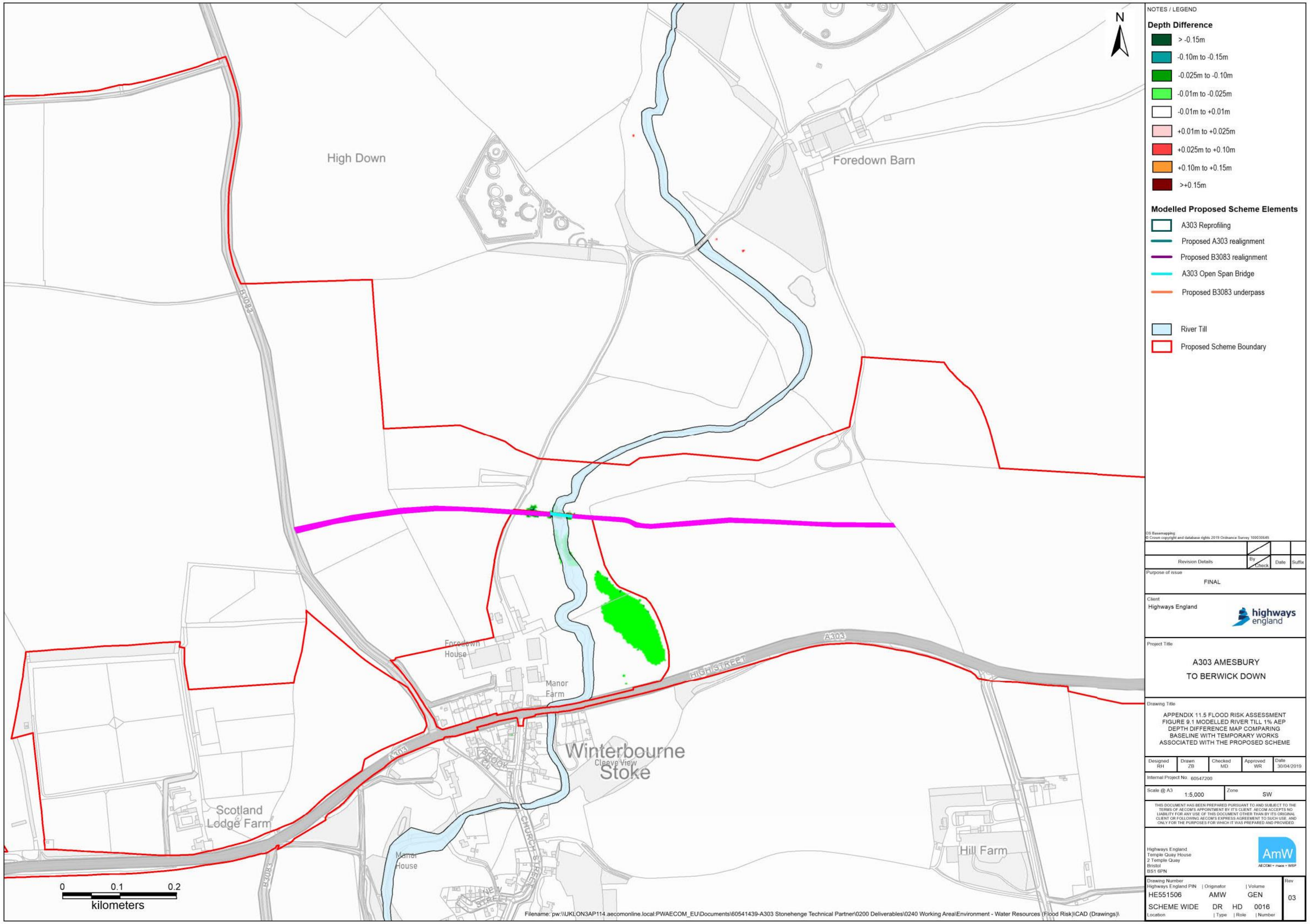


Figure 9.1: Modelled River Till 1% AEP Depth Difference Map comparing baseline with temporary works associated with the proposed scheme

9.3 Surface Water Flood Risk

9.3.1 The temporary scheme elements which have the potential to alter surface water flooding are:

a) Site compounds due to an increase in impermeable area.

9.3.2 Any scheme elements which will result in an increase in impermeable area will have design mitigation incorporated. Site compounds will be designed to manage surface water runoff so there is no increase in surface water flooding to other receptors, in accordance with the Outline CEMP.

9.3.3 The risk to receptors from surface water flooding as a result of the temporary works associated with the proposed scheme would be Low.

9.4 Groundwater Flood Risk

9.4.1 The temporary scheme elements will not alter groundwater flood risk, therefore, the risk to receptors from groundwater flooding as a result of the proposed scheme would be Negligible.

9.5 Sewer Flood Risk

9.5.1 The temporary scheme elements will not alter sewer flood risk, therefore, the risk to receptors from sewer flooding as a result of the proposed scheme would be Negligible.

10 Summary and Conclusions

10.1 Key Flood Risk Sources

10.1.1 The main sources of flood risk within the study area are fluvial, surface water (pluvial) and groundwater. The risk of sewer flooding is localised given the limited extent of any sewers within the study area.

10.1.2 The majority of the study area is within Flood Zone 1 (low probability), except where it traverses the two river channels, where areas of Flood Zone 2 and 3 are present. The baseline modelling flood extents for the River Till and the River Avon, largely coincide with the corresponding Flood Zones produced by the Environment Agency.

10.1.3 The majority of surface water flood risk in the study area is categorised as 'Low'; with some small 'pockets' of 'Medium' or 'High' flood risk. These are typically in valley bottoms and where surface water flow paths are impeded by artificial structures. The baseline modelling flood extents show some differences in extent to the Environment Agency Flood Risk from Surface Water mapping, particularly along the River Till valley (to the north of where the proposed scheme will be located).

10.1.4 The risk of groundwater flooding in the study area is considered to be high. The baseline groundwater model predicts that peak groundwater levels can be above the ground level and therefore, groundwater flooding is likely to occur, along the rivers and dry valleys, such as Stonehenge Bottom.

10.1.5 The proposed scheme is not impacted by flooding during the design event and should always remain operational during periods of nearby flooding. Due to this conclusion, there is no further requirement to consider safe access and egress as part of this FRA.

10.2 Flood Risk to the Proposed Scheme

Fluvial flood risk

10.2.1 The permanent scheme elements at risk from fluvial flooding include:

a) The provision of utilities to provide power to the eastern portal crosses the River Avon floodplain and therefore is located within the 1% AEP flood extent; and,

b) The piers of the River Till viaduct are located within the 1% AEP flood extent.

10.2.2 To mitigate potential impacts to the proposed scheme the installation of above ground utilities structures, such as electricity pylons or substation extensions, would be located outside of the River Avon 1% AEP plus climate change flood extent. The pier foundations for the River Till viaduct have been designed to withstand fluvial flood flows interacting with the piers.

10.2.3 With design mitigation, the risk to the proposed scheme from fluvial flooding would be Low.

Surface water flood risk

10.2.4 The permanent scheme elements at risk from surface water flooding include:

a) Longbarrow Junction upgrades;

b) Twin-bore tunnel, including portals;

c) Countess Roundabout flyover;

d) Embankments and cuttings;

e) Road drainage; and

f) High Load route.

10.2.5 The road is designed to minimise the risk of flooding by incorporating current design standards and future climate change allowances to improve its resilience through the use of sustainable drainage techniques.

10.2.6 With design mitigation, the risk to the proposed scheme from surface water flooding would be Low.

Groundwater flood risk

10.2.7 There are no permanent scheme elements at risk from groundwater flooding; therefore, the risk to the proposed scheme from groundwater flooding would be Low.

Sewer flood risk

10.2.8 Drainage of the Countess Roundabout flyover has the potential to impact on sewer flooding due to high river levels preventing discharge of road runoff.

10.2.9 The road is designed to minimise the risk of flooding with attenuation features to detain runoff from all events expected to occur with 1% annual probability or more frequently.

10.2.10 With design mitigation, the risk to the proposed scheme from sewer flooding would be Low.

10.3 Flood Risk from the Proposed Scheme – Permanent Works

Fluvial flood risk

10.3.1 The permanent scheme elements which have the potential to alter fluvial flooding are:

a) *Countess Roundabout flyover:* the introduction of embankments and infill of the existing subway has the potential to alter flood flow pathways associated with the River Avon; and

b) *River Till viaduct:* the introduction of piers into the River Till floodplain has potential to interrupt flood flows and create a local backwater effect.

The hydraulic modelling for the River Avon shows that there is an increase in water levels on the floodplain to the south of the existing A303 within the proposed scenario when compared to the baseline, this increase is between 0.01m and 0.025m and is limited to an area of approximately 0.003km^2. Since this area is already at risk of flooding, the small increase in flood depths is unlikely to increase the overall flood risk. Given that hydraulic modelling results indicate that there will be a small increase in flood depth upon third party land as a result of the proposed scheme within the 1% AEP + 40% climate change event, mitigation will be discussed with the Environment Agency at the detailed design stage where this is considered necessary

10.3.2 The hydraulic modelling for the River Till shows an area of relatively shallow flood depth difference (>-0.20m) overlaying the proposed A303 reprofiling and greater flood depth (0.10m to 0.20m) towards the southern end of the embankment, near the river channel. Since this area is already at risk of flooding the minor changes in flood depths will not increase the risk since there are no sensitive receptors that could be impacted by this change.

10.3.3 With design mitigation, such as specifying embankment locations and river crossing dimensions, the risk to receptors from fluvial flooding as a result of the proposed scheme would be Low.

Surface water flood risk

10.3.4 The permanent scheme elements which have the potential to alter surface water flooding are:

a) *Longbarrow Junction upgrades*: an increase in impermeable ground at this area which could potentially increase surface water flood risk;

b) *Embankments and cuttings:* adjustments to the land profile to facilitate the creation of embankments and cuttings has the potential to change the catchment characteristics, such as altering surface water overland flow paths;

c) *Landscaping:* permanent topographic changes following deposition of tunnel excavated material and embankment creation may altering surface water overland flow paths;

d) *Increased road surface:* introduction of new impermeable areas as part of the proposed scheme has the potential to increase the amount of surface water runoff; and

e) *High Load route:* a minor increase in impermeable ground is expected due to the road widening at this section of the route.

10.3.5 Any scheme elements which will result in an increase in impermeable area have design mitigation incorporated. The road is designed to minimise the risk of flooding with attenuation features to detain runoff from all events expected to occur with 1% annual probability or more frequently.

10.3.6 The surface water hydraulic modelling for the Parsonage Down area shows flood depth differences to the existing surface water overland flow path. The proposed mitigation is to implement a managed land drainage solution to enable the overland flow path to continue towards the River Till.

10.3.7 Within the proposed scenario surface water from Parsonage Down ultimately flows into the River Till floodplain at the same location as within the baseline. The redirection of surface water conveyance to the River Till has resulted in an increase in peak flows supplied to the River Till floodplain, this increase is $0.17m^3/s$ for the design event and equates to an increase of 18%. It should be noted that the additional flow supplied to the River Till floodplain does not lead to a significant increase in flood depths on the floodplain, whilst flood risk to the B3083 is decreased due to the managed drainage arrangement.

10.3.8 With design mitigation, the risk to receptors from surface water flooding as a result of the proposed scheme would be Low.

Groundwater flood risk

10.3.9 The presence of structures below the groundwater level in the Chalk, such as the twin-bore tunnel has the potential to interfere with groundwater flow.

10.3.10 Groundwater levels are predicted to rise up hydraulic gradient (north) of the tunnel in the order of 0.5m-1.0m in the vicinity of the tunnel, reducing to less than 0.2m in the area of Larkhill. Groundwater level rise beneath built up areas around Larkhill is in an area where the water table is in excess of 10m deep. Therefore this predicted rise does not result in an increased risk from groundwater flooding.

10.3.11 A rise in water table elevation in areas with a baseline shallow water table during the 2014 peak or groundwater levels above surface would indicate an increased risk of groundwater flooding when the tunnel is in place. Areas where this occurs are limited to very small parts of Stonehenge Bottom valley.

10.3.12 A rise in water table is not predicted in the areas where groundwater discharges to the River Avon and River Till.

10.3.13 With design mitigation, such as influencing design and re-profiling of land east of Parsonage Down NNR, the risk to receptors from groundwater flooding as a result of the proposed scheme would be Low.

Sewer flood risk

10.3.14 The permanent scheme elements will not alter sewer flood risk, therefore, the risk to receptors from sewer flooding as a result of the proposed scheme would be Negligible.

10.4 Flood Risk from the Proposed Scheme – Temporary Works

Fluvial flood risk

10.4.1 The temporary scheme elements which have the potential to alter fluvial flooding are:

a) The stockpile area located northeast of the Countess Roundabout located within an area at very low risk (less than 0.1% AEP) of surface water flooding and both Flood Zone 1 and Flood Zone 2 of the River Avon;

b) Temporary River Till crossing; and

c) Haul route crossing the River Till valley.

10.4.2 The hydraulic modelling for the River Avon shows a change in flood extent in the vicinity of the stockpile area near Countess Roundabout such that it is no longer identified within the 1% AEP +40% climate change flood extent.

10.4.3 The hydraulic modelling for the River Till shows a variation in flood depth (between -0.01m to -0.10m) in an area to the south of the temporary bridge and haul route when the baseline and proposed temporary works are compared.

10.4.4 The risk to receptors from fluvial flooding as a result of the temporary works associated with the proposed scheme would be Low.

Surface Water Flood Risk

10.4.5 Site compounds have the potential to alter surface water flooding due to an increase in impermeable area.

10.4.6 Any scheme elements which will result in an increase in impermeable area will have design mitigation incorporated. Site compounds will be designed to manage surface water runoff so there is no increase in surface water flooding to other receptors, in accordance with the CEMP.

10.4.7 The risk to receptors from surface water flooding as a result of the temporary works associated with the proposed scheme would be Low.

Groundwater Flood Risk

10.4.8 The temporary scheme elements will not alter groundwater flood risk, therefore, the risk to receptors from groundwater flooding as a result of the proposed scheme would be Negligible.

Sewer Flood Risk

10.4.9 The temporary scheme elements will not alter sewer flood risk, therefore, the risk to receptors from sewer flooding as a result of the proposed scheme would be Negligible.

10.5 Conclusion

10.5.1 It is concluded that the flood risk to and from the permanent features of the proposed scheme from fluvial, surface water, groundwater and sewer flooding, would be either Low or Negligible.

10.5.2 The assessment of flood risk from the temporary features of the proposed scheme has concluded that the risk to other receptors from fluvial and surface water flooding is Low. The temporary features will not alter groundwater or sewer flood risk, therefore, the risk to receptors from groundwater or sewer flooding as a result of the proposed scheme would be Negligible.

10.5.3 When considering fluvial flood risk, any changes to modelled flood depths within the proposed scenario are limited to areas of agricultural land or undeveloped green space. For both the River Till and River Avon there are both areas of increase and decrease in maximum flood depth as a result of the proposed scheme. As these areas are already at risk of flooding, the changes in flood depths would not increase flood risk because they do not coincide with sensitive receptors, for example properties and businesses.

10.5.4 Surface water modelling undertaken suggests that the proposed scheme and drainage arrangement at Parsonage Down leads to a small increase in peak flow onto the River Till floodplain from Parsonage Down. For the design event, modelling shows that the increase in peak flow is $0.17m^3/s$. Modelling results show that this increase in peak flow does not lead to a significant change in flood depths on the River Till floodplain.

10.5.5 Sequential Test – The Secretary of State confirmed the selection of the final route for the proposed scheme. The application of the Sequential Test was therefore undertaken through this process.

Exception Test - The assessment of flood risk to and from the proposed development where an encroachment within Flood Zone 3 exists has been undertaken through site specific hydraulic modelling. Within Annex 1 (Part A and B) of this report, it is demonstrated that under both proposed and temporary scenarios, neither have a detrimental impact on flooding, to the satisfaction of the Exception Test.

References

Ref 2.1 N. Mortimore. Making sense of Chalk: a total-rock approach to its engineering geology. The Eleventh Glossop Lecture. Quarterly Journal of Engineering and Hydrogeology 45, 252-334. 2012.

Ref 2.2 AAJV. Preliminary Sources Study Report, 2016.

Ref 3.1 Defra, 2015, Sustainable Drainage Systems Non-statutory technical standards for sustainable drainage systems, 2015. Available online at: https://www.gov.uk/government/uploads/system/uploads/attachment_data/file/415773/sustainable-drainage-technical-standards.pdf.

Ref 3.2. Highways England, unpublished 2018 version. Design Manual for Roads and Bridges ("DMRB") Volume 11, Section 3, Part 10 HD45.

Ref 4.1 Environment Agency, 2016, National Planning Policy Framework Planning Practice Guidance. Available online at https://www.gov.uk/government/collections/planning-practice-guidance.

Ref 5.1 AmW, February 2018, Flood Risk Hydraulic Modelling Methodology.

Ref 5.2 Environment Agency, 2016, Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities.

Ref 6.1 RMA Short Form Tilshead and Orcheston Flood Attenuation Scheme Business Case, Version 4, June 2017.

Ref 6.2 Kay, 2011. Snow in Britain: the historical picture and future projections. Centre for Ecology & Hydrology, Wallingford, UK, February 2016

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Level 3 Flood Risk Assessment

Annex 1A – Fluvial Hydraulic Modelling Report

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

May 2019

Table of contents

1 Introduction

1.1 Background

1.1.1 In order to robustly assess the impact of the proposed scheme upon flood risk, and provide quantitative information to inform the Flood Risk Assessment (FRA), hydraulic modelling was undertaken for the River Till and River Avon. This Hydraulic Modelling Report has been produced in order to document the technical work undertaken in support of the FRA.

1.1.2 This Hydraulic Modelling report forms Annex 1 of the FRA, and the reader is referred to the FRA document for further context relating to the River Till and River Avon, along with details of the proposed scheme.

1.1.3 Annex 1 is formed of two parts. Hydraulic modelling of the fluvial regime of both River Till and River Avon are discussed within Annex 1 – Part A (this report), whereas the pluvial (direct rainfall modelling) is discussed within Annex 1 – Part B.

1.1.4 This Hydraulic Modelling report is accompanied by one further annex (Annex 2) which documents the hydrological analysis undertaken for the fluvial River Till, fluvial River Avon and pluvial catchment east of Parsonage Down Natural Nature Reserve (NNR). For the avoidance of duplication, the reader is referred to Annex 2 of the FRA for further details of the approach used to generate design inflows for the two watercourses.

1.1.5 The first version of this report documented the methodology and results obtained from initial fluvial hydraulic modelling, and was submitted as part of the Environmental Statement on 19th October.

1.1.6 The current version of the report contains updates to the methodology and results from additional fluvial hydraulic modelling undertaken between January 2019 and April 2019. Additional hydraulic modelling has been undertaken in order to reflect comments received from the Environment Agency on the first version of the report. The key updates documented within this report are:

- Re-simulation of the River Avon model with updated hydrological inflows.
- Change in the indicative areas assigned to highways drainage ponds, close to Countess Roundabout on the River Avon.
- Addition of sensitivity analysis results for both watercourses, including incorporation of Continuous Simulation Modelling (CSM) for Salisbury for the River Avon. The Salisbury Modelling has been undertaken by JBA Consulting in partnership with the Environment Agency. As this modelling overlaps the study area for the River Avon, the Environment Agency have requested that this be considered within this assessment.
- Model verification with respect to available observations, including historical flood records, aerial photographs and flood extents.

1.2 Objectives

1.2.1 In order to provide an appropriate assessment of flood risk from the River Till and River Avon in the context of the proposed scheme, the following primary objectives have been completed;

1. To assess fluvial flood risk within the existing (baseline) scenario for the River Till and River Avon;
2. To assess fluvial flood risk to/from the proposed temporary development scenario during construction, including temporary crossing in the River Till catchment, north of Winterbourne Stoke;
3. To assess fluvial flood risk to/from the proposed scheme scenario in the River Till catchment; and
4. To assess fluvial flood risk to/from the proposed scheme, in the River Avon catchment.

1.3 Design Simulations and Climate Change

1.3.1 To meet the objectives outlined in Section 1.2, and also to ensure compliance with relevant planning policy[1], the fluvial hydraulic modelling for the River Till and River Avon has been undertaken for the baseline, temporary and proposed scenarios for a range of design events. These are discussed in more detail within the respective sections for each watercourse.

1.3.2 In line with Environment Agency guidance[2], the 1% annual exceedance probability (AEP) design event including an allowance for climate change (1% AEP + 40% increase in peak flows) has also been simulated for the baseline, temporary and proposed scenarios. The allowance of +40% corresponds to the Higher Central allowance for the South West river basin district.

1.3.3 As a sensitivity analysis, fluvial modelling for the River Till and River Avon was undertaken for the 1% AEP design event, inclusive of an uplift in peak flow of +85%. This corresponds to the Upper climate change allowance for South West river basin district for the baseline, temporary and proposed scheme scenarios.

1.4 Report Structure

1.4.1 The River Till and River Avon hydraulic models were built using a consistent approach and methodology, hence the common aspects of model set up and development which are outlined in Section 2. Prior to the development of all hydraulic models as part of this study, the methodology presented herein is consistent with the AmW methodology report[3] confirmed by the Environment Agency and Wiltshire Council during the preparatory stage.

1.4.2 Specific information relating to the setup of the baseline, temporary and proposed scenario River Till model is included within Section 3, whilst specific information

[1] HM Government (2018) Revised National Planning Policy Framework
[2] Environment Agency (2016) Adapting to Climate Change: Advice for Flood and Coastal Management Authorities.
[3] AmW (2017) A303 Stonehenge. Flood Risk Hydraulic Modelling Methodology.

relating to the setup of the baseline, temporary and proposed scenario River Avon model is included within Section 4.

1.4.3 Results from the River Till hydraulic modelling are presented and discussed in Section 5, whilst results for the River Avon are documented in Section 6.

1.4.4 A statement of the limitations associated with the fluvial hydraulic modelling work undertaken is included within Section 7.

1.4.5 Conclusions based upon the fluvial hydraulic modelling work undertaken are outlined in Section 8.

2 Hydraulic Modelling Methodology

2.1 Hydrological Analysis

2.1.1 The catchments of the River Till and River Avon have been subject to hydrological analysis in order to estimate design inflows for the fluvial hydraulic models. This has been undertaken using industry standard techniques, namely the Flood Estimation Handbook (FEH) and Revitalised Flood Hydrograph 2 (ReFH2) methods.

2.1.2 For a detailed description of the flow estimation undertaken, the reader is referred to Annex 2 of the FRA.

2.2 Software

2.2.1 The 1D River Till and River Avon channels have been represented in Flood Modeller Pro (FMP). FMP is a one-dimensional (1D) package used for modelling river channels, including bridges, culverts, weirs and other structures. FMP calculates the varying water levels within the channel and associated transference of flow to the floodplain when hydraulically linked to a 2D model (TUFLOW).

2.2.2 TUFLOW is a two-dimensional (2D) hydraulic modelling package that simulates hydrodynamic behaviour of flood waters across the land surface using a grid based approach.

2.2.3 Combining FMP and TUFLOW allows for full hydraulic linking between the channel and the floodplain allowing the water from the channel (1D) to enter the floodplain (2D) and vice versa.

2.2.4 Models were simulated using Flood Modeller Pro (FMP) version 4.3 and TUFLOW version 2018-03-AC.

2.3 1D Model - River Channel Survey

2.3.1 Highways England commissioned a survey of the River Till which was carried out by AP Land Surveys in 2018 which comprised of 51 channel cross-sections and 63 structure cross-sections. A survey of the River Avon was also carried out by AP Land Surveys in 2018 and comprised 74 channel cross sections and 31 structures.

2.3.2 The channel cross sectional and structure surveys were utilised to build the 1D FMP models for the River Till and River Avon channels.

2.4 2D Model - Floodplain Topography

2.4.1 The topographical data utilised within both fluvial hydraulic models is a composite Digital Terrain Model (DTM) with a 2m grid resolution.

2.4.2 The primary source of topographical data within the composite DTM is provided by a 2m resolution Environment Agency LiDAR DTM. Gaps present within the LiDAR DTM were filled in the first instance by a high resolution (1m) photogrammetric DTM. Any remaining gaps were then filled by a 5m Synthetic Aperture Radar (SAR) DTM, although it should be noted that the 5m SAR DTM was not utilised within any areas of interest for this study.

2.4.3 For both fluvial models a grid resolution of 4m was used within the 2D TUFLOW model domain. A 4m model resolution represented the finest resolution that could be achieved whilst retaining practical model run times. Both fluvial models typically take between 10 and 20 hours to simulate.

2.5 Roughness

2.5.1 Channel and floodplain friction was represented in the hydraulic model by defining a varying Manning's Roughness Coefficient across both the 1D and 2D model domain.

2.5.2 Within the 1D FMP model, Manning's Roughness Coefficients were assigned based upon cross sectional survey and accompanying photos, alongside relevant guidance[4].

2.5.3 Within the 2D TUFLOW model, OS Mastermap was used to define floodplain land cover, allowing the Manning's Roughness Coefficients to be spatially distributed throughout the domain.

2.5.4 Buildings were represented as areas of elevated roughness, where a Manning's Roughness Coefficient of 0.5 was specified, as per best practice guidance for fluvial hydraulic modelling.

2.5.5 An extract of the Manning's Roughness Coefficients used in the hydraulic model are found below in Table 2.1.

Table 2.1: Hydraulic Model – Manning's Roughness Coefficients ('n')

Surface	'n'
2D	
Building	0.5
Roads and Paved areas	0.025
Grass	0.06
1D	
Smooth channel bed	0.04
Rough Grass	0.06

[4] Chow (1959) Open Channel Hydraulics

Concrete	0.025
Brick Walls	0.03

2.5.6 To understand the influence of assumptions made during the model development phase, sensitivity analysis associated with roughness has been undertaken. Within the model sensitivity runs, the Manning's Roughness Coefficients ('n') on channel and floodplain values were changed by +/- 20%. The impact on maximum flood depth of the 1% AEP event was then assessed, with results presented in Sections 5.4 and 6.3.

2.6 Model Timestep

2.6.1 For both the River Till and River Avon, the 2D TUFLOW model was simulated with a timestep of 2 seconds, in line with best practice guidance, which suggests that the 2D timestep should be half of the 2D grid cell size (4m).

2.6.2 The 1D FMP model time step was set to be half of the 2D timestep, at 1 second, as per best practice guidelines.

2.7 Model Boundary Conditions

2.7.1 For both the River Till and River Avon, model inflows comprised direct inflows at the primary watercourse upstream boundary, significant tributaries, and lateral inflows to represent inflows from the intervening catchments. In all cases these were defined as flow-time boundaries based upon results of the hydrological analysis (Annex 2).

2.7.2 For both River Till and River Avon hydraulic models there was no known recorded hydrological data (stage or flow) at the downstream extent of the model that could be utilised to define a downstream boundary condition. Therefore, for both models a normal depth boundary was applied to the 1D FMP model, which calculates outflow based upon the gradient of the upstream channel bed. Within the 2D TUFLOW model a stage-flow (HQ) boundary was included to represent natural propagation of water across the floodplain according to local topography.

2.7.3 Saliently, for both models the upstream and downstream boundaries are considered remote from the proposed scheme and the configuration of the boundaries has been proven through assessment of the results to have no impact upon hydraulics at the location of the proposed scheme.

3 River Till - Specific Approach

3.1 Model Setup – Baseline Scenario

3.1.1 The extent of the River Till model is shown within the schematic in Figure 3.1.

3.1.2 The upstream boundary of the model is located to the south of Tilshead, whilst the downstream boundary is located downstream of Berwick St James. The length of the modelled reach is approximately 13km. This model extent is considered more than sufficient to assess flood risk to and from the proposed scheme.

3.1.3 A small tributary watercourse is included within the model, which confluences with the River Till at Shrewton. This tributary supplied a relatively small flow into the River Till.

3.1.4 The River Till model was set up to simulate for 35 hours, in order to fully capture the flood event from the 12 hour storm event estimated within the hydrological analysis.

3.1.5 Small footbridges with a shallow deck depth were deemed unlikely to have a significant hydraulic impact and so were excluded from the model build. Small localised areas of topography around the narrow channel were deemed inaccurate and as such amendments were made to raise bridges and bank levels in 2D that were affected.

3.2 Model Setup - Permanent Scenario

3.2.1 The proposed scheme design was incorporated into the hydraulic model. For the River Till proposed scenario, modifications to the model set up were only required to be made within the 2D TUFLOW model only. Further details of the proposed permanent features are provided within Section 3.2 of the FRA document.

3.2.2 Amendments were made to the model topography in order to add in the embankments for the proposed route of the A303 around the River Till viaduct.

3.2.3 The River Till viaduct has not been represented directly within the model as the bridge is open span and the soffit is elevated far above feasible flood levels.

3.2.4 Piers for the River Till viaduct are located within the floodplain and have been represented within the 2D model as flow constriction units, which facilitate blockage of flow through cells and mimic the obstruction to flood flow attributable to the piers.

3.2.5 A number of new highways drainage ponds were included as part of the proposed scheme design, close to the River Till viaduct. Interrogation of baseline model results revealed that these drainage ponds did not fall within the fluvial floodplain for the 0.1% AEP event, and thus these were not included within the proposed scenario fluvial model.

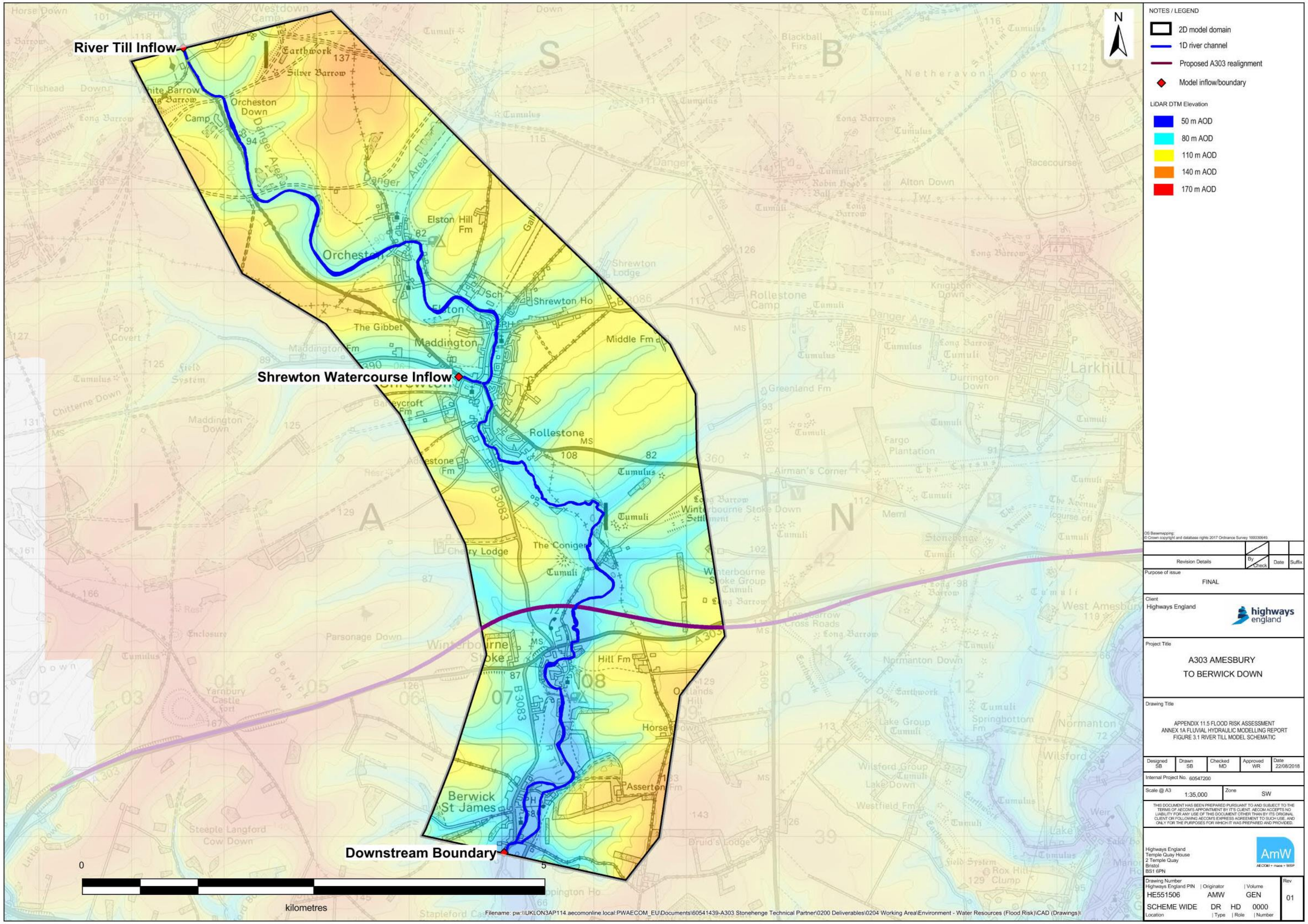


Figure 3.1: River Till Model Schematic

3.3 Model Setup - Temporary Scenario

3.3.1 The temporary construction design features a haul road and Bailey bridge. The haul route cuts across the floodplain of the River Till, approximately parallel to the route of the proposed River Till viaduct for the A303. The Bailey bridge and haul road facilitates access over the River Till watercourse and floodplain. Further details of the temporary features are included within Section 3.3 of the FRA document.

3.3.2 In order to facilitate hydraulic modelling of the Bailey bridge and impacts upon flood risk, and feed into the design of these temporary works, the following design has been included within the hydraulic model;

1. The Bailey bridge is located 60m to the south of the centre of the proposed River Till viaduct;
2. The Bailey bridge is 7.5m wide;
3. The soffit level of the bridge deck (72.83m AOD) is 300mm above the maximum 1% AEP water level, taken from the baseline simulation; and,
4. The Bailey bridge deck thickness is 1m.

3.3.3 The design of the temporary haul route has been included within the hydraulic model to be 7.5m in width, and the crest level of the haul route is set equivalent to the deck level of the bridge. The crest level of the haul route is uniform across the floodplain, and grades into the slope at the sides of the Till Valley.

3.3.4 The Bailey bridge was incorporated into the 1D FMP model, with appropriate adjustments made to the 2D TUFLOW model.

3.3.5 The temporary haul route is represented through amendments to topography within the 2D model domain.

4 River Avon - Specific Approach

4.1 Model Setup – Baseline Scenario

4.1.1 The extent of the River Avon model is shown within the schematic within Figure 4.1. The upstream boundary of the model is located close to Figheldean, whilst the downstream boundary is located upstream of Great Durnford. The modelled reach is approximately 14km in length. This model extent is considered more than sufficient to assess flood risk to and from the proposed scheme.

4.1.2 No tributaries are directly represented within the hydraulic model as river channels, although the Nine Mile River is represented as an inflow to the River Avon close to Bulford.

4.1.3 Inclusion of a hydraulic representation of the Nine Mile River was not considered a requirement, as the confluence was located sufficiently upstream from the location of the proposed scheme. Hence it is unlikely that there would be any change in river flows or levels, attributable to the proposed scheme, which would influence flood risk on the Nine Mile River. This is confirmed through presentation of modelling results in Section 6.

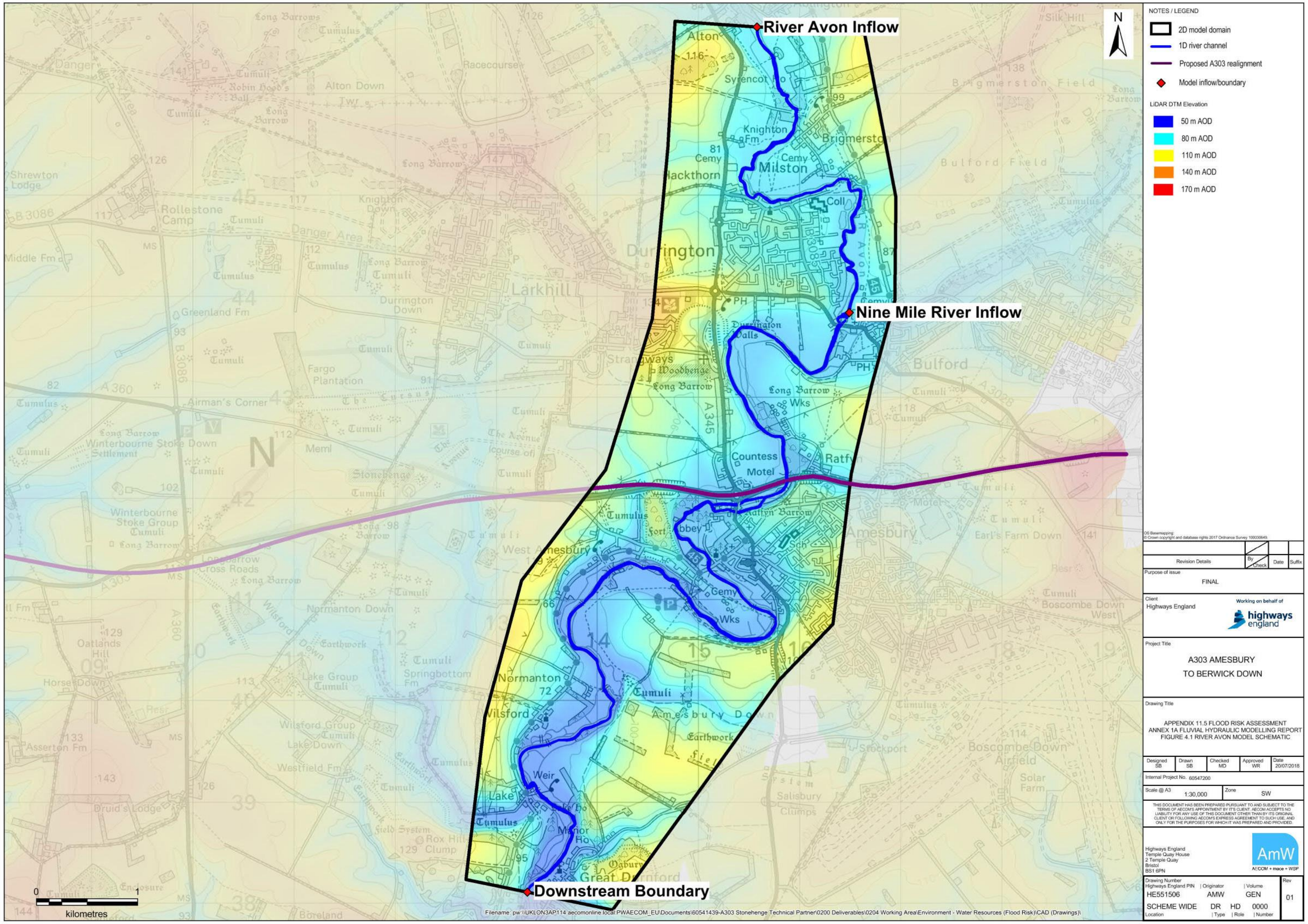


Figure 4.1: River Till Model Schematic

4.1.4 The River Avon model was set up to simulate for 60 hours, in order to fully capture the flood event from the 16 hour storm event estimated within the hydrological analysis.

4.2 Model Setup - Permanent Scenario

4.2.1 The proposed scheme design was incorporated into the hydraulic model. For the River Avon permanent scenario, modifications to the model set up were made within the 2D TUFLOW model only as no changes were proposed to take place within the River Avon channel. Further details of the proposed permanent features of the scheme are included within Section 3.2 of the FRA document.

4.2.2 Modifications to the topography of the A303 embankments were included within the 2D model domain through topographical amendments.

4.2.3 Highways drainage ponds were included through topographical amendments within the 2D model domain. As the drainage ponds are designed for the storage of water from the A303 carriageway only, the levels of the bunds were raised above conceivable flood levels as 'glass walls'. This approach ensured a conservative representation of the impacts of flood risk, whilst peak flood levels around the ponds were communicated to the highways team in order to inform design of the ponds.

4.2.4 Infill of the Countess Roundabout was represented through amendments to 2D model topography, much like for the drainage ponds a 'glass wall' approach was adopted. However it should be noted that through the assessment of baseline and proposed model scenario results, this location did not fall within the floodplain for the 0.1% AEP event.

4.2.5 The scheme design included the closure and infill of a pedestrian subway/underpass close to the Countess Roundabout. This underpass was included within the baseline model as a 1D ESTRY culvert element, which was subsequently removed within the proposed scenario model. It should be noted that the underpass was not shown to flood within the baseline scenarios.

5 River Till Model Results

5.1 Baseline Scenario Results

5.1.1 The baseline fluvial model was simulated for the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 3.33% AEP, 2% AEP, 1.33% AEP, 1% AEP, 1% AEP + 40% climate change and 0.1% AEP events. Sensitivity was tested through simulation of the 1% AEP event including an uplift in peak flow of 85%, corresponding to the Upper climate change allowance.

5.1.2 Comparison of modelled flood extents with corresponding Environment Agency Flood Zones for the River Till is included within the main FRA document and hence this is not repeated here.

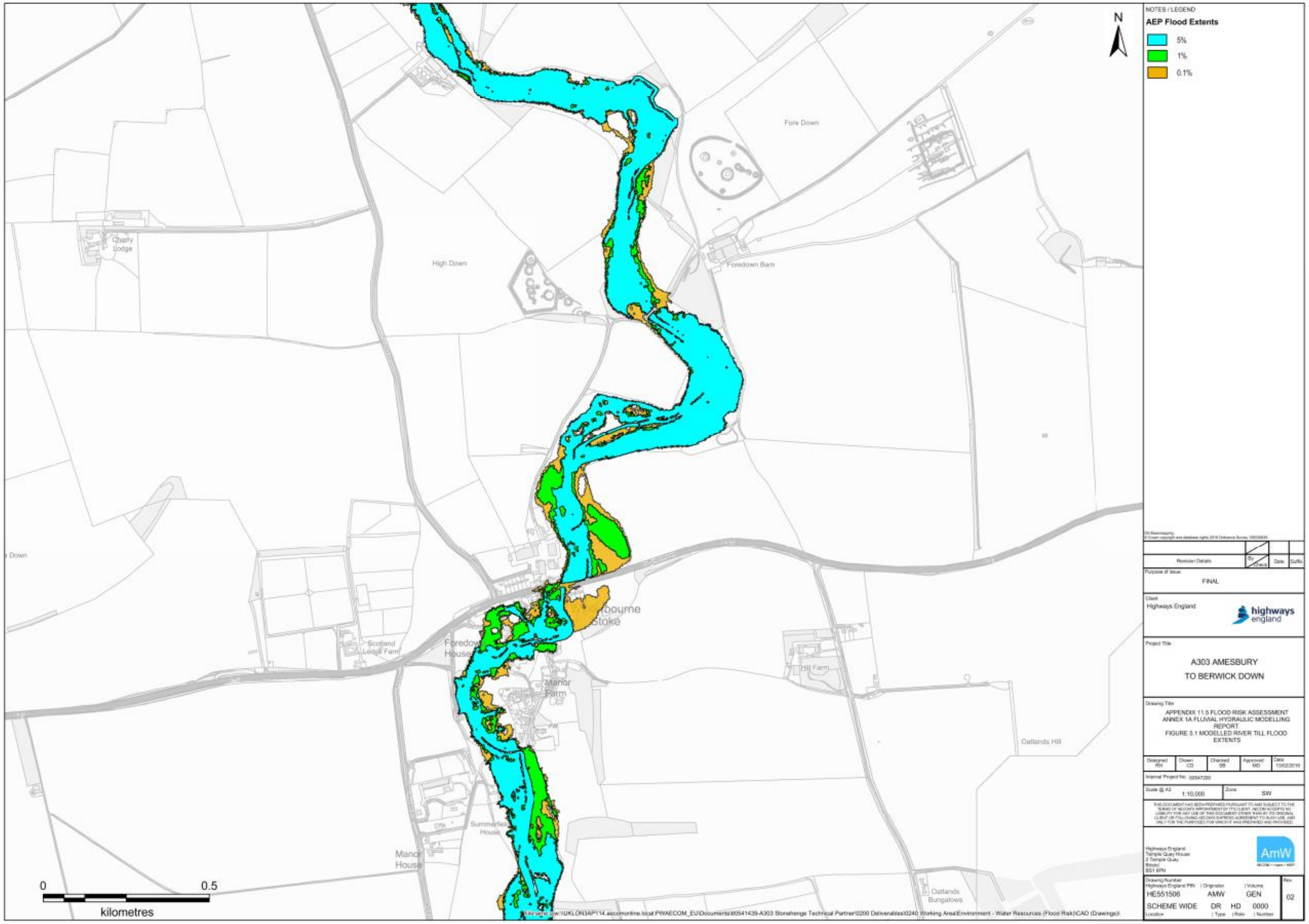


Figure 5.1: River Till Baseline Maximum Flood Extent Comparison

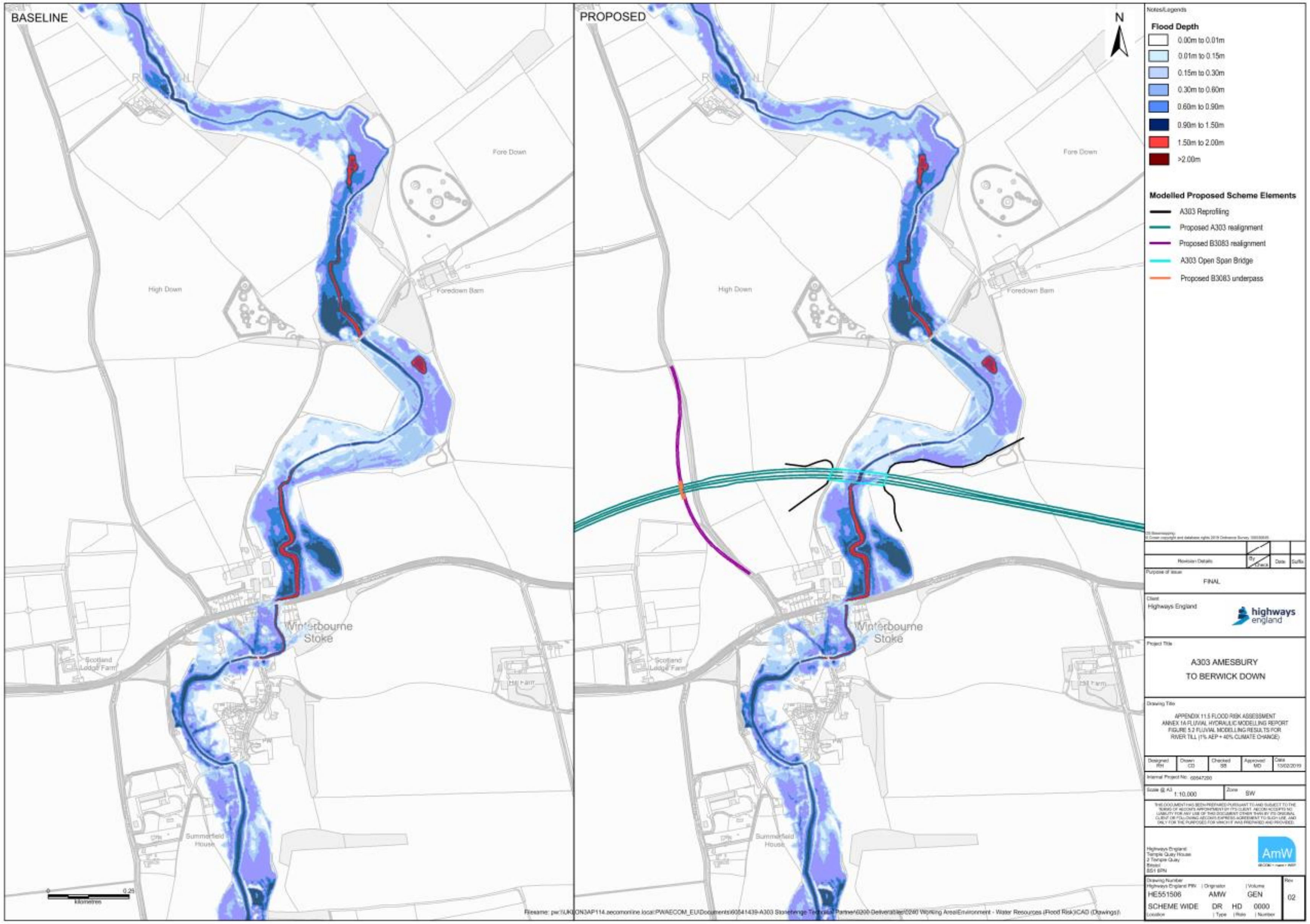


Figure 5.2: River Till Maximum Flood Depth Comparison (Permanent) 1% AEP Plus Climate Change (40%)

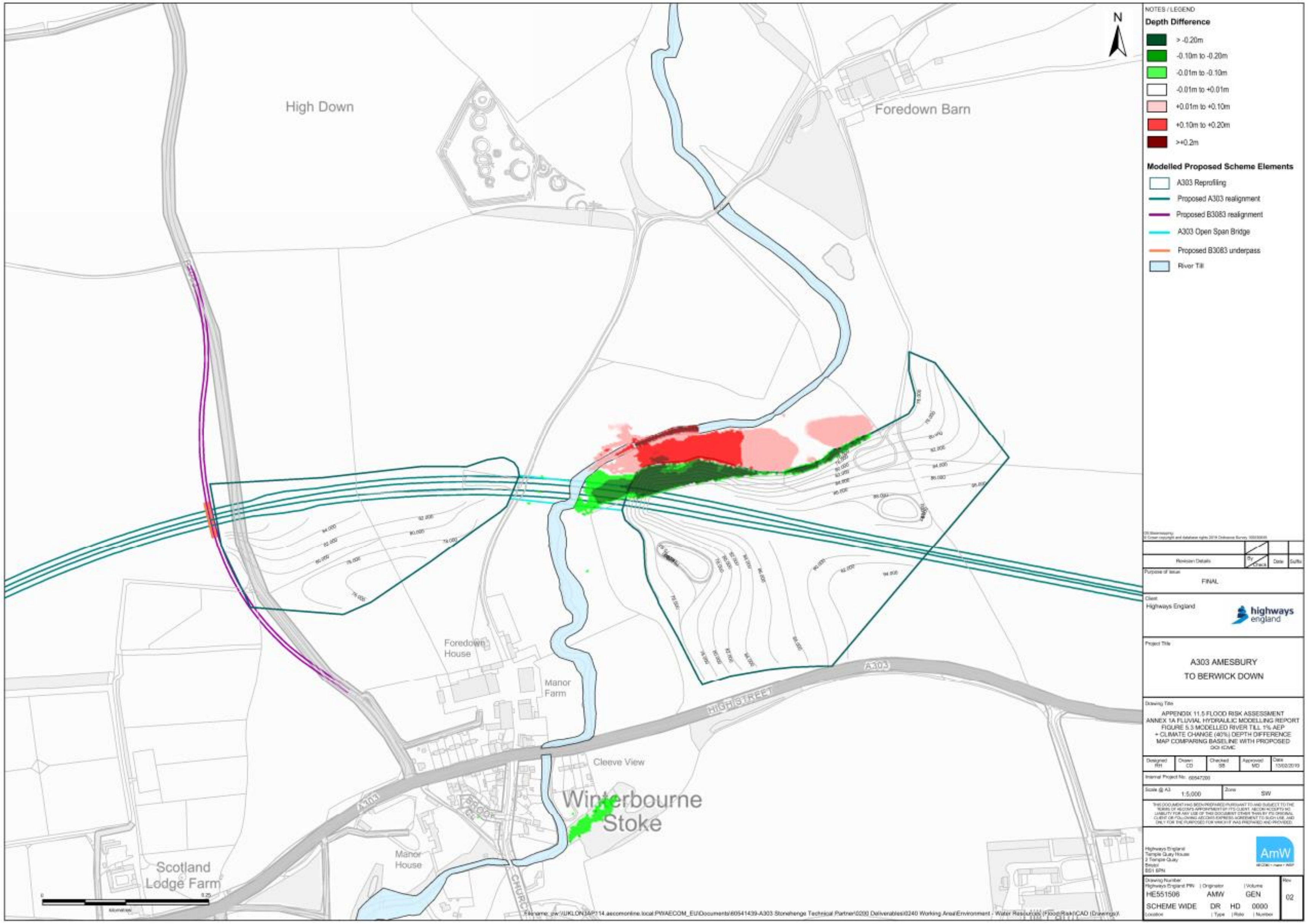


Figure 5.3: River Till Maximum Flood Depth Difference Plot (Permanent) 1% AEP Plus Climate Change (40%)

5.1.3 Figure 5.1 shows a comparison of the maximum flood extents for the modelled 5% AEP, 1% AEP and 0.1% AEP events for the River Till. This comparison figure shows that upstream of the A303 around the meander bend, the maximum flood extents are quite similar for the range of events shown. This suggests that the Avon valley bottom is filled during the 5% AEP flood event at this location. Directly upstream of the A303, and downstream of the A303 crossing there is a larger difference in maximum extent of the different design events.

5.1.4 The comparison plot shown within Figure 5.2 shows the maximum flood depth for the baseline scenario within the 1% AEP design event, including an uplift in peak flow of +40% to account for climate change. This map shows that maximum depths within the River Till channel upstream of the A303 crossing are typically 1.5m-2.0 m, whilst the depth of flooding on the adjacent floodplain reaches depths of up to 1.5m.

5.1.5 Within the baseline scenario flood depths downstream of the A303 crossing are lower both within the channel and on the floodplain. This demonstrates that the bridge crossing of the current A303 exerts an attenuating impact upon flows downstream within the baseline, causing water to back-up upstream of the crossing. It should be noted that water flows over the A303 within the 1% AEP plus climate change and 0.1% AEP design events.

5.2 Proposed Permanent Scenario Results

5.2.1 The proposed fluvial model was simulated for the 5% AEP, 1% AEP, 1% AEP + 40% climate change and 0.1% AEP events. Sensitivity was tested through simulation of the 1% AEP event including an uplift in peak flow of 85%, corresponding to the Upper climate change allowance.

5.2.2 Figure 5.2 shows a comparison of maximum modelled flood depths for the 1% AEP plus climate change event for the baseline and proposed permanent scenario. Figure 5.3 shows a maximum flood depth difference plot, which shows the increases and decreases in flood depth that are attributable to the proposed scheme elements. Key elements of the proposed scheme at this location are highlighted within these figures, and the reader is referred to Section 3.2 of the main FRA document for further detailed information.

5.2.3 Based upon Figure 5.2 it is difficult to visually identify changes attributable to the inclusion of the proposed scheme elements at this location, including the River Till viaduct, associated embankments and level changes, as well as the viaduct piers. This reflects the fact that the River Till viaduct is an open span bridge which is raised high above the floodplain, whilst the embankments are generally outside the extent of the floodplain.

5.2.4 There is a slight increase in modelled floodplain depth to the east of the River Till upstream of the viaduct where the embankment does encroach into the 1% AEP plus climate change floodplain.

5.2.5 The observation documented in Section 5.2.3 is corroborated within Figure 5.3, which shows the associated changes in flood depth. Importantly, the figure shows that the changes associated with the scheme are very localised, and there is no discernible change in flood depth more than 500m upstream or downstream of the River Till viaduct.

5.2.6 There are some localised increases in flood depth upstream of the viaduct and embankments, these are generally less than 0.2m within the 1% AEP + 40% climate change event and are located to the east of the River Till channel. Although there are some increases in depth within the River Till channel itself and upon the floodplain to the west.

5.2.7 It should be noted that there are some commensurate decreases in flood depth where embanking has taken place, and just downstream beneath the River Till viaduct within the 1% AEP + 40% climate change event.

5.2.8 The impacts attributable to the proposed permanent elements of the scheme are similar for the remaining simulated return periods, as shown within the figures within Appendix A. Intuitively, the 0.1% AEP event demonstrates the largest magnitude of change.

5.2.9 Results for the sensitivity analysis for the 1% AEP event, including an uplift of +85% in peak flow, are included within Appendix A.

5.2.10 Overall impacts of the scheme are localised, and there appear to be no increases in flood risk in the region of any vulnerable elements such as properties.

5.3 Proposed Temporary Scenario Results

5.3.1 Figure 5.4 shows a maximum flood depth comparison for the baseline and temporary scenario for the River Till, whilst Figure 5.5 shows the corresponding maximum depth difference map. Both figures correspond to the 1% AEP design event.

5.3.2 These figures demonstrate the impact attributable to the temporary scenario elements, primarily the Bailey bridge and raised haul route. The location of these elements is highlighted upon the figures, and for further information the reader is referred to Section 3.3 of the main FRA document.

5.3.3 Based upon the comparison presented within Figure 5.4 it is difficult to visually assess differences within the maximum flood depth for the baseline and proposed scenario, with the exception of the temporary haul route which is raised above flood levels and not inundated in the 1% AEP modelled event.

5.3.4 The depth difference plot within Figure 5.5 demonstrates that the changes in maximum depth associated with the temporary scheme elements are localised. There are some decreases in maximum depth, directly downstream of the A303, generally up to 0.1m. It should be noted that there are slight increases in maximum depth, directly upstream of the haul route and bridge, however there are less than 0.01m and are therefore not shown on Figure 5.5.

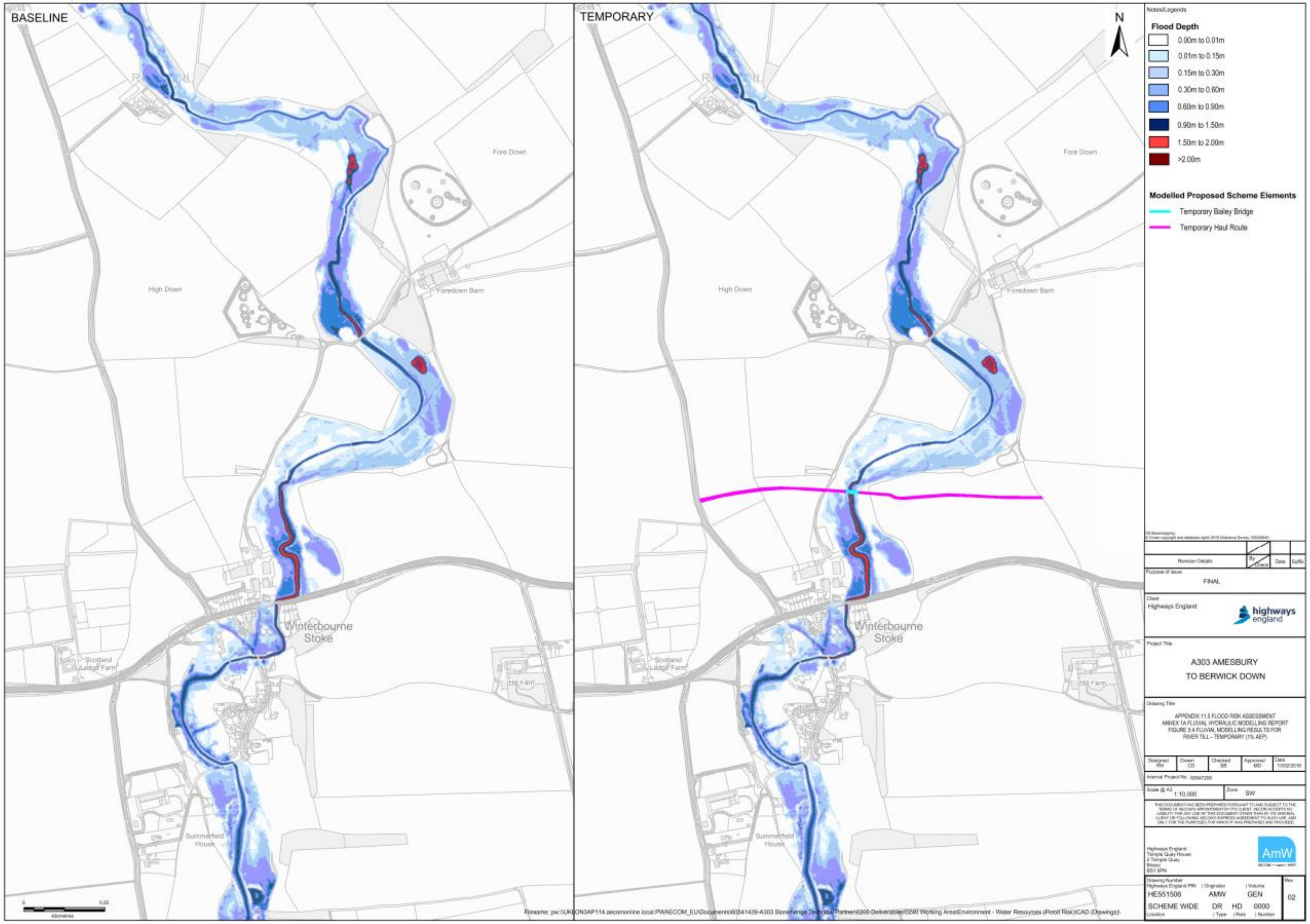


Figure 5.4: River Till Maximum Flood Depth Comparison (Temporary) 1% AEP

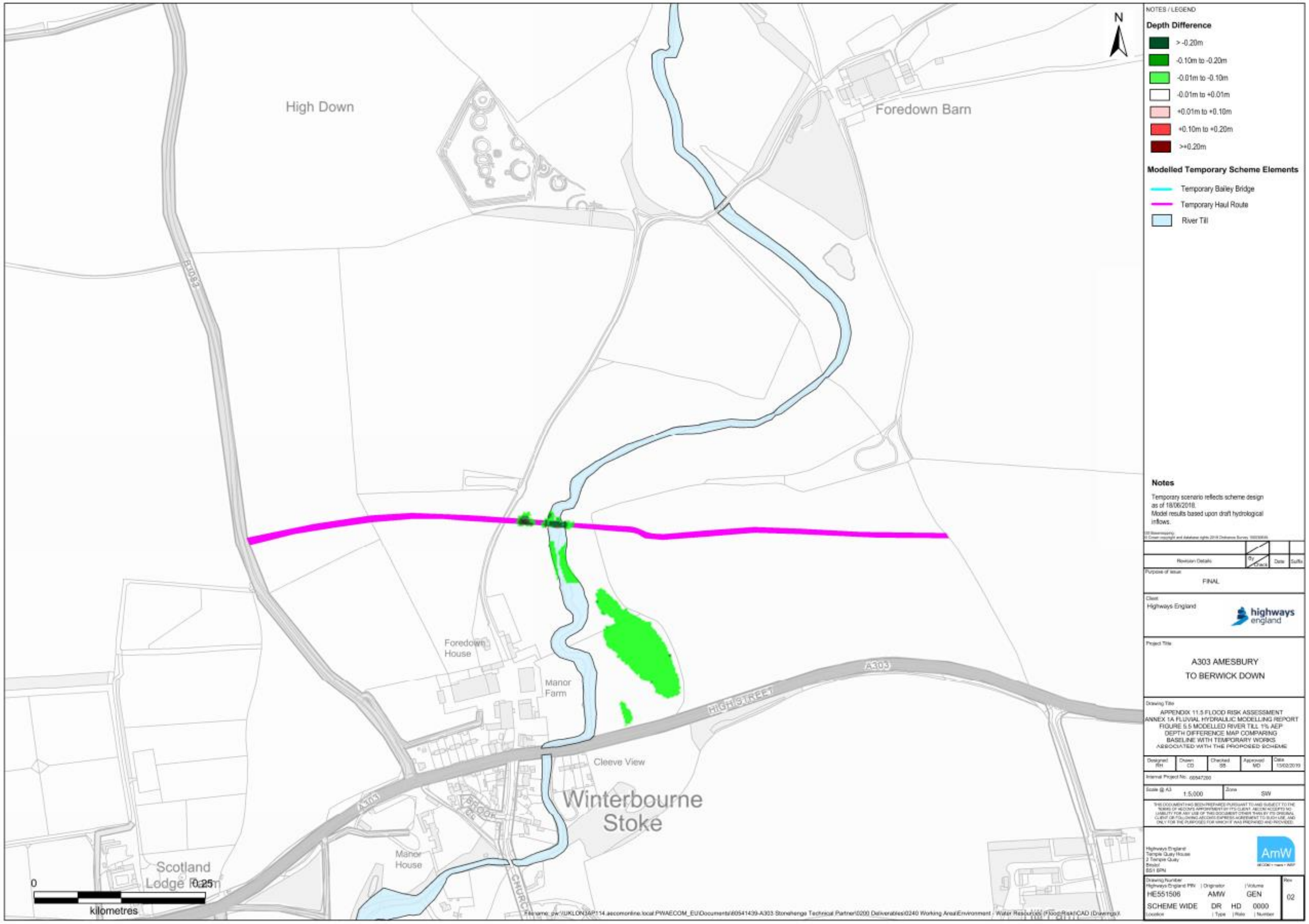


Figure 5.5: River Till Maximum Flood Depth Difference Plot (Temporary) 1% AEP

5.3.5 Further examination of model results suggests that the changes in depth observed can be attributed primarily to a very small decrease in conveyance through the channel, due to the Bailey bridge, in comparison to the baseline. Within the baseline scenario floodplain flow velocities are very low around the location of the haul route and Bailey bridge, indicating that out of bank flow from the River Till ponds and is stored on the floodplain, rather than flowing downstream parallel to the channel. It is thought that this is the reason why raising of the haul route does not exert a more discernible impact upon flood depths. The minimal impact on flood depths shows that the proposed temporary conveyance system works very effectively.

5.3.6 Overall, the modelling undertaken shows that the impact of proposed temporary elements of the scheme upon flood depth and extent is very localised for the 1% AEP event. Furthermore, temporary elements of the scheme should not lead to an increase in flood risk within the region of any vulnerable receptors for the 1% AEP event.

5.4 Sensitivity Analysis Results

5.4.1 A global 20% increase and decrease was applied to Manning's Roughness Coefficient ('n') value representation within the 1D channel and 2D floodplain to assess how sensitive the model is to changes in roughness and build confidence in the assumptions made to channel characteristics during the model build. In-channel water levels were extracted within the area of interest (Table 5.1) to assess the sensitivity of channel and floodplain roughness. The location of the nodes used for data extraction are shown in Figure 5.6.

Table 5.1: In-Channel Water Levels from Manning's Roughness Sensitivity Analysis (River Till)

Node	Node Type	Baseline (1% AEP) (m AOD)	-20% n (1% AEP) (m AOD)	Difference (m)	+20% n (1% AEP) (m AOD)	Difference (m)
RT_XS_16	Channel	73.72	73.70	- 0.02m	73.75	+ 0.03m
RT_18	Channel	73.56	73.53	- 0.03m	73.59	+ 0.03m
RT_17B	Channel	73.19	73.15	- 0.04m	73.22	+ 0.03m
RT_17	Channel	71.66	71.51	- 0.15m	71.70	+ 0.04m
RT_17_BU	Access Bridge	71.66	71.51	- 0.15m	71.70	+ 0.04m
RT_XS_14	Channel	71.56	71.43	- 0.13m	71.59	+ 0.03m
RT_XS_13	Channel	71.54	71.41	- 0.13m	71.56	+ 0.02m
RT_XS_12	Channel	71.53	71.40	- 0.13m	71.54	+ 0.01m
RT_16	A303 Bridge	71.46	71.34	- 0.12m	71.47	+ 0.01m
RT_16_DS	Channel	71.16	71.08	- 0.08m	71.20	+ 0.04m
RT_XS_11	Channel	71.08	71.01	- 0.07m	71.10	+ 0.02m

5.4.2 Table 5.1 generally shows that in-channel levels decrease when the Manning's Roughness Coefficient values are reduced throughout the model and increase when the Manning's Roughness Coefficient values are raised. Within the area of interest, the model appears to be more sensitive when the roughness is decreased, with the change in water level ranging between -0.15m and -0.02m, compared to +0.01m and +0.04m when roughness is increased.

5.4.3 When looking at the entire model, there is an average increase of +0.04m when the roughness is increased (with a maximum increase of +0.11m) and an average decrease of -0.05m when the roughness is decreased (with a maximum decrease of -0.19m). Overall, the change in water level for both roughness sensitivity simulations is not considered significant and consequently, the assumptions currently used within the model are deemed appropriate.

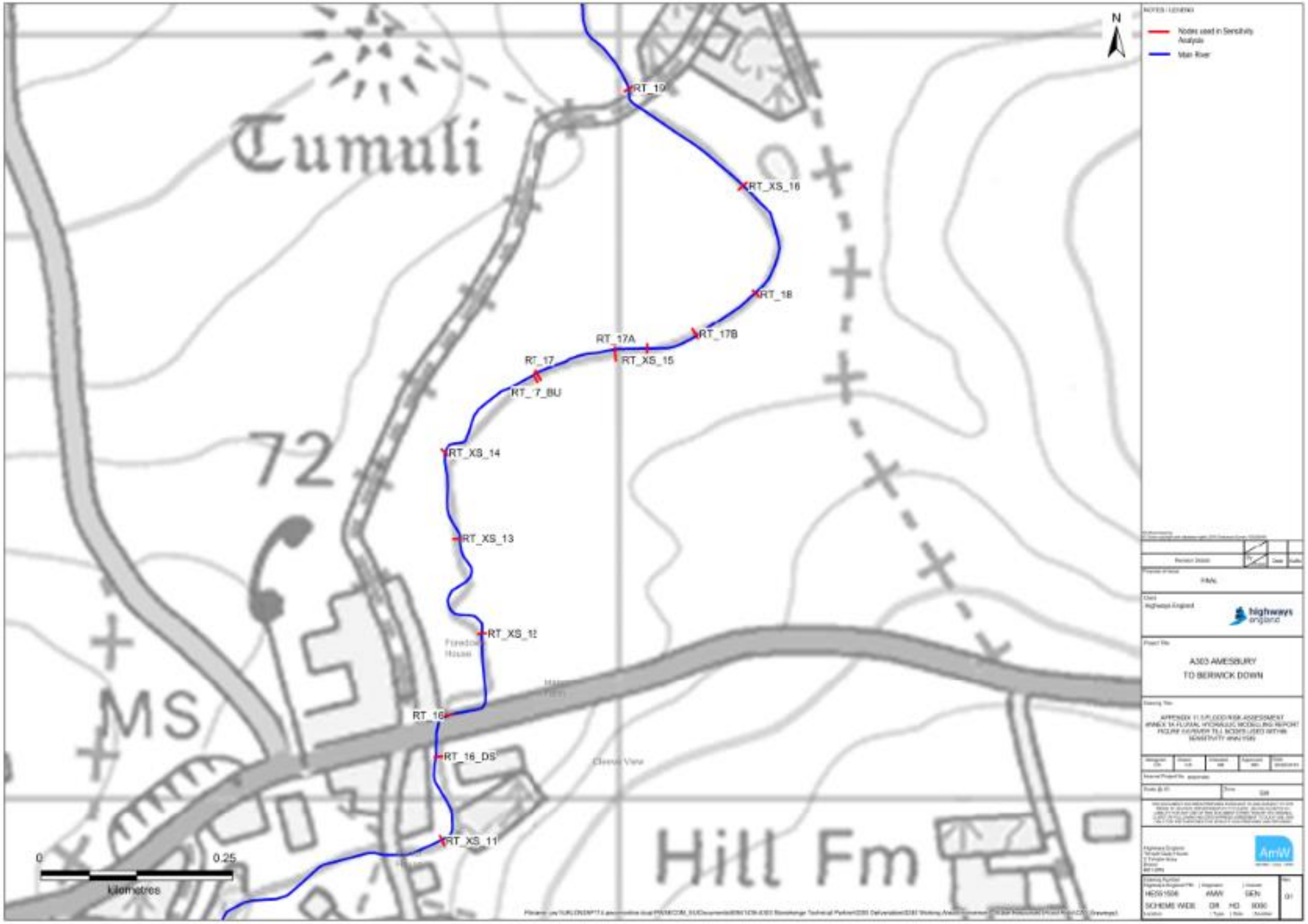


Figure 5.6: River Till Nodes Within Area of Interest for Sensitivity Analysis

6 River Avon Model Results

6.1 Baseline Scenario Results

6.1.1 The baseline fluvial model was simulated for the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 3.33% AEP, 2% AEP, 1.33% AEP, 1% AEP, 1% AEP + 40% climate change and 0.1% AEP events. Sensitivity was tested through simulation of the 1% AEP event including an uplift in peak flow of 85%, corresponding to the Upper climate change allowance.

6.1.2 Comparison of modelled flood extents with corresponding Environment Agency Flood Zones for the River Avon is included within the main FRA document and hence this is not repeated here.

6.1.3 Figure 6.1 shows a comparison of the maximum flood extents for the modelled 5% AEP, 1% AEP and 0.1% AEP events around the location of the A303 crossing of the River Avon. During the 5% AEP event over bank flow occurs immediately upstream and downstream of the A303 where the River Avon flows beneath the highway. In the area where the River Avon splits i.e. to the south of the main A303 roundabout, water is largely contained within bank during this event. For the 1% AEP event more over bank flow occurs within this area, while there a substantial increase in the maximum flood extent for the 0.1% AEP event, where the majority of the River Avon valley becomes inundated. The A303 is not inundated within any of the modelled design events.

6.1.4 Figure 6.2 shows the depth and extent of flooding within the baseline scenario for the 1% AEP + 40% climate change event. Upstream of the A303 crossing depths of flooding are generally up to 1.5m deep upon the floodplain. Immediately downstream of the A303 crossing out of bank flooding occurs directly to the south of the A303 and in-between the two River Avon channels. Depths of flooding are generally up to 1.5m deep, the exception being within the small ditches either side of the River Avon where depths reach approximately 2.0m.

Figure 6.1: River Avon Baseline Maximum Flood Extent Comparison

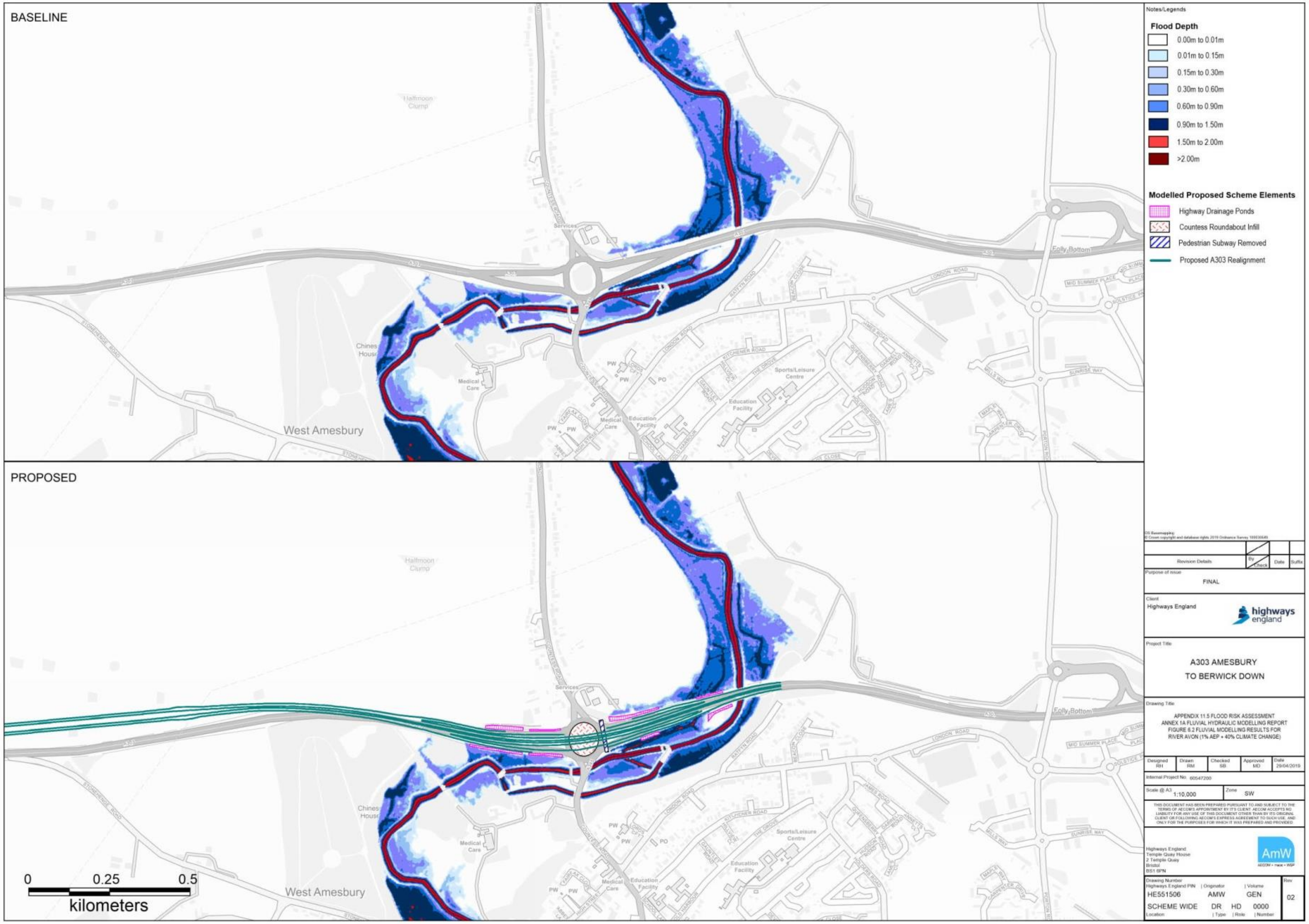


Figure 6.2: River Avon Maximum Flood Depth Comparison- 1% AEP Plus Climate Change (40%)

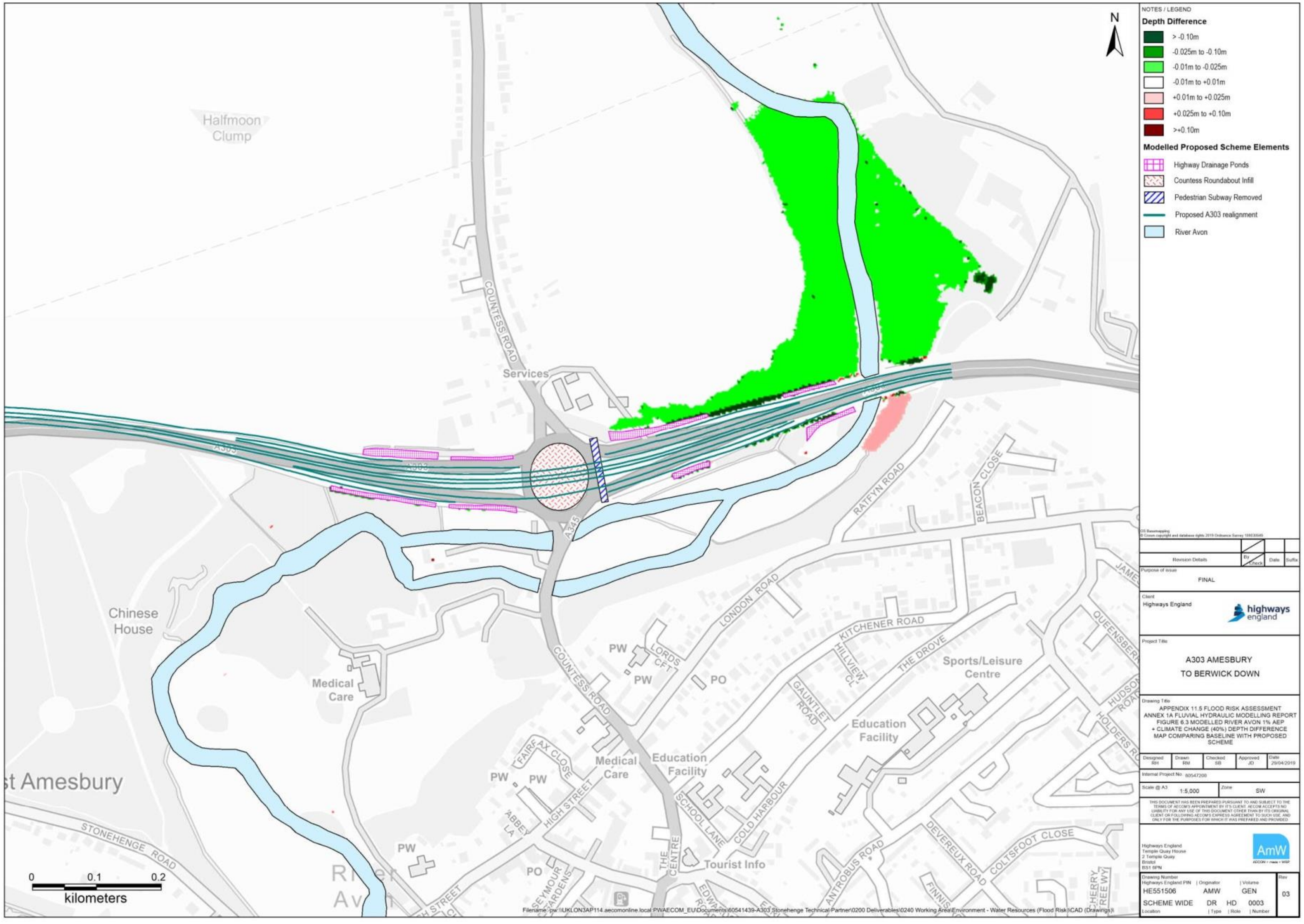


Figure 6.3: River Avon Maximum Flood Depth Difference Plot- 1% AEP Plus Climate Change (40%)

6.2 Proposed Scenario Results

6.2.1 The proposed fluvial model was simulated for the 5% AEP, 1% AEP, 1% AEP + 40% climate change and 0.1% AEP events. Sensitivity was tested through simulation of the 1% AEP event including an uplift in peak flow of +85%, corresponding to the Upper climate change allowance.

6.2.2 Figure 6.2 shows a comparison of maximum modelled flood depths for the 1% AEP + 40% climate change event for the baseline and proposed scenario. Figure 6.3 shows a maximum flood depth difference plot, which shows the increases and decreases in flood depth that are attributable to the proposed scheme elements. Key elements of the proposed scheme at this location are highlighted within these figures, and the reader is referred to Section 3.2 of the main FRA document for further detailed information.

6.2.3 Based upon Figure 6.2 it is difficult to visually identify any changes in maximum flood extent or depth as a result of inclusion of the proposed permanent elements of the scheme, namely the road drainage ponds, infill of Countess Roundabout and removal of the pedestrian underpass. Countess Roundabout and the pedestrian underpass fall outside the flood extent for this design event.

6.2.4 The observations made in Section 6.2.3 are corroborated within the maximum flood depth difference plot presented in Figure 6.3. This figure illustrates that the only discernible changes attributable to the scheme are decreases in depth observed within the area of the road drainage ponds, which are no longer flooded in the proposed scenario due to the presence of raised bund crest levels. However model results show that there is a slight displacement of floodwater within the area immediately to the south of the A303 to the east of the Avon. This area experiences a small flood depth increase that is between 0.01m and 0.025m for the 1% AEP + 40% climate change event.

6.2.5 Comparison results for the remaining simulated return periods are presented within Appendix B, including for the 1% AEP + 85% climate change sensitivity simulations. Intuitively, for the 5% AEP and 1% AEP events the magnitude of changes attributable to the scheme is lesser than the 1% AEP + 85% climate change event.

6.2.6 Overall, hydraulic modelling for the River Avon has demonstrated that any changes in depth attributable to the proposed scheme are localised. During the 1% AEP + 40% climate change design event, there are small increases in flood depths to the south of the proposed attenuation drainage ponds, the magnitude of this increase is 0.01m to 0.025m (Figure 6.3). This is discussed in Section 6.5. It should be noted however that these increases in depth are localised and do not coincide with any vulnerable receptors such as properties, businesses and infrastructure.

6.3 Sensitivity Analysis Results

6.3.1 A global 20% increase and decrease was applied to all Manning’s Roughness Coefficient (‘n’) values within the 1D channel and 2D floodplain to assess how sensitive the model is to changes in roughness and build confidence in the assumptions made to channel characteristics during the model build. Results from the area of interest (i.e. the proposed A303 alignment) are shown in Table 6.1

and the locations of the nodes used during the Sensitivity Analysis are displayed in Figure 6.4.

Table 6.1: In-Channel Water Levels from Manning's Roughness Sensitivity Analysis (River Avon)

Label	1D Node	Baseline (1% AEP) (m AOD)	-20% n (1% AEP) (m AOD)	Difference (m)	+20% n (1% AEP) (m AOD)	Difference (m)
RA_XS_35	Channel	70.12	70.13	+ 0.01	70.13	+ 0.01
RA_ST_19CU	A303 Bridge	70.03	70.04	+ 0.01	70.03	0.00
RA_XS_34	Channel	69.89	69.90	+ 0.01	69.89	0.00
RA_ST_18d	Channel	69.48	69.50	+ 0.02	69.48	0.00
RA_XS_33	Channel	69.40	69.41	+ 0.01	69.40	0.00
RA_ST_52CUA	A345 Bridge	69.39	69.40	+ 0.01	69.39	0.00
RA_ST_50	Access Bridge	69.32	69.34	+ 0.02	69.32	0.00
RA_XS_31	Channel	68.80	68.80	0.00	68.80	0.00
RA_XS_30u	Channel	68.80	68.80	0.00	68.80	0.00
RA_ST_15	Channel	68.66	68.66	0.00	68.66	0.00
RA_ST_17u	2nd Channel	69.85	69.85	0.00	69.85	0.00
RA_ST_17CU	A345 Bridge	69.69	69.69	0.00	69.69	0.00
RA_ST_16d	2nd Channel	68.86	68.86	0.00	68.86	0.00

6.3.2 Table 6.1 generally shows marginal differences when the Manning's Roughness Coefficient is changed within the 1D model. Within the area of interest, the model does not appear to be significantly sensitive when the roughness is altered, with the change in water level ranging between +0.01m and +0.02m when roughness is decreased, compared to +0.01m when roughness is increased.

6.3.3 When looking at the entire model, there is an average increase of +0.01m when the roughness is increased (with a maximum increase of +0.03m) and an average decrease of -0.01m when the roughness is decreased (with a maximum decrease of -0.03m). Overall, the change in water level for both roughness sensitivity simulations is not considered significant and consequently, the assumptions currently used within the model are deemed appropriate.

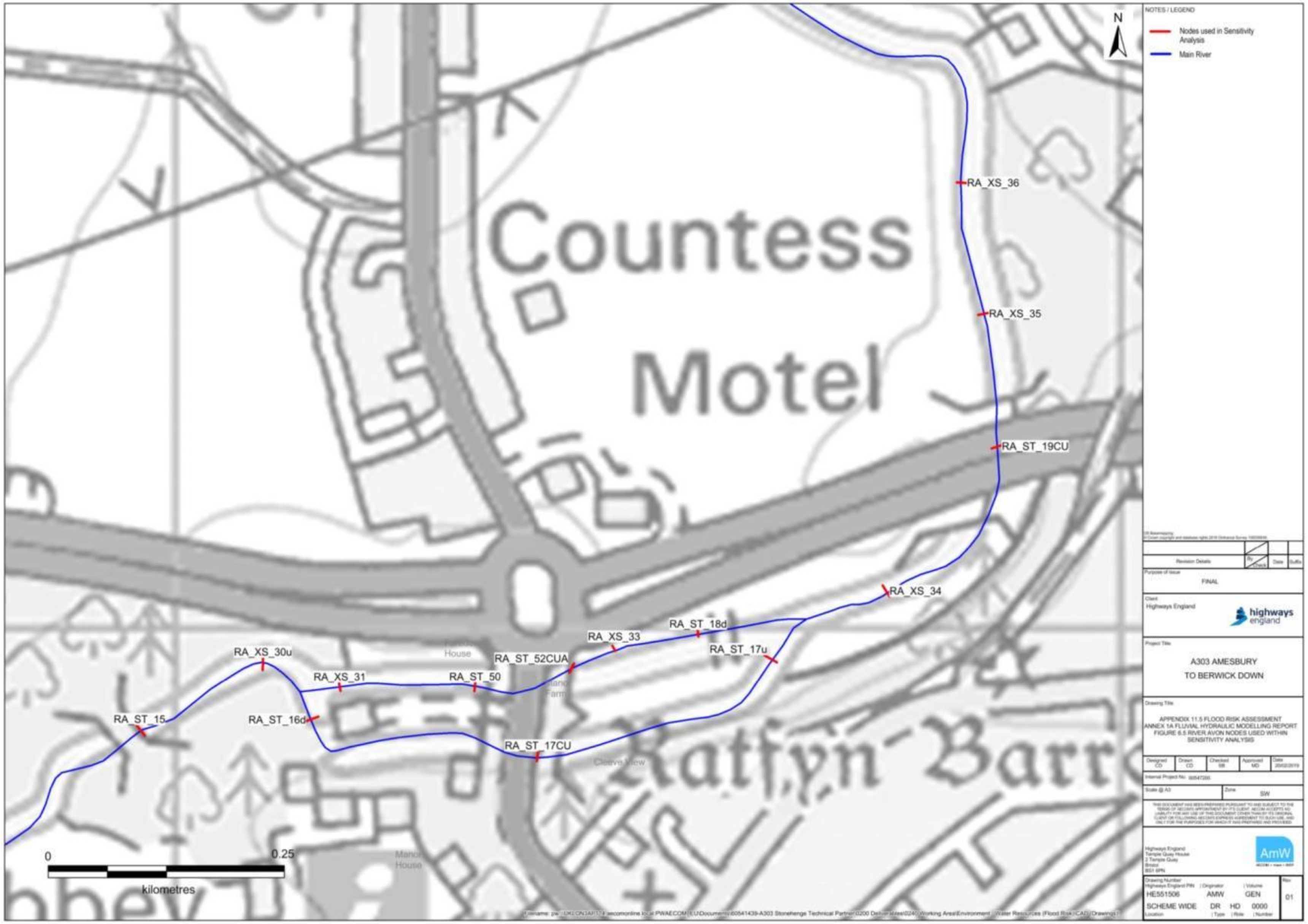


Figure 6.4: River Avon Nodes Within Area of Interest for Sensitivity Analysis

6.3.4. A further sensitivity test was undertaken in order to test the model response to longer duration flood events for the River Avon. Groundwater fed watercourses such as the River Avon are often characterised by extended periods of raised water levels, hence it was deemed appropriate to further explore the potential impacts upon flood risk within the modelled reach.

6.3.5. The sensitivity test involved re-simulation of the baseline hydraulic model using a hydrograph supplied by the Environment Agency. This hydrograph was extracted from Continuous Simulation Modelling (CSM) for Salisbury, currently being undertaken by the Environment Agency and JBA Consulting. The simulation was undertaken for the 1% AEP event for a total of 800 hours, in line with the duration of the supplied hydrograph. Key conclusions of the sensitivity test are outlined below, for further details on the methodology and results the reader is referred to Appendix C of this report.

6.3.6. The following key conclusions were drawn based upon the sensitivity testing undertaken:

- Maximum flood depths are significantly higher within the 1% AEP CSM sensitivity simulation when compared to the 1% AEP design event. The maximum flood extent for the 1% AEP design event and the 1% AEP CSM sensitivity simulation are shown within Figure 6.5. This shows that in a few locations across the reach, the 800 hour CSM sensitivity results have a larger flood extent than the 60 hour results. However, in general the flood extents of the two model results are equivalent.
- The peak flow within the CSM hydrograph is significantly higher than the modelled design event with an equivalent AEP. Comparison of results strongly suggests that the greater maximum depths observed within the CSM sensitivity simulation can be attributed to peak flow.
- It does not appear that longer duration hydrographs associated with greater flow volumes lead to an increase in maximum flood depth, or extent.
- The results of the sensitivity test provide further evidence that the modelled design events for the River Avon are robust, and appropriate for assessment of the proposed scheme.

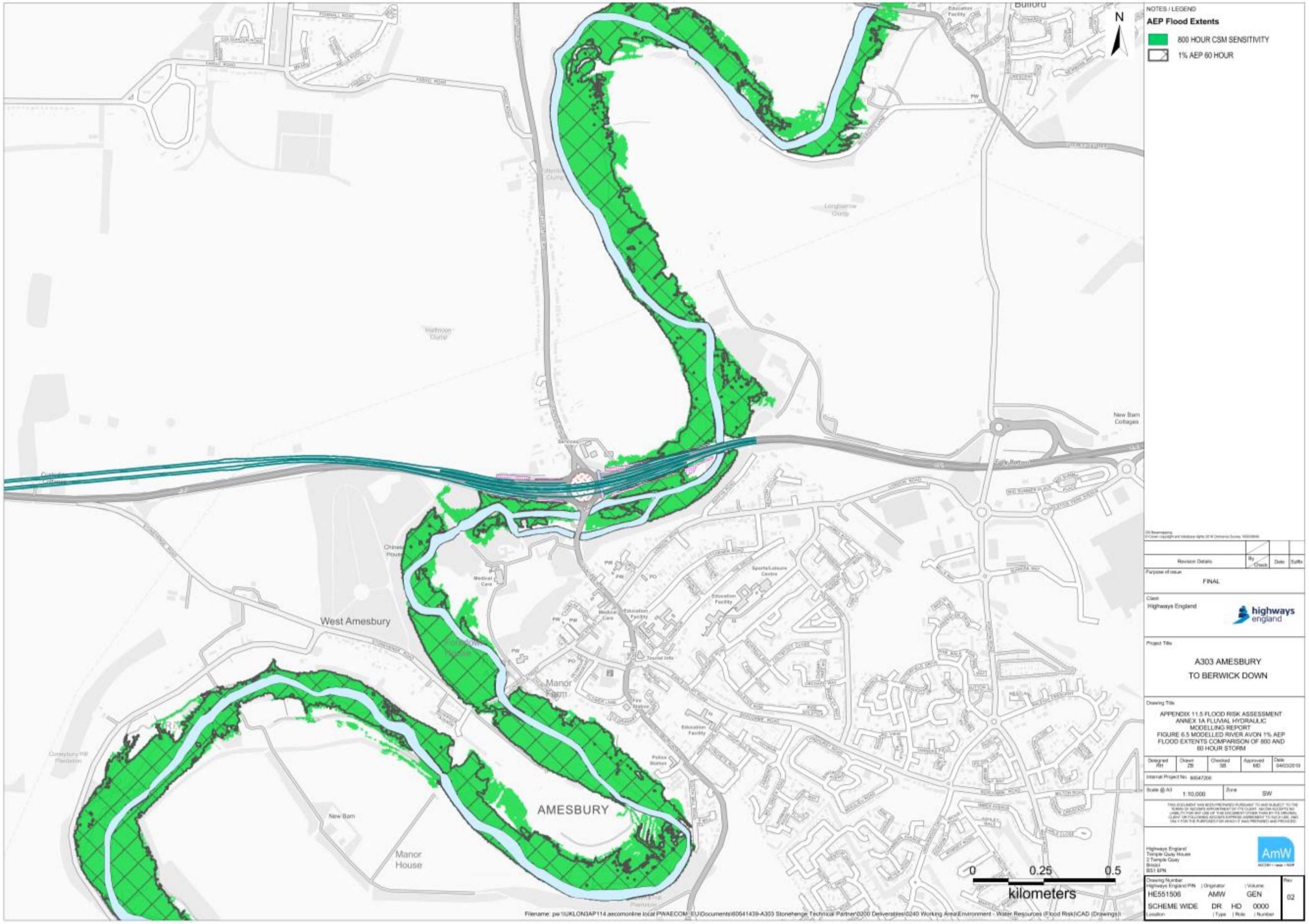


Figure 6.5: Flood Extent Comparison - 1% AEP Flood Event

6.4 Check Files

6.4.1 As part of the outputs from the model, check files are produced which are useful for confirming the file inputs to the model and appropriateness of the setup. The check files can be used to ensure that certain features are being represented correctly and allow for the accuracy of the DTM to be checked.

6.4.2 The key warnings and checks which remain in the model (according to the messages layer which forms part of the check files) are presented within Table 6.2. Following a review of the type and location of these warnings/checks it is considered that they are unlikely to have a significant effect on the modelling results.

Table 6.2: Model Warnings and Checks

Model	Warning/Check ID	Warning/Check Description
River Till	Warning 0305	Projection of .mif is different to that specified by the MI projection
	Check 2370	Ignoring coincident point found in TIN layer
	Check 2077 / 2078	Beginning/end of 3D TIN breakline dangling
	Check 2099	Ignored repeat application of boundary to 2D cell
River Avon	Warning 1317	WLL does not cross or snap to 1D channel
	Warning 2118	Lowered SX ZC Zpt to 1D node bed level
	Check 2370	Ignoring coincident point found in TIN layer
	Check 2099	Ignored repeat application of boundary to 2D cell

6.5 Floodplain Volume Displacement

6.5.1 With the proposed highway attenuation drainage ponds located within the floodplain, there is potential for an associated loss in floodplain storage within modelled design events with a magnitude greater than 5% AEP. Modelling results presented within Section 6.2 demonstrates that increases in floodplain depth resulting from displacement of floodwater by highways drainage ponds are localised and do not coincide with vulnerable receptors. There are no discernible changes to flood extents. Furthermore any displacement of volume does not lead to changes in flood extents and flows further upstream or downstream on the River Avon. The Environment Agency have confirmed that although the drainage ponds are likely to have only a small impact compared with the extent of the wider floodplain, mitigation should still be required to compensate for the loss of storage within the floodplain.

6.5.2 Volume calculations were undertaken for drainage ponds that form part of the proposed scheme around the River Avon close to Countess Roundabout. In total, from the six proposed drainage ponds that are within the 1% AEP plus climate change floodplain, the total amount of volume displaced within this event is approximately 1,230m^3. It should be noted that drainage ponds located both to the north and south of the A303 contribute towards the displacement of flood water within the 1% AEP + 40% climate change event, however this displacement only leads to an increase in flood depths to the south of the A303. Discussion regarding the displacement of flood waters by highways drainage ponds will continue through the development of the scheme.

6.6 Model Validation

6.6.1 A comparison of historic flood areas in the communities surrounding the River Till and the River Avon (within the study area) and the modelled 1% AEP event has been undertaken. The comparison between modelled results and historical observations of flooding are displayed in Figure 6.6 and Figure 6.7. Table 6.3 provides further information regarding when the historic flood took place, type of flooding and where this information was derived from, for both the River Till and the River Avon.

6.6.2 Seven out of the ten historic flood areas along the River Till are situated within the modelled 1% AEP scenario extent. The remaining three areas are situated nearby a small tributary which flows into the River Till. This tributary is not included within the model and this explains why the historic flood areas are not within the flooded extent.

6.6.3 All three historic flood areas along the River Avon are outside of the modelled 1% AEP scenario extent. The two flood areas in Bulford are situated in close proximity to the Nine Mile River (which has not been included within the model) and the flood area near Amesbury is just outside the modelled flood extent.

6.6.4 It should be noted that it is not possible to estimate the AEP for the flood event that each flood area corresponds to for the River Till as there are no gauging stations along this section. The River Avon does have a gauge station to the south of Amesbury, however the 1% AEP modelled event has been selected for comparison to the historical flood observations.

6.6.5 The limitations of this historic flood event comparison include:

- To protect sensitive property information, individual properties which are known to have flooded internally or externally due to fluvial sources have not been pinpointed. Instead, broader flood risk areas have been displayed based on the approximate location of any historic flooding.
- Difficulty placing the location of where some historic flood photos were taken.
- Residents may not inform their local council or the Environment Agency if a flood incident occurs within their property in fear that their property price may be affected. Therefore, not all records are available to compare.
- The observed flood outline presented in Figure 6.8 covers a limited area and does not appear to reflect the distribution of inundation that would be expected during the occurrence of a flood event. The data has been taken from the Environment Agency Flood Reconnaissance Database, although more specific information on how this data was obtained and the methodology for generation of the flood outline, is unknown. It is also not known whether any quality assurance has been carried out on the flood outline and therefore the reliability of the data cannot be confirmed.
- The aerial images presented in Figure 6.9 were supplied from the Environment Agency Flood Reconnaissance Database, although no further information detailing the precise location and time/date of the photos were

supplied. Without this information it was not possible to use gauge data at Amesbury to determine which modelled specific event extent should be used for comparison. In lieu of this information, the photographs have been compared with the 1% AEP event.

Flooded Areas/Properties at Risk- Winterbourne Stoke

6.6.6 In addition, flood extents from flooding in 1994/95 have been recorded within a drawing titled ‘Flooded Areas / Properties at Risk Plan Winterbourne Stoke’ which was derived from the Environment Agency. These flood extents have been overlaid with the modelled 10% AEP scenario extents, as displayed in Figure 6.8.

6.6.7 The historic flood outline shows a greater flood extent than the results shown for the modelled 10% AEP scenario to the north of the A303.

6.6.8 The modelled 10% AEP results show a similar flood extent to the historical flood extents shown on Church Street, south of the A303. The flood extent on the right bank of the channel immediately downstream of the A303 is smaller for the 10% AEP results than the historic flood event outline. Additionally, on the left bank of this channel section, flooding is shown during the 10% AEP event but not during the historic flood event outline. Where the channel then meanders west, the flood extents are similar. Further downstream, the 10% AEP results have a greater flood extent than the historic flood extents outline on the right bank of the River Till. Additionally, during the 10% AEP event, flooding occurs on both the right and left bank of the River Till in this location, and the historic flooding extent is only recorded on the left bank.

6.6.9 Although there are some discrepancies between modelled and reported flood extents, any differences could also be attributable to the fact that flood extents may have not been reported, or reported accurately.

Historic Flood Areas

6.6.10 Aerial photographs displaying flood extents were received from the Environment Agency flood reconnaissance archive. Those images that clearly represent areas within the study area are displayed within Figure 6.9.

6.6.11 As seen in Figure 6.9, the flood extents within the photographs and the modelled 1% AEP scenario extents are similar.

6.6.12 The photograph displaying the River Avon floods between Durrington and Bulford (furthest to the north) show large flood extents (similar to the modelling results) which overtopped the right bank of the river channel.

6.6.13 The second photograph, along the straight River Avon channel section shows historic flooding overtopping the left bank of the river channel. These flooding extents are again similar to the modelling results.

6.6.14 The historic photograph of flooding near the Countess Roundabout shows a large area of flooding towards the south and the north of the A303 extending from the River Avon channel. The modelling results also display this.

6.6.15 The photograph displaying flooding along the River Avon meander to the west of Amesbury displays extensive flooding which overtopped the left and right banks of the river channel, particularly along straight sections of the channel. Very similar results are shown in the modelling results.

6.6.16 The photograph of the River Till historic flooding shows narrower flooding extents near the current A303 road, than upstream, with a small ponded area to the east. Wider extents are displayed to the north of the proposed A303 and viaduct, particularly along the left river bank. The modelling results present similar flooding extents to those in the photograph.

Table 6.3: Historic Flood Areas

Community	Date of Flooding	Type of flooding	Flood Source Information	File Name
Orcheston	December 2000, 1995	Fluvial and Groundwater	Salisbury District Council and Resident	Orcheston – File.pdf, Environment Agency's Flood Incident Response photos
Orcheston	September 2001	Fluvial	Salisbury District Council	Orcheston – File.pdf
Orcheston	January 2014	Fluvial	Environment Agency	Environment Agency's Flood Incident Response photos
Orcheston	January 2014	Fluvial	Environment Agency	Environment Agency's Flood Incident Response photos
Shrewton	January 2013	Fluvial	Resident informed Environment Agency	Till Valley – 2003
Shrewton	1990 and December 2000/ January 2001	Fluvial	Environment Agency	Spreadsheet from Environment Agency: Shrewton.xls
Shrewton	January 2014	Fluvial	Environment Agency	Environment Agency's Flood Incident Response photos
Shrewton	January 2014	Fluvial and Groundwater	Environment Agency	Environment Agency's Flood Incident Response photos
Winterbourne Stoke	November 1998 (when map was created)	Fluvial	Map drawn by Salisbury District Council	Winterbourne Stoke – Ward 11 Land Drainage Temporary File – PDF
Winterbourne Stoke	November 1998 (when map was created)	Fluvial	Map drawn by Salisbury District Council	Winterbourne Stoke – Ward 11 Land Drainage Temporary File – PDF
Bulford	January 2014	Fluvial	Environment Agency	Spreadsheet from Environment Agency: Wessex MASTER Properties Flooded Winter 2013 - 2014
Bulford	January 2014	Fluvial	Environment Agency	Spreadsheet from Environment Agency: Wessex MASTER Properties Flooded Winter 2013 - 2014
Amesbury	January 2014	Fluvial	Environment Agency	Environment Agency's Flood Incident Response photos

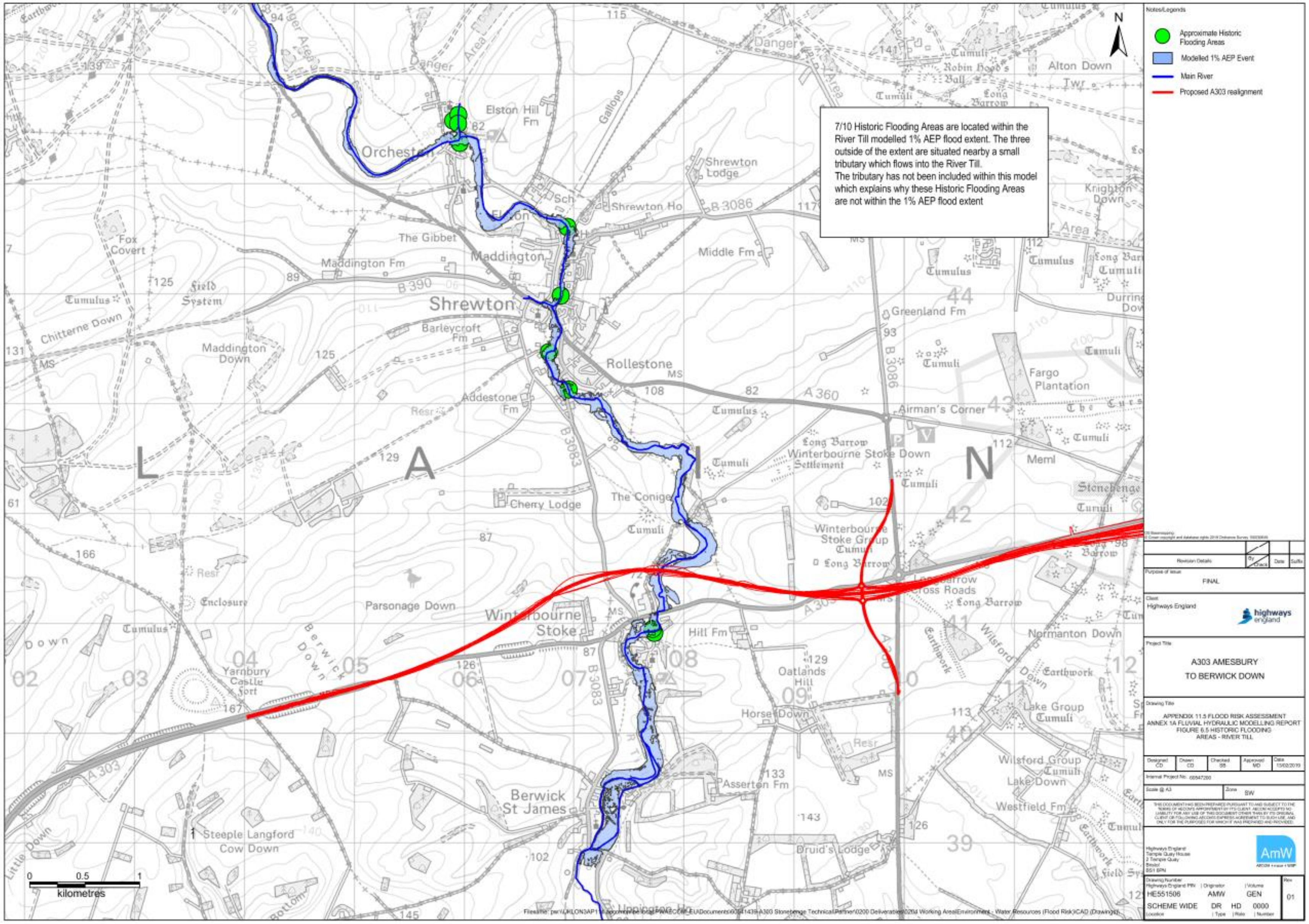


Figure 6.6: Historic Flooding Areas – River Till

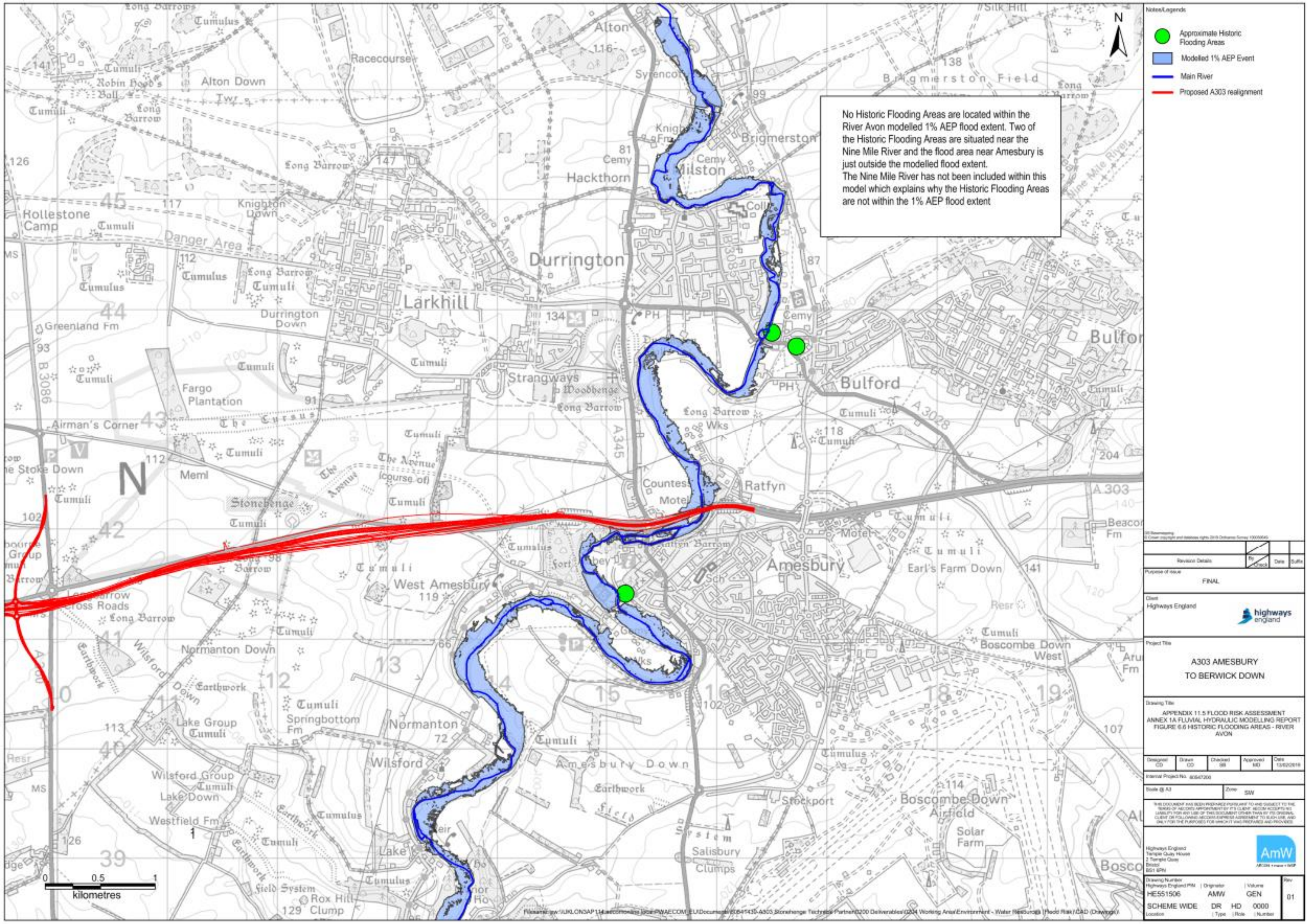


Figure 6.7: Historic Flooding Areas – River Avon

Figure 6.8: Flood Extent Comparison

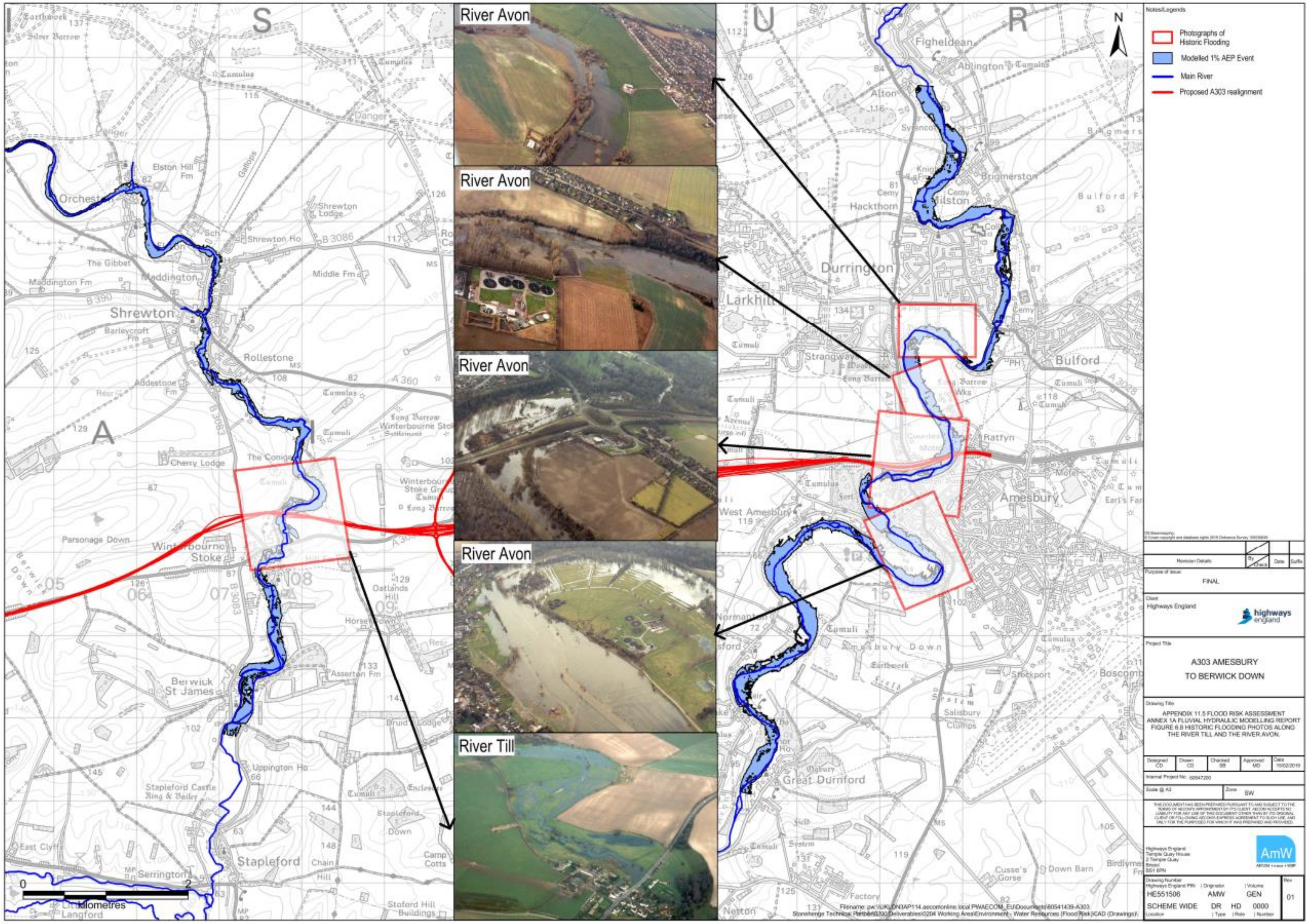


Figure 6.9: Historical Flood Photos

7 Limitations

7.1 General Limitations

7.1.1 Hydraulic modelling for the River Till and River Avon was completed using a broadly similar approach, thus a number of the uncertainties are common amongst both approaches. It is important that the model results and changes in flood risk associated with the proposed scheme are considered within the context of these uncertainties.

7.1.2 Uncertainties associated with hydrological inflows generated through FEH methods are typically the largest source of uncertainty associated with hydraulic modelling. For ungauged catchments peak flows estimated through the best available FEH methods are associated with an uncertainty of +/- 40%, this level of uncertainty is generally lower where the catchments are gauged.

7.1.3 Another large source of uncertainty commonly associated with hydraulic modelling is attributable to the data utilised to define floodplain topography. The composite DTM utilised here comprises a combination of Environment Agency LiDAR, high resolution photogrammetric DTM, and a SAR DTM. The stated accuracy of these data sources is included within Table 7.1.

7.1.4 It should be noted that within independent ground truthing, the vertical accuracy of Environment Agency LiDAR was shown to be superior to the photogrammetric DTM, thus the vertical accuracy of the photogrammetric DTM should be regarded as lower than +/-150mm.

Table 7.1: Accuracy of Topographic Data Sources

Topographical Data Source	Spatial Resolution (m)	Stated Vertical Accuracy
Environment Agency LiDAR DTM	2	+- 150 mm
Photogrammetric DTM	1	+- 40 mm
SAR DTM	5	+- 1000 mm

7.1.5 Model sensitivity has only been tested in terms of the ‘Upper’ climate change allowance, which represents a +85% uplift in peak flows for the 1% AEP event. This sensitivity test demonstrated that the modelled flood extents and depths responded in the expected manner for the increase in flow. No further tests of sensitivity to other model parameters were undertaken.

7.1.6 Within the channel cross sectional survey for both River Till and River Avon, a significant number of small footbridges were surveyed, these were typically several metres wide and with a small deck thickness.

7.1.7 In instances where these bridges were remote from the proposed scheme and were deemed to be hydraulically insignificant, they were removed from the 1D model in order to improve model performance. A large number of these bridges were present upon the River Till through Shrewton and Orcheston. Importantly, it is not thought that the absence of these structures would lead to any meaningful change in the overall conveyance of flow to the A303.

7.1.8 The model software used is not able to directly take into account links between fluvial flows and groundwater. Water is only able to enter the model via the specified hydrological inflows and can only leave the model at the downstream boundary, thus no exchange with groundwater can be represented. As both the River Till and River Avon catchments are permeable, there is potential for significant interaction between rivers and underlying groundwater. This is a fundamental limitation with the approach adopted, although it should be noted that other industry standard hydraulic models are also unable to directly represent interactions with groundwater.

7.1.9 In light of Section 7.1.2, the reader is referred to the hydrological analyses for the River Till and River Avon, which document in more detail the methodology utilised for estimation of flood flows within the two watercourses. For the River Till, flows have been estimated using a groundwater flow variability method which considers groundwater interactions and utilises outputs from the groundwater modelling undertaken and documented within Environmental Statement Appendix 11.4 (Groundwater Risk Assessment).

8 Conclusions

8.1 River Till

8.1.1 Hydraulic modelling of the River Till has been completed for a range of design flood events and changes in flood risk attributable to the proposed scheme assessed for a range of flood conditions.

8.1.2 Within the proposed scenario, changes in flood depth attributable to the scheme are localised within the 1% AEP + 40% climate change design event. The observed changes are largely a result of a slight encroachment of the proposed A303 embankment into the existing/baseline River Till floodplain, and associated displacement of floodwater. Importantly, any increases in flood depth do not coincide with any vulnerable receptors.

8.1.3 Within the temporary scenario, changes in maximum flood depth are highly localised and attributable to a slight constriction of flow through the River Till channel by the Bailey bridge and temporary haul route. As for the proposed permanent scenario, the observed increases in flood depth occur within a limited area upstream of the Bailey bridge and do not coincide with any vulnerable receptors.

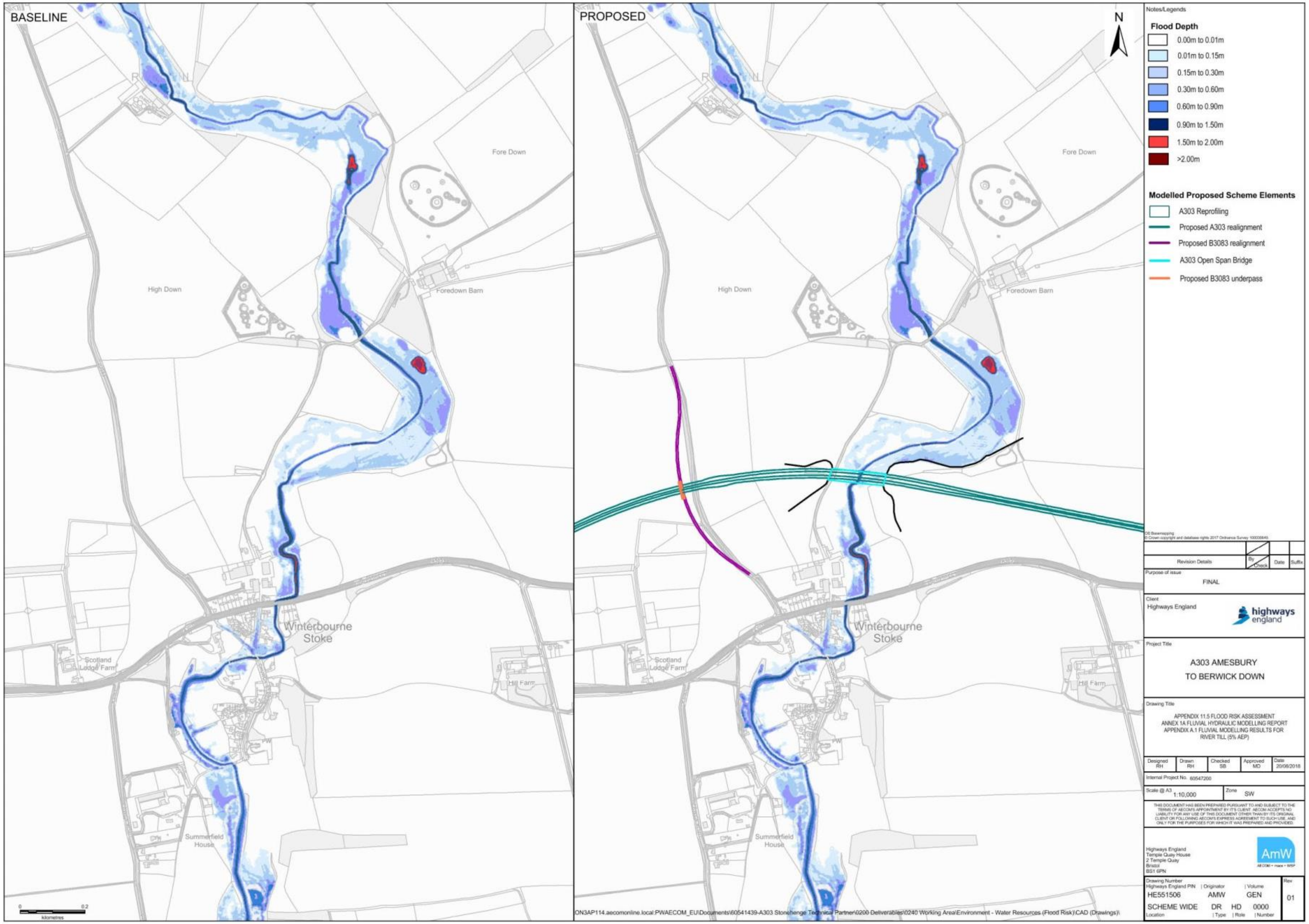
8.2 River Avon

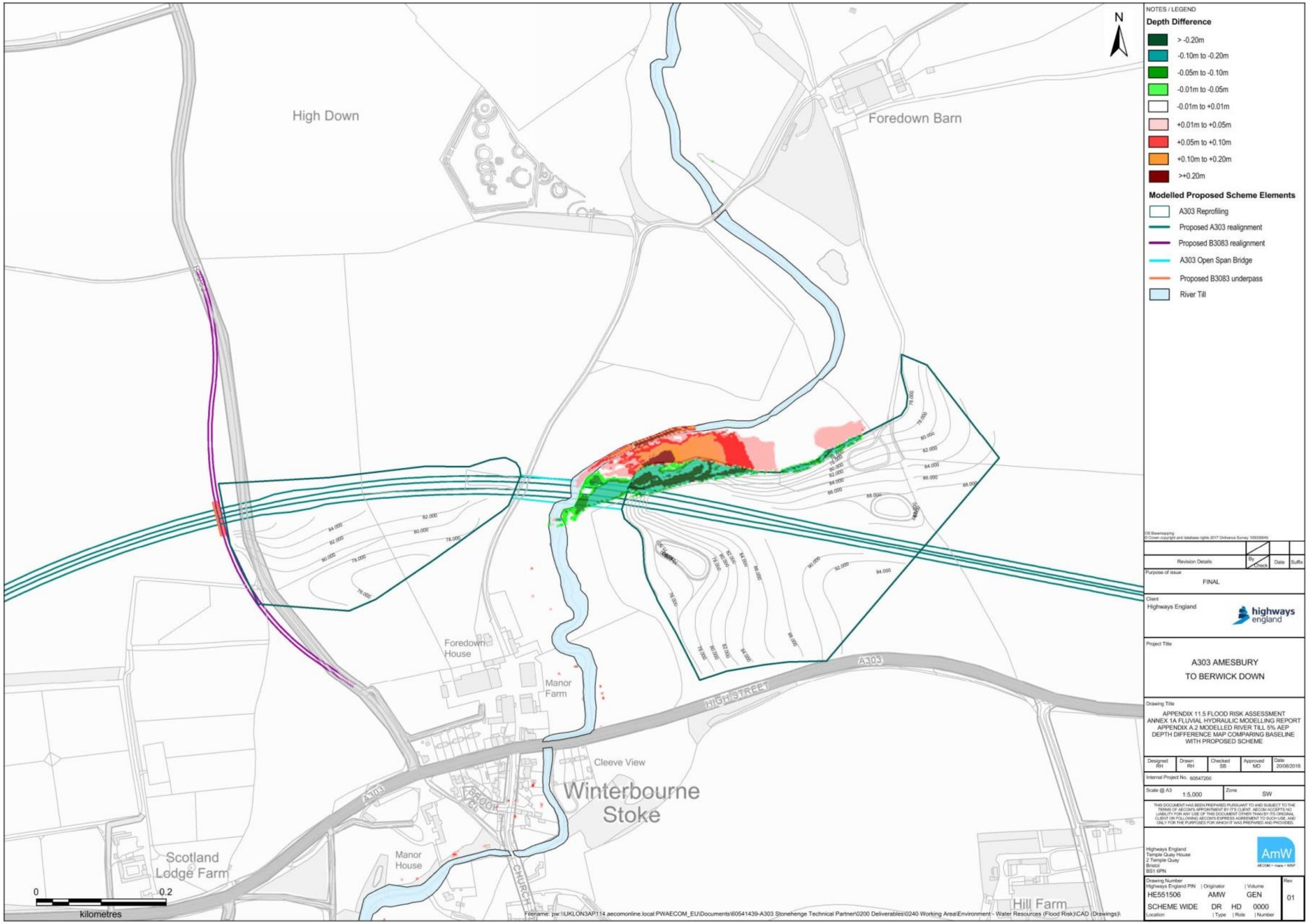
8.2.1 Hydraulic modelling of the River Avon has been completed for a range of design flood events and changes in flood risk attributable to the proposed scheme assessed for a range of flood conditions.

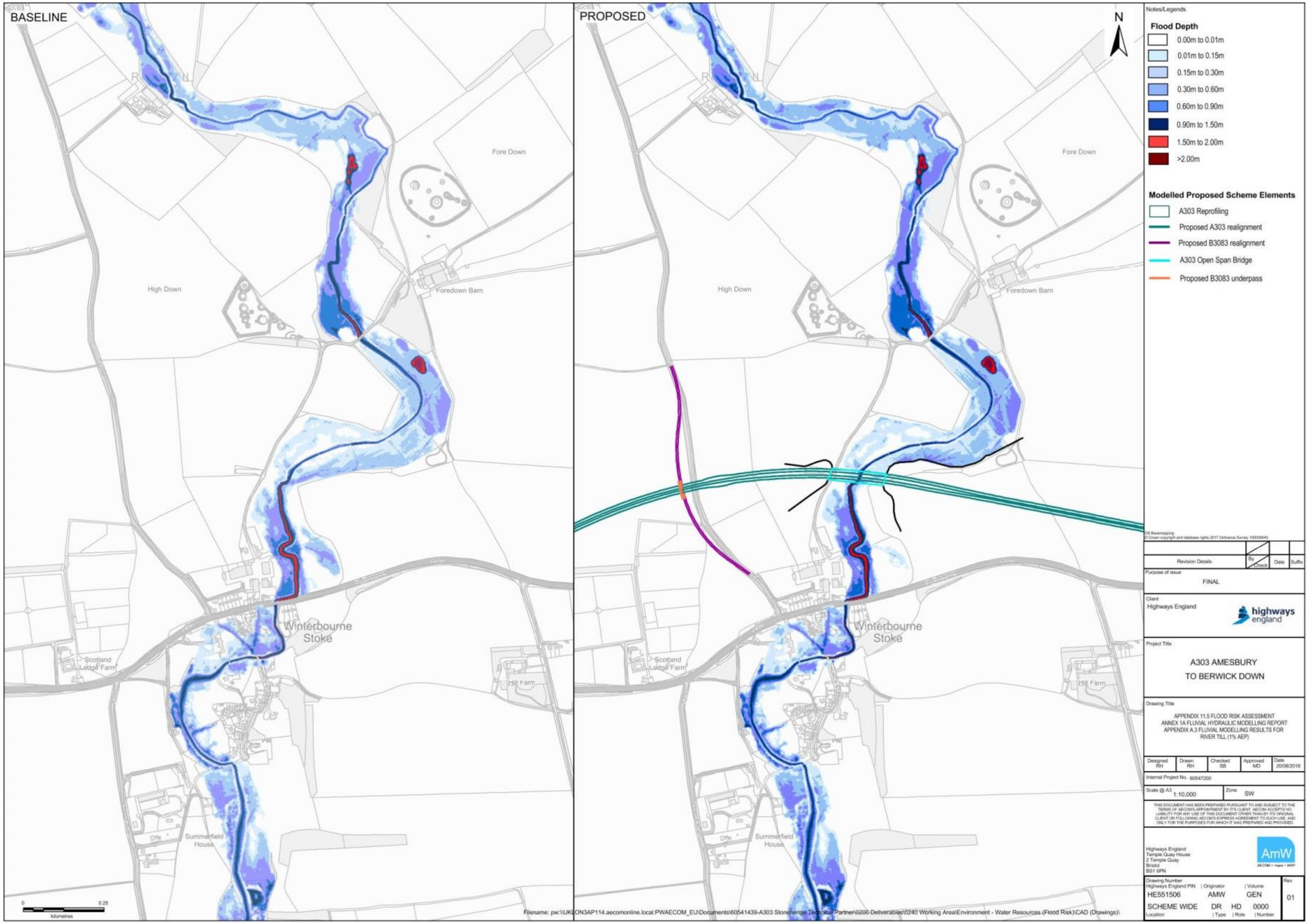
8.2.2 Within the proposed scenario, small decreases in flood depth are observed within the footprint of road drainage ponds for the 1% AEP + 40% climate change design event. As these drainage ponds reduce storage within the floodplain, there are small localised increases in flood depth, between 0.01m and 0.025m, upon the Avon floodplain. However these increases in depth do not intersect any vulnerable elements of the scheme or residential/commercial properties, whilst there is no discernible change in the flood extent.

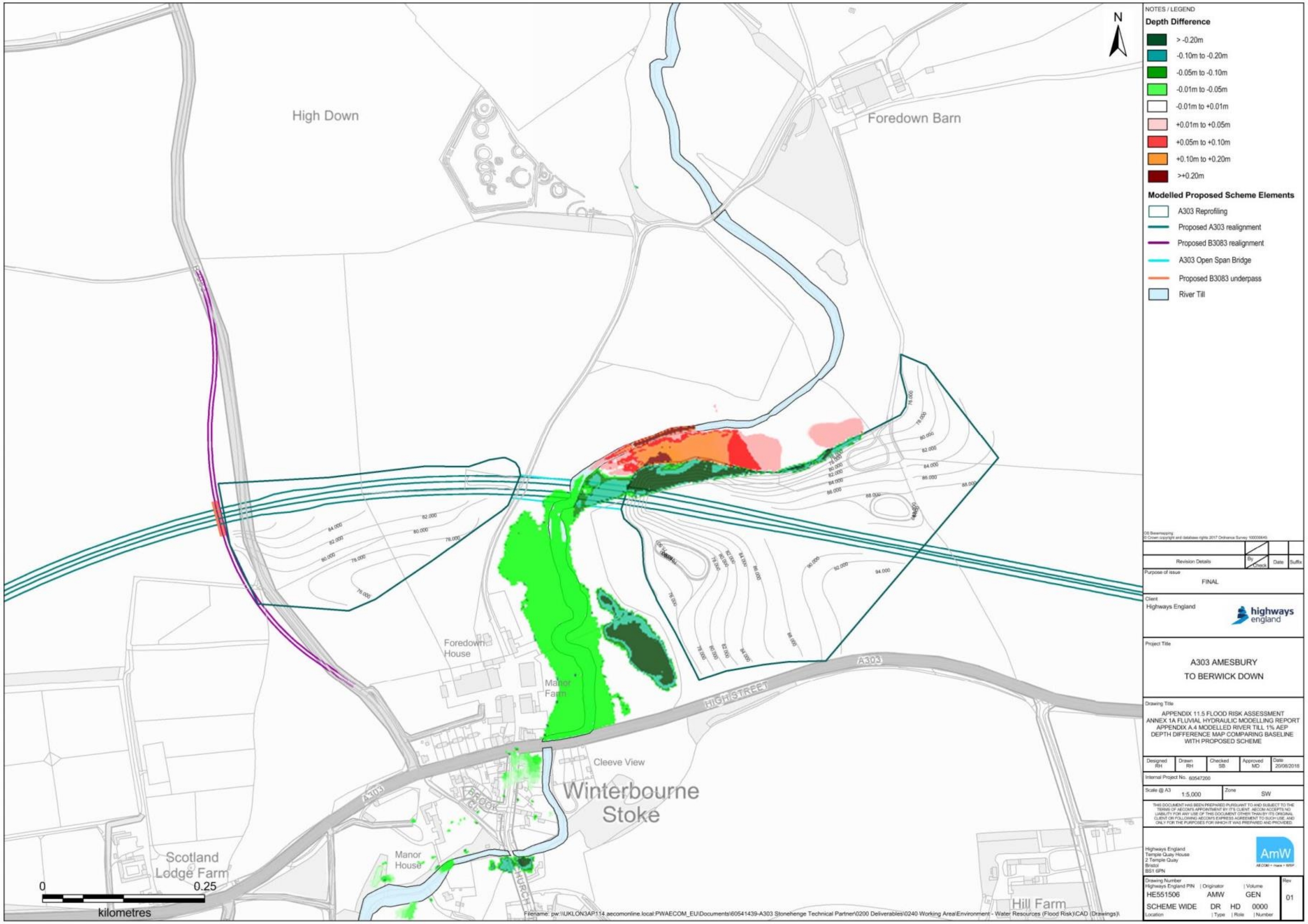
8.2.3 Within the 0.1% AEP event, along with the other climate change sensitivity simulation (+85% uplift in peak flow) there are also some localised increases in maximum flood depth, although these do not intersect any vulnerable elements of the scheme.

9 Appendix A- River Till Flood Mapping

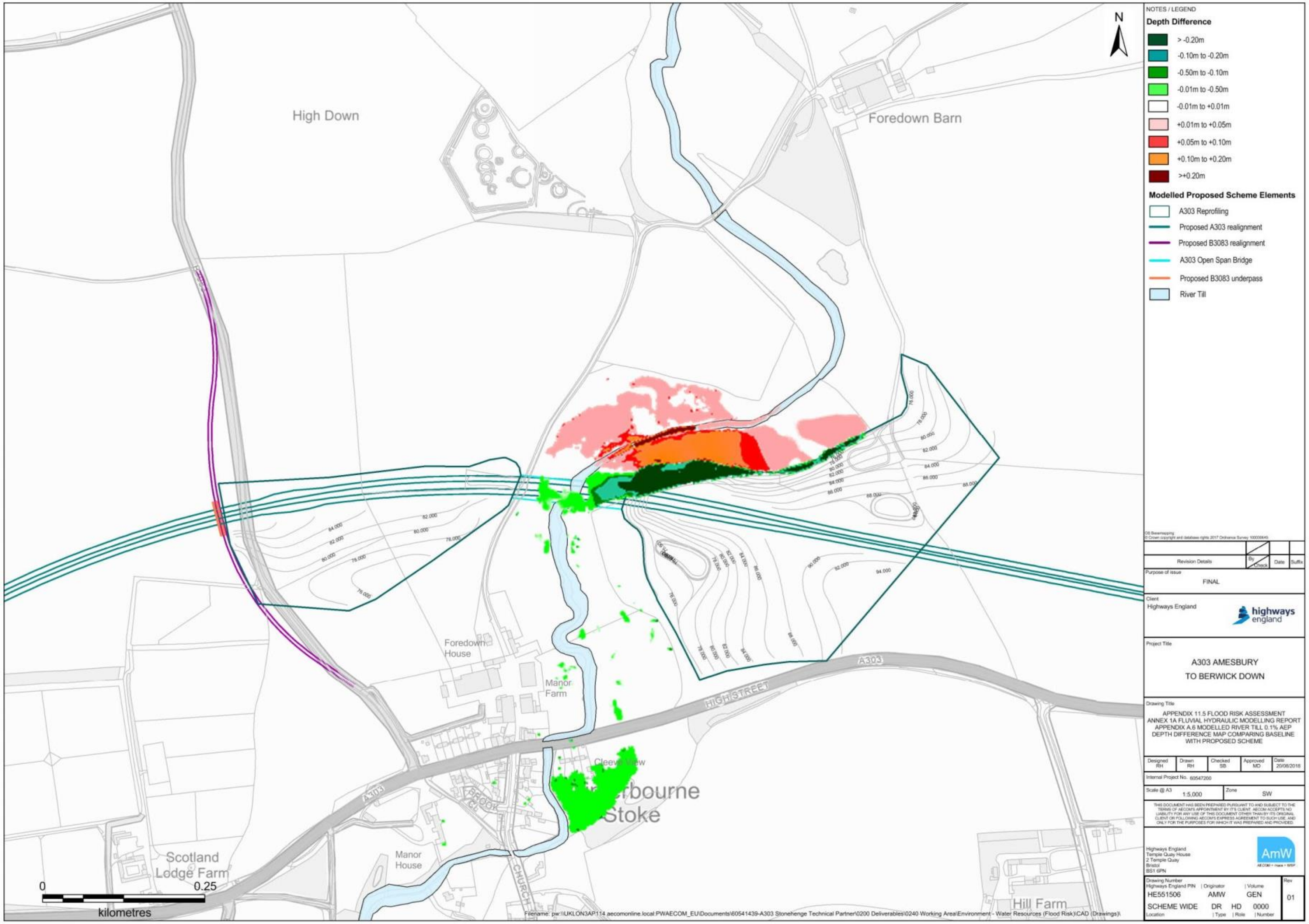

BASELINE
PROPOSED
N
Fore Down
High Down
Foredown Barn
Winterbourne
Stoke
Scotland
Lodge Farm
Hill Farm
Summerfield
House
0
0.2
kilometres
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
OS Basemapping
© Crown copyright and database rights 2017 Ordnance Survey 100030649
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.1 FLUVIAL MODELLING RESULTS FOR
RIVER TILL (5% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/06/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM • mace • WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01


N
High Down
Foredown Barn
Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Manor House
Scotland Lodge Farm
Hill Farm
HIGH STREET
A303
0
0.2
kilometres
NOTES / LEGEND
Depth Difference
> -0.20m
-0.10m to -0.20m
-0.05m to -0.10m
-0.01m to -0.05m
-0.01m to +0.01m
+0.01m to +0.05m
+0.05m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
River Till
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.2 MODELLED RIVER TILL 5% AEP
DEPTH DIFFERENCE MAP COMPARING BASELINE
WITH PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01

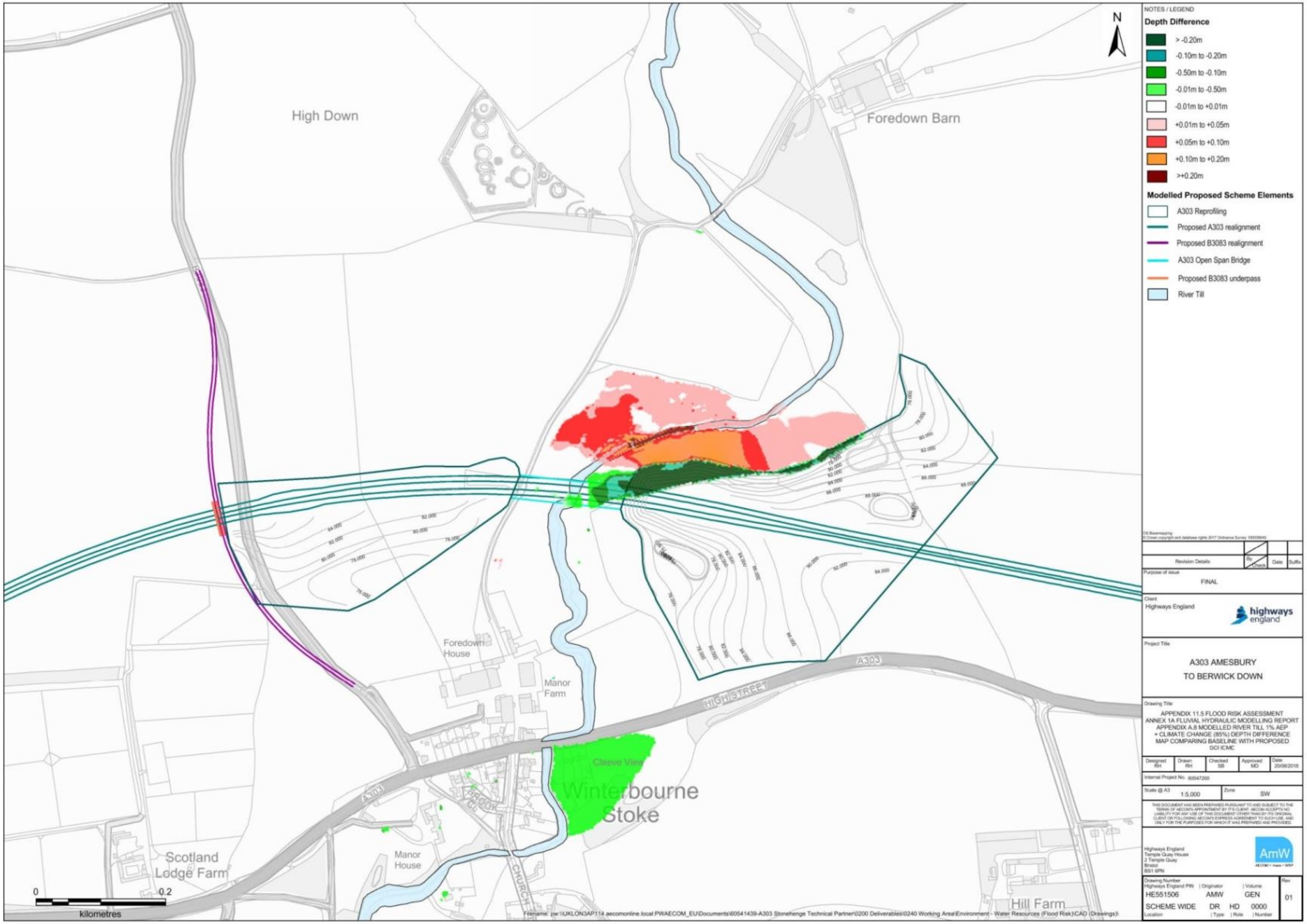

BASELINE
PROPOSED
N
High Down
Fore Down
Foredown Barn
Winterbourne
Stoke
Scotland
Lodge Farm
Summerfield
House
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.3 FLUVIAL MODELLING RESULTS FOR
RIVER TILL (1% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01


N
High Down
Foredown Barn
Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Scotland Lodge Farm
Manor House
Hill Farm
HIGH STREET
A303
0
0.25
kilometres
NOTES / LEGEND
Depth Difference
> -0.20m
-0.10m to -0.20m
-0.05m to -0.10m
-0.01m to -0.05m
-0.01m to +0.01m
+0.01m to +0.05m
+0.05m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
River Till
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.4 MODELLED RIVER TILL 1% AEP
DEPTH DIFFERENCE MAP COMPARING BASELINE
WITH PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01


BASELINE
PROPOSED
N
High Down
Fore Down
Foredown Barn
Scotland Lodge Farm
Summerfield House
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.5 FLUVIAL MODELLING RESULTS FOR
RIVER TILL (0.1% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/06/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN
Originator
Volume
HE551506
AMW
GEN
SCHEME WIDE
DR
HD
0000
Location
Type
Role
Number
Rev
01

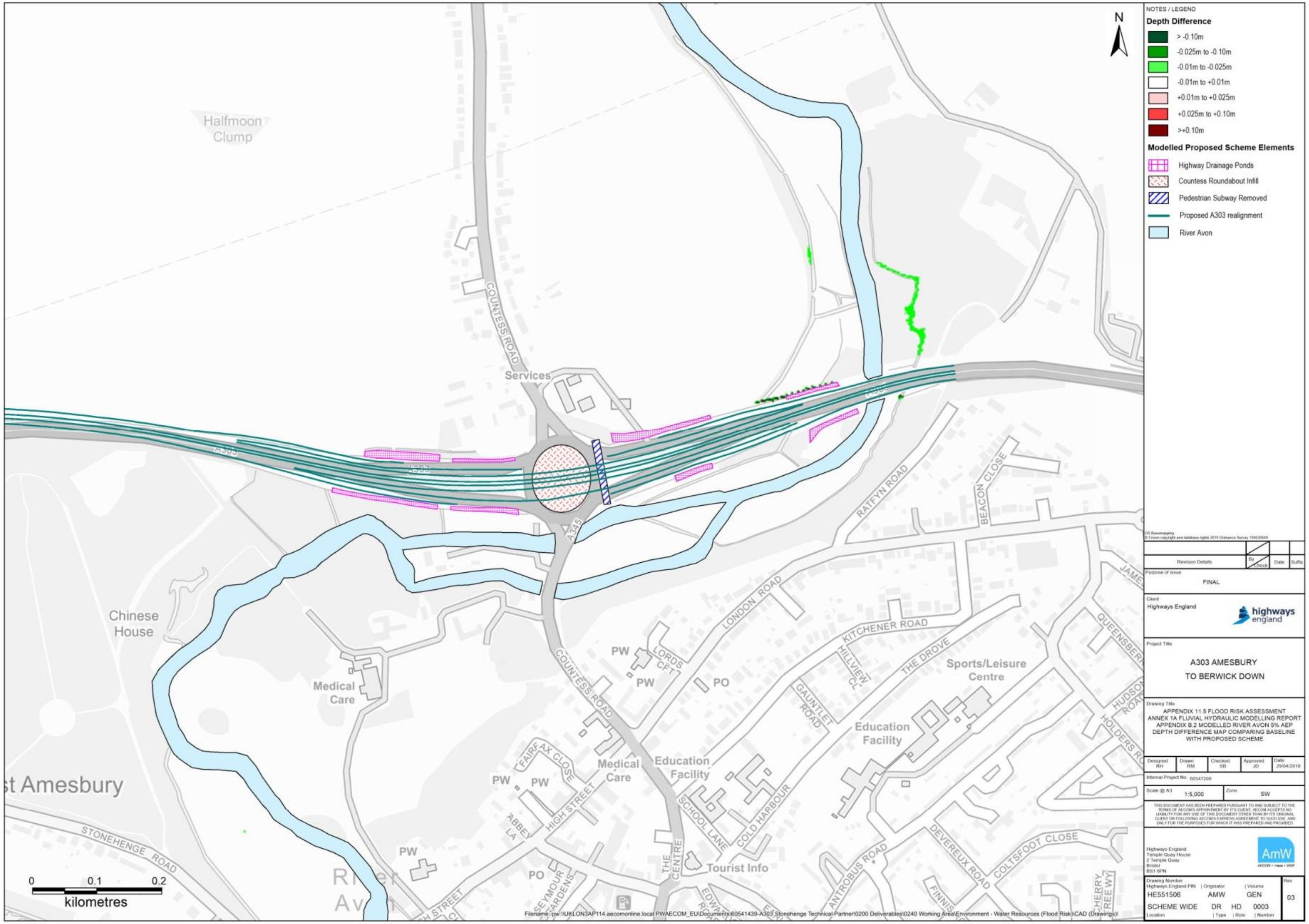

N
High Down
Foredown Barn
Foredown House
Manor Farm
Cleeve View
Stoke
Scotland Lodge Farm
Manor House
Hill Farm
HIGH STREET
A303
CHURCH
0
0.25
kilometres
NOTES / LEGEND
Depth Difference
> -0.20m
-0.10m to -0.20m
-0.50m to -0.10m
-0.01m to -0.50m
-0.01m to +0.01m
+0.01m to +0.05m
+0.05m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
River Till
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.6 MODELLED RIVER TILL 0.1% AEP
DEPTH DIFFERENCE MAP COMPARING BASELINE
WITH PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01

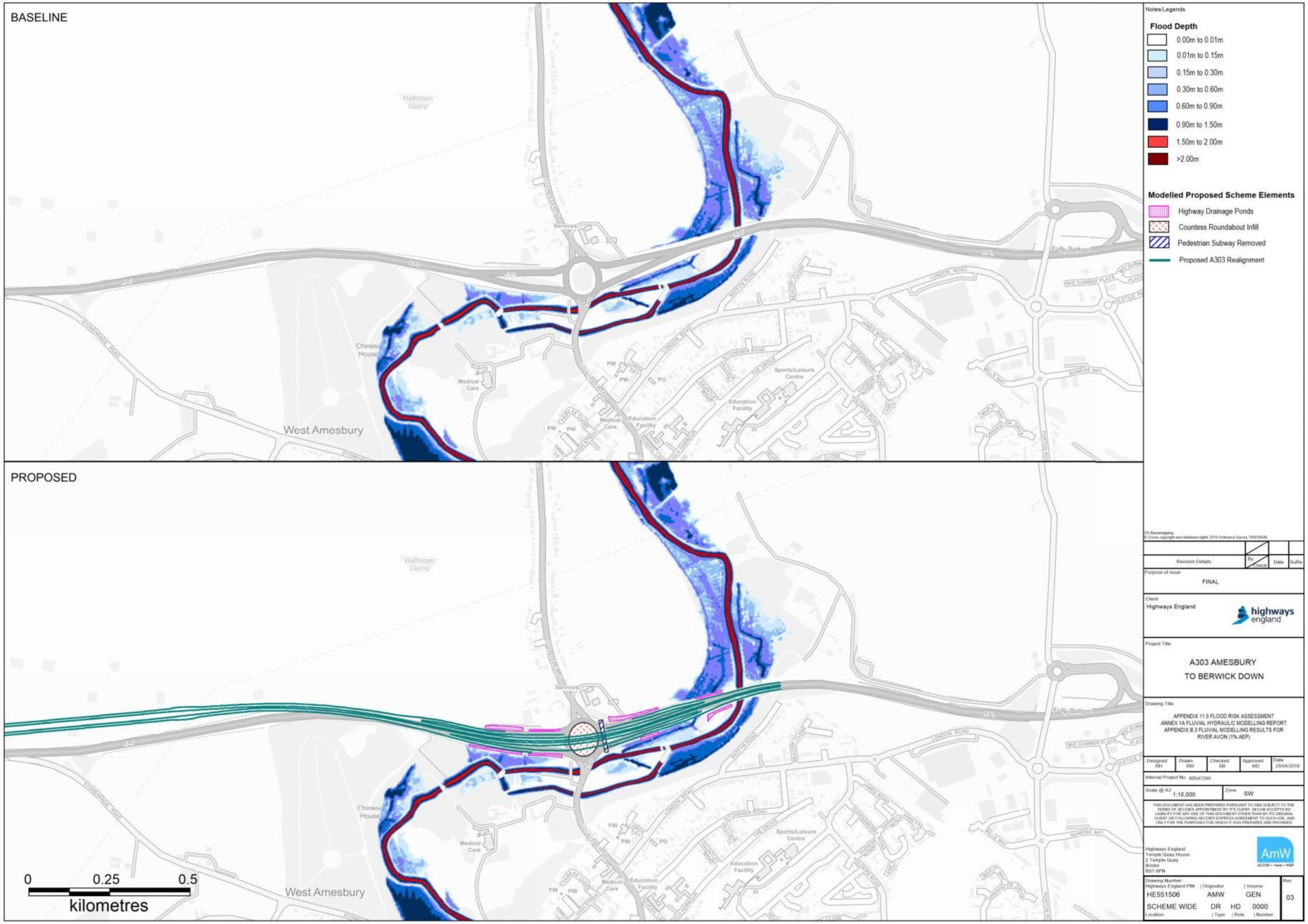

BASELINE
PROPOSED
N
Fore Down
High Down
Foredown Barn
Scotland Lodge Farm
Hill Farm
Summerfield House
kilometres
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
OS Basemapping
© Crown copyright and database rights 2017 Ordnance Survey 100030649
Revision Details
By Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.7 FLUVIAL MODELLING RESULTS FOR RIVER TILL (1% AEP + 85% CLIMATE CHANGE)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM • mace • WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01

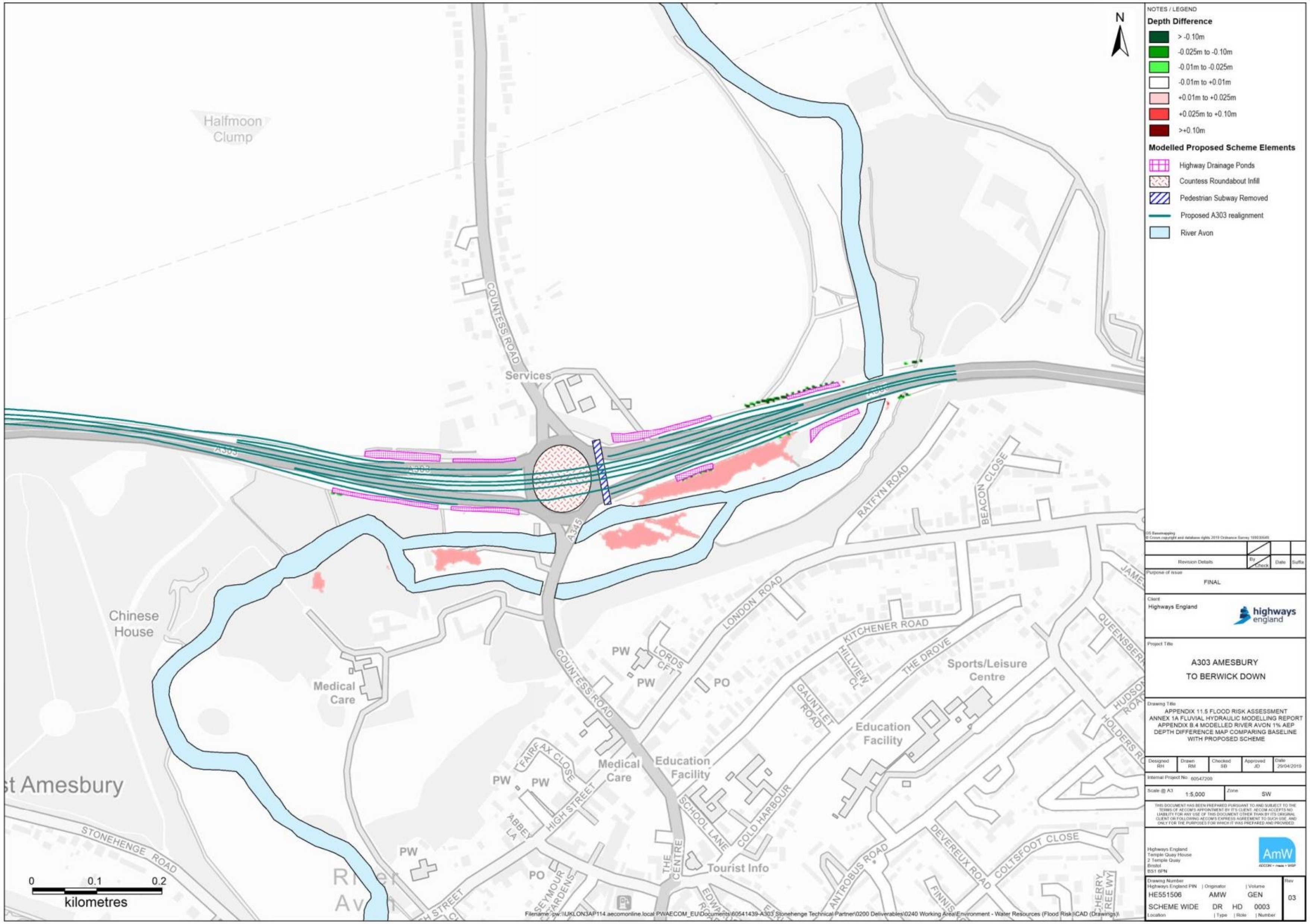

High Down
Foredown Barn
Foredown House
Manor Farm
HIGH STREET
A303
Winterbourne Stoke
Cleeve View
Scotland Lodge Farm
Manor House
CHURCH
Hill Farm
0
0.2
kilometres
N
NOTES / LEGEND
Depth Difference
> -0.20m
-0.10m to -0.20m
-0.50m to -0.10m
-0.01m to -0.50m
-0.01m to +0.01m
+0.01m to +0.05m
+0.05m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
A303 Reprofiling
Proposed A303 realignment
Proposed B3083 realignment
A303 Open Span Bridge
Proposed B3083 underpass
River Till
Revision Details
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.8 MODELLED RIVER TILL 1% AEP
+ CLIMATE CHANGE (85%) DEPTH DIFFERENCE
MAP COMPARING BASELINE WITH PROPOSED
DCI ICMC
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01

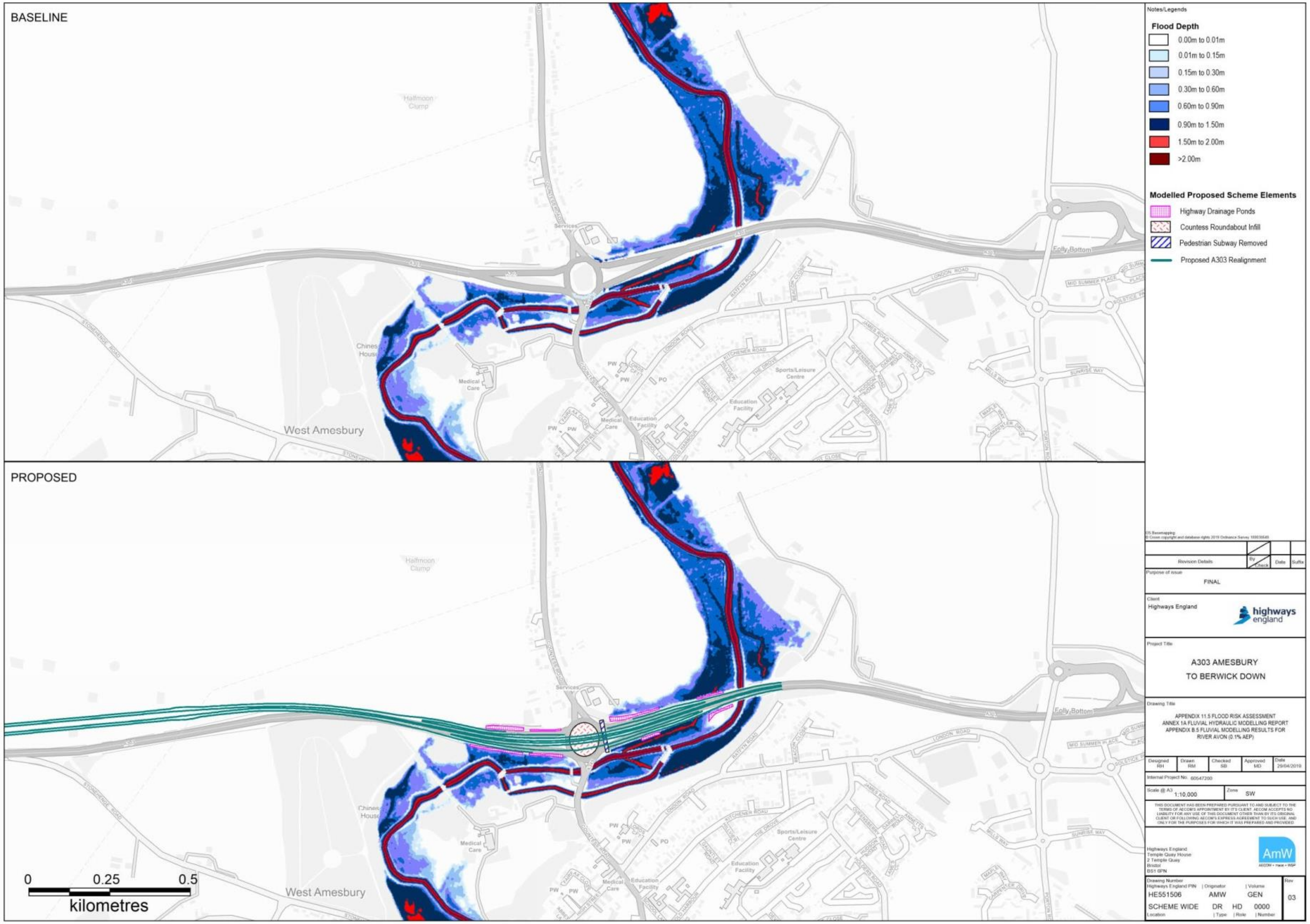
10 Appendix B- River Avon Flood Mapping

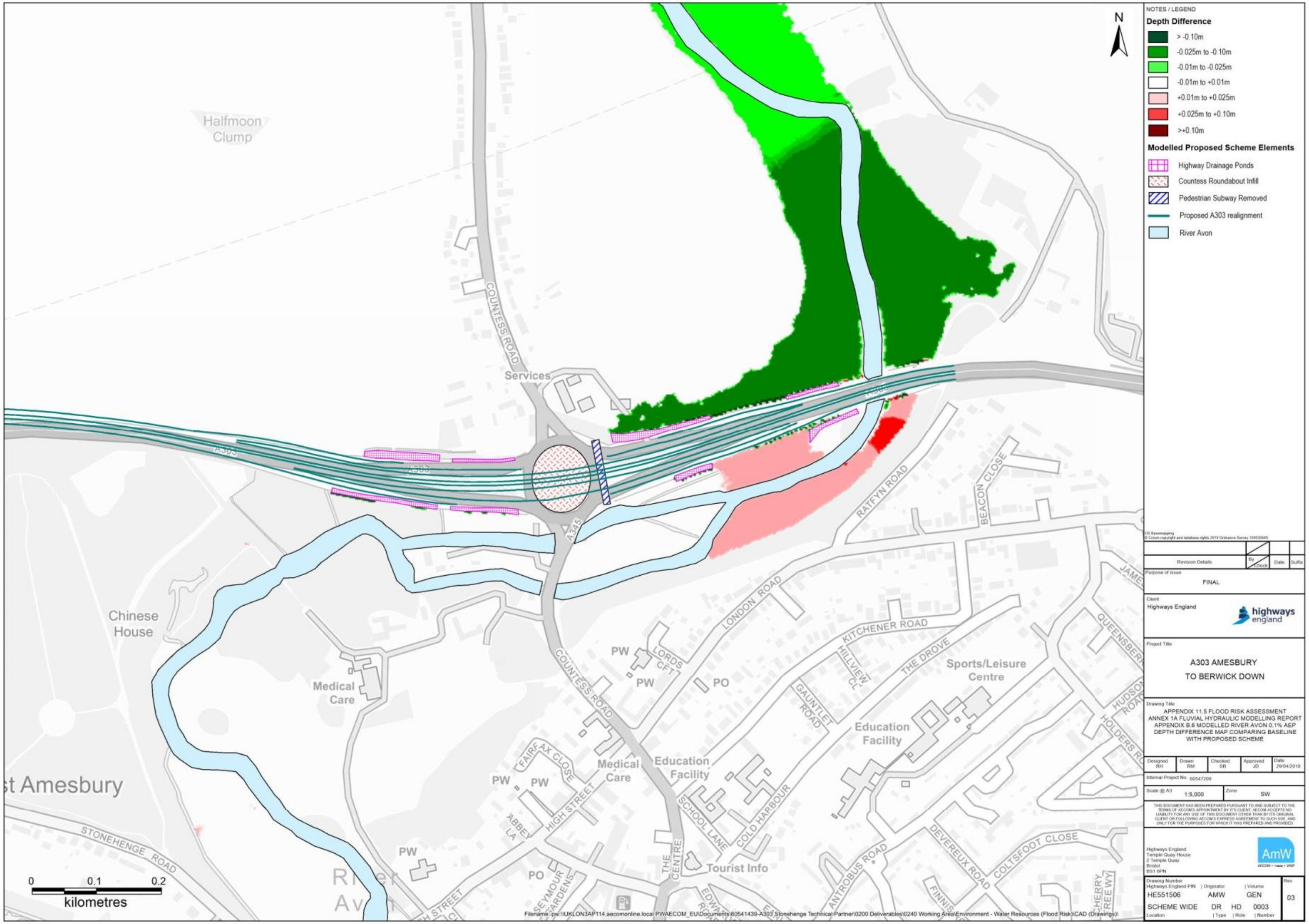

BASELINE
PROPOSED
Halfmoon Clump
Chinese House
Medical Care
West Amesbury
Services
Folly Bottom
London Road
PW
PO
Education Facility
Sports/Leisure Centre
0
0.25
0.5
kilometres
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 Realignment
Revision Details
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.1 FLUVIAL MODELLING RESULTS FOR
RIVER AVON (5% AEP)
Designed RH
Drawn RM
Checked SB
Approved MD
Date 29/04/2019
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
03

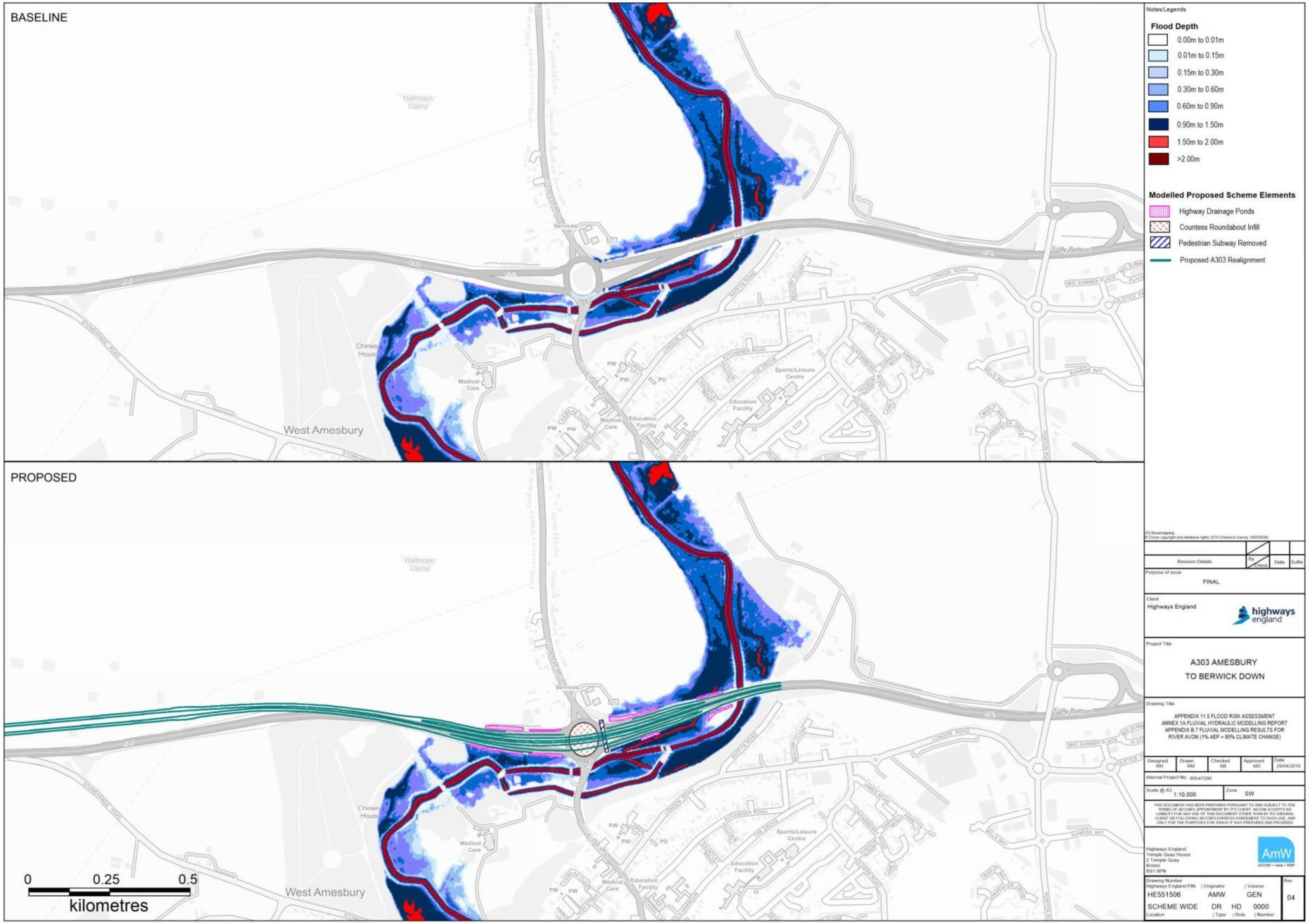

N
NOTES / LEGEND
Depth Difference
> -0.10m
-0.025m to -0.10m
-0.01m to -0.025m
-0.01m to +0.01m
+0.01m to +0.025m
+0.025m to +0.10m
>+0.10m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
Halfmoon Clump
Countess Road
Services
A303
A345
Chinese House
Medical Care
Education Facility
Sports/Leisure Centre
Tourist Info
PW
PO
t Amesbury
Stonehenge Road
London Road
Kitchener Road
The Drove
Ratfyn Road
Beacon Close
High Street
School Lane
Coltsfoot Close
Devereux Road
0 0.1 0.2
kilometres
OS Basemapping
Revision Details
By Check Date Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.2 MODELLED RIVER AVON 5% AEP
DEPTH DIFFERENCE MAP COMPARING BASELINE
WITH PROPOSED SCHEME
Designed RH
Drawn RM
Checked SB
Approved JD
Date 29/04/2019
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0003
Location | Type | Role | Number
Rev
03

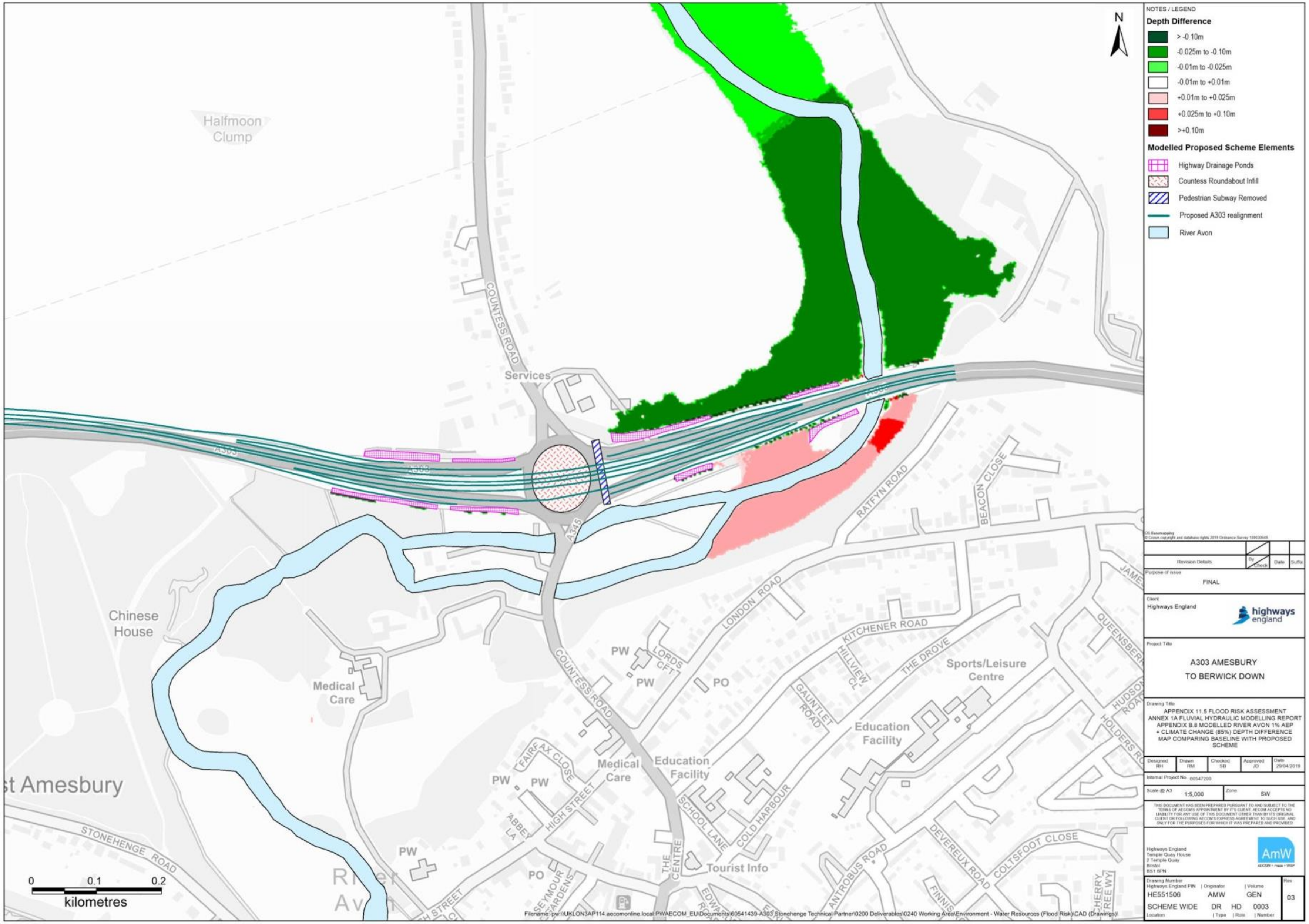

BASELINE
PROPOSED
West Amesbury
0
0.25
0.5
kilometres
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 Realignment
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.3 FLUVIAL MODELLING RESULTS FOR
RIVER AVON (1% AEP)
Designed RH
Drawn RM
Checked SB
Approved MD
Date 29/04/2019
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
03


N
NOTES / LEGEND
Depth Difference
> -0.10m
-0.025m to -0.10m
-0.01m to -0.025m
-0.01m to +0.01m
+0.01m to +0.025m
+0.025m to +0.10m
>+0.10m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
Halfmoon Clump
Chinese House
Medical Care
Services
COUNTESS ROAD
A303
A345
RATFYN ROAD
BEACON CLOSE
LONDON ROAD
KITCHENER ROAD
THE DROVE
Sports/Leisure Centre
Education Facility
PW
PO
Tourist Info
STONEHENGE ROAD
HIGH STREET
SCHOOL LANE
COLD HARBOUR
ANTROBUS ROAD
DEVEREUX ROAD
COLTSFOOT CLOSE
FAIRFAX CLOSE
0
0.1
0.2
kilometres
Revision Details
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.4 MODELLED RIVER AVON 1% AEP
DEPTH DIFFERENCE MAP COMPARING BASELINE
WITH PROPOSED SCHEME
Designed RH
Drawn RM
Checked SB
Approved JD
Date 29/04/2019
Internal Project No 60547200
Scale @ A3 1:5,000
Zone SW
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0003
Location | Type | Role | Number
Rev
03


BASELINE
PROPOSED
West Amesbury
Medical Care
Education Facility
Sports/Leisure Centre
Folly Bottom
0
0.25
0.5
kilometres
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 Realignment
Revision Details
By Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.5 FLUVIAL MODELLING RESULTS FOR
RIVER AVON (0.1% AEP)
Designed RH
Drawn RM
Checked SB
Approved MD
Date 29/04/2019
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
03


N
NOTES / LEGEND
Depth Difference
> -0.10m
-0.025m to -0.10m
-0.01m to -0.025m
-0.01m to +0.01m
+0.01m to +0.025m
+0.025m to +0.10m
>+0.10m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
Halfmoon Clump
Services
COUNTESS ROAD
A303
A345
RATFYN ROAD
BEACON CLOSE
LONDON ROAD
KITCHENER ROAD
THE DROVE
Sports/Leisure Centre
Education Facility
Medical Care
Chinese House
Amesbury
STONEHENGE ROAD
PW
PO
Tourist Info
0
0.1
0.2
kilometres
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.6 MODELLED RIVER AVON 0.1% AEP
DEPTH DIFFERENCE MAP COMPARING BASELINE
WITH PROPOSED SCHEME
Designed RH
Drawn RM
Checked SB
Approved JD
Date 29/04/2019
Internal Project No 60547200
Scale @ A3 1:5,000
Zone SW
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
HE551506 AMW GEN
SCHEME WIDE DR HD 0003
Rev 03


BASELINE
PROPOSED
West Amesbury
0
0.25
0.5
kilometres
Notes/Legends
Flood Depth
0.00m to 0.01m
0.01m to 0.15m
0.15m to 0.30m
0.30m to 0.60m
0.60m to 0.90m
0.90m to 1.50m
1.50m to 2.00m
>2.00m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 Realignment
OS Basemapping:
© Crown copyright and database rights 2019 Ordnance Survey 100030649
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.7 FLUVIAL MODELLING RESULTS FOR
RIVER AVON (1% AEP + 85% CLIMATE CHANGE)
Designed RH
Drawn RM
Checked SB
Approved MD
Date 29/04/2019
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
04


N
NOTES / LEGEND
Depth Difference
> -0.10m
-0.025m to -0.10m
-0.01m to -0.025m
-0.01m to +0.01m
+0.01m to +0.025m
+0.025m to +0.10m
>+0.10m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
Halfmoon Clump
Services
Countess Road
A303
A345
Chinese House
Medical Care
Education Facility
Sports/Leisure Centre
Tourist Info
London Road
Kitchener Road
The Drove
Ratfyn Road
Beacon Close
Stonehenge Road
High Street
School Lane
Coltsfoot Close
Devereux Road
Antrobus Road
0 0.1 0.2
kilometres
Revision Details
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1A FLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.8 MODELLED RIVER AVON 1% AEP
+ CLIMATE CHANGE (85%) DEPTH DIFFERENCE
MAP COMPARING BASELINE WITH PROPOSED SCHEME
Designed RH
Drawn RM
Checked SB
Approved JD
Date 29/04/2019
Internal Project No 60547200
Scale @ A3 1:5,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
HE551506 AMW GEN
SCHEME WIDE DR HD 0003
Rev 03

11 Appendix C- River Avon CSM Sensitivity Testing

11.1 Overview

11.1.1 The Environment Agency have provided a hydrograph for the River Avon, generated from Continuous Simulation Modelling (CSM) currently being undertaken for Salisbury. The CSM hydrograph supplied has been extracted at the location of the Amesbury gauge (Station ID 43005) and has a duration of 800 hours, which is significantly longer than the current simulation length of 60 hours for the modelled design events. The CSM hydrograph also has multiple peaks present throughout the duration of the storm event as opposed to the single peak used in standard design simulations.

11.1.2 The CSM hydrograph has been used to undertake a sensitivity test for the River Avon hydraulic model, in order to support the assessment of flood risk and the design simulations already completed. The aim of this sensitivity test is to evaluate whether the duration of the event has an impact on the model results where the flow volumes are significant, as well as the peak flow. Permeable chalk catchments can be associated with river flows which remain high for extended durations, and thus such a sensitivity test will be useful in the context of the studied reach of the River Avon.

11.1.3 It should be noted that as the supplied CSM hydrograph has been extracted from the Amesbury gauge, which does not coincide with the upstream boundary of the River Avon hydraulic model, therefore the hydrograph has been modified to facilitate inclusion within the model. The sensitivity test has been completed for the baseline scenario only.

11.2 Methodology

11.2.1 The CSM sensitivity test was simulated using the same baseline model set up as the design runs, with the only change being the inflow hydrographs. In order to facilitate inclusion of the CSM hydrograph at the Amesbury gauge into the River Avon hydraulic model a number of steps have been completed.

11.2.2 The hydrological analysis undertaken for the River Avon demonstrates that peak flow increases from 25.9m^3/s at the upstream model boundary (AVON01) to 32.5m^3/s at the downstream boundary (AVON04), an increase of 6.7m^3/s or equivalent to <20% of the peak model inflow. Furthermore, an analysis of modelled hydrographs at the upstream and downstream boundaries of the model suggest that the lag time from the peak flow upstream, to the peak flow downstream is approximately 11.5 hours. The Amesbury gauge is located between the AVON03 and AVON04 estimation points, detailed within the hydrological assessment[5].

11.2.3 The above analysis suggests that the CSM hydrograph supplied at Amesbury should not be simply used as an inflow to the model at the upstream boundary. To account for the propagation of flooding through the modelled reach peak flows calculated within the hydrological analysis, along with results from the 1% AEP baseline model simulation were utilised in order to adjust the CSM hydrograph for

[5] AmW (2018) Appendix 11.5 Annex 2B River Avon Hydrological Analysis.

application at each of the flow estimation points, taking into account both peak flow and lag time.

11.2.4 To adjust peak flow, the ratio of peak flow between the Amesbury gauge and each of the flow estimation points was calculated based upon the FEH peaks calculated within the hydrological analysis for the River Avon. These ratios were then applied to the CSM hydrograph in order to create an adjusted CSM hydrograph for each flow estimation point. These ratios and flows are shown within Table C.1.

11.2.5 To account for the lag time through the modelled reach, the timing of the peak flows at the flow estimation points was extracted from the model and compared to the timing of peak flow at the Amesbury gauge (Table C-1). The calculated lag times to each flow estimation point were then used to adjust the timing of the CSM hydrograph at each flow estimation point.

Table C.1: River Avon Peak Inflows

AEP Event	Peak Inflows (m^3/s)				
	AVON01	AVON02	AVON03	Amesbury Gauge	AVON04
1%	25.9	27.2	29.7	30.3	32.5
1% CSM	34.5	36.3	39.6	40.4	43.4
1% + 40CC	36.3	38.1	41.6	42.4	45.5
Peak flow ratio*	0.85	0.90	0.98	-	1.07
Lag Time (Hours)*	-7.5	-5.0	-2.0	0	4.0

* Ratio/lag calculated between flow estimation point and Amesbury gauge

11.2.6 Completion of these two adjustments created an ‘adjusted’ CSM hydrograph for each flow estimation point which could then be applied within the hydraulic model. These hydrographs are shown in Figure C.1.

11.2.7 Peak flows estimated from FEH are compared to the adjusted CSM peak flows at the flow estimation points, as well as the Amesbury gauge in Table C.1. It should be noted that the CSM peak flow is substantially higher than peak flow estimated within FEH, and as a result are more comparable to the peak flows from the 1% AEP plus higher central climate change (40%). As a result, the 1% AEP CSM results will also be compared to the 1% AEP plus higher central climate change results to add additional context to the sensitivity analysis.

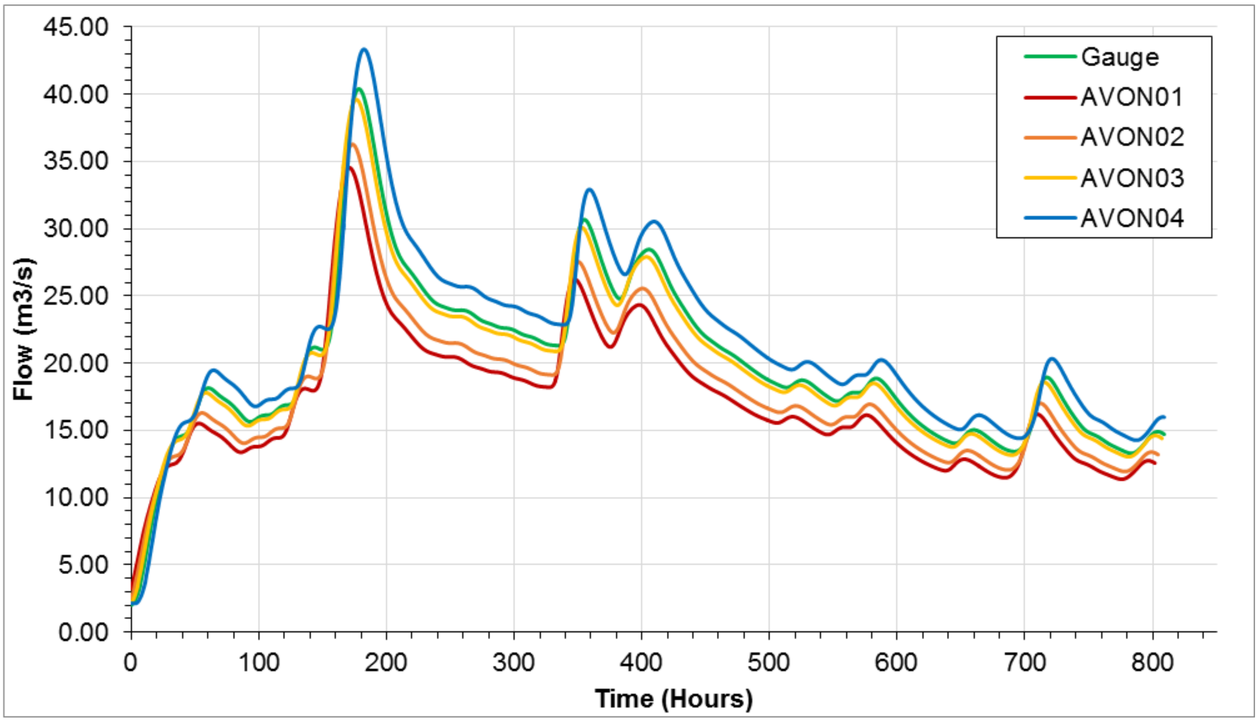


Figure C.1: Adjusted CSM Hydrograph for Each Flow Estimation Point

11.3 Results

11.3.1 The maximum modelled flood depth from the CSM sensitivity simulation can be seen in Figure C.2. This section details comparisons to the baseline design results from the 1% AEP and 1% AEP plus climate change simulations. A visual comparison of maximum flood depth from the CSM simulation shows a noticeable increase in depth in comparison to the 1% AEP design event, although the CSM and 1% AEP + 40% climate change event appear visually similar. Maximum modelled flood levels from the River Avon (upstream and downstream of the proposed A303 alignment) are shown in Table C.2.

11.3.2 Table C.2 generally shows that there is an increase in the 1D peak in-channel water levels for the CSM inflows, when compared with the 1% AEP event results, with the largest increase in water levels being approximately 0.5m.The location of the nodes that the water levels are extracted from is shown by Figure 6.4. A comparison of peak modelled water levels within the channel between the 1% AEP CSM simulation and the 1% AEP plus climate change design simulation suggest that the levels are virtually identical. Based upon peak modelled water levels within the channel it is thought that the higher peak levels within the 1% AEP CSM simulation can be attributed to the higher peak flow in comparison to the 1% AEP design event, rather than the longer duration and multiple peaks present within the CSM simulation.

Table C.2: River Avon Peak Modelled Stage

1D Node	Description	CSM 1% AEP (m AOD)	Baseline 1% AEP (m AOD)	Baseline 1% AEP Difference (m)	Baseline (1% 40CC AEP) (m AOD)	Baseline (1% 40CC AEP) Difference (m)
RA_XS_36	Channel	70.44	70.18	0.26	70.44	0.00
RA_XS_35	Channel	70.39	70.12	0.27	70.39	0.00
RA_ST_19CU	A303 Bridge	70.24	70.03	0.21	70.24	0.00
RA_XS_34	Channel	70.05	69.89	0.16	70.05	0.00
RA_ST_18d	Channel	69.98	69.48	0.50	69.98	0.00
RA_XS_33	Channel	69.93	69.4	0.53	69.93	0.00
RA_ST_52CUA	A345 Bridge	69.89	69.39	0.50	69.89	0.00
RA_ST_50	Access Bridge	69.82	69.32	0.50	69.82	0.00
RA_XS_31	Channel	69.07	68.8	0.27	69.07	0.00
RA_XS_30u	Channel	69.07	68.8	0.27	69.07	0.00
RA_ST_15	Channel	68.87	68.66	0.21	68.87	0.00
RA_ST_17u	2nd Channel	70.03	69.85	0.18	70.03	0.00
RA_ST_17CU	A345 Bridge	69.86	69.69	0.17	69.86	0.00
RA_ST_16d	2nd Channel	69.08	68.86	0.22	69.08	0.00

11.3.4 A depth difference plot was produced to illustrate the change in flood depths and extents between the 1% AEP design event and the CSM simulation using the 2D results (Figure C.3). There is a general increase in peak flood depth between 0.1m and 0.3m across the floodplain within results from the CSM simulation. In the area of interest, there is a larger increase of approximately 0.6m in the flood depths behind the node RA_ST_16d. In the results of the 1% AEP design event, there is a general flood depth of approximately 0.15m between the River Avon and A303, as demonstrated by Appendix B.3. Based upon this figure alone it is unclear whether the increase in modelled depths can be attributed solely to the increase in peak flow, or whether the extended duration also contributes towards the depth increase shown.

11.3.5 In order to further evaluate whether the duration of the model simulation has an impact on the model results, a comparison was made between the water levels of the 1% AEP CSM simulation and the 1% AEP + 40% climate change event results. A depth difference plot of these results has been produced and is shown in Figure C.4. This plot demonstrates that when the peak inflows into the simulation are comparable, the resulting modelled water depths are not significantly different. This is also confirmed with the 1D peak in-channel water levels shown in Table C.2.

11.3.6 Therefore overall, based upon the sensitivity testing conducted here it can be concluded that the increase in maximum flood depths and levels along the modelled reach of the River Avon shown within the CSM sensitivity simulation can be primarily attributed to the increase in peak flow compared to the 1% AEP design event, rather than the longer duration and multiple peaks of the CSM hydrograph. This conclusion demonstrates that design event modelling for the River Avon is robust, and further strengthens the conclusions made within the hydraulic modelling report and FRA.

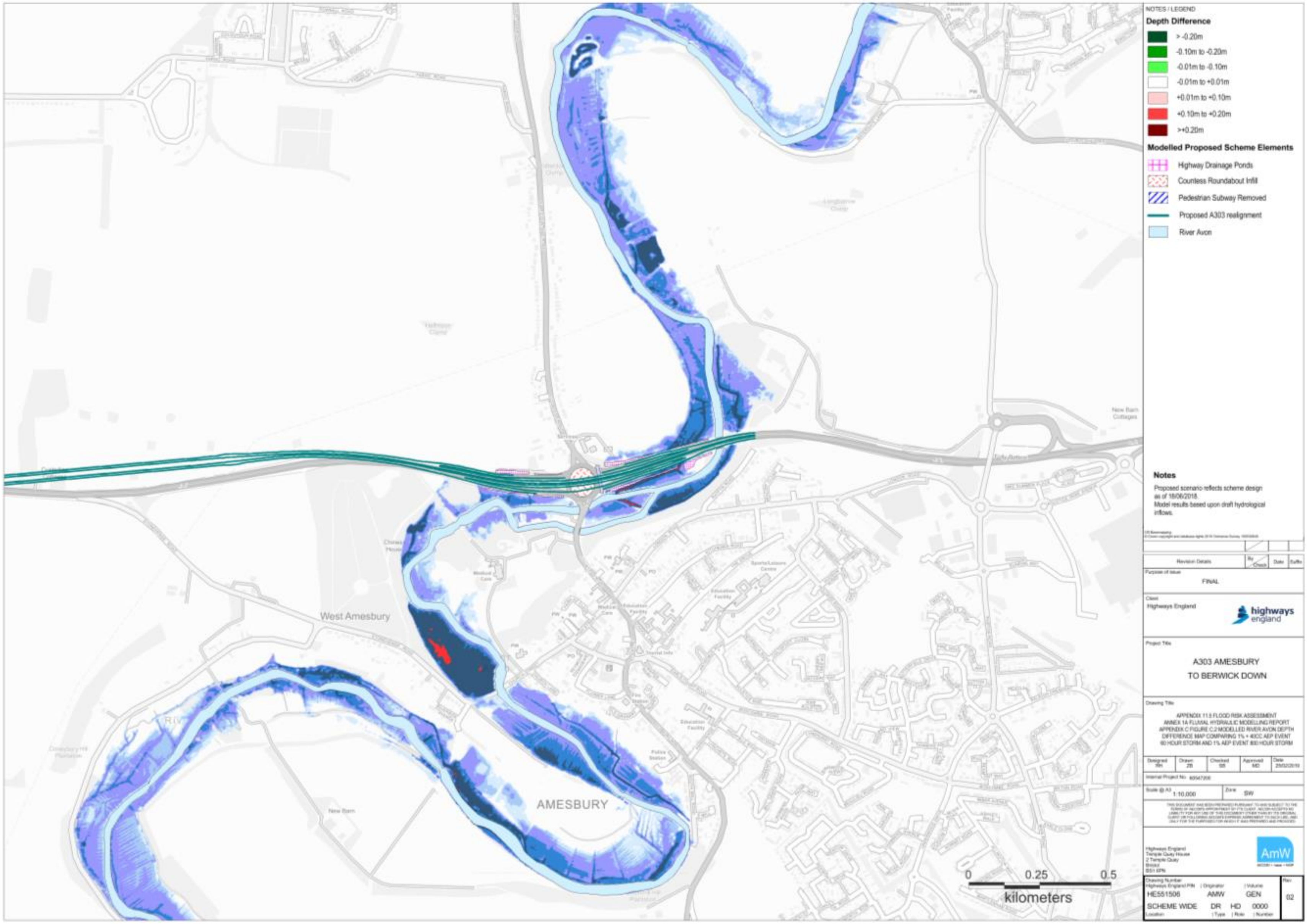


Figure C.2: River Avon CSM Sensitivity Flood Depth Map

Figure C.3: Depth Difference Map Comparing 1% AEP Event 60 Hour and 800 Hour Storm

Figure C.4: Depth Difference Map Comparing 1% AEP + 40% Climate Change 60 Hour Storm and 1% AEP 800 Hour Storm

A303 Amesbury to Berwick Down

TR010025

Volume 6

6.3 Environmental Statement Appendices

Appendix 11.5 Level 3 Flood Risk Assessment

Annex 1B – Pluvial Hydraulic Modelling Report

APFP Regulation 5(2)(a)

Planning Act 2008

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

May 2019

Table of contents

1 Introduction

1.1 Background

1.1.1 Initial examination of the Environment Agency (EA) surface water flood risk map revealed that there was one area where a substantial surface water flow pathway, within the 'dry valley' at Parsonage Down, interacted with significant changes to the landscape as part of the proposed scheme.

1.1.2 In order to robustly assess the impact of the proposed scheme upon surface water flood risk, and to provide quantitative information to inform the Flood Risk Assessment (FRA), pluvial hydraulic modelling was undertaken for the Parsonage Down catchment. This Pluvial Hydraulic Modelling Report has been produced in order to document the technical work undertaken in support of the FRA.

1.1.3 The surface water hydraulic modelling undertaken and documented within this report utilises aspects of the fluvial hydraulic modelling undertaken for the River Till, along with the hydrological analysis completed for the River Till. These aspects are documented within Annex 1 Part A and Annex 2 respectively, and the reader is referred to these reports for full details of the hydrological analysis and hydraulic modelling undertaken.

1.1.4 The first version of this report documented the methodology and results obtained from the surface water hydraulic modelling undertaken, culminating on 19th October 2018 when it was submitted to PINS as part of the Environmental Statement.

1.1.5 The current version of this report (v2) contains updates to the methodology and results from additional surface water hydraulic modelling undertaken between January 2019 and May 2019. The additional modelling resulted from discussions with Wiltshire Council that have taken place since submission of the original Environmental Statement, to reflect their comments on the initial methodology and results. The main updates in this second version are summarised as:

- Additional surface water hydrological calculations and model simulations to demonstrate initial moisture conditions and critical storm duration for design runs.
- Change in representation of the boundary with the River Till within the hydraulic model.
- Modelling of revised drainage solution at Parsonage Down, including changes to infiltration rates due to deposition of chalk tunnel arisings within this area.
- Additional sensitivity testing, including blockage analysis for the culvert beneath the A303.

1.1.6 The reader is referred to the FRA document (of which this report forms an annex to) for further context and information relating to the proposed scheme.

1.2 Objectives

1.2.1 In order to provide an appropriate assessment of surface water flood risk within the Parsonage Down catchment, in the context of the proposed scheme, the following objectives have been completed;

1. To assess surface water flood risk to the proposed scheme areas in order to determine flood risk within the existing (baseline) scenario;
2. To assess surface water flood risk as a result of the proposed scheme, including the new course of the A303, realignment of the B3083, reprofiling/landscaping at Parsonage Down and installation of a surface water drainage arrangement.

1.2.2 It was not considered necessary to include a proposed scenario model for the temporary phase of works. Works associated with the temporary phase did not intersect with surface water flow pathways within the EA Flood Map from Surface Water (FMfSW), and hence the assessment of the temporary scenario within the fluvial modelling was therefore considered sufficient for the purposes of the FRA.

1.3 Design Simulations and Climate Change

1.3.1 To meet the objectives outlined in Section 1.2 in compliance with relevant planning policy[1] surface water modelling for Parsonage Down has been undertaken for the baseline and proposed scenario for design rainfall events with the following Annual Exceedance Probability (AEP) scenarios; 3.33% AEP, 1% AEP and 0.1% AEP. This corresponds to the AEPs displayed within the EA FMfSW.

1.3.2 In line with Environment Agency (EA) guidance[2] the 1% AEP design rainfall event including an allowance for climate change (+40% increase in peak rainfall intensity) has also been simulated for the baseline and proposed scheme scenarios.

1.4 Report Structure

1.4.1 Section 2 details the approach adopted for the estimation of rainfall hydrology and losses, which form the primary input into the surface water hydraulic model. This was distinct from the fluvial hydrology estimated and documented within Annex 2 of the FRA.

1.4.2 The hydraulic modelling methodology employed is outlined within Section 3, including details of the representation of the proposed scenario.

1.4.3 Section 4 presents results from the baseline and proposed scenario modelling, whilst also documenting changes in surface water flood risk as a result of the proposed scheme.

1.4.4 Section 5 presents sensitivity analysis undertaken to evaluate the hydraulic model.

[1] HM Government (2018) Revised National Planning Policy Framework
[2] Environment Agency (2016) Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities.

1.4.5 Section 6 documents the limitations associated with the work presented, and Section 7 provides a summary of the overall conclusions drawn based upon the surface water hydraulic modelling at Parsonage Down.

2 Rainfall Hydrology and Losses Approach

2.1 Representation of Hydrology with the Hydraulic Model

2.1.1 The 2D TUFLOW hydraulic modelling software package was utilised for the surface water modelling undertaken, a further description of the modelling software and methodology is provided in Section 3. However, for the purposes of outlining the approach to rainfall hydrology and losses, it should be noted that TUFLOW does not contain a sophisticated hydrological model. This is reflected in the approach adopted.

2.1.2 TUFLOW enables a representation of direct rainfall onto the model grid, whilst a basic representation of losses to infiltration and evapotranspiration can be attained through specification of an initial loss (mm) and continuing loss rates (mm/hr). There is some scope to represent spatial variation in rainfall and losses, although importantly TUFLOW is unable to represent any further interaction between water on the surface and in the subsurface. Therefore, once water has been lost from the surface within the model it cannot return to the domain.

2.1.3 As a result of these constraints, rainfall and losses are commonly represented via two different methods when utilising TUFLOW:

1. Representation of losses within the TUFLOW model. Rainfall profiles inputted into the model and losses represented as an initial loss (mm) and continuing loss rate (mm/hr) through the simulation within the hydraulic model.
2. Application of effective rainfall to the model domain. Rainfall profiles are generated and losses calculated outside the model, effective rainfall (run off) only is applied within the model and no losses are represented directly within the TUFLOW model.

2.1.4 Within this study, the second approach was utilised. Rainfall profiles and losses were calculated through use of a hydrological model, Revitalised Flood Hydrograph 2 (ReFH2). The resulting effective rainfall (rainfall-losses) was applied directly to the TUFLOW model grid. This method was chosen as it allows the ReFH2 model to be harnessed. This is considered to provide a better representation of losses and effective rainfall than the basic losses that can be applied within TUFLOW.

2.1.5 Use of the second approach was also predicated upon the fact that the modelled rainfall catchment is predominantly rural, meaning that catchment averaged net rainfall generated by ReFH2 could be applied across the whole rainfall catchment.

2.1.6 This section of the report outlines the rationale and approach for the hydrological aspects of the work undertaken. Section 3 of the report documents how this is incorporated into the hydraulic model, to estimate surface water flood risk.

2.2 Revitalised Flood Hydrograph (ReFH) 2 Model

2.2.1 ReFH2 is an industry standard rainfall-runoff model that is used to estimate peak design flows and hydrographs for catchments across the UK. ReFH2 was published in 2015 and is an update to the previous ReFH model that was first published in 2005. ReFH2 is a recommended method within the 2015 CIRIA SuDS manual for the estimation of greenfield runoff volumes.

2.2.2 The ReFH model has three components: a loss model, a routing model and a base flow model. The loss model uses a soil moisture accounting approach to define the amount of rainfall that is converted to direct runoff. The routing model functions according to the unit hydrograph concept, whilst the base flow model is based upon the linear reservoir concept. For more detailed information the reader is referred to the ReFH2 Technical Guidance[3].

2.2.3 ReFH2 contains a number of updates with respect to the original ReFH model. In particularly ReFH2 utilises the FEH13 Depth Duration Frequency (DDF) rainfall model, an update from the FEH99 model utilised within the original ReFH model.

2.2.4 Given that the Parsonage Down rainfall catchment is small and ungauged, it is considered that ReFH2 represents a viable method for estimation of rainfall hydrology in this area.

2.3 Estimation of Effective Rainfall using ReFH2

2.3.1 The catchment descriptors for the Shrewton (S01) inflow, detailed further within the fluvial hydrological analysis for the River Till (Annex 2 Part A), were input into ReFH2 as the descriptors at this location were considered most appropriate for application to the Parsonage Down catchment. Catchment descriptors were amended based upon GIS analysis undertaken for the Parsonage Down catchment, the values are included within Table 2.1.

Table 2.1 Amended Catchment Descriptors

Catchment Descriptor	Description	Amended Value
Catchment Area	Area of modelled catchment.	8.93 km2
URBEXT2000	Urban extent within the catchment.	0
DPLBAR	Drainage path length.	3.99 km
DPSBAR	Drainage path slope.	13.78 m/km

2.3.2 The model, along with amended parameters, was used to generate rainfall hydrology and calculate losses for the 3.33% AEP, 1% AEP, 0.1% AEP storm event with durations of 60, 180,360 and 720 minutes. This was undertaken for the winter storm profile, as the Parsonage Down catchment is essentially rural. An allowance of +40% was applied to the 1% AEP event to account for climate change.

[3] Wallingford Hydrosolutions (2016) The Revitalised Flood Hydrograph Model ReFH2 Technical Guidance

2.3.3 Initial hydraulic testing was undertaken with the 1% AEP event in order to determine the critical storm duration. This was found to be 360 minutes. Model results to justify this choice of critical duration are presented within Section 4.

2.3.4 Net rainfall, which corresponds to the surface runoff calculated after application of the loss model, was taken from the ReFH2 model output for the winter storm event and used for input directly into the TUFLOW model as effective rainfall.

2.4 Antecedent Conditions

2.4.1 As the Parsonage Down catchment falls within the chalk catchment of the River Till, where groundwater is known to exhibit significant variability, ground water levels and antecedent conditions prior to a storm event were considered when generating rainfall hydrology. Antecedent conditions were investigated using a sensitivity test of several key ReFH2 loss model parameters.

2.4.2 The two key parameters associated with the ReFH2 loss model are C_{max}, which represents the maximum soil moisture capacity, and C_{ini} which represents the initial water content within the soil moisture store.

2.4.3 Using the ReFH2 Technical Guide, a review of parameter estimates for C_{Max} and C_{Ini} has been undertaken. A summary of the findings are provided below with further details of the technical details provided in Appendix A.

2.4.4 The ReFH2 Technical Guide provides the predictive performance of the ReFH2 equations for estimating C_{Max} and other model parameters. The standard error (S.E.) for C_{Max} is reported as 1.29 (see Table 3, ReFH2.2 Technical Guidance), therefore taking the initial value derived by ReFH2, a sensitivity test for upper and lower value for +/-1 S.E. of C_{Max} can be investigated (Table 2.2).

Table 2.2: Lower and upper C_{Max} values for 1 +/- S.E.

Initial C_{Max} value from ReFH2	Standard error	Lower limit (C_{Max} – 1 S.E)	Upper limit (C_{Max} + 1 S.E)
1341	1.29	1040	1730

2.4.5 The results of varying C_{Max} values to reflect the lower and upper S.E. are provided in Table 2.3.

2.4.6 The initial soil moisture content is represented by the C_{Ini} initial condition. Within ReFH2, C_{Ini} is related to BFIHOST for all catchments irrespective of permeability when using the FEH13 rainfall model. The relationship between C_{Ini}, C_{Max} and BFIHOST for all stations in the National River Flow Archive (NRFA) dataset flagged as being suitable for Q_{Med} estimation is provided in Figure 20 of the ReFH2 Technical Guidance and reproduced Appendix A.

Table 2.3: Impact of increasing/decreasing C_{Max} on ReFH2 estimates 1% AEP event and C_{Ini} model parameter

Parameter/Model Estimate	Initial estimate	Lower limit (C_{Max} – 1 S.E)	Upper limit (C_{Max} + 1 S.E)
C_{Max}	1341	1040	1730
C_{Ini}	50.51	39.71	65.16
Flow (m^3s^{-1})	0.79	0.87	0.78
Direct Runoff Volume (Ml)	28.13	30.98	25.91

2.4.7 A review of BFIHOST throughout the River Till catchment indicates that there is little variation (0.963 – 0.967), therefore adjusting values for this catchment descriptor has not been considered. Using the relationship between C_{Ini}, C_{Max} and BFIHOST, upper and lower values have been estimated for C_{Ini} based on the BFIHOST (0.96) for the catchment (Table 2.4)

Table 2.4: Upper and lower values for C_{Ini} based on NRFA catchments suitable for QMed with a BFIHOST of ~0.96.

	Ln C_{Ini} (as a proportion of C_{Max})	C_{Max}	Updated C_{Ini} for initial soil moisture conditions
Upper	-2.90	1341	73.79
Lower	-4.45	1341	15.66

2.4.8 The results of varying C_{Ini} values to reflect the upper and lower initial conditions based on observed C_{Ini} for lower and upper S.E. are provided in Table 2.5. This demonstrates that ReFH2 is more sensitive to adjustments in C_{Ini} when compared with adjusting C_{Max}.

Table 2.5: Impact of increasing/decreasing the C_{Ini} initial condition on ReFH2 estimates for 1% AEP event.

Parameter/Model Estimate	Initial C_{Ini} for initial soil moisture condition	Lower C_{Ini} for initial soil moisture condition	Upper C_{Ini} for initial soil moisture condition
C_{Ini}	50.51	15.66	73.79
C_{Max}	1341	1341	1341
Flow (m^3s^{-1})	0.79	0.44	1.12
Direct Runoff Volume (Ml)	28.13	15.51	36.56

2.4.9 Based upon the analysis presented above, a sensitivity analysis was undertaken in order to quantify the response of the model to changes in effective rainfall resulting from varying the value of the C_{Ini} parameter in ReFH2.

2.4.10 The model was simulated using effective rainfall profiles based upon the default C_{Ini} value (50.51), along with the calculated lower bound C_{Ini} (15.66) and calculated upper bound C_{Ini} (73.79). Results from this sensitivity analysis are presented within Section 4. In summary the upper bound C_{Ini} value (73.79) was selected for application within the design simulations as a conservative assumption, taking into account the potential for higher groundwater levels within the Parsonage Down area.

2.5 Representation of the River Till

2.5.1 As stated within Section 1.2, the focus of the pluvial hydraulic modelling undertaken was to assess the existing flood risk from the surface water flow path at Parsonage Down and to assess changes in this surface water flow path attributable to the proposed scheme.

2.5.2 It was identified from the EA FMfSW that the surface water flow pathway through Parsonage Down discharges into the River Till to the north of Winterbourne Stoke. Hence it is clear that the River Till will act as a receptor for water flowing from this surface water pathway.

2.5.3 Interrogation of the LiDAR DTM shows that the level of the River Till floodplain adjacent to the channel is approximately 71m AOD. The level of the B3083 road, which presents a topographical obstruction to flow through Parsonage Down, is associated with a level of approximately 77m AOD at the location where it obstructs overland flow. Levels within the Parsonage Down valley to the west of the B3083 generally range from 77-80m AOD. The difference in level highlighted above, along with results presented within Section 6 of this report, demonstrate that flood levels on the River Till floodplain would not exert backwater effects upon the flow pathway through Parsonage Down.

2.5.4 The work undertaken here is intended to assess flood risk arising from a short duration localised storm event, these events commonly lead to surface water flood inundation. It is assumed that the modelled storm event at Parsonage Down would not coincide with a large fluvial event on the River Till. As such, no assessment of joint probability of these events is considered further within this report.

2.5.5 As a result of the above considerations it was deemed appropriate to include a basic representation of the River Till through application of an outflow boundary condition within the surface water hydraulic model. Further information on this boundary condition will be presented within Section 3 although saliently, the boundary condition was configured in order to reflect maximum water levels within the River Till channel for a 50% AEP fluvial flood event.

2.5.6 A sensitivity test was undertaken to further explore the sensitivity of modelled flows through Parsonage Down to the downstream boundary condition applied to the River Till. This sensitivity testing is further detailed within Section 5. Importantly, the results demonstrate that water level applied at the River Till boundary exerts a negligible impact upon flows through Parsonage Down, and suggests that the application of a boundary corresponding to maximum stage within the River Till within the 50% AEP fluvial event is appropriate.

3 Surface Water Hydraulic Modelling Methodology

3.1 Modelling Approach and Software

3.1.1 The surface water hydraulic modelling was undertaken using TUFLOW software version 2018_03_AB.

3.1.2 TUFLOW is a two-dimensional (2D) hydraulic modelling package that simulates hydrodynamic behaviour of flood waters across the land surface using a grid based approach. TUFLOW possesses the relevant functionality to facilitate the hydraulic modelling of surface water flows within a catchment in response to direct rainfall. This includes the ability to apply rainfall to the model grid, represent losses to infiltration and evapotranspiration, whilst the double precision version of TUFLOW enables shallow flow depths commonly associated with direct rainfall models to be simulated

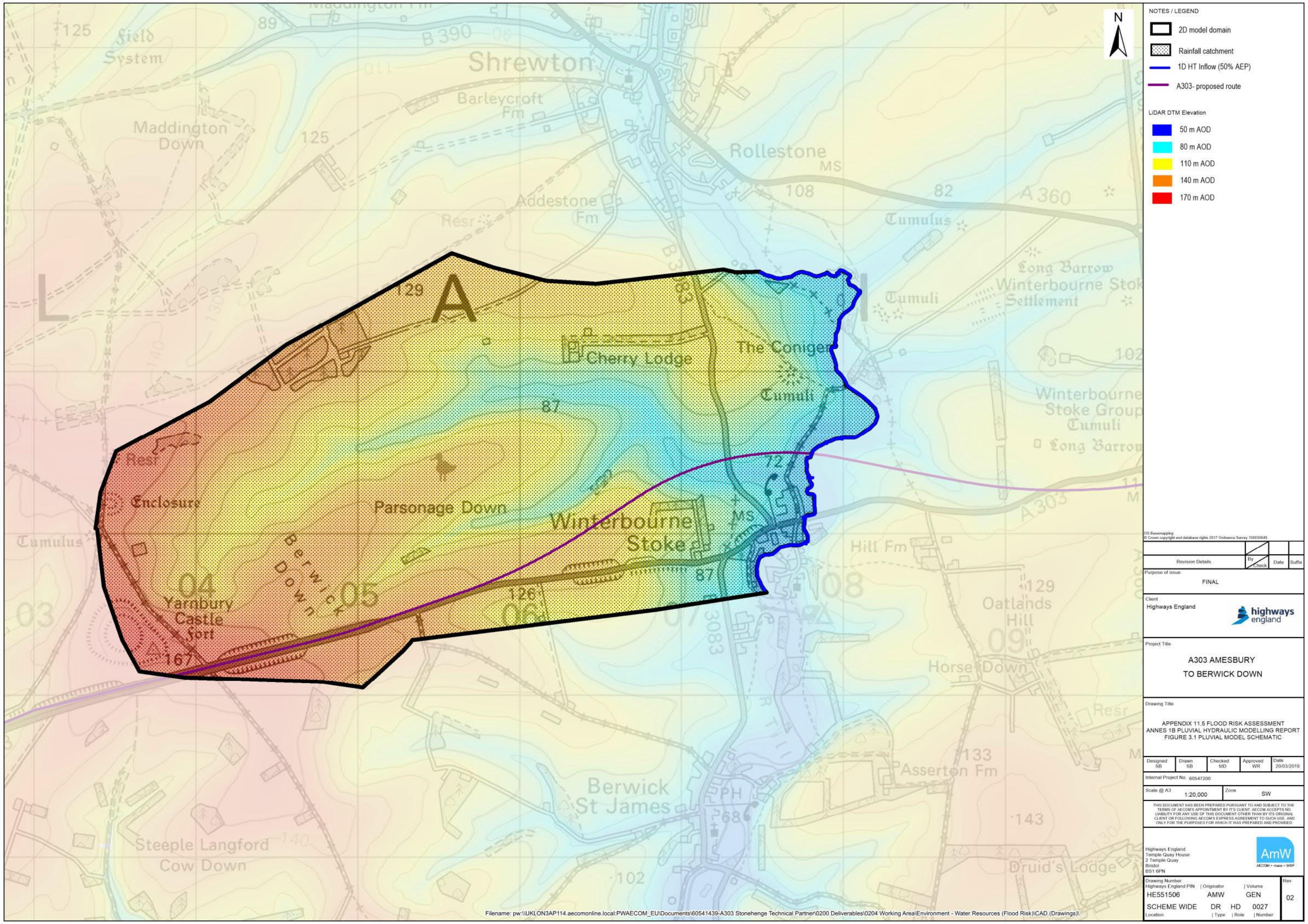
3.1.3 As outlined in Section 2 of this report, the primary input into the hydraulic model was effective rainfall hyetographs, whilst the River Till is represented as a boundary condition within the model, corresponding to water levels in a 50% AEP event. This section of the report details the setup of the hydraulic model in order to incorporate the rainfall hydrology and determine surface water flood risk.

3.2 Model Extent and Rainfall Catchment

3.2.1 The extent of the 2D model domain, along with the contributing rainfall catchment is shown within schematic Figure 3.1.

3.2.2 Within Figure 3.1, it is demonstrated that the model rainfall catchment covers the entire area that contributes surface water to the flow pathway through Parsonage Down at the location where it intersects the River Till. The model domain and rainfall catchment has been defined in order to encompass the entire contributing rainfall catchment to the west of the River Till.

3.2.3 It should be noted that the River Till channel was used to delineate the limit of the model domain and rainfall catchment, based upon the assumption that any rainfall to the east of the River Till would be conveyed into the watercourse and would not contribute towards surface water flooding within the Parsonage Down catchment, as this would be represented through the application of a River Till boundary (Figure 3.1).


NOTES / LEGEND
2D model domain
Rainfall catchment
1D HT Inflow (50% AEP)
A303- proposed route
LiDAR DTM Elevation
50 m AOD
80 m AOD
110 m AOD
140 m AOD
170 m AOD
Shrewton
Rollestone
Winterbourne Stoke
Parsonage Down
Cherry Lodge
Berwick St James
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNES 1B PLUVIAL HYDRAULIC MODELLING REPORT
FIGURE 3.1 PLUVIAL MODEL SCHEMATIC
Designed SB
Drawn SB
Checked MD
Approved WR
Date 20/03/2019
Internal Project No. 60547200
Scale @ A3 1:20,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0027
Location | Type | Role | Number
Rev
02

3.3 2D Model - Topography

3.3.1 The underlying topographical data utilised within surface water hydraulic model is a composite Digital Terrain Model (DTM) with a 2m grid resolution.

3.3.2 The primary source of topographical data within the composite DTM is provided by a 2m resolution EA LiDAR DTM. Metadata shows that this LiDAR was flown in 2010, and is the most recent LiDAR available for this location. Gaps present within the LiDAR DTM were filled in the first instance by a high resolution (1m) photogrammetric DTM.

3.3.3 The 1m resolution photogrammetric DTM was produced by Atkins within the previous project stage in 2016. This DTM was quoted to have a vertical accuracy of +/-40mm, although independent ground truthing suggested that its vertical accuracy was actually lower than the EA LiDAR DTM which is quoted to have a vertical accuracy of +/-150mm.

3.3.4 The 2D TUFLOW model was set up with a grid resolution of 4m and agreed with Wiltshire Council and the Environment Agency as appropriate.

3.3.5 Given the grid resolution of 4m, it was not considered appropriate to represent buildings through raising of threshold levels within the 2D domain, as is commonplace in direct rainfall models. Initial model testing demonstrated that building footprints were not clearly resolved within the grid, hence buildings were represented using raised Manning's Roughness Coefficient values.

3.4 Manning's Roughness Coefficient ('n')

3.4.1 Spatial variations in land cover within the model domain were defined using OS Mastermap data. This was used to define appropriate Manning's Roughness Coefficients throughout the model, shown within Table 3.1.

3.4.2 In line with best practice guidance for direct rainfall modelling, depth varying Manning's Roughness Coefficients were specified where appropriate within the model domain. This is based upon the rationale that effective roughness exerted upon surface water flows varies significantly depending upon the depth of water in relation to the scale of the roughness elements[4].

3.4.3 Depth varying roughness is significant within direct rainfall models, as these models are commonly characterised by very shallow flows across large areas of the model domain. Effective roughness for these shallow flows is much higher in comparison to deeper fluvial flows, for which standard Manning's Roughness Coefficients are applied.

3.4.4 Depth varying roughness values were applied for areas of grass and green space as this comprises the majority of the model domain. In general, elevated roughness values were applied for shallow flow depths, whilst this transitions to a standard roughness coefficient as flow exceeds a specified depth threshold. For grass and green space a Mannings 'n' coefficient of 0.1 was applied for depths

[4] Boyte (2014) The Application of Direct Rainfall Models as Hydrologic Models Incorporating Hydraulic Resistance at Shallow Depths.

less than 0.1 m and the standard value of 0.06 was applied when flow depths exceed this threshold. Depth varying roughness values were adapted from TUFLOW guidance[5]

3.4.5 For surfaces characterised by lower roughness such as roads Mannings 'n' values were retained as standard, as the influence of roughness elements on shallower depths was deemed to be less significant.

3.4.6 Buildings were represented within the model through application of an elevated Manning's Roughness Coefficient of 0.5.

Table 3.1 Mannings 'n' Roughness Values

Surface	'n'
Building	0.5
Roads and Paved areas	0.025
Grass	0.06
General Surface	0.04

3.5 Downstream Boundary

3.5.1 The River Till has been represented within the surface water hydraulic model as a constant head-time boundary, aligned with the right (west) bank of the river channel (Figure 3.1). This boundary defines the eastern most extent of the model domain. Based upon the rationale outlined within Section 2.5, it is considered that this will provide an appropriate representation of the River Till channel for the purposes of modelling surface water flooding through the Parsonage Down valley.

3.5.2 Time series of stage were extracted from the River Till fluvial model nodes for the 50% AEP event and were used to define levels for the head time boundary applied within the surface water model. During model development, a time varying head-time boundary was tested along with a constant head-time boundary, in which the peak stage was applied as a conservative assumption. Sensitivity testing, presented in Section 5 demonstrates that the boundary applied along the River Till has no impact upon the flow hydrograph and flood extents through Parsonage Down.

3.5.3 The 1D head-time boundaries were linked to the 2D model domain through a TUFLOW HX link, which allowed the water level to be interpolated between the different nodes. Adoption of this schematisation required creation of a dummy ESTRY channel network, which should be considered when evaluating model health indicators associated with the model.

3.5.4 As the 1D ESTRY element comprises a dummy network only, the volume of water stored within the 1D model remains constant despite the inflow and outflow volumes, which has a negative impact upon volume and mass error reporting. Therefore, it is recommended that the health of the model is assessed based upon diagnostics for the 2D TUFLOW element of the model only. This is

[5] BMT (2017) TUFLOW Manual 2017-09.

considered defensible as there are no other 1D ESTRY elements within the baseline model.

3.5.5 An additional model boundary was located along the southern limit of the model domain approximately 1 km downstream of Winterbourne Stoke, and comprised a head flow (HQ) relationship defined based upon the slope of the floodplain at this location.

3.5.6 Based upon the presence of several key attenuating structures upstream and the distance from the proposed scheme, the southern HQ boundary has no influence upon hydraulics within the area of interest for this report.

3.6 Model Timestep and Simulation Duration

3.6.1 The 2D TUFLOW model was simulated with a timestep of 2 seconds, in line with best practice guidance, which suggests that the 2D time step should be half of the 2D grid cell size (4m). The 1D model time step was set to be half of the 2D timestep, at 1 second. The model was run for a duration of 10 hours.

3.7 Proposed Scheme Scenario

3.7.1 The proposed scheme design was incorporated into the hydraulic model. For additional information relating to the proposed scheme, the reader is referred to the main FRA document which this report forms an annex to.

3.7.2 Amendments were made to the model topography and roughness layers in order to add in the cutting and embankments for the proposed route of the A303 within the model domain.

3.7.3 Amendments to the model topography were made in order to represent the land reprofiling at Parsonage Down.

3.7.4 Topographical amendments include a small increase in levels to the west of the farm access track which runs parallel to the River Till, north of Winterbourne Stoke. This increase in levels is intended to hold back small volumes of water flowing towards the River Till floodplain from the Parsonage Down valley, to encourage formation of a meadow habitat within the area.

3.7.5 The River Till Viaduct has not been represented directly within the model as the bridge is open span and the soffit is elevated far above feasible flood levels.

3.7.6 Piers for the River Till Viaduct are located within the floodplain and have been represented within the 2D model as flow constriction units, which facilitate blockage of flow through cells and mimic the obstruction to flood flow attributable to the piers.

3.7.7 The realignment of the B3083 road was represented within the model through amendments to topography and roughness layers. The underpass of the B3083 beneath the A303 was represented directly within the model grid, as ground levels were lowered to the level of the B3083.

3.7.8 Following consultation with Wiltshire Council, a modified drainage solution was added to the model, this was included within the proposed scheme design in order to retain conveyance of surface water flow through Parsonage Down east to the River Till (Figure 3.2).

3.7.9 The solution comprised a 1.2m diameter circular culvert, the inlet of which was located immediately to the west of the underpass of the B3083, flowing in southerly direction beneath the A303. The culvert conveys water beneath the A303, outfalling to an open drainage channel for approximately 85m. An additional three 0.45m diameter circular culverts convey water from this drainage channel, from west to east below the B3083. From the outfall of this culvert surface water is able to flow east overland towards the River Till, closely following the original overland flow paths prior to construction of the proposed scheme. This solution was added into the model as a combination of 1D ESTRY culverts and topographical modifications within the 2D domain. The proposed scheme arrangement at Parsonage Down is shown here within Figure 3.2. More information relating to this drainage solution is contained within the Road Drainage Strategy document (A303 Amesbury to Berwick Down Environmental Statement- Appendix 11.3).

3.7.10 It should be noted that the drainage arrangement, namely the culvert beneath the A303 and culverts below the B3083, are also intended to drain groundwater from the Parsonage Down valley.

3.7.11 The schematisation of the current route of the A303 was retained as within the baseline model.

3.7.12 The landscaping planned at Parsonage Down involves deposition of unstructured chalk, sourced from the tunnel arisings, upon the current ground surface. As the deposited chalk is unstructured it is expected that its permeability will be reduced when deposited in-situ. The change in permeability within the area of deposited tunnel arisings has been represented within the proposed scenario model, along with the installation of an engineering solution to manage surface water run off rates. This report describes how the tunnel arisings and associated engineering solution are applied within the hydraulic model.

3.7.13 To account for the decrease in permeability of the deposited chalk, along with the engineering solution that will be installed to improve permeability within this area, varying infiltration rates have been applied within Parsonage Down in the model. The varying infiltration rates below have been applied across the area of chalk fill. The engineering solution will be zoned according to the depth of fill within the Parsonage Down area. Varying infiltration rates stated below have been applied within the model based upon spatial maps of calculated fill depth, these are included within Appendix C.

1. 0-2m depth of chalk fill- engineering solution installed- infiltration retained at existing (baseline) rates
2. 2-4m depth of chalk fill- engineering solution installed- infiltration assumed to be 50% of existing (baseline) rate
3. >4m depth of chalk fill- no engineering solution installed- infiltration rate assumed to be 0% of existing (impermeable)

3.7.14 In summary where the depth of chalk fill is 0-2m and 2-4m, an engineering solution will be installed to improve permeability. Where the depth of chalk fill is greater than 4m no engineering solution is applied and the chalk is assumed to be associated with a very low infiltration rate and is treated as impermeable within the hydraulic model. Infiltration rates calculated and applied to the model as detailed within 3.7.13 are included within Appendix D.

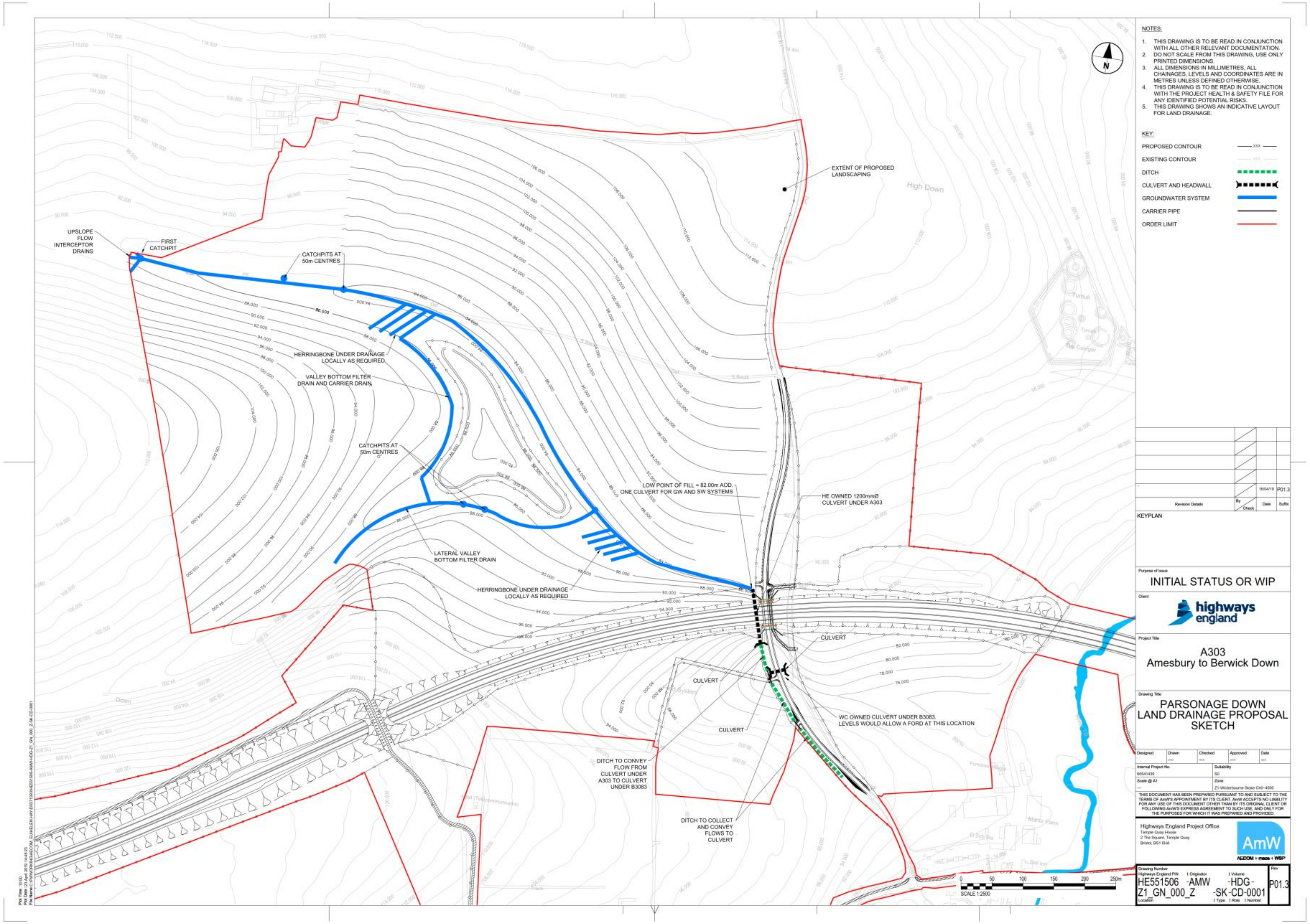


Figure 3.2 Parsonage Down Drainage Arrangement

3.8 Filtering of Model Results

3.8.1 Within pluvial hydraulic models, rainfall is applied across a large area within the model, hence a large proportion of the grid is commonly characterised by very shallow flood depths. Therefore in all cases it is necessary that model results are filtered prior to presentation within map format.

3.8.2 Within the proposed surface water model some erroneously high depths were observed at the edge of the A303 carriageway, particularly within the length of the road which falls within a cutting. Upon further investigation it was confirmed that these depths were an artefact of the modelling representation within TUFLOW, occurring as a result of shallow depths of water flowing at high velocities down the steep banks within the A303 cutting.

3.8.3 For the presentation of surface water modelling results within this report, filtering was undertaken in order to remove the erroneously high depths outlined above along with small isolated areas of ponding water of less than 6 grid cells size, deemed to be sufficiently minor and were removed to not impact presenting the results.

3.9 Model Simulation List

3.9.1 Table 3.2 contains a list of the model simulations that have been completed as part of the work documented within this hydraulic modelling report. The following simulations were undertaken.

Table 3.2 Simulation List

Simulation	AEP	Storm Duration
Baseline Scenario	3.33%	6hr
	1%	6hr
	1% + 40% CC	6hr
	0.1%	6hr
	1%+ 40% CC	12hr
Baseline Scenario Sensitivity +20% roughness	1%	6hr
Baseline Scenario Sensitivity -20% roughness	1%	6hr
Baseline Scenario Sensitivity- River Till Time Varying 1 in 2yr Water Level Boundary	1%	6hr
Baseline Scenario Sensitivity- River Till Peak 100yr Water Level Boundary	1%	6hr
Proposed Scenario	3.33%	6hr
	1%	6hr
	1% + 40% CC	6hr
	0.1%	6hr
	1% + 40% CC	12hr
Proposed Scenario Sensitivity- 25% Blockage	1% + 40% CC	6hr
Proposed Scenario Sensitivity- 50% Blockage	1% + 40% CC	6hr

4 Surface Water Hydraulic Modelling Results

4.1 Overview

4.1.1 Within this section hydraulic model results are presented in order to document any impacts the proposed scheme may have upon surface water flooding. Results are presented as follows:

4.1.2 Section 4.2 establishes the hydrological conditions to be applied within the design simulations.

4.1.3 Section 4.3 presents the baseline scenario hydraulic model results.

4.1.4 Section 4.4 presents the proposed scenario hydraulic model results and demonstrates changes with respect to the baseline scenario.

4.1.5 Within this section, results will be presented in the form of maps, along with hydrographs. Key locations where model results have been extracted are visualised within Figure 4.1. Hydrographs have been extracted from both within Parsonage Down valley, and also at the location where the flow pathway reaches the River Till floodplain.

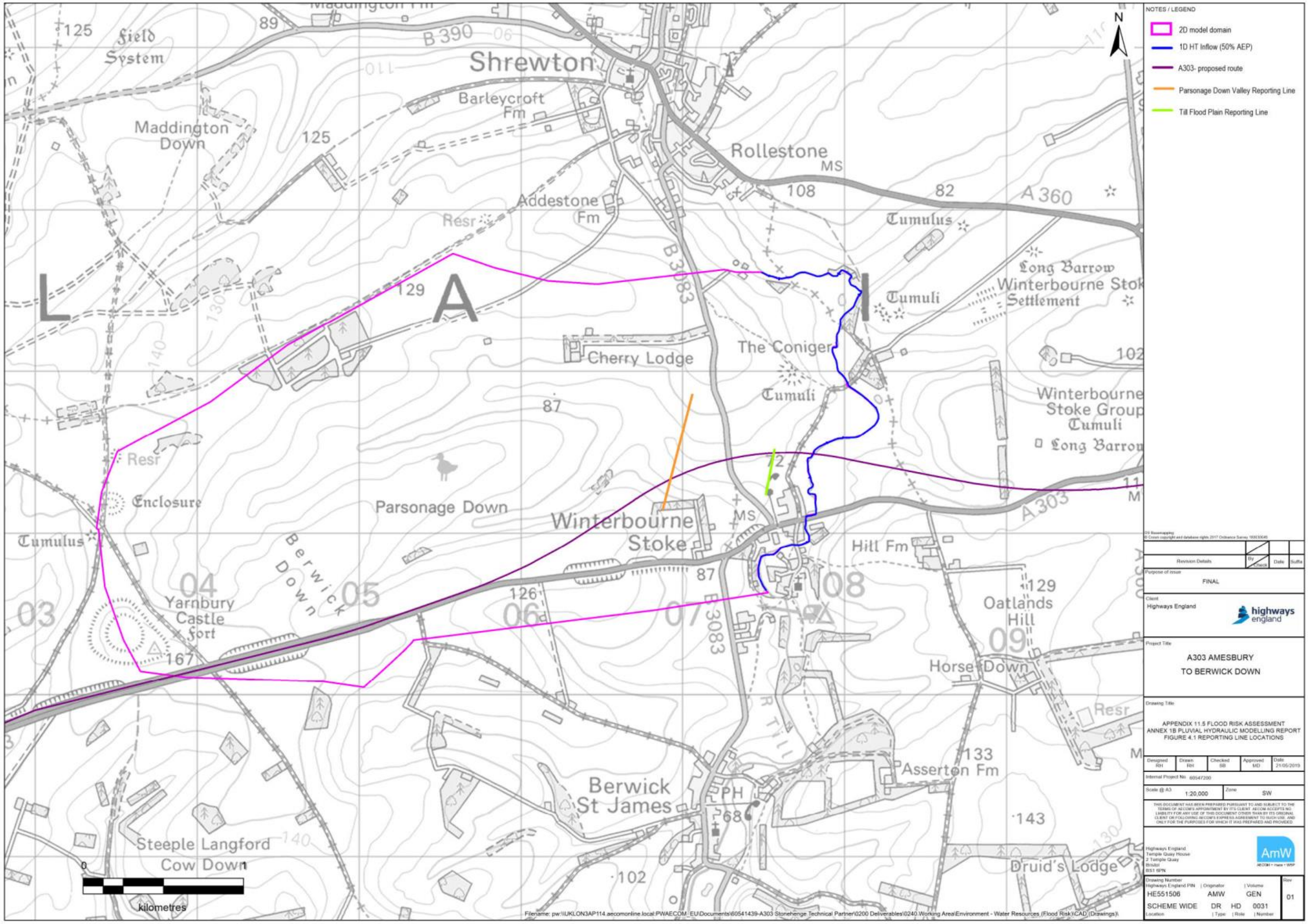


Figure 4.1 Model Results Extraction Locations

4.2 Critical Storm Duration and Antecedent Conditions

4.2.1 In order to establish the hydrological conditions, a series of hydraulic model simulations were undertaken. These simulations were undertaken for the 1% AEP event using the baseline model set up.

4.2.2 The aim of these simulations was to determine the critical storm duration for the catchment at Parsonage Down. Furthermore, an additional set of simulations were completed in order to select the C_{Ini} value which is most appropriate for the representation of antecedent conditions at the site.

Table 4.1 Peak Flow Matrix- Parsonage Down Valley

	Storm Duration			
	1 hour	3 hours	6 hours	12 hours
C_{Ini} Lower	0.13 m^3/s	0.25 m^3/s	0.40 m^3/s	0.39 m^3/s
C_{Ini} Default	0.33 m^3/s	0.69 m^3/s	0.89 m^3/s	0.76 m^3/s
C_{Ini} Upper	0.55 m^3/s	1.05 m^3/s	1.25 m^3/s	1.02 m^3/s

Table 4.2 Cumulative Volume Matrix- Parsonage Down Valley

	Storm Duration			
	1 hour	3 hours	6 hours	12 hours
C_{Ini} Lower	2,294 m^3	4,021 m^3	7,476 m^3	10,160 m^3
C_{Ini} Default	5,373 m^3	8,879 m^3	14,169 m^3	18,213 m^3
C_{Ini} Upper	7,499 m^3	12,281 m^3	18,686 m^3	23,748 m^3

4.2.3 Table 4.1 presents a matrix of peak flow through Parsonage Down (location shown on Figure 4.1), whilst Table 4.2 presents cumulative volume flowing through Parsonage Down. The data contained within these tables demonstrates that the model is sensitive to storm duration, and that critical storm duration is 6 hours when considered in terms of peak flow. The 12 hour storm is critical when considered in terms of cumulative volume flowing through Parsonage Down valley.

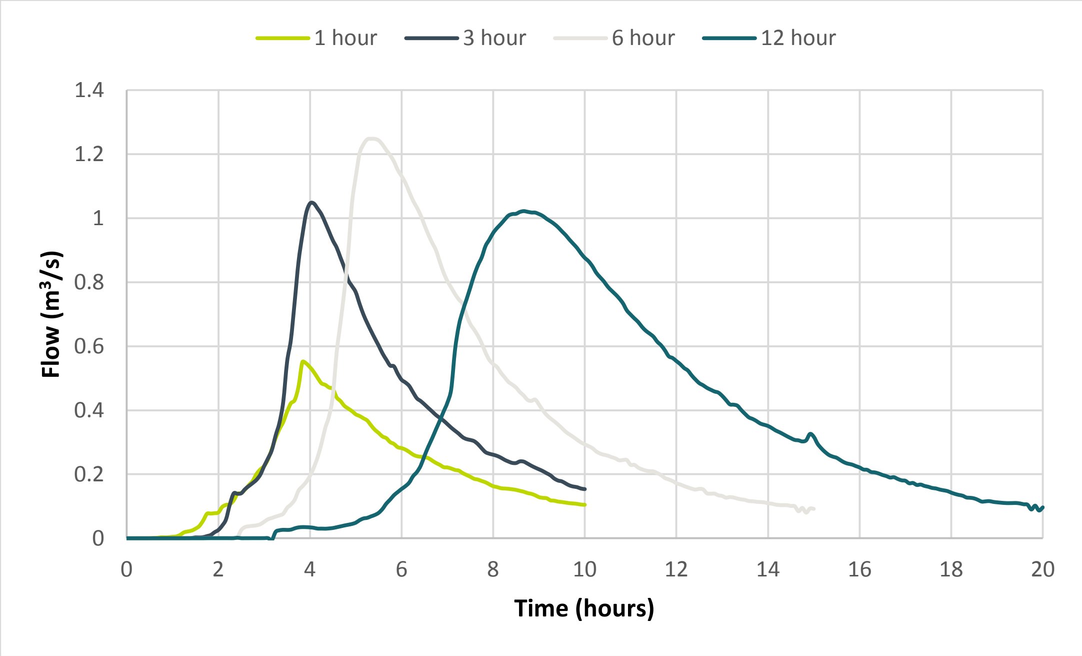


Figure 4.2 Parsonage Down Valley Flow 1% AEP - Varying Storm Duration (C_{ini} Upper)

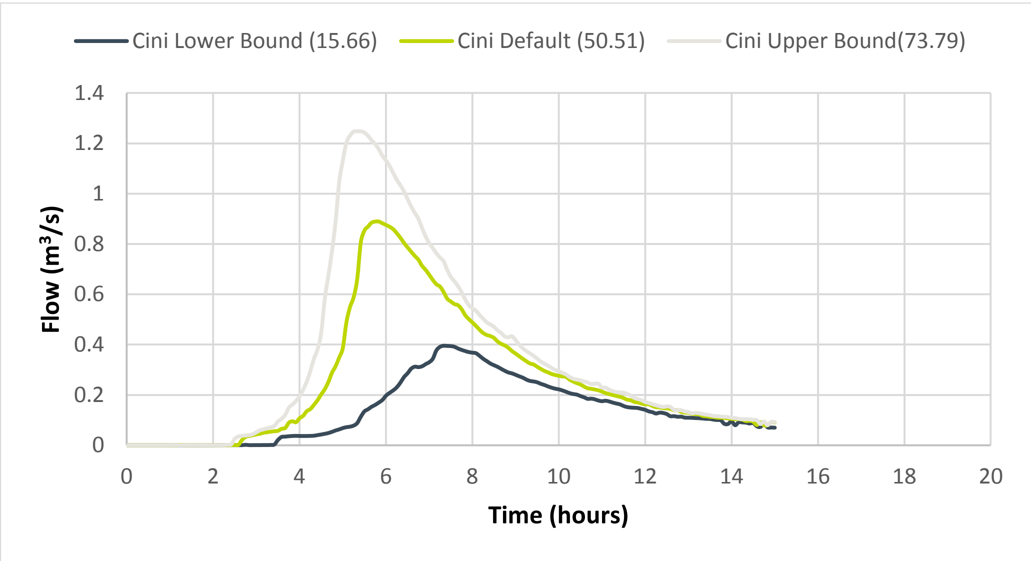


Figure 4.3 Parsonage Down Valley Flow 1% AEP- Varying C_{Ini} (6 hour storm)

4.2.4 Tables 4.1-4.2 shows that modelled flow through Parsonage Down is sensitive to variations in the ReFH2 C_{Ini} parameter. For all storm durations the upper bound value of C_{Ini} produces both the greatest peak flow and cumulative volume.

4.2.5 Based upon results presented within Tables 4.1 - 4.2 and Figures 4.2 – 4.3 it was decided to adopt the 6 hour storm duration and the upper limit C_{ini} value for the design model simulations. As the 12 hour storm was shown to be critical in terms of cumulative volume, this storm duration was also selected for simulation of the design event (1% AEP plus climate change).

4.3 Baseline Scenario Results

4.3.1 Figure 4.4 shows a comparison of the 1% AEP surface water flood extent output from the Parsonage Down model and the EA medium surface water flood extent, which corresponds to the equivalent AEP event.

4.3.2 Figure 4.4 shows a good level of agreement for the surface water flow pathway through Parsonage Down to the point where it intersects the B3083. Within the 1% AEP event, the two flow paths sourced from the catchment to the west join to form one primary flow path to the south east of Cherry Lodge. This flow pathway continues to the east, with surface water accumulating behind the current course of the B3083.

4.3.3 Within the baseline scenario, modelled 1% AEP results, surface water flows over the top of the B3083 road and down to the bottom of the River Till valley, whilst in the EA FMfSW this does not occur. This suggests that the modelled flow through the Parsonage Down catchment is higher within the site specific model, or that the crest level of the B3083 is more well defined within the model grid. Within the baseline scenario, the B3083 constitutes the only blockage to the conveyance of flow within the flow pathway, and the baseline depth map demonstrates that water accumulates to depths of over 1.5m to the west of the road within the 1% AEP event.

4.3.4 It should be noted that there is a significant difference between the inundation shown within the River Till channel floodplain between the EA FMfSW and the Parsonage Down baseline scenario flood extents. As described previously, within the site specific modelling undertaken here the River Till is as a boundary condition corresponding to the peak level within the 50% AEP fluvial event. Therefore it would not be expected that site specific model results would match the EA FMfSW. This is not considered to be significant given that the focus of this modelling is the surface water flow pathway through Parsonage Down, up-catchment of the River Till fluvial floodplain.

4.4 Proposed Scenario Results

4.4.1 Figure 4.5 shows a comparison of maximum flood depth outputs from the baseline and proposed permanent scenario models for the 1% AEP event, including a +40% allowance for climate change for the critical 6 hour storm duration. Figure 4.6 presents a difference plot which spatially maps the differences in maximum flood depths within the baseline and proposed scenario. Green colouration reflects a decrease in flood depth within the proposed scenario, whilst red colouration demonstrates an increase in depth within the proposed scenario.

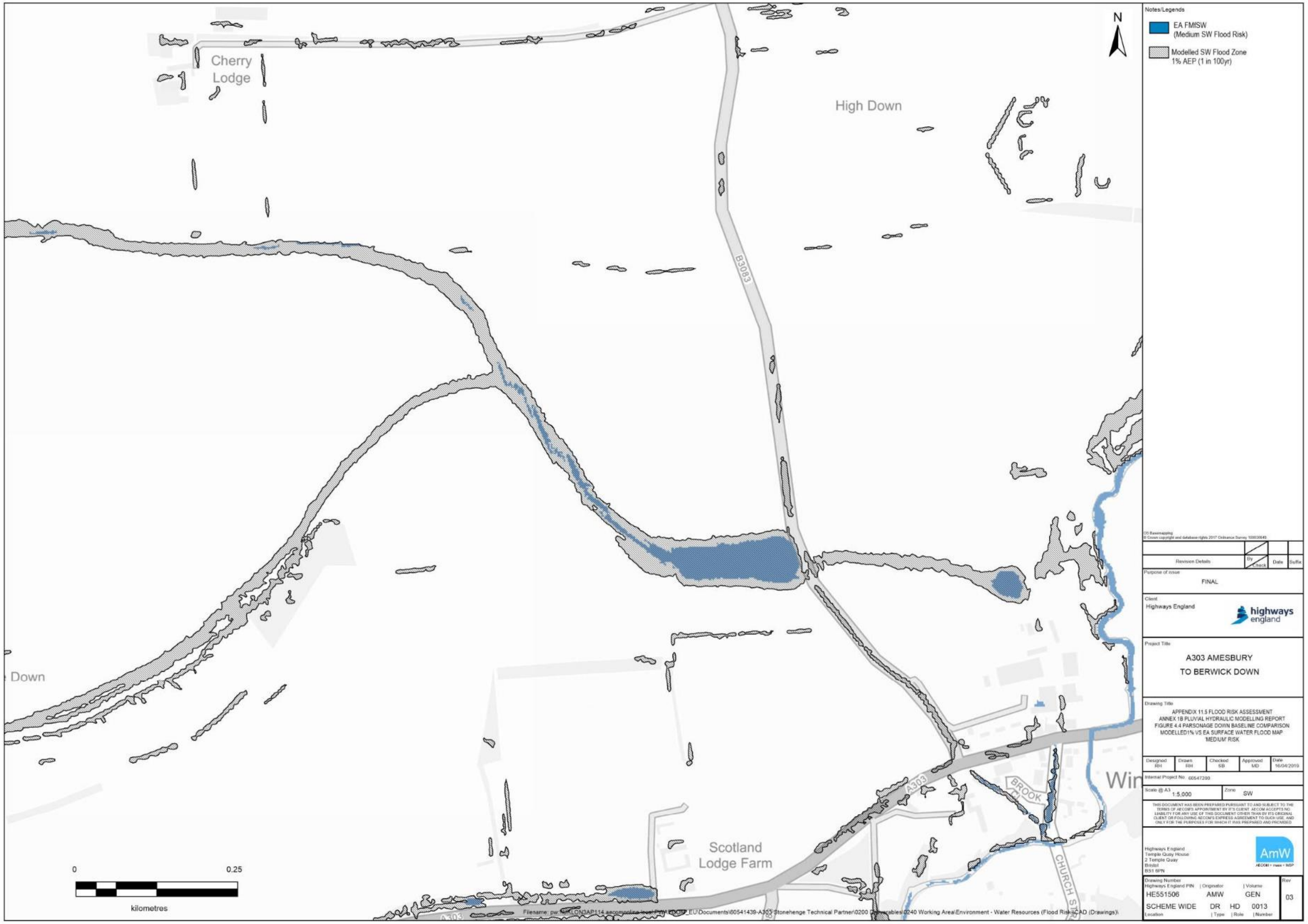


Figure 4.4 Surface Water Flood Extent Comparison - 1% AEP

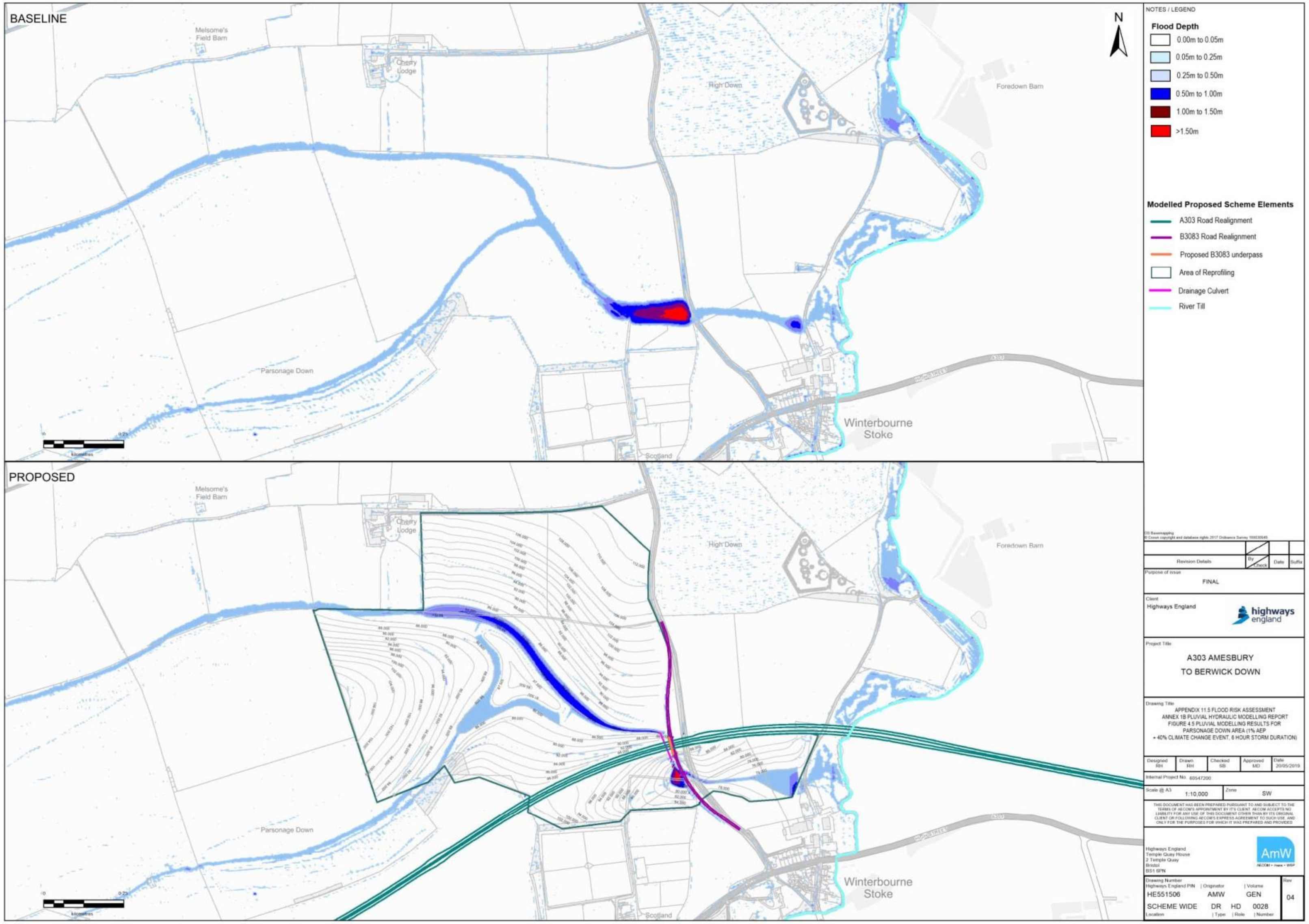


Figure 4.5 Maximum Flood Depth Comparison- 1% AEP Plus Climate Change (40%) - 6 Hour Storm Duration

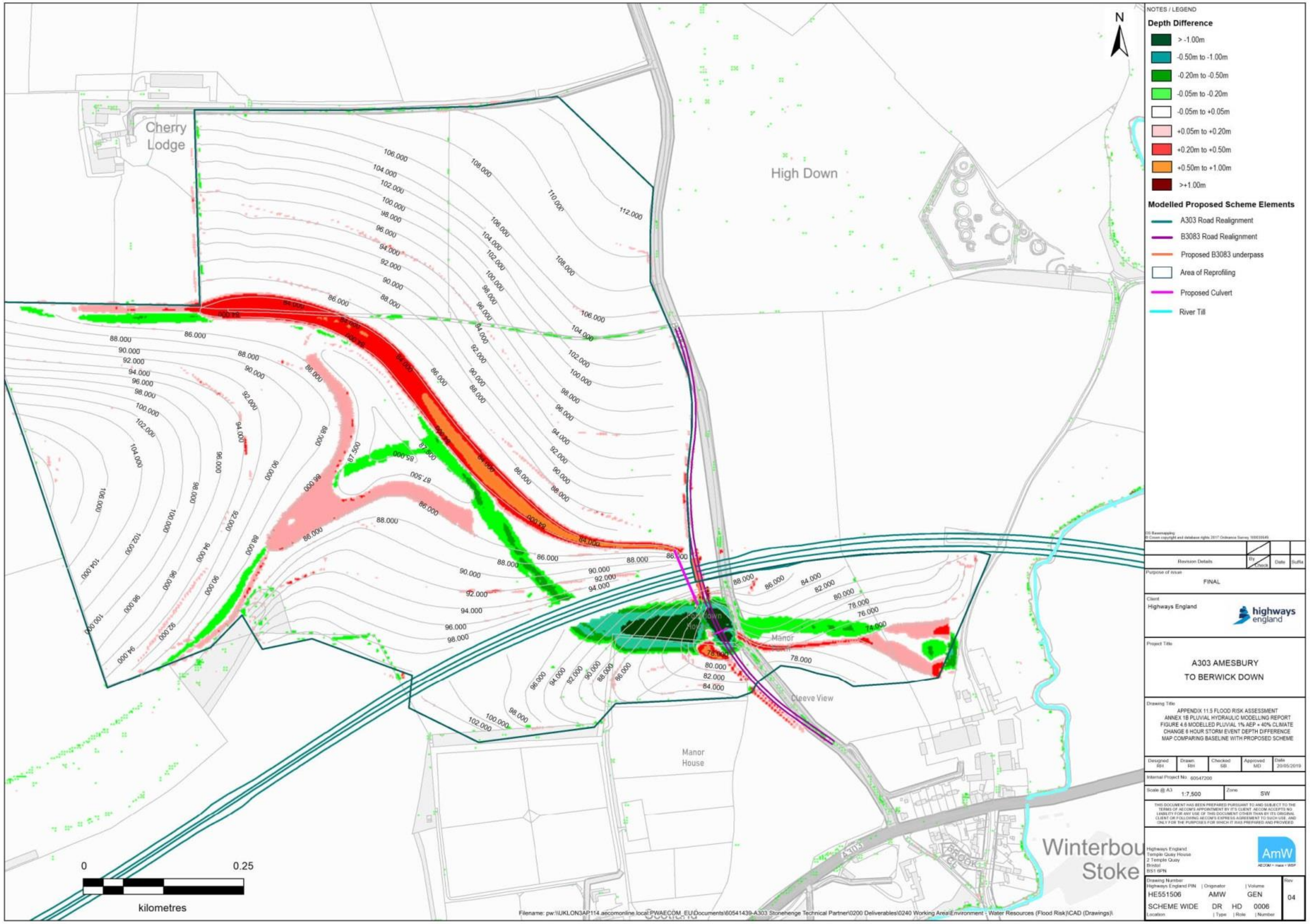


Figure 4.6 Maximum Flood Depth Difference Plot- 1% AEP Plus Climate Change (40%) - 6 Hour Storm Duration

4.4.2 The proposed scheme has a substantial impact upon the flow of surface water through Parsonage Down. Within the proposed scenario, the land reprofiling, along with new alignment of the A303 and B3083, present a combined constriction on conveyance along the surface water flow pathway through Parsonage Down. This is mitigated by the presence of the drainage culvert which collects surface water to the north west of the intersection of the A303 and B3083, and passes water approximately 85m to the south beneath the A303. Water subsequently flows through a drainage ditch and through three parallel culverts below the B3083, before draining overland towards the River Till, south of the proposed route of the A303.

4.4.3 Modelling results demonstrate that the combination of elements of the proposed scheme lead to a change in the location of the flow route, and deepening of the surface water flow through Parsonage Down to the north of the A303. Depths within the surface water flow pathway are typically up to 1m in depth.

4.4.4 It should be noted that a substantial volume of water ponding to the west of the B3083 road within the baseline scenario is no longer present within the proposed scenario. Furthermore, the model results suggest that the B3083 road is flooded within the baseline scenario, whilst the drainage solution implemented within the proposed scenario means that the B3083 does not flood within the equivalent event. This is a benefit of the scheme.

4.4.5 Despite the changes described above, within the proposed scenario surface water sourced from the Parsonage Down catchment outfalls to the River Till at approximately the same location as in the baseline scenario.

4.4.6 The maximum depth difference plot presented within Figure 4.6, shows that for the 1% AEP event including an allowance for climate change, there is no significant change in maximum flood depths upon the River Till floodplain as a result of surface water flow from the Parsonage Down catchment.

4.4.7 Hydrographs extracted showing flow into the River Till floodplain (Figure 4.7), provide a further indication of the impact of the scheme upon flow of surface water onto the River Till floodplain. Figure 4.7 indicates that for the modelled design event, there is a small increase in peak surface water flow onto the River Till floodplain. The peak flow increases from $0.97m^3/s$ in the baseline scenario, to $1.14m^3/s$ in the proposed scenario, an increase of $0.17\ m^3/s$. It should also be noted that the results demonstrate an increase in the overall volumes of surface water supplied to the Till floodplain.

4.4.8 Figure 4.7 suggests that there is an increase in peak flow and volume to the Till floodplain within the 1% AEP event including an allowance for climate change. These hydrographs were extracted just before the surface water flow pathway reaches the Till floodplain (Figure 4.1). The lack of difference in maximum flood depth observed on the Till floodplain within Figure 4.6 suggests that the additional surface water flow is distributed across a larger response hydrograph, where the resulting changes in flood depth are not sufficient to create a significant change in maximum depth.

4.4.9 Figures 4.8-4.10 present equivalent flood mapping and hydrographs for the 1% AEP plus climate change design event for the longer 12 hour duration storm, which was shown to be critical in terms of cumulative volumes of flow through Parsonage Down.

4.4.10 Results from the longer duration storms can be considered similar to those for the shorter 6 hour duration storm. Saliently, the increase in peak flow within the proposed scenario compared to the baseline is slightly less than for the shorter duration storm (+0.11m^3/s). As for the shorter duration storm, the maximum flood depth difference plot indicates there is no change in maximum modelled flood depth upon the River Till floodplain within the proposed scenario when simulated for the 12 hour storm duration.

4.4.11 Importantly, the results outlined above indicate that longer duration critical volume events are not associated with additional detrimental impacts relating to the proposed scheme, when compared to shorter duration peak flow events.

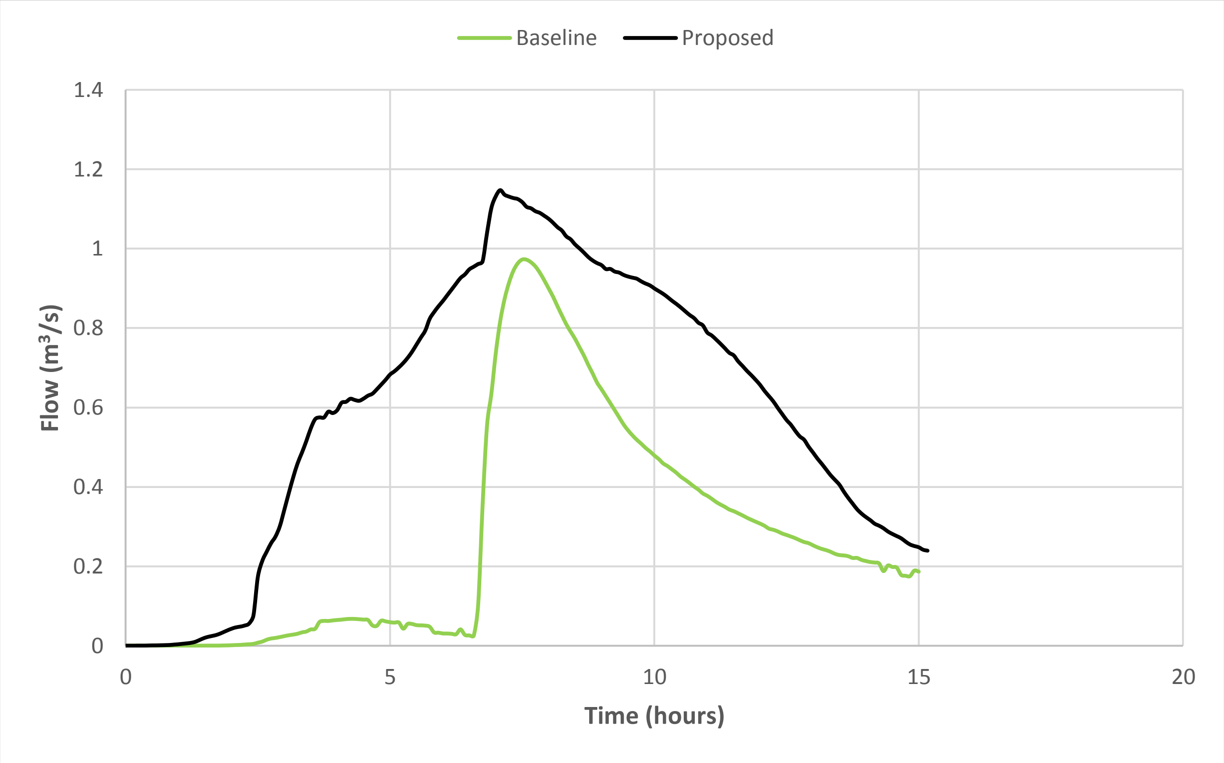


Figure 4.7 Flow on to the River Till Floodplain 1% AEP + 40% 6 hours CC Comparison

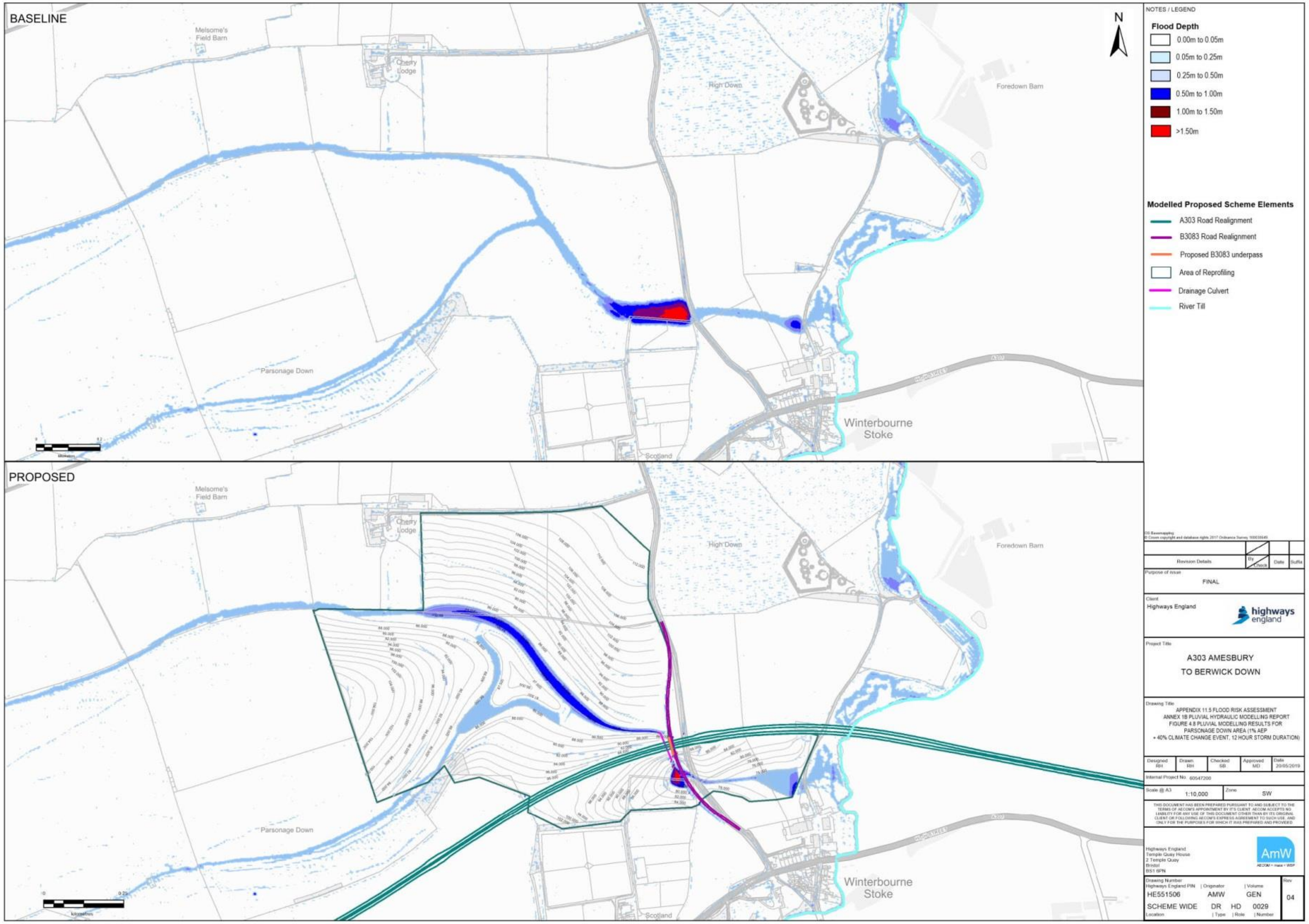


Figure 4.8 Maximum Flood Depth Comparison Plot - 1% AEP Plus Climate Change (40%) 12 Hour Storm Duration

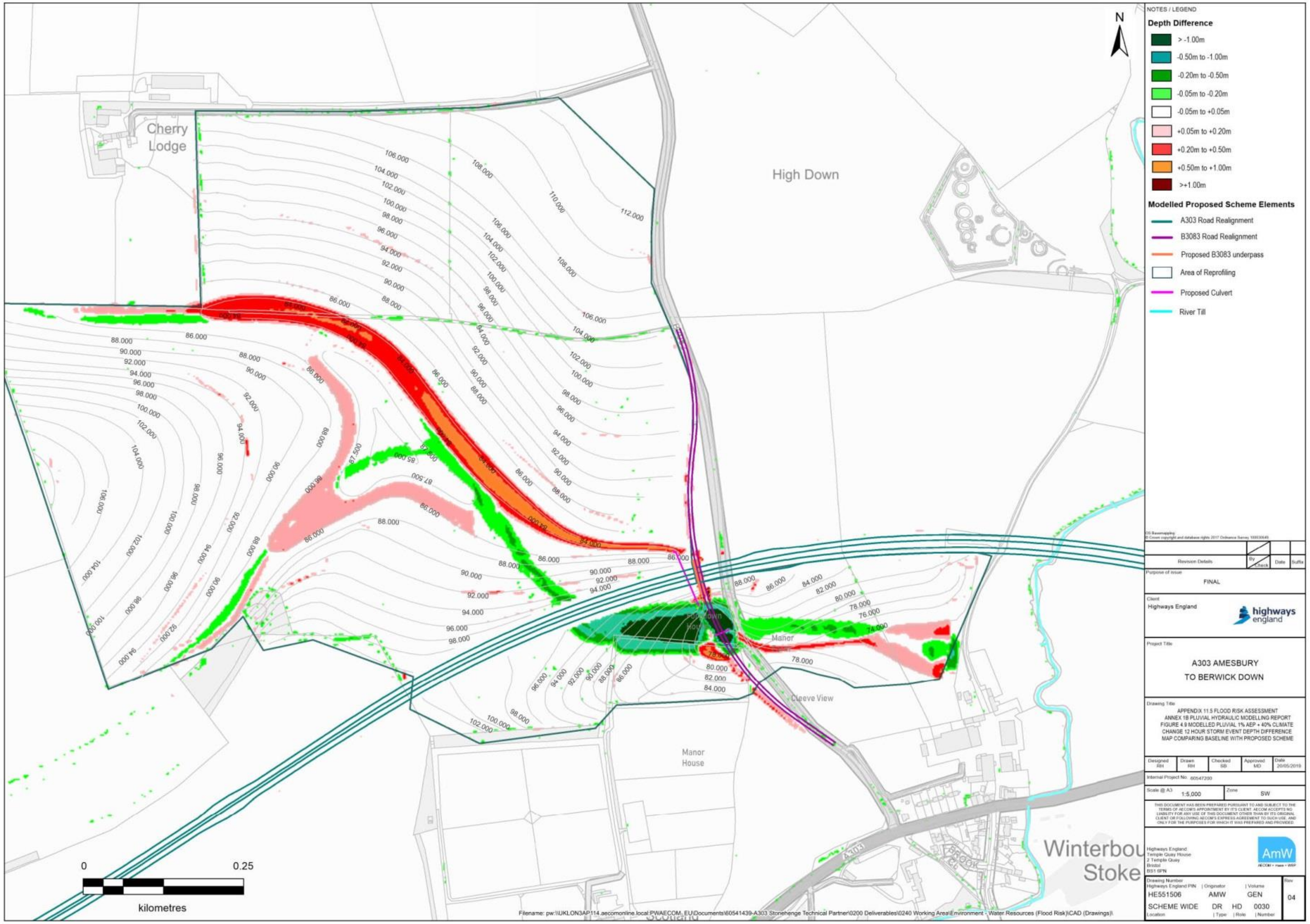


Figure 4.9 Maximum Flood Depth Difference Plot- 1% AEP Plus Climate Change (40%)- 12 Hour Storm Duration

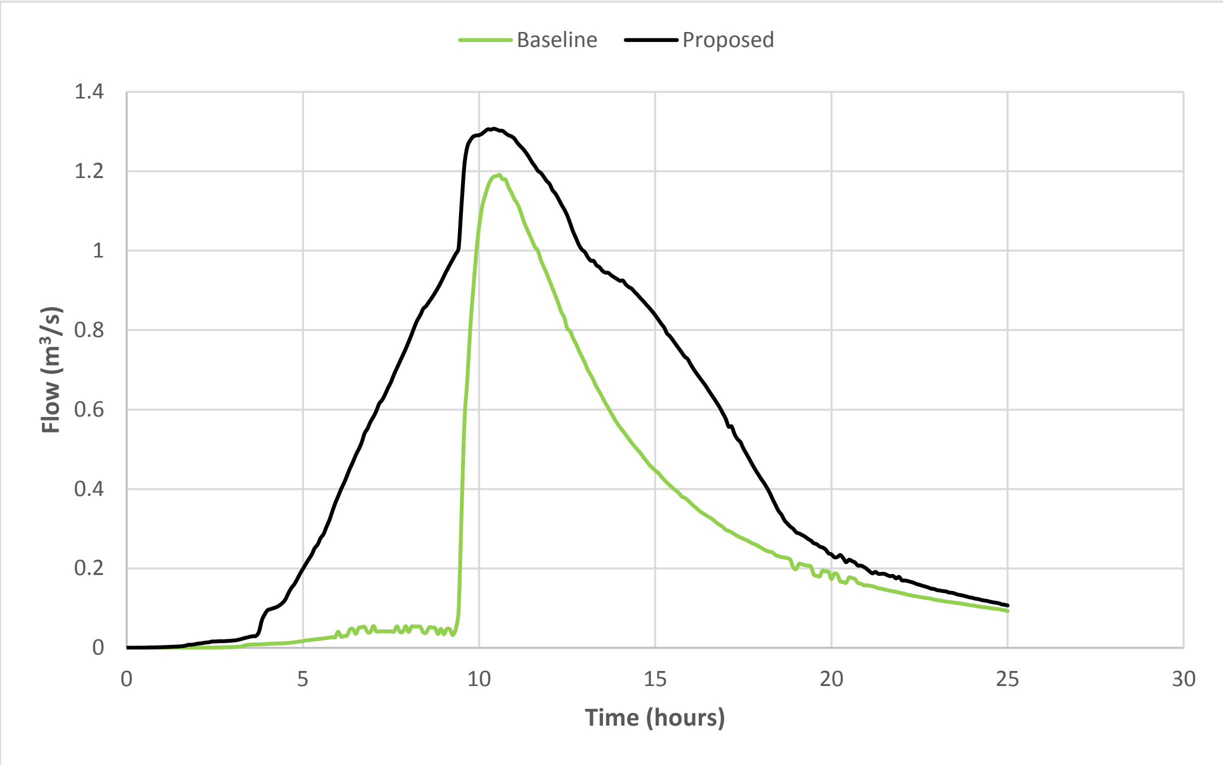


Figure 4.10 Flow on to the River Till Floodplain 1% AEP + 40% CC 12 hours Comparison

4.4.12 Equivalent comparison maps and depth difference plots for the 3.33% AEP, 1% AEP and 0.1% AEP events (6 hour storm duration) are included within Appendix B. In general, the modelling results from the other AEP events are in line with the 1% AEP plus climate change design event. It should be noted that an increase in maximum flood depth of between 0.05m-0.20m is shown to occur within an area of the Till floodplain within the 3.33% AEP event. This can be attributed to the fact that in 3.33% AEP baseline scenario a large proportion of surface water is trapped by the B3083 carriageway and retained upslope of the Till floodplain, whilst in the proposed scenario the drainage solution installed removes the attenuating impact of the B3083.

5 Sensitivity Analysis

5.1 Overview

5.1.1 An absence of observed data for the Parsonage Down area means that verification of model outputs is not possible. In lieu of verification, sensitivity analysis provides an alternative means to assess the quality of a hydraulic model.

5.1.2 Sensitivity analysis involves running of the hydraulic model with variations in key model inputs or parameters. The response of the model can be used to provide insight to model performance, and identify potential shortcomings in assumptions.

5.1.3 Specifically, sensitivity to Manning's 'n' Roughness Coefficient and the boundary condition applied to represent the River Till were assessed for the baseline model.

5.1.4 Several sensitivity tests were also undertaken using the proposed scenario model, in order to reflect aspects of the scheme which may impact upon surface water flood risk at Parsonage Down.

5.2 Baseline Scenario- Manning's 'n' Roughness Coefficient

5.2.1 Sensitivity to modelled roughness was assessed through globally increasing and decreasing 2D Manning's Roughness Coefficient (Manning's 'n') by 20%.

5.2.2 The modelled response to variations in Manning's 'n' in terms of flow through Parsonage Down are shown within Figure 5.1. The model responds in a reasonable manner to variations in Manning's 'n', with flow increasing with respect to the baseline when roughness is lowered, and vice versa. Overall peak flow through Parsonage Down within the baseline scenario is 1.26 m^3/s, whilst peak flows are 1.21 m^3/s and 1.27 m^3/s for the increased and decreased Manning's 'n' simulations respectively. This corresponds to a range of 0.06 m3/s over the range of Manning's 'n' values tested.

5.2.3 It should be noted that the change in maximum modelled flood depths and extents across the range of roughness values tested was minimal, and in line with the magnitude of change in flows documented within Figure 5.1, and therefore results were not presented within map format.

5.2.4 Overall, the hydraulic model shows an intuitive and consistent response to variations in roughness equal to what would be anticipated for a hydraulic representation of such a region.

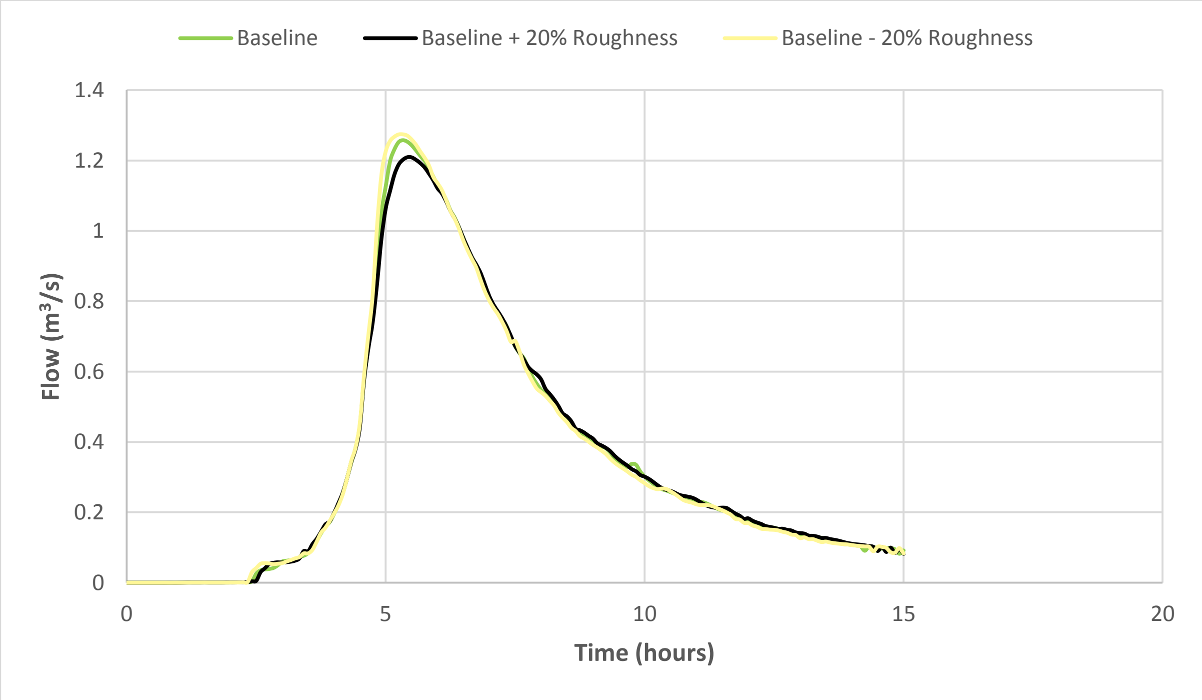


Figure 5.1 Parsonage Down Flow 1% AEP 6 hours Roughness Comparison

5.3 Baseline Scenario - River Till Boundary Condition

5.3.1 Based upon the difference in elevation between the River Till floodplain and through the dry valley at Parsonage Down, it was thought that fluvial flood inundation from the River Till would exert a minimal impact upon flow of surface water through the area of interest. Nevertheless, further sensitivity analysis has been undertaken in order to further justify the representation of the River Till applied within the hydraulic model.

5.3.2 The design simulations were undertaken with a constant HT boundary, with the water level applied corresponding to the peak water level extracted from the River Till fluvial model for the 50% AEP event. Two additional sensitivity simulations are presented here, the first is a time varying water level for the 50% AEP event extracted from the River Till fluvial model, whilst the second is a constant water level corresponding to the modelled 1% AEP event peak for the River Till.

5.3.3 Hydrographs of flow through Parsonage Down are presented for the three boundary configurations within Figure 5.2. Overall, the boundary condition applied to represent the River Till exerted no impact upon flow of surface water through Parsonage Down, justifying the boundary setup adopted for the design simulations.

5.3.4 It should be noted that the change in maximum modelled flood depths and extents at Parsonage Down for the range of boundary configurations tested was minimal and therefore results were not presented within map format.

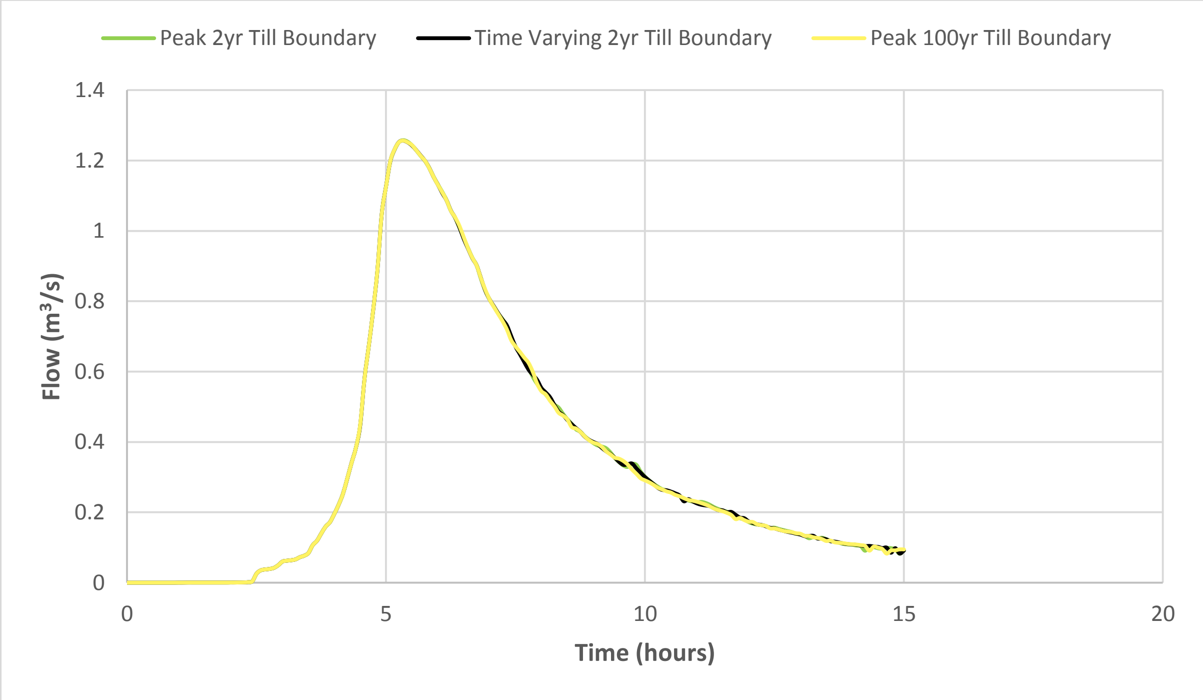


Figure 5.2 Parsonage Down Flow 1% AEP 6 hours Boundary Comparison

5.4 Proposed Scenario- Blockage Analysis

5.4.1 Additional sensitivity analysis has been undertaken using the proposed scenario model in order to account for potential blockage to the culvert which conveys water beneath the A303. Two scenarios were tested, involving representation of a 25% and 50% blockage to the culvert, applied using the blockage attribute of the ESTRY culvert within the model.

5.4.2 Figures 5.3 and 5.4 show a maximum flood depth comparison plot and maximum flood depth difference plot for the 50% blockage scenario for the 1% AEP + CC design event. Plots 5.3 and 5.4 demonstrate that when blockage is applied maximum depth and extent of surface water flooding upstream of the A303 culvert shows a slight increase with respect to the proposed scenario for the same design event. It should be noted that the integrity of the scheme is maintained, and key flow pathways are retained with respect to the proposed scenario.

5.4.3 Plots were not produced for the 25% blockage scenario as differences were minimal when compared to the proposed scenario, and application of 50% blockage provided a more conservative assessment.

5.4.4 Figure 5.5 shows hydrographs of surface water flow onto the River Till floodplain for the baseline scenario, along with the proposed scenario including potential blockages to the culvert beneath the A303. Representation of 50% blockage of the A303 culvert results in a reduction in peak flow to the Till floodplain, restricting peak flow in line with baseline representation. It should be noted that blockage only exerts an impact upon flow onto the Till floodplain around the peak of the event (5-10 hours).

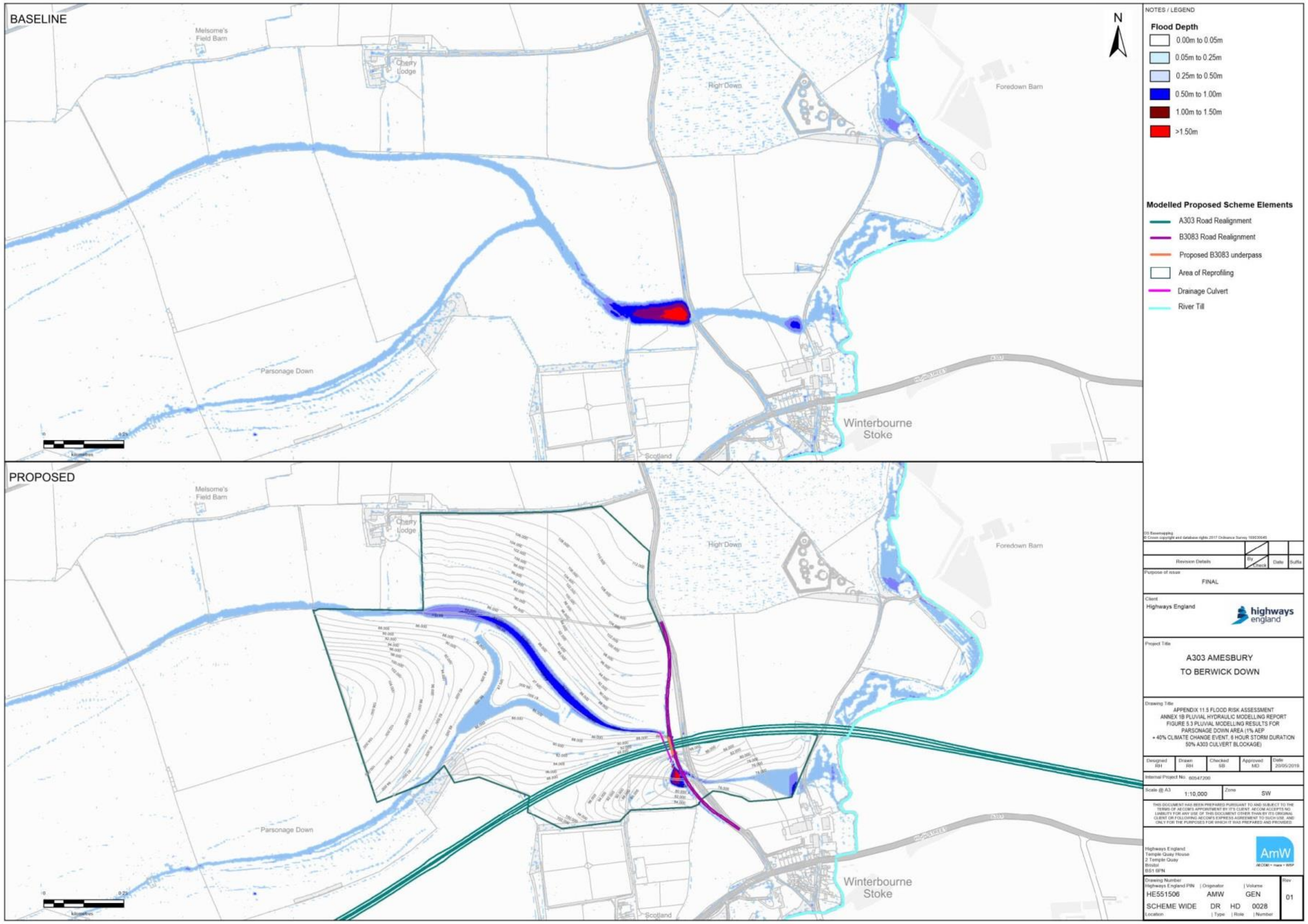


Figure 5.3 Maximum Flood Depth Comparison Plot- 1% AEP Plus Climate Change (40%) 6 Hour Storm Duration 50% Blockage Scenario

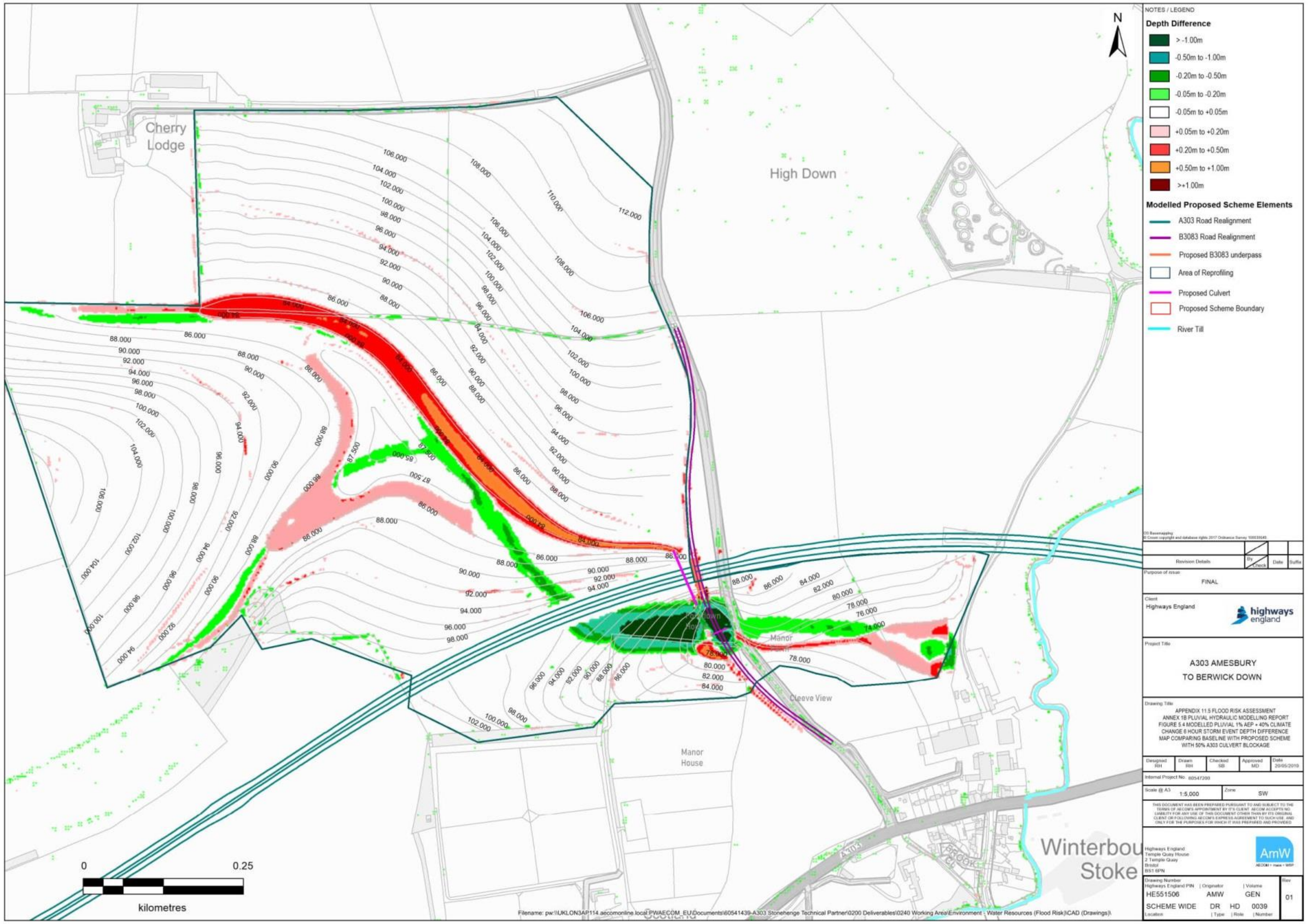


Figure 5.4 Maximum Flood Depth Difference Plot- 1% AEP Plus Climate Change (40%) 6 Hour Storm Duration 50% Blockage Scenario

5.4.5 The relatively small magnitude of impact on results can be attributed to the fact that the large diameter of the culvert beneath the A303 (1.2m), set by minimum size requirements for culverts beneath highways for access reasons, means that full capacity of the culvert is not utilised within the 1% AEP + CC event. Therefore even when partial blockage of the culvert is simulated, there is still sufficient capacity within the culvert to convey a significant volume of surface water.

5.4.6 Overall it can be concluded that 25% and 50% blockage of the culvert beneath the A303 would not be associated with detrimental impacts in terms of flood risk, should it occur. In general, blockage of the culvert leads to a small increase in maximum flood depth within Parsonage Down valley and a reduction in flow rate to the Till floodplain. No increases in flood risk to vulnerable receptors is shown within the presented blockage analysis.

5.4.7 It should be noted that the drainage system considered within this blockage analysis is designed to drain surface water during infrequent but high magnitude rainfall events, rather than a perennial watercourse.

5.4.8 Model results suggests that the A303 culvert is of a sufficient size to comfortably convey surface water flows for the 1% AEP + CC design event. Model output files demonstrate that the culvert is at ~70% capacity at the peak of the design event, suggesting it has additional unused capacity for the conveyance of flow. This is corroborated by the blockage analysis, which demonstrates that the culvert and scheme still operate effectively when a 50% blockage is applied. These results suggest that the A303 culvert would possess the capacity to convey water associated with high groundwater levels in Parsonage Down, even if this were to coincide with a high order surface water flood event.

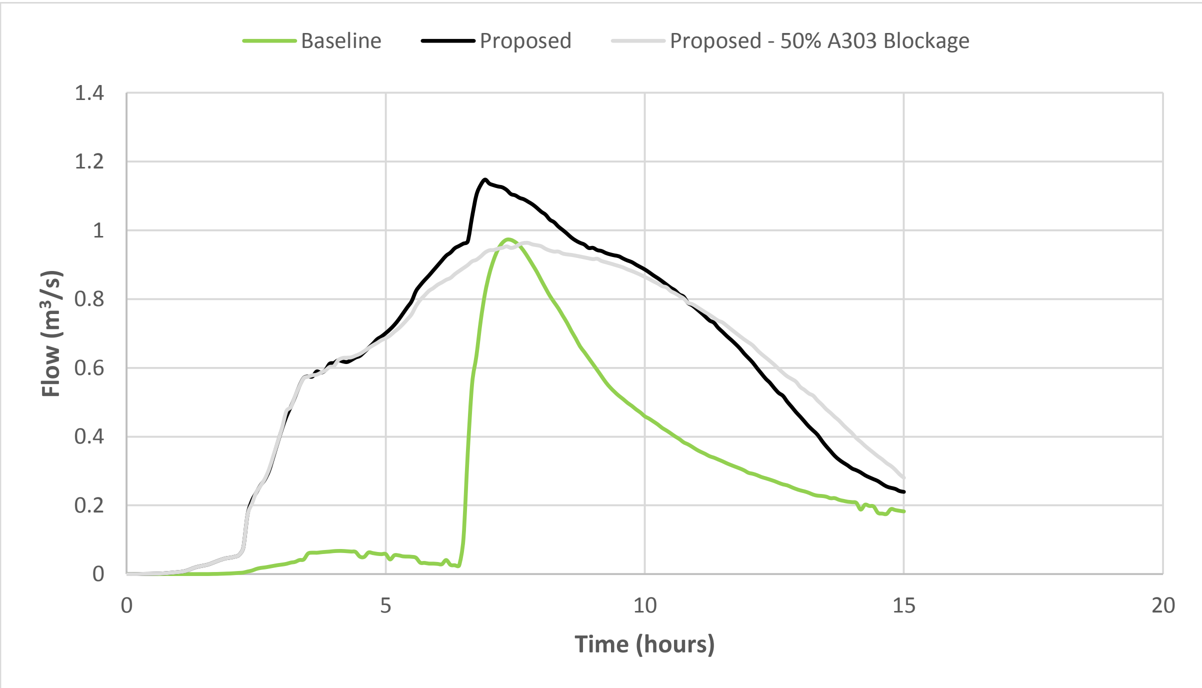


Figure 5.5 Flow onto the River Till Floodplain 1% AEP + 40% CC 6 hours-Blockage Analysis

6 Limitations

6.1 Rainfall Hydrology and Losses

6.1.1 There is currently no defined best practice guidance for the estimation of rainfall hydrology and losses, and subsequent inclusion within surface water hydraulic modelling. As such,sensitivity simulations have been undertaken in order to test the response of the model to different antecedent wetness conditions as a result of high groundwater levels (C_{ini}) and storm durations.

6.1.2 Based upon the methodology adopted and sensitivity analysis it is thought that the approach developed and adopted here represents an appropriately conservative representation of surface water flood risk within the Parsonage Down catchment.

6.1.3 The ReFH2 rainfall runoff model is associated with a number of limitations and the reader is referred to the technical guidance for ReFH2 for a more detailed account of these limitations and uncertainties[6].

6.1.4 The ReFH2 model produces a catchment wide average estimate of net rainfall. In the approach adopted within this study, this is applied uniformly across the rainfall catchment within the hydraulic model for the design events. This assumption is considered valid as the modelled rainfall catchment at Parsonage Down is predominantly rural with very limited area of pavement, buildings and hard standing.

6.1.5 For the proposed scenario model, varying infiltration rates have been applied within Parsonage Down in order to reflect the permeability of the chalk tunnel arisings, including an engineering solution to encourage infiltration. In order to reflect the modelling methodology utilised here, adjusted effective rainfall profiles to reflect the anticipated permeability were calculated based upon ReFH2 outputs. These calculations are included within Appendix D.

6.1.6 The Parsonage Down catchment is ungauged and thus there is no quantitative historic data available for calibration or verification of the hydrological model.

6.2 Hydraulic Model

6.2.1 A large source of uncertainty commonly associated with hydraulic modelling is associated with the data utilised to define floodplain topography. The composite DTM utilised here comprises a combination of EA LiDAR and high resolution photogrammetric DTM. The stated accuracy of these data sources is included within Table 5.1.

6.2.2 It should be noted that through independent ground truthing, the vertical accuracy of EA LiDAR was shown to be superior to the photogrammetric DTM, thus the vertical accuracy of the photogrammetric DTM should be regarded as lower than +/-150mm.

[6] Wallingford Hydrosolutions (2016) The Revitalised Flood Hydrograph Model ReFH2 Technical Guidance

Table 5.1 Accuracy of Topographic Data Sources

Topographical Data Source	Spatial Resolution (m)	Stated vertical Accuracy
EA LiDAR DTM	2	+/- 150 mm
High Resolution Photogrammetric DTM	1	+/- 40 mm

6.2.3 Calibration and validation of the hydraulic model was not able to be undertaken as part of the work presented in this report. This is due to a lack of appropriate historic data. Therefore there is no way to quantitatively assess the accuracy of the results of the modelling work undertaken.

6.2.4 Depth varying Manning's Roughness Coefficients have been incorporated based upon guidance from the software developers, although there is currently not an accepted industry wide set of depth varying roughness values for direct rainfall models. There is some uncertainty relating to the raised values of roughness for shallower flow depths, and associated depth thresholds, although it is considered that this offers an improvement compared to definition of standard Manning's Roughness Coefficients.

6.3 Combined Limitations

6.3.1 The primary limitation with the overall approach adopted can be attributed to the hydrological component of the TUFLOW model software. The ability to represent losses, along with interactions between surface and sub-surface flows, within the model can be attributed as a limitation. The input of effective rainfall profiles directly within the model domain, harnessing the ReFH2 hydrological model, offered an alternative approach to better capture losses within this uniform catchment.

6.3.2 The basic hydrological component present within the TUFLOW software also precludes a representation of interactions between groundwater and surface water within the hydraulic model. Such interactions are known to be significant given the permeable chalk bed rock within the Till catchment. The results presented therefore do not account directly for groundwater interactions. Consideration of groundwater is limited to the investigation of antecedent wetness in Section 2, along with the blockage analysis in Section 5.

7 Summary and Conclusions

7.1 Summary

7.1.1 Surface water flood risk through the Parsonage Down catchment has been modelled using TUFLOW for the baseline and proposed permanent scenario.

7.1.2 Rainfall hydrology has been calculated using ReFH2, producing effective rainfall profiles which have been included within the hydraulic model.

7.1.3 Design rainfall events for the 3.33% AEP, 1% AEP, 1% AEP +40% allowance for climate change, and 0.1% AEP have been modelled. Winter rainfall profiles with a critical duration of 360 minutes were simulated as this was found to be the critical storm duration for the catchment. Additional simulations were undertaken for the 12 hour storm duration as this was shown to be critical when considering flow volumes.

7.2 Conclusions

7.2.1 Overall several key elements of the proposed permanent A303 scheme design, specifically the realignment of the A303, B3083, land reprofiling and changes in permeability due to deposition of chalk tunnel arisings, have an impact upon the location and conveyance of surface water flows through Parsonage Down.

7.2.2 Inclusion of a series of culverts and drainage channels maintains connectivity between the surface water flow pathway and the River Till floodplain within the proposed scenario. Model results suggest that the drainage arrangement within this area conveys overland flows into the River Till at a similar location to the baseline scenario.

7.2.3 Overall, the modelling conducted suggests that the proposed scheme results in an increase in peak flow and volume of surface water flow into the River Till from the Parsonage Down catchment. For the 1% AEP +CC design event the increase in peak flow rate into the floodplain of the River Till is +0.17 m^3/s. It should be noted that flood depth difference maps demonstrate that the increase in surface water flow to the Till floodplain do not lead to a significant change in depth of flooding on the floodplain. This can be explained by the fact that the modest increase in flow is spread over a wide area of the Till floodplain over the course of the storm event.

7.2.4 The reasons for this increase is due to an increase in the volume of surface water generated within Parsonage Down due to the decrease in permeability associated with unstructured chalk deposition, with respect to existing infiltration rates. Whilst this is partially mitigated by the inclusion of an engineering drainage solution, to increase permeability of the deposited chalk, there is still an overall increase in the volume of surface water generated.

7.2.5 Within the proposed scenario, additional surface water from Parsonage Down is now conveyed to the River Till via a managed system of culverts and drainage channels, rather than flowing freely through the Parsonage Down valley. Nevertheless, modelling results suggest that this increase in flow from Parsonage Down does not lead to a significant change in maximum flood depths upon the Till floodplain, which is the primary receptor to surface water flow through

Parsonage Down. It should be noted that flood risk to the B3083 roadway is substantially reduced as a result of the scheme, providing functional access at all times for residents and emergency services.

Appendix A- ReFH2 Loss Model Parameters

The ReFH2.2 Technical Guide provides users with information on the relationship between optimal C_{Ini} values and BFIHOST for stations in the NRFA Peak Flows dataset that are considered suitable for Q_{Med} estimation when using the FEH13 rainfall model (see Figure A.1)

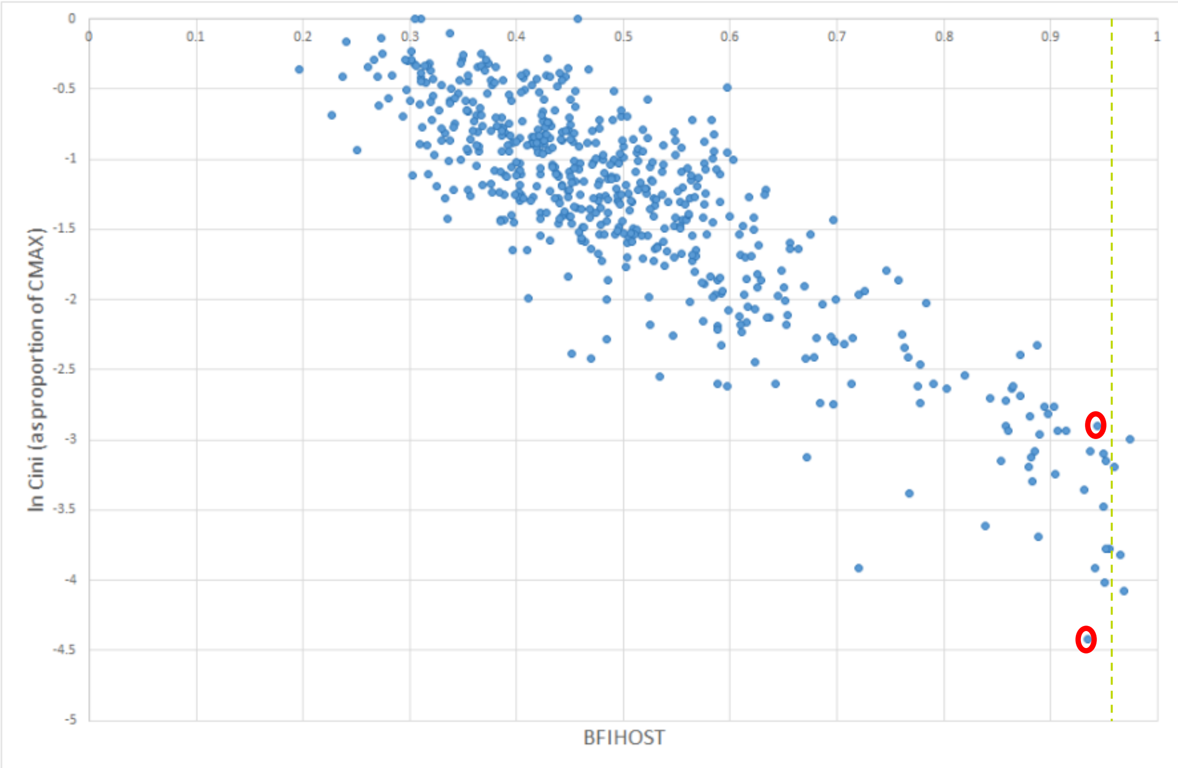
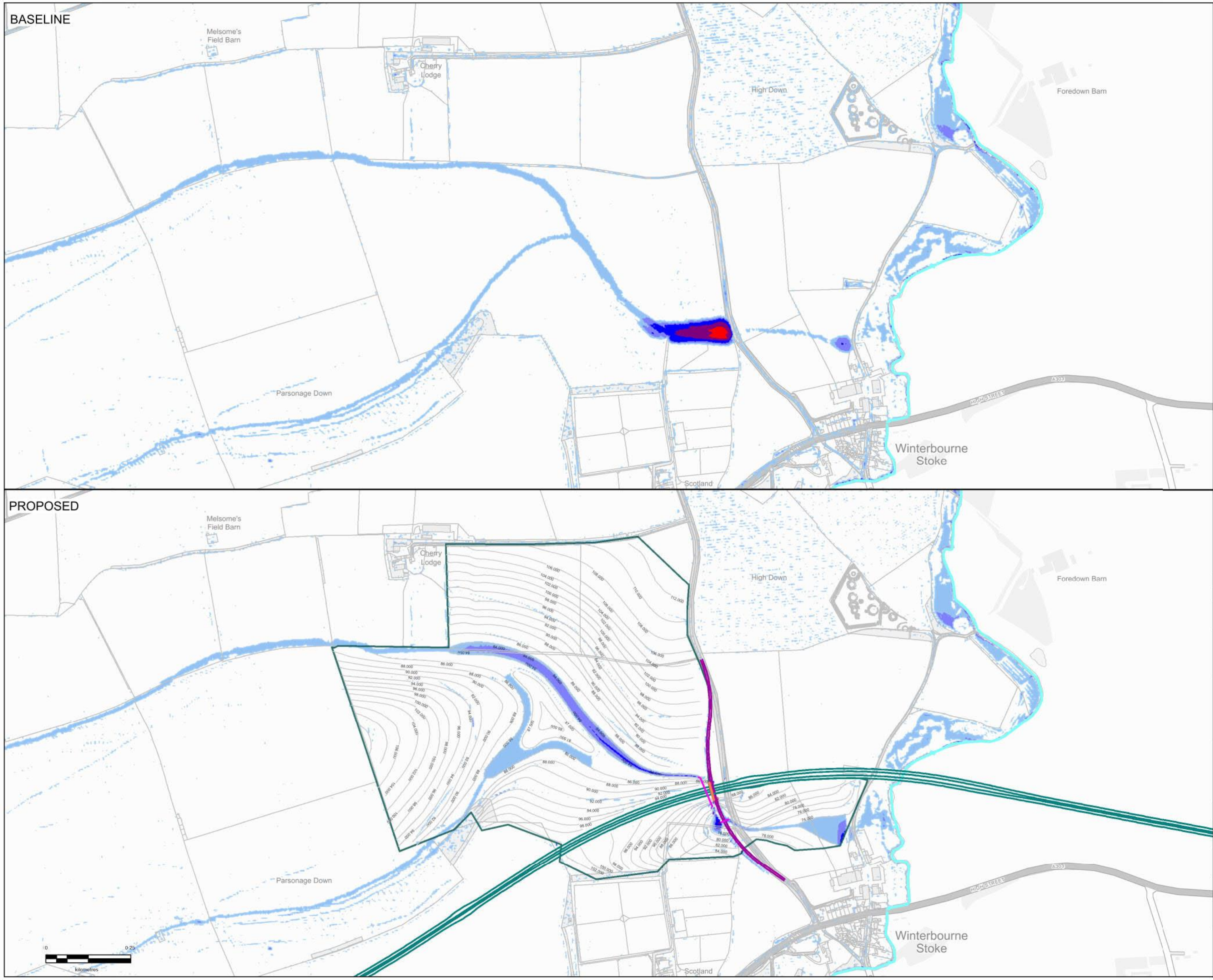


Figure 20 The relationship between the optimal Cini$_{(2)}$ and the BFIHOST value for stations from the NRFA Peak Flows dataset flagged as being suitable for Q_{MED} estimation.

Figure A.1. Reproduced from ReFH2.2 Technical Guidance (Figure 20). The dashed line illustrates the approximate BFIHOST value of the Parsonage Down catchment and the red circles indicate the upper and lower Ln C_{Ini} values used within the analysis.

Using Figure A.1, upper and lower values of Ln C_{Ini} (as a proportion of C_{Max}) have been identified for BFIHOST (0.96) in the Parsonage Down Catchment. These have then been converted using the EXP() function within Microsoft Excel and then multiplied by C_{Max} to provide upper and lower values of C_{Ini} to test the sensitivity of the surface water modelling for the Parsonage Down catchment.

Appendix B- Additional Flood Mapping



NOTES / LEGEND

Flood Depth

- 0.00m to 0.05m
- 0.05m to 0.25m
- 0.25m to 0.50m
- 0.50m to 1.00m
- 1.00m to 1.50m
- >1.50m

Modelled Proposed Scheme Elements

- A303 Road Realignment
- B3083 Road Realignment
- Proposed B3083 underpass
- Area of Reprofiling
- Drainage Culvert
- River Till

Revision Details	By / Check	Date	Suffix

Purpose of issue: FINAL

Client: Highways England

Project Title: A303 AMESBURY TO BERWICK DOWN

Drawing Title: APPENDIX 11.5 FLOOD RISK ASSESSMENT ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT APPENDIX B.1 PLUVIAL MODELLING RESULTS FOR PARSONAGE DOWN AREA (3.33% AEP)

Designed	Drawn	Checked	Approved	Date
RH	RH	SB	MD	20/05/2019

Internal Project No. 60547200

Scale @ A3: 1:10,000 | Zone: SW

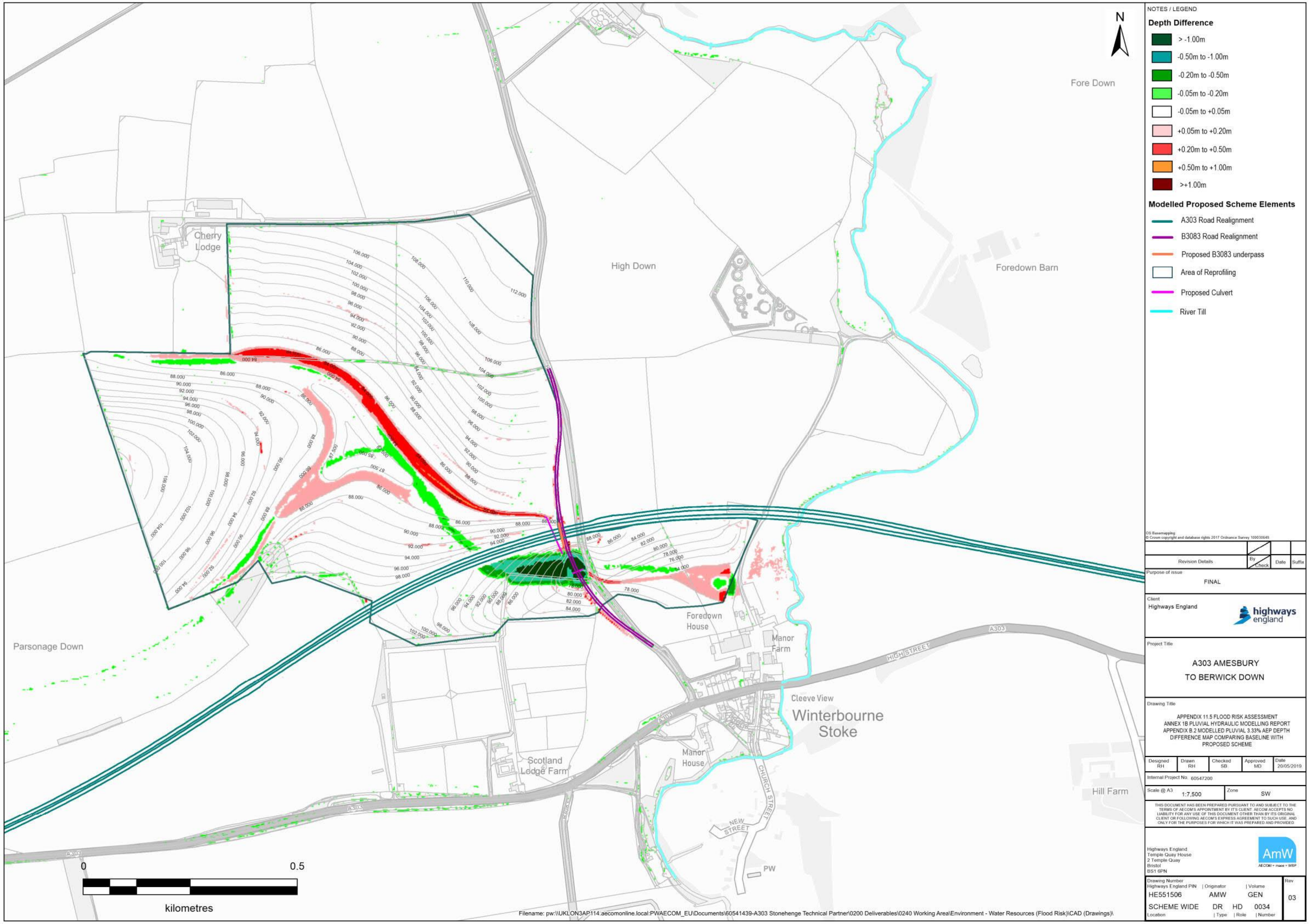
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN

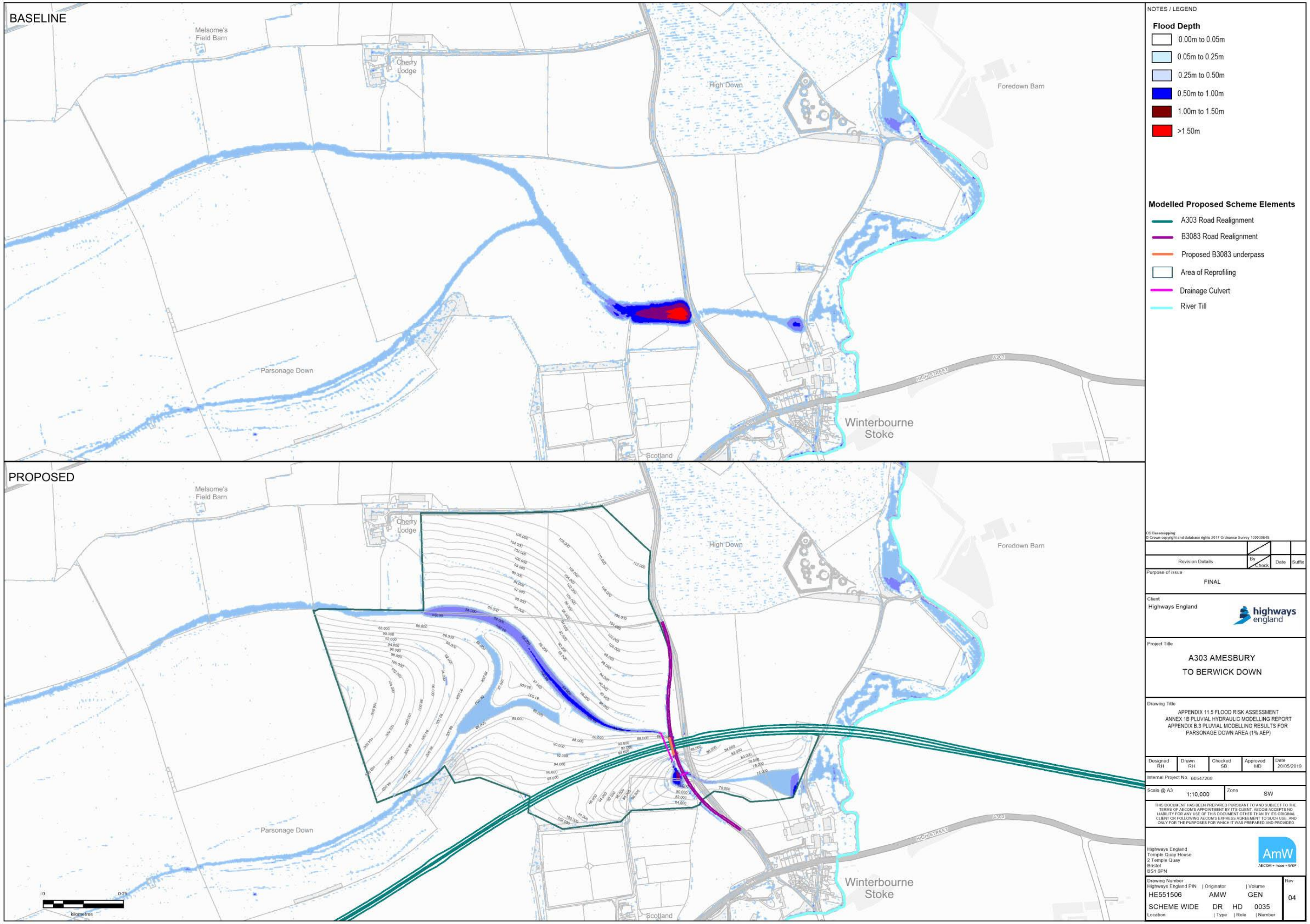
AmW
AECOM + mace + WSP

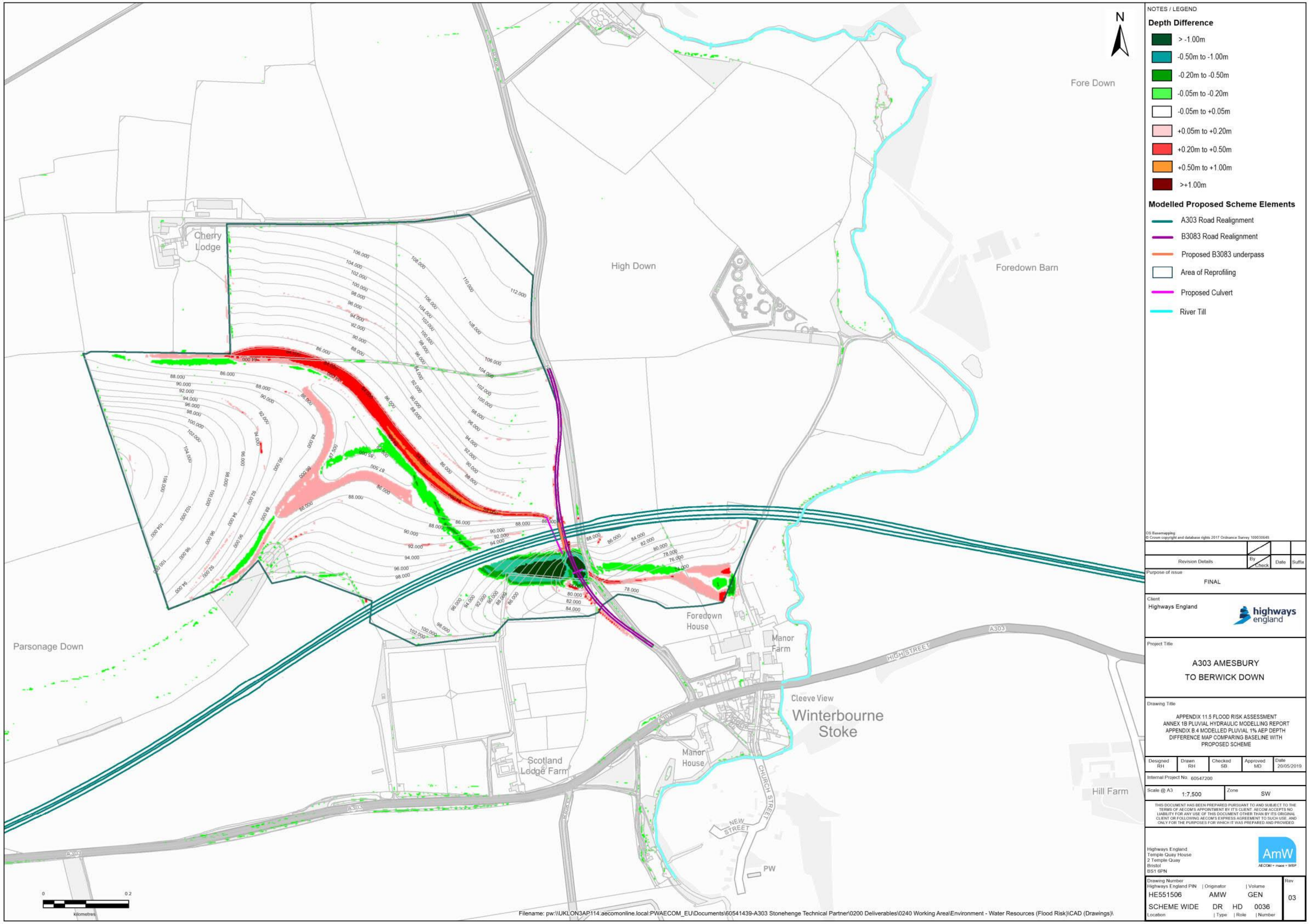
Drawing Number

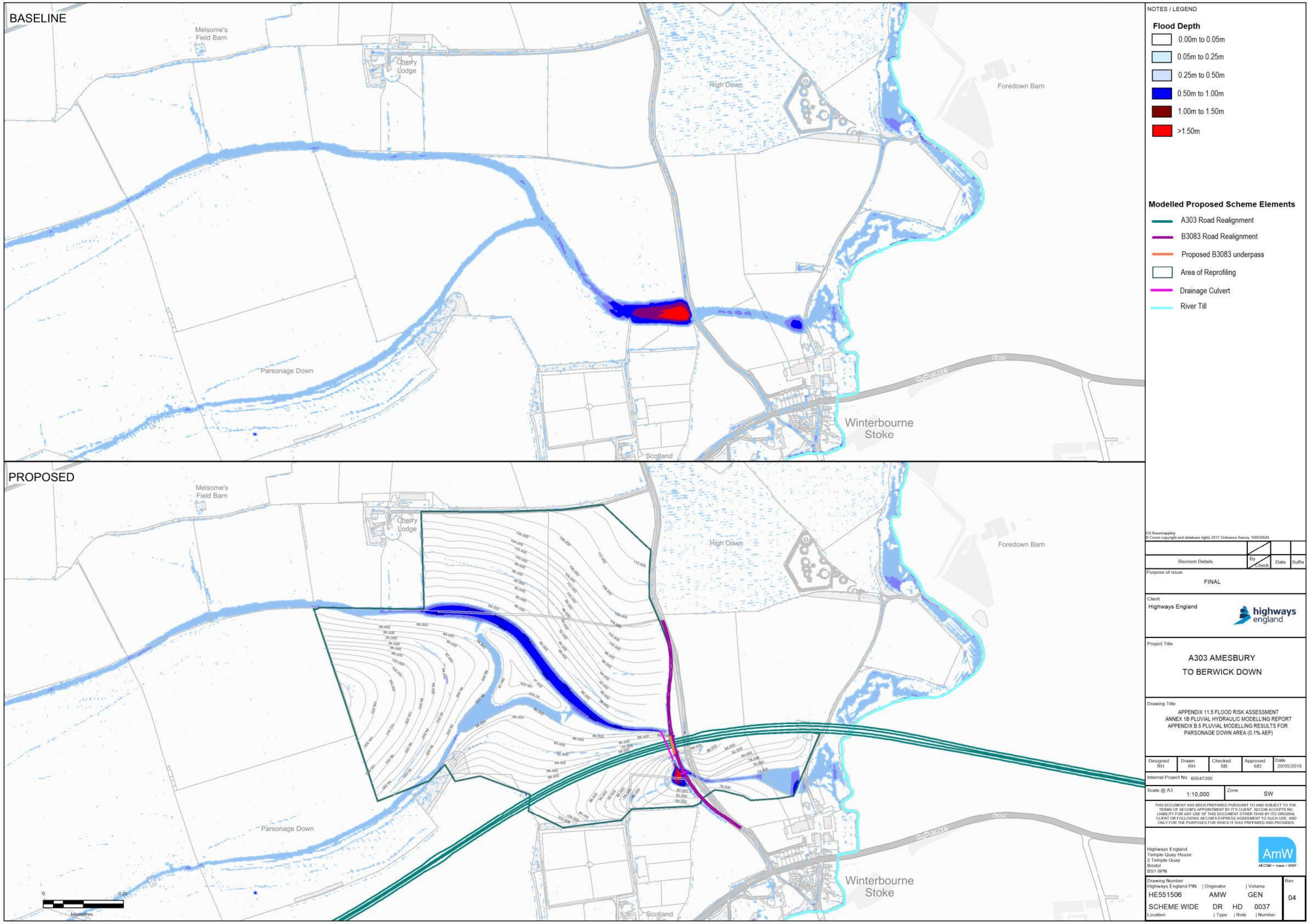
Highways England PIN	Originator	Volume	Rev
HE551506	AMW	GEN	04

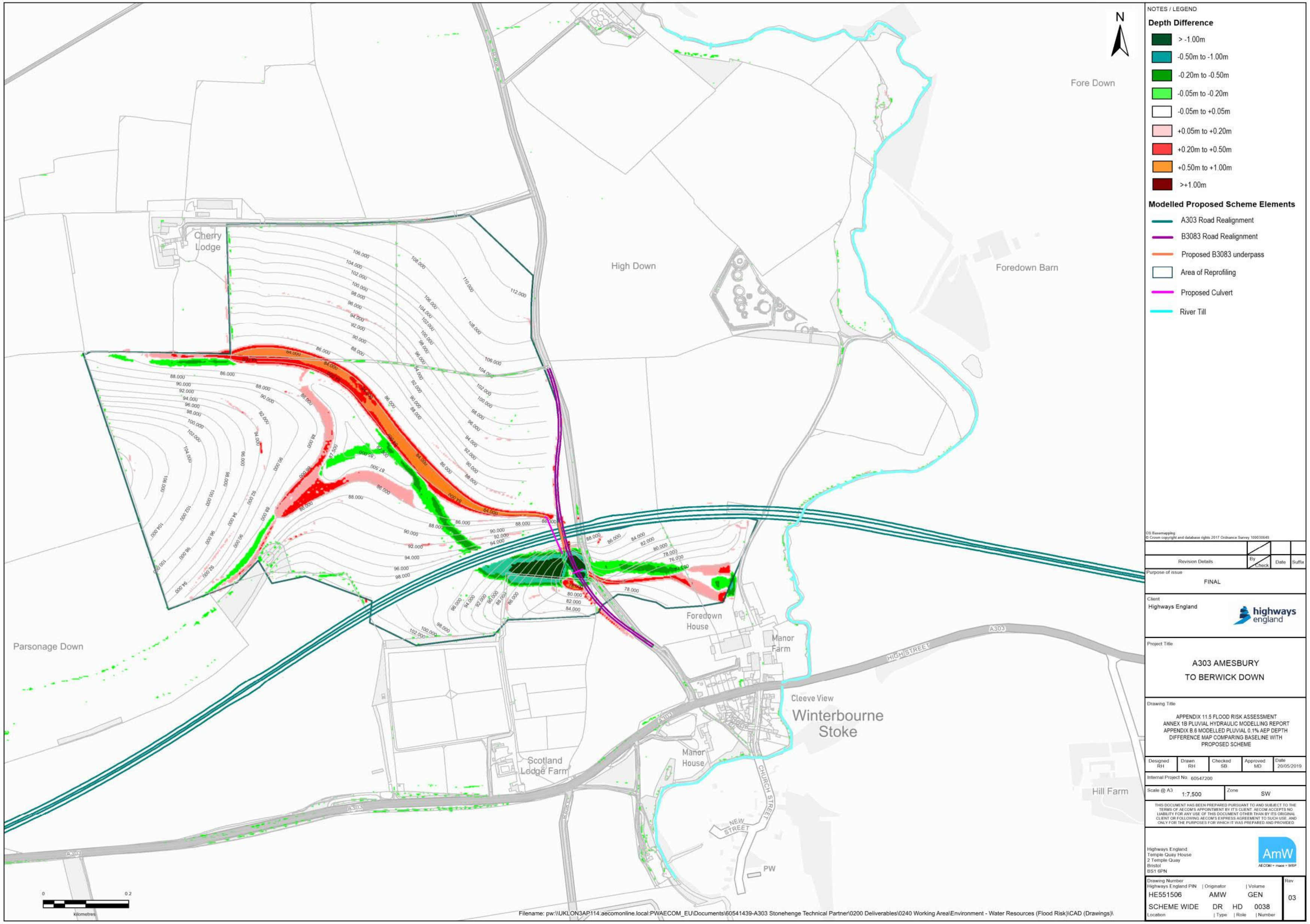
Location	Type	Role	Number
SCHEME WIDE	DR	HD	0033


N
NOTES / LEGEND
Depth Difference
> -1.00m
-0.50m to -1.00m
-0.20m to -0.50m
-0.05m to -0.20m
-0.05m to +0.05m
+0.05m to +0.20m
+0.20m to +0.50m
+0.50m to +1.00m
>+1.00m
Modelled Proposed Scheme Elements
A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Proposed Culvert
River Till
Fore Down
High Down
Foredown Barn
Cherry Lodge
Parsonage Down
Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Manor House
Scotland Lodge Farm
Hill Farm
HIGH STREET
CHURCH STREET
NEW STREET
A303
PW
0
0.5
kilometres
OS Basemapping:
© Crown copyright and database rights 2017 Ordnance Survey 100030649
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.2 MODELLED PLUVIAL 3.33% AEP DEPTH
DIFFERENCE MAP COMPARING BASELINE WITH
PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/05/2019
Internal Project No. 60547200
Scale @ A3 1:7,500
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0034
Location | Type | Role | Number
Rev
03

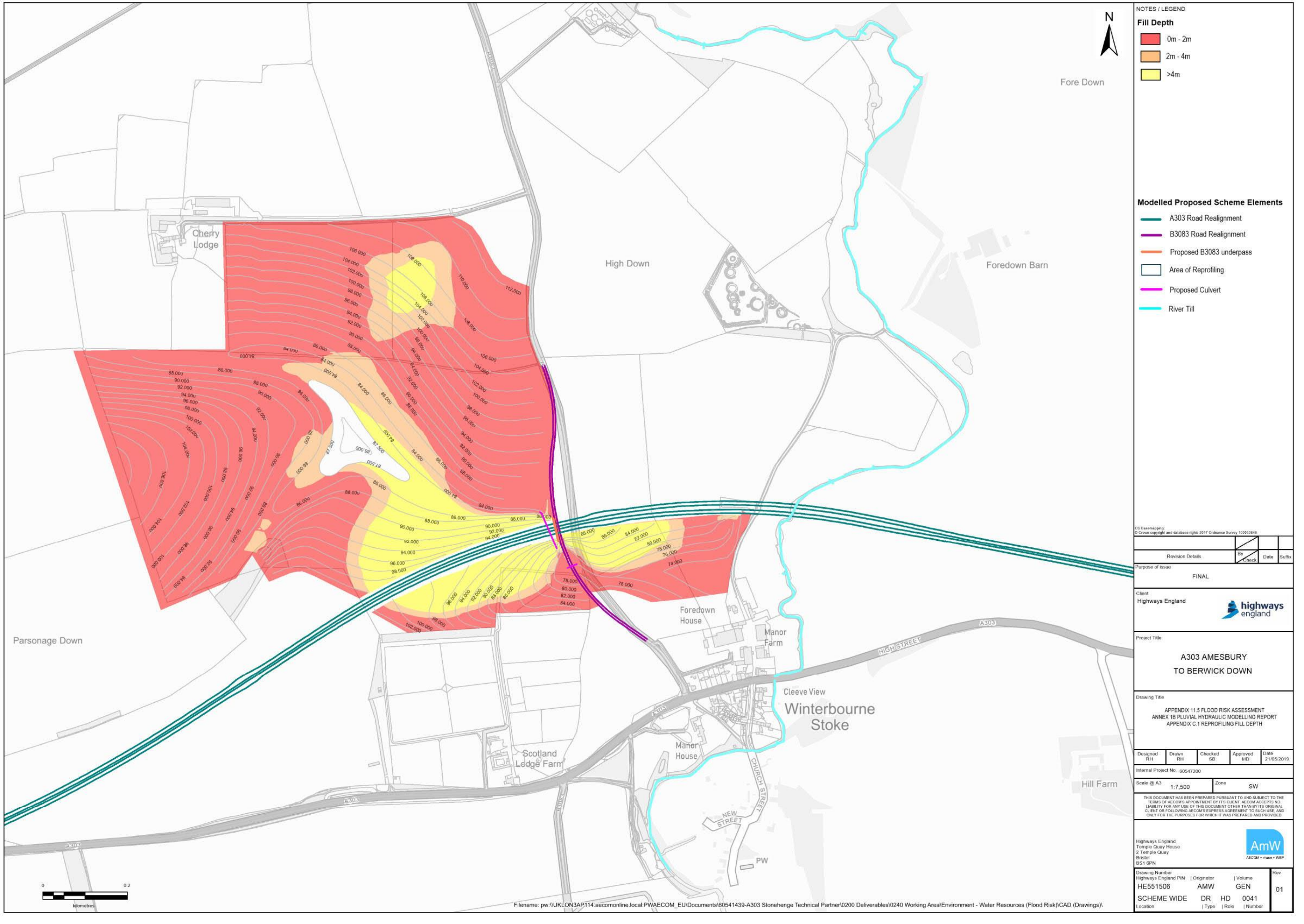

BASELINE
PROPOSED
Melsome's Field Barn
Cherry Lodge
High Down
Foredown Barn
Parsonage Down
Winterbourne Stoke
Scotland
NOTES / LEGEND
Flood Depth
0.00m to 0.05m
0.05m to 0.25m
0.25m to 0.50m
0.50m to 1.00m
1.00m to 1.50m
>1.50m
Modelled Proposed Scheme Elements
A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Drainage Culvert
River Till
OS Basemapping
© Crown copyright and database rights 2017 Ordnance Survey 100030649
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.3 PLUVIAL MODELLING RESULTS FOR
PARSONAGE DOWN AREA (1% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/05/2019
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0035
Location | Type | Role | Number
Rev
04


N
Fore Down
Cherry Lodge
High Down
Foredown Barn
Parsonage Down
Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Manor House
Scotland Lodge Farm
Hill Farm
HIGH STREET
CHURCH STREET
NEW STREET
A303
NOTES / LEGEND
Depth Difference
> -1.00m
-0.50m to -1.00m
-0.20m to -0.50m
-0.05m to -0.20m
-0.05m to +0.05m
+0.05m to +0.20m
+0.20m to +0.50m
+0.50m to +1.00m
>+1.00m
Modelled Proposed Scheme Elements
A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Proposed Culvert
River Till
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.4 MODELLED PLUVIAL 1% AEP DEPTH
DIFFERENCE MAP COMPARING BASELINE WITH
PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/05/2019
Internal Project No. 60547200
Scale @ A3 1:7,500
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0036
Location | Type | Role | Number
Rev
03
0 0.2
Kilometres


BASELINE
PROPOSED
Melsome's Field Barn
Cherry Lodge
High Down
Foredown Barn
Parsonage Down
Winterbourne Stoke
Scotland
A303
HIGH STREET
NOTES / LEGEND
Flood Depth
0.00m to 0.05m
0.05m to 0.25m
0.25m to 0.50m
0.50m to 1.00m
1.00m to 1.50m
>1.50m
Modelled Proposed Scheme Elements
A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Drainage Culvert
River Till
OS Basemapping
© Crown copyright and database rights 2017 Ordnance Survey 100030649
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.5 PLUVIAL MODELLING RESULTS FOR
PARSONAGE DOWN AREA (0.1% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/05/2019
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0037
Location | Type | Role | Number
Rev
04


N
NOTES / LEGEND
Depth Difference
> -1.00m
-0.50m to -1.00m
-0.20m to -0.50m
-0.05m to -0.20m
-0.05m to +0.05m
+0.05m to +0.20m
+0.20m to +0.50m
+0.50m to +1.00m
>+1.00m
Modelled Proposed Scheme Elements
A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Proposed Culvert
River Till
Fore Down
High Down
Foredown Barn
Cherry Lodge
Parsonage Down
Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Manor House
Scotland Lodge Farm
Hill Farm
HIGH STREET
CHURCH STREET
NEW STREET
Kilometres
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX B.6 MODELLED PLUVIAL 0.1% AEP DEPTH
DIFFERENCE MAP COMPARING BASELINE WITH
PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/05/2019
Internal Project No. 60547200
Scale @ A3 1:7,500
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY IT'S CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0038
Location | Type | Role | Number
Rev
03

Appendix C- Fill Depth Map


N
NOTES / LEGEND
Fill Depth
0m - 2m
2m - 4m
>4m
Modelled Proposed Scheme Elements
A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Proposed Culvert
River Till
Fore Down
High Down
Foredown Barn
Cherry Lodge
Parsonage Down
Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Manor House
Scotland Lodge Farm
Hill Farm
HIGH STREET
CHURCH STREET
NEW STREET
A303
PW
0
0.2
kilometres
OS Basemapping
© Crown copyright and database rights 2017 Ordnance Survey 100030649
Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
Drawing Title
APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX C.1 REPROFILING FILL DEPTH
Designed RH
Drawn RH
Checked SB
Approved MD
Date 21/05/2019
Internal Project No. 60547200
Scale @ A3 1:7,500
Zone SW
THIS DOCUMENT HAS BEEN PREPARED PURSUANT TO AND SUBJECT TO THE TERMS OF AECOM'S APPOINTMENT BY ITS CLIENT. AECOM ACCEPTS NO LIABILITY FOR ANY USE OF THIS DOCUMENT OTHER THAN BY ITS ORIGINAL CLIENT OR FOLLOWING AECOM'S EXPRESS AGREEMENT TO SUCH USE, AND ONLY FOR THE PURPOSES FOR WHICH IT WAS PREPARED AND PROVIDED.
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0041
Location | Type | Role | Number
Rev
01

Appendix D- Parsonage Down Effective Rainfall Calculations

Guide to calculations in tables below

Column 1- Time (hours)

Column 2- Design Rainfall (FEH 2013)

Column 3- Net rainfall- total rainfall less calculated losses (ReFH2), equates to effective rainfall applied to model

Column 4- Total loss- total loss calculated by ReFH2

Column 5- Loss rate- total loss divided by timestep to give a rate of loss

Column 6- 50% loss rate- calculated loss rate is halved to calculate the 50% loss rate

Column 7- Total loss (50% loss rate)- calculated loss when 50% loss rate is applied.

Column 8- Net rainfall (50% loss rate)- calculated net rainfall (effective rainfall) when 50% loss rate is applied, for application to appropriate area of Parsonage Down.

Time	30 year design rainfall - FEH 2013 model (mm)	30 year total net rainfall (mm)	Total Loss (mm)	Loss rate (mm/hr)	50% Loss rate (mm/hr)	Total Loss- 50% loss rate (mm)	30 year net rainfall with 50% Loss rate (mm)
00:00	1.046	0.058	0.988	1.482	0.741	0.494	0.552
00:40	1.962	0.111	1.851	2.777	1.388	0.926	1.036
01:20	3.648	0.214	3.434	5.151	2.576	1.717	1.931
02:00	6.664	0.416	6.248	9.371	4.686	3.124	3.540
02:40	9.951	0.683	9.268	13.902	6.951	4.634	5.317
03:20	6.664	0.499	6.165	9.248	4.624	3.083	3.581
04:00	3.648	0.287	3.361	5.041	2.521	1.680	1.968
04:40	1.962	0.159	1.803	2.705	1.353	0.902	1.060
05:20	1.046	0.086	0.960	1.440	0.720	0.480	0.566

Time	100 year design rainfall - FEH 2013 model (mm)	100 year total net rainfall (mm)	Total Loss(mm)	Loss rate (mm/hr)	50% Loss rate (mm/hr)	Total Loss- - 50% loss rate (mm)	100 year net rainfall with 50% Loss rate (mm)
00:00	1.326	0.074	1.252	1.879	0.939	0.626	0.700
00:40	2.488	0.142	2.346	3.519	1.760	1.173	1.315
01:20	4.626	0.276	4.350	6.525	3.263	2.175	2.451
02:00	8.450	0.545	7.905	11.858	5.929	3.953	4.497
02:40	12.619	0.913	11.706	17.559	8.780	5.853	6.766
03:20	8.450	0.678	7.773	11.659	5.829	3.886	4.564
04:00	4.626	0.393	4.232	6.349	3.174	2.116	2.510
04:40	2.488	0.218	2.270	3.404	1.702	1.135	1.353
05:20	1.326	0.118	1.208	1.812	0.906	0.604	0.722

Time	100 year + CC design rainfall - FEH 2013 model (mm)	100 year + CC total net rainfall (mm)	Total Loss (mm)	Loss rate (mm/hr)	50% Loss rate (mm/hr)	Total Loss- 50% loss rate (mm)	100 year + CC net rainfall with 50% Loss rate (mm)
00:00	1.856	0.103	1.753	2.630	1.315	0.877	0.980
00:40	3.483	0.198	3.285	4.927	2.464	1.642	1.841
01:20	6.476	0.386	6.090	9.135	4.568	3.045	3.431
02:00	11.830	0.763	11.068	16.601	8.301	5.534	6.296
02:40	17.666	1.278	16.389	24.583	12.292	8.194	9.472
03:20	11.830	0.949	10.882	16.322	8.161	5.441	6.389
04:00	6.476	0.551	5.925	8.888	4.444	2.963	3.513
04:40	3.483	0.305	3.178	4.766	2.383	1.589	1.894
05:20	1.856	0.165	1.691	2.536	1.268	0.845	1.011

Time	1000 year design rainfall - FEH 2013 model (mm)	1000 year total net rainfall (mm)	Total Loss (mm)	Loss rate (mm/hr)	50% Loss rate (mm/hr)	Total Loss- 50% loss rate (mm)	1000 year net rainfall with 50% Loss rate (mm)
00:00	2.222	0.124	2.098	3.147	1.573	1.049	1.173
00:40	4.169	0.243	3.926	5.889	2.944	1.963	2.206
01:20	7.751	0.486	7.265	10.898	5.449	3.633	4.119
02:00	14.159	1.003	13.156	19.734	9.867	6.578	7.581
02:40	21.145	1.776	19.368	29.052	14.526	9.684	11.461
03:20	14.159	1.376	12.783	19.175	9.588	6.392	7.768
04:00	7.751	0.817	6.935	10.402	5.201	3.467	4.284
04:40	4.169	0.458	3.711	5.567	2.783	1.856	2.313
05:20	2.222	0.249	1.973	2.959	1.479	0.986	1.236

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Level 3 Flood Risk Assessment

Annex 2A – River Till Hydrological Analysis

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

May 2019

Table of contents

Table of Figures

Table of Tables

1 Introduction

1.1 Overview

This document provides a record of the calculations and decisions made during the production of flood estimates for the River Till, Wiltshire. It is a supporting Annex to the hydraulic modelling work being undertaken for the wider A303 Amesbury to Berwick Down project.

The information provided here should enable the work to be reproduced by others in the future. It is formed of a method statement, locations where flood estimates are required, the Flood Estimation Handbook (FEH) methods used, a discussion and summary of results plus supporting information.

2 Method Statement

2.1 Overview of requirement for flood estimates

2.1.1 The purpose of the study is to provide flow estimates for use within hydraulic modelling to define Flood Zone 2 and Flood Zone 3 in accordance with the National Planning Policy Framework (Ref 1), associated practice guidance (Ref 2) and National Policy Statement (Ref 3) for National Networks. In addition, the 3.33% AEP event will be run to define the functional floodplain as described within the NPPF (Ref 1).

2.1.2 Peak flow estimates and hydrographs are required for the 3.3% AEP, 1% AEP, and 0.1% AEP events at five locations. Allowances for climate change are also required for the South West River Basin District, these are 30% (Central), 40% (Higher Central) and 85% (Upper) (Ref 4).

2.2 Overview of catchment

2.2.1 The River Till catchment is approximately 40 km^2 at the upstream boundary of the hydraulic model and 124 km^2 at the downstream boundary. The catchment is underlain by chalk (Upper Cretaceous – Upper and Middle chalk Series) with superficial deposits of sands and gravels in the valley base.

2.2.2 The watercourse is a 'winterbourne' and experiences ephemeral flows during periods of high groundwater levels, typically in the period between October and March. There is a minor tributary that joins the River Till in Shrewton and has the same winterbourne characteristics. A review of the 1:20,000 British Geological Survey (BGS) mapping (Sheets 8 and 9) indicate that the groundwater catchment coincides well with the surface water catchment.

2.2.3 The main settlements within the catchment are Tilshead, Orcheston, Shrewton and Winterbourne Stoke. These villages have no significant future development planned based on the Wiltshire Council Local Plan.

2.2.4 Figure 3-1 in the following section provides a map of the catchment, model extent and flow estimation points.

2.3 Source of flood peak data

2.3.1 Version 6 (released in February 2018) of the National River Flow Archive (NRFA) Peak Flows dataset has been used.

2.4 Flood History

2.4.1 A range of sources have been used to identify the flood history in the River Till catchment. These include:

- Journal papers;
- BHS Chronology of British Hydrological Events;
- Information provided by the Environment Agency and Wiltshire Council that includes reports, photos and other information;
- Internet searches including newspaper articles, photos and planning applications.

2.4.2 Annex B provides a full list of the flood history within the River Till catchment. Based on the flood history, a combination of sources including fluvial, surface water and groundwater sources are the primary mechanisms of flooding within the catchment. An exceptional event in 1841 (the Great Till Flood) is attributed to a combination of snow melt, frozen ground and rainfall. However, this mechanism of flooding is not considered as a primary source when compared with fluvial, surface water and groundwater.

2.5 Gauging stations (flow or level)

2.5.1 There are no gauging stations on the River Till. Potential donor sites from neighbouring catchments are discussed in Section 4.

2.6 Other available data

2.6.1 A range of additional data has been obtained to further support information for flow estimation. These are variable in quality and a summary has been provided in Table 2-1.

Table 2-1: Summary of additional data available

Type of data	Data relevant to this study	Data available	Source of data	Details
Check flow gaugings (if planned rating review)	n/a	n/a	n/a	n/a
Historic flood data	Yes	Yes	Internet, Met Office Library, Wiltshire Council, Environment Agency	A range of historic flood information is available, in particular, the 1841 'Great Till Flood'. Whilst some data provides the date of flooding, observations are limited with little information on the mechanisms, flow, extent and timing of flooding. These are summarised in the 'Flood History' in Annex B.
Flow data for events	No	No	n/a	There are no flow gauges within the River Till catchment.
Rainfall data for events	Yes	Yes	Environment Agency, Met Office	A range of daily and sub-daily data are available for stations within and around the catchment.
Results from previous studies	Yes	Yes	Journal, Internet, Wiltshire Council	Flow estimation for the 1841 flood. Flow estimation for a number of studies (Tilshead Flood Alleviation Reservoir, 2017; River Till Flood Alleviation Works, 1996; Flood Risk Assessment for Karrick House, 2007)
Other information e.g. groundwater, tides etc	Yes	Yes	Environment Agency	Groundwater monitoring levels at Tilshead, groundwater emergence chainage (indicates location of emergence over a period of years), regional groundwater model outputs.

2.7 Initial choice of approach

2.7.1 The FEH statistical method is normally the most appropriate method on highly permeable catchments according to the Environment Agency Flood Estimation Guideline (2017) (Ref 5).

Conceptual model

2.7.2 The main site of interest is the proposed open span bridge structure to the north of Winterbourne Stoke and the potential impacts this may exert on flood extents upstream and downstream of the structure.

2.7.3 Due to the ephemeral nature of the winterbourne, the emergence of flow within the river channel varies depending on the time of year and underlying groundwater levels. The catchment is highly permeable and catchment wetness influences runoff and flow within the channel. The primary likely cause of flooding within the catchment is groundwater with prolonged periods of elevated flows (i.e. flood volume). There is also the potential for a high rainfall event to result in flooding when combined with high groundwater levels (i.e. catchment is saturated and therefore catchment reacts like an impermeable catchment).

2.7.4 The historic flood of 1841 was attributed to a combination of cold weather, snowmelt and heavy rainfall. Whilst flooding of this type is noted, this historic event was within the ‘Little Ice Age’ period circa 1300 – 1850 AD where climatic conditions do not reflect the current conditions of milder, wetter winters. The flood record is not considered to be stationary and the use of earlier records should not be used to assess present day flooding. Furthermore, a review of the Met Office ‘Days of Snow Lying’ annual average for the period 1961 to 1990 against the period 1981 to 2010 indicates that there is a decrease in snow lying days. The River Till catchment receives 5 to 10 days of snow lying on average and this is likely to decrease with climate change based on Kay (2016) (Ref 6).

2.7.5 The likelihood of the coincidence of significant snow depths combined with heavy rainfall and frozen ground is considered to be very low and not considered further within this analysis.

Unusual catchment features

2.7.6 The catchment is highly permeable with BFIHOST values all greater than 0.96 at the flow estimation points.

2.7.7 SPRHOST is less than 20%, however, this is only relevant to the stations within the WINFAP pooling group because there is no gauge within the River Till catchment.

2.7.8 WINFAP v4 doesn’t allow user defined values of L-CV and L-SKEW to be entered following permeable adjustment. An alternative approach of removing of ‘non-flood’ years (QMED less than QMED/2) from the AMAX series for stations within the pooling group with an SPRHOST less than 20% will be undertaken to compare with the unadjusted pooling group. This approach is a compromise on the permeable adjustment procedure described within FEH although its application has minor effects on the growth curve factors (similar to the permeable adjustment procedure).

2.7.9 The catchment is not highly urbanised (largest value of URBEXT2000 is 0.0051 at the downstream boundary) and there is no significant development planned in the future.

2.7.10 The catchment is not influenced by pumping, reservoirs or extensive floodplain storage. It is noted that a flood alleviation scheme is planned to the north of Tilshead to manage flows from the West Down area (to the north east of Tilshead). These are unlikely to impact flows estimates at the point of interest in Winterbourne Stoke.

Initial choice of method and reasons

2.7.11 A range of QMED methods have been assessed (see Section 4.5) to identify the preferred method. The FEH statistical pooling group method has been selected to obtain peak flow estimates. These peak flow estimates will be used to scale hydrographs derived using ReFH2.2 software to provide inflows to the hydraulic model.

2.7.12 Flow estimates using ReFH2.2 have also been undertaken to provide an independent comparison with the FEH statistical values and also generate design hydrographs to scale final flow estimates.

2.7.13 WINFAPv4 and ReFH2.2 software versions have been used in this study.

3 Location of flood estimates

3.1 Summary of subject sites

3.1.1 Table 3-1 lists the locations of subject sites that are illustrated in Figure 3-1. There are no major inflows on the River Till apart from a minor tributary at Shrewton (S01). Subject sites, T01 and S01 are model inflow locations with T02, T03 and T04 used as check locations and to distribute intervening flows. T03 is located at the existing crossing of the A303 at Winterbourne Stoke and used as a check on combined flows from T02 and S01.

Table 3-1: Summary of subject sites.

Site Code	Watercourse	Site	Easting	Northing	Area on FEH web service (km^2)	Revised area if altered
T01	Till	Upstream model extent at Tilshead	403450	147650	39.54	Not amended
T02	Till	At confluence with unnamed tributary in Shrewton	406850	144100	72.89	Not amended
S01	Unnamed tributary from east of Shrewton	At confluence with River Till in Shrewton	406500	143950	15.97	Not amended
T03	Till	Winterbourne Stoke at existing A303 crossing	407800	141200	113.99	Not amended
T04	Till	Downstream model extent	407100	138900	123.73	Not amended

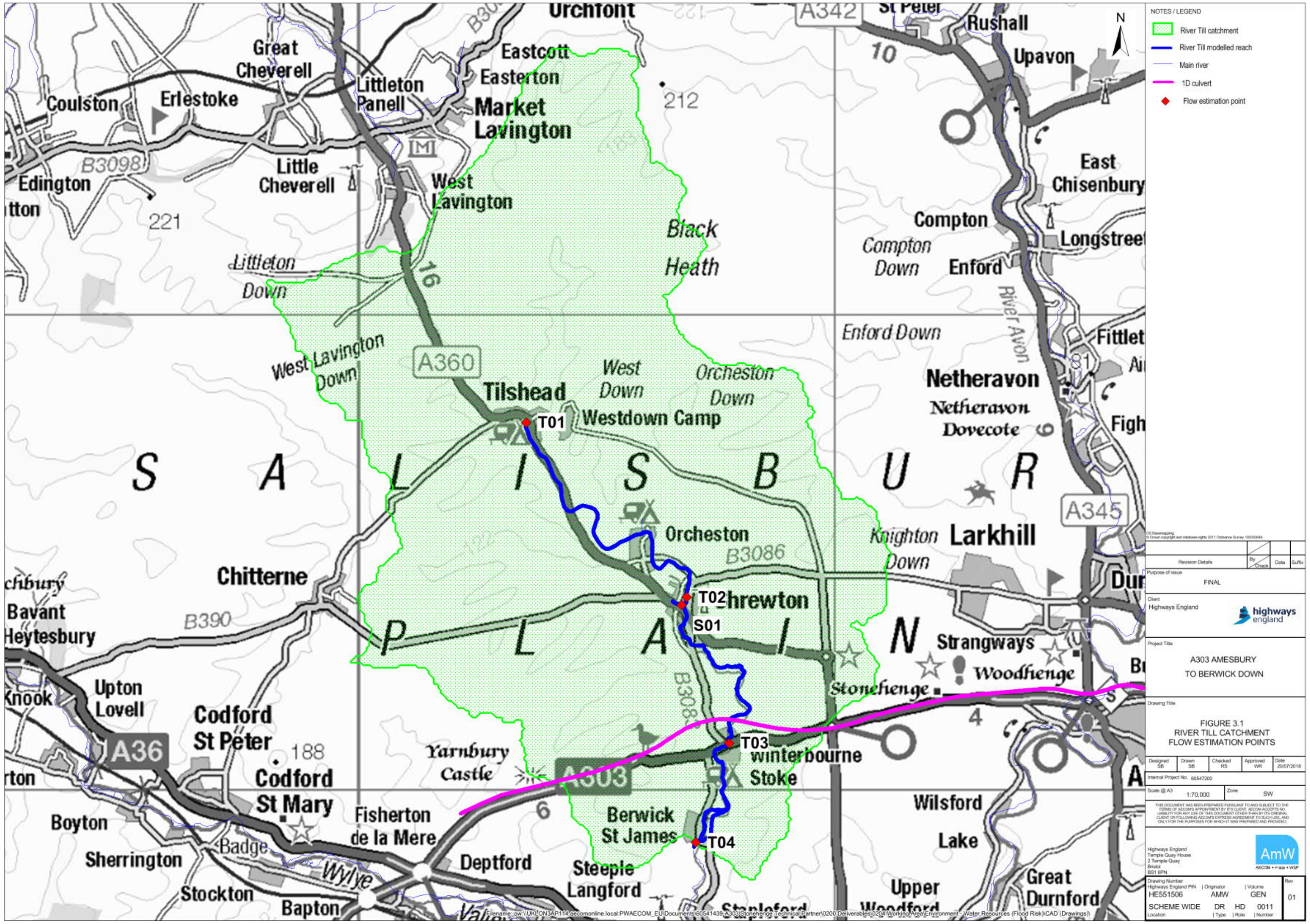


Figure 3-1: Flow estimation points

3.2 Subject site catchment descriptors

3.2.1 Table 3-2 lists the key catchment descriptors for each of the subject sites, these remain unchanged based on the following review commentary.

3.2.2 The catchment boundaries were checked through visual inspection against OS 1:25,000 mapping. These correspond well to OS mapping and therefore no amendments were made to catchment areas.

3.2.3 Soils were checked through inspection of Soilscapes (http://www.landis.org.uk/soilscapes/), these are identified as shallow lime rich over chalk across the majority of the catchment. Within the valley base, soils are freely draining lime rich loamy soils. Thin soils and chalk were noted during a site visit in October 2017. In addition, the underlying bedrock and superficial deposits correspond well with overlying soil type based on an inspection of the BGS Geology of Britain (http://mapapps.bgs.ac.uk/geologyofbritain/home.html).

Table 3-2: Important catchment descriptors at subject sites.

Site Code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT2000	FPEXT
T01	1.00	0.35	0.966	5.49	56.7	751	5.03	0.0021	0.0289
T02	1.00	0.35	0.967	9.75	50.7	748	4.97	0.0042	0.0372
S01	1.00	0.35	0.963	4.73	49.2	775	5.30	0.0002	0.0320
T03	1.00	0.35	0.965	11.34	49.5	752	5.10	0.0046	0.0371
T04	0.99	0.35	0.965	13.67	50.8	754	5.12	0.0051	0.0377

3.2.4 URBEXT2000 values from the FEH web service have been used. The catchment is not heavily urbanised and whilst minor adjustment could be mode to extents in Tilshead and Shrewton, these are unlikely to impact flow estimates or flows at the point of interest.

3.2.5 Whilst the catchments are not considered to be urbanised with the largest URBEXT2000 value of 0.0051 (T04), the Environment Agency Flood Estimation Guidelines (2017) recommend carrying out an urban adjustment for all QMED estimates to avoid a discontinuity even when URBEXT2000 is equal or less than 0.03.

3.2.6 WINFAP v4 adjusts both QMED (using the UAF) and L-moments (L-CV and L-Skew) within the software. UAF ranges between a minimum of 1.001 (S01) and 1.037 (T04) therefore resulting in an increase in QMED at all locations. The change in L-CV and L-Skew is minimal when applying urbanisation with growth curve factors decreasing by a maximum of 0.003 (see Section 4 for further analysis).

4 FEH Statistical Method

4.1 Review of potential QMED donor sites

4.1.1 There are no level or flow gauges within the catchment or model reach. Potential donor sites have been identified and are provided in Table 4-1. Further information on the data available and rating equations for the donor sites are provided in Table 4-2 and Table 4-3. All donor stations identified are within neighbouring catchments and are within the wider Hampshire Avon catchment of which the River Till is a sub-catchment.

4.1.2 In terms of catchment area, stations 43801, 43014 and 43017 are considered most suitable when compared with the main site of interest (T03).

4.1.3 With regard to BFIHOST, Station 43801 is preferable when comparing this parameter, which was confirmed from comparison with BGS geological and hydrogeological mapping. Stations 43014 and 43017 are considered less suitable as donors when comparing BFIHOST, with lower values, due the differing geology and large areas of 'moderate permeability' bedrock when viewing the NRFA catchment information.

4.1.4 All catchments are considered suitable when comparing FARL and when comparing URBEXT2000 all are considered to be 'essentially rural'.

4.1.5 Flood peak data for station 43801 has been reviewed because the NRFA highlights large periods of missing data. This is potentially associated with low flows experienced within the catchment, therefore below the gauged limit. Comparison of the AMAX series against data for stations on the River Wylye indicates that the timings of AMAX at 43801 are comparable and therefore considered a reasonable representation of the flood series.

4.1.6 Depending on the point of interest, additional donors have been identified within WINFAP v4. Whilst the headwaters of 53002 (Semington Brook @ Semington) are located to the north of Tilshead, the catchment drains into the Bristol Avon and has a 'sharp' response to rainfall due to the Kimmeridge and Gault Clay underlying large areas of the catchment. The BFIHOST is 0.564 and therefore not representative of the River Till catchment.

4.1.7 Station 43003 (Avon @ East Mills) has been identified as a potential donor due to the relative locations of the catchment centroids. However, the catchment area for 43003 is 1459 km2 and is greater than 10x the area of the catchments being investigated. This catchment is not considered comparable due to the significant difference in catchment area and the likely differences in flood response.

4.1.8 Additional checks on QMED are being undertaken using groundwater emergence data from the regional groundwater model. These have been used to estimate daily mean flows on the River Till and assess the flows exceeded 5% and 10% of the time from the flow duration curve. This allows the 'Flow variability' function within WINFAP v4 to be utilised (QMED Linking Equation). In addition, this allows a sensibility check when comparing to the QMED value from catchment descriptors (see Annex C for further details).

Table 4-1: Local gauging stations

Watercourse	Station Name	NRFA number	Grid Reference	Catchment area (km^2)	BFIHOST	FPEXT	URBEXT2000
Chitterne Brook	Codford	43801	ST970401	69.7	0.974	0.0246	0.0008
East Avon	Upavon	43014	SU133559	85.8	0.838	0.0700	0.0117
West Avon	Upavon	43017	SU133559	84.6	0.872	0.1188	0.0112
Avon	Amesbury	43005	SU151413	323.7 (326.5*)	0.903	0.0710	0.0132
Wylye	Stockton Park	43024	ST975393	254.8	0.925	n/a	n/a
Wylye	South Newton	43008	SU086342	445.4	0.937	0.0518	0.0102
Bourne	Laverstock	43004	SU156303	163.6	0.952	0.0561	0.0237

* catchment area in brackets from FEH catchment descriptors and differs slightly area provided by NRFA.

4.2 Data available at each flow gauging station

4.2.1 Table 4-2 provides a summary of the data available for each of the potential donor sites from neighbouring catchments.

Table 4-2: Data availability at local gauging stations

Station Name	Start and end date on NRFA	Updated for this study?	Suitable for QMED?	Suitable for pooing?	Data quality check needed?	Other comments on station and flow data quality e.g. information from NRFA Peak Flows, trends in peaks, outliers.
Codford	Jan 1972 to present	No	Yes	No	Yes	Whilst NRFA indicates start date as 1972, peak flow (AMAX) data is only available from 1993 onwards. There are large periods of missing data in early record (up to 1998). There are 'non' flood years within the record (AMAX < QMED/2). Refer to Station Info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43801
Upavon (East	Jan 1970 to	No	Yes	Yes	No	No missing data according to NRFA,

Station Name	Start and end date on NRFA	Updated for this study?	Suitable for QMED?	Suitable for pooing?	Data quality check needed?	Other comments on station and flow data quality e.g. information from NRFA Peak Flows, trends in peaks, outliers.
Avon)	present					long period of record and gauged above QMED (within 29% of AMAX3). Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43014
Upavon (West Avon)	Jan 1970 to present	No	Yes	No	Yes	No missing data according to NRFA, long period of record and gauged to within 17% of QMED. However, rating not validated beyond QMED due to too few high flow gaugings. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43017
Amesbury	Jan 1965 to present	No	Yes	Yes	No	Long period of record and station measures over the full range of flows with no bypassing or out of bank flow. Gauged beyond AMAX3. Small amount of data missing over period of record (73 days in total). Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43005
Stockton Park	May 1994 to present	No	No	No	No	This station is not within the HiFlows dataset and information is only available for daily mean flows. This hasn't been used further. within the analysis. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43024
South Newton	Jan 1966 to	No	Yes	Yes	Yes	Long period of record and gauged above

Station Name	Start and end date on NRFA	Updated for this study?	Suitable for QMED?	Suitable for pooing?	Data quality check needed?	Other comments on station and flow data quality e.g. information from NRFA Peak Flows, trends in peaks, outliers.
	present					QMED and AMAX3. Data between 1986 and 1991 missing but no explanatory notes on NRFA. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43008
Laverstock	Oct 1964 to present	No	Yes	Yes	No	Long period of record and gauged above QMED and AMAX3. Data between 1984 and 1992 missing but no explanatory notes on NRFA. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43004.

4.3 Rating equations

4.3.1 Whilst commentary on rating equations has been provided in Table 4-3, for the purposes of this study, a detailed review of existing rating equations does not form part of the required deliverables for this project.

Table 4-3: Summary of information on 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 rating
Codford	Theoretical rating. Upper limit of rating is above QMED. Extrapolated beyond stage of 0.80 m.	No	Note: few spot flow gaugings, none are above QMED. Weir drowns at stage of 0.44 m but no significant bypassing. Two ratings have been applied over period of record, however, these are the same on NRFA notes.
Upavon (East Avon)	Theoretical rating. Upper limit of rating is above QMED. Extrapolated beyond stage of 0.73 m.	No	Note: few spot flow gaugings available but gauged to within 29% of AMAX3.

Upavon (West Avon)	Theoretical rating. Upper limit of rating is below QMED. Extrapolated beyond stage of 0.4 m.	No	Note: few high flow gaugings available and rating only validated to QMED (gauged to within 17% of QMED).
Amesbury	Empirical rating, extrapolated beyond stage of 1 m. Re-rated in 2001 to include exceptional event in December 2000. Environment Agency is very confident in stage/discharge relationship.	No	Note: large range of spot flow gaugings across full range of flow and above AMAX3.
Stockton Park	Unavailable on NRFA	No	This station is not within the HiFlows dataset and information is only available for daily mean flows.
South Newton	Empirical rating, extrapolated based on flood gaugings.	No	Note: large range of spot flow gaugings across full range of flow and above AMAX3.
Laverstock	Theoretical rating, re-calibrated at low flows. Upper limit of rating is above QMED. Extrapolated beyond upper limit of rating at 0.8 m.	No	Note: large range of spot flow gaugings across full range of flow and above AMAX3.

4.4 Selected donor sites

4.4.1 Table 4-4 provides an overview of the selected donor site for adjusting QMED from catchment descriptors based on the discussion in Section 4.1.

Table 4-4: Selected donor sites

NRFA Number	Reasons for choosing or rejecting	Method (AMAX or POT)	Adjusted for climatic variation?	QMED from flow data (gauged) (m^3s^{-1}) (A)	QMED from flow data – urban influence removed (m^3s^{-1})*	$QMED_{CDs}$ (m^3s^{-1}) (B)	Adjustment Ratio (A/B)
43801	See comments in Section 4.1	AMAX	No	3.19	3.17	1.59	1.99

* This was undertaken within WINFAPv4.

4.4.2 The urban adjustment approach within WINFAPv4 has been applied to QMED estimates.

4.5 Estimation of QMED at subject sites

4.5.1 Two methods of estimating QMED were undertaken, these were QMED adjusted by donor transfer and a variation on the 'Flow variability' (QMED Linking Equation) method available within WINFAPv4.

QMED donor transfer method

4.5.2 As identified in Section 4.4, data transfer using donor site 43801has been undertaken. This procedure is fully explained in Science Report SC050050 (Ref 7). The QMED adjustment ratio A/B as provided in Table 4-4 is moderated using a power term, 'a', which is a function of the distance between the centroids of the subject site catchment and the donor catchment. The final estimate of QMED is (A/B) a multiplied by the initial estimate from catchment descriptors. As only a single donor has been used, no weights have been applied to the moderation term.

4.5.3 The donor adjusted QMED values are provided in Table 4-5. QMED has been adjusted for urbanisation as per the Environment Agency Flood Estimation Guidelines (2017) (Ref 5).It is noted that caution should be taken when adjusting for urbanisation in permeable catchments. However, the changes to QMED in general are less than 0.1 m3 s-1 and are therefore not considered significant.

Table 4-5: Adjusted QMED values using data transfer using full AMAX series at Station 43801

Site Code	$QMED_{CDs}$ (m^3s^{-1}) (rural)	Method	Donor site NRFA number	Distance between centroids (km)	Moderated adjustment factor (*a*)	If more than one donor used		Final estimate of $QMED_{CDs}$ (rural)	Final estimate of $QMED_{CDs}$ (urban)
						Weight if WINFAPv4 method not used	Weighted average of moderated adjustment factor (*a*)		
T01	0.886	DT	43801	8.00	0.404	n/a	n/a	1.11	1.131
T02	1.467	DT	43801	8.46	0.398	n/a	n/a	1.84	1.896
S01	0.450	DT	43801	7.54	0.410	n/a	n/a	0.57	0.569
T03	2.201	DT	43801	8.76	0.394	n/a	n/a	2.75	2.845
T04	2.367	DT	43801	8.95	0.392	n/a	n/a	2.96	3.068

4.5.4 The values of QMED are consistent at successive points and increase in a downstream direction. The sum of the flows for T02 and S01 are less than flow at T03, therefore allowing for intervening flows between Shrewton and Winterbourne Stoke.

QMED flow variability method

4.5.5 As the River Till is ungauged and heavily influenced by groundwater flows, a novel approach using outputs from the Wessex Regional Groundwater Model has been utilised. Outputs from the groundwater model have been used to create and assess the flow duration curve statistics for flows at or exceeding 5% (Q5) and 10% (Q10) of the time at T01, T02, T03 and T04 on the River Till. These have then been used to estimate QMED using the ‘Catchment Descriptors and Flow Variability’ function within WINFAPv4. The results of this method are provided in Table 4-6 and further information on the approach, justification and limitations are provided in Annex C.

Table 4-6: Parameter values and QMED estimates using flow variability method

Site Code	Q5 (m^3s^{-1})	Q10 (m^3s^{-1})	BFI	$QMED_{FV}$ (m^3s^{-1}) (rural)	$QMED_{FV}$ (m^3s^{-1}) (urban)
T01	0.097	0.018	0.966	0.78	0.80
T02	1.299	0.793	0.967	3.36	3.36
T03	2.439	1.650	0.965	4.59	4.74
T04	3.275	2.370	0.965	5.35	5.55

4.6 Discussion on QMED

4.6.1 Two approaches have been applied, as described in the sections above, to estimate QMED. The first approach utilises donor transfer from a local site in a neighbouring catchment to improve QMED estimates from catchment descriptors.

4.6.2 The influence of using a donor site reduces the Factori**al** Standard Error (F.S.E) when compared to solely using catchment descriptors (Ref 7). The reduction in F.S.E for each site is illustrated in the following tables for the 68% confidence interval (Table 4-7) and 95% confidence interval (Table 4-8) for ‘as rural’ estimates**.**

Table 4-7: F.S.E – 68% confidence interval

Site Code	$QMED_{CDs}$ (m^3s^{-1})	F.S.E ($QMED_{CDs}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})		$QMED_{Adj}$ (m^3s^{-1})	F.S.E ($QMED_{Adj}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})
T01	0.89	1.431	0.62	1.27		1.11	1.388	0.80	1.55
T02	1.47	1.431	1.03	2.10		1.84	1.390	1.32	2.55
S01	0.45	1.431	0.31	0.64		0.57	1.387	0.41	0.79
T03	2.20	1.431	1.54	3.15		2.75	1.390	1.98	3.83
T04	2.37	1.431	1.65	3.39		2.96	1.391	2.13	4.11

Table 4-8: F.S.E – 95% confidence interval

Site Code	$QMED_{CDs}$ (m^3s^{-1})	F.S.E ($QMED_{CDs}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})		$QMED_{Adj}$ (m^3s^{-1})	F.S.E ($QMED_{Adj}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})
T01	0.89	1.431	0.43	1.81		1.11	1.388	0.58	2.15
T02	1.47	1.431	0.72	3.00		1.84	1.390	0.95	3.55
S01	0.45	1.431	0.22	0.92		0.57	1.387	0.30	1.09
T03	2.20	1.431	1.07	4.51		2.75	1.390	1.42	5.32
T04	2.37	1.431	1.16	4.85		2.96	1.391	1.53	5.72

The second approach utilises an adapt approach using ‘flow variability’ through use of groundwater model outputs (see Annex C for further information). Whilst the application of this adapted approach does have limitations, it is noted that the QMED estimate provided in Table 4-6 fall between 68% and 95% upper confidence intervals for QMED from catchment descriptors adjusted by donor transfer (see Table 4-7 and Table 4-8).

For impermeable catchments, QMED is typically considered to be equivalent to bankfull level where the channel has adapted to the hydrological regime (Ref 8). For baseflow dominated hydrologic regimes (i.e. permeable catchments), channels typically adjust to rarer floods (20% AEP to 10% AEP).Initial model runs of QMED from both methods were undertaken to assess levels and flood extents. Flows in general remained in channel for both QMED estimates with some floodplain inundation.

4.6.3 Whilst acknowledging the limitation of using data from the regional groundwater model, QMED estimates for inflows (and distributed inflows) on the River Till have been based on the adapted flow variability approach.

4.7 Derivation of pooling groups

4.7.1 Pooling groups were created for each subject site in WINFAPv4 using an URBEXT2000 threshold value of 0.03 and minimum record length of 500 years of station data.

4.7.2 The Heterogeneity statistic (H2) for each pooling group was assessed using WINFAPv4. This provides an indication of whether a review of the pooling group is required (no, optional, desirable or essential). The similarity of the subject site against stations within the pooling group is assessed by the Similarity Distance Measure (SDM) and is a function of Area, SAAR, FARL an FPEXT. However, it is noted that this has limitations when estimating growth curves on permeable catchments (Ref 9) therefore a review of the pooling groups has been undertaken. The composition of the initial and revised pooling groups is provided in the Annex A.

4.7.3 As per the Environment Agency guidelines, modifications to the pooling group tend to have a relatively minor effect on the final design flow (compared with, for example, the selection of donor sites for QMED). In particular, 'Section 6.7. – Example: a pooling group' in Science Report SC0500505 (Ref 9) indicates that apart from the first four or five stations within a pooling group (i.e. lowest SDM), the record length at a station will only have a modest effect its weight within the pooling group (unless the record is very short). The review of the pooling group has therefore mainly focused on the first five stations within each pooling group unless others have been identified that potentially require review. The review of stations is provided in Table 4-9.

Table 4-9: Review of stations from initial pooling groups

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
T01	T01	No	Sites Investigated 39033 – Winterbourne Stream @ Bagnor **RETAIN** - SDM is closest to subject site. - Chalk dominated catchment with a high BFIHOST similar to subject catchment. - Long period of record (54 years) covering flood rich and flood poor episodes. - AMAX1 is +7 times greater than QMED. This is associated with surface water runoff contributions in July 2007 event.

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			- Single Site Growth curve is steep, however subject site is likely to show similar response if groundwater flows are high and coupled with exceptional rainfall. 24007 – Browney @ Lanchester **REMOVE** - BFIHOST is 0.33 and dis-similar in underlying geology. - Hydrographs are prominently peaked and often multi-peaked. - Period of record is 1968 – 1983 (15 AMAX in total) and is considered to be in a 'Flood Poor' period of record (Ref 10 and Ref 11). 26803 - Water Forlornes @ Driffield **RETAIN** - Chalk dominated catchment with a high BFIHOST similar to subject catchment. - AMAX series covers a 'Flood Rich' period (1997 onwards). - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (6.81). 28058 - Henmore Brook @ Ashbourne **REMOVE** - 12 years of usable record but coincides with a 'Flood Poor' period of record (1970s) - Large period of record rejected following construction of Carsington Reservoir - Responsive catchment 53017 - Boyd @ Bitton **RETAIN** - Long period of record (43 years) covering flood rich and flood poor episodes. - BFIHOST is 0.49 and clay catchment. Decided to retain because may mimic flow response at subject site when ground is saturated. 44003 - Asker @ Bridport **RETAIN** - BFIHOST is 0.696. - Station replaced by 44011 (channel modifications but in same location). 44011 – Asker @ East Bridge Bridport **RETAIN** - BFIHOST 0.696

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			- Period of record from 1996 onwards covering 'flood rich' episodes. Station replaced 44003 (see above).
T02	T02	No	Sites Investigated 20007 - Gifford Water @ Lennoxlove **RETAIN** - Long period of record (43 years) covering flood rich and flood poor episodes. - BFIHOST is 0.53. Inspection of geological mapping on NRFA indicates large areas of high permeability in the lower catchment with lower permeability in the headwaters. - Whilst geographically a long way from the subject site (Scotland), catchment area, SAAR, FARL and FPEXT are similar. The site isn't discordant and lies within the central area of the L-moments graph. 42008 - Cheriton Stream @ Sewards Bridge **RETAIN** - Long period of record (46 years) covering flood rich and flood poor episodes. - Chalk dominated catchment with a high BFIHOST similar to subject catchment. Influenced by groundwater and is ephemeral in upper reaches. Surface water runoff can produce minor hydrograph spikes on top of underlying groundwater dominated flows. - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (6.89). 20005 - Birns Water @ Saltoun Hall **RETAIN** - Long period of record (45 years) covering flood rich and flood poor episodes. - BFIHOST is 0.54. Inspection of geological mapping on NRFA indicates large areas of high permeability in the lower catchment with lower permeability In the headwaters. - Whilst geographically a long way from the subject site (Scotland), catchment area, SAAR, FARL and FPEXT are similar. The site isn't discordant and lies within the central area of the L-moments graph. 51001 – Doniford Stream @ Swill Bridge **RETAIN** - Long period of record (50 years) covering flood rich and flood poor

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			episodes. - BFIHOST is 0.63, therefore close to being considered permeable (BFIHOST > 0.65 = permeable) - AMAX1 is +4.75 times greater than QMED and exhibits steepest growth curve within the pool. AMAX is a verified flood event that affected large parts of Somerset in July1968. - Although flashy response, decided to retain because provides analogous flow response at subject site when ground is saturated. 42006 – Meon @Mislingford **RETAIN** - Long period of record (57 years) covering flood rich and flood poor episodes. - Chalk dominated catchment with a high BFIHOST similar to subject catchment. Influenced by groundwater. Lower and Middle Chalk causes a more flashy response when compared with neighbouring chalk catchments. - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (5.54).
S01	S01	No	<u>Sites Investigated</u> 26802 – Gypsey Race @ Kirby Grindalythe **RETAIN** - Short period of record (17 years) but covers a flood rich episode (2000 onwards). - Chalk dominated catchment with a high BFIHOST similar to subject catchment. Groundwater dominated flow regime and similar catchment descriptors to subject site. - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (5.67). 25019 – Leven @ Easby **RETAIN** - Medium period of record (38 years) covering both flood rich and flood poor episodes. - Steep growth curve due to large peak in 1976 (AMAX1). 27010 – Hodge Beck @ Bransdale Weir **RETAIN**

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			- Station closed in 1978 although has a long period of record (41 years) covering flood rich and flood poor episodes. - Whilst catchment is not representative of the study catchment, in particular only medium to low permeability bedrock geology (BFIHOST 0.34), the site is not discordant and fits well with others within the pool. No reason to exclude. 49005 – Bolingey Stream @ Bolingey Cocks Bridge **REMOVE** - Short record of 6 years. 44008 – South Winterbourne @ Winterbourne Steepleton **RETAIN** - Small chalk dominated catchment with a high BFIHOST similar to subject catchment and dominated by groundwater flows. - Medium period of record (37 years) covering both flood rich and flood poor episodes. - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (19.53). 28058 - Henmore Brook @ Ashbourne **REMOVE** - 12 years of usable record but coincides with a 'Flood Poor' period of record (1970s) - Large period of record rejected following construction of Carsington Reservoir - Responsive catchment 24007 – Browney @ Lanchester **REMOVE** - BFIHOST is 0.33 and dis-similar in underlying geology. - Hydrographs are prominently peaked and often multi-peaked. - Period of record is 1968 – 1983 (15 AMAX in total) and is considered to be in a 'Flood Poor' period of record The following Stations have been **removed** due to SAAR values being significantly greater than the subject site (SAAR = 775 mm): - 47022 Tory Brook @ Newnham Park – SAAR = 1403 mm - 49006 Camel @ Camelford – SAAR = 1418 mm

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			- 27032 Hebden Beck @ Hebden – SAAR = 1433 mm - 73015 Keer @ High Keer Weir – SAAR = 1158 mm - 25011 Langdon Beck @ Langdon – SAAR = 1463 mm These stations have been **replaced** with stations that have more appropriate SAAR values and are: - 20002 West Peffer Burn @ Luffness - 28041 Hamps @ Waterhouses - 49004 Gannel @ Gwills - 39033 Winterbourne Stream @ Bagnor
T03	T03	No	<u>Sites Investigated</u> 21016 – Eye Water @ Eyemouth Mill **RETAIN** - Long period of record (39 years) covering flood rich and flood poor episodes. - BFIHOST is 0.60. - SAAR, FARL and FPEXT are similar. The site isn’t discordant and lies within the main cluster of the L-moments graphs. 39208 – Dun @ Hungerford **RETAIN** - Long period of record (48 years) covering flood rich and flood poor episodes. - Chalk dominated catchment with a high BFIHOST similar to subject catchment. - Single site analysis growth curve is relatively flat with GCF value of 2 for 1% AEP. When comparing AMAX1 to QMED (i.e. QAMAX1/QMED), this is approximately 2. - Retained based on catchment similarities with subject site. 53028 – By Brook @ Middlehill **RETAIN** - Moderate period of record (35 years) covering flood rich and flood poor episodes. - BFIHOST is 0.42. Inspection of geological mapping on NRFA indicates large areas of high permeability bedrock across catchment

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			(approx. 97%) with slowly permeable soils in river valleys. - Single site analysis growth curve is relatively flat with GCF value of 1.6 for 1% AEP. When comparing AMAX1 to QMED (i.e. QAMAX1/QMED) over the 35 year record, the ratio is approximately 1.36. - Whilst BFIHOST suggests that the catchment is relatively impermeable, the low growth curve factor and inspection of underlying bedrock geology on NRFA suggest that the site exhibits flow characteristics of a permeable catchment, therefore retained. 39020 – Coln @ Bibury **RETAIN** - Long period of record (53 years) covering flood rich and flood poor episodes. - Baseflow dominated catchment with a high BFIHOST and similar land uses to subject catchment. - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (5.54). 20005 - Birns Water @ Saltoun Hall **RETAIN** - Long period of record (45 years) covering flood rich and flood poor episodes. - BFIHOST is 0.54. inspection of geological mapping on NRFA indicates large areas of high permeability in the lower catchment with lower permeability In the headwaters. - Whilst geographically a long way from the subject site (Scotland), catchment area, SAAR, FARL and FPEXT are similar. The site isn't discordant and lies within the central area of the L-moments graph. 27055 – Rye @ Broadway Foot **RETAIN** - AMAX1 is an exceptional event that occurred in June 2005. This is a considerable outlier within the AMAX series, however, NRFA indicates that robust hydraulic modelling has been used to estimate the peak flow. - The effect of AMAX1 is a strongly skewed single site growth curve. However, due to the location within the pooling group and the period

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			of record (38 years), the influence of this station on the pooled growth curve factor if retained or removed is likely to be minimal.
T04	T04	No	Sites Investigated 21016 – Eye Water @ Eyemouth Mill **RETAIN** - Long period of record (39 years) covering flood rich and flood poor episodes. - BFIHOST is 0.60. - SAAR, FARL and FPEXT are similar. The site isn't discordant and lies within the main cluster of the L-moments graphs. 39020 – Coln @ Bibury **RETAIN** - Long period of record (53 years) covering flood rich and flood poor episodes. - Baseflow dominated catchment with a high BFIHOST and similar land uses to subject catchment. - Check on non-flood years to be undertaken as identified as a permeable catchment based SPRHOST (5.54). 39208 – Dun @ Hungerford **RETAIN** - Long period of record (48 years) covering flood rich and flood poor episodes. - Chalk dominated catchment with a high BFIHOST similar to subject catchment. - Single site analysis growth curve is relatively flat with GCF value of 2 for 1% AEP. When comparing AMAX1 to QMED (i.e. QAMAX1/QMED), this is approximately 2. - Retained based on catchment similarities with subject site. 53028 – By Brook @ Middlehill **RETAIN** - Moderate period of record (35 years) covering flood rich and flood poor episodes. - BFIHOST is 0.73. Inspection of geological mapping on NRFA indicates large areas of high permeability bedrock across catchment (approx. 97%) with slowly permeable soils in river valleys.

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			- Single site analysis growth curve is relatively flat with GCF value of 1.6 for 1% AEP. When comparing AMAX1 to QMED (i.e. QAMAX1/QMED) over the 35 year record, the ratio is approximately 1.36. 33018 – Tove @ Cappenham Bridge **RETAIN** - Long period of record (53 years) covering flood rich and flood poor episodes. - Predominantly chalk catchment although low BFIHOST (0.36) due to overlying boulder clay. 27055 – Rye @ Broadway Foot **RETAIN** - AMAX1 is an exceptional event that occurred in June 2005. This is a considerable outlier within the AMAX series, however, NRFA indicates that robust hydraulic modelling has been used to estimate the peak flow. - The effect of AMAX1 is a strongly skewed single site growth curve. However, due to the location within the pooling group and the period of record (38 years), the influence of this station on the pooled growth curve factor if retained or removed is likely to be minimal.

4.8 Derivation of flood growth curves at subject sites

4.8.1 The revised pooling groups for each subject site were updated and the Goodness of Fit statistic used within WINFAPv4 to identify the best fitting distribution. Table 4-10 provides a summary of the main factors used in derivation of the growth curves for each subject site.

Table 4-10: Main factors for derivation of growth curves

Site Code	Method (SS, P, ESS, FH)	If P, ESS or FH, name of pooling group	Distribution used and reason for choice	Notes on urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustment	Growth Curce Factor (GCF) for 1% AEP
T01	Pooled	T01	GEV Distribution – Whilst GL distribution is recommended for UK catchments, this distribution fitted best to the pooling group.	Adjusted for urbanisation using WINFAPv4. No permeable adjustment undertaken (see assumptions section)	Location =0.829 Scale =0.466 Shape =-0.009	3.018
T02	Pooled	T02	GL Distribution – GL Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Adjusted for urbanisation using WINFAPv4. No permeable adjustment undertaken (see assumptions section)	Location = 1.00 Scale =0.282 Shape =-0.208	3.173
S01	Pooled	S01	GL Distribution – GL Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Adjusted for urbanisation using WINFAPv4. No permeable adjustment undertaken (see assumptions section)	Location = 1.00 Scale = 0.298 Shape = -0.229	3.426
T03	Pooled	T03	GL Distribution – GL Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Adjusted for urbanisation using WINFAPv4. No permeable adjustment made as only one station considered as being permeable (39020) but only has one ‘non-flood’ year within AMAX series.	Location = 1.00 Scale = 0.260 Shape = -0.176	2.840
T04	Pooled	T04	GL Distribution – GL Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Adjusted for urbanisation using WINFAPv4. No permeable adjustment made as only one station considered as being permeable (39020) but only has one ‘non-flood’ year within AMAX series.	Location =1.00 Scale =0.271 Shape =-0.182	2.947

4.9 Flood estimates from statistical method

4.9.1 For sites on the River Till, QMED estimates using urbanised results from the flow variability method have been applied. For the minor tributary at Shrewton (S01), QMED has been estimated from donor adjusted catchment descriptors as groundwater modelling output s in this location have not been assessed. Flood estimates are provided in Table 4-11 and have been rounded to three significant figures.

Table 4-11: Peak flood estimates (m^3s^{-1}) for a range of AEP's using FEH statistical method

Site Code	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
T01	0.80	1.22	1.50	1.78	1.93	2.13	2.29	2.40	3.30
T02	3.36	4.88	6.00	7.20	7.98	9.04	9.95	10.7	18.0
S01	0.45	0.67	0.84	1.02	1.13	1.30	1.44	1.55	2.72
T03	4.74	6.68	8.05	9.50	10.4	11.6	12.7	13.5	21.4
T04	5.55	7.93	9.62	11.4	12.5	14.1	15.4	16.4	26.4

5 Revitalised flood hydrograph method (ReFH2)

5.1 Parameters for ReFH2 model

5.1.1 The values reported within this section have been estimated using the ReFH2.2 software. These flow estimates have utilised the FEH13 rainfall model and therefore provide an independent comparison against flow estimates derived from the FEH statistical pooling method.

Table 5-1: Parameter values used within ReFH2

Site Code	Method OPT: Optimisation BR: base flow recession fitting CD: catchment descriptors DT: data transfer	Tp (hours) – Time to peak	C_{max} (mm) – Maximum storage capacity	BL (hours) – Base flow lag	BR – Base flow recharge
T01	CD	4.66	1362	71.15	2.67
T02	CD	6.71	1366	80.70	2.67
S01	CD	4.48	1351	68.74	2.66
T03	CD	7.37	1359	82.29	2.67
T04	CD	8.13	1359	86.76	2.67

5.1.2 There are no flow or level gauges on the River Till, therefore no flood event analysis has been undertaken. However, accounts of historic flooding indicate that for the Great Till Flood (Ref 12) on 16th January 1841:

1. Antecedent conditions appear to have been a significant factor in the flooding mechanism. According to Cross (1967) (Ref 12) the autumn of 1840 was wet and early December had a long severe spell of frost and snow. The cold weather returned on 4th January 1841 with heavy snow on 9th January, there was further frost and snow between the 12th and 15th January. A rapid thaw was accompanied by heavy rain on the 16th January causing widespread damage and loss of life in the Till valley.

2. The duration of out-of-bank flows was approximately 12 hours in Shrewton/Maddington based on newspaper reports (Ref 13). Anecdotal reports suggest that flows increased from approximately 3pm with the peak of the flood event in Maddington occurred

between approximately 8 pm – 10 pm with roads being dry again by 3 am. This suggests the time to peak was approximately 5 to 7 hours.

5.1.3 Whilst this provides an indication of catchment response, it is noted that the time to peak and duration of events are partially dependent on the antecedent conditions within the catchment.

5.1.4 Reliance cannot be placed on a single event for estimating time to peak or duration. In particular, this was a catastrophic flood that caused loss of life (three people) and destroyed 72 homes and made approximately 200 peoples homeless. Clark (2003) (Ref 14) provides cautionary remarks regarding historic flood frequency analysis and indicates where floods have been largely been caused by runoff from frozen ground and snowmelt combined with recent climatic warming that the flood record is unlikely to be stationary. Furthermore, it indicates that for the purposes of prediction the earlier record should not be used.

5.1.5 Flooding can occur from a range of sources within the River Till catchment and are likely to be a combination of groundwater, surface water and fluvial contributions. The duration of flood events that coincide with high groundwater levels and low intensity rainfall such as in 2013/14 may cause prolonged flooding over extended periods (i.e. weeks rather than hours).

5.2 Design events for ReFH2 method

5.2.1 Table 5-2 provides general information on the ReFH2 design events. The catchment is predominately rural with the exception of the settlements of Tilshead, Orcheston, Maddington, Shrewton and Winterbourne Stoke. No amendments have been made to the urbanisation model parameters because there has been no significant development or planned future development that is likely to significantly impact flooding.

Table 5-2: Design event information

Site Code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area at subject site)	Source of design rainfall (FEH13 or FEH99)
T01	Winter	8.5	Catchment area	FEH13
T02	Winter	11.0	Catchment area	FEH13
S01	Winter	7.5	Catchment area	FEH13
T03	Winter	13.0	Catchment area	FEH13
T04	Winter	15.0	Catchment area	FEH13

5.2.2 It should be noted that summer storms typically produce a 'flashy' response and peak flows from ReFH2 are greater for the summer design event. However, the upstream reaches at T01, T02, S01 are predominantly dry during the summer period (April to September) as illustrated by groundwater emergence data from 1993-2007 (see Annex C). The winter season has therefore been selected for the design events.

5.3 Flood estimates from ReFH2

5.3.1 Table 5-2 provides the peak flow estimates generated using the ReFH2 method. As per the Technical Guidance Document: ReFH 2.2, the urban results are reported. These results take account of the urban extent within the catchment based on URBEXT2000 and are therefore representative of existing conditions.

5.3.2 Flood volumes have also been provided in Table 5-4.

Table 5-3: Peak flood estimates (m^3s^{-1}) for a range of AEP's using ReFH2 method

Site Code	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
T01	1.13	1.62	1.99	2.38	2.64	3.00	3.33	3.60	7.29
T02	1.65	2.35	2.87	3.42	3.79	4.30	4.78	5.18	10.63
S01	0.49	0.71	0.87	1.05	1.16	1.32	1.46	1.58	3.28
T03	2.52	3.54	4.31	5.14	5.68	6.46	7.19	7.79	16.14
T04	2.61	3.67	4.45	5.31	5.86	6.67	7.43	8.06	16.77

Table 5-4: Flood volume estimates (m^3) for a range of AEP's using ReFH2 method

Site Code	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
T01	32716	46826	57312	68624	75971	86279	95867	103664	209403
T02	65955	93781	114100	136107	150570	170947	189874	205348	419646
S01	13424	19273	23651	28425	31488	35816	39814	43067	88877
T03	110572	155749	189155	225552	248983	282996	314775	340802	703077
T04	126678	177943	215754	256936	283616	322291	358841	389026	805735

6 Discussion and summary of results

6.1 Comparison of results from different methods

6.1.1 Table 6-1 and Table 6-2 provide a comparison of peak flow estimates from the FEH Statistical method and ReFH2 method for QMED and the 1% AEP event, respectively.

6.1.2 These illustrate that with the exception of the upstream inflow point at Tilshead (T01), flow estimates from the FEH statistical method are typically around 50% greater than those from the ReFH2 method on the River Till. This is likely to be a function of using the 'flow variability' method to estimate QMED.

6.1.3 For the minor inflow at Shrewton (S01), flow estimates compare well between methods. It is noted that QMED for this location has been estimated from catchment descriptors and adjusted by donor transfer.

Table 6-1: Comparison of FEH Statistical and ReFH2 peak flow estimates (m^3s^{-1}) for QMED

Site Code	FEH Statistical	ReFH2	Ratio (ReFH2/FEH Statistical)
T01	0.80	1.13	1.41
T02	3.36	1.65	0.49
S01	0.45	0.49	1.09
T03	4.74	2.52	0.53
T04	5.55	2.61	0.47

Table 6-2: Comparison of FEH Statistical and ReFH2 peak flow estimates (m^3s^{-1}) for 1% AEP event

Site Code	FEH Statistical	ReFH2	Ratio (ReFH2/FEH Statistical)
T01	2.40	3.60	1.50
T02	10.7	5.18	0.49
S01	1.55	1.58	1.02
T03	13.5	7.79	0.58
T04	16.4	8.06	0.49

6.2 Final choice of method

6.2.1 The final choice of method is to use the FEH Statistical Pooling Group method to estimate peak flows.

6.2.2 Hydraulic modelling runs have been undertaken to assess flood extents and compare estimates of QMED using catchment descriptors with a donor adjustment applied against those using the QMED linking equation for flow variability (as discussed In Section 4.6). This was undertaken prior to applying the growth curve factors.

6.2.3 For flows on the River Till, the final QMED estimates are from the QMED linking equation based on flow variability. For the incoming tributary at Shrewton, QMED has been estimated using from catchment descriptors and then donor adjusted from the Chitterne Brook @ Codford (with non-flood years removed from the AMAX series as the donor is considered to be a permeable catchment).

6.3 Assumptions, limitations and uncertainty

6.3.1 There are a number of assumptions with flow estimates for the River Till. These are:

1. The catchment is ungauged, permeable and has ephemeral flows, therefore greater focus has been given in determining QMED because this has a greater influence on the final flow estimates compared to modifications to stations within the pooling group.

2. There are a limited number of stations within each pooling group that are considered to be permeable. A permeable adjustment of these stations has not been undertaken as WINFAPv4 does not allow adjustments to L - CV and L-Skew. A check of non-flood years indicates that an adjustment is unlikely to significantly alter resultant growth curve factors.

3. Flows within the catchment are influenced by groundwater due to the permeable nature of the catchment. Surface water runoff may also contribute depending on catchment wetness i.e. the catchment may respond differently to the same rainfall event depending on antecedent conditions.

4. The catchment is essentially rural with limited development planned in the future. As per the Environment Agency Flood Estimation Guidelines, the effects of urbanisation have been applied even though the catchment is considered to be rural.

5. Historic flood events have been identified through data review. The flood generating processes for these events are variable and include snowmelt combined with frozen ground (1841), high groundwater levels with prolonged low intensity rainfall (2013/14).

6. The period of historic record is not considered to be stationary with regard to climate (1789 and 1841 events within the Little Ice Age period). A review of Met Office information indicates a reduction in average number of snow lying days through comparison of 1960 – 1991 and 1981 – 2010 records for the UK that indicates the climate is not stationary.

7. The results from this study imply that the flood estimate of 48 m^3s^{-1} at Shrewton for the 1841 Great Till Flood by Clark (2004) (Ref 15) are likely to be very conservative with large uncertainty surrounding the estimate (see below for further discussion).

6.3.2 The following limitations with regard to the methods applied in this study are acknowledged:

1. The performance of FEH methods for flood estimation in permeable catchments is acknowledged to be less certain than for catchments where BFIHOST is < 0.65.

2. The FEH statistical method is considered suitable for up to the 0.5% AEP. This method has been used to estimate 0.1% AEP and therefore caution should be used with these flows as they are outside of the range for AEP's.

6.3.3 With regard to uncertainty, the following points are noted:

1. The F.S.E for QMED has been provided for the 68% and 95% confidence intervals to illustrate the upper and lower limit of QMED using a) catchment descriptors only and b) catchment descriptors with a donor adjustment applied (reduces the F.S.E). These are provided in Section 4.5.

2. To help reduce uncertainty in QMED, the use of long term groundwater model outputs have been utilised to assess QMED using the flow variability through the QMED Linking Equation in WINFAPv4. A cross check was undertaken with a neighbouring catchment to compare flow duration statistics from daily mean flows against groundwater model outputs (see Annex C). At the required flows (Q5 and Q10), a good comparison was observed. The F.S.E for the QMED Linking Equation is reported to be 25% smaller than using the catchment descriptor approach. However, the F.S.E has not been calculated in this report because flows from the groundwater model are on a tri-monthly time step and is therefore a limitation in the application of this approach.

3. Due to the permeable nature of the catchment and absence of gauged data within the catchment, there is likely to be greater uncertainty in the growth curve estimates.

6.3.4 The flood estimates in this report have been developed for the purposes of this study only and to assess the impact of the proposed viaduct structure at Winterbourne Stoke. The results may be applicable for other studies, although users should undertake necessary checks for additional data (e.g. updates to AMAX data for QMED and stations within the pooling group, more recent flooding, updated estimation techniques).

6.3.5 Whilst this study is for the purposes of an individual project, it is noted that it would aid future studies if spot flow measurements were undertaken at bank full level to aid validation of future models.

6.4 Checks

6.4.1 A series of checks have been undertaken to assess the flow estimates.

6.4.2 The results are consistent with an increase in flow in a downstream direction. The flow at Winterbourne Stoke (T03) is greater than the sum of the flows (T02 + S01) upstream at Shrewton where there is an incoming minor tributary (S01).

6.4.3 The catchment is ungauged and therefore assessment of AEP's against flooding is not feasible in this instance. A number of floods both recent and historical have occurred within the catchment as provided in the flood history (see Annex B). There is very limited flow data available (two spot flow gaugings outside the reach of interest).

6.4.4 Cross verification between flow estimates and hydraulic modelling results have been undertaken to provide a 'sensibility' check. i.e. are the flows too low (no flooding at more frequent AEP's) or too high (significant flood extents at QMED).

6.4.5 With regard to growth curve factors, the typical range is considered to be between 2.1 to 4.0 for the 1% AEP event. In this study, the growth curve factors are:

- T01 = 3.018
- T02 = 3.173
- S01 = 3.426
- T03 = 2.840
- T04 = 2.947

6.4.6 The above values all sit within the typical range and are therefore considered realistic.

6.4.7 In addition, a check on T04 was undertaken to compare the 1% AEP growth curve factor using the standard FEH approach where the similarity distance measure (SDM) defines stations within the pooling group against a pooling group created from stations identified as permeable only (SPRHOST < 20). The SDM does not include BFIHOST and as discussed in Science Report SC050050 does not pay special attention to growth curve estimation in permeable catchments. The growth curve factor using only permeable stations for T04 = 3.009 and is therefore comparable to the standard FEH statistical approach.

6.4.8 The 0.1%/1% AEP event ratios using the FEH statistical method range between 1.4 and 1.8. These values are generally within the expected range for UK catchments. The ratio is lower where the GEV distribution has been applied to the pooling group and is expected as the GEV distribution generally results in shallower growth curves than the GL distribution.

6.4.9 The specific runoff for the 1% AEP event, the specific discharge rates are:

- T01 = 0.61 l/s/ha
- T02 =1.46 l/s/ha
- S01 = 0.97 l/s/ha
- T03 = 1.18 l/s/ha
- T04 = 1.32 l/s/ha

6.4.10 Whilst these are considered to be lower than normal, this is to be expected due to the permeable nature of the catchment.

6.4.11 A cross check with the gauged data in the neighbouring Chitterne Brook catchment (Station 43801) indicates a specific discharge of between 0.41 l/s/ha (full AMAX series) and 0.80 l/s/ha (non-flood years excluded from AMAX series). Flow estimation point T02 has a similar catchment area (69.7 km2) to Station 43801 (72.9 km2) and the specific discharge for QMED is comparable at 0.46 l/s/ha.

6.4.12 Table 6-3 provides a list of studies within the catchment where previous flow estimates have been undertaken for a range of studies.

6.4.13 It is considered that the results from the more recent studies are comparable and is likely due to the change from early flow estimates using Flood Study report techniques to those adopted within FEH. Whilst flows are comparable, the assumptions, limitations and uncertainty with the techniques used should still be referred to.

6.4.14 Updated FEH methods have been applied since the previous studies and include extended series of records. This extends record lengths giving greater certainty for QMED at donor stations and may reduce the number of stations within pooling groups that have similar hydrological characteristics.

Table 6-3: Previous studies

Study and date	Purpose of study	Nearest flow estimation point	Flow estimates (m^3s^{-1})	Comments
Tilshead and Orcheston Flood Attenuation Scheme, June 2017 Atkins (on behalf of Wiltshire Council)	Business case for proposed attenuation scheme upstream of Tilshead	T01	QMED = 0.71 20% AEP = 1.01 10% AEP = 1.22 4% AEP = 1.53 2% AEP = 1.80 1% AEP = 2.10 0.5% AEP = 2.45	This report provides a summary for the purpose of the business case. Information pertaining to the flood estimation is therefore limited to a table of flows and acknowledgements within the text that there is 'technical uncertainty' with flow estimation. This is expanded to say that whilst best practice has been used, the nature of the permeable catchment means that flows are uncertain but provides no further information on uncertainty, limitations or assumptions. FEH statistical method was used and QMED derived from catchment descriptors adjusted using West Avon@Upavon (Station 43017). The report indicates that FEH design flows were compared to gauge flow records but provides no indication of which gauge has been used, it appears to suggest that a temporary flow gauge has been installed. Estimated flows and growth curve factors are slightly lower when compared to AECOM analysis but are not dissimilar.
Shrewton Steam Laundry, Flood Risk Assessment, March 2013 RPS	Flood Risk Assessment for Planning	T02	QMED = 2.8 1% AEP = 9.0	The Environment Agency indicated in a previous version of the FRA that it would be accept the FRA if these flows were used in the hydraulic modelling undertaken. It appears that these flows are based on the estimates provided by JBA consulting for a separate FRA at High Street, Shrewton (see below). Estimated flows used within this FRA are slightly lower than estimated from the AECOM analysis. However, growth curve factors are comparable and less than 1% different.

Study and date	Purpose of study	Nearest flow estimation point	Flow estimates (m^3s^{-1})	Comments
High Street, Shrewton, January 2007 JBA Consulting (on behalf of Such Salinger Peters)	Supporting flow estimates for Flood Risk Assessment	T02	QMED = 2.8 20% AEP = 3.9 10% AEP = 4.8 4% AEP = 6.1 2% AEP = 7.4 1.33% AEP = 8.2 1% AEP = 8.9 0.75% AEP = 9.9 0.5% AEP = 10.7	The main FRA report by Such Salinger Peters argues that the FSSR flow rates 'more closely reflect the known situation'. However, the FSSR estimates were based on analysis from 1998 (report not available). JBA consulting provided supporting flow data (Appendix 2 within the FRA) and derived flows using the FEH statistical method. It is noted that Hi-Flows v1.1 was used and that the version of WINFAP-FEH pre-dates the revised methods currently used for selecting pooling groups in WINFAPv4. Whilst the pooling group composition is predominantly formed of permeable sites for the 2007 study and the number of station years within the pooling group was 443, growth curve estimates are comparable for the 1 in 1% AEP event (JBA = 3.18, AECOM 3.17). Flow estimates are approximately 20% higher in the AECOM assessment and is due to the approach adopted for QMED.
Estimating extreme floods, 2004 Colin Clark	Journal Paper in: International Water Power	T02	1789 Flood = 25-40 m^3s^{-1}, return period estimated at 126 years 1841 Flood = 48 m^3s^{-1}, return period estimated at 307 years 1915 Flood = 12 m^3s^{-1}, return period estimated at 80 years	This paper estimates the flood flow for the 1841 event using historical information including newspaper accounts and flood markers within Shrewton. Extrapolation of the FEH statistical growth curve factors would result in a return period between 50,000 to 100,000 year, which is much greater than the reported estimate of ca. 300 years. The estimates have been made using Manning's equation based on reported flood water depths and assumed cross section profile but no values are given for channel dimensions in Shrewton (only a cross section figure). Whilst based on reported values, caution should be used when estimating flows from such data. No account has been taken for the potential debris and blockage caused by destruction of the cob walled houses within the vicinity that would affect observed water levels. In addition, these

Study and date	Purpose of study	Nearest flow estimation point	Flow estimates (m^3s^{-1})	Comments
				estimates contradict the cautionary remarks on use of historic flood frequency analysis present by the author in a separate article in 2003.
Information in Wiltshire Council archive (Tilshead, Shrewton & Orcheston – Land Drainage.pdf – p52) Tilshead FAS – Flow calculations using FSSR4	Tilshead Flood Alleviation Scheme (early 2000s)	T01	MAF = 1.3 20% AEP = 1.62 10% AEP = 1.91 4% AEP = 2.21 1.33% AEP = 2.65	Calculations relate to proposed channel and drainage improvements at Tilshead and undertaken using FSSR4. A comparison peak flow estimates from the FSSR4 approach with FEH statistical indicates that estimates of QMED are greater using FSSR4 (likely to be a function of using the mean rather than median). However, it is noted that whilst peak estimates for the return periods are greater, the growth curve from FSSR4 has a lower gradient than FEH statistical: FSSR4 - Q75/MAF = 2.03 FEH Statistical Q75/QMED = 2.86
Winterbourne Stoke, PIDS report, August 2001 Environment Agency	Problem Identification Study (PIDS)	T04	Spot flow measurements 1995 (no date provided) = 7.22 11/12/2000 = 6.86	Report indicates flows taken '3km downstream of Winterbourne Stoke at Bury Bridge'. No grid reference provided or context to timing of spot flow measurements (i.e. was this at the peak of the event?). In addition, a comparison is made with Posford Duvivier FSR calculations (Prefeasibility Report, March 1996) and assumes a 'conservative' flow of 4 m^3s^{-1} at Winterbourne Stoke on 11/12/2000. The report indicates this would equate to between a 1 in 10 and 1 in 20 year event but may be greater. It is noted that there is a flow split on the River Till in the vicinity to Bury Bridge, therefore spot flow measurements should be taken with caution. Also there appears to be no valid reasoning for the assumption of a 4 m^3s^{-1} flow at Winterbourne Stoke. Due to the limited information, it is not possible to draw sound conclusions from this report.
G4640 Shrewton Flood	Feasibility study and	T02	FSSR4	Hydrological analysis undertaken using FSSR4 and

Study and date	Purpose of study	Nearest flow estimation point	Flow estimates (m^3s^{-1})	Comments
Alleviation Scheme: Pat 1 Scheme Viability, August 1996 Posford Duvivier	appraisal report		MAF = 2.64 20% AEP = 3.33 10% AEP = 3.91 4% AEP = 4.46 3.33% AEP = 4.81 2% AEP = 5.23 1% AEP = 5.26 <u>FSSR16</u> MAF = 4.0 20% AEP = 6.30 10% AEP = 8.19 5% AEP = 10.09 3.33% AEP = 11.16 2% AEP = 12.96 1% AEP = 15.34 0.1% AEP = 28.24	FSSSR16 techniques. Study adopted FSSR4 as considered suitable for permeable (chalk) catchments but suggested that FSSR16 values could represent frozen ground conditions. The estimated flows at T02 up to the 1% AEP event lie between the FSSR4 and FSSR16 estimates. It is noted that the growth curve factors using the FSSR4 method are very low with Q100/MAF = 1.99. As this study was taken 20+ years ago, it should be noted that the flood series used to derive estimates has been significantly extended for present day estimates and comparisons made with caution.

7 ANNEX A – Pooling Groups

7.1 Initial Pooling Groups

7.1.1 Table A-1 to Table A-5 provide the initial pooling groups derived from WINFAPv4 for each subject site.

Table A-1: Initial Pooling Group for Site T01

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
39033 (Winterbourne Stream @ Bagnor)	0.214	54	0.404	0.344	0.386	2.436
24007 (Browney @ Lanchester)	0.259	15	10.981	0.222	0.212	2.561
26803 (Water Forlornes @ Driffield)	0.323	17	0.437	0.3	0.112	0.336
28058 (Henmore Brook @ Ashbourne)	0.368	12	9.006	0.155	-0.064	1.932
53017 (Boyd @ Bitton)	0.378	43	13.82	0.247	0.106	0.106
44003 (Asker @ Bridport)	0.492	14	12.354	0.224	0.17	1.092
44011 (Asker @ East Bridge Bridport)	0.492	21	16.8	0.239	0.112	0.429
42011 (Hamble @ Frogmill)	0.543	44	8.282	0.167	0.073	0.99
20006 (Biel Water @ Belton House)	0.551	28	11.748	0.375	0.128	1.002
43806 (Wylye @ Brixton Deverill)	0.593	25	2.08	0.376	0.211	0.447
44013 (Piddle @ Little Puddle)	0.608	23	1.103	0.463	0.254	1.898
41020 (Bevern Stream @ Clappers Bridge)	0.632	47	13.9	0.205	0.17	0.685
49004 (Gannel @ Gwills)	0.636	47	15.022	0.258	0.105	0.315
41022 (Lod @ Halfway Bridge)	0.668	46	16.26	0.288	0.181	0.315
36004 (Chad Brook @ Long Melford)	0.702	49	5.321	0.292	0.178	0.481
36010 (Bumpstead Brook @ Broad Green)	0.709	49	7.59	0.365	0.173	0.974
Total		534				
Weighted means				0.284	0.168	

Table A-2: Initial Pooling Group for Site T02

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
20007 (Gifford Water @ Lennoxlove)	0.205	43	16.19	0.325	0.204	0.503
42008 (Cheriton Stream @ Sewards Bridge)	0.326	46	1.348	0.26	0.4	1.055
20005 (Birns Water @ Saltoun Hall)	0.354	44	18.215	0.303	0.222	0.106
51001 (Doniford Stream @ Swill Bridge)	0.385	50	11.98	0.325	0.385	1.177
42006 (Meon @ Mislingford)	0.396	57	3.003	0.257	0.217	0.468
38002 (Ash @ Mardock)	0.396	75	6.764	0.285	0.081	1.879
20006 (Biel Water @ Belton House)	0.408	28	11.748	0.375	0.128	2.097
27059 (Laver @ Ripon)	0.418	39	21.878	0.234	0.342	1.478
30004 (Lymn @ Partney Mill)	0.431	54	6.983	0.231	0.046	0.835
53023 (Sherston Avon @ Fosseway)	0.432	40	7.333	0.227	0.187	0.383
43014 (East Avon @ Upavon)	0.433	45	3.958	0.206	0.061	1.017
Total		521				
Weighted means				0.276	0.208	

Table A-3: Initial Pooling Group for Site S01

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
26802 (Gypsey Race @ Kirby Grindalythe)	0.049	17	0.116	0.274	0.24	0.112
25019 (Leven @ Easby)	0.207	38	5.333	0.338	0.391	0.998
27010 (Hodge Beck @ Bransdale Weir)	0.575	41	9.42	0.224	0.293	0.584
49005 (Bollingey Stream @ Bolingey Cocks Bridge)	0.581	6	6.511	0.265	0.063	1.638
44008 (South Winterbourne @ Winterbourne Steepleton)	0.635	37	0.448	0.416	0.326	1.094
22003 (Usway Burn @ Shillmoor)	0.791	13	16.17	0.282	0.311	0.692
203046 (Rathmore	0.869	34	10.788	0.146	0.136	0.622

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
Burn @ Rathmore Bridge)						
36010 (Bumpstead Brook @ Broad Green)	0.939	49	7.585	0.365	0.173	1.422
27051 (Crimple @ Burn Bridge)	0.978	44	4.539	0.223	0.156	0.116
26803 (Water Forlornes @ Driffield)	1.015	17	0.437	0.3	0.112	0.497
44013 (Piddle @ Little Puddle)	1.182	23	1.103	0.463	0.254	2.044
47022 (Tory Brook @ Newnham Park)	1.22	23	7.123	0.262	0.115	0.144
49006 (Camel @ Camelford)	1.223	10	11.35	0.12	-0.269	3.717
41020 (Bevern Stream @ Clappers Bridge)	1.246	47	13.9	0.205	0.17	0.6
28058 (Henmore Brook @ Ashbourne)	1.269	12	9.006	0.155	-0.064	1.439
27032 (Hebden Beck @ Hebden)	1.269	50	3.923	0.207	0.253	0.781
73015 (Keer @ High Keer Weir)	1.271	25	12.239	0.174	0.191	0.481
25011 (Langdon Beck @ Langdon)	1.272	28	15.878	0.238	0.318	1.021
Total		514				
Weighted means				0.266	0.205	

Table A-4: Initial Pooling Group for Site T03

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
21016 (Eye Water @ Eyemouth Mill)	0.086	39	36.964	0.275	0.151	0.741
39028 (Dun @ Hungerford)	0.257	48	2.207	0.219	-0.002	0.774
53028 (by Brook @ Middlehill)	0.283	35	10.692	0.171	-0.083	0.915
39020 (Coln @ Bibury)	0.29	53	3.61	0.191	0.08	0.946
20005 (Birns Water @ Saltoun Hall)	0.311	44	18.215	0.303	0.222	0.659
27086 (Skell @ Alma Weir)	0.404	30	27.498	0.265	0.436	1.785
13001 (Bervie @ Inverbervie)	0.426	27	35.577	0.212	0.141	0.626
33018 (Tove @ Cappenham Bridge)	0.437	52	17.059	0.273	0.182	0.131

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
27055 (Rye @ Broadway Foot)	0.443	38	41.699	0.364	0.575	2.776
38004 (Rib @ Wadesmill)	0.475	57	11.798	0.308	0.166	0.908
23002 (Derwent @ Eddys Bridge)	0.475	11	48.41	0.171	0.032	0.798
19011 (North Esk @ Dalkeith Palace)	0.477	44	36.856	0.324	0.282	1.529
21024 (Jed Water @ Jedburgh)	0.481	34	71.477	0.216	0.151	0.412
Total		512				
Weighted means				0.254	0.175	

Table A-5: Initial Pooling Group for Site T04

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
21016 (Eye Water @ Eyemouth Mill)	0.087	39	36.964	0.275	0.151	0.662
39020 (Coln @ Bibury)	0.337	53	3.61	0.191	0.08	0.719
39028 (Dun @ Hungerford)	0.342	48	2.207	0.219	-0.002	0.73
53028 (by Brook @ Middlehill)	0.368	35	10.692	0.171	-0.083	0.978
33018 (Tove @ Cappenham Bridge)	0.392	52	17.059	0.273	0.182	0.035
13001 (Bervie @ Inverbervie)	0.399	27	35.577	0.212	0.141	0.879
27086 (Skell @ Alma Weir)	0.4	30	27.498	0.265	0.436	1.717
27055 (Rye @ Broadway Foot)	0.405	38	41.699	0.364	0.575	2.636
20003 (Tyne @ Spilmersford)	0.407	55	34.345	0.377	0.223	2.305
21024 (Jed Water @ Jedburgh)	0.419	34	71.477	0.216	0.151	0.604
20005 (Birns Water @ Saltoun Hall)	0.42	44	18.215	0.303	0.222	0.293
38004 (Rib @ Wadesmill)	0.426	57	11.798	0.308	0.166	0.442
Total		512				
Weighted means				0.265	0.181	

7.2 Revised Pooling Groups

7.2.1 Following the pooling group review outlined in Section 4.7, Table A-6 and Table A-7 provide details of the revised pooling groups for subject sites T01 and S01 respectively. The remaining pooling groups (T02, T03 and T04) remain unchanged from the initial groups following review.

Table A-6: Revised Pooling Group for Site T01

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
39033 (Winterbourne Stream @ Bagnor)	0.214	54	0.404	0.344	0.386	3.139
26803 (Water Forlornes @ Driffield)	0.323	17	0.437	0.3	0.112	0.466
53017 (Boyd @ Bitton)	0.378	43	13.82	0.247	0.106	0.244
44003 (Asker @ Bridport)	0.492	14	12.354	0.224	0.17	0.962
44011 (Asker @ East Bridge Bridport)	0.492	21	16.8	0.239	0.112	0.976
42011 (Hamble @ Frogmill)	0.543	44	8.282	0.167	0.073	1.153
20006 (Biel Water @ Belton House)	0.551	28	11.748	0.375	0.128	1.146
43806 (Wylye @ Brixton Deverill)	0.593	25	2.08	0.376	0.211	0.364
44013 (Piddle @ Little Puddle)	0.608	23	1.103	0.463	0.254	1.74
41020 (Bevern Stream @ Clappers Bridge)	0.632	47	13.9	0.205	0.17	0.662
49004 (Gannel @ Gwills)	0.636	47	15.022	0.258	0.105	0.666
41022 (Lod @ Halfway Bridge)	0.668	46	16.26	0.288	0.181	0.724
36004 (Chad Brook @ Long Melford)	0.702	49	5.321	0.292	0.178	0.481
36010 (Bumpstead Brook @ Broad Green)	0.709	49	7.585	0.365	0.173	1.276
Total		507				
Weighted means				0.296	0.176	

Table A-7: Revised Pooling Group for Site S01

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
26802 (Gypsey Race @ Kirby Grindalythe)	0.049	17	0.116	0.274	0.24	0.037
25019 (Leven @ Easby)	0.207	38	5.333	0.338	0.391	0.812
27010 (Hodge Beck @ Bransdale Weir)	0.575	41	9.42	0.224	0.293	0.555
44008 (South Winterbourne @ Winterbourne Steepleton)	0.635	37	0.448	0.416	0.326	0.877
22003 (Usway Burn @ Shillmoor)	0.791	13	16.17	0.282	0.311	1.332
203046 (Rathmore Burn @ Rathmore Bridge)	0.869	34	10.788	0.146	0.136	1.097
36010 (Bumpstead Brook @ Broad Green)	0.939	49	7.585	0.365	0.173	0.947
27051 (Crimple @ Burn Bridge)	0.978	44	4.539	0.223	0.156	0.459
26803 (Water Forlornes @ Driffield)	1.015	17	0.437	0.3	0.112	0.766
44013 (Piddle @ Little Puddle)	1.182	23	1.103	0.463	0.254	1.798
41020 (Bevern Stream @ Clappers Bridge)	1.246	47	13.9	0.205	0.17	0.559
20002 (West Peffer Burn @ Luffness)	1.322	41	3.299	0.292	0.015	2.334
28041 (Hamps @ Waterhouses)	1.34	31	26.664	0.22	0.295	1.147
49004 (Gannel @ Gwills)	1.434	47	15.022	0.258	0.105	0.818
39033 (Winterbourne Stream @ Bagnor)	1.465	54	0.404	0.344	0.386	1.462
Total		533				
Weighted means				0.291	0.229	

8 ANNEX B – Historical Flood Record

8.1 Flood History

8.1.1 A range of sources have been used to identify the flood history in the River Till catchment. These include:

- Journal papers;
- BHS Chronology of British Hydrological Events;
- Information provided by the Environment Agency and Wiltshire Council that includes reports, photos and other information;
- Internet searches including newspaper articles, photos and planning applications.

8.1.2 Table B-1 provides a chronological history of flooding within the River Till catchment. The detail of information in some instances is very poor and only indicates that flooding has occurred but with little further information on the source, magnitude or impacts.

Table B-1: Flood chronology for the River Till catchment

Date	Description
January 1790	Wiltshire Independent article from 21st January1841 indicates '51 years last Monday an inundation from melted snow took place'. The quantity of water is said to have been greater than the 1841 flood event. However, only damage to walls and out buildings occurred with no destruction of properties or loss of life. No further details have been found on this event.
1809	BHS chronology of British hydrological events indicates a flood and high springs at Shrewton
1827	BHS chronology of British hydrological events indicates 'the springs in the valley were so prone to flood that at times in Orcheston St Mary the officiating clergyman who would escape damp was obliged, to wear clogs while ministering at the altar, to raise him above the wet'.
16th January 1841	Report within the Wiltshire Independent, 21st January 1841 indicates that snow fall on 14th and 15th January, coupled with frozen ground followed by a rapid thaw and intense rainfall on the 16th January caused widespread flooding within the Till catchment. This includes information on reported flood depths and timing plus the loss of three lives, livestock and destruction of property. This is also reported in articles by Brodie (1841), Cross (1967). Clark (2003, 2004) provides an estimate of flood discharge in Tilshead and Shrewton based on the above articles but disputes that the ground was frozen.
1905	Flooding in Shrewton – no further information available on extent, properties affected or source.
5th January 1915	Flooding in and around Elston and Shrewton reported in BHS Chronology of flood events, Cross (1967) and Clark (2004). In addition, series of photographs provided within information provided by the Environment Agency illustrates extent of flooding at various points in Shrewton and Elston'.
March 1925	BHS chronology of British hydrological events indicates 'Tilshead has many shallow wells in gravel on chalk, which in March 1925 were overflowing into the gardens and street'.
1940	Flooding in Tilshead, Orcheston and Winterbourne Stoke – no further information available on extent, properties affected or source.

Date	Description
1944	Flooding in Tilshead referred to in Atkins, 2017 report – no further information available on extent, properties affected or source.
1947	Flooding in Tilshead due to snowmelt, no further information available on extent or properties affected.
1949	Flooding in Tilshead referred to in Atkins, 2017 report – no further information available on extent, properties affected or source
1960	Overtopping of River Till in High Street, Shrewton – noted from JBA flow estimation report for FRA. Flooding in Tilshead referred to in Atkins, 2017 report – no further information available on extent, properties affected or source
1976	Flooding in Winterbourne Stoke – no further information available on extent, properties affected, suggested source is fluvial and groundwater.
1977	Flooding in Tilshead from groundwater / springs – no further information available on extent, properties affected.
1986	Flooding in Tilshead and Orcheston – no further information available on extent, properties affected. Source of flooding a combination of groundwater and surface water.
1990	Flooding in Tilshead, Orcheston, Shrewton and Winterbourne Stoke from a combination of sources (groundwater, surface water and fluvial)
1991/1992	Flooding in Shrewton – no further information available on extent, properties affected or source.
1992	Flooding in Tilshead and Orcheston – no further information available on extent, properties affected. Source of flooding from springs and groundwater.
1993	Flooding in Tilshead, Orcheston, Shrewton and Winterbourne Stoke – no further information available on extent, properties affected or source.
January / February 1995	National Rivers Authority Report (referenced in JBA flow estimates) indicates eight houses flooded in Shrewton, four from fluvial combined with groundwater and the others from groundwater. Significant plant growth, debris and in channel obstructions noted + a cob wall collapsed into the river. JBA report suggests a return period of between 5 and 60 years for this event. At this point in time, was thought to be the worst event in Shrewton since 1841 based on historical evidence although evidence pre-1960 is limited. Thought to be the only event since 1960 when overtopping has taken place on the High Street in Shrewton. Reported flood depths in Shrewton are consistent with a flow of approximately 6 – 6.5 m^3s^{-1} estimated using a hydraulic model.
1998	Flooding in Orcheston, Shrewton and Winterbourne Stoke – no further information available on extent, properties affected. Source of flooding is thought to be from a combination of groundwater, surface water and fluvial.
1999	Flooding in Tilshead and Orcheston – no further information available on extent, properties affected. Source of flooding predominantly groundwater.
2000	Flooding within the wider River Till catchment – no further information available on extent, properties affected or source of flooding although likely to be a combination of high groundwater levels coupled with rainfall causing out of bank flows on River Till.
Winter 2013 /2014	Eleven residential properties in Tilshead and A360 main road affected.

Date	Description
	Four residential properties in Orcheston affected and also the Caravan Park. Combination of high groundwater flows coupled with rainfall causing out of bank fluvial flows on River Till.

9 ANNEX C – QMED Linking Equation & Flow Variability

9.1 Background

9.1.1 In Section 4.5, an additional method of estimating QMED has been utilised within WINFAPv4. The following information provides the rational in using this approach and a novel approach to its application for estimating QMED on the River Till.

9.1.2 The QMED Linking Equation has been developed for use within WINFAPv4. This method utilises gauged records for within bank, non-flood flows for estimating QMED. The requirements for estimating QMED using this method are:

- Gauged estimates of the Daily Mean Flow (DMF) that are equalled or exceeded for 5% of the time (Q5DMF) and 10% (Q10DMF) of the time; and
- BFI – the value of Base Flow Index calculated directly from the daily mean flow series for a gauging station (not to be confused with BFIHOST).

9.1.3 In addition, the average drainage path slope (DPSBAR) is required from the FEH catchment descriptors.

9.2 Available data and approach

9.2.1 As identified in Section 4, there are no flow gauges present within the River Till catchment. A novel approach has therefore been adopted to with cross reference to available data on the neighbouring River Avon at Amesbury and use of data outputs from the Wessex Regional Groundwater Model. The following steps have been taken:

1. Assess DMF for Station 43005 (River Avon @ Amesbury) using NRFA data for the period of record 1965-2016. This required analysing the DMF within HEC-DSSVue to calculate Q5DMF and Q10DMF. BFI was identified as 0.91 from the NRFA.

2. Assess outputs from the Wessex Regional Groundwater Model for the same location as Station 43005. Due to the spatial and temporal resolution of the model, data are available as tri-monthly outputs. Outputs were analysed using HEC-DSSVue to calculate Q5DMF and Q10DMF.

3. Q5DMF and Q10DMF were compared from the two data sources and also for the wider flow duration curve (see Figure C.1 and Table C.1). These illustrate that Q5DMF and Q10DMF are considered to be reasonably similar with less than +/- 2% difference between the values (although -9% at Q2). It is noted that whilst greater differences (up to +24%) are observed from Q50 to Q99, this is likely to be a function of the temporal resolution of the output data from the Wessex Groundwater Model. This is expected because this is when a greater percentage of a given flow is exceeded and when there is likely to be greatest variability in flow i.e. due to the temporal resolution of the groundwater model this flow variability is diluted.

4. Comparison of emergence surveys for the period April 1993 – July 2007 against outputs from the Wessex Regional Groundwater Model for flow estimation points at Tilshead (T01), Shrewton (T02), Winterbourne Stoke (T03) and the downstream fluvial model boundary (T04). Figure C.2 provides winterbourne emergence profiles for the River Till, these have then been compared with the outputs from the

groundwater model. A semi quantitative check indicates that there is a good comparison between observed and predicted timing of emergence across the period of record. Based on these observations and the analysis undertaken in Step 3, the use of these data are appropriate in applying the QMED Linking Equation based on outputs of the groundwater model.

5. Analyse outputs from the Wessex Regional Groundwater Model for each flow estimation point on the River Till (T01, T02, S01, T03, T04) using HEC-DSSVue to calculate Q5DMF and Q10DMF.

6. Use Q5DMF and Q10DMF values within WINFAPv4 to estimate QMED using QMED Linking Equation. In the absence of BFI from a mean daily flow series, the use of BFIHOST in this instance was considered appropriate. This is justified when comparing the BFI (0.91) for the DMF at Station 43005 and the BFIHOST value (0.903) from FEH catchment descriptors at the same location.

9.3 Wessex Groundwater Model Limitations

9.3.1 The Wessex Model comprises separately a recharge model and a groundwater model, this is described in further detail in the Numerical Model Report, Appendix 11.4: Annex 1 that covers the groundwater modelling aspects of the project. A brief summary of key model components are as follows:

- Grid cells are 250 m by 250 m
- Model time interval is 10 day stress periods (tri-monthly)
- Model time horizon is 1965 to 2016. The period 1965 -1969 is a ‘warm up period’ to allow initial conditions to be set and well calibrated at periods of interest early in the simulation period (e.g. 1976 drought).
- The recharge model requires rainfall inputs, potential evapotranspiration (PE), land use, soil type, geology, crop type and urban mains leakage.
- Runoff is routed according to Digital Terrain Mapping and stream cells mapped according to OS mapping.
- The recharge model calculates recharge to the underlying aquifer and runoff to streams (directly and via interflow). This creates a MODFLOW recharge file and stream file for use as input to the groundwater model.

9.3.2 Whilst appropriate for modelling recharge and groundwater at the basin scale, it is acknowledged that the grid cell and time steps introduce uncertainties when applying to a higher resolution. The regional model has been calibrated by the Environment Agency to groundwater levels and stream flows through their Wessex Basin Groundwater Modelling Study Phase 4 (Ref 16).

9.3.3 A further limitation of using the Wessex Regional Groundwater Model to predict flows is the assumption of connectivity between the groundwater and the surface water that has to be decided by the model developer rather than based on measured flows. Connectivity can vary along a watercourse and therefore the use of the model to reflect inflows at several locations along an ungauged reach is highly uncertain.

9.4 Summary

9.4.1 In the absence of gauged data on the River Till, the estimation of QMED for this ephemeral stream is challenging. QMED from catchment descriptors should be used as a 'last resort' and it is preferable to utilise local data where available (e.g. donor transfer). The QMED Linking Equation provides a new method in catchments where high flow data may not be available but the use of daily mean flows can provide a refined estimation over catchment descriptors.

9.4.2 Whilst the River Till is ungauged, the use of emergence flows from the Wessex Regional Groundwater Model has been considered. Flow duration statistics for flow equal or exceeded for 5% (Q5) of the time and 10% (Q10) of the time are comparable from the groundwater model when compared with daily mean flows on the River Avon at Amesbury. In addition, timing of modelled groundwater emergence on the River Till at a range of locations compares well with observed emergence for the period of record between April 1993 and July 2007. It is therefore considered that the use of Q5 and Q10 flows from the groundwater model are suitable for use within the QMED Linking Equation.

9.4.3 It is noted that there are limitations with the outputs of the Wessex Regional Groundwater Model, in particular, the temporal resolution being tri-monthly timesteps. Whilst these limitations exist, the use of this approach is considered appropriate due to the ungauged and complex nature of the catchment (i.e. highly permeable).

9.4.4 Fluvial hydraulic model runs are proposed to assess the range of QMED values derived from the separate FEH techniques and outputs compared with historical data to provide a 'sensibility check'. This provides an iterative approach between the hydrological and hydraulic modelling analysis to aid in a better representation of the fluvial flooding process occurring within the River Till catchment.

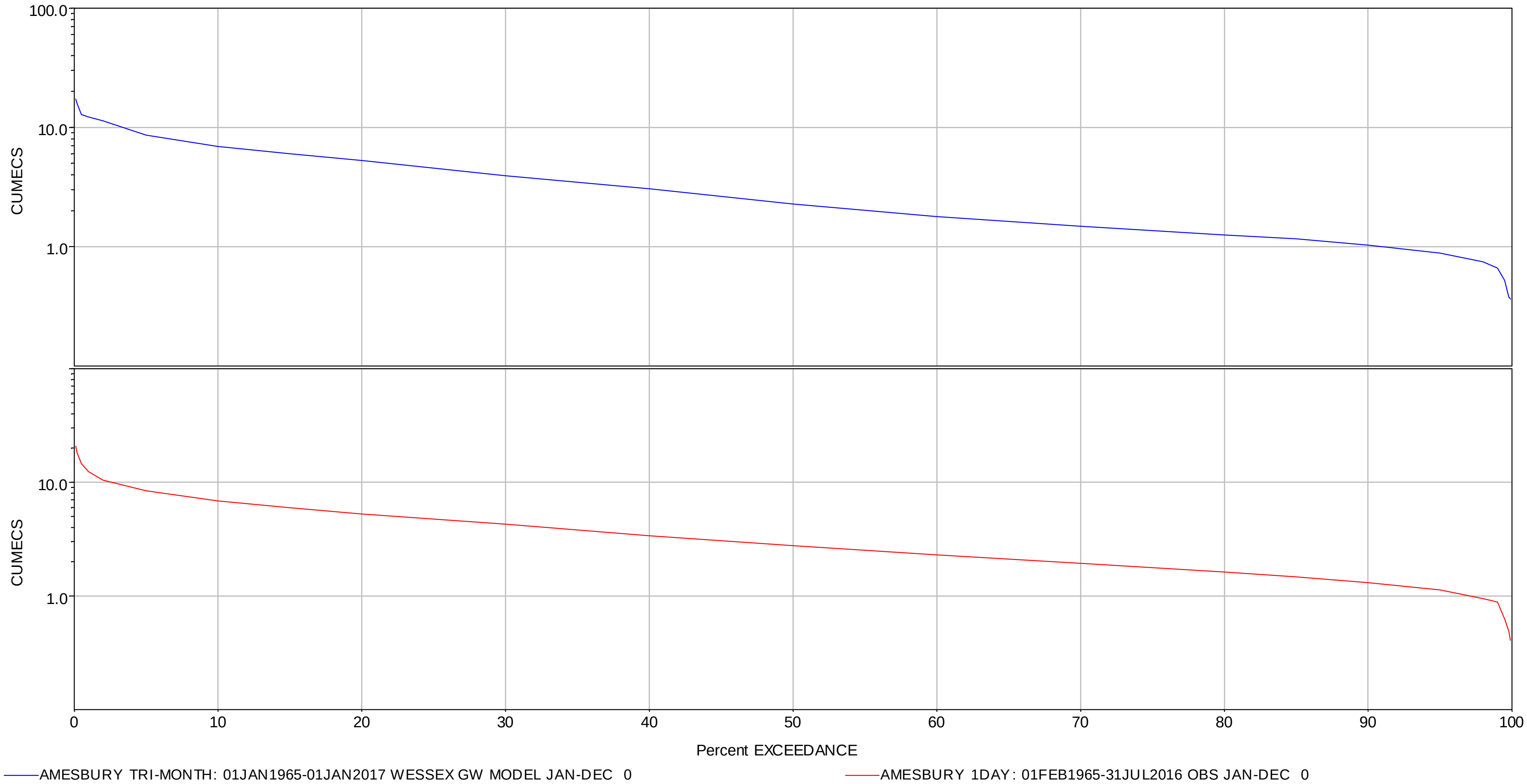
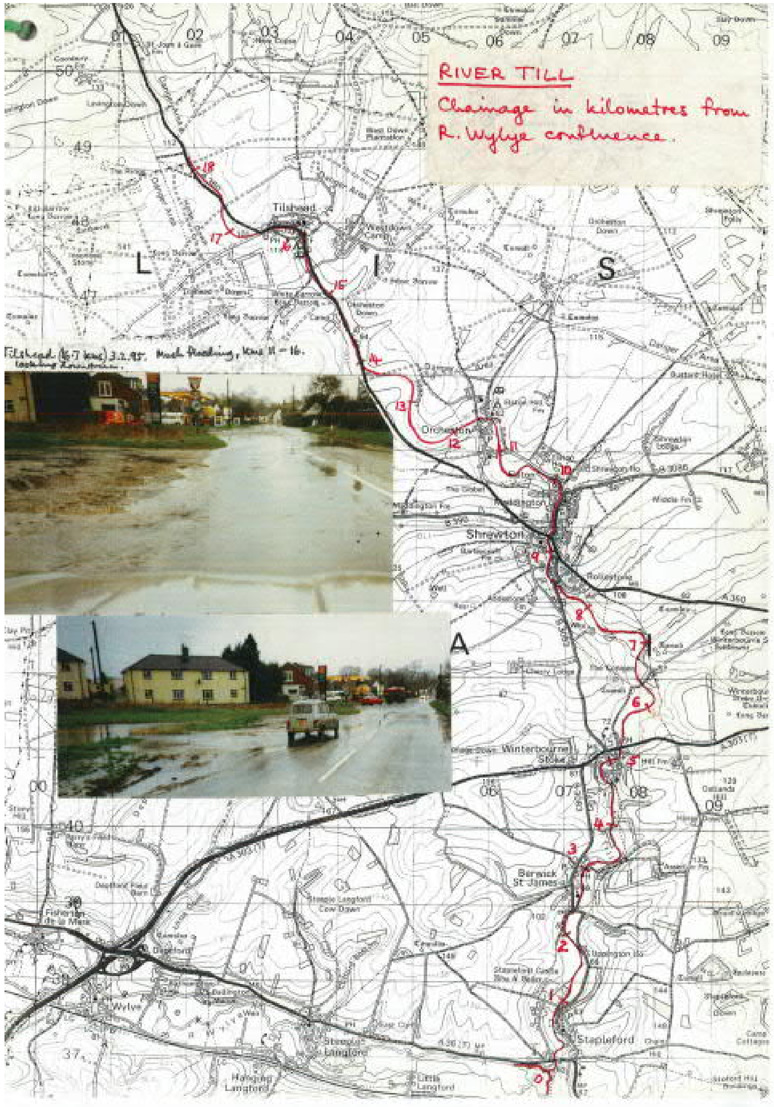


Figure C.1: Comparison of flow duration curves from Wessex Groundwater Model (upper graph) and Daily Mean Flows at Amesbury (lower graph)

Table C.1: Comparison of Flow Duration Curve statistics

Flow Duration Curve	Percentage of time flow (m^3s^{-1}) at or exceeded															
	1	2	5	10	15	20	30	40	50	60	70	80	85	90	95	99
Wessex Groundwater Model	12.21	11.32	8.60	6.91	6.01	5.27	3.94	3.06	2.28	1.79	1.48	1.25	1.16	1.03	0.88	0.66
Avon @ Amesbury	12.35	10.40	8.39	6.83	5.95	5.25	4.27	3.38	2.77	2.30	1.93	1.62	1.47	1.31	1.13	0.88
% difference between flows	1%	-9%	-2%	-1%	-1%	0%	8%	9%	18%	22%	24%	23%	21%	21%	22%	25%



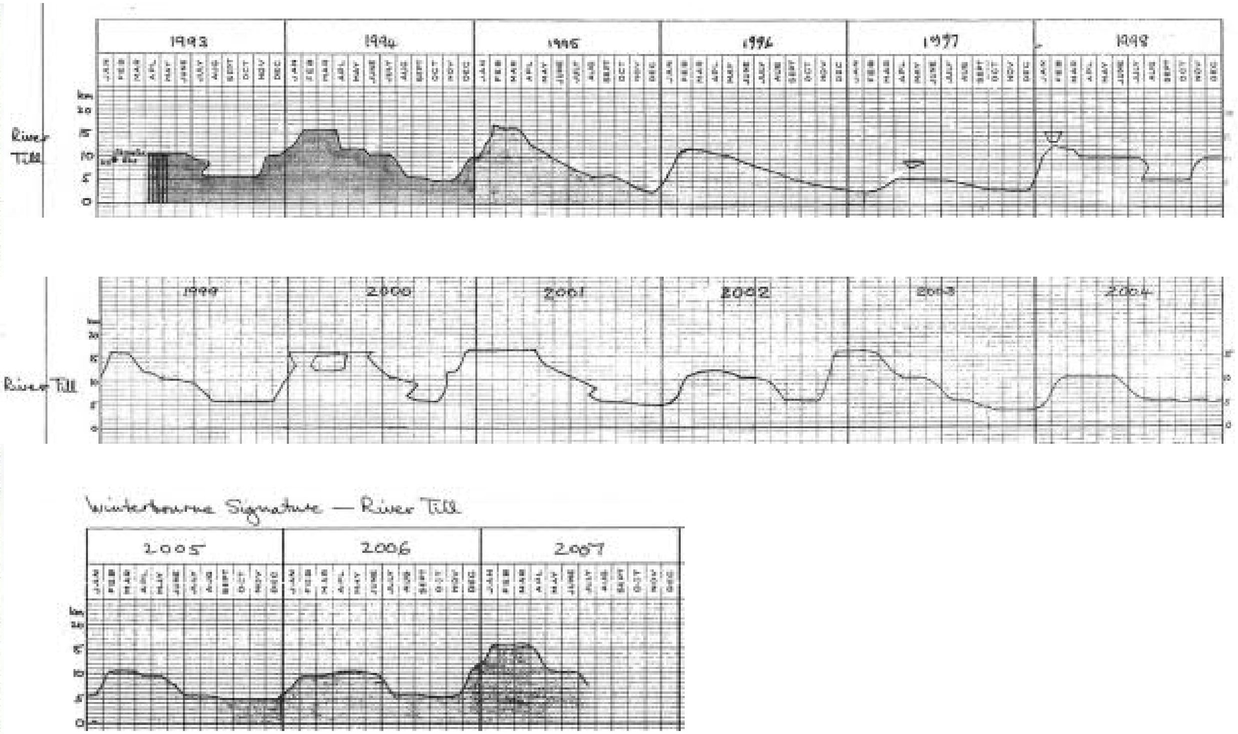


Figure C.2: River Till emergence survey for period 1993 to 2007 and associated chainage

Abbreviations List

AM	Annual maxima
AREA	Catchment area (km^2)
BFI	Base flow index
BFIHOST	Base flow index derived using the HOST soil classification
DPLBAR	Mean drainage path length (km)
DPSBAR	Mean drainage path slope (m/km)
EA	Environment Agency
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FPEXT	Floodplain extent
FSR	Flood Studies Report
HOST	Hydrology of soil types
NRFA	National River Flow Archive
POT	Peaks over threshold
QMED	Median annual flood (50% AEP)
ReFH	Revitalised flood hydrograph method – used for rainfall runoff method
SAAR	standard average annual rainfall (SAAR)
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
WINFAP	Windows Frequency Analysis Package – used for FEH statistical method

References

Ref 1 National Planning Policy Framework accessed July 2018, available at: https://www.gov.uk/government/publications/national-planning-policy-framework--2

Ref 2 Planning Practice Guidance accessed July 2018, available at: https://www.gov.uk/government/collections/planning-practice-guidance

Ref 3 National Policy Statement for National Networks accessed July 2018, available at: https://www.gov.uk/government/publications/national-policy-statement-for-national-networks

Ref 4 Flood risk assessment: climate change allowances accessed July 2018, available at: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

Ref 5 Environment Agency (2017), Flood Estimation Guidelines, Technical Guidance 197_08.

Ref 6 Kay, A.L. (2016), Snow in Britain: the historical picture and future projections. Centre for Ecology & Hydrology, pp 24.

Ref 7 Kjeldsen, T.R., Jones, D.A., and Morris, D.G. (2014), Using multiple donor sites for enhanced flood estimation in ungauged catchments, Water Resources Research, 50, 6646-6657.

Ref 8 Harvey, A.M. (1969), Channel capacity and the adjustment of streams to hydrologic regime, Journal of Hydrology, Vol 8(1), p82-98.

Ref 9 Kjeldson, T.R., Jones, D.A. and Bayliss, A.C. (2008) Improving the FEH statistical procedures for flood frequency estimation, Science Report SC050050, Environment Agency/Defra.

Ref 10 MacDonald, N. and Sangster, H. (2017) High-magnitude flooding across Britain since AD 1750, Hydrology and Earth System Sciences, 21, 1631-1650.

Ref 11 Lane, S.N. (2009) Flood rich periods, flood poor periods and the need to look beyond instrumental records, Geophysical Research Abstracts, 11, 13743.

Ref 12 Cross, D.A.E. (1967), The Great Till Flood of 1841, Weather, 22(11), p430-433.

Ref 13 The Wiltshire Independent, Thursday 21 January 1841

Ref 14 Clark, C. (2003), The role of historic flood data in estimating extreme flood events. In 'Watershed Hydrology', p482-493.

Ref 15 Clark, C. (2004), Estimating extreme floods, Water Power and Dam Construction (http://www.waterpowermagazine.com/features/featureestimating-extreme-floods/)

Ref 16 Environment Agency, 2011. Wessex Basin Groundwater Modelling Study. Phase 4 final report (prepared by Entec UK).

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Level 3 Flood Risk Assessment
Annex 2B – River Avon Hydrological Analysis

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

May 2019

Table of contents

Chapter

Table of Figures

Table of Tables

1 Introduction

1.1 Overview

1.1.1 This document provides a record of the calculations and decisions made during the production of flood estimates for the River Avon and Nine Mile River, Wiltshire. It is a supporting Annex to the hydraulic modelling work being undertaken for the wider A303 Amesbury to Berwick Down project.

1.1.2 The information provided here should enable the work to be reproduced by others in the future. It is formed of a method statement, locations where flood estimates are required, the Flood Estimation Handbook (FEH) methods used, a discussion and summary of results plus supporting information.

2 Method Statement

2.1 Overview of requirement for flood estimates

2.1.1 The purpose of the study is to provide flow estimates for use within hydraulic modelling to define Flood Zone 2 and Flood Zone 3 in accordance with the National Planning Policy Framework (Ref 1), associated practice guidance (Ref 2) and National Policy Statement for National Networks (Ref 3). In addition, 3.33% AEP event will be run to define the functional floodplain as described within the NPPF (Ref 1).

2.1.2 Peak flow estimates and hydrographs are required for the 3.3% AEP, 1% AEP, and 0.1% AEP events at six locations. Allowances for climate change are also required for the South West River Basin District, these are 30% (central), 40% (higher central) and 85% (upper end).

2.2 Overview of catchment

2.2.1 The River Avon catchment is approximately 269 km^2 at the upstream boundary of the hydraulic model and 366 km^2 at the downstream boundary. The catchment is underlain by chalk (Upper Cretaceous – Upper and Middle chalk Series) with superficial deposits of sands and gravels in the valley base.

2.2.2 The Nine Mile River is a tributary that joins the River Avon at Bulford. The watercourse is a ‘winterbourne’ and experiences ephemeral flows during periods of high groundwater levels, typically during the winter period. Conversely, there are periods where the river has no flow within its channel, typically during the summer period.

2.2.3 A review of the 1:20,000 British Geological Survey (BGS) mapping (Sheets 8 and 9) indicate that the groundwater catchment for the River Avon and Nine Mile River coincide well with the surface water catchments.

2.2.4 The main settlements within the catchment are Pewsey, Amesbury, Bulford and Durrington (Figure 3-1). There are developments with planning permission that are committed to be built between 2017 and 2026 based on the Wiltshire Council Local Plan documents. There are approximately 180 committed in Pewsey and 1056 committed with the Amesbury/Bulford/Durrington area and another 60 identified in Bulford/Durrington. The remainder of the catchment is rural and consists of predominantly grassland with small areas of arable and woodland. As per the NPPF, new development should ensure that there is no increase in flood risk to and from the development, therefore no changes in urban response are expected from the future development.

2.3 Source of flood peak data

2.3.1 Version 6 (released in February 2018) of the National River Flow Archive (NRFA) Peak Flows dataset has been used.

2.4 Flood History

2.4.1 A range of sources have been used to identify the flood history in the River Avon catchment. These include:

- Journal papers;
- BHS Chronology of British Hydrological Events;
- Information provided by the Environment Agency and Wiltshire Council that includes reports, photos and other information;
- Internet searches including newspaper articles, photos and planning applications.

2.4.2 Annex B provides a full list of the flood history within the River Avon catchment (including downstream to Salisbury).

2.4.3 Based on the flood history, a combination of sources including fluvial, surface water and groundwater sources are the primary mechanisms of flooding within the catchment.

2.4.4 An exceptional event in the neighbouring River Till catchment in 1841 (the Great Till Flood) is attributed to a combination of snow melt, frozen ground and rainfall. However, this mechanism of flooding is not considered as a primary source when compared with fluvial, surface water and groundwater.

2.5 Gauging stations (flow or level)

2.5.1 There is one gauging station within the modelled reach (Avon at Amesbury, NRFA Station 43005) and two gauging stations upstream (East Avon at Upavon, NRFA Station 43014 and West Avon at Upavon, NRFA Station 43017). Further information on these are provided in Section 4 alongside other potential donor sites from neighbouring catchments.

2.6 Other data available and how it has been obtained

2.6.1 A range of additional data are available to provide further supporting information for flow estimation. These are variable in quality and a summary has been provided in Table 2-1.

Table 2-1: Summary of additional data available

Type of data	Data relevant to this study	Data available	Source of data	Details
Check flow gaugings (if planned rating review)	n/a	n/a	n/a	n/a
Historic flood data	Yes	Yes	Internet, Met Office Library, Wiltshire Council, Environment Agency	A range of historic flood information is available, in particular, the British Hydrological Society (BHS) Chronology of British Hydrological Events. Whilst some data provides the date of flooding, observations are limited with little information on the mechanisms, flow, extent and timing of flooding. These are summarised in the 'Flood History' in Annex B.
Flow data for events	Yes	Yes	Environment Agency, National River Flow Archive	15 minute and daily flow data for River Avon at Amesbury, East Avon and West Avon.
Rainfall data for events	Yes	Yes	Environment Agency, Met Office	A range of daily and sub-daily data are available for stations within and around the catchment. These are variable in record length.
Results from previous studies	Yes	No	Journal, Internet, Wiltshire Council	Very limited data are available from previous studies after an extensive review of data provided.
Other information e.g. groundwater, tides etc	Yes	Yes	Environment Agency	Wessex regional groundwater model outputs.

2.7 Initial choice of approach

2.7.1 The FEH statistical method is normally the most appropriate method on highly permeable catchments according to the Environment Agency Flood Estimation Guidelines (2017) (Ref 5).

Conceptual model

2.7.2 The main site of interest is the existing crossing of the A303 over the River Avon in the vicinity of the Countess Roundabout and the potential impacts this exerts on flood extents.

2.7.3 The catchment is highly permeable and catchment wetness influences runoff and flow within the channel. The primary likely cause of flooding within the catchment is groundwater with prolonged periods of elevated flows (i.e. flood volume). There is also the potential for a high rainfall event to result in flooding when combined with high groundwater levels (i.e. catchment is saturated and therefore catchment reacts like an impermeable catchment).

2.7.4 The Nine Mile River is ephemeral and flows are heavily influenced by groundwater levels.

Unusual catchment features

2.7.5 The catchment is highly permeable with BFIHOST values all >0.89 and up to 0.96 at the flow estimation points.

2.7.6 SPRHOST is less than 20%, and therefore relevant to assess stations within the WINFAP pooling group and the gauge at Amesbury (used for Enhanced Single Site analysis).

2.7.7 WINFAP v4 doesn't allow user defined values of L-CV and L-SKEW to be entered following permeable adjustment. An alternative approach of removing of 'non-flood' years (QMED less than QMED/2) from the AMAX series for stations within the pooling group with an SPRHOST less than 20% will be undertaken to compare with the unadjusted pooling group. This approach is a compromise on the permeable adjustment procedure described within FEH although its application has minor effects on the growth curve factors (similar to the permeable adjustment procedure).

2.7.8 The catchment is not highly urbanised (largest value of URBEXT2000 is 0.0147 at the downstream boundary) and whilst there is development planned up to 2026, planning policy should ensure that there is no increase in flood risk to and from these sites including allowances for climate change.

2.7.9 The catchment is not influenced by pumping, reservoirs or extensive floodplain storage.

Initial choice of method and reasons

2.7.10 QMED has been estimated using AMAX data from the Avon at Amesbury (NRFA Station 43005) for the main River Avon flows. The FEH statistical method has been selected to obtain peak flow estimates. These peak flow estimates will be used to scale hydrographs derived using ReFH2.2 software, to provide inflows to the hydraulic model. Use of local data from Station 43005 (Avon at Amesbury) will be utilised as it is located within reach of interest. This includes using Enhanced Single Site (ESS) analysis.

2.7.11 In addition, a comparison will be made between the ESS growth curve and historic data analysis using the maximum likelihood method available within WINFAPv4.

2.7.12 Use FEH statistical method will be used in conjunction with a donor adjusted QMED for Nine Mile River. The pooling group and growth curve factors will be compared with those derived from ESS of the Avon at Amesbury and separate growth curves used for the River Avon and Nine Mile River as required.

2.7.13 Flow estimates using ReFH2.2 have also been undertaken to provide an independent comparison with the FEH statistical values and to generate design hydrographs to scale final flow estimates.

2.7.14 WINFAPv4 and ReFH2.2 software version have been used in this study.

3 Location of flood estimates

3.1 Summary of subject sites

3.1.1 Table 3-1 lists the locations of subject sites that are illustrated in Figure 3-1. There are no major inflows on the River Avon within the model reach apart from the Nine Mile River (NM01, NM02) at Durrington. Subject sites, AVON01 and NMR01 are model inflow locations with AVON02, AVON03, AVON04 and NMR02 used as check locations and to distribute intervening flows. AVON03 is located at approximately the location of the crossing of the A303 to the east of Countess Roundabout.

Table 3-1: Summary of subject sites.

Site Code	Watercourse	Site	Easting	Northing	Area on FEH web service (km^2)	Revised area if altered
AVON01	Avon	Upstream model extent on River Avon	415600	146550	268.7	Not amended
AVON02	Avon	Immediately upstream of confluence with Nine Mile River	416250	143350	277.77	Not amended
NMR01	Nine Mile River	Upstream model extent on Nine Mile River	419150	145100	31.41	Not amended
NMR02	Nine Mile River	Immediately upstream of confluence with River Avon	416350	143300	39.82	Not amended
AVON03	Avon	River Avon at existingA303 crossing	415850	142200	324.66	Not amended
AVON04	Avon	Downstream model extent	413150	137550	366.03	Not amended

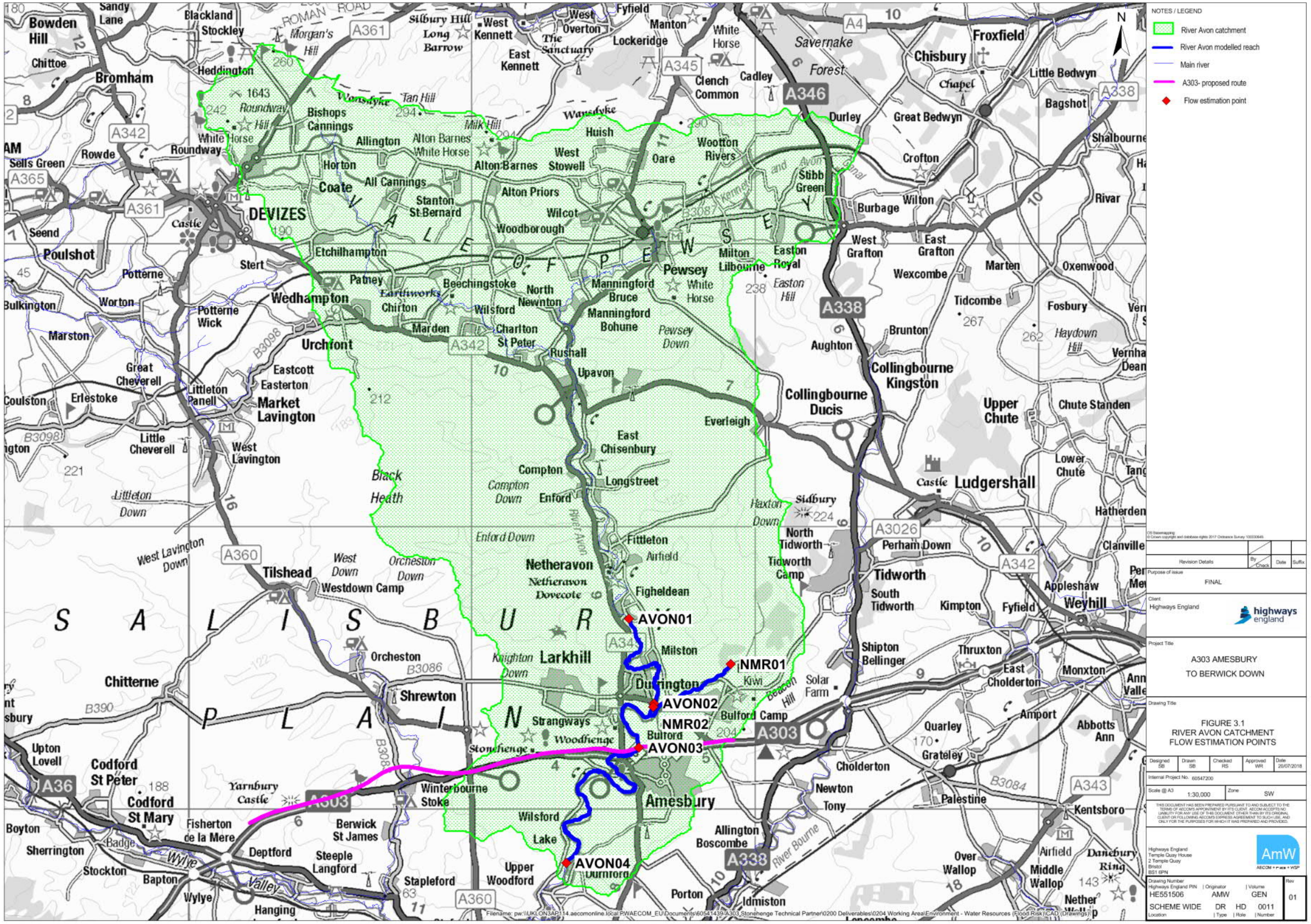


Figure 3-1: Flow estimation points

3.2 Subject site catchment descriptors

3.2.1 Table 3-2 lists the key catchment descriptors for each of the subject sites, these remain unchanged based on the following review commentary.

3.2.2 The catchment boundaries were checked through visual inspection against OS 1:25,000 mapping. These correspond well to the OS mapping and therefore no amendments were made to catchment areas.

3.2.3 Soils were checked through inspection of Soilscapes (http://www.landis.org.uk/soilscapes/), these are identified as shallow lime rich over chalk across the majority of the catchment. Within the valley base, soils are freely draining lime rich loamy soils. Thin soils and chalk were noted during a site visit in October 2017. In addition, the underlying bedrock and superficial deposits correspond well with overlying soil type based on an inspection of the BGS Geology of Britain (http://mapapps.bgs.ac.uk/geologyofbritain/home.html).

Table 3-2: Important catchment descriptors at subject sites (incorporating any changes made).

Site Code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT2000	FPEXT
AVON01	1.00	0.33	0.894	19.63	51.6	746	11.70	0.0094	0.0744
AVON02	1.00	0.33	0.894	24.35	51.2	745	11.59	0.0112	0.0750
NMR01	1.00	0.34	0.969	7.49	44.6	747	4.74	0.0006	0.0347
NMR02	1.00	0.34	0.966	9.55	46.5	742	4.95	0.0219	0.0367
AVON03	1.00	0.35	0.903	24.93	50.6	744	10.69	0.0131	0.0704
AVON04	1.00	0.33	0.894	19.63	51.6	746	11.70	0.0094	0.0744

3.2.4 URBEXT2000 values from the FEH web service have been used. The catchment is not heavily urbanised and whilst minor adjustments could be made to urban extents, these are unlikely to impact flow estimates or flows at the point of interest when considering the large upstream catchment area

3.2.5 Whilst the catchments are not considered to be urbanised, with the largest URBEXT2000 value of 0.0147 (AVON04) on the River Avon and 0.0219 (NMR02) on the Nine Mile River, the Environment Agency Flood Estimation Guidelines (2017) (Ref 5) recommend carrying out an urban adjustment for all QMED estimates to avoid a discontinuity even when URBEXT2000 is equal or less than 0.03.

3.2.6 WINFAP v4 adjusts both QMED (using the UAF) and L-moments (L-CV and L-Skew) within the software. UAF ranges between a value of 1.039 (AVON01) and 1.065 (AVON04) on the River Avon and between 1.005 (NMR01) and 1.116 (NMR02) on the Nine Mile River, therefore increasing QMED at all locations. The change in L-CV and L-Skew is minimal when applying urbanisation to the growth curve factors with a maximum of 0.003 for L-CV and 0.004 for L-Skew.

4 Statistical Method

4.1 Review of potential QMED donor sites

4.1.1 Potential donor sites have been identified and are provided in Table 4-1. Further information on the data available and rating equations for the donor sites are provided in Table 4-2 and Table 4-3. There are a number of donor sites available with one being within the model domain, two upstream of the model domain and the remainder within wider Hampshire Avon catchment.

4.1.2 For the River Avon, Station 43005 (Avon at Amesbury) is preferable when comparing BFIHOST and BGS Geological and Hydrogeological Mapping. It is considered suitable when considering FARL (>0.95) and is also essentially rural (URBEXT2000 < 0.03). The station also has a long period of record (51 years).

4.1.3 For the Nine Mile River, Stations 43014 and 43017 are considered suitable as donors when comparing BFIHOST. Both stations have a long record length although the upper limit of the rating for Station 43017 is below QMED and therefore less confidence can be placed on flow values from this station.

4.1.4 The preferred donor station for QMED estimation on the River Avon is Station 43005 (River Avon at Amesbury). This gauge is located within the modelled reach, has a long record and a reliable rating based on the NRFA. In terms of potential donors for the Nine Mile River catchment, Stations 43014 and 43017 are considered most suitable as the catchment area of Station 43005 is approximately 10 times the catchment area of the subject sites for this tributary.

Table 4-1: Local gauging stations

Watercourse	Station Name	NRFA number (used in FEH)	Grid Reference	Catchment area (km^2)	BFIHOST	FPEXT	URBEXT2000
Chitterne Brook	Codford	43801	ST970401	69.7	0.974	0.0246	0.0008
East Avon	Upavon	43014	SU133559	85.8	0.838	0.0700	0.0117
West Avon	Upavon	43017	SU133559	84.6	0.872	0.1188	0.0112
Avon	Amesbury	43005	SU151413	323.7 (326.47)*	0.903	0.0710	0.0132
Wylye	Stockton Park	43024	ST975393	254.8	0.925	n/a	n/a
Wylye	South Newton	43008	SU086342	445.4	0.937	0.0518	0.0102
Bourne	Laverstock	43004	SU156303	163.6	0.952	0.0561	0.0237

* catchment area in brackets from FEH catchment descriptors and differs slightly area provided by NRFA.

4.2 Data available at each flow gauging station

4.2.1 Table 4-2 provides a summary of the data available for each of the potential donor sites from neighbouring catchments.

Table 4-2: Local gauging stations

Station Name	Start and end date on NRFA	Updated for this study?	Suitable for QMED?	Suitable for pooing?	Data quality check needed?	Other comments on station and flow data quality e.g. information from NRFA Peak Flows, trends in peaks, outliers.
Codford	Jan 1972 to present	No	Yes	No	Yes	Whilst NRFA indicates start date as 1972, peak flow (AMAX) data is only available from 1993 onwards. There are large periods of missing data in early record (up to 1998). There are 'non' flood years within the record (AMAX < QMED/2). Refer to Station Info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43801
Upavon (East	Jan 1970 to	No	Yes	Yes	No	No missing data according to NRFA,

Station Name	Start and end date on NRFA	Updated for this study?	Suitable for QMED?	Suitable for pooing?	Data quality check needed?	Other comments on station and flow data quality e.g. information from NRFA Peak Flows, trends in peaks, outliers.
Avon)	present					long period of record and gauged above QMED (within 29% of AMAX3). Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43014
Upavon (West Avon)	Jan 1970 to present	No	Yes	No	Yes	No missing data according to NRFA, long period of record and gauged to within 17% of QMED. However, rating not validated beyond QMED due to too few high flow gaugings. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43017
Amesbury	Jan 1965 to present	No	Yes	Yes	No	Long period of record and station measures over the full range of flows with no bypassing or out of bank flow. Gauged beyond AMAX3. Small amount of data missing over period of record (73 days in total). Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43005
Stockton Park	May 1994 to present	No	No	No	No	This station is not within the HiFlows dataset and information is only available for daily mean flows. This hasn't been used further. within the analysis. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43024
South Newton	Jan 1966 to	No	Yes	Yes	Yes	Long period of record and gauged above

Station Name	Start and end date on NRFA	Updated for this study?	Suitable for QMED?	Suitable for pooing?	Data quality check needed?	Other comments on station and flow data quality e.g. information from NRFA Peak Flows, trends in peaks, outliers.
	present					QMED and AMAX3. Data between 1986 and 1991 missing but no explanatory notes on NRFA. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43008
Laverstock	Oct 1964 to present	No	Yes	Yes	No	Long period of record and gauged above QMED and AMAX3. Data between 1984 and 1992 missing but no explanatory notes on NRFA. Refer to Station info on NRFA for further information: http://nrfa.ceh.ac.uk/data/station/info/43004.

4.3 Rating equations

4.3.1 Whilst commentary on rating equations has been provided in Table 4-3, for the purposes of this study, a detailed review of existing rating equations does not form part of the required deliverables for this project.

Table 4-3: Summary of information on 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 rating
Codford	Theoretical rating. Upper limit of rating is above QMED. Extrapolated beyond stage of 0.80 m.	No	Note: few spot flow gaugings, none are above QMED. Weir drowns at stage of 0.44 m but no significant bypassing. Two ratings have been applied over period of record, however, these are the same on NRFA notes.
Upavon (East Avon)	Theoretical rating. Upper limit of rating is above QMED. Extrapolated beyond stage of 0.73 m.	No	Note: few spot flow gaugings available but gauged to within 29% of AMAX3.

Upavon (West Avon)	Theoretical rating. Upper limit of rating is below QMED. Extrapolated beyond stage of 0.4 m.	No	Note: few high flow gaugings available and rating only validated to QMED (gauged to within 17% of QMED).
Amesbury	Empirical rating, extrapolated beyond stage of 1 m. Re-rated in 2001 to include exceptional event in December 2000. Environment Agency is very confident in stage/discharge relationship.	No	Note: large range of spot flow gaugings across full range of flow and above AMAX3.
Stockton Park	Unavailable on NRFA	No	This station is not within the HiFlows dataset and information is only available for daily mean flows.
South Newton	Empirical rating, extrapolated based on flood gaugings.	No	Note: large range of spot flow gaugings across full range of flow and above AMAX3.
Laverstock	Theoretical rating, re-calibrated at low flows. Upper limit of rating is above QMED. Extrapolated beyond upper limit of rating at 0.8 m.	No	Note: large range of spot flow gaugings across full range of flow and above AMAX3.

4.4 Selected donor sites

4.4.1 Table 4-4 provides an overview of the selected donor site for adjusting QMED from catchment descriptors.

Table 4-4: Selected donor sites

NRFA Number	Reasons for choosing or rejecting	Method (AMAX or POT)	Adjusted for climatic variation?	QMED from flow data (gauged) (m^3s^{-1}) (A)	QMED from flow data – urban influence removed (m^3s^{-1})*	$QMED_{CDs}$ (m^3s^{-1}) (B)	Adjustment Ratio (A/B)
43005	Suitable for QMED for River Avon flow estimation points	AMAX	No	10.8	10.2	7.49	1.36
43014	Suitable for QMED for Nine Mile River flow estimation points (see notes above)	AMAX	No	3.96	3.82	3.58	1.07

* This was undertaken within WINFAPv4.

4.4.2 The urban adjustment approach within WINFAPv4 has been applied to QMED estimates.

4.5 Estimation of QMED at subject sites

4.5.1 For flow estimation points on the River Avon, QMED has been estimated from gauged records and adjusted based on catchment area.

4.5.2 For flow estimates on the Nine Mile River, two methods of estimating QMED were undertaken; QMED adjusted by donor transfer and a variation on the 'Flow variability' (QMED Linking Equation) method available within WINFAPv4.

QMED donor transfer method

4.5.3 As identified in Section 4.4, data transfer using donor site 43005has been undertaken for flow estimation points on the River Avon. This procedure is fully explained in Science Report SC050050 (Ref 6). The QMED adjustment ratio A/B as provided in Table 4-4 is moderated using a power term, 'a', which is a function of the distance between the centroids of the subject site catchment and the donor catchment. The final estimate of QMED is (A/B)a multiplied by the initial estimate from catchment descriptors. However, checks on flow estimates in a downstream direction on the River Avon indicate a reduction in QMED at AVON04 when using the power term.

4.5.4 The moderation term for flow estimation points on the River Avon has therefore been removed to ensure flow estimates increase in a downstream direction (see example in Environment Agency Flood Estimation Guidelines (2017), Figure 13 (Ref 5)). The moderation term has been retained for flow estimates on the Nine Mile River.

4.5.5 The donor adjusted QMED values are provided in Table 4-5 (full AMAX series for donor stations). QMED has been adjusted for urbanisation as per the Environment Agency Flood Estimation Guidelines (2017) (Ref 5).It is noted that caution should be taken when adjusting for urbanisation in permeable catchments. Urban permeable catchments are beyond the range of catchments used to develop the PRUAF (Percentage Runoff Urban Adjustment Factor) equation within the FEH methods.

4.5.6 The values of QMED increase in a downstream direction for the River Avon. However, it is noted that the sum of the AVON02 and NMR02 is greater than QMED at AVON03 by 0.1 m3s-1. It is considered that the timing of peaks are unlikely to coincide and therefore whilst the sum of the estimates for AVON02 and NMR02 are greater than AVON03, the QMED estimates are considered realistic.

Table 4-5: Adjusted QMED values using data transfer using full AMAX series for donor sites

Site Code	$QMED_{CDs}$ (m^3s^{-1}) (rural)	Method	Donor site NRFA number	Distance between centroids (km)	Moderated adjustment factor (*a*)	If more than one donor used		Final estimate of $QMED_{CDs}$ (rural)	Final estimate of $QMED_{CDs}$ (urban)
						Weight if WINFAPv4 method not used	Weighted average of moderated adjustment factor (*a*)		
AVON01	6.61	DT	43005	~~2.06~~*	~~0.64~~*	n/a	n/a	8.99	9.34
AVON02	6.88	DT	43005	~~1.66~~*	~~0.69~~*	n/a	n/a	9.36	9.79
NMR01	0.71	DT	43014	10.9	0.37	n/a	n/a	0.74	0.74
NMR02	0.86	DT	43014	12.1	0.36	n/a	n/a	0.90	1.05
AVON03	7.46	DT	43005	~~0.07~~*	~~0.98~~*	n/a	n/a	10.2	10.7
AVON04	8.05	DT	43005	~~1.55~~*	~~0.70~~*	n/a	n/a	11.0	11.7

*These values have been struck through as the moderation term is not being applied as per reasons provided in the text.

QMED flow variability method

4.5.7 As the Nine Mile River is ungauged and heavily influenced by flows from groundwater emergence, a novel approach using outputs from the Wessex Regional Groundwater Model has been utilised. Outputs from the groundwater model have been used to create and assess the flow duration curve statistics for flows at or exceed 5% (Q5) and 10% (Q10) of the time at NMR01 and NMR02 the Nine Mile River. These have then been used to estimate QMED using the ‘Catchment Descriptors and Flow Variability’ function within WINFAPv4. The results of this method are provided in Table 4-6 and further information on the approach, justification and limitations are provided in Annex C.

4.5.8 This method suggests greater flows than those from catchment descriptors with donor transfer. In addition, the sum of the flows at the confluence with the River Avon (AVON02 + NMR02) gives a greater value than observed flows at Station 43005 although it is appreciated that the timings of peaks are unlikely to coincide based on differing catchment areas.

Table 4-6: Parameter values and QMED estimates using flow variability method

Site Code	Q5 (m^3s^{-1})	Q10 (m^3s^{-1})	BFI	$QMED_{FV}$ (m^3s^{-1}) (rural)	$QMED_{FV}$ (m^3s^{-1}) (urban)
NMR01	0.53	0.38	0.969	1.04	1.05
NMR02	0.88	0.66	0.966	1.60	1.86

4.6 Discussion on QMED

4.6.1 For flow estimation points on the River Avon, QMED has been estimated from gauged records and adjusted based on catchment area. This approach utilises local data from the Avon at Amesbury that has a long record length (51 year) and a gauge that has high confidence based on Environment Agency comments in the NRFA. Section 4.5 identifies that the use of the moderation term to adjust QMED for flow estimation points based on catchment centroids does not provide consistent flows in a downstream direction and therefore the moderation term has not been applied in this instance.

4.6.2 For flow estimates on the Nine Mile River, two methods of estimating QMED were undertaken; QMED adjusted by donor transfer and a variation on the ‘flow variability’ (QMED Linking Equation) method available within WINFAPv4.

4.6.3 The influence of using a donor site reduces the Factorial Standard Error (F.S.E) when compared to solely using catchment descriptors (Ref 7). The reduction in F.S.E for estimation points on the Nine Mile River is illustrated in the following tables for the 68% confidence interval (Table 4-7) and 85% confidence interval (Table 4-8). Note that these tables illustrate QMED adjusted for ‘rural QMED estimates.

Table 4-7: F.S.E – 68% confidence interval using Station 43014 as a donor for QMED estimates on the Nine Mile River

Site Code	$QMED_{CDs}$ (m^3s^{-1})	F.S..E ($QMED_{CDs}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})		$QMED_{Adj}$ (m^3s^{-1})	F.S..E ($QMED_{Adj}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})
NMR01	0.71	1.431	0.49	1.01		0.74	1.395	0.53	1.03
NMR02	0.86	1.431	0.60	1.24		0.90	1.397	0.65	1.26

Table 4-8: F.S.E – 95% confidence interval using Station 43014 as a donor for QMED estimates on the Nine Mile River

Site Code	$QMED_{CDs}$ (m^3s^{-1})	F.S..E ($QMED_{CDs}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})		$QMED_{Adj}$ (m^3s^{-1})	F.S..E ($QMED_{Adj}$)	Lower (m^3s^{-1})	Upper (m^3s^{-1})
NMR01	0.89	1.431	0.34	1.45		0.74	1.395	0.38	1.44
NMR02	1.47	1.431	0.42	1.77		0.90	1.397	0.46	1.76

4.6.4 When comparing with results from the donor transfer method with the 'flow variability' approach, the rural QMED estimates for NMR01 (1.04 m3 s-1) and NMR02 (1.60 m3 s-1) are outside the upper limit for the 68% confidence interval (1.03 m3 s-1 and 1.26 m3 s-1 respectively) but within the 95% confidence interval (1.44 m3 s-1 and 1.76 m3 s-1 respectively).

4.6.5 Whilst the 'flow variability' approach utilising groundwater model predictions provides an alternative method to estimate QMED, an assessment of the F.S.E for flow variability is not easily applied in this instance. There are no level or flow gauges on the Nine Mile River and also there no groundwater emergence surveys available to compare against the outputs of the Wessex Regional groundwater model. It is therefore considered that QMED estimates using donor transferred are the preferred method and have been applied for the purposes of deriving design flows on the Nine Mile River.

4.7 Derivation of pooling groups

4.7.1 For flow estimation points on the River Avon, enhanced single site (ESS) analysis has been selected as the preferred method to estimate growth curve factors. This is considered pragmatic for the following reasons:

4.7.2 Station 43005 (Avon @ Amesbury) has a 48 year period record when non-flood years have been removed. Growth curve factors up to 4% AEP are considered to be representative of peak flows events up to this AEP;

4.7.3 A check was undertaken to compare the stations within the pooling groups for AVON01 to AVON04 against those within the ESS analysis pool of stations. Of the eleven stations within the ESS pool, between seven (AVON01 pooling group) and eleven (AVON03 pooling group) of the stations were also included in the pooling groups for the flow estimation points on the River Avon.

4.7.4 For the Nine Mile River, there is only 8.4 km^2 difference in catchment area between the upstream and downstream flow estimation points (approximately 25% of total catchment at the downstream point). It is considered that a single pooling group is assessed at the downstream boundary and applied to estimate flows for both the upstream and downstream flow estimation points within this catchment.

4.7.5 The Heterogeneity statistic (H2) for each pooling group was assessed using WINFAPv4. This provides an indication of whether a review of the pooling group is required (no, optional, desirable or essential). The similarity of the subject site against stations within the pooling group is assessed by the Similarity Distance Measure (SDM) and is a function of Area, SAAR, FARL an FPEXT. However, it is noted that this has limitations when estimating growth curves on permeable catchments (Ref 6), therefore a review of the pooling groups has been undertaken. The composition of the initial and revised pooling groups is provided in the Annex A.

4.7.6 As per the Environment Agency guidelines, modifications to the pooling group tend to have a relatively minor effect on the final design flow (compared with, for example, the selection of donor sites for QMED). In particular, ‘Section 6.7. – Example: a pooling group’ in Science Report SC0500505 (Ref 6) indicates that apart from the first four or five stations within a pooling group (i.e. lowest SDM), the record length at a station will only have a modest effect its weight within the pooling group (unless the record is very short). The review of the pooling group has therefore mainly focused on the first five stations within each pooling group unless others have been identified that potentially require review. The review of stations is provided in Table 4-9.

Table 4-9: Review of stations from initial pooling groups

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
RIVER AVON	Station 43005	Yes	Sites Investigated 10002 – Ugie @ Inverugie **RETAIN** - SDM is closest to subject site. - Medium period of record (35 years) covering flood rich and flood poor episodes. - Single Site Growth curve is similar to subject site. 20001 – Tyne @ East Linton **RETAIN** - Long period of record (47 years) covering flood rich and flood poor episodes. - Single Site Growth curve is similar to subject site. 53008 – Avon @ Great Somerford **RETAIN** - Long period of record (53 years) covering flood rich and flood poor episodes. - Single Site Growth curve is similar to subject site. 22007 – Wansbeck @ Mitford **RETAIN** - Long period of record (54 years) covering flood rich and flood poor

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			episodes. - Single Site Growth curve is steeper than subject site but likely to be influenced by significant floods in 1962 and 2007. 39006 – Windrush @ Newbridge **RETAIN** - Long period of record (66 years) covering flood rich and flood poor episodes. - BFIHOST is 0.79 and considered permeable. Single site growth curve is less steep than the subject site. The relative frequency of sites with a similar BFIHOST is very low when considering sites available for pooling across the UK. As ESS analysis is being used, the weight apportioned to Station 43005 is greater within the pool and therefore the influence of other sites is lower. Sites with a higher BFIHOST exist but their SDM is lower, therefore sites lower in the ESS pooling group remain unchanged. Stations 39006 and 42010 are considered permeable based on SPRHOST < 20. A review of the AMAX series for each of these stations indicates that for both stations there is only one water year where QMED/2 is less than QMED. No permeable adjustment of these stations has been undertaken as it will have a minimal effect on the resultant growth curve factors.
NINE MILE RIVER	NMR02	No	Sites Investigated 39033 – Winterbourne Stream @ Bagnor **RETAIN** - SDM is closest to subject site. - Chalk dominated catchment with a high BFIHOST similar to subject catchment. - Long period of record (54 years) covering flood rich and flood poor episodes. - AMAX1 is +7 times greater than QMED. This is associated with surface water runoff contributions in July 2007 event. 24007 – Browney @ Lanchester **REMOVE**

Name of pooling group	Site code from whose descriptors pooling group was derived	Subject site treated as gauged (i.e. Enhanced Single Site Analysis)	Changes made to default pooling group, with reasons. Includes sites that were investigated and either retained or removed.
			- BFIHOST is 0.33 and dis-similar in underlying geology. - Hydrographs are prominently peaked and often multi-peaked. - Period of record is 1968 – 1983 (15 AMAX in total) and is considered to be in a 'Flood Poor' period of record. 53017 - Boyd @ Bitton **RETAIN** - Long period of record (43 years) covering flood rich and flood poor episodes. - BFIHOST is 0.49 and clay catchment. 26803 - Water Forlornes @ Driffield **RETAIN** - Chalk dominated catchment with a high BFIHOST similar to subject catchment. - AMAX series covers a 'Flood Rich' period (1997 onwards). 28058 - Henmore Brook @ Ashbourne **REMOVE** - 12 years of usable record but coincides with a 'flood poor' period of record (1970s) - Large period of record rejected following construction of Carsington Reservoir - Responsive catchment 44003 - Asker @ Bridport **RETAIN** - BFIHOST is 0.696. - Station replaced by 44011 (channel modifications but in same location). 44011 – Asker @ East Bridge Bridport **RETAIN** - BFIHOST 0.696 - Period of record from 1996 onwards covering 'flood rich' episodes. Station replaced 44003 (see above).

4.8 Derivation of flood growth curves at subject sites

4.8.1 The revised pooling groups were updated where required and the Goodness of Fit statistic used within WINFAPv4 to identify the best fitting distribution. Table 4-10 provides a summary of the main factors used in derivation of the growth curves for each subject site.

Table 4-10: Selected donor sites

Site Code	Method (SS, P, ESS, FH)	If P, ESS or FH, name of pooling group	Distribution used and reason for choice	Notes on urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustment	Growth Curce Factor (GCF) for 1% AEP
AVON01	ESS	RIVER AVON	GL Distribution – Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Growth curve adjusted for urbanisation at station 43005 but not adjusted further for individual site as only very minor differences noted. Non-flood years removed from Station 43005 to account for permeable nature of catchment.	Location =1.00 Scale =0.227 Shape =-0.214	2.767
AVON02	ESS	RIVER AVON	GL Distribution – Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Growth curve adjusted for urbanisation at station 43005 but not adjusted further for individual site as only very minor differences noted. Non-flood years removed from Station 43005 to account for permeable nature of catchment.	Location =1.00 Scale =0.227 Shape =-0.214	2.767
NMR01	Pooled	NINE MILE RIVER	GEV Distribution – GL Distribution is recommended for UK catchments but GEV distribution fitted best to	Growth curve not adjusted for urbanisation.	Location = 0.837 Scale = 0.445 Shape = -0.001	2.888

Site Code	Method (SS, P, ESS, FH)	If P, ESS or FH, name of pooling group	Distribution used and reason for choice	Notes on urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustment	Growth Curce Factor (GCF) for 1% AEP
			the pooling group.			
NMR02	Pooled	NINE MILE RIVER	GEV Distribution – GL Distribution is recommended for UK catchments but GEV distribution fitted best to the pooling group.	Growth curve not adjusted for urbanisation.	Location = 0.837 Scale = 0.445 Shape = -0.001	2.888
AVON03	ESS	RIVER AVON	GL Distribution – Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Growth curve adjusted for urbanisation at station 43005 but not adjusted further for individual site as only very minor differences noted. Non-flood years removed from Station 43005 to account for permeable nature of catchment.	Location =1.00 Scale =0.227 Shape =-0.214	2.767
AVON04	ESS	RIVER AVON	GL Distribution – Distribution is recommended for UK catchments and this distribution fitted best to the pooling group.	Growth curve adjusted for urbanisation at station 43005 but not adjusted further for individual site as only very minor differences noted. Non-flood years removed from Station 43005 to account for permeable nature of catchment.	Location =1.00 Scale =0.227 Shape =-0.214	2.767

4.9 Flood estimates utilising historic data

4.9.1 The enhanced single site analysis (as described above) provides refined flood estimates based on local gauged data. However, the use of historic data has been explored to identify if additional improvements to these estimates could be made. Peak flow data at Amesbury extends back to 1965 and there is potential to further refine flood estimates using historic data.

4.9.2 Guidelines within the FEH recommend that 'at-site' methods can be used to estimate peak flood flows for AEP's for up to approximately half of the available record length. For the Avon at Amesbury, based on a 48 year period of record (accounting for the removal of non-flood years), this can be used to reliably estimate the 4% AEP event. However, as identified in Section 2.1, design flows up to and beyond the 1% AEP are required.

4.9.3 A review of historic flood records was undertaken for Amesbury but no significant historic flooding was identified. Within the gauged record, the flood on 3rd January 2003 is the largest flow recorded at Amesbury at ca. 28 m3s-1. An expanded search to the wider catchment indicates that Salisbury Cathedral (approximately 19 km downstream) has been subject to flooding on 10 separate occasions since 1309 to the present day.

4.9.4 This anecdotal evidence infers that a flow less than 28 m3s-1 at Amesbury is unlikely to result in flooding at Salisbury, a perception threshold of 30 m3 s-1 was therefore set for historic analysis. It is noted that this is a gross assumption and flooding at Salisbury Cathedral is also influenced by other tributaries of the Hampshire Avon including the River Wylye and River Nadder, which collectively contribute about two thirds of the total catchment area.

4.9.5 As peak flows for the historic events are unknown, Method 1b 'event only' information has been used within WINFAPv4 for the Maximum Likelihood Estimate (MLE) (Ref 8). This requires the gauged annual maxima from the gauge of interest (Amesbury), the number (k) of historical events that have exceed the perception threshold (X0) and the length of historical period represented (h).

4.9.6 The MLE outputs for a range of AEP's are provided in Table 4-11 alongside the growth curve factors. In this instance, WINFAPv4 indicated that 'This data does not appear to fit a generalised logistic distribution'. No confidence intervals were generated using this approach due to the limitations set within the software based on the historic data provided.

4.9.7 A comparison of flows provided in Table 4-11 with those provided for AVON03 and AVON04 (Amesbury gauge located between the two) from enhanced single site analysis (see Table 4-12) indicates that peak flows up to and including the 3.33% AEP event are similar. For peak flow estimates above the 3.33% AEP event, there is an increase above flow estimates at AVON04 (downstream of Amesbury gauge) which reflects a steeper growth curve.

4.9.8 Whilst attempts have been used to reduce uncertainty in flood estimates using local data, the built in limitations of the software, application of method 1b where flows are unknown, the sensitivity of the perception threshold, the potential for floods not to be recorded in the historic record due to other factors (war, plague etc) and the potential influence of other tributaries contributing to inundation of Salisbury Cathedral, it is considered that the Enhanced Single Site Analysis approach be adopted for peak flow estimates.

Table 4-11: Peak flood estimates (m^3s^{-1}) for a range of AEP's using Method 1b for historic data

	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
Growth Curve Factor	1	1.37	1.66	2.00	2.23	2.55	2.84	3.07	5.77
Peak Flow Estimate	11.2	15.2	18.5	22.2	24.7	28.3	31.6	34.1	64.0

4.10 Flood estimates from statistical method

4.10.1 For sites on the River Avon, QMED estimates have been taken directly from the AMAX data at Amesbury and adjusted by donor transfer (including an adjustment for urbanisation). For the Nine Mile River (NMR01 & NMR02), QMED has been estimated from donor adjusted catchment descriptors. QMED estimates have then been multiplied by the respective growth curve factors to provide flood estimates (see Table 4-12) and have been rounded to three significant figures.

Table 4-12: Peak flood estimates (m^3s^{-1}) for a range of AEP's using FEH statistical method

Site Code	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
AVON01	9.34	12.8	15.3	18.0	19.8	22.2	24.3	25.9	46.1
AVON02	9.79	13.4	16.0	18.9	20.8	23.3	25.5	27.2	48.3
NMR01	0.74	1.08	1.31	1.57	1.72	1.93	2.11	2.44	3.58
NMR02	1.05	1.53	1.87	2.22	2.44	2.74	2.99	3.18	5.08
AVON03	10.7	14.6	17.5	19.2	22.7	25.5	27.9	29.7	52.8
AVON04	11.7	16.0	19.2	22.6	24.8	27.8	30.5	32.5	57.7

5 Revitalised flood hydrograph method (ReFH2)

5.1 Parameters for ReFH2 model

5.1.1 The values reported within this section have been estimated using the ReFH2.2 software. These flow estimates have utilised the FEH13 rainfall model and therefore provide an independent comparison against flow estimates derived from the FEH statistical pooling method. Model parameters have only been estimated from catchment descriptors and have not been not been estimated from gauged records (flow and rainfall) as the ReFH calibration utility tool is only applicable for ReFH and not ReFH2.

Table 5-1: Parameter values used within ReFH2

Site Code	Method OPT: Optimisation BR: base flow recession fitting CD: catchment descriptors DT: data transfer	Tp (hours) – Time to peak	C_{max} (mm) – Maximum storage capacity	BL (hours) – Base flow lag	BR – Base flow recharge
AVON01	CD	10.65	1144	92.06	2.38
AVON02	CD	12.07	1144	96.49	2.38
NMR01	CD	6.21	1381	77.29	2.65
NMR02	CD	7.04	1370	81.35	2.64
AVON03	CD	11.87	1164	96.26	2.44
AVON04	CD	13.80	1176	102.0	2.45

5.1.2 Flooding in the River Avon and Nine Mile River catchments is heavily influenced by groundwater levels and baseflow component. The duration of flood events that coincide with high groundwater levels and low intensity rainfall such as in 2013/14 may cause prolonged flooding over extended periods (i.e. weeks rather than hours).

5.1.3 Inspection of hydrographs using 15 min data from the gauge at Amesbury illustrates the responsiveness of the catchment to rainfall combined with groundwater levels through extended hydrograph shape and duration. This is illustrated using two examples provided in Figure 5-1 and Figure 5-2.

5.1.4 Figure 5-1 illustrates that the rising limb is approximately 8 days in duration and the recession limb takes approximately 4 months to recede back to similar levels.

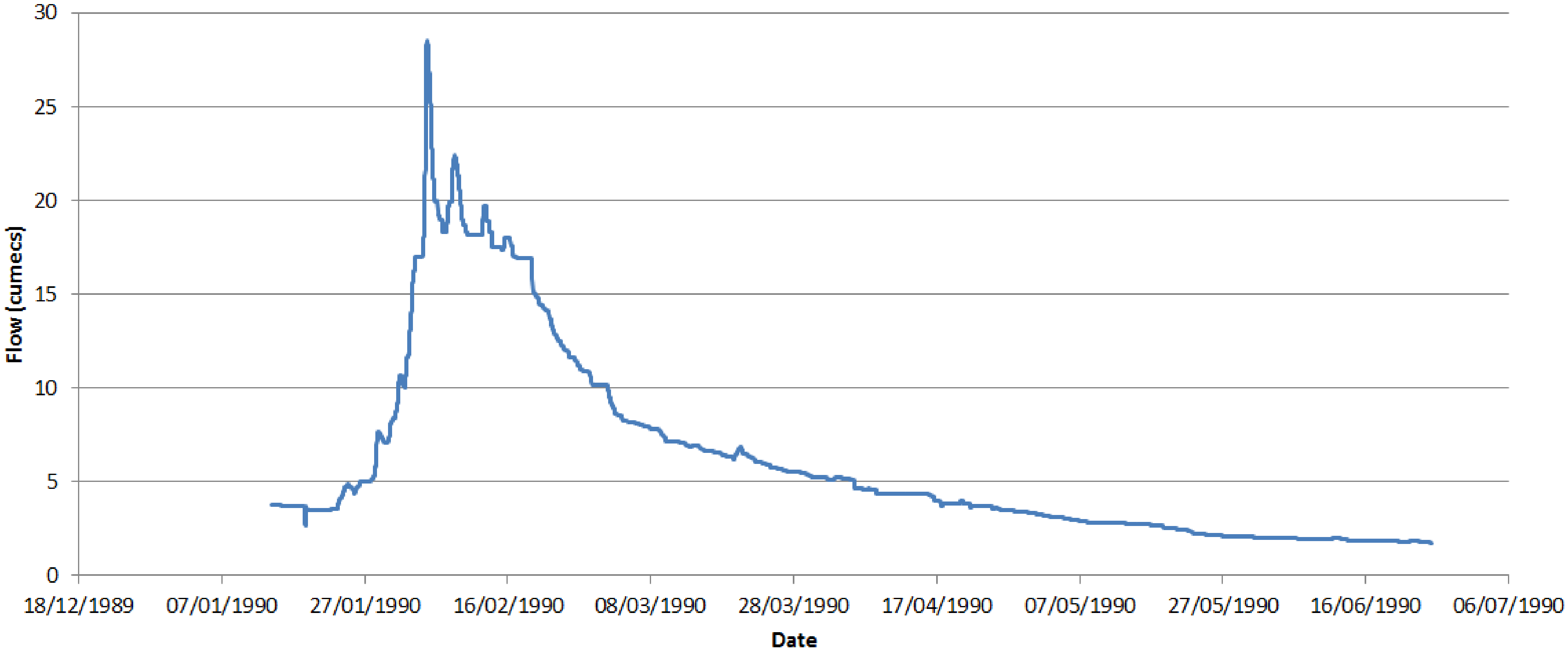


Figure 5-1: Hydrograph from the Avon @ Amesbury illustrating significant duration of rising and falling limbs.

5.1.5 Figure 5-2 illustrates a separate scenario from the winter of 2013/2014. This shows the response to a succession of low intensity, long duration rainfall events that cause a number of peaks over elevated river levels (driven by baseflow). It also illustrates the difficulty with assessing hydrographs to provide a 'design shape' that is representative of 'typical conditions.

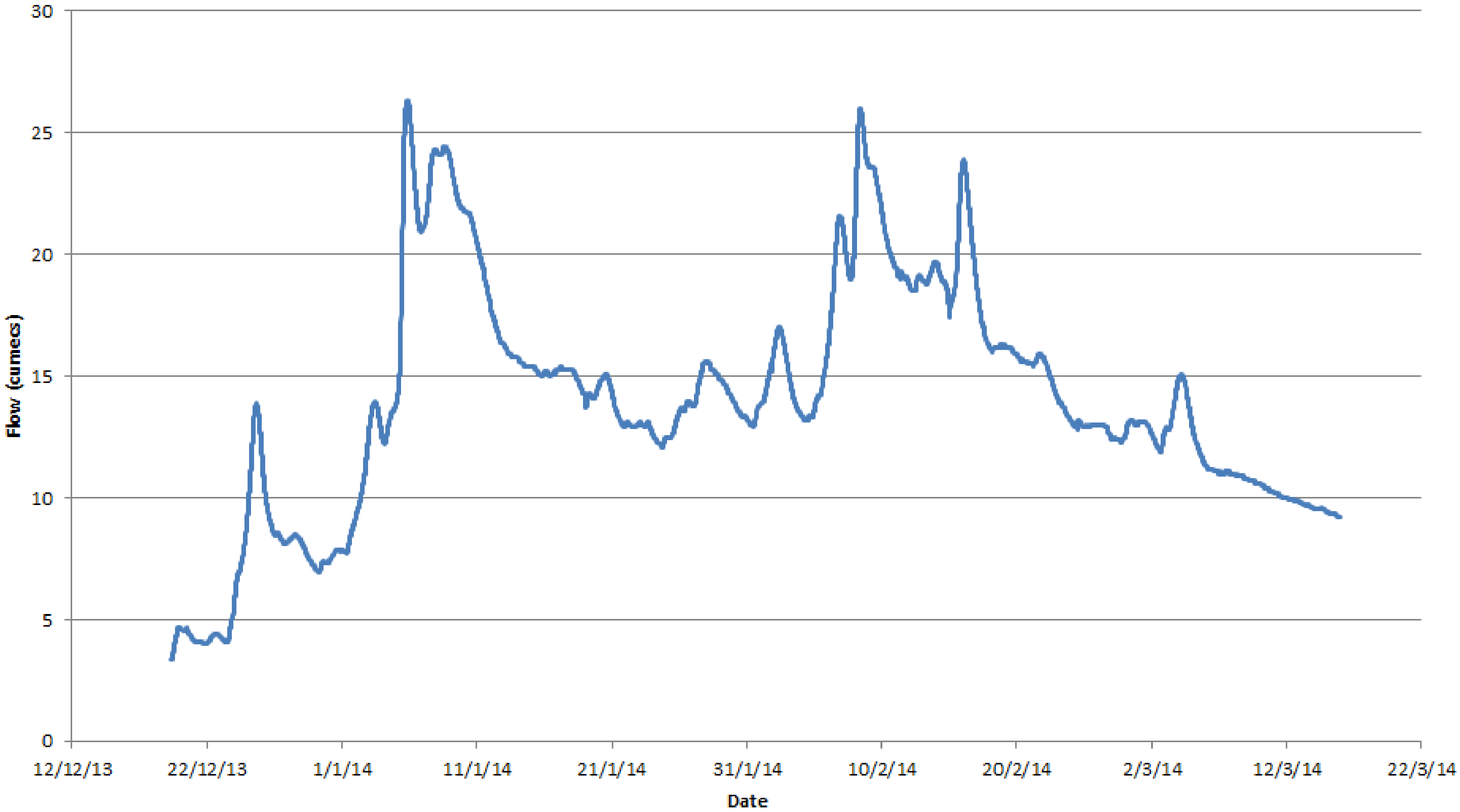


Figure 5-2: Hydrograph from the Avon @ Amesbury illustrating influence of underlying baseflow coupled with low intensity rainfall events.

5.1.6 In order to explore the influence of flood event duration and volume upon flood risk within the study reach of the River Avon, sensitivity testing was undertaken and is documented within the fluvial hydraulic modelling report[1]. The sensitivity testing utilised an extended 800 hour hydrograph containing multiple peaks extracted from Continuous Simulation Modelling (CSM) for Salisbury, undertaken by JBA consulting on behalf of the Environment Agency. Analysis of model outputs indicates that flood duration and volume does not have a significant impact upon maximum flood extent and depth within the modelled reach of the River Avon. However, the duration associated with overbank flows may affect recovery times. Peak flows determine the maximum flood extent and depth, indicating that the hydrological analysis and hydraulic modelling completed is appropriate for assessment of flood risk in the context of the proposed scheme.

5.2 Design events for ReFH2 method

5.2.1 Table 5-2 provides general information on the ReFH2 design events. The catchment is predominately rural with the exception of Pewsey, Amesbury, Bulford and Durrington. No amendments have been made to the urbanisation model parameters because there has been no significant development or planned future development that is likely to significantly impact flooding at the site of interest.

Table 5-2: Design event information

Site Code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Source of design rainfall (FEH13 or FEH99)
AVON01	Winter	18.00	Catchment area	FEH13
AVON02	Winter	22.00	Catchment area	FEH13
NMR01	Winter	11.00	Catchment area	FEH13
NMR02	Winter	13.00	Catchment area	FEH13
AVON03	Winter	22.00	Catchment area	FEH13
AVON04	Winter	26.00	Catchment area	FEH13

[1] AmW (2019) A303 Amesbury to Berwick Down. Environmental Statement. Appendix 11.5: Level 3 Flood Risk Assessment. Annex 1 Part A Fluvial Hydraulic Modelling Report.

5.2.2 It should be noted that summer storms within ReFH2 produce a ‘flashier’ response and greater peak flows. However, due to groundwater inflows being a controlling factor in river levels on the River Avon and that the Nine Mile River is ephemeral, the winter season has been selected for design events.

5.3 Flood estimates from ReFH2

5.3.1 Table 5-2 provides peak flow estimates generated using the ReFH2 method. As per the Revitalised Flood Hydrograph Model ReFH 2.2 Technical Guidance (Ref 9), the urban results are reported. These results take account of the urban extent within the catchment based on URBEXT2000 and are therefore representative of existing conditions.

5.3.2 Flood volumes have also been provided in Table 5-4. Similar to the results for the FEH statistical method, it is noted in Table 5-2 that the sum of the flows from AVON02 and NMR02 exceed flow estimates at AVON03 and a function of different time to peaks. The sum of the volumes for these flow estimation points is consistent in a downstream direction. This illustrates the volume of flow is a key factor in permeable catchments.

Table 5-3: Peak flood estimates (m^3s^{-1}) for a range of AEP's using ReFH2 method

Site Code	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
AVON01	6.12	8.47	10.2	11.7	13.1	14.7	16.2	17.3	33.7
AVON02	6.00	8.27	9.89	11.6	12.7	14.2	15.6	16.8	32.5
NMR01	0.74	1.06	1.29	1.53	1.68	1.88	2.05	2.19	4.26
NMR02	0.98	1.37	1.66	1.95	2.13	2.38	2.60	2.77	5.38
AVON03	6.71	9.32	11.2	13.2	14.4	16.1	17.7	19.0	37.0
AVON04	6.93	9.53	11.4	13.4	14.6	16.4	18.0	19.2	37.5

Table 5-4: Flood estimates (m^3) for a range of AEP's using ReFH2 method

Site Code	50% AEP	20% AEP	10% AEP	5% AEP	3.33% AEP	2% AEP	1.33% AEP	1% AEP	0.1% AEP
AVON01	1197160	1670550	2015660	2370050	2607750	2931230	3226030	3462200	6707260
AVON02	1326510	1842860	2211810	2606150	2853530	3204390	3522940	3780850	7373510
NMR01	97330	139150	168970	200330	219570	245430	268290	286330	556600
NMR02	135590	191300	231840	273320	299270	334660	365770	390200	762830
AVON03	1507000	2093850	2512700	2956840	3237610	3633170	3990320	4270690	8371960
AVON04	1784500	2459070	2950980	3458950	3782540	4240120	4651010	4972610	9753540

6 Discussion and summary of results

6.1 Comparison of results from different methods

6.1.1 Table 6-1 and Table 6-2 provide a comparison of peak flow estimates from the FEH Statistical and ReFH2 methods for QMED and the 1% AEP event, respectively.

6.1.2 These illustrate that for flow estimates on the River Avon, ReFH2.2 typically underestimates flows compared with the FEH statistical method. QMED estimates for the River Avon using the FEH statistical method are based on donor transfer using local data from the gauge at Amesbury. These are preferable over ReFH2 as this method does not utilise local data to generate flow estimates.

6.1.3 For the Nine Mile River, flow estimates are comparable between methods. It is noted that QMED for this location has been estimated from catchment descriptors and adjusted by donor transfer.

Table 6-1: Comparison of FEH Statistical and ReFH2 peak flow estimates (m^3s^{-1}) for QMED

Site Code	FEH Statistical	ReFH2	Ratio (ReFH2/FEH Statistical)
AVON01	9.34	6.12	0.66
AVON02	9.79	6.00	0.61
NMR01	0.74	0.74	1.00
NMR02	1.05	0.98	0.93
AVON03	10.7	6.71	0.63
AVON04	11.7	6.93	0.59

Table 6-2: Comparison of FEH Statistical and ReFH2 peak flow estimates (m^3s^{-1}) for 1% AEP event

Site Code	FEH Statistical	ReFH2	Ratio (ReFH2/FEH Statistical)
AVON01	25.9	17.3	0.67
AVON02	27.2	16.8	0.62
NMR01	2.44	2.19	0.90
NMR02	3.18	2.77	0.87
AVON03	29.7	19.0	0.64
AVON04	32.5	19.2	0.59

6.2 Final choice of method

6.2.1 The final choice of method is to use the FEH Statistical method to estimate peak flows. For flow estimates on the River Avon, enhanced single site analysis is used and for the Nine Mile River pooled analysis is used.

6.2.2 For flows on the River Avon, the final QMED estimates are taken from observed flows at the donor station Avon at Amesbury and scaled accordingly. The moderation term has been omitted following checks on downstream estimates at AVON04.

6.2.3 For the incoming tributary (Nine Mile River), QMED has been estimated using catchment descriptors and then adjusted based on the donor East Avon at Upavon (NRFA Station 43014).

6.2.4 ReFH2 hydrographs have been rescaled using the FEH statistical outputs for the River Avon and Nine Mile River to provide design event hydrographs for use within the hydraulic modelling. These are provided in Annex D.

6.3 Assumptions, limitations and uncertainty

6.3.1 A number of assumptions were required to be made with flow estimates for the River Avon and Nine Mile River. These are:

1. The catchment has a long term gauge (Avon at Amesbury – NRFA Station 43005) within the reach of interest. The Environment Agency has high confidence in the gauge and there is sufficient length of record to determine QMED from flow data (51 years). It is considered that this gauge is also suitable for enhanced single site analysis. The range of stage (level) is only 1 m across all gauged records with no out of bank flows or bypassing records at the gauge.

2. There are a limited number of stations within each pooling group that are considered to be permeable. A permeable adjustment of these stations has not been undertaken as WINFAPv4 does not allow adjustments to L - CV and L-Skew. A check of non-flood years indicates that an adjustment is unlikely to significantly alter resultant growth curve factors.

3. Peak flows within the catchment are influenced by groundwater due to the permeable nature of the catchment. Surface water runoff may also contribute to peak flows depending on catchment wetness i.e. the catchment may respond differently to the same rainfall event depending on antecedent conditions.

4. The catchment is essentially rural with limited development planned in the future. Future development will have limited impact on runoff due to the requirements to manage flood risk set out within the NPPF. As per the Environment Agency Flood Estimation Guidelines (2017) (Ref 5), the effects of urbanisation have been applied even though the catchment is considered to be rural.

5. Historic flood events have been identified through data review. The flood generating processes for these events are variable and include snowmelt combined with frozen ground (1309, 1841), high groundwater levels with prolonged low intensity rainfall (2013/14). Typically, high flows are experienced through a combination of elevated groundwater levels that provide baseflow and rainfall.

6.3.2 The following limitations with regard to the methods applied in this study are acknowledged:

1. The performance of FEH methods for flood estimation in permeable catchments is acknowledged to be less certain than for catchments where BFIHOST is < 0.65.

This is due to the smaller number of permeable catchments within the NRFA dataset when compared to the number of impermeable catchments which FEH methods are predominantly based upon.

2. The FEH statistical method is considered suitable for up to the 0.5% AEP. This method has been used to estimate 0.1% AEP and therefore caution should be used with these flows as they are outside of the range for AEP's.

6.3.3 With regard to uncertainty, the following points are noted:

1. The F.S.E for QMED has been provided for the 68% and 95% confidence intervals to illustrate the upper and lower limit of QMED using a) catchment descriptors only and b) catchment descriptors with a donor adjustment applied (reduces the F.S.E). These have only been provided for QMED estimates on the Nine Mile River as estimates on the River Avon are from observed records. These are provided in Section 4.5.

2. To help reduce uncertainty in QMED from catchment descriptor for the Nine Mile River, a donor station has been used (East Avon at Upavon).

3. Due to the permeable nature of the catchment and absence of gauged data within the Nine Mile River catchment, there is likely to be greater uncertainty in the growth curve estimates. For the River Avon, the use of enhanced single site analysis has been undertaken to give greater weight to local data. In addition, further use of local historic data was utilised, however, the findings from this were not incorporated into the final flows due to the limitations identified by WINFAPv4.

6.3.4 The flood estimates in this report have been developed for the purposes of this study only to assess the impact of flooding within the vicinity of a new flyover at Countess Roundabout. The results may be applicable for other studies, although users should undertake necessary checks for additional data (e.g. updates to AMAX data for QMED and stations within the pooling group, more recent flooding, updated estimation techniques).

6.3.5 It is noted that emergence surveys and spot flow measurements, undertaken at bank full level, would aid understanding of flows in the Nine Mile River and help to reduce uncertainty.

6.4 Checks

6.4.1 A series of checks have been undertaken to assess the flow estimates.

6.4.2 The results are consistent with an increase in flow in a downstream direction.

6.4.3 The Environment Agency 'Flood Map for Planning' illustrates that the flood extent for 1% AEP event at the gauge (Avon at Amesbury) is significantly out of bank. A review of information on the NRFA site indicates that the highest recorded flow on record is 28.2 m3s-1 (1st March 2003) and is considered to be between a 2% AEP and a 1.33% AEP event. No out of bank flows have been experienced over the period of record (51 years) and bankfull stage is indicated to be 1.37 m. Based on the stage discharge relationship, the approximate 1% AEP event stage is 1.1m and is therefore below bankfull level. The hydraulic model has been run for the 1% AEP event and indicates that flows remain in bank upstream and out of bank downstream of the Church Street Bridge. This corresponds with observed out of bank flows downstream of the bridge in 2014 (although noting that the 2014 event was not a 1% AEP event).

6.4.4 The typical range for growth curve factors is between 2.1 to 4.0 for the 1% AEP event. In this study, the growth curve factors are:

- RIVER AVON = 2.767
- NINE MILE RIVER = 2.888

6.4.5 Both values sit within the typical range and are therefore considered realistic.

6.4.6 The 0.1%/1% AEP event ratios using the FEH Statistical method range between 1.47 and 1.78. These values are generally within the expected range for UK catchments. The ratio is lower where the GEV distribution has been applied to the pooling group and is expected as the GEV distribution generally results in shallower growth curves than the GL distribution.

6.4.7 The specific runoff for the 1% AEP event, the specific discharge rates are:

- AVON01 = 1.01 l/s/ha
- AVON 02 =1.01 l/s/ha
- NMR01 = 0.78 l/s/ha
- NMR02 = 0.80 l/s/ha
- AVON 03 = 0.94 l/s/ha
- AVON 04 = 0.91 l/s/ha

6.4.8 These are considered to be lower than normal based on recommended limiting discharges of between 2 l/s/ha and 6 l/s/ha for development plots (typically 1 to 10 ha). The low specific discharges are due to the permeable nature of the catchment.

6.4.9 No information from previous studies (e.g. Flood Risk Assessments, Journal Papers) is available against which to compare previous flow estimates.

7 ANNEX A – Pooling Groups

7.1 Initial Pooling Groups

7.1.1 Table A-1 to Table A-5 provide the initial pooling groups derived from WINFAPv4 for each subject site.

Table A-1: Initial Pooling Group for River Avon

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
43005 (Avon @ Amesbury)	0	48	11.106	0.228	0.275	0.976
10002 (Ugie @ Inverugie)	0.201	35	45.871	0.291	0.243	0.431
20001 (Tyne @ East Linton)	0.272	47	57.803	0.32	0.193	1.525
53008 (Avon @ Great Somerford)	0.312	53	37.08	0.264	0.216	0.209
22007 (Wansbeck @ Mitford)	0.323	54	98.399	0.309	0.284	0.711
39006 (Windrush @ Newbridge)	0.344	66	11.2	0.202	0.252	0.809
14001 (Eden @ Kemback)	0.404	39	40.417	0.176	0.032	1.09
55021 (Lugg @ Butts Bridge)	0.415	45	45.768	0.165	0.066	1.069
42010 (Itchen @ Highbridge & Allbrook Total)	0.451	58	9.676	0.158	0.193	0.518
39034 (Evenlode @ Cassington Mill)	0.459	45	20.9	0.164	0.286	2.818
55029 (Monnow @ Grosmont)	0.494	43	155.954	0.141	0.036	1.664
22009 (Coquet @ Rothbury)	0.55	41	133.004	0.262	0.245	0.179
Total		574				
Weighted means				0.228	0.212	

Table A-2: Initial Pooling Group for Nine Mile River

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
39033 (Winterbourne Stream @ Bagnor)	0.198	54	0.404	0.344	0.386	2.411
24007 (Browney @ Lanchester)	0.323	15	10.981	0.222	0.212	2.838
53017 (Boyd @ Bitton)	0.333	43	13.82	0.247	0.106	0.069
26803 (Water Forlornes @ Driffield)	0.374	17	0.437	0.3	0.112	0.319
28058 (Henmore Brook @ Ashbourne)	0.396	12	9.006	0.155	-0.064	1.678
44011 (Asker @ East Bridge Bridport)	0.52	21	16.8	0.239	0.112	0.427
44003 (Asker @ Bridport)	0.52	14	12.354	0.224	0.17	1.124
42011 (Hamble @ Frogmill)	0.523	44	8.282	0.167	0.073	0.89
20006 (Biel Water @ Belton House)	0.566	28	11.748	0.375	0.128	1.225
41020 (Bevern Stream @ Clappers Bridge)	0.578	47	13.9	0.205	0.17	0.669
43806 (Wylye @ Brixton Deverill)	0.6	25	2.08	0.376	0.211	0.594
41022 (Lod @ Halfway Bridge)	0.628	46	16.26	0.288	0.181	0.317
36004 (Chad Brook @ Long Melford)	0.634	49	5.321	0.292	0.178	0.605
44013 (Piddle @ Little Puddle)	0.658	23	1.103	0.463	0.254	2.045
30004 (Lymn @ Partney Mill)	0.665	54	6.983	0.231	0.046	0.5
49004 (Gannel @ Gwills)	0.669	47	15.022	0.258	0.105	0.289
Total		539				
Weighted means				0.275	0.16	

7.2 Revised Pooling Groups

7.2.1 Following the pooling group review outlined in Section 4.7, Table A-3 provides details of the revised pooling groups for the Nine Mile River. The pooling group for the River Avon remains unchanged from the initial group following review.

Table A-3: Revised Pooling Group for Nine Mile River

Station	SDM	Years of Data	QMED from AMAX	L-CV	L-SKEW	Discordancy
39033 (Winterbourne Stream @ Bagnor)	0.198	54	0.404	0.344	0.386	2.88
53017 (Boyd @ Bitton)	0.333	43	13.82	0.247	0.106	0.157
26803 (Water Forlornes @ Driffield)	0.374	17	0.437	0.3	0.112	0.409
44011 (Asker @ East Bridge Bridport)	0.52	21	16.8	0.239	0.112	1.097
44003 (Asker @ Bridport)	0.52	14	12.354	0.224	0.17	1.023
42011 (Hamble @ Frogmill)	0.523	44	8.282	0.167	0.073	0.952
20006 (Biel Water @ Belton House)	0.566	28	11.748	0.375	0.128	1.528
41020 (Bevern Stream @ Clappers Bridge)	0.578	47	13.9	0.205	0.17	0.651
43806 (Wylye @ Brixton Deverill)	0.6	25	2.08	0.376	0.211	0.534
41022 (Lod @ Halfway Bridge)	0.628	46	16.26	0.288	0.181	0.817
36004 (Chad Brook @ Long Melford)	0.634	49	5.321	0.292	0.178	0.694
44013 (Piddle @ Little Puddle)	0.658	23	1.103	0.463	0.254	1.824
30004 (Lymn @ Partney Mill)	0.665	54	6.983	0.231	0.046	0.768
49004 (Gannel @ Gwills)	0.669	47	15.022	0.258	0.105	0.666
Total		512				
Weighted means		512		0.286	0.167	

8 ANNEX B – Historical Flood Record

8.1 Flood History

8.1.1 A range of sources have been used to identify the flood history in the River Till catchment. These include:

- Journal papers;
- BHS Chronology of British Hydrological Events;
- Information provided by the Environment Agency and Wiltshire Council that includes reports, photos and other information;
- Internet searches including newspaper articles, photos and planning applications.

8.1.2 Table B-1 provides a chronological history of flooding within the River Avon and Nine Mile River catchment of significant note. The detail of information in some instances is very poor and only indicates that flooding has occurred but with little further information on the source, magnitude or impacts.

Table B-1: Flood chronology for the River Avon and Nine Mile River catchment

Date	Description
19 January 1309	BHS Chronology of British Hydrological Events indicates – "*A sudden thaw after a great frost caused the water so fast to rise that Salisbury Cathedral was flooded".* In addition *"On the 17th and 18th January the water rose so high as it had not been known to do for many years before; even so as to come to the feet of Kings, which stand at the west door of the choir of Salisbury Cathedral. The stone niches in the Chore Screen are still there and if this level is determined it would appear that this might well be the second oldest flood-mark in England*."
February 1635	BHS Chronology of British Hydrological Events indicates *"Salisbury cathedral has been in past generations liable to serious floods. In February 1635 the officiants rode on horseback into the choir to perform divine service*". No further information available.
1637	BHS Chronology of British Hydrological Events indicates *"There was [in Salisbury Cathedral] a flood again in 1637"*. No further information available.
1724	BHS Chronology of British Hydrological Events – reference from 1774 indicates a 1724 flood at Salisbury. No further information available.
1726	BHS Chronology of British Hydrological Events indicates *"An exceptional flood in Salisbury inundated the cathedral to a depth of a foot" and "In 1726, the water in the Cathedral rose so rapidly during divine service that a pulpit for preaching was erected in the choir as the water in the body of the church being nearly a foot deep"*.
1774	BHS Chronology of British Hydrological Events indicates 1774 *"Greater than 1724"* flood at Salisbury. No further information available.
20th September 1775	BHS Chronology of British Hydrological Events indicates *"In the afternoon, a most violent storm of rain and hail, accompanied with more dreadful thunder and vivid lightening than had ever been remembered by the oldest person living, fell in Oxford and Salisbury, and other places in their neighbourhood. Several streets were overflowed; the lightning was almost one continued flash for two hours, the fourth-western firmament, in particular, frequently appeared one vast expanse of fire".* This suggests that pluvial flooding was the main source of flooding in Salisbury.

Date	Description
1824	BHS Chronology of British Hydrological Events indicates *"More recently about 1824 in the time of George IV. the late Mr. J. Harding remembered the Cathedral [at Salisbury] being more than once flooded with water to the depth of several inches over the nave and the aisles. But he never recollected to have seen it reach the level of the choir though the water was standing underground a little below the pavement of the S.E. choir transept"*
1828	BHS Chronology of British Hydrological Events indicates *"Fisherton Street area flooded for several days. Nave, Cloisters, and Chapter House of Cathedral inundated"*. No further information available.
16th January 1841	Report within the Wiltshire Independent, 21st January 1841 on the Great Till Flood indicates that the flood passed downstream to affect Salisbury and water from the Avon came up to the cathedral doors. Flooding mechanism from snowmelt, frozen ground and rainfall. Further information available various articles but main focus is the River Till.
26th November 1852	BHS Chronology of British Hydrological Events – water came up in pools in Salisbury Cathedral and Chapter House. As the autumn previously had been excessively wet it is likely that this was a groundwater effect.
February 1883	BHS Chronology of British Hydrological Events – rainfall observer noted that *"The Avon Valley was flooded from the 11th to 24th, the flood on the 12th being highest for many years".* No further information for upstream of Salisbury, main information for Downton (downstream of Salisbury).
20th December 1910	BHS Chronology of British Hydrological Events – Observer at Salisbury noted 'Great Floods'. No further information available.
21st August 1912	BHS Chronology of British Hydrological Events – Quote from the Times about the difficult conditions in Salisbury Plain military camps due to the wet cold weather. *"Many of the camping places are half flooded with surface water, the ground being practically waterlogged".* No further information identified.
5th January 1915	Series of postcards/photos of flooding on Countess Road & Countess Bridge, Amesbury provided by the Environment Agency. Flooding reported elsewhere in the catchment in BHS Chronology of British Hydrological Events, in particular, Salisbury Cathedral where reports of between 3 to 13 inches are reported (search term 'Salisbury'). Three photos provided by Environment Agency of flooding in Bulford, indicates flooding by Bulford Church and also near Bulford Manor where overbank flows caused flooding to the hospital.
24th July 1915	3 photos provided by Environment Agency (all same location). Information limited to 'Floods followed the thunderstorm which occurred at Amesbury on Saturday. It was accompanied by a heavy hailstorm' and 'Glimpse of Saturday's remarkable storm. Scene at Amesbury, Salisbury Plain. Those districts which experienced the violent downpour on Saturday will not soon forget it. The hailstones were as large as marbles, and "snow-drifts" were common, the general appearance of lawns and housetops being for half an hour more that of the depth of winter'.
1943	Single photo of flooded field provided by Environment Agency. No further information provided on location, date or properties affected.
23rd June 1946	BHS Chronology of British Hydrological Events indicates *"At Salisbury the rain exceeded all local records since the great storm of 28th June 1917, when the centre of the downfall [sic] was at Bruton, Somerset. The serious [urban] flooding which resulted was said to be unparalleled in living memory. "Salisbury (Manor Road) raingauge caught 2.04 inches in 40 minutes, and Salisbury (Atherton House) 2.12 inches in 50 minutes*". This suggests pluvial flooding was the main contributing source as

Date	Description
	opposed to fluvial.
1947	BHS Chronology of British Hydrological Events – "*the (Hampshire) Avon river had already overflowed its banks above Salisbury*". No further information available.
1974	Single photo of South Mill Road provided by Environment Agency. No further information provided on location, date or properties affected.
2000/1	Photos taken from Queenstown Bridge and also across the water meadows to south west of Amesbury – provided by the Environment Agency. No further information on date, timing, properties affected. Information from Wiltshire Council that one property flooded in Durrington. Unknown source and no further information available.
12th September 2008	Information from Wiltshire Council – number of properties affected in Durrington by 'flash flooding' caused by intense rainfall and exceedance of drainage capacity (and blocked drains).
Winter 2013 /2014	Flooding of High Street in Bulford (January and February 2014), two properties affected by fluvial flooding based on Environment Agency information.

9 ANNEX C – QMED Linking Equation & Flow Variability

9.1 Background

9.1.1 In Section 4.5, an additional method of estimating QMED has been utilised within WINFAPv4. The following information provides the rational in using this approach and a novel approach to its application for estimating QMED on the Nine Mile River.

9.1.2 The QMED Linking Equation has been developed for use within WINFAPv4. This method utilises gauged records for within bank, non flood flows for estimating QMED. The requirements for estimating QMED using this method are:

- Gauged estimates of the Daily Mean Flow (DMF) that are equalled or exceeded for 5% of the time (Q5DMF) and 10% (Q10DMF) of the time; and
- BFI – the value of Base Flow Index calculated directly from the daily mean flow series for a gauging station (not to be confused with BFIHOST).

9.1.3 In addition, the average drainage path slope (DPSBAR) is required from the FEH catchment descriptors.

9.2 Available data and approach

9.2.1 As identified in Section 4.1, there are no flow gauges present within the Nine Mile River catchment. A novel approach has therefore been adopted to cross reference with available data on the River Avon at Amesbury and use of data outputs from the Wessex Regional Groundwater Model. The following steps have been taken:

1. Assess DMF for Station 43005 (River Avon @ Amesbury) using NRFA data for the period of record 1965-2016. This required analysing the DMF within HEC-DSSVue to calculate Q5DMF and Q10DMF. BFI was identified as 0.91 from the NRFA.

2. Assess outputs from the Wessex Regional Groundwater Model for the same location as Station 43005. Due to the spatial and temporal resolution of the model, data are available as tri-monthly outputs. Outputs were analysed using HEC-DSSVue to calculate Q5DMF and Q10DMF.

3. Q5DMF and Q10DMF were compared from the two data sources and also for the wider flow duration curve (see Figure C.1 and Table C.1). These illustrate that Q5DMF and Q10DMF are considered to be reasonably similar with less than +/- 2% difference between the values. It is noted that whilst greater differences are observed from Q50 to Q99, this is likely to be a function of the temporal resolution of the output data from the Wessex Groundwater Model. This is expected because this is when a greater percentage of a given flow is exceeded and when there is likely to be greatest variability in flow i.e. due to the temporal resolution of the groundwater model this flow variability is diluted.

4. Analyse outputs from the Wessex Regional Groundwater Model for each flow estimation point on the Nine Mile River (NMR01 and NMR02) using HEC-DSSVue to calculate Q5DMF and Q10DMF.

5. Use Q5DMF and Q10DMF values within WINFAPv4 to estimate QMED using QMED Linking Equation. In the absence of BFI from a mean daily flow series, the use of BFIHOST in this instance was considered appropriate. This is justified when comparing the BFI (0.91) for the DMF at Station 43005 and the BFIHOST value (0.903) from FEH catchment descriptors at the same location.

9.3 Wessex Groundwater Model Limitations

9.3.1 The Wessex Model comprises separately a recharge model and a groundwater model, this is described in further detail in the Numerical Model Report, Appendix 11.4: Annex 1 that covers the groundwater modelling aspects of the project. A brief summary of key model components are as follows:

- Grid cells are 250 m by 250 m
- Model time interval is 10 day stress periods (tri-monthly)
- Model time horizon is 1965 to 2016. The period 1965 -1969 is a 'warm up period' to allow initial conditions to be set and well calibrated at periods of interest early in the simulation period (e.g. 1976 drought).
- The recharge model requires rainfall inputs, potential evapotranspiration (PE), land use, soil type, geology, crop type and urban mains leakage.
- Runoff is routed according to Digital Terrain Mapping and stream cells mapped according to OS mapping.
- The recharge model calculates recharge to the underlying aquifer and runoff to streams (directly and via interflow). This creates a MODFLOW recharge file and stream file for use as input to the groundwater model.

9.3.2 Whilst appropriate for modelling recharge and groundwater at the basin scale, it is acknowledged that the grid cell and time steps introduce uncertainties when applying to a higher resolution. The regional model has been calibrated by the Environment Agency to groundwater levels and stream flows through their Wessex Basin Groundwater Modelling Study Phase 4 (Ref 10).

9.3.3 It is noted that no groundwater emergence data is available for the Nine Mile River, therefore comparison against the Wessex Regional Groundwater Model outputs is unachievable. This introduces a limitation to the application of these data for the QMED Linking Equation based on outputs of the groundwater model.

9.4 Summary

9.4.1 In the absence of gauged data on the Nine Mile River, the estimation of QMED for this ephemeral stream is challenging. QMED from catchment descriptors should be used as a 'last resort' and it is preferable to utilise local data where available (e.g. donor transfer). The QMED Linking Equation provides a new method in catchments where high flow data may not be available but the use of daily mean flows can provide a refined estimation over catchment descriptors.

9.4.2 Whilst the Nine Mile River is ungauged, the use of emergence flows from the Wessex Regional Groundwater Model has been considered. Flow duration statistics for flow equal or exceeded for 5% (Q5) of the time and 10% (Q10) of the time are comparable from the groundwater model when compared with daily mean flows on the River Avon at Amesbury.

9.4.3 It is noted that there are limitations with the outputs of the Wessex Regional Groundwater Model, in particular, the temporal resolution being tri-monthly timesteps. In addition, there is no groundwater emergence data for comparison with groundwater model outputs. This introduces greater uncertainty in application of this method when compared to QMED from catchment descriptors and adjusted by a donor station. The use of a donor adjusted QMED has therefore been adopted for the Nine Mile River.

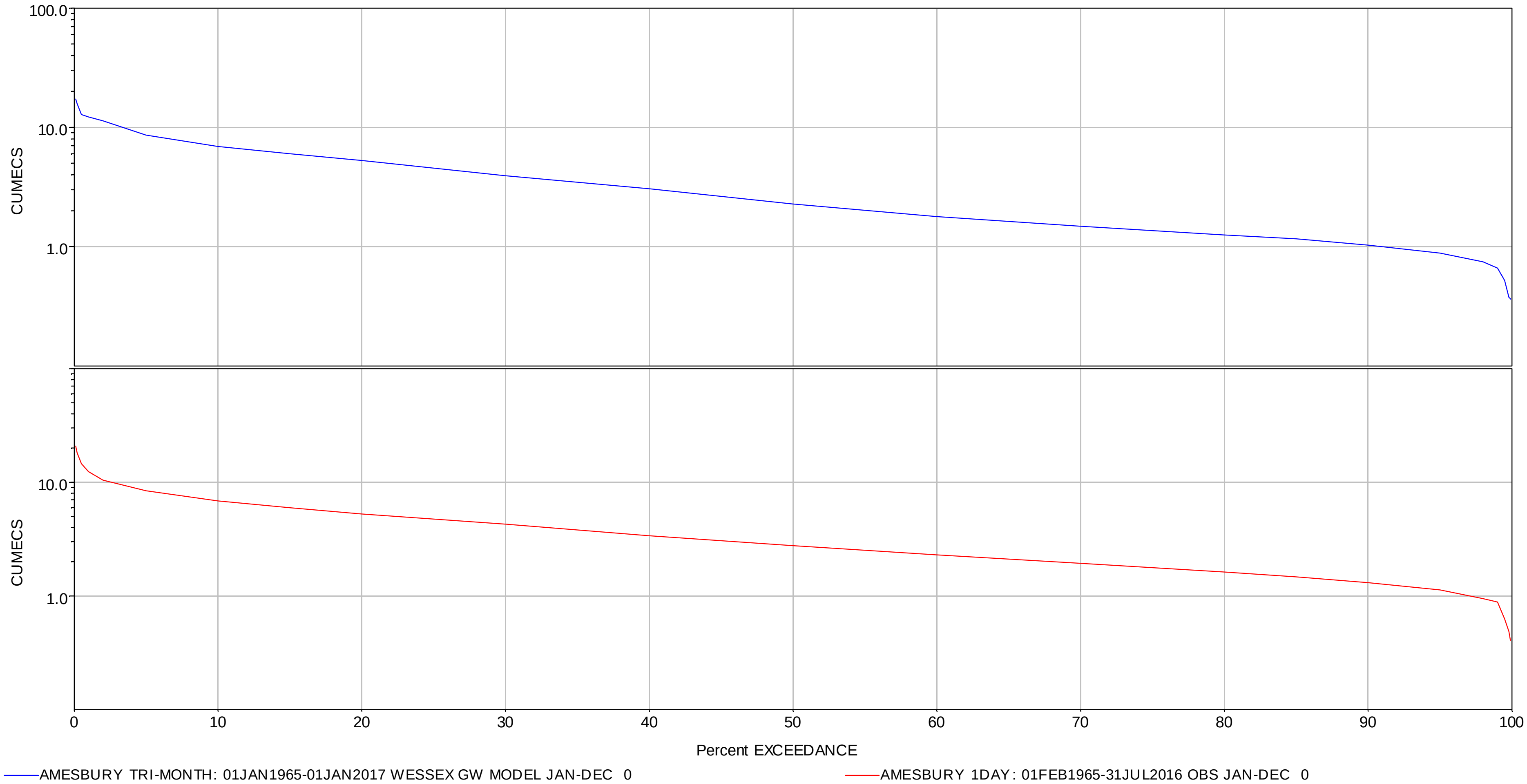


Figure C.1: Comparison of flow duration curves from Wessex Groundwater Model (upper graph) and Daily Mean Flows at Amesbury (lower graph)

Table C.1: Comparison of Flow Duration Curve statistics

Flow Duration Curve	Percentage of time flow (m^3s^{-1}) at or exceeded															
	1	2	5	10	15	20	30	40	50	60	70	80	85	90	95	99
Wessex Groundwater Model	12.21	11.32	8.60	6.91	6.01	5.27	3.94	3.06	2.28	1.79	1.48	1.25	1.16	1.03	0.88	0.66
Avon @ Amesbury	12.35	10.40	8.39	6.83	5.95	5.25	4.27	3.38	2.77	2.30	1.93	1.62	1.47	1.31	1.13	0.88
% difference between flows	1%	-9%	-2%	-1%	-1%	0%	8%	9%	18%	22%	24%	23%	21%	21%	22%	25%

10 ANNEX D – Design Event Hydrographs

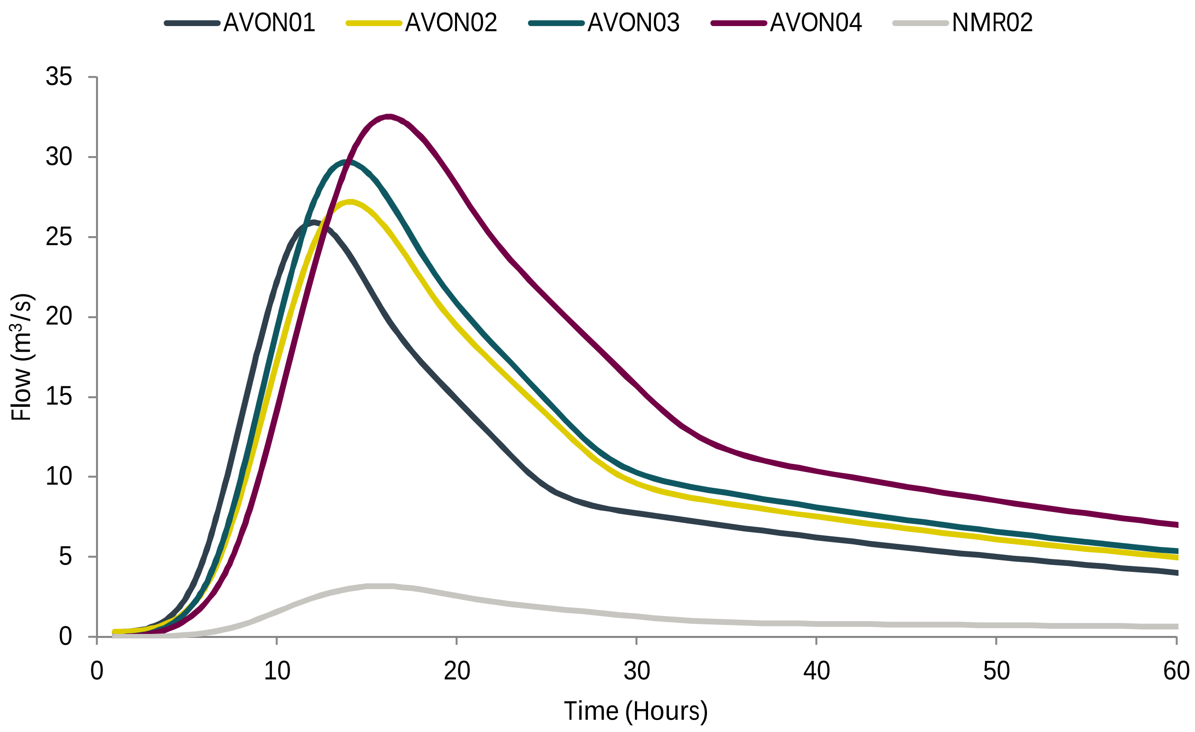


Figure D.1: Inflow Hydrographs Scaled to FEH Peak Flow- 1% AEP

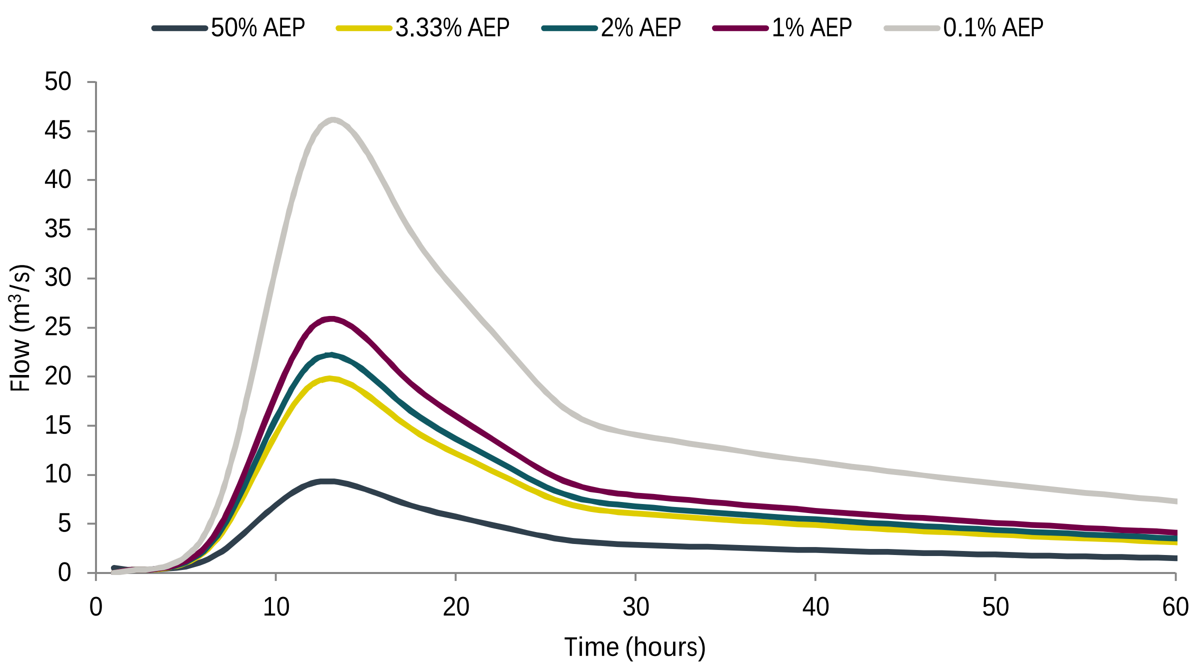


Figure D.2: AVON01 Inflow Hydrographs Scaled to FEH Peak Flow

Abbreviations List

AM	Annual maxima
AREA	Catchment area (km^2)
BFI	Base flow index
BFIHOST	Base flow index derived using the HOST soil classification
DPLBAR	Mean drainage path length (km)
DPSBAR	Mean drainage path slope (m/km)
EA	Environment Agency
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FPEXT	Floodplain extent
FSR	Flood Studies Report
HOST	Hydrology of soil types
NRFA	National River Flow Archive
POT	Peaks over threshold
QMED	Median annual flood (with a AEP of 50%)
ReFH	Revitalised flood hydrograph method – used for rainfall runoff method
SAAR	standard average annual rainfall (SAAR)
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
WINFAP	Windows Frequency Analysis Package – used for FEH statistical method

References

Ref 1 National Planning Policy Framework accessed July 2018, available at: https://www.gov.uk/government/publications/national-planning-policy-framework--2

Ref 2 Planning Practice Guidance accessed July 2018, available at: https://www.gov.uk/government/collections/planning-practice-guidance

Ref 3 National Policy Statement for National Networks accessed July 2018, available at: https://www.gov.uk/government/publications/national-policy-statement-for-national-networks

Ref 4 Flood risk assessment: climate change allowances accessed July 2018, available at: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances.

Ref 5 Environment Agency (2017), Flood Estimation Guidelines, Technical Guidance 197_08.

Ref 6 Kjeldson, T.R., Jones, D.A. and Bayliss, A.C. (2008) Improving the FEH statistical procedures for flood frequency estimation, Science Report SC050050, Environment Agency/Defra.

Ref 7 Kjeldsen, T.R., Jones, D.A., and Morris, D.G. (2014), Using multiple donor sites for enhanced flood estimation in ungauged catchments, Water Resources Research, 50, 6646-6657.

Ref 8 Environment Agency Technical Guidance 12_17 – 'Using local data to reduce uncertainty in flood frequency estimation'. Available only on request from the Environment Agency.

Ref 9 The Revitalised Flood Hydrograph Model – ReFH2.2 Technical Guidance (2016), Wallingford Hydrosolutions Ltd.

This document is also available on our website at **www.gov.uk/highways**

If you have any enquiries about this publication email
info@highwaysengland.co.uk
or call **0300 123 5000***.

*Calls to 03 numbers cost no more than a national rate call to an 01 or 02 number and must count towards any inclusive minutes in the same way as 01 and 02 calls.
These rules apply to calls from any type of line including mobile, BT, other fixed line on payphone.
Calls may be recorded or monitored