A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Level 3 Flood Risk Assessment

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

October 2018

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Executive Summary

This Flood Risk Assessment (FRA) has been prepared to assess the risk of flooding to 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 Environmental Management Plan (OEMP) (Appendix 2.2).

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 largely coincide with the corresponding Flood Zones produced by the Environment Agency. The baseline modelling flood extents for the River Avon show a substantial decrease in comparison to the equivalent Flood Zones produced by the Environment Agency. The current Environment Agency fluvial flood maps of the study site at both River Till and River Avon are based on broad-scale hydraulic modelling information. As such, the Environment Agency have agreed to our hydraulic modelling methodology to create a more detailed and site specific assessment of the design fluvial flood events of the River Till and River Avon.

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 pluvial 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). The current Environment Agency Flood Risk from Surface Water flood maps are based on broad-scale hydraulic modelling information. As such, the Environment Agency have agreed to our hydraulic modelling methodology 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.

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. 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 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 Flood Risk Assessment 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.

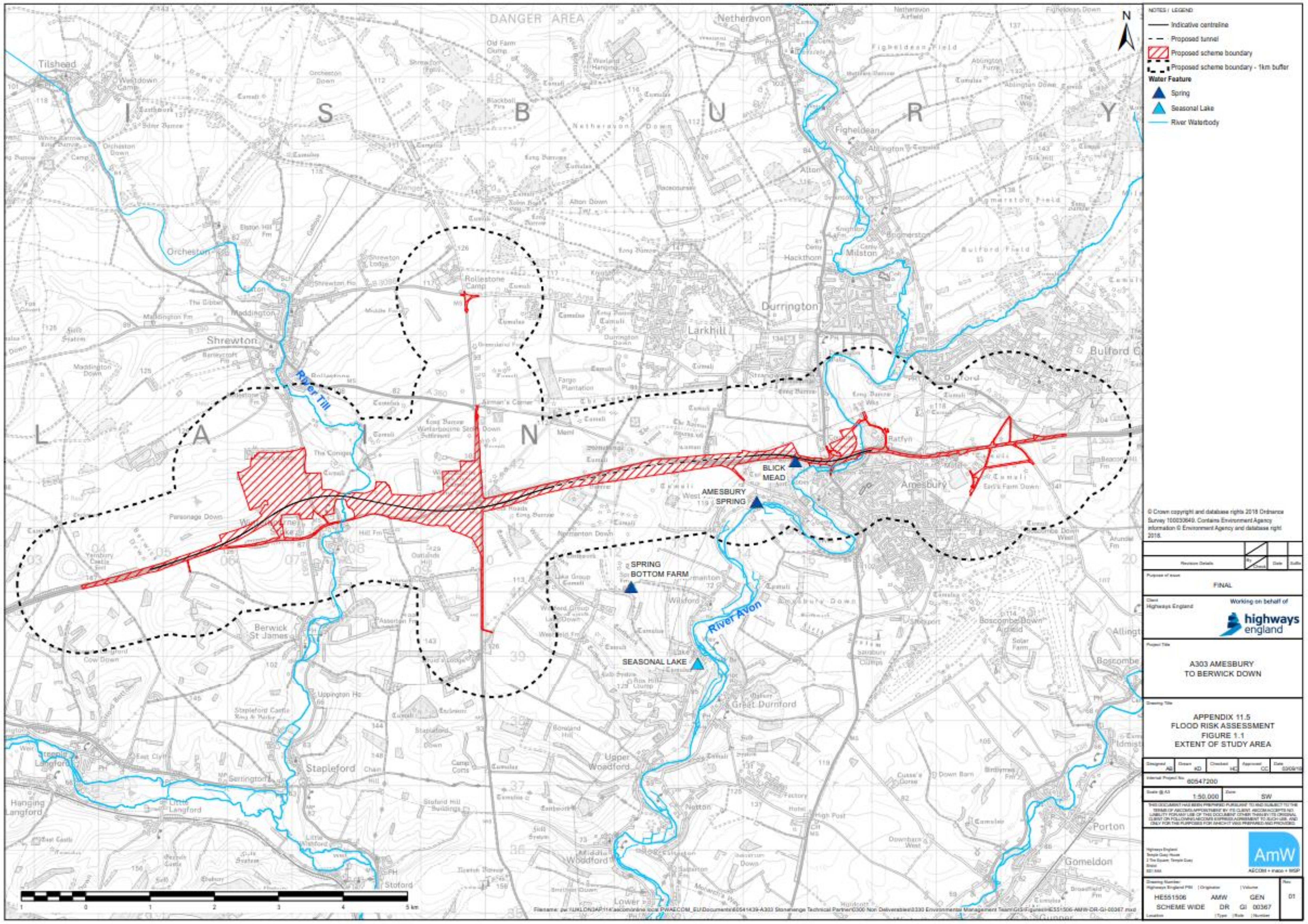

Figure 1.1: Extent of Study Area

2 Study Area Hydrological Context

2.1 Overview

2.1.1 The 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 EA 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.

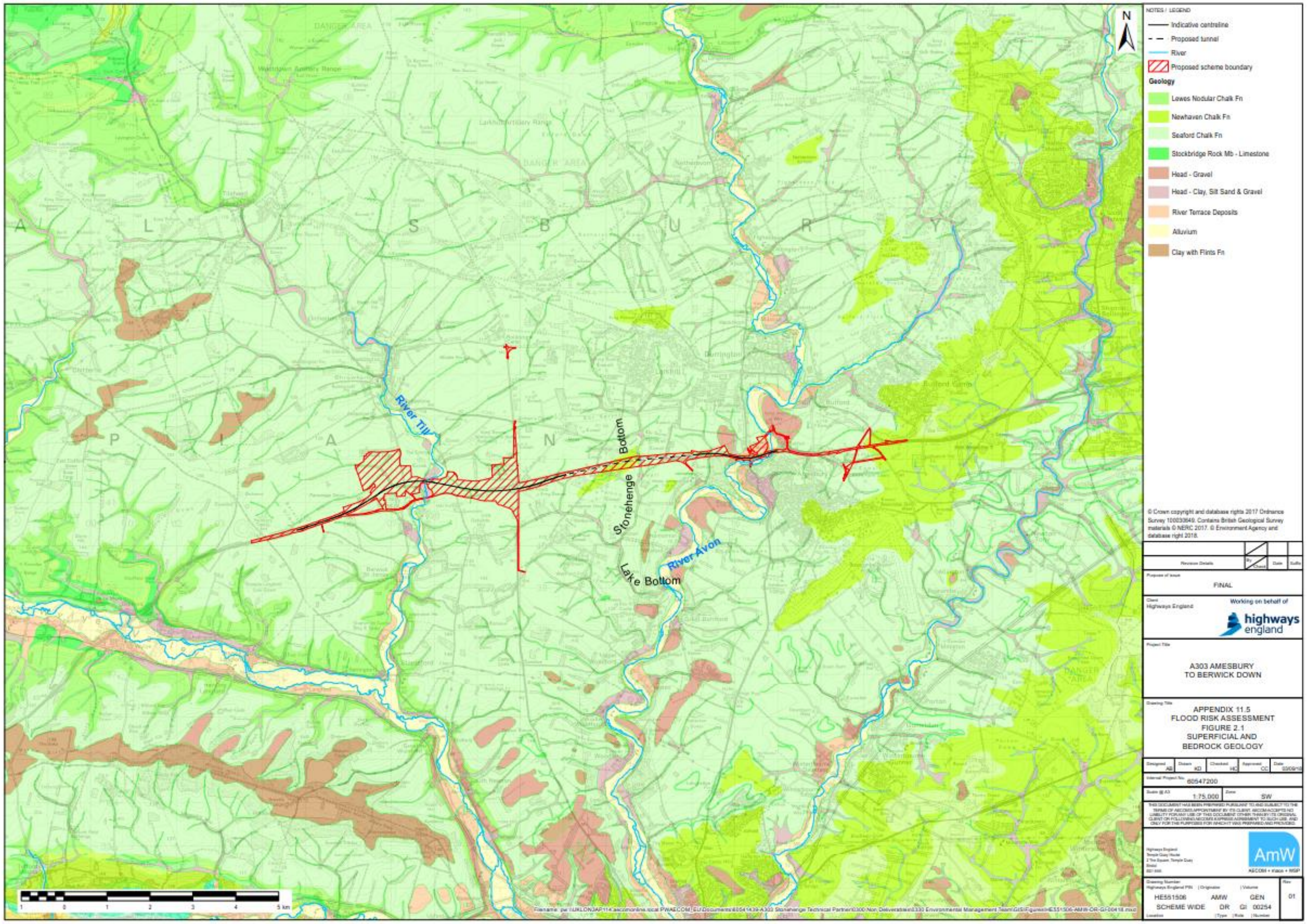

Figure 2.1: Superficial and Bedrock Geology

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 EA 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
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
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
December	80.2	86	73.0	81.7
Annual Average	**753.1**	**770.4**	**760.9**	**748.6**

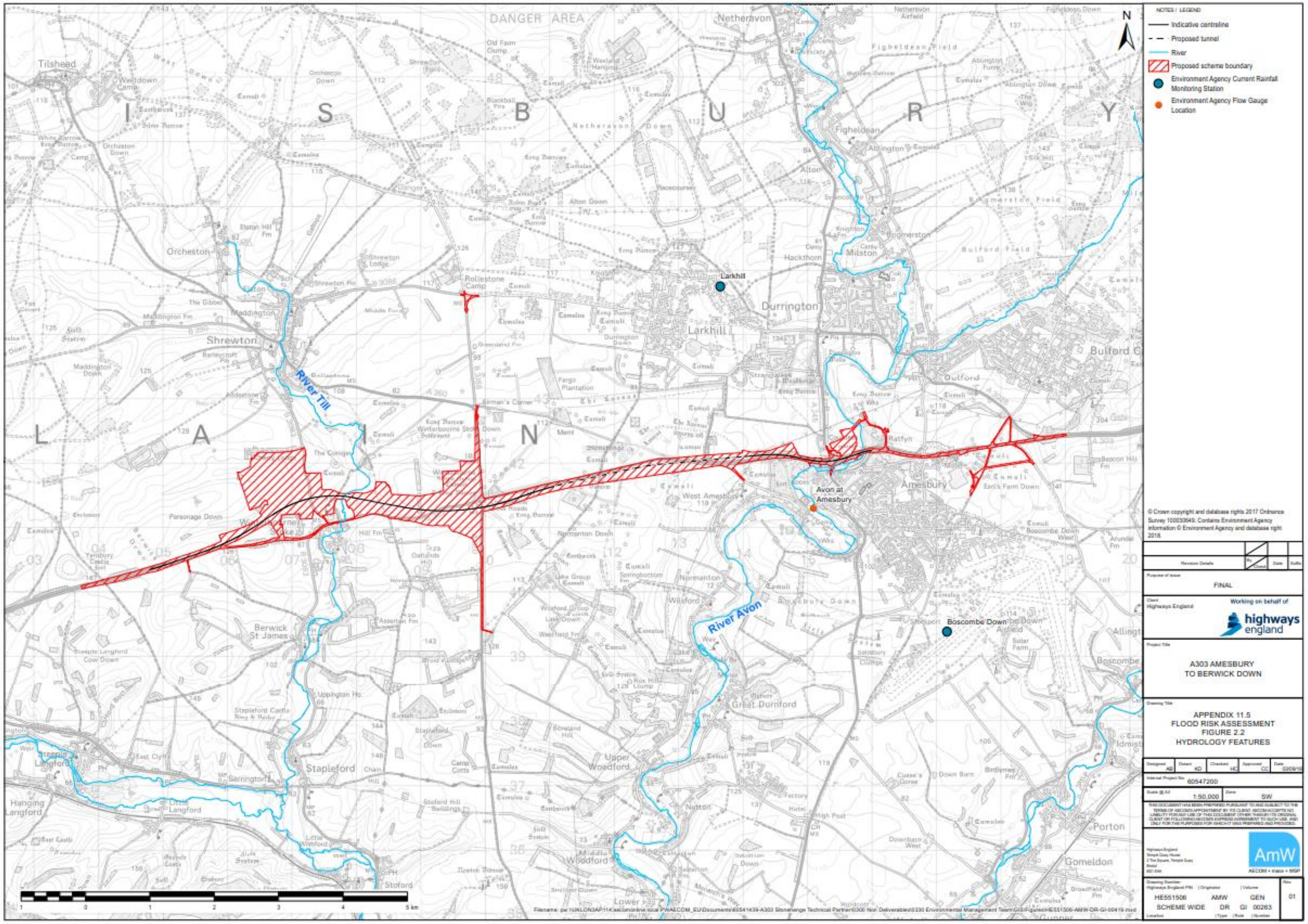

Figure 2.2: Hydrology features

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 The Scheme

3.1.1 The 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 junction between the A303 and A360 to the west of and outside the WHS, replacing the existing Longbarrow roundabout.

c) A twin-bore tunnel approximately 2 miles (3.3km) long, past Stonehenge.

d) A new junction between the A303 and A345 at the existing Countess roundabout.

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

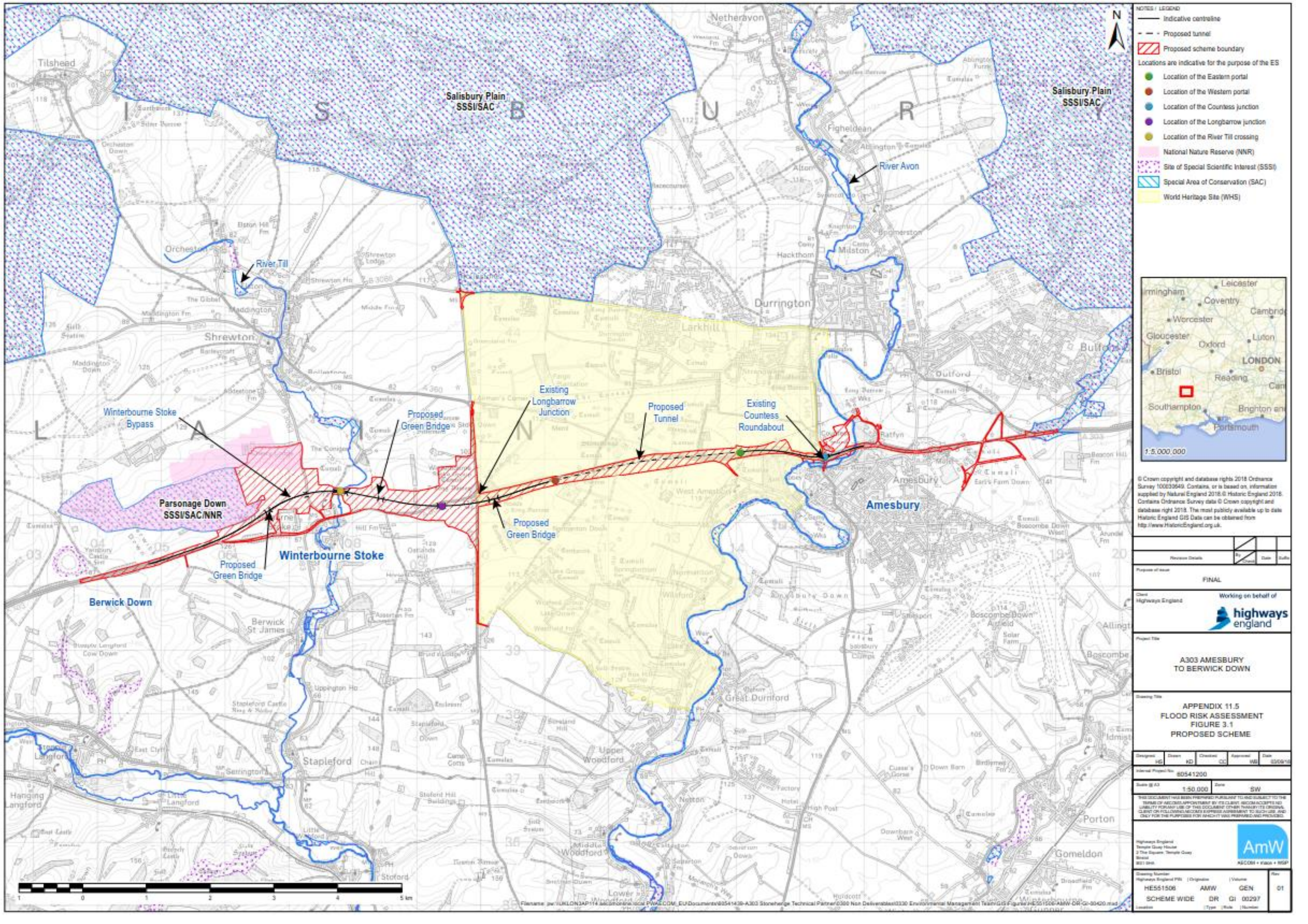

Figure 3.1: Proposed scheme

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.

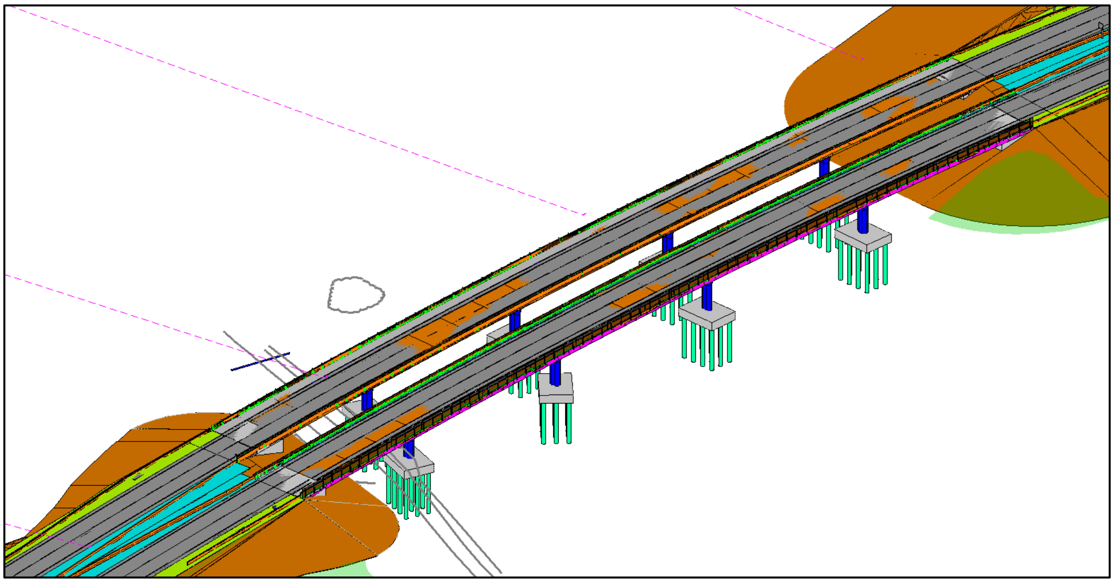

Figure 3.2: Proposed River Till Viaduct Configuration

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.

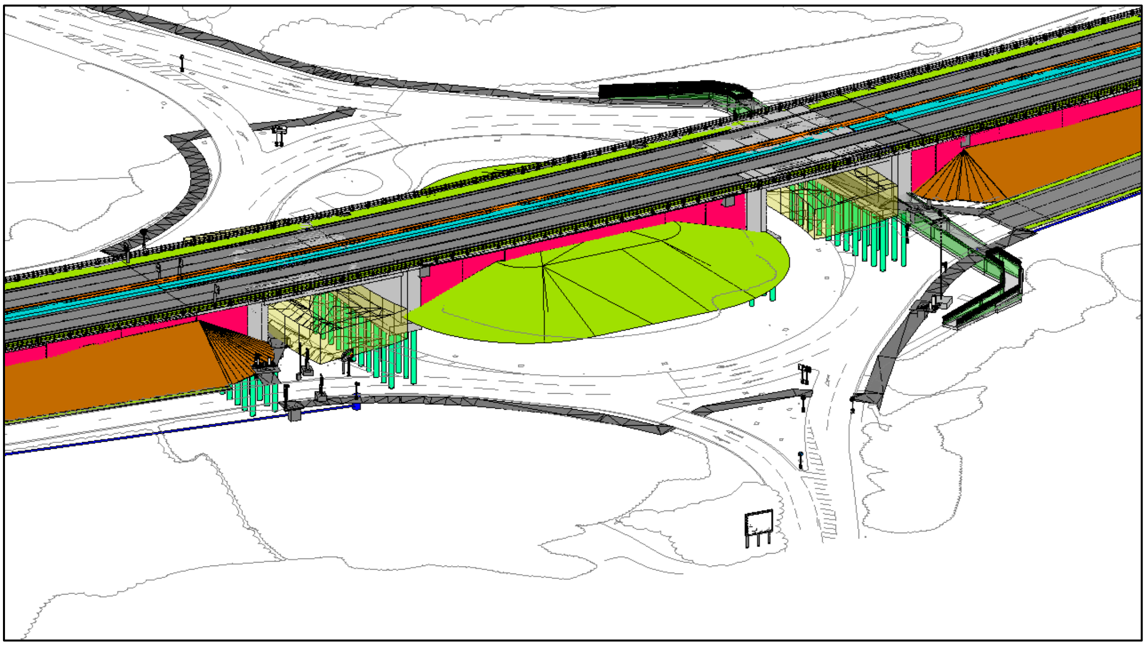

Figure 3.3: Proposed Countess Roundabout Flyover Configuration

3.2.10 The new flyover would be approximately seven metres 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 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 would also have areas of exposed chalk and within Parsonage Down some areas would have the 100mm topsoil rotovated into the chalk to provide a specific habitat. There is a possibility that this topsoil could affect the conveyance of surface water flows in these areas.

Provision of new utilities

3.2.15 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.

3.2.16 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 SAC

Increased road surface

3.2.17 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

3.2.18 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.

3.2.19 Road widening at the crossroads near Rollestone Camp (red shaded area at north of Figure 2.1) would take place in order 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"/Maybe 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.

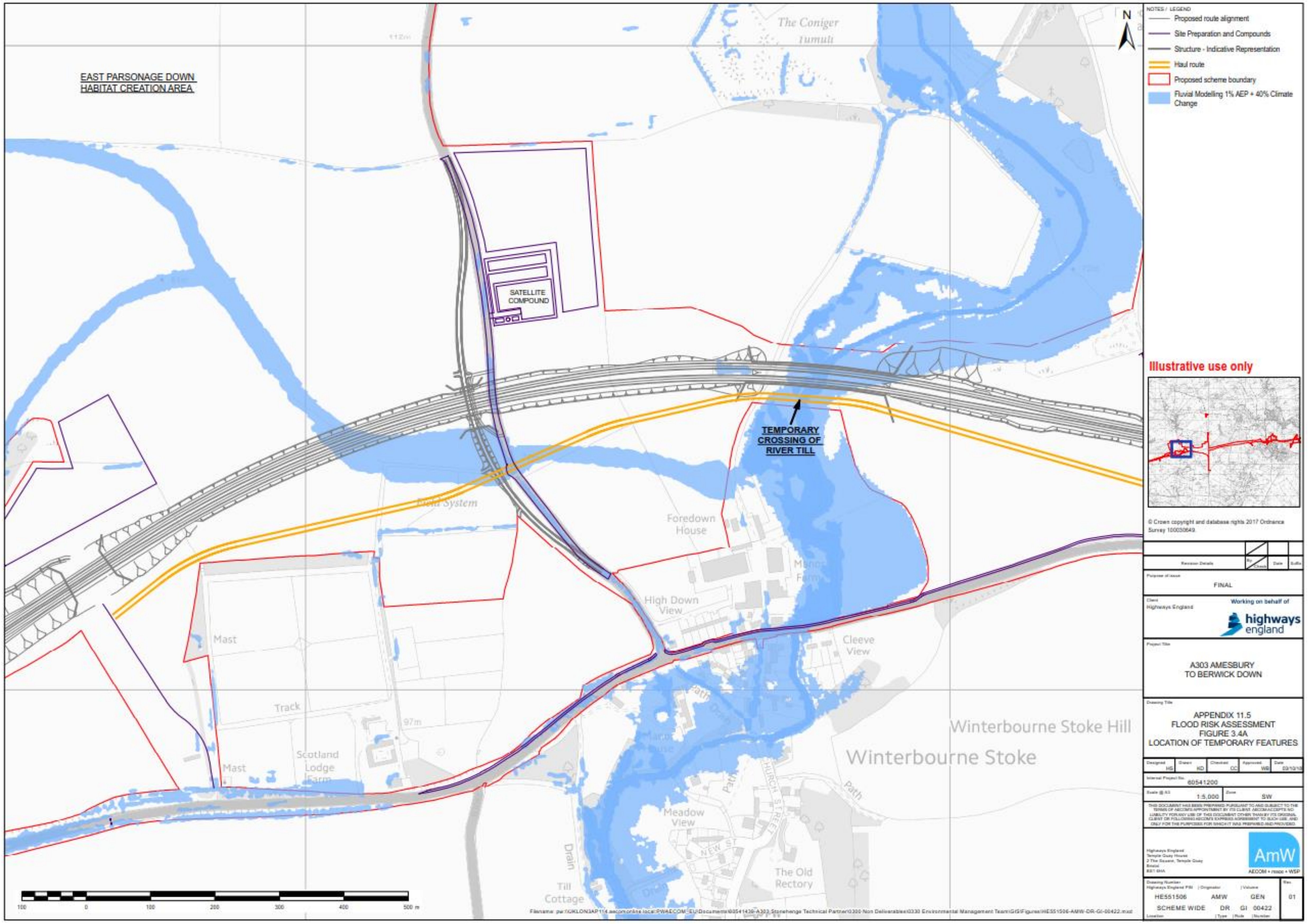

Figure 3.4A: Location of temporary features

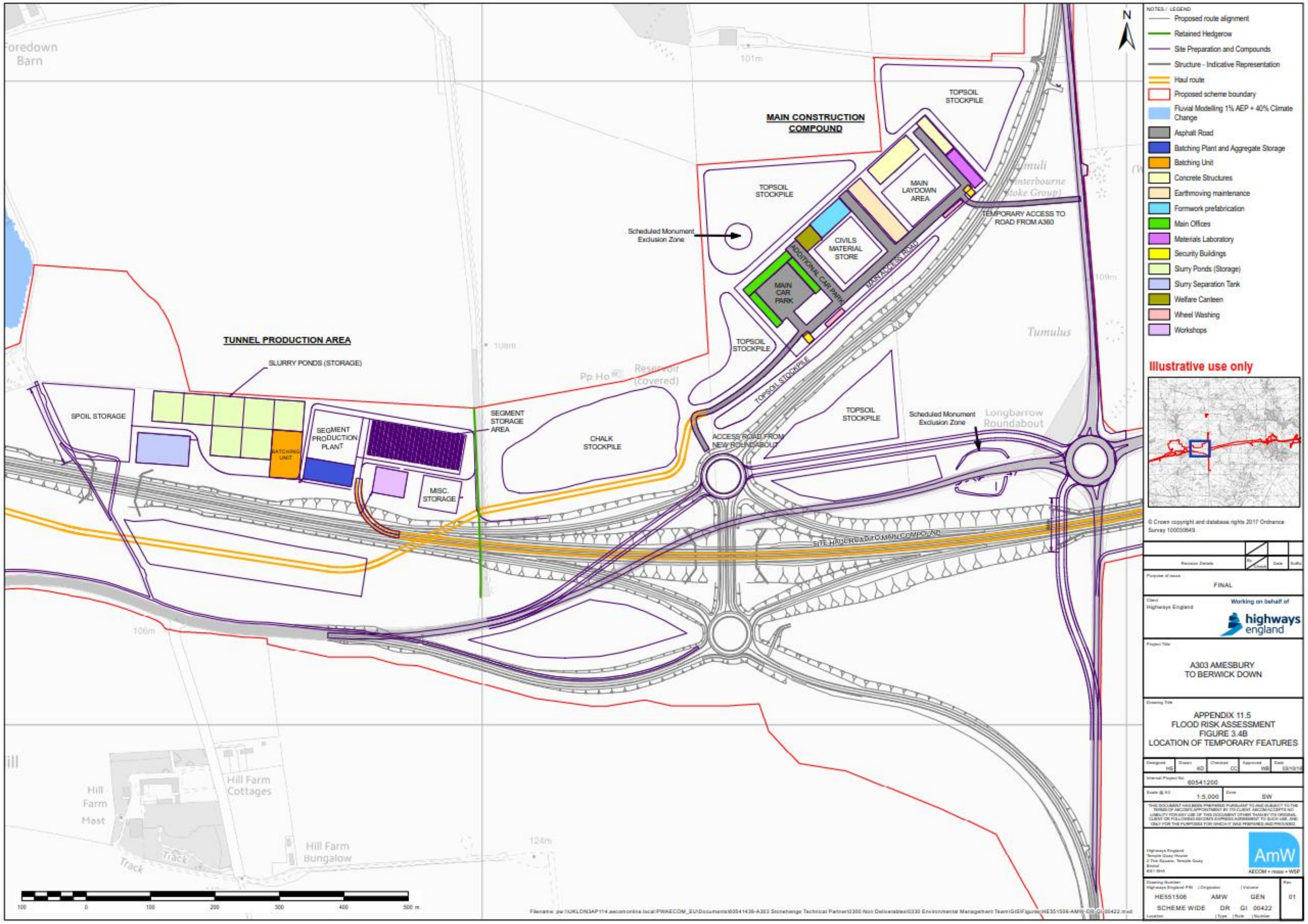

Figure 3.4B Location of temporary features

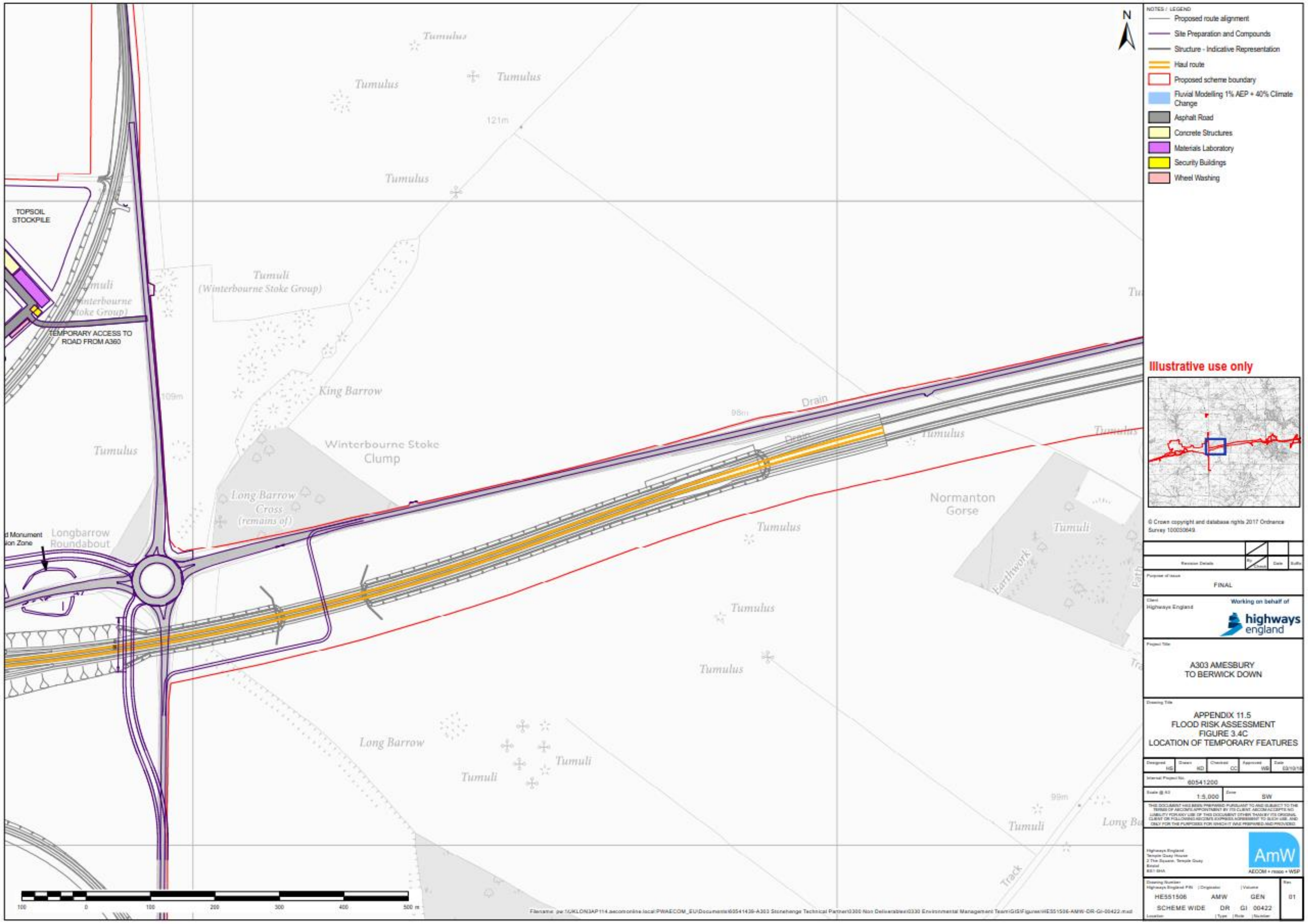

Figure 3.4C Location of temporary features

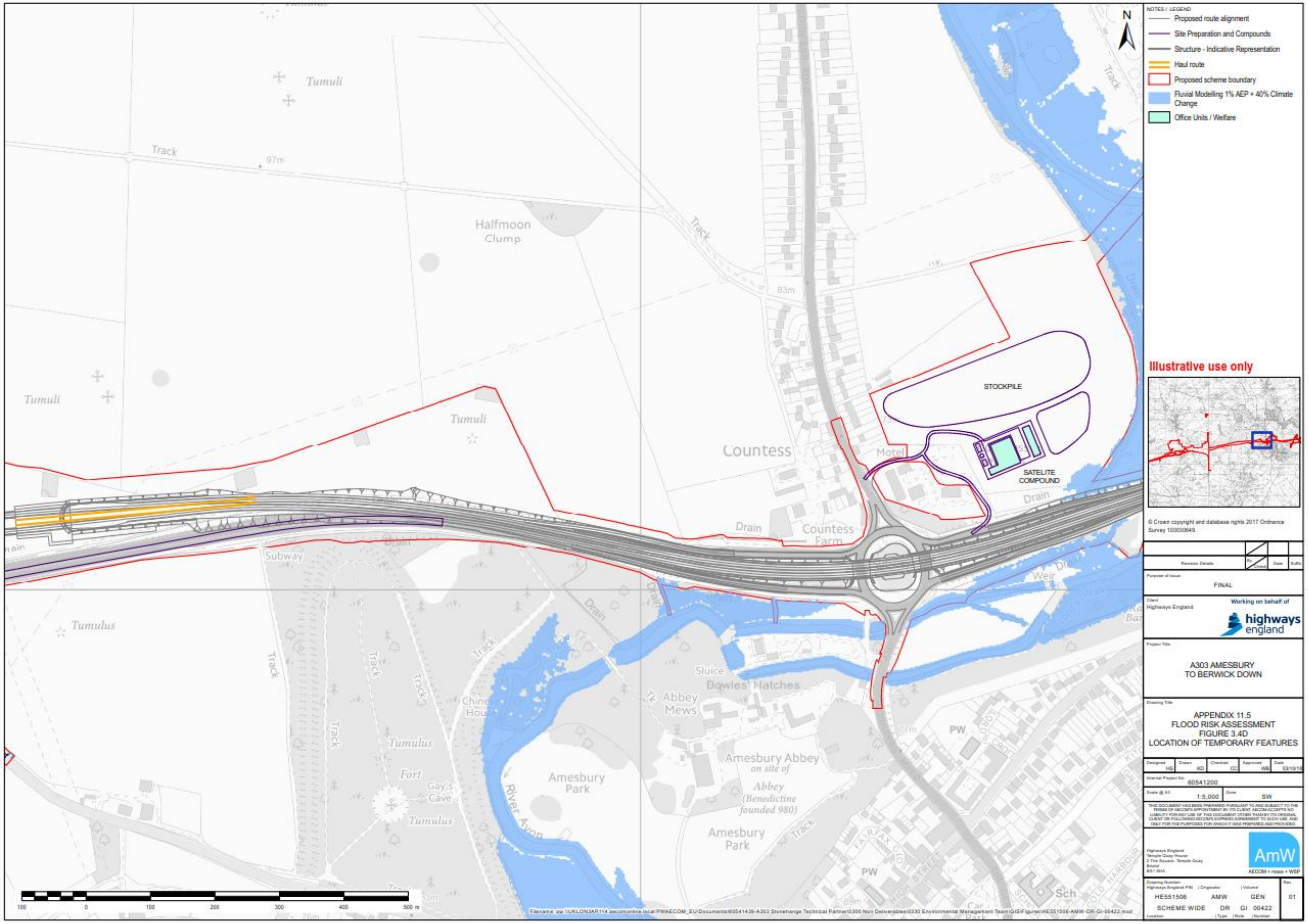

Figure 3.4D Location of temporary features

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 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 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 exit 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 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 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.

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

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

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 escape 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 ‘*all 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.’*

Consultation

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

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

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 Lead Local Flood Authority (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 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 The 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 CEMP derived from the Outline EMP (Appendix 2.2).

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

a) 1% Annual Exceedance Probability (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, and liaison with stakeholders has been used to confirm that modelled outputs replicate as closely as possible to flood events experienced. Model calibration for the River

Avon has been completed using data from the EA flow/level gauge at Amesbury.

5.3.6 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)

5.3.7 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.

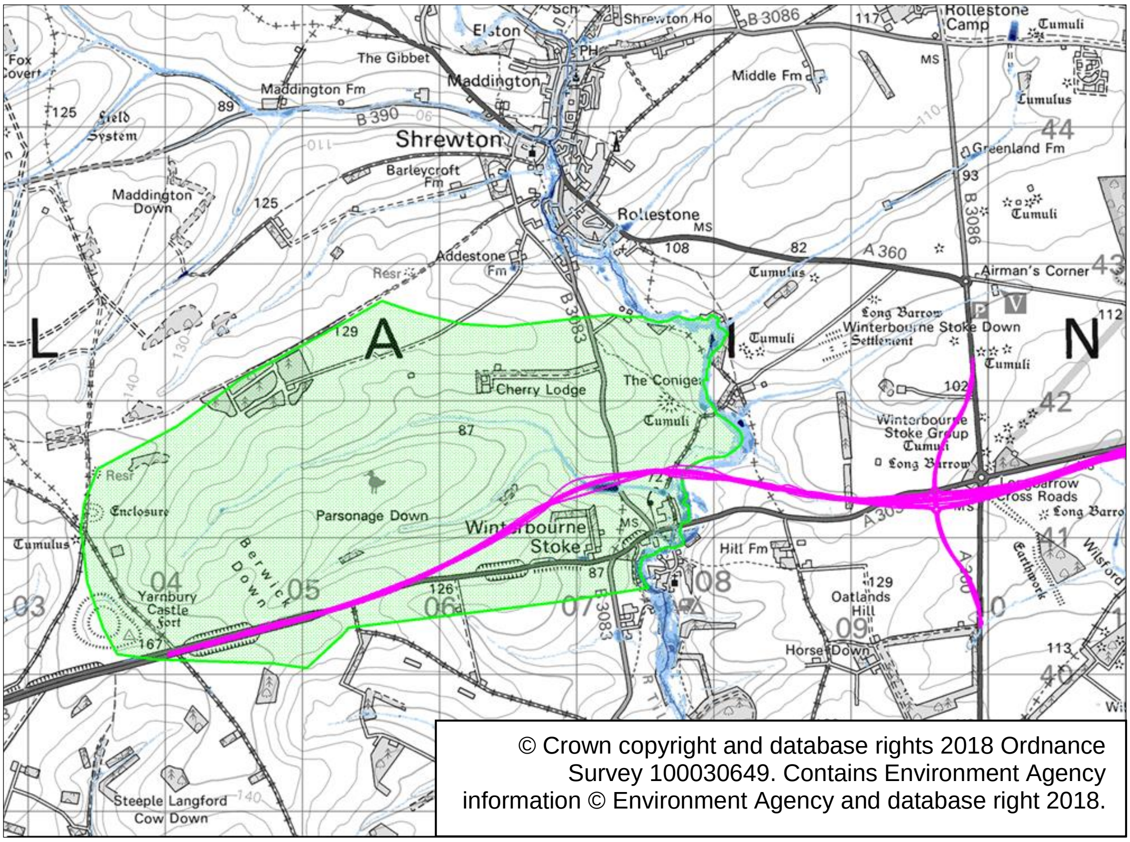

Figure 5.1: Parsonage Down Pluvial Model Extent (Green Outline)

5.3.8 Multiple rainfall event scenarios were modelled, including:

a) 1% AEP event; and

b) 1% AEP plus climate change event.

5.3.9 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

5.3.10 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.

5.3.11 The peak flow/groundwater level conditions within the model were modified for climate change predictions by altering the recharge stress period, with an increase of 20%. 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).

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 (HADDMS) 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 HADDMS 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.

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 It was agreed with the Environment Agency that the hydraulic modelling baseline results for the study area would be used in place of the Environment Agency Flood Map for Planning for the purposes of the FRA. As the current Environment Agency flood mapping is based on coarse data (JFLOW) and not considered to provide a site specific representation of the region, this approach has been considered to be the most appropriate course of action.

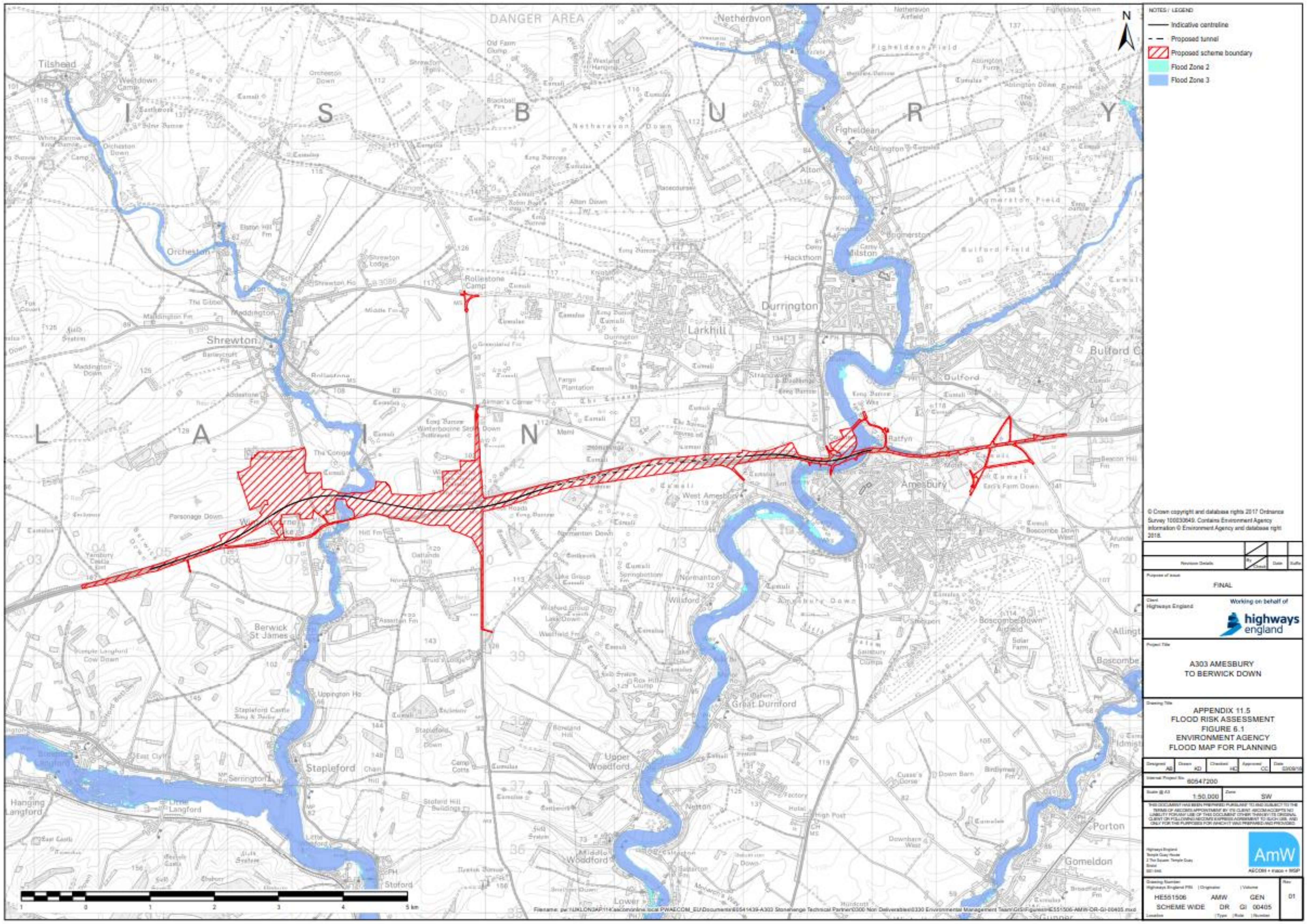

Figure 6.1: Flood Map for Planning (Source: Environment Agency)

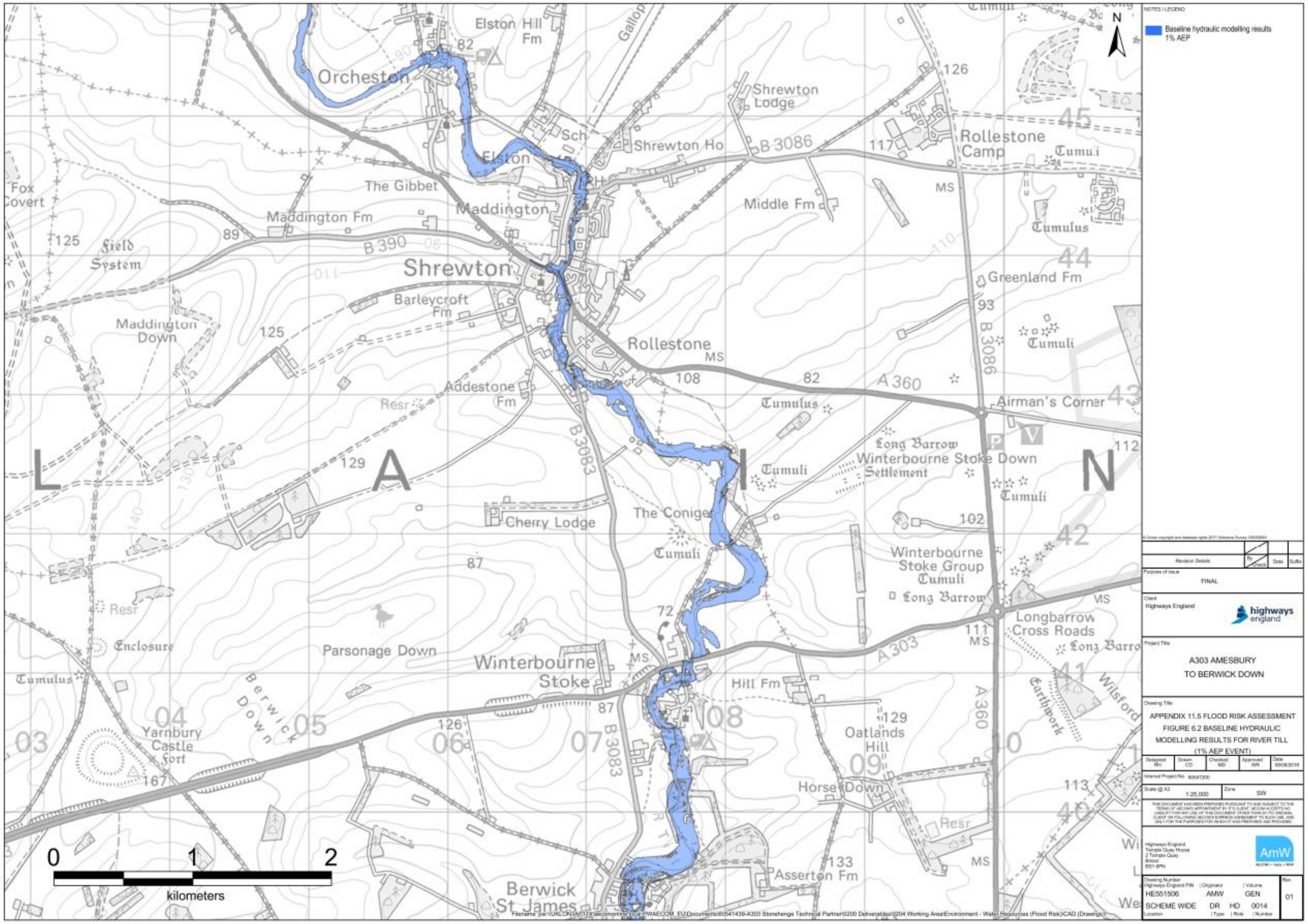

Figure 6.2: Baseline hydraulic modelling results for River Till (1% AEP event)

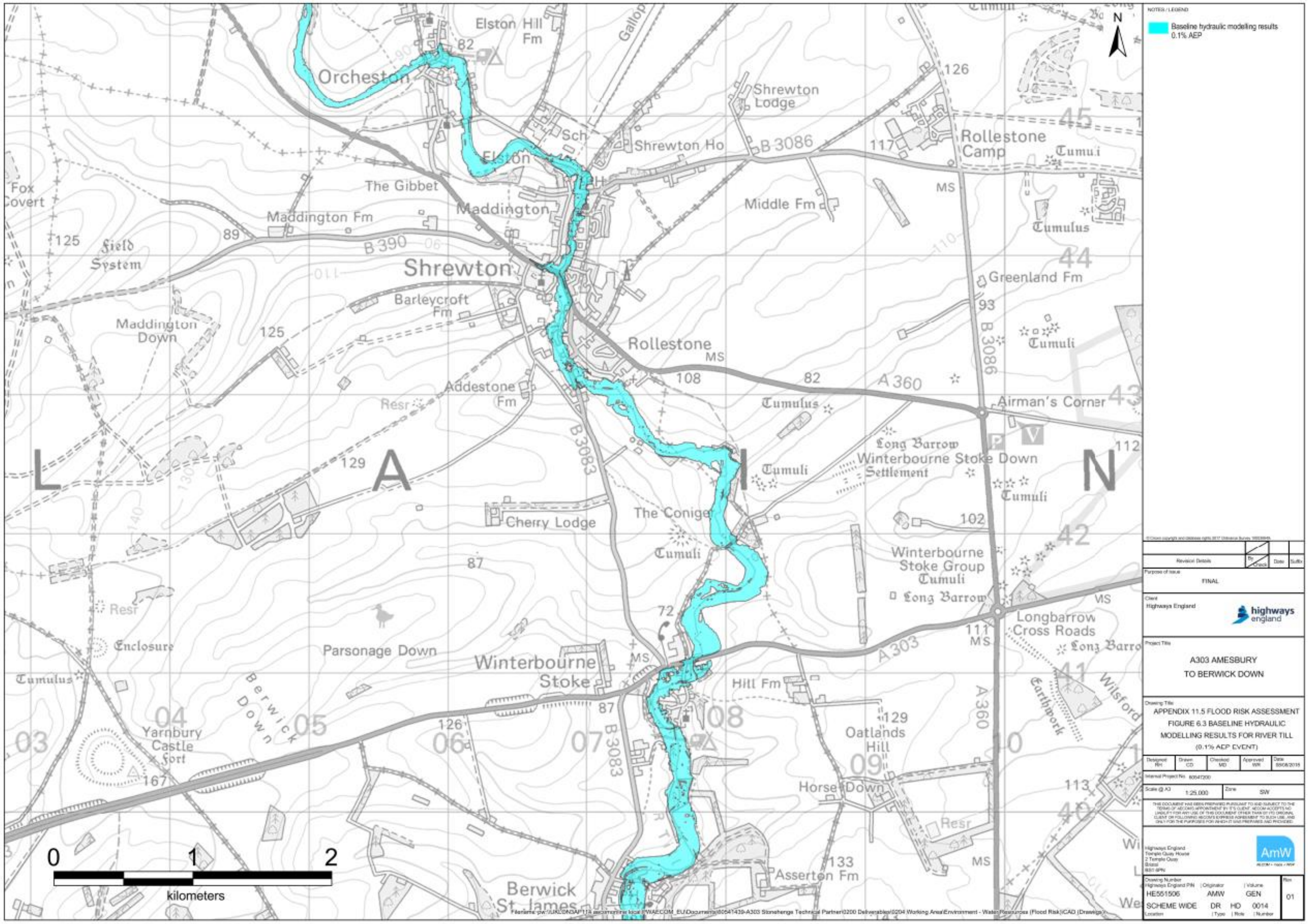

Figure 6.3: Baseline hydraulic modelling results for River Till (0.1% AEP event)

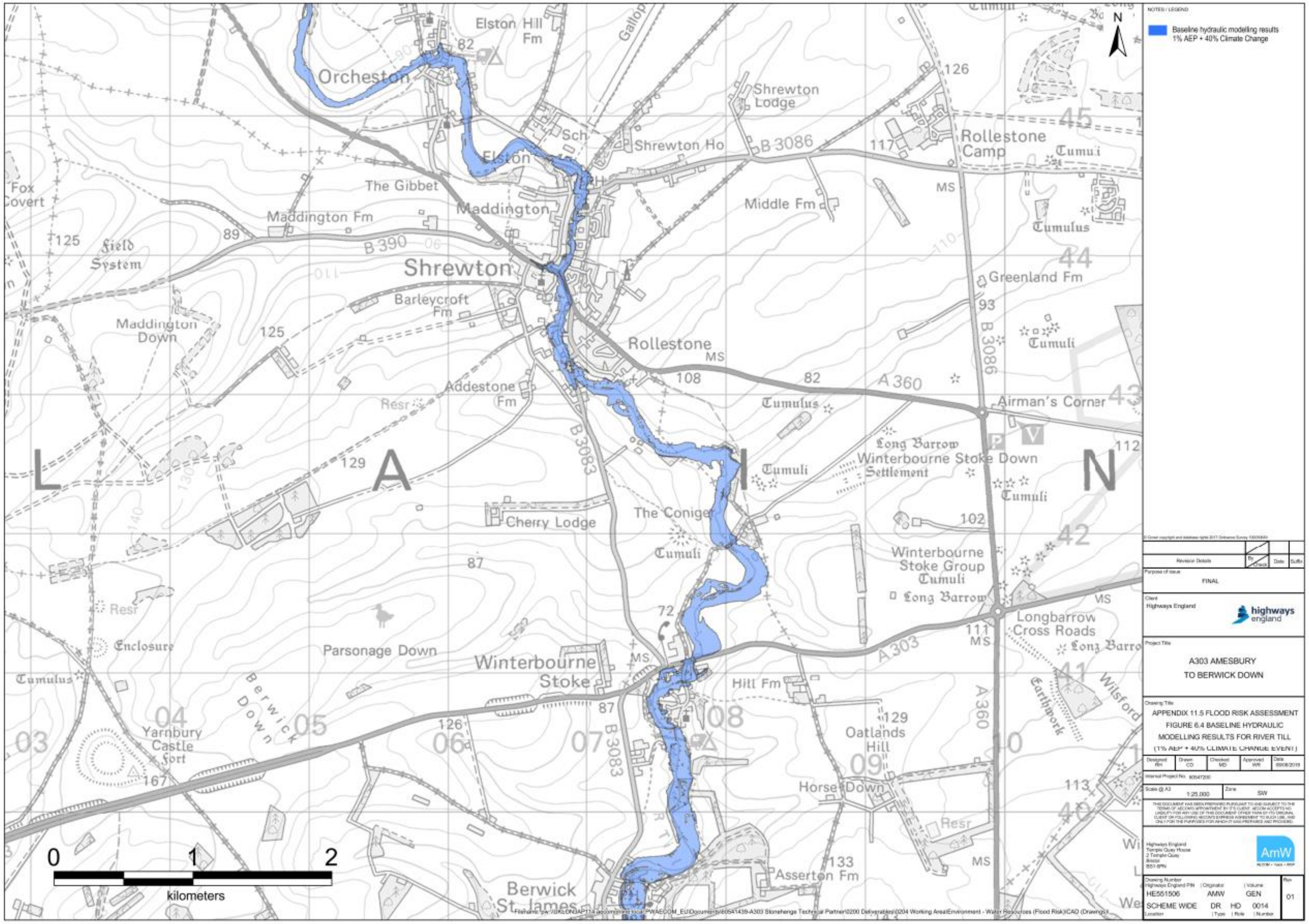

Figure 6.4: Baseline hydraulic modelling results for River Till (1% AEP + 40% climate change event)

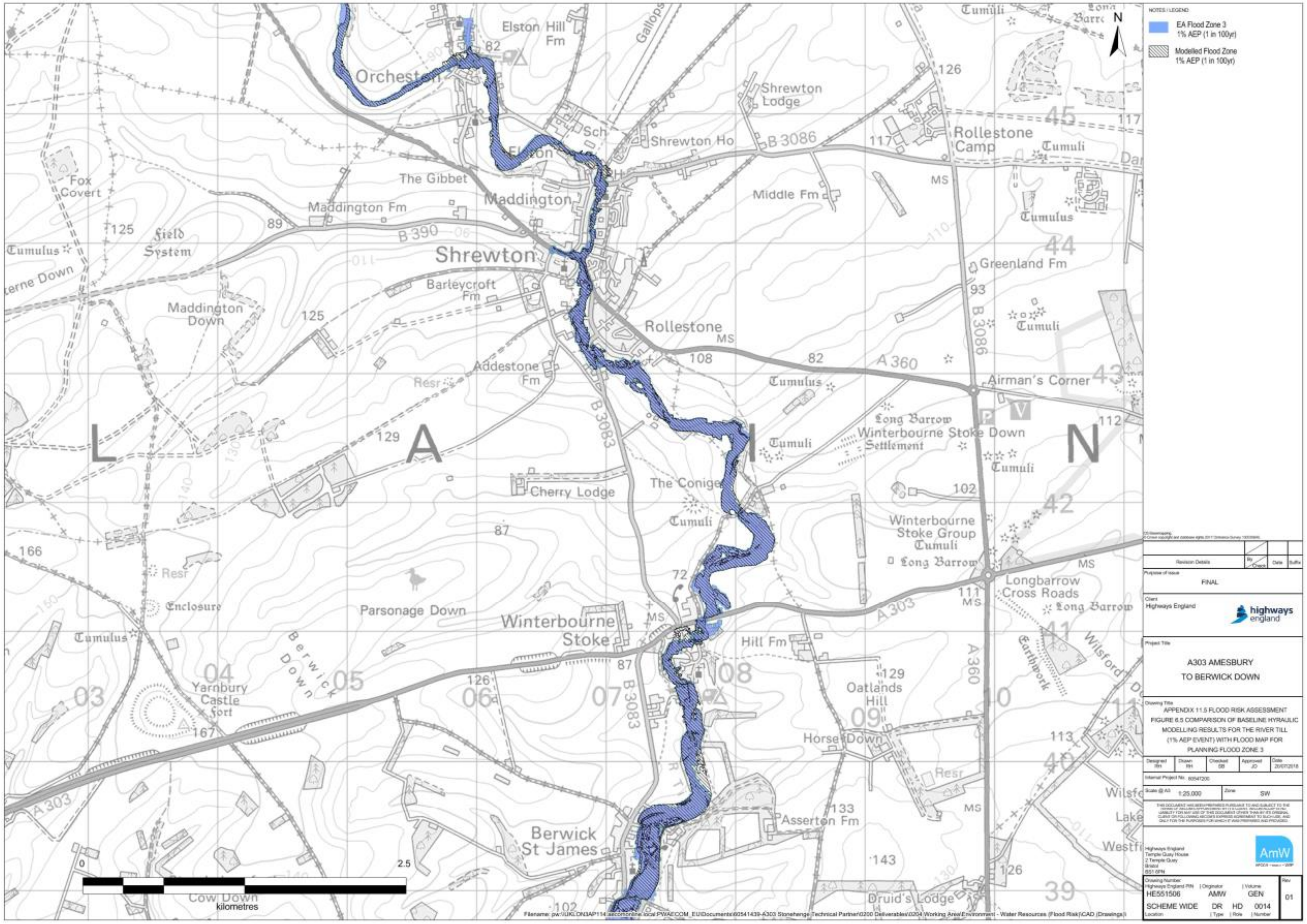

Figure 6.5: Comparison of baseline hydraulic modelling results for River Till (1% AEP event) with Flood Map for Planning (Flood Zone 3)

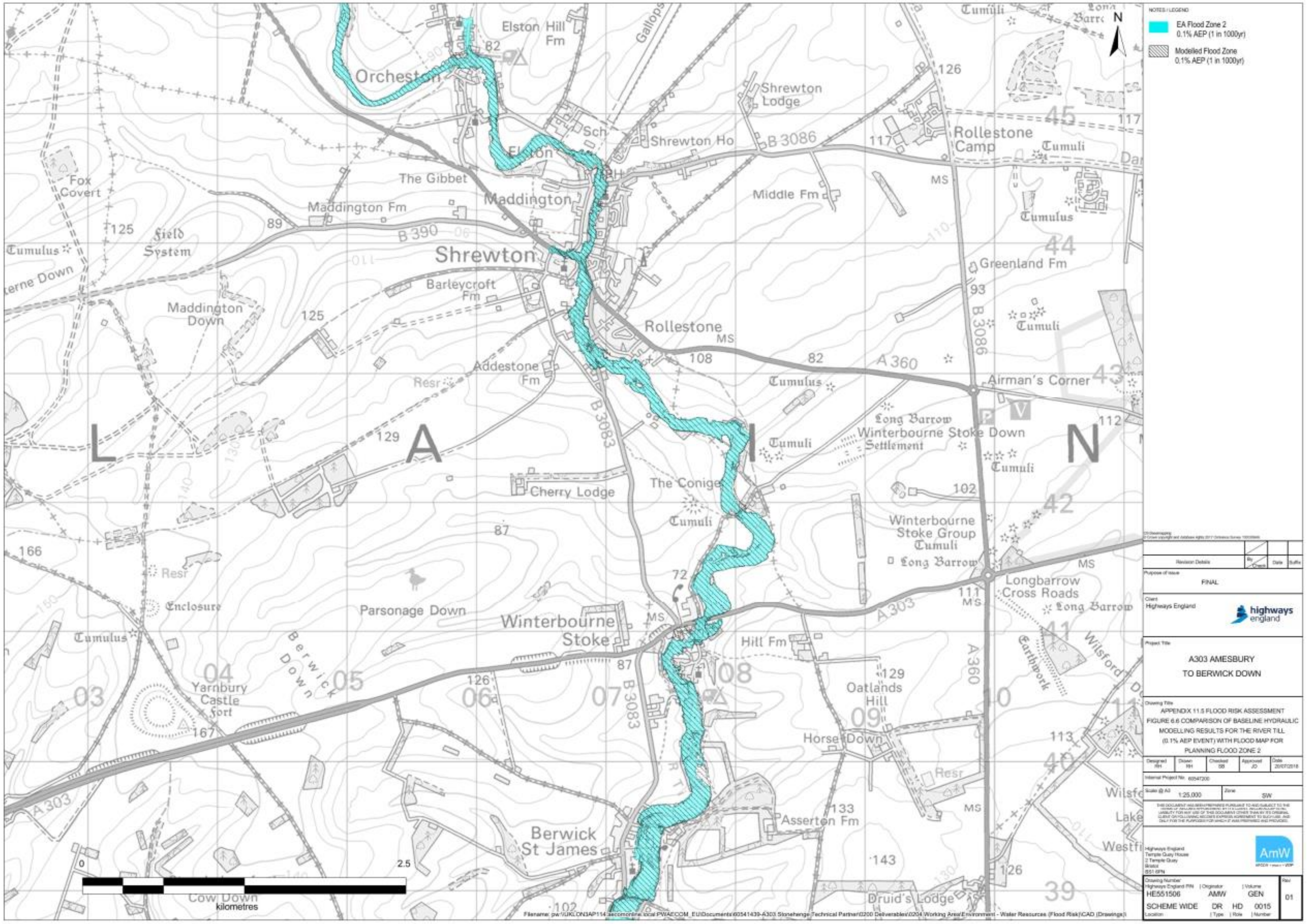

Figure 6.6: Comparison of baseline hydraulic modelling results for River Till (0.1% AEP event) with Flood Map for Planning (Flood Zone 2)

River Avon

6.2.12 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.

6.2.13 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.

6.2.14 For the 1% AEP event, the compared flood extents seen in Figure 6.10 shows a substantial decrease in fluvial modelling extents, particularly within Amesbury Park and just west and southwest of Bulford. To the south of the existing A303, Flood Zone 3 covers the entire valley bottom, whilst the study model results suggest there is minimal out of bank flow on the floodplain.

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

6.2.16 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).

6.2.17 Once reviewed and approved by the Environment Agency and Wiltshire Council, it was agreed that the site specific hydraulic modelling baseline results for the study area would be adopted in place of the Environment Agency Flood Map for Planning.

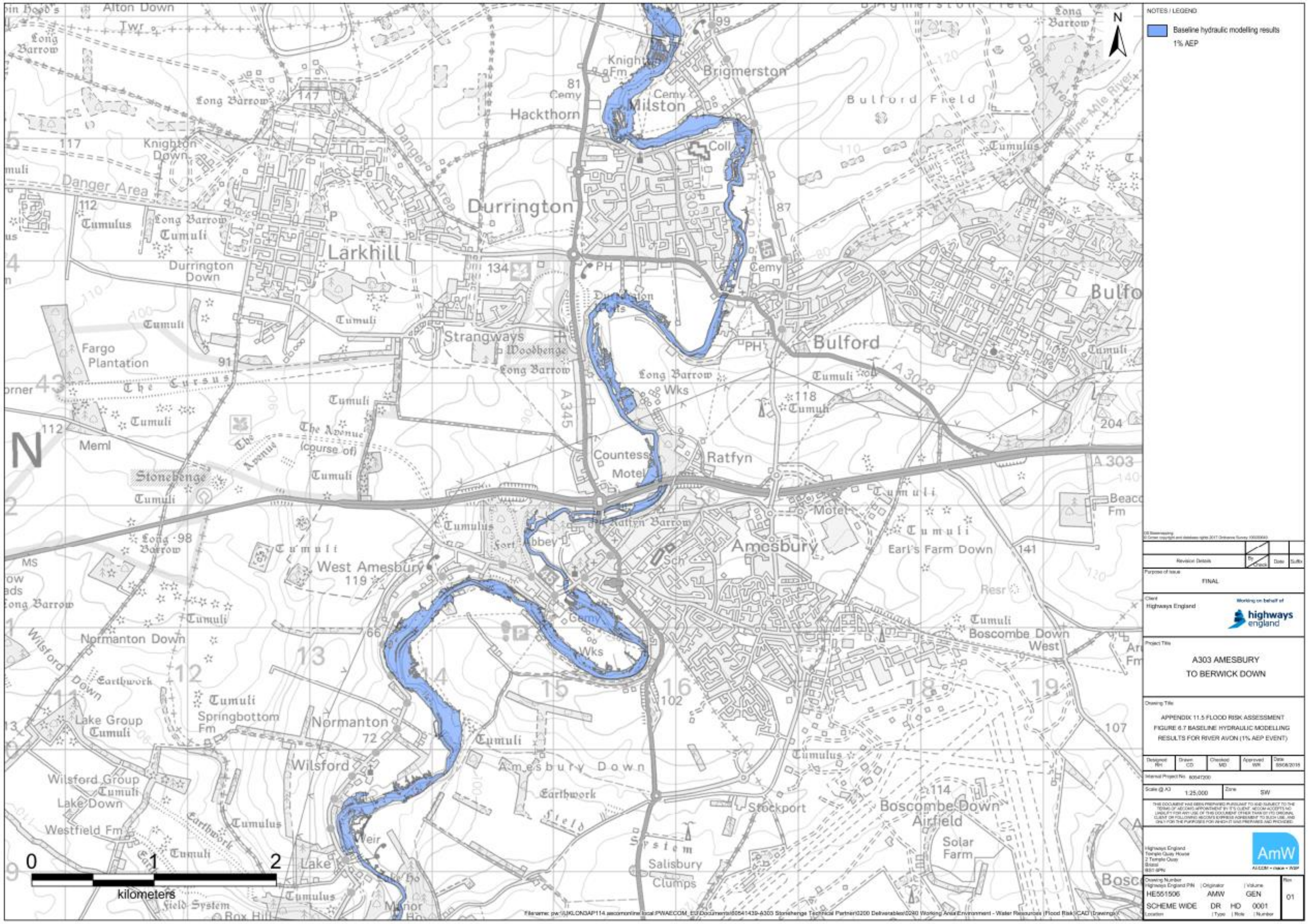

Figure 6.7: Baseline hydraulic modelling results for River Avon (1% AEP event)

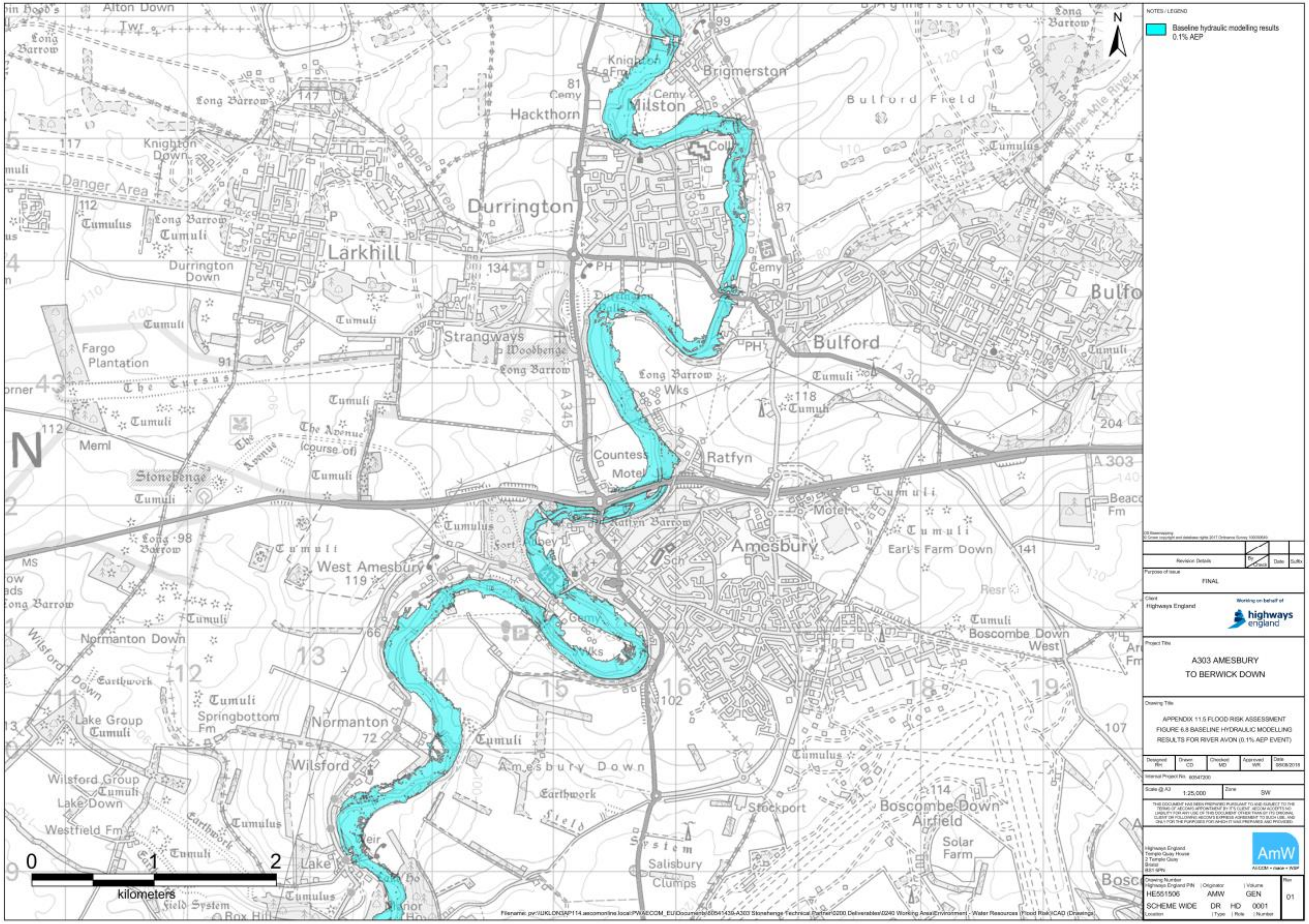

Figure 6.8: Baseline hydraulic modelling results for River Avon (0.1% AEP event)

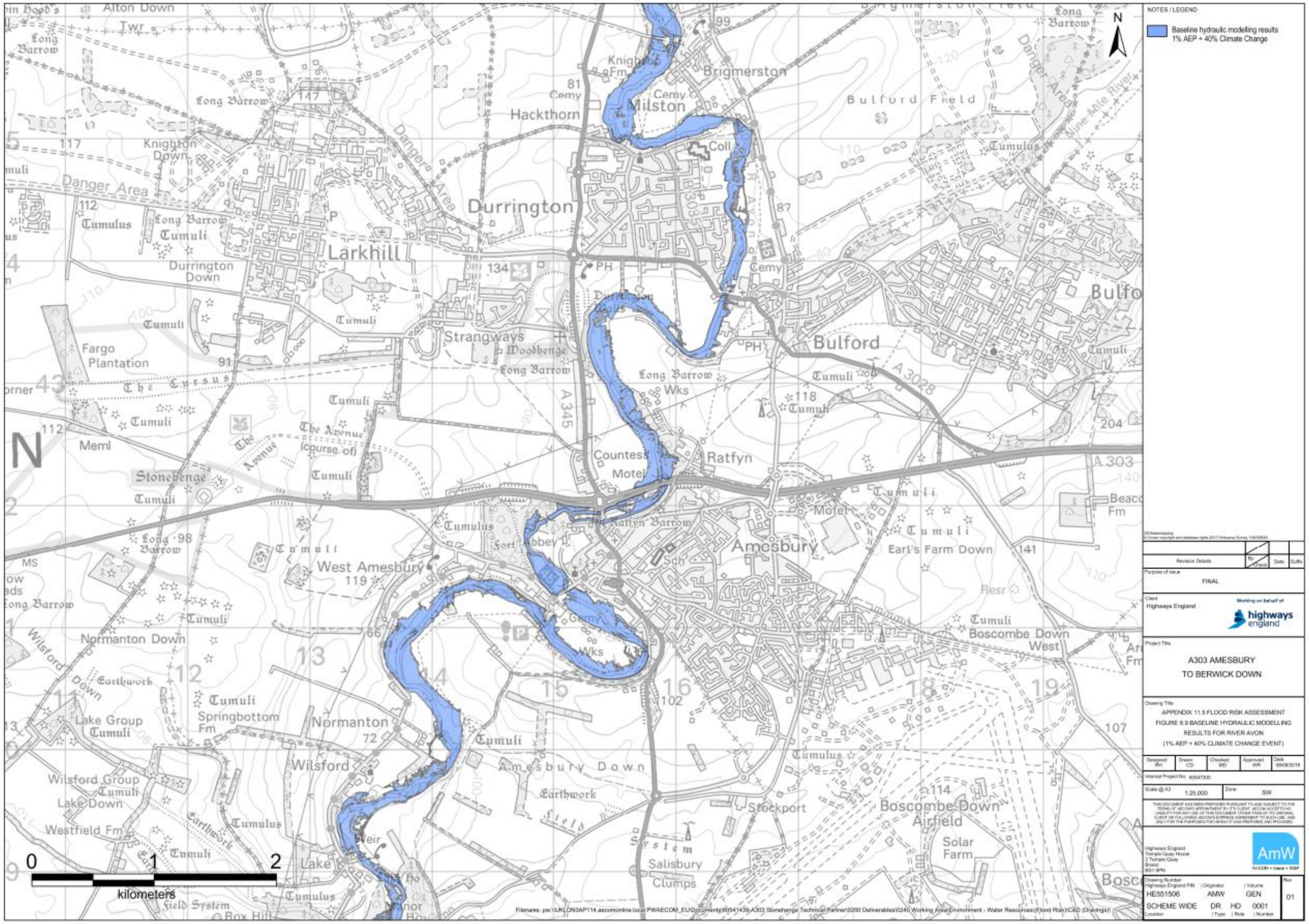

Figure 6.9: Baseline hydraulic modelling results for River Avon (1% AEP + 40% climate change event)

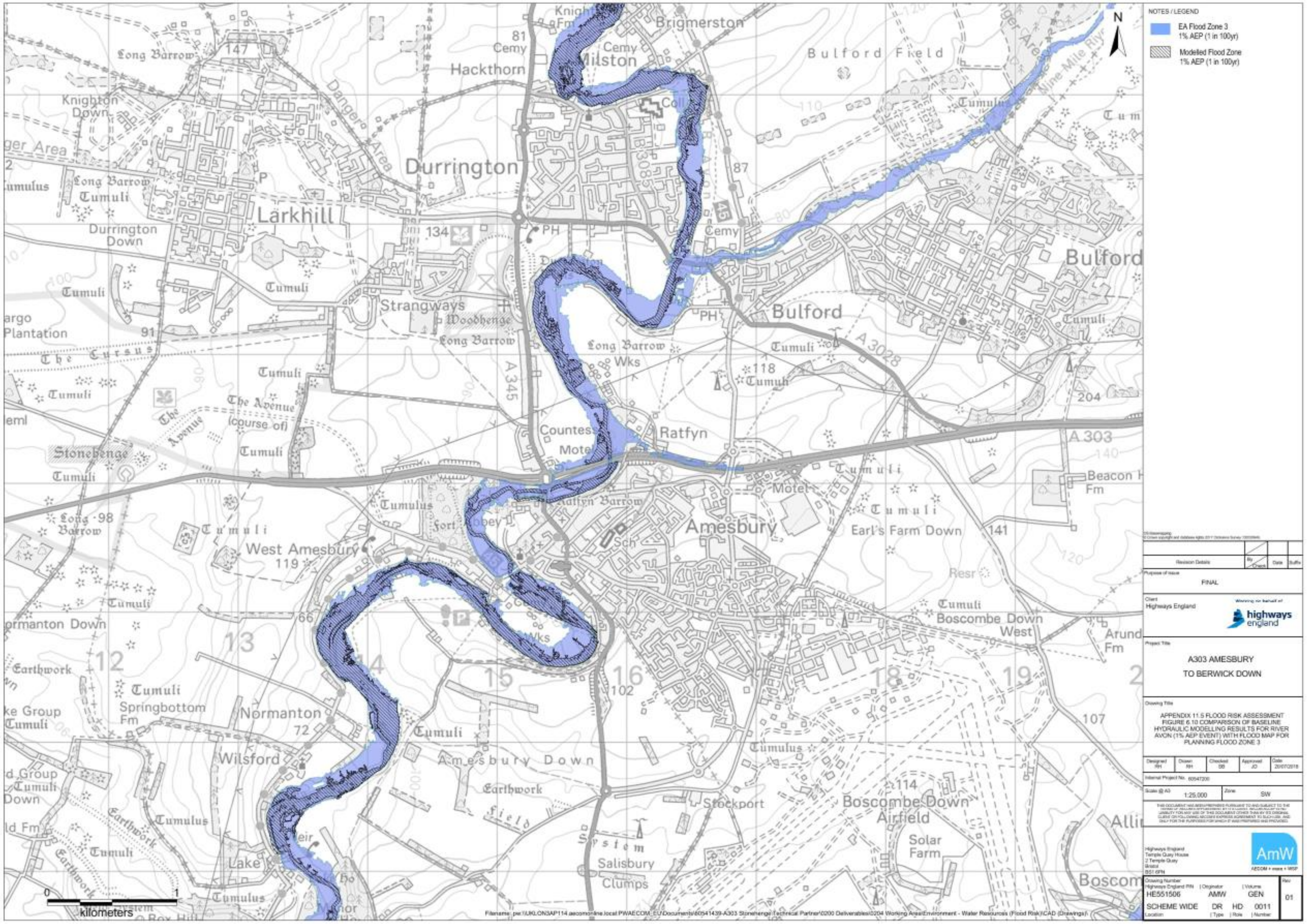

Figure 6.10: Comparison of baseline hydraulic modelling results for River Avon (1% AEP event) with Flood Map for Planning (Flood Zone 3)

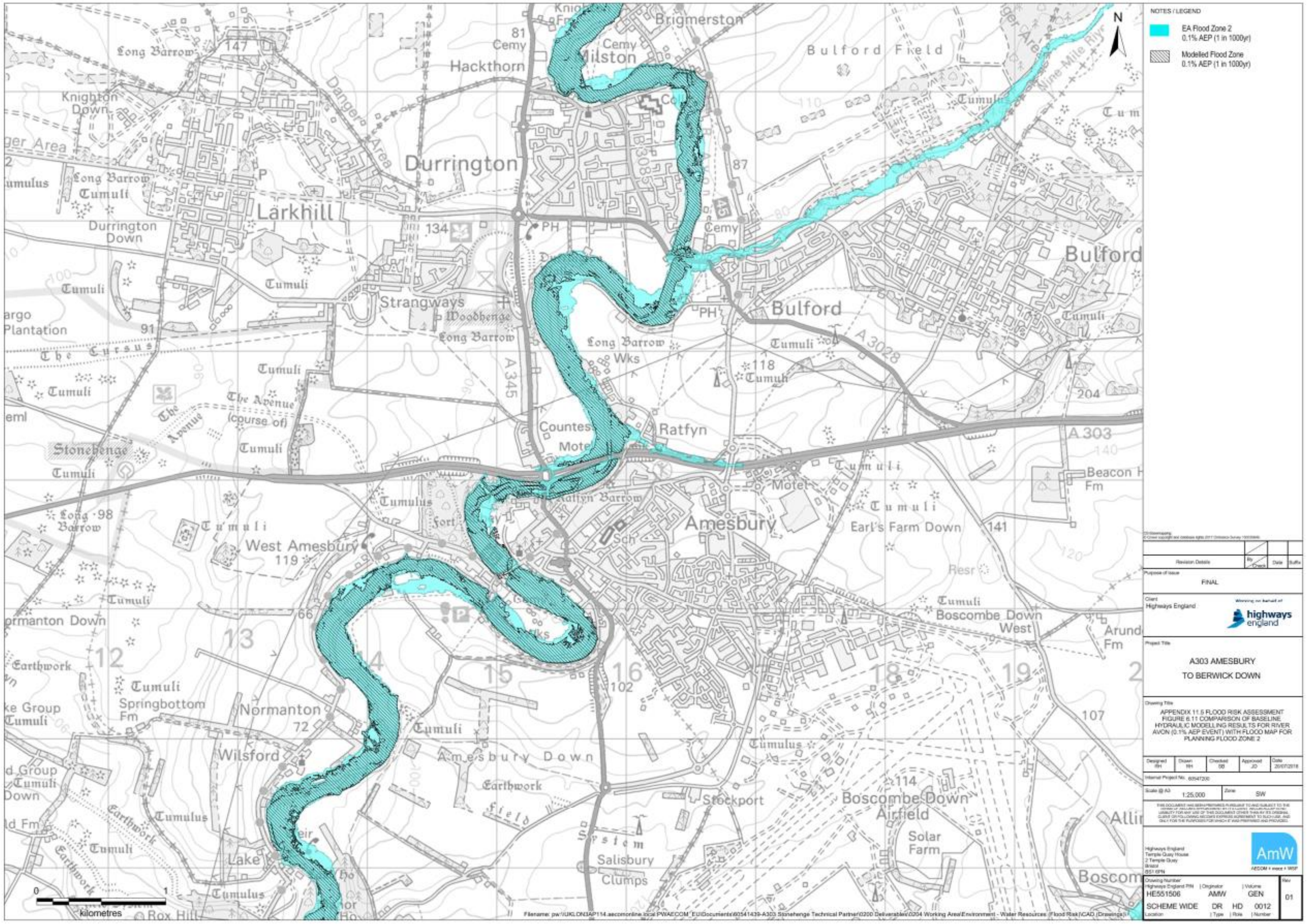

Figure 6.11: Comparison of baseline hydraulic modelling results for River Avon (0.1% AEP event) with Flood Map for Planning (Flood Zone 2)

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
Chitterene	1986, 1992, 2003
River Avon	
Durrington	1980, 2008
Amesbury	1999
Enford	2000, 2001, 2003
Great Durnford	1977
Wilsford-Cum-Lake	1995

6.3.4 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

6.3.5 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.

6.3.6 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 1% AEP flood event in Figure 6.15.

6.3.7 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 through the dry valley at Parsonage Down.

6.3.8 A difference can be observed along the River Till valley to the north of where the proposed scheme will be located. The pluvial model was set up to simulate a localised 1% AEP rainfall event applied to the direct rainfall catchment. In contrast, the Environment Agency surface water flood extent shown within the Till Valley at this location reflects a 1% AEP rainfall event across the entire upstream catchment. In addition the model was set up to receive a fluvial inflow for the River Till corresponding to a 50% AEP event at the upstream model boundary of the River Till. Therefore the difference in fluvial flow supplied from upstream and extent of upstream rainfall catchment can be attributed as the reason for the observed difference in flood extent within the Till Valley.

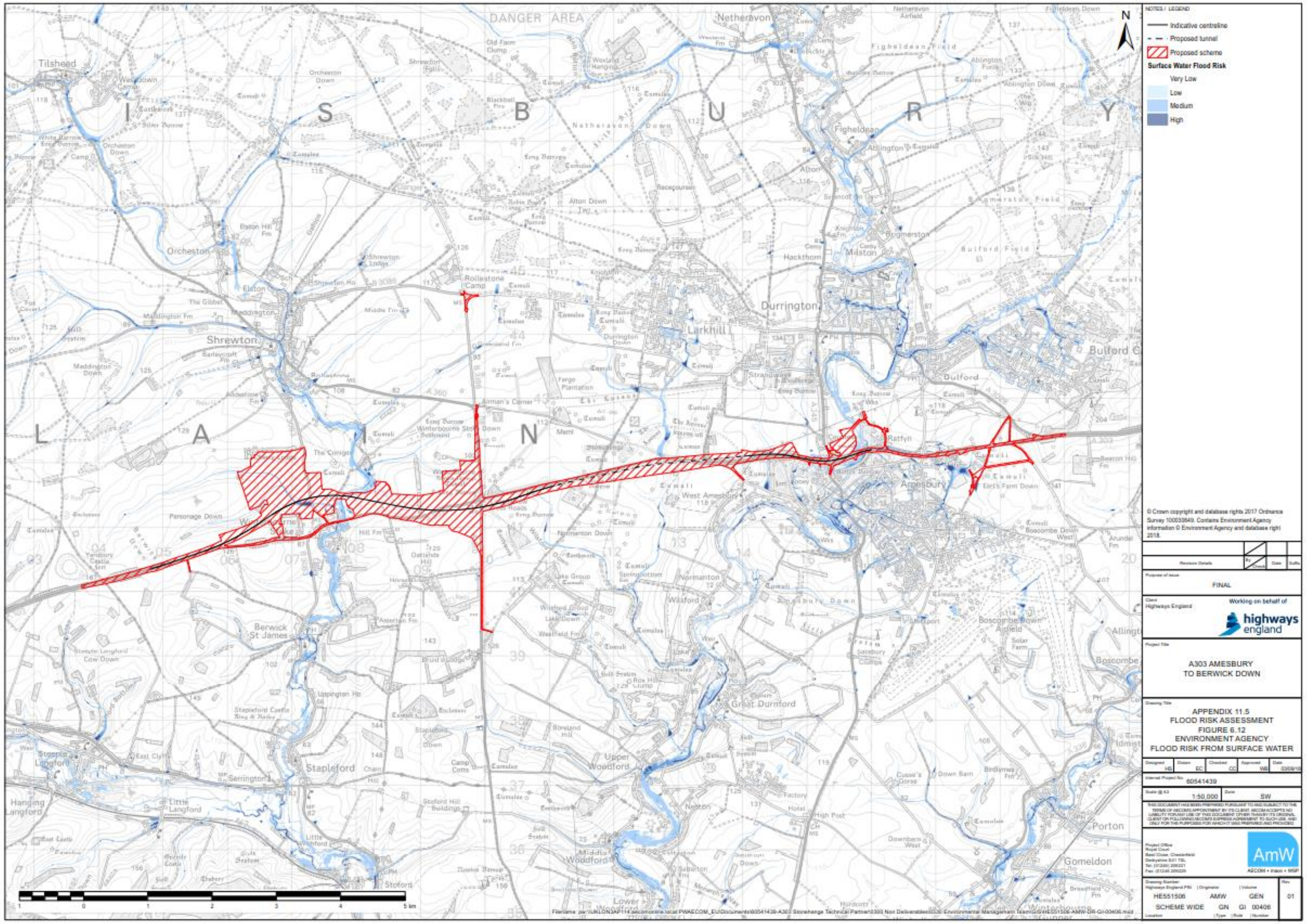

Figure 6.12: Flood Risk from Surface Water Mapping (Source: Environment Agency)

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 to 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: Historic Groundwater Flood Events

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 place that are actioned when there is a risk of flooding.

Baseline Groundwater Modelling

6.4.6 The groundwater model 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, 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.

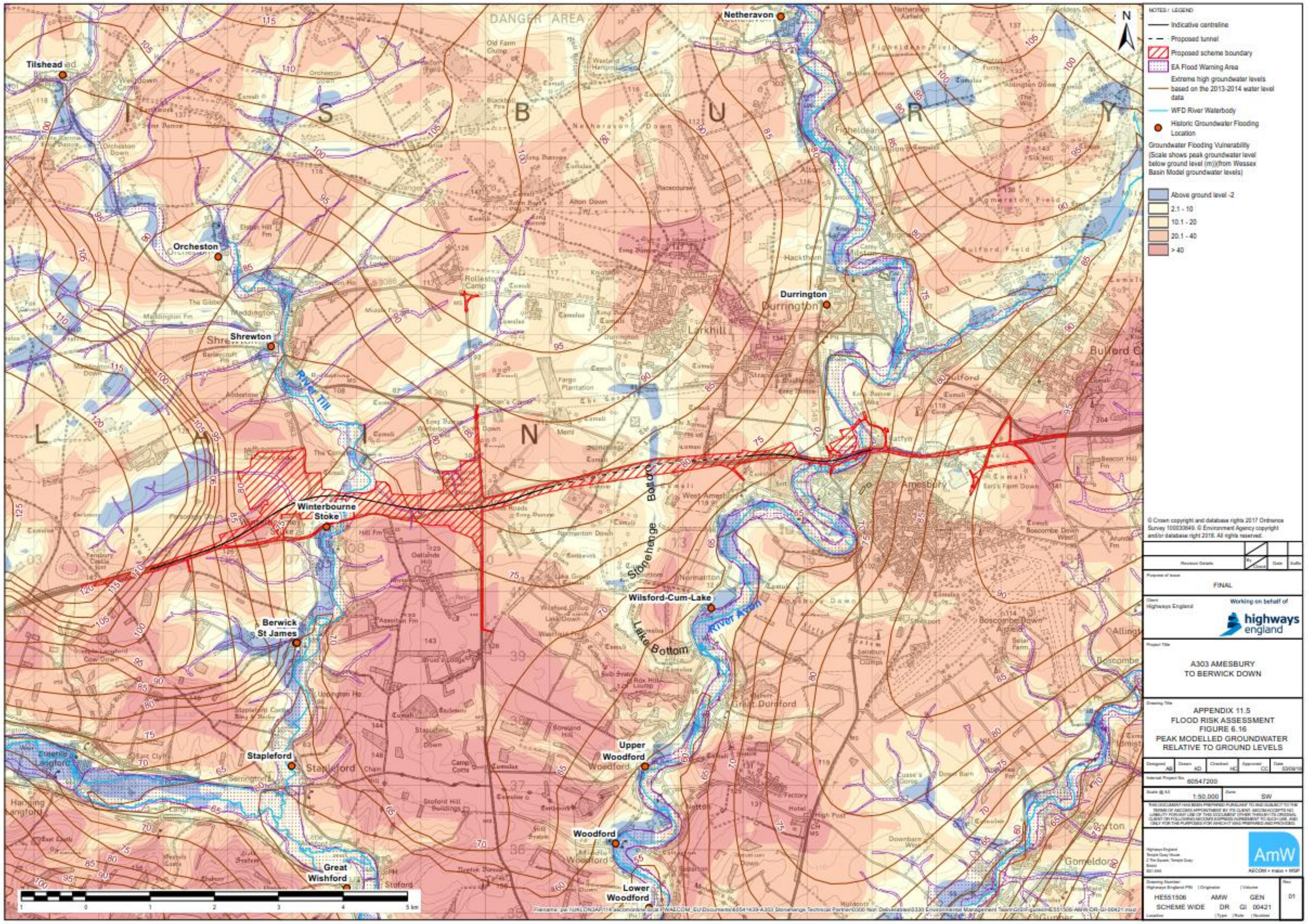

Figure 6.16: Peak modelled groundwater levels relative to the ground level

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 to be considered as 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 (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 (1 in 100 year) + 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 hydraulic modelling shows no areas of increased flood depth compared to the baseline as a result of the proposed scheme, as illustrated in Figure 8.1.

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 Flood Risk from Surface Water 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 11.3: 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 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 Topographical alterations will be required to support the A303 flyover and infilling of the subway underneath of the Countess Roundabout has the potential to alter fluvial flooding paths from the River Avon. To determine the potential change in flood risk from these scheme elements, hydraulic modelling has been undertaken.

8.2.2 Hydraulic modelling shows no areas of increased flood depth compared to the baseline as a result of the proposed scheme, as illustrated in Figure 8.1.

8.2.3 Relatively shallow flood depth differences can be viewed within two drainage ponds to the east of Countess Roundabout in Figure 8.1. Bunding around the highway drainage attenuation ponds has been assumed to be at a height above the floodplain which could explain the modelled minor depth reductions in these areas. The flood difference map confirms that any localised displaced water as a result of the highway drainage attenuation ponds remains at an increase below +10mm (acceptable tolerances for planning purposes). Displacement of floodwater beyond the immediate site area shown is not recorded.

8.2.4 Details of how the scheme elements have been represented are included in Fluvial Hydraulic Modelling Report (Annex 1 Part A) and the Pluvial Hydraulic Modelling Report (Annex 1 Part B).

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

8.2.6 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 of this report, it is also demonstrated that the proposed scenario does not have a detrimental impact on flooding.

Third Party Flood Risk

8.2.7 As displayed in Figure 8.1, all areas where depth has decreased are within the red line boundary, where the land is owned by Highways England. There are no areas of flood increase at this location and so no flood risk to third parties is expected for a 1% AEP +40% climate change event.

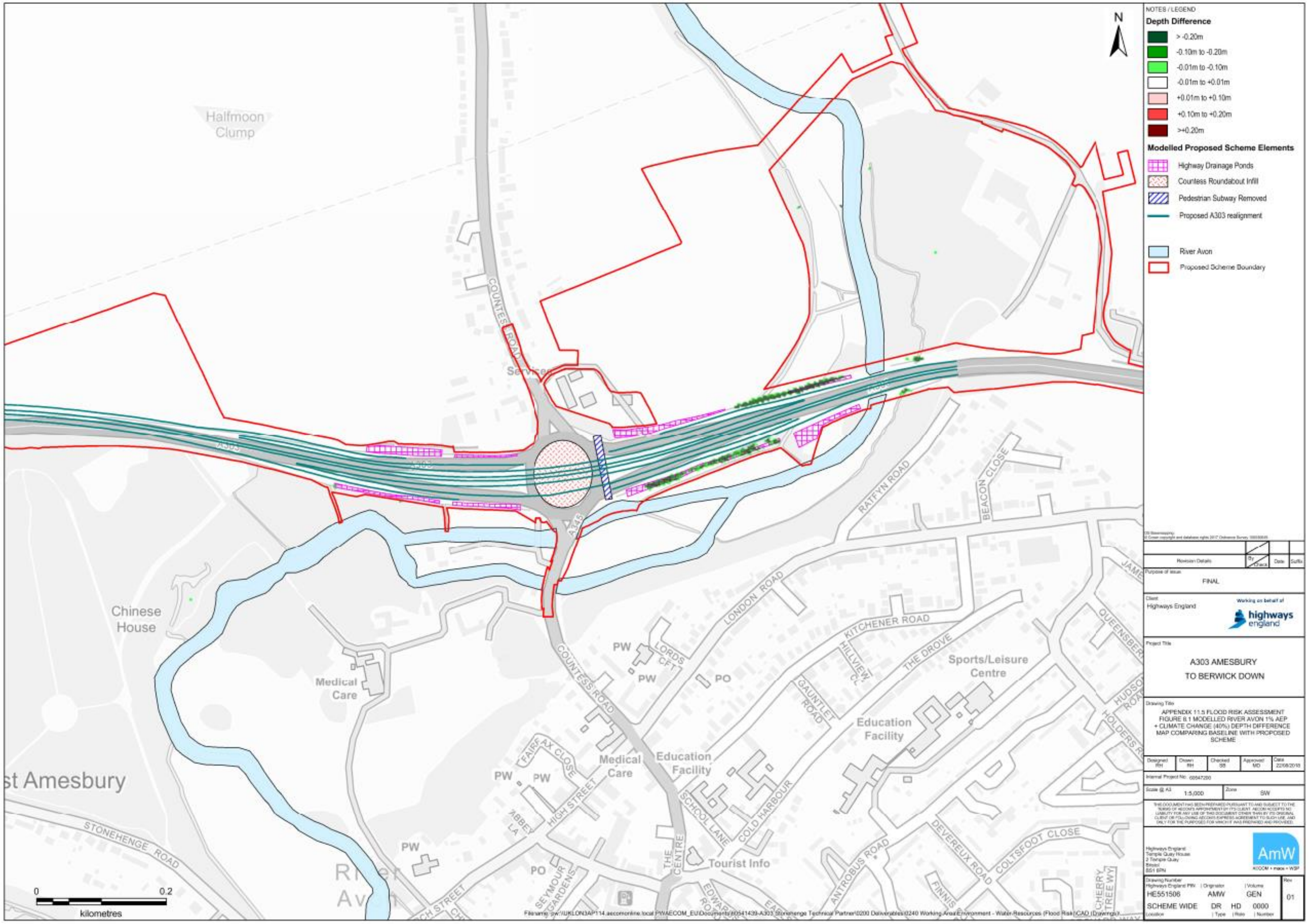

Figure 8.1: Modelled River Avon 1% AEP + Climate Change (+40%) Depth Difference Map comparing baseline with proposed scheme

River Till

8.2.8 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.

8.2.9 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).

8.2.10 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.

8.2.11 The upstream increase in flood risk towards the river channel is due to any 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.

8.2.12 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.

8.2.13 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.

8.2.14 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

8.2.15 As displayed in Figure 8.2, all areas where depth has decreased are within the red line boundary, where the land is owned by Highways England. 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. In order to mitigate this third party flooding in a 1% AEP +40% climate change event, the land owner(s) and Environment Agency will be consulted.

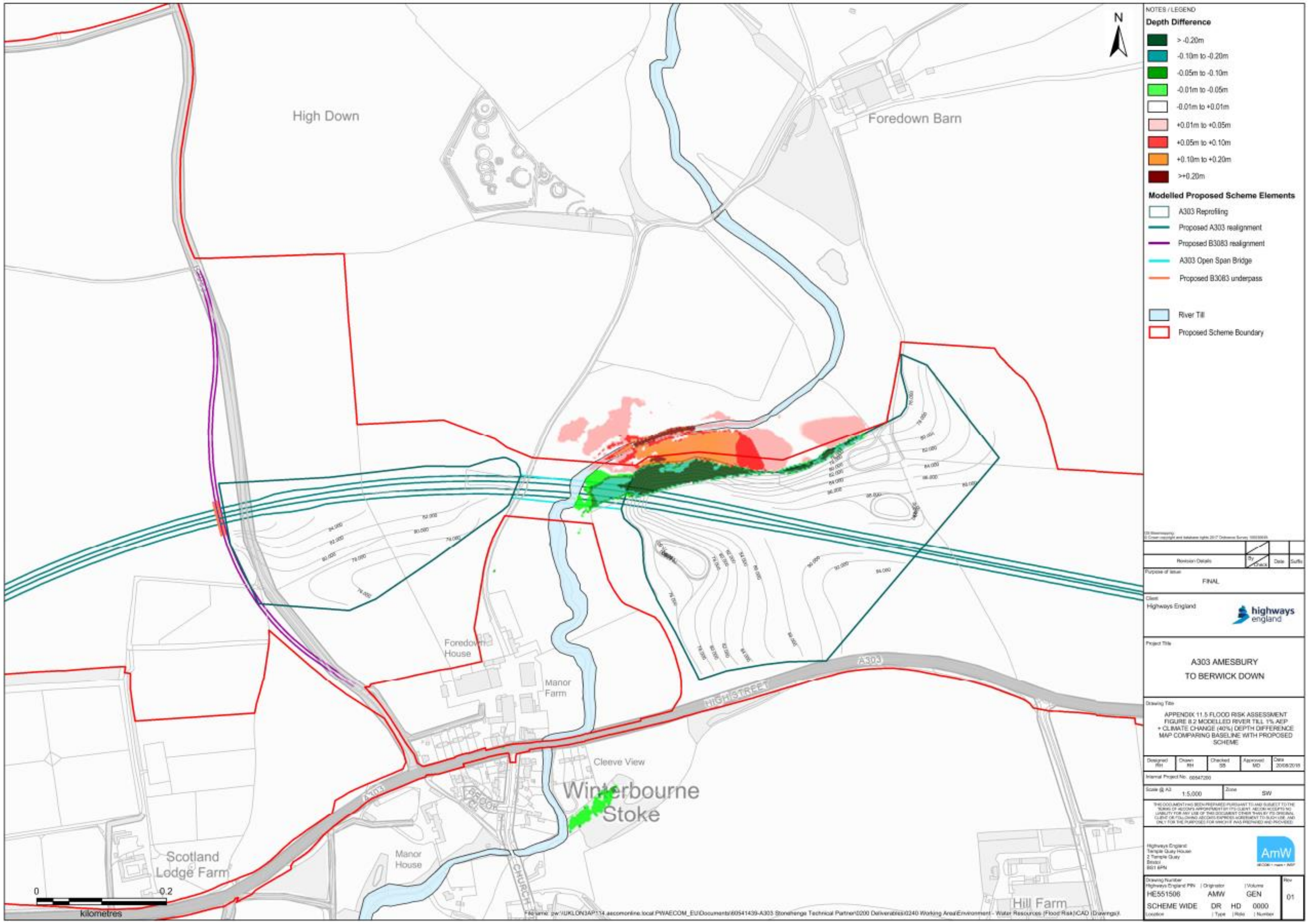

Figure 8.2: Modelled River Till 1% AEP plus Climate Change (+40%) Depth Difference Map comparing baseline with proposed scheme

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 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.

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.

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 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 general increases in depth within the reprofiled area south of Cherry Lodge of up to 0.40m with a few isolated spots within this area having an increased depth of up to 0.65m. There is a large area of decreased flood risk (depth difference of > -1.50m) just to the west, this is likely due to the proposed A303 north of this location cutting off the surface water flow path. In addition, there is a large area of reduced depth between the B3083 and the River Till (depth difference of 0.05m to 0.50m). This decrease in floodwater flowing into the River Till is due to the existing flow path being blocked by the realigned A303.

8.3.5 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 the River Till. A concept solution has been modelled and the depth difference results are illustrated in Figure 8.3.

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

Third Party Flood Risk

8.3.7 As displayed in Figure 8.3, the majority of depth differences are within the red line boundary, where the land is owned by Highways England. A small area of depth decrease is located outside the boundary along the River Till and near Winterbourne Stoke. Small areas of flood increase can be seen outside the red line boundary towards the western perimeter. The changes in depths outside of the red line boundary are located within agricultural land. As discussed in Chapter 11 of the Environmental Statement, the impact magnitude of flooding on agricultural land is considered to be low. In order to mitigate this third party flooding in a 1% AEP +40% climate change event, the land owner(s) will be consulted with.

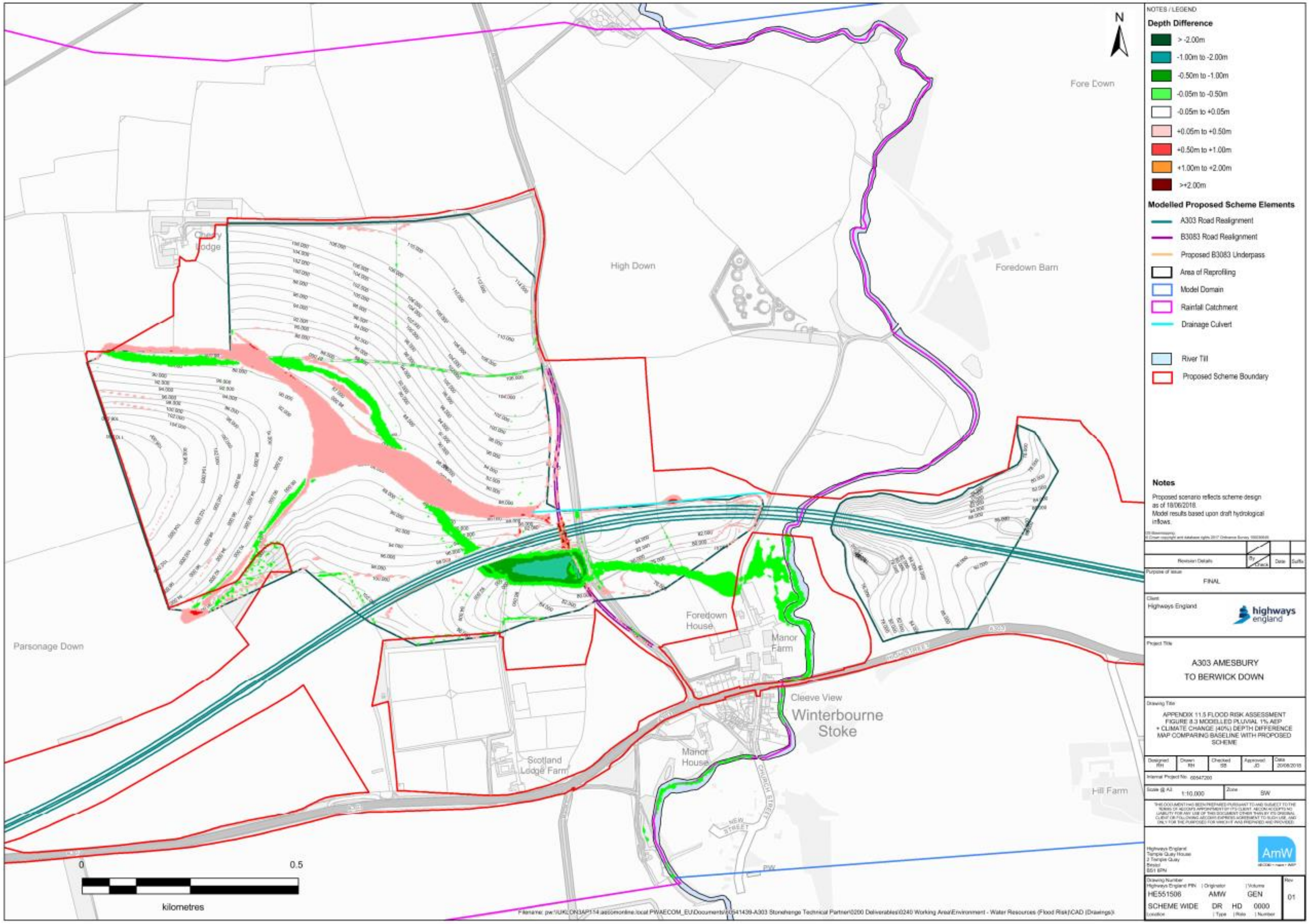

Figure 8.3: Modelled Pluvial 1% AEP plus Climate Change (+40%) Depth Difference Map comparing baseline with proposed scheme

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 11.3: 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 0.5-1.0m 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.

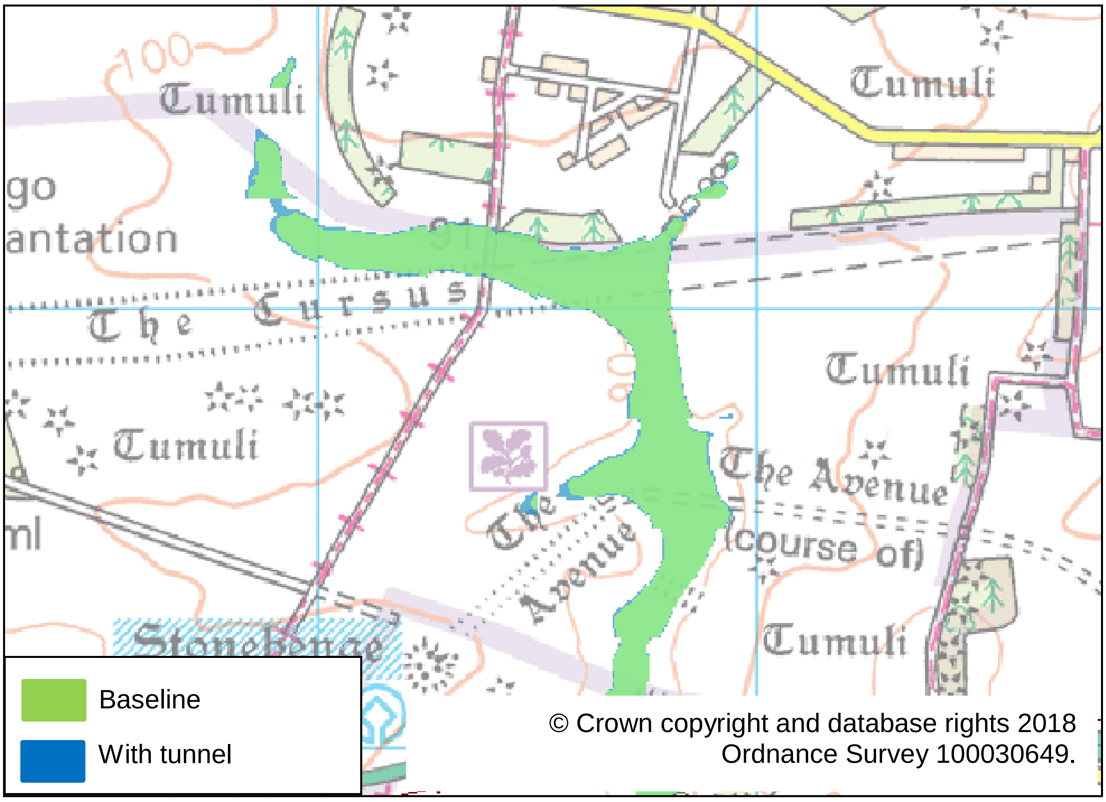

Figure 8.4: Groundwater level at depth shallower than 2m bgl

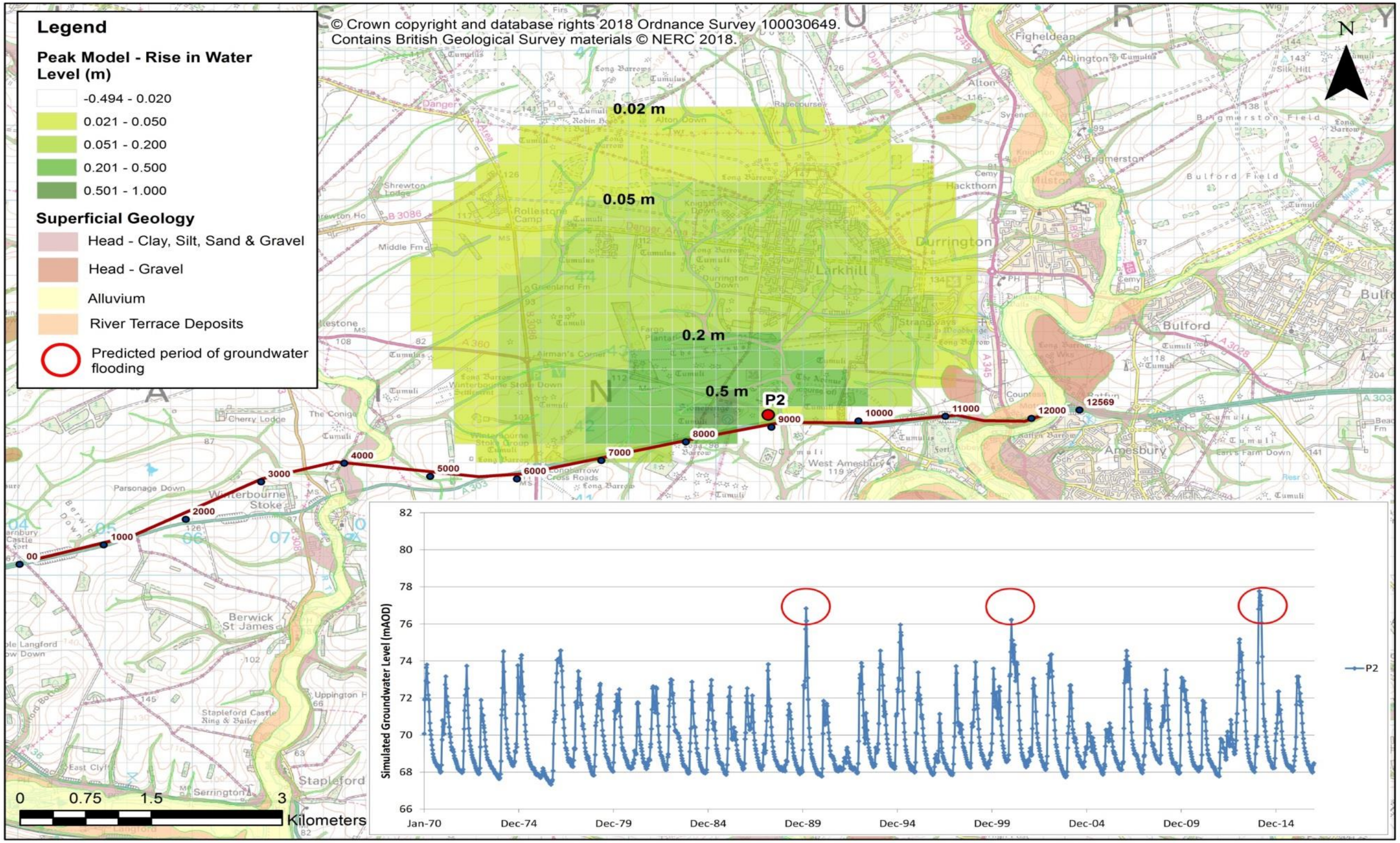

Figure 8.5: Modelled rise in groundwater level at peak (flood) groundwater condition

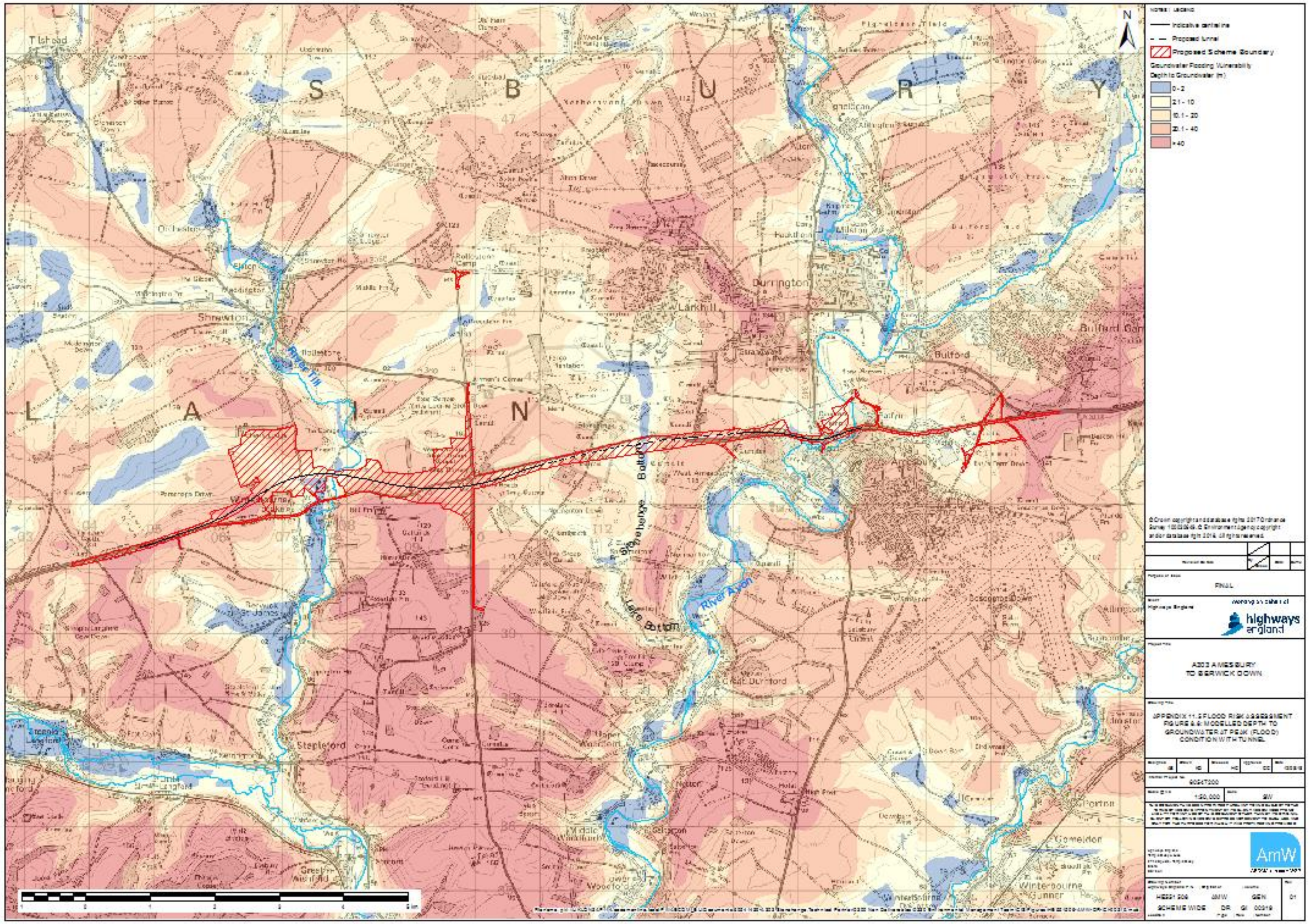

Figure 8.6: Modelled depth to groundwater at peak (flood) condition with tunnel

Increase to River Base flow

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

8.4.5 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 500m3/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.

8.4.6 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.

8.4.7 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.

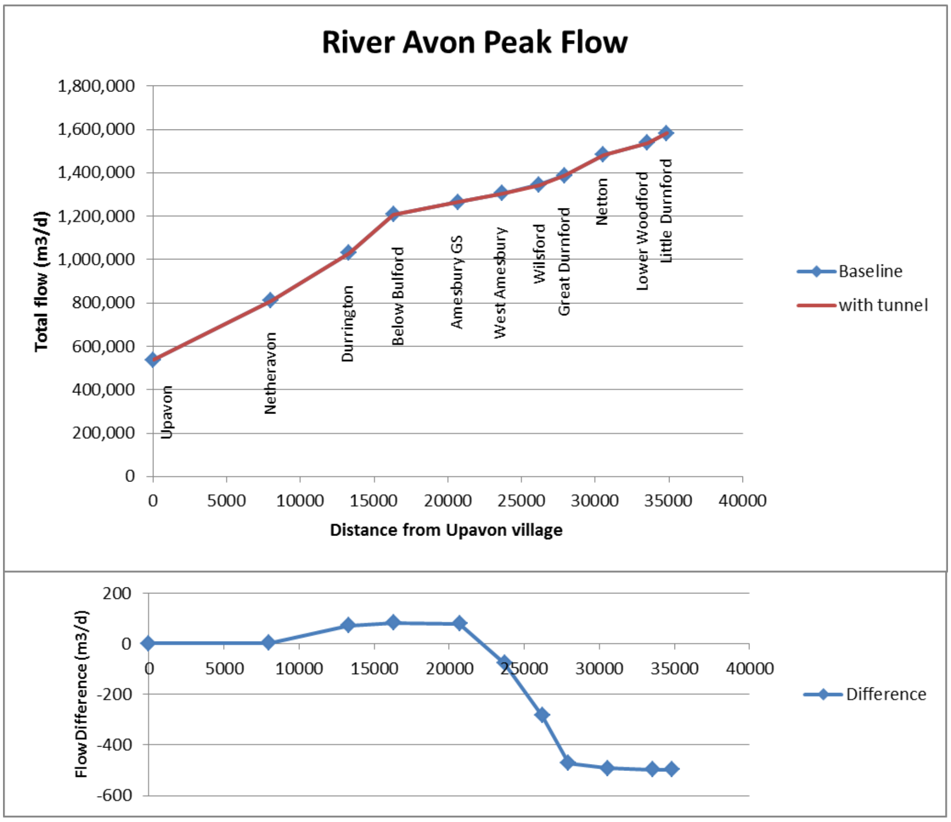

Figure 8.7 River Avon peak flow accretion profile

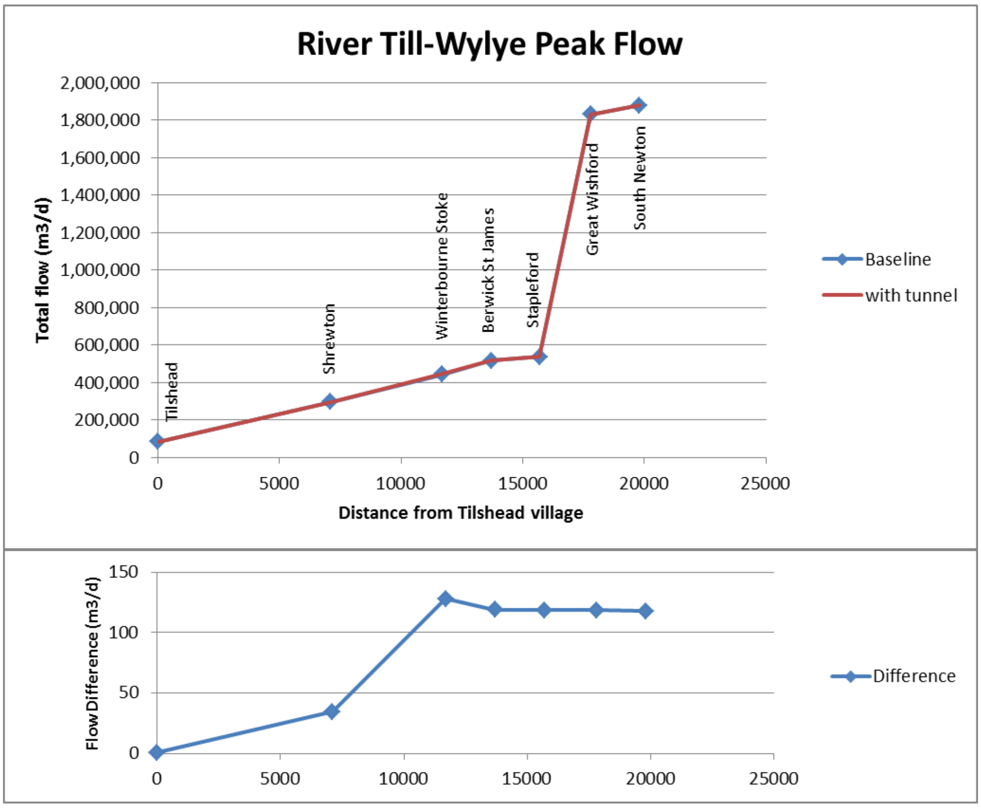

Figure 8.8 River Till and Wylye peak flow accretion profile

8.4.8 Modelled results show that the effects of the tunnel on high river flows would be negligible.

8.4.9 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 other 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 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 plus 40% climate change flood extent when compared to the existing Environment Agency flood maps.

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 variation in flood depth (between -0.09m to -0.04m) in an area to the south of the temporary Bailey Bridge and haul route, and an area of increased flooding (<0.02m) to the north of the temporary Bailey Bridge, 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 depth decrease is shown within the red line boundary, where the land is owned by Highways England. A depth decrease can be seen to the south of the red line boundary within the River Till and along its river bank. Similarly, two small areas of depth increase are located outside of the boundary, to the north and south, either within the River Till or along its river bank. The depth increase is between 0.01m to 0.1m within agricultural land. This surrounding the River Till is agricultural land. As discussed in Chapter 11 of the Environmental Statement, the impact magnitude of flooding on agricultural land is considered to be low. In order to mitigate this third party flooding in a 1% AEP event, the land owner(s) and Environment Agency will be consulted.

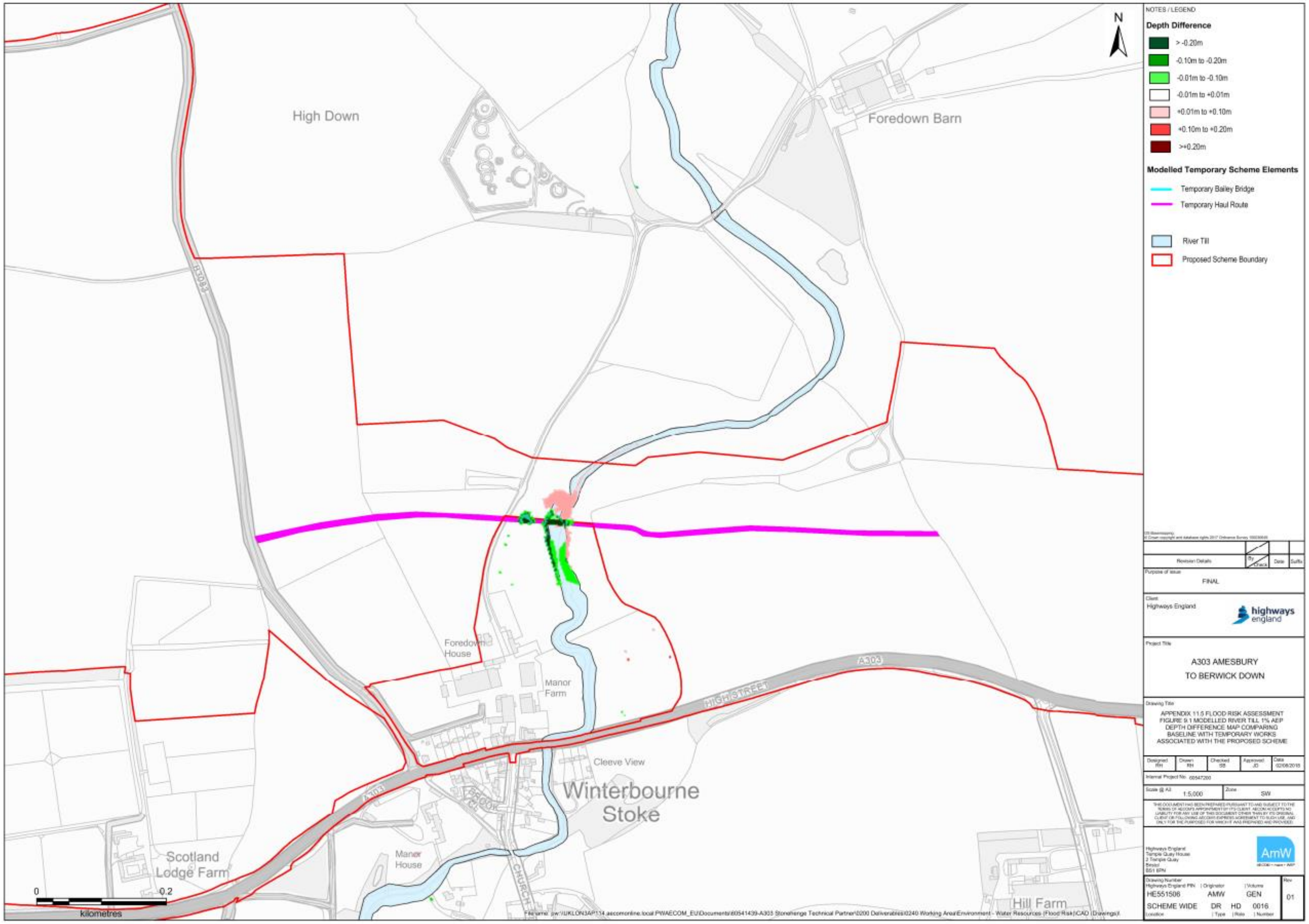

Figure 9.1: Modelled River Till 1% AEP Depth Difference Map comparing baseline with temporary works associated with the proposed scheme

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 in no increase in surface water flooding to other receptors, in accordance with the OEMP.

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, largely coincide with the corresponding Flood Zones produced by the Environment Agency. The baseline modelling flood extents for the River Avon show a substantial decrease in comparison to the equivalent 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 0.1% 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 it 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.

10.3.2 The hydraulic modelling for the River Avon shows no areas of increased flood depth compared to the baseline as a result of the proposed scheme.

10.3.3 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.

10.3.4 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

10.3.5 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.

10.3.6 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% AEP or more frequently.

10.3.7 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 land drainage solution to enable the overland flow path to continue towards the River Till.

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 0.5-1.0m 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 plus climate change (+40%) 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 and an area of minor increase in flood depth (approximately +0.02m) towards the east of the River Till. 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.

10.4.4 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 in no increase in surface water flooding to other receptors, in accordance with the OEMP.

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 Where any alterations to modelled flood depth have been identified, the areas affected are limited to areas of agricultural land. Since these areas are already at risk of flooding the minor changes in flood depths would not increase the risk since there are no sensitive receptors that could be impacted by this change.

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

10.5.5 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

Annex 1 Part A – Fluvial Hydraulic Modelling Report

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Annex 1A Fluvial Hydraulic Modelling Report

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

October 2018

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Chapter

1 Introduction

1.1 Background

1.1.1 In order to robustly assess the impact of the proposed development 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 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.

1.2 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 development, 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 River Till catchment.
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, 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 design events with the following Annual Exceedance Probabilities (AEPs); 5% AEP, 1% AEP and 0.1% AEP.

[1] HM Government (2018) Revised National Planning Policy Framework

1.3.2 In line with Environment Agency (EA) guidance[2], the 1% 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 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 EA 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 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] Environment Agency (2016) Adapting to Climate Change: Advice for Flood and Coastal Management Authorities.
[3] AmW (2017) A303 Stonehenge. Flood Risk Hydraulic Modelling Methodology.

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 4.2 and TUFLOW version 2016-03-AB.

2.3 1D Model - River Channel Survey

2.3.1 Highways England commissioned a survey of 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 EA 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.

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;

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
Concrete	0.025
Brick Walls	0.03

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 inflows 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.

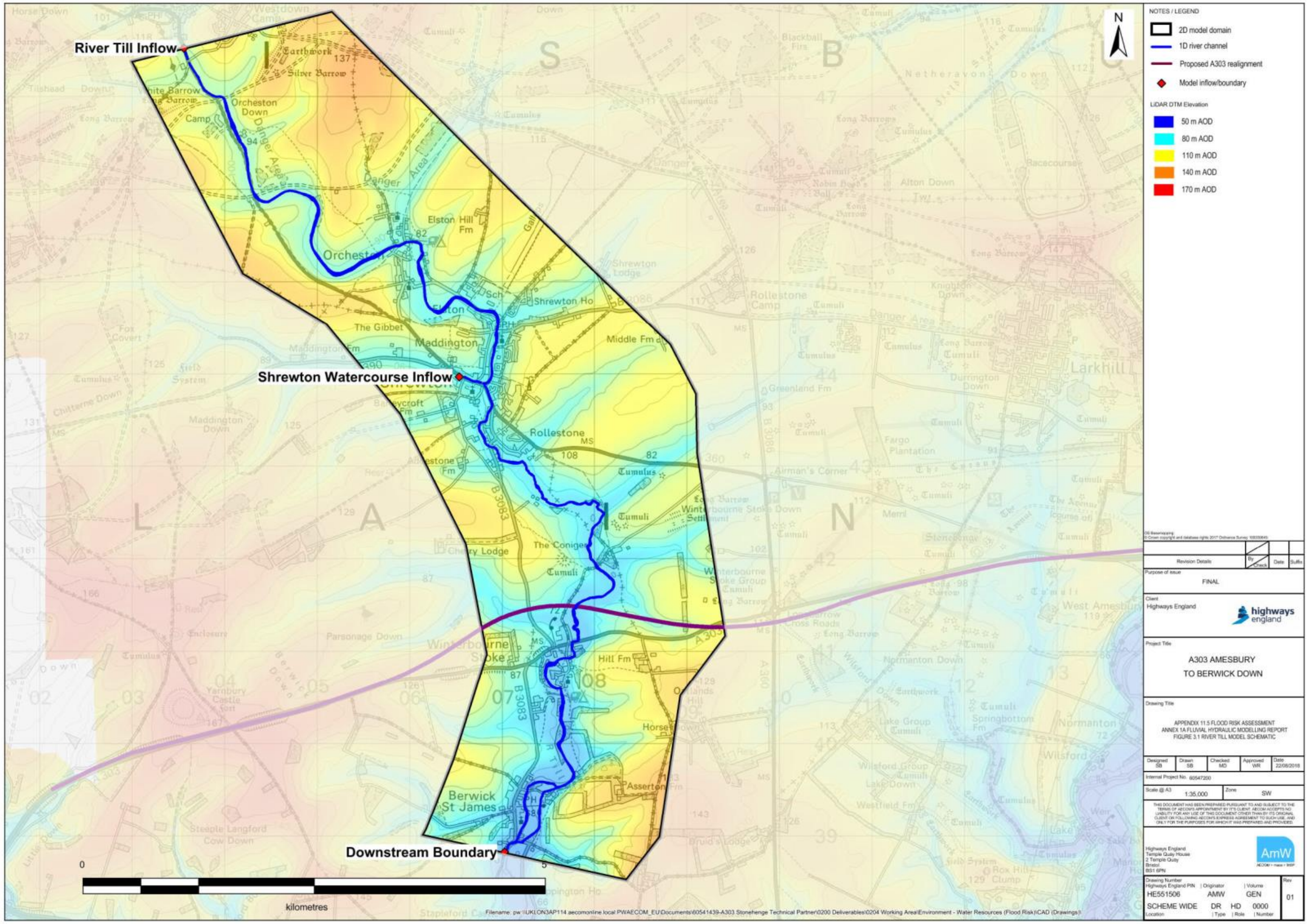


Figure 3.1 River Avon Hydraulic Model Schematic

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 100yr water level, taken from 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.

4.1.2 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 14 km in length. This model extent is considered more than sufficient to assess flood risk to and from the proposed scheme.

4.1.3 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.

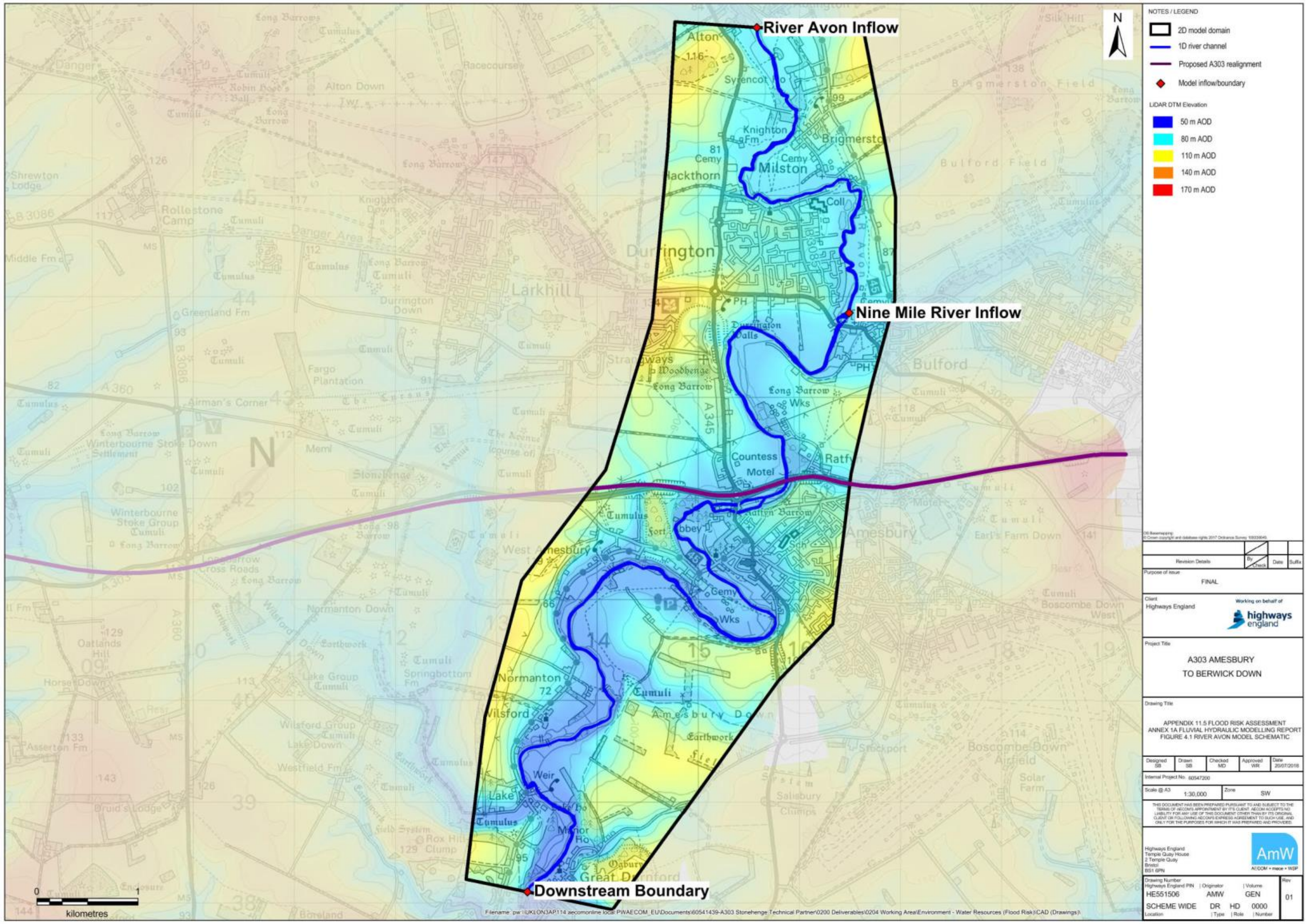


Figure 4.1 River Till Hydraulic Model Schematic

4.1.4 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.

4.1.5 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 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.

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

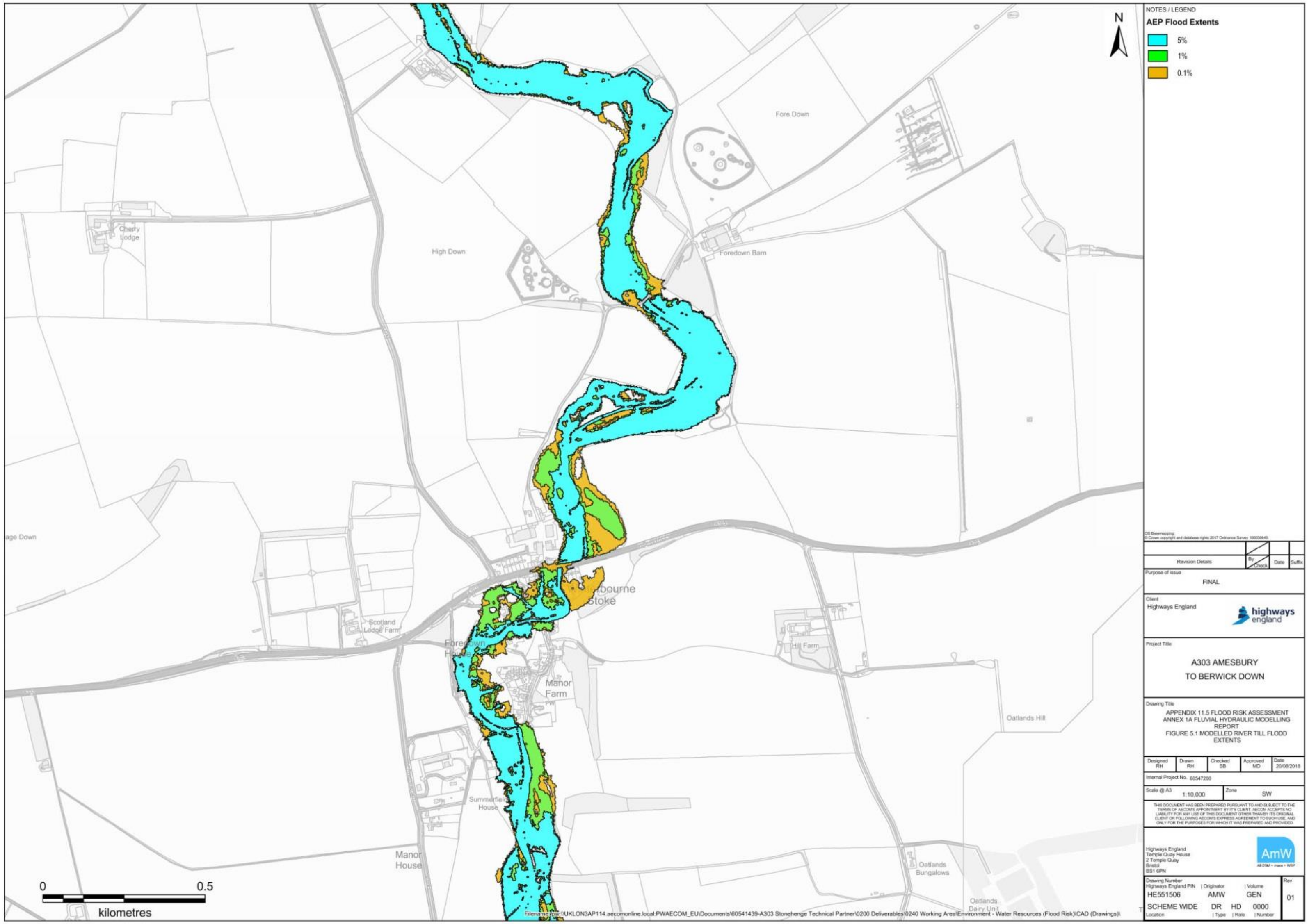


Figure 5.1 River Till Baseline Maximum Flood Extent Comparison

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

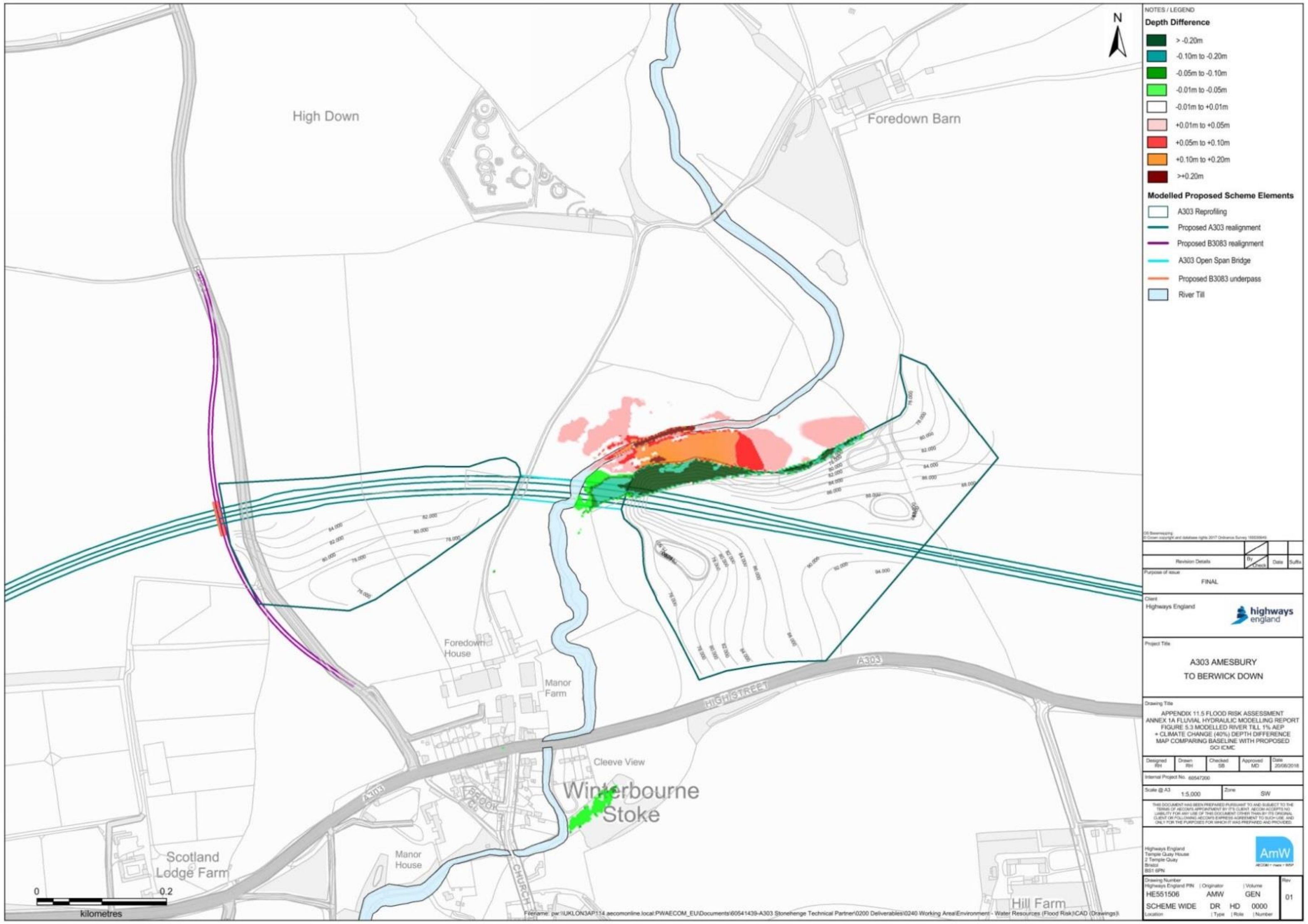


Figure 5.3 River Till Maximum Flood Depth Difference Plot (Permanent)- 1% AEP Plus Climate Change (40%)

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 within 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 1.5-2.0 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 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 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 The 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 The figures above 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 increases in maximum depth, generally up to 0.1m, directly upstream of the haul route and bridge. There are commensurate decreases in modelled depth downstream, although these are also confined to within the channel.

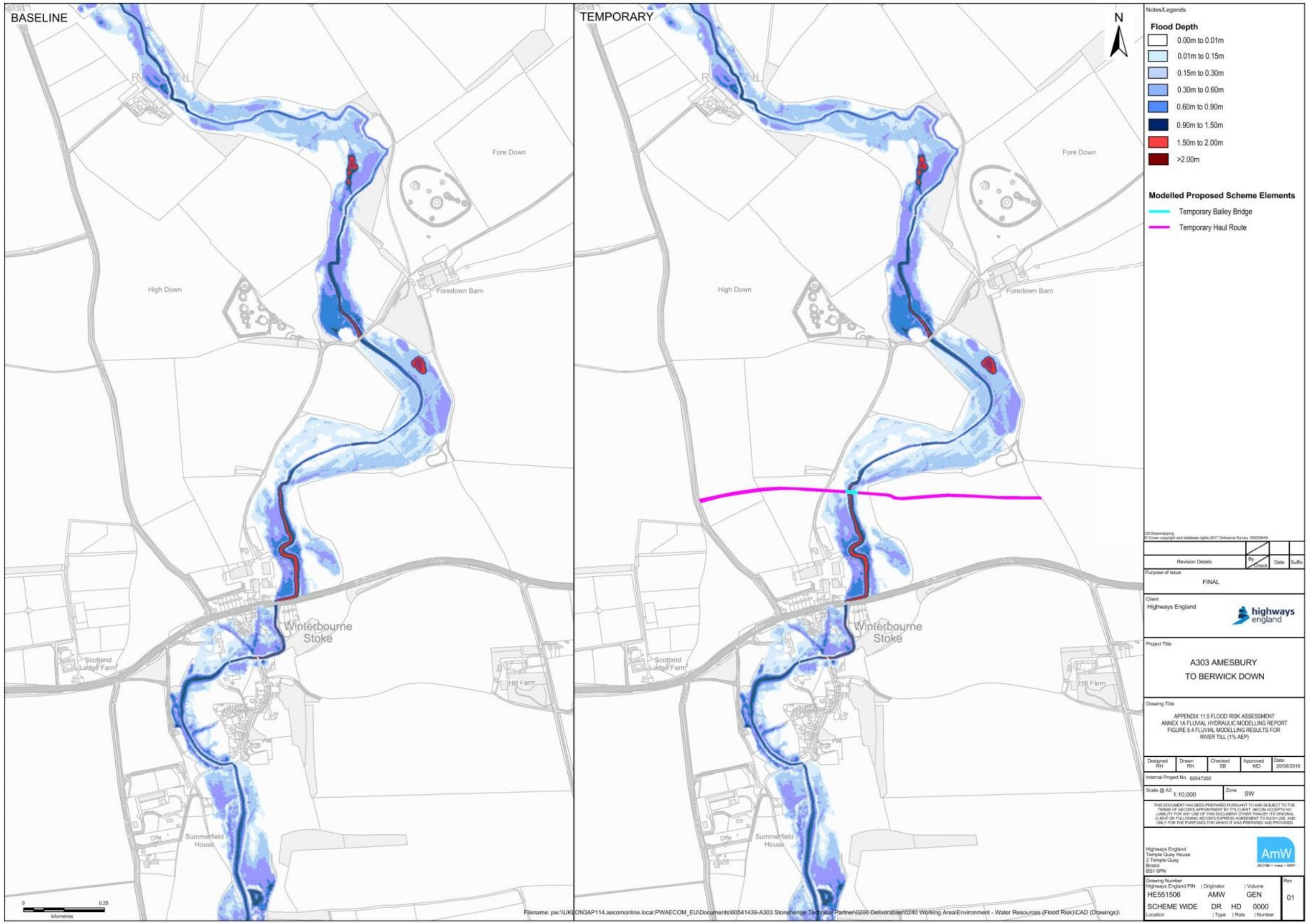


Figure 5.4 River Till Maximum Flood Depth Comparison (Temporary)- 1% AEP Plus Climate Change (40%)

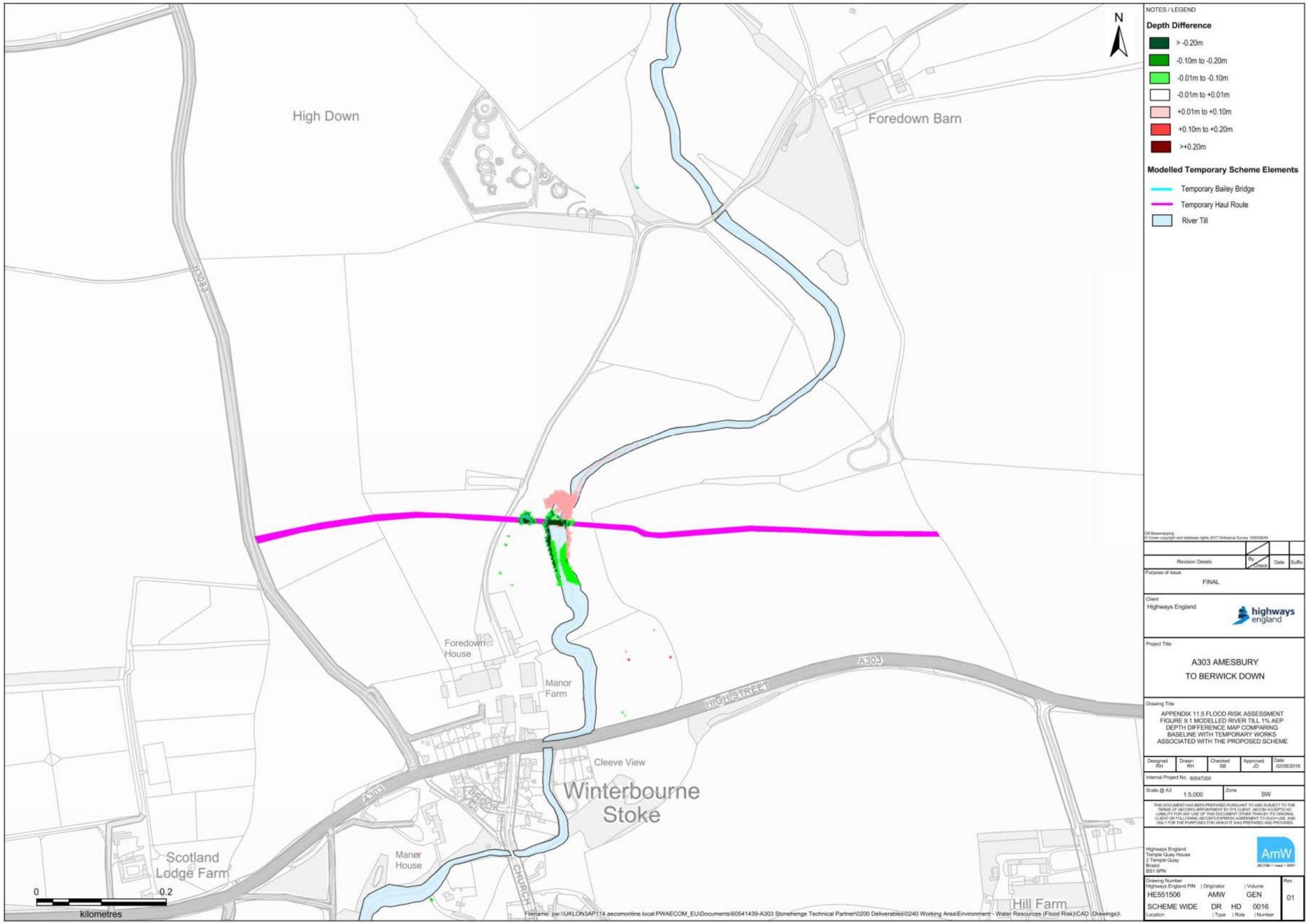


Figure 5.5 River Till Maximum Flood Depth Difference Plot (Temporary)- 1% AEP Plus Climate Change (40%)

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

5.3.6 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.

5.3.7 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 not lead to an increase in flood risk within the region of any vulnerable receptors for the 1% AEP event.

6 River Avon Model Results

6.1 Baseline Scenario Results

6.1.1 The baseline 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.

6.1.2 Comparison of modelled flood extents with corresponding EA 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. The 5% AEP event is largely contained within bank for this reach of the Avon, whilst out of bank flow for the 1% AEP event is also limited. There is a substantial increase in the maximum flood extent for the 0.1% AEP event, and the majority of the River Avon valley is inundated within this event. 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 event including a +40% uplift in peak flow to allow for climate change. 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 flood inundation is limited in extent, indicating the presence of raised bank or floodplain ground levels.

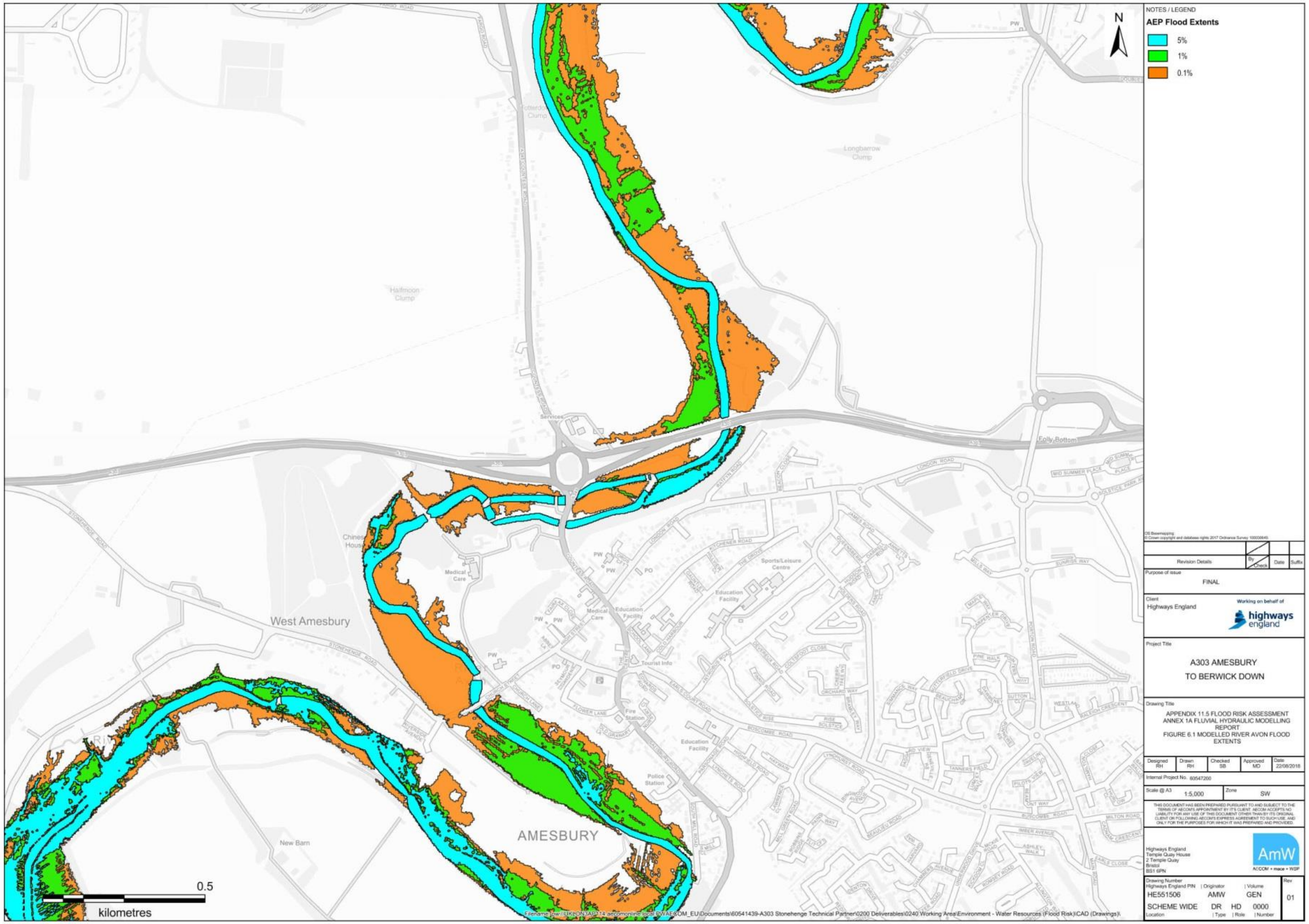


Figure 6.1 River Avon Baseline Maximum Flood Extent Comparison

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

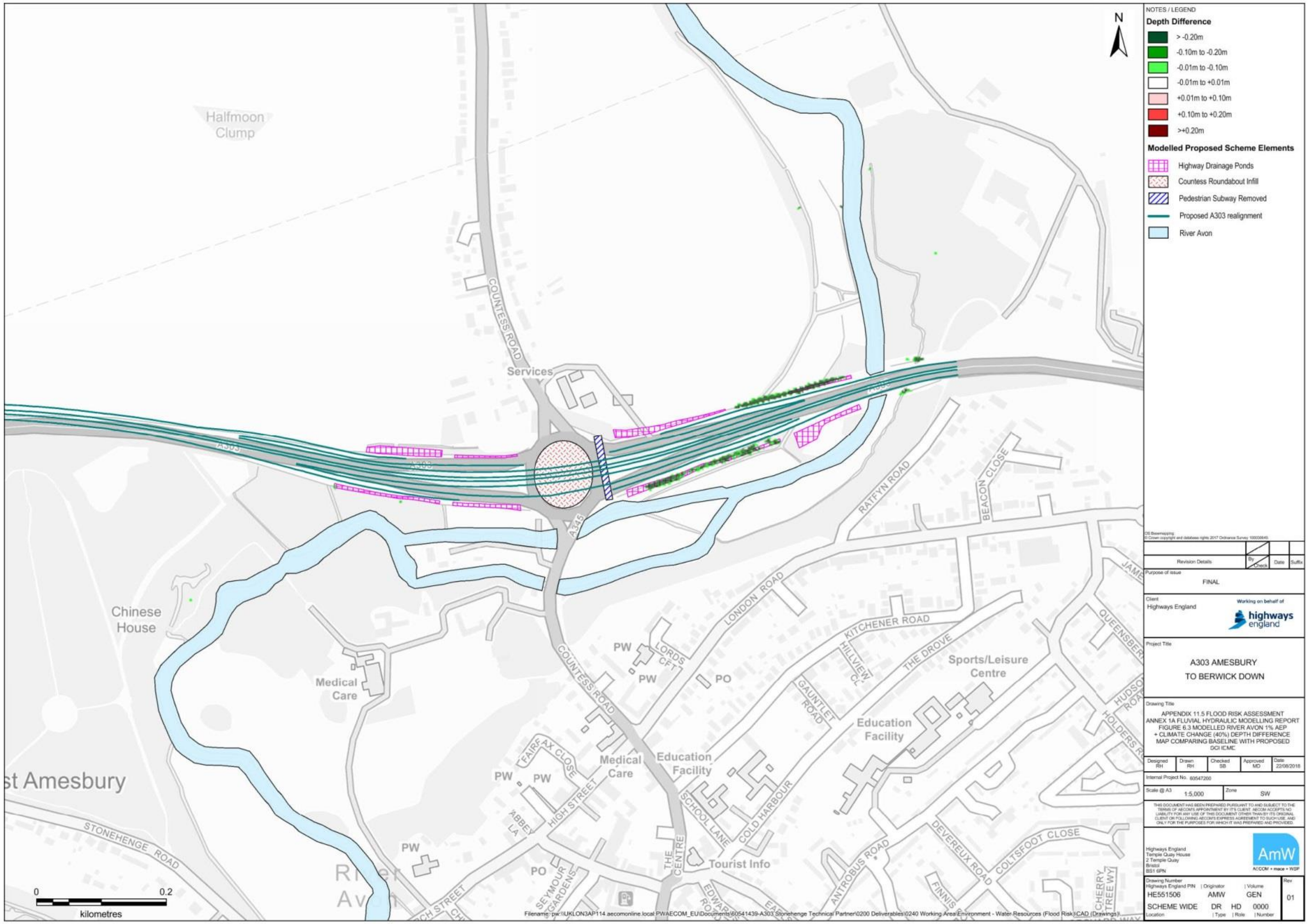


Figure 6.3 River Avon Maximum Flood Depth Difference Plot- 1% AEP Plus Climate Change (40%)

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, along with the majority of the road drainage ponds.

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. The footprint of these bunds is limited and hence model outputs suggest that there are no significant increases in depth observed elsewhere due to the displacement of flood water. This suggests that the volume of water displaced is distributed across the remainder of the floodplain and is not sufficient to cause a significant increase in maximum flood depth.

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 3.33% 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, and for the 1% AEP +40% climate change design event no increases in flood depths are observed. Whilst some increases in maximum flood depth are observed for the 0.1% AEP event, these increases are localised and do not coincide with any vulnerable receptors.

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 development 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 EA 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 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.

Table 7-1 Accuracy of Topographic Data Sources

Topographical Data Source	Spatial Resolution (m)	Stated Vertical Accuracy
EA 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. 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.

7.1.7 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.

7.1.8 In light of Section 7.1.7, 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.

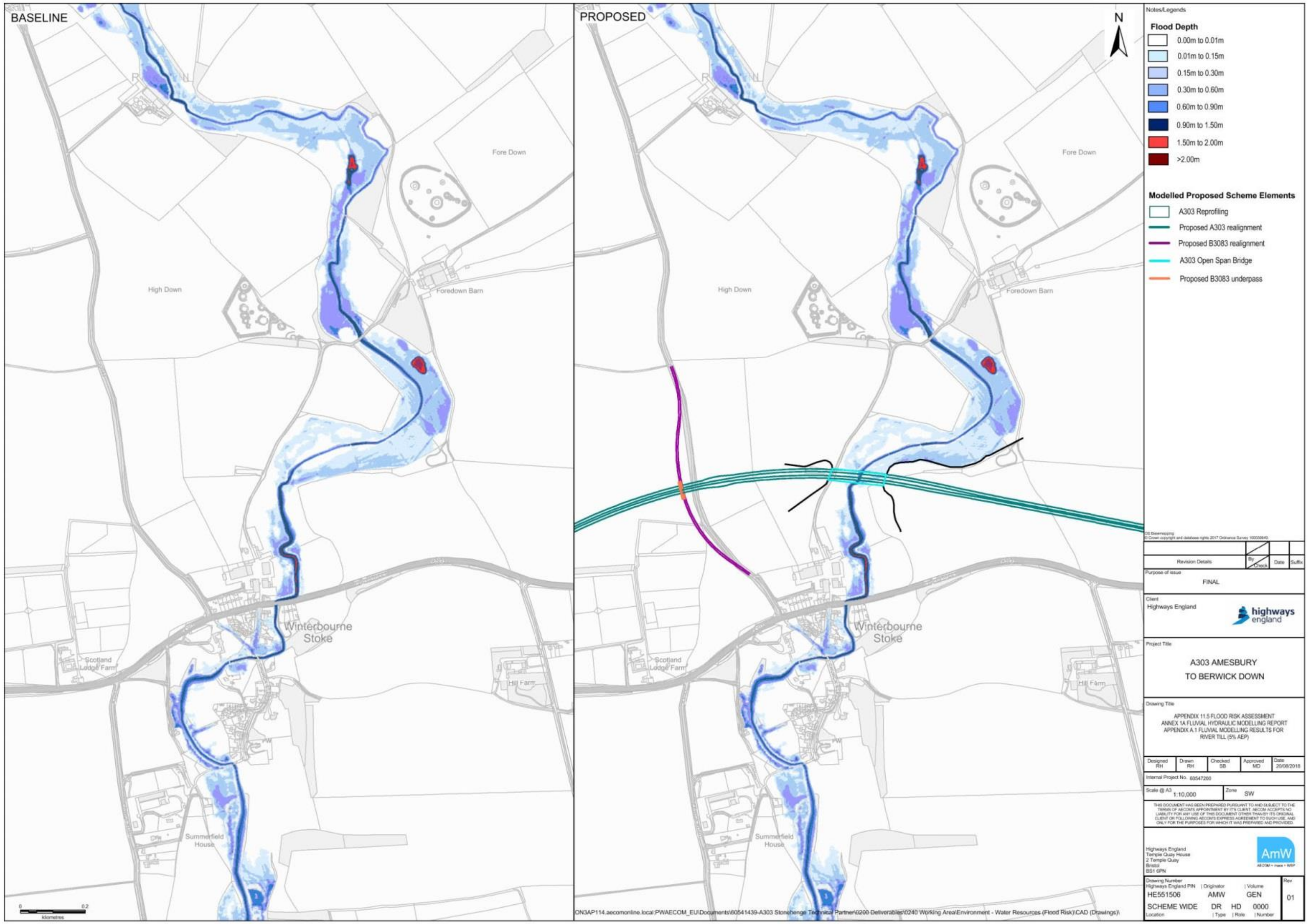
8.2 River Avon

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, only small decreases in flood depth are observed within the footprint of road drainage ponds for the 1% AEP +40% climate change design event, whilst no discernible increases in flood depth are present.

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

Appendix A- River Till Flood Mapping


BASELINE
PROPOSED
N
Fore Down
High 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.1 FLUVIAL MODELLING RESULTS FOR RIVER TILL (5% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
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2 Temple Quay
Bristol
BS1 6PN
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Drawing Number
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SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
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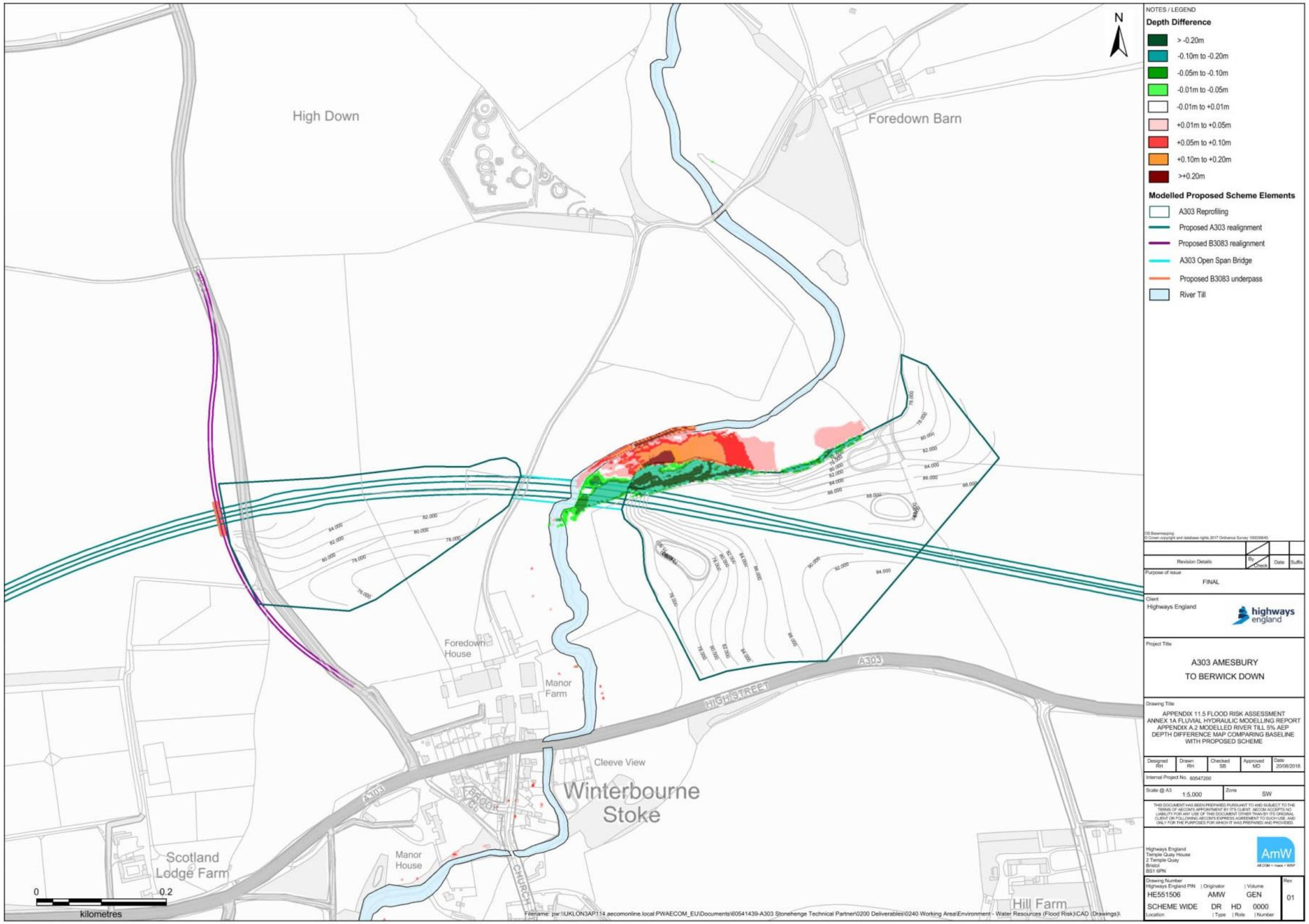
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High Down
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Foredown House
Manor Farm
Cleeve View
Winterbourne Stoke
Manor House
Scotland Lodge Farm
Hill Farm
A303
HIGH STREET
CHURCH
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0.2
kilometres
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
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>+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
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
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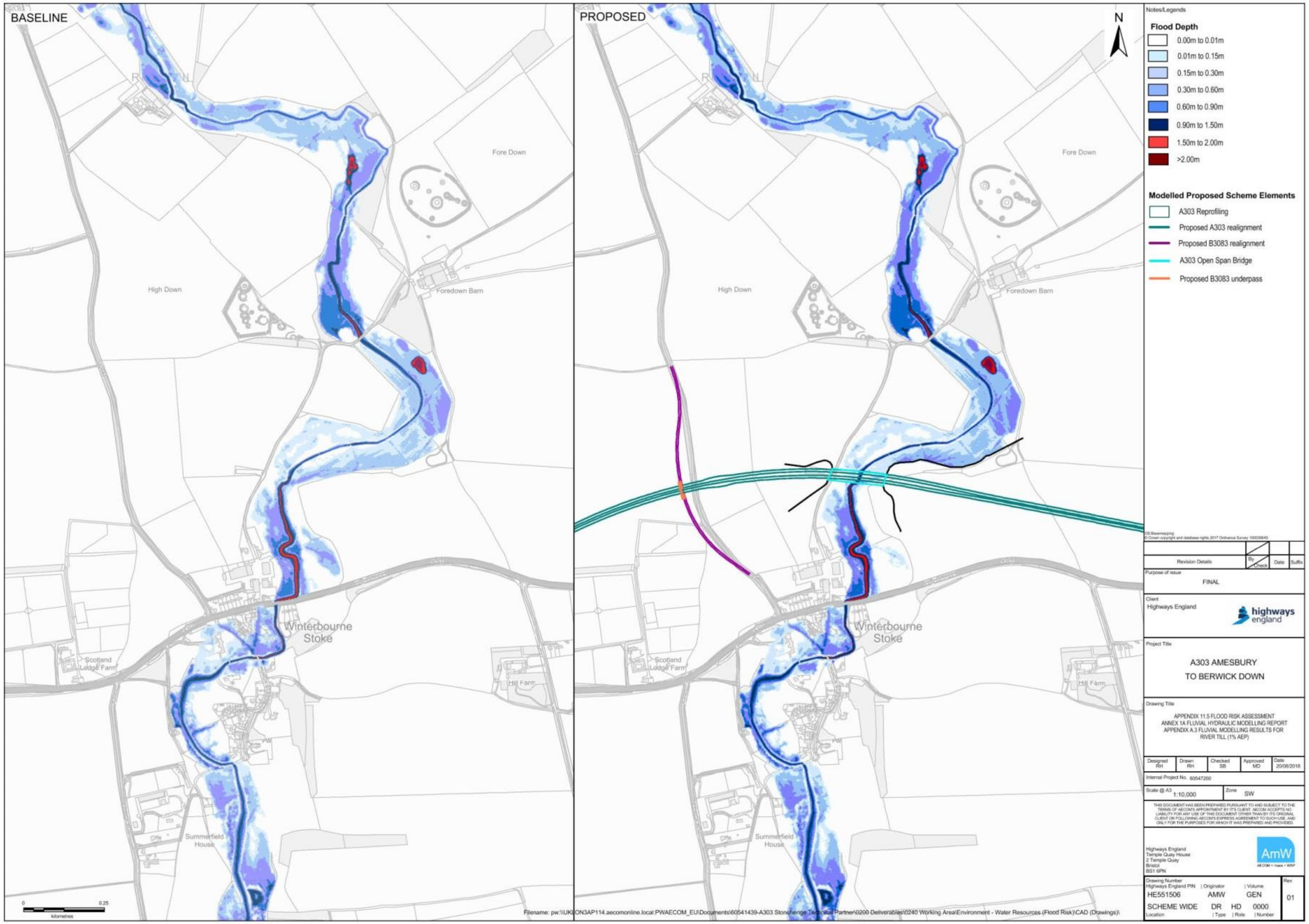
BASELINE
PROPOSED
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High Down
Fore Down
Foredown Barn
Winterbourne
Stoke
Scotland
Lodge Farm
Summerfield
House
Notes/Legends
Flood Depth
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Proposed B3083 underpass
Revision Details
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Date
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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
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AmW
Drawing Number
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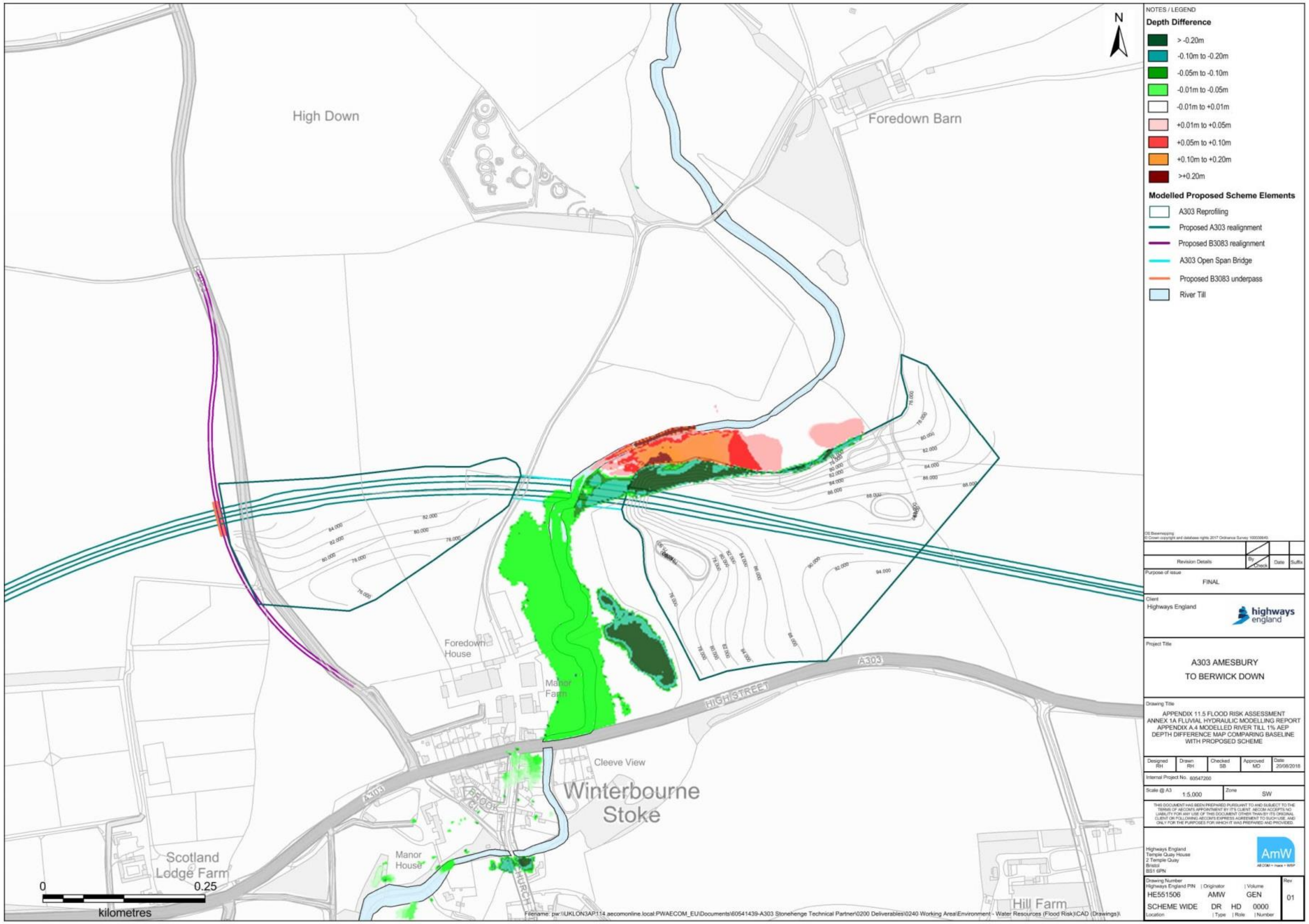
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01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High Down
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Winterbourne Stoke
Scotland Lodge Farm
Manor House
Hill Farm
HIGH STREET
A303
0
0.25
kilometres
NOTES / LEGEND
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
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.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
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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
Winterbourne Stoke
Summerfield House
0
0.25
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
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 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
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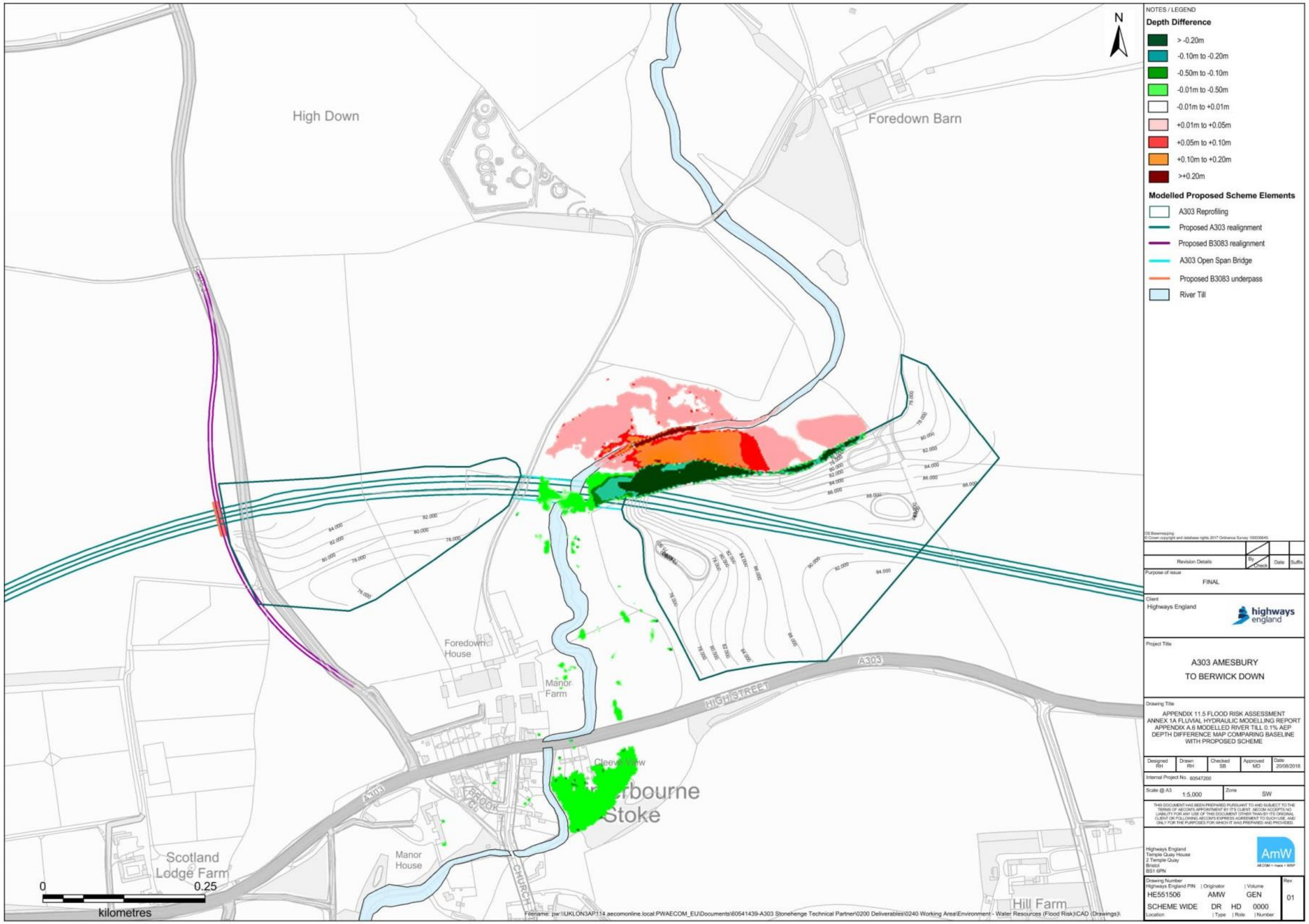
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM • mace • WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01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Depth Difference
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-0.10m 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
High Down
Foredown Barn
Foredown House
Manor Farm
HIGH STREET
A303
Cleeve View
Winterbourne Stoke
Scotland Lodge Farm
Manor House
CHURCH
Hill Farm
0
0.25
kilometres
OS Basemapping
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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
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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
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
Fore Down
High Down
Foredown Barn
Scotland Lodge Farm
Summerfield House
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
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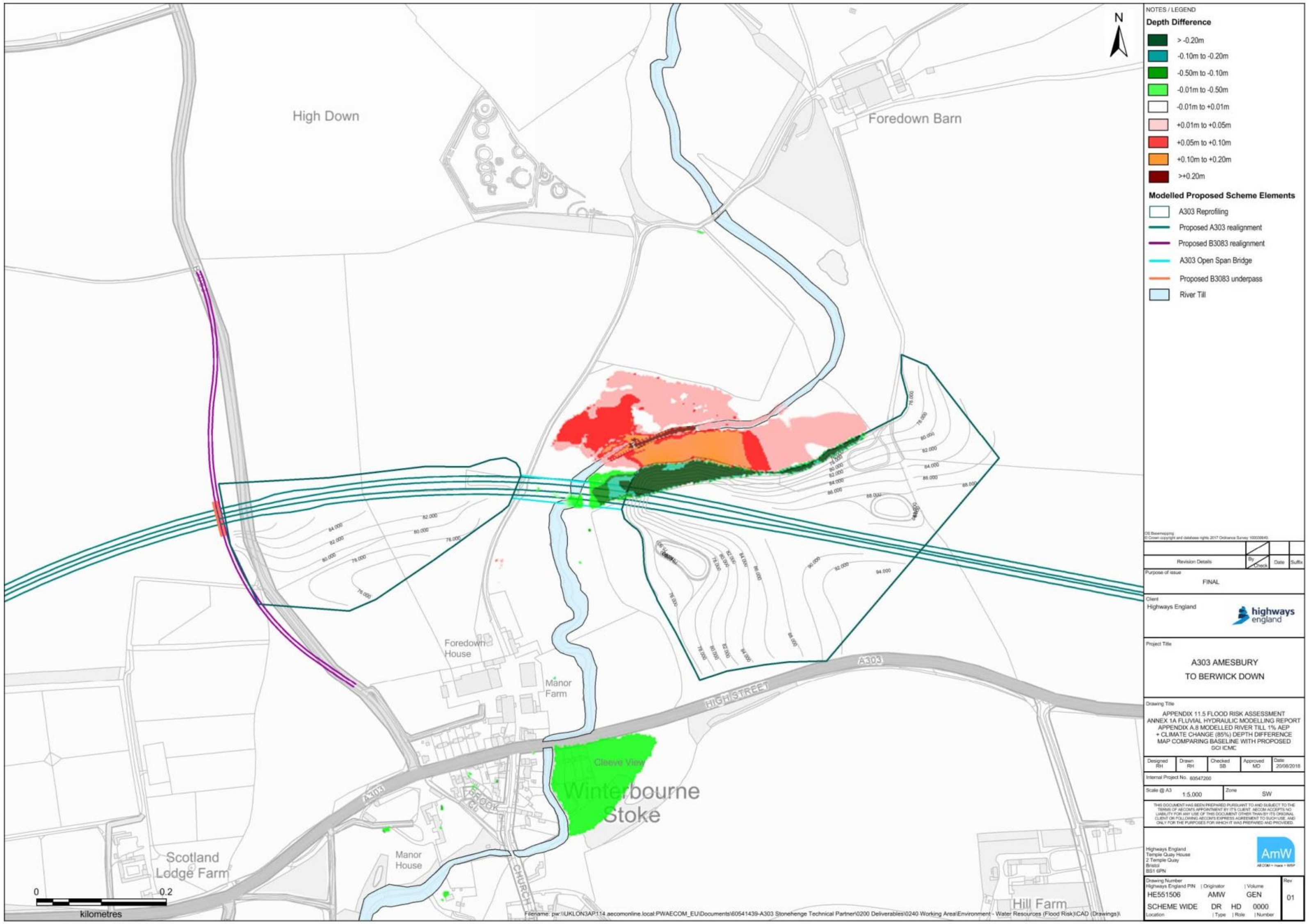
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01
0
0.25
Kilometres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NOTES / LEGEND
Depth Difference
> -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
High Down
Foredown Barn
Foredown House
Manor Farm
HIGH STREET
A303
Cleeve View
Winterbourne Stoke
Scotland Lodge Farm
Manor House
CHURCH
Hill Farm
0
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 A.8 MODELLED RIVER TILL 1% AEP
+ CLIMATE CHANGE (85%) DEPTH DIFFERENCE
MAP COMPARING BASELINE WITH PROPOSED
SCI ICMC
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
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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

Appendix B- River Avon Flood Mapping


BASELINE
PROPOSED
West Amesbury
Chinese House
0
0.5
kilometres
NOTES / LEGEND
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
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Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
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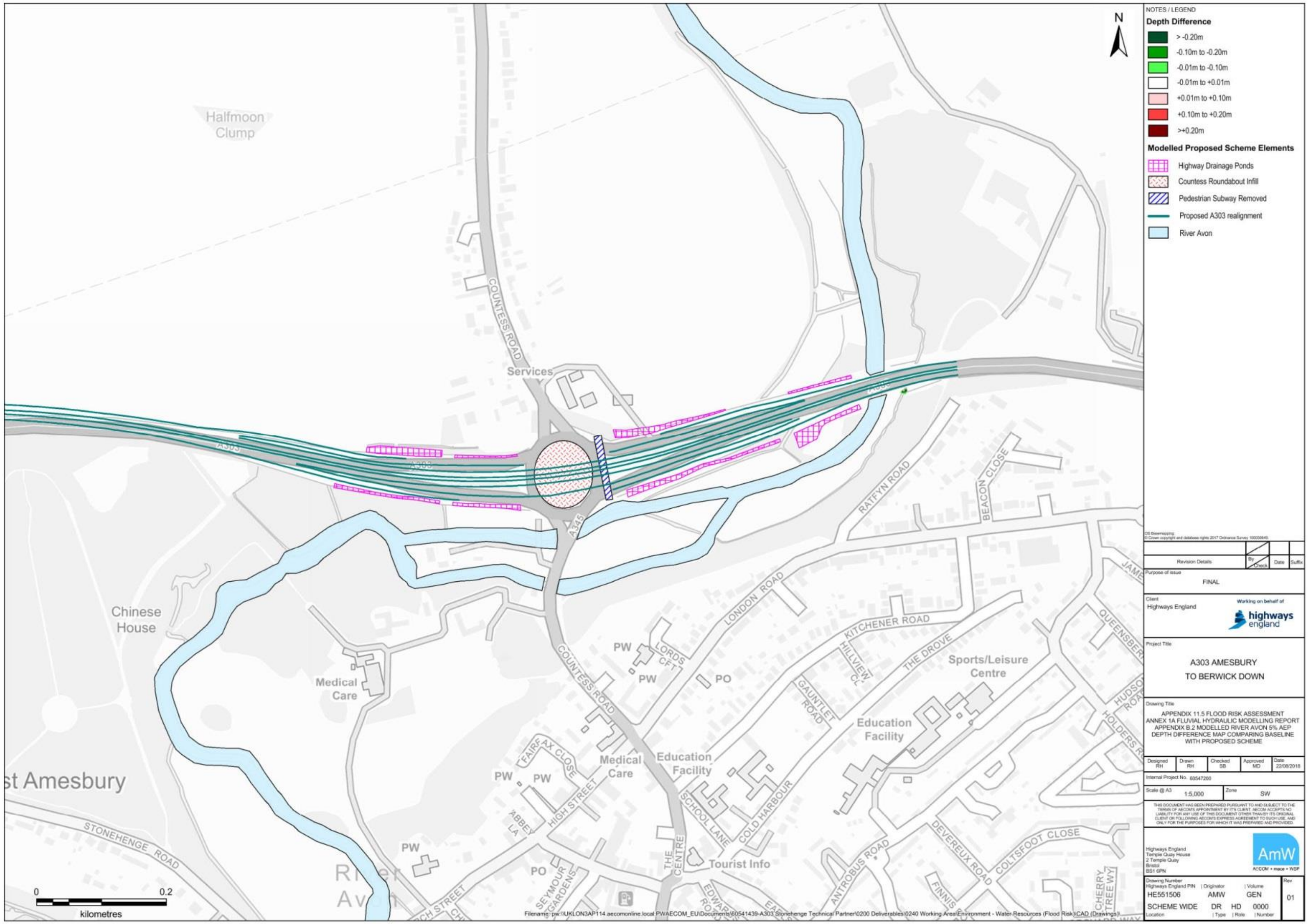
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01


NOTES / LEGEND
Depth Difference
> -0.20m
-0.10m to -0.20m
-0.01m to -0.10m
-0.01m to +0.01m
+0.01m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
N
0
0.2
kilometres
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
PW
PO
Tourist Info
STONEHENGE ROAD
HIGH STREET
SCHOOL LANE
COLD HARBOUR
ANTROBUS ROAD
DEVEREUX ROAD
COLTSFOOT CLOSE
FAIRFAX CLOSE
st Amesbury
Purpose of issue
FINAL
Client
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 22/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
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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


BASELINE
PROPOSED
West Amesbury
Chinese House
0
0.5
kilometres
NOTES / LEGEND
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
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Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
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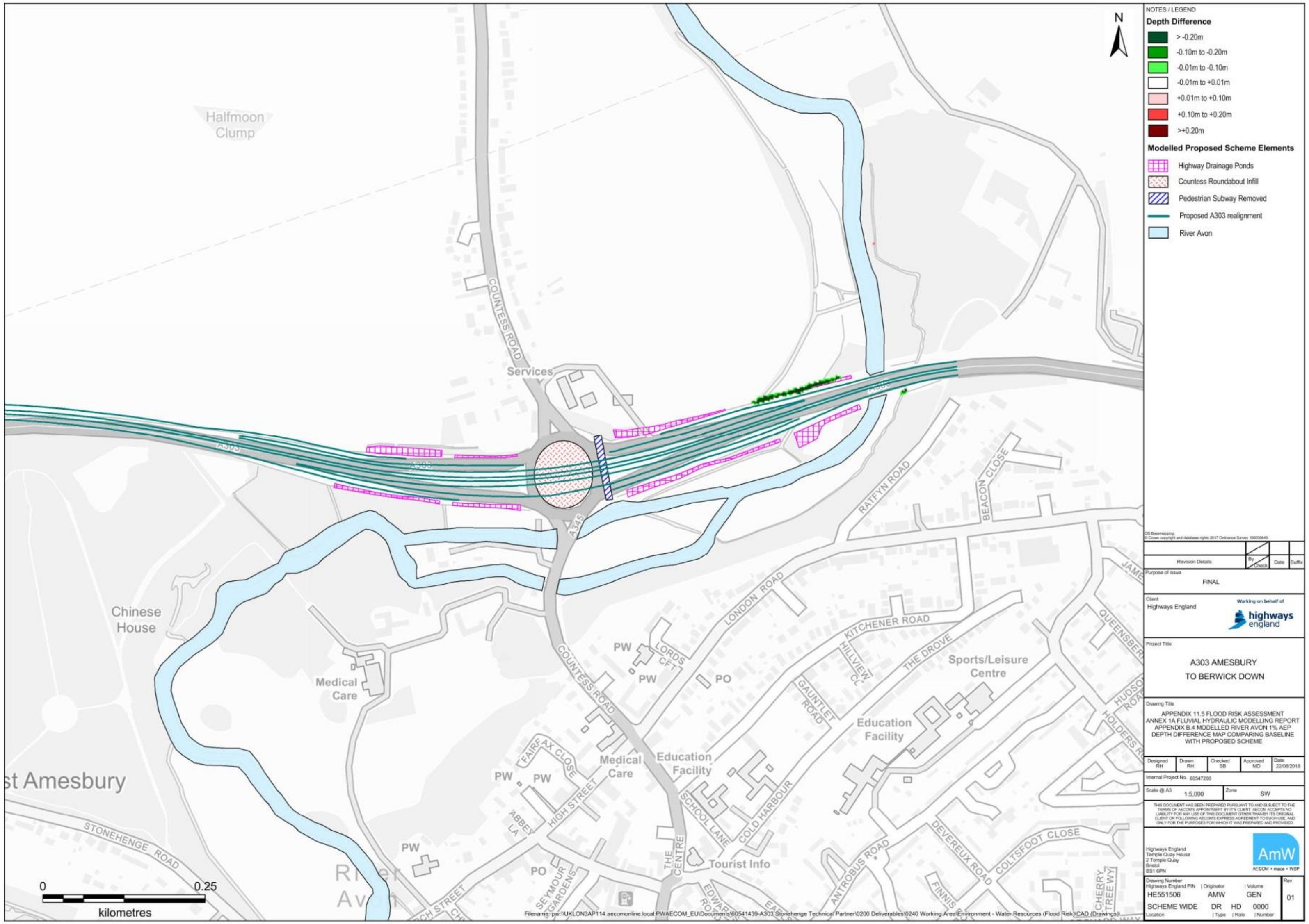
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM • mace • WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01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NOTES / LEGEND
Depth Difference
> -0.20m
-0.10m to -0.20m
-0.01m to -0.10m
-0.01m to +0.01m
+0.01m to +0.10m
+0.10m to +0.20m
>+0.20m
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
HILLVIEW CL
GAUNTLET ROAD
Sports/Leisure Centre
Education Facility
Chinese House
Medical Care
PW
PO
LORDS CFT
FAIRFAX CLOSE
HIGH STREET
SCHOOL LANE
COLD HARBOUR
ANTROBUS ROAD
DEVEREUX ROAD
COLTSFOOT CLOSE
FINNIS
Tourist Info
THE CENTRE
SEYMOUR GARDENS
ABBEY LA
STONEHENGE ROAD
River Avon
st Amesbury
QUEENSBERRY
HUDSON ROAD
HOLDERS ROAD
CHERRY TREE WY
0 0.25
kilometres
OS Basemapping
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Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Client
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 22/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
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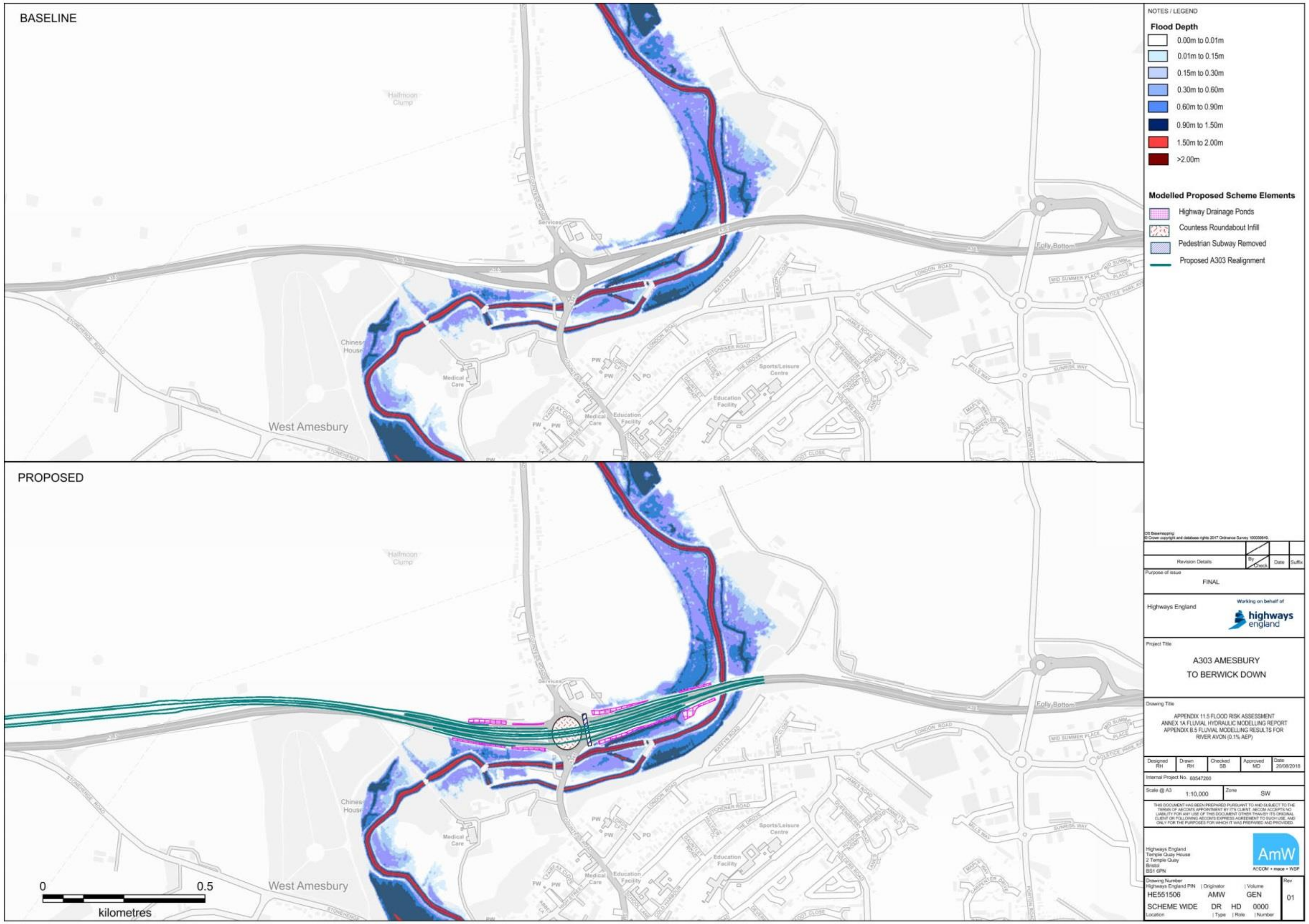
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01


BASELINE
West Amesbury
PROPOSED
West Amesbury
0
0.5
kilometres
NOTES / LEGEND
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
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
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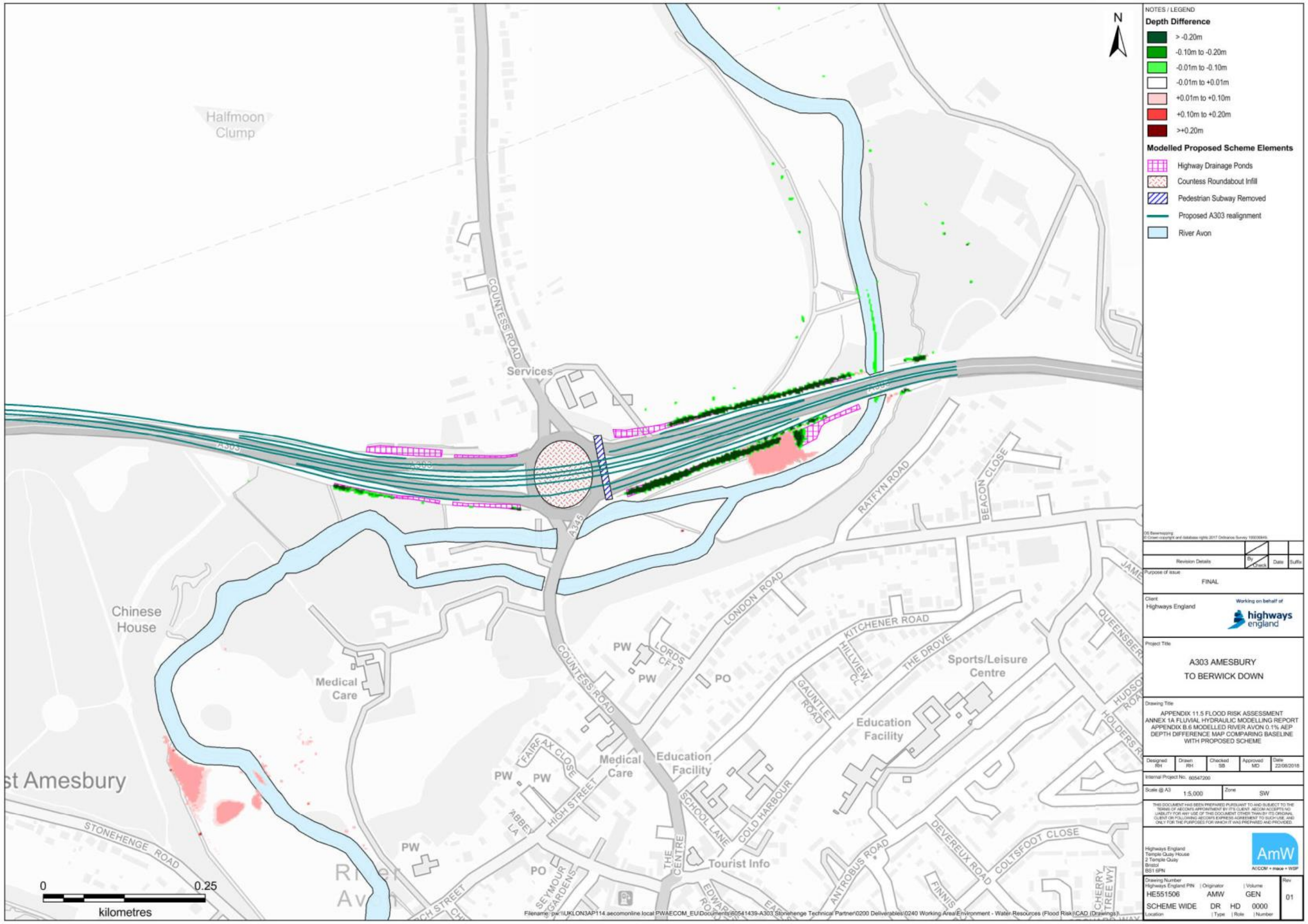
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM • mace • WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01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-0.01m to +0.01m
+0.01m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
N
0
0.25
kilometres
Halfmoon Clump
Chinese House
Services
Medical Care
Education Facility
Sports/Leisure Centre
Tourist Info
COUNTESS ROAD
A303
A345
LONDON ROAD
KITCHENER ROAD
RATFYN ROAD
BEACON CLOSE
THE DROVE
STONEHENGE ROAD
HIGH STREET
SCHOOL LANE
COLD HARBOUR
ANTROBUS ROAD
DEVEREUX ROAD
COLTSFOOT CLOSE
FAIRFAX CLOSE
Revision Details
Purpose of issue
FINAL
Client
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 22/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
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Highways England
Temple Quay House
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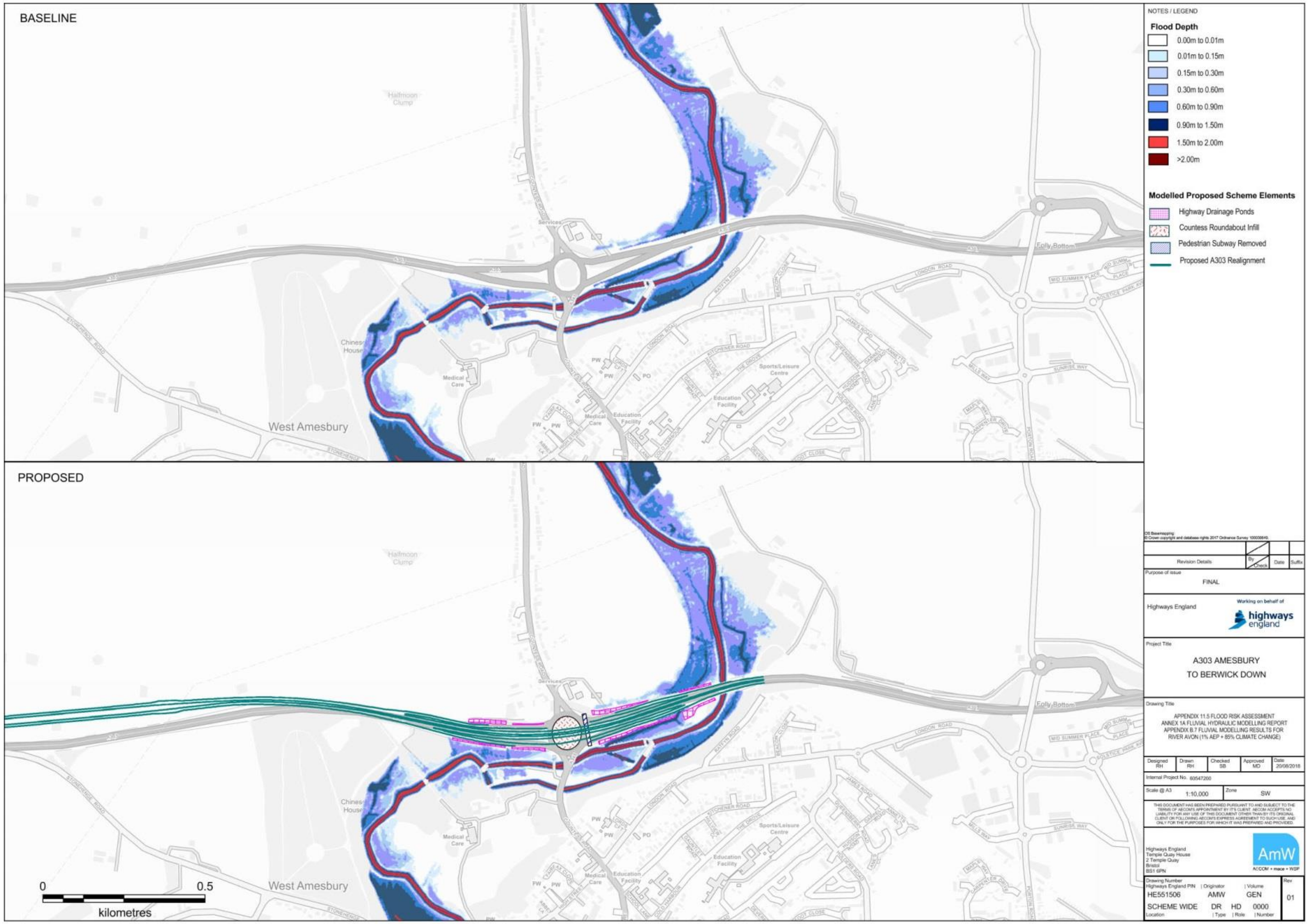
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01


BASELINE
PROPOSED
West Amesbury
0
0.5
kilometres
NOTES / LEGEND
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
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Revision Details
By
Check
Date
Suffix
Purpose of issue
FINAL
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
Scale @ A3 1:10,000
Zone SW
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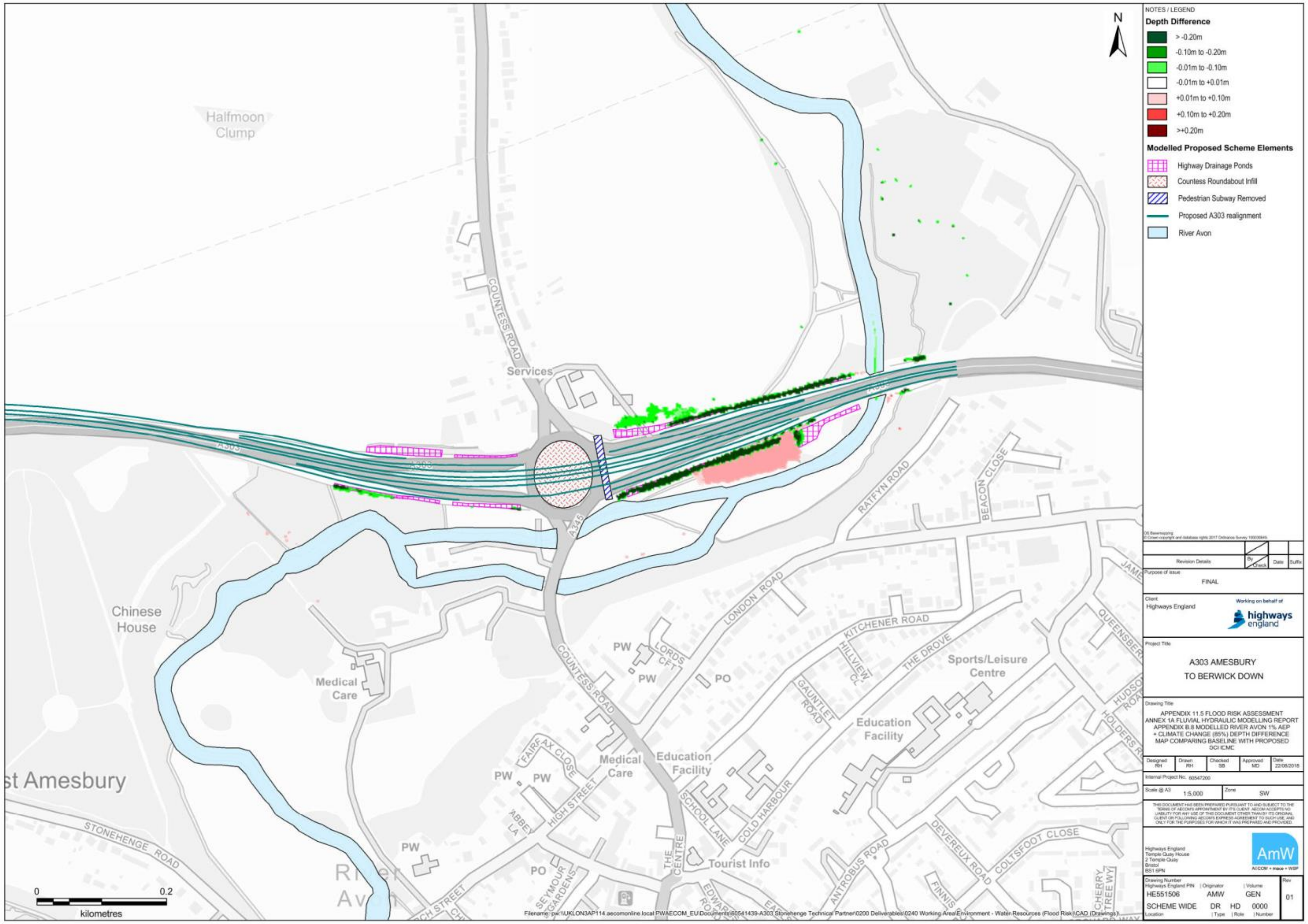
Highways England
Temple Quay House
2 Temple Quay
Bristol
BS1 6PN
AmW
AECOM + mace + WSP
Drawing Number
Highways England PIN | Originator | Volume
HE551506 AMW GEN
SCHEME WIDE DR HD 0000
Location | Type | Role | Number
Rev
01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Depth Difference
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-0.10m to -0.20m
-0.01m to -0.10m
-0.01m to +0.01m
+0.01m to +0.10m
+0.10m to +0.20m
>+0.20m
Modelled Proposed Scheme Elements
Highway Drainage Ponds
Countess Roundabout Infill
Pedestrian Subway Removed
Proposed A303 realignment
River Avon
Halfmoon Clump
Services
Chinese House
Medical Care
Education Facility
Sports/Leisure Centre
Tourist Info
st Amesbury
COUNTESS ROAD
A303
A345
LONDON ROAD
KITCHENER ROAD
THE DROVE
RATFYN ROAD
BEACON CLOSE
STONEHENGE ROAD
HIGH STREET
SCHOOL LANE
COLD HARBOUR
ANTROBUS ROAD
DEVEREUX ROAD
COLTSFOOT CLOSE
FAIRFAX CLOSE
0 0.2
kilometres
Revision Details
Purpose of issue
FINAL
Client
Highways England
Working on behalf of
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 RH
Checked SB
Approved MD
Date 22/08/2018
Internal Project No. 60547200
Scale @ A3 1:5,000
Zone SW
AmW
HE551506
AMW
GEN
SCHEME WIDE
DR
HD
0000
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Annex 1 Part B – Pluvial Hydraulic Modelling Report

highways england

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Annex 1B Pluvial Hydraulic Modelling Report

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

October 2018

Table of contents

Chapter

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 This report is intended to document the methodology and results obtained from the surface water hydraulic modelling undertaken. The reader is referred to the FRA document 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 documents the limitations associated with the work presented, and Section 6 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 rate (mm/hr). There is some scope to represent spatial variation in rainfall and losses, although importantly TUFLOW is unable to represent any further interaction

[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.

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, with a very small proportion of paved/urbanised areas. This is important, as the uniform nature of the modelled area at Parsonage Down meant 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].

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

2.2.3 ReFH2 contains a number of updates with respects 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 and 360 minutes. This was undertaken for both summer and winter storm profiles. An allowance of +40% was applied to the 1% AEP event to account for climate change.

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 180 minutes. Further testing compared the summer and winter storm events, demonstrating that the summer storm event was more critical for the Parsonage Down catchment.

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 summer 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 is characterised by a high level of permeability, 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 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 In order to replicate antecedent conditions and raised groundwater levels within ReFH2 the value of C_{ini} was raised from the default value of 50.51mm to 675mm. The value of 675mm represented approximately 50% of C_{max} which was 1341.04mm.

2.4.4 The ReFH2 model showed a high level of sensitivity to the change in the value of C_{ini}. Raising the C_{ini} value increased peak effective rainfall by an order of magnitude, and when incorporated into the hydraulic model produced unrealistically high flows and volumes within the Parsonage Down catchment. As a result the original C_{ini} value was retained for calculation of effective rainfall for input into the model.

2.5 River Till Fluvial Flow

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. Whilst it is not thought that flood depths or extents on the River Till floodplain are likely to exert significant backwater effects upon the flow pathway through Parsonage Down, the River Till will be a receptor for water flowing from this surface water pathway.

2.5.3 The work undertaken here is intended to assess flood risk arising from a short duration localised storm event, which commonly lead to surface water flood inundation. Hence it is assumed that the modelled storm event at Parsonage Down would not coincide with a large fluvial event on the River Till.

2.5.4 As a result of the above considerations it was considered appropriate to include a basic representation of the River Till within the surface water hydraulic model as a downstream boundary. A 50% AEP hydrograph was applied for the River Till, which was phased to coincide with the peak of the rainfall event, to replicate approximately bank full conditions within the River Till. This fluvial inflow hydrograph was applied within all simulations undertaken.

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_AA.

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

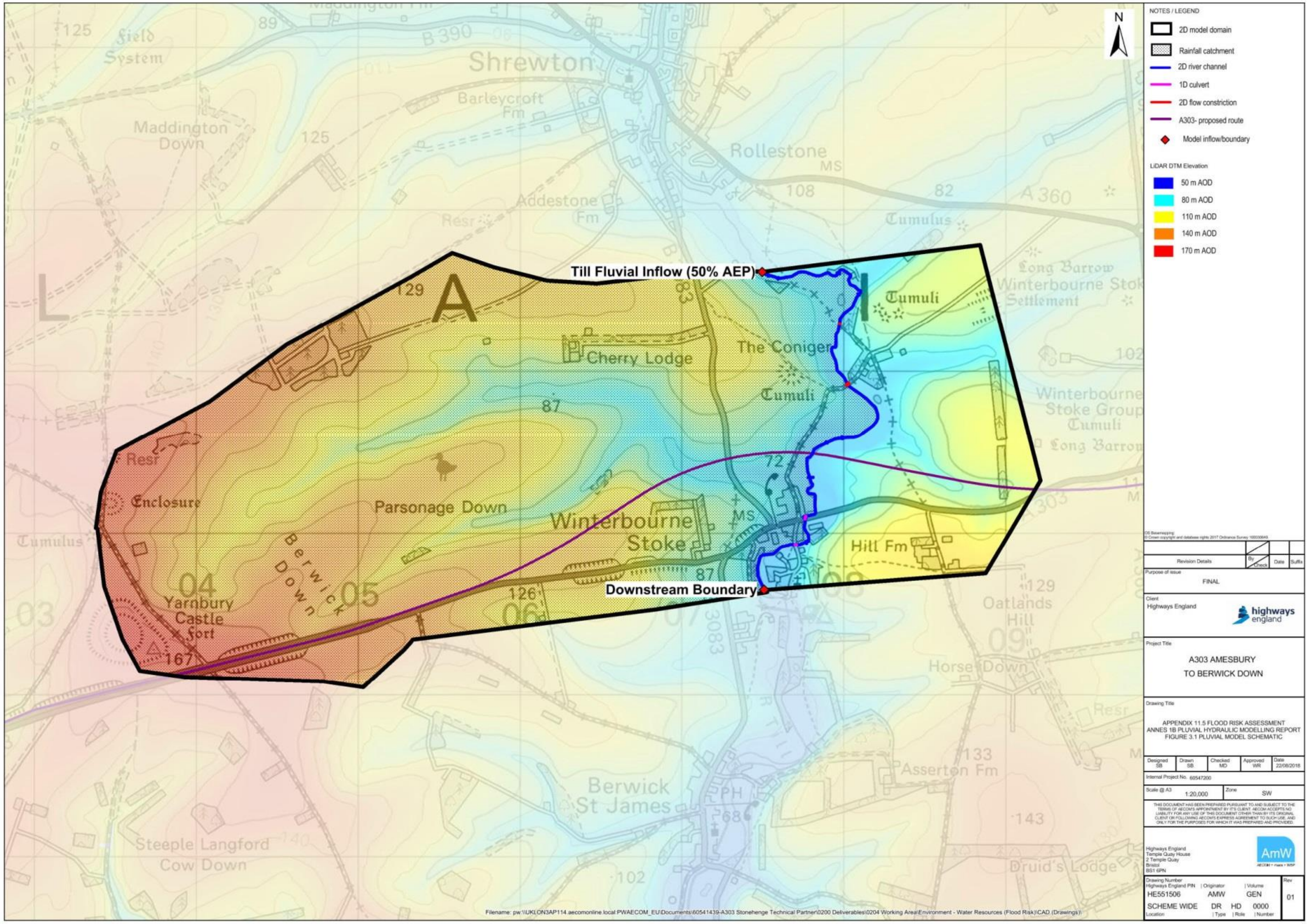

Figure 3.1 Pluvial Hydraulic Model Schematic

3.1.3 As outlined in Section 2 of this report, the primary input into the hydraulic model was effective rainfall hyetographs, whilst a small fluvial flow was represented in the model to replicated bank full conditions on the River Till. The estimation of these flows is documented within the Section 2. Section 3 of the report herein 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 can be seen 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 was defined in order to encompass the contributing rainfall catchment, whilst it also extends across the River Till valley to the east, in order to facilitate a representation of the river channel.

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. 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. 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. This was chosen as 4m resolution represented the finest resolution that could be achieved whilst retaining practical model run times. The model typically takes between 5 and 10 hours to simulate.

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 2D Model - River Till

3.4.1 The River Till channel has been represented within the surface water hydraulic model. Open channel reaches of the River Till have been represented through lowering levels within the 2D grid, whilst significant structures such as the existing A303 bridge have been included as ESTRY 1D elements. The reach of the channel represented, along with key structures are shown on the schematic in Figure 3.1.

3.4.2 Open channel sections of the River Till have been represented through leveraging cross sectional survey information utilised within the fluvial hydraulic model. Cells within the model domain which fall within the River Till channel have been lowered to the channel bed level extracted from the survey, whilst cells adjacent to the channel have been raised to surveyed bank top levels. Given that the River Till channel is primarily a receptor for surface water, this was considered an appropriate estimate of the geometry of the River Till.

3.4.3 Key structures along the reach of the River Till, particularly those which were likely to impact significantly upon flow conveyance, were included within the model as 1D ESTRY components. The channel structural survey information was used in order to accurately represent the geometry of these structures. Locations of these structures are shown within Figure 3.1.

3.4.4 Several less significant structures along the River Till, such as small bridges located upstream of Winterbourne Stoke, were included within the 2D model through the use of flow constrictions. Flow constrictions facilitate an approximate representation of attenuation associated with structures through blockage/partial blockage of flow through cells within the 2D model domain. Locations of these flow constrictions are shown within Figure 3.1.

3.4.5 The model was initially simulated in order to extract an initial water level for the River Till channel, this was applied so that the river channel contained a nominal flow at the start of subsequent design simulations.

3.5 Manning's Roughness Coefficient ('n')

3.5.1 Spatial variations of 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.

3.5.2 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].

3.5.3 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.

3.5.4 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 less than 0.1 m and the standard value of 0.06 was applied when flow depths

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

exceed this threshold. Depth varying roughness values were adapted from TUFLOW guidance[5]

3.5.5 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.

3.5.6 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

3.6 Downstream Boundary

3.6.1 The downstream model boundary was located approximately 1 km downstream of Winterbourne Stoke, and comprised a depth flow (HQ) relationship defined based upon the slope of the floodplain at this location.

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

3.7 Model Timestep and Simulation Duration

3.7.1 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.

3.8 Proposed Scheme Scenario

3.8.1 The proposed scheme design was initially incorporated into the hydraulic model. For additional information relating to the proposed scheme, the reader is referred to the main FRA document.

3.8.2 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.

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

3.8.4 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.

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

3.8.5 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.

3.8.6 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.

3.8.7 A surface water 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. The solution comprised a 750mm pipe, the inlet of which was located immediately to the north west of the underpass of the B3083 beneath the A303. An impounding bund was included to prevent flow of surface water though the underpass. The outfall of the pipe was located m to the east, to the north of the new course of the A303 carriageway. This solution was added into the model as a 1D ESTRY culvert. 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)

3.8.8 The schematisation of the current route of the A303 was retained as per the baseline model.

3.9 Filtering of Model Results

3.9.1 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.

3.9.2 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 cutting.

3.9.3 For the presentation of surface water modelling results within this report, filtering was undertaken in order to remove the erroneously high depths outlined within Section 3.9.2 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.

4 Surface Water Hydraulic Modelling Results

4.1 Baseline Scenario Results

4.1.1 Figure 4.1 shows a comparison of the 1% AEP surface water flood extent output from the Parsonage Down model and the EA low surface water flood extent, which corresponds to the equivalent AEP event

Figure 4.1 Surface Water Flood Extent Comparison- 1% AEP

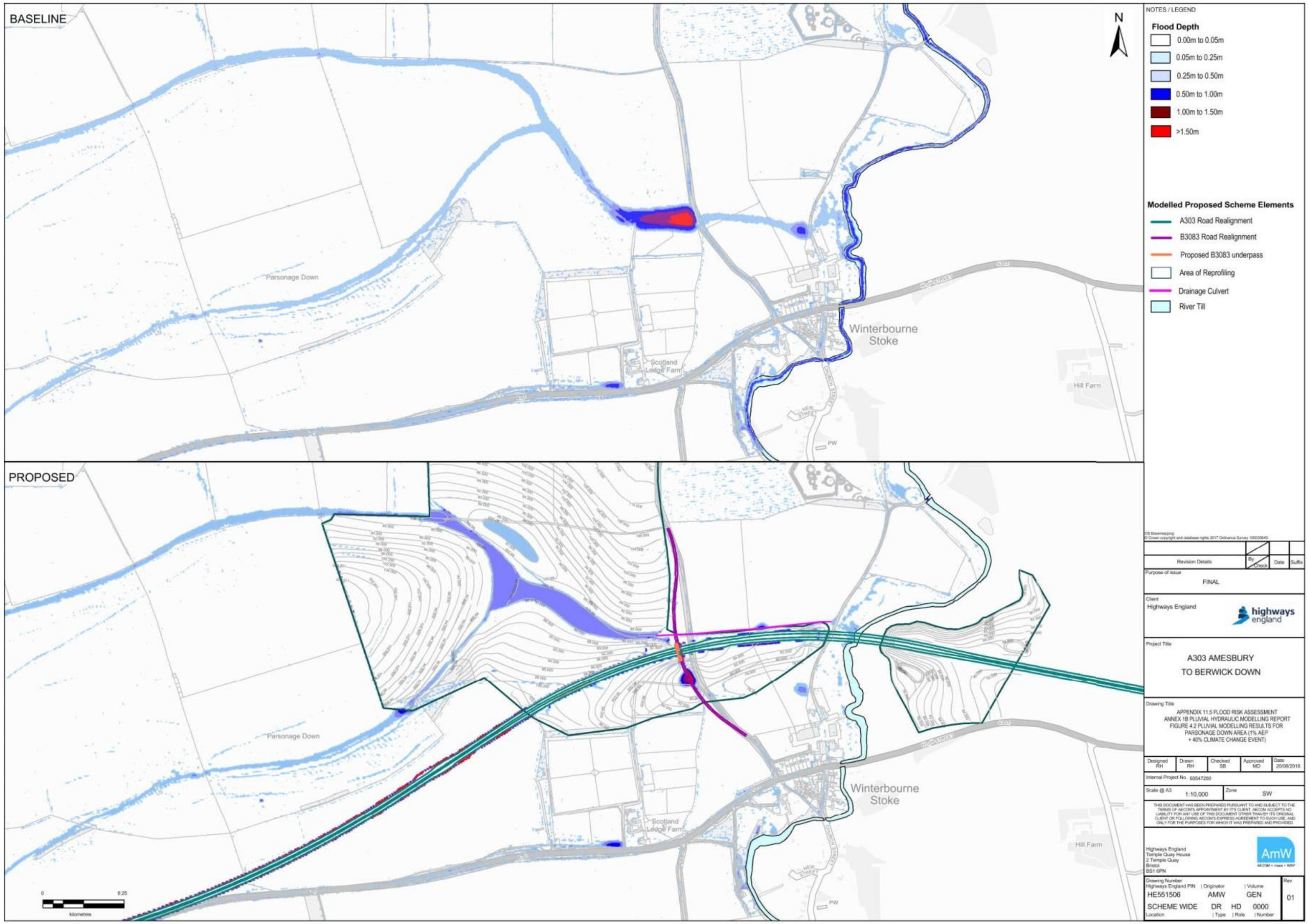


Figure 4.2 Maximum Flood Depth Comparison- 1% AEP Plus Climate Change (40%)

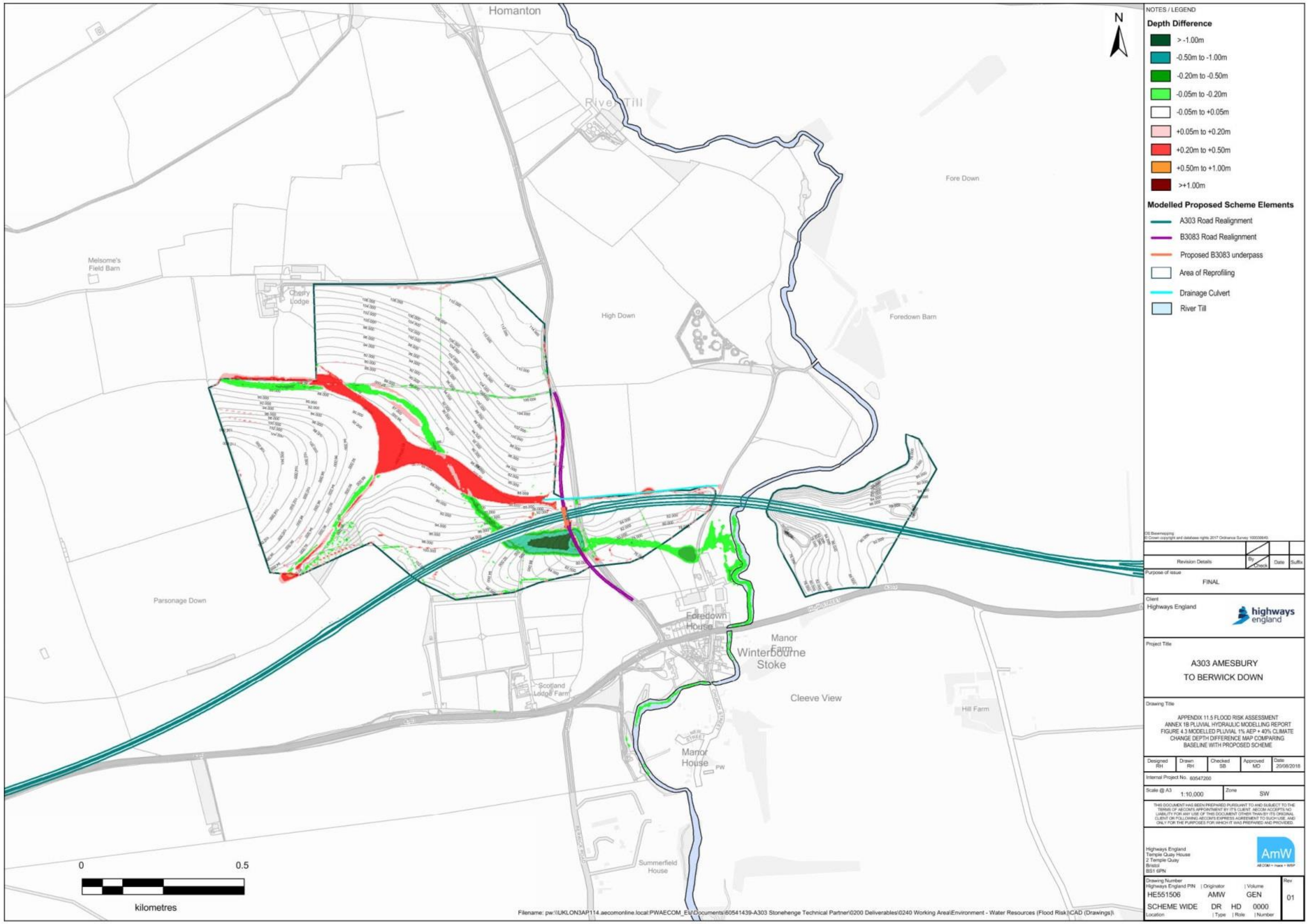


Figure 4.2 Maximum Flood Depth Difference Plot- 1% AEP Plus Climate Change (40%)

4.1.2 Figure 4.1 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.

4.1.3 Within the EA FMfSW water flows over the top of the road and down to the bottom of the River Till valley, whilst in the modelled 1% results this does not occur. This suggests that the modelled flow through the Parsonage Down catchment is lower within the localised 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.

4.1.4 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. Within the modelled results there is very little water out of bank and the extent is smaller than the EA FMfSW (Figure 4.1).

4.1.5 The difference described in 4.1.3 is to be expected as the localised Parsonage Down model includes a fluvial inflow for a 50% AEP event, whilst within the EA surface water map shows an extent which effectively incorporates a 1% AEP event across the whole River Till catchment. 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.

4.2 Proposed Scenario Results

4.2.1 Figure 4.2 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. Whilst Figure 4.3 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.

4.2.2 It is clear that 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 a drainage pipe which collects surface water to the north east of the intersection of the A303 and B3083, and passes water approximately 500m to the east before outfalling to the River Till, north of the proposed route of the A303.

4.2.3 Modelling results demonstrate that the combination of elements of the proposed scheme lead to a widening and deepening of the surface water flow pathway through Parsonage Down to the north of the A303. Depths within the surface water flow pathway are typically between 0.2m and 0.5m. Furthermore, within the proposed scenario surface water sourced from the Parsonage Down catchment outfalls to the River Till further to the north than in the baseline scenario. Based upon the hydraulic modelling results presented, changes in the surface water flow paths are expected and have been designed as such to not lead to an increase in flood risk to any vulnerable receptors.

4.2.4 The maximum depth difference plot presented within Figure 4.3 provides a further reflection of the change in the location and characteristics of the surface water flow pathway as described above. Significantly, the map demonstrates that there is a reduction of between 0.05m and 0.2m in flood depth within the River Till and neighbouring floodplain close to Winterbourne Stoke. This demonstrates that the proposed scenario has an overall attenuating affect upon surface water flows through Parsonage Down, reducing the amount of water supplied to the River Till during an intense rainfall event.

4.2.5 Equivalent comparison maps and depth difference plots for the 3.33% AEP, 1% AEP and 0.1% AEP events are included within Appendix A. Importantly, for all modelled return periods the proposed scheme has an overall attenuating impact upon surface water flows reaching the River Till. For the 3.33% AEP event there is no change in maximum flood depth within the River Till valley between the A303 viaduct and Winterbourne Stoke. For all other modelled events reductions in maximum flood depth of 0.05m-0.20m are observed within this area.

5 Limitations

5.1 Rainfall Hydrology and Losses

5.1.1 There is currently no defined best practice guidance for the estimation of rainfall hydrology and losses, and subsequent inclusion within surface water hydraulic modelling. Therefore it is thought that the approach developed and adopted represents an appropriately conservative representation of surface water flood risk within the catchment, based upon both hydrological and engineering judgement.

5.1.2 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].

5.1.3 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. 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.

5.1.4 The Parsonage Down catchment is ungauged and thus there is no quantitative historic data available for calibration or verification of the hydrological model.

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

5.2 Hydraulic Model

5.2.1 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.

5.2.2 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.

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

5.2.3 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.

5.2.4 The model features a hydraulic representation of the River Till. The River Till is relatively narrow (< 8m width) with respect to the model grid resolution (4m), hence in some parts of the reach it is possible that conveyance of flow may be sub optimal. However, whilst associated with a lower accuracy than a full 1D channel representation, this approach is considered acceptable within the context of this study and is unlikely to impact upon conclusions.

5.2.5 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.

5.3 Combined Limitations

5.3.1 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.

5.3.2 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 for groundwater interactions.

6 Summary and Conclusions

6.1 Summary

6.1.1 Surface water flood risk through the Parsonage Down catchment has been modelled using TUFLOW for the baseline and proposed permanent scenario.

6.1.2 Rainfall hydrology has been calculated using ReFH2, producing effective rainfall profiles which have been included within the hydraulic model.

6.1.3 Design rainfall events for the 3.33% AEP, 1% AEP, 1% AEP +40% allowance for climate change, and 0.1% AEP have been modelled. Summer rainfall profiles with a critical duration of 180 minutes were simulated as this was shown to be the critical storm duration for the catchment.

6.2 Conclusions

6.2.1 Overall several key elements of the proposed permanent A303 scheme design, specifically the realignment of the A303, B3083 and land reprofiling, have an impact upon the location and conveyance of surface water flows through Parsonage Down.

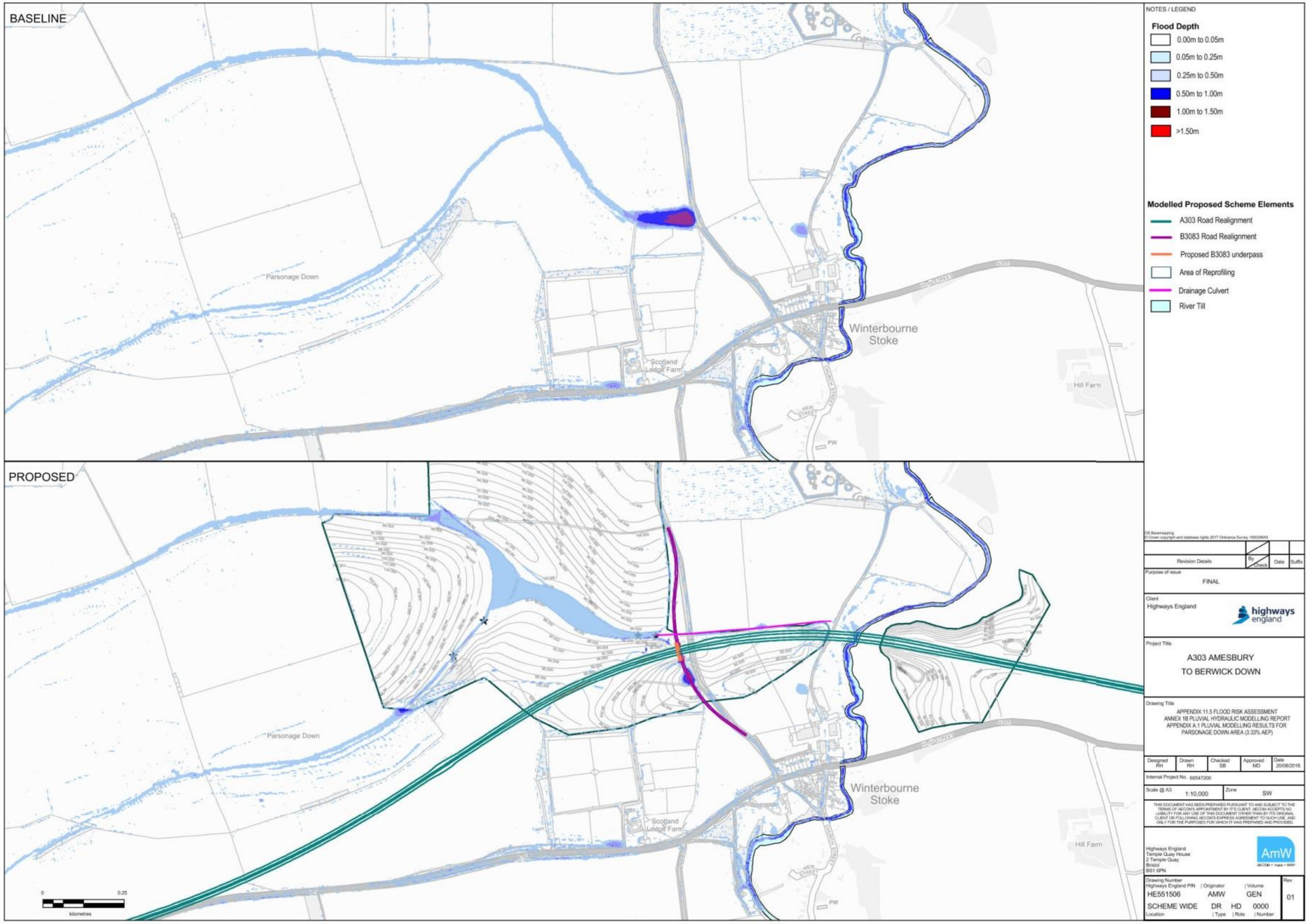
6.2.2 Inclusion of a pipe maintains connectivity between the surface water flow pathway and the River Till floodplain, with the pipe outfall discharging water north of the proposed A303 route.

6.2.3 Overall, the proposed scheme leads to a slight change in the location where surface water is routed, and leads to a widening and deepening of the flow path through Parsonage Down. Within the proposed scenario, surface water joins the River Till north of its existing flow pathway as expected.

6.2.4 Overall, the proposed scheme has an attenuating impact upon surface water flow into the River Till from Parsonage Down, reflected in the decreases in flood depth shown within the River Till valley.

6.2.5 Changes in surface water flow pathways shown within the modelling undertaken do not indicate an increase in surface water flood risk to any vulnerable receptors within the Parsonage Down or neighbouring areas.

Appendix A- Additional Flood Mapping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Winterbourne Stoke
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kilometres
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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 A.1 PLUVIAL MODELLING RESULTS FOR
PARSONAGE DOWN AREA (3.33% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
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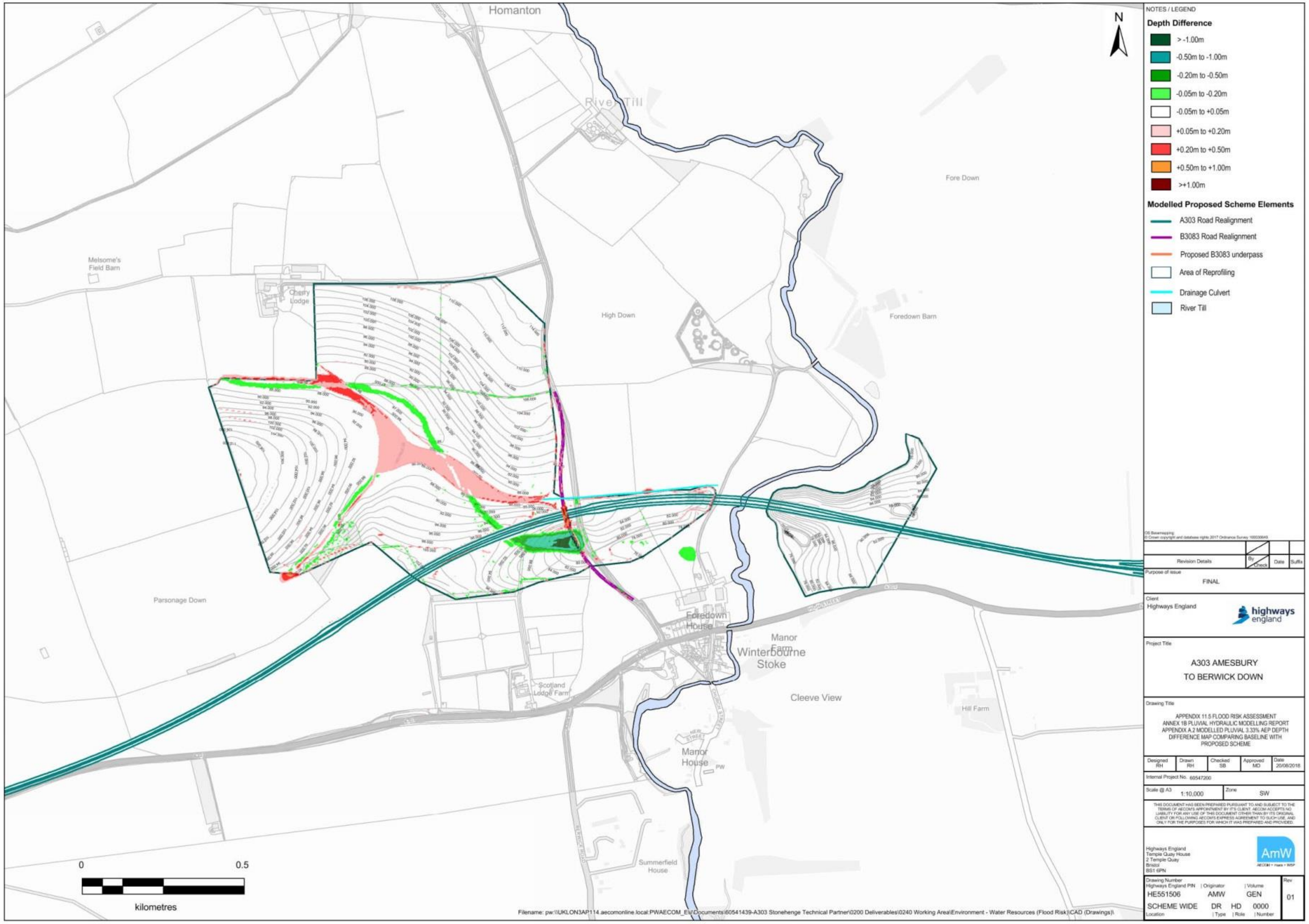
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B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Drainage Culvert
River Till
N
Homanton
River Till
Fore Down
Melsome's Field Barn
High Down
Foredown Barn
Parsonage Down
Foredown House
Manor Farm
Winterbourne Stoke
Cleeve View
Scotland Lodge Farm
Hill Farm
Manor House
Summerfield House
0
0.5
kilometres
Purpose of issue
FINAL
Client
Highways England
Project Title
A303 AMESBURY
TO BERWICK DOWN
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DIFFERENCE MAP COMPARING BASELINE WITH
PROPOSED SCHEME
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
Internal Project No. 60547200
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Zone SW
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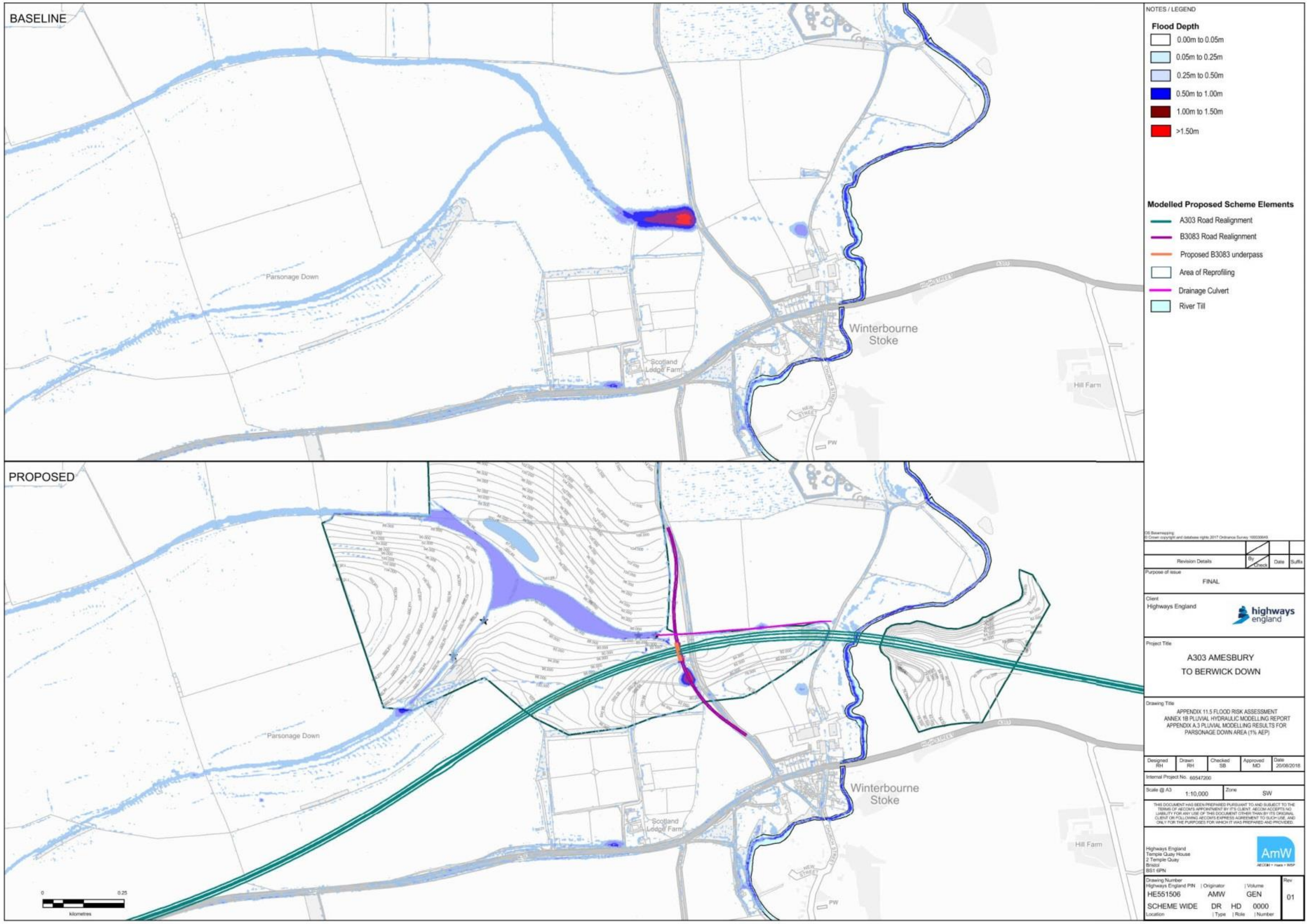
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0.50m to 1.00m
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A303 Road Realignment
B3083 Road Realignment
Proposed B3083 underpass
Area of Reprofiling
Drainage Culvert
River Till
0
0.25
kilometres
Revision Details
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A303 AMESBURY
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APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 1B PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.3 PLUVIAL MODELLING RESULTS FOR
PARSONAGE DOWN AREA (1% AEP)
Designed RH
Drawn RH
Checked SB
Approved MD
Date 20/08/2018
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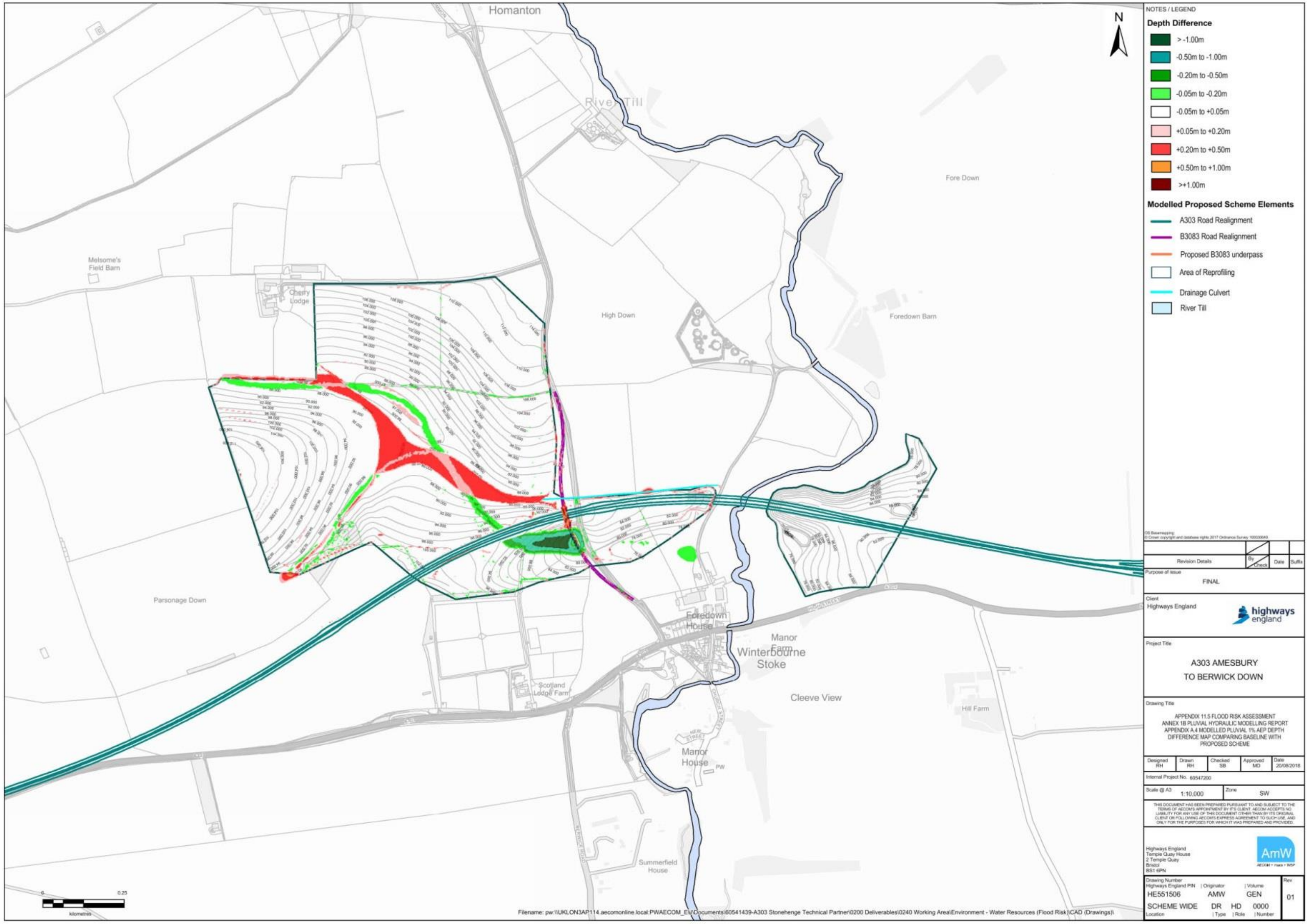
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High Down
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Suffix
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APPENDIX 11.5 FLOOD RISK ASSESSMENT
ANNEX 18 PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.4 MODELLED PLUVIAL 1% AEP DEPTH
DIFFERENCE MAP COMPARING BASELINE WITH
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Drawn RH
Checked SB
Approved MD
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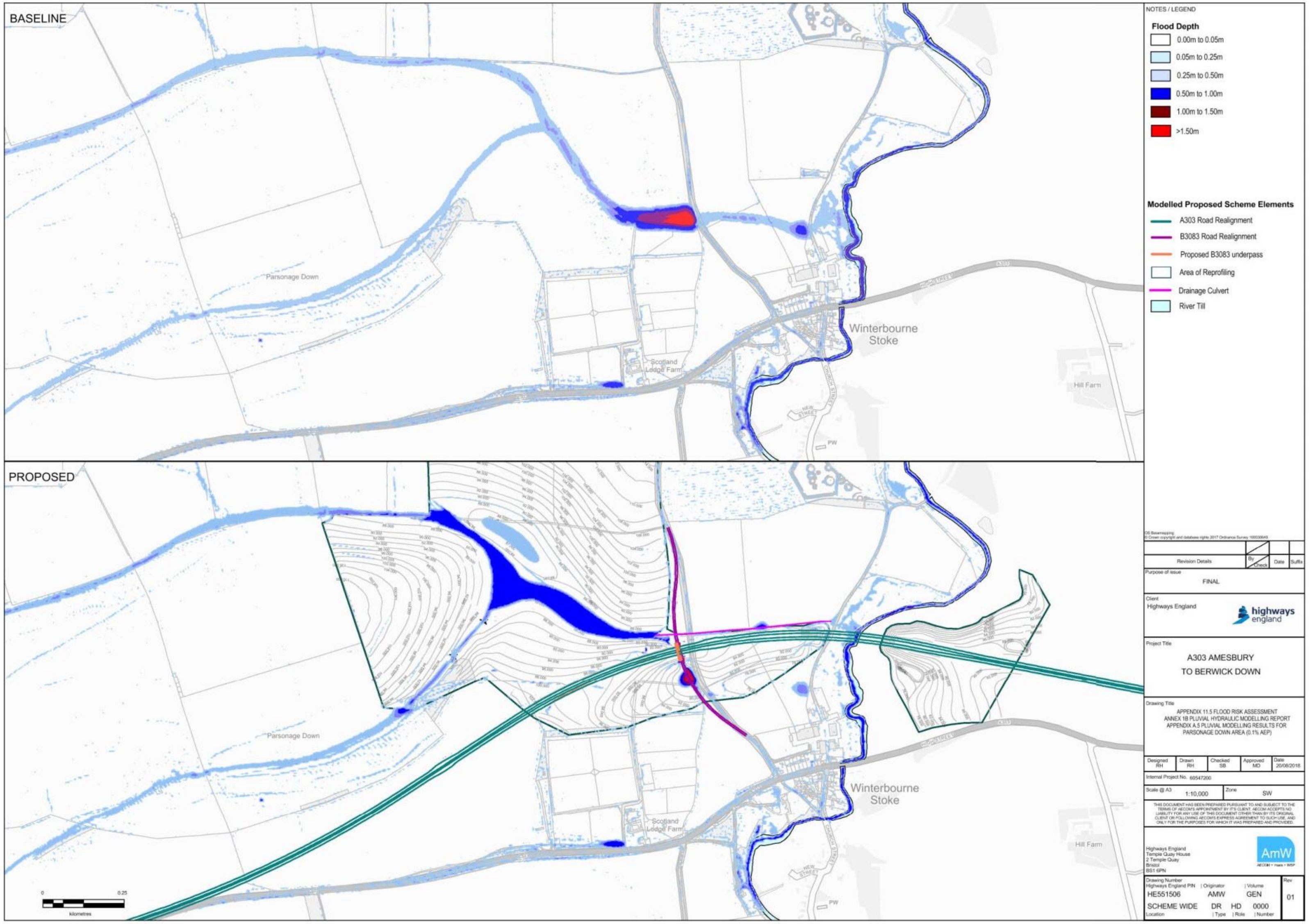
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APPENDIX A.5 PLUVIAL MODELLING RESULTS FOR PARSONAGE DOWN AREA (0.1% AEP)
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Approved MD
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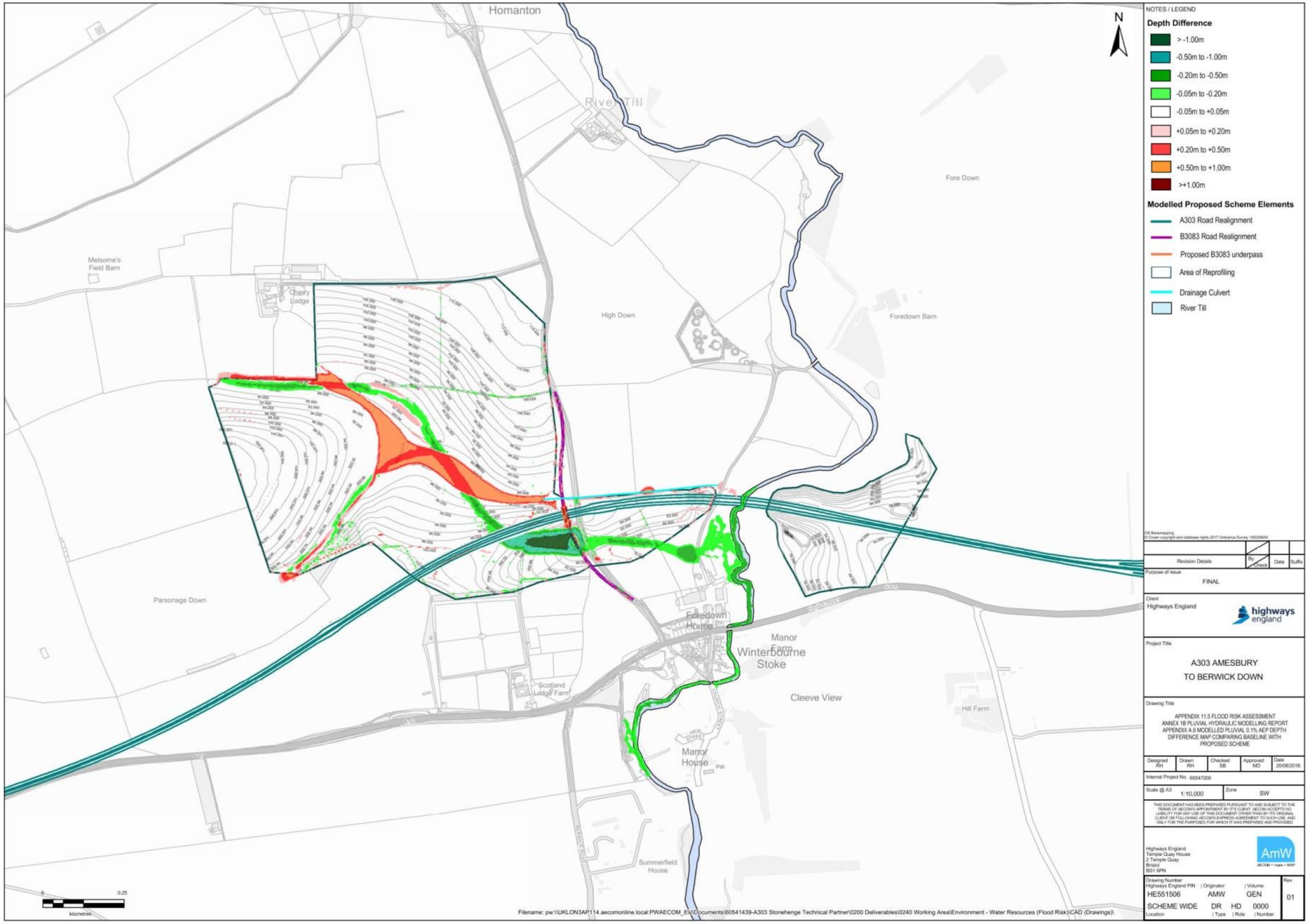
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ANNEX 18 PLUVIAL HYDRAULIC MODELLING REPORT
APPENDIX A.6 MODELLED PLUVIAL 0.1% AEP DEPTH
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PROPOSED SCHEME
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Annex 2 Part A – River Till Hydrological Analysis

highways england

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Annex 2A River Till Hydrological Analysis

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

October 2018

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Chapter

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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 km2 at the upstream boundary of the hydraulic model and 124 km2 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

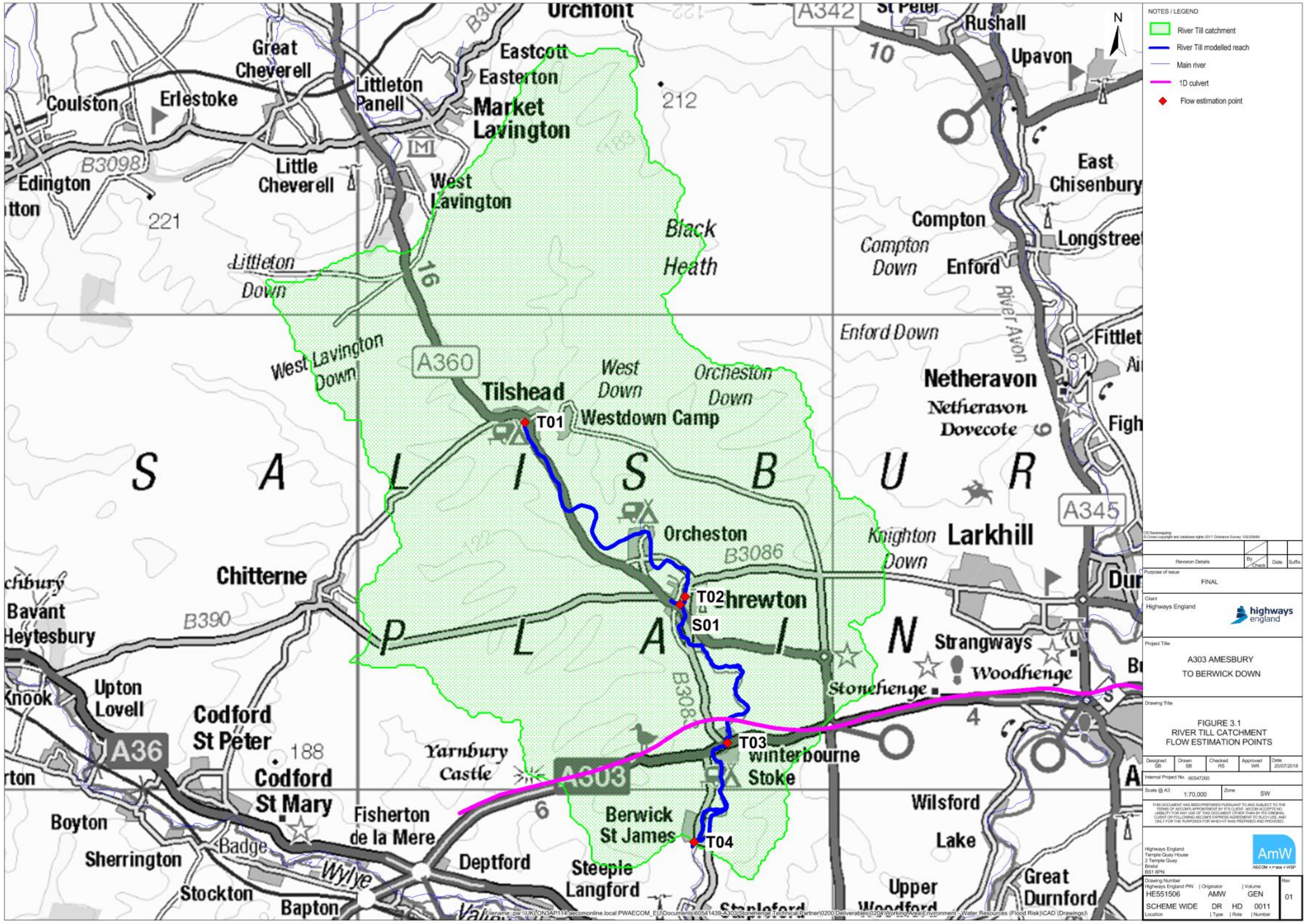

Figure 3-1: Flow estimation points

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	$QMED_{CDs}$ ($m^3 s^{-1}$) (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 Factorial 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

4.6.3 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).

4.6.4 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.

4.6.5 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	Sites Investigated 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 FSSR16 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.

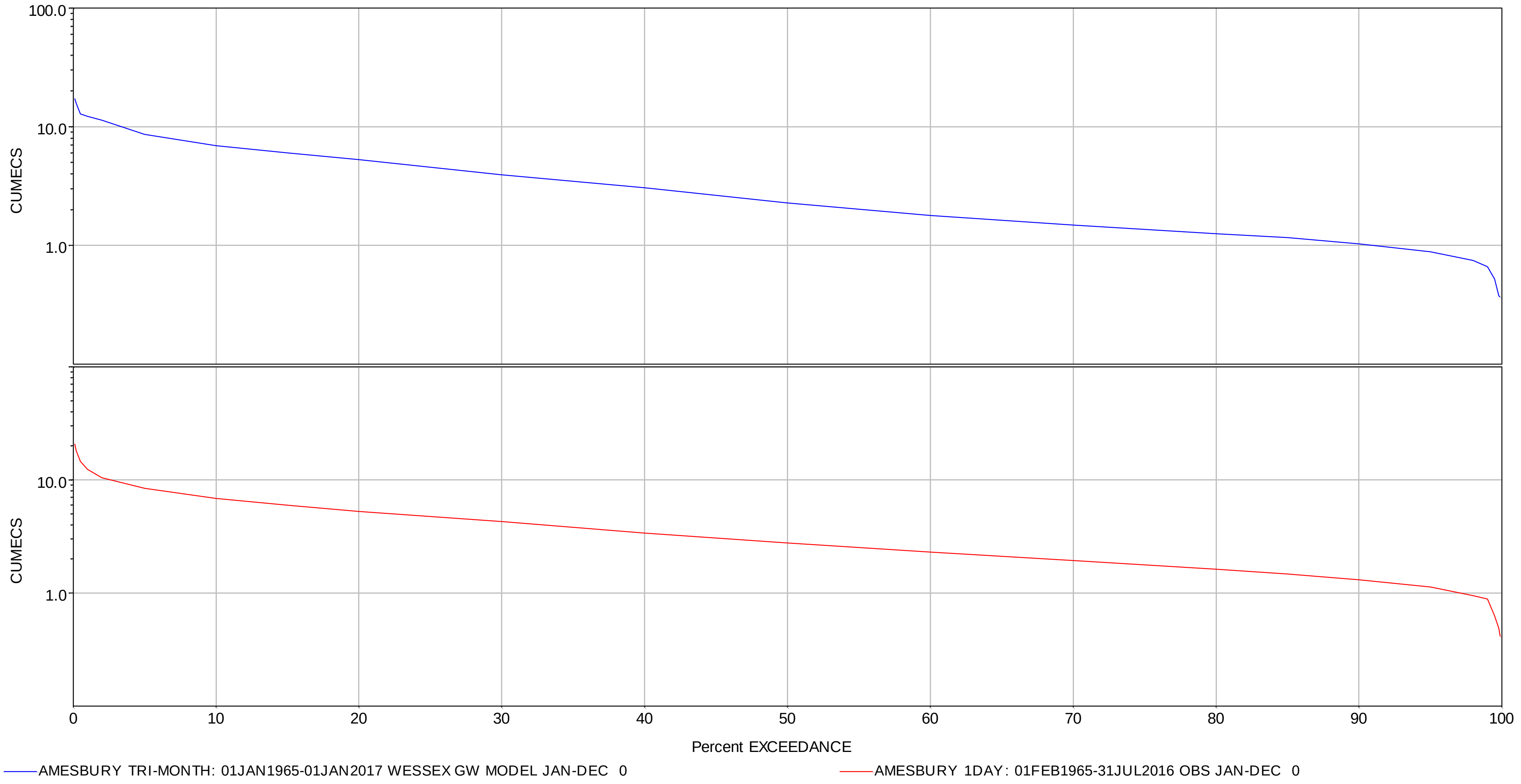

Figure C.1: Comparison of flow duration curves from Wessex Groundwater Model (upper graph) and Daily Mean Flows at Amesbury (lower graph)

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%

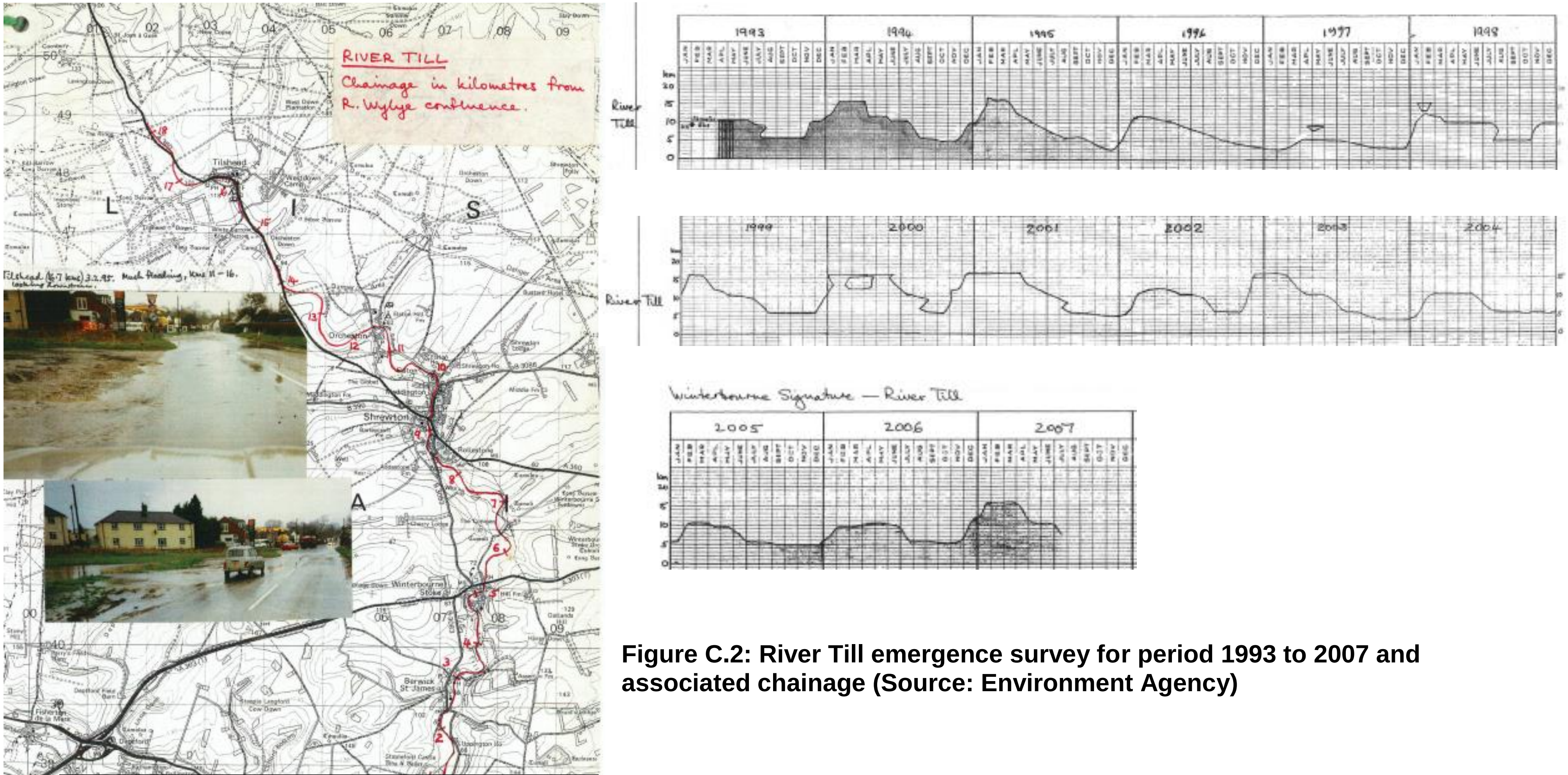

Figure C.2: River Till emergence survey for period 1993 to 2007 and associated chainage (Source: Environment Agency)

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.

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Annex 2 Part B – River Avon Hydrological Analysis

A303 Amesbury to Berwick Down

TR010025

6.3 Environmental Statement Appendices

Appendix 11.5 Annex 2B River Avon Hydrological Analysis

Volume 6

APFP Regulation 5(2)(a)

Planning Act 2008

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

October 2018

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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 km2 at the upstream boundary of the hydraulic model and 366 km2 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

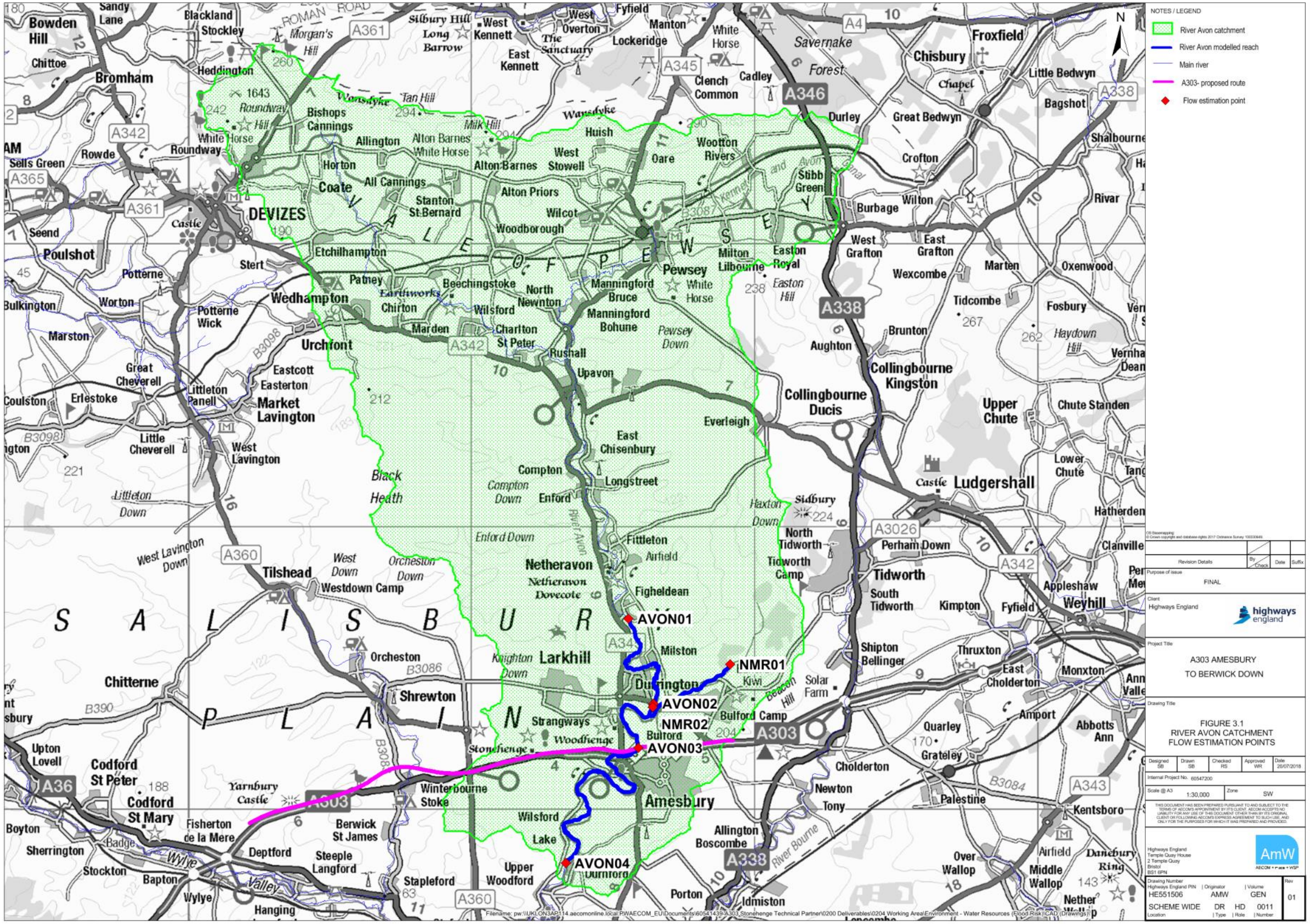

Figure 3-1: Flow estimation points

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
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 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 km2 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.

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.

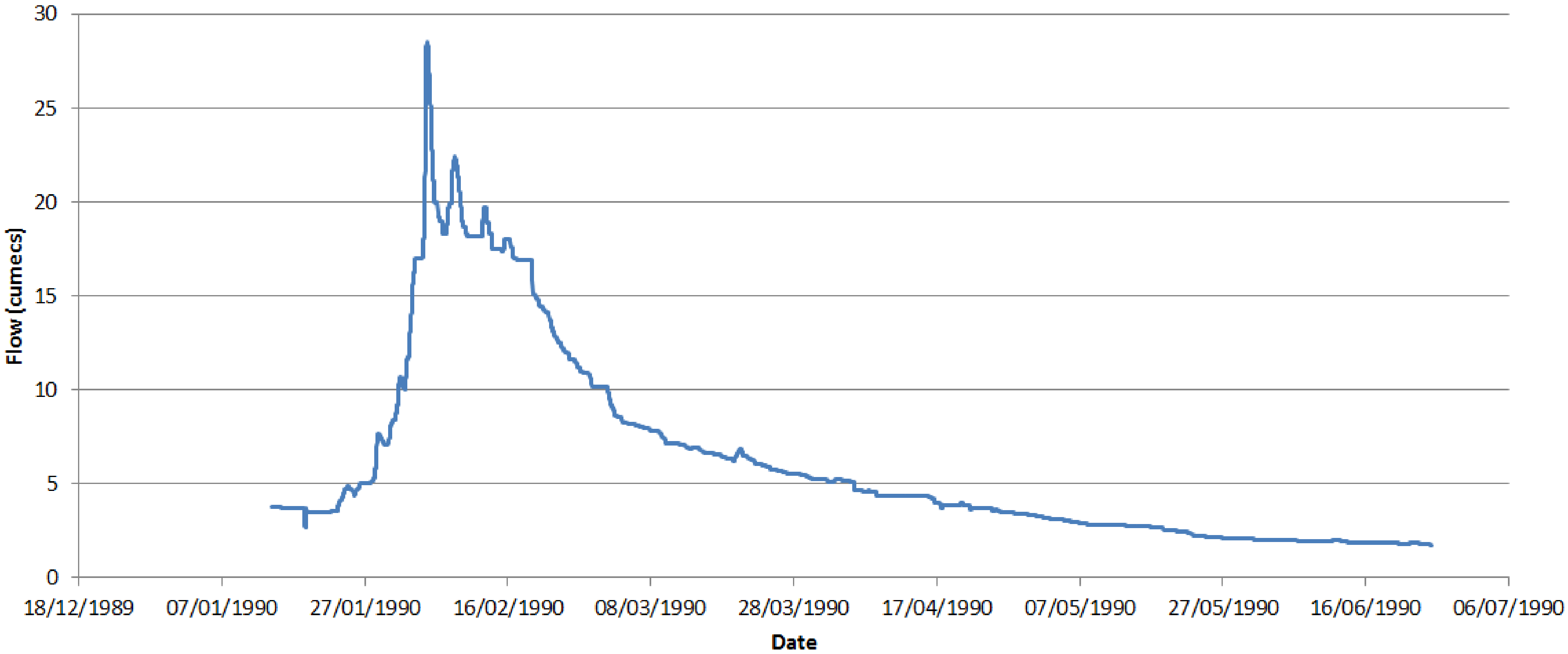

Figure 5-1: Hydrograph from the Avon @ Amesbury illustrating significant duration of rising and falling limbs.

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.

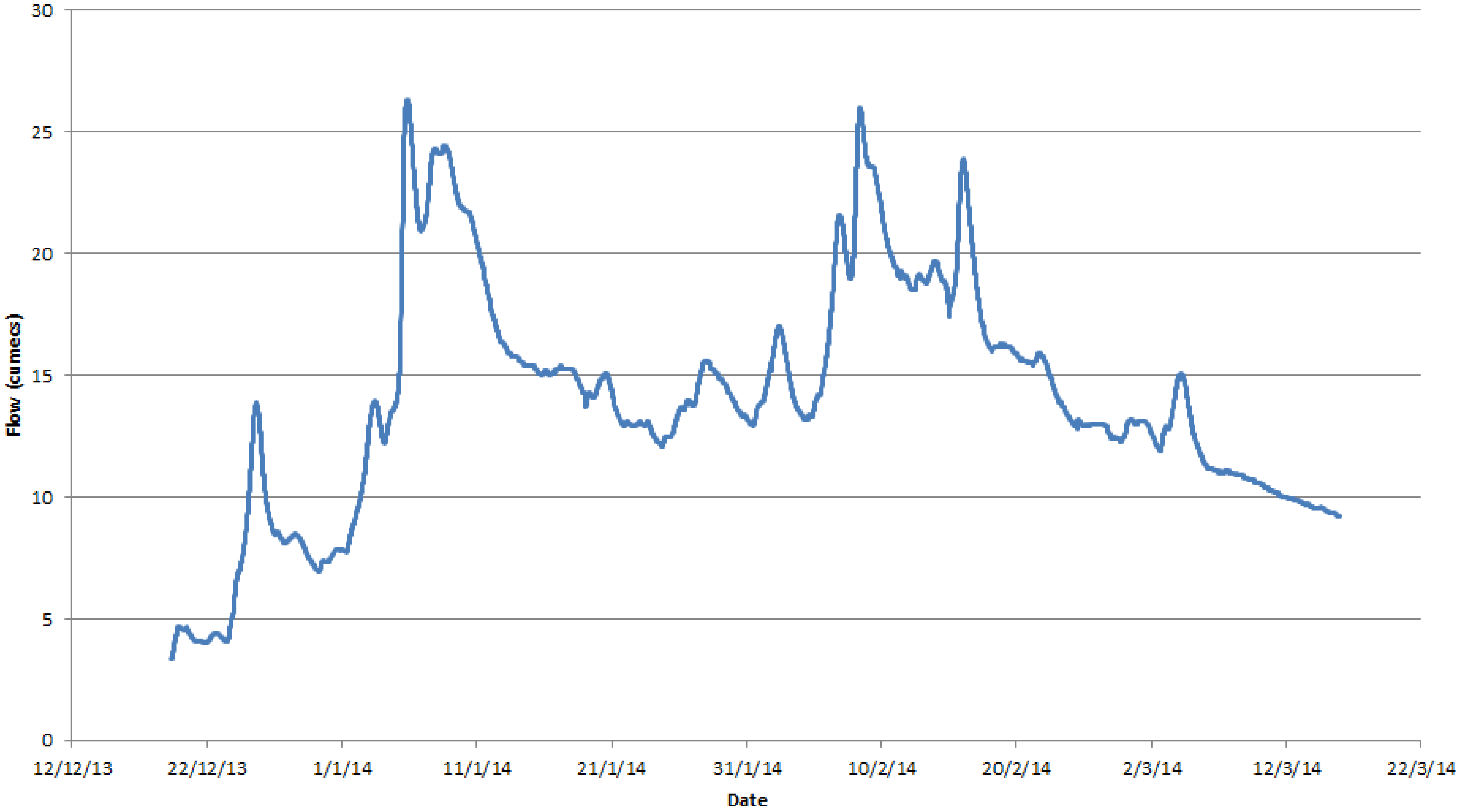

Figure 5-2: Hydrograph from the Avon @ Amesbury illustrating influence of underlying baseflow coupled with low intensity rainfall events.

5.2 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

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).

6.3 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.

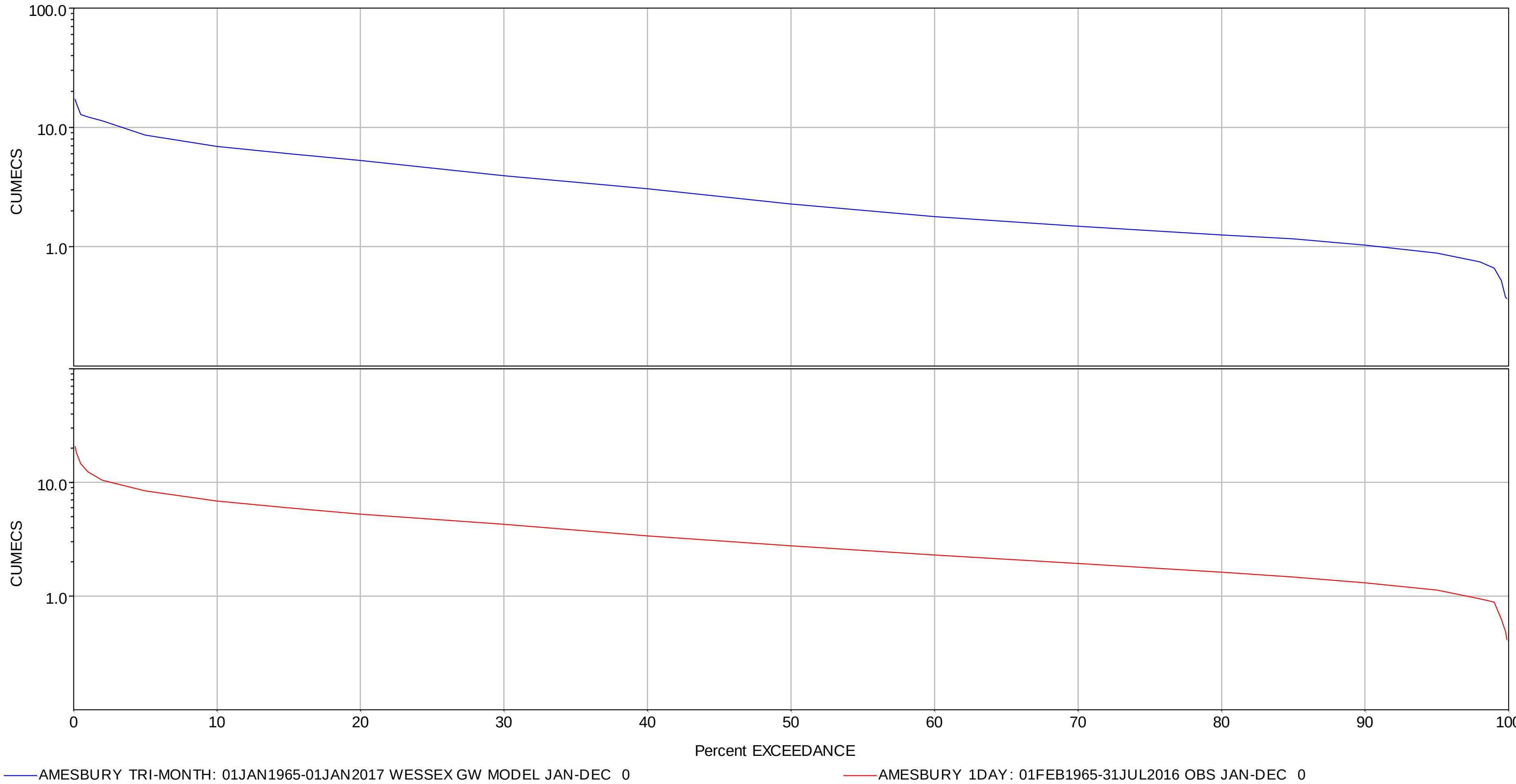

Figure C.1: Comparison of flow duration curves from Wessex Groundwater Model (upper graph) and Daily Mean Flows at Amesbury (lower graph)

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%

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.

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