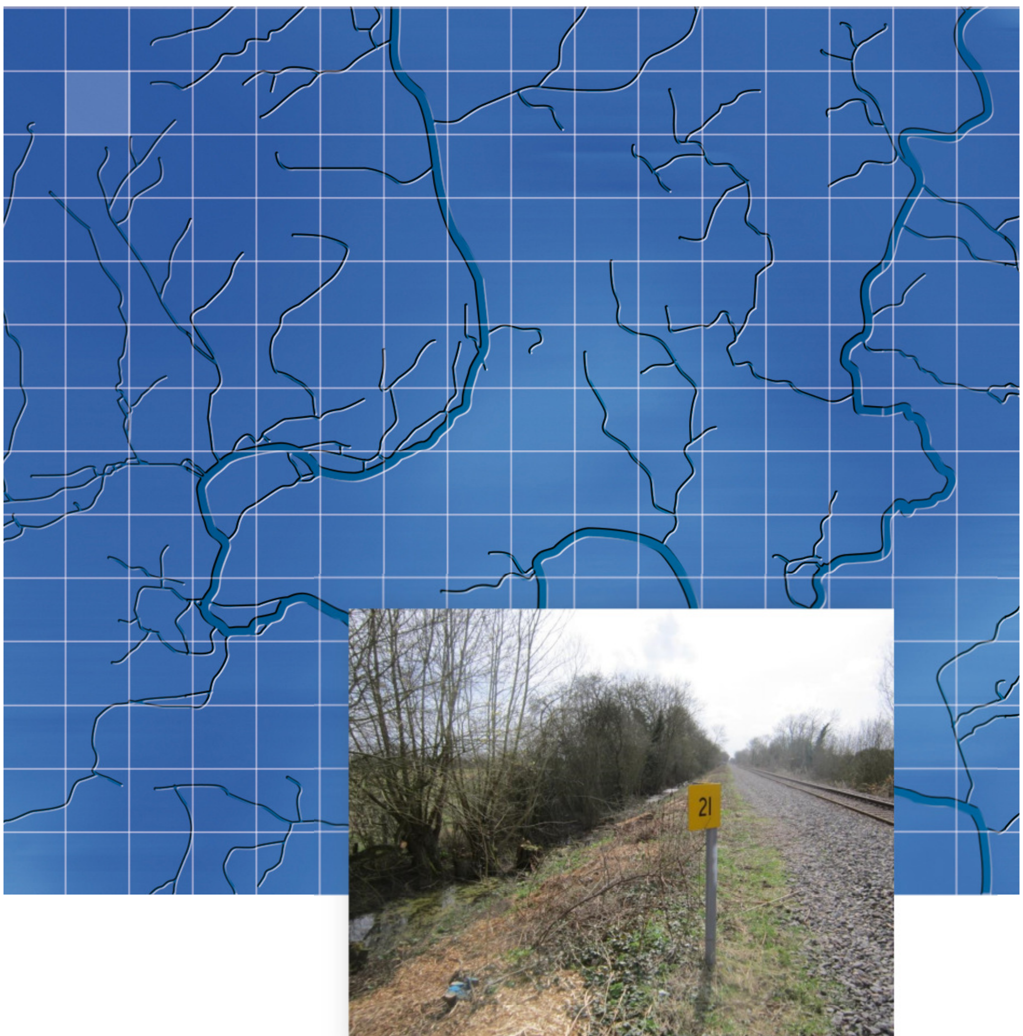
Network Rail and Chiltern Railways

December 2015

EWR P1 – SW Drainage Assessment (AP2 & AP3)



Wallingford HydroSolutions Limited

Network Rail and Chiltern Railways

EWR P1 – SW Drainage Assessment (AP2 & AP3)

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WHS1160

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Date **21 December 2015**

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Contents

1 Purpose

This document constitutes a surface water drainage assessment (SWDA), as required by Condition 13 of the Order under the Transport and Works Act 1992 (TWA) obtained by Chiltern Railways for the construction of the East West Rail Phase 1 (EWR P1) project between Bicester and Oxford. This document also provides the information required by the National Planning Policy Framework (NPPF) in considering the surface water drainage aspects of a Flood Risk Assessment for new development.

This surface water drainage assessment considers the requirements for the development of the following elements of the railway scheme:

- AP2 – Tubbs Lane Footbridge – Cherwell District Council Planning Ref:13/00235/DISC
- AP3 – Bicester Town Station – Cherwell District Council Planning Ref:13/00281/DISC

Figure 1 shows the locations of these Assessment Points in relation to the overall railway development.

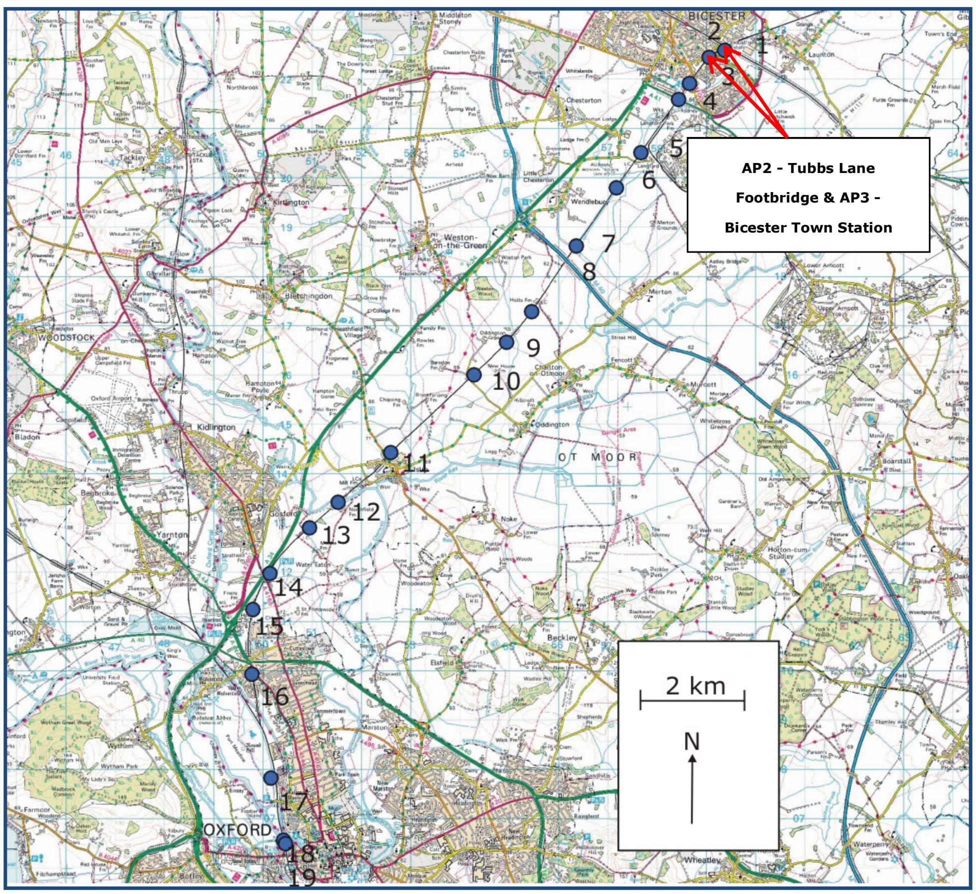


Figure 1 - Overview of the scheme with Assessment Points shown.

Condition 13 of the TWA Order requires that:

'No construction of any one of the following elements of development shall commence until a surface water drainage assessment and scheme for that element (as identified in the Level 2 Flood Risk Assessment Revised, July 2010 (Inquiry document CD/2.22), unless stated otherwise here) has been submitted to and approved in writing by the local planning authority, in consultation with the Environment Agency:

- *AP1 Bicester Chord.*
- *AP2 Tubbs Lane footbridge.*
- *AP3 Bicester Town station.*
- *AP4 A41 overbridge.*
- *AP6 Elm Tree Farm/Langford Lane Overbridge (modified to accord with the revised proposal shown on Revised Sheets 8b, 35 and 37 of the Deposited Plans and Sections (Inquiry Document CD/1.28).*
- *AP7 Merton footbridge.*
- *AP8 Holts Farm overbridge.*
- *AP9 Oddington Footbridge No 5.*
- *AP10 Oddington overbridge.*
- *AP11 Islip station in Phase 1.*
- *AP11 Islip station in Phase 2.*
- *AP13 Water Eaton No5 overbridge.*
- *AP14 Water Eaton Parkway.*
- *AP15 Gosford and Water Eaton Footbridge No 10.*
- *AP17 Banbury Road Sidings*
- *AP18 Sheepwash Bridge.*
- *AP19 Oxford station.*

The surface water drainage assessments shall follow the methodology set out in the Scope of Surface Water Drainage Assessment, July 2010, agreed by the Environment Agency. Each surface water drainage assessment shall demonstrate that surface water discharge rates and volumes from that element of the development will not increase flood risk, or taken together with other relevant works in the same catchment, can be maintained at or below the agreed limits, using sustainable drainage techniques. Development shall be in accordance with the approved surface water drainage assessment and scheme.'

Therefore the purpose of this document is to obtain approval of the local planning authority, in consultation with the Environment Agency (EA), for the surface water drainage assessment for AP2 Tubbs Lane Footbridge & AP3 Bicester Town Station, thus discharging the requirements of Condition 13 of the TWA Order and meeting the surface water drainage requirements of NPPF.

2 Proposed Development

2.1 Overview

EWR P1 is a major package of infrastructure investments including: the doubling of the line between Bicester town and Oxford North Junction; a new independent line being built between Oxford North Junction and Oxford station, using a disused track bed parallel to the existing railway; the existing stations at Bicester Town and Islip will be rebuilt, and a new station built at Water Eaton Parkway; and at Oxford the disused parcels platforms at the north end of the station will be removed and replaced for passenger use for Chiltern Railways services. The following sections describe the proposed works at both AP2 & AP3 in more detail.

2.2 AP2 – Tubbs Lane Footbridge

An existing pedestrian level crossing at Tubbs Lane, Bicester is to be replaced with a new footbridge crossing the upgraded railway (please see Figure 2). There is an existing impermeable level crossing structure in place at the proposed development site.

This structure is to be steel on earth embankments no more than 4m high. Adjacent to the railway, the embankment will be supported via a retaining wall structure.

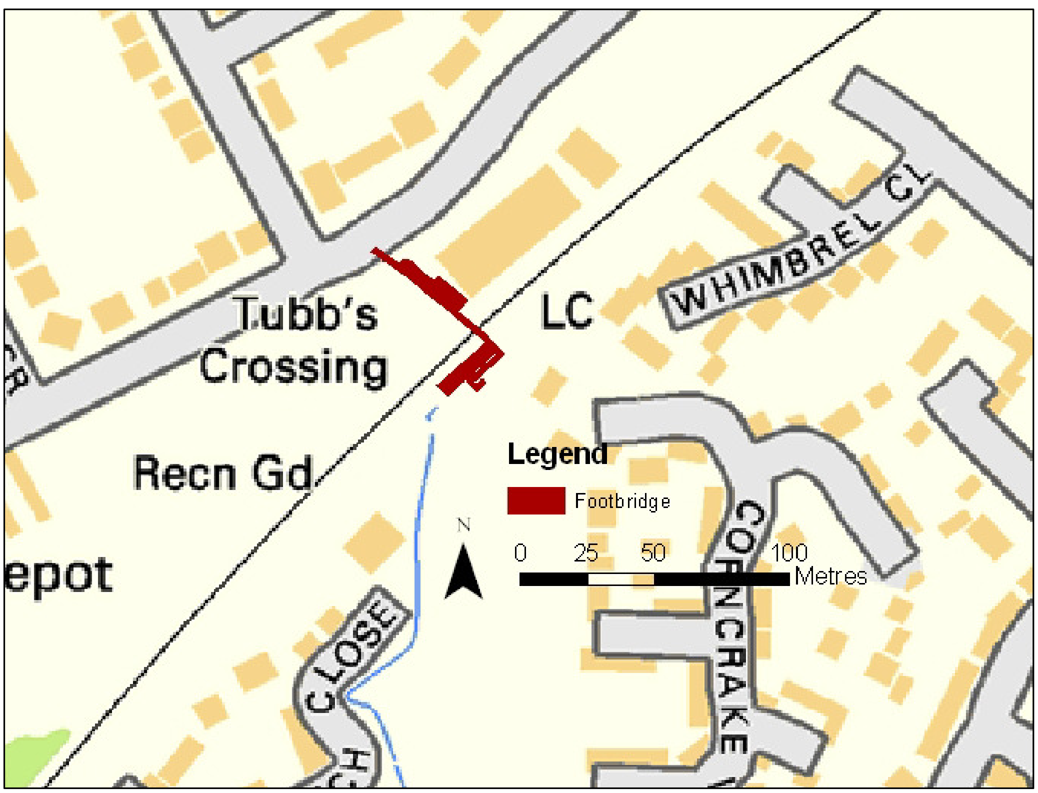
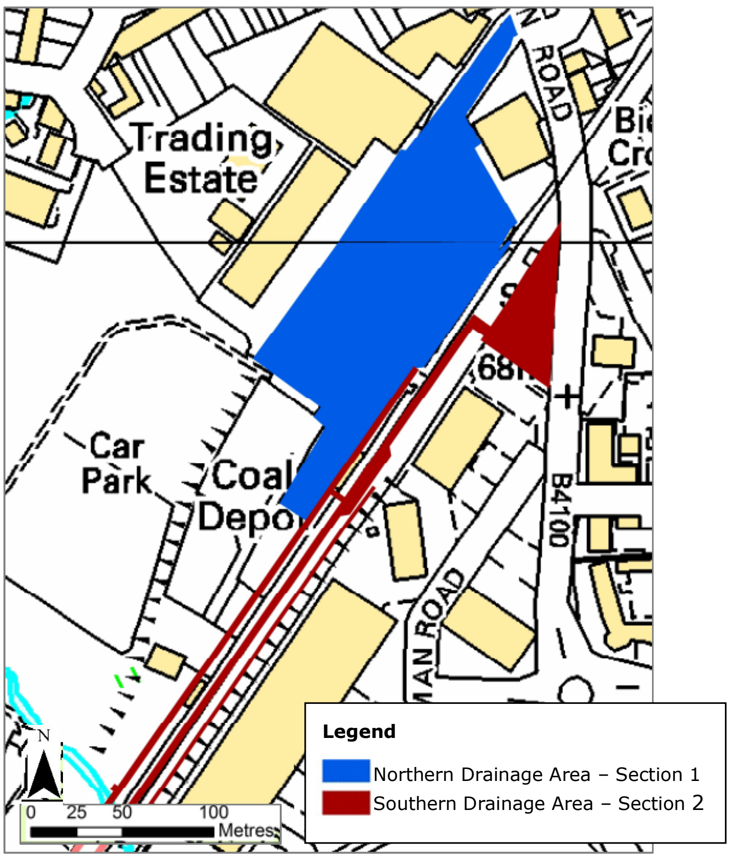


Figure 2 – AP2: Tubbs Lane Footbridge. (Contains Ordnance Survey Data © Crown copyright and database right 2013)

2.3 AP3 – Bicester Town Station

Bicester Town station will be rebuilt with two platforms, to allow reinstatement of a double track (please see Figure 3). The existing station is to be demolished. An existing industrial area is to be replaced with car parking suitable for the new station. A new drop-off area is to be located on a section of the site that is currently grassed, which is the only new area of hardstanding. The new platforms will accommodate 8-car trains and new station buildings will be constructed. Level access will be provided throughout, together with improved links to both the town centre and the adjacent Bicester Village Outlet Centre. A footbridge is also being built across the railway tracks at this point. It should be noted that there is an additional 50m of 'passive provision' for platform extension shown on the Atkins plans[1] which has been considered in the current drainage designs to ensure that the drainage infrastructure can accommodate additional runoff from any future platform expansion.



[1] Atkins Design Drawings. Bicester Town Station Proposed Drainage General Arrangement (Dwg Ref – 5114534-ATK-DRG-DR-023310)

Figure 3 – AP3: Bicester Town Station. (Contains Ordnance Survey Data © Crown copyright and database right 2013)

3 Management of Surface Water Runoff

3.1 Planning Requirements

It is a recognised development requirement that post-development the stormwater runoff rates discharged from any new development should not be greater than flows currently generated from the site, whether this be at greenfield or existing brownfield run-off rates. Exceptions generally only apply where it is not practical to achieve this due to the size of the hydraulic control unit. In this situation overcompensation at neighbouring sites will be provided to ensure that over the whole scheme surface water runoff is reduced. These commitments are in line with guidance set out in the NPPF and through discussions with the EA. The following sections describe the calculation procedure followed to obtain these rates.

3.2 Runoff Assessments

Pre-development (i.e greenfield or brownfield) peak surface water runoff rates have been calculated for the 1 in 1 year and the 1 in 100 year storm events for the development sites at AP2 & AP3. Appendix 1 outlines the methodology used in the estimation of the peak surface water runoff rates. It should be noted that following discussions with the EA this analysis has not considered runoff from embankments, as these are permeable hence can be assumed to generate runoff at the greenfield rate. The following sections present the data used and the results of the surface water runoff calculations.

3.2.1 Area Assessment

Table 1~~Table 1~~ details the areas of permeable and impermeable surfaces at each assessment point pre- and post-development.

Table 1 - Surface types and areas at assessment points.

Assessment Point	Type	Existing Brownfield		Post Development		Increase in Impermeable area (ha)
		Impermeable extent (ha)	Permeable extent (ha)	Impermeable extent (ha)	Permeable extent (ha)	
AP2 Tubbs Lane Footbridge	**Greenfield**	0	0	0.05	0	0.05
AP3 Bicester Town Station	**Brownfield**	1.06	0.37	1.44	0	0.37

3.2.2 Surface Water Runoff Rates

Both greenfield and brownfield runoff rates were calculated as described in Appendix 1. Brownfield runoff rates have been based on a total area of 1.44ha which comprises of 1.06ha of existing impermeable surfacing and the remaining 0.37ha is greenfield land on which the proposed platforms will be and small carpark will be constructed. Runoff rates for both AP2 Tubbs Lane Footbridge & AP3 Bicester Town Station are presented in Table 2 and Table 3 below.

Table 2 – Surface Water Runoff Rates (AP2 Tubbs Lane Footbridge).

Return Period	Greenfield Runoff Rate (l/s)
1:1	0.10
1:100	0.38

Table 3 – Surface Water Runoff Rates (AP3 Bicester Town Station).

Return Period	Greenfield Runoff Rate (l/s)	Brownfield Runoff Rate (l/s)
1:1	3.08	9.96
1:100	11.55	38.10

4 Design Statements & Commitments

4.1 Target Discharge Rates

As it is not practicable to provide attenuation at AP2 Tubbs Lane footbridge, to account for this the combined pre-development runoff rate for both AP2 & AP3 has been reduced to reflect the post-development runoff rate for AP2. For example, for the 1 in 100 year plus climate change event the difference between post-development and greenfield flow of 1.91l/s for Tubbs Lane has been subtracted from the brownfield flow of 38.10l/s for Bicester Town station resulting in a reduced target discharge rate of 36.19l/s for Bicester Town station. This results in the reduced target discharge rates presented in Table 4~~Table 3~~ below for AP3 Bicester Town station.

Table 4~~3~~ – AP3 Bicester Town Station Target Discharge Rates (l/s).

Return Period	Limiting Discharge rate (l/S)
1:1	9.96*
1:100	36.19*

**an allowance for the Tubbs Lane Footbridge has been included.*

4.2 AP2 – Tubbs Lane Footbridge

Drainage of Tubbs Lane footbridge will be achieved through a combination of a formal piped drainage system and filter drains that will ultimately discharge directly into the adjacent drainage ditch (Please see Appendix 2 for details). The drainage system is split into two systems that discharge to the ditch via outfall headwalls labelled as HW23-18 and HW24-11 on Drawing No 5114534-ATK-SKT-DR-0003 in Appendix 2.

It should be noted that this formal drainage system drains directly into the watercourse with no formal attenuation of flows, although there will be some nominal attenuation provided by the filter drains. As the impermeable area of this footbridge is small the resulting greenfield limiting runoff rates are extremely small at around 0.4l/s for the 1 in 100 year event. It is considered impracticable to provide attenuation at this site as such a small flow control device would easily block and result in maintenance issues. Therefore, the increase in runoff caused by this footbridge has been incorporated into the design of Bicester Town Station.

4.3 AP3 - Bicester Town Station Drainage Strategy

Jacobs has prepared the surface water drainage design for Bicester Town Station which is provided in Appendix 3. The surface water drainage design at Bicester Town Station is split into 2 sections; drainage to the north of the railway line and drainage to the south of the railway line.

- **Section 1** – This comprises the largest section of ~~D~~drainage ~~from~~ serving the large northern car park and other impermeable areas located to the north of the railway line. This section is drained by a series of lined drainage channels and conventional piped drainage network that discharge to an underground attenuation structure situated under the car park via a bypass oil separator to treat the runoff from the car parking area. The system discharges to a Thames Water sewer via an outfall located at manhole labelled as CD2 on Drawing No SKM-DRG-BTS-UN60312-4003 provided in Appendix 3, additionally labelled as "location of discharge into public sewer". The drainage design and discharge rates have been technically approved by Geoff Nokes of Thames Water.

- Th~~is~~e attenuation structure provides 315m^3 of storage volume to cater for the 1 in 30 year rainfall event. This tank discharges into the existing public sewer that crosses the site with a hydro-brake that limits the peak discharge rate to 28.4l/s. The system is designed to ensure no flooding up to and including 1 in 30 year +30% climate change event, however there is some contained flooding in some 100 year storm events. The critical storm event is the 1 in 100 year, 15min winter +30% climate change avent and this leads to a total flood volume of 2.544m^3 at several gulley positions connected to drainage runs between SW4 and SW5. It has been calculated that this small flooded volume can be contained within the kerb lines at the low points located in the area of flooding. The total volume of above ground storage against the 100mm high kerbs is 10.393m^3. It is noted that situated within the flood area are gulleys connected to pipes down steam of SW5 that are only surcharged, therefore ponding water will drain into those gullies during and immediately after the storm event, hence the flooded volume will actually be less than shown by the modelling results. ~~For all larger rainfall events flooding of strategic areas of the car park is proposed to ensure storage within the site for events up to and including the 1 in 100 year (plus an allowance for climate change) rainfall event. This tank discharges into an existing public sewer that crosses the site with a hydro-brake that limits the peak discharge rate to 28.4l/s.~~

- **Section 2 –** This smaller section of drainage is concerned with the drainage from the disabled car park/taxi drop off area and other impermeable areas located to the south of the railway line. Jacobs' drainage design solution for this section is to collect runoff though a combination of linear drainage channels and gullies into a piped drainage network under the roadway. The attenuation structure for this section provides 98.8m^3 of storage volume, of which 73m^3 being provided in the geocellular system under the car park and the rest in oversized (600mm diameter) pipes. A hydro-brake located at SW37 limits the peak discharge rate to 5.0l/s. This then discharges to an existing headwall connecting into an existing 375mm diameter pipe that subsequently discharges to the Bicester Brook. This smaller system is designed to ensure no above ground flooding up to the 1 in 100 year plus 30% allowance for climate change event.

The total storage volume being provided is 413.8m^3 and the total peak discharge rate is limited to 33.4l/s, below the required maximum discharge rate of 36.2l/s.

~~Jacobs has ensured that the drainage systems at Bicester Town Station have been designed to the following criteria;~~

- ~~Drainage systems are designed not to surcharge in events up to and including the 1 in 1 year rainfall event~~
- ~~There is no flooding of the site for events up to and including the 1 in 30 year rainfall event.~~
- ~~Flooding is controlled on site up to the 1 in 100 year (plus a 30% increase in rainfall to account for climate change) rainfall event.~~

5 Conclusion

The proposed drainage arrangements at AP3 Bicester Town Station have been designed to achieve betterment over the target discharge rates presented in Table 4~~Table 3~~. The design will ensure a degree of over attenuation is provided to account for the runoff generated by AP2 Tubbs Lane footbridge. This will ensure that post-development runoff rates from both AP2 & AP3 are reduced to pre-development equivalents. We consider that the information provided in this surface water drainage assessment is sufficient to comply with Condition 13 of the TWA Order and the surface water drainage requirements of NPPF.

Appendix 1 – Surface Runoff Calculations Methodology

1.1 Introduction

Guidance issued by DEFRA[2] states that post development the stormwater runoff discharges from urban developments should approximate to the site greenfield response over an extended range of storm frequencies of occurrence (return periods). However, it is accepted that drainage proposals may be measured against the existing drainage performance of the site (brownfield). In addition the peak rate of runoff into a watercourse should be no greater than the undeveloped rate of runoff, although similarly exceptions apply where it is not practical to achieve this. The guidance outlines methodologies for estimating storage volumes for stormwater control for development sites and also provides methodologies for the estimation of peak rates of runoff from greenfield sites.

For clarification, the greenfield rate refers to the volumes and peak flows associated with an undeveloped site whilst brownfield relates to a site which has been previously developed hence a proportion of the site is impermeable.

As part of the East West Rail Phase 1 development surface water runoff volumes for greenfield and brownfield conditions are required. In addition, peak runoff rates are also required for greenfield and brownfield conditions. Section 1.2 outlines the methodology for the estimation of the surface water runoff volumes whilst Section 1.3 outlines the methodology for estimating the peak runoff rates. Note that there is no guidance on estimating brownfield peak runoff rates, and the guidance states that greenfield runoff rates should be considered as indicative only due to the limitations of the methodologies.

1.2 Surface Water Runoff Methodology

The DEFRA guidance recommends the use of Institute of Hydrology Report 124 (IH124)[3] for estimating surface water runoff. However, recent research into flood design for small catchments[4] suggests that the FEH statistical method[5] and the Revitalised Flood Hydrograph (ReFH)[6] event-based method both outperform the older methods. The report states that these are applicable across the range of catchment sizes used in their development and that the continued recommendation of outdated methods such as IH124 and ADAS 345 is inappropriate. The research notes that there is little evidence to suggest that the accuracy of the FEH methods when applied to ungauged catchments is particularly scale dependent and recommends the use of current versions of the FEH statistical approach or the ReFH rainfall-runoff model except on highly permeable (BFIHOST > 0.65) or urbanised catchments (URBEXT2000>0.15) where the results of the ReFH model can be less reliable. The research recommends that for catchments smaller than 0.5 km^2 and plot scale, which is relevant for the development sites within the East West Rail Phase 1 development, runoff estimates should be derived from FEH methods applied to the nearest suitable catchment above 0.5 km^2 for which descriptors can be derived from the FEH CD-ROM and scaled down by the ratio of catchment areas.

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[2] Kellagher R, 2012, Preliminary rainfall runoff management for developments, DEFRA R&D Technical Report W5-074/A/TR/1 Revision E

[3] Marshall D, C, W. Bayliss, A, C,. Flood Estimation for small catchments. Institute of Hydrology Report 124.

[4] Environment Agency, 2012, Estimating flood peaks and hydrographs for small catchments: Phase 1, SC090031

[5] Robson, A.J. and Reed, D.W. (1999) Statistical procedures for flood frequency estimation. Volume 3 of the Flood Estimation Handbook. Centre for Ecology & Ecology.

[6] NERC (CEH). 2005. Revitalised FSR/FEH rainfall runoff method. Spreadsheet application version 1.4.http://www.ceh.ac.uk/feh2/SpreadsheetimplementationofReFH.html

Following the guidance, and taking into account this research, greenfield runoff hydrographs were calculated using 6.25 hour duration design rainfall events for the required return period event using a conjunction of the IH124 and ReFH rainfall runoff method.

IH124

Greenfield peak runoff rates have been calculated using the small catchment statistical method, IH124 methodology, in conjunction with the growth curves factors specified within the NERC Flood Studies Supplementary Reports 2[7] and 14[8]

A catchment area of 50 ha was assumed for each site with the results expressed as runoff rates per unit area to facilitate scaling to the development area. A key catchment descriptor within the method is the soil class(es) as defined by the Winter Rainfall Acceptance Potential (WRAP) map[9]. This is an extremely coarse map which is mapped at a scale of 1:625,000 and as such does not contain sufficient information for determining local soil and underlying substrate permeability. At design level the selection of appropriate soil class values would be informed by local soil maps coupled within infiltration tests. For the purposes of defining runoff rates for this assessment the soil permeability classes and substrate classes within the Hydrology of Soil Types (HOST) classification[10] were used to guide soil class selection. The HOST classification has replaced the WRAP map in all current flood estimation procedures.

ReFH

Given that there is no available flood event data on which to calibrate the ReFH model, the catchment descriptors for each site were obtained from the FEH CD ROM v3. The nearest 1km cell to each site was used to obtain the rainfall parameters required for the rainfall Depth Duration Frequency (DFF) ReFH model. Where this is not possible catchment scale parameters were obtained for the nearest small river reach.

The ReFH model was run using the 6.25 hour event for the 1 in 1 year, 1 in 30 year and 1in 100 year events. Allowances for climate change were made for the 1:100 year event by increasing the rainfall intensity by 30%. Note that current DEFRA[11] guidance advises increasing rainfall intensities by 20% for 2080 and beyond, so the adopted values are conservative. A catchment area of 50 hectares was assumed and results are then scaled to the site level.

Development of final runoff rates

The ReFH and IH124 methodologies produce independent runoff rates for the given return periods. Current research into small catchments[12] indicates that more recent methodologies are generally more reliable that the older (IH124) methodologies. The differences between the peak runoff rates were resolved by adjusting the BFIHOST or WRAP classes. For most of the sites the peak runoff from IH124 was rescaled to be similar to ReFH. Since ReFH is not considered as reliable in high

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[7] Faulkner, D.S. 1999. Rainfall Frequency Estimation. Flood Estimation Handbook Vol. 2, Institute of Hydrology, Wallingford, UK.

[8] Institute of Hydrology,1983 Review of regional growth curves. Flood Studies Supplementary Report 14. Institute of Hydrology, Wallingford, UK

[9] Natural Environment Research Council, 1975. Flood Studies Report.

[10] Boorman, D. B., Hollis, J. M. and Lilly, A., Hydrology of soil types: a hydrologically-based classification of the soils of the United Kingdom. Institute of Hydrology Report 126.

[11] Kellagher R, 2012, Preliminary rainfall runoff management for developments, DEFRA R&D Technical Report W5-074/A/TR/1 Revision E

[12] Environment Agency, 2012, Estimating flood peaks and hydrographs for small catchments: Phase 1, SC090031.

permeability catchments (taken to be where the BFIHOST is greater than 0.6) in highly permeable catchments the IH124 estimates for peak runoff were given a greater weighting.

Calculation of current brownfield and potential post development runoff volumes

The assessment of current brownfield and potential post-development runoff volumes for each return period is conducted:

- by assuming a runoff coefficient of unity for impermeable areas;
- calculating a gross direct runoff volume by taking the product of the areal extent of the impermeable area and the corresponding rainfall event profile;
- calculating the equivalent greenfield runoff profile for the impermeable area by taking the product of the greenfield runoff hydrograph (expressed in units of runoff per unit area) and the impermeable areas, and estimating the net runoff volume for the impermeable area.

This nett runoff volume represents the runoff volume that has to be captured, and preferably infiltrated to maintain runoff at the greenfield rate. For the 1:100 year event the runoff calculations have included an overall increase in event rainfall depth of 30% for the impermeable runoff estimate to allow for climate change.

Surfaces assumed to be impermeable in this outline design level assessment include roofs, car parks, pavements, roads, bridge structures and platforms. As such this represents a worst case scenario as it ignores the detailed design potential for at-source mitigation.

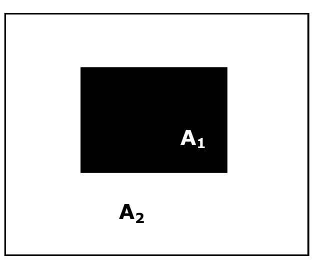
1.3 Brownfield Peak Runoff

The greenfield peak runoff can be obtained from the IH124 and ReFH methodologies. However, DEFRA[13] do not provide guidance on producing peak runoff for brownfield sites. Whilst ideally runoff volumes and peak runoff should be returned to the greenfield level, it is accepted that this is not always possible. In these circumstances maintaining the current runoff or peak flows is acceptable hence brownfield peak runoff values are required.

It is widely accepted that increasing the impermeable extents within a catchment, or development site in this case, increases runoff volume and decrease the response time within the catchment[14].

The following methodology has been developed to calculate the Brownfield peak flow:

1) Consider a site to contain an impermeable surface of area A_1 (m^2) and permeable surface of area A_2 (m^2), as per diagram below



[13] Kellagher R, 2012, Preliminary rainfall runoff management for developments, DEFRA R&D Technical Report W5-074/A/TR/1 Revision E

[14] Chow V. T., Maidment D. R. and Mays L. W., 1988, Applied Hydrology, McGraw-Hill, New York, USA.

2) Calculations within ReFH assume that A_1 and A_2 are both greenfield hence we already have the design rainfall *P* (mm) and the greenfield runoff *Q* (mm) for the design hydrograph.

3) For a completely impermeable surface, A_1 and A_2 are impermeable, the following is proposed:

$$Q = 0.7 \times P + 0.3 \times Q$$

It is assumed that 70% of the rainfall becomes direct runoff. The value of 70% is used as this is generally recommended for use within the UK[15,16]. A proportion of the rainfall is also delayed through the system and this is reflected by adding 30% of the greenfield runoff.

The result is a hydrograph which has a faster time to peak, higher peak and greater total runoff than the greenfield hydrograph.

4) For a mixed impermeable/greenfield site these two components are combined according to the proportion of each within the development site.

$$Q = \left[\frac{A_2}{A_1 + A_2} \times Q\right] + \left[\frac{A_1}{A_1 + A_2} \times 0.7 \times P\right] + \left[\frac{A_1}{A_1 + A_2} \times 0.3 \times Q\right]$$

5) The peak flows can then be extracted from the hydrographs and rescaled to cumecs.

An example is presented within Figure 4~~Figure 5~~.

[15] Institute of Hydrology, 1999, Flood Estimation Handbook, Vols 1 – 5.
[16] Department of Environment/National Water Council, 1981, Design and analysis of Urban Storm Drainage:the Wallingford Procedure, National Water Council, UK.

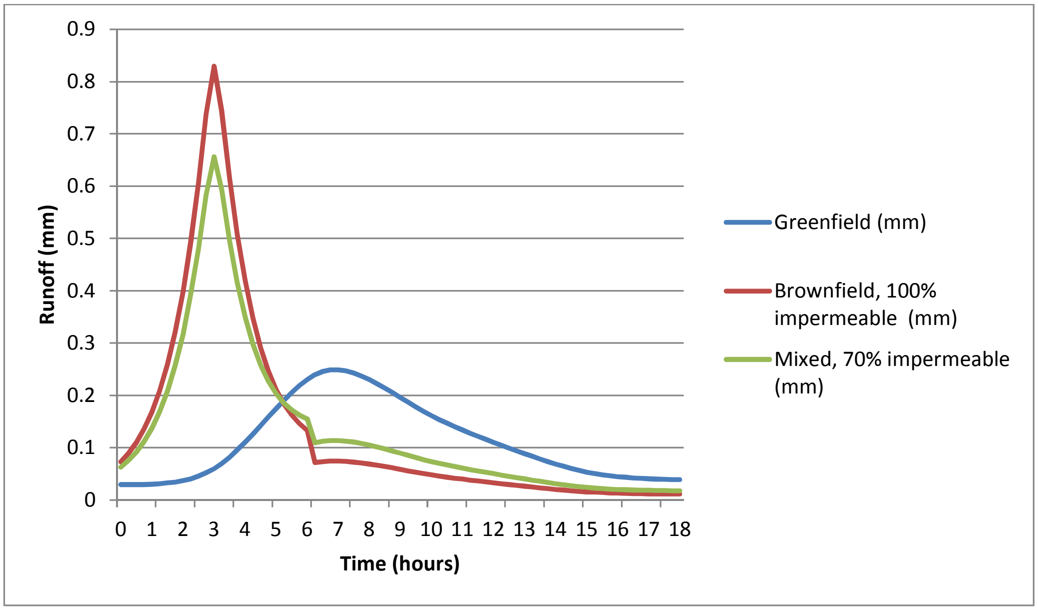


Figure 4~~5~~ Example Hydrograph for a Greenfield, 100% impermeable and 70% impermeable site.

~~1.4~~2.4 Determining the development site area

For most sites the development site considered is the same as the footprint of the development thus the post development will be 100% impermeable. i.e. if a footbridge is being built then the footprint of the footbridge is considered to be the development site and the site is initially 100% greenfield and post development 100% impermeable.

Some sites are more complex, for example the development of Islip and Water Eaton Parkway Stations. The proposal indicates that the aim will be to retain the runoff associated with the existing site (or greenfield where possible) which means that agreement of the development site extent may affect the amount of flood storage which must be allowed for. In these cases the development site is considered to be the addition of the existing and proposed development site. Post development all sites will be 100% impermeable unless land at any of the sites is returned to greenfield which is unlikely. This is illustrated for Islip Station, Figure 2, where the development site is the combined area of existing and proposed developments.

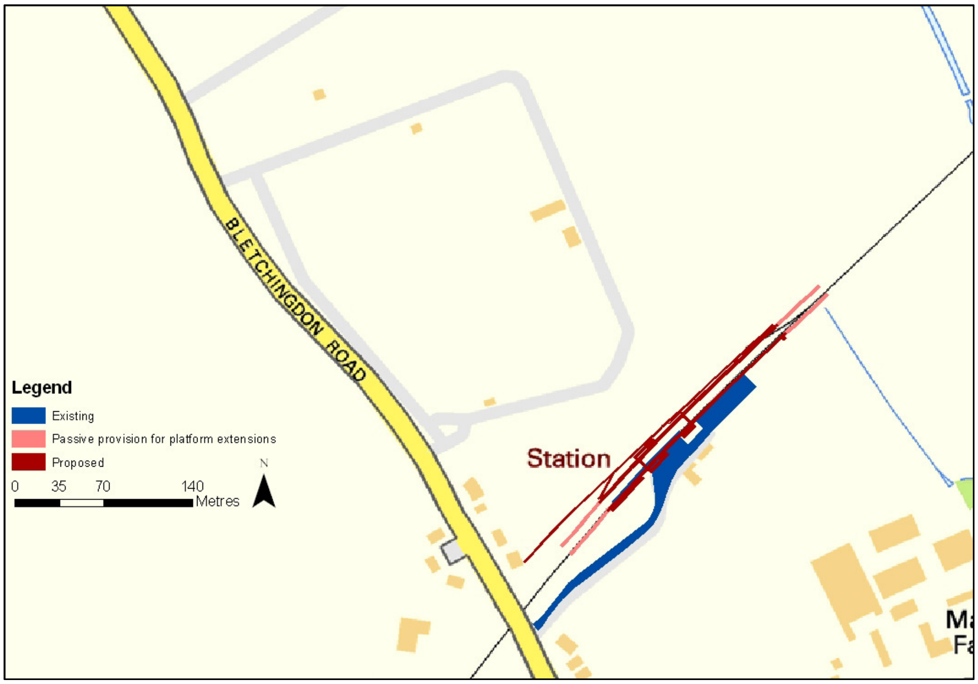


Figure ~~6~~5 Existing and post development site at Islip Station.

Appendix 2 – (AP2) Tubbs Lane Drainage Design

Appendix 3 – (AP3) Bicester Town Station Drainage Design