E119

High Speed Two Phase 2a (West Midlands - Crewe)

Background Information and Data

CA2: Colwich to Yarlet

Hydraulic modelling report - Hopton (BID-WR-004-006)

July 2017

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High Speed Two (HS2) Limited,
Two Snowhill
Snow Hill Queensway
Birmingham B4 6GA

Telephone: 08081 434 434

General email enquiries: HS2enquiries@hs2.org.uk

Website: www.gov.uk/hs2

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1 Introduction

1.1 Background

1.1.1 This document presents the results of the hydraulic modelling carried out in the Colwich to Yarlet area (CA2) relevant to High Speed Rail (West Midlands - Crewe).

1.1.2 The Hydraulic modelling report – Great Haywood Viaduct (Background Information and Data 004: BID-WR-004-004) is also relevant to the Colwich to Yarlet area.

1.1.3 The water resources and flood risk assessment is detailed in the High Speed Rail (West Midlands - Crewe) Environmental Statement (ES)[1]. Volumes 2, 3 and 4 discuss water resource and flood risk effects and Volume 5, Appendices sets out the following relevant to the Colwich to Yarlet area:

- a route-wide Water Framework Directive compliance assessment (Volume 5: Appendix WR-001-000);
- a water resources assessment (Volume 5: WR-002-002);
- a flood risk assessment (Volume 5: WR-003-002); and
- a route-wide draft water resources and flood risk operation and maintenance plan (Volume 5: Appendix WR-005-000).

1.2 Aims

1.2.1 The Proposed Scheme includes a number of locations where the route will cross watercourses and their floodplains. The Proposed Scheme crossing locations have the potential to increase flood risk where they restrict flood flows or change floodplain dynamics.

1.2.2 At the locations discussed in this report, the route, as well as a new balancing pond access track, cross an unnamed watercourse at Hopton on a small embankment within the Hopton South cutting, approximately 500m downstream of the Hopton Pools, as shown in CT06 Map Book, drawing CT-06-216.

1.2.3 A hydraulic model in the location of the unnamed watercourse was created to simulate the risk of flooding for an approximate 1km stretch of the watercourse due to a breach of the Hopton Pools. This report documents the methods used and discusses the results, assumptions and limitations imposed by them.

1.2.4 Hydraulic models of the existing conditions and with the Proposed Scheme included have been evaluated to assess the impact of the Proposed Scheme on

[1] HS2 Ltd (2017), *High Speed Rail (West Midlands - Crewe) Environmental Statement (ES)*, www.gov.uk/hs2

flood risk and to derive peak flood water levels relative to the proposed structures.

1.2.5 This report details the existing hydrological and hydraulic processes of the reaches modelled and how these will be affected by the Proposed Scheme.

1.3 Objectives

1.3.1 The objectives were to:

- conduct, where feasible, a site visit to inform understanding of existing conditions, including existing channel and floodplain characteristics, hydraulic structures and flow paths;
- estimate breach hydrographs for the Hopton Pools;
- develop a hydraulic model, commensurate with the level of detail required and available at this stage, to provide peak levels at key structures for the Proposed Scheme based on the most suitable data available and flow hydrographs developed; and
- analyse the impact of the Proposed Scheme on flood risk levels obtained from the results of a breach analysis of the Hopton Pools.

1.4 Justification of approach

1.4.1 The breach analysis was carried out following the Reservoir Flood Maps: external guidance[2]. This assumes a simplified breach outfall hydrograph to reflect a credible worst case dam breach scenario.

1.4.2 The model has been constructed to provide an awareness of existing flood risk to inform the Proposed Scheme design. The detail included identifies potential impacts of the Proposed Scheme on surrounding land and ensures 1.0m freeboard to track level is provided in the breach scenario.

1.4.3 A 2D hydraulic model was selected for this study as detailed 1D channel information was not available at the time of study and the Light Detection and Ranging (LiDAR) survey adequately portrayed the existing channels and features. Using a 2D approach still allows for structures to be represented using the ESTRY solver within Two-dimensional Unsteady FLOW (TUFLOW).

1.5 Scope

1.5.1 The scope of the study was to undertake hydraulic modelling to enable an assessment to be made of the impact of the Proposed Scheme on the local environment. The models should be detailed enough to allow for future assessment of different options associated with each crossing location, to allow the management of flood risk and correct sizing of crossing openings.

[2] Environment Agency (2014), *Reservoir Flood Maps: external guidance*, Environment Agency 2014, LIT 6882

1.5.2 The report focuses upon:

- discussion of all relevant datasets, quality and gaps;
- hydrological analysis undertaken, approach used and calculation steps;
- integration of the hydrological analysis with the hydraulic modelling;
- hydraulic modelling methodology chosen, with clear identification of general methodologies and justification; and
- hydraulic modelling parameters, assumptions, limitations and uncertainty.

2 Site characteristics

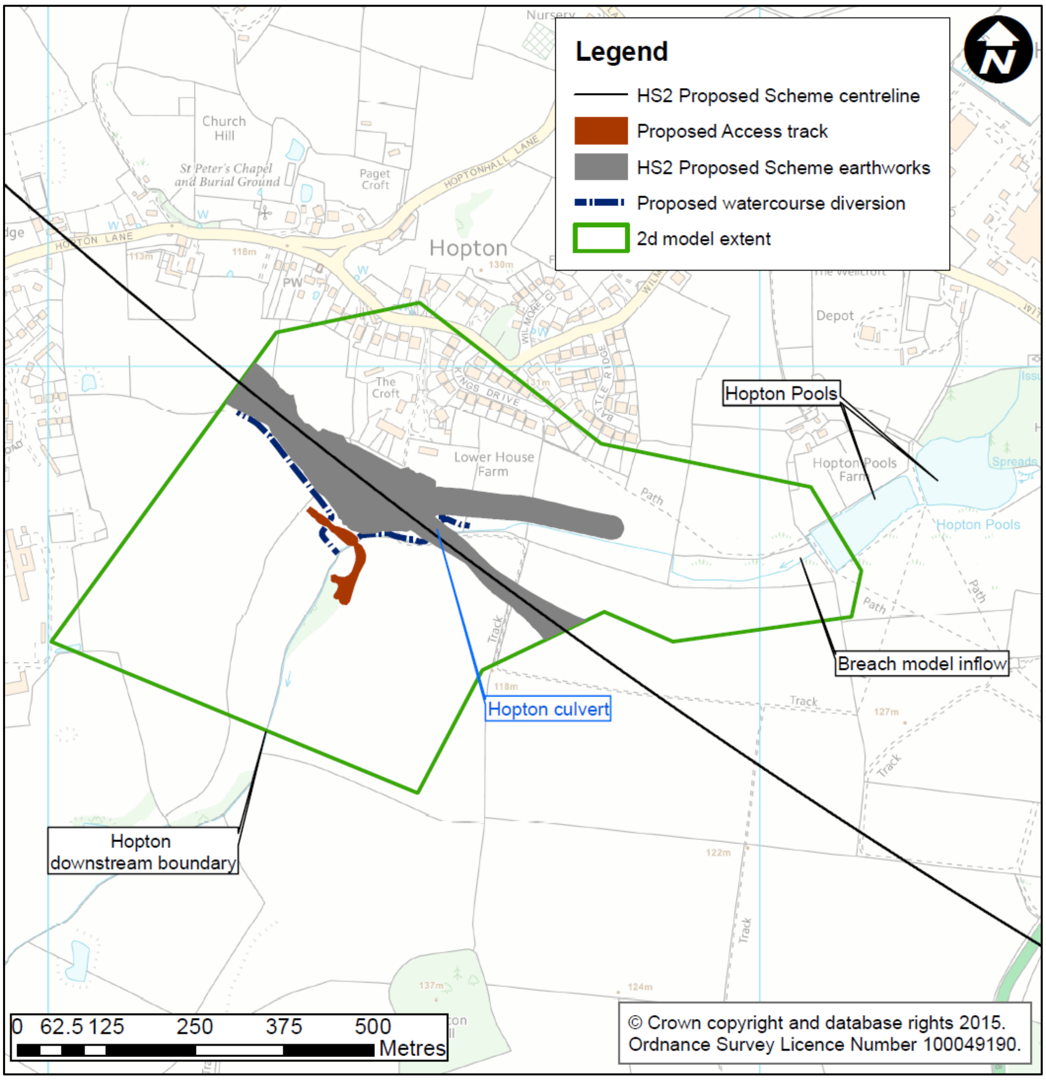
2.1 Description of the study area

2.1.1 The study area is situated to the south of the village of Hopton. Figure 1 shows the modelled extent, with the model upstream boundary located at the downstream face of Hopton Pools and the downstream boundary located approximately 600m south of Hopton.

2.1.2 The catchment is generally rural in nature, though it does include a part of the Hopton village and an industrial park. The watercourse flows in a westerly direction before turning south in the vicinity of the Proposed Scheme.

2.1.3 The Hopton Pools are located approximately 500m upstream (east) of the Proposed Scheme. The Hopton Pools consist of two online pools, with the upper pool receiving inflows from two watercourses, and the lower pool outlet feeding the watercourse downstream.

Figure 1: Schematic of key features within the study area



Hydrological description

2.1.4 The watercourses originate just upstream of the Hopton Pools in an area of woodland, to the north-east of the Proposed Scheme.

2.1.5 The catchment area contributing to the Hopton Pools is 2.2km^2, and is predominantly rural.

2.1.6 The unnamed watercourse at Hopton is ungauged along its entire length.

2.1.7 The method for defining the breach hydrograph hydrology for this model has followed the recommended approach as summarised within the following documents:

- Reservoir Flood Maps: external guidance[2]; and
- Guide to risk assessment for reservoir safety management[3].

2.1.8 This approach is designed to create a simplified breach outfall hydrograph to reflect a credible worst case breach scenario.

Railway alignment

2.1.9 The route of the Proposed Scheme crosses the study area in a south-east to north-westerly direction grading down to the north-west. The route is on an embankment across the valley, with the proposed Hopton North cutting and Hopton South cutting on either side. The unnamed watercourse passes under the Proposed Scheme through Hopton culvert.

2.1.10 In the valley, the Proposed Scheme is partially bounded by Hopton retaining wall on the upstream side, linking into a landscape bund extending out to the east. The screening bund will include a culvert to allow surface water flows from Hopton to enter the unnamed watercourse. On the downstream side, a balancing pond access track crosses the watercourse with a culvert crossing.

Flood mechanisms

2.1.11 Hopton is to the north of the Proposed Scheme. However, due to its elevation it is unlikely to be impacted by flooding of the watercourse. Lower House Farm on the southern edge of the village is a possible receptor and has been assessed for flood risk as a part of the flood modelling. Hopton Pools Farm to the north of Hopton Pools is unlikely to be impacted by the Proposed Scheme.

2.1.12 Downstream of the study area, the watercourse joins Kingston Brook to flow through Stafford. Any increase in flows due to the construction of the Proposed Scheme could impact on flood levels within Stafford. This is unlikely, with the culvert under the Proposed Scheme more likely to provide some flow attenuation and reduce the risk of flooding downstream in a breach scenario of the Hopton Pools.

[3] Environment Agency (2013), *Guide to risk assessment for reservoir safety management*, Volume 2, Environment Agency, SC090001/R2

2.1.13 Figure 2 shows a surface flow path from Wilmore Hill Lane, following Lower Lane and passing Lower House Farm to discharge into the watercourse.

2.2 Existing understanding of flood risk

Sources of information

2.2.1 Sources of Environment Agency data were assessed as below:

- Flood Map for Planning (Rivers and Sea)[4]; and
- updated Flood Map for Surface Water (uFMfSW)[5].

2.2.2 The Environment Agency Flood Map for Planning does not show any flood zones in the area of interest.

2.2.3 No reservoir breach mapping is available for the Hopton Pools.

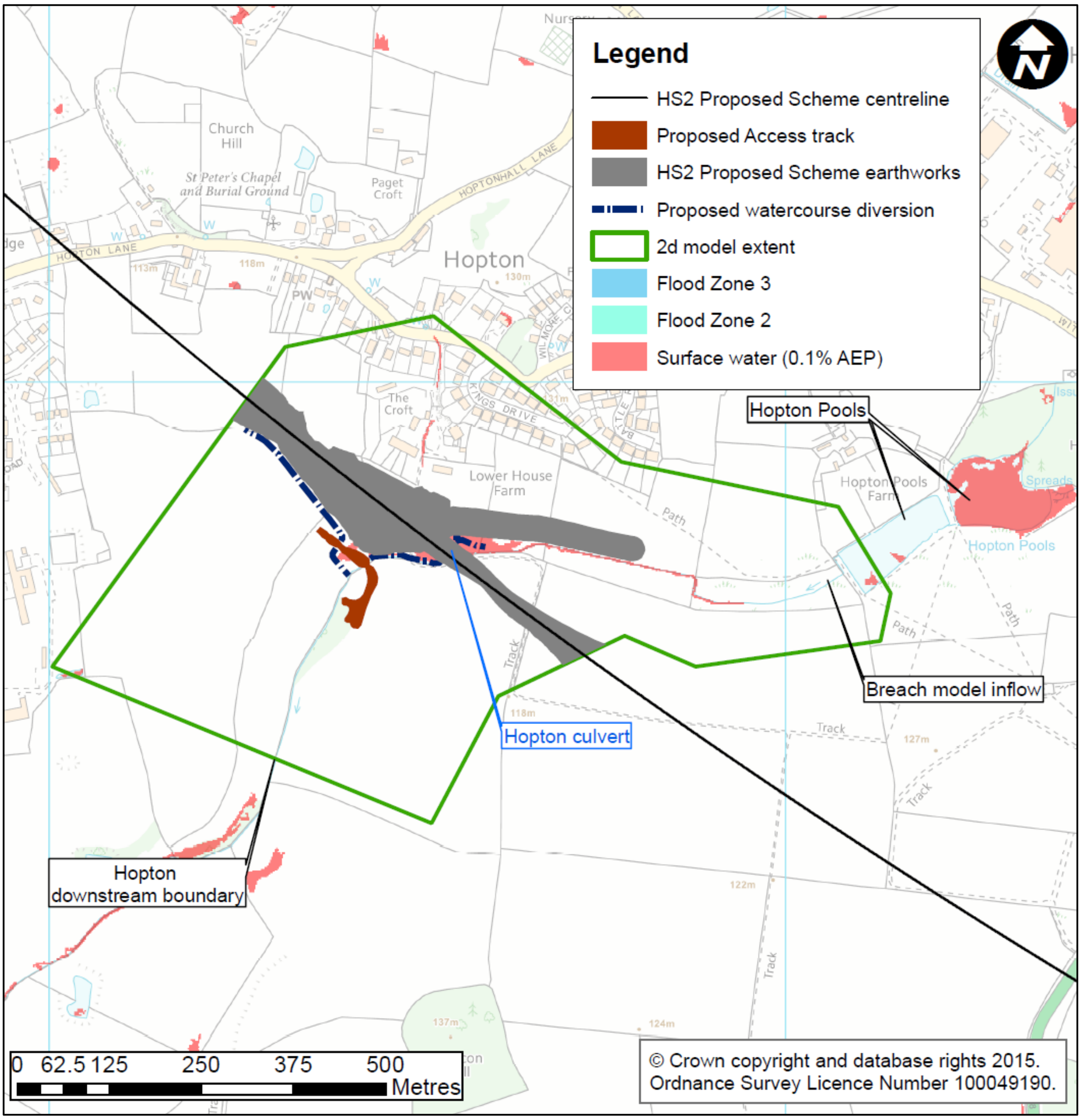
2.2.4 Figure 2 shows that the main flow path is confined largely to the channel of the unnamed watercourse, with a ponding location upstream of the farm track for the Lower House Farm (in the immediate vicinity of the Proposed Scheme). However, as the culvert under the existing farm access track is not included in the surface water mapping, this is not expected to be a realistic representation.

2.2.5 Available information does not indicate the presence of any flood defences within the area of interest.

2.2.6 The Hopton Pools generate a risk to the Proposed Scheme, due to the potential for an embankment breach.

[4] Gov.uk, *Flood map for planning*, https://flood-map-for-planning.service.gov.uk
[5] Gov.uk, *Long term flood risk information*, https://flood-warning-information.service.gov.uk/long-term-flood-risk/map?map=SurfaceWater

Figure 2: Environment Agency uFMfSW (0.1% Annual Exceedance Probability (AEP)) at Hopton



2.3 Availability of existing hydraulic models

2.3.1 There were no existing models for the unnamed watercourse at Hopton identified for this study.

2.4 Site visit

2.4.1 A site visit was undertaken in October 2016 to determine the dimensions of the watercourse, pools and any existing infrastructure.

2.4.2 Several structures were visited along the unnamed watercourse at Hopton and the pools however not all could be visited due to site access restrictions and general accessibility issues. For the structures that were visited, images were taken to ascertain dimensions and roughness.

2.4.3 Downstream of Hopton Pools, the watercourse is approximately 1.0-1.5m wide and 0.5m deep. Evidence of zero flow conditions was present.

2.4.4 Downstream of the point where the watercourse turns to the south, the channel becomes non-existent at times. Around this turn to the south (also the location of the Proposed Scheme), the left bank became 2m deep and incised. Larger trees were present in this location.

2.4.5 Where the access track for Lower House Farm crosses the watercourse a 0.6m diameter culvert was identified.

2.4.6 Photos from the site visit are presented in Figure 3 and show the lower Hopton Pool and the watercourse downstream.

Figure 3: Images of Hopton Pools and downstream watercourse

3 Model approach and justification

3.1 Model conceptualisation

3.1.1 Model extents were carefully selected to ensure that the model boundaries did not have any impact on the flood extent in the area of interest.

3.1.2 The breach hydrograph was applied immediately downstream of the Hopton Pools lower embankment. The upstream model extent was located on the upstream side of this embankment.

3.1.3 The downstream model extent was located approximately 400m downstream of the Proposed Scheme.

3.1.4 A 2D hydraulic model was adopted for the watercourse and floodplain due to the suitability of the catchment and for breach scenario stability, with culvert structures modelled in ESTRY. The watercourse downstream of the Hopton Pools is small and its limited capacity is not expected to significantly influence the floodplain extents during a breach scenario.

3.2 Software

3.2.1 TUFLOW (2016-AA) has been used. This is in line with standard practice for a 2D only model and uses the latest available TUFLOW build at the time modelling commenced.

3.3 Topographic survey

3.3.1 No additional topographic survey has been commissioned at this stage.

3.4 Input data

3.4.1 The elevation data for the study area was produced using 200mm LiDAR flown specifically for HS2 Ltd and covers 500m either side of the route centreline.

4 Technical method and implementation

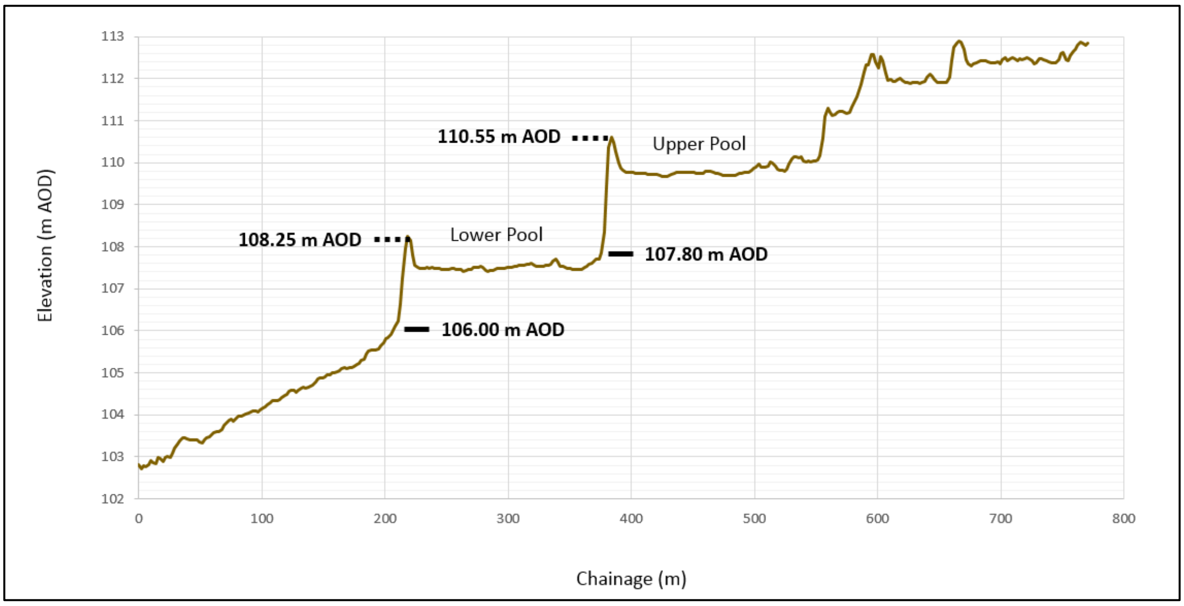
4.1 Hydrological assessment

Pool volume

4.1.1 For this assessment, the volume of water within the Hopton Pools has been estimated based on the LiDAR, assessing existing bank profiles, and making assumptions with regards to the depth of pools.

4.1.2 Figure 4 shows the topographical profile from the LIDAR survey information through the two Hopton Pools. The ground level at the toe of the downstream embankment is approximately 106.0 metres above Ordnance Datum (mAOD)and was adopted as the lower pool invert level. Assessing the profile further also resulted in an invert level of 107.8mAOD being assumed for the upstream pool. The crest levels were assessed across the width of each crest and levels were adopted for each pool as shown in Table 1.

Figure 4: Profile through Hopton Pools based on 200mm LiDAR



4.1.3 The surface areas of each pool were calculated based on Ordnance Survey (OS) mapping and aerial photography, with the surface area being multiplied by the depth to invert to conservatively estimate the volume for each pool. The volumes calculated are shown in Table 1.

4.1.4 As the pools are online (or impounding), the Reservoir Flood Maps: external guidance[2] recommends that the maximum water level is assumed to be super-elevated to 0.5m above the crest level, to account for inflows from the upstream catchment without modelling them explicitly. This approach is assumed to be conservative, due to the estimated upstream flows being relatively small. The impact this has on the impounded volume is shown in the last column of Table 1.

4.1.5 The total combined volume caused by a breach of both pools is 58,370m^3 (including 0.5m super-elevation above the crest level).

Table 1: Adopted pool levels and volumes

Pool	Crest level (mAOD)	Adopted invert level (mAOD)	Pool depth (m)	Surface area (m^2)	Volume (m^3) (area x depth) (rounded to nearest 5 m^3)	Volume (m^3) (including 0.5 m super-elevation above crest level) (rounded to nearest 5 m^3)
Upper Pool	110.55	107.80	2.75	10,950	30,115	35,590
Lower Pool	108.25	106.00	2.25	8,285	18,640	22,785
SUM	-	-	-	-	-	58,375

Breach hydrograph methodology

4.1.6 A simplified dam breach hydrograph methodology, as outlined in the Reservoir Flood Maps: external guidance[2], has been used for this assessment. This assumes a simplified breach outfall hydrograph to reflect a credible worst-case dam breach scenario, as outlined in Appendix A.

4.1.7 As the Hopton Pools are in close proximity (the upper pool would breach directly into the lower pool) the possible breach scenarios are combined. The guidance notes that it is common for a failure of a downstream dam to be triggered if the dam crest is overtopped as a result of an upstream dam failure. This is likely to happen in quick succession for Hopton Pools.

4.1.8 The levels and volumes presented in Table 1 were used to calculate the breach hydrograph from the Hopton Pools for three scenarios, which are outlined as follows:

- estimated breach hydrograph;
- cumulative breach hydrograph; and
- worst case volume breach hydrograph.

Estimated breach hydrograph

4.1.9 To simplify the approach for this study, one breach outflow hydrograph for the combined volume of the pools has been modelled using the levels of the lower pool. This assumes the lower pool breaches as a result of inflows from the breaching of the upper pool, with the combined volume of both pools behind the breach. The resultant breach hydrograph is presented in Table 2 and Figure 5.

Cumulative breach hydrograph

4.1.10 In order to test the peak outflow conditions and the assumptions made in regards to the estimated breach hydrograph, the breach hydrographs for each pond were calculated independently then added together such that the peak flows coincided. This makes no allowance for the attenuation of peak flow from the upper pool spilling into the lower pool but provides a peak flow scenario. The resultant breach hydrograph is presented in Table 2 and Figure 5.

Worst case volume breach hydrograph

4.1.11 Due to the uncertainty surrounding the depth assumptions, a worst case volume scenario was also derived and modelled, assuming the invert levels of both pools were at the same level (106.0mAOD). This increased the depth of the upper pool to 4.55m + 0.5m super-elevation above the crest, with a resultant combined volume of 78,080m^3. The calculation was carried out as for the estimated breach scenario, but with the increased volume. The resultant breach hydrograph is presented in in Table 2 and Figure 5.

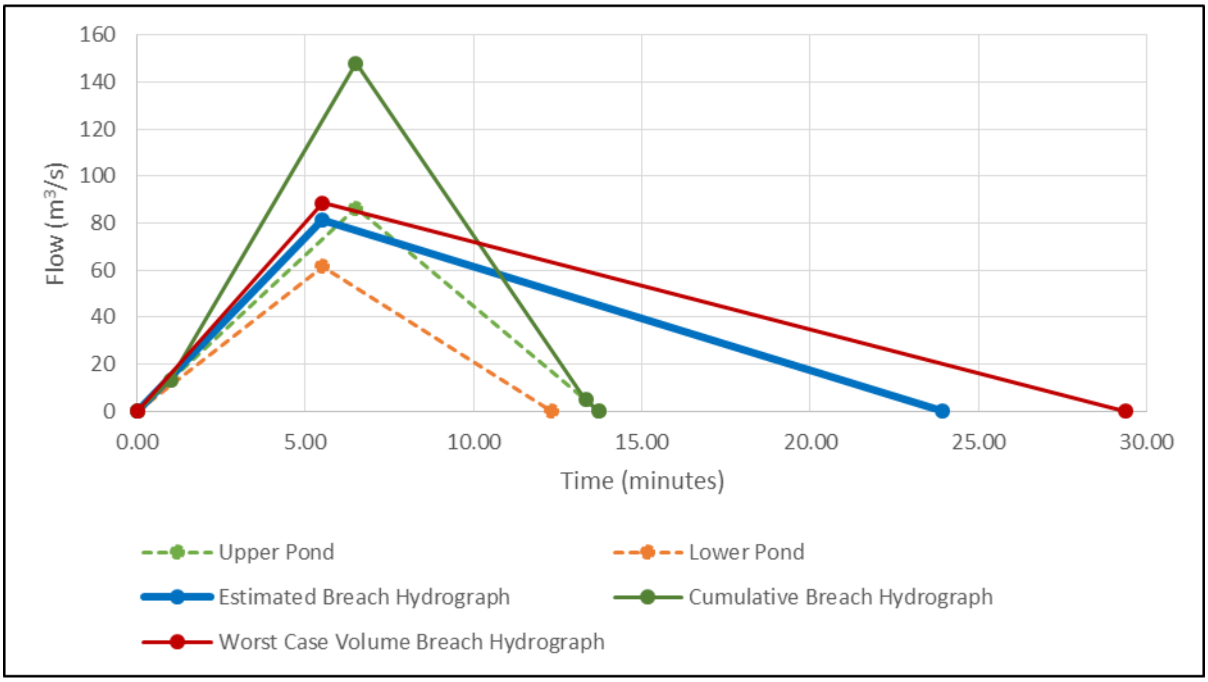
4.1.12 Further information on the results of the cumulative and worst case volume breach hydrographs can be found in the sensitivity analysis in Section 6.5.

4.1.13 The breach hydrographs were applied as inflows to the hydraulic model at the toe of the Hopton Pools downstream embankment.

Table 2: Breach hydrograph scenario details

Hydrology scenario	**Q_p, peak flow (m^3/s)**	**T_p, time to peak (minutes)**	**T_e, time to empty (minutes)**	**Volume (m^3) (including 0.5 m super-elevation above crest)**
Estimated breach hydrograph	81.3	5.5	23.9	58,370
Cumulative breach hydrograph	148.0	6.5	13.7	58,370
Worst case volume breach hydrograph	88.6	5.5	29.4	78,080

Figure 5: Breach hydrographs for scenarios proposed



4.2 Hydraulic model build – baseline model

1D representation

4.2.1 There is one structure to be represented in 1D within the baseline model, a culvert under an existing farm track. This was represented in ESTRY.

2D representation

4.2.2 The cell size of the model was set as 2m. Cell size for the 2D model grid was optimised to ensure appropriate representation of the flow pathways whilst maintaining reasonable run times.

Inflow boundaries

4.2.3 The breach hydrograph was applied immediately downstream of the Hopton Pools lower embankment. A region inflow was required to distribute flows for stability due to the magnitude of the breach hydrograph.

Downstream boundary

4.2.4 A normal depth boundary was used at the downstream extent of the unnamed watercourse, and also in the floodplain at the downstream extent. This generates a stage-discharge curve based on the bed slope which varies across the floodplain.

4.2.5 A normal depth slope of 0.0079 m/m (1 in 127) was used within the channel and floodplain. This was derived from LiDAR.

Key structures

4.2.6 There are a number of structures within the model extent that were modelled in a variety of ways. Additionally, there are a number of structures which are not modelled as no information is available. Those included in the model and deemed to be key hydraulic controls are detailed in Table 3 and shown in Figure 6.

Table 3: Key structures present within the modelled extent at Hopton

Structure reference	Structure description	Modelling representation and justification
C1	Small circular culvert. 8.91m (L) x 0.6m (D)	1D culvert structure. Dimensions taken from site visit

Roughness

4.2.7 Roughness values utilised are in line with the recommended values stated within Chow, 1959[6].

4.2.8 The Reservoir Flood Maps: external guidance[2] recommends the use of a high Manning's 'n' roughness value for the downstream channel and floodplain, with a recommended value of 0.10 globally. This high value is recommended to allow for an estimation of the effects of sediment entrainment (dam embankment and scour), buildings and other obstructions that are not detailed specifically, and has been adopted globally within this model.

4.3 Hydraulic model build – Proposed Scheme

4.3.1 The Proposed Scheme model has been edited from the baseline to include the following:

Viaduct piers

4.3.2 There are no piers present in the Proposed Scheme at Hopton.

Topographic changes

4.3.3 The Proposed Scheme earthworks, balancing ponds and access tracks have been included using the relevant heights for embankment crests and access track alignments incorporated. These features of the Proposed Scheme are based on the design as shown in Map CT-06-216 in the Volume 2 Map Book.

Replacement floodplain storage areas

4.3.4 There are no replacement floodplain storages areas within the Proposed Design at Hopton.

[6] Chow, V.T (1959), *Open-channel hydraulics*, McGraw-Hill, New York

Key Structures

4.3.5 Four structures were included in the Proposed Scheme model, as outlined in Table 4 and presented in Figure 6. However, none of these are deemed to be key hydraulic controls.

Table 4: Proposed Scheme structures

Structure reference	Structure description	Modelling representation and justification
P1	Watercourse culvert under the Proposed Scheme. Three 4.2 x 1.35m box culverts.	1D culvert structure.
P2	Culvert through Hopton screening bund. 1.35 x 1.35m box culvert.	1D culvert structure – to provide connectivity for flows from the Lower Lane catchment area.
P3	Watercourse culvert under balancing pond access track. 1.35 x 1.35m box culvert.	1D culvert structure.
P4	Perimeter drain culvert under balancing pond access track. 0.45m diameter circular culvert.	1D culvert structure.

Channel realignment or diversions

4.3.6 Upstream of the Proposed Scheme, a channel realignment is required to facilitate a perpendicular culvert crossing under the Proposed Scheme.

4.3.7 Downstream of the Proposed Scheme, a channel realignment is required to facilitate a perpendicular culvert crossing of the Proposed Scheme and to suit the environmental earthworks extents.

4.3.8 No diversions of the river channel have been proposed.

Production of flood extents

4.3.9 Flood extents have been derived using the direct output options now available in TUFLOW to produce ASCII output for the maximum depth and height. This has then been converted into a polygon, and cleaned to remove all bow ties (where two polygons overlap) as well as any dry islands less than $48m^2$.

Modelling assumptions made

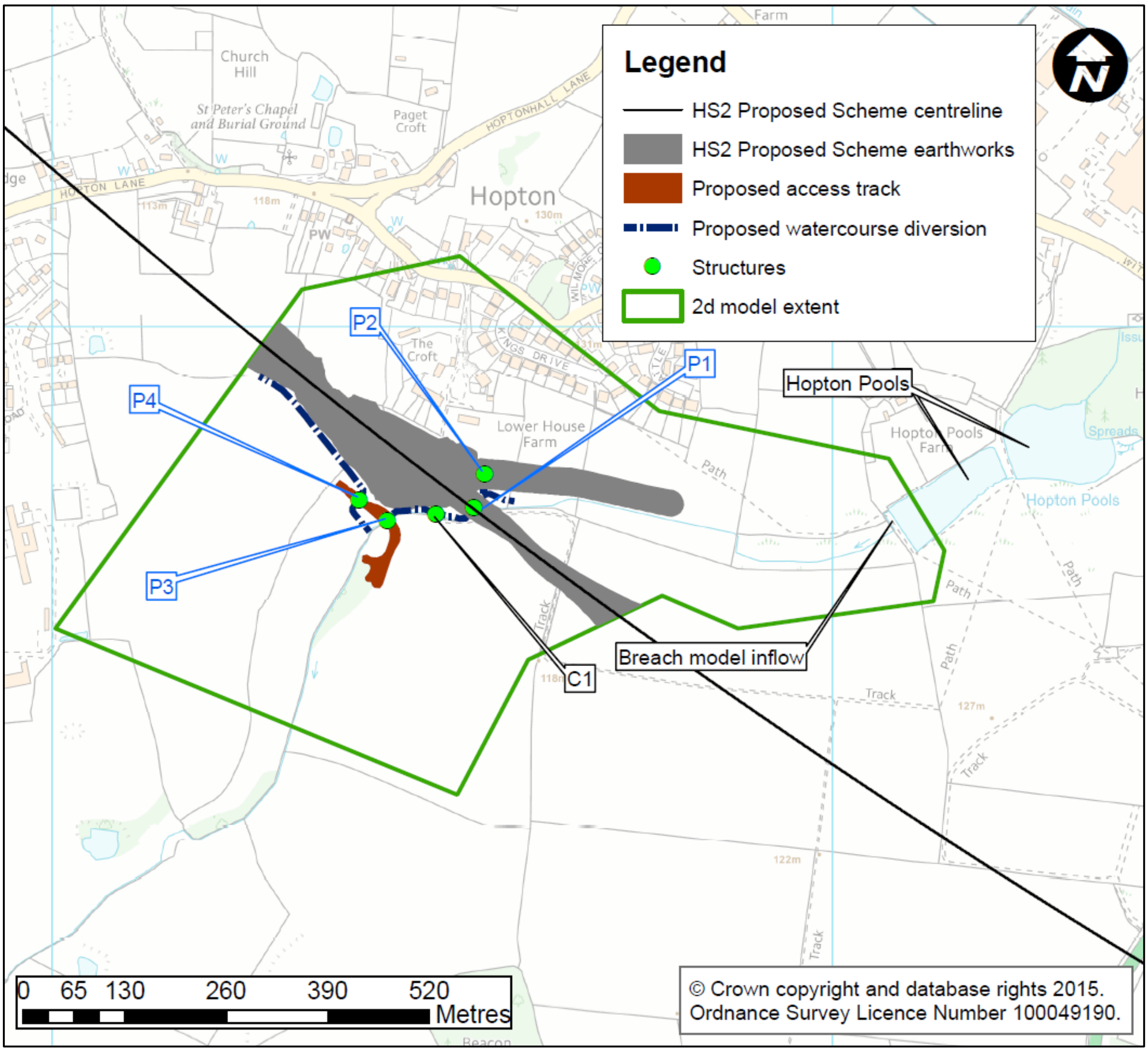
4.3.10 Existing LiDAR is assumed to be correct as no other information is available.

4.3.11 Culvert sizes have been assumed in a number of places within the model. Where a site visit to provide photos or measurements was not possible, they have been approximated based on LiDAR information. This provided road levels and ground levels, and the measured width of the top of structures from aerial photography.

4.4 Climate Change

4.4.1 The inclusion of climate change factors is not applicable to a breach assessment.

Figure 6: Plan of hydraulic model structures



5 Model results

5.1.1 The following scenarios have been modelled:

- baseline: modelling of a breach scenario for the existing site conditions. This provides a baseline breach floodplain against which to assess the impacts of the Proposed Scheme; and
- proposed: incorporating the Proposed Scheme into the baseline model to assess the impact on a breach scenario. Key features of the Proposed Scheme that will influence the flood risk have been incorporated and include earthworks, retaining walls and culverts is built into the model. The adopted design consists of three 4.2m wide by 1.35m high box culverts, including an allowance for 50% blockage due to the expected nature of the breach flows.

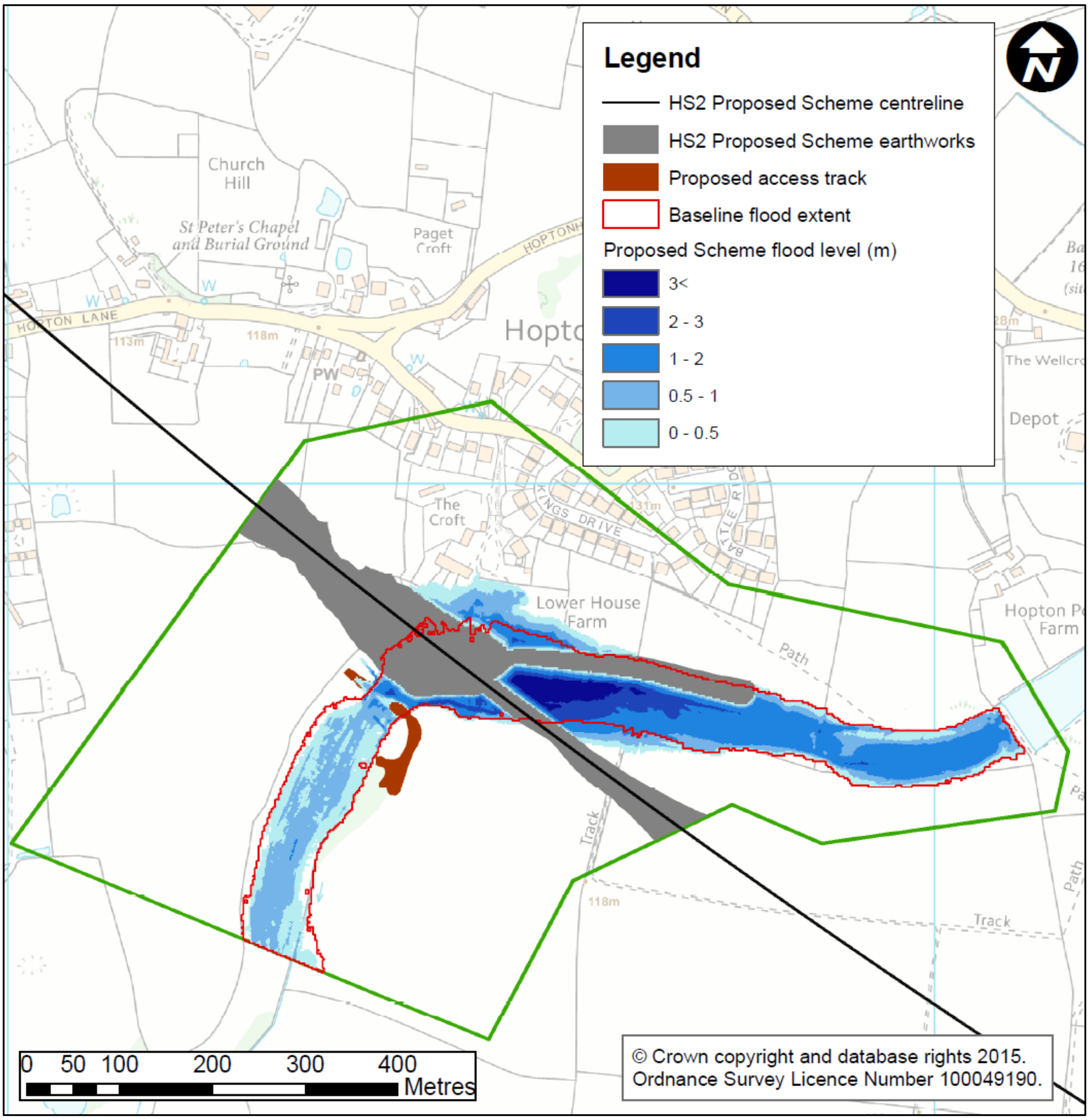
5.1.2 Figure 7 compares the floodplain extents for the baseline (red outline) and the Proposed Scheme (blue floodplain). Refer to Appendix B for the breach assessment flood level impact map.

5.1.3 Under ordinary design flow conditions (i.e. 1.0%+climate change and 0.1% AEP), the design flows in the watercourse are small and the minimum size culvert for the Proposed Scheme will have adequate capacity. However, the modelling demonstrated that during a breach scenario, a greater culvert capacity is required to provide the 1m freeboard requirement to the top of track level and to have no impact on the flood receptors at Lower House Farm.

5.1.4 The Proposed Scheme increases flood levels by approximately 2.1m immediately upstream of the Proposed Scheme during a breach scenario. Impacts return to existing breach conditions after approximately 400m upstream of the Proposed Scheme.

5.1.5 The proposed top of track level for the Proposed Scheme is 104.05mAOD at the crossing location, while the breach analysis based on the estimated breach hydrograph scenario estimates a peak flood level of 103.02mAOD, satisfying the 1m freeboard requirement. The design includes a 50% blockage allowance for the main watercourse crossing under the Proposed Scheme, due to the expected nature of the breach flows.

Figure 7: Baseline and Proposed Scheme flood extent comparison



5.1.6 The culvert through the screening bund, in normal flood conditions, provides an outlet to flows from Hopton and Proposed Scheme perimeter drainage. During the modelled breach simulations, water (up to 1.8 m^3/s) flows back through this culvert, resulting in ponding of water on the upstream side.

5.1.7 The flood extent on the upstream side of the screening bund, as shown in Figure 7, extends significantly further than the baseline floodplain. No impacts on residential dwellings at Lower House Farm have been identified.

5.1.8 For the watercourse between the Proposed Scheme and the access track for the downstream balancing pond, the flood level is elevated when compared to the baseline results by a maximum of 0.5m. This is due to the confined nature of the proposed channel. Immediately downstream of the access track, the flood levels reduce, showing an improvement compared to the baseline due to the attenuating effect of the Proposed Scheme during a breach scenario. No attenuation is expected during normal design flow conditions.

6 Model proving

6.1 Introduction

6.1.1 This section of the report presents the analysis of the model undertaken to ensure confidence in the stability of the model build, its response to input values and consistency with previous modelling.

6.2 Run performance

6.2.1 Model output has been assessed across all model structures to assess model stability and overall model performance.

6.2.2 Final cumulative mass balance error is within +/-1.0% for all baseline, mitigation, blockage and sensitivity cases simulated.

6.3 Calibration and validation

6.3.1 There is no gauge situated within an appropriate distance of this location to provide calibration or validation data.

6.3.2 There is no additional anecdotal evidence available for any effective model verification exercise.

6.4 Verification

6.4.1 There are no breach assessment maps available for comparison for this location.

6.5 Sensitivity analysis

6.5.1 The following sensitivity scenarios were undertaken:

- increase in roughness (Manning's n) by 20%; and
- type of breach inflow hydrograph peak and volume.

Roughness

6.5.2 A high global Manning's n roughness of 0.10 was adopted for the modelling, based on the Reservoir Flood Maps: external guidance[2] for breach analysis scenarios. The roughness for the 1D culverts and 2D domain was increased by 20% to assess the sensitivity to the roughness values adopted.

6.5.3 The result of increasing the roughness by 20% was an increase in peak flood level of between 50mm and 100mm immediately upstream of the Hopton culvert and minor increases in the flood extent. This demonstrates that the breach analysis is not sensitive to the adopted roughness values and changes in the adopted roughness will only result in minor changes to the peak flood level.

Breach inflow hydrograph

6.5.4 As discussed in Section 4.1, three different breach inflow hydrographs were generated to assess the impact of the breach hydrograph assumptions. These hydrographs are outlined as follows:

- estimated breach hydrograph: assumed the lower pool breaches as the result of inflows from the breaching of the upper pool, with the combined volume of both pools behind the breach. This hydrograph has been adopted for the inflow to the hydraulic model;
- cumulative breach hydrograph: assumed the breach hydrographs for each pool are calculated independently, then added together resulting in a higher peak inflow; and
- worst case volume breach hydrograph: calculated as for the estimated breach hydrograph, but assuming the upper pool has the same invert level as the lower pool, resulting in a 33% increase in total breach volume.

6.5.5 A comparison of the flood levels generated by the estimated, cumulative and worst case volume breach hydrographs is presented in Table 5.

Table 5: Flood levels for the varying breach scenarios immediately upstream of the Proposed Scheme

Hydrology scenario	Baseline flood level	Proposed Scheme flood level
Estimated breach hydrograph	100.92 mAOD	103.02 mAOD
Cumulative breach hydrograph	+0.31 m	+0.20 m
Worst case volume breach hydrograph	+0.10 m	+0.49 m

6.5.6 During the baseline modelling where there is no restriction to the free flow of flood waters downstream, the cumulative breach hydrograph (peak flow scenario) generates the highest flood level.

6.5.7 For the Proposed Scheme where the flood waters pass through a culvert under the Proposed Scheme, the worst-case volume breach hydrograph generates the highest flood level with an increase of close to 0.5m at the Proposed Scheme for a 33% increase in pool volume.

Summary

6.5.8 The sensitivity analysis shows the model is not sensitive to changes in roughness values at the Hopton culvert.

6.5.9 The breach inflow hydrograph demonstrates that the breach analysis is moderately sensitive to the volume assumptions for the Hopton Pools.

6.6 Blockage analysis

6.6.1 Blockage was considered as a part of the breach analysis for the Proposed Scheme, due to the sediment entrainment and gross pollutants that can be expected in the breach flows. The design for the Proposed Scheme includes a 50% blockage factor.

6.7 Run parameters

6.7.1 There is no deviation from default run parameters for the model runs.

6.7.2 The time step parameters used were 0.5 seconds for the 1D structures and 1 second for the 2D model. This is the suggested approach for a grid size of 2m.

7 Limitations

7.1.1 No volume information or bathymetric survey was available for the Hopton Pools, with their volumes being estimated as described in Section 4.1. The pool volumes have been conservatively estimated and the results are suitable for the purpose of reducing the risk to the Proposed Scheme. There is an opportunity to obtain bathymetric survey, increasing the confidence of the breach analysis.

7.1.2 No survey data was available for the watercourse and the model has been developed based on the LiDAR provided. Culvert dimensions have been estimated based upon ground levels and watercourse size, which may impact flood extent and level predictions if these were to change.

7.1.3 Calibration has not been able to be carried out due to a lack of available data.

7.1.4 Model resolution may limit the representation of flow paths along the Proposed Scheme. Consequently, consideration may need to be given to the proximity of the flood levels to the track corridor level. There may be an opportunity to design the retaining wall for the mitigation earthworks to tie into the track formation, preventing the possibility of breach flows entering the track corridor and interacting with the track drainage.

8 Conclusions and recommendations

8.1.1 The aim of developing a hydraulic model of the unnamed watercourse at Hopton to simulate the baseline and Proposed Scheme scenarios, to determine the peak water levels and flows throughout the catchment in a breach scenario has been met.

8.1.2 Increases in water level are observed due to the Proposed Scheme during a breach scenario of Hopton Pools. This breach analysis has demonstrated that culverts can be designed under the Proposed Scheme, providing 1m freeboard to the top of track level and providing protection to the key flood receptors.

8.1.3 It is recommended that bathymetric survey of the Hopton Pools be undertaken, providing the opportunity to increase the confidence of the breach analysis and refine the culvert design under the Proposed Scheme for the unnamed watercourse.

9 References

Chow, V.T (1959), *Open-channel hydraulics*, McGraw-Hill, New York.

Environment Agency (2013), *Guide to risk assessment for reservoir safety management, Volume 2*, Environment Agency, SC090001/R2.

Environment Agency (2014), *Reservoir Flood Maps: external guidance*, Environment Agency, LIT 6882.

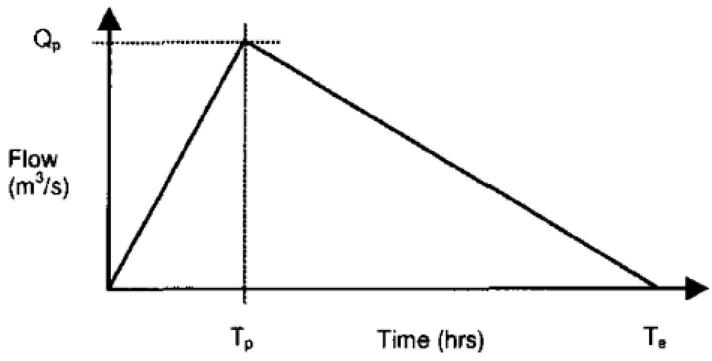
Gov.uk, *Flood map for planning*. Available online at: https://flood-map-for-planning.service.gov.uk.

Gov.uk, *Long term flood risk information.* Available online at: https://flood-warning-information.service.gov.uk/long-term-flood-risk/map?map=SurfaceWater.

HS2 Ltd (2017), *High Speed Rail (West Midlands- Crewe) Environmental Statement (ES)*. www.gov.uk/hs2.

Appendix A: Hydrology

1.1.1 A simplified dam breach hydrograph methodology, as outlined in the Reservoir Flood Maps: external guidance[2], has been used for this assessment. This assumes a simplified breach outfall hydrograph to reflect a credible worst case dam breach scenario, as follows:



Where:

Q_p is the peak discharge, calculated for an earth filled embankment as:

$$Q_p = FOS\,[0.607\,(V_w^{0.295} \times H_w^{1.24})]$$

Where:

- FOS = factor of safety (equal to 1.5);
- V_w = reservoir volume at top water level (m^3); and
- H_w = Maximum dam height (m). This is calculated as 0.5m above the crest level for impounding dams, with the subsequent overtopping leading to a breach over the entire height of the dam.

T_p is the time to peak discharge (in seconds), calculated for an earth filled embankment as 120 times the maximum dam height (m).

T_e is the time to end of discharge, so that the volume under the hydrograph matches the dam volume, calculated as:

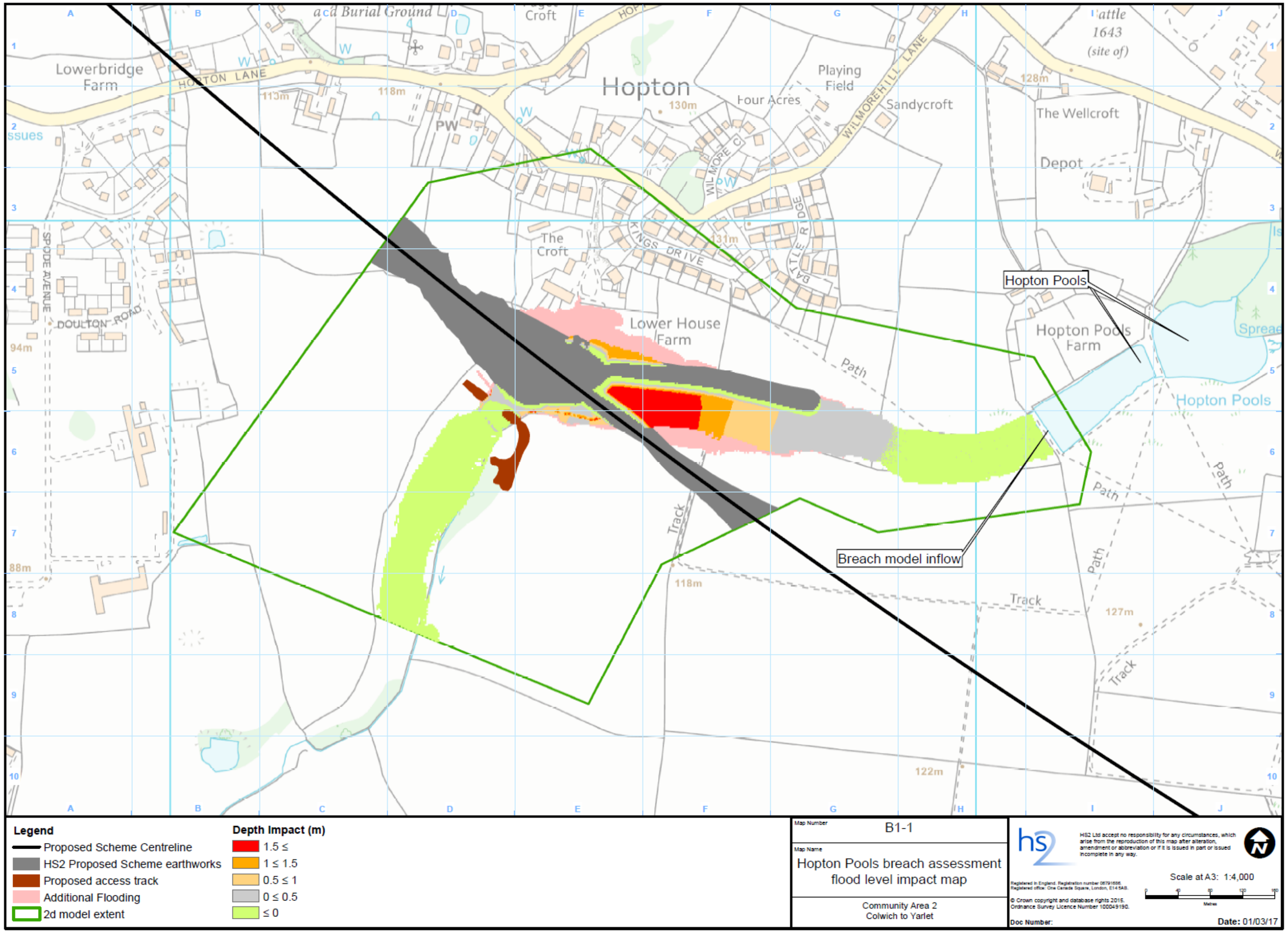
$$V_w = 0.5\,(Q_p \times T_p) + 0.5\,[Q_p\,(T_e - T_p)]$$

1.1.2 In some conditions, where the storage volume is relatively small but the height of the embankment is relatively high, the calculation of the breach hydrograph will not be possible, due to T_e being less than T_p. The Guide to risk assessment for reservoir safety management, Vol. 2[3] states that when the value to T_e is less than 2 x T_p, then keeping Q_p constant, T_p should be reduced, keeping T_e = 2 x T_p, until the breach hydrograph volume is matched. It is noted that T_p should not be reduced to less than 40 x H_w

Appendix B: Flood level impact map

1.1.1 A breach assessment flood level impact map can be seen in this section as described in Section 5, see Figure B-1.

Figure B-1: Hopton Pools breach assessment impact map



High Speed Two (HS2) Limited
Two Snowhill
Snow Hill Queensway
Birmingham B4 6GA

08081 434 434
HS2Enquiries@hs2.org.uk

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