Appendix B4: Flood Risk Assessment -Proposed Ribble Crossing

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Haweswater Aqueduct Resilience Programme - Proposed Bowland Section

Supplementary Environmental Information

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February 2022

Water for the North West

Proposed Bowland Section Supplementary Environmental Information Appendix B4: Flood Risk Assessment - Proposed Ribble Crossing
United Utilities Water Ltd

Haweswater Aqueduct Resilience Programme - Proposed Bowland Section

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Contents

1. Introduction

1.1 Purpose

1) This Flood Risk Assessment (FRA) has been prepared to support the planning application for the Proposed Marl Hill and Bowland Sections[1], which form part of the wider Haweswater Aqueduct Resilience Programme (HARP). The assessment of flood risk has been carried out in combination with design development during the Environmental Impact Assessment (EIA) process and informs Chapter 8 (Flood Risk) of the June 2021 Environmental Statement.

2) This FRA has been carried out with consideration of the National Planning Policy Framework (NPPF)2 and the Planning Practice Guidance (PPG).[3] Complying with planning policy would promote a development that would be appropriate given the level of local flood risks, would be safe during the construction and operational phases of its lifetime, and would not increase flood risk both on site and elsewhere.

3) This report presents an update to the FRA report submitted in the June 2021 Environmental Statement. This report describes the results of the detailed hydraulic modelling work undertaken of the River Ribble and its tributaries replacing and superseding the qualitative assessment of fluvial flood risk which was presented in the June 2021 Environmental Statement.

1.2 HARP Overview

4) The existing 110 km Haweswater Aqueduct takes raw water from Haweswater Reservoir in the Lake District National Park along a 16 km section of the aqueduct to a Water Treatment Works (WTW) near Kendal. From the WTW the aqueduct conveys treated water to customers in Greater Manchester, Cumbria and Lancashire through service reservoirs and water mains which branch off the main aqueduct.

5) The aqueduct comprises five unpressurised single line tunnels and conduit sections (generally 2.6 m internal diameter) in addition to multi-line sections. The flow of water along the entire length of the aqueduct is achieved by gravity, with no energy-consuming pumps involved in supplying the water from north to south. Out of the total 110 km length of the aqueduct, the Proposed Programme of Works on the single line sections accounts for just under half this distance, about 53 km.

6) The Proposed Programme of Works is to replace part of an ageing strategic asset to secure a major water supply serving over two million people, and to mitigate potential risks to drinking water quality. The proposed solution is to provide a full replacement of the five single line tunnel sections. To facilitate the construction works along the tunnel sections, a series of highways improvements are proposed of which the Proposed Ribble Crossing is one.

1.3 Development Proposals

7) The proposed Ribble Crossing forms part of the enabling works of the Proposed Marl Hill and Bowland Sections. It would facilitate construction traffic movements to the Newton-in-Bowland, Braddup and Bonstone compounds.

8) The crossing would be temporary and would be in place for the duration of the construction of the Proposed Marl Hill and Bowland Sections. Enabling works on the Proposed Ribble Crossing would start in 2023 and would be followed by a main construction phase which would last for approximately one year. The Proposed Ribble Crossing would then be operational for approximately seven years until the

[1] The proposed Ribble Crossing would serve both the Proposed Marl Hill Section, and the south compound – Newton-in-Bowland – of the Proposed Bowland Section. While this report has been prepared as part of the January 2022 supplementary environmental information for the Proposed Marl Hill Section, the Proposed Bowland Section is referred to as the same flood risk assessment applies to both planning applications.

[2] Department for Communities and Local Government (2019) National Planning Policy Framework. [Online] Available at: https://www.gov.uk/government/publications/national-planning-policy-framework--2. [Accessed: 09 March 2021].

[3] Department for Communities and Local Governments (2019) Planning Practice Guidance. [Online] Available at: https://www.gov.uk/government/collections/planning-practice-guidance. [Accessed: 09 March 2021].

end of the tunnel construction works. Following completion of the construction works on the Proposed Marl Hill and Bowland Sections, the Proposed Ribble Crossing would be decommissioned.

9) The Proposed Ribble Crossing would be located approximately 2 km north of Clitheroe town centre and adjacent to West Bradford (to the south and west of the village). Furthermore, the proposed development is located to the west of the Hanson Cement factory (buildings and a quarry), off Clitheroe Road and West Bradford Road. The National Grid Reference (NGR) for the Proposed Ribble Crossing is SD 74404 43875 as indicated on Figure 1.

10) The Local Planning Authority is Ribble Valley Borough Council and the Local Lead Flood Authority is Lancashire County Council.

11) The outline design for the Proposed Ribble Crossing would combine the following key elements:

- Enabling works including the establishment of temporary construction compounds and temporary access roads
- Temporary drainage around the compound locations
- Construction of a temporary three-span bridge across the River Ribble
- Construction of three, temporary single span bridges across the Ordinary Watercourses; Coplow Brook, Greg Sike and an unnamed watercourse referred to as Unnamed Watercourse 2097
- Construction of a temporary tarmac road approximately 7.7 m wide with associated drainage between a junction with West Bradford Road to the south of the River Ribble at SD 74517 43833 and another junction also with West Bradford Road to the north of the River Ribble at SD 73344 44007.

12) Drawings showing the indicative design of the Proposed Ribble Crossing are presented in Annexe B.

13) The temporary laydown compounds would be used for less than one year and would then be reinstated back to agricultural land for the operational phase of the Proposed Ribble Crossing. The Proposed Ribble Crossing itself including the road with the bridge and crossings over Ordinary Watercourses would have a design life of approximately seven years after which it would be decommissioned and the land re-instated to its previous condition. The decommissioning works would require the re-establishment of the compounds that were re-instated following the completion of the construction phase. Following the completion of the decommissioning, the compounds would once again be re-instated to their pre-development condition.

14) The road would be a two-lane carriageway approximately 1.45 km in length. During the construction works at the main compounds the road would be reserved for the use of all construction traffic. Public access to the road would be prohibited through the provision of vehicle barriers at either end of the road.

15) The road would be suitable for heavy duty use and would be surfaced with a tarmac construction based on a stone aggregate foundation.

16) A temporary bridge crossing of the River Ribble would be incorporated in the road. The bridge would be a Bailey bridge type clear span construction supported on columns either side of the river and 72 m in length. The bridge would extend over the adjacent floodplain with additional bridge sections either side of the river bridge. Overall, the bridge would be approximately 140 m in length (see drawing 80061155-01-JAC-TR3-97-DR-C-00008). Earthwork abutments would be required either side of the bridge.

17) The Proposed Ribble Crossing has been designed to include the following embedded mitigation measures:

- A three-span structure that crosses the floodplain of the River Ribble on slender piers that would be orientated parallel to the direction of flood flows and rounded at the ends
- Bridge soffit levels over the Ribble to be set 600 mm above the 1% AEP peak flood level
- Clear span crossings of Ordinary Watercourses with no use of culverts or in channel structures

- The road surface constructed at grade across areas of floodplain except for a slight camber and where Ordinary Watercourse crossings are required
- Drainage systems based on Sustainable Drainage System (SuDS) to manage runoff from construction compounds and roads at a rate not exceeding greenfield run off
- All topsoil stripped as part of the construction would be stored within Flood Zone 1 at upper end of the Coplow Brook to provide visual screening to the local school from the site compound within this area.

18) The route of the road has been selected to keep to the periphery of field boundaries where possible. Gated crossing points would be provided for landowners and tenants to enable access to land that the road crosses.

19) The road crosses several public rights of way (PROWs) including the Ribble Way. The temporary bridge across the River Ribble would cross over the Ribble Way with sufficient clearance to avoid any disruption to access apart from during the bridge construction which may require a temporary diversion. Gated crossing points would be provided to ensure continuity of access for any other affected public rights of way.

2. Methodology & Scope

2.1 Assessing Flood Risk

20) The assessment of flood risk has been undertaken in line with the development of the EIA and the Proposed Ribble Crossing design. A summary of the main HARP FRA methodology aligned with this reports' methodology is presented in the following sections along with key datasets, assumptions and limitations.

2.1.1 Source-pathway-receptor

21) Flood risk is conceptualised using the source-pathway-receptor model. For a flood risk to be present each of the three elements is required:

- A **source** of flood water such as a river or groundwater body
- A **pathway** that enables the flow of flood water from a 'source' to a 'receptor'. This could include low lying land within a floodplain or permeable strata that enable groundwater to seep to the surface, or construction activities such a tunnelling
- A **receptor** such as a person, property or habitat that may be impacted by a flood event.

22) Flood risk is therefore dependent on all elements being present and is assessed in terms of the probability (likelihood) of an event occurring and the consequence of the flood.

2.1.2 Probability

23) The probability of flooding in this report is defined using Annual Exceedance Probability (AEP). This is the preferred approach in comparison to the annual maximum return period (e.g. 1 in 100-year event). This is due to the potential misconception that return periods are associated with a regular occurrence rather than an average recurrence interval. For example, it is sometimes assumed that the 1 in 100-year event flood would occur once every 100-years. However, events with a magnitude of the 1 in 100-year event have a 1 % chance of being exceeded in any one year. Table 1 provides a comparison of AEP to return periods to aid the understanding of flood frequency.

Table 1: Equivalent annual exceedance probabilities and return periods

AEP	10%	3.33%	2%	1.33%	1%	0.1%
Return Period	1 in 10-year	1 in 30-year	1 in 50-year	1 in 75-year	1 in 100-year	1 in 1000-year

2.1.3 Consequence

24) The consequence of flooding is dependent on two factors:

- Exposure – For example, the number of people or properties potentially affected
- Vulnerability – The potential for people or property to be harmed or damaged.

25) Floods impact both individuals and communities, and have social, economic, and environmental consequences. These can be both negative and positive and can include direct and indirect loss.

26) With regards to development and flood risk, vulnerability is largely driven by the type of development proposed or affected.

2.1.4 Impacts

27) The assessment of the flood risk impacts as a result of the Proposed Ribble Crossing and the magnitude of the change in flood risk, considers the potential effects on all elements of flood risk including flood frequency, extent, depth, velocity and combinations of these components.

28) The duration of changes to flooding is also considered when assessing flood risk impacts, where a distinction is made between permanent changes and temporary changes where the effect would cease to be felt after a period. Temporary changes can be long-term or short term in nature.

29) Embedded mitigation measures are also considered when determining potential impacts on flood risk. These measure form part of an optimised design used to reduce the significance of flood risk effects, for example:

- Following the sequential approach to avoid placing assets, features and activities within areas at high flood risk where possible
- Designing the Ribble Crossing, including construction phase, in accordance with established good practice
- Discharge surface water run-off as high up the drainage hierarchy and implementing Sustainable Drainage Systems (SuDS) where possible, to minimise the impact on the receiving watercourse.

2.1.5 Links to the Environmental Statement

30) The EIA process adopts a slightly different assessment model to flood risk (sensitivity x magnitude of change = significance), where:

- The **sensitivity** of a feature or resource is typically determined by, among other things, its level of designation or protection (e.g. importance, value or rarity), its susceptibility to or ability to accommodate change. Within the context of this FRA, sensitivity is a function of the likelihood of flooding and the potential consequences (i.e. baseline flood risk)
- The **magnitude of change** is a measure of the scale or extent of the change in the baseline condition, irrespective of the value of the feature or resource(s) affected (i.e. impact on flood risk)
- The **significance** of the overall flood risk is a product of the sensitivity of the resource or feature and the magnitude of the impacts.

31) Whilst the flood risk assessment model (probability x consequence = risk) will be used within this FRA, technical evidence provided in this FRA will be used to inform Chapter 8 (Flood Risk) of the Proposed Ribble Crossing ES. Annexe A therefore provides a set of assessment criteria used within the ES to define sensitivity, magnitude of change and significance.

2.2 Scope

2.2.1 Sources of information and data

32) The following readily available sources of information and datasets have been reviewed and assessed for the purpose of this FRA:

- Conceptual designs for the construction and operation of the Proposed Ribble Crossing provided by United Utilities
- Environment Agency Flood Map for Planning[4]
- Environment Agency Risk of Flooding from Surface Water Mapping[5]
- Environment Agency Reservoir Flood Mapping[4]
- British Geological Survey (BGS) mapping[6]
- British Geological Survey (BGS) groundwater flooding susceptibility maps[7]

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[4] Environment Agency (2020) Flood Map for Planning. [Online] Available at: https://flood-map-for-planning.service.gov.uk/. [Accessed: January 2021].

[5] Environment Agency (2020) Risk of Flooding from Surface Water Mapping. [Online] Available at: https://flood-warning-information.service.gov.uk/long-term-flood-risk/map. [Accessed: January 2021].

[6] British Geological Survey (2020) Geology of Britain viewer (classic). [Online] Available at: https://mapapps.bgs.ac.uk/geologyofbritain/home.html. [Accessed: January 2021].

[7] BGS (2020) BGS Groundwater Flooding Susceptibility Dataset [Accessed: January 2021]

- Ordnance Survey Datasets including 1:25,000 scale mapping
- The Ribble Valley Strategic Flood Risk assessment[8]
- United Utilities asset data
- A web search of historical flood incidents.

33) These datasets were then used to undertake a qualitative assessment of flood risk and potential flood risk impacts from all sources of flood risk.

34) Following completion of the draft FRA and in consultation with the Environment Agency, it was agreed that that the Flood Map for Planning was not suitable (as it was based on based on national scale hydraulic modelling) for use as the sole source of fluvial flood risk information to inform this site-specific FRA.

35) Without an existing hydraulic model available of the River Ribble at this location, a detailed hydraulic model was constructed for the watercourses that would be crossed by the proposed Ribble Crossing. This model includes a representation of the baseline (existing) situation and a representation of the Proposed Ribble Crossing to inform the design of the structures and to quantitatively assess their impacts. Full details of the hydraulic model are presented in Annexe C.

2.3 Assessment Area and Sources of flood risk

36) The assessment area of the Proposed Ribble Crossing FRA is focussed on the EIA red line boundary but also extends along the River Ribble approximately 1 km upstream and approximately 500 m downstream of the B6478 road to identify sources of flood risk and the extents of possible impacts on the proposed development.

37) With the Proposed Ribble Crossing located within the floodplain of the River Ribble and with crossings of Ordinary Watercourses required, fluvial flood risk has been identified as the main source of potential flood risk. Environment Agency Mapping has also identified a potential risk from surface water, reservoirs and groundwater.

38) The Proposed Ribble Crossing is approximately 40 km from the River Ribble Estuary at an elevation of more than 50 m AOD. Therefore, no coastal flood risk has been identified and no further assessment is necessary. A review of Ordnance Survey Mapping has not identified any canals in the vicinity of the Proposed Ribble Crossing that could pose a risk of flooding.

39) United Utilities have not identified any areas of sewer flood risk in close proximity to the Proposed Ribble Crossing and no discharges to the public sewer network are proposed. Failure of water mains are a potential source of flooding due to surcharging of man-made drainage systems but are unlikely to impact this type of development. Therefore, no further assessment of sewer and water mains have been undertaken.

40) No data is available on the location of local land drainage assets such as drains, channels and outflow pipes. Where these features are identified on site and affected, they would be replaced if necessary, with assets that have the same performance. Therefore, the risk of flooding, which is most commonly the result of obstructions, poor maintenance and/or blockages, is unlikely to change and no further assessment would be necessary.

41) The lifetime of the Proposed Ribble Crossing would be approximately seven years starting in 2023 as part of the enabling and construction phase of the main HARP. Therefore, the effects of climate change should not be considered in relation to this development.

42) In summary, this FRA would be focus on the following flood sources:

- Fluvial flooding
- Surface water flooding

[8] Ribble Valley (2010) Level 1 Strategic Flood Risk Assessment. [Online] Available at: https://www.ribblevalley.gov.uk/download/downloads/id/7085/strategic_flood_risk_assessment.pdf. [Accessed: 09 March 2021].

- Groundwater flooding
- Reservoir flooding.

2.4 Limitations and assumptions

43) The flood risk assessment was undertaken with the following limitations and assumptions:

- The assessment was based on outline design details. There are minor elements of the design, which may still be subject to change for example precise compound area locations
- The Construction Code of Practice (CCoP) has been produced to provide an overview of appropriate flood design principles, standards and good practice. It is assumed that these would be applied at later stages of the design process
- The approach to hydraulic river modelling and hydrological analysis inherently contains a number of limitations and assumptions which are outline in Annexe C
- No Ground Investigation (GI) has yet been undertaken and no BGS historical boreholes are located within or in the vicinity of the study area to provide data on groundwater levels. No springs are annotated on current Ordinance Survey maps within the study area. Therefore, the groundwater assessment is based on BGS mapping only.
- The assessment of surface water and reservoir flood risk is based on a conceptual assessment using national scale mapping. These datasets often do not include local features.

3. Planning Policy

44) The legislation and planning policies relevant to the Proposed Bowland Section are considered in Chapter 5 of the Proposed Ribble Crossing Environmental Statement.

45) The legislation and planning policies relevant to Water Environment are also considered in Section 7.3 of the Proposed Bowland Section Environmental Statement Chapter 7 and 8.

3.1 National Planning Policy

3.1.1 National Planning Policy Framework

46) The National Planning Policy Framework (NPPF) 9 was published by the Ministry of Housing, Communities and Local Government in February 2019. This sets out the government's policies for planning in England. The Planning Practice Guidance 10 is available online to support the policy documented within the NPPF.

47) The principle aim of the NPPF assessment of flood risk is that: "Inappropriate development in areas at risk of flooding should be avoided by directing development away from areas at highest risk (whether existing or future). Where development is necessary in such areas, the development should be made safe for its lifetime without increasing flood risk elsewhere."

48) The NPPF requires an FRA to be produced in the following scenarios:

- All proposals for new development (including minor development and change of use) in Flood Zone 2 and 3
- Proposals in an area within Flood Zone 1 which has critical drainage problems (as notified to the local planning authority by the Environment Agency) or greater than one hectare in size
- Where land identified in a strategic flood risk assessment as being at increased flood risk in future
- Where proposed development or a change of use to a more vulnerable class may be subject to other sources of flooding.

3.1.2 Assessment of Flood Risk

49) The flood risk from fluvial (Main Rivers) and tidal flooding is assessed through the use of the Environment Agency Flood Map for Planning (rivers and sea). This map defines three flood zones of different flood risk (the third of which is subdivided into two categories), as detailed in Table 1 of the Planning Practice Guidance (PPG):

- Zone 1 "Low probability of flooding" – This zone comprises land assessed as having a less than 1 in 1,000 annual probability of river or sea flooding (<0.1 % Annual Exceedance Probability (AEP))
- Zone 2 "Medium probability of flooding" – This zone comprises land assessed as having between a 1 in 100 and 1 in 1,000 annual probability of river flooding (1 % – 0.1 % AEP), or between a 1 in 200 and 1 in 1,000 annual probability of sea flooding (0.5 % – 0.1 % AEP) in any year
- Zone 3a "High probability of flooding" – This zone comprises land assessed as having a 1 in 100 or greater annual probability of river flooding (>1 % AEP), or a 1 in 200 or greater annual probability of flooding from the sea (>0.5 % AEP) in any year
- Zone 3b "The Functional Floodplain" – A sub-part of Zone 3, this zone comprises land where water has to flow or be stored in times of flood. This zone is not normally included within the national Flood Map for Planning and is calculated where necessary using detailed hydraulic modelling and is typically defined as areas having a 1 in 20 or greater annual probability of flooding (>5 % AEP).

[9] Ministry of Housing, Communities and Local Government (2019). National Planning Policy Framework. [Online] Available at: https://www.gov.uk/government/publications/national-planning-policy-framework--2 [Accessed: 09 March 2021].
[10] Ministry of Housing, Communities and Local Government (2019). Planning practice guidance. [Online] Available at: https://www.gov.uk/guidance/flood-risk-and-coastal-change [Accessed: 09 March 2021].

50) The NPPF requires that developers consider not just the flood risk to a development but also the impact that a development might have on flood risk elsewhere. As well as Main Rivers and the sea, it is also necessary to consider flood risk from other sources, including surface water, groundwater, Ordinary Watercourses, artificial drainage systems, canals and reservoirs, where relevant.

3.1.3 Sequential Test

51) The Sequential Test ensures that a sequential approach is followed to steer new development to areas with the lowest probability of flooding i.e. to steer new development to Flood Zone 1 (areas with a low probability of river or sea flooding). Where there are no reasonably available sites in Flood Zone 1, sites in Flood Zone 2 should be considered. Only where there are no reasonably available sites in Flood Zones 1 or 2 should the suitability of sites in Flood Zone 3 (areas with a high probability of river or sea flooding) be considered.

52) The proposed development would be difficult to locate entirely within Flood Zone 1 as it is required to provide access over the River Ribble to provide linkages between west and east bank of the river and through the agricultural land to West Bradford Road to avoid traffic through West Bradford. However, a sequential approach has still be applied, with the proposed development located in one of the narrowest areas of the floodplain (Flood Zone 2 and 3 is approximately 125 m wide) and in an area with no property immediately upstream, so that flood risk and potential impacts are limited as far as possible through the placement of the proposed development. It is therefore considered that the Sequential Test has been passed.

3.1.4 Vulnerability Classification

53) Since the proposed development would be in Flood Zone 3, it is necessary to take into account the flood risk vulnerability of land uses and if necessary, apply the Exception Test.

54) Table 2[11] illustrates the flood risk vulnerability categories and flood zone compatibility matrix for England from the NPPF. The Proposed Ribble Crossing is a component of the proposed Marl Hill and Bowland Sections of HARP. Therefore, as supporting infrastructure for a water transmission development, it is classified as 'water compatible'.

55) As water compatible development, the proposed Ribble Crossing would be suitable in all Flood Zones and the Exception Test would not be applicable. However, in order to provide a conservative approach, the principles of the exception test have been applied.

Table 2: Flood Risk Vulnerability and Flood Zone Compatibility.

Flood Zones	Flood Risk Vulnerability Classification				
	Essential infrastructure	Highly vulnerable	More vulnerable	Less vulnerable	Water-compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test required	✓	✓	✓
Zone 3a	**Exception Test required**	✗	Exception Test required	✓	✓
Zone 3b	**Exception Test required**	✗	✗	✗	✓

[11] Ministry of Housing, Communities and Local Government (2019). Planning practice guidance. Table 3: Flood risk vulnerability and flood zone 'compatibility'. [Online] Available at: https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/575184/Table_3_-_Flood_risk_vulnerability_and_flood_zone__compatibility_.pdf [Accessed: 09 March 2021].

3.1.5 Exception Test

56) The Exception Test, as set out in paragraph 160 of the NPPF, is a method to demonstrate and help ensure 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. For the Exception Test to be passed it should be demonstrated that:

- *"The development would provide wider sustainability benefits to the community that outweigh the flood risk*
- *The development will be safe for its lifetime taking account of the vulnerability of its users, without increasing flood risk elsewhere, and, where possible, will reduce flood risk overall."*[12]

57) The Proposed Ribble Crossing is predominately located within Flood Zone 2 and 3 and would comprise vital support infrastructure for the construction of the HARP, which would secure access to safe drinking water for millions of customers across the North West England. It would also aim to reduce in the volume of construction traffic through the neighbouring communities, causing less noise and air pollution. Therefore, it is assumed that the first element of the Exception Test would be passed.

58) This FRA has been prepared to address the second element of the Exception Test i.e. will the proposed development be safe for its lifetime, without increasing flood risk elsewhere and where possible reduce flood risk overall?

3.2 Local Planning Policy

59) NPPF is supplemented at a local level by development and flood risk policies put in place by the local authorities. The main documents which form Ribble Valley Borough Council's local planning policy relating to flood risk management is detailed below.

3.2.1 Ribble Valley Strategic Flood Risk Assessment (SFRA) Level One – May 2010[13]

60) The SFRA forms an integral part of the Ribble Valley Borough Council's flood risk and planning evidence base in terms of identifying locations for development and preparation of flood risk policies. The SFRA assess all types of flood risk across the Ribble catchment by using generalised and detailed model results supplied by the Environment Agency and existing Flood Zone mapping.

61) For the Clitheroe Policy Unit, the preferred policy is to take further action to reduce flood risk in this area. Policy DP9 also encourages new development to include adaptation to climate change. It emphasises the protection of the most versatile agricultural land and the use of SuDS techniques for transport development. Mitigation measures must be incorporated in any development that, exceptionally, must be placed in areas of current or future flood risk.

3.2.2 Consultation Draft Local Flood Risk Management Strategy for Lancashire 2021 - 2027[14]

62) This strategy is focused on the development of flood risk management schemes and includes details of how Risk Management Authorities can work together effectively across Lancashire to address local flood risks and challenges whilst maximising local opportunities.

63) One of the key themes identified within the strategy is "supporting sustainable flood resilient development".

[12] Ministry of Housing, Communities and Local Government (2019). National Planning Policy Framework. [Online] Available at: https://www.gov.uk/guidance/national-planning-policy-framework/14-meeting-the-challenge-of-climate-change-flooding-and-coastal-change#para160 [Accessed: 09 March 2021].

[13] Ribble Valley Strategic Flood Risk Assessment (2010). [Online] Available at: https://www.ribblevalley.gov.uk/download/downloads/id/7085/strategic_flood_risk_assessment.pdf [Accessed: 09 March 2021].

[14] Lancashire County Council and Blackpool Council (2021). Consultation Draft Local Flood Risk Management Strategy for Lancashire 2021 - 2027. [Online] Available at: https://www.blackpool.gov.uk/Your-Council/Documents/Local-Flood-Risk-Management-Strategy-2021-to-2027-Accessible.pdf [Accessed: December 2021].

64) "We will ensure that guiding principles for sustainable development are applied and inappropriate development is avoided in existing and future areas at risk of flooding and coastal erosion while elsewhere, carefully managing other land to avoid increasing the risks. We will work with our Local Planning Authorities to ensure Local Plans fully take account of food risks and have policies in place which manage these risks and make sure that all developments take account of them."

3.2.3 Ribble: Catchment Flood Management Plan – Dec 2009[15]

65) This plan provides an overview of the flood risk across the River Ribble Catchment and recommended ways of managing the flood risk now and over the next 50 to 100 years.

66) The Clitheroe sub-area is located downstream from the proposed development, which falls into areas of moderate to high flood risk where we can generally take further action to reduce flood risk. Appropriate highways drainage would need to be considered to avoid increased flood risk downstream. The policy promotes the application of rigorous planning control for any new development in and around Clitheroe using the principles in Planning Policy Statement 25 and encourage the implementation of SuDS.

[15] Environment Agency (2009). Ribble: Catchment flood management plan. [Online] Available at: https://www.gov.uk/government/publications/ribble-catchment-flood-management-plan [Accessed: 09 March 2021].

4. Assessment of Flood Risk

67) This section of the FRA focuses on both the flood risk to the Proposed Ribble Crossing and potential impacts it would have on flood risk. The assessment includes consideration of temporary construction compound sites, associated features, temporary access tracks and surface water drainage.

4.1 Existing Land Use and Topography

68) The landscape in the vicinity of the proposed Ribble Crossing consists predominantly of agricultural fields with grassland and farmsteads. There are numerous footpaths throughout the red line boundary of the Proposed Ribble Crossing.

69) Designated Sites[16] in proximity of the Proposed Ribble Crossing include:

- Forest of Bowland, which is an Area of Outstanding Natural Beauty (AONB) and located adjacent to the other side of West Bradford Road to the north from the proposed development
- Cross Hill Quarry, which is a Local Nature Reserve (LNR) and located approximately 500 m to the south from the Proposed Ribble Crossing
- Coplow Quarry, which is a Sites of Specific Scientific Interest (SSSI) and located approximately 1000 m to the south from the Proposed Ribble Crossing.

70) Light Detection and Ranging (LiDAR) data shows the changes in height across the surface within the vicinity of the Proposed Ribble Crossing. Topographically, the high point within the red line boundary is at the northern end of the proposed development on the West Bradford Road at the junction with the proposed access road. The land here is nearly 81 m Above Ordnance Datum (AOD) and it falls to the southeast to a level of approximately 60 m AOD at bridge crossing of the Proposed Ribble Crossing. South of the Ribble, land rises up to approximately 63 m AOD at the east end of the proposed development and the southern junction with West Bradford Road.

4.2 Hydrology

71) The Main Rivers within the assessment area are the River Ribble, Waddington Brook and West Bradford Brook.

72) The River Ribble is the main hydrological point of interest and source of flooding in the area. The River Ribble flows from northeast to southwest direction past the Proposed Ribble Crossing. The catchment area[17] for the River Ribble upstream (NGR: SD 74404 43875) of the Proposed Ribble Crossing is approximately 390 km^2. The watershed runs approximately from north to south from Gayle Moor in the Yorkshire Dales National Park to the Clitheroe area. The River Ribble has a predominately agricultural land use with small urban areas dispersed throughout the catchment.

73) Approximately 100 m upstream of the Proposed Ribble Crossing, the River Ribble is crossed by an existing bridge. This historic bridge comprises four masonry arches which are supported by three piers within the river channel.

74) The Ordinary Watercourses within the study area from east to west along the River Ribble are Moor Roads Sike, Unnamed Watercourse 2100, Unnamed Watercourse 2099, Unnamed Watercourse 2097, Greg Sike, Coplow Brook, and Unnamed Watercourse 2098. Four of them are present within the Proposed Ribble Crossing red line boundary. These are identified on Figure 2 and are summarised below:

- Coplow Brook would be crossed approximately 200 m downstream of the existing West Bradford Road culvert crossing. The catchment area at the proposed crossing location would be 0.95 km^2. A simple slab crossing is located approximately 200 m downstream of the Proposed Ribble Crossing

[16] DEFRA Multi-Agency Geographic Information for the Countryside (MAGIC) online mapping tool. [Online] Available at: https://magic.defra.gov.uk/MagicMap.aspx. [Accessed: February 2021].
[17] Flood Estimation Handbook (FEH) Web Service. [Online] Available at: https://fehweb.ceh.ac.uk/. [Accessed: February 2021].

- Greg Sike would be crossed at a point approximately 650 m south of West Bradford Road. Its catchment at this point would be approximately 1 km^2. A simple bridge to provide field access has been identified approximately 50 m upstream of the proposed crossing
- Unnamed Watercourse 2097 would be crossed at a point approximately 650 m south of West Bradford Road. The catchment of this watercourse is approximately 0.1 km^2. A footbridge providing field access is located approximately 30 m upstream of the proposed crossing
- Unnamed Watercourse 2099 is located to the south of the River Ribble. It would not be crossed by the Proposed Ribble Crossing and with a catchment area of less than 0.1 km^2 it is likely to be ephemeral.

75) Outside of the red line boundary, immediately to the north of the Proposed Ribble Crossing is the confluence of the River Ribble and West Bradford Brook (Main River, NGR: SD 74588 44167) and approximately 40 m further upstream is the confluence of the Moor Roads Sike (Ordinary Watercourse, NGR: SD 74627 44194). West Bradford Brook at this point has a catchment area of approximately 4.59 km^2 and Moor Roads Sike has a catchment area of approximately 1 km^2.

76) Immediately to the south of the planning application boundary of the Proposed Ribble Crossing there is the confluence of the River Ribble and Waddington Brook (Main River, NGR: SD 74012 43438). Waddington Brook at this point has a catchment area of approximately 5.20 km^2.

77) Existing receptors within the assessment area include:

- The local road network (Clitheroe Road, West Bradford Road)
- Approximately 10 residential properties within Flood Zone 2 in West Bradford upstream of the Proposed Ribble Crossing
- The Hanson Cement quarry located on the left (southern) bank
- Isolated farm properties and pastoral farmland.

78) A detailed hydrological study is presented in Annexe C to support the hydraulic modelling of the watercourses within the study area. This includes photographs of existing structures present along the watercourses assessed.

4.3 Soils, Geology and Hydrogeology

79) Pending the results of a detailed ground investigation (GI), geological data has been obtained from the British Geological Survey (BGS) online mapping viewer.[18] Hydrogeological information has been obtained from the DEFRA MAGIC online mapping tool.

80) The majority of the area consists of slowly permeable seasonally wet acid loamy and clayey soils, which drains to the stream network. However, the soil of the east side of the River Ribble is freely draining slightly acid but base-rich, which drains to the groundwater.[19]

81) The underlying bedrock geology comprises the Clitheroe Limestone Formation and Hodder Mudstone Formation. This was formed approximately 337 to 347 million years ago in an environment dominated by shallow carbonate seas. The rocks comprising carbonate material (coral, shell fragments), forming beds and locally reefs.

82) The overlying superficial deposits largely comprise Alluvium - Clay, Silt, Sand and Gravel. These rocks were formed up to 2 million years ago in the Quaternary Period.

83) There are no GI or historical BGS boreholes within the site of the Proposed Ribble Crossing to confirm the geology or prove otherwise. However, the crossing and the access road linking it to West Bradford Road is expected to directly cross all the superficial lithologies.

84) The Environment Agency's aquifer designation maps indicate that the study area is underlain by bedrock designated as a Secondary A aquifer. This classification refers to aquifers with 'permeable layers capable

[18] The British Geological Survey online tool [Online]. Available at: http://mapapps.bgs.ac.uk/geologyofbritain/home.html.
[19] Cranfield University, Soilscapes mapping tool [Online]. Available at: http://www.landis.org.uk/soilscapes/. [Accessed: February 2021].

of supporting water supplies at a local level rather than a strategic scale, and in some case forming an important source of base flow to rivers. These are generally aquifers formally classified as minor aquifers.'[20]

85) The superficial deposits are designated as Secondary (undifferentiated) aquifers. These are layers assigned in cases where it has not been possible to attribute either category A or B to a rock type. In most cases, this means that the layer in question has previously been designated as both minor and non-aquifer in different locations due to the variable characteristics of the rock type.

86) The groundwater-bearing Glacial Till is designated as a Secondary Undifferentiated aquifer by the Environment Agency and the BGS with each bedrock formation designated as Secondary A aquifers. This means that each of the bedrock aquifers contain permeable layers of rock capable of supporting water supplies at a local scale with Glacial Till having the potential to store and yield limited amounts of groundwater which are potentially important to river baseflow and abstractions at a local scale only.

87) The assessment area falls within a Groundwater Vulnerability Zone classified as Medium at the east bank of the River Ribble, which become Medium-Low and Low for the majority of the study area. The whole study area lies within soluble rock risk category.

4.4 Fluvial Flood Risk

88) This section of the report includes details of the baseline fluvial flood risk, the potential effects and the likely magnitude of impacts as a result of the proposed development.

89) Fluvial flooding refers to flooding from rivers, streams and other inland watercourses. Fluvial flooding is usually caused by prolonged or intense rainfall, generating high rates of runoff which overwhelm the capacity of the channel. When this occurs, excess water spills onto low-lying areas of land adjacent to the channel.

90) Fluvial flood risk can be divided between risk from Main Rivers and risk from Ordinary Watercourses. Main Rivers are usually larger rivers and streams where the Environment Agency carries out maintenance, improvement or construction work to manage flood risk. Ordinary Watercourses are any other watercourses not designated as Main Rivers.

4.4.1 Fluvial flood risk to the Proposed Ribble Crossing

91) During the outline design stage of the Proposed Ribble Crossing, several temporary road routes and bridge location were considered taking into account a range of design and environmental considerations including flood risk. The location of the proposed temporary road and temporary bridge crossing has been confirmed in January 2021, as it is believed to be the best location due to the stable straight channel, the relatively narrow floodplain and its proximity to the existing road network (see Drawing 80061155-01-JAC-TR3-97-DR-C-00009).

Risk from Main Rivers

92) The Environment Agency Flood Map for Planning (FMfP) as illustrated in Figure 2 shows the extents of Flood Zone 2 and 3. The Strategic Flood Risk Assessments (SFRA) for Ribble Valley adds that all rural/undeveloped sites within Flood Zone 3 should also be considered as "potential" areas of Functional Floodplain (Flood Zone 3b) as a precautionary approach to development. Therefore, the development envelope of the Proposed Ribble Crossing potentially could be associated with Flood Zone 3b.

93) Significant elements of the proposed Ribble Crossing would take place within Flood Zone 3 and would therefore be at high risk from fluvial flood events with a probability of greater than 1 % AEP

94) As identified in Section 2.2, the assessment of fluvial flood risk has been informed by new hydraulic modelling. Details of the hydraulic model are presented in Annexe C along with flood maps showing depths and extents during modelled flood events. Flood events modelled include the 50 %, 20 %, 10 %, 3.33 %, 1.33 % and 1 % AEP events.

[20] Environment Agency – Aquifers. [Online] Available at: http://apps.environment-agency.gov.uk/wiyby/117020.aspx. [Accessed: February 2021].

95) Hydraulic modelling identifies that the onset of out of bank flooding along the River Ribble is the 50 % AEP flood event with flow overtopping the right bank immediately downstream of the existing road bridge. During the 20 % AEP flood event, floodwater overtops the right bank, upstream of the existing bridge and flows through the right bank floodplain, bypassing the existing bridge before re-entering the river channel approximately 100 m downstream of the location of the proposed bridge across the River Ribble. The right bank floodplain then increases in width approximately 300 m downstream of the existing bridge to form an extensive area of flooding at the outside of a downstream meander. Figure 8-1 of Annexe C details the depth of flooding and the direction of flood flows through the use of velocity vectors.

96) Flooding of the left bank is largely limited to an area upstream of the existing bridge during flood events with a magnitude of 3.33 % AEP and greater. Downstream of the existing bridge, ground levels rise relatively steeply to ensure that the floodplain is limited to a narrow strip of open land adjacent to the bank.

97) The modelled flood extents are generally smaller in the vicinity of the Proposed Ribble Crossing than those shown by the existing Flood Map for Planning.

98) Table 3 provides a summary of peak fluvial flood depths at the construction phase elements of the Proposed Ribble Crossing.

Table 3: Summary of approximate maximum baseline fluvial flood depth from Main Rivers

Enabling phase works	20% AEP	3.33% AEP	1% AEP
Left bank laydown area	Not in modelled flood extent		
Right bank downstream laydown area	0.5 m	1 m	1.2 m
Right bank upstream laydown area	0.5 m	0.8 m	1.1 m
Northern laydown area	Not in modelled flood extent		
Temporary access track to right bank compound	0.7 m	1 m	1.3 m

99) Hydraulic modelling confirms that the probability of flooding, to the right bank laydown areas (required for construction of the bridge) would be very high with flooding occurring during the 50 % AEP flood event. Maximum flood depths during the 50 % AEP flood event would be approximately 0.5 m although maximum velocities would be less than 1 m/s. The predicted depth of floodwater would necessitate the pre-evacuation of staff from this area and the removal material and equipment to avoid damage or potential risk to life.

100) A very short section of the track would be located in an area that would also flood during the 20 % AEP flood event. To reduce the probability of flooding, the vertical alignment of the access road will be raised above the 20 % AEP event flood level. This will limit flooding of the road to the 3.33 % AEP where only a short 100m of the track will be at risk of flooding to a depth of 200mm. The flow velocity during the 3.33% AEP event at this section of track would be less than 1 m/s. The modelled flood depth and velocity at this location would be low and would typically represent a low hazard to construction traffic and would be unlikely to result in significant damage to the road if inundated. However, it is recommended that the contractor considers the risk of road closures due to flooding within their construction programme.

101) The piers of the proposed bridge across the River Ribble would be located within an area with a high probability of flooding. The piers have been designed to withstand the likely loading and erosive forces from flood waters. Embedded mitigation measures include the construction of the piers to be parallel to the direction of flood flows and to incorporate a rounded leading edge.

102) The bridge soffit would be placed above the level of the 1 % AEP flood level + 600 mm freeboard (as detailed on drawing 80061155-01-JAC-TR3-97-DR-C-00009). Therefore, the probability of floodwater overtopping the bridge structure would be low.

103) Element of the Proposed Ribble Crossing are located within areas with a high risk of flooding. However, good practice measures detailed within the CCoP would include:

- The adoption of a sequential approach to the layout of construction compounds to help to ensure that sensitive equipment avoids the areas of highest risk
- Monitoring of water levels
- Subscription to the Environment Agencies Flood Warning Area for the Upper River Ribble and Hodder
- The preparation of flood response plans to ensure the safe evacuation of staff and plant from the area of risk and the inspection and repair (if necessary) of any damaged sections of the proposed Ribble Crossing.

104) These measures would result in the likely consequences of flooding being limited to short-term disruption impacting the construction works and/or operation of the access road. Therefore, the overall flood risk to the enabling works is considered to be low and no additional mitigation requirements have been identified.

105) There would be a residual risk from fluvial flood events greater than 3.33 % AEP which may result in longer periods of disruption and potential damage to elements of the proposed Ribble Crossing, but given the assumed seven-year design life and the freeboard allowance within the bridge design this is considered to be low.

Risk from Ordinary Watercourses

106) The Proposed Ribble Crossing would also involve the crossing of three Ordinary Watercourses which are tributaries of the River Ribble. These are from upstream to downstream along the River Ribble, Unnamed Watercourse 2097, Greg Sike, and Coplow Brook. The Ordinary Watercourses do not individually have Flood Zones as represented within the Environment Agency FMfP although their confluences are located within Flood Zones 2 and 3 associated with the River Ribble.

107) Along with the River Ribble, these Ordinary Watercourses have been individually hydraulically modelled to determine their potential flood risk to the Proposed Ribble Crossing. Given the potential complex interactions with the River Ribble, they have been modelled independently. A summary of the flood risk at the crossing locations is provided in the sections below with fuller details of the model outputs including flood depth maps and velocity vectors is provided in Annexe C.

108) As identified in Section 4.2, these watercourses are small, steep watercourses within narrow channels which are crossed in multiple places by existing structures.

109) The hydraulic models show that out of bank flooding would first occur at Unnamed Watercourse 2097 in the vicinity of the proposed crossing during the 1.33 % AEP flood event. Peak flow during this event would be 0.56 m3/s and would overtop the banks upstream of the Proposed Ribble Crossing at an existing crossing location. This would result in shallow (less than 100 mm) flood water flowing through a short (less than 100m) section of floodplain before returning to the channel in the location of the proposed Ribble crossing.

110) At Greg Sike, out of bank flooding during the baseline scenario would first occur during the 50 % AEP flood event where peak flood flows would be 1.1 m^3/s. Flow would overtop the banks upstream of the Proposed Ribble Crossing at an existing crossing with shallow (less than 300 mm) flood water passing over the proposed road location and extending along the right bank of the watercourse towards the confluence of Coplow Brook and the River Ribble downstream. Flood depths across the road would between approximately 50 mm during the 20 % AEP flood event and approximately 100 mm during the 1 % AEP flood event. Flow velocities would be less than 1 l/s during both flood events.

111) Coplow Brook would flood during the 50 % AEP flood event with flows of 2.46 m^3/s predicted. Flow in the baseline scenario is predicted to overtop a low section of the right bank approximately 15 m downstream of the Proposed Ribble Crossing. During the 1 % AEP flood event, flood flows would spill

to the south and west of the channel and would result in flooding of a residential property before joining the River Ribble.

112) The proposed bridge across Coplow Brook would surcharge during the 50 % AEP flood event. However, flow would not overtop until the 3.33% AEP event. During the 3.33 % AEP flood event this area of flooding would spill over the road and a new flow path would form along the road. During the 3.33 % AEP flood event, flood flows along the road would be approximately 50 mm deep whilst during the 1 % AEP flood event, depths of approximately 100 mm are predicted. Flow overtopping the road would be less than 1 l/s in both predicted flood events.

113) Crossings of these watercourses would be single span structures. Lancashire County Council advised that their standard design requirement for crossings would be 600 mm above the 1 % AEP flood level. However, crossings with this standard of service would require large ramps and long spans to cross the extensive areas of predicted flooding. This is considered to be disproportionate given the small capacity of existing upstream crossings and the lack of vulnerable receptors to flooding. These larger structures would also act as potential barriers to River Ribble floodplain flows. Therefore, to avoid the potential for adverse impacts on the River Ribble floodplain the proposed design is based on bridge soffits being set at the bank top level with the bridge deck set approximately 600 mm above this. This would be in line with existing upstream crossings.

114) As the Proposed Ribble Crossing would not be raised above the predicted flood level of the Ordinary Watercourses, the likelihood of flooding along the road would be high. Similar embedded mitigation to that applied for the risk of flooding from Main Rivers would be applied, including proactive monitoring and planning, but although flood depths and velocities are predicted to be relatively low, it is recommended that the contractor plans for road closures during periods of flooding. With flood risk from Ordinary Watercourses managed effectively by the contractor, the consequences of this flooding would be limited to short term disruption to the road and the flood risk to the Proposed Ribble Crossing is considered to be low.

115) There would be a residual risk from events greater than 1 % AEP but given the assumed seven-year design life this is considered to be low.

4.4.2 Impacts on fluvial flood risk from the Proposed Ribble Crossing

116) Embedded mitigation has been incorporated into the design of the Proposed Ribble Crossing to avoid or reduce the magnitude of the following potential fluvial flood risk impacts:

- Temporary loss of storage volume within the River Ribble floodplain
- The temporary restriction of flood flows within the River Ribble floodplain
- The temporary constriction of flood flows within the three Ordinary Watercourses that would be crossed
- Temporary increase in runoff rates entering watercourses due to an increase in hard standing associated with access road and compound sites
- Temporary increase in the risk of blockages
- Temporary discharges of groundwater entering watercourses from excavations activities.

Impacts of the bridge across the River Ribble

117) The results of the hydraulic modelling (presented within Annexe C) demonstrate that the Proposed Ribble Crossing would have a negligible impact on the magnitude of flooding from the River Ribble with changes in predicted flood levels generally less than 10 mm during all flood events modelled. During the 1 % AEP flood event, localised depth increases of no greater than 50 mm are predicted immediately upstream of the piers and a small area downstream where the level of the proposed track needs to be raised over Unnamed Watercourse 2097. However, these areas of increased flood depth are limited to areas of pastoral farmland that would already be inundated during equivalent flood events in the baseline scenario. For example, during the 1 % AEP flood events the areas which would experience

increases would be flooded to depths of up to 1 m. Therefore, the overall impact on flood risk is considered to be negligible.

118) Across all of the flood events modelled, increases in flood extent are predicted to be negligible. Any changes in flood depth at nearby sensitive receptors, including the existing bridge crossing and nearby residential property are also shown to be negligible. For example, during the 1 % AEP flood event, flood levels along Clitheroe Road would increase by a maximum of 2 mm compared to a baseline flood depth of more than 1 m. Upstream of Clitheroe Road, changes in flood depth are less than 1 mm. No change in flood risk is predicted at any residential properties.

119) Any increase in blockage risk due to the proposed bridge across the River Ribble is considered to be unlikely. The existing upstream bridge has a smaller capacity than the proposed bridge across the River Ribble and structures within the channel. Therefore, it is more likely to trap any large debris from reaching the proposed bridge crossing downstream. The proposed bridge across the River Ribble would also have a soffit level 600 mm above the 1 % AEP flood level so is unlikely to block as a result of any debris passing through the existing bridge. Good practice measures including monitoring and subsequent debris removal by United Utilities would also ensure that the residual risk is managed.

120) The construction compounds associated with the proposed bridge across the Ribble have been designed to ensure that there is an 8 m standoff between the top of bank and the edge of the compound to ensure that access to the riverbank for any future inspection and maintenance by the EA during the operational period is maintained. This easement is shown in drawings within Annexe B.

Impacts of the Ordinary Watercourse crossings

121) Full, “with scheme” model results are presented in Section 8.2 of Annexe C and are summarised below.

122) The proposed bridge across Coplow Brook would surcharge during the 50 % AEP flood event. However, flow would not overtop the road until the 3.33 % AEP event. During the 3.33% AEP flood event, flooding would spill over the road and a new flow path would form along the road. Flood depths would be approximately 50 mm deep, increasing to 100 mm during the 1 % AEP flood event. During such events road closures may be required. Post event inspections of the track crossing and track surface would be required, and any damage repaired. There is potential for this flow path to convey flood water towards a barn building located along the left bank of Coplow Brook. This Barn is not at risk of flooding within the baseline flood event from Coplow Brook or the River Ribble. However, the potential diversion of flows would result in a moderate adverse impact. Measures to manage exceedance flow along the track would need to be developed by the contractor as part of the detailed design and could include:

- Measures to divert flow off the road and back into Coplow Brook before flow reaches the barn such as small changes to the vertical alignment of the road or cross drainage to divert flow into Coplow Brook
- Measures to retain flow within the track and route flow away from the barn and towards the River Ribble such as a low roadside bund.

123) Whilst the flow path along the track would remain, initial testing of these measures using the hydraulic modelling has demonstrated that either of these would be effective and would ensure that the impact of flood risk to the barn would be negligible.

124) The increase in flood depth upstream of the road would be limited to an area of pastoral farmland along an existing flood flow path and no vulnerable receptors would be affected. However, by diverting flow to the left bank, there will be no increase in risk to the residential property downstream which is south and west of Coplow Brook. Flood levels along the flow path towards this property, would generally reduce by a negligible magnitude (less than 10 mm) although small areas are predicted to experience depth reductions of up to 50 mm.

125) Greg Sike is predicted to experience out of bank flooding during the 50 % AEP flood event as flow spills from the channel upstream of an existing footbridge located upstream of the Proposed Ribble Crossing. The soffit of the proposed crossing would be set at the existing bank level with the bridge deck approximately 600 mm above this level. The ramps up to the bridge deck would result in a small

displacement of flood flows with flow passing over a section of unraised track immediately west of the proposed crossing.

126) Unnamed Watercourse 2097 is not predicted to experience out of bank flooding until the 1.33 % AEP flood event with lower magnitude events remaining in channel and freely passed by the proposed crossing. During events which result in out of bank flooding, the ramps over the crossing would restrict and divert flow, increasing flood depths by up to 150 mm during the 1 % AEP flood event as well as increasing flood extents to areas of pastoral farmland.

127) The localised adverse impacts predicted at all three crossing locations would be limited to the road that forms part of the Proposed Ribble Crossing and pastoral farmland and would be temporary in nature. No adverse impacts are predicted to any vulnerable receptors. The areas impacted by the crossings of Greg Sike and the Unnamed Watercourse 2097 are within the area at risk of flooding from the River Ribble and it is likely that during a flood event, the impact on these Ordinary Watercourses would be imperceptible when compared to flooding from the River Ribble. Therefore, given the low sensitivity of the land that would be impacted, the short-term nature of the impacts and the insignificance of this source of flooding compared to Main River flooding, the overall impact on flood risk is considered to be negligible and no additional mitigation is considered to be necessary.

128) However, the contractor will need to be aware of the high probability of fluvial flooding and the potential disruption and damage to the road if flooding was to occur.

Impacts of changes in runoff rates on fluvial flooding

129) The proposed access road and the proposed construction compounds are located on existing greenfield sites currently comprising agricultural land. The compaction of soil and the creation of impermeable surfaces associated with the proposed construction compounds and roads have the potential to increase the rate of surface water runoff which could have impacts on or fluvial flood risk within the receiving watercourses. However, the management of surface water runoff using a proposed sustainable surface water drainage system based on roadside swales would reduce runoff to greenfield rates would ensure that magnitude of any effects on surface water or fluvial flood risk would be negligible.

130) In summary, although the impact on surface water runoff and blockage on fluvial flood risk would be negligible, it is predicted that there are likely to be temporary minor adverse impacts on fluvial flood risk due to the constriction of flows and the loss of floodplain volume. Therefore, additional mitigation would be required which is detailed in Section 4.8.

4.5 Surface Water Flood Risk

131) Surface water flooding is defined as water flowing over the ground that has not yet entered a drainage system or watercourse. It usually occurs as a result of an intense period of rainfall, which exceeds the infiltration capacity of the ground or sewer system.

4.5.1 Surface water flood risk to the Proposed Ribble Crossing

132) The Environment Agency's Risk of Flooding from Surface Water mapping indicates that the probability of surface water flooding across the red line boundary is generally very low (less than 0.1 % AEP). However, localised areas of risk along surface water flowpaths or areas or ponding have been identified as shown on Figure 3. These areas include:

- At the eastern end of the Proposed Ribble Crossing, the junction of West Bradford Road to the south of the Ribble would be at risk of flooding to a depth of less than 300 mm during the 1% AEP rainfall event
- The construction compound on the right (north) bank of the River Ribble at the eastern end of the red line boundary would be at risk of flooding to a depth of less than 300 mm during the 1 % AEP flood event
- A section of the temporary road approximately 5 m long east to Greg Sike would flood to a depth of less than 300 mm during the 0.1 % AEP flood event

- A surface water flowpath up to 900 mm deep would form during the 3.33 % AEP flood event immediately to the south of the proposed crossing of Coplow Brook.

133) Embedded mitigation in the form of drainage around construction compounds and along the road would ensure that shallow surface water flooding is managed safely and would ensure that the risk of flooding to the Proposed Ribble Crossing would be low.

134) Good practice detailed with the CCoP would be applied during the design of the track and compounds to ensure that additional drainage features such as cross drains are included to reduce the risk of flooding along the road. In the event of an extreme rainfall event and localised surface water flooding, a flood response plan would be in place to manage the safety of staff and equipment on site.

135) In summary, although localised areas of moderate to high surface water flood risk have been identified, embedded mitigation measures and good practice would ensure that the risk to the Ribble Crossing from surface water would be low.

4.5.2 Impacts on surface water flood risk from the Proposed Ribble Crossing

136) The proposed locations for the study area currently comprise agricultural land. The development of the temporary access road and associated features are likely to increase the area of impermeable surfaces due to the road surface, which is proposed to be a tarmac construction based on a stone aggregate foundation. Therefore, it would increase the rate of surface water runoff. Uncontrolled, any increase in runoff could increase the risk of surface water flooding downstream through the surface water catchment or to the discharge location.

137) In line with NPPF, surface water management strategies would be developed for the Proposed Ribble Crossing study area with the focus on the temporary access road and bridge.

138) The proposed drainage strategy would include:

- The placement of stockpiles of materials outside of areas of surface water flood risk
- A system serving the compounds that captures runoff and drain to attenuation lagoons prior to discharge to the River Ribble.

139) The impacts on surface water runoff associated with the compounds and laydown areas during construction phase would not be experienced in the operational phase of the proposed Ribble Crossing as they would be restored back to agricultural land.

140) The establishment of temporary compounds and laydown areas to enable the decommissioning would have the potential to result in increase in surface water runoff as during construction phase. Mitigation would be the same as during construction with temporary drainage managing runoff to ensure that the impact on flood risk would be negligible.

141) The proposed surface water drainage would manage any potential increase in surface water runoff rates as a result of the Proposed Ribble Crossing and as a result, the impact on surface water flood risk would be negligible.

4.6 Groundwater Flood Risk

142) Groundwater flood risk refers to either a rise in the water table or lowering of the ground level leading to an increased likelihood of flooding at the ground surface. The magnitude of the change in groundwater levels relative to the ground surface and spatial extent affected is considered for this assessment of groundwater flood risk impacts.

4.6.1 Groundwater flood sources

143) Groundwater is stored in both superficial aquifers, typically of Glacial Till, and underlying bedrock aquifers which is discussed in the Water Environment section of the main ES report (Chapter 7).

144) The aquifer units present are described in Section 4.3 and include Alluvium and underlying bedrock comprising Clitheroe Limestone Formation and Hodder Mudstone Formation.

145) No springs are annotated on current Ordinance Survey maps within the study area. However, it is considered likely that shallow groundwater flow follows the topography towards the River Ribble.

4.6.2 Groundwater flood risk to the Proposed Ribble Crossing

146) The temporary access route across the River Ribble is immediately adjacent to the River Ribble itself and minor tributaries. As groundwater is likely to be in continuity with river levels, emergence of groundwater is likely to be indistinguishable from fluvial flooding which is assessed in Section 4.3.

147) Below ground elements of the construction and enabling works such as foundations would be designed in accordance with good practice following a ground investigation to ensure that they would be designed with regard to local groundwater conditions. This would ensure that the risk from groundwater flooding would be low.

4.6.3 Impacts on groundwater flood risk from the Proposed Ribble Crossing

148) Earthworks associated with the construction of laydown areas, topsoil storage, welfare and generator locations, attenuation ponds as part of the drainage system have the potential to encounter groundwater. These works have therefore the potential to allow groundwater to flood excavation areas and reach the surface.

149) Excavations including the construction of bridge foundations would have the potential to result in localised disturbance of groundwater flow and the potential release of artesian pressures. However, the localised scale of the works combined with good practice design following a GI would ensure that the magnitude of any impacts on groundwater flooding would be negligible.

4.7 Reservoir Flood Risk

150) Flooding could also occur due to the collapse and/or failure of man-made water retaining features such as hydro-dams, water supply reservoirs, canals, flood defences structures, underground conduits, and water treatment tanks or pumping stations. No canals or flood defences have been identified within the vicinity of the Proposed Ribble Crossing and therefore, the assessment of flooding from artificial infrastructure is focussed on reservoirs.

151) Reservoir failure can be a particularly dangerous form of flooding as it results in the sudden release of large volumes of water that can travel at high velocity. This can result in deep and widespread flooding, potentially resulting in significant damage. The likelihood of reservoir flooding occurring is however extremely low even with all large reservoirs (over 25,000 m^3) managed in accordance with the Reservoirs Act 1975.

4.7.1 Reservoir flood sources

152) There is one covered reservoir located approximately 2 km upstream of the northern extent of Proposed Ribble Crossing, called West Bradford Reservoir.

153) The Environment Agency's online reservoir flood mapping (Figure 5) illustrates the maximum flood extents from reservoir failure along the Proposed Ribble Crossing.

4.7.2 Reservoir flood risk to the Proposed Ribble Crossing

154) Environment Agency reservoir flood mapping indicates that the Proposed Ribble Crossing would be located within the maximum extent of potential reservoir flooding. Therefore, failure of this reservoir would pose a risk to the access road and the temporary bridge across the River Ribble and the construction works associated with these elements of the development. Maximum flood depths of more than 2 m and maximum flow velocities of more than 2 m/s are predicted along the River Ribble in the event of a reservoir failure.

155) Failure of any reservoir would be however highly unlikely during the enabling and construction phase of the Proposed Ribble Crossing. Good practice mitigation in the form a flood response plan which included subscription to flood warnings and an evacuation plan is outlined in the CCoP. This would also ensure

that the consequences of a reservoir flood to the proposed development would be low. Therefore, the overall risk is low, and no additional mitigation is needed.

4.7.3 Impacts on reservoir flood risk from the Proposed Ribble Crossing

156) The Proposed Ribble Crossing would be located downstream and remote from West Bradford Reservoir. Therefore, no mechanism has been identified by which the Proposed Ribble Crossing would increase the likelihood of reservoir failure.

157) Hydraulic modelling has demonstrated that the proposed temporary bridge across the River Ribble has a negligible impact on flood flow during the 1 % AEP fluvial flood event. Therefore, in the unlikely event of a reservoir failure (which could follow the same flood flow routes), it is inferred that any adverse impacts would be localised within areas of pastoral farmland. These increases in flood depth would have a negligible impact on overall flood risk.

158) The impact of the enabling and construction phase activities of the Proposed Ribble Crossing on reservoir flooding would therefore be negligible.

4.8 Mitigation

159) Several elements of the Proposed Ribble Crossing would have a high probability of flooding and it is likely that flooding may cause disruption to the construction and operation of the temporary development. However, the mitigation measures embedded within the design of the Proposed Ribble Crossing and good practice measures detailed within the CCoP would enable the Proposed Ribble Crossing to be managed in a safe and effective manner during its design life.

160) Flood risk impacts elsewhere have been assessed to be generally negligible with the exception of the potentially increased risk to the barn building adjacent to Coplow Brook. Additional mitigation such as cross drains, or a roadside bund would effectively manage the shallow flooding predicted and mitigate the impacts to the barn. It is proposed that this mitigation would be developed by the contractor during the detailed design phase. No further requirements for additional mitigation measures have been identified.

5. Summary and Conclusion

5.1 Summary

161) This FRA has been prepared to support the planning application for the Proposed Ribble Crossing, which includes temporary features to provide road access to construction traffic associated with the Haweswater Aqueduct Resilience Programme (HARP). The Proposed Ribble Crossing would be located approximately 2 km north of Clitheroe town centre and immediately adjacent to West Bradford (to the south of the village).

162) The FRA focuses on the following key high risk or high impact activities or features associated with the enabling, construction and decommission of the Proposed Ribble Crossing:

- Temporary laydown sites, associated features, temporary construction access track and surface water drainage associated with the enabling and construction phases of the Proposed Ribble Crossing
- Management of groundwater intercepted during excavation works
- A temporary access road and bridge crossing over the River Ribble
- Temporary access road crossings of three Ordinary Watercourses.

163) The Proposed Ribble Crossing would be difficult to locate entirely within Flood Zone 1 as it is required to provide linkages between west and east bank of the River Ribble and through the agricultural land to West Bradford Road to avoid traffic through West Bradford (and the associated noise and air pollution impacts). However, a sequential approach has still be applied, with the proposed development located in one of the narrowest areas of the floodplain (Flood Zone 2 and 3 is approximately 125 m wide) and in an area with no property immediately upstream, so that flood risk and potential impacts are limited as far as possible through the placement of the proposed development. It is therefore assumed that the Sequential Test has been passed.

164) As supporting infrastructure for a water compatible development, the proposed Ribble Crossing is considered suitable within Flood Zone 3b. However, the principles of the Exception Test have also been applied. As the Proposed Ribble Crossing is required as part of the wider development to secure safe drinking water for properties across the North West England, the first element of the test is assumed to be passed. This FRA provides the evidence to support the second element of the Exception Test, i.e. the development would be safe from flooding and not increase flood risk elsewhere.

165) The assessment of flood risk is largely based on detailed hydraulic modelling of the River Ribble and Ordinary Watercourses that would be crossed. However, the assessment of other sources is conceptual and based on national scale datasets and a conceptual understanding of flood mechanisms. As the design life of the Proposed Ribble Crossing is approximately seven years, the impacts of climate change have not been considered as part of the assessment and all risks and impacts are based on present day hydrological conditions.

166) Table 5 provides a summary of the assessment of flood risk. This assessment has considered the embedded mitigation contained (e.g. flood design standards) within the design process and good practice that would be applied during the construction and operational phases of the Proposed Ribble Crossing. Where additional mitigation over and above that embedded into the design or covered by good practice, this has been identified.

Table 5: Flood risk assessment summary

Flood Assessment	Fluvial (Main River)	Fluvial (Ordinary Watercourse)	Surface Water	Groundwater	Reservoir
Flood Risks	High	High	Low	Low	Low
Flood Risk Impacts	Negligible	Moderate	Negligible	Negligible	Negligible
Additional Mitigation	No	Yes	No	No	No

167) Several elements of the Proposed Ribble Crossing would have a high probability of fluvial flooding with elements of the construction works within an area that is predicted to flood from the River Ribble during the 50 % AEP flood event and sections of track that are likely to flood during the 3.33 % AEP flood event. It is likely that this fluvial flooding may cause disruption to the construction and operation of this development with the closure of the road likely to be required for periods. However, the mitigation measures embedded within the design of the Proposed Ribble Crossing and good practice measures detailed within the CCoP would enable the Proposed Ribble Crossing to be managed in a safe and effective manner during its design life.

168) Whilst adverse impacts are predicted, particularly at Ordinary Watercourse crossing locations these impacts would be largely localised within small areas of pastoral farmland with no vulnerable receptors adversely impacted. Potential impacts to a barn on the left bank of Coplow Brook have been identified, which would need to be mitigated through the detailed design of road drainage measures.

169) All impacts would limited to the approximately seven year design life of the Proposed Ribble Crossing and would therefore be temporary and reversible. In addition, most adverse impacts would occur in areas that are already subject to a high level of flood risk. Therefore, with the additional mitigation to address impacts to the barn adjacent to Coplow Brook in place, the overall impact on flood risk is considered to be negligible.

170) Risks from all other identified sources are generally low, except for localised areas with a high probability of surface water flooding. Surface water drainage associated with the compound sites and the road, together with the application of good practice, would ensure that the risk posed to these features and the temporary impacts of the development would be low.

5.2 Conclusion

171) This FRA has been supported by detailed hydraulic modelling and confirms that based on the proposed design and the good practice mitigation included within the CCoP, the Proposed Ribble Crossing would be safe from flooding and would not increase the risk of flooding elsewhere. Therefore, the Proposed Ribble Crossing fully meets the requirements of the NPPF and local policy and guidance with regard to the potential impacts of the development on fluvial flood risk.

Annexe A: EIA Assessment Criteria

A.1 Baseline Sensitivity

The baseline sensitivity for flood sources considers the:

- Probability (likelihood) of flooding from the flood source considered e.g. Main Rivers, Ordinary Watercourses, groundwater etc. (the primary receptor) using probability values used by the Environment Agency on flood zone data; and
- Consequences of flooding as indicated by the vulnerability of receptors at risk (property, infrastructure, agricultural land etc.) using vulnerability classifications within NPPF.

Table A-1: Baseline sensitivity criteria

Sensitivity Importance	Criteria
Low	▪ Fluvial - Land having a less than 0.1 % AEP of river flooding (Flood Zone 1) ▪ Surface water - Land having between 1 % and 0.1 % AEP of flooding from surface water ▪ Groundwater - areas with limited potential for groundwater flooding to occur ▪ Artificial infrastructure - Areas at risk of flooding from failures of water infrastructure or ▪ Land use that is defined within the NPPF as water compatible.
Medium	▪ Fluvial- Land having between 1 % and 0.1 % AEP of river flooding (Flood Zone 2) ▪ Surface water - Land having between a 1 % and 3.3 % AEP of flooding from surface water ▪ Groundwater - Areas with potential for groundwater flooding to receptors situated below ground level ▪ Land use including productive farmland or unclassified roads.
High	▪ Fluvial - Land having a greater than 1 % AEP of river flooding (Flood Zone 3) ▪ Surface water - Land having a greater than 3.3 % AEP of flooding from surface water ▪ Groundwater - Areas with potential for groundwater flooding to occur at surface level or ▪ Land uses classified as Less Vulnerable within the NPPF or local transport networks and infrastructure.
Very High	▪ Fluvial – Land where water has to flow or be stored in times of flood, referred to as Functional Floodplain (Flood Zone 3b) ▪ Land uses classified as Essential Infrastructure; More Vulnerable; or Highly Vulnerable; or where the increase in flood risk would result in a risk to life (i.e. a flood hazard that is dangerous for all).

A.2 Magnitude of Change Criteria

The magnitude of change is a measure of the scale or extent of the change in the baseline condition, irrespective of the value of the resource(s) affected. However, flood risk can be influenced by several factors, including:

- Potential changes associated with the source of flooding linked to a change (or combination in changes) in run-off/higher discharge, flood storage volume, conveyance, flood frequency, depth/extent, velocity and/or peak flow

- Temporal changes to flooding such as permanent or temporary changes such as those that would be limited in duration to the construction period and those that would remain for the full duration of the operational life of the development
- "Embedded" mitigation measures that form part of an optimised design used to manage the likely significant flood risk effects.

The magnitude of change has been determined based on the factors listed above, the data available for flood sources and the criteria set within Table A-2. The term "Magnitude" of effects has been used to describe the severity of impacts within both the FRA and the Environmental Statement.

The overall baseline sensitivity was determined by the availability of data to determine probability for all flood sources and the potential for multiple receptors to be at risk. Where there was uncertainty regarding whether a receptor would be at risk, a precautionary approach was taken.

Table A-2: Magnitude of change criteria

Magnitude	Criteria
Major	A large adverse or beneficial change in flood depth, flood extent, velocity, or peak flow, that may have an impact some distance upstream or downstream. Potential to significantly change flood frequency. Potential change in risk to life. A large adverse or beneficial change in groundwater levels and flows which would affect groundwater flooding susceptibility over catchment scale.
Moderate	A moderate adverse or beneficial change in flood depth, flood extent or peak flow that may have limited impact some distance upstream or downstream. Potential for some change in flood frequency. Minor changes in floodplain flow pathways that increase velocity or extent of flooding but does not lead to new areas being inundated or new flow pathways forming. A moderate adverse or beneficial change in groundwater levels and flows which would affect groundwater flooding susceptibility over catchment scale or a large adverse or beneficial change in groundwater levels and flows which would affect groundwater flooding susceptibility over local scale.
Minor	A small or very localised adverse or beneficial change in flood depth, extent or peak flow with no perceptible impact upstream or downstream or in the floodplain. Small changes in flood frequency. A small adverse or beneficial change in groundwater levels and flows which would affect groundwater flooding susceptibility over catchment scale or a moderate adverse or beneficial change in groundwater levels and flows which would affect groundwater flooding susceptibility over local scale.
Negligible	Very limited potential for change. No change in flood frequency.

A.3 Significance of Impacts

The Significance of the overall flood risk is a product of the likelihood (sensitivity/value) and the magnitude of the impacts. Should the overall significance of flood risk be classified as Moderate, Large or Very Large, then additional mitigation would be required. Any effects that cannot be mitigated would be recorded as residual effects.

The overall risk of flooding during the construction and operational phases is a product of the likelihood of occurrence and the severity of impact as indicated in Table A-3.

Table A-3: Significance of flood risk Impacts

		Magnitude of Impact			
		Negligible	**Minor**	**Moderate**	**Major**
Baseline Flood Risk	**Low**	Neutral	Neutral	Slight	Moderate/Large
	Medium	Neutral	Slight	Moderate	Large
	High	Neutral	Slight/Moderate	Moderate/Large	Large/Very Large
	Very High	Neutral	Moderate/Large	Large/Very Large	Very Large

Annexe B: Figures

Figure 1 – Proposed Ribble Crossing Location

Figure 2 – The Flood Map for Planning

Figure 3 – The Risk of Flooding from Surface Water Map

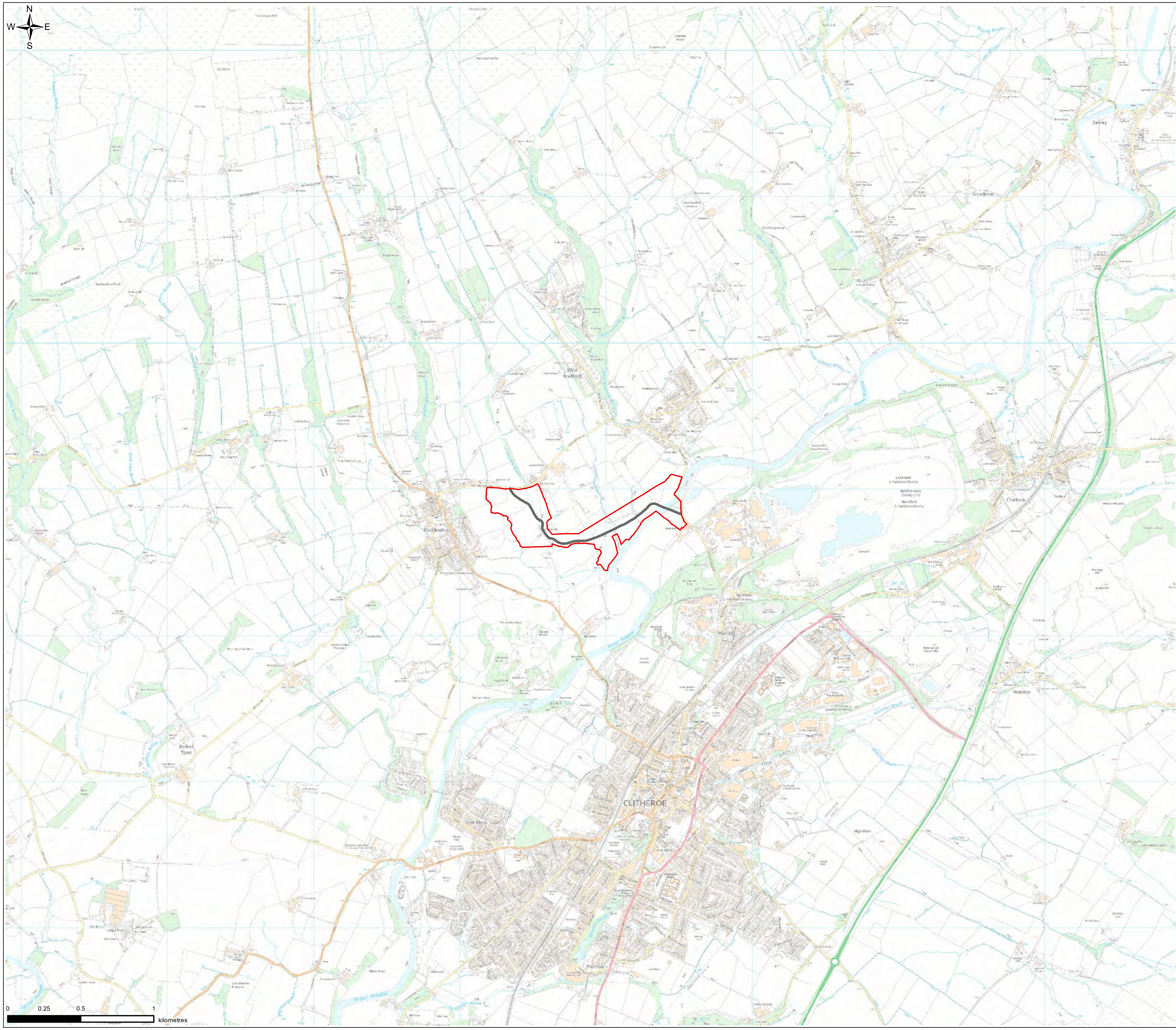
Figure 4 – Risk of flooding from Reservoirs Map

Drawing 80061155-01-JAC-TR3-97-DR-C-00009 - PROPOSED RIBBLE CROSSING BRIDGE GENERAL ARRANGEMENT AND ELEVATIONS

Drawing 80061155-01-JAC-TR3-97-DR-C-00006 - Highways Works Proposals (RIBBLE CROSSING HAUL ROAD - Sheet 1 of 2)

Drawing 80061155-01-JAC-TR4-97-DR-C-00008 - Highways Works Proposals (RIBBLE CROSSING HAUL ROAD - Sheet 2 of 2)

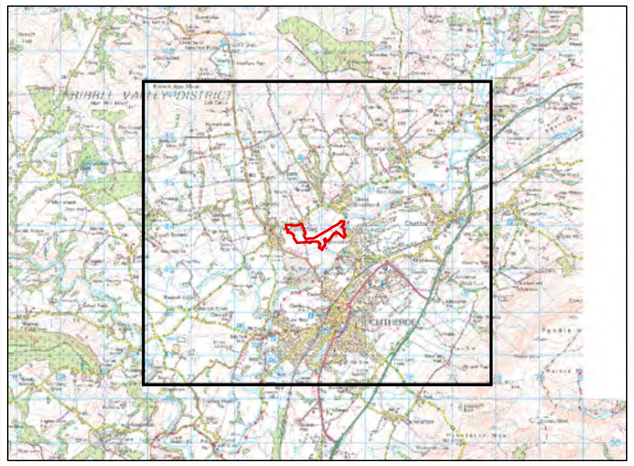
FIGURE 1



Legend

Planning Application Boundary

Indicative Route Alignment



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VERSION	AUTH	CHKD	REVD	REASON FOR ISSUE	DATE

United Utilities

Water for the North West

UNITED UTILITIES WATER LIMITED
HAWESWATER AQUEDUCT RESILIENCE PROGRAMME
SITE LOCATION
PROPOSED RIBBLE CROSSING
PAGE 1 OF 1

SCALE	SHEET SIZE
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DRAWING NUMBER	REVISION
LCC_RVBC-BO-RC-FIG-01	0

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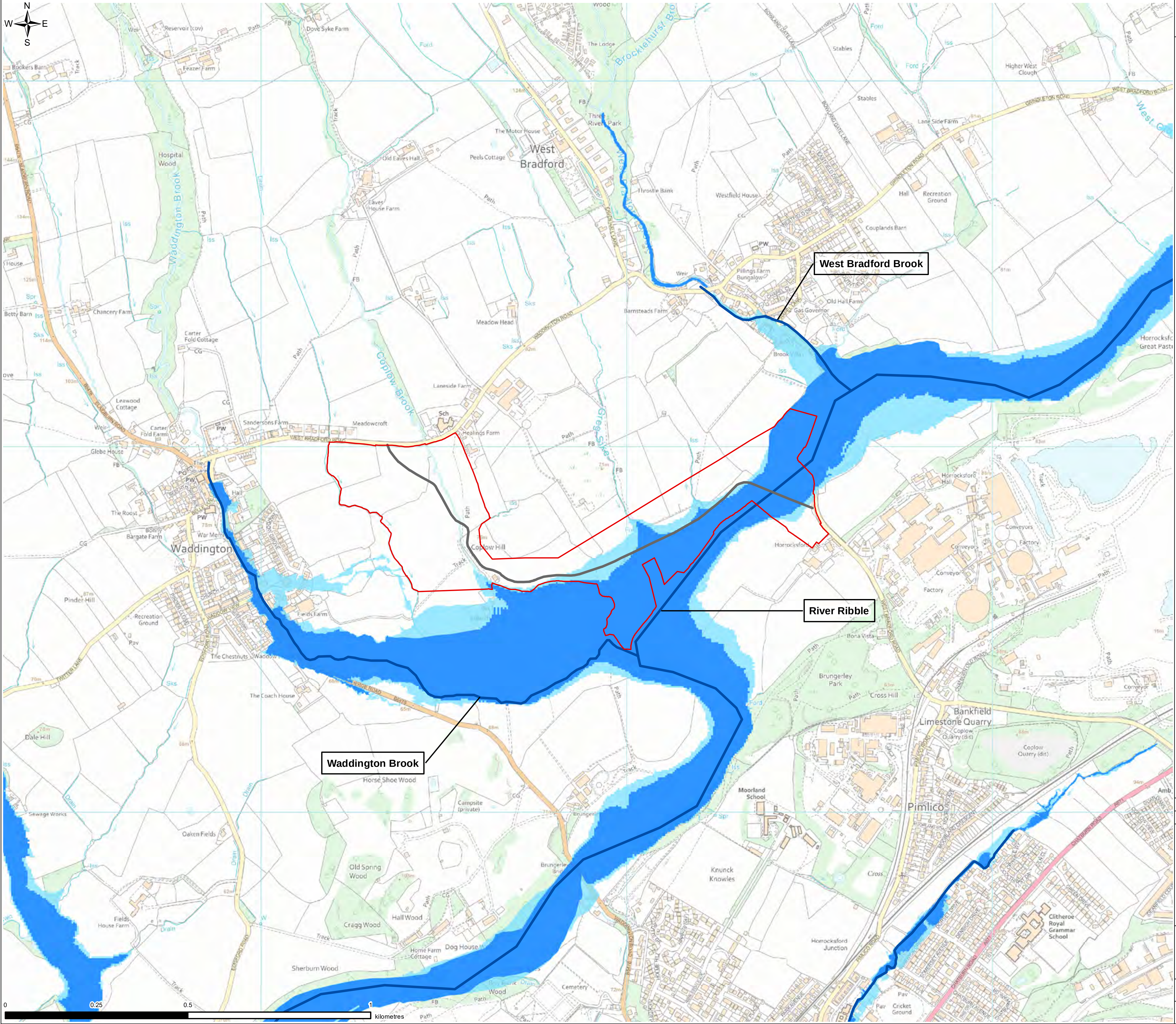
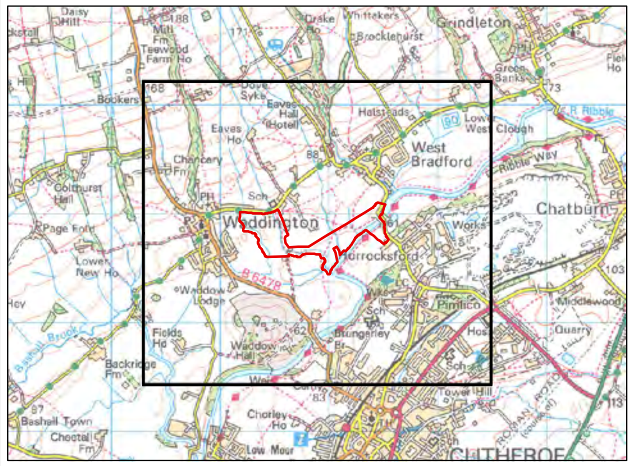
FIGURE 2



Legend

- Planning Application Boundary
- Indicative Route Alignment
- Main Rivers
- Flood Zone 2
- Flood Zone 3



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HAWESWATER AQUEDUCT RESILIENCE PROGRAMME
FLOOD MAP FOR PLANNING
PROPOSED RIBBLE CROSSING
PAGE 1 OF 1

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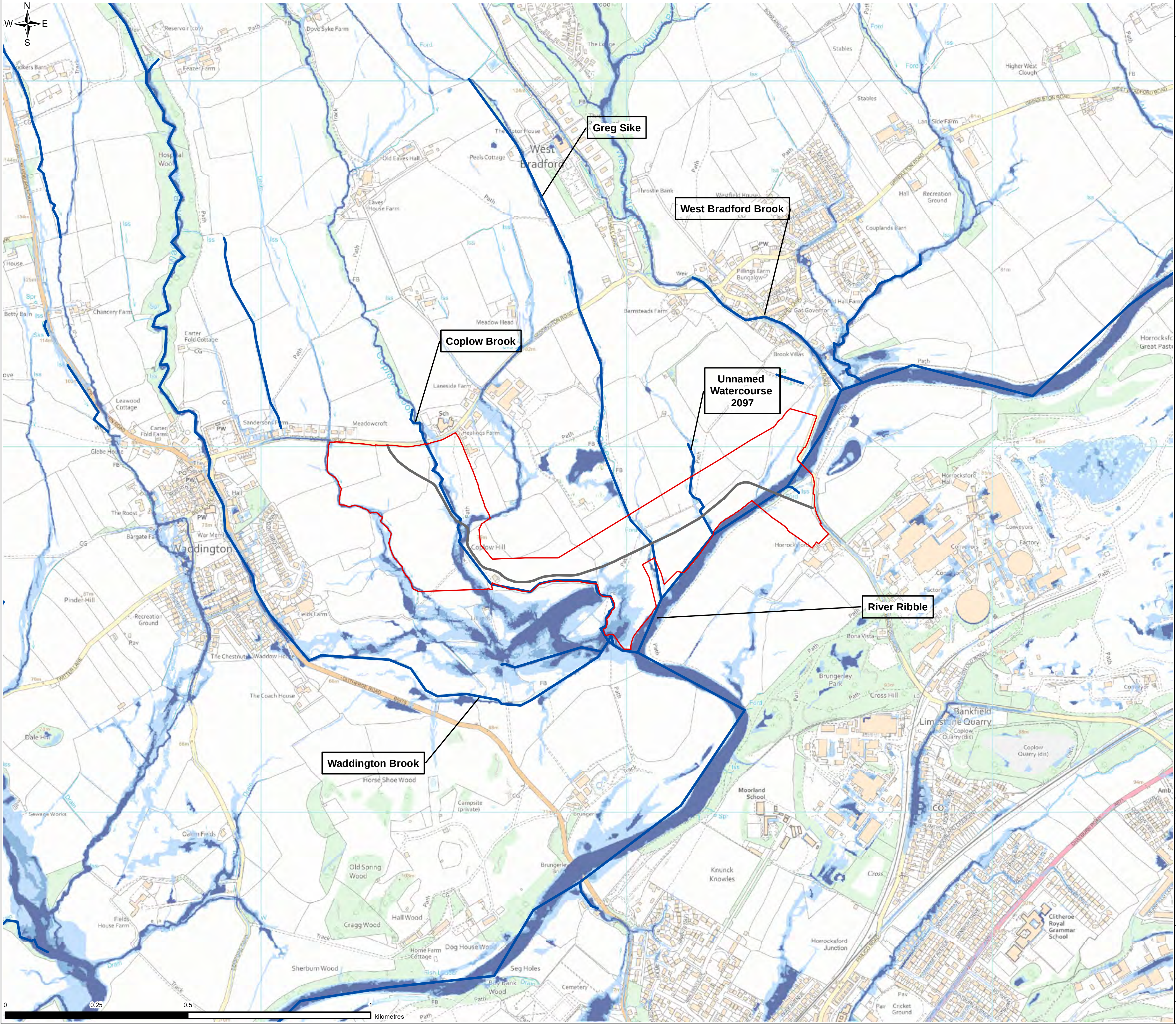
FIGURE 3

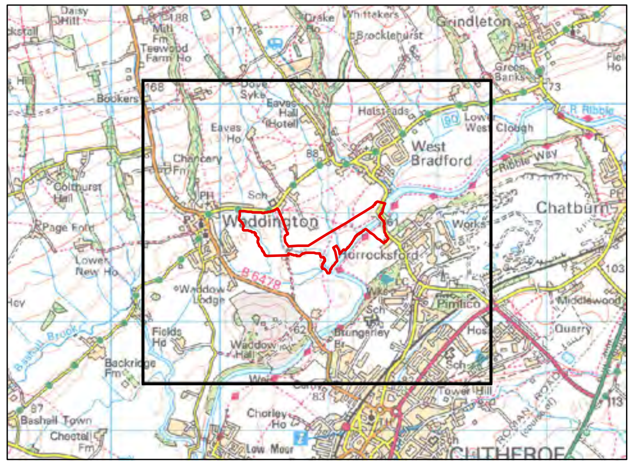


Legend

- Planning Application Boundary
- Indicative Route Alignment
- Watercourses

Risk of Flooding from Surface Water

- Surface Water Flood Extent 3.33% AEP
- Surface Water Flood Extent 1% AEP
- Surface Water Flood Extent 0.1% AEP



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HAWESWATER AQUEDUCT RESILIENCE PROGRAMME
RISK OF FLOODING FROM SURFACE WATER
PROPOSED RIBBLE CROSSING
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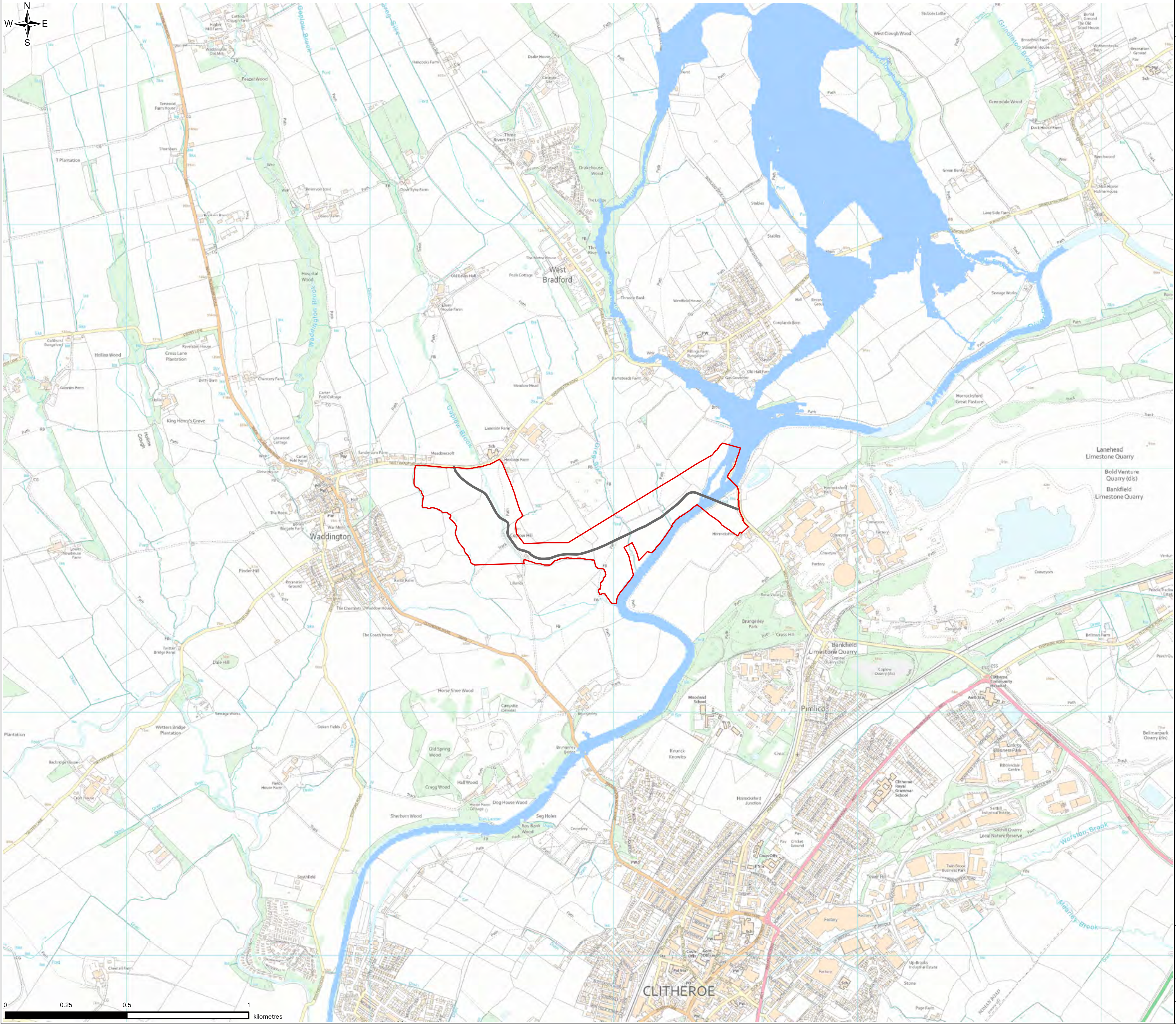
FIGURE 4

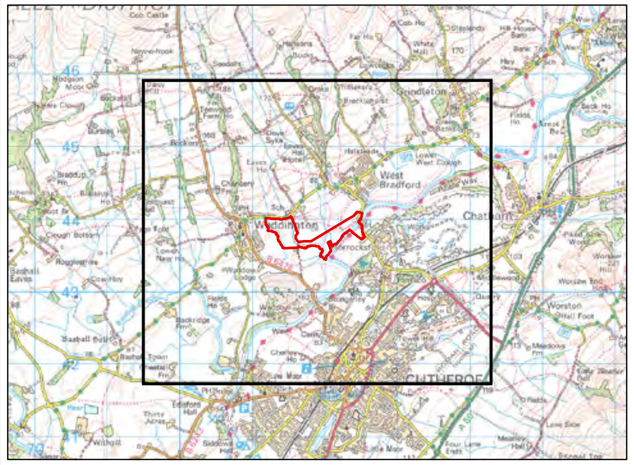


Legend

- Planning Application Boundary
- Indicative Route Alignment

Risk of Flooding from Reservoirs

- Maximum Extent of Flooding



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RESERVOIR FLOOD EXTENT
PROPOSED RIBBLE CROSSING
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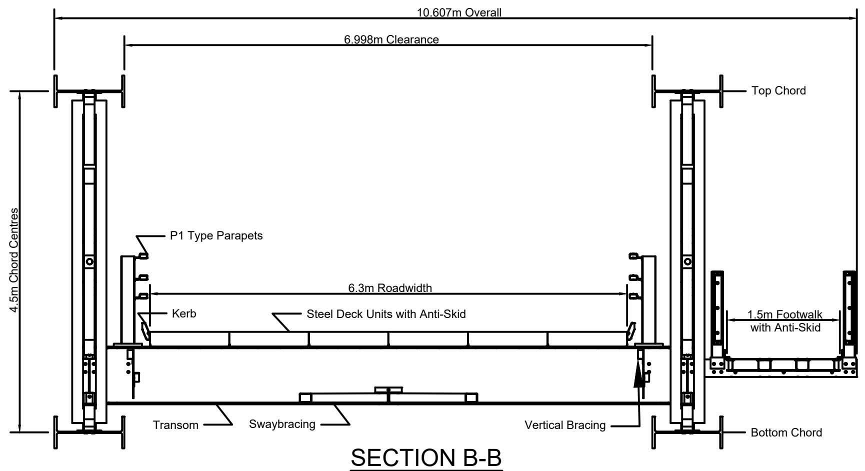

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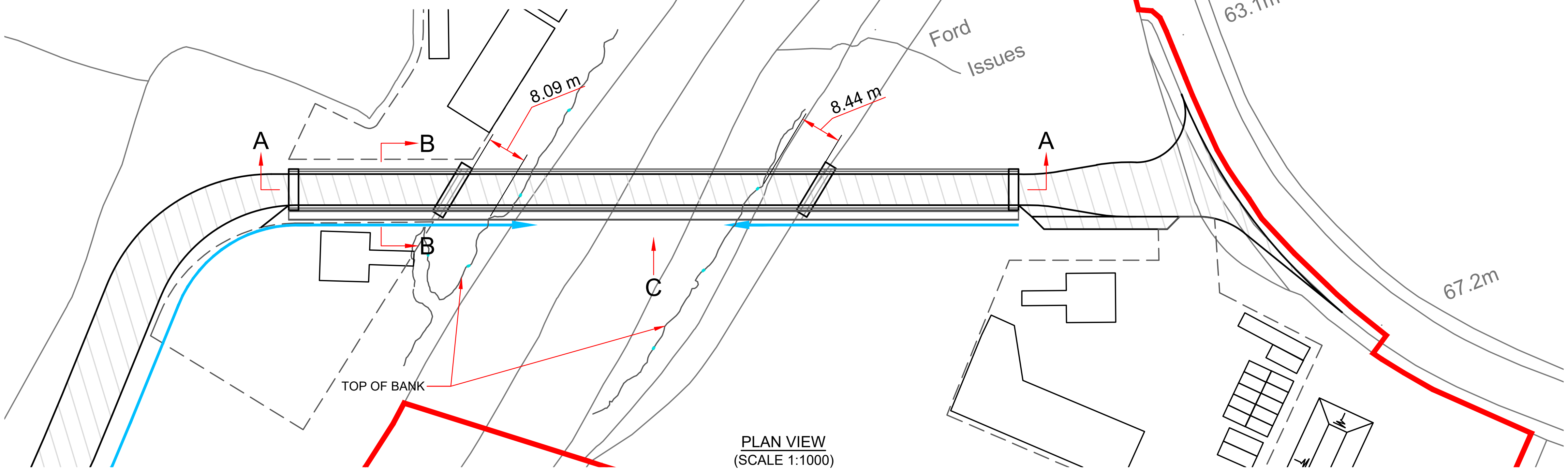

PLAN VIEW
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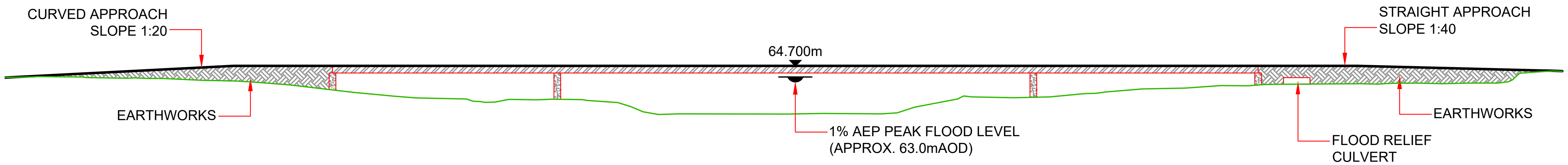

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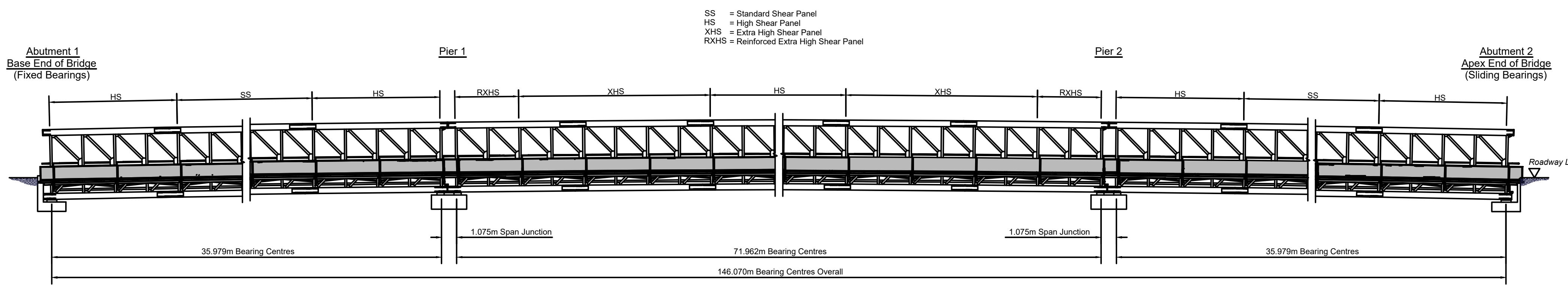

ELEVATION C
(SCALE 1:150)

NOTES

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2. TEMPORARY BRIDGE BASED ON INDICATIVE SUPPLIER INFORMATION (MABEY BRIDGE DRG 24813-24651-MAB-DEL-001-DR-CD-101-P01)

OS MAP REFERENCE

SD7343

LEGEND

SITE APPLICATION BOUNDARY

ACCESS ROAD SURFACE WATER DRAINAGE

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P03	PC	WB	WB	Issued for Planning Application	07/12/21

SUITABILITY CODE	SUITABILITY DESCRIPTION
S2	Issued for Information

United Utilities
Water for the North West

UNITED UTILITIES WATER COMPANY
HAWESWATER AQUEDUCT RESILIENCE PROGRAMME
RIBBLE VALLEY BOROUGH COUNCIL
(BOWLAND SECTION)
PROPOSED RIBBLE CROSSING BRIDGE
GENERAL ARRANGEMENT AND ELEVATIONS

SCALE: AS SHOWN	SHEET SIZE: A3
DRAWING NUMBER: 80061155-01-JAC-TR3-97-DR-C-00009	REVISION: P03

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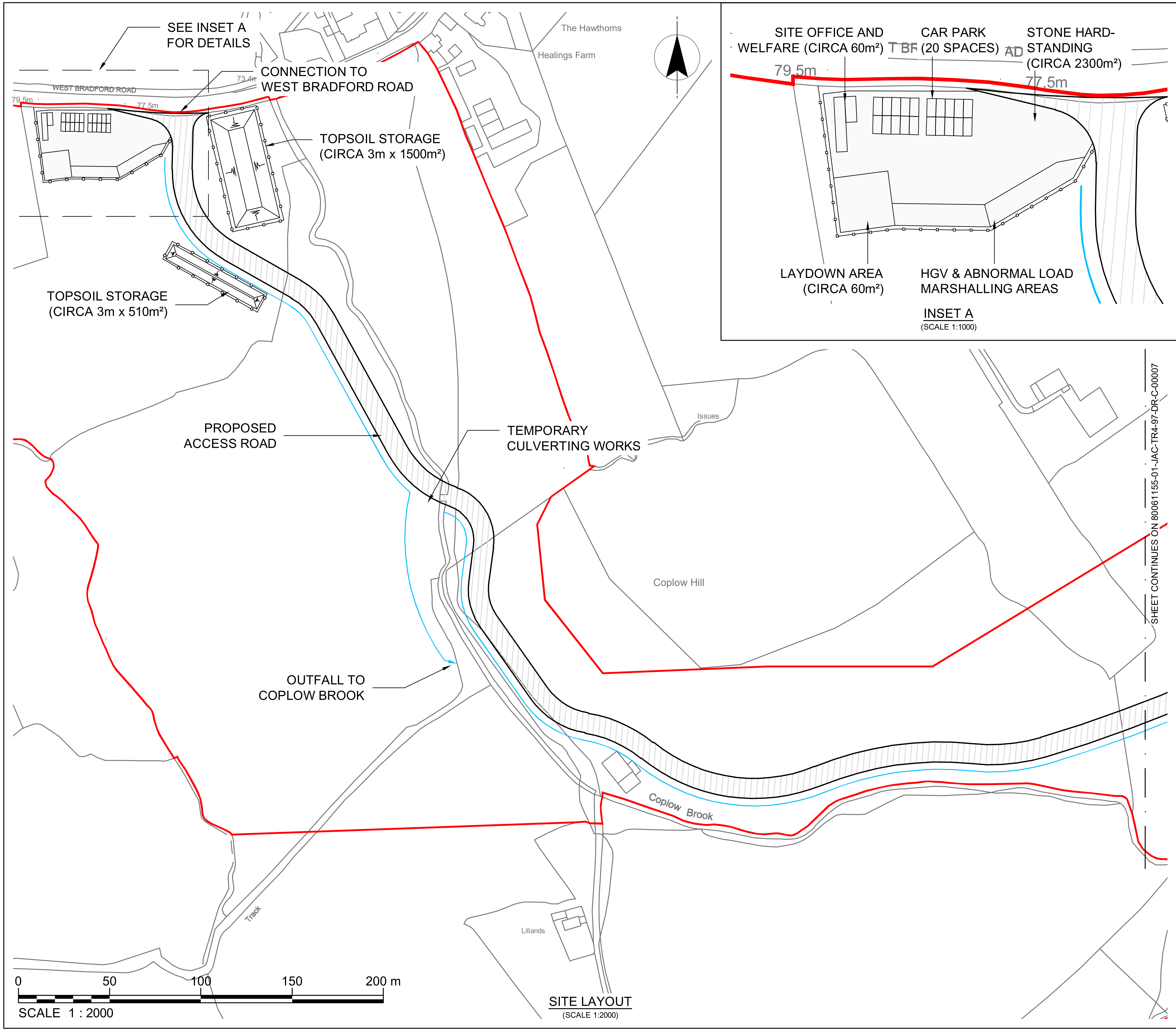

SITE LAYOUT

(SCALE 1:2000)

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Water for the North West

UNITED UTILITIES WATER LIMITED
HAWESWATER AQUEDUCT RESILIENCE PROGRAMME
RIBBLE VALLEY BOROUGH COUNCIL
(BOWLAND SECTION) PROPOSED TEMPORARY
RIBBLE CROSSING HAUL ROAD
SHEET 1 OF 2

SCALE	SHEET SIZE
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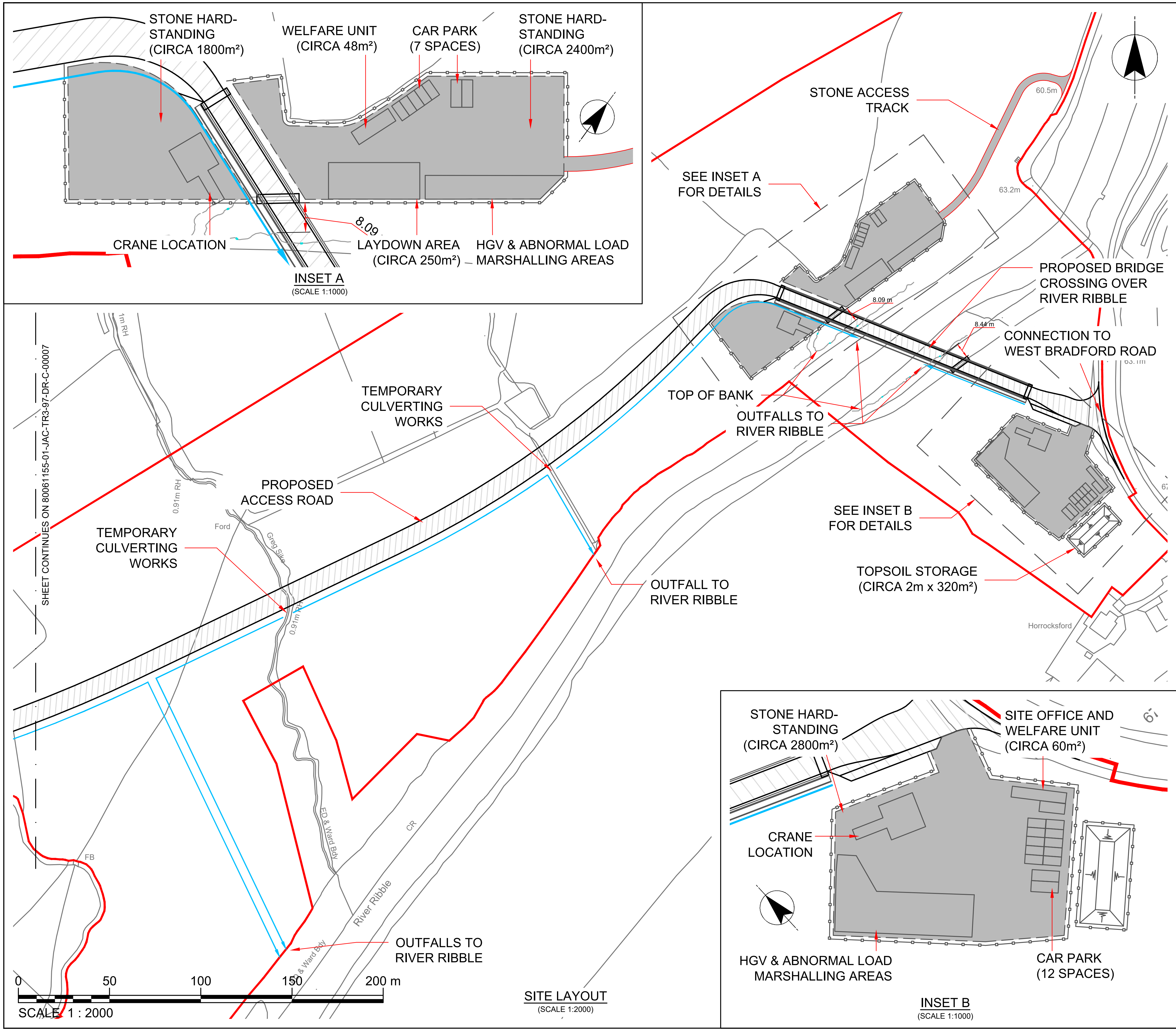

NOTES

1. ALL DIMENSIONS AND LEVELS IN METRES AOD UNLESS NOTED OTHERWISE.

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RIVER CROSSING DETAIL DRAWING NO. 80061155-01-JAC-TR3-97-DR-C-00009

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UNITED UTILITIES WATER COMPANY
HAWESWATER AQUEDUCT RESILIENCE PROGRAMME
RIBBLE VALLEY BOROUGH COUNCIL
(BOWLAND SECTION) PROPOSED TEMPORARY
RIBBLE CROSSING HAUL ROAD
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Annexe C: Hydraulic Modelling Report [21]

[21] Jacobs (2021). B27070CT HARP River Ribble Hydraulic Modelling Report.

Jacobs

Haweswater Aqueduct Resilience Programme - Proposed Bowland Section

Appendix 4: Flood Risk Assessment

Annexe C: Hydraulic Modelling Report - Proposed Ribble Crossing

February 2022

United Utilities
Water for the North West

Haweswater Aqueduct Resilience Programme - Proposed Bowland Section

Project No:	B27070CT
Document Title:	Proposed Bowland Section – Appendix B4: Flood Risk Assessment Annexe C: Hydraulic Modelling Report – Proposed Ribble Crossing
Document Ref.	LCC_RVBC-BO-TA-008-B4
Revision:	0
Date:	February 2022
Client Name:	United Utilities Water Ltd

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Contents

1. Introduction

1.1 Purpose of the report

1) United Utilities proposes to construct a temporary haul road and a temporary crossing of the River Ribble, near Clitheroe, to service the Proposed Bowland and Marl Hill Sections of the Haweswater Aqueduct Resilience Programme (HARP). The design life for the temporary works is estimated to be five years.

2) A Flood Risk Assessment (FRA) is required to meet relevant local and national planning legislation and inform the design and planning process. The FRA was submitted with the June 2021 Environmental Statements and planning applications for the Proposed Bowland Section and Proposed Marl Hill Sections of HARP.

3) This hydraulic modelling report supports an updated FRA, prepared as part of a Supplementary Environmental Information (SEI) report in January 2022. The updated FRA now provides the results of the detailed hydraulic modelling work undertaken of the River Ribble and its tributaries replacing the qualitative assessment of fluvial flood risk.

4) Computational hydraulic model was undertaken of the River Ribble with associated catchment hydrology. The impact of the Proposed Ribble Crossing on water level both upstream and downstream and the associated flood extents was determined for a range of storm flood events.

5) To inform the design of the temporary bridge crossing, hydraulic modelling was required to:

- Define baseline (existing situation) flood extents and set peak flood design levels
- Test the performance of outline bridge design options to inform the design process
- Provide information required to support the future permitting application.

6) This report details the methodology and the results of the hydraulic modelling carried out for the River Ribble, to assess the baseline situation and the consequences of the temporary works and any flood risk mitigation measures required. The haul road is also required to cross three small Ordinary Watercourses, which are tributaries on the right (north) side of the Ribble valley. These comprise: Coplow Brook, Greg Syke and an unnamed watercourse. The unnamed watercourse is referred to as 'Unnamed Watercourse 2097' in the FRA but is referred to throughout the remainder of this document as Unnamed Watercourse. Assessment of the flood risk impact of the proposed works at these locations is also presented.

7) Throughout this report, reference is made to 'the Proposed Ribble Crossing', which encompasses the proposed works covered by this assessment, including coring of the Ribble and the three Ordinary Watercourses. This is a technical report, focused on the hydraulic modelling, and therefore the intended audience is those with a reasonable understanding and knowledge of hydraulic modelling principles, although no specific knowledge of particular software is needed.

1.2 Methodology

8) River modelling was carried out using a linked one-dimensional/two-dimensional (1D/2D) schematisation, where the river channel is represented as a 1D component and is linked to the floodplain, represented by a 2D domain. The river modelling package Flood Modeller version 4.6 (Jacobs, 2021) was used for the 1D component and TUFLOW version 2018-03-AE-iDP-w64 (BMT WBM, 2018) for the 2D component.

9) A separate model was developed for each watercourse, i.e. one model for the main River Ribble, and three separate models for the three key tributaries that are crossed by the Proposed Ribble Crossing; Coplow Brook, Greg Syke and the Unnamed Watercourse.

1.3 Study Area

10) The River Ribble arises in the Yorkshire Dales and runs south over 80 km to Clitheroe, and from there it continues for another 30 km before reaching the sea to the west of Preston. The section of the River

Ribble under investigation in this study is approximately 3 km long and flows in a south-westerly direction (Figure 1-1).

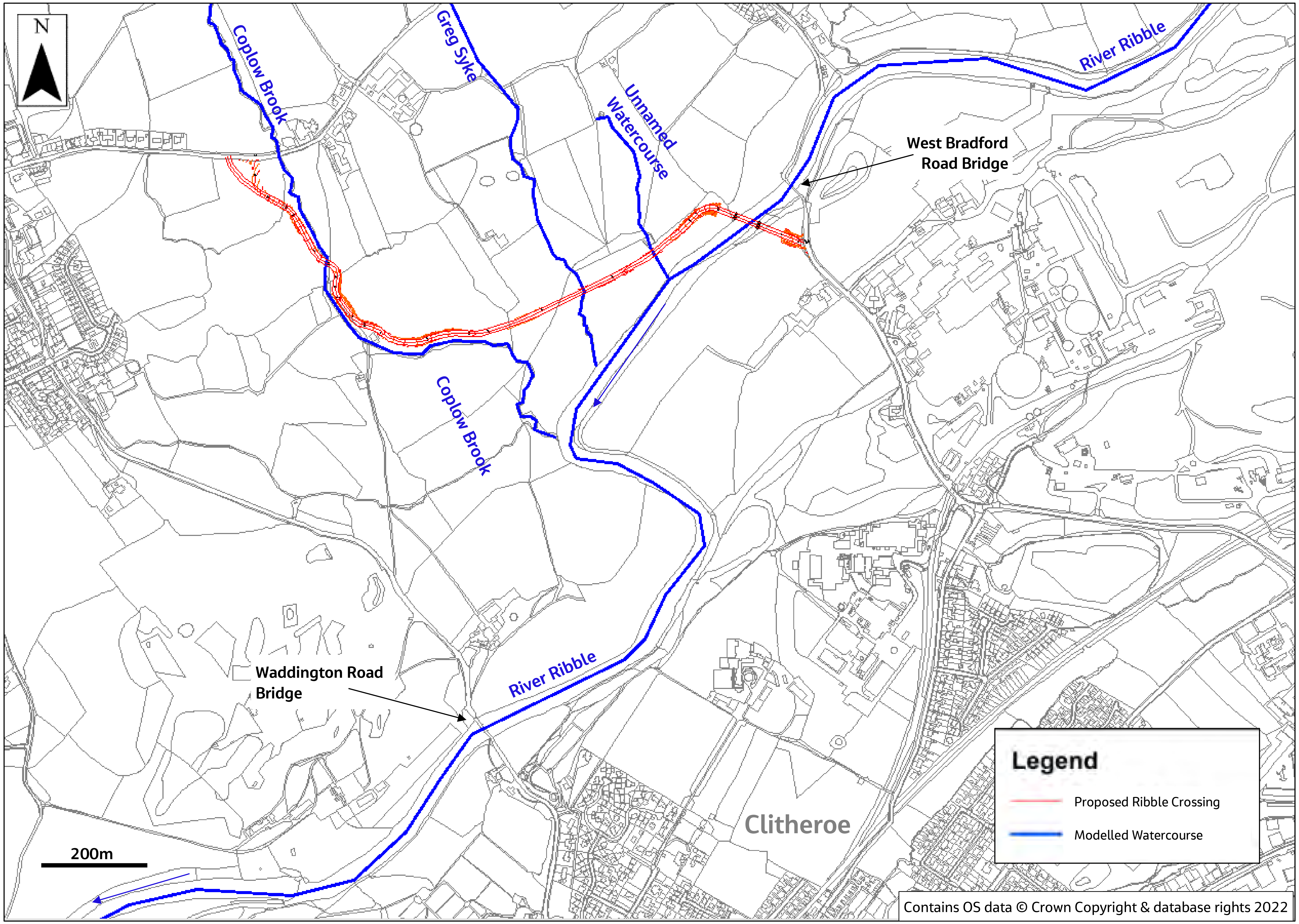


Figure 1-1: Modelled reaches of River Ribble and tributaries

2. Input Data

11) The data used to construct the hydraulic models for the River Ribble and tributaries is summarised in Table 2-1.

Table 2-1: Data used to build the hydraulic model.

Data	Description	Source
LiDAR	1m resolution filtered Digital Terrain Model (DTM) from LiDAR (Light Detection and Ranging) data. Used to inform the hydraulic model (2D floodplain) with ground level information.	Defra 2021
OS Mastermap	Land use data. Used to specify roughness values across the 2D floodplain model.	United Utilities 2021
Channel Survey	Topographic survey of In-channel cross-sections and hydraulic structures. Used for 1D model representation of the channel.	United Utilities 2021
Site Photographs	Photographs taken upstream and downstream of each surveyed cross-section.	United Utilities 2021
Outline Design Drawings	Drawings of the Proposed Ribble Crossing design.	United Utilities 2021
Flood Zone Mapping	EA Flood Zone 2 flood maps.	Environment Agency 2021

3. Hydrology

3.1 Background

12) To inform the design of the temporary bridge crossing of the River Ribble, hydrological analysis of the River Ribble catchment is required to derive estimates of peak flood flow at six flood estimation points (FEPs) for the following range of flood events:

- 50 % AEP (Annual Exceedance Probability) (1 in 2)
- 20 % AEP (1 in 5)
- 10 % AEP (1 in 10)
- 3.33 % AEP (1 in 30)
- 1.33 % AEP (1 in 75)
- 1 %AEP (1 in 100).

13) Design flood hydrographs were determined for the six FEPs:

- Ribble Upstream at West Bradford Road Bridge
- Ribble Downstream at Waddington Road Bridge
- Wadddington Brook
- Coplow Brook
- Greg Syke
- Unnamed Watercourse.

14) Figure 3-1 shows the modelling extent (i.e. study area) and flow estimation locations, and Figure 3-2 shows the hydraulic model schematisation and required inflow locations.

15) Table 3-1 lists the subject FEPs (shown in Figure 3-2) alongside their contributing upstream catchment area.

16) The modelled catchment inflows form part of the wider River Ribble catchment. Underlying geology is comprised of moderate permeability Carboniferous Limestone overlain by Boulder Clay in valleys with intermittent Millstone Grit. Soils are typically slowly permeable loamy and clayey soils, with wet peaty upland soils present in the upper catchment. Land use is largely rural and agricultural land, particularly in the north and west. The lower catchment contains the settlements of Chatburn and Croft. There are also dispersed villages in the upper catchment, such as Settle, Long Preston and Hellifield.

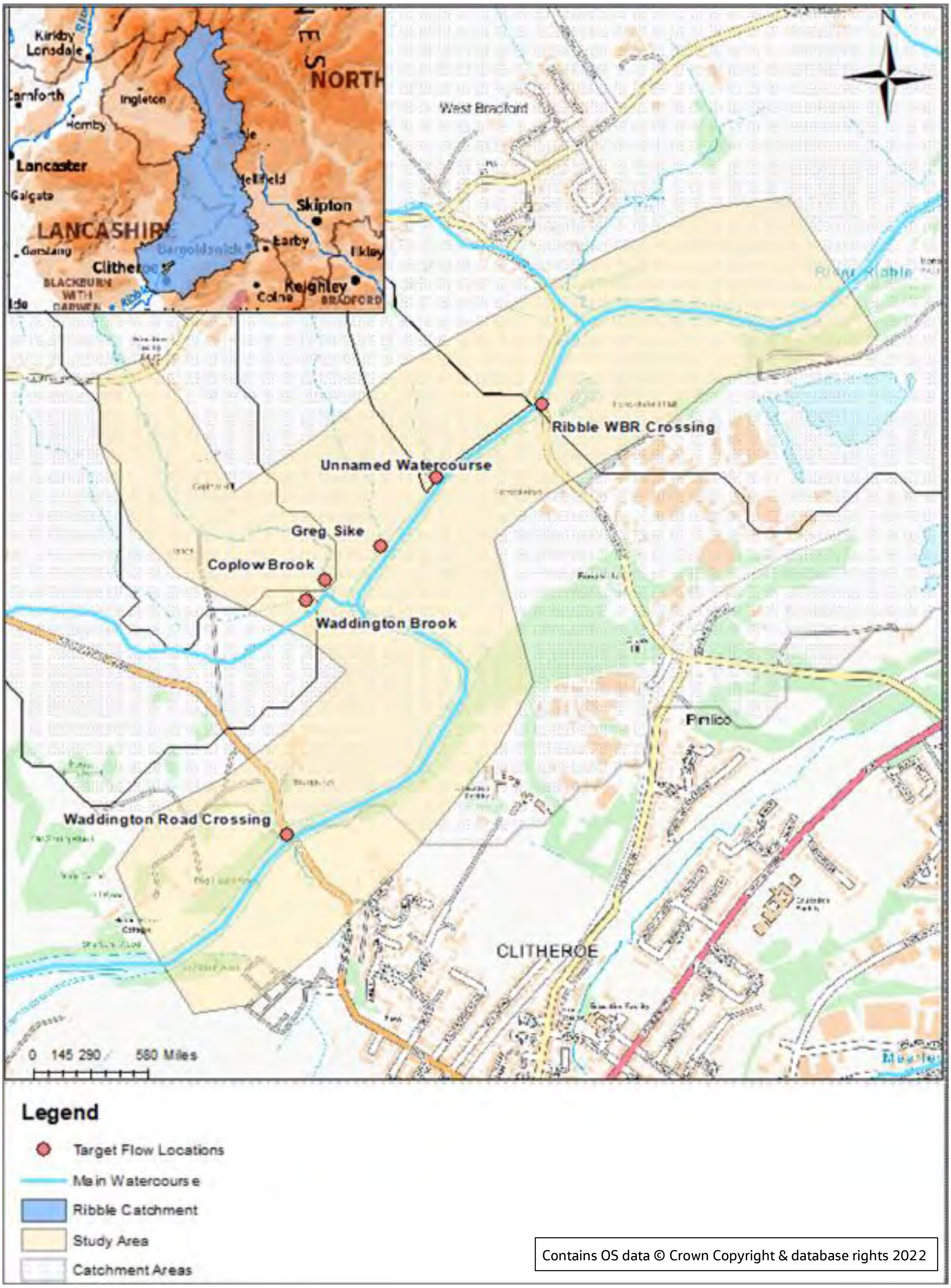


Figure 3-1: Location of the temporary bridge crossing, the proposed model extent and flow estimation points (in orange).

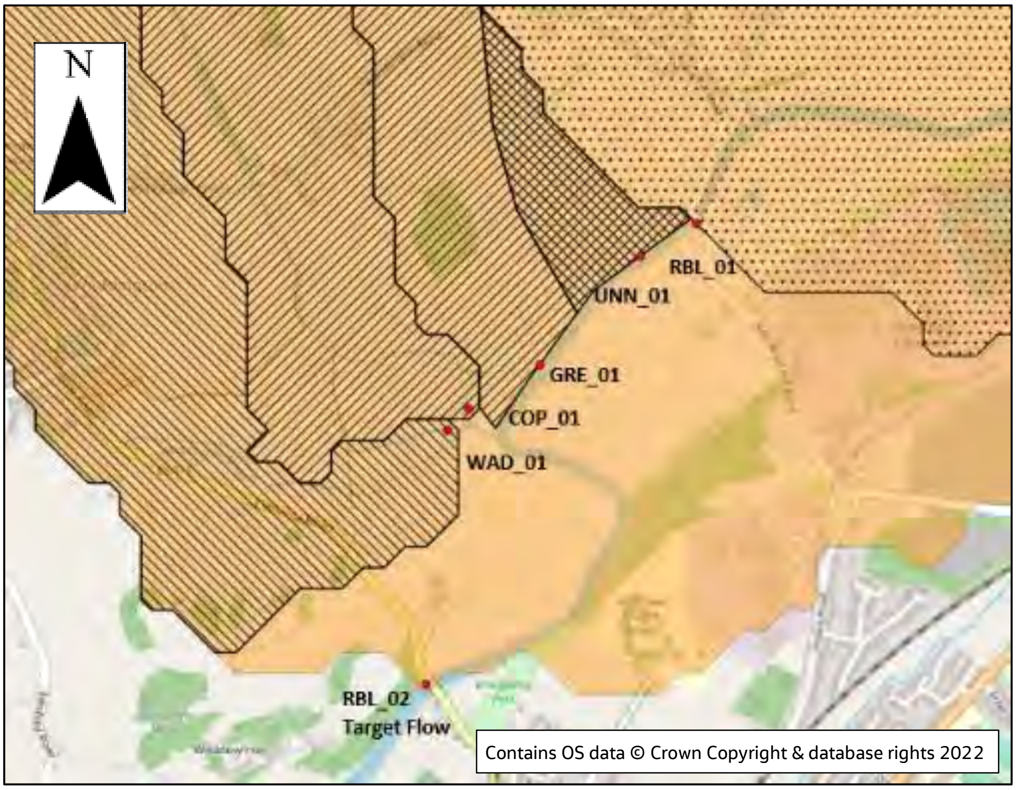


Figure 3-2: Required inflow locations.

Table 3-1: Summary of subject sites

FEP	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
RBL_01	S	R. Ribble	West Bradford Road	374500	443950	392.2	N/A
UNN_01	S	Unnamed Watercourse	Outflow to Ribble	374234	443763	N/A	0.12
GRE_01	S	Greg Sike	Outflow to Ribble	374090	443586	N/A	0.62
COP_01	S	Coplow Brook	Coplow / Waddington Confluence	373950	443500	1.39	N/A
WAD_01	S	Waddington Brook	Coplow / Waddington Confluence	373900	443450	3.79	N/A
Res_01	S	Residual catchment	Residual catchment	N/A	N/A	N/A	1.25
RBL_02	L	R.Ribble	Waddington Road	373850	442850	399.4	N/A

3.2 Methodology

17) The scope of work included estimation of design peak flood flows using two methods (FEH Statistical and ReFH2). Flows derived by either method for the River Ribble are shown to be generally consistent, particularly at the 1 % AEP event. For the tributary inflows, flows derived by the ReFH2 method are significantly lower than are derived by the FEH Statistical method.

18) For deriving the upstream inflow and the downstream target flow for the River Ribble, the FEH Statistical single site method as undertaken at Station 71006 was adopted. For the tributary inflows, the ReFH2 design hydrographs are scaled to the higher peak flow estimates derived by the FEH Statistical method and have been reconciled to agree with the flood estimates derived by the FEH Statistical method at RBL_02.

19) The scope of work included deriving design model inflows and hydrographs based on the 'theoretical' critical storm duration for each of the assessed AEP events. The critical duration is defined here as that which gives the highest flow at the flow estimation point and has been assessed through an iterative process whereby the storm duration was incrementally increased until peak flow was no longer observed to increase but rather decrease.

20) A single flow hydrograph representing residual flow was also calculated. Residual flow is defined as an inflow distributed across multiple river reaches as opposed to applying a point inflow. The hydrograph was distributed over the model reach according to reach length and accounted for an area of 3.1 km^2.

21) Assessment of the critical storm duration at the upstream and downstream model extent was identical and assessed as 15 hours. The storm duration for the residual catchment and the tributary inflows was determined to be 3.25 hours. The adjustment of the onset of the tributary flows was carried out in the subsequent hydraulic modelling in order to achieve concurrent peak timing and a worst-case scenario for flood risk analysis.

22) Two sets of runs were simulated to account for the differences in critical storm duration of the Ordinary Watercourses in comparison to the River Ribble. The runs are as follows:

- **Run 1** adopted a 15 hour storm duration. It is observed from Figure 3-3 that for the design hydrographs derived within ReFH2 the flood peak in the minor watercourses occurs at 10 hours whereas the design hydrograph for the River Ribble (RBL_01) peaks at approximately 24 hours, thus a difference in the time to peak of14 hours. Similarly, the hydrograph from ReFH2 for the River Ribble (not shown here) shows that the peak flow occurs at 15 hours, thus indicating the difference in time to peak of 5 hours.

  Therefore, for Run 1, the flow reconciliation at the downstream model extent has been achieved by aligning the hydrographs of the River Ribble and the minor watercourse by the difference in time to peak from 5 to 14 hours, and scaling down the minor watercourses inflows (as inflow hydrograph peaks are adopted from the statistical peaks) as required. Results of the un-reconciled simulation showed no scaling was required to obtain the target flow at RBL_02. Run 1 hydrology was used to assess the baseline and with-Proposed Ribble Crossing scenarios for the River Ribble only

- **Run 2** adopted the sub-catchment specific storm duration of 3.25 hours for the three Ordinary Watercourses, which were scaled to match with the statistical peaks. Run 1 50 %AEP (1 in 2) maximum water levels within the Ribble were extracted at each confluence with the three Ordinary Watercourses and applied within the Run 2 models to replicate a combined event. The 50 %AEP (1 in 2) Ribble water levels were chosen because the chances of a higher magnitude event in the Ribble peaking at the same time as its tributaries is highly unlikely when the critical storm durations are so different. Run 2 hydrology was used to assess the baseline and with-Proposed Ribble Crossing scenarios for the Ordinary Watercourses only.

23) Due to the temporary nature of the bridge and road structures, as per the scope of work, no allowance for climate change was required to be applied to the estimated design peak flood flows.

24) Run 1 Final flood estimates from the statistical method, based on the critical storm duration and areal reduction factor as calculated at RBL_02, are presented in Table 3-2, and the flood hydrographs for the 1 %AEP (1 in 100) storm event are plotted in Figure 3-3.

25) Full details of the hydrological analysis undertaken, and decisions made are contained within the Hydrology Calculation Record presented in Appendix A of this report.

Table 3-2: Final adopted flood estimates (Run 1).

Site code	Flood peak (m^3/s) for the following return periods (in years)					
	2	5	10	30	75	100
	Flood peak (m^3/s) for the following AEP (%) events					
	50	20	10	3.33	1.33	1
RBL_01	202	252	287	349	410	431
UNN_01	0.21	0.28	0.34	0.45	0.56	0.60
GRE_01	1.10	1.48	1.78	2.34	2.92	3.13
COP_01	2.46	3.32	3.99	5.24	6.54	7.02
WAD_01	6.67	9.00	10.8	14.2	17.7	19.0
Res_01	2.20	2.97	3.57	4.69	5.86	6.28
RBL_02	205	255	291	354	415	436

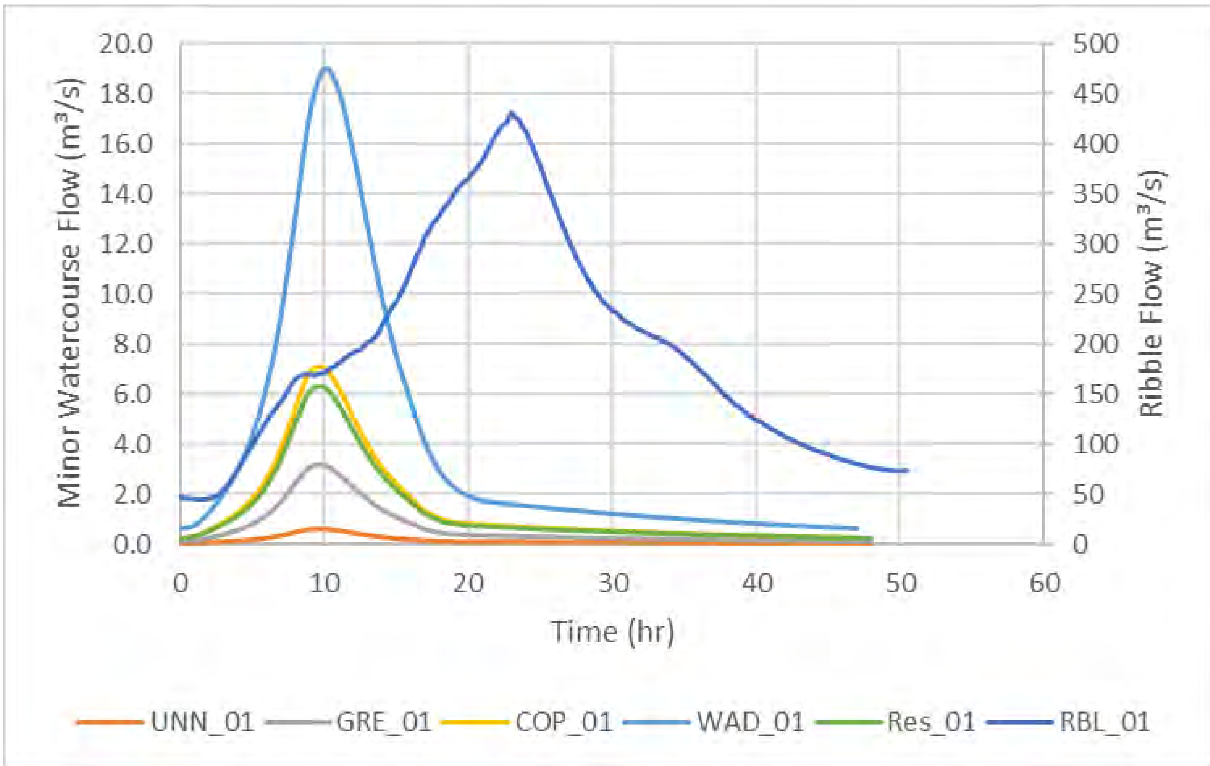


Figure 3-3: Rive Ribble model inflow hydrographs 1%AEP (1 in 100) event

4. Baseline Modelling

26) The following sections describe the schematisation of each separate model developed for this study.

4.1 Main River Ribble Watercourse Schematisation – 1D Domain

River Geometry

27) The 1D model covers a 2895 m reach of River Ribble. The surveyed cross-section data has been used to inform the in-channel geometry of the modelled watercourse. The locations of the surveyed cross-sections are shown in Figure 4-2.

28) Interpolated cross-sections were added between the surveyed cross-sections at the downstream end of the model to assist with model performance around a local change in bed level. In general the model cross sections are set at no more than 200 m spacing.

Hydraulic Friction

29) Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using site photographs taken during the survey. A uniform value of N = 0.04 was utilised for the in-channel roughness values, taken from standard guidance (Chow, 1959) as a high value for a clean straight channel with some stones, or mid value for a clean winding channel with some pools and shoals. Figure 4-1 shows a typical section of the River Ribble within the model domain.

Figure 4-1: Site visit photograph facing downstream, cross-section Ribb_1112 (location shown in Figure 4.5).

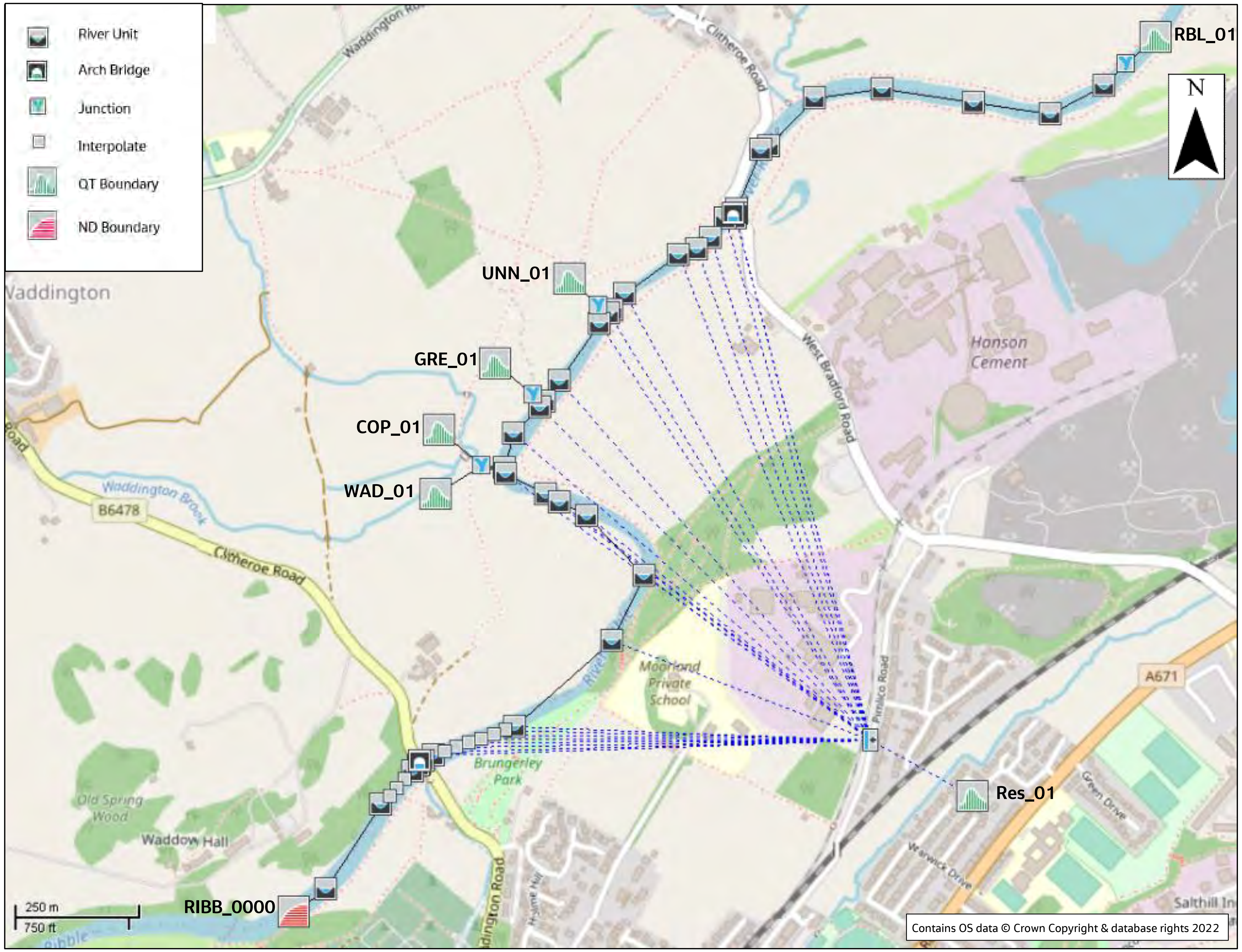


Figure 4-2: Baseline schematisation of River Ribble Model 1D domain and inflows.

Structures

30) Two hydraulic structures were included in the baseline model. Arch bridge units were used to represent the river crossings at West Bradford Road and Waddington Road, which are pictured in Figure 4-3 and Figure 4-4 respectively. The locations of the structures are shown in Figure 4-2.

Figure 4-3: Site photograph of West Bradford Road bridge (facing downstream).

Figure 4-4: Site photograph of Waddington Road bridge (facing downstream).

Boundary Conditions

31) The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4-1. Derivation of the hydrological boundaries are detailed in Section 3. Inflow locations are shown in Figure 4-2.

Table 4-1: Boundary conditions - 1D domain.

Type of Boundary	Flood Modeller Node	Description
QT boundary	RBL_01	Applied at the upstream end of the modelled reach of the River Ribble
QT boundary	UNN_01	Applied at the confluence with Unnamed Watercourse.
QT boundary	GRE_01	Applied at the confluence with Greg Syke
QT boundary	COP_01	Applied at the confluence with Coplow Brook
QT boundary	WAD_01	Applied at the confluence with Coplow Brook *
QT boundary	Res_01	Applied laterally along the main modelled reach of the River Ribble
Normal Depth Downstream Boundary	RIBB01_0000	Applied to the downstream end of the modelled reach of the River Ribble. A normal depth boundary calculates a flow-head relationship based on the channel characteristics. This downstream boundary type is considered suitable as there is no influence of downstream structures. The suitability of the downstream boundary is further discussed in Section 6.4.3.

*Waddington Brook outfalls into Coplow Brook just upstream from the outfall of Coplow into the River Ribble.

4.2 Main River Ribble Floodplain Schematisation – 2D Domain

32) The 2D domain covers an area of 1.8 km^2.

33) The topography of the floodplain is represented in the model using a 5 m resolution square grid. The levels for the grid cells are based on a DTM derived from 1 m resolution LiDAR data.

34) Breaklines were applied along two key road alignments where detailed topographic survey long sections had been collected. Breaklines (gully lines) were also applied to enforce flowpaths on the hillside tributaries. A review of the floodplain using available aerial and OS mapping has shown that there are no further structures within the floodplain that require representation in the model. Therefore, no other modifications were made to the LiDAR DTM.

35) Figure 4-5 shows the 2D model extent, links between the 1D and 2D model components and the land use type, which was used in the 2D representation of the floodplain.

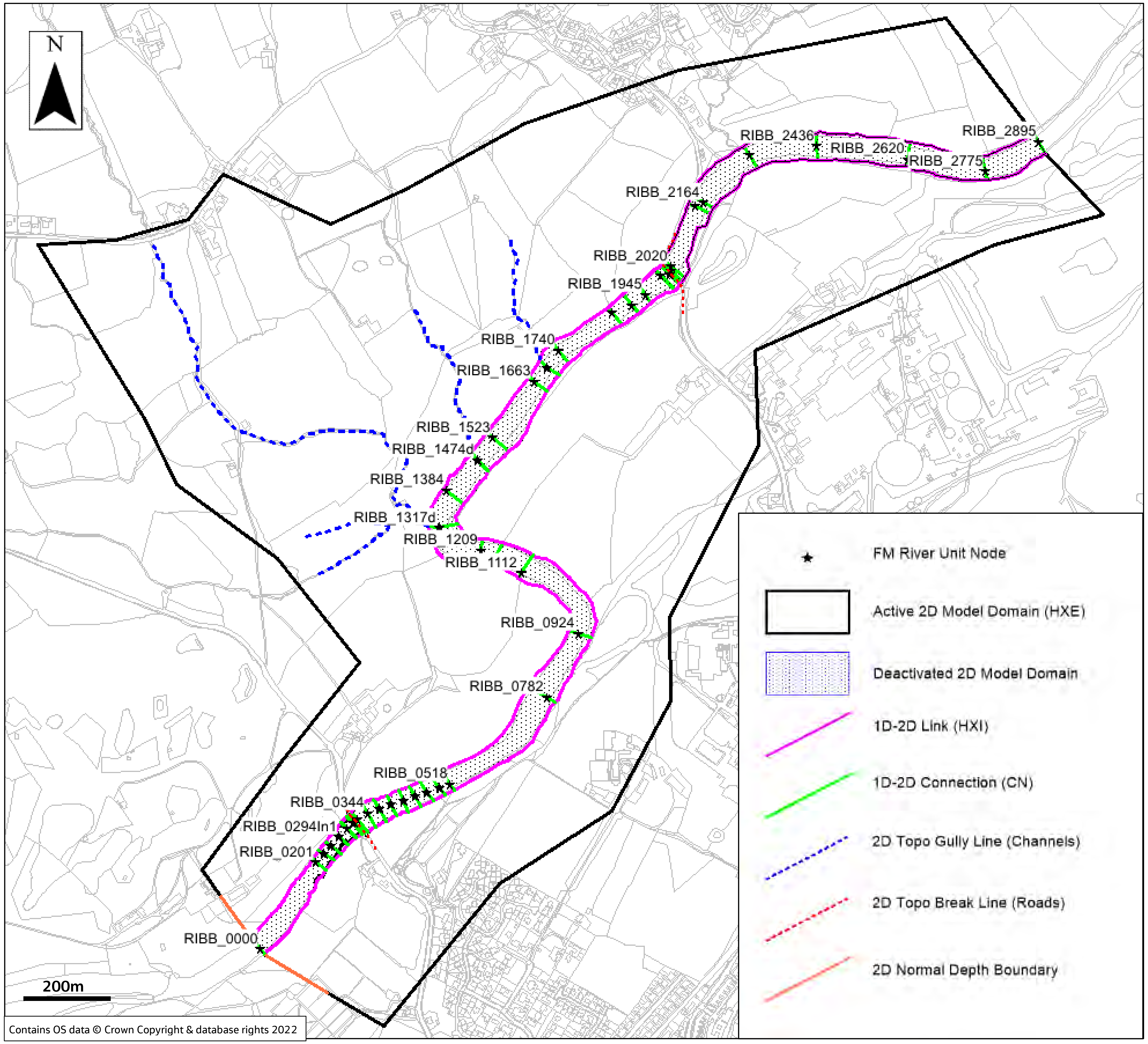


Figure 4-5: River Ribble 2D model schematisation.

36) Hydraulic roughness coefficients are applied over each grid cell of the 2D domain, as shown in Table 4-2, using on land use categories taken from OS Mastermap data. Roughness values adopted were based on commonly used values for each land use type with reference to standard guidance (Chow, 1959). The distribution of land use types within the active model domain is almost exclusively the natural surface type, with some very limited coverage of manmade surface, roads, inland water and thick vegetation.

Table 4-2: Manning's 'n' coefficients of roughness - 2D domain.

Land Use	Manning’s ‘n’
Manmade surfaces	0.025
Natural surfaces	0.04
Inland Water	0.02
Gardens	0.05
Roads/Tracks/Paths	0.025
Thick Vegetation	0.07
Structures	0.025

37) No inflow has been applied directly in the 2D domain. Any flow across the 2D domain is as a result of the 1D channel being overtopped, simulating out of bank conditions. A flow-stage (HQ) boundary was applied at the downstream model extent to allow water to flow freely out of the model domain, which was determined based on the average slope at the downstream model extent taken from LiDAR data.

38) The link between the 1D and 2D domains was defined along the bank tops of the River Ribble, using a “HX” schematisation which directly transfers the water levels between the 1D and 2D domains. The grid cell levels along the 1D/2D boundary alignment were based on the LiDAR data without any adjustment.

4.3 Unnamed Watercourse Schematisation – 1D Domain

River Geometry

39) The 1D model covers a 265 m reach of the Unnamed Watercourse. The Surveyed cross-section data has been used to inform the in-channel geometry of the modelled watercourse. The locations of the surveyed cross-sections are shown in Figure 4-7.

40) Interpolated cross-sections were added to the upper half of the model to assist with model performance. In general, the model cross sections are set at no more than 30 m spacing.

Hydraulic Friction

41) Hydraulic roughness (Manning’s ‘n’ coefficient) values were determined primarily using site photographs taken during the survey, a uniform value of N = 0.055 was utilised for the in-channel roughness values, taken from standard guidance (Chow, 1959) as a mid to high value for a winding channel with stones. Figure 4-6 shows a typical section of the Unnamed Watercourse within the model domain.

Figure 4-6: Site visit photograph facing downstream, cross-section UNN01_1808.

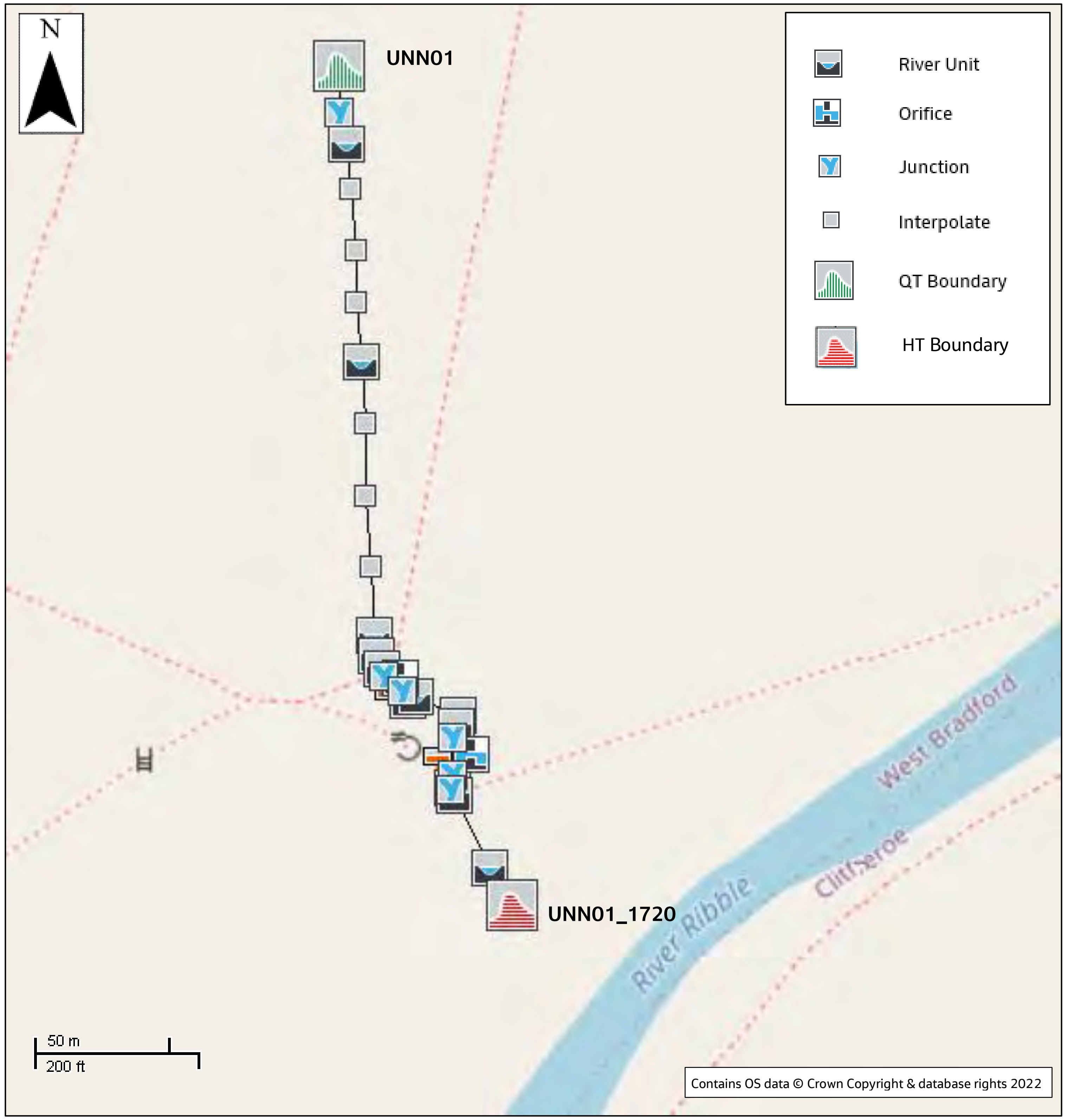


Figure 4-7: Baseline schematisation of Unnamed Watercourse Model 1D domain and inflows.

Structures

42) Two hydraulic structures were included in the baseline model, using Flood Modeller Orifice units with associated inline spill units to represent a simple footbridge and a boundary wall (see Figure 4-8). The location of the structures is shown in Figure 4-7.

Figure 4-8: Site photographs of A: footbridge at UNN01_1781 (facing upstream), and B: boudary wall crossing at UNN01_1824 (facing downstream).

Boundary Conditions

43) The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4-3. Derivation of the hydrological boundaries are detailed in Section 3. Inflow locations are shown in Figure 4-7.

Table 4-3: Boundary conditions - 1D domain.

Type of Boundary	Flood Modeller Node	Description
QT boundary	UNN_01	Applied at the upstream end of the modelled reach of the Unnamed Watercourse
HT Boundary	UNN01_1720	Applied to the downstream end of the Unnamed watercourse reach. A constant head boundary condition set to the peak of the 50%AEP (1 in 2) event taken from the River Ribble model results. The suitability of the downstream boundary is discussed in Section 3.2.

4.4 Unnamed Watercourse Floodplain Schematisation – 2D Domain

44) The 2D domain covers an area of 0.02 km^2.

45) The topography of the floodplain is represented in the model using a 5 m resolution square grid. The levels for the grid cells are based on a DTM derived from 1 m resolution LiDAR data. A review of the floodplain using available aerial and OS mapping has shown that there are no structures within the floodplain that require additional representation. Therefore, no modifications were made to the LiDAR DTM. Figure 4-9 shows the 2D model extent, links between the 1D and 2D model components and the land use type, which was used in the 2D representation of the floodplain.

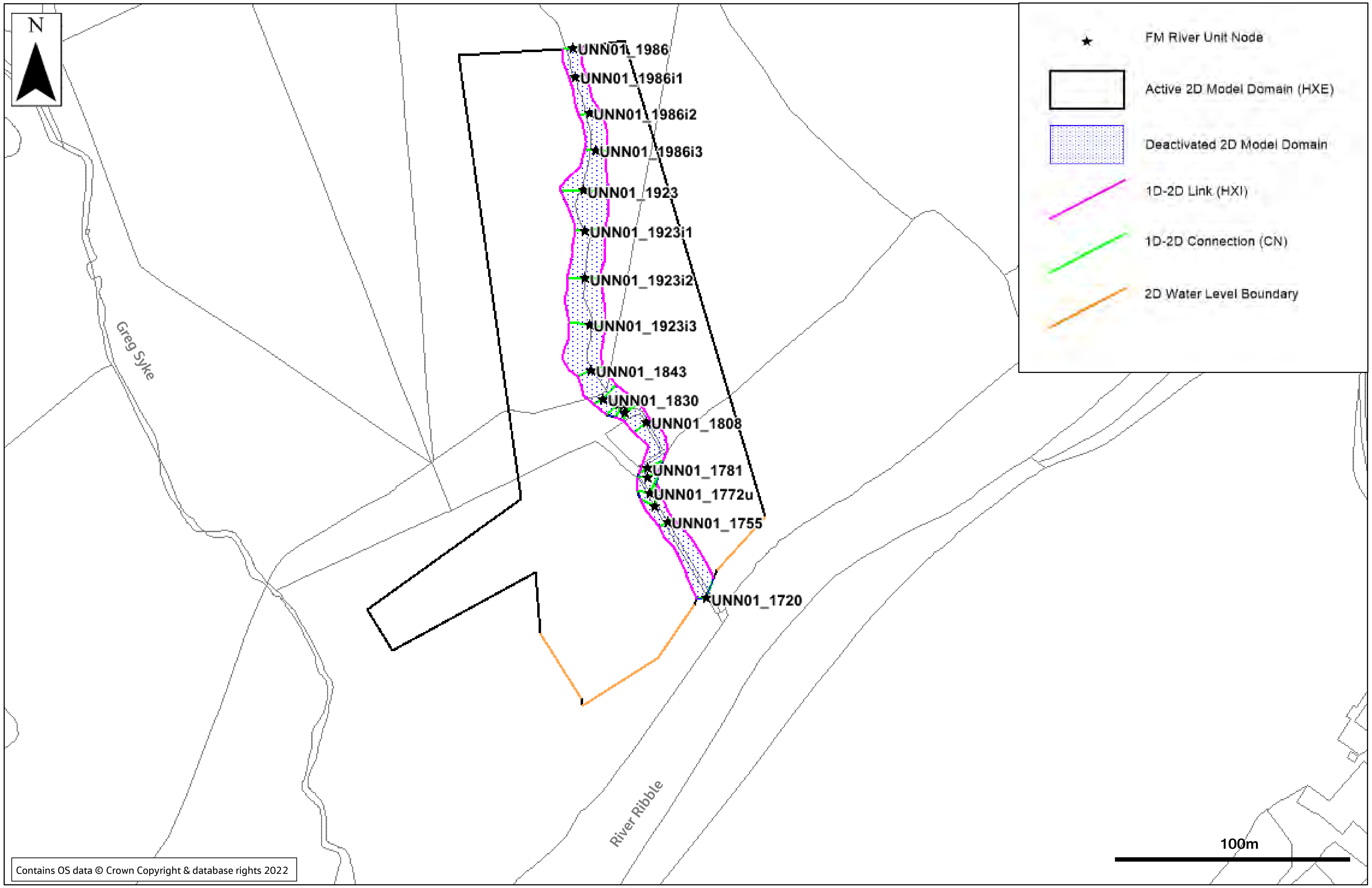


Figure 4-9: Unnamed Watercourse 2D model schematisation.

46) Hydraulic roughness coefficients are applied over each grid cell of the 2D domain, as shown in Table 4-4 using land use categories taken from OS Mastermap data. Roughness values adopted were based on commonly used values for each land use type with reference to standard guidance (Chow, 1959). The type of land use within the active model domain is entirely the natural surface type.

Table 4-4: Manning's 'n' coefficients of roughness - 2D domain.

Land Use	Manning's 'n'
Manmade surfaces	0.025
Natural surfaces	0.04
Inland Water	0.02
Gardens	0.05
Roads/Tracks/Paths	0.025
Thick Vegetation	0.07
Structures	0.025

47) No inflow has been applied directly in the 2D domain. Any flow across the 2D domain is as a result of the 1D channel being overtopped, simulating out of bank conditions. A Water Level boundary was utilised at the downstream extents of the 2D model, this utilised a constant water level taken from the peak of the River Ribble 50 % AEP (1 in 2) event, at the confluence of the Unnamed Watercourse and the Ribble.

48) The link between the 1D and 2D domains was defined along the bank tops of the Unnamed Watercourse, using a "HX" schematisation which directly transfers the water levels between the 1D and 2D domains. The grid cell levels along the 1D/2D boundary alignment were based on the LiDAR data without any adjustment.

4.5 Greg Syke Watercourse Schematisation – 1D Domain

River Geometry

49) The 1D model covers a 319 m reach of Greg Syke. The Surveyed cross-section data has been used to inform the in-channel geometry of the modelled watercourse. The locations of the surveyed cross-sections are shown in Figure 4-11.

50) Interpolated cross-sections were added along the full modelled reach to assist with model performance. In general, the model cross sections are set at no more than 20 m spacing.

Hydraulic Friction

51) Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using site photographs taken during the survey, a uniform value of N = 0.05 was utilised for the in-channel roughness values, taken from standard guidance (Chow, 1959) as a high value for a winding channel with weeds and stones, or mid value for a winding channel with more stones. Figure 4-10 shows a typical section of the Greg Syke within the model domain.

Figure 4-10: Site visit photograph facing downstream, cross-section GRE_1765.

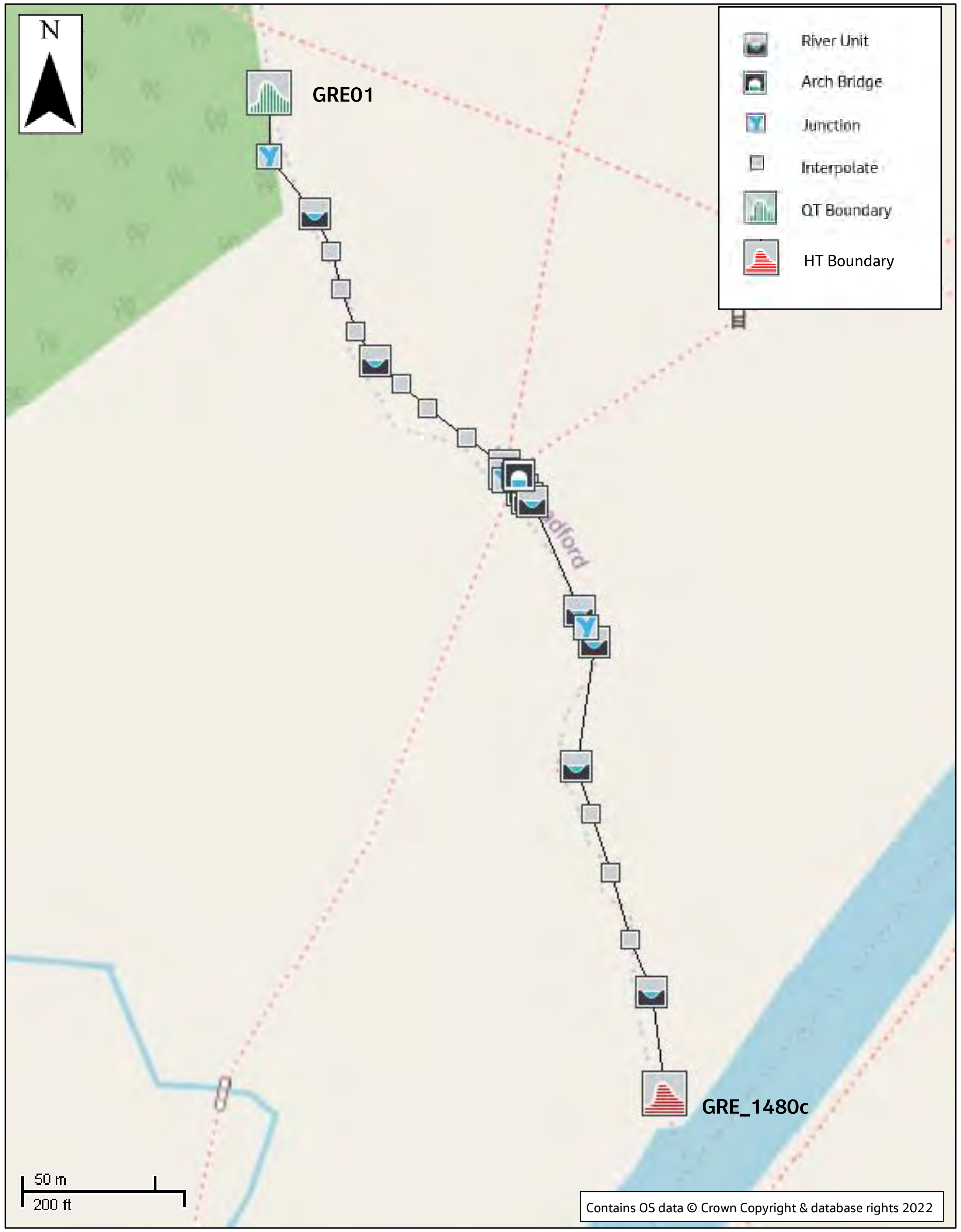


Figure 4-11: Baseline schematisation of Greg Syke Model 1D domain and inflows.

Structures

52) One hydraulic structure was included in the baseline model. A Flood modeller arch bridge unit with associated inline spill unit was used to represent the simple slab footpath crossing which is pictured in Figure 4-12. The location of the structure is shown in Figure 4-11.

Figure 4-12: Site photograph of Farm Bridge (facing downstream).

Boundary Conditions

53) The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4-5. Derivation of the hydrological boundaries are detailed in Section 3. Inflow locations are shown in Figure 4-11.

Table 4-5: Boundary conditions - 1D domain.

Type of Boundary	Flood Modeller Node	Description
QT boundary	GRE01	Applied at the upstream end of the modelled reach of Greg Syke
HT Boundary	GRE01_1480c	Applied to the downstream end of the Greg Syke modelled reach. A constant head boundary condition set to the peak of the 50%AEP (1 in 2) event taken from the River Ribble model results. The suitability of the downstream boundary is discussed in Section 3.2.

4.6 Greg Syke Watercourse Floodplain Schematisation – 2D Domain

54) The 2D domain covers an area of 0.06 km^2. The topography of the floodplain is represented in the model using a 4 m resolution square grid. The levels for the grid cells are based on a DTM derived from 1 m resolution LiDAR data. A review of the floodplain using available aerial and OS mapping has shown that there are no structures within the floodplain that require additional representation. Therefore, no modifications were made to the LiDAR DTM. Figure 4-13 shows the 2D model extent, links between the 1D and 2D model components and the land use type, which was used in the 2D representation of the floodplain.

Jacobs

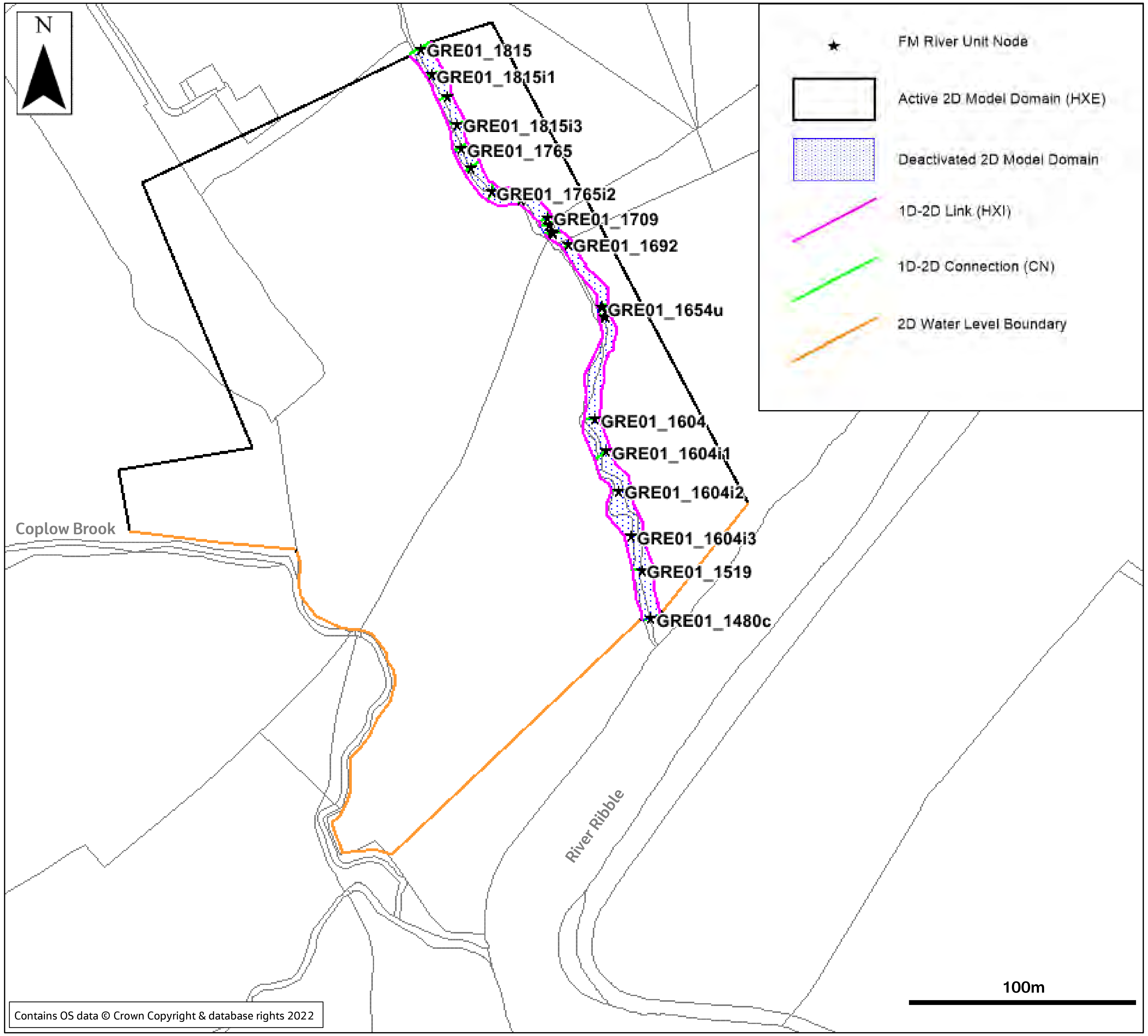


Figure 4-13 Greg Syke 2D model schematisation.

55) Hydraulic roughness coefficients are applied over each grid cell of the 2D domain, as shown in Table 4-6, using on land use categories taken from OS Mastermap data. Roughness values adopted were based on commonly used values for each land use type with reference to standard guidance (Chow, 1959). The type of land use within the active model domain is entirely the natural surface type.

Table 4-6: Manning's 'n' coefficients of roughness - 2D domain.

Land Use	Manning's 'n'
Manmade surfaces	0.025
Natural surfaces	0.04
Inland Water	0.02
Gardens	0.05
Roads/Tracks/Paths	0.025
Thick Vegetation	0.07
Structures	0.025

56) No inflow has been applied directly in the 2D domain. Any flow across the 2D domain is as a result of the 1D channel being overtopped, simulating out of bank conditions. A Water Level boundary was utilised at the downstream extents of the 2D model, this utilised a constant water level taken from the peak of the River Ribble 50 % AEP (1 in 2) event, at the confluence of Greg Syke and the River Ribble.

57) The link between the 1D and 2D domains was defined along the bank tops of Greg Syke, using a "HX" schematisation which directly transfers the water levels between the 1D and 2D domains. The grid cell levels along the 1D/2D boundary alignment were based on the LiDAR data without any adjustment.

4.7 Coplow Brook Watercourse Schematisation – 1D Domain

River Geometry

58) The 1D model covers a 562 m reach of Coplow Brook. The surveyed cross-section data has been used to inform the in-channel geometry of the modelled watercourse. The locations of the surveyed cross-sections are shown in Figure 4-15.

59) Interpolated cross-sections were added between selected cross-sections to assist with model performance. In general, the model cross sections are set at no more than 130 m spacing.

Hydraulic Friction

60) Hydraulic roughness (Manning's 'n' coefficient) values were determined primarily using site photographs taken during the survey, a uniform value of N = 0.045 was utilised for the in-channel roughness values, taken from standard guidance (Chow, 1959) as a high value for a clean winding channel with some pools and shoals, or mid value for a winding channel with weeds and stones. Figure 4-14 shows a typical section of the Coplow Brook within the model domain.

Figure 4-14: Site visit photograph facing downstream, cross-section COPL_1927.

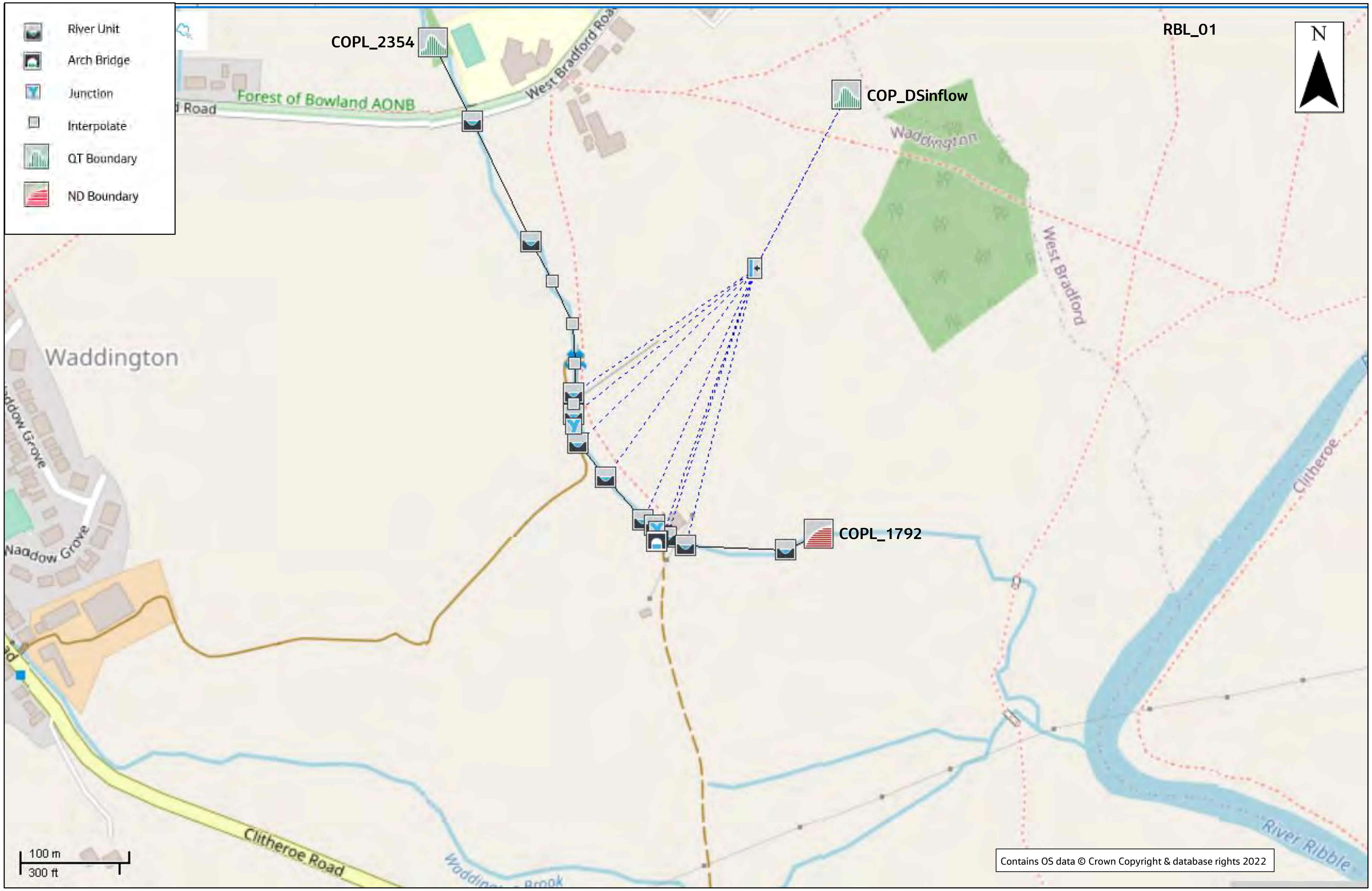


Figure 4-15: Baseline schematisation of Coplow Brook Model 1D domain and inflows.

Structures

61) One hydraulic structure was included in the baseline model. A Flood modeller arch bridge unit, with associated inline spill unit was used to represent the simple slab farm bridge crossing which is pictured in Figure 4-16. The location of the structure is shown in Figure 4-15.

Figure 4-16: Site photograph of Farm Bridge (facing downstream).

Boundary Conditions

62) The upstream and downstream boundary conditions applied to the 1D domain are described in Table 4-7. Derivation of the hydrological boundaries are detailed in Section 3. Inflow locations are shown in Figure 4-15.

Table 4-7: Boundary conditions - 1D domain.

Type of Boundary	Flood Modeller Node	Description
QT boundary	COPL_2354	Applied at the upstream end of the modelled reach of Coplow Brook
QT boundary	COP_DSinflow	Applied laterally along the main modelled reach of Coplow Brook
Normal Depth Downstream Boundary	COPL_1792	Applied to the downstream end of the modelled reach of Coplow Brook. A normal depth boundary calculates a flow-head relationship based on the channel characteristics. This downstream boundary type is considered suitable as there is no influence of downstream structures. The suitability of the downstream boundary is further discussed in Section 9.

4.8 Coplow Brook Watercourse Floodplain Schematisation – 2D Domain

63) The 2D domain covers an area of 0.11 km^2.

64) The topography of the floodplain is represented in the model using a 4 m resolution square grid. The levels for the grid cells are based on a DTM derived from 1 m resolution LiDAR data. A review of the floodplain using available aerial and OS mapping has shown that there are no structures within the floodplain that require additional representation. Therefore, no modifications were made to the LiDAR DTM.

65) A single topographic patch was applied close to the downstream boundary of the model, the patch smooths the terrain via interpolation of the elevations around its perimeter and fixes a localised area of poor mass balance performance.

66) Figure 4-17 shows the 2D model extent, links between the 1D and 2D model components and the land use type, which was used in the 2D representation of the floodplain.

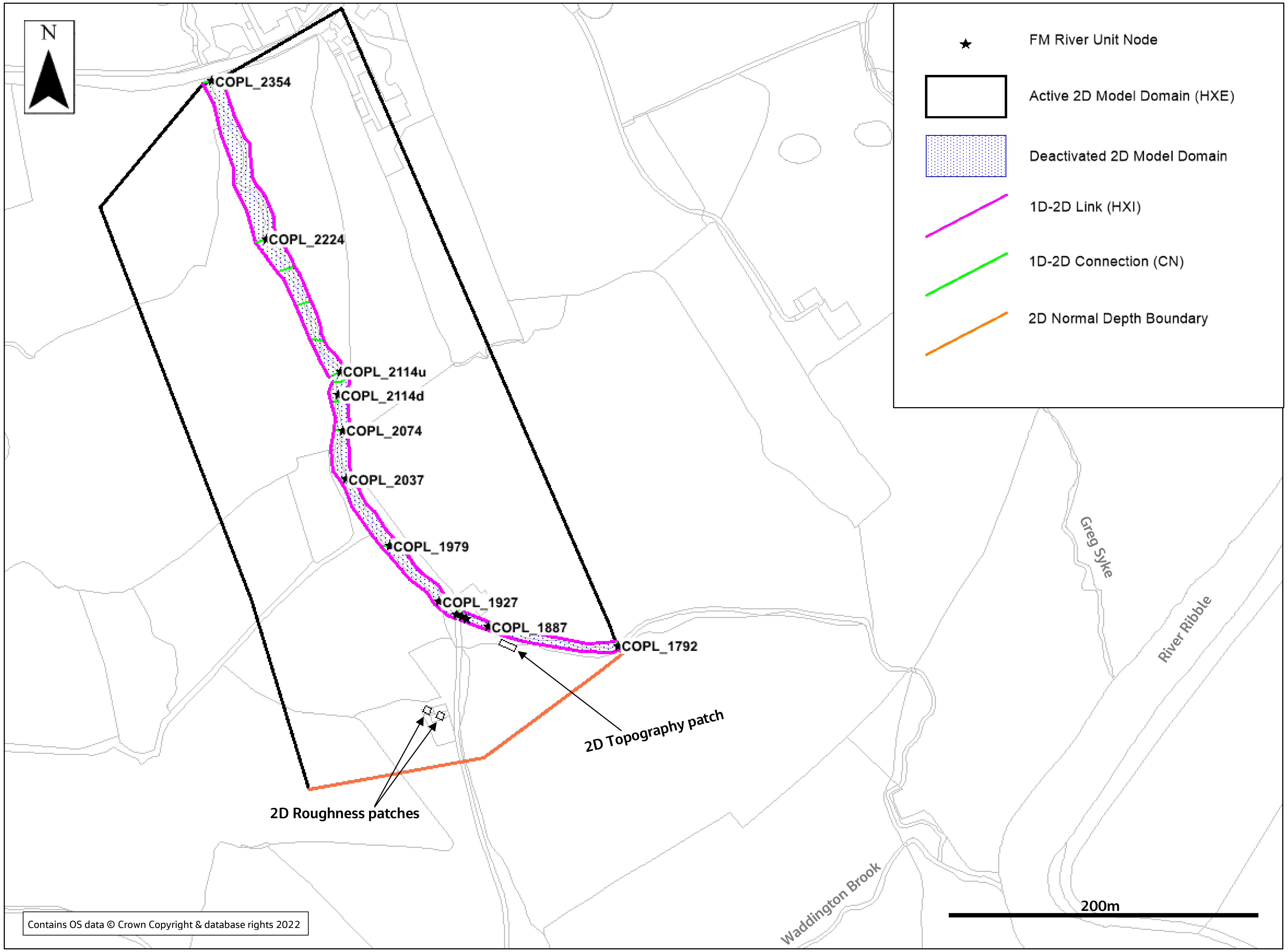


Figure 4-17: 2D Coplow Brook 2D model schematisation.

67) Hydraulic roughness coefficients are applied over each grid cell of the 2D domain, as shown in Table 4-8, using land use categories taken from OS Mastermap data. Roughness values adopted were based on commonly used values for each land use type with reference to standard guidance (Chow, 1959). Depth varying roughness is utilised in order to optimise model convergence and achieve appropriate mass balance performance. This approach utilises increased roughness, beyond the standard values, for shallow flow depths (see Table 4-8).

68) The distribution of land use types within the active model domain is almost exclusively the natural surface type, with some very limited coverage of manmade surface, roads, inland water and thick vegetation. Two very small roughness patches were applied to correct the land use type at a house and garden within the modelled flood extent, which was not correctly represented in the OS Mastermap coverage.

Table 4-8: Manning's 'n' coefficients of roughness - 2D domain.

Land Use	Manning’s ‘n’ depth <0.15m	Manning’s ‘n’ depth >0.15m
Manmade surfaces	0.05	0.025
Natural surfaces	0.08	0.04
Inland Water	0.04	0.02
Gardens	0.10	0.05
Roads/Tracks/Paths	0.50	0.025
Thick Vegetation	0.14	0.07
Structures	0.50	0.025
Buildings	1.00	1.00

69) No inflow has been applied directly in the 2D domain. Any flow across the 2D domain is as a result of the 1D channel being overtopped, simulating out of bank conditions. A normal depth boundary was utilised at the downstream extents of the 2D model, allowing free discharge for flows leaving the 2D domain. The boundary utilised bed slope value taken from inspection of the lidar topography local to the boundary. It is considered that the boundary is situated suitably downstream from the area of interest that boundary conditions will not affect model results in the area of interest.

70) The link between the 1D and 2D domains was defined along the bank tops of the Coplow Brook, using a “HX” schematisation which directly transfers the water levels between the 1D and 2D domains. The grid cell levels along the 1D/2D boundary alignment were based on the LiDAR data without any adjustment.

5. Modelled Events

71) Table 5-1 shows the Annual Exceedance Probability (AEP) events and model scenarios that were simulated with the hydraulic models.

72) In order to test the model sensitivity to key hydraulic parameters, a series of simulations were undertaken for the baseline 1 % AEP event. The assessed hydraulic parameters were: Manning's 'n' roughness coefficients, hydrological inflows and downstream boundary slope.

73) As discussed in Section 3, there is no requirement to simulate storm event with future scenario climate change adjustment factors, for the flood risk assessment of the temporary works under consideration.

Table 5-1: Modelled events.

Scenario	AEP Event					
	50%	20%	10%	3.33%	1.33%	1%
River Ribble						
Baseline	✓	✓	✓	✓	✓	✓
Roughness Sensitivity (1D and 2D)						✓
Hydrological Inflow Sensitivity						✓
Downstream Boundary Sensitivity						✓
With Proposed Ribble Crossing	✓	✓	✓	✓	✓	✓
Tributaries						
Unnamed Watercourse Baseline	✓	✓	✓	✓	✓	✓
Unnamed Watercourse With Proposed Ribble Crossing	✓	✓	✓	✓	✓	✓
Greg Syke Baseline	✓	✓	✓	✓	✓	✓
Greg Syke With Proposed Ribble Crossing	✓	✓	✓	✓	✓	✓
Coplow Baseline	✓	✓	✓	✓	✓	✓
Coplow Brook With Proposed Ribble Crossing	✓	✓	✓	✓	✓	✓

6. Model Proving

6.1 Introduction

74) The following sections discuss the model's numerical performance and the verification process. Details relating to the additional runs carried out to test the sensitivity of the model (River Ribble only) to key variables are also presented.

6.2 Model Numerical Performance

75) Run performance has been for each simulation carried out, to ensure a suitable model convergence was achieved. Convergence is the ability of the modelling software to arrive at a stable solution within a specified number of iterations, for each model timestep. The convergence of the 1D models was checked and seen to be within the recommended tolerance for all simulations. These model diagnostics are considered to be well within the acceptable range, providing good confidence in the computational solution. Figure 6-1 shows typical convergence plots for the events modelled for the main River Ribble and tributary models developed for this study.

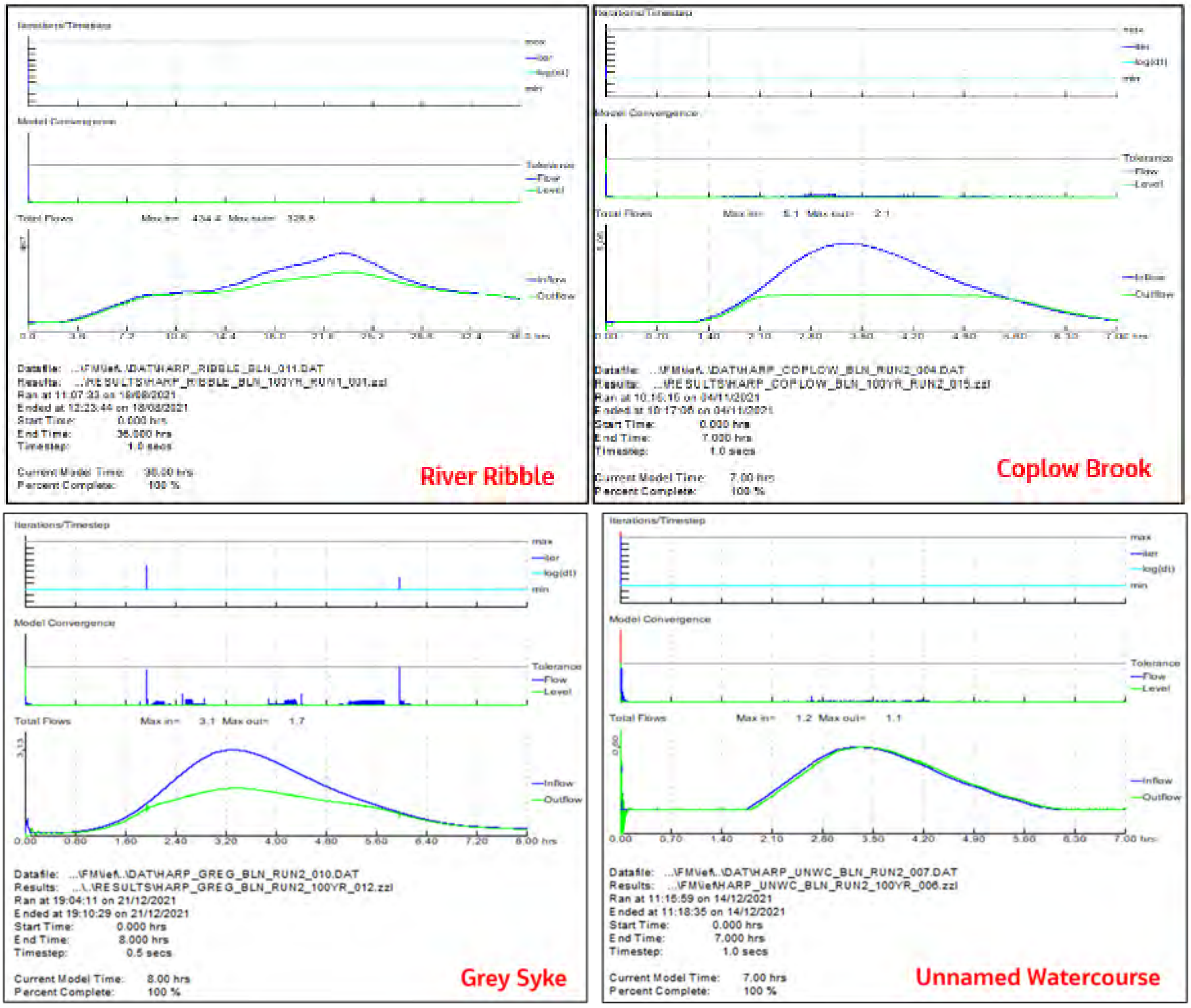
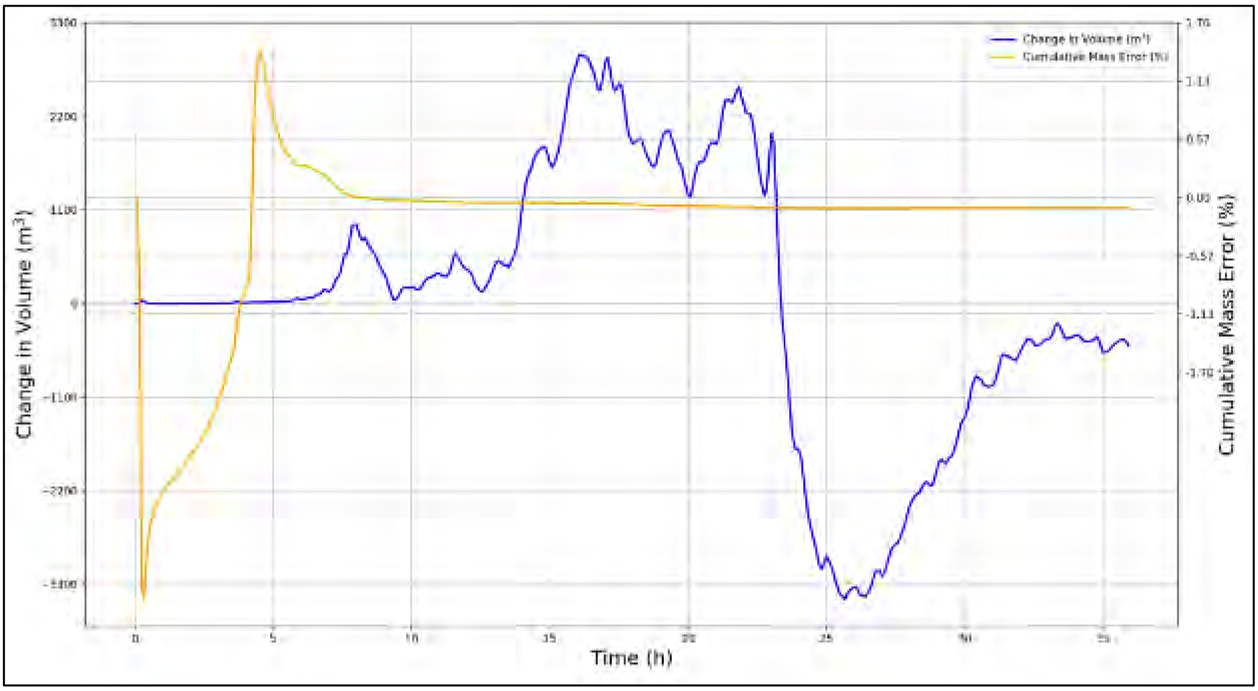


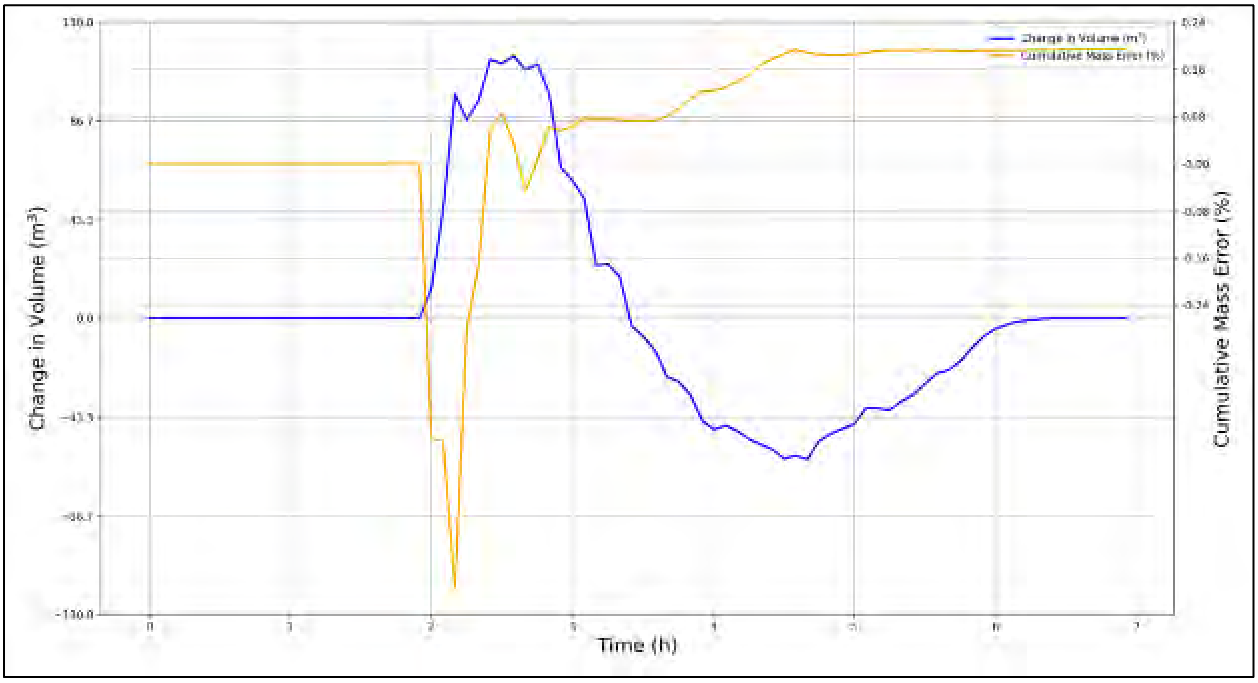
Figure 6-1: Flood Modeller 1D convergence plots - 1% AEP event - baseline scenario

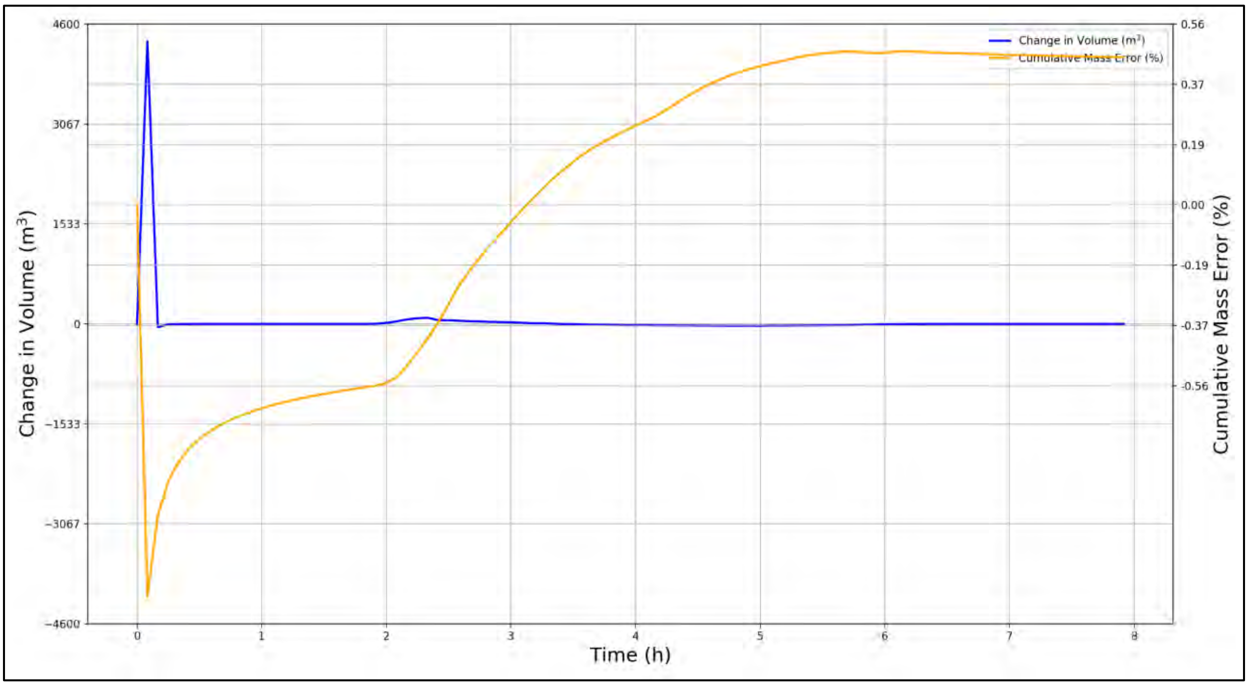
76) The cumulative mass error (Cum ME) reports output from the TUFLOW 2D models have been checked for all simulated events. The accepted tolerance range recommended by the software manual is ±1 % mass error. Figure 6-2 shows that for the 1 % AEP flood event, Cum ME is well within this tolerance range for all models under consideration. Initial high mass error at the beginning of a simulation is

expected and relates to the onset of water flow from 1D to 2D before any significant volume of water is present in the 2D domain.

77) Smooth variation of the change in volume (dVol) through the model simulation is another indicator of good convergence in the 2D model (Figure 6-2). These 2D mass error and dVol diagnostics are considered acceptable and are seen to be typical, for all events simulated with each model.







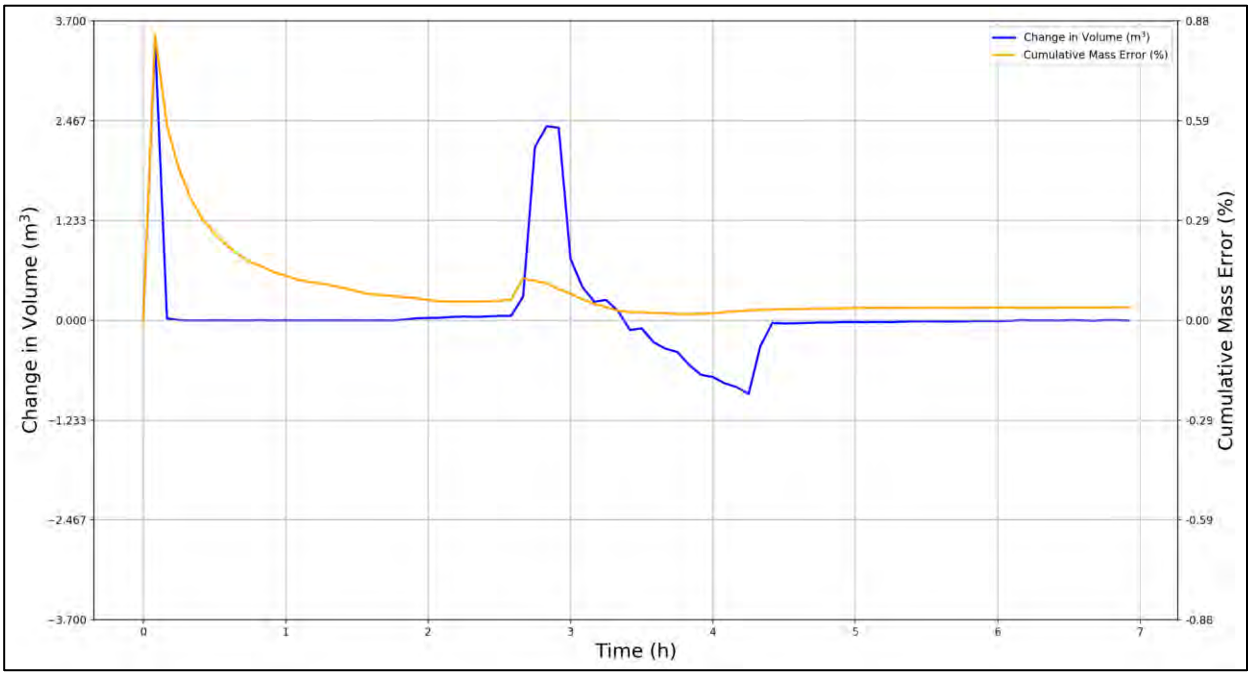


Figure 6-2: Cumulative Mass Error and change in volume plots - 1% AEP event – baseline scenario

6.3 Calibration and Verification

6.3.1 Calibration

78) Calibration of the hydraulic models was not possible because the River Ribble catchment is ungauged within the study area.

6.3.2 Verification Using Historical Data

79) EA historic flood maps show the maximum extent of all individual recorded flood events and areas of land that have been previously subject to flooding in England. An assessment of this data showed that there is no historic flood data for the River Ribble at this location.

6.3.3 Verification Using Environment Agency (EA) Flood Maps

80) Figure 6-3 shows the modelled 1 % AEP flood extent and the EA published Flood Zone 2 for the River Ribble model extent. The comparison shows that the model results generally well match the published flood map, with just some limited areas where the modelled extent has larger coverage.

81) There is no available published fluvial flood map data that is suitable for the verification of the tributary watercourses.

6.3.4 Verification Conclusion

82) In conclusion, there is limited data available for verification of the River Ribble model. There are some discrepancies between the EA flood map and the model, however this is due to the present study being a more detailed assessment of flooding than the EA flood map.

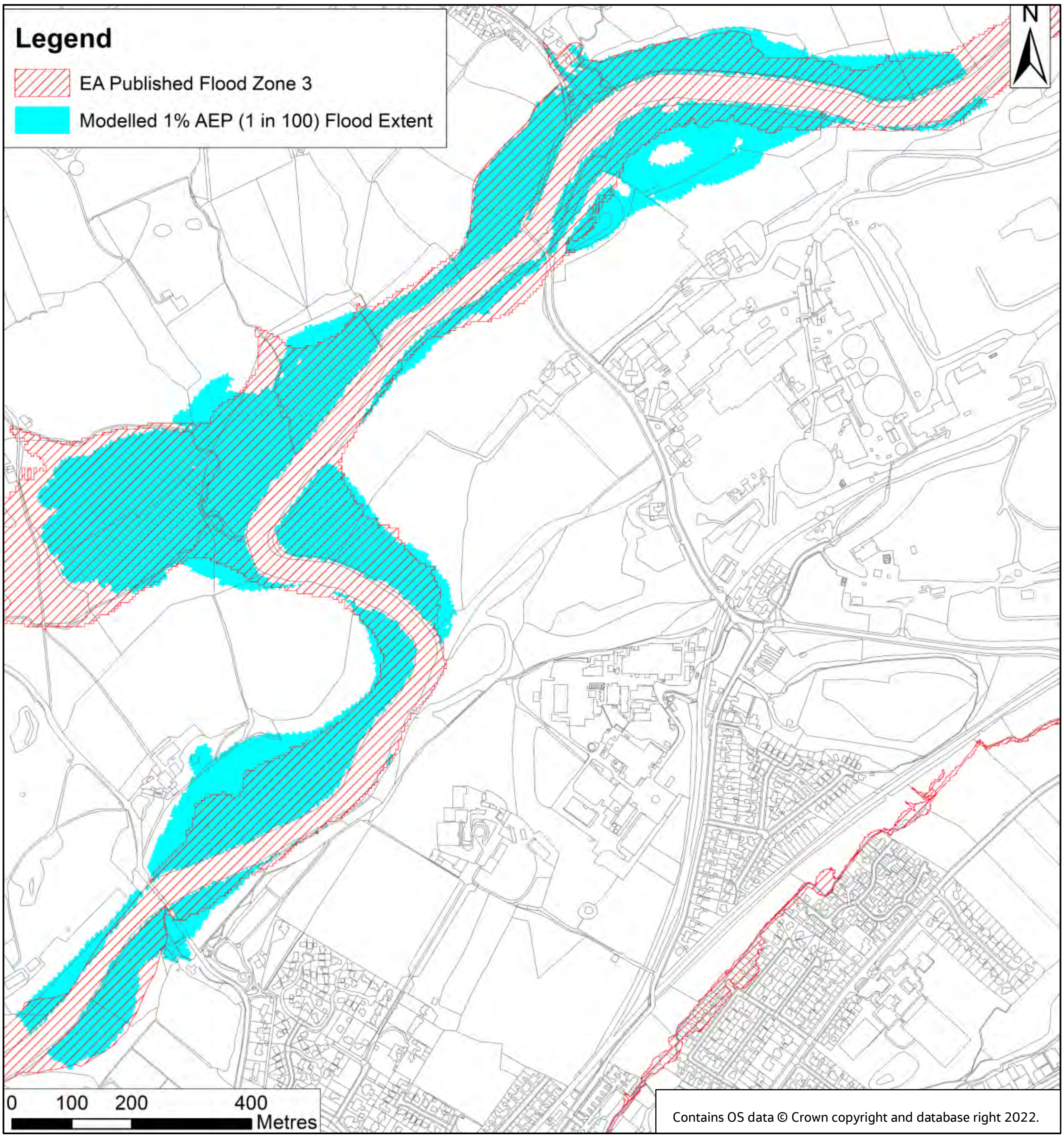


Figure 6-3: Baseline 1% AEP event modelled flood extent compared to the EA Flood Zone 3 mapping.

6.4 Sensitivity Analysis

83) Sensitivity tests have been carried out to investigate the robustness of the River Ribble model and quantify uncertainty. The following sensitivity tests have been carried out for the 1 % AEP event and compared to the baseline 1D water levels and 2D flood extents.

6.4.1 Roughness Sensitivity

84) In-channel and floodplain roughness coefficients (Manning's 'n') were adjusted by +20 % and -20 %. Table 6-1 shows the impact of changing the model roughness on predicted peak water levels. The results show that the in-channel water levels are sensitive to changes in roughness. The majority of the floodplain is sensitivity to changes in roughness, as illustrated in Figure 6-4, experiencing a 150-250 mm increase when roughness is increased by 20 %. The flood extent is also increased slightly, resulting in potential flooding to one property on Clitheroe Road.

Table 6-1: Roughness sensitivity results.

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Roughness	+0.32	+0.16	+0.21
-20% Roughness	-0.43	-0.15	-0.24

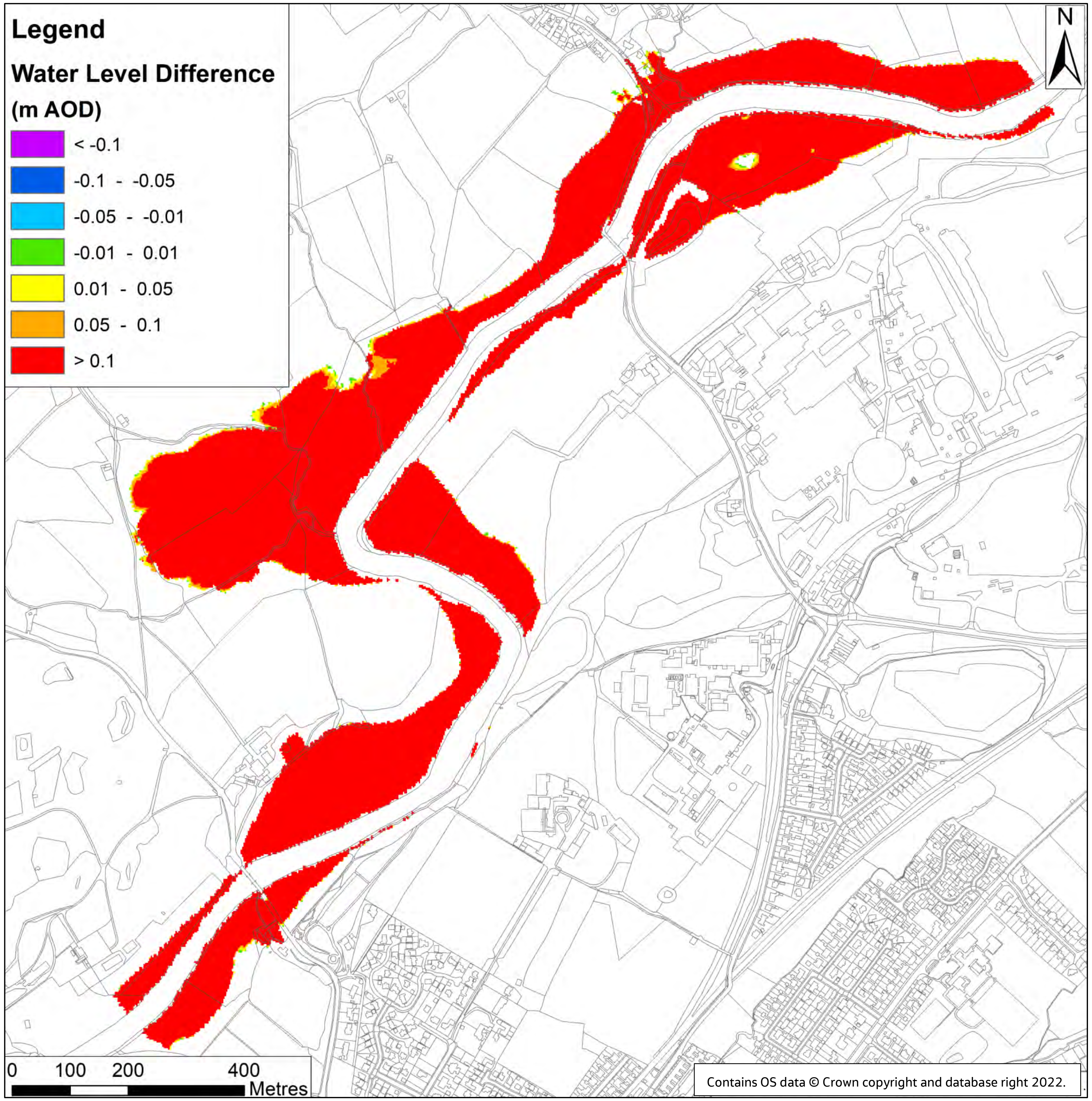


Figure 6-4: Difference in modelled flood levels when in-channel and floodplain roughness values are increased by 20%.

6.4.2 Hydrological Inflow Sensitivity

85) The flows into the model were adjusted by +20 % and -20 %. Table 6-2 shows the impact of changing the model inflows on predicted peak water levels. The results show that the in-channel water levels are sensitive to changes in flow. The majority of the floodplain is majorly sensitivity to changes in flow, as illustrated in Figure 6-5, experiencing a 150-350 mm increase when inflows are increased by 20 %. The flood extent is also slightly increased, resulting in potential flooding to one property on Clitheroe Road.

Table 6-2: Hydrological inflow sensitivity results.

Sensitivity	Water Level Difference (m)		
	Max	Min	Average
+20% Flow	+0.34	+0.15	+0.26
-20% Flow	-0.39	-0.18	-0.30

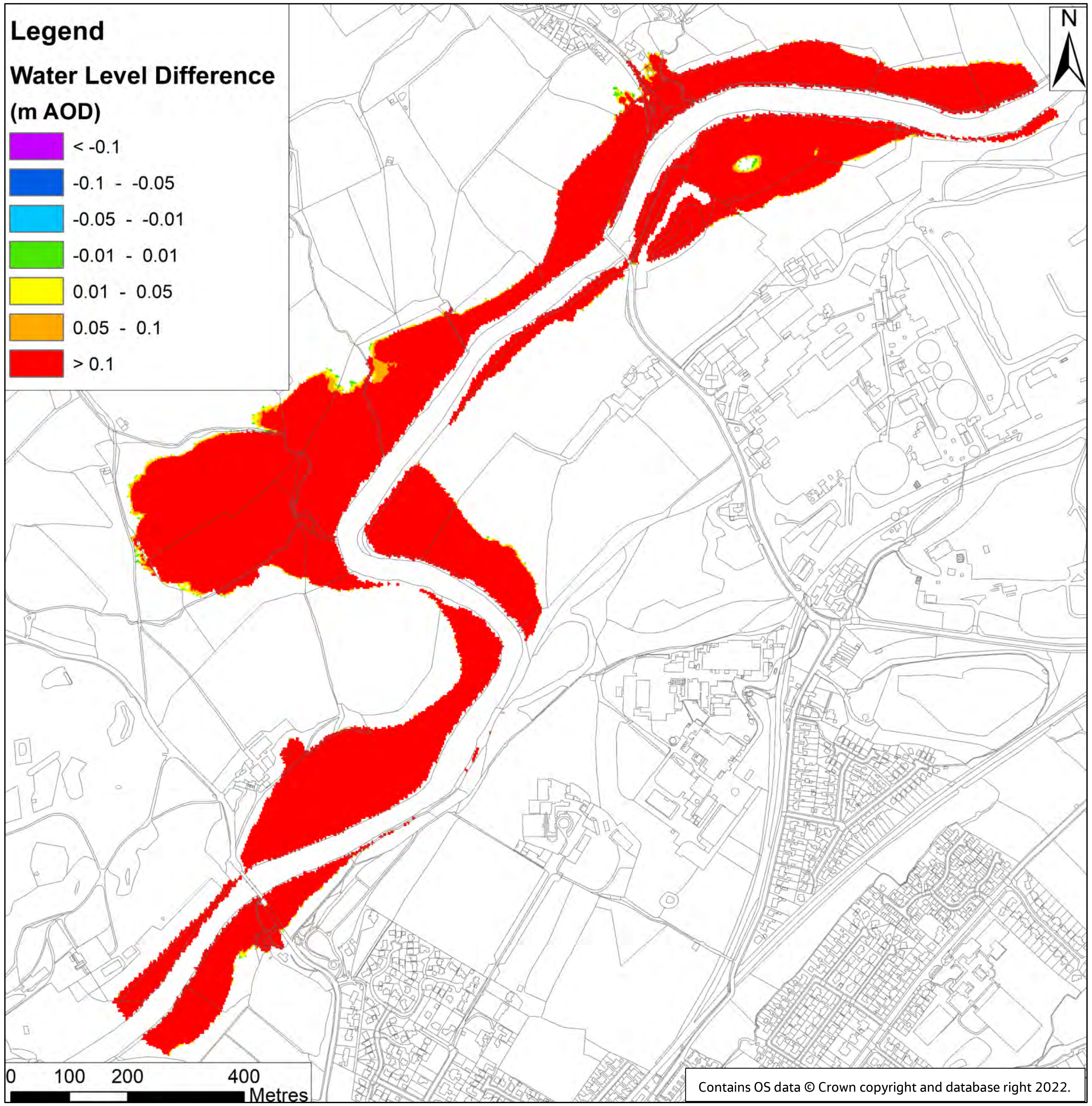


Figure 6-5: Difference in modelled flood levels when hydrological inflows to the model are increased by 20%.

6.4.3 Downstream Boundary Condition Sensitivity

86) The slope of the normal depth downstream boundary was adjusted in the 1D domain by +20 % and -20 %. Table 6-3 shows the response at the downstream end of the model (Flood Modeller node RIBB_0000). The location at which there is no change in water level as a result of changing the downstream boundary has been identified. Distances from this location, in relation to the downstream end of the model (tailwater distance) and in relation to the Proposed Ribble Crossing are also shown. The results show that the effect of the downstream boundary does not reach the proposed development location. The flood outline also remains unchanged at the proposed development location, with just minor changes in extent local to the downstream boundary (see Figure 6-6). This indicates that the downstream boundary is suitably removed from the area of interest and the boundary assumption is appropriate.

Table 6-3: Downstream boundary slope sensitivity results.

Sensitivity	Water Level Difference (m) at RIBB_0000	Tailwater Distance (m)	Distance to Proposed Ribble Crossing (m)
+20% Slope	-0.11	782	1128
-20% Slope	+0.11	1112	798

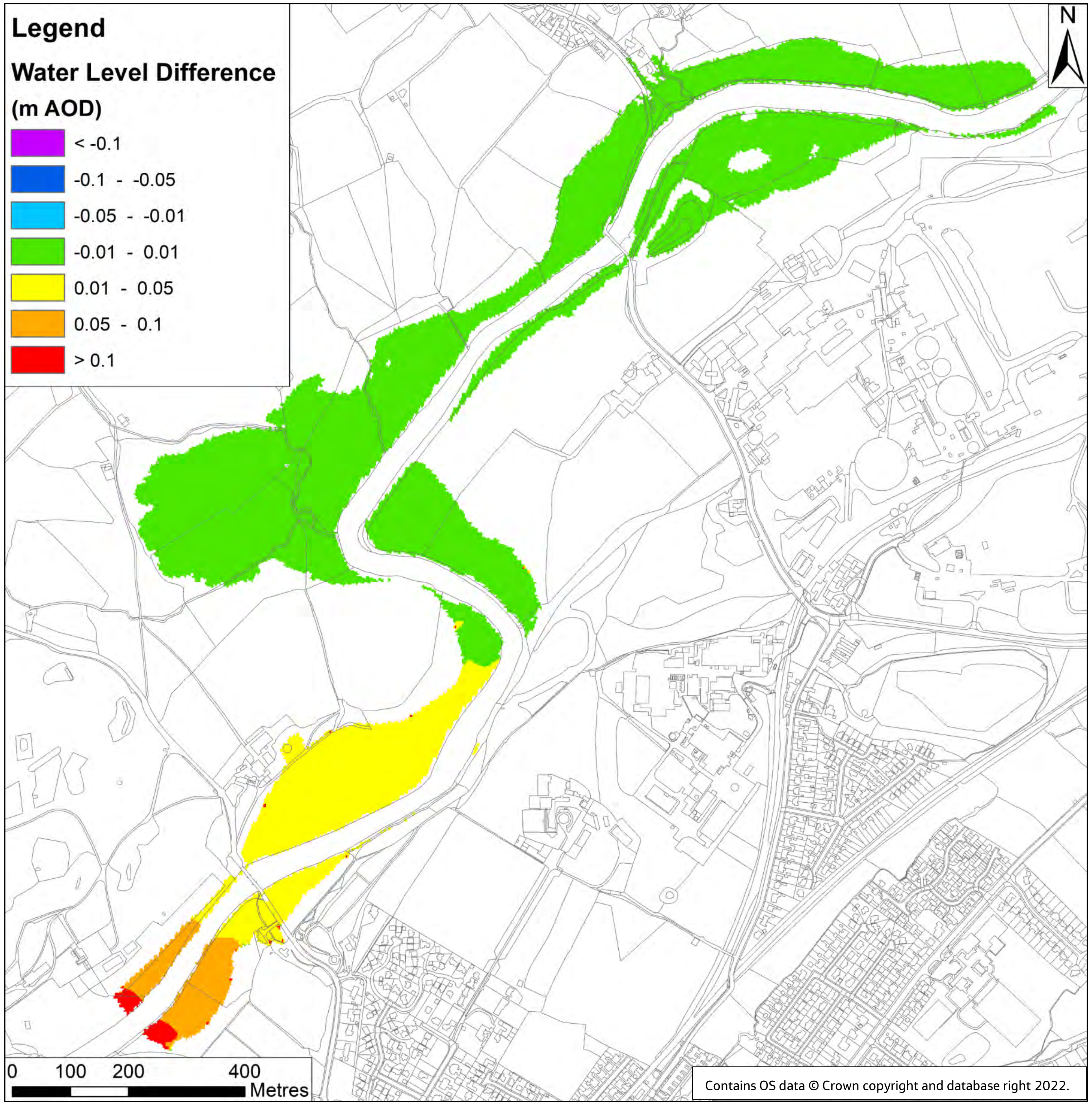


Figure 6-6: Difference in modelled flood levels when slope at the downstream boundary is decreased by 20%.

7. Proposed Ribble Crossing Modelling

87) The Proposed Ribble Crossing comprises a haul route of approximately 1.5 km crossing the Ribble Valley to the west of Clitheroe (see Figure 7-1). The River Ribble will be spanned by a steel frame road bridge to allow the crossing of heavy vehicles to gain access to a construction site that will be located on the hillside north of the Ribble. The bridge structure will be supported by two piers placed just outside of the River Ribble bank tops and oriented to the river flow path. There are also two construction compounds to be set up next to the bridge, one on each bank of the Ribble. The compound design has been confirmed to have a finished surface level that does not exceed the current existing ground level, these features therefore do not require representation in the model. The bridge and road are estimated to be in service for five years and will then be removed.

88) The haul road will traverse the Ribble valley along the north hillside and crosses the three tributary watercourses. To achieve a suitable vertical alignment for the required traffic, the finished road level will include some cut and some fill of the existing topography along its route. The basic specification for the three minor watercourse crossings will be assessed within the present study.

89) Outline design drawings are available in Appendix F.

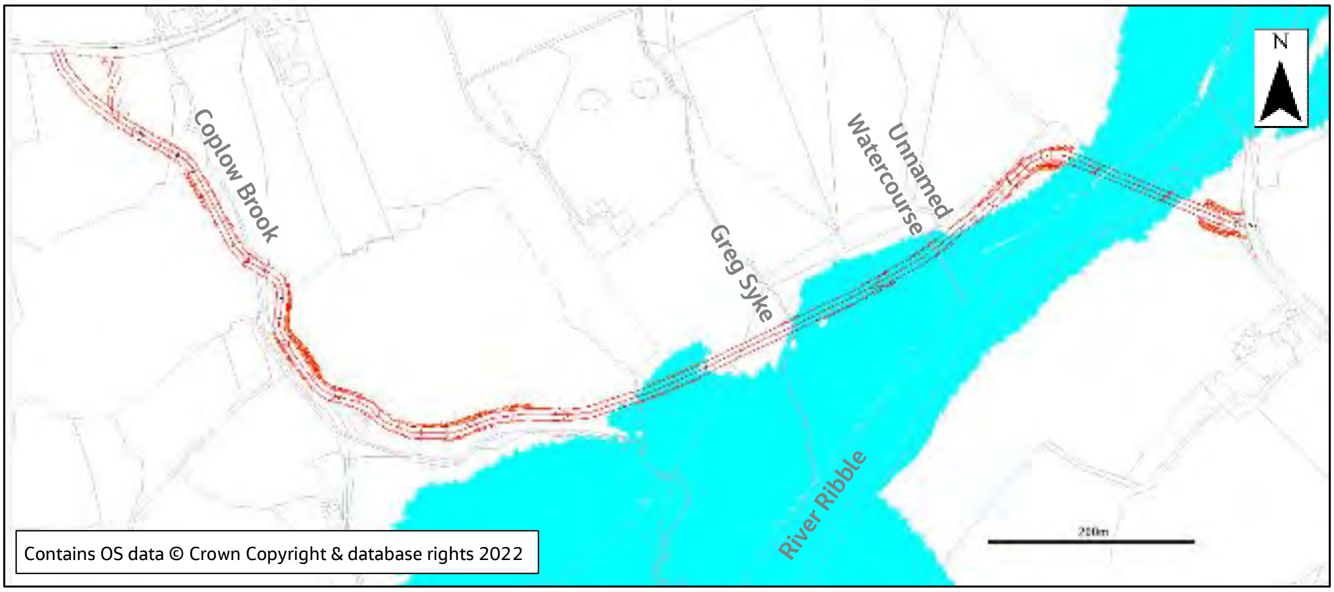


Figure 7-1: Proposed Ribble Crossing haul road route (United Utilities 2021) with Modelled 1%AEP (1 in 100) flood extent.

7.1 Model Schematisation

90) The proposed River Ribble bridge soffit level of 63.6 mAOD is designed to be clear of the 1 % AEP (1 in 100) fluvial flood level, there was therefore no requirement to represent the bridge explicitly within the 1D model. The two piers are situated in the 2D domain and were represented in the 2D model with a flow constriction applied to the relevant 2D model cells.

91) The bridge abutments and haul road surface were introduced to the Ribble model and the tributary models using a Z-tin (triangular irregular network), extracted from the 3D CAD drawing, combined with additional Z-shape topographic adjustment polygons, to adjust the required finished road levels at each of the tributary crossings. The model surface roughness was modified within the footprint of the road to represent the change from natural ground to road surface.

92) Bridge units with associated inline spill units were added to the relevant locations in the 1D tributary models, with the soffit levels tying into the bank tops. The finished road level for the tributary crossings was set to 0.6 m above the bank top levels in order to provide sufficient cover for heavy traffic. Figure

7-4 shows a long section of the road running from upstream to downstream of the River Ribble (east to west). Individual cross-sections of each crossing can be found in Appendix B.

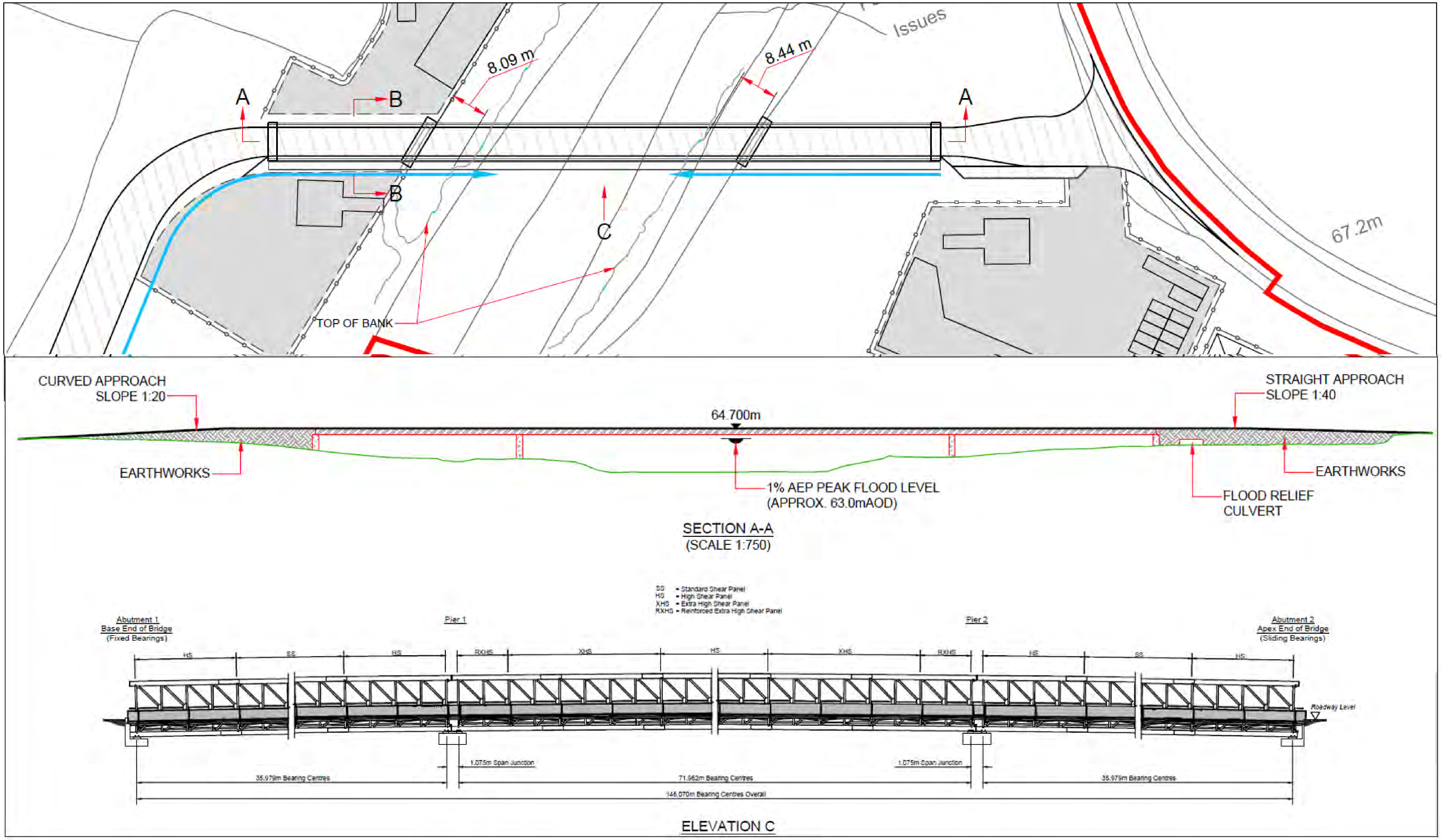


Figure 7-2: Proposed Ribble Crossing bridge outline design drawing: 80061155-01-JAC-TR4-97-DR-C-00008 (United Utilities 2021)

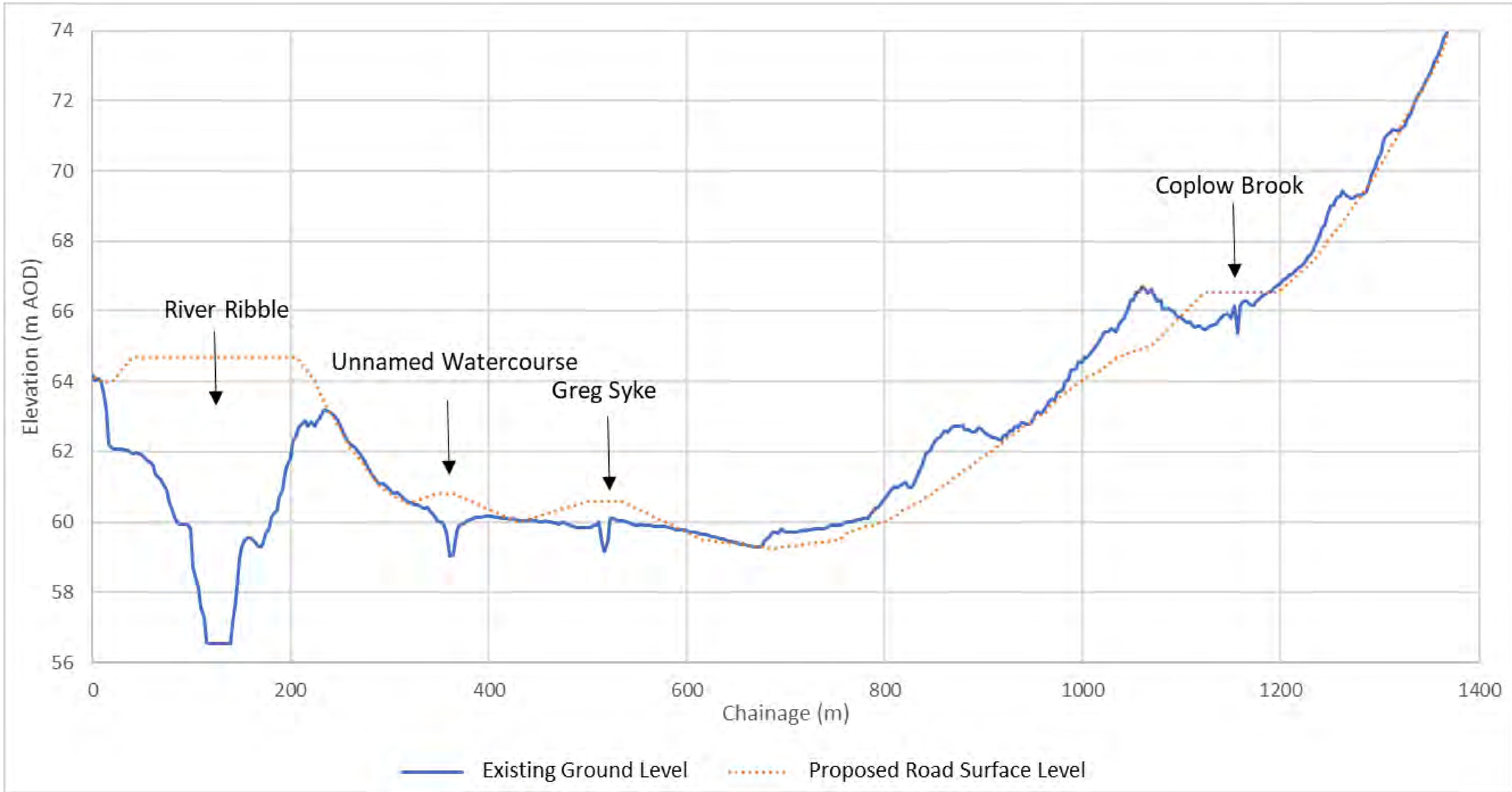


Figure 7-4: Long section of proposed road levels compared to existing road levels going from upstream to downstream.

8. Model Results

8.1 Baseline Results

River Ribble Model

93) The River Ribble in-channel maximum water levels have been inspected at key locations (see Figure 8-1) in relation to the Proposed Ribble Crossing. Table 8-1 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

94) Figure 8-1 shows the maximum flood depths and velocity vectors for the River Ribble Baseline1 % AEP event. The flood extents for all modelled events are shown in Appendix D.

95) The model results show that the river floodplain is inundated for all of the events simulated. The higher events exhibit significant flooding, with the wider river valley operating in a fully channelised fashion. Maximum floodplain depths in the 1 % AEP (1 in 100) event are as high as 2 m at locations where the cross section is narrower. The land affected is predominantly farmland and some small patches of woodland. The 1 % AEP (1 in 100) event causes external flooding to one property on Clitheroe Road, inundating the front and rear gardens and driveway.

96) For the 1 % AEP (1 in 100) event the West Bradford Road Bridge runs relatively full, with water levels high up the arches, but does still have almost 1 m freeboard from the maximum water level to the crown of the main arches. The downstream Waddington Road Bridge has greater capacity with almost 2 m of freeboard from the maximum water level to the crown of the arches.

97) The location of the proposed river crossing is well selected for flood risk. The out of bank ground levels are relatively high and flooding is limited to a narrow area immediately alongside each bank. The baseline 1 %AEP (1 in 100) maximum water level at this location is 60.86 m AOD. The soffit of the proposed bridge is in excess of 63.00 m AOD, which provides a significant freeboard allowance of over 2 m.

98) The footprint of the haul road route intersects the modelled flood extent for events above the 10 % AEP event, at low lying portions of the hillside between 150 and 500 m downstream from the proposed bridge crossing.

Unnamed Watercourse Model

99) The Unnamed Watercourse in-channel maximum water levels have been inspected at key locations (see Figure 8-2) in relation to the Proposed Ribble Crossing and Table 8-2 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

100) Figure 8-2 shows the maximum flood depths and velocity vectors for the Unnamed Watercourse Baseline 1 % AEP event. The flood extents for all modelled events are shown in Appendix D.

101) The floodplain around the downstream end of the model has minor flooding as a result of the modelled downstream boundary water level on the River Ribble. Flooding from the Unnamed Watercourse itself occurs from a low point on the right bank of the watercourse, around the proposed crossing location for events at or larger than the 1.33 % AEP (1 in 75) event, where high flows spill from the right bank about 50 m upstream from the Proposed Ribble Crossing location, flooding some low-lying ground including a small area intersecting with the proposed haul road footprint.

Greg Syke Model

102) The Greg Syke in-channel maximum water levels have been inspected at key locations (see Figure 8-3) in relation to the Proposed Ribble Crossing and Table 8-3 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

103) Figure 8-3 shows the maximum flood depths and velocity vectors for the Greg Syke Baseline 1 % AEP event. The flood extents for all modelled events are shown in Appendix D.

104) The River Ribble 50 % AEP (1 in 2) event was adopted as the downstream boundary condition for the Greg Syke Model. The Greg Syke model was allowed to run with just the downstream boundary water level, prior to initiating the design storm on Greg Syke itself, in order to allow the 2D floodplain in the lower portion of the Greg Syke model to fill, as would occur during the River Ribble 50 % AEP (1 in 2) event.

105) The model results indicate that Greg Syke spills from its right bank around the proposed crossing location, for events at or of larger magnitude than the 50 % AEP (1 in 2) event and flows down the hillside alongside the watercourse. The hillside topography routes portion of the flood water westwards towards Coplow Brook.

Coplow Brook Model

106) The Coplow Brook in-channel maximum water levels have been inspected at key locations (see Figure 8-4) in relation to the Proposed Ribble Crossing and Table 8-4 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

107) Figure 8-4 shows the maximum flood depths and velocity vectors for the Coplow Brook Baseline 1 % AEP event. The flood extents for all modelled events are shown in Appendix D.

108) The model results indicate that Coplow Brook runs out of bank around the proposed crossing location, for events at or larger than the 3.33 % AEP (1 in 30) event. On the left bank water surcharges out of channel and loops back into channel a short way downstream. Modelled flood water also fills back up an ephemeral tributary of Coplow brook on the left bank immediately upstream of the haul road.

109) The main flood risk from Coplow Brook is due to out of bank flooding on the right bank between 20 and 100 m downstream from the proposed crossing location. Events at or larger than the 20 % AEP (1 in 5) event flow downhill alongside and roughly parallel to the brook. The floodplain topography direct flows towards a farmhouse property which is within the modelled flood extent for events at or larger than the 20 % AEP (1 in 5) event.

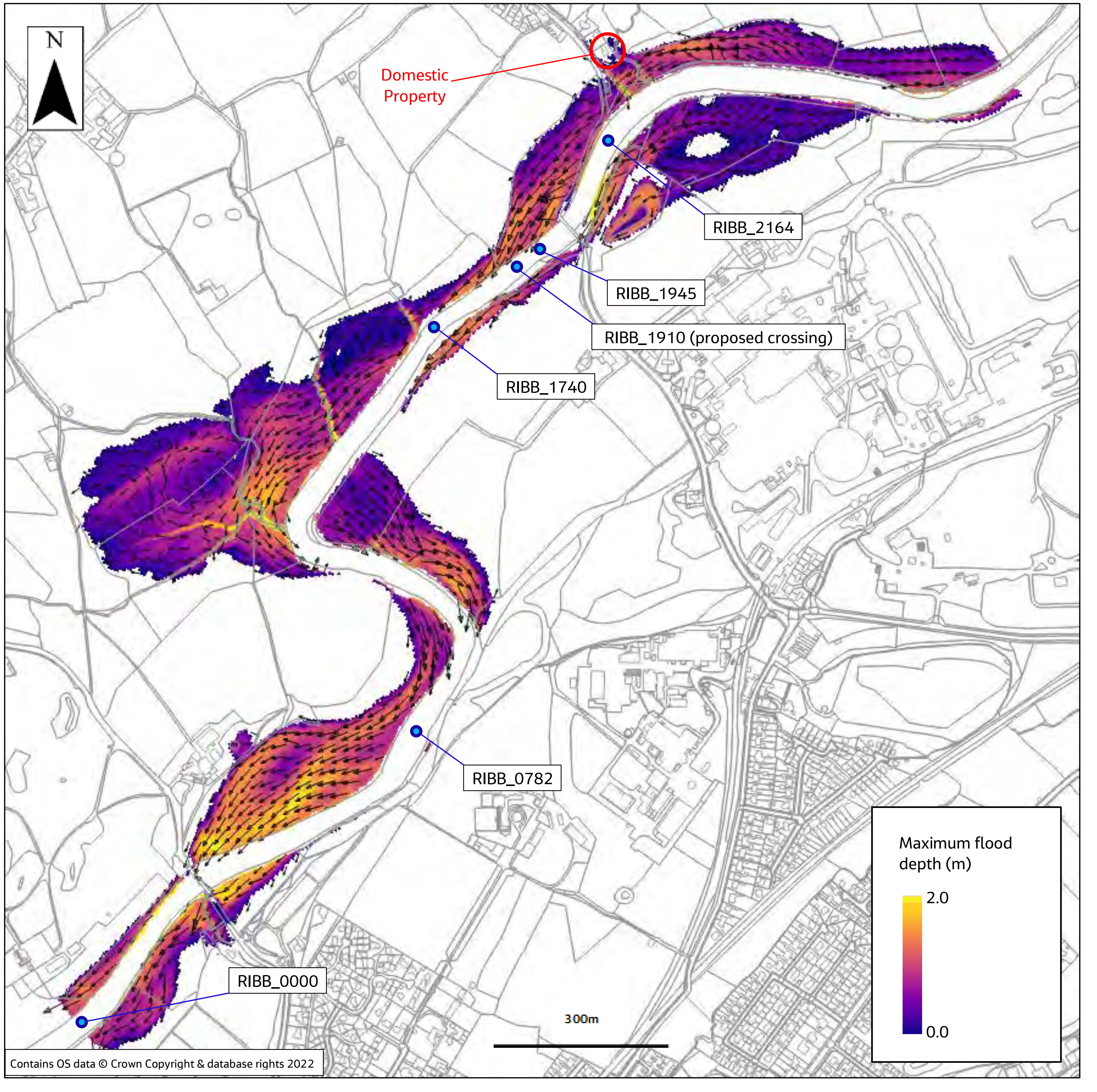


Figure 8-1: River Ribble Baseline 1% AEP event modelled maximum flood depth and velocity vectors.

Figure 8-2: Unnamed Watercourse Baseline 1% AEP event modelled maximum flood depth and velocity vectors.

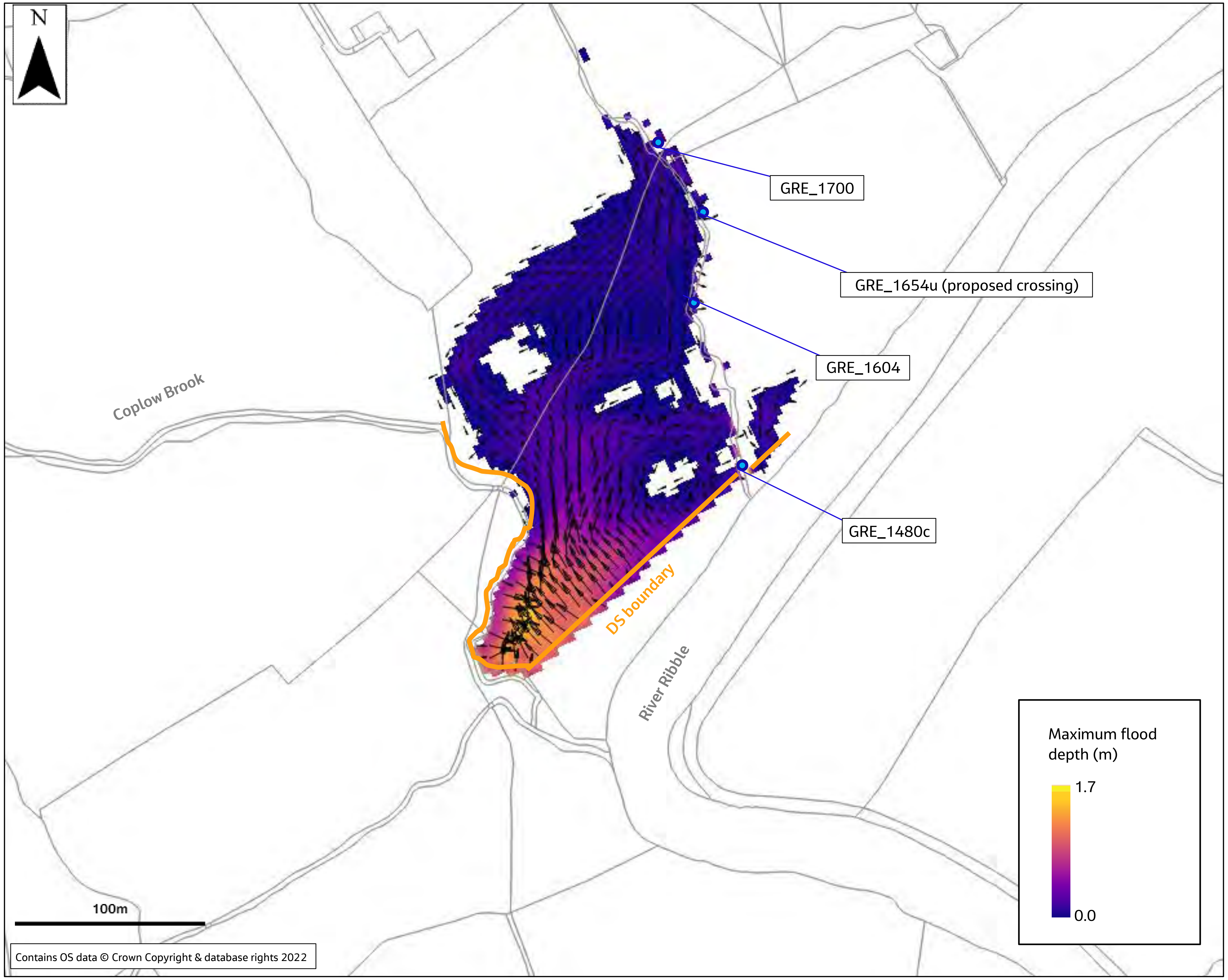


Figure 8-3: Greg Syke Baseline 1% AEP event modelled maximum flood depth and velocity vectors

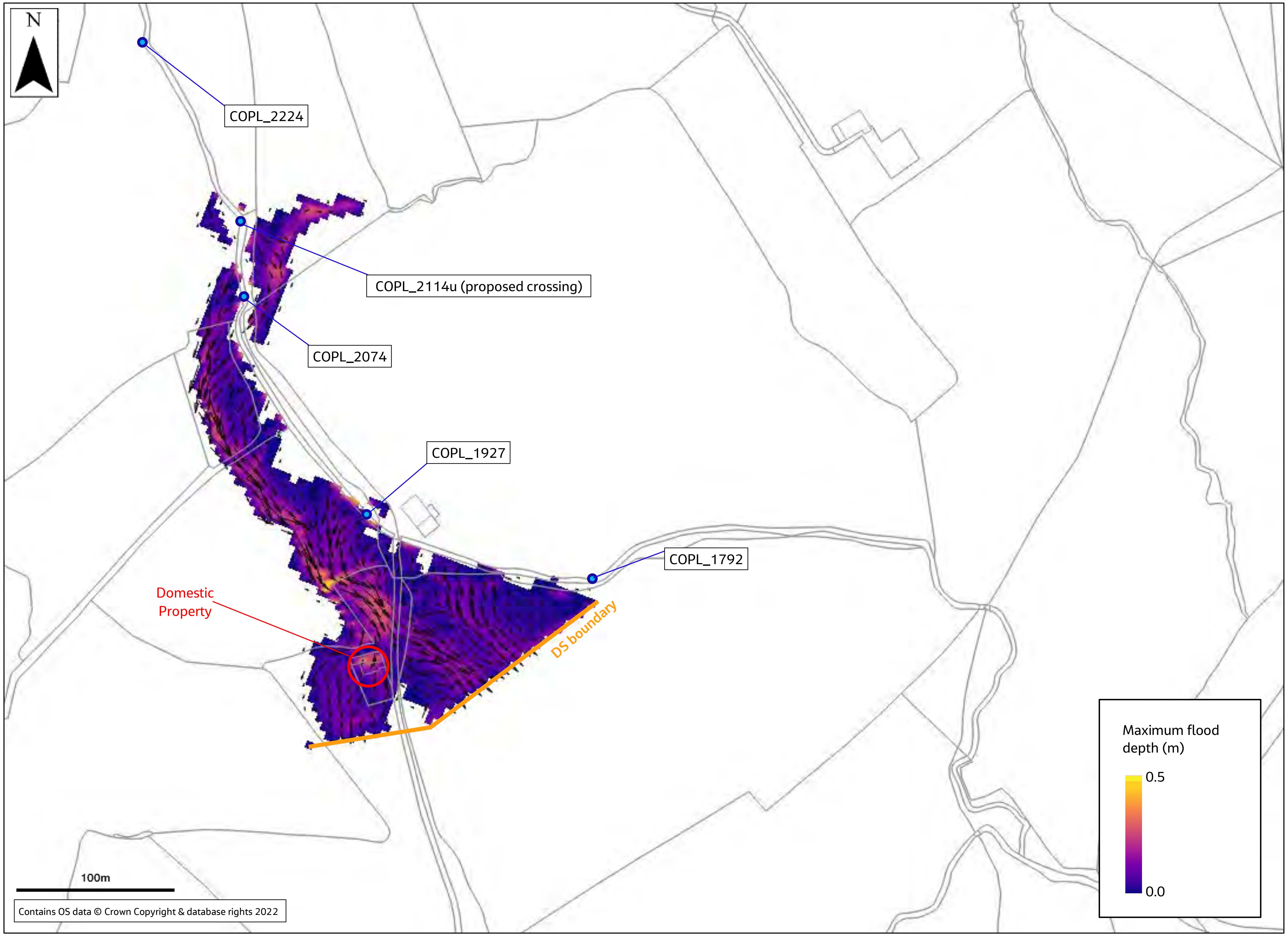


Figure 8-4: Coplow Brook Baseline 1% AEP event modeled maximum flood depth and velocity vectors.

8.2 With Proposed Ribble Crossing Results

River Ribble Model

110) The River Ribble in-channel maximum water levels have been inspected at key locations in relation to the Proposed Ribble Crossing. Table 8-1 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C. The results show an insignificant tailwater effect upstream of the temporary bridge, in the order of +2 mm immediately upstream of the bridge, and some minor increases of up to 5 mm on the watercourse downstream from the bridge. Further downstream the model shows zero change in maximum water level, i.e. over the lower 1.5 km of the modelled reach.

111) Figure 8-5 shows the difference in modelled maximum flood level caused by the implementation of the Proposed Ribble Crossing for the 1 % AEP event. Water level differences maps for all AEP events can be found in Appendix E. This analysis demonstrates that there is a small localised tailwater upstream of each abutment and some redistribution of flood depths in the right-hand side floodplain where the haul road cut and fill has been implemented. Inspection of the model results show that for all locations away from the proposed development elements, there is negligible change in modelled water levels, including the property on Clitheroe Road.

Unnamed Watercourse Model

112) The Unnamed Watercourse in-channel maximum water levels have been inspected at key locations in relation to the Proposed Ribble Crossing. Table 8-2 shows the in-channel maximum water levels for the 1 % AEP (1 in 100) event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

113) The 1D model results show no change to flow or stage in the channel as a result of the implementation of the Proposed Ribble Crossing.

114) Figure 8-6 shows the difference in modelled maximum flood depth caused by the implementation of the Proposed Ribble Crossing for the 1 % AEP event. Water level differences maps for all AEP events can be found in Appendix E. The flooding on the hillside on the right bank out of channel area, that was noted for the baseline scenario 1 % AEP (1in 100) and 1.33 % AEP (1 in 75) events, is exacerbated by the haul road levels that are required to install the temporary bridge across the Unnamed Watercourse. However, the haul road does not over top for any of the events modelled. It is recommended that a lateral surface drainage pipe is required beneath the haul road at the lowest point in the floodplain to drain both flood water and to ensure the basin formed uphill of the haul road does not become a pond. It is presumed that the required perimeter drainage will be addressed at detailed design phase and will not affect the key findings of the fluvial flood modelling related here.

Greg Syke Model

115) The Greg Syke in-channel maximum water levels have been inspected at key locations in relation to the Proposed Ribble Crossing.

116) Table 8-3 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

117) The 1D model results show a very minor in-channel tailwater effect due to the implementation of the watercourse crossing, along with some water level and flow reduction on the reach downstream from the temporary bridge. The model shows that the bridge runs clear with approximately 0.93 m of freeboard to the soffit in the 1 % AEP (1 in 100) event.

118) Figure 8-7 shows the difference in modelled maximum flood depth caused by the implementation of the Proposed Ribble Crossing for the 1 % AEP event. Water level differences maps for all AEP events can be found in Appendix E. The flooding of the right bank out of channel area, that was noted for the baseline scenario, is exacerbated by the elevated road levels that are required to install the temporary bridge across Greg Syke. Pass forward flows down the hillside are re-distributed somewhat by the haul

road topography, with increased flow routed around the far right of the Proposed Ribble Crossing causing increased flood depths. This is combined with a reciprocal reduction in flow and depth in the area immediately downhill from the crossing. The bottom half of the model shows only insignificant change in water level due to the implementation of the Proposed Ribble Crossing.

Coplow Brook Model

119) The Coplow Brook in-channel maximum water levels have been inspected at key locations in relation to the Proposed Ribble Crossing.

120) Table 8-4 shows the in-channel maximum water levels for the 1 % AEP event. The in-channel water levels at key locations for all modelled events are shown in Appendix C.

121) The 1D model results show the tailwater effect from the bridge crossing, which extends approximately 30 m upstream from the proposed bridge location. There is a small reduction of in-channel flow downstream from the Proposed Ribble Crossing due to the left out of bank flow path. However, the lower portion of the modelled reach, downstream from the location where the left bank flows re-join the channel, exhibits no change in water level or flow from the baseline model, aside from a small lag in hydrograph timing.

122) Figure 8-8 shows the difference in modelled maximum flood depth caused by the implementation of the Proposed Ribble Crossing for the 1 % AEP event. Water level differences maps for all AEP events can be found in Appendix E. This analysis shows that the tailwater effect from the temporary bridge and haul road structure causes an increase in the modelled flood extent immediately upstream from the crossing, particularly in the low-lying field on the left bank of the brook.

123) The raised road levels required to form the Coplow brook crossing, along with the finished road levels assigned to the portion of haul road which runs alongside the left bank of the brook, causes out of bank flows, which run along the haul road itself for approximately 230 m before re-joining the brook. The haul road is flooded in this location for events larger than the 10 %AEP (1 in 10) event. Where flow on the road re-joins the brook, it inundates a farm building (assumed to be a barn) to depths of up to 80 mm in a 3.33 % AEP (1 in 30) event, increasing to 120 mm in a 1 % AEP (1 in 100) event.

124) The Proposed Ribble Crossing does not worsen flooding to the property on the right floodplain that floods in the baseline scenario from a 20 % AEP (1 in 5) event.

125) As a result of the flood volume spilled upstream of the crossing and the redistributed pass forward flow from the Proposed Ribble Crossing there is a minor reduction in flood risk to the right-hand side out of bank floodplain where the farmhouse property is located.

Coplow Brook Mitigation Scenario

126) The potential impact of the Proposed Ribble Crossing on the barn for the 3.33 % AEP (1 in 30) event and higher magnitude events meant mitigation measures are deemed necessary.

127) Measures to manage exceedance flow along the haul road would need to be developed by the contractor as part of the detailed design and could include:

- Measures to divert flow off the road and back into Coplow Brook before flow reaches the barn such as small changes to the vertical alignment of the road or cross drainage to divert flows into Coplow Brook
- Measures to retain flow within the haul road and route flow away from the barn and towards the River Ribble such as a low roadside bund.

128) Whilst the flow path along the haul road would remain, initial testing of these measures using the hydraulic model has demonstrated that either of these would be effective and would ensure that the residual impact of flood risk to the barn would be negligible.

129) Figure 8-9 shows the impact of adding a speed hump with a height of 150 mm across the road, as an example of a potential mitigation option.

Table 8-1: River Ribble in-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)*
RIBB_2164	154m upstream from Temporary Bridge	61.492	61.493	0.001
RIBB_1945	35m upstream from Temporary Bridge	60.960	60.963	0.003
RIBB_1910	Location of Temporary Bridge	60.859	60.861	0.002
RIBB_1740	70m downstream from Temporary Bridge	60.379	60.384	0.005
RIBB_0782	1.2km downstream from Temporary Bridge	58.093	58.093	0.000
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.967	56.967	0.000
* With Proposed Ribble Crossing scenario minus Baseline scenario				

Table 8-2: Unnamed Watercourse in-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)*
UNN01_1808	110m upstream from Temporary Bridge	60.813	60.813	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.863	59.863	0.000
UNN01_1755	40m downstream from Temporary Bridge	59.488	59.487	-0.001
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000
* With Proposed Ribble Crossing scenario minus Baseline scenario				

Table 8-3: Greg Syke in-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)*
GRE_1700	46m upstream from Temporary Bridge	61.230	61.230	0.000
GRE_1654u	Immediately upstream of Temporary Bridge	59.905	59.907	0.002
GRE_1604	50m downstream from Temporary Bridge	59.074	59.057	-0.017
GRE_1480c	174m downstream from Temporary Bridge	58.922	58.922	0.000

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)*
GRE_1700	46m upstream from Temporary Bridge	61.230	61.230	0.000
	Model d/s boundary			
* With Proposed Ribble Crossing scenario minus Baseline scenario				

Table 8-4: Coplow Brook in-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)*
COPL_2224	110m upstream from Temporary Bridge	68.327	68.326	-0.001
COPL_2114u	Immediately upstream of Temporary Bridge	66.204	66.636	0.432
COPL_2074	40m downstream from Temporary Bridge	65.553	65.546	-0.007
COPL_1927	147m downstream from Temporary Bridge	62.985	62.985	0.000
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000
* With Proposed Ribble Crossing scenario minus Baseline scenario				

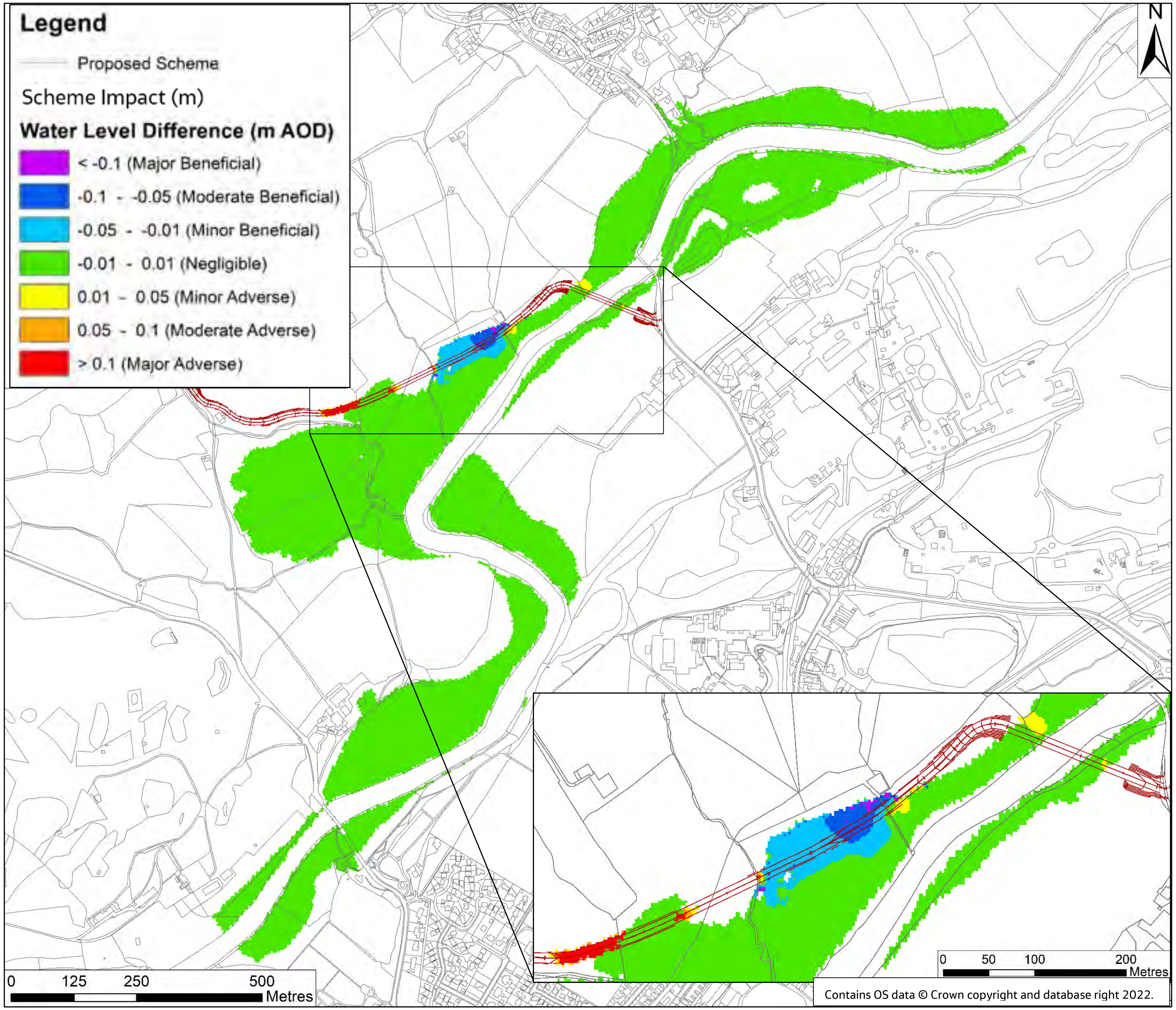


Figure 8-5: River Ribble Flood level difference map for the 1%AEP (1 in 100) event. With Proposed Ribble Crossing scenario minus Baseline scenario.

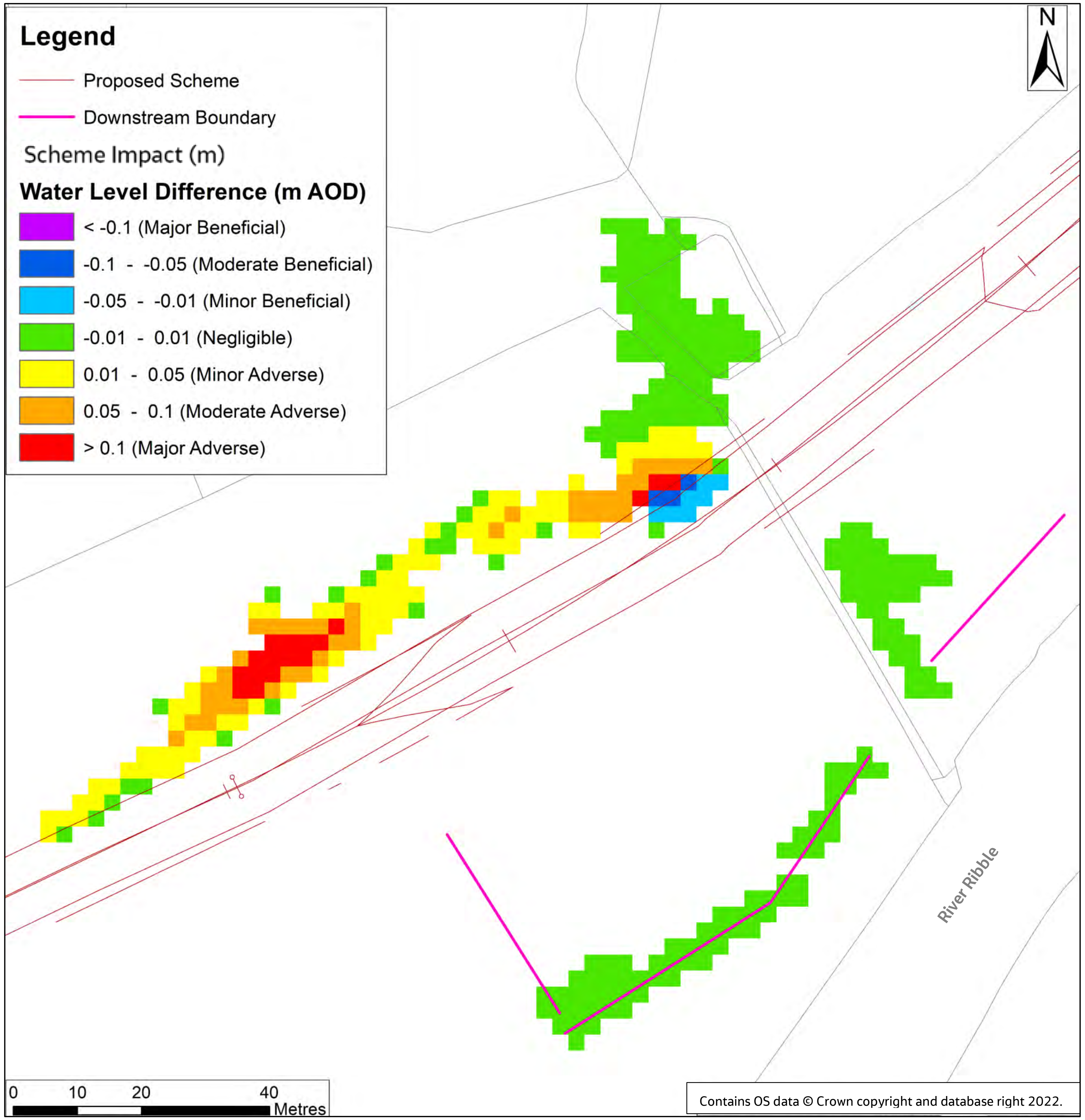


Figure 8-6: Unnamed Watercourse Flood level difference map for the 1%AEP (1 in 100) event. With Proposed Ribble Crossing scenario minus Baseline scenario.

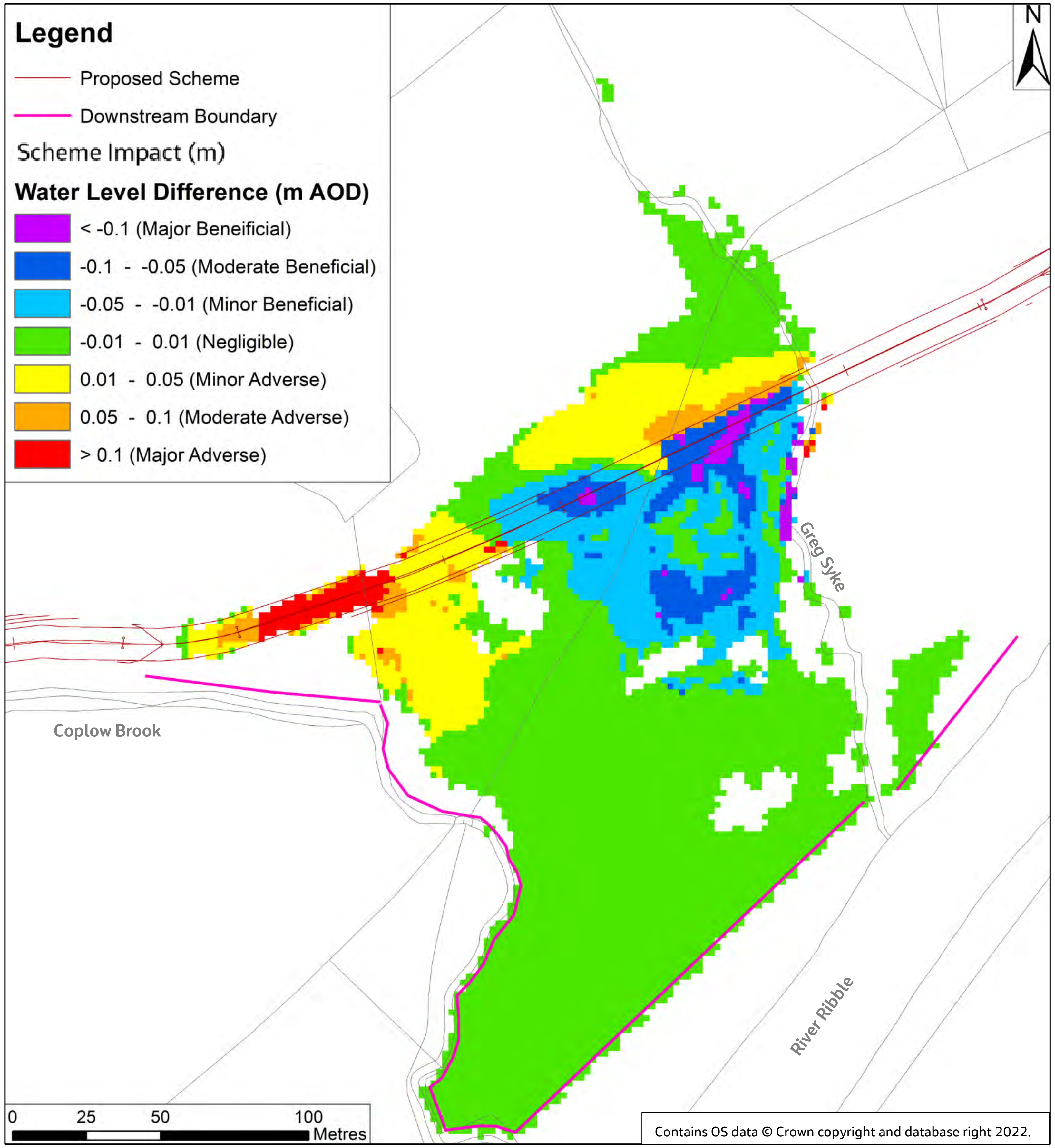


Figure 8-7: Greg Syke flood level difference map for the 1%AEP (1 in 100) event. With Proposed Ribble Crossing scenario minus Baseline scenario.

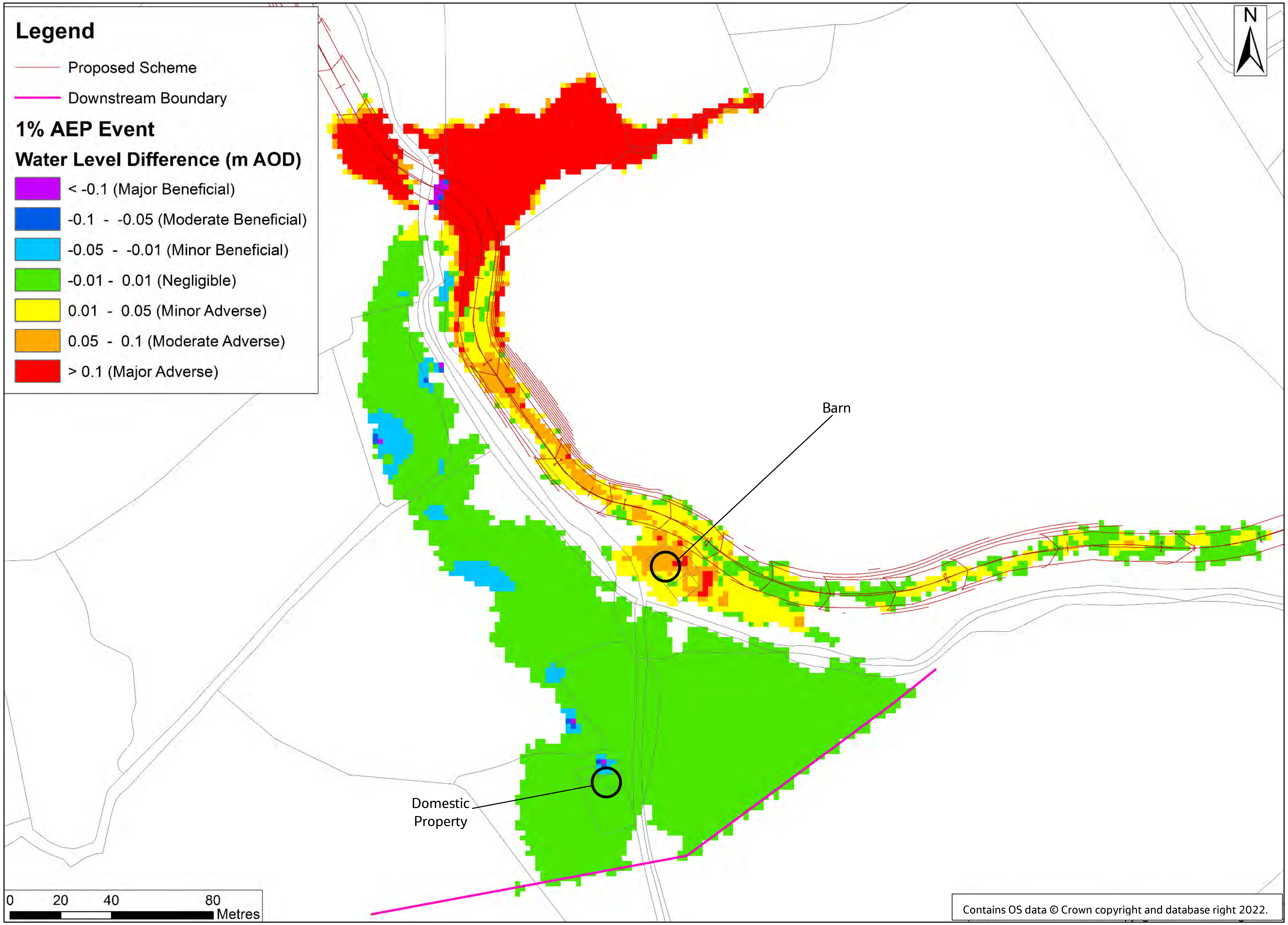


Figure 8-8: Coplow Brook Flood level difference map for the 1%AEP (1 in 100) event. With Proposed Ribble Crossing scenario minus Baseline scenario.

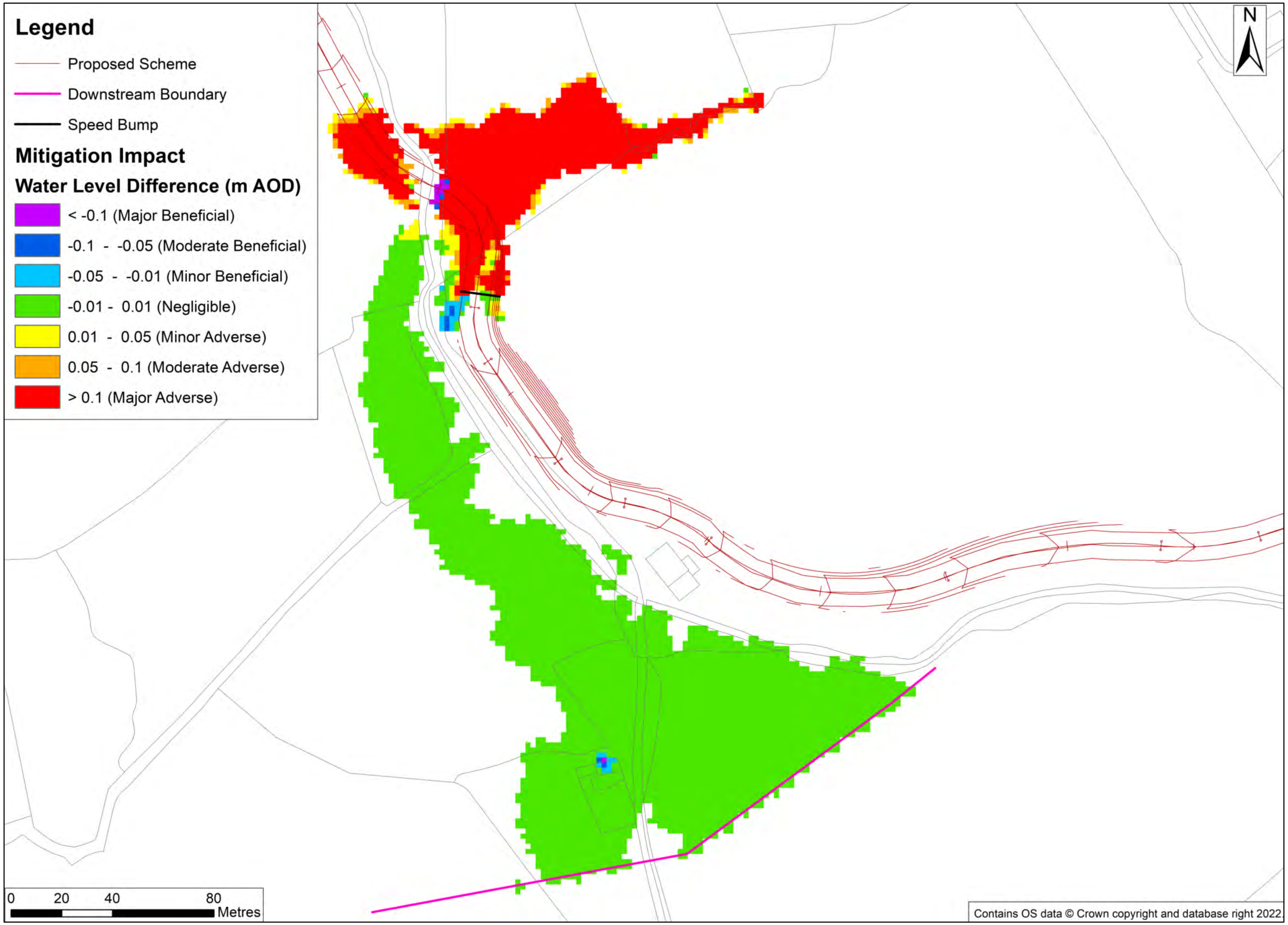


Figure 8-9: Coplow Brook Flood level difference map for the 1%AEP (1 in 100) event. Mitigation scenario minus Baseline scenario

9. Model Assumptions and Limitations

130) The accuracy and validity of the hydraulic model results are heavily dependent on the accuracy of the hydrological and topographic data included in the models. While the most appropriate available information has been used to construct the models to represent fluvial flooding mechanisms, there are uncertainties and limitations associated with the models. These include assumptions made as part of the model build process.

131) Efforts have been made to assess and reduce levels of uncertainty in each aspect of the modelling process. The assumptions made are considered to be generally conservative for modelled water levels at the Proposed Ribble Crossing and are therefore appropriate for the flood risk assessment.

9.1 Hydrology

132) The key sources of uncertainty and the limitations associated with the hydrological analysis undertaken for the River Ribble and the tributaries are outlined in Section 6.3 of the Hydrology report (see Appendix A).

9.2 Hydraulic Modelling

133) The key sources of uncertainty and the limitations associated with the modelling undertaken for the River Ribble and the tributaries are as follows:

- Channel roughness has been assigned using the best available information (survey data and aerial photographs). The roughness values used are based on available guidance (Chow, 1959)
- Hydraulic coefficients for structures have been applied using available guidance within the Flood Modeller and TUFLOW software. The dimensions for structures have been based on detailed survey measurements
- A normal depth boundary was used at the downstream end of the Ribble model. The sensitivity analysis has shown that changes to the downstream boundary do not impact upon modelled levels at the area of interest. Therefore, it was deemed appropriate to use a normal depth boundary
- A normal depth boundary was used at the downstream end of the Coplow Brook model. No sensitivity analysis has been undertaken to assess whether the downstream boundary has an impact upon modelled levels at the area of interest. However, the downstream boundary is located approximately 320 m away from the proposed crossing and is based on a steep bed slope. The boundary assumption is not expected to influence the results at the area of interest
- A water level boundary applying the 50 % AEP event peak water level within the Ribble was used at the downstream end of the Unnamed Watercourse and Greg Syke models. No sensitivity analysis has been undertaken to assess whether the downstream boundary has an impact upon modelled levels at the area of interest. However, the boundary conditions chosen were deemed to be conservative, assuming that 1 % AEP peak flows in the Ordinary Watercourses coincide with 50 % AEP peak flows in the River Ribble
- The LiDAR data is assumed to appropriately represent the floodplain
- A 5 m grid has been used for the Ribble and Unnamed Watercourse models. A 4 m grid has been used for the Greg Syke and Coplow Brook models. These grid cell sizes are deemed to provide a sufficient level of detail to represent floodplain topography and flooding mechanisms demonstrated by the models
- No calibration of the model was possible due to lack gauged information within the modelled area.

10. Conclusion

134) This report has detailed the modelling carried out to assess the flood risk for the River Ribble and three tributaries with reference to the location of the Proposed Ribble Crossing.

135) The model results for the River Ribble baseline scenario have shown that the Ribble valley experiences significant flood risk in the immediate floodplain during the design events simulated.

136) Model simulations were carried out to assess the effect of the Proposed Ribble Crossing. These results showed that the temporary road bridge causes a minor localised tailwater effect from the abutments. The bridge soffit runs clear with a significant freeboard of over 2 m to the modelled 1 % AEP (1 in 100) event maximum water level.

137) There is some minor redistribution of flow paths and depths at the periphery of the flood extent where ground levels are modified for the installation of the haul road. However, at locations downstream of the proposed works there is negligible change to modelled flow and water level.

138) At the Unnamed Watercourse there is only limited out of bank flooding, but this does occur in the location of the proposed haul road footprint. The 'with Proposed Ribble Crossing' modelling shows that the haul road blocks the hillside overland flow path on the right (west) of the channel. Floodwater is held back and fills a depression on the hillside, which is pre-existing, but made more significant by the haul road topography. The modelled floodwater does not overtop the haul road and it is recommended that a drainage solution is required in this area to ensure both floodwater and general surface water does not permanently pond uphill from the road. There is no significant effect on pass forward flow from the Unnamed Watercourse into the River Ribble.

139) At Greg Syke the baseline event shows significant flooding of the right bank side (east) of the channel. This flood water fills a depression in the hillside then runs down to the River Ribble. The implementation of the haul road and bridge crossing causes some backing up of floodwater in the floodplain on the uphill side of the road, affecting the right out of bank area. Floodwater overtops the haul road on the far-right floodplain, near to Coplow Brook. This flooding affects the haul road for events in excess of 50 % AEP (1 in 5). There is no significant effect on pass forward flow from Greg Syke into the River Ribble.

140) At Coplow Brook there is significant flooding of the hillside to the right bank side (east) of the channel and there is one domestic property within the baseline modelled flood extent. The implementation of the haul road and bridge crossing causes some backing up of flood water on the uphill side of the road. The proposed works also divert some flood water along a new flow path along the haul road itself on the Coplow Brook left out of bank, parallel to the watercourse channel. This flood water returns to the brook a short distance downhill from the crossing, where it floods a barn for the 3.33 % AEP (1 in30) event and higher magnitude events. The new floodplain flows on the left out of bank cause a reduction in flow within the right (west) side floodplain, conferring minor flood betterment to the single property. There is no significant effect on pass forward flow from Coplow Brook into the River Ribble. Results of the mitigation modelling demonstrate that measures can be put in place to avoid flooding of the barn for all AEP events simulated.

11. References

Chow, V.T. (1959) Open Channel Hydraulics. McGraw-Hill, New York.

Appendix A. Hydrology Calculation Record

Flood estimation report:

Environment Agency – Ribble Model Update

Introduction

This report provides a record of the hydrological context, the method statement, the calculations and decisions made during flood estimation and the results.

Contents

Approval

Revision stage	Analyst / Reviewer name & qualifications	Amendments	Date
Method statement preparation	K. Bhattarai, PhD (Hydrology), CEng MIEI, MCIWEM CWEM		04/03/2021
Method statement sign-off			
Calculations preparation	M.McParland K.Samson MA, MSc, CEnv		21/04/2021
Ccalculations sign-off	K. Bhattarai, PhD (Hydrology), CEng MIEI, MCIWEM CWEM		14/05/2021

Abbreviations

AEP ....Annual Exceedance Probability

AM..Annual Maximum

AREA ....Catchment area (km^2)

BFI ....Base Flow Index

BFIHOST ....Base Flow Index derived using the HOST soil classification

CPRE......................................Council for the Protection of Rural England

FARL......................................FEH index of flood attenuation due to reservoirs and lakes

FEH ....Flood Estimation Handbook

FSR...Flood Studies Report

HOST.....................................Hydrology of Soil Types

NRFA ....National River Flow Archive

OS...Ordnance Survey

POT ....Peaks Over a Threshold

QMED....................................Median Annual Flood (with return period 2 years)

ReFH......................................Revitalised Flood Hydrograph method

ReFH2 ....Revitalised Flood Hydrograph 2 method

SAAR ....Standard Average Annual Rainfall (mm)

SPR...Standard percentage runoff

SPRHOST..............................Standard percentage runoff derived using the HOST soil classification

Tp(0) ....Time to peak of the instantaneous unit hydrograph

URBAN...................................Flood Studies Report index of fractional urban extent

URBEXT1990.......................FEH index of fractional urban extent

URBEXT2000.......................Revised index of urban extent, measured differently from URBEXT1990

WINFAP-FEH ....Windows Frequency Analysis Package – used for FEH statistical method

1 SUMMARY OF ASSESSMENT

1.1 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections.

Catchment location	The study area is located within the Ribble Catchment, in North Yorkshire and Lancashire.
Purpose of study and scope	As part of United Utilities Haweswater Aqueduct Resilience Programme (HARP), a temporary construction access road is to be constructed from the West Bradford Road, just south of the existing River Ribble crossing, to the same road just west of the existing Coplow Brook crossing, east of Waddington. To inform the design of the proposed link road, hydraulic modelling is required to predict return period flood levels at the location of the proposed crossing(s).
Key catchment features	The catchments are predominately rural and do not exhibit any unusual catchment features that limit the use of standard FEH methodologies.
Flooding mechanisms	The main source of flood risk is considered fluvial. Flooding is suspected to result from a combination of peak flows and flood volumes that exceed the channel capacity. Flooding is also suspected to be influenced by the timing of flood peaks i.e. peak flood flows coinciding in contributing watercourses.
Gauged / ungauged	The study reach of the River Ribble is ungauged. The nearest gauge is the peak flow rated Ribble at Henthorn (NRFA station number 71006) station which is located approximately 5km downstream of the study area. No gauge exists on any of the minor watercourses considered in the study.
Final choice of method	For deriving the upstream inflow and the downstream target flow for the River Ribble, the FEH Statistical method is adopted. For the tributary inflows, the ReFH2 flows are adopted and will be reconciled to agree with the flood estimates derived by the FEH Statistical method at RBL_02.
Key limitations / uncertainties in results	Limitations to the study include the lack of gauged data for the tributary inflows. However as the tributary inflows will be scaled to agree with the estimated flow at RBL_02 (derived from gauged data) this is considered a minor limitation. The unnamed watercourse has a catchment area of 0.12km^2. There is considerable uncertainty estimating peak flows for very small catchments (<0.5km^2) and FEH methods were not originally developed with the intension of applying them to catchments smaller than this size.

1.2 Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval.

The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001

Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

2 METHOD STATEMENT

2.1 Requirements for flood estimates

<table>
<tr><td>Overview</td><td><u>Background</u>

As part of United Utilities Haweswater Aqueduct Resilience Programme (HARP), a link road is to be constructed joining the West Bradford Road, from just south of the existing River Ribble crossing, and the same road just west of the existing Coplow Brook crossing.

The proposed link road would cross the River Ribble approximately 100m to the west of the existing West Bradford Road crossing, as well as three other minor watercourses, namely, an unnamed watercourse, the Greg Sike, and the Coplow Brook.

The proposed link road alignment is shown in Figure A-1 within the Annex to this report.

Flood risk to the area is largely from fluvial sources whereby the capacity of watercourses are exceeded during intense rainfall events. Accordingly, a flood risk assessment and hydrological/hydraulic modelling is required.

<u>Purpose of study</u>

To inform the design of the proposed link road, hydraulic modelling is required to predict return period flood levels at the location of the proposed crossing(s).

<u>Model design event inputs</u>

As inputs to the hydraulic model, hydrological boundaries in the form of design fluvial hydrographs are required for the following AEP events:

50%, 20%, 10%, 33%, 1.33% and 1% AEP, equivalent to a 2yr, 5yr, 10yr, 30yr, 75yr and 100yr return period event, respectively.

It is understood that the proposed link road is temporary infrastructure and hence, no consideration of climate change is required. The locations where flood estimates are required are shown in Figure 3-1.

<u>Purpose of this document</u>

Section 2 ‘Method Statement’ of this document, summarises the tasks and the proposed approach and methodologies relating to deriving appropriate flood estimates and the required hydrological boundaries for input to the hydraulic model.

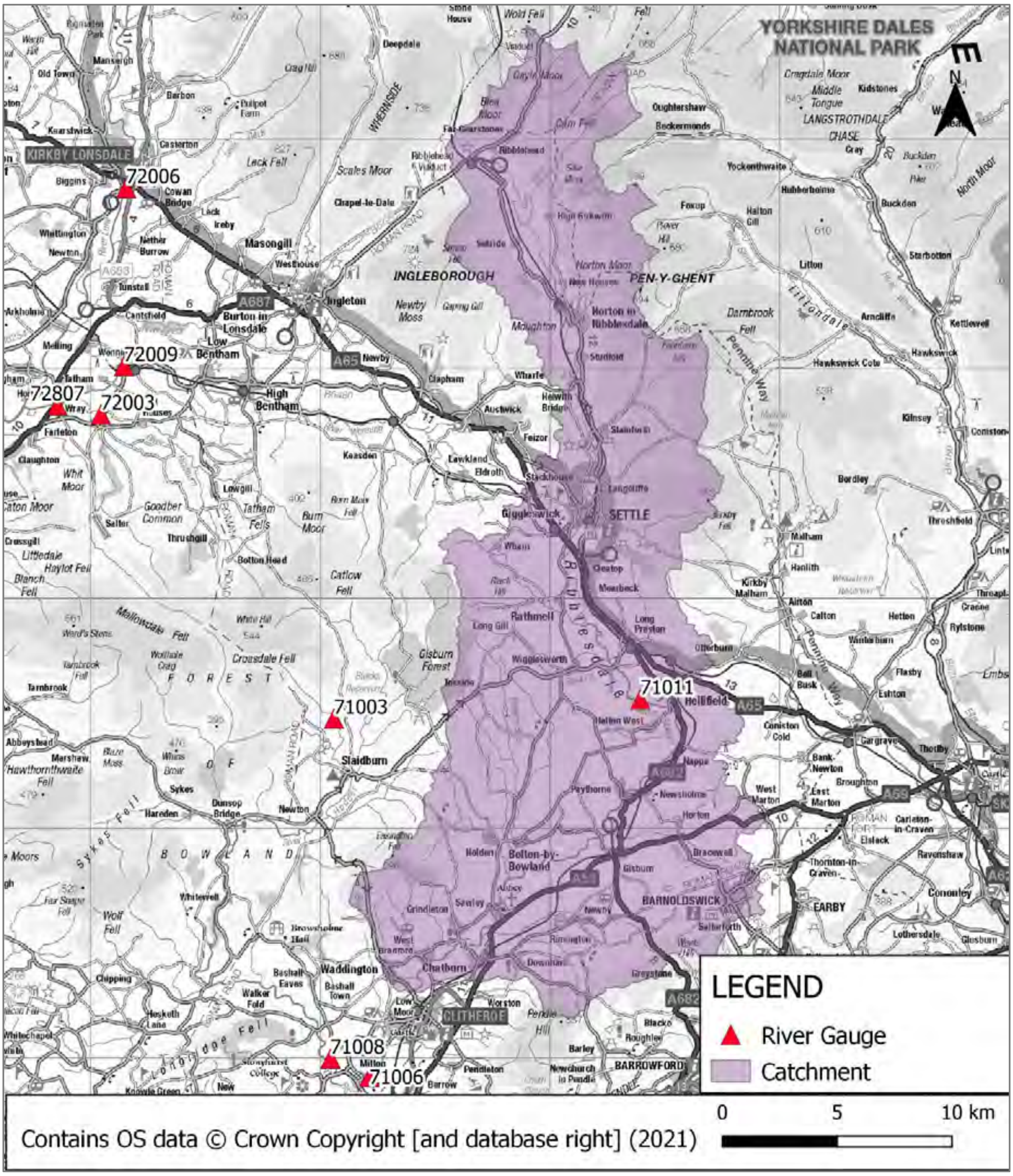
Subsequent sections of this document present the data made available for the study and provide a record of the calculations, decisions made and the results.</td></tr>
<tr><td>Project scope</td><td><u>Project Scope</u>
The project scope is ultimately to derive design return period flood levels at the location of the proposed crossing of the River Ribble.

Design inflow hydrographs are required for the hydraulic analysis and for the minor watercourses, are derived by the ReFH2 method for ungauged catcments. Where gauged data is available (i.e. the River Ribble), data recorded at Station 71006 is adopted to inform a representative hydrograph shape derived from averaging the five largest historic flood event hydrographs recorded at the gauge.

Design peak flows for the River Ribble are derived based on the FEH Statistical method whereas design peak flows for the minor watercourses are derived using both the FEH Statistical and ReFH2.3 methods and the most appropriate peaks adopted.</td></tr>
</table>

2.2 The catchment

The catchment is shown below alongside peak flow rated river gauges



Description	The Ribble originates high in the Pennines at Newby Head Moss at an altitude of 422m. At the downstream study extent, the River Ribble drains a catchment area of approximately 400 km². Catchment geology is mainly moderate permeability Carboniferous Limestone overlain by Boulder Clay in valleys with intermittent Millstone Grit. Soils are typically slowly permeable loamy and clayey soils, with wet peaty upland soils present in the upper catchment. The catchment contains extensive areas of rural and agricultural land particularly in the north and west. The lower catchment contains the settlements of Chatburn and Croft. There are also dispersed villages in the upper catchment, such as Settle, Long Preston and Hellifield. Average annual rainfall (SAAR) for the Ribble catchment, at the downstream study extent, is 1350mm.

2.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 9, released 24th September 2020. This contains data up to the end of September 2019 and provisional data for water year 2019/2020 at stations which set new records.

2.4 Gauging stations (flow or level)

Water-course	Station name	Gauging authority number	NRFA number	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start of record and end if station closed
Ribble	Ribble at Henthorn	710305	71006	456	Rated	30/08/1968 -

2.5 Data available at each flow gauging station in Table 2.4

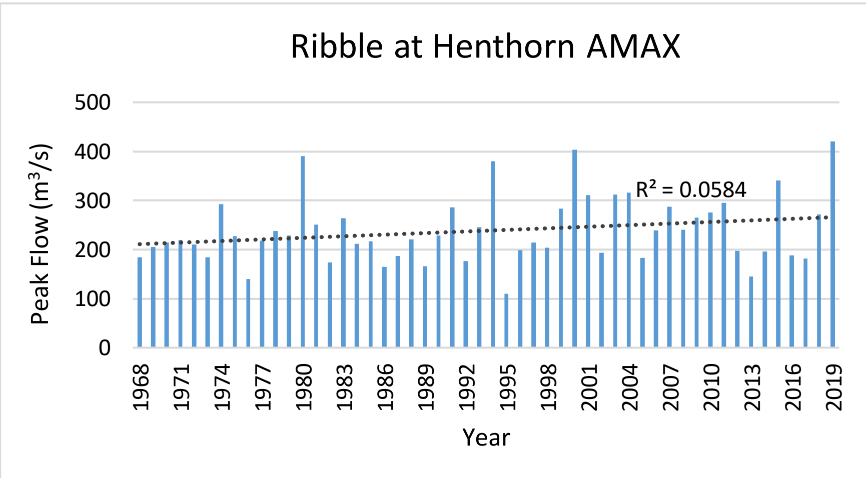
Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling ?	Data quality check needed?	Other comments on station and flow data quality
Ribble at Henthorn	30/08/1968 - 01/10/2020	No	Yes - Good fit to gauging.	Yes	N/A	Weir drowns at high flows, and insensitive at low flow due to its breadth. Algal build-up and leaks affect low flows. No bypassing reported.

2.6 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Check flow gaugings	No	N/A	N/A	No rating review is required or requested.
Historical flood data	Yes	No	Internet search	No formal historic flood data, in the form of recorded flood levels, has been provided. From a brief internet search it is understood that the West Bradford Road was closed in both directions during November 2020 due to flooding at Ribblesdale Works. During Storm Ciara which occurred in February 2020, graves at Clitheroe graveyard, just downstream of the study extent at NGR SD 73881 42555, were submerged. Severe flooding to the area was also recorded during Storm Eva which occurred in December 2015.

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Flow or river level data for events	Yes	Yes	Environment Agency	15-minute flow data has been provided by the Environment Agency for the River Ribble at Henthorn for the period of record 03/07/1985 to 23/02/2021.
Rainfall data for events	No	N/A	N/A	No event verification has been requested and hence utilising event rainfall data is outwith the scope of the current study.
Potential evaporation data	No	N/A	N/A	As above
Results from previous studies	Yes	No	N/Av	No previous studies have been identified following a brief internet search.
Other data or information	N/A	N/A	N/A	N/A

2.7 Hydrological understanding of catchment

Plots of flood peak data and interpretation	 The AMAX series as recorded at the Ribble at Henthorn (NRFA Station 71006) is plotted above. Fitting a simple linear trendline shows a potential increase in flood peaks over the period of record. 3 of the 10 largest flood events recorded at the gauge have occurred in the last 10 years with the largest AMAX event recorded, with a peak flow of 420.2 m^3/s, occurring on 09 February 2020.

Conceptual model	The main sites of interest are at the locations where the proposed link road crosses one of several watercourses. Figure A-1 shows the proposed link road would cross the River Ribble approximately 100m to the west of the existing crossing on the West Bradford Road, as well as three other minor watercourses, namely, an unnamed watercourse, Greg Sike and Coplow Brook. The main source of flood risk is considered fluvial. Flooding is suspected to result from a combination of peak flows and flood volumes that exceed the capacity of the channel. Flooding is also suspected to be influenced by the timing of flood peaks i.e. peak flood flows coinciding in contributing watercourses and; from culverted sections of the River Ribble and contributing watercourses that may not have sufficient hydraulic capacity to transmit water downstream during intense rainfall events, leading to out of bank flow.
Unusual catchment features	There are no catchment features that limit the applicability of applying standard FEH methodologies.

2.8 Initial choice of approach

Is FEH appropriate?	Yes, standard FEH methodologies are appropriate. The catchments within the study are considered rural and do not exhibit any unusual catchment features. The unnamed watercourse has a catchment area of approximately 0.12km^2 and hence FEH catchment descriptor data is not available. Flows for the unnamed watercourse (and Greg Sike) will be derived by scaling the flows of the Coplow Brook catchment by the ratio of catchment areas.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	Design peak flows for the River Ribble are derived based on the FEH Statistical method whereas design peak flows for the minor watercourses are derived using both the FEH Statistical and ReFH2.3 methods and the most appropriate peaks adopted. Design inflow hydrographs for the minor watercourses are derived by the ReFH2 method for ungauged catcments based on (1) the actual design storm (both duration and ARF) for the specific catchment and; (2) based on the design storm (duration and ARF) as calculated at the downstream study extent, i.e. the most downstream flow estimate on the River Ribble. For the main River Ribble inflow, data recorded at Station 71006 is adopted to inform a representative hydrograph shape derived from averaging the five largest historic flood event hydrographs recorded at the gauge. This is compared to the ReFH2 design hydrograph and the adopted hydrograph reported in Section 6. The wider Ribble catchment has been split into five sub-catchments. The sub-catchments (from upstream to downstream) are; • Ribble at West Bradford Road Crossing (392 km^2) • Unnamed watercourse (0.12 km^2)

	• Greg Sike (0.62 km^2) • Coplow Brook (1.39 km^2) • Waddington Brook (3.79 km^2) There is a residual catchment area of 1.25 km^2. This has been accounted for by multiplying the average specific discharge rate (m^3/s/km^2) calculated for the tributary branches of COP_01 and WAD_01, by the residual area of 1.25 km^2
Software to be used (with version numbers)	FEH Web Service[1] / WINFAP 4 / ReFH 2.3

[1] CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, UK.

3 LOCATIONS WHERE FLOOD ESTIMATES REQUIRED

3.1 Locations of flood estimation points (FEPs)

The locations of flood estimation points (FEPs) are shown in Figure 3-1, below.

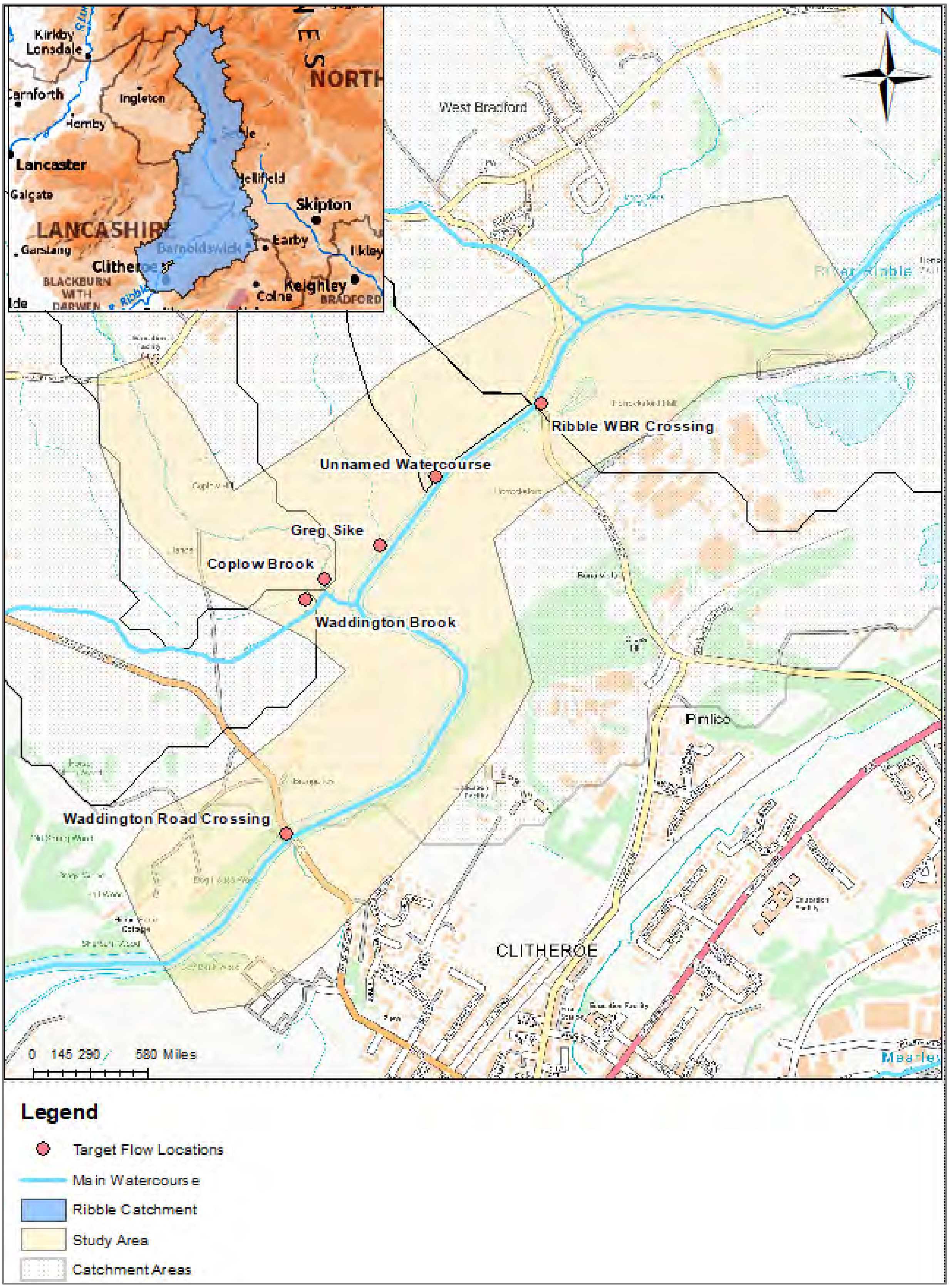


Figure 3-1 Locations of flood estimation points (FEPs)

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

3.2 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
RBL_01	S	R. Ribble	West Bradford Road	374500	443950	392.2	N/A
UNN_01	S	Unnamed	Outflow to Ribble	374234	443763	N/A	0.12
GRE_01	S	Greg Sike	Outflow to Ribble	374090	443586	N/A	0.62
COP_01	S	Coplow Brook	Coplow / Waddington Confluence	373950	443500	1.39	N/A
WAD_01	S	Waddington Brook	Coplow / Waddington Confluence	373900	443450	3.79	N/A
Res_01	S	Residual catchment	Residual catchment	N/A	N/A	N/A	1.25
RBL_02	L	R.Ribble	Waddington Road	373850	442850	399.4	N/A

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

Site code	FARL	PROPWET	BFIHOST	BFIHOST19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000*	FPEXT
RBL_01	0.998	0.61	0.372	0.364	31.6	88.8	1350	0.011	0.09
UNN_01**	1.000	0.6	0.311	0.329	1.95	58.2	1347	-	0.12
GRE_01**	1.000	0.6	0.311	0.329	1.95	58.2	1347	-	0.12
COP_01	1.000	0.6	0.311	0.329	1.95	58.2	1347	0.005	0.12
WAD_01	0.998	0.6	0.289	0.308	4.01	92.3	1418	0.025	0.06
Res_01	-	-	-	-	-	-	-	-	-
RBL_02	0.998	0.61	0.372	0.364	32.9	88.6	1350	0.012	0.09

*Reported URBEXT 2000 values are updated to the current year (2021)

**No FEH Catchment Descriptor data is available at either site and design flows at these locations are derived by scaling the rural flows derived for COP_01 by the ratio of catchment areas. Design flows at UNN_01 and GRE_01 are derived from the rural flow estimates as no urban areas are present in either of the catchmetns where FEH data is unavailable.

3.4 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	Catchment boundaries have been checked based on 2m LiDAR and; against the surface water network as depicted on Ordnance Survey mapping. No changes to the FEH catchment boundaries were made.
Record how other catchment descriptors were checked and describe any changes.	Values for BFIHOST and SPRHOST have been sense checked against Soilscapes 1:250,000 scale soils dataset and British Geological Survey 1:625,000 scale geology mapping. Values are found to be sensible based on the underlying moderate permeability sedimentary bedrock and typically slowly permeable, wet peaty upland soils. FARL values have been sense checked by a review of Ordnance Survey mapping. No changes made to default catchment descriptors.
Source of URBEXT	URBEXT2000
Method for updating of URBEXT	CPRE formula from 2006 CEH report on URBEXT2000. Urban Adjustment Factor (UAF) based on WINFAP v4 procedure.

4 STATISTICAL METHOD

4.1 Application of Statistical method

What is the purpose of applying this method?	The FEH Statistical method has been undertaken, for comparison with peak design flows derived for the minor watercourses by the ReFH2 rainfall runoff method and; as the River Ribble is gauged at Henthorn, some 5km downstream of the study extent, the FEH Statistical method is used to derive design peak flow estimates at RBL_01 and RBL_02 i.e. the model upstream inflow and downstream flow reconciliation location.

4.2 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	For refining the estimate of QMED for the River Ribble, the obvious donor site is NRFA Station number 71006 Ribble at Henthorn, located some 5km downstream of RBL_02. For refining the estimate of QMED for the Waddington Brook and Coplow Brook, the six closest stations given by WINFAP v4 are adopted except where there are multiple entries for the same watercourse. Where multiple entries are shown, the lower ranked station is discounted in favour of the next closest station. For the two study catchments where FEH catchment desctriptor data is unavailable (UNN_01 and GRE_01), refinement of QMED is based on the adjustment calculated for the neighbouring Coplow Brook.

4.3 verview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m^3/s)	Final method	Data transfer					Urban adjust-ment factor UAF	Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids d_{ij} (km)	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor: Weight	If more than one donor: Weighted ave. adjustment		
RBL_01	216.6	DT	71006	2.26	0.924	-	-	1.010	202.2
UNN_01	0.17	DT	-	-	-	-	1.21	1.000	0.21
GRE_01	0.91	DT	-	-	-	-	1.21	1.000	1.10
COP_01	2.03	DT	72002, 72817, 72003, 72017, 27084, 69046	N/A	N/A	0.314, 0.313, 0.305, 0.299, 0.285, 0.266	1.21	1.004	2.46
WAD_01	5.29	DT	72016, 72817, 72003, 72017, 27084, 69046	N/A	N/A	0.320, 0.315, 0.313, 0.306, 0.280, 0.260	1.24	1.021	6.67
RBL_02	220.0	DT	71006	1.96	0.920	-	-	1.011	204.7

Are the values of QMED spatially consistent?	Yes QMED increases sensibly with downstream distance.
Method used for urban adjustment for subject and donor sites	WINFAP v4[2]

Parameters used for WINFAP v4 urban adjustment if applicable

Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010)**Error! Bookmark not defined.** in which PRUAF is c alculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016)[2].

2 Wallingford HydroSolutions (2016). WINFAP 4 Urban adjustment procedures.

4.4 Derivation of pooling groups

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments L-CV and L-skew, (before urban adjustment)
71006 (Ribble @ Henthorn)	71006	Yes (ESS)	**Removed:** 27027, duplicate entry for Wharfe **Added:** 76008, to increase total number of station years	L-CV = 0.152 L-Skew = 0.140
		No (P)	**Removed:** 27027, duplicate entry for Wharfe **Added:** 76008, to increase total number of station years	L-CV = 0.139 L-Skew = 0.141
Ribble WBR Crossing	RBL_01	No	**No changes to default group**	L-CV = 0.135 L-Skew = 0.163
Waddington Brook	WAD_01	No	**Removed:** 26016, SPRHOST <20%; 106002 and 47022, FARL <0.95 **Added:** 57017 and 27010, to increase total number of station years	L-CV = 0.214 L-Skew = 0.259
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

4.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
71006	SS	71006 (Ribble @ Henthorn)	GL, preferred distribution for UK flood data	Growth curve adjusted using WINFAP v4 urban adjustment procedure.	Location = 1.0 Scale = 0.154 Shape = -0.191	2.13
	ESS		GL, for direct comparison with SS growth		Location = 1.0 Scale = 0.151 Shape = -0.142	1.98

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution	Growth factor for 100-year return period / 1% AEP
	P		curve	No permeable adjustment required.	Location = 1.0 Scale = 0.139 Shape = -0.141	1.90
RBL_01	P	Ribble WBR Crossing	GL, for direct comparison with growth curves derived at 71006		Location = 1.0 Scale = 0.133 Shape = -0.165	1.91
WAD_01	P	Waddington Brook	GL, best fit as indicated by Z-value		Location = 1.0 Scale = 0.209 Shape = -0.259	2.85

Notes
Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis
Urban adjustments are all carried out using the method of Kjeldsen (2010).
Growth curves were derived using the procedures from Science Report SC050050 (2008).

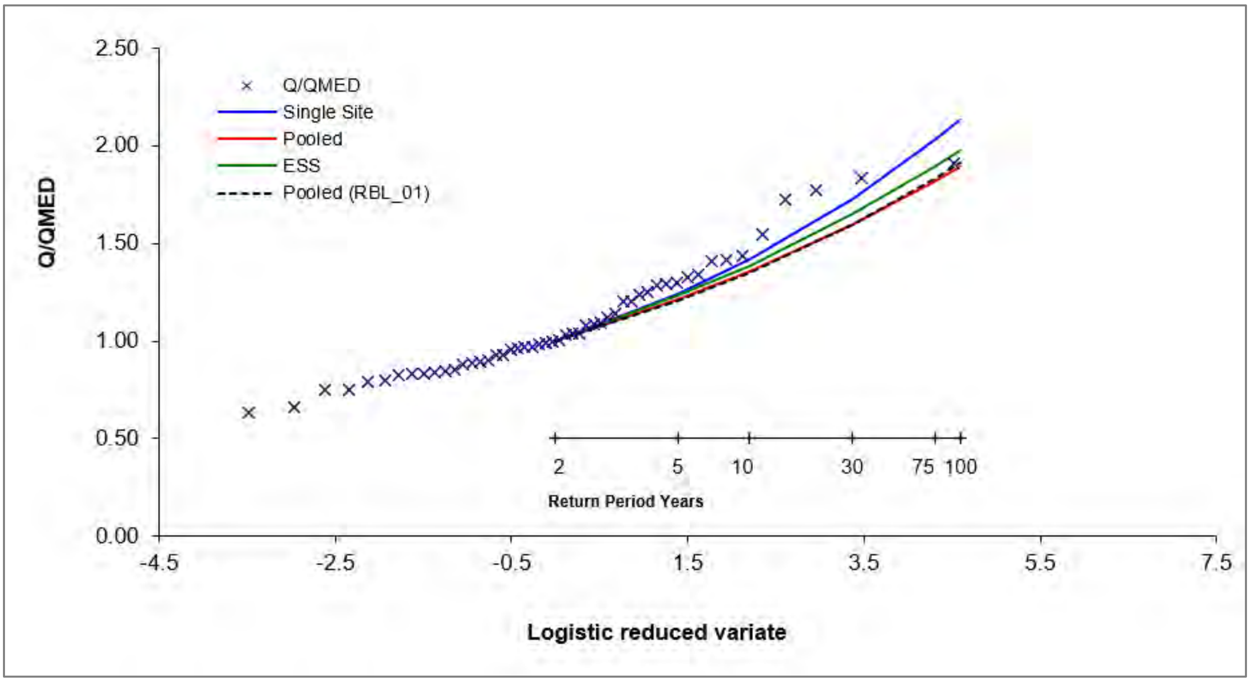


Figure 4-1 Flood growth curves

Flood growth curves reported in Section 4.5 are plotted in Figure 4-1. It is observed that the steepest growth curve is derived by single site analysis at 71006.. The two growth curves derived by pooling analysis are observed to be flatter and the enhanced single site growth curve lies between the single site and pooled growth curves.

The single site growth curve places the largest event (February 2020) as having an estimated return period of approximately 50-years while the pooled and enhanced single site growth curve places the event rarity as approximately the 100-year or 75-years return period, respectively. Adopting either of the enhanced single site or pooled growth curve would mean the estimated 30-year return period peak flow is exceeded 4 times, the estimated 50-year return period peak flow exceeded 2-4 times and the 75 to 100 year return period event exceeded one time, in a record of 52 years. As the station data is of good quality, with a single rating applied throughout the gauged period and no bypassing, the single site growth curve is adopted for deriving peak flows for the River Ribble.
Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)					
	2	5	10	30	75	100
	Flood peak (m^3/s) for the following AEP (%) events					
	50	20	10	3.33	1.33	1
RBL_01	202	252	287	349	410	431
UNN_01	0.21	0.28	0.34	0.45	0.56	0.60
GRE_01	1.10	1.48	1.78	2.34	2.92	3.13
COP_01	2.46	3.32	3.99	5.24	6.54	7.02
WAD_01	6.67	9.00	10.8	14.2	17.7	19.0
Res_01	2.20	2.97	3.57	4.69	5.86	6.28
RBL_02	205	255	291	354	415	436

5 REVITALISED FLOOD HYDROGRAPH 2 (ReFH2) METHOD

5.1 Application of ReFH2 method

What is the purpose of applying this method?	The ReFH2 method has been used to derive distributed and lumped flood hydrographs and peak flows for each of the sub-catchments within the study area.

5.2 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	C_{max} (mm)	PR_{imp} % runoff for impermeable surfaces	BL (hours)	2-yr BR
RBL_01	CD	5.85	4.39	253.64	70%	44.8	1.11
UNN_01*	CD	1.39	1.04	232.41	70%	23.13	0.93
GRE_01*	CD	1.39	1.04	232.41	70%	23.13	0.93
COP_01	CD	1.39	1.04	232.41	70%	23.13	0.93
WAD_01	CD	1.81	1.36	220.07	70%	26.00	0.74
Res_01	-	-	-	-	-	-	-
RBL_02	CD	5.99	4.49	253.64	70%	45.19	1.07
Brief description of any flood event analysis carried out		N/A					
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

*Flows at UNN_01 and GRE_01 are scaled from the rural flow estimates derived at COP_01 by the ratio of catchment areas. The parameters for COP_01 are hence shown for UNN_01 and GRE_01 also.

5.3 Design events for ReFH2 method: Lumped catchments

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)
RBL_01	Urban	Winter	13.0
UNN_01*	Urban	Winter	3.25
GRE_01*	Urban	Winter	3.25
COP_01	Urban	Winter	3.25
WAD_01	Urban	Winter	4.25
Res_01	Urban	Winter	-
RBL_02	Urban	Winter	15.0

5.4 Flood estimates from the ReFH2 method (lumped)

Lumped flows are generated using the individual catchment critical storm duration for comparison with the FEH Statistical estimates.

Site code	Flood peak (m^3/s) for the following return periods (in years)					
	2	5	10	30	75	100
	Flood peak (m^3/s) for the following AEP (%) events					
	50	20	10	3.3	1.3	1
RBL_01	170	213	245	303	367	390
UNN_01*	0.11	0.17	0.20	0.27	0.33	0.35
GRE_01*	0.60	0.87	1.07	1.40	1.72	1.83
COP_01	1.34	1.94	2.39	3.13	3.84	4.09
WAD_01	3.79	5.33	6.46	8.40	10.2	10.9
Res_01	1.23	1.75	2.13	2.79	3.41	3.62
RBL_02	176	220	252	311	375	398

5.5 Design events for ReFH2 method: Sub-catchments and intervening areas

Distrubuted (sub-catchment) flows are generated adopting a common design storm in terms of both duration and ARF, calculated at RBL_02, i.e. the downstream model extent.

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
RBL_01	Winter	15.0	399.4 km^2 (ARF = 0.901)	Design storm duration for downstream model boundary
UNN_01*	Winter	15.0		
GRE_01*	Winter	15.0		
COP_01	Winter	15.0		
WAD_01	Winter	15.0		
Res_01	Winter	15.0		
RBL_02	Winter	15.0	Catchment area	

5.6 Flood estimates from the ReFH2 method (distributed)

Site code	Flood peak (m^3/s) for the following return periods (in years)					
	2	5	10	30	75	100
	Flood peak (m^3/s) for the following AEP (%) events					
	50	20	10	3.33	1.33	1
RBL_01	175	219	251	309	373	397
UNN_01*	0.10	0.13	0.16	0.19	0.23	0.25
GRE_01*	0.53	0.70	0.81	1.02	1.22	1.29
COP_01	1.19	1.55	1.82	2.28	2.72	2.89
WAD_01	3.50	4.57	5.34	6.67	7.95	8.42
Res_01	1.11	1.45	1.70	2.12	2.53	2.68
RBL_02	176	220	252	311	375	398

6 DISCUSSION AND SUMMARY OF RESULTS

6.1 Comparison of results from different methods

Site code	Ratio of peak flow to FEH Statistical peak	
	Return period 2 years / 50% AEP	Return period 100 years / 1% AEP
	ReFH2 (lumped)	ReFH2 (lumped)
RBL_01	0.84	0.90
UNN_01*	0.54	0.58
GRE_01*	0.54	0.58
COP_01	0.54	0.58
WAD_01	0.57	0.57
Res_01	0.56	0.58
RBL_02	0.86	0.91

6.2 Final choice of method

Choice of method and reasons	Flows derived by either method for the River Ribble are shown to be generally consistent, particularly at the 100-year return period. For the tributary inflows, flows derived by the ReFH2 method are significantly lower than are derived by the FEH Statistical method. For deriving the upstream inflow and the downstream target flow for the River Ribble, the FEH Statistical single site method as undertaken at Station 71006 is adopted. For the tributary inflows, the ReFH2 design hydrographs are scaled to the higher peak flow estimates derived by the FEH Statistical method and will be reconciled to agree with the flood estimates derived by the FEH Statistical method at RBL_02.
How will the flows be applied to a hydraulic model?	Inflow hydrographs will be applied at RBL_01, UNN_01, GRE_01, COP_01 and WAD_01. Flow contributed from the residual area will be distributed along the model reach between RBL_01 and RBL_02. Target flow has been estimated at RBL_02 by the FEH Statistical method and the tributary inflows will be scaled accordingly so as their total contributing flows agree with that estimated at RBL_02. In addition, model flow will also be provided for the individual critical storm duration, in order to check any crititcal flow at the minor watercourse crossings.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The study assumes that the design hydrographs derived for the tributary inflows in ReFH2.3 and scaled to peak flows estimated by the FEH Statistical method are a satisfactory estimate of the true peak flows and hydrographs generated at the study site(s). As no gauged data is available for the tributary catchments, this assumption is uncertain. The flood frequency estimates for the River Ribble are derived based on data recorded at Station 71006, some 5km downstream of the study extent and hence are considered to be more robust. Within the hydraulic model, the tributary inflows will be adjusted accordingly to agree with the more robust estimate of flow derived at RBL_02.

Discuss any particular limitations	There are no catchment characteristics that limit the appropriateness of standard FEH methodologies. Limitations to the study include the lack of gauged data for the tributary inflows. However as the tributary inflows will be scaled to agree with the estimated flow at RBL_02 (derived from gauged data) this is considered a minor limitation.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	There is currently no published conventional method of quantifying uncertainty in the results derived by the ReFH2.3 method and hence uncertainty bounds are derived for the adopted FEH Statistical estimates and are reported in Section 7.6.
Comment on the suitability of the results for future studies	Results are suitable for the purpose of the current study only.
Give any other comments on the study	Flows should be revisited following the release of a revised, fuller flood peak data set and/or following any significant flood events on the River Ribble.

6.4 Checks

Are the results consistent, for example at confluences?	Yes, design flows on the main River Ribble increase sensibly with downstream distance. The sum of flow from the tributary inflows is generally in good agreement with the flow estimates derived at the downstream model extent (i.e. RBL_02).
What do the results imply regarding the return periods / frequency of floods during the period of record?	The adopted single site growth curve places the largest AMAX event as having a return period of approximately 50-years, this is considered a more realistic estimate of the event rarity than is derived by either the enhanced single site or pooled growth curves which place the event as having a return period of approximately 75-years or 100-years, respectively.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The 100-year growth factor for the River Ribble is 2.13 and for the tributary inflows, ranges from 2.40 – 2.43 which falls within the range of the regional 100-year growth factors reported in the Flood Studies Report which varied from 2.1 to 4.0.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the 1000-year flow to the 100-year flow is 1.5 for the River Ribble and 1.6 for the tribitary inflows.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	No previous studies were provided by the client nor found from a brief internet search.
Are the results compatible with the longer-term flood history?	The results of the study and how they fit with the flood frequency understanding will be developed an reported during the course of the hydraulic modelling.
Describe any other checks on the results	N/A

6.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)					
	2	5	10	30	75	100
	Flood peak (m^3/s) for the following AEP (%) events					
	50	20	10	3.33	1.33	1
RBL_01	202	252	287	349	410	431
UNN_01	0.21	0.28	0.34	0.45	0.56	0.60
GRE_01	1.10	1.48	1.78	2.34	2.92	3.13
COP_01	2.46	3.32	3.99	5.24	6.54	7.02
WAD_01	6.67	9.00	10.8	14.2	17.7	19.0
Res_01	2.20	2.97	3.57	4.69	5.86	6.28
RBL_02	205	255	291	354	415	436

6.6 Uncertainty bounds

This table reports the flows derived from the uncertainty analysis detailed in Section 6.3. The 'true' value is more likely to be near the estimate reported in Section 0 than the bounds. However, it is possible that the 'true' value could still lie outside these bounds.

There is currently no published conventional method of quantifying uncertainty in the results derived by the ReFH2.3 method and hence uncertainty bounds are derived for the adopted FEH Statistical estimates only. The table below shows that design peak flows from the ReFH2 method presented in Section 5.6, lie within the uncertainty bounds of the statistical peak flows, for the corresponding design events.

Uncertainty bounds for the River Ribble are based on 95% confidence intervals fitted to the single site growth curve as derived at Station 71006 while uncertainty for the sub-catchments flow estimates are based on the 95% confidence intervals presented in Technical guidance 197_08 for design flows at an ungauged rural site using six donors.

Site code	Flood peak (m^3/s) for the following return periods (in years)							
	2		5		10		100	
	Flood peak (m^3/s) for the following AEP (%) events							
	50		20		10		1.0	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
RBL_01	202	202	243	264	273	311	378	531
UNN_01	0.11	0.41	0.14	0.56	0.17	0.68	0.29	1.23
GRE_01	0.56	2.17	0.74	2.94	0.89	3.55	1.54	6.42
COP_01	1.26	4.85	1.66	6.58	2.00	7.94	3.44	14.4
WAD_01	3.40	13.1	4.50	17.8	5.41	21.5	9.32	39.0
Res_01	1.12	4.34	1.49	5.89	1.79	7.11	3.08	12.9
RBL_02	205	205	246	268	276	315	382	538

6.7 Flood Hydrographs

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Design inflow hydrographs are required for the hydraulic analysis. For the minor watercourses, hydrographs are derived by the ReFH2 method for ungauged catchments and scaled to the peak flow estimates derived by the FEH Statistical method. For the main River Ribble, data recorded at Station 71006 is adopted to inform a representative hydrograph shape derived from averaging the five largest historic flood event hydrographs recorded at the gauge as shown in Figure 6-1. Design hydrographs are provided in 'B27070CT_Ribble Design Hydrographs.xls'

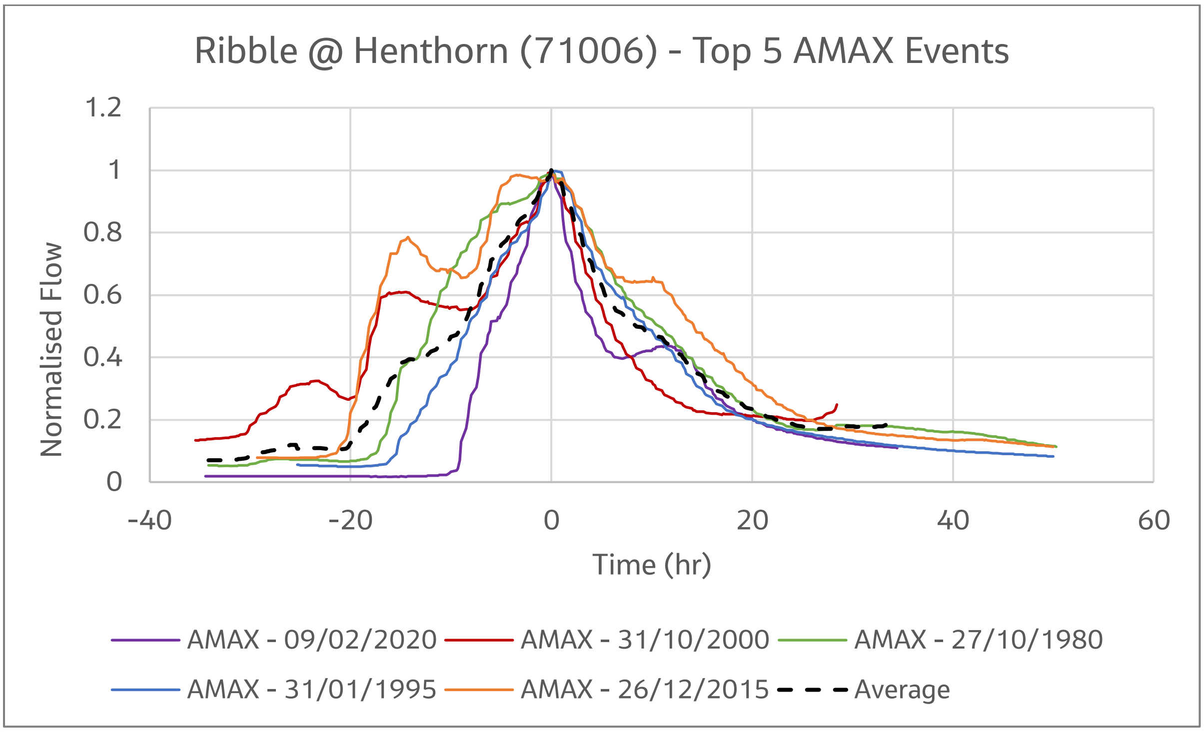


Figure 6-1 Average flood hydrograph

The average flood event hydrograph is plotted against design hydrographs derived within ReFH 2.3 at RBL_01 in Figure 6-2.

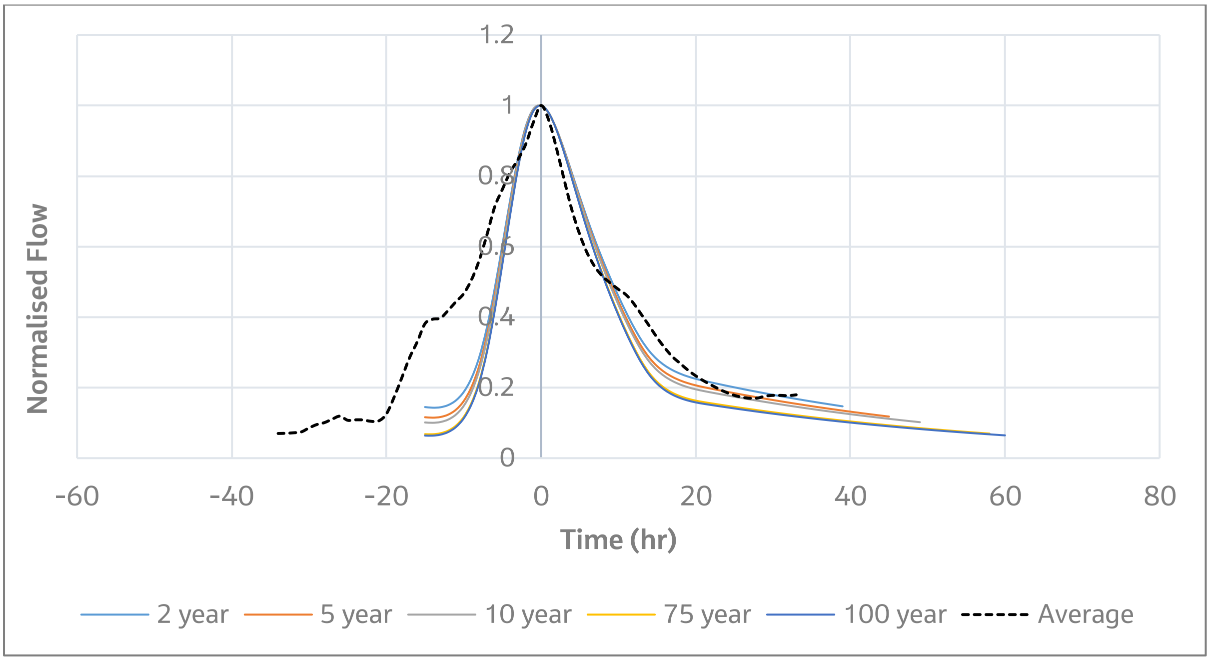


Figure 6-2 Average flood hydrograph compared against ReFH2 design hydrographs.

It is observed from Figure 6-2 that the design hydrographs derived within ReFH 2.3 potentially underestimate the typical flood hydrograph volume. The averaged event hydrograph shows a greater volume on the rising limb. The receding limb matches the design hydrograph shape well, however, given that the design hydrographs potentially underestimate typical hydrograph volume, the averaged hydrograph shaped derived from the gauge data is adopted. The design hydrographs are plotted in Figure 6-3.

6.8 Model runs and flow reconciliation

The design inflow hydrographs for Run 1 (Storm Duration as calculated for the River Ribble downstream model extent) for the River Ribble and for the minor watercourses (ReFH2.3 hydrograph scaled to Statistical peak) for the 100-year return period event are shown in Figure 6.3.

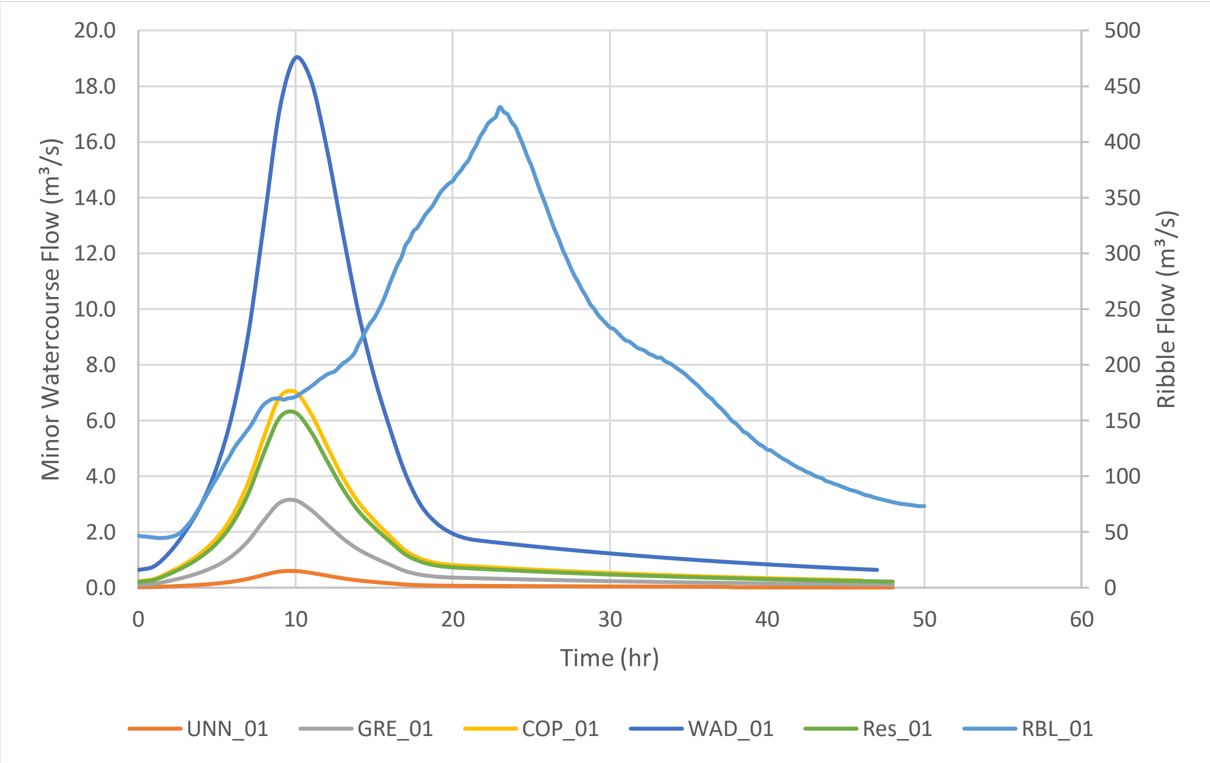


Figure 6-3 Design hydrographs for the minor watercourses and River Ribble.

It is observed from Figure 6-3 that for the design hydrographs derived within ReFH2 the flood peak in the minor watercourses occurs at 10 hours whereas the design hydrograph for the River Ribble (RBL_01) peaks at approximately 24 hours, thus a difference in the time to peak by 14 hours. Similarly, the hydrograph from ReFH2 for the River Ribble (not shown here) shows that the peak flow occurs at 15 hours, thus indicating the difference in time to peak of 5hours.

Therefore, for Run 1, the flow reconciliation at the downstream model extent will be achieved by aligning the hydrographs of the River Ribble and the minor watercourse by the difference in time to peak from 5 to 14 hours, and scaling down the minor watercourses inflows (as inflow hydrograph peaks are adopted from the statistical peak) as required.

For Run 2, flow reconciliation is not necessary as the minor watercourses peaks are already scaled to the statistical peak and the hydrograph shapes are adopted for the theoretical critical storm duration suggested by ReFH2 for the corresponding crossing location (which is 3.25 hour).

7 ANNEX

7.1 Proposed link road alignment

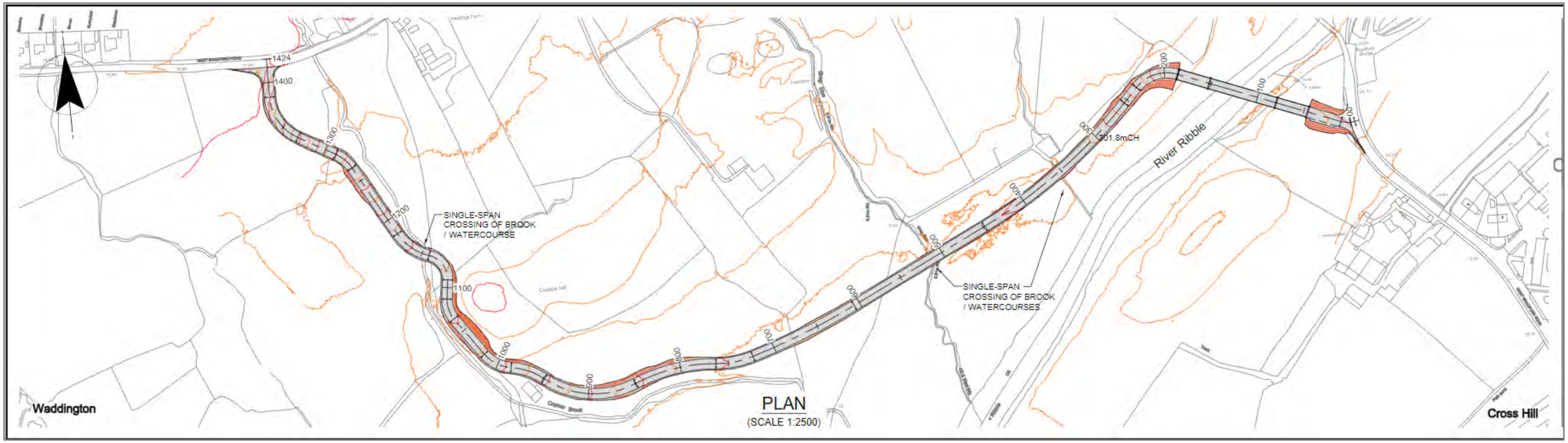


Figure A-1: Proposed link road (reproduced from Drawing 80061155-01-JAC-TR4-XX-M3-C-00007)

7.2 Pooling Group Composition

River Ribble: Pooling and Enhanced Single Site group at 71006

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
71006 (Ribble @ Henthorn)	0	52	220	0.157	0.188	0.17	446	1343	0.091	0.997	0.015
83005 (Irvine @ Shewalton)	0.35	30	200	0.145	0.196	0.17	368	1228	0.081	0.980	0.027
202001 (Roe @ Ardnargle)	0.47	44	151	0.087	-0.016	1.01	366	1250	0.059	0.993	0.006
54014 (Severn @ Abermule)	0.54	51	185	0.165	0.270	1.21	575	1256	0.060	0.970	0.004
27034 (Ure @ Kilgram Bridge)	0.54	52	250	0.134	0.076	0.10	511	1337	0.045	0.990	0.004
83006 (Ayr @ Mainholm)	0.55	31	249	0.157	0.217	0.36	579	1212	0.058	0.992	0.006
236005 (Colebrooke @ Ballindarragh Bridge)	0.58	38	109	0.093	0.157	1.86	314	1156	0.082	0.987	0.001
79006 (Nith @ Drumlanrig)	0.60	39	337	0.133	0.132	0.04	469	1485	0.041	0.990	0.002
203020 (Moyola @ Moyola New Bridge)	0.61	47	112	0.089	-0.145	2.64	304	1225	0.112	0.992	0.008
7001 (Findhorn @ Shenachie)	0.62	59	277	0.162	0.231	0.96	416	1217	0.039	0.982	0.000
27043 (Wharfe @ Addingham)	0.64	46	263	0.170	0.058	1.96	430	1385	0.035	0.975	0.004
76008 (Irthing @ Greenholme)	0.65	52	151	0.141	0.139	1.52	333	1073	0.067	0.994	0.003
Total		**541**									
Weighted means				**0.152**	**0.14**						

Waddington Brook: Adopted Pooling group at WAD_01

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy	AREA	SAAR	FPEXT	FARL	URBEXT 2000
45816 (Haddeo @ Upton)	1.00	26	3.46	0.300	0.406	0.99	6.81	1210	0.011	1.00	0.005
28033 (Dove @ Hollinsclough)	1.17	44	4.18	0.228	0.371	0.51	7.92	1346	0.007	1.00	0.000
76011 (Coal Burn @ Coalburn)	1.29	42	1.84	0.163	0.301	0.65	1.63	1096	0.074	1.00	0.000
91802 (Allt Leachdach @ Intake)	1.48	34	6.35	0.153	0.257	0.80	6.54	2554	0.003	0.99	0.000
27051 (Crimple @ Burn Bridge)	1.52	47	4.52	0.218	0.156	0.38	8.17	855	0.013	1.00	0.006
71003 (Croasdale Beck @ Croasdale Flume)	1.61	37	10.9	0.212	0.323	0.20	10.7	1882	0.016	1.00	0.000
25003 (Trout Beck @ Moor House)	1.65	46	15.1	0.168	0.290	0.58	11.4	1905	0.041	1.00	0.000
54022 (Severn @ Plynlimon Flume)	1.66	38	15.0	0.156	0.171	0.66	8.75	2481	0.010	1.00	0.000
25011 (Langdon Beck @ Langdon)	1.77	33	15.6	0.232	0.328	0.63	12.8	1463	0.012	1.00	0.001
206006 (Annalong @ Recorder)	1.94	48	15.3	0.189	0.052	2.21	14.4	1704	0.023	0.98	0.000
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	2.13	9	5.78	0.271	0.151	3.22	16.1	1044	0.023	0.99	0.006
25019 (Leven @ Easby)	2.22	41	5.09	0.342	0.386	2.11	15.1	830	0.019	1.00	0.004
57017 (Rhondda Fawr @ Tynewydd)	2.37	19	24.3	0.150	0.062	1.02	16.6	2458	0.012	1.00	0.016
27010 (Hodge Beck @ Bransdale Weir)	2.40	41	9.42	0.224	0.293	0.07	18.8	987	0.009	1.00	0.001
Total		**505**									
Weighted means				**0.214**	**0.259**						

Appendix B. Cross-Sections of Proposed Crossings

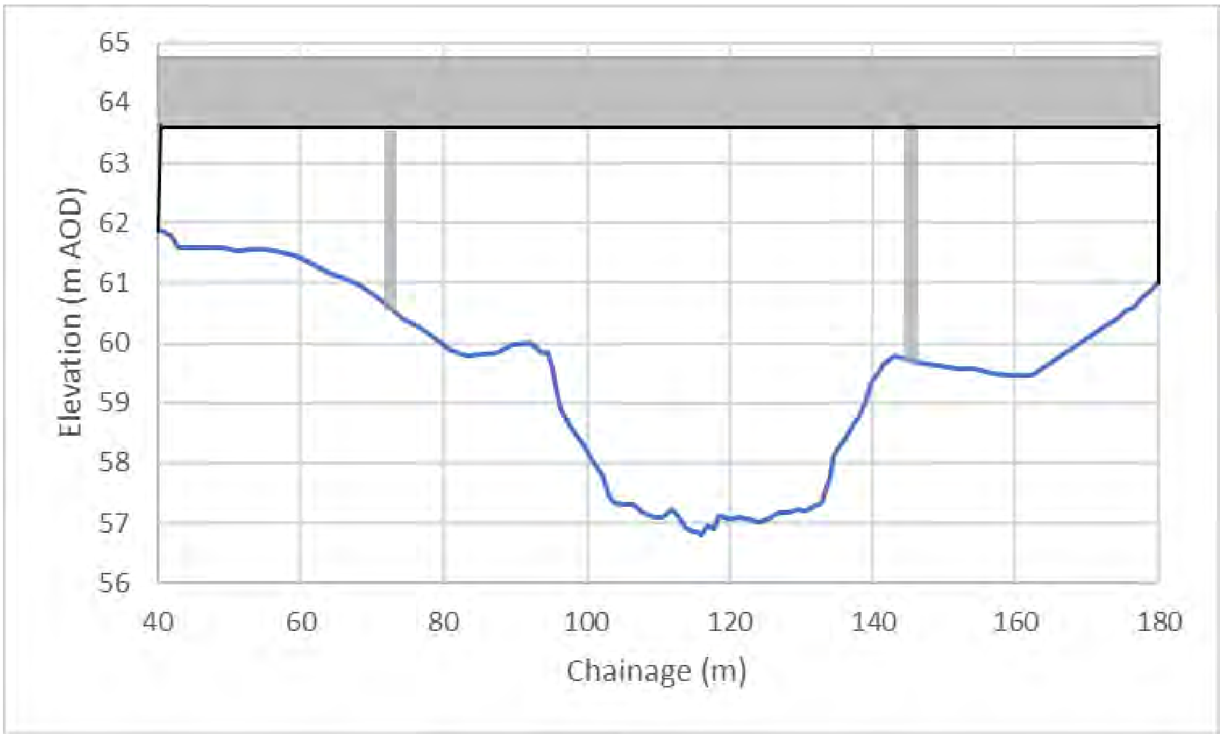


Figure B-1: Proposed River Ribble crossing.

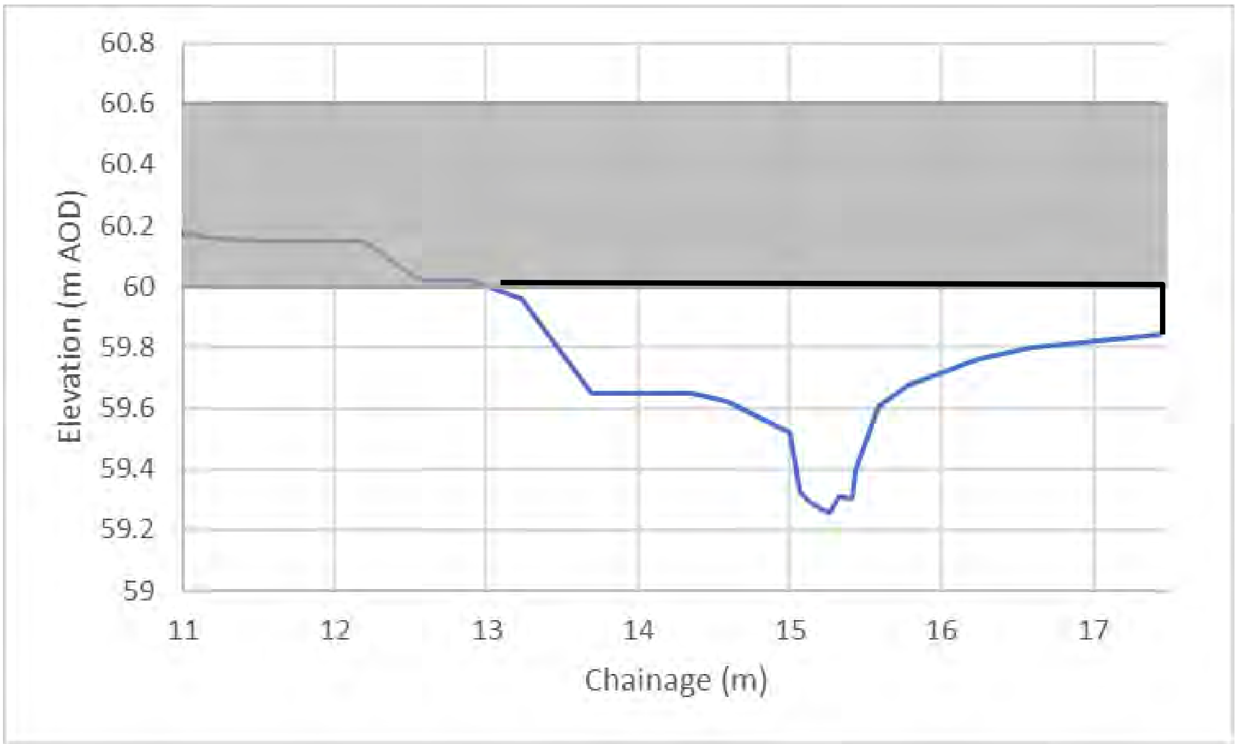


Figure B-2: Proposed Unnamed Watercourse crossing.

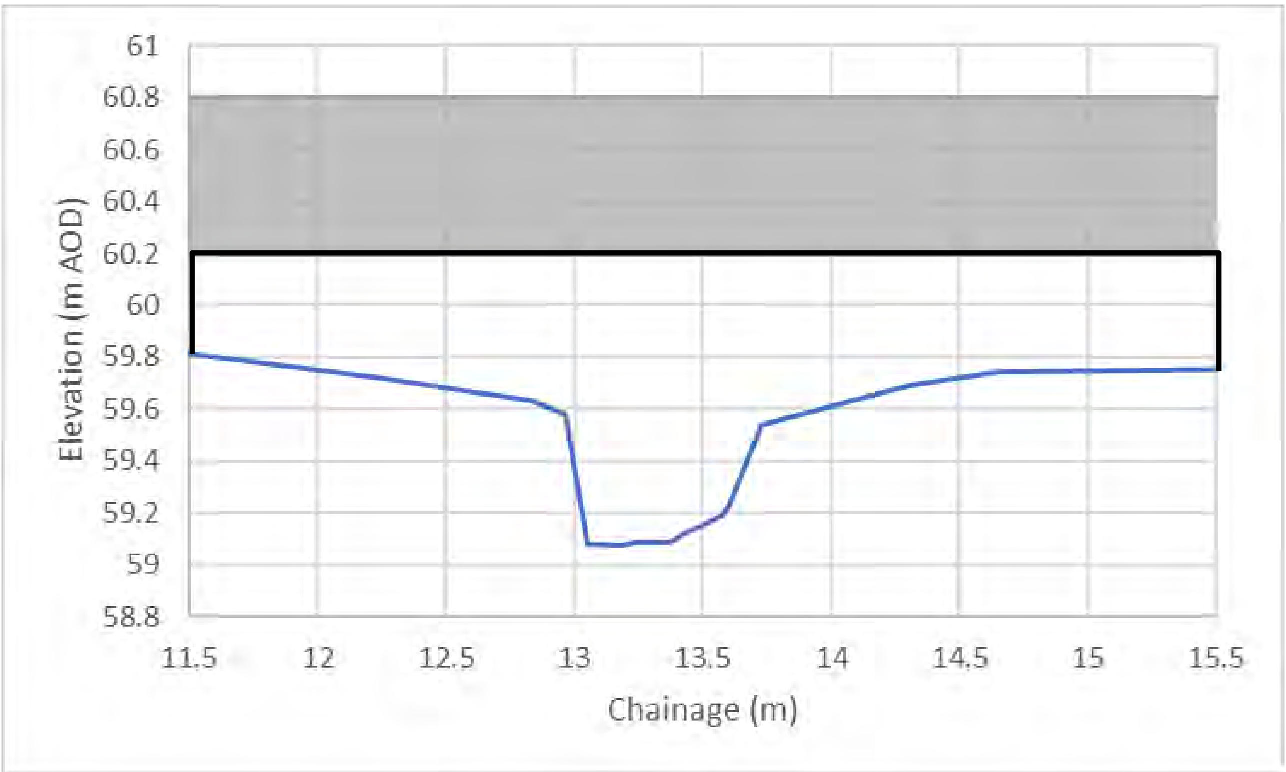


Figure B-3: Proposed Greg Syke crossing.

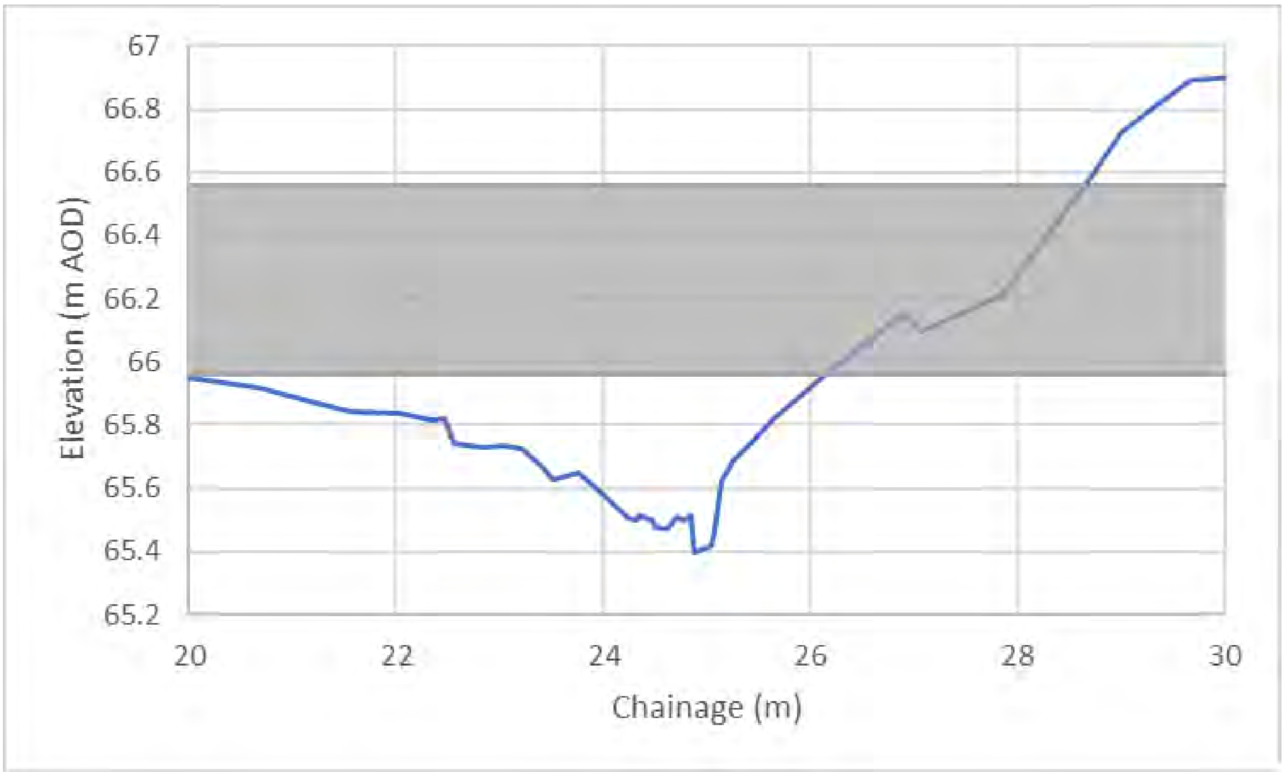


Figure B-4: Proposed Coplow Brook crossing.

Appendix C. In-Channel Water Levels

C.1 River Ribble

Table C-1: In-channel maximum water level at key locations for the 50% AEP event.

Node	Description	Baseline 50% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 50% AEP Event Max Stage (m AOD)	Water level difference (m)
RIBB_2164	154m upstream from Temporary Bridge	60.672	60.672	0.000
RIBB_1945	35m upstream from Temporary Bridge	59.920	59.919	-0.001
RIBB_1910	Location of Temporary Bridge	59.843	59.843	0.000
RIBB_1740	70m downstream from Temporary Bridge	59.546	59.546	0.000
RIBB_0782	1.2km downstream from Temporary Bridge	57.148	57.149	0.001
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.035	56.035	0.000

Table C-2: In-channel maximum water level at key locations for the 20% AEP event.

Node	Description	Baseline 20% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 20% AEP Event Max Stage (m AOD)	Water level difference (m)
RIBB_2164	154m upstream from Temporary Bridge	60.917	60.917	0.000
RIBB_1945	35m upstream from Temporary Bridge	60.192	60.193	0.001
RIBB_1910	Location of Temporary Bridge	60.128	60.128	0.000
RIBB_1740	70m downstream from Temporary Bridge	59.808	59.808	0.000
RIBB_0782	1.2km downstream from Temporary Bridge	57.398	57.398	0.000
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.298	56.298	0.000

Table C-3: In-channel maximum water level at key locations for the 10% AEP event.

Node	Description	Baseline 10% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 10% AEP Event Max Stage (m AOD)	Water level difference (m)
RIBB_2164	154m upstream from Temporary Bridge	61.042	61.042	0.000
RIBB_1945	35m upstream from Temporary Bridge	60.381	60.381	0.000
RIBB_1910	Location of Temporary Bridge	60.311	60.311	0.000
RIBB_1740	70m downstream from Temporary Bridge	59.970	59.970	0.000
RIBB_0782	1.2km downstream from Temporary Bridge	57.557	57.557	0.000
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.458	56.458	0.000

Table C-4: In-channel maximum water level at key locations for the 3.33% AEP event.

Node	Description	Baseline 3.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 3.33% AEP Event Max Stage (m AOD)	Water level difference (m)
RIBB_2164	154m upstream from Temporary Bridge	61.242	61.242	0.000
RIBB_1945	35m upstream from Temporary Bridge	60.648	60.650	0.002
RIBB_1910	Location of Temporary Bridge	60.566	60.567	0.001
RIBB_1740	70m downstream from Temporary Bridge	60.164	60.164	0.000
RIBB_0782	1.2km downstream from Temporary Bridge	57.797	57.798	0.001
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.694	56.694	0.000

Table C-5: In-channel maximum water level at key locations for the 1.33% AEP event.

Node	Description	Baseline 1.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1.33% AEP Event Max Stage (m AOD)	Water level difference (m)
RIBB_2164	154m upstream from Temporary Bridge	61.428	61.429	0.001
RIBB_1945	35m upstream from Temporary Bridge	60.883	60.886	0.003
RIBB_1910	Location of Temporary Bridge	60.787	60.788	0.001
RIBB_1740	70m downstream from Temporary Bridge	60.328	60.333	0.005
RIBB_0782	1.2km downstream from Temporary Bridge	58.017	58.017	0.000
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.901	56.901	0.000

Table C-6: In-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)
RIBB_2164	154m upstream from Temporary Bridge	61.492	61.493	0.001
RIBB_1945	35m upstream from Temporary Bridge	60.960	60.963	0.003
RIBB_1910	Location of Temporary Bridge	60.859	60.861	0.002
RIBB_1740	70m downstream from Temporary Bridge	60.379	60.384	0.005
RIBB_0782	1.2km downstream from Temporary Bridge	58.093	58.093	0.000
RIBB_0000	1.7km downstream from Temporary Bridge Model d/s boundary	56.967	56.967	0.000

C.2 Unnamed Watercourse

Table C-7: In-channel maximum water level at key locations for the 50% AEP event.

Node	Description	Baseline 50% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 50% AEP Event Max Stage (m AOD)	Water level difference (m)
UNN01_1808	110m upstream from Temporary Bridge	60.701	60.701	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.679	59.679	0.000
UNN01_1755	40m downstream from Temporary Bridge	59.461	59.461	0.000
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000

Table C-8: In-channel maximum water level at key locations for the 20% AEP event.

Node	Description	Baseline 20% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 20% AEP Event Max Stage (m AOD)	Water level difference (m)
UNN01_1808	110m upstream from Temporary Bridge	60.735	60.735	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.701	59.701	0.000
UNN01_1755	40m downstream from Temporary Bridge	59.464	59.464	0.000
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000

Table C-9: In-channel maximum water level at key locations for the 10% AEP event.

Node	Description	Baseline 10% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 10% AEP Event Max Stage (m AOD)	Water level difference (m)
UNN01_1808	110m upstream from Temporary Bridge	60.762	60.762	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.721	59.721	0.000
UNN01_1755	40m downstream from Temporary Bridge	59.467	59.467	0.000
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000

Table C-10: In-channel maximum water level at key locations for the 3.33% AEP event.

Node	Description	Baseline 3.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 3.33% AEP Event Max Stage (m AOD)	Water level difference (m)
UNN01_1808	110m upstream from Temporary Bridge	60.794	60.794	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.755	59.755	0.000
UNN01_1755	40m downstream from Temporary Bridge	59.475	59.475	0.000
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000

Table C-11: In-channel maximum water level at key locations for the 1.33% AEP event.

Node	Description	Baseline 1.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1.33% AEP Event Max Stage (m AOD)	Water level difference (m)
UNN01_1808	110m upstream from Temporary Bridge	60.810	60.810	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.782	59.782	0.000
UNN01_1755	40m downstream from Temporary Bridge	59.484	59.484	0.000
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000

Table C-12: In-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)
UNN01_1808	110m upstream from Temporary Bridge	60.813	60.813	0.000
UNN01_1772u	Immediately upstream of Temporary Bridge	59.792	59.791	-0.001
UNN01_1755	40m downstream from Temporary Bridge	59.488	59.487	-0.001
UNN01_1720	322m downstream from Temporary Bridge Model d/s boundary	59.456	59.456	0.000

C.3 Greg Syke

Table C-13: In-channel maximum water level at key locations for the 50% AEP event.

Node	Description	Baseline 50% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 50% AEP Event Max Stage (m AOD)	Water level difference (m)
GRE_1700	46m upstream from Temporary Bridge	60.919	60.919	0.000
GRE_1654u	Immediately upstream of Temporary Bridge	59.828	59.828	0.000
GRE_1604	50m downstream from Temporary Bridge	58.989	58.989	0.000
GRE_1480c	174m downstream from Temporary Bridge Model d/s boundary	58.922	58.922	0.000

Table C-14: In-channel maximum water level at key locations for the 20% AEP event.

Node	Description	Baseline 20% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 20% AEP Event Max Stage (m AOD)	Water level difference (m)
GRE_1700	46m upstream from Temporary Bridge	61.122	61.122	0.000
GRE_1654u	Immediately upstream of Temporary Bridge	59.853	59.853	0.000
GRE_1604	50m downstream from Temporary Bridge	59.010	59.009	-0.001
GRE_1480c	174m downstream from Temporary Bridge Model d/s boundary	58.922	58.922	0.000

Table C-15: In-channel maximum water level at key locations for the 10% AEP event.

Node	Description	Baseline 10% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 10% AEP Event Max Stage (m AOD)	Water level difference (m)
GRE_1700	46m upstream from Temporary Bridge	61.155	61.155	0.000
GRE_1654u	Immediately upstream of Temporary Bridge	59.864	59.863	-0.001
GRE_1604	50m downstream from Temporary Bridge	59.019	59.017	-0.002
GRE_1480c	174m downstream from Temporary Bridge Model d/s boundary	58.922	58.922	0.000

Table C-16: In-channel maximum water level at key locations for the 3.33% AEP event.

Node	Description	Baseline 3.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 3.33% AEP Event Max Stage (m AOD)	Water level difference (m)
GRE_1700	46m upstream from Temporary Bridge	61.193	61.193	0.000
GRE_1654u	Immediately upstream of Temporary Bridge	59.882	59.881	-0.001
GRE_1604	50m downstream from Temporary Bridge	59.040	59.033	-0.007
GRE_1480c	174m downstream from Temporary Bridge Model d/s boundary	58.922	58.922	0.000

Table C-17: In-channel maximum water level at key locations for the 1.33% AEP event.

Node	Description	Baseline 1.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1.33% AEP Event Max Stage (m AOD)	Water level difference (m)
GRE_1700	46m upstream from Temporary Bridge	61.221	61.220	-0.001
GRE_1654u	Immediately upstream of Temporary Bridge	59.898	59.899	0.001
GRE_1604	50m downstream from Temporary Bridge	59.064	59.049	-0.015
GRE_1480c	174m downstream from Temporary Bridge Model d/s boundary	58.922	58.922	0.000

Table C-18: In-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)
GRE_1700	46m upstream from Temporary Bridge	61.230	61.230	0.000
GRE_1654u	Immediately upstream of Temporary Bridge	59.905	59.907	0.002
GRE_1604	50m downstream from Temporary Bridge	59.074	59.057	-0.017
GRE_1480c	174m downstream from Temporary Bridge Model d/s boundary	58.922	58.922	0.000

C.4 Coplow Brook

Table C-19: In-channel maximum water level at key locations for the 50% AEP event.

Node	Description	Baseline 50% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 50% AEP Event Max Stage (m AOD)	Water level difference (m)
COPL_2224	110m upstream from Temporary Bridge	68.001	68.005	0.004
COPL_2114u	Immediately upstream of Temporary Bridge	65.984	66.114	0.130
COPL_2074	40m downstream from Temporary Bridge	65.358	65.356	-0.002
COPL_1927	147m downstream from Temporary Bridge	62.880	62.877	-0.003
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000

Table C-20: In-channel maximum water level at key locations for the 20% AEP event.

Node	Description	Baseline 20% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 20% AEP Event Max Stage (m AOD)	Water level difference (m)
COPL_2224	110m upstream from Temporary Bridge	68.087	68.078	-0.009
COPL_2114u	Immediately upstream of Temporary Bridge	66.041	66.275	0.234
COPL_2074	40m downstream from Temporary Bridge	65.428	65.426	-0.002
COPL_1927	147m downstream from Temporary Bridge	62.947	62.945	-0.002
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000

Table C-21: In-channel maximum water level at key locations for the 10% AEP event.

Node	Description	Baseline 10% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 10% AEP Event Max Stage (m AOD)	Water level difference (m)
COPL_2224	110m upstream from Temporary Bridge	68.131	68.127	-0.004
COPL_2114u	Immediately upstream of Temporary Bridge	66.081	66.407	0.326
COPL_2074	40m downstream from Temporary Bridge	65.465	65.460	-0.005
COPL_1927	147m downstream from Temporary Bridge	62.966	62.963	-0.003
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000

Table C-22: In-channel maximum water level at key locations for the 3.33% AEP event.

Node	Description	Baseline 3.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 3.33% AEP Event Max Stage (m AOD)	Water level difference (m)
COPL_2224	110m upstream from Temporary Bridge	68.220	68.215	-0.005
COPL_2114u	Immediately upstream of Temporary Bridge	66.146	66.593	0.447
COPL_2074	40m downstream from Temporary Bridge	65.513	65.512	-0.001
COPL_1927	147m downstream from Temporary Bridge	62.980	62.980	0.000
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000

Table C-23: In-channel maximum water level at key locations for the 1.33% AEP event.

Node	Description	Baseline 1.33% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1.33% AEP Event Max Stage (m AOD)	Water level difference (m)
COPL_2224	110m upstream from Temporary Bridge	68.300	68.298	-0.002
COPL_2114u	Immediately upstream of Temporary Bridge	66.192	66.628	0.436
COPL_2074	40m downstream from Temporary Bridge	65.543	65.539	-0.004
COPL_1927	147m downstream from Temporary Bridge	62.984	62.984	0.000
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000

Table C-24: In-channel maximum water level at key locations for the 1% AEP event.

Node	Description	Baseline 1% AEP Event Max Stage (m AOD)	Proposed Ribble Crossing 1% AEP Event Max Stage (m AOD)	Water level difference (m)
COPL_2224	110m upstream from Temporary Bridge	68.327	68.326	-0.001
COPL_2114u	Immediately upstream of Temporary Bridge	66.204	66.636	0.432
COPL_2074	40m downstream from Temporary Bridge	65.553	65.546	-0.007
COPL_1927	147m downstream from Temporary Bridge	62.985	62.985	0.000
COPL_1792	322m downstream from Temporary Bridge Model d/s boundary	61.127	61.127	0.000

Appendix D. Modelled Flood Extents

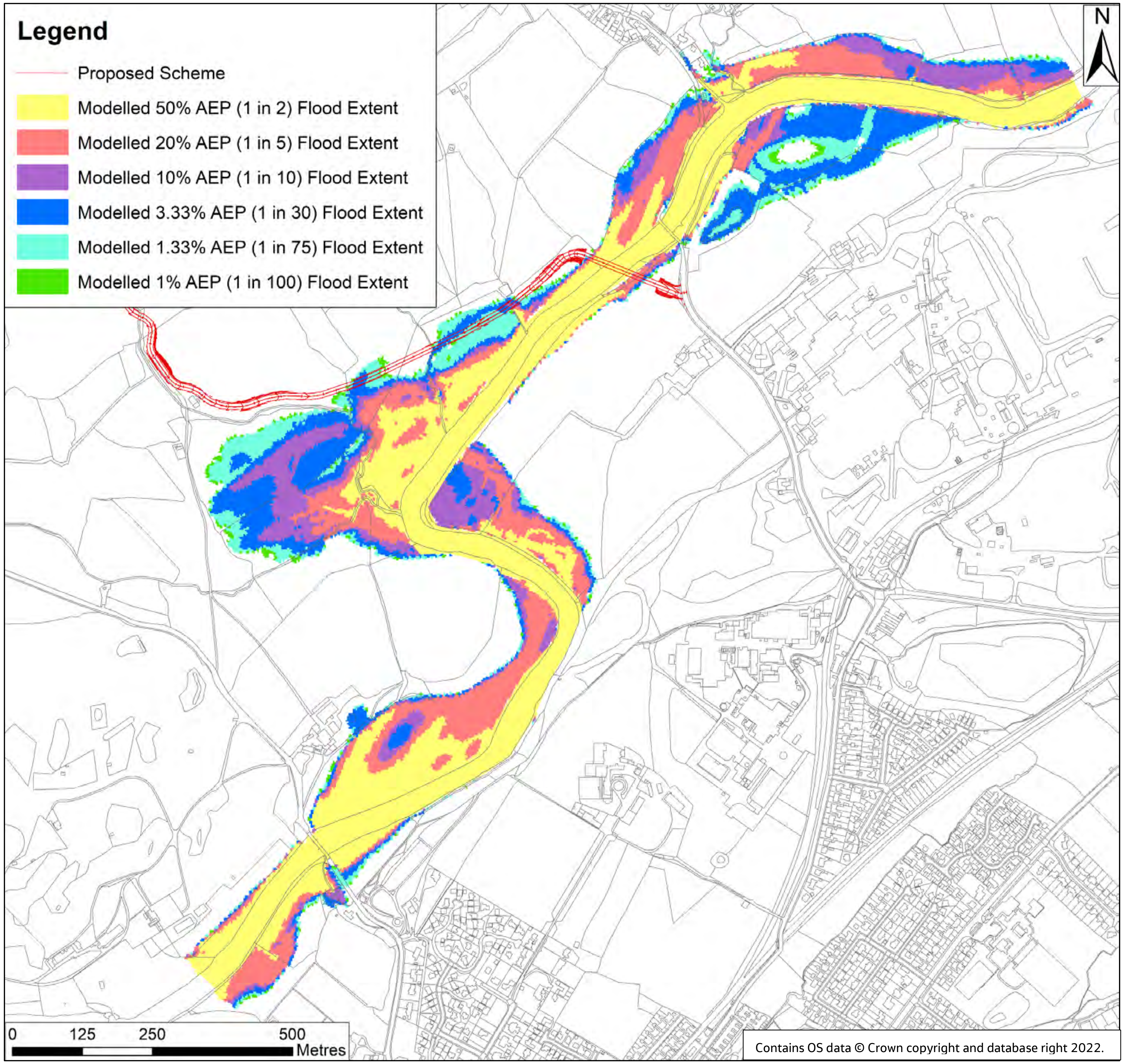


Figure D-1 River Ribble Baseline modelled maximum flood extents, with proposed crossing and haul road footprint overlaid.

Figure D-2 Unnamed Watercourse Baseline modelled maximum flood extents, with proposed haul road footprint overlaid.

Figure D-3 Greg Syke Baseline modelled maximum flood extents, with proposed haul road footprint overlaid.

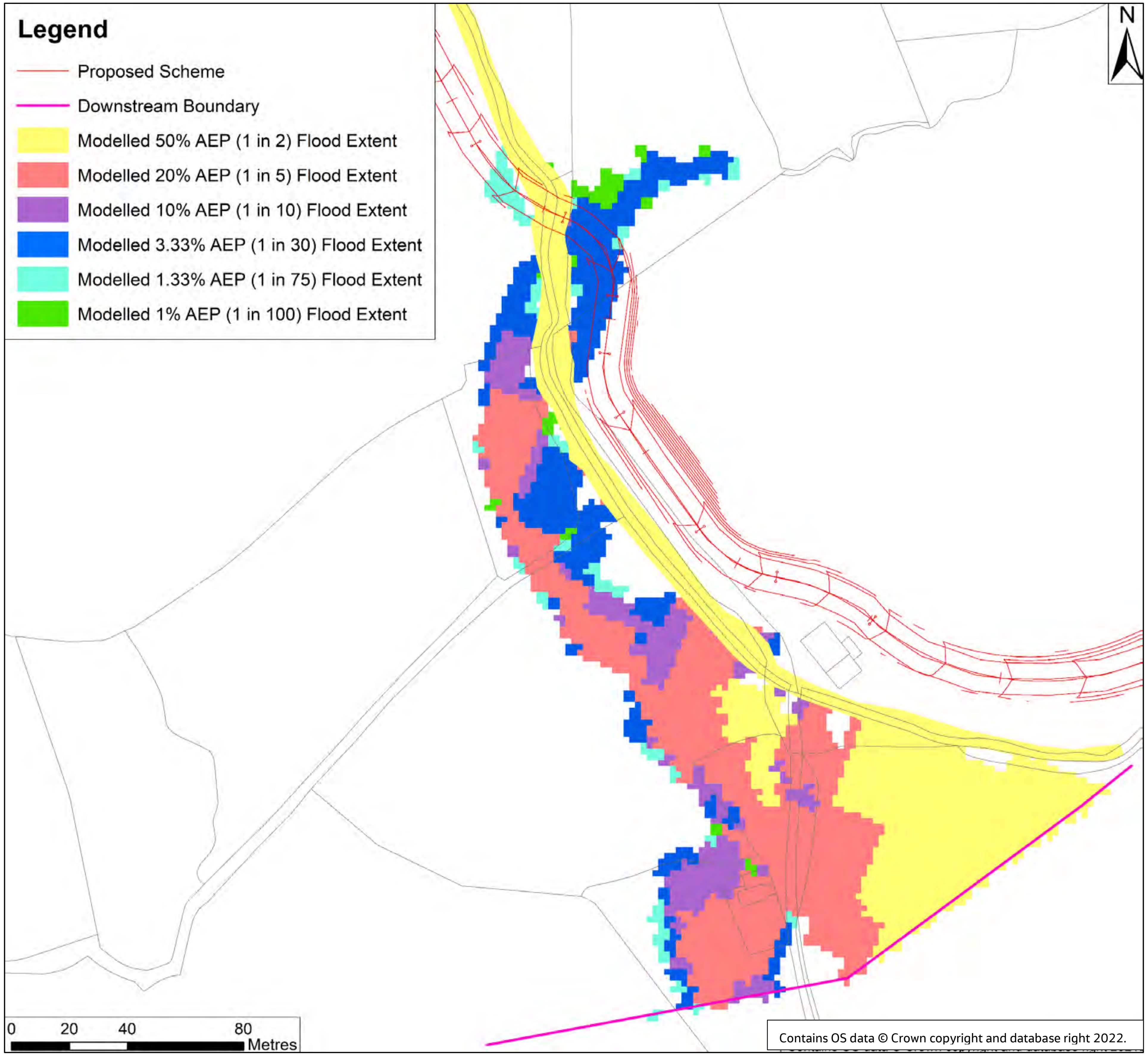


Figure D-4 Coplow Brook Baseline modelled maximum flood extents, with proposed haul road footprint overlaid.

Appendix E. Water Level Difference Maps

E.1 River Ribble

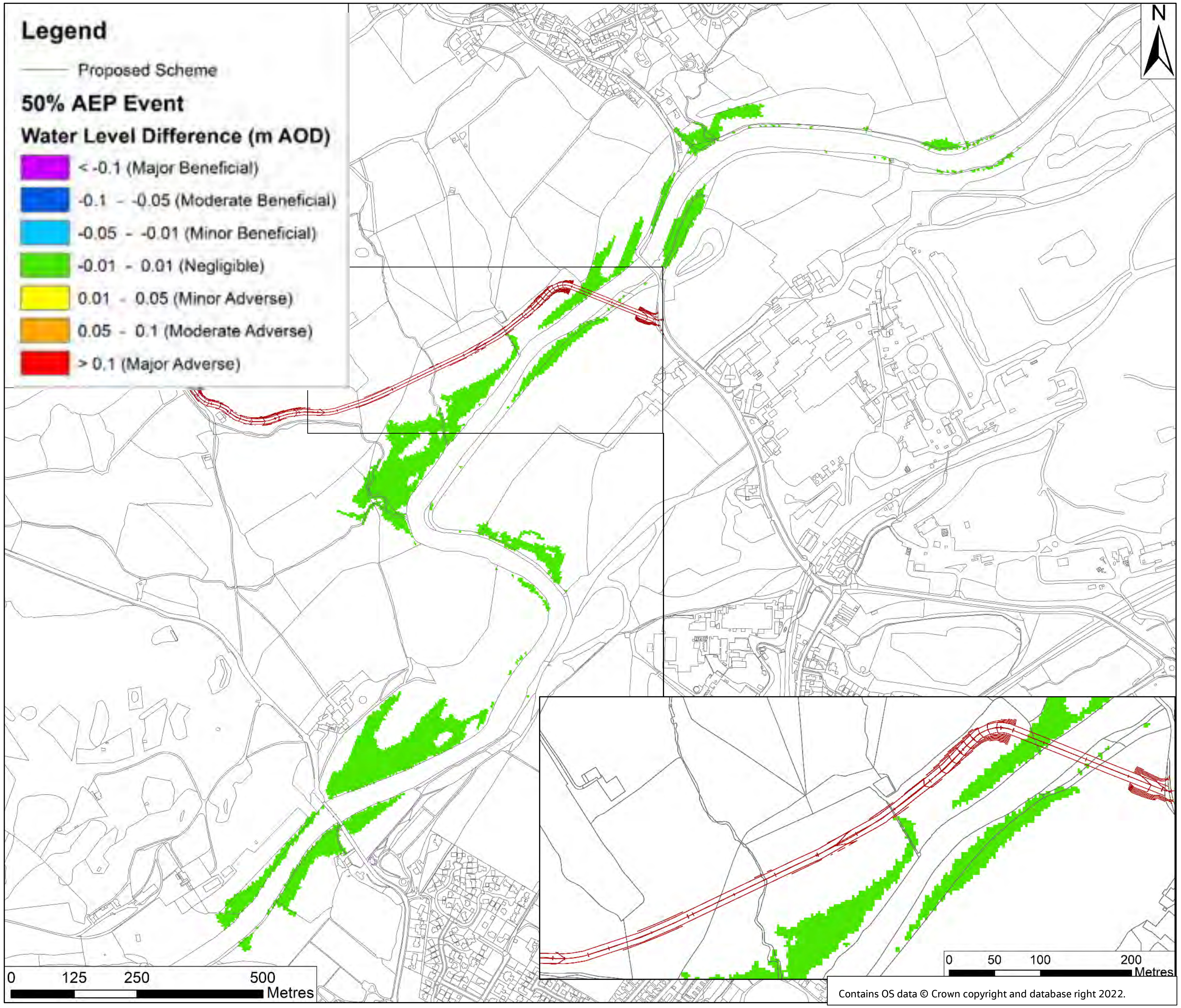


Figure E-1 Flood level difference map for the 50% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

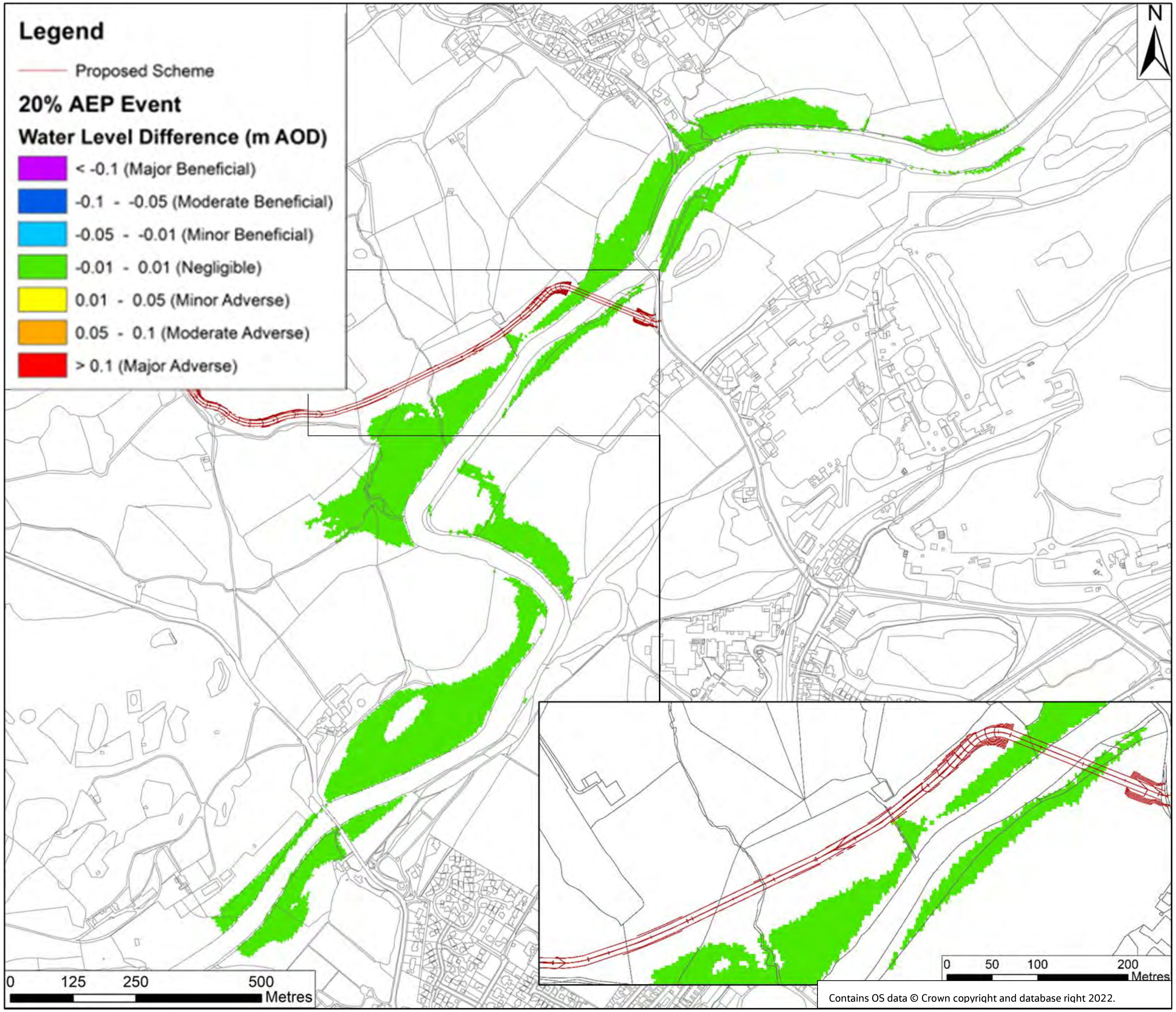


Figure E-2 Flood level difference map for the 20% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

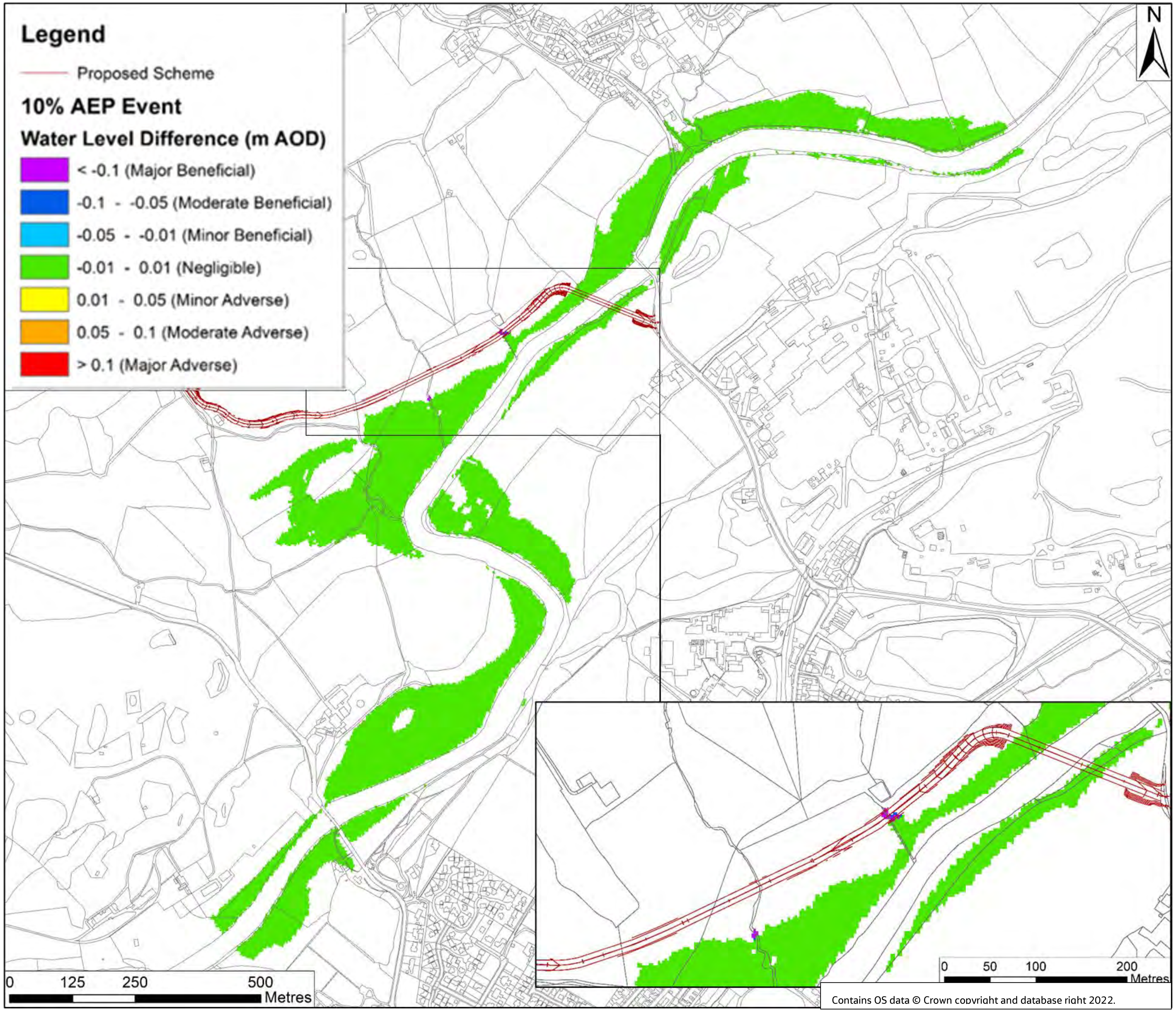


Figure E-3 Flood level difference map for the 10% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

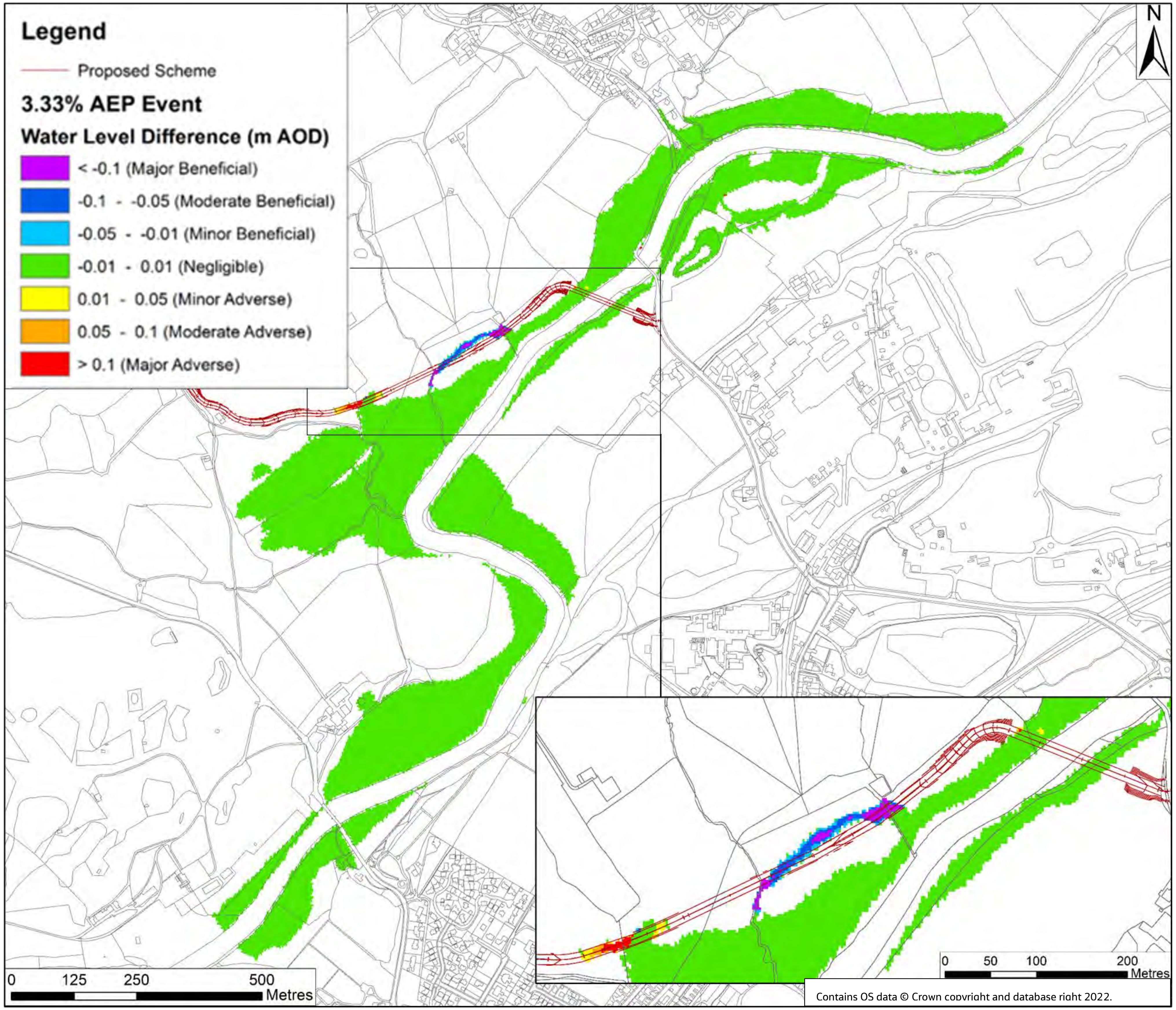


Figure E-4 Flood level difference map for the 3.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

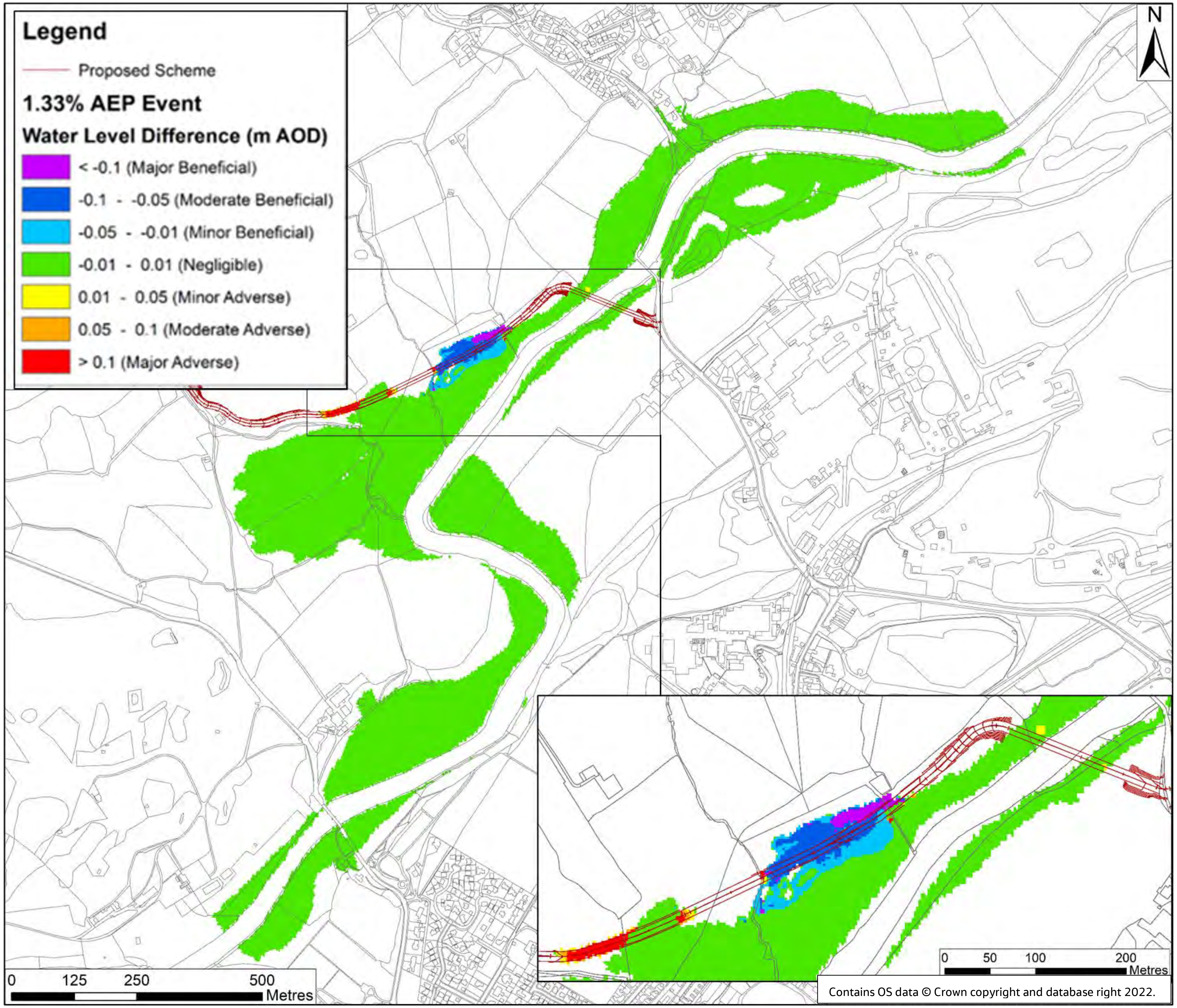


Figure E-5 Flood level difference map for the 1.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

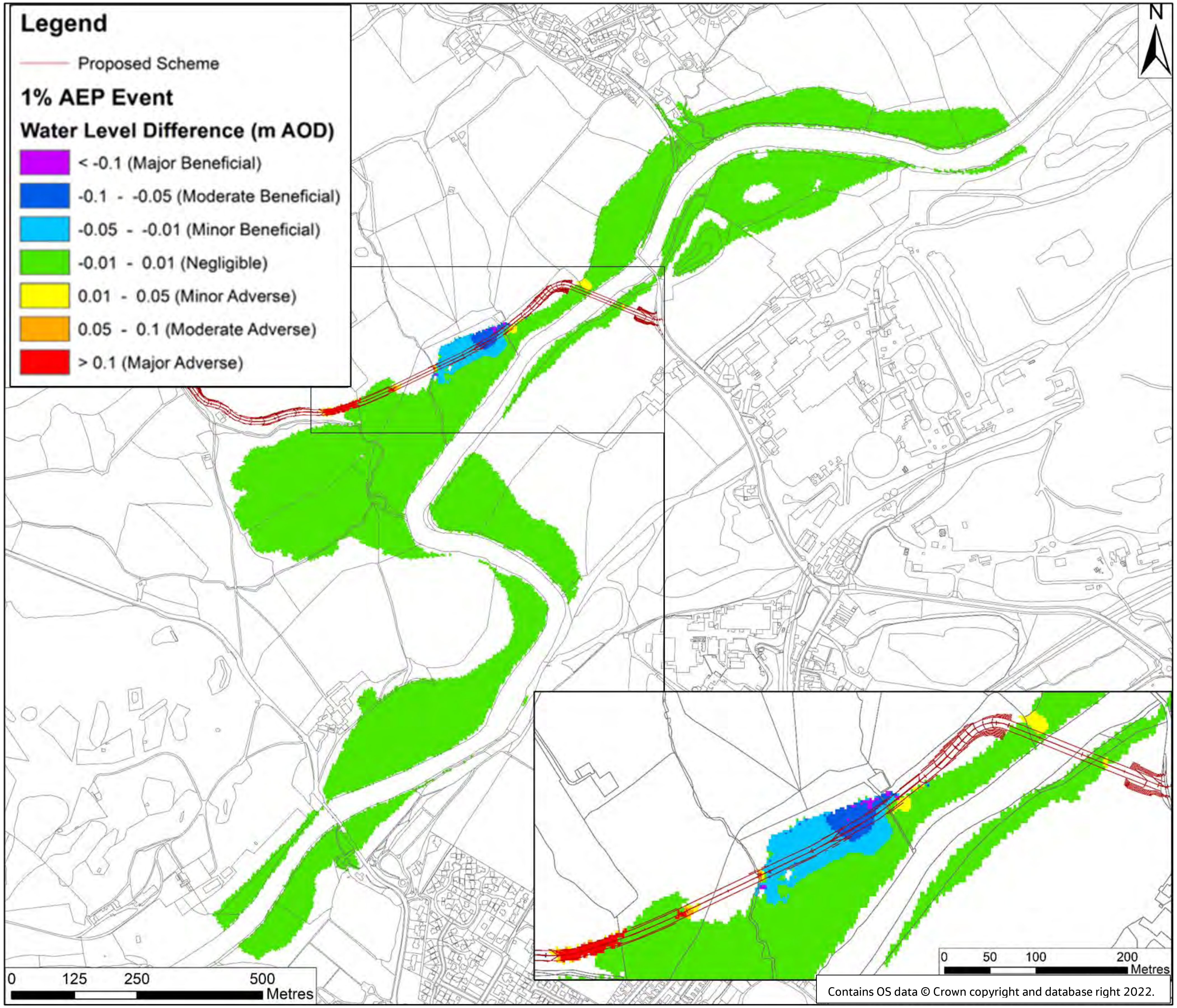


Figure E-6 Flood level difference map for the 1% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

E.2 Unnamed Watercourse

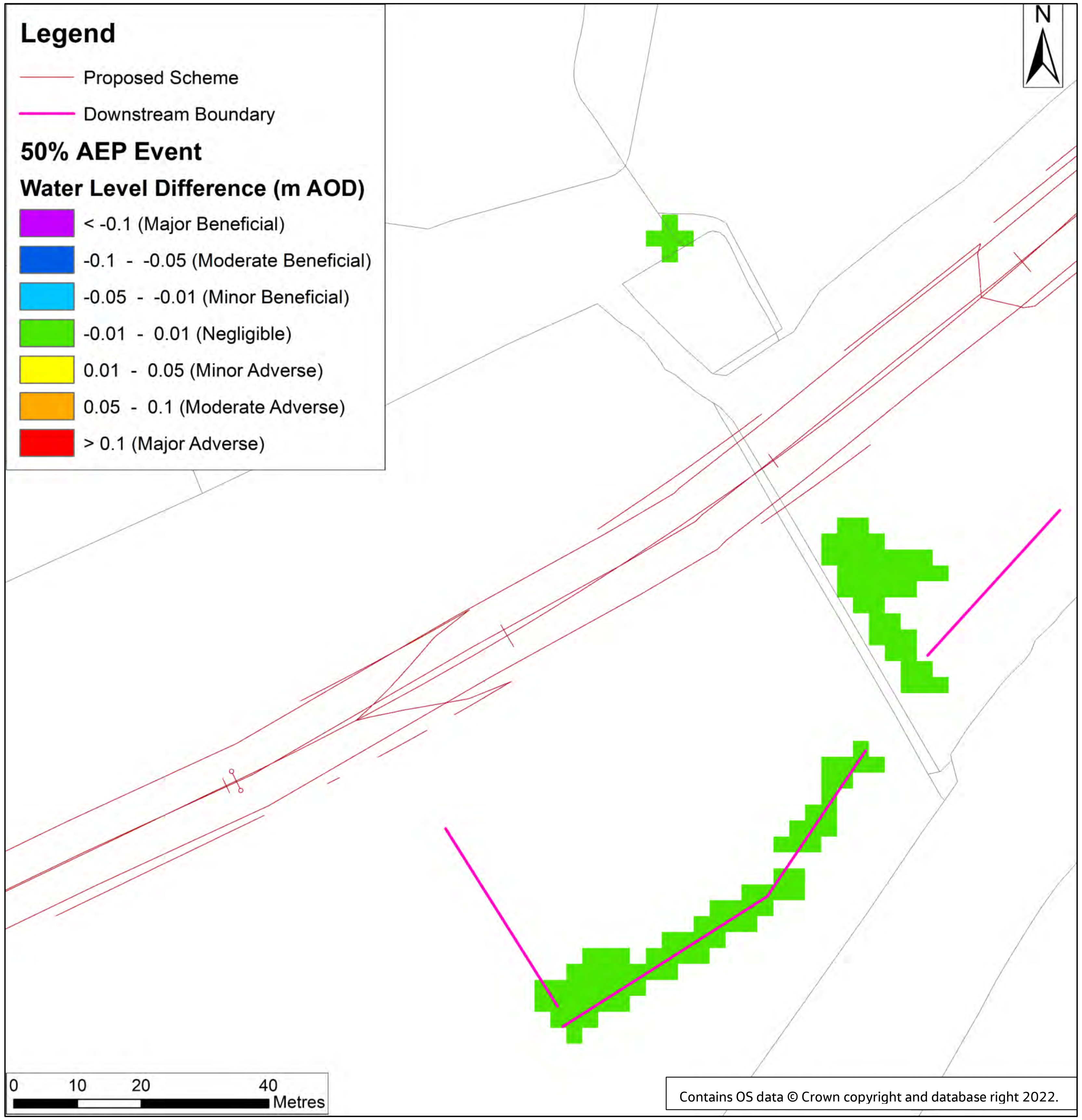


Figure E-7 Flood level difference map for the 50% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

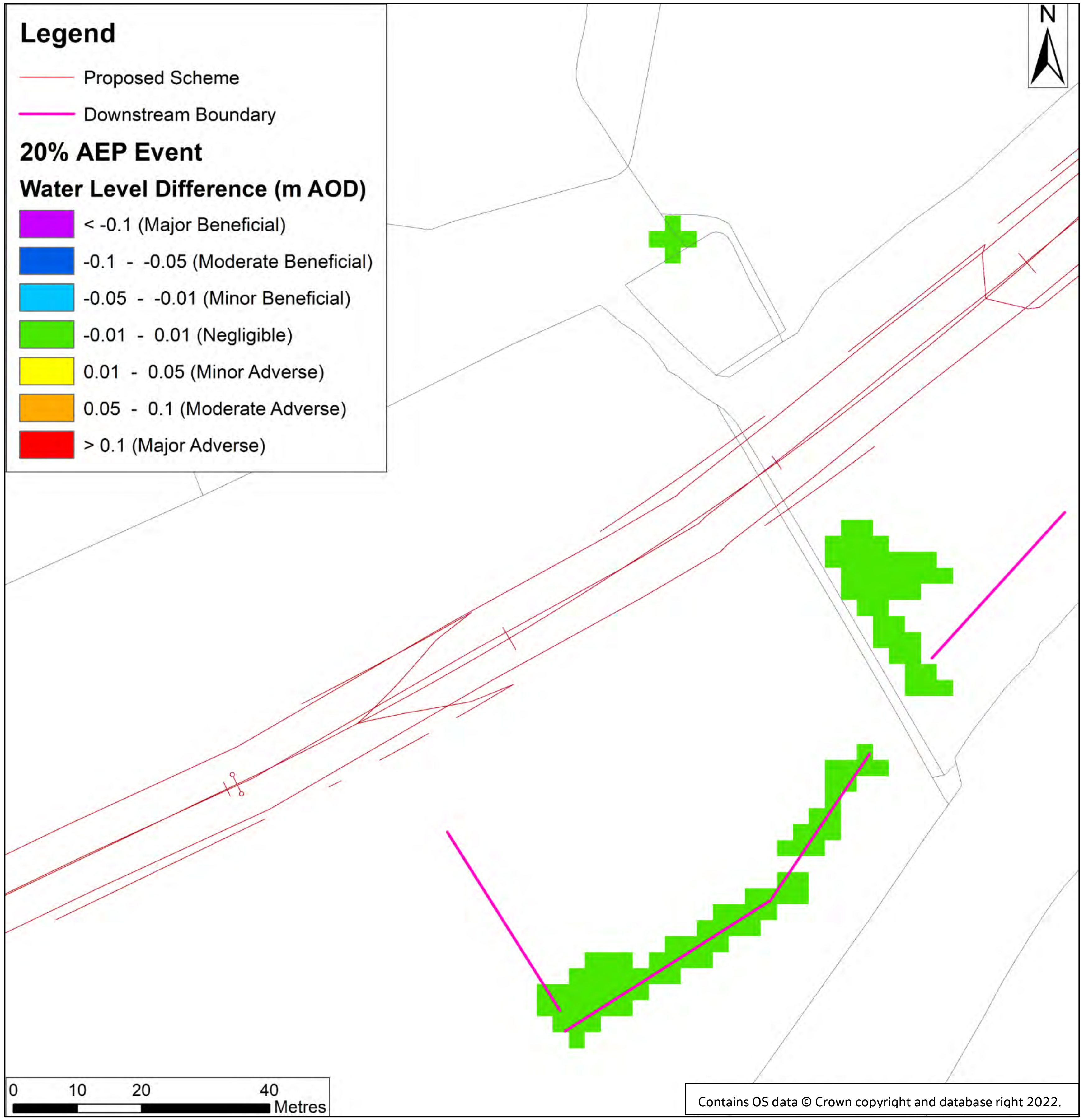


Figure E-8 Flood level difference map for the 20% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

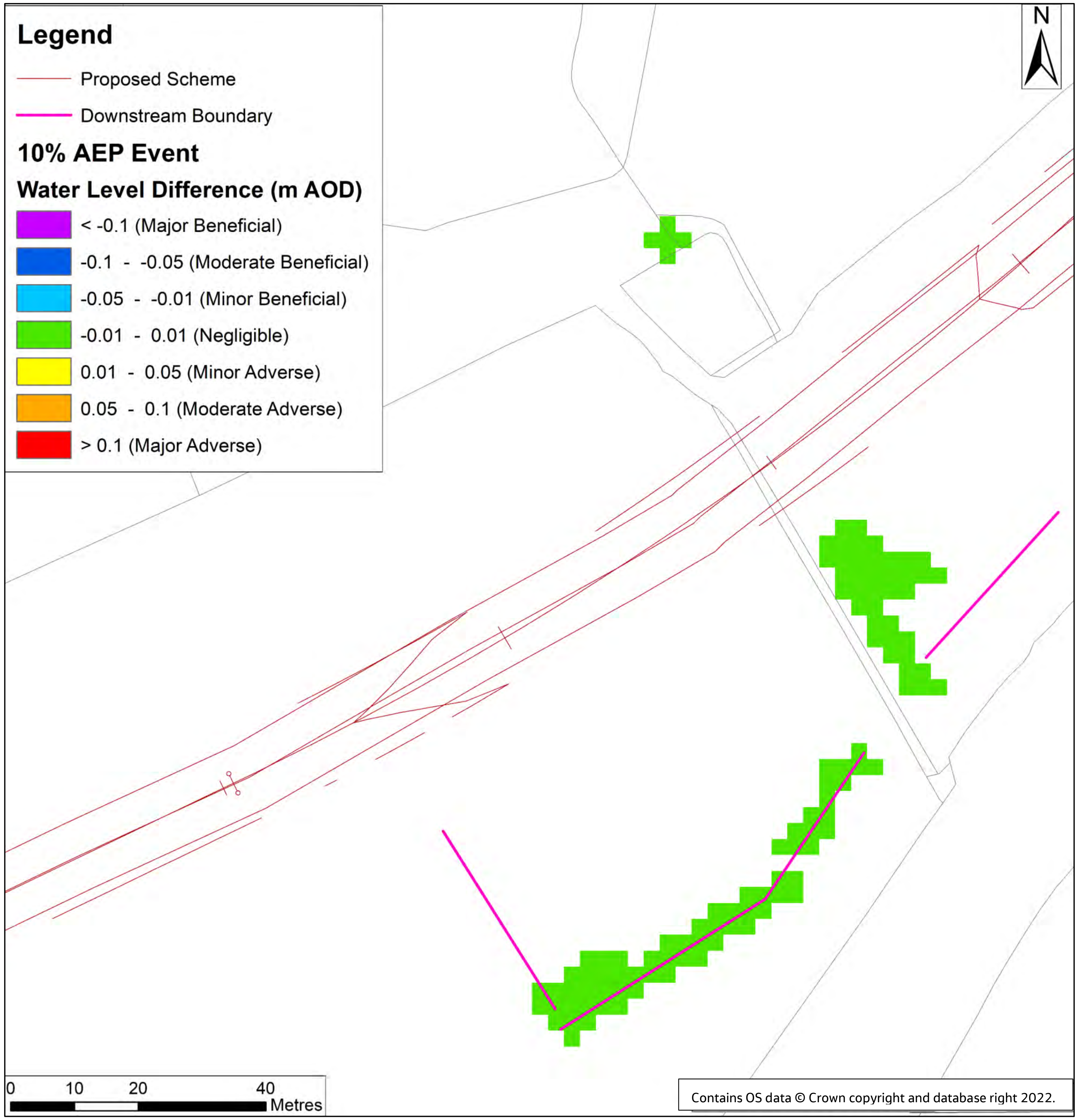


Figure E-9 Flood level difference map for the 10% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

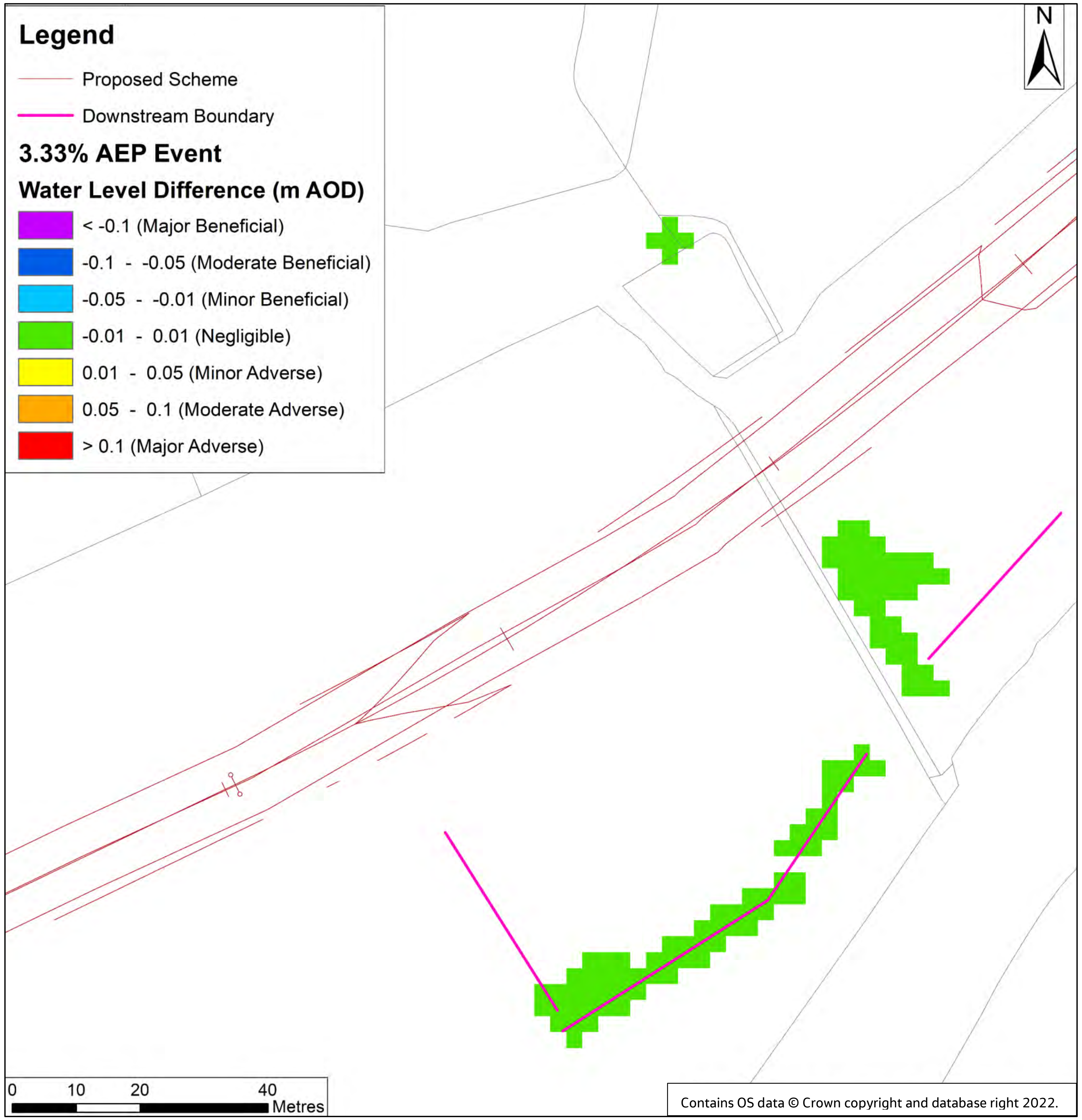


Figure E-10 Flood level difference map for the 3.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

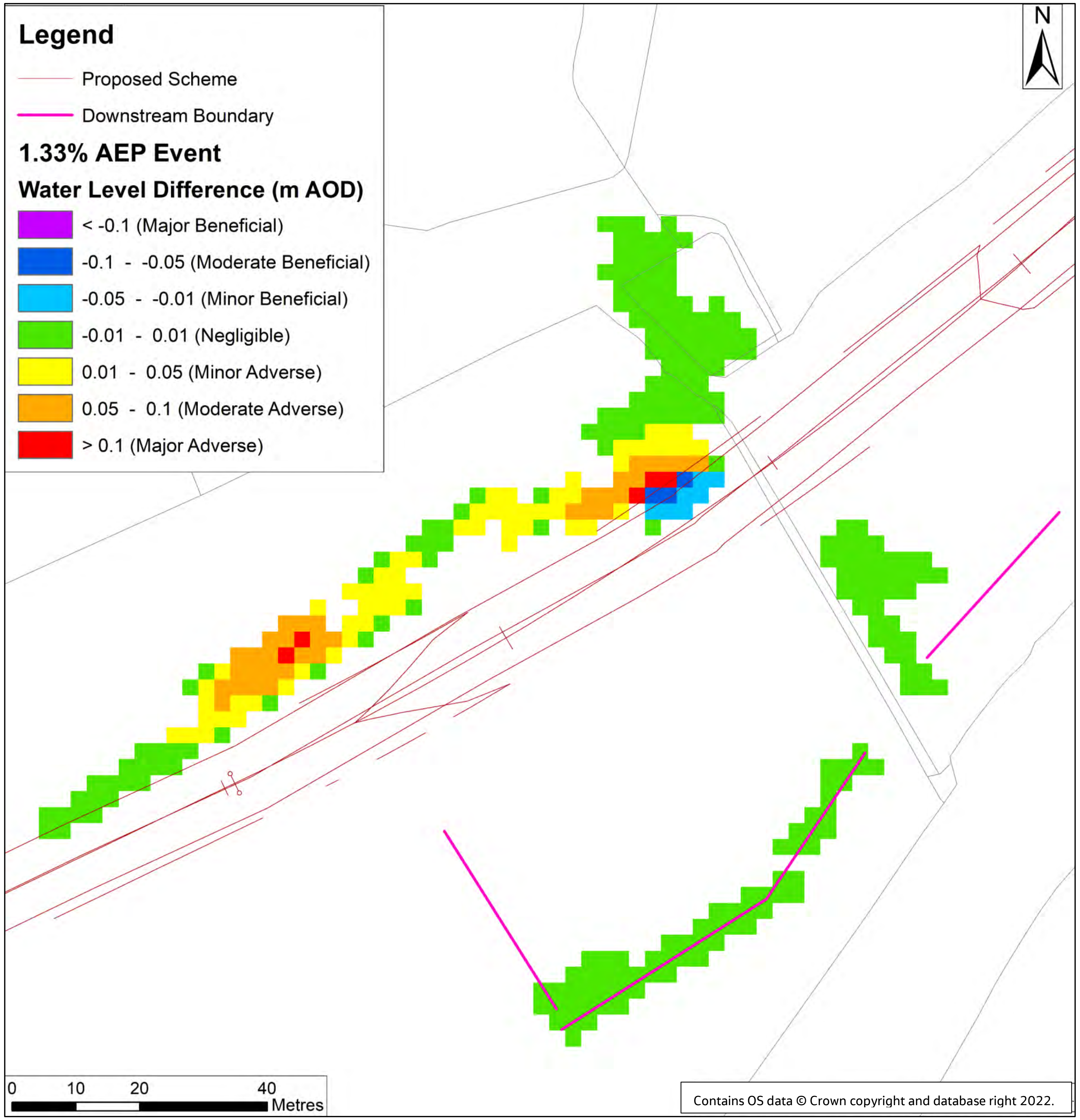


Figure E-11 Flood level difference map for the 1.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

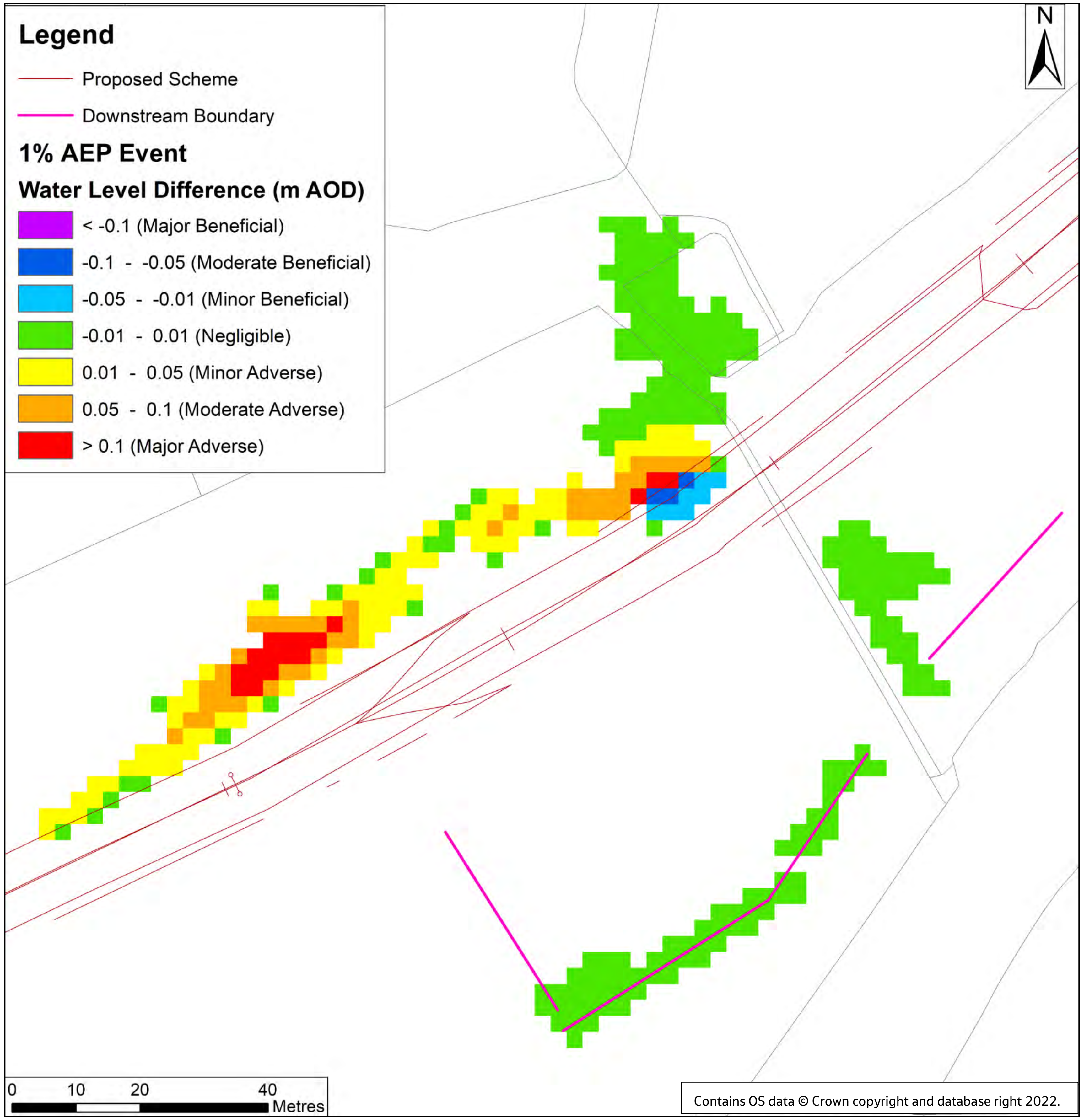


Figure E-12 Flood level difference map for the 1% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

E.3 Greg Syke

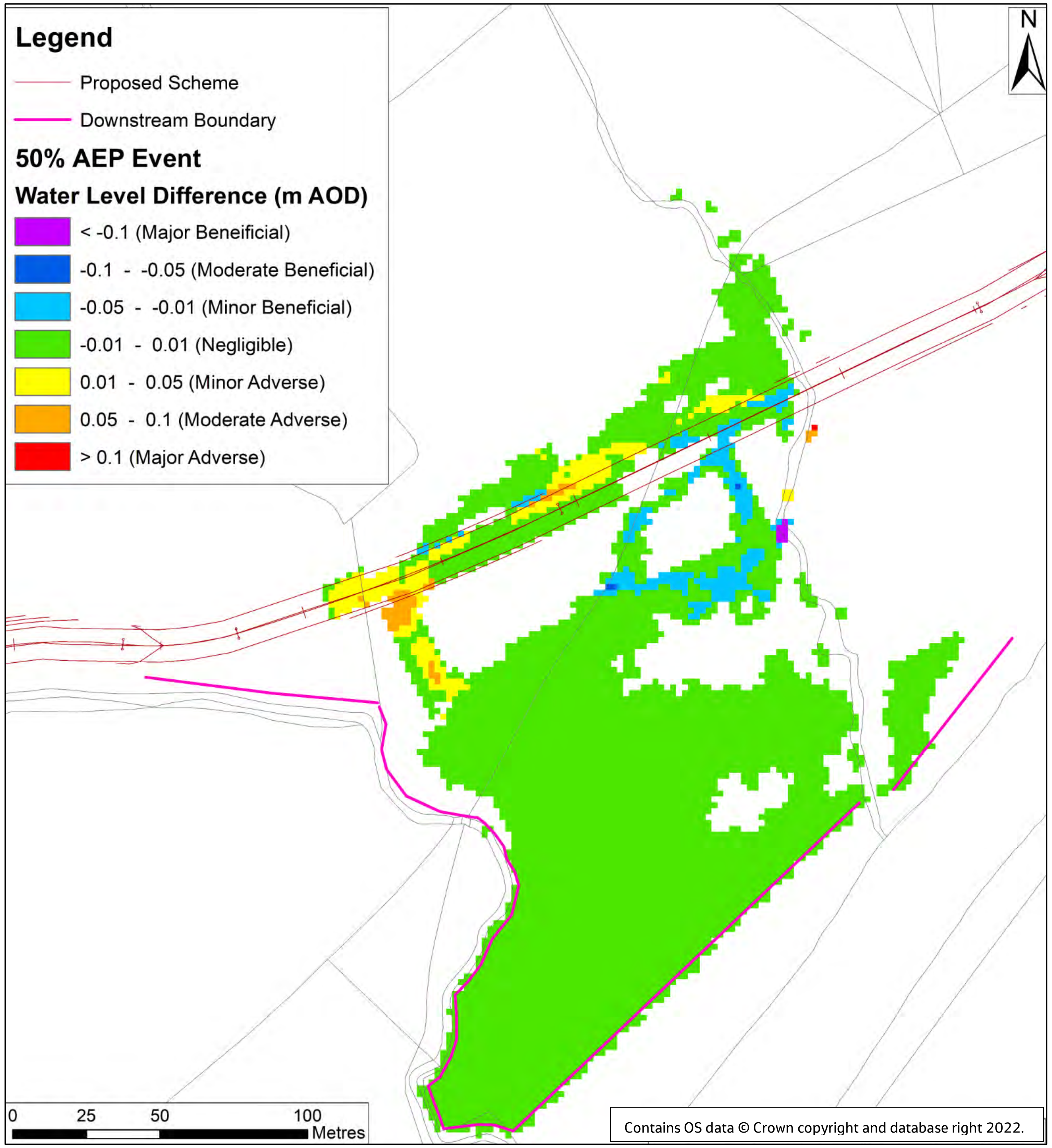


Figure E-13 Flood level difference map for the 50% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

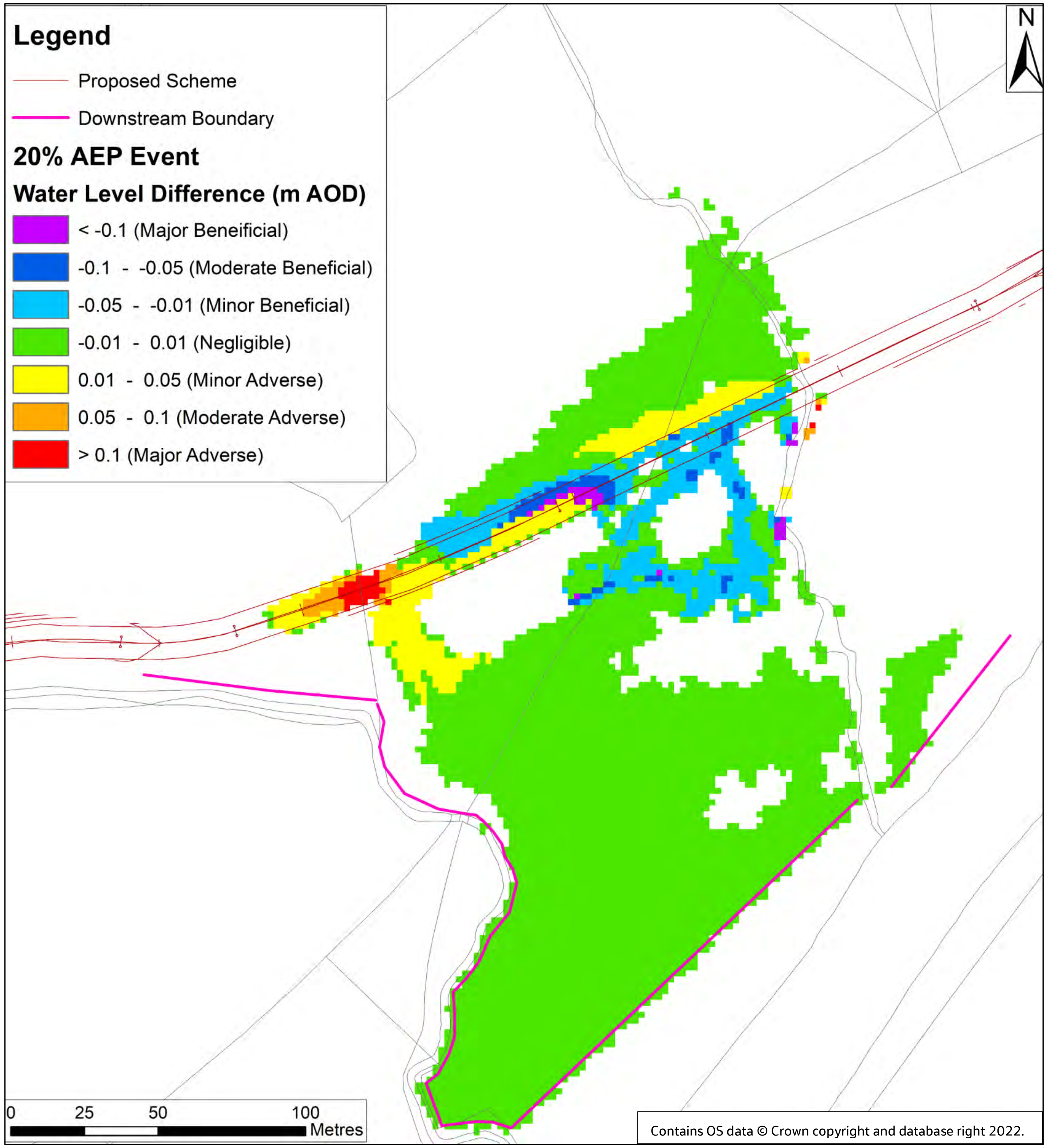


Figure E-14 Flood level difference map for the 20% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

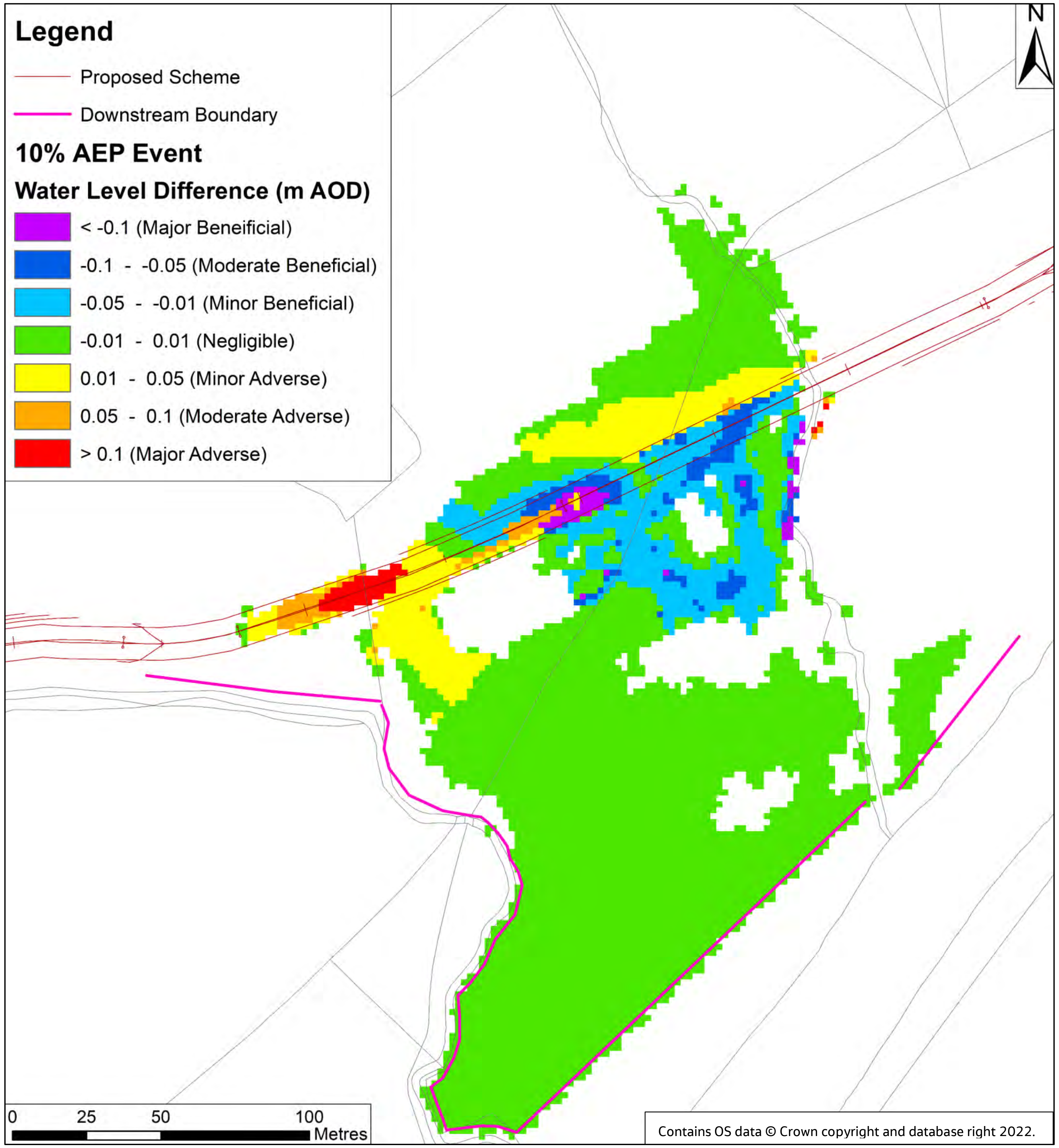


Figure E-15 Flood level difference map for the 10% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

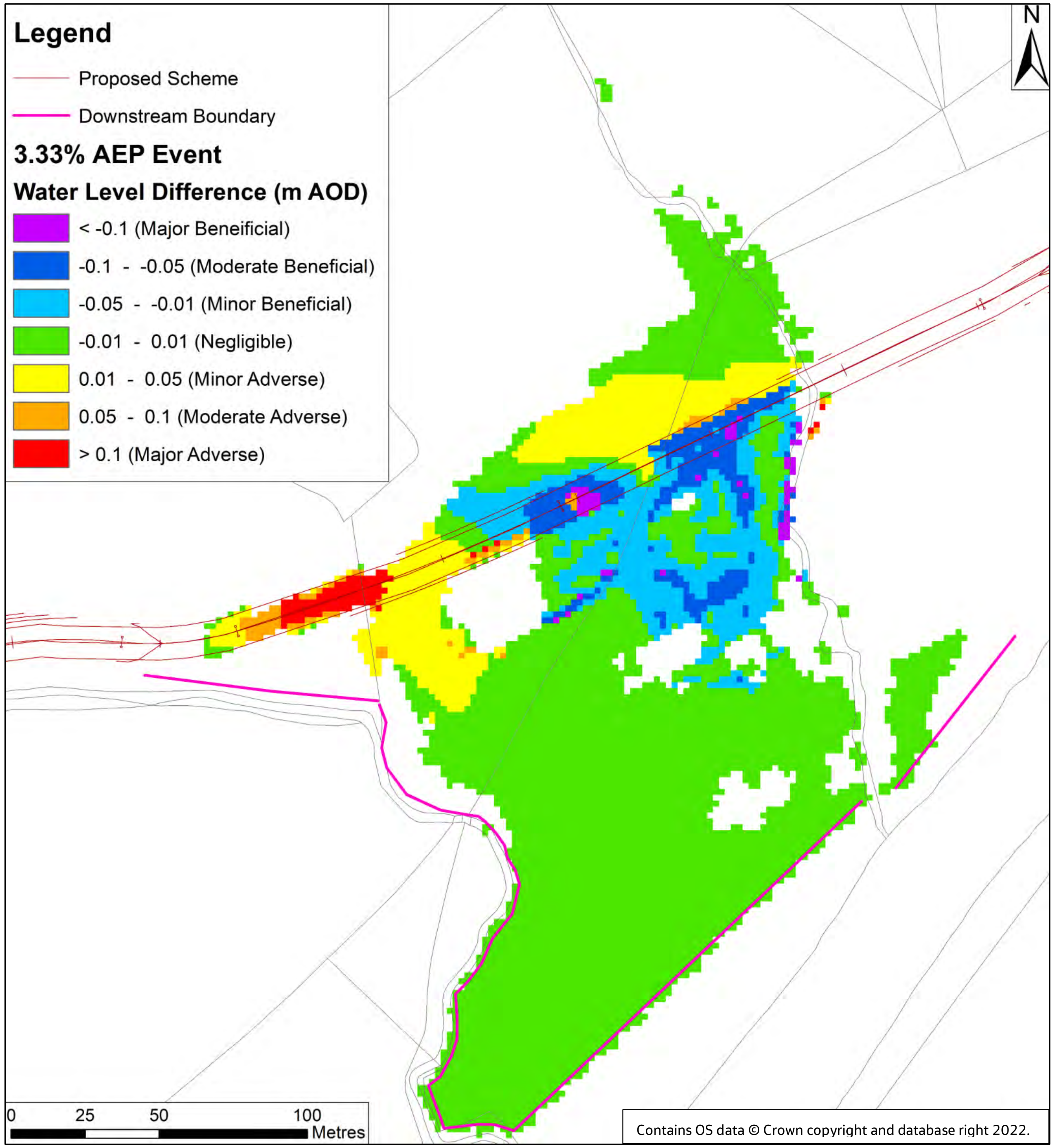


Figure E-16 Flood level difference map for the 3.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

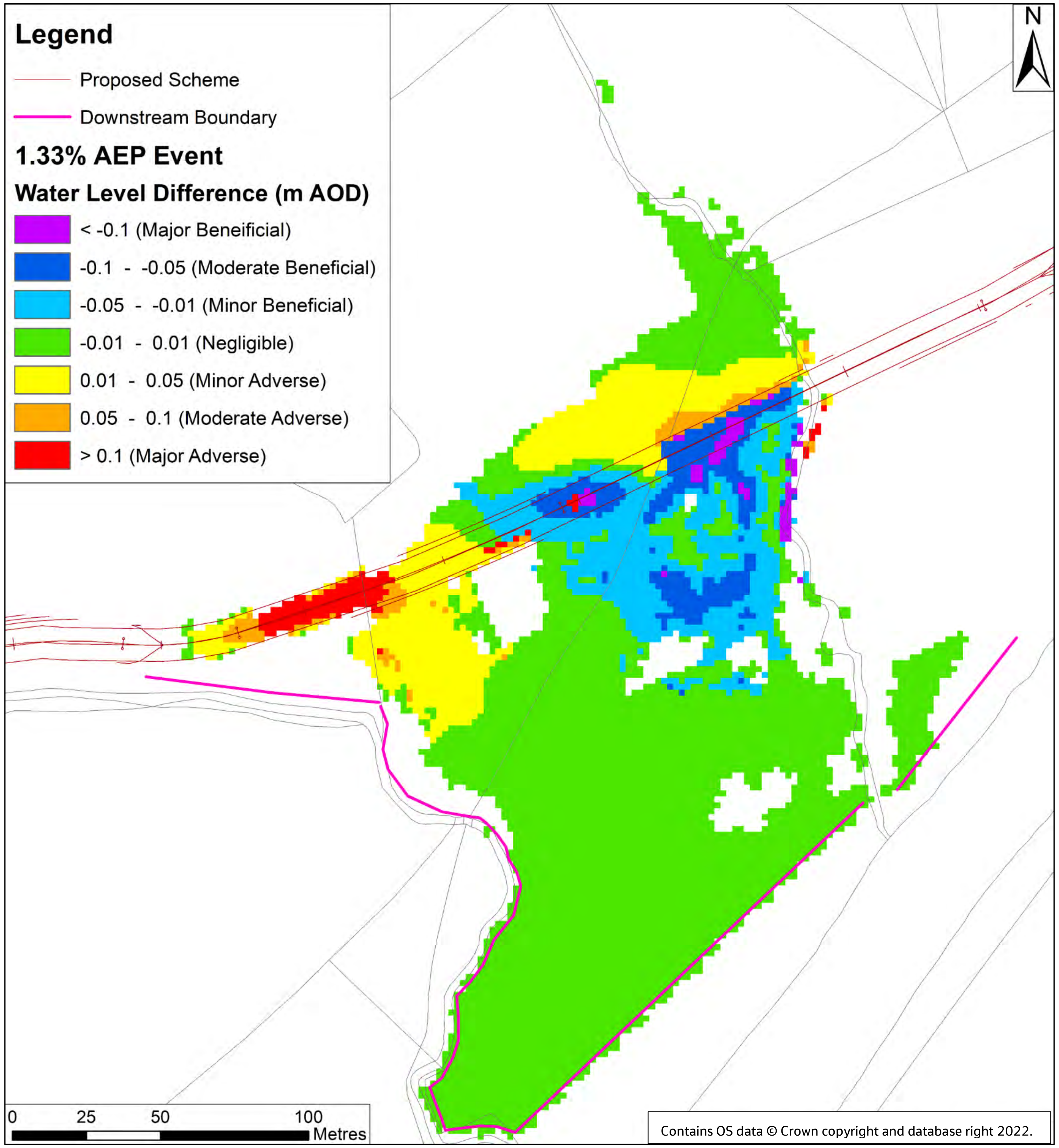


Figure E-17 Flood level difference map for the 1.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

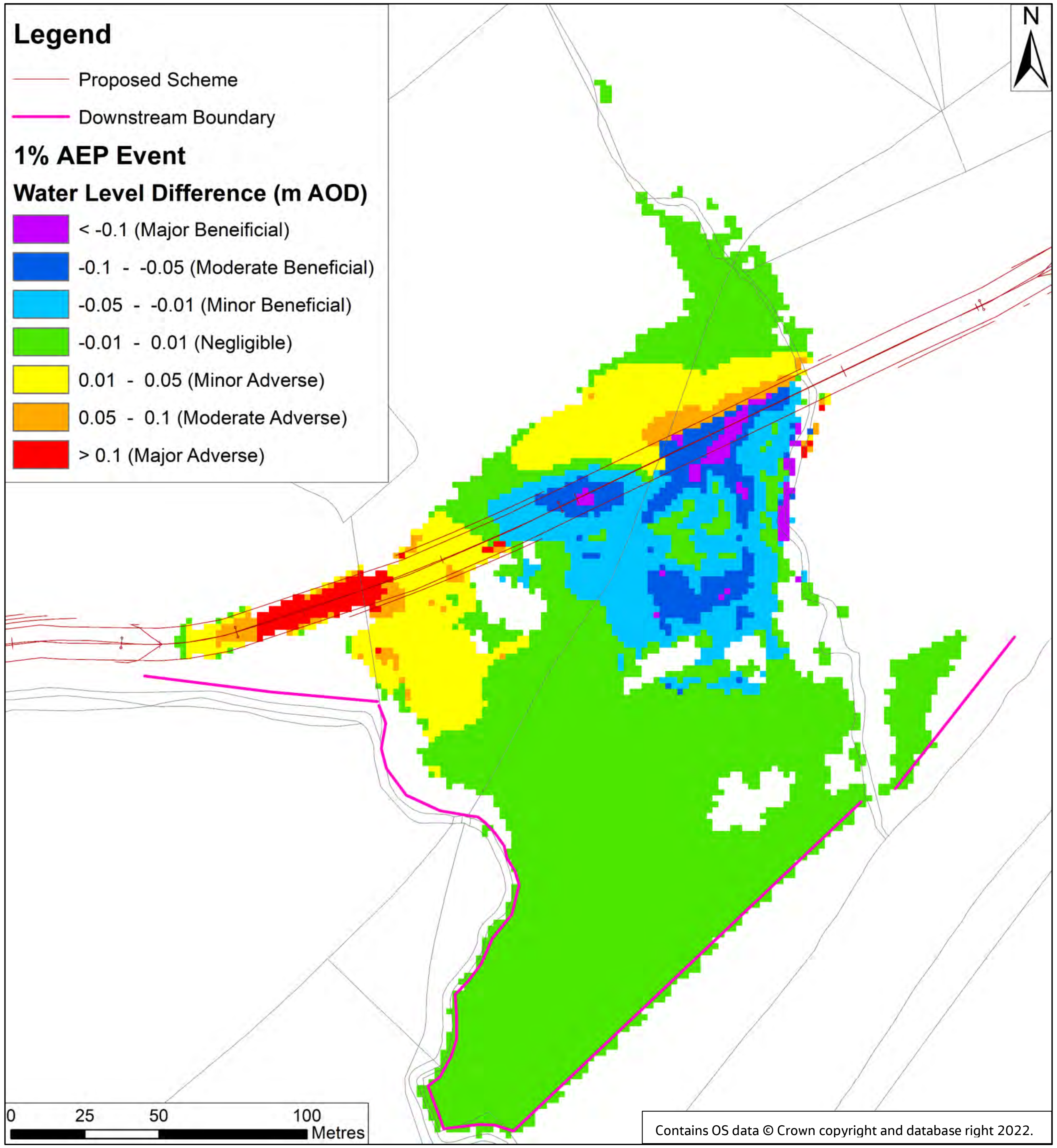


Figure E-18 Flood level difference map for the 1% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

E.4 Coplow Brook

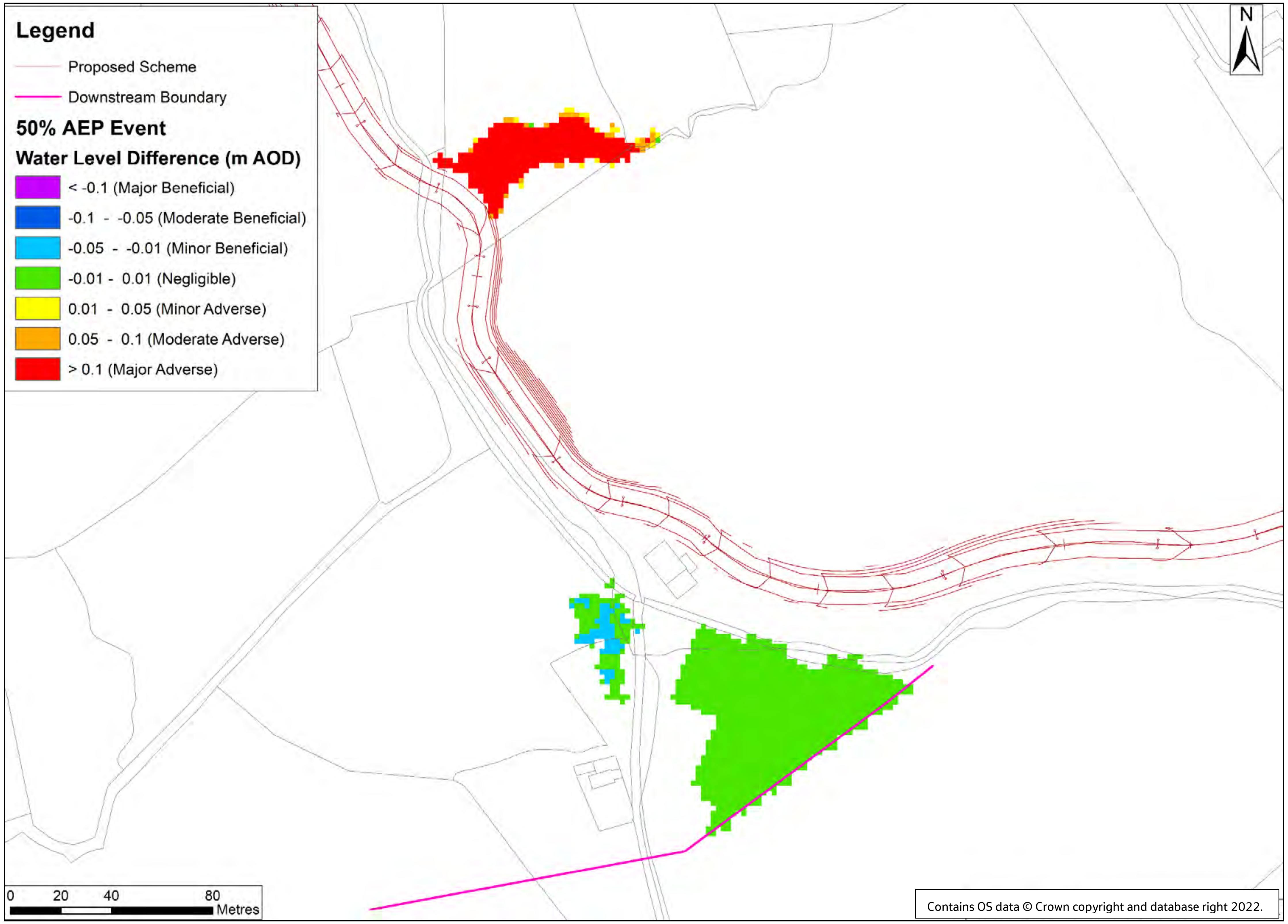


Figure E-19 Flood level difference map for the 50% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

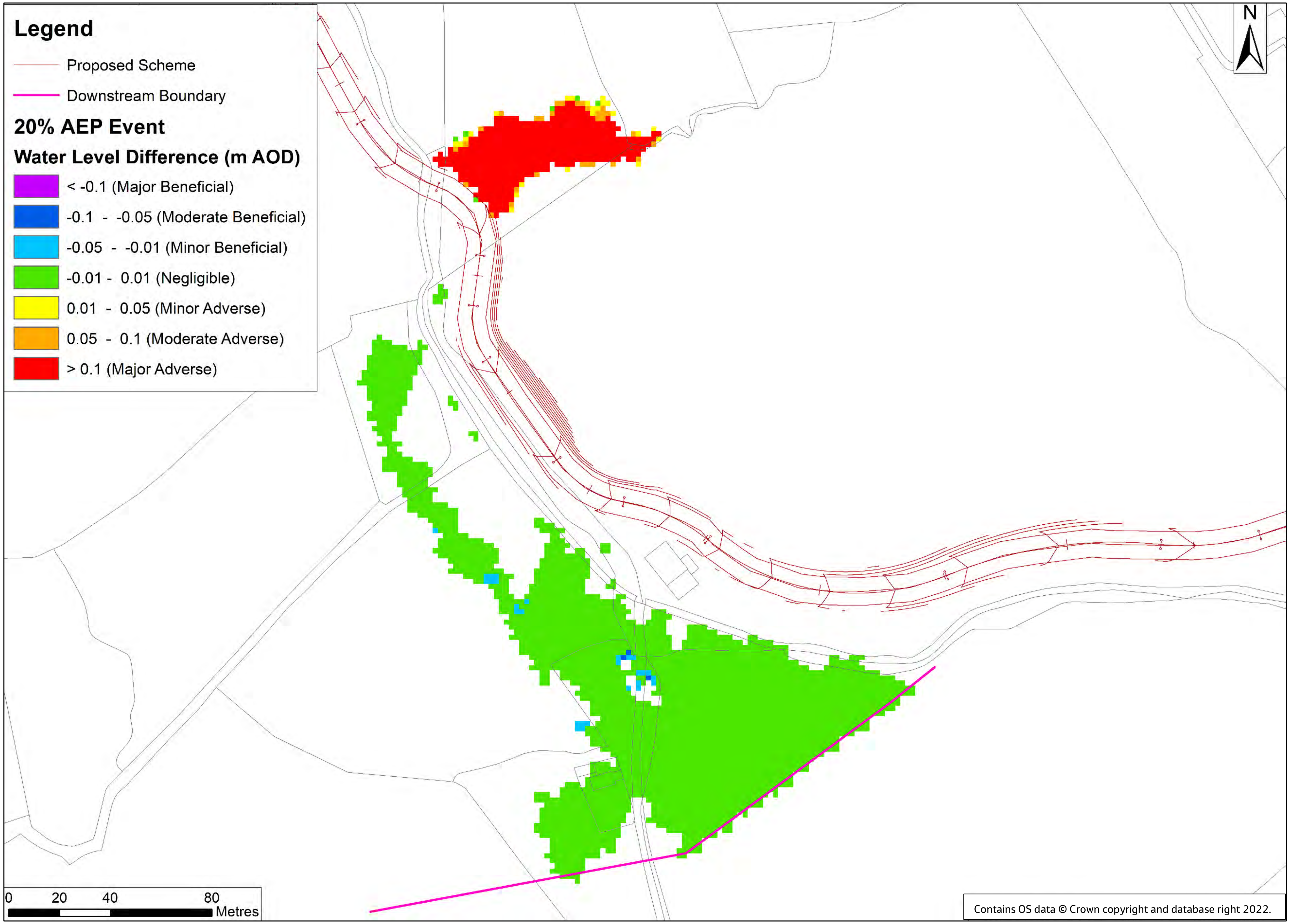


Figure E-20 Flood level difference map for the 20% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

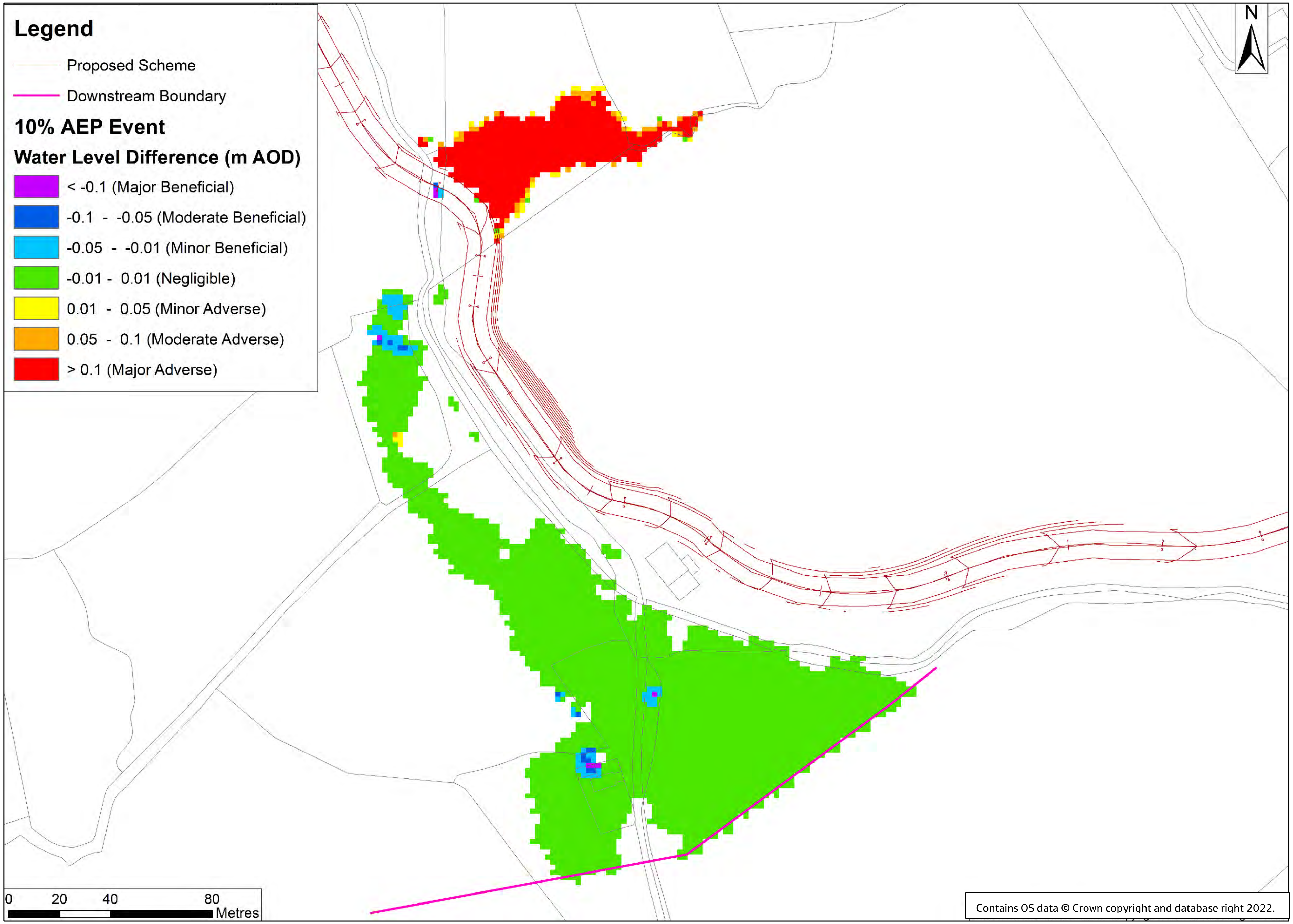


Figure E-21 Flood level difference map for the 10% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

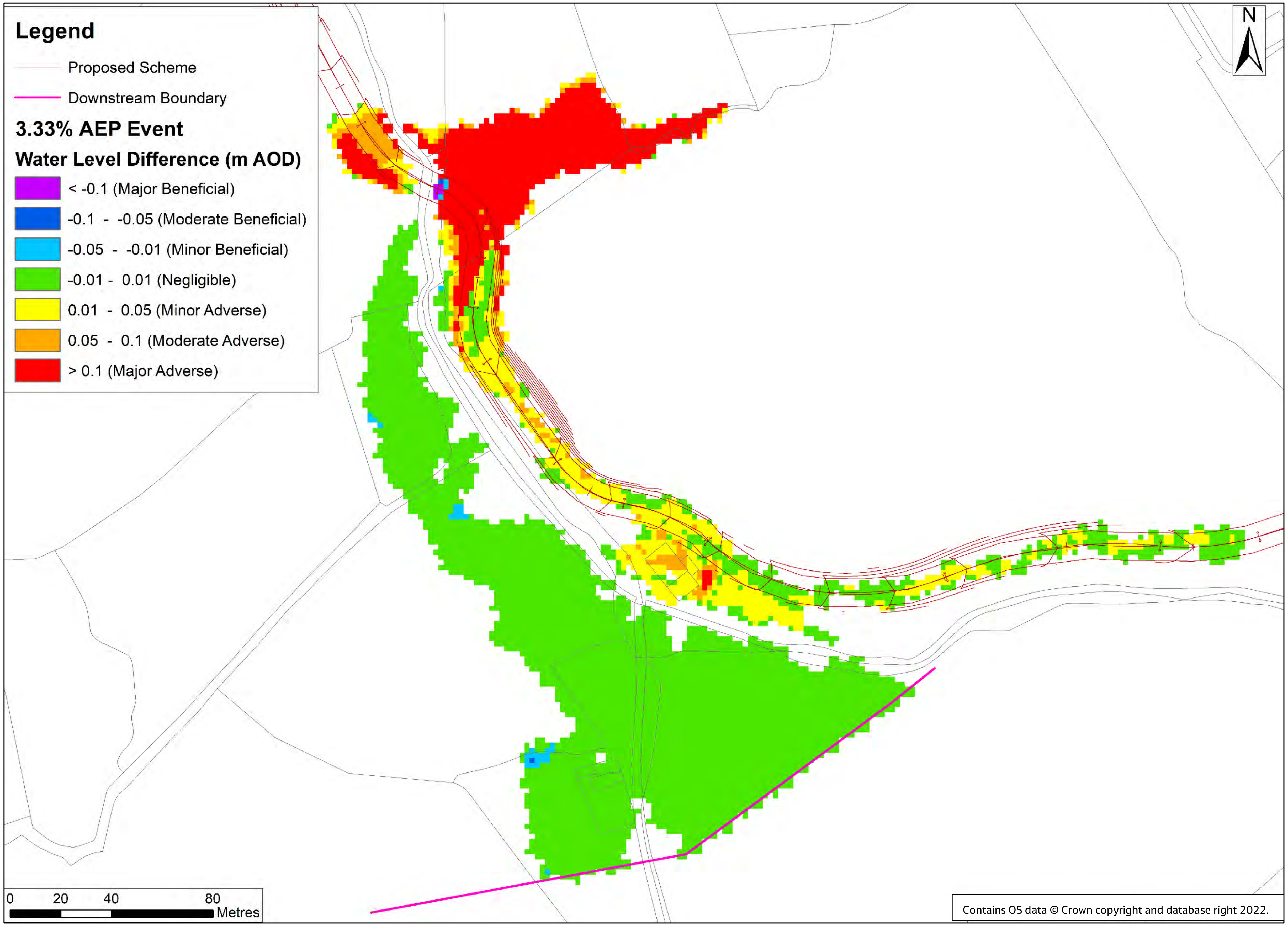


Figure E-22 Flood level difference map for the 3.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

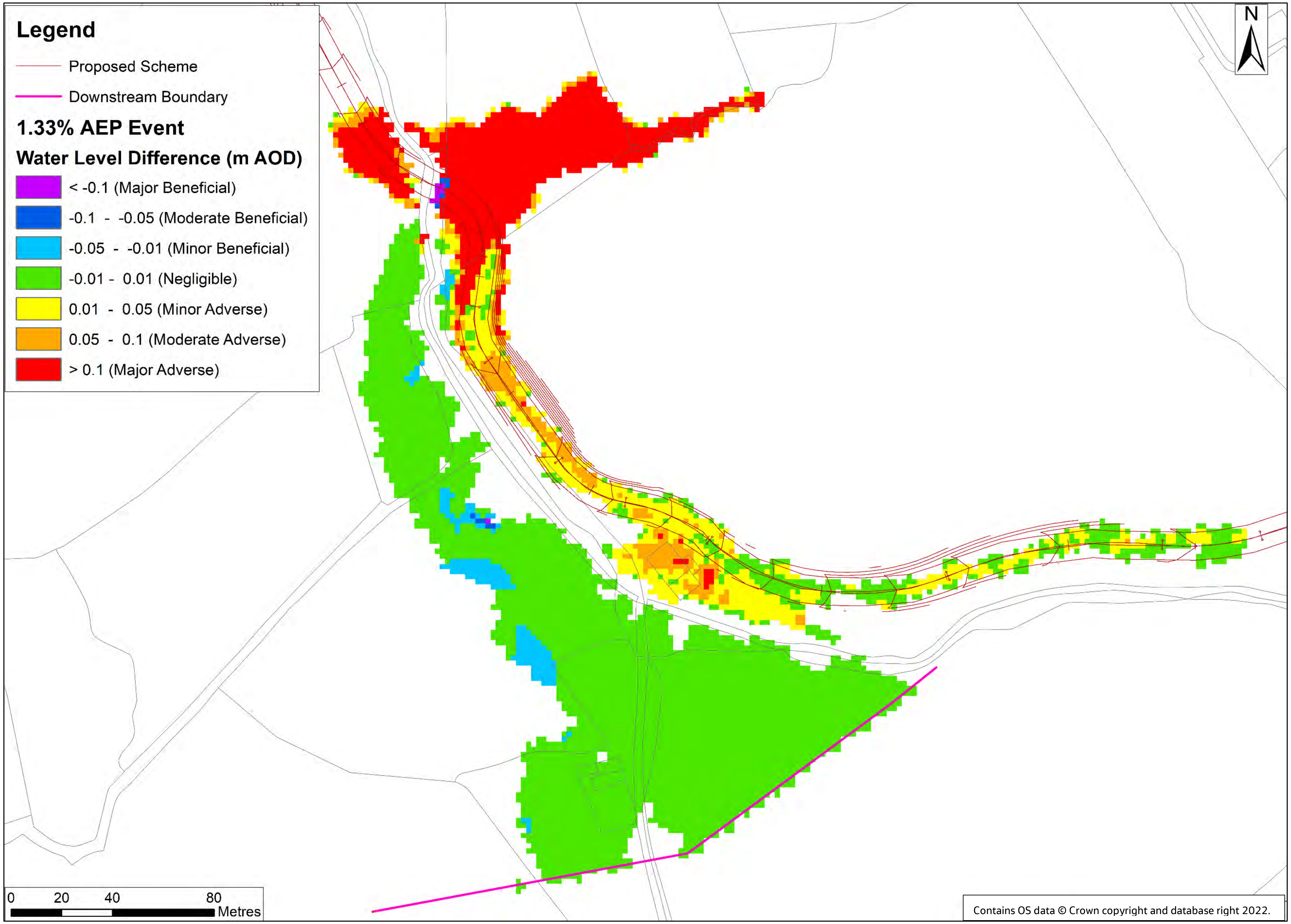


Figure E-23 Flood level difference map for the 1.33% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

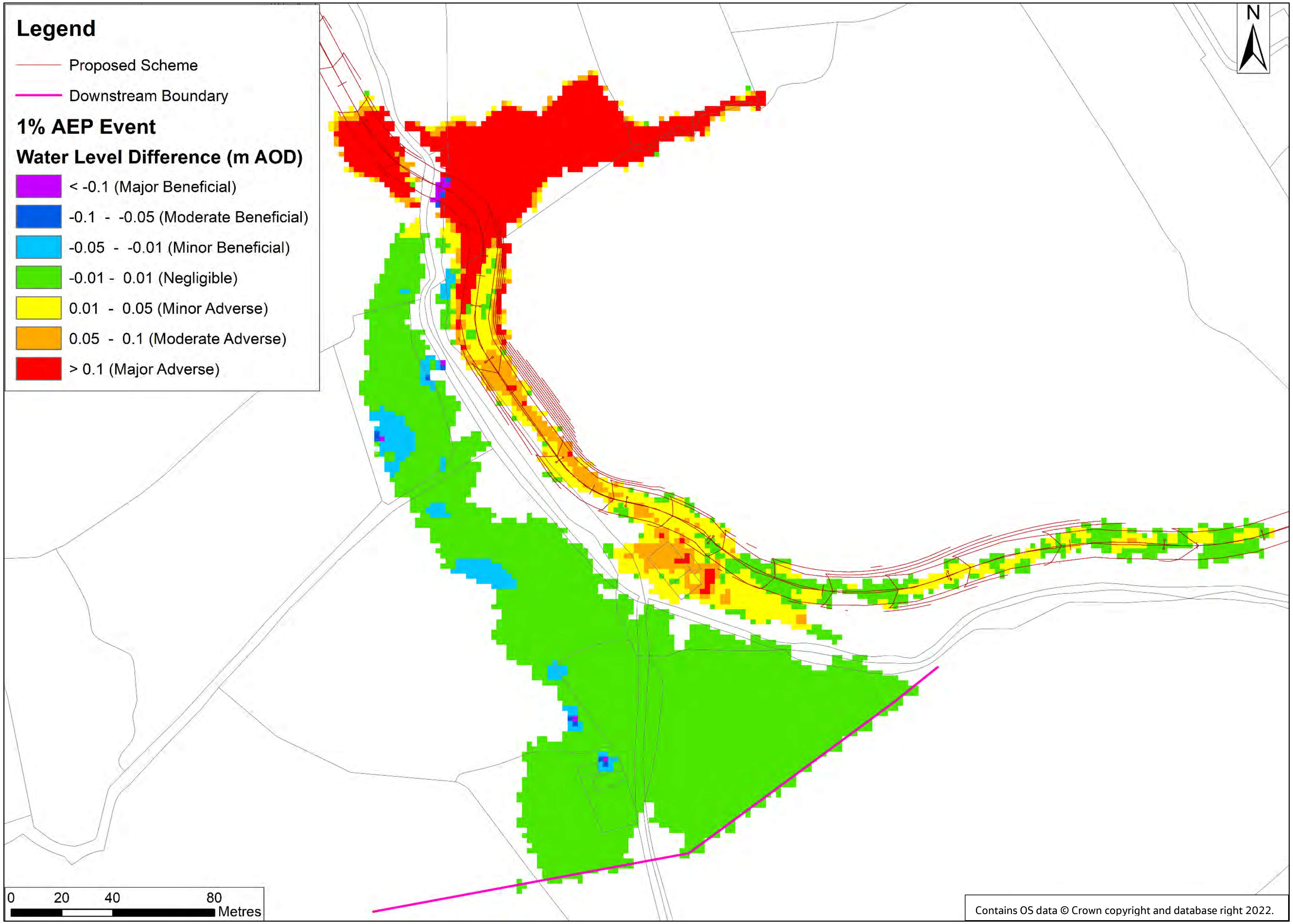


Figure E-24 Flood level difference map for the 1% AEP event. With-Proposed Ribble Crossing scenario minus baseline scenario.

Appendix F. Notional Design Drawings

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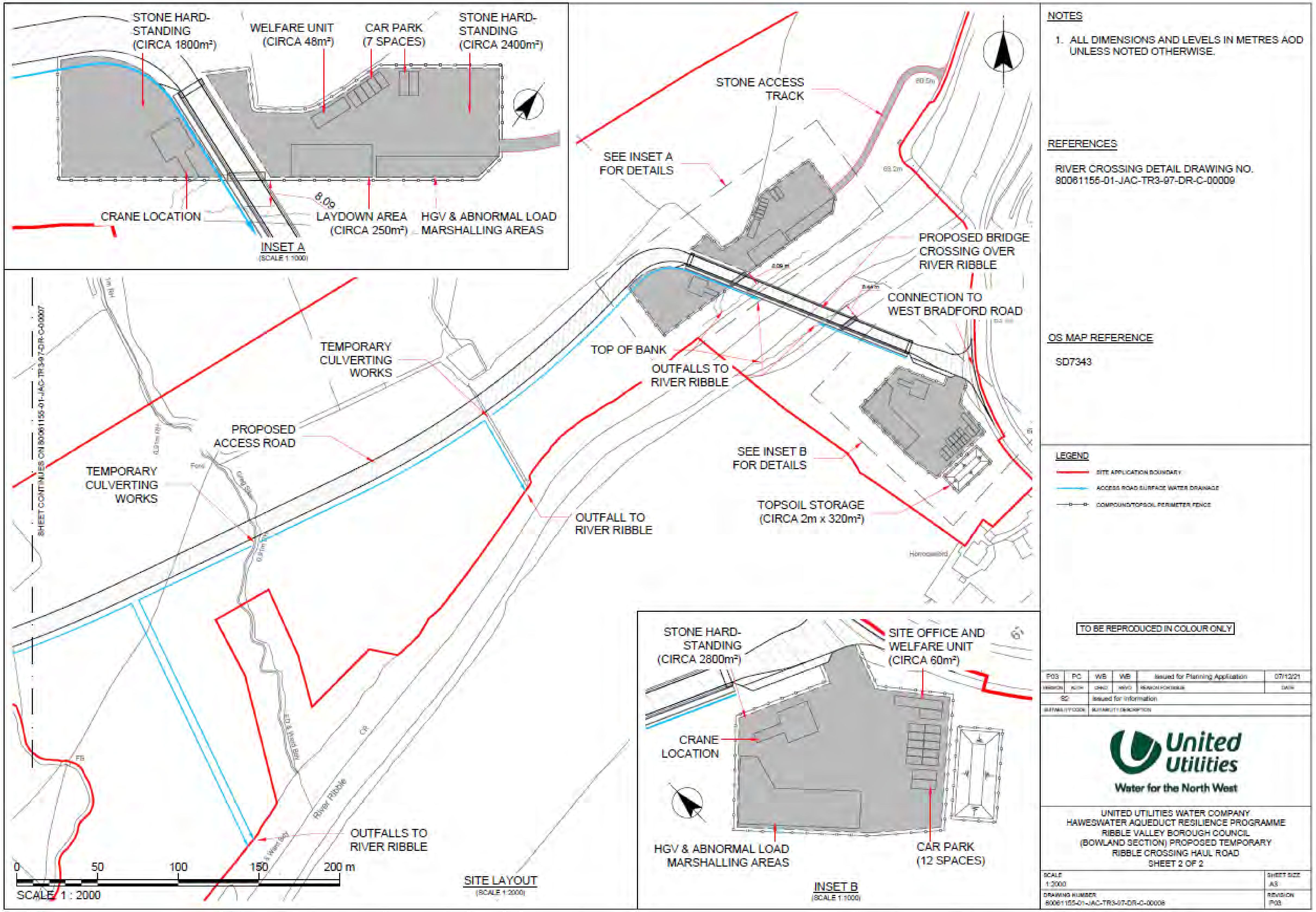

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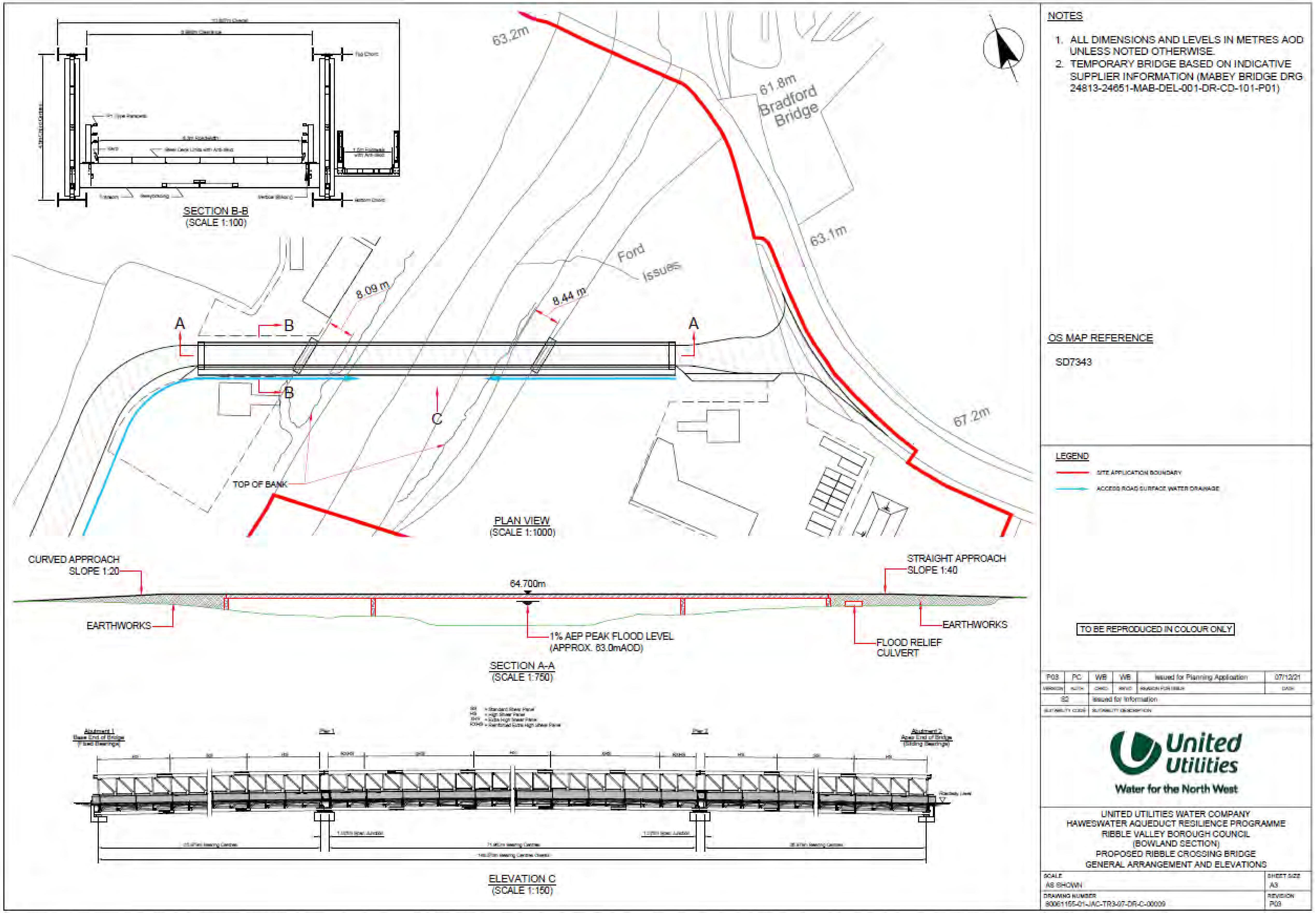

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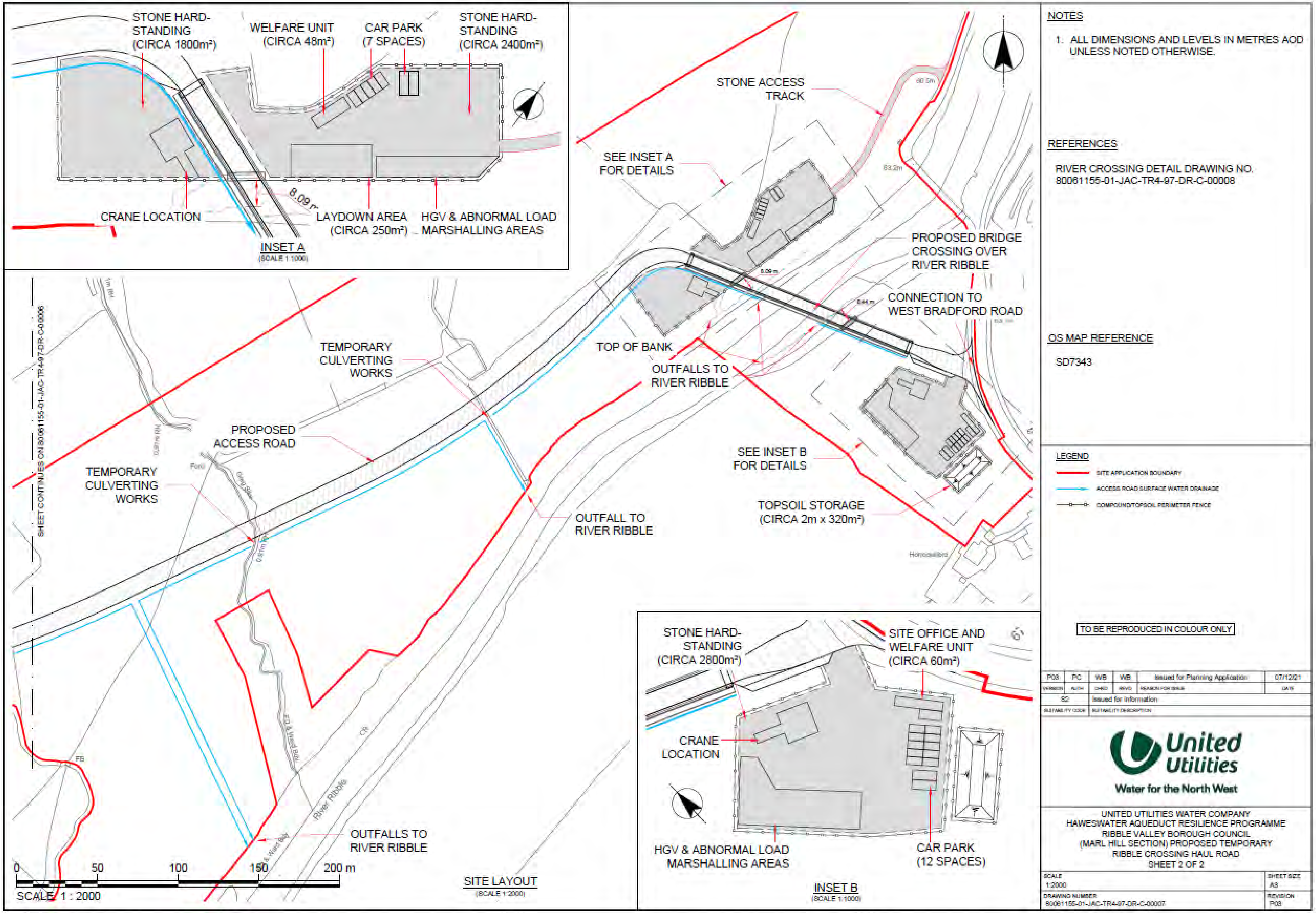

Jacobs

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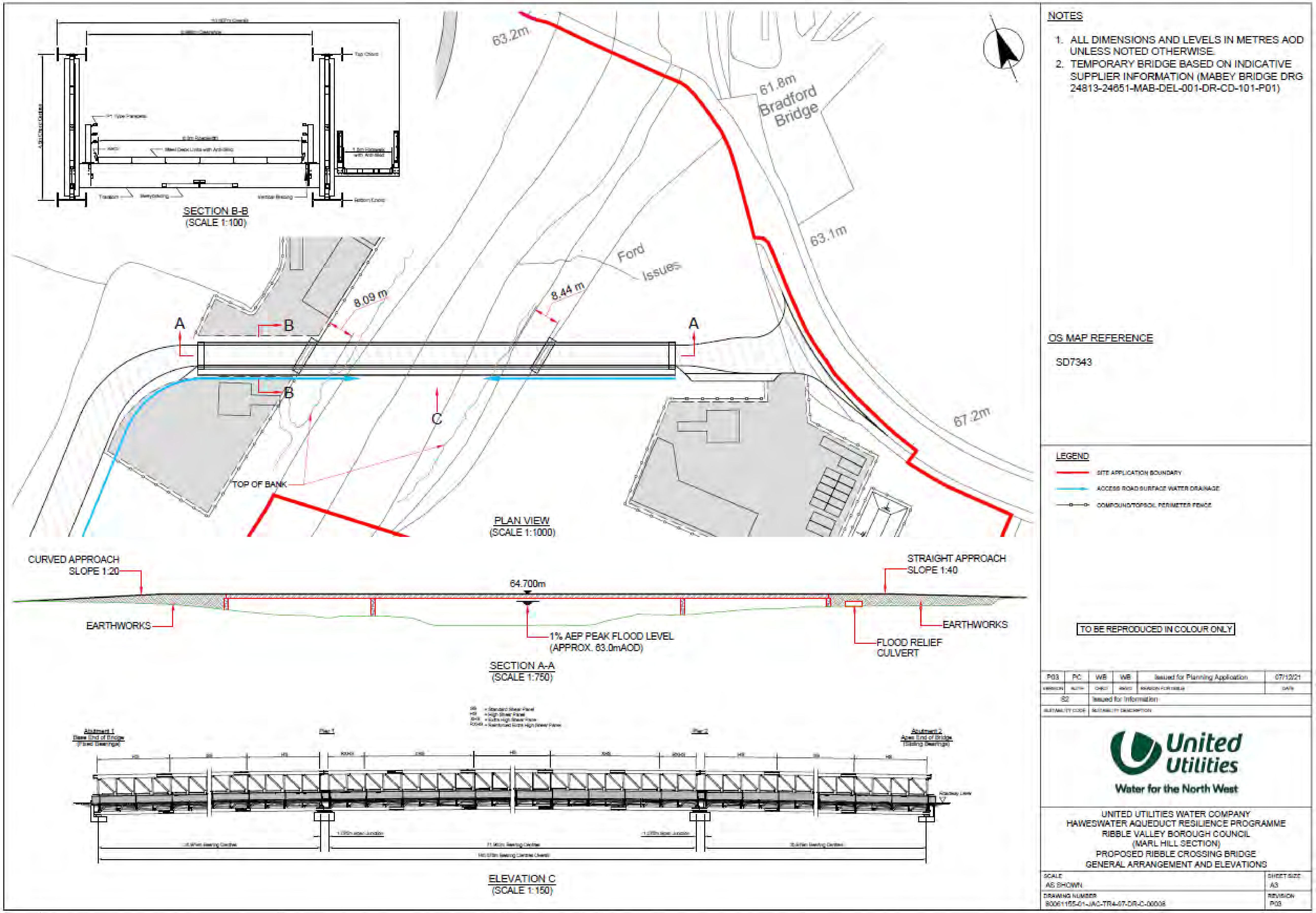

F.5 80061155-01-JAC-TR4-XX-DR-C-00029.pdf