Tier Consult
Richmond House, Sandpiper Court,
Chester Business Park, Chester CH4 9QZ
01244 684900

Flood Risk Assessment

Symmetry Park Oxford North

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Contents

Executive Summary

The proposed development would be expected to remain dry in all but the most extreme conditions. Providing the recommendations made in this FRA are instigated, flood risk from all sources would be minimised, the consequences of flooding are acceptable, and the development would be in accordance with the requirements of the NPPF.

This FRA demonstrates that the proposed development would be operated with minimal risk from flooding, would not increase flood risk elsewhere and is compliant with the requirements of the NPPF. The development should not therefore be precluded on the grounds of flood risk.

1. Introduction

1.1 Background

This Flood Risk Assessment (FRA) has been prepared by Tier Consult at the request of Tritax Symmetry Limited and Siemens Healthineers for the proposed development at Symmetry Park Oxford North. This FRA utilises a hydraulic model of the Wendlebury Brook and Gagle Brook[1] supplied by the Environment Agency to further assess the flood risk to the site and the impact of the proposed development on flood risk.

This FRA has been carried out in accordance with guidance contained in the National Planning Policy Framework (NPPF)[2], associated Planning Practice Guidance (PPG)[3] and the PPG 'Site-specific flood risk assessment checklist (para 068 Reference ID: 7-068-20140306. This FRA identifies and assesses the risks of all forms of flooding to and from the development and demonstrates how these flood risks will be managed so that the development remains safe throughout the lifetime, taking climate change into account.

It is recognised that developments which are designed without regard to flood risk may endanger lives, damage property, cause disruption to the wider community, damage the environment, be difficult to insure and require additional expense on remedial works. The development design should be such that future users will not have difficulty obtaining insurance or mortgage finance, or in selling all or part of the development, as a result of flood risk issues.

1.2 Planning Policy Context

The Flood and Water Management Act 2010

The legislative framework for flood and coastal risk management is set out principally in The Flood and Water Management Act 2010 The legislation endorses the principle of an integrated approach to water and drainage management. The intentions of the Act are summarised below:

- Deliver improved security, service and sustainability for people and their communities;
- Clarify responsibilities for managing all sources of flood risk;
- Protect essential water supplies by enabling water companies to control more non-essential uses of water during droughts;
  - Modernise the law for managing the safety of reservoirs;
  - Encourage more sustainable forms of drainage in new developments through new arrangements for adoption and future operation of such features; and

[1] Wendlebury Brook Flood Risk Mapping Study, IMSE500106, JBA Consulting on behalf of the EA, 2014.
[2] Ministry of Housing, Communities and Local Government (2021) National Planning Policy Framework.
[3] Department for Communities and Local Government (2014) Planning Practice Guidance - Flood Risk and Coastal Change.

- Make it easier to resolve misconnections to sewers.

Water Framework Directive

The Water Framework Directive 2000/60/EC is a European Union directive designed to improve and integrate the way water, from all sources, is managed throughout Europe. In the UK, much of the implementation work is undertaken by competent authorities such as the Environment Agency and Local Authorities. It came into force in December 2000 and was transposed into UK law in 2003. Member States are required to achieve good chemical and ecological status for their inland and coastal waters by 2015.

Water Resources Act 1991

Under the Act, it is an offence to "cause or knowingly permit poisonous, noxious or polluting matter or any solid waste to enter controlled waters" unless it is covered by a consent to discharge issued by the Environment Agency. Failure to comply may result in a fine. This includes discharge to surface water drains.

National Planning Policy Framework

The National Planning Policy Framework (NPPF) aims to ensure that flood risk is taken into account by all relevant statutory bodies from regional to local authority planning departments to avoid inappropriate development in areas at risk of flooding and to direct development away from areas of high risk. Where new development is, exceptionally necessary in high risk areas, the policy framework aims to make it safe, ensure that it will not increase flood risk elsewhere and, where possible, reduce overall flood risk in the local area (see Paragraph 159 of the NPPF).

A risk-based approach is adopted at stages of the planning process, applying a source pathway receptor model to planning and flood risk. To demonstrate this, an FRA is required and should include:

- whether a proposed development is likely to be affected by current or future flooding from all sources;
- whether it will increase flood risk elsewhere;
- whether the measures proposed to deal with these effects and risks are appropriate;
- if necessary, provide the evidence to the Local Planning Authority (LPA) that the Sequential Test can be applied; and
- whether the development will be safe and pass part c) of the Exception Test if this is appropriate.

Local Authorities should only consider development in flood risk areas as appropriate where it is informed by a Site-specific Flood Risk Assessment, based upon the Environment Agency's Standing Advice on flood risk. The Assessment should identify and assess the risks of all forms of flooding to and from the development and demonstrate how flood risks will be managed so that the development remains safe throughout its lifetime, taking climate change into account (see Paragraph 167 of the NPPF).

Adopted Cherwell Local Plan 2011-2031 (Part 1)

The Adopted Cherwell Local Plan 2011-2031 (Part 1) contains strategic planning policies for development and the use of land. It forms part of the statutory Development Plan for Cherwell to which regard must be given in the determination of planning applications.

The Plan was formally adopted by the Council on 20 July 2015. Policy Bicester 13 was re-adopted on 19 December 2016.

Policy ESD 6: Sustainable Flood Risk Management states:

"The Council will manage and reduce flood risk in the District through using a sequential approach to development; locating vulnerable developments in areas at lower risk of flooding. Development proposals will be assessed according to the sequential approach and where necessary the exceptions test as set out in the NPPF and NPPG. Development will only be permitted in areas of flood risk when there are no reasonably available sites in areas of lower flood risk and the benefits of the development outweigh the risks from flooding.

In addition to safeguarding floodplains from development, opportunities will be sought to restore natural river flows and floodplains, increasing their amenity and biodiversity value. Building over or culverting of watercourses should be avoided and the removal of existing culverts will be encouraged.

Existing flood defences will be protected from damaging development and where development is considered appropriate in areas protected by such defences it must allow for the maintenance and management of the defences and be designed to be resilient to flooding.

Site specific flood risk assessments will be required to accompany development proposals in the following situations:

- *All development proposals located in flood zones 2 or 3*
- *Development proposals of 1 hectare or more located in flood zone 1*
- *Development sites located in an area known to have experienced flooding problems*
- *Development sites located within 9m of any watercourses.*

Flood risk assessments should assess all sources of flood risk and demonstrate that:

- *There will be no increase in surface water discharge rates or volumes during storm events up to and including the 1 in 100 year storm event with an allowance for climate change (the design storm event)*

- *Developments will not flood from surface water up to and including the design storm event or any surface water flooding beyond the 1 in 30 year storm event, up to and including the design storm event will be safely contained on site.*

Development should be safe and remain operational (where necessary) and proposals should demonstrate that surface water will be managed effectively on site and that the development will not increase flood risk elsewhere, including sewer flooding."

1.3 Report Structure

This FRA has the following report structure:

- Section 2 details the sources of information that have been consulted;
- Section 3 describes the location area and the existing and proposed development;
- Section 4 outlines the flood risk to the existing and proposed development;
- Section 5 details the proposed surface water drainage for the site;
- Section 6 describes the risk management methods used to mitigate all sources of flood risk; and
- Section 7 presents a summary and conclusions.

2. Sources of Information

2.1 Environment Agency

The Flood and Water Management Act 2010 gives the Environment Agency a strategic overview role for all forms of flooding and coastal erosion. They also have direct responsibility for the prevention, mitigation and remediation of flood damage for main rivers and coastal areas. The Environment Agency is the statutory consultee with regards to flood risk and planning. Information regarding the current flood risk at the application site, local flood defences and flood risk has been obtained from the Environment Agency (see Appendix B).

2.2 Cherwell District Council

Cherwell District Council is the Local Planning Authority (LPA) Planning guidance written by Cherwell District Council regarding flood risk was consulted to assess the policies in place. The Cherwell District Council Strategic Flood Risk Assessment (SFRA) which covers the site has been reviewed.

2.3 Oxfordshire County Council

Oxfordshire County Council is the Lead Local Flood Authority (LLFA) and has responsibilities for 'local flood risk', which includes surface runoff, groundwater and ordinary watercourses. Planning guidance written by Oxfordshire County Council regarding flood risk was consulted to assess the policies in place. The Oxfordshire County Council Preliminary Flood Risk Assessment (PFRA) which covers the site has been reviewed.

2.4 Thames Water

Thames Water is responsible for the disposal of waste water and supply of clean for this area. Information with regards to sewer and water main flooding contained within the Cherwell District Council SFRA and the Oxfordshire County Council PFRA have been consulted. All Water Companies have a statutory obligation to maintain a register of properties/areas which are at risk of flooding from the public sewerage system, and this is shown on the DG5 Flood Register.

3. Location & Description

3.1 Site Location

The site is located on land to the north of the A41 and east of the M40 near Junction 9 (NGR: SP 55368 19504) (see Drawing T/2407/FRA/1). The area of land within the red line boundary is 19.353 hectares (ha).

The boundary of the site fronts the A41 road and extends across several open fields that are currently in agricultural use. There are a number of buildings in agricultural or commercial use located in the north east part of the site.

The eastern extent of the site is defined by field boundaries and hedgerows, the Grange Farm Industrial Estate, and Lower Grange Farm. The Wendlebury Brook defines the western edge of the site, flowing from north to south towards a small area of woodland, where its course then changes to flow east across the Site, before passing under the A41.

Fields within the site are enclosed by hedgerows having few associated mature trees. The arable use offers negligible ecological value. A site survey has found the agricultural land to be of moderate quality (Grade 3b), which is not categorised as the best and most versatile.

3.2 Proposed Development

Full planning application for the erection of a new high quality combined research, development and production facility comprising of Class B2 floorspace and ancillary office floorspace with associated infrastructure including: formation of signal-controlled vehicular access to the A41 and repositioning of existing bus stops; ancillary workshops; staff gym and canteen; security gate house; a building for use as an energy centre (details of the energy generation reserved for future approval); loading bays; service yard; waste management area; external plant; vehicle parking; landscaping including permanent landscaped mounds; sustainable drainage details; together with the demolition of existing agricultural buildings within the red line boundary; and the realignment of an existing watercourse.

The proposed development will include the diversion of the Wendlebury Brook for approximately 450m and the creation of a 10m buffer strip on each bank of the Wendlebury Brook. The proposed site layout is shown within Appendix C.

The extent of the application site area includes the land needed to undertake construction, the re-alignment of the Wendlebury Brook, and landscaping including landscape mounds. Development would also require the removal of the existing agricultural buildings located within the north-east part of the Site.

The proposal will be delivered in 2 phases: Phase 1 due to become operational in 2024 and, Phase 2 due to become operational in 2030, subject to projected demand.

When complete and fully operational, the facility would provide approximately 1,345 jobs, including third party and visitor jobs. The production staff work in a shift pattern over a 24-hour period, 7-days a week.

3.3 Ground Levels

A topographical survey of the site has been undertaken (see Appendix D). Generally, ground levels fall from north to southeast, from approximately 77.50 metres Above Ordnance Datum (mAOD) to approximately 64.00mAOD.

3.4 Catchment Hydrology / Existing Drainage

The Wendlebury Brook flows through the site, the Wendlebury Brook outfalls into the Langford Brook downstream from the site, after flowing through Wendlebury Village, as shown in Figure 3.1. There is a drainage ditch running along the east of the M40 which reduces flows reaching the northern end of the site. This was observed to be dry during the site visit, and therefore suggests that it is not the primary flow route.

Currently, the site is unlikely to be served by a positive surface water drainage system, with rainfall currently infiltrating into the ground where geological and hydrogeological conditions allow, and then runoff once the infiltration capacity of the ground has been exceeded.

3.5 Ground Conditions

The British Geological Survey (BGS) Map indicates that the site is underlain by areas of River Terrace Deposits, 2 - sand and gravel, locally with lenses of silt, clay or peat and Alluvium - clay, silt, sand and gravel.

The bedrock deposits underlying the site consist of the Kellaway Clay Member -mudstone, the Kellaway Sand Member - sandstone and siltstone, interbedded and the Peterborough Member - mudstone.

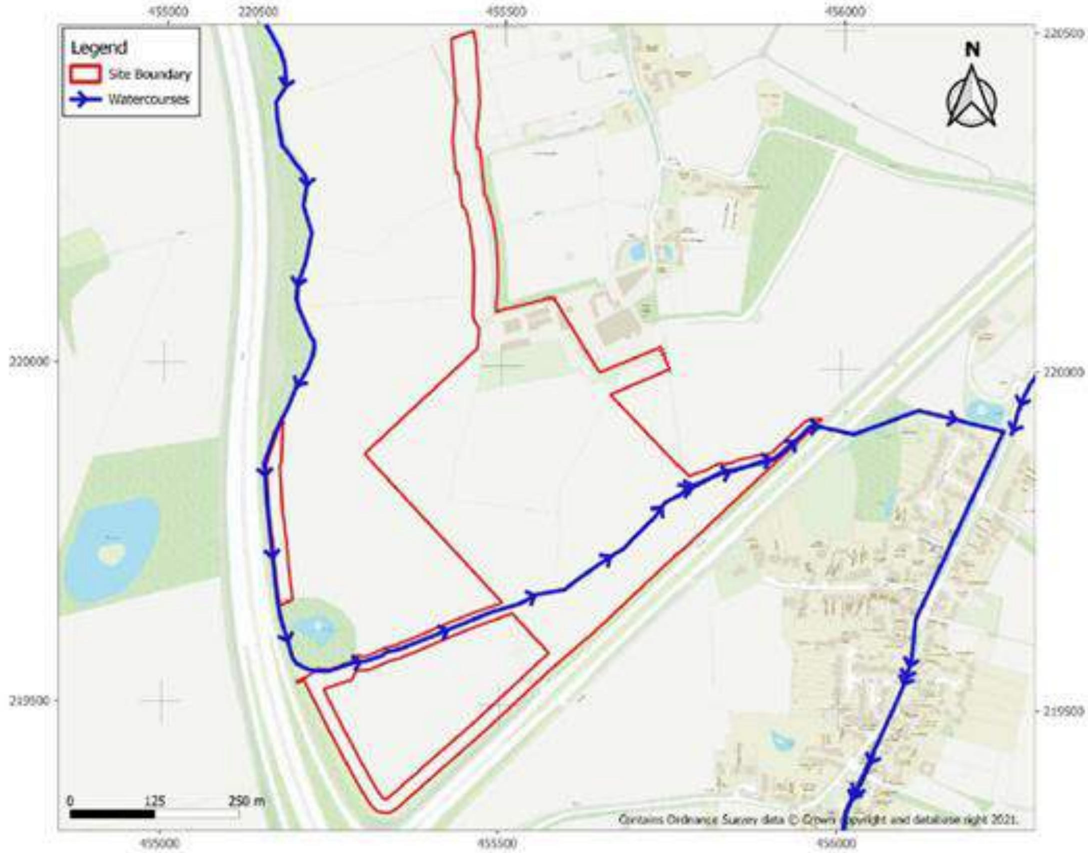

Figure 3.1 - Wendlebury Brook

4. Flood Risk

4.1 Sources of Flooding

All sources of flooding have been considered, these are; fluvial (river) flooding, tidal (coastal) flooding, groundwater flooding, surface water (pluvial) flooding, sewer flooding and flooding from artificial drainage systems/infrastructure failure.

4.2 Historic Flooding

The Environment Agency historic flood map shows that the site has not historically flooded. There are no records of anecdotal information of flooding at the site including within the British Hydrological Society "Chronology of British Hydrological Events[4]". No other historical records of flooding for the site have been recorded. Therefore, it has been assumed that the site has not flooded within the recent past.

It is understood that areas of Wendlebury Village, downstream from the site, have experienced localised flooding in the recent past.

4.3 Existing and Planned Flood Defence Measures

There are no existing flood defences which protect the site against flooding. Further risk management measures will be used to protect the site from flooding and these are discussed in Section 5.0.

4.4 Environment Agency Flood Zones

A review of the Environment Agency's flood map indicates that the majority of the site is located within Flood Zone 1 and therefore has a 'low probability' of fluvial flooding, with less than a 1 in 1000 annual probability of river (<0.1%) (see Drawing T/2407/FRA/2). However, a small proportion of the site, to the east on the left bank of the Wendlebury Brook, is located within Flood Zone 2 and therefore has a 'medium' probability of fluvial flooding, with between a 1 in 100 and 1 in 1000 annual probability of river flooding (1% - 0.1%) in any year. These areas are located immediately adjacent to the Wendlebury Brook.

The Environment Agency Flood Zones and acceptable development types are explained in Table 4.1. The Flood Zones are the current best information on the extent of the extremes of flooding from rivers or the sea that would occur without the presence of flood defences, because these can be breached, overtopped and may not be in existence for the lifetime of the development.

Table 4.1 - Environment Agency Flood Zones and Appropriate Land Use

Flood Zone	Probability	Explanation	Appropriate Land Use
Zone 1	Low	Less than 1 in 1000 annual probability of river or sea flooding in any year (<0.1%)	All development types generally acceptable
Zone 2	Medium	Between a 1 in 100 and 1 in 1000 annual probability of river flooding (1% - 0.1%) or between a 1 in 200 and 1 in 1000 annual probability of sea flooding (0.5% 0.1%) in any year	Most development type are generally acceptable
Zone 3a	High	A 1 in 100 or greater annual probability of river flooding (>1%) or a 1 in 200 or greater annual probability of flooding from the sea (>0.5%) in any year	Some development types not acceptable
Zone 3b	'Functional Floodplain'	Land where water has to be flow or be stored in times of flood. SFRAs should identify this zone (land which would flood with an annual probability of 1 in 20 (5%) or greater in any year or is designed to flood in an extreme (0.1% flood, or at another probability to be agreed between the LPA and the Environment Agency, including water conveyance routes)	Some development types not acceptable

4.5 Flood Risk Vulnerability

In the Planning Practice Guidance to the NPPF (Table 1) appropriate uses have been identified for the Flood Zones. Applying the Flood Risk Vulnerability Classification in Table 2 and 3 of the Planning Practice Guidance to the NPPF, the development is classified as 'less vulnerable'. Table 3 of the Planning Practice Guidance to the NPPF states that 'less vulnerable' uses are appropriate within Flood Zones 1, 2 and 3.

Table 4.2 - Flood Risk Vulnerability and Flood Zone 'Compatibility' as identified in Table 3 of the Planning Practice Guidance to the NPPF

Flood Risk Vulnerability Classification	Essential Infrastructure	Water Compatible	Highly Vulnerable	More Vulnerable	Less Vulnerable
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	✓	Exception test required	✓	✓
Zone 3a	Exception test required	✓	×	Exception test required	✓
Zone 3b 'Functional Floodplain'	Exception test required	✓	×	×	×

Key: ✓: Development is appropriate, ×: Development should not be permitted.

4.6 Climate Change

Projections of future climate change, in the UK, indicate more frequent, short-duration, high intensity rainfall and more frequent periods of long duration rainfall. Guidance included within the NPPF recommends that the effects of climate change are incorporated into FRA. Recommended precautionary sensitivity ranges for peak rainfall intensities and peak river flows are outlined in the associated Planning Practice Guidance to the NPPF[4].

Table 4.3 shows peak river flow climate change allowances by river basin district. The flood risk assessments: climate change allowances guidance recommends that for 'less vulnerable' uses in Flood Zone 2 that the central allowances are used. Therefore, the design flood level for the site is the 1 in 100 year (+15%) event.

Table 4.3 - Peak River Flow Allowances by River Basin District (use 1961 to 1990 baseline)

River Basin District	Allowance Category	2020s	2050s	2080s
Cherwell and Ray Management Catchment	Upper	+24%	+27%	+49%
	Higher	+11%	+10%	+25%
	Central	+6%	+4%	+15%

[4] https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances#high-allowances.

4.7 Fluvial (river) Flooding

Existing Scenario

This section discusses the existing flood risk posed to the site from the Wendlebury Brook based on its current alignment and watercourse location. The Wendlebury Brook poses the primary flood risk to the site, as it flows through the site. The Environment Agency modelled flood outlines are shown in Figures 4.1 with the modelled water levels shown in Table 4.4. Figure 4.1 shows that the site will not be inundated with floodwater for all events up to and including the 1 in 100 year (+20%) event, the site will be flood free during the 1 in 100 year (+20%) event.

Table 4.4 - Environment Agency Modelled Water Levels (mAOD)

Node Label	100	100 (+20%)	1000
061_14_2014_001WB_2872_MN	66.81	66.87	67.02
061_14_2014_001WB_2733_MN	66.35	66.41	66.56
061_14_2014_001WB_2599_MN	65.89	65.95	66.09
061_14_2014_001WB_2457_MN	65.38	65.45	65.59
061_14_2014_001WB_2320_MN	65.04	65.10	65.17
Floodplain 1	65.70	65.77	65.90
Floodplain 2	65.94	66.00	66.14
Floodplain 3	Null	Null	66.40

The site will only be inundated with floodwater during the 1 in 1000 year event. These areas are located immediately adjacent to the Wendlebury Brook and are located on the left bank of the Wendlebury Brook.

Figure 4.2 shows the Environment Agency hazard mapping. This shows that the majority of the site is not at risk of fluvial flooding and therefore, does not have a hazard rating. A very small proportion of the site, immediately adjacent to the Wendlebury Brook on the left bank, are shown to have a very low hazard rating.

These results are consistent with the Environment Agency Flood Zones discussed in Section 4.4.

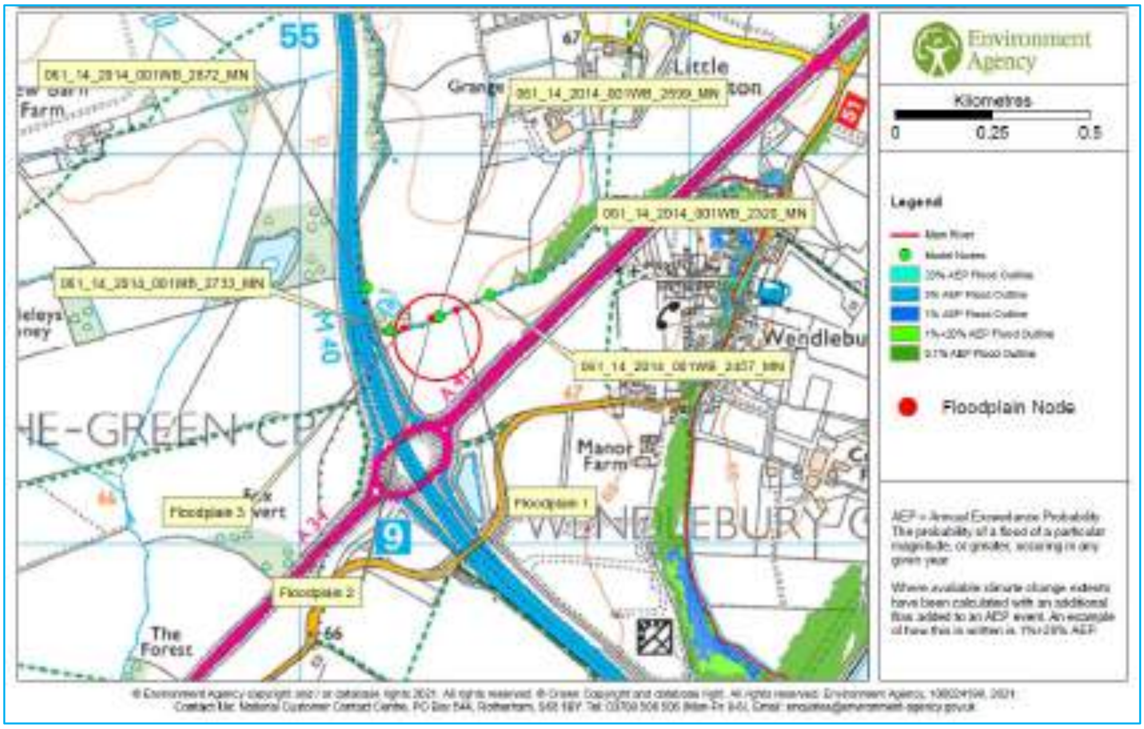

Figure 4.1 - Environment Agency Modelled Flood Outlines

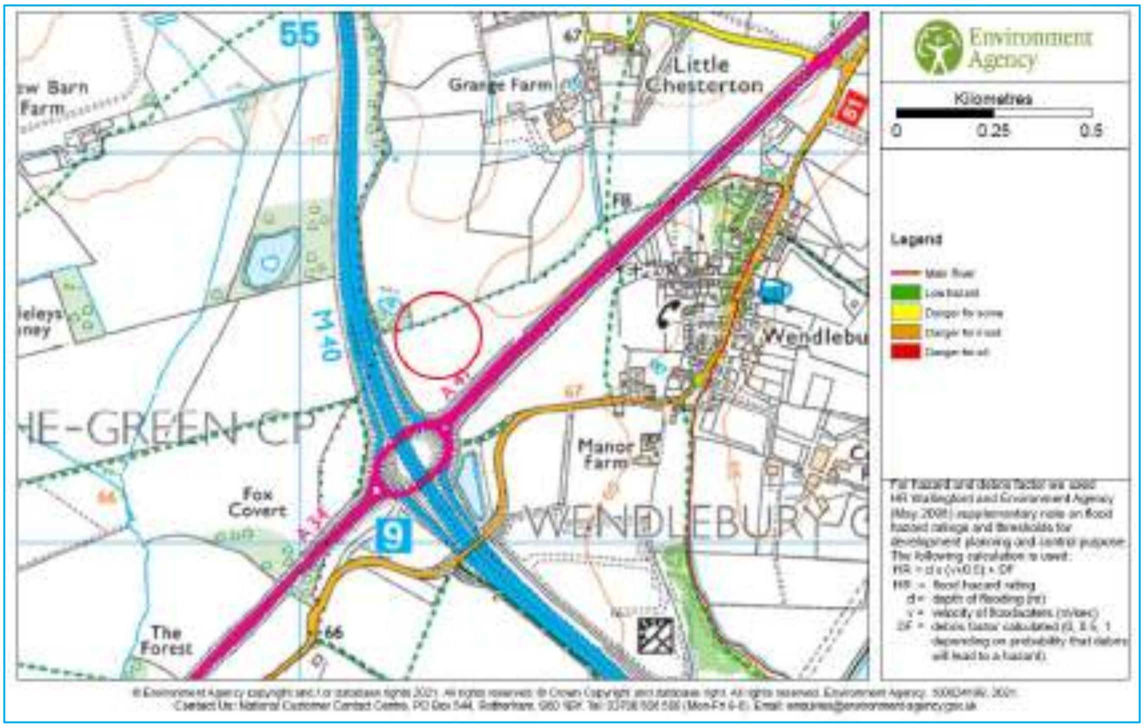

Figure 4.2 - Environment Agency Hazard Mapping

This FRA utilises a hydraulic model of the Wendlebury Brook and Gagle Brook[5] supplied by the Environment Agency to further assess the flood risk to the site. The baseline model has been updated to use the latest software version, updated climate change allowances to be suitable for use in this study. The modelling report and modelling outputs are contained within Appendix E.

During the 1 in 100 year event there are no significant out of bank flows for the Wendlebury Brook upstream of the A41. As such the site is not at risk of fluvial flooding during the 1 in 100 year baseline event, as shown in Figure 4.3.

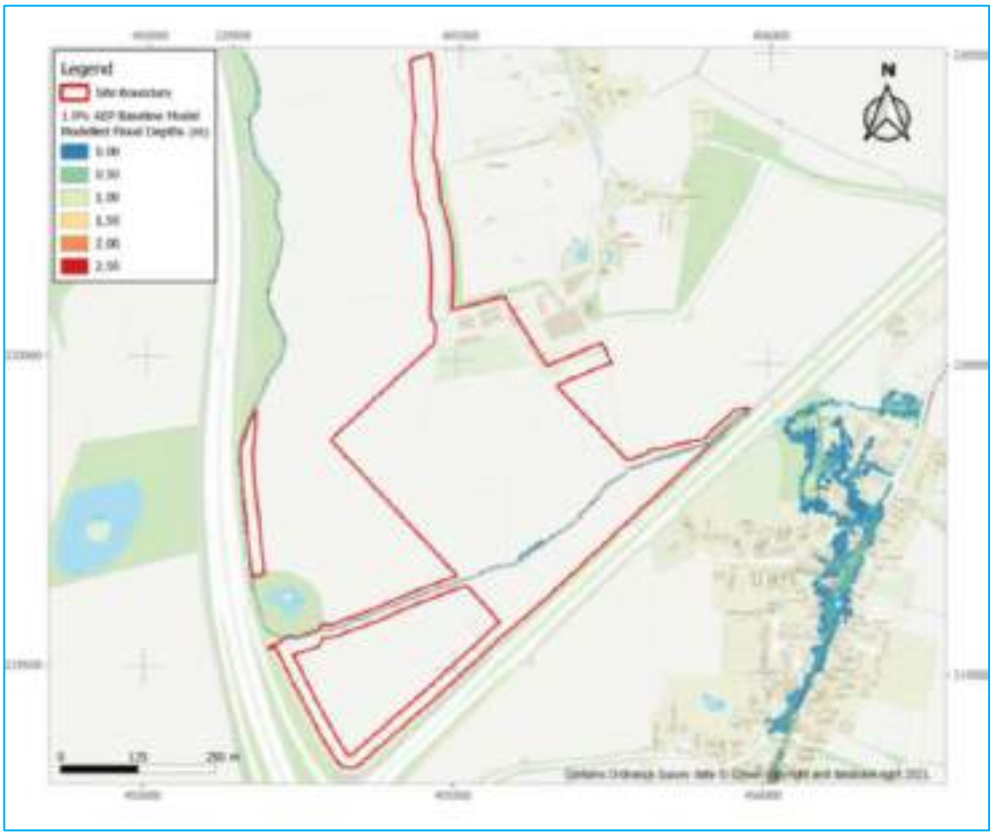

Figure 4.3 - Baseline Condition: 1 in 100 Year Event Modelled Results

[5] Wendlebury Brook Flood Risk Mapping Study, IMSE500106, JBA Consulting on behalf of the EA, 2014.

During the 1 in 100 year (+15%) event there are no significant out of bank flows within the development site, as in the 1 in 100 year event, as shown in Figure 4.4. As such the site is not at risk of fluvial flooding during the 1 in 100 year (+15%) event.

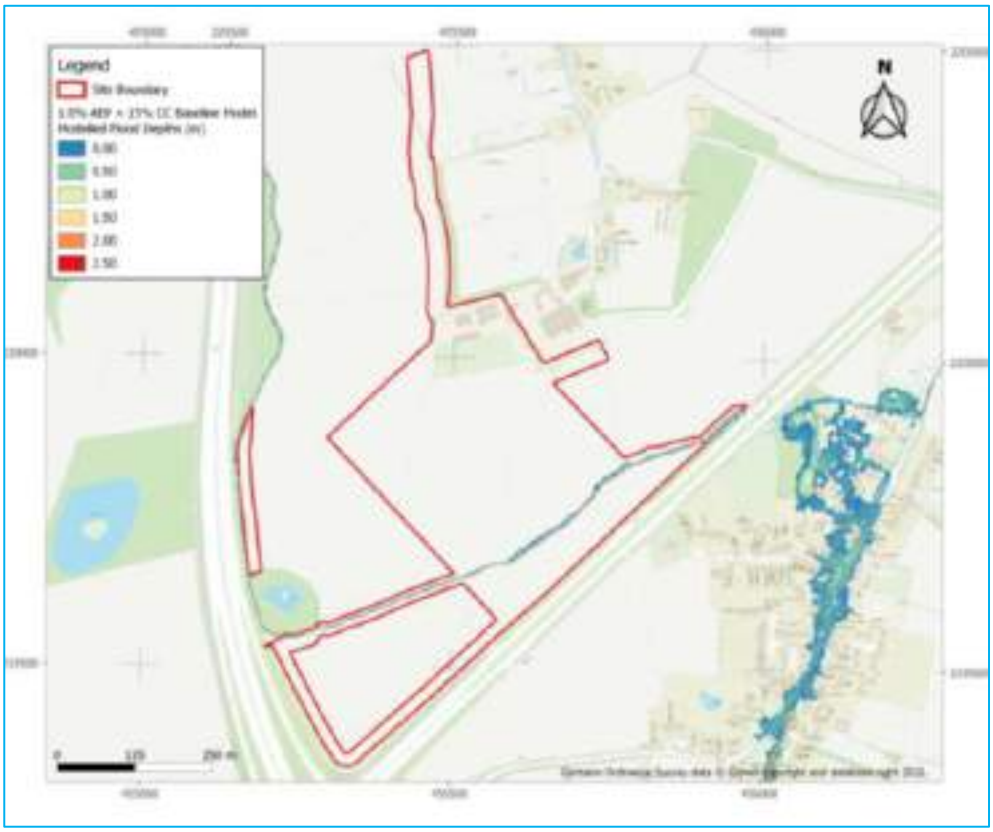

Figure 4.4 - Baseline Condition: 1 in 100 Year (+15%) Event Modelled Results

During the more extreme 1 in 1000 year event some fluvial flooding can be seen within the eastern area of the site, as shown in Figure 4.5. As such some of the site is at risk of fluvial flooding during the 1 in 1000 year baseline event. These areas are located immediately adjacent to the Wendlebury Brook and are located on the left bank of the Wendlebury Brook. These results are consistent with the Environment Agency Flood Zones discussed in Section 4.4 and with the Environment Agency data discussed above.

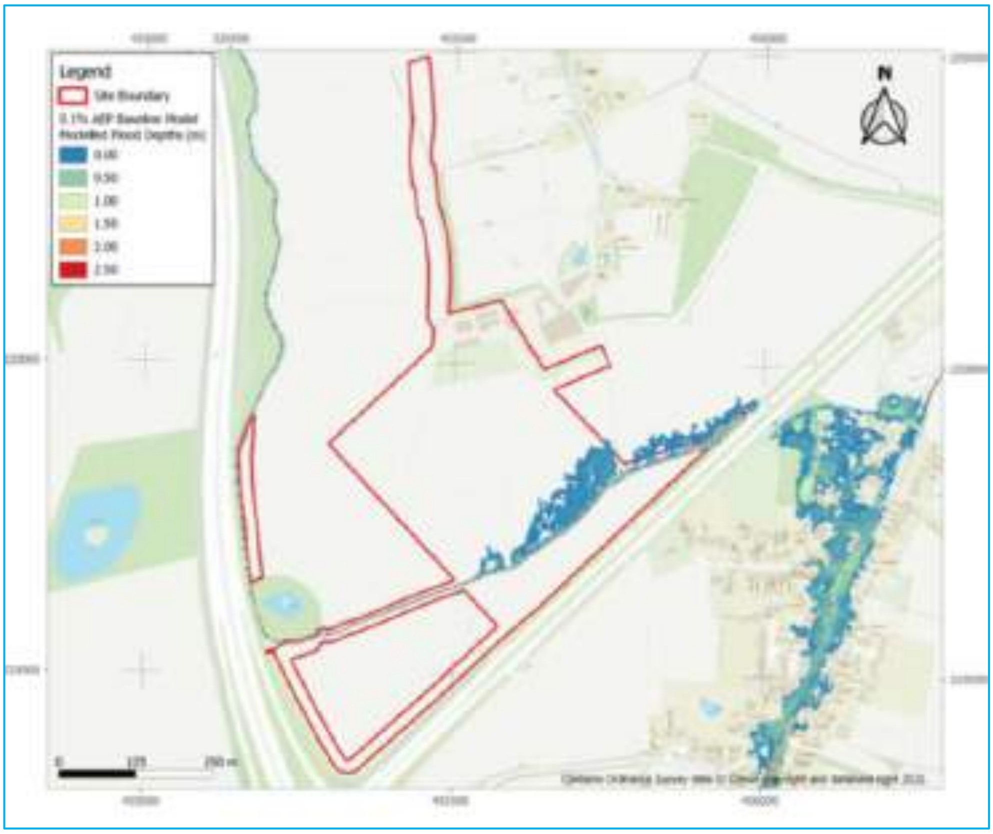

Figure 4.5 - Baseline Condition: 1 in 1000 Year Event Modelled Results

The modelled results indicate that the site is not at high risk of fluvial flooding. Flooding within the site is confined to the east of the site boundary, adjacent to the banks of the watercourse. The majority of the site is not at risk of flooding. The floodwaters are restricted from entering the site boundary by the surrounding topography.

In conclusion, the outputs from the detailed hydraulic modelling show that the majority of the site will not be inundated with floodwater for all events up to and including the 1 in 1000 year event.

The likelihood of a rapid river level rise and possible rapid inundation of areas posing a risk to life is considered to be minimal. The site is located within a low risk area where the onset of flooding is gradual as per Flood Risk Assessment Guidance for New Development Phase 2, R&D Technical Report FD2320/TR2. The speed of inundation and rate of floodwater rise would be low.

Any overbank flow would follow the contours of the surrounding area and would flow directly to the west rather than flowing towards the site. The flood risk can also be considered to be limited due to the difference in elevations. The ground levels of the site are located above the normal water level of the Wendlebury Brook. The flood risk would be confined to the river channel and areas immediately adjacent.

Given the scale and nature of the proposed development and the size and location of the fluvial flooding sources it has been concluded that fluvial flooding poses a low flood risk to the site therefore, the risk of fluvial flooding is considered to be of low significance.

4.8 Tidal (coastal) Flooding

The site is not located within the vicinity of tidal flooding sources and the risk of tidal flooding is considered to be not significant.

4.9 Groundwater Flooding

Groundwater flooding is defined as the emergence of groundwater at the ground surface or the rising of groundwater into man-made ground under conditions where the normal range of groundwater levels is exceeded. Groundwater flooding tends to occur sporadically in both location and time. When groundwater flooding does occur, it tends to mostly affect low-lying areas, below surface infrastructure and buildings (for example, tunnels, basements and car parks) underlain by permeable rocks (aquifers).

The susceptibility of the site to groundwater flooding, based on the underlying geological conditions, is low. There are no records of groundwater flooding at or near to the site. It can therefore be concluded that the risk of groundwater flooding is not significant.

4.10 Surface Water (pluvial) Flooding

The soil condition at the site and within the vicinity of the site and the topography of the site suggest that the site is relatively well drained and surface water flooding would not be expected to accumulate to any significant depths. Surface water flooding tends to occur sporadically in both location and time.

The Environment Agency Surface Water flood map shows that the majority of the site has very low risk of surface water flooding with a chance of flooding of less than 1 in 1000 (0.1%) years (see Drawing T/2407/3). However, small areas of the site have a low to high risk of surface water flooding with a chance of flooding of 1 in 1000 (0.1%) years to greater than 1 in 30 (3.3%) years.

This is associated with the Wendlebury Brook, low spots within the site and ponds within the site. The surface water runoff is shown to follow the topographical lows with runoff flowing the local topography. This may result in ponding of 300-900mm in depth and water velocities of more 0.25m/s on a very small proportion of the site. The risk of flooding from surface water flooding is considered to be of low significance.

4.11 Sewer Flooding

Sewer flooding occurs when urban drainage networks become overwhelmed and maximum capacity is reached. This can occur if there is a blockage in the network causing water to back up behind it or if the sheer volume of water draining into the system is too great to be handled. Sewer flooding tends to occur sporadically in both location and time such flood flows would tend to be confined to the streets around the development.

There are existing public sewers within roads adjacent to the site these will inevitably have a limited capacity so in extreme conditions there would be surcharges, which may in turn cause flooding. Flood flows could also be generated by burst water mains, but these would tend to be of a restricted and much lower volume than weather generated events and so can be discounted for the purposes of this assessment. Given the design parameters normally used for drainage design in recent times and allowing for some deterioration in the performance of the installed systems, which are likely to have been in place for many years, an appropriate flood risk probability from this source could be assumed to have a return period in the order of 1 in 10 to 1 in 30 years.

The provision of adequate level difference between the ground floors and adjacent ground level would reduce the annual probability of damage to property from this source to 1 in 100 years or less. Therefore, sewer flooding poses a low flood risk to the site. It can therefore be concluded that the risk of sewer flooding is considered to be not significant.

4.12 Flooding from Artificial Drainage Systems/Infrastructure Failure

There are no other nearby artificial water bodies, water channels and artificial drainage systems that could be considered a flood risk to the site. The Environment Agency Reservoir flood map shows that the site is not at risk of reservoir flooding (see Drawing T/2407/FRA/4). The risk of flooding from reservoir failure is considered to be not significant.

4.13 Effects of the Development on Flood Risk

Proposed Scenario

The proposed development will include the diversion of the Wendlebury Brook for approximately 450m and the creation of a 10m buffer strip on each bank of the Wendlebury Brook. The diverted channel will have, as a minimum, the same dimensions to the existing channel.

The proposed development scenario has been modelled to assess the impact of the proposed development on flood risk. A number of different options for the channel dimensions have been modelled to ascertain the optimum solution which does not increase flood risk.

Figure 4.6 shows the flood depths during the proposed scenario 1 in 100 year event, there are no out of bank flows upstream of the A41 culvert as in the baseline model. Figure 4.7 shows the change in flood depth during the proposed scenario 1 in 100 year event, there is no modelled impact.

Figure 4.6 - Proposed Development (including watercourse diversion) 1 in 100 Year Event Modelled Results

Figure 4.7 - Proposed Development (including watercourse diversion) 1 in 100 Year Event Modelled Change in Modelled Flood Depth

Figure 4.8 shows the flood depths during the proposed scenario 1 in 100 year (+15%) event, there are no out of bank flows upstream of the A41 culvert as in the baseline model. Figure 4.9 shows the change in flood depth during the proposed scenario 1 in 100 year (+15%) event, there is no modelled impact.

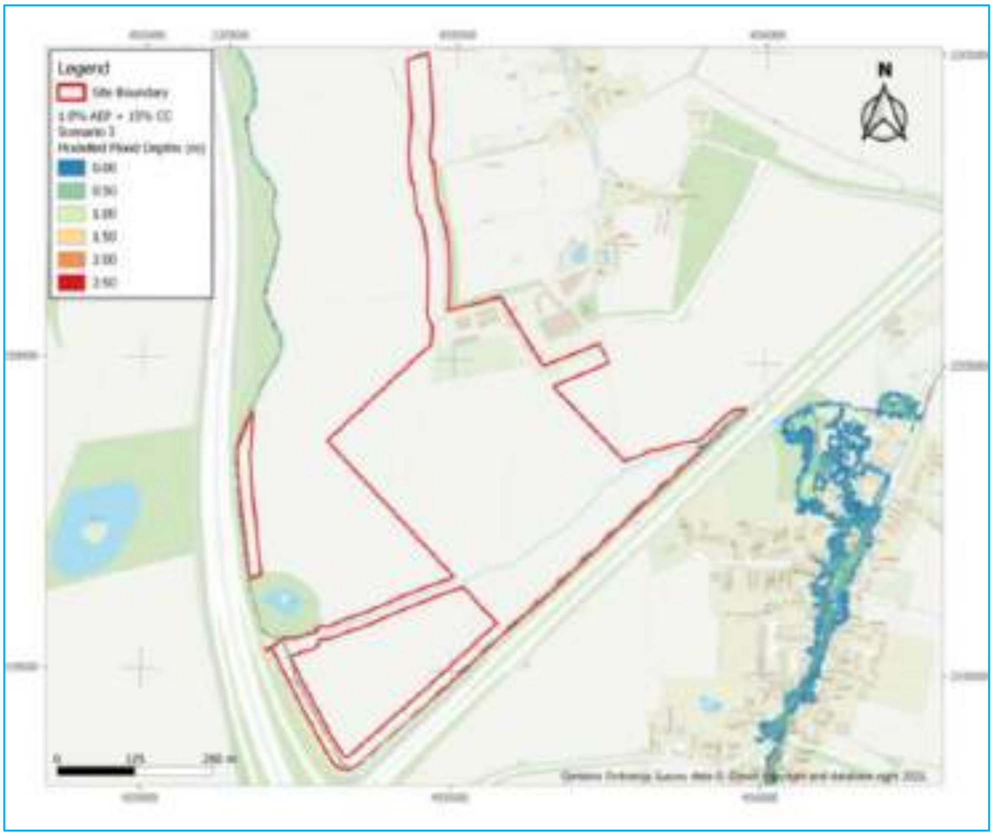

Figure 4.8 - Proposed Development (including watercourse diversion) 1 in 100 Year (+15%) Event Modelled Results

Figure 4.9 - Proposed Development (including watercourse diversion) 1 in 100 Year (+15%) Event Modelled Change in Modelled Flood Depth

Figure 4.10 shows the flood depths during the proposed scenario 1 in 1000 year event and shows that the flood outline upstream of the A41 culvert is reduced compared to the baseline scenario 1 in 1000 year event. The post development scenario shows a reduced out of bank flooding upstream of Wendlebury.

Figure 4.11 shows the change in flood depth during the proposed scenario 1 in 1000 year event. At the upstream extremity of the realigned channel, some increases in depth can be seen. However, for the majority of the modelled extent, no impact can be seen. Upstream of the A41 culvert, the majority of changes in depth lie within the +/- 10mm band, but both some small extents of increases and decreases in depth beyond 10mm can be seen. There are some isolated cells of flood depth increases beyond 10mm in Wendlebury. Due to the inherited instabilities in the baseline model through Wendlebury, the cause of these increases is thought to be a result of oscillations in the modelling results rather than as a result of the proposed development.

The proposed development will have no impact on flood risk. The flood risk will not be increased elsewhere. There will be no detriment to the flood storage capacity of the site. The overall direction of the movement of water will be maintained within the developed site and surrounding area. The conveyance routes (flow paths) will not be blocked or obstructed. The proposed development will have no impact on the movement of floodwater across the site. There will be no increase in the floodwater levels due to the proposed development. There will be no loss in flood storage capacity and no change in the on-site and off-site flood risk.

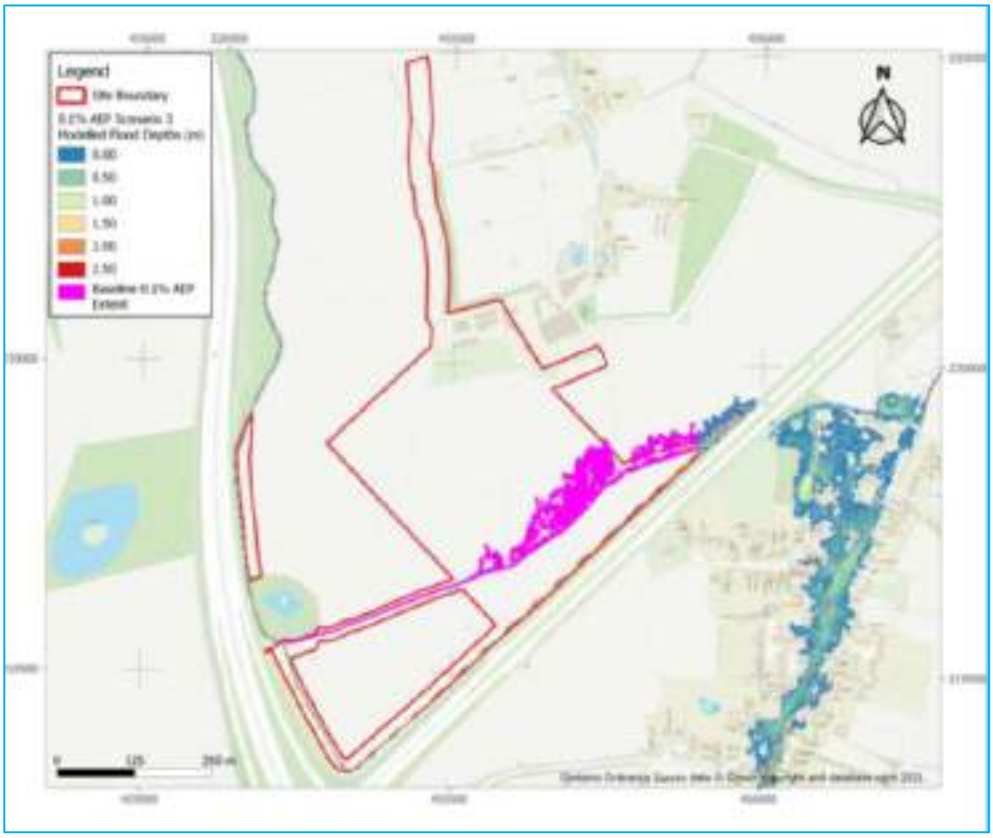

Figure 4.10 - Proposed Development (including watercourse diversion) 1 in 1000 Year Event Modelled Results

Figure 4.11 - Proposed Development (including watercourse diversion) 1 in 1000 Year Event Modelled Change in Modelled Flood Depth

To assess the impact of a blockage of the culvert under the A41, a blockage scenario was run. Following a review of the risk of a blockage occurring, it was determined that the risk at this location was low, as a result of the large culvert size and lack of potential debris sources. As such a 33% blockage ratio is considered appropriate for this location.

Figure 4.12 below shows the modelled flood depths during the 1 in 100 year (+15%) event with a 33% blockage ratio for the culvert under the A41. As can be seen a small out of bank flood extent can be seen just upstream of the A41 culvert as a result of the blockage scenario. This small increase in extent does not pose any additional risk to the proposed development site and is limited to a small area of agricultural land.

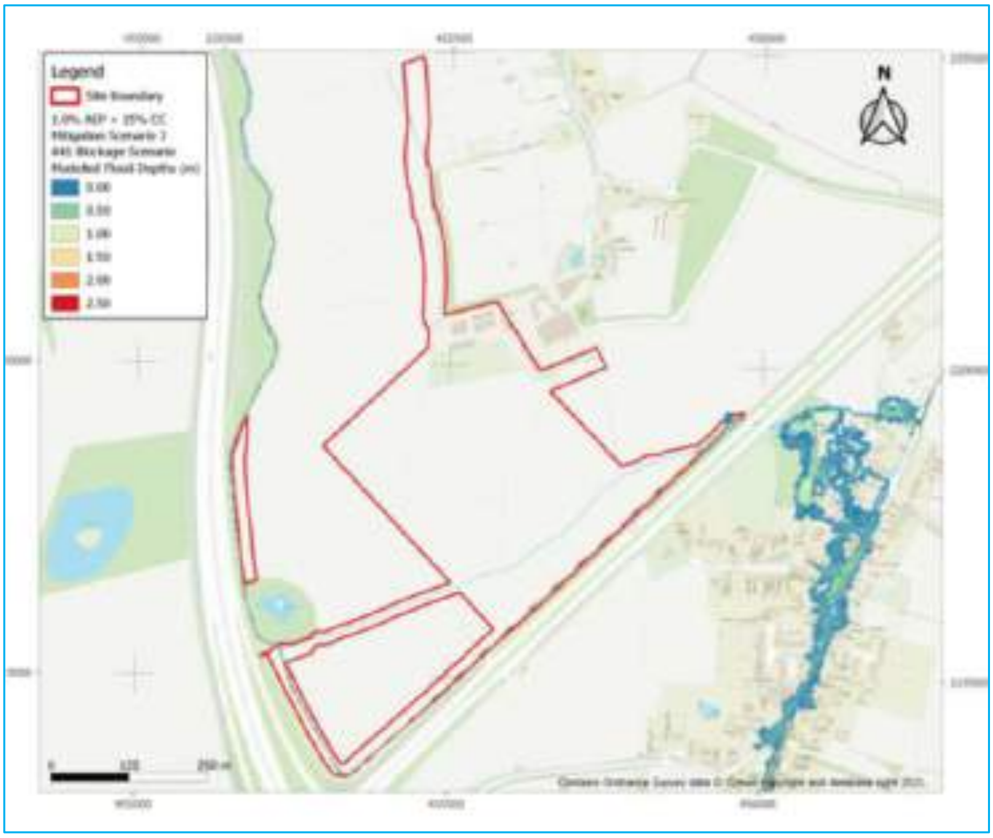

Figure 4.12 - Proposed Development (including watercourse diversion) 1 in 100 Year (+15%) Event Modelled Results

4.14 Summary of Site-Specific Flood Risk Assessment

A summary of the sources of flooding and a review of the risk posed by each source at the site is shown in Table 4.5.

Table 4.5 - Risk Posed by Flooding Sources

Sources of Flooding	Potential Flood Risk	Potential Source	Probability/Significance
Fluvial (river) Flooding	Yes	Wendlebury Brook	Existing Scenario: Low Proposed Scenario: Low
Tidal (coastal) Flooding	No	None Reported	Not significant
Groundwater Flooding	No	None Reported	Not significant
Surface Water (pluvial) Flooding	Yes	Low Spots	Low
Sewer Flooding	No	None Reported	Not significant
Flooding from Artificial Drainage Systems/Infrastructure Failure	No	None Reported	Not significant

The site is unlikely to flood except in extreme conditions. The primary, but unlikely, flood risk posed to the site is from fluvial flooding from Wendlebury Brook however, the site has no history of flooding.

The Environment Agency's flood map indicates that the majority of the site is located within Flood Zone 1 and therefore has a 'low probability' of fluvial flooding, with less than a 1 in 1000 annual probability of river (<0.1%). However, a small proportion of the site, to the east on the left bank of the Wendlebury Brook, is located within Flood Zone 2 and therefore has a 'medium' probability of fluvial flooding, with between a 1 in 100 and 1 in 1000 annual probability of river flooding (1% - 0.1%) in any year. These areas are located immediately adjacent to the Wendlebury Brook.

The site will not be inundated with floodwater for all events up to and including the 1 in 100 year (+20%) event, the site will be flood free during the 1 in 100 year (+20%) event. This has been confirmed by the Environment Agency data and the hydraulic modelling undertaken as part of this FRA.

A small area of the site will only be inundated with floodwater during the 1 in 1000 year event. These areas are located immediately adjacent to the Wendlebury Brook and are located on the left bank of the Wendlebury Brook.

Given the scale and nature of the proposed development and the size and location of the fluvial flooding sources it has been concluded that fluvial flooding poses a low flood risk to the site therefore, the risk of fluvial flooding is considered to be of low significance. A secondary flooding source has been identified which may pose a low risk to the site. This is:

- Surface Water (pluvial) Flooding

The risk of flooding from all sources is considered to be low or not significant. The flooding sources will only inundate part of the site to a relatively low water depth and water velocity, will only last a short period of time, in very extreme cases and will not have an impact on the whole of the proposed development site.

The proposed development scenario has been modelled to assess the impact of the proposed development on flood risk including the diversion of Wendlebury Brook. A number of different options for the channel dimensions have been modelled to ascertain the optimum solution which does not increase flood risk.

During the proposed scenario 1 in 100 year (+15%) event, there are no out of bank flows upstream of the A41 culvert as in the baseline model. During the proposed scenario 1 in 1000 year event and shows that the flood outline upstream of the A41 culvert is reduced compared to the baseline scenario 1 in 1000 year event. The post development scenario shows a reduced out of bank flooding upstream of Wendlebury.

There will be no detriment to the flood storage capacity of the site. The overall direction of the movement of water will be maintained within the developed site and surrounding area. The conveyance routes (flow paths) will not be blocked or obstructed. The proposed development will have no impact on the movement of floodwater across the site. There will be no increase in the floodwater levels due to the proposed development. There will be no loss in flood storage capacity and no change in the on-site and off-site flood risk.

The flood risk at the site, will be further managed and mitigated by using a number of risk management techniques, and mitigation strategies to manage and reduce the overall flood risk at the site The application is for a new, suitable flood-resilient design. The exposure of people and property will be reduced and minimised compared to existing site conditions. The chance of flooding each year is low each year. This takes into account the effect of any flood defences that may be located within the vicinity of the site as well property level protection measures.

5. Surface Water Drainage

5.1 Surface Water Drainage Overview

It is recognised that consideration of flood issues should not be confined to the floodplain. The alteration of natural surface water flow patterns through developments can lead to problems elsewhere in the catchment, particularly flooding downstream. For example, replacing vegetated areas with roofs, roads and other paved areas can increase both the total and the peak flow of surface water runoff from the development site. Changes of land use on previously developed land can also have significant downstream impacts where the existing drainage system may not have sufficient capacity for the additional drainage.

A SuDS Strategy for the site proposals should be developed to manage and reduce the flood risk posed by the surface water runoff from the site. An assessment of the surface water runoff rates should be undertaken, in order to determine the surface water options and attenuation requirements for the site. The assessment considers the impact of the development compared to current conditions. Therefore, the surface water attenuation requirement for the developed site can be determined and reviewed against existing arrangements.

The surface water drainage arrangements for any development site should be such that the volumes and peak flow rates of surface water leaving a developed site are no greater than the rates prior to the proposed development, unless specific off-site arrangements are made and result in the same net effect.

It should be acknowledged that the satisfactory collection, control and discharge of surface water runoff are now a principle planning and design consideration. This is reflected in recently implemented guidance and released National Sustainable Drainage Systems (SuDS) Standards.

5.2 Discharge of Surface Water

There are three possible options to discharge the surface water runoff in accordance with requirement H3 of the Building Regulations, this hierarchy is also promoted within the NPPF. Rainwater shall discharge to one of the following, listed in order of priority:

- an adequate soakaway or some other adequate infiltration system; or, where that is not reasonably practicable,
- a watercourse; or where that is not reasonably practicable,
- a sewer.

It is necessary to identify the most appropriate method of controlling and discharging surface water. The design should seek to improve the local runoff profile by using systems that can either attenuate runoff and reduce peak flow rates or positively impact on the existing surface water runoff.

5.3 Soakaway/Infiltration System

In determining the future surface runoff from the site, the potential of using infiltration devices has been considered. Infiltration methods are unlikely to be suitable across the majority of the site due to the likely presence of low permeability strata. Therefore, it will not be possible to discharge surface water runoff from the site via infiltration methods.

5.4 Watercourse

Should infiltration be found to be unsuitable, the next option is discharge to a watercourse. Wendlebury Brook flows through the site. Therefore, it will be possible to discharge surface water runoff from the site into a watercourse and this is the preferred option. All surface water runoff that cannot be discharged via infiltration will be managed on site and discharged to the Wendlebury Brook. The QBAR runoff rate has been calculated to be 22.50 litres/second therefore, it is proposed the surface water for the development will discharge into Wendlebury Brook at the QBAR runoff rate of 22.50l/s for all events up to and including the 1 in 100 year (+40%) events.

5.5 Sewer

In the event that discharge of surface water via infiltration or discharge to a watercourse is deemed unsuitable, then discharge to the public sewer is the next option. However, there is also no surface water sewer within a reasonable distance from the site furthermore, it is proposed to discharge the surface water into a watercourse therefore, this option is not required.

5.6 SuDS and Water Quality

Current guidance promotes sustainable water management through the use of SuDS. SuDS measures should be used to control the surface water runoff from the proposed development site therefore, managing the flood risk to the site and surrounding areas from surface water runoff.

1. A hierarchy of techniques is identified6:
2. Prevention – the use of good site design and housekeeping measures on individual sites to prevent runoff and pollution (e.g. minimise areas of hard standing).
3. Source Control – control of runoff at or very near its source (such as the use of rainwater harvesting, permeable paving, soakaways and/or green roofs).
4. Site Control – management of water from several sub-catchments (including routing water from roofs and car parks to one/several large soakaways for the whole site, swales and/or infiltration trenches).
5. Regional Control – management of runoff from several sites, typically in a detention pond, basins, tanks and/or wetland.

It is generally accepted that the implementation of SUDS as opposed to conventional drainage systems, provides several benefits by:

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[6] CIRIA (2004) Report C609, Sustainable Drainage Systems – Hydraulic, Structural and Water Quality advice.

- reducing peak flows to watercourses or sewers and potentially reducing the risk of flooding downstream;
- reducing the volumes and frequency of water flowing directly to watercourses or sewers from developed sites;
- improving water quality over conventional surface water sewers by removing pollutants from diffuse pollutant sources;
- reducing potable water demand through rainwater harvesting;
- improving amenity through the provision of public open spaces and wildlife habitat; and
- replicating natural drainage patterns, including the recharge of groundwater so that base flows are maintained.

The most appropriate attenuation system will need to satisfy three main characteristics, firstly, provide the required volume of storage, secondly, minimise the loss of developable land and thirdly, where possible provide local amenity. The application of the SuDS Manual requires that the runoff from sites is not only restricted to meet the pre-development runoff characteristics but also that SuDS systems are utilised to improve the quality of the runoff prior to outfall to watercourses. The SuDS Manual and Environment Agency guidance applies a sustainability hierarchy to the various types of SuDS systems, this is summarised in Table 5.1.

Table 5.1 - Sustainability Hierarchy

Most Sustainable	**SuDS Technique**	**Flood Reduction**	**Pollution Reduction**	**Landscape & Wildlife**
	Green / Living Roofs / Living Walls	✓	✓	✓
	Basins and ponds - Constructed wetlands - Balancing ponds - Detention basins - Retention ponds	✓	✓	✓
	Filter strips and swales	✓	✓	✓
	Infiltration Devices - Soakaways	✓	✓	✓
	Permeable Surfaces and Filter Drains - Gravelled areas - Solid paving blocks - Permeable paving	✓	✓	
Least Sustainable	Tanked systems - Over-sized pipes/tanks - Cellular storage	✓		

Systems at the top of the hierarchy provide a combination of attenuation, treatment and ecology and are deemed the most sustainable options. There are always specific scenarios where systems are more suitable than others and at this stage it is not possible to guide the development towards a particular strategy.

The usual approach is to consider the 'SUDS train' where each of the above options are considered in turn until a suitable solution is found. Thus, source control techniques such as soakaways, rainwater harvesting and/or infiltration trenches, if suitable on a site, are considered preferable to permeable conveyance and passive treatment systems such as tanks or ponds. The various options are considered in outlined in Table 5.2.

Table 5.2 - SuDS Techniques

SuDS Technique	Comments	Suitability for Development
Green / Living Roofs / Living Wall	A green roof is a multi-layered system that covers the top of a building with soil and vegetation and which can provide a degree of rain storm attenuation and a reduction in site runoff. Can be used to reduce the volume and rate of runoff so that other SUDS techniques in the scheme can be significantly reduced in size.	Not a practical option for the proposed development. A green/living roof/living wall would not provide all of the attenuation storage requirements alone.
Basins / Ponds	Provides storage of runoff and flow attenuation. Vegetated surfaces can be used to support the prevention of runoff from the site for small rainfall events (interception) and improve water quality associated with the removal of sediment and buoyant materials.	The required area is available and it is proposed to use an attenuation pond within the SuDS Strategy.
Filter Strips / Swales	Good removal of urban pollutants, reduces runoff rates and volumes.	The required area is available and it is proposed to use a swale within the SuDS Strategy.
Infiltration Devices (e.g. soakaways)	Reduces total runoff volume from the development.	Soakaways are not suitable due to site ground conditions and limited space.
Permeable Surfaces and Filter Drains	Permeable surfaces together with their associated substructures are an efficient means of intercepting runoff, reducing the volume and frequency of runoff and providing a treatment medium.	While possible as there is no infiltration potential it would merely be a storage option with only 30% void. It is far more cost effective and maintainable to collect surface water and use attenuation pond, swale and attenuation cellular storage.

Tanked / Cellular Systems	Ideal for sites with insufficient space for basins etc., provide a volume of below ground storage with a high void ratio.	Potential to be installed under the main central area with clearance from the foundations. The required area is available and it is proposed to use cellular storage within the SuDS Strategy.
Flow Reduction	Manages and reduces the flood risk to the local surface water sewers and watercourses.	A hydrobrake can be installed downstream of attenuation tanks and control flows to the natural greenfield run off rates. It is proposed to use hydrobrakes.

5.7 SuDS Strategy

The objective of this SuDS Strategy is to ensure that a sustainable drainage solution can be achieved which reduces the peak discharge rate to manage and reduce the flood risk posed by the surface water runoff from the site. One of the aims of the NPPF is to provide not only flood risk mitigation but also to maximise additional gains such as improvements in runoff quality and provision of amenity and biodiversity. The SuDS Strategy takes into account the following principles:

- No increase in the volume or runoff rate of surface water runoff from the site.
- No increase in flooding to people or property off-site as a result of the development.
- No surface water flooding of the site.
- The proposals take into account a 40% increase in rainfall intensity due to climate change during the next 100 years which is the lifetime of the development.

For all development, a hierarchical approach to surface water management. This approach has been adopted within this SuDS Strategy with a restricted discharge to the Wendlebury Brook being utilised and will take the form of:

- Attenuation pond.
- Swale.
- Cellular attenuation storage.

It is proposed the surface water for the development will discharge into Wendlebury Brook at the QBAR runoff rate of 22.50l/s for all events up to and including the 1 in 100 year (+40%) events, as shown in Appendix F.

As a consequence of limiting the rate of discharge from the site, at times of heavy rainfall the volume of water leaving the site will be significantly less than that draining from it. In order to prevent this water backing up in the system and causing flooding, attenuation storage will be incorporated into the site layout of attenuation storage will be provided. The size of this attenuation storage has been calculated such that the proposed development has the capacity to accommodate the 100 year rainfall event including a 40% increase in rainfall intensity that is predicted to occur as a result of climate change.

The remainder of the site that is not formally drained, i.e. landscaped areas, will be permeable (grass). The majority of rainwater falling on these areas will soak into the ground. Surface water runoff would be directed to the drainage system through drainage gullies located around the perimeter of the buildings and through contouring of the hardstanding areas.

These methods will reduce peak flows, the volume of runoff, and slow down flows and will provide a suitable SuDS solution for this site. The adoption of a SuDS Strategy for the site represents an enhancement from the current conditions as the current surface water runoff from the site is uncontrolled, untreated, unmanaged and unmitigated. In adopting these principles, it has been demonstrated that a scheme can be developed that does not increase the risk of flooding to adjacent properties and development further downstream.

5.8 Designing for Local Drainage System Failure/Design Exceedance

The exceedance flow routes are shown in Appendix F. When considering residual risk, it is necessary to make predictions as to the impacts of a storm event that exceeds the design event, or the impact of a failure of the local drainage system. The SuDS Strategy applies a safe and sustainable approach to discharging rainfall runoff from the site and this reduces the risk of flooding however, it is not possible to completely remove the risk. This section of the FRA is therefore associated with the way the residual risk is managed.

As part of the SuDS Strategy it must be demonstrated that the flooding of property would not occur in the event of local drainage system failure and/or design exceedance. It is not economically viable or sustainable to build a drainage system that can accommodate the most extreme events. Consequently, the capacity of the drainage system may be exceeded on rare occasions, with excess water flowing above ground[7].

The attenuation requirements have been designed to accommodate the 1 in 100 year event plus climate change (+40%). The design of the site layout provides an opportunity to manage this local drainage system failure/exceedance flow and ensure that indiscriminate flooding of property does not occur.

There will not be an extensive sewerage network on the proposed development site and therefore any potential exceedance flooding would be from the drains connecting the buildings to the attenuation storage. It is very unlikely that a catastrophic failure would occur. An exceedance or blockage event of the drains would not affect the proposed buildings because the finished floor level will be raised above surrounding ground levels, ensuring any exceedance flooding would not affect the buildings.

Exceedance flows would be contained within the highways within the site and adjacent to the site and would flow to the lower ground levels where the landscaped areas are located. In particular, the landscaped areas will include preferential flow paths that convey water away from buildings. Surface

[7] CIRIA (2006) Designing for exceedance in urban drainage – good practice.

water runoff would be directed to the drainage system through drainage gullies located around the perimeter of the buildings and through contouring of the hardstanding areas. It is not considered that there is an increased risk to the properties on the site or located adjacent to the site.

When considering the impacts of a storm event that exceeds the 1 in 100 year (+40%) event, there is safety factor for attenuation storage, even under the design event conditions. Consequently, if this event were to be exceeded there is additional capacity with the system to accommodate this. If this freeboard was to be exceeded the consequences would be similar, if not less than for the local drainage system failure. Drainage gullies, manholes and pipework will provide additional water storage and provide betterment. Consequently, the impact of an exceedance event is not considered to represent any significant flood hazard.

The above manages and mitigates the flood risk from surface water runoff from surface water runoff generated by the site development and to offsite locations as well as the risk from surface water runoff generated offsite.

6. Risk Management

6.1 Introduction

In this flood zone, developers and local authorities should seek opportunities to reduce the overall level of flood risk in the area through the layout, form of the development and the use of flood mitigation measures including SuDS techniques. The flooding sources will have to be mitigated on the site by using a number of techniques, and mitigation strategies to manage and reduce the overall flood risk at the site. These will be used to ensure the development will be safe and there is:

- Minimal risk to life;
- Minimal disruption to people living and working in the area;
- Minimal potential damage to property;
- Minimal impact of the proposed development on flood risk generally; and;
- Minimal disruption to natural heritage.

6.2 Minimum Floor Level

There is no minimum finished floor level proposed as a result of flooding required. However, it is recommended that generally all building floor levels are located above the highways by 150mm (apart from HGV loading areas) to enable the full capacity of any secondary flood conveyance to be utilised.

6.3 Flood Resilience and Resistance

Relatively simple measures such as raising utility entry points, using first floor or ceiling down electrical circuits and sloping landscaping away from the buildings can be easily and economically incorporated into the development of the site.

6.4 Access and Egress

The site and surrounding area is not located within the floodplain therefore a permanently safe and dry access can be maintained.

6.5 Buffer Strip

The proposed development will include the creation of a 10m buffer strip on each bank of the Wendlebury Brook within the site. This will allow access for maintenance of the watercourse while also accommodating any flooding.

6.6 Flooding Consequences

The mitigation measures detailed above show that the flood risk can be effectively managed and therefore the consequences of flooding are acceptable. In conclusion, the flood risk to the site can be considered to be limited, with a low annual probability of flooding and from all sources.

7. Summary and Conclusion

7.1 Introduction

This report presents an FRA in accordance with the NPPF for the proposed development at Symmetry Park Oxford North. This FRA identifies and assesses the risks of all forms of flooding to and from the development and demonstrates how these flood risks will need to be managed so that the development remains safe throughout the lifetime, taking climate change into account.

7.2 Flood Risk

The site is unlikely to flood except in extreme conditions. The primary, but unlikely, flood risk posed to the site is from fluvial flooding from Wendlebury Brook however, the site has no history of flooding.

The Environment Agency's flood map indicates that the majority of the site is located within Flood Zone 1 and therefore has a 'low probability' of fluvial flooding, with less than a 1 in 1000 annual probability of river (<0.1%). However, a small proportion of the site, to the east on the left bank of the Wendlebury Brook, is located within Flood Zone 2 and therefore has a 'medium' probability of fluvial flooding, with between a 1 in 100 and 1 in 1000 annual probability of river flooding (1% - 0.1%) in any year. These areas are located immediately adjacent to the Wendlebury Brook.

The site will not be inundated with floodwater for all events up to and including the 1 in 100 year (+20%) event, the site will be flood free during the 1 in 100 year (+20%) event. This has been confirmed by the Environment Agency data and the hydraulic modelling undertaken as part of this FRA.

A small area of the site will only be inundated with floodwater during the 1 in 1000 year event. These areas are located immediately adjacent to the Wendlebury Brook and are located on the left bank of the Wendlebury Brook.

Given the scale and nature of the proposed development and the size and location of the fluvial flooding sources it has been concluded that fluvial flooding poses a low flood risk to the site therefore, the risk of fluvial flooding is considered to be of low significance. A secondary flooding source has been identified which may pose a low risk to the site. This is:

- Surface Water (pluvial) Flooding

The risk of flooding from all sources is considered to be low or not significant. The flooding sources will only inundate part of the site to a relatively low water depth and water velocity, will only last a short period of time, in very extreme cases and will not have an impact on the whole of the proposed development site.

The proposed development scenario has been modelled to assess the impact of the proposed development on flood risk including the diversion of Wendlebury Brook. A number of different options for the channel dimensions have been modelled to ascertain the optimum solution which does not increase flood risk.

During the proposed scenario 1 in 100 year (+15%) event, there are no out of bank flows upstream of the A41 culvert as in the baseline model. During the proposed scenario 1 in 1000 year event and shows that the flood outline upstream of the A41 culvert is reduced compared to the baseline scenario 1 in 1000 year event. The post development scenario shows a reduced out of bank flooding upstream of Wendlebury.

There will be no detriment to the flood storage capacity of the site. The overall direction of the movement of water will be maintained within the developed site and surrounding area. The conveyance routes (flow paths) will not be blocked or obstructed. The proposed development will have no impact on the movement of floodwater across the site. There will be no increase in the floodwater levels due to the proposed development. There will be no loss in flood storage capacity and no change in the on-site and off-site flood risk.

The flood risk at the site, will be further managed and mitigated by using a number of risk management techniques, and mitigation strategies to manage and reduce the overall flood risk at the site The application is for a new, suitable flood-resilient design. The exposure of people and property will be reduced and minimised compared to existing site conditions. The chance of flooding each year is low each year. This takes into account the effect of any flood defences that may be located within the vicinity of the site as well property level protection measures.

7.3 SuDS Strategy

The SuDS Strategy ensures that a sustainable drainage solution can be achieved which reduces the peak discharge rate to manage and reduce the flood risk posed by the surface water runoff from the site. One of the aims of the NPPF is to provide not only flood risk mitigation but also to maximise additional gains such as improvements in runoff quality and provision of amenity and biodiversity. The SuDS Strategy takes into account the following principles:

- No increase in the volume or runoff rate of surface water runoff from the site.
- No increase in flooding to people or property off-site as a result of the development.
- No surface water flooding of the site.
- The proposals take into account a 40% increase in rainfall intensity due to climate change during the next 100 years which is the lifetime of the development.

For all development, a hierarchical approach to surface water management. This approach has been adopted within this SUDS Strategy with a restricted discharge to the Wendlebury Brook being utilised and will take the form of:

- Attenuation pond.
- Swale.
- Cellular attenuation storage.

It is proposed the surface water for the development will discharge into Wendlebury Brook at the QBAR runoff rate of 22.50l/s for all events up to and including the 1 in 100 year (+40%) events, as shown in Appendix F.

As a consequence of limiting the rate of discharge from the site, at times of heavy rainfall the volume of water leaving the site will be significantly less than that draining from it. In order to prevent this water backing up in the system and causing flooding, attenuation storage will be incorporated into the site layout of attenuation storage will be provided. The size of this attenuation storage has been calculated such that the proposed development has the capacity to accommodate the 100 year rainfall event including a 40% increase in rainfall intensity that is predicted to occur as a result of climate change.

The remainder of the site that is not formally drained, i.e. landscaped areas, will be permeable (grass). The majority of rainwater falling on these areas will soak into the ground. Surface water runoff would be directed to the drainage system through drainage gullies located around the perimeter of the buildings and through contouring of the hardstanding areas.

These methods will reduce peak flows, the volume of runoff, and slow down flows and will provide a suitable SuDS solution for this site. The adoption of a SuDS Strategy for the site represents an enhancement from the current conditions as the current surface water runoff from the site is uncontrolled, untreated, unmanaged and unmitigated. In adopting these principles, it has been demonstrated that a scheme can be developed that does not increase the risk of flooding to adjacent properties and development further downstream.

7.4 Risk Management

The flooding sources will be managed on the site by using a number of mitigation strategies to manage and reduce the overall flood risk at the site and will ensure the development will be safe. Measures used:

Minimum Floor Level - There is no minimum finished floor level proposed as a result of flooding required. However, it is recommended that generally all building floor levels are located above the highways by 150mm (apart from HGV loading areas) to enable the full capacity of any secondary flood conveyance to be utilised.

Flood Resilience and Resistance - Relatively simple measures such as raising utility entry points, using first floor or ceiling down electrical circuits and sloping landscaping away from the buildings can be easily and economically incorporated into the development of the site.

Access and Egress - The site and surrounding area is not located within the floodplain therefore a permanently safe and dry access can be maintained.

Buffer Strip - The proposed development will include the creation of a 10m buffer strip on each bank of the Wendlebury Brook within the site. This will allow access for maintenance of the watercourse while also accommodating any flooding.

7.5 Conclusion

In conclusion, the proposed development, would be expected to remain dry in all but the most extreme conditions. Providing the recommendations made in this FRA are instigated, flood risk from all sources would be minimised, the consequences of flooding are acceptable, and the development would be in accordance with the requirements of the NPPF.

This FRA demonstrates that the proposed development would be operated with minimal risk from flooding, would not increase flood risk elsewhere and is compliant with the requirements of the NPPF. The development should not therefore be precluded on the grounds of flood risk.

Appendix A – Drawings

KEY

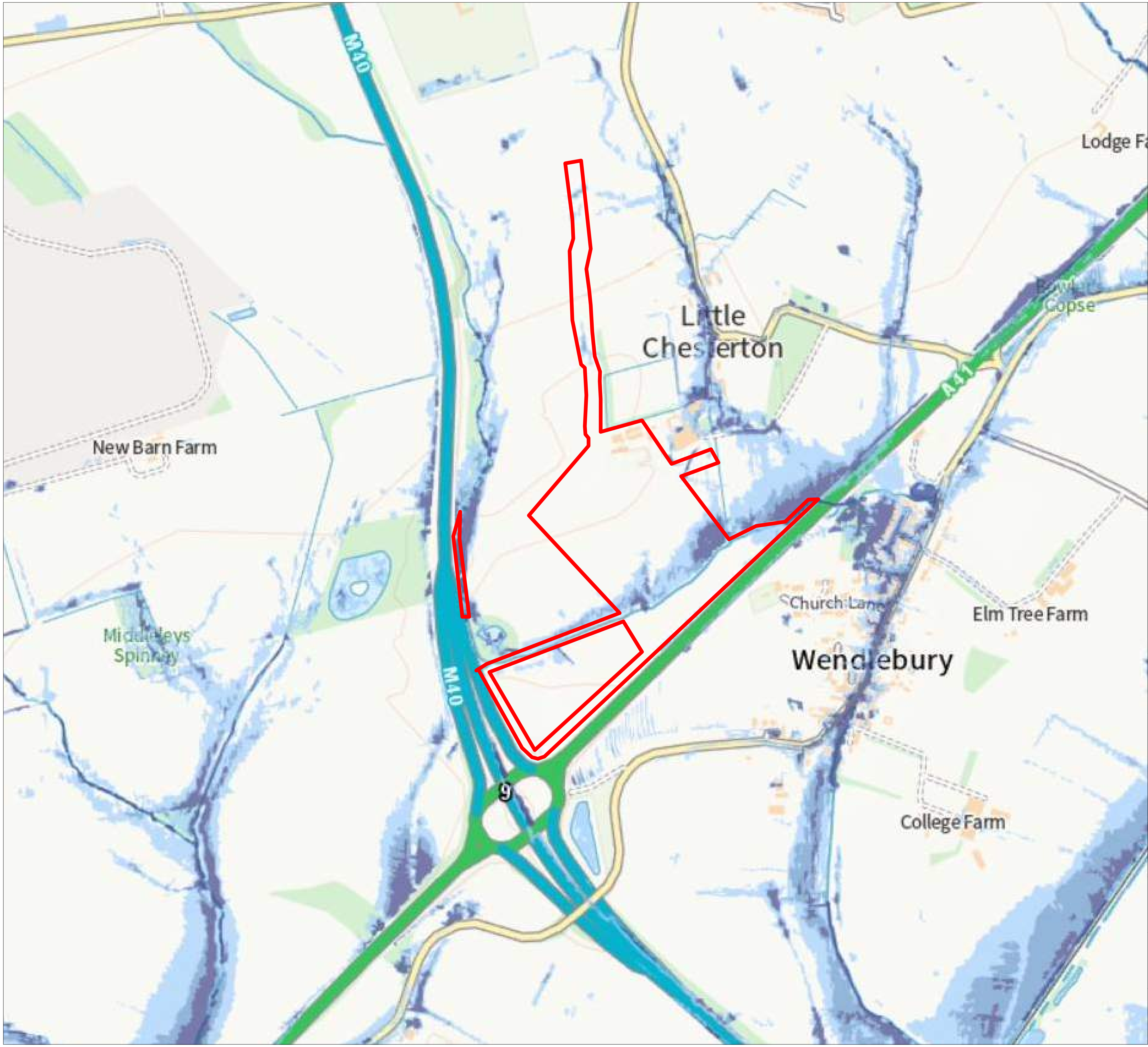
Site Boundary

PROJECT	Symmetry Park Oxford North
CLIENT	Tritax Symmetry Limited and Siemens Healthineers
TITLE	Site Location
PROJECT REF	T/2407/FRA
DRAWING NO	T/2407/FRA/1
SCALE	NTS@A3
DATE	July 2021

KEY

- Site Boundary
- Flood Zone 3
- Flood Zone 2
- Flood Zone 1

PROJECT	**Symmetry Park Oxford North**
CLIENT	**Tritax Symmetry Limited and Siemens Healthineers**
TITLE	**Environment Agency Flood Zones**
PROJECT REF	**T/2407/FRA**
DRAWING NO	**T/2407/FRA/2**
SCALE	**NTS@A3**
DATE	**July 2021**

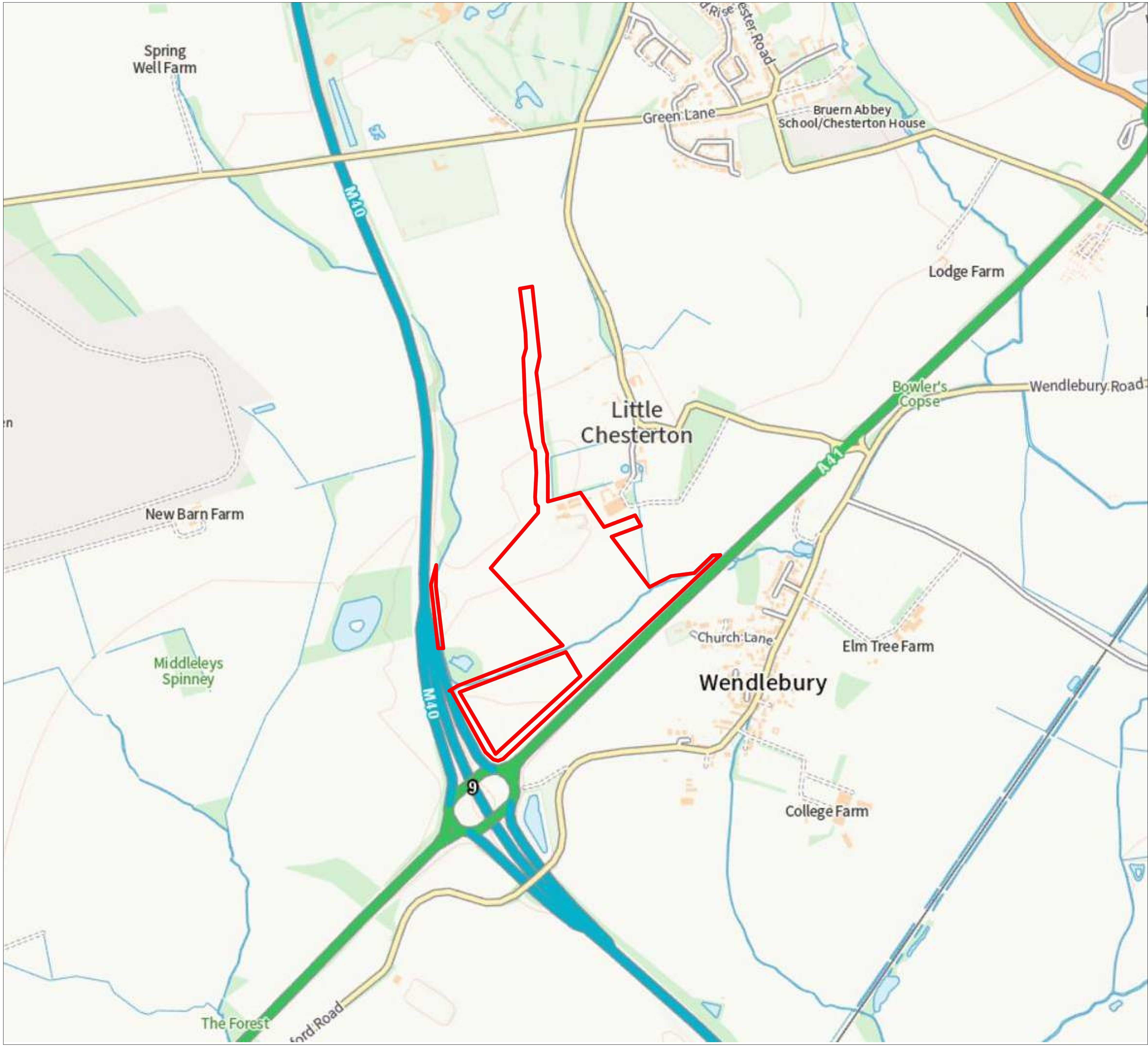


KEY

- Site Boundary
- Very Low
- Low
- Medium
- High

TIER CONSULT

PROJECT	**Symmetry Park Oxford North**
CLIENT	**Tritax Symmetry Limited and Siemens Healthineers**
TITLE	**Environment Agency Surface Water Flood Map**
PROJECT REF	**T/2407/FRA**
DRAWING NO	**T/2407/FRA/3**
SCALE	**NTS@A3**
DATE	**July 2021**



KEY

Site Boundary

Reservoir Flood Outline

TIER CONSULT

PROJECT
Symmetry Park Oxford North

CLIENT
Tritax Symmetry Limited and Siemens Healthineers

TITLE
Environment Agency Reservoir Flood Map

PROJECT REF
T/2407/FRA

DRAWING NO
T/2407/FRA/4

SCALE
NTS@A3

DATE
July 2021

Appendix B – Environment Agency Correspondence

Product 4 (Detailed Flood Risk) for OX25 3QE
Our Ref: THM220769

Product 4 is designed for developers where Flood Risk Standing Advice FRA (Flood Risk Assessment) Guidance Note 3 Applies. This is:

i) "all applications in Flood Zone 3, other than non-domestic extensions less than 250 sq metres; and all domestic extensions", and

ii) "all applications with a site area greater than 1 ha" in Flood Zone 2.

Product 4 includes the following information:

Ordnance Survey 1:25k colour raster base mapping;

Flood Zone 2 and Flood Zone 3;

Relevant model node locations and unique identifiers (for cross referencing to the water levels, depths and flows table);

Model extents showing *defended* scenarios;

FRA site boundary (where a suitable GIS layer is supplied);

Flood defence locations (where available/relevant) and unique identifiers; (supplied separately)

Flood Map areas benefiting from defences (where available/relevant);

Flood Map flood storage areas (where available/relevant);

Historic flood events outlines (where available/relevant, not the Historic Flood Map) and unique identifiers;

Statutory (Sealed) Main River (where available within map extents);

A table showing:

i) Model node X/Y coordinate locations, unique identifiers, and levels and flows for *defended* scenarios.

ii) Flood defence locations unique identifiers and attributes; (supplied seperately)

iii) Historic flood events outlines unique identifiers and attributes; and

iv) Local flood history data (where available/relevant).

Please note:

If you will be carrying out computer modelling as part of your Flood Risk Assessment, please request our guidance which sets out the requirements and best practice for computer river modelling.

This information is based on that currently available as of the date of this letter. You may feel it is appropriate to contact our office at regular intervals, to check whether any amendments/ improvements have been made. Should you re-contact us after a period of time, please quote the above reference in order to help us deal with your query.

This information is provided subject to the enclosed notice which you should read.

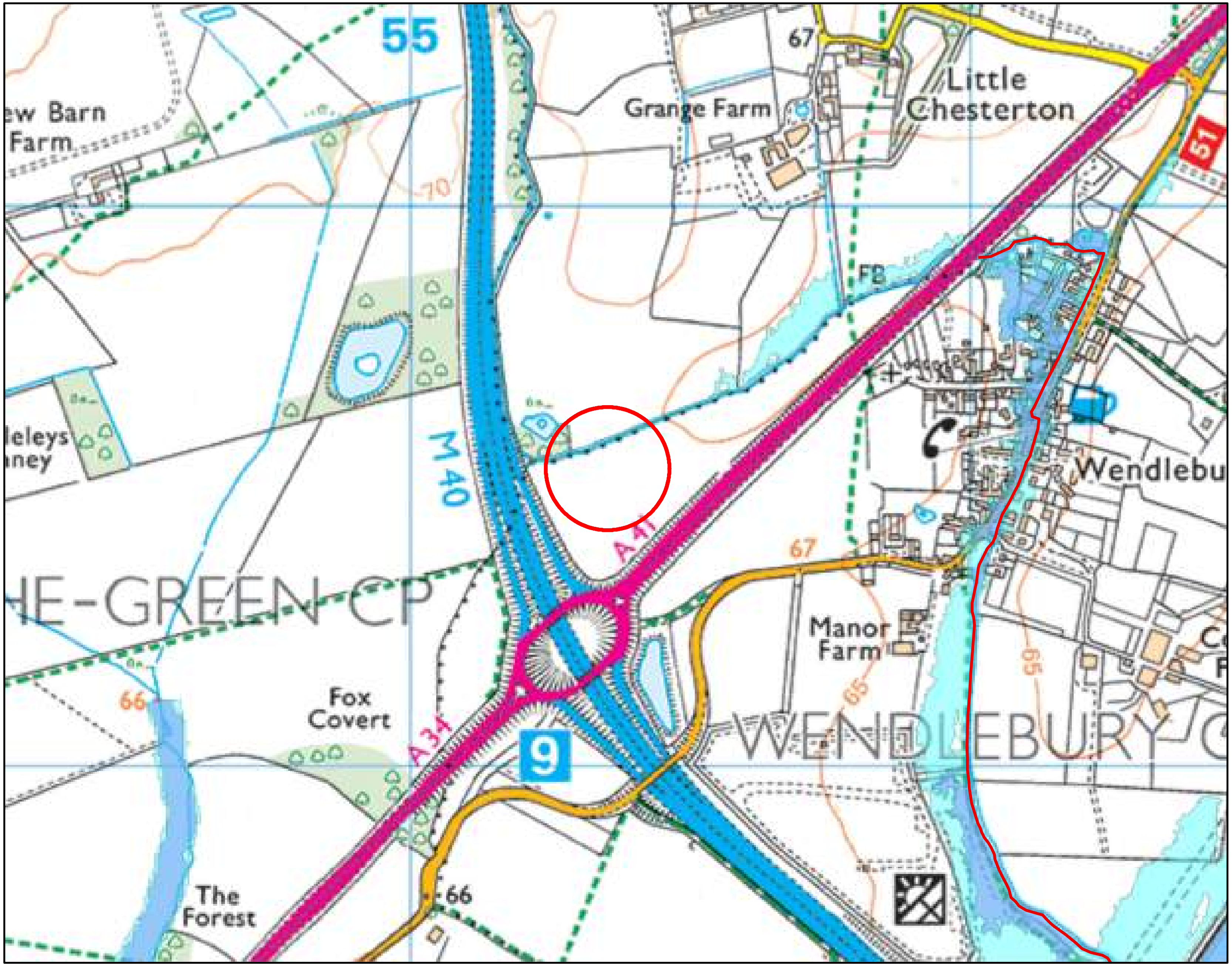
This letter is not a Flood Risk Assessment. The information supplied can be used to form part of your Flood Risk Assessment. Further advice and guidance regarding Flood Risk Assessments can be found on our website at:

https://www.gov.uk/guidance/flood-risk-assessment-local-planning-authorities

If you would like advice from us regarding your development proposals you can complete our pre application enquiry form which can be found at:

https://www.gov.uk/government/publications/pre-planning-application-enquiry-form-preliminary-opinion

Flood Map for Planning centred on OX25 3QE
Created on 8/6/2021 REF: THM220769



Environment Agency

Kilometres

0 0.25 0.5

Legend

- Main River
- Flood defences
- Areas benefiting from flood defences
- Flooding from rivers or sea (FZ3)
- Extent of extreme flood (FZ2)
- Flood Map - flood storage areas

Flooding from rivers or sea without defences (Flood Zone 3) shows the area that could be affected by flooding:
- from the sea with a 1 in 200 or greater chance of happening each year
- or from a river with a 1 in 100 or greater chance of happening each year.

The Extent of an extreme flood (Flood Zone 2) shows the extent of an extreme flood from rivers or the sea with up to a 1 in 1000 chance of occurring each year.

Contact Us: National Customer Contact Centre, PO Box 544, Rotherham, S60 1BY. Tel: 03708 506 506 (Mon-Fri 8-6). Email: enquiries@environment-agency.gov.uk

Defence information

THM220769

Defence Location:	No defences on Main River
Description:	This location is not currently protected by any formal defences and we do not currently have any flood alleviation works planned for the area. However we continue to maintain certain watercourses and the schedule of these can be found on our internet pages.

Model information

THM220769

Model: Wendlebury Brook 2014

Description: The information provided is from the Wendlebury Brook Flood Mapping Study completed in April 2014. The study was carried out using 2D modelling software (ISIS-Tuflow).

Model design runs:
1 in 5 / 20% Annual Exceedance Probability (AEP); 1 in 10 / 10% AEP; 1 in 20 / 5% AEP; 1 in 30 / 3.3% AEP; 1 in 50 / 2% AEP; 1 in 75 / 1.33% AEP; 1 in 100 / 1% AEP, 1 in 100+20% / 1% AEP plus 20% increase in flows and 1 in 1000 / 0.1% AEP.

Mapped outputs:
1 in 5 / 20% AEP; 1 in 10 / 10% AEP; 1 in 20 / 5% AEP; 1 in 30 / 3.3% AEP; 1 in 50 / 2% AEP; 1 in 75 / 1.33% AEP; 1 in 100 / 1% AEP and 1 in 1000 / 0.1% AEP.

Model accuracy:
Levels ± 250mm

Modelled in-channel flood flows and levels

THM220769

The modelled flood levels and flows for the closest most appropriate model node points for your site that are within the river channel are provided below:

Node label	Model	Easting	Northing	Flood Levels (mAOD) 20% AEP	5% AEP	1% AEP	1% AEP (+20% increase in flows)	1% AEP (+25% increase in flows)	1% AEP (+35% increase in flows)	1% AEP (+70% increase in flows)	0.1% AEP
061_14_2014_001WB_2872_MN	Wendlebury Brook 2014	455170	219657	0.00	0.00	66.81	66.87	0.00	0.00	0.00	67.02
061_14_2014_001WB_2733_MN	Wendlebury Brook 2014	455231	219548	0.00	0.00	66.35	66.41	0.00	0.00	0.00	66.56
061_14_2014_001WB_2599_MN	Wendlebury Brook 2014	455357	219583	0.00	0.00	65.89	65.95	0.00	0.00	0.00	66.09
061_14_2014_001WB_2457_MN	Wendlebury Brook 2014	455489	219637	0.00	0.00	65.38	65.45	0.00	0.00	0.00	65.59
061_14_2014_001WB_2320_MN	Wendlebury Brook 2014	455614	219686	0.00	0.00	65.04	65.10	0.00	0.00	0.00	65.17

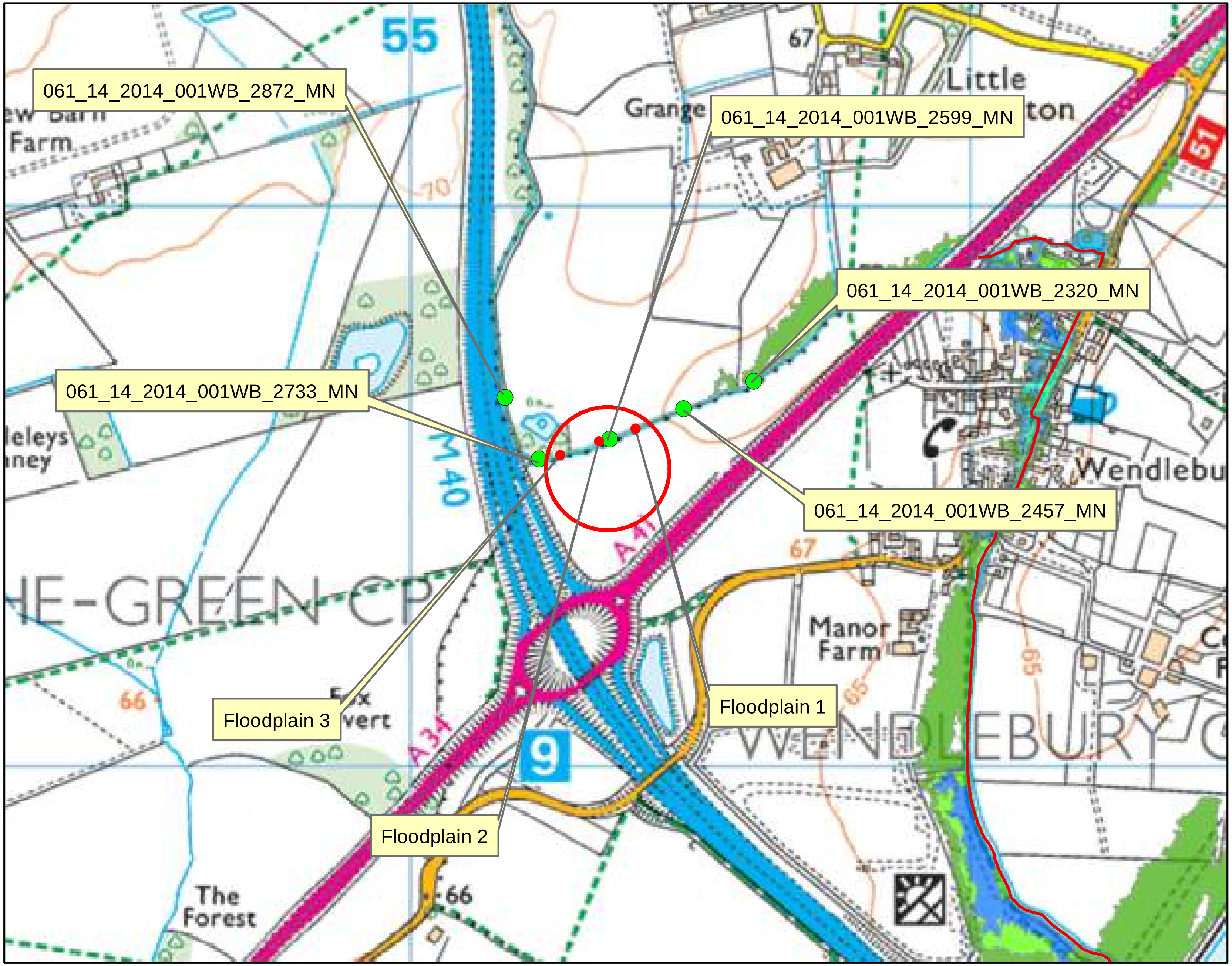
Node label	Model	Easting	Northing	Flood Flows (m3/s) 20% AEP	5% AEP	1% AEP	1% AEP (+20% increase in flows)	1% AEP (+25% increase in flows)	1% AEP (+35% increase in flows)	1% AEP (+70% increase in flows)	0.1% AEP
061_14_2014_001WB_2872_MN	Wendlebury Brook 2014	455170	219657	0.00	0.00	0.82	0.98	0.00	0.00	0.00	1.44
061_14_2014_001WB_2733_MN	Wendlebury Brook 2014	455231	219548	0.00	0.00	0.88	1.05	0.00	0.00	0.00	1.55
061_14_2014_001WB_2599_MN	Wendlebury Brook 2014	455357	219583	0.00	0.00	0.94	1.13	0.00	0.00	0.00	1.66
061_14_2014_001WB_2457_MN	Wendlebury Brook 2014	455489	219637	0.00	0.00	1.00	1.20	0.00	0.00	0.00	1.77
061_14_2014_001WB_2320_MN	Wendlebury Brook 2014	455614	219686	0.00	0.00	1.07	1.28	0.00	0.00	0.00	1.82

Note:

Due to changes in guidance on the allowances for climate change, the 20% increase in river flows should no longer to be used for development design purposes.
The data included in this Product can be used for interpolation of levels as part of an intermediate level assessment.

For further advice on the new allowances please visit
https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

Detailed FRA Map centred on OX25 3QE
Created on 8/6/2021 REF: THM220769



Environment Agency

Kilometres

0 0.25 0.5

Legend

- Main River
- Model Nodes
- 20% AEP Flood Outline
- 5% AEP Flood Outline
- 1% AEP Flood Outline
- 1%+20% AEP Flood Outline
- 0.1% AEP Flood Outline

Floodplain Node

AEP = Annual Exceedance Probability
The probability of a flood of a particular magnitude, or greater, occuring in any given year

Where available climate change extents have been calculated with an additional flow added to an AEP event. An example of how this is written is 1%+20% AEP.

Contact Us: National Customer Contact Centre, PO Box 544, Rotherham, S60 1BY. Tel: 03708 506 506 (Mon-Fri 8-6). Email: enquiries@environment-agency.gov.uk

Modelled floodplain flood levels

THM220769

The modelled flood levels for the closest most appropriate model grid cells for your site are provided below:

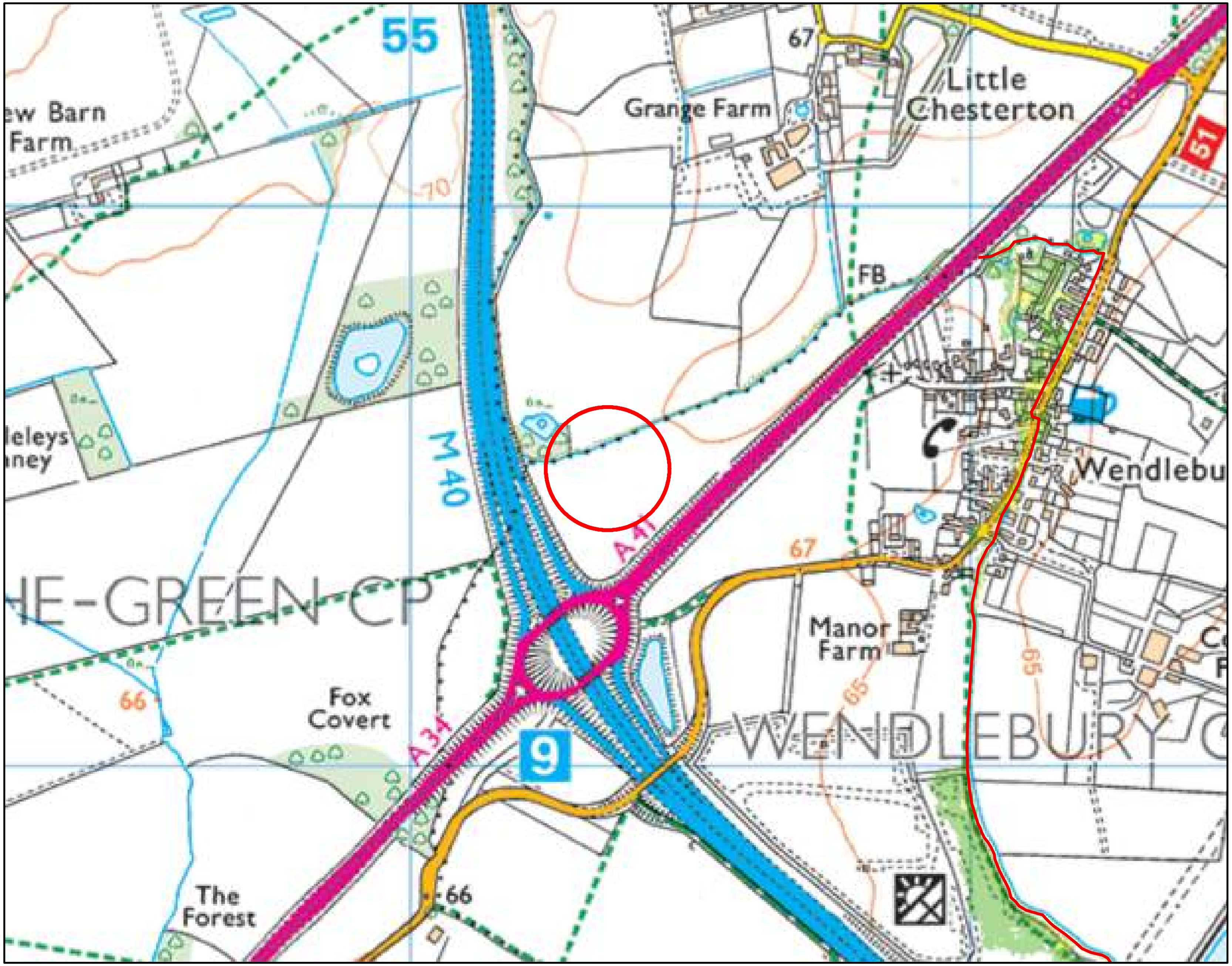
				flood levels (mAOD)				
2D grid cell reference	Model	Easting	Northing	20% AEP	5% AEP	1% AEP	1% AEP (+20% increase in flows)	0.1% AEP
Floodplain 1	Wendlebury Brook 2014	455,404	219,603	65.48	65.58	65.70	65.77	65.90
Floodplain 2	Wendlebury Brook 2014	455,342	219,581	65.72	65.81	65.94	66.00	66.14
Floodplain 3	Wendlebury Brook 2014	455,272	219,559	no data	no data	no data	no data	66.40

This flood model has represented the floodplain as a grid.
The flood water levels have been calculated for each grid cell.

Note:
Due to changes in guidance on the allowances for climate change, the 20% increase in river flows should no longer to be used for development design purposes. The data included in this Product can be used for interpolation of levels as part of an intermediate level assessment.

For further advice on the new allowances please visit
https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

Hazard Map centred on OX25 3QE
Created on 8/6/2021 REF: THM220769



Environment Agency

Kilometres

0 0.25 0.5

Legend

- Main River
- Low hazard
- Danger for some
- Danger for most
- Danger for all

For hazard and debris factor we used HR Wallingford and Environment Agency (May 2008) supplementary note on flood hazard ratings and thresholds for development planning and control purpose. The following calculation is used:

HR = d x (v+0.5) + DF
HR = flood hazard rating
d = depth of flooding (m)
v = velocity of floodwaters (m/sec)
DF = debris factor calculated (0, 0.5, 1 depending on probability that debris will lead to a hazard)

Contact Us: National Customer Contact Centre, PO Box 544, Rotherham, S60 1BY. Tel: 03708 506 506 (Mon-Fri 8-6). Email: enquiries@environment-agency.gov.uk

Hazard Mapping (for the 1%+35% climate change scenario) THM22076

Hazard Mapping methodology:

To calculate flood hazard with the debris factor we have used the supplementary note to Flood Risk to People Methodology (see below).
The following calculation is used:

$HR = d \times (v+0.5) + DF$

Where HR = flood hazard rating
d = depth of flooding (m)
v = velocity of floodwaters (m/sec)
DF = debris factor calculated (0, 0.5, 1 depending on probability that debris will lead to a hazard)

The resultant hazard rating is then classified according to:

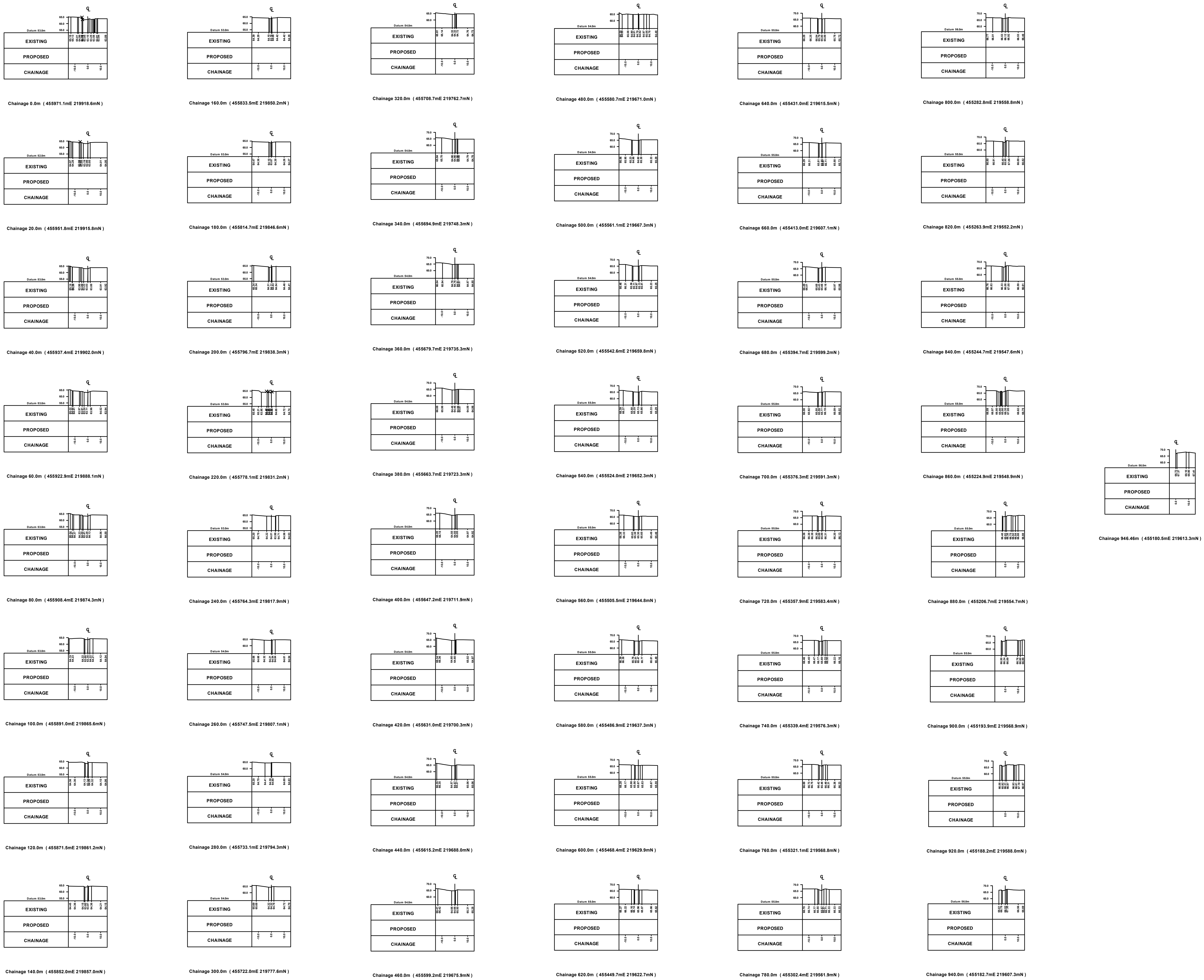
Flood Hazard	Colour	Hazard to People Classification	
Less than 0.75		Very low hazard	- Caution
0.75 to 1.25		Danger for some	- includes children, the elderly and the infirm
1.25 to 2.0		Danger for most	- includes the general public
More than 2.0		Danger for all	- includes the emergency services

REF: HR Wallingford and Environment Agency (May 2008) Supplementary note of flood hazard ratings and thresholds for development planning and control purpose – Clarification of the Table 113.1 of FD2320/TR2 and Figure 3.2 of FD2321/TR1

Red Kite House, Howbery Park, Wallingford, Oxon OX10 8BD
Customer services line: 08708 506 506
Email: WTenquiries@environment-agency.gov.uk

Appendix C – Proposed Site Layout

Appendix D – Topographical Survey



N

Notes :

This plan should only be used for its original purpose. SV Surveying Ltd accepts no responsibility for this plan if supplied to any other party other than the original client.

All dimensions / levels should be checked on site prior to design and construction.

Drainage information (where applicable) has been visually inspected from the surface and should be treated as approximate only.

Tree information (where applicable) has been surveyed from ground level and therefore should be treated as approximate only.

Contours are drawn at 0.2m intervals

The survey has been fixed to GPS OSGB 1936 using the active GPS network.

Legend:

LAND SURVEYS + COMPUTER MODELLING
BUILDING SURVEYS + SITE ENGINEERING

SV SURVEYING LTD

76B MARKET STREET
ASHBY-DE-LA-ZOUCH
LEICESTERSHIRE
LE65 1AP

Tel 01530 560837
Fax 01530 560123
Email: info@svsurveying.co.uk
www.svsurveying.co.uk

Project

TIER UK
NORTH OXFORD

Drawing

SECTIONS
SITE SURVEY

Scales	Drawn/Paper Size
SCALE	AB/A0
Survey Date	Drawing No
25/05/2021	21103-21-01

DO NOT SCALE

N

Notes :

This plan should only be used for its original purpose. SV Surveying Ltd accepts no responsibility for this plan if supplied to any other party other than the original client.

All dimensions / levels should be checked on site prior to design and construction.

Drainage information (where applicable) has been visually inspected from the surface and should be treated as approximate only.

Tree information (where applicable) has been surveyed from ground level and therefore should be treated as approximate only.

Contours are drawn at 0.2m intervals

The survey has been fixed to GPS OSGB 1936 using the active GPS network.

Legend:

	Building Canopy / Overhang
	Buildings / walls
	Kerb line
	Concrete edge
	Centre Line
	Barrier
	Hedge
	Security Fence
	Panel Fence
	Open Fence
	Overhead Powerline
	Chambers
STN1	Survey Station & Name
BM 100.00	Bench Mark
TBM 100.00	Temporary Bench Mark
SAP	Tree / Sapling
	Area Of Undergrowth
	Gate
IC	Inspection chamber
CL	Cover level
IL	Invert level
Inv Ø0.25	Pipe invert (diameter)
Gy	Gully
Bg	Back Gully
MH	Manhole

Trig	Trig Pillar
Ridg	Ridge Level
Eave	Eave Level
DPC	Dpc Level
FL	Floor Level
Soff	Soffit Level
T/Bo	Control Box
LP	Lamp Post
SP	Sign Post
post	Post
FS	Flag Staff
Mrk	Gas Marker
EP	Electric Post
TP	Telegraph Post
invt	Pipe Invert
GU	Gully
RP	Rodding Point
BT	Bt Cover
CO	Cover Unknown
EC	Electricity Cover
FH	Fire Hydrant
GAS	Gas Cover
SV	Stop Valve
TV	Tv Cover
WM	Water Meter
BB	Bellisha Beacon
bol	Bollard
BS	Bus Stop
ER	Earthing Rod
Box	Post Box
MS	Mile Stone
TL	Traffic Light
GV	Gas Valve
peg	Peg

LAND SURVEYS + COMPUTER MODELLING
BUILDING SURVEYS + SITE ENGINEERING

SV SURVEYING LTD
76B MARKET STREET
ASHBY-DE-LA-ZOUCH
LEICESTERSHIRE
LE65 1AP

Tel 01530 560837
Fax 01530 560123
Email: info@svsurveying.co.uk
www.svsurveying.co.uk

Project

TIER UK NORTH OXFORD

Drawing

TOPOGRAPHIC SITE SURVEY

Scales	Drawn/Paper Size
NOT TO SCALE	AB/A0
Survey Date	Drawing No
25/05/2021	21103-21-01

DO NOT SCALE

Appendix E – Hydraulic Modelling Report

Technical Note

KRS Environmental

November 2021

Symmetry Park Oxford North Hydraulic Modelling

Project	Oxford North Hydraulic Modelling
Project Number	WHS1869
Title	**Symmetry Park Oxfo rd North Hydraulic Modelling**
Description	This technical note details the updates made to the baseline model of the Wendlebury Brook and Gagle Brook and quantifies the impact on flood risk of a proposed development on a greenfield site, using hydraulic modelling
Prepared by	Clement Ehall *(Consultant)*
Reviewed by	Brett Park *(Principal Consultant)*
Date	November 2021
Version	1.1

1 Introduction

1.1 Background

Wallingford Hydrosolutions Ltd has been commissioned by KRS Environmental to update the existing Wendlebury Brook and Gagle Brook model in support of a Flood Risk Assessment (FRA) for a commercial development on an approximate 31 ha greenfield site, to the immediate east of junction 9 on the M40, Wendlebury, Oxfordshire (NGR: SP 55368 19504) as shown in Figure 1. This document contains the technical method related to the modelling and results, this should be read in conjunction with the original model report[1], and FRA which will be submitted separately by KRS Environmental.

Figure 1 - Site Location

[1] Wendlebury Brook Flood Risk Mapping Study Final Report, IMSE500106, JBA Consulting on behalf of the EA, 2014

1.2 Scope

The site is situated partially in flood zone 2, with the risk being fluvial from the Wendlebury Brook which runs through the site. The proposed development is to consist of realignment of the Wendlebury Brook and cut and fill earthworks to reprofile ground levels. This hydraulic modelling study is required to quantify the risk to the proposed development and assess the impacts of realigning the Wendlebury Brook.

In summary, this technical note will summarise the following:

- Local review of the existing Environment Agency (EA) model.
- Updates made to the baseline model.
- Results of baseline modelling.
- Approach to modelling the channel realignment.
- Impacts of the proposed development on flood risk.

1.3 Approach

This study utilises a hydraulic model of the Wendlebury Brook and Gagle Brook[2] supplied by the Environment Agency (EA) to assess the flood risk to the site. The baseline model has been updated to use the latest software version and climate change allowances to be suitable for use in this study.

The development proposals are then incorporated into the model and the post-development results are compared to the baseline to quantify impacts on flood risk.

[2] Wendlebury Brook Flood Risk Mapping Study, IMSE500106, JBA Consulting on behalf of the EA, 2014

2 Wendlebury Brook and Gagle Brook Model (2014)

2.1 Baseline Updates

In total three key updates were made to the baseline model;

- Updated software version,
- Updated climate change allowances,
- Updated 1D panel markers.

Both the 1D and 2D software versions were updated to use the best available software for the estimation of flood levels at the site. These are summarised in Table 1.

Table 1 - Software Version Updates

Software	Previous Version	Updated Version
Flood Modeller (1D)	ISIS V6.6	Flood Modeller 5.0
TuFLOW (2D)	2013-12-AA	2020-10-AA

The previous approach for climate change has been superseded in 2021 by the new national climate change guidance, dependant on which management catchment the catchment falls in. The Cherwell and Ray management catchment is within the Thames River basin district. The Central climate change allowance for this catchment is +15% ("less vulnerable" development), replacing the +20% value used in the 2014 model.

The 1.0% AEP + 15CC event uses the central climate change estimate. The hydrographs for this event were created by increasing the 'fit to peak of value' in the Flood Modeller ReFH boundaries for the 1.0% AEP event by 15%. For the WB03_IA hydrograph, the 'by a factor of' method was used instead of the to 'fit to peak of' method, with the value increased as described for the other hydrographs above. The 'fit to peak of' and 'by a factor' values used for the 1.0% AEP and 1.0% AEP + 15CC events can be seen in Table 2.

Table 2 - Climate Change Peak Flow Updates

Hydrograph Node Label	1.0% AEP Peak Flow	1.0% AEP + 15CC Peak Flow
WB01a	0.380	0.437
WB02a	0.830	0.955
TRIB01	0.610	0.702
WB03_IA	2.000*	2.300*
GB01	2.190	2.519

**Scale factor used instead of peak flow fit.*

It was identified that some 1D Flood Modeller nodes had decreasing conveyance at bank level, as such the panel marks for these nodes were updated, to ensure conveyancy increased with river stage.

In addition to the updates specified above, the model timestep was increased from 0.5s to 1.0s in Flood Modeller. This was done to improve the stability of the model. Some minor updates were also made to baseline model TuFLOW files, a description of the updates made to the baseline model are specified in Table 3 below.

Table 3 - Summary of baseline model updates

Updated Files	Description	Descriptions of Updates
WBs_~s1~_~s2~_e1~_003.tcf		
Global Settings	GIS Format	GIS format updated to use shapefiles.
	Simulation management	TCF updated to use event and scenarios functionality, so all model runs read the same TCF file.
	File locations	Check, Result and Log files location updated to use simulation scenario and event name in the folder structure.
	Output intervals	Map Output interval updated from 900s to 300s. Time Series Output interval updated from 120s to 900s Screen Displace interval. updated from 10s to 30s.
	Map Output Data Types	MB1D, MB, MB2D and z9 outputs removed. MB1 output added.
Projection.shp	Shapefile setting the projection system to British National Grid (OSGB 1936).	Projection is now set from shapefile instead of MIF file projection.
1D_ND_ISIS_08.shp	1D FMP Node Locations	File updated from MIF to SHP format.
1d_x1d_ISIS_nwk_WB_04.shp	1D FMP Network	File updated from MIF to SHP format.
1d_WLL_03.shp	1D FMP Water Level Lines	File updated from MIF to SHP format.
2d_po_WB.shp	PO line	File updated from MIF to SHP format.
WB_21_SLext_with_SP_002.tgc		
2d_loc_WB.shp	2D grid location and orientation	File updated from MIF to SHP format.
2d_code_WB_01.shp	2D active cells, Wendlebury Brook	File updated from MIF to SHP format.
2d_code_GB_extended_2.shp	2D active cells, Gagle Brook	File updated from MIF to SHP format.
2d_code_WB_1d_05.shp	2D de-active cells, Wendlebury Brook	File updated from MIF to SHP format.
2d_code_GB_1d_02.shp	2D de-active cells, Gagle Brook	File updated from MIF to SHP format.
2d_zln_bank_levels_03_L.shp | 2d_zln_bank_levels_03_P.shp	Wendlebury Brook bank levels from topographic survey	File updated from MIF to SHP format.

2d_zsh_WB_Road_banks_02_R.shp | 2d_zsh_WB_Road_banks_02_P.shp	Road bank levels from topographic survey	File updated from MIF to SHP format.
2d_zln_GB_Topbanks_survey_L.shp | 2d_zln_GB_Topbanks_survey_P.shp	Gagle Brook bank levels from topographic survey	File updated from MIF to SHP format.
2d_zln_Ch2_top_of_bank_L.shp | 2d_Ch2_top_of_bank_P.shp	Channel 2 bank levels from topographic survey	File updated from MIF to SHP format.
2d_zsh_Topo_Check_Points_R.shp | 2d_zsh_Topo_Check_Points_P.shp	Topographic survey check points behind houses at Rectory Close	File updated from MIF to SHP format.
2d_zln_bank_levels_North_Rectory_Close_03_L.shp | 2d_zln_bank_levels_North_Rectory_Close_03_P.shp	Wendlebury Brook bank levels north of Rectory Close, from topographic survey	File updated from MIF to SHP format.
2d_zln_Channel_Bed_behind_Rectory_Close_L.shp | 2d_zln_Channel_Bed_behind_Rectory_Close_P.shp	Channel bed levels along drains behind Rectory Close	File updated from MIF to SHP format.
2d_zln_Road_Level_L.shp | 2d_zln_Road_Level_P.shp	Road levels	File updated from MIF to SHP format.
2d_zln_Gagle_Brook_Channel_Lower_Points_L.shp | 2d_zln_Gagle_Brook_Channel_Lower_Points_P.shp	Not surveyed Gagle Brook channel (2D domain) low point taken from LiDAR	File updated from MIF to SHP format.
2d_zln_bank_levels_Upstream_A41_L.shp | 2d_zln_bank_levels_Upstream_A41_P.shp	Wendlebury Brook bank levels upstream of A41 from topographic survey	File updated from MIF to SHP format.
2d_mat_WB_01.shp	2D materials codes	File updated from MIF to SHP format
2d_mat_stability_patch_03.shp	2D materials stability patch	File updated from MIF to SHP format
WB_16SLext.tbc		
2d_bc_hxi_GB_04.shp	1D 2D link from Gagle Brook and Channel 1	File updated from MIF to SHP format
2d_bc_GB_1d_01.shp	Downstream end of 1D Gagle Brook spilling in the 2D	File updated from MIF to SHP format

2d_bc_GB_downstream_01.shp	Downstream 2D boundary of Gagle Brook	File updated from MIF to SHP format
2d_bc_Railway_boundary.shp	Downstream 2D boundary at the railway line	File updated from MIF to SHP format
2d_bc_hxi_wb_07_L.shp | 2d_bc_hxi_07_P.shp	1D 2D link for the Wendlebury Brook	File updated from MIF to SHP format
Wbs_Events_001.tef		
-	Defines model events.	File added to runs folder and read in by TCF
WB_33.dat/WB_34.dat		
WB_35a.dat/WB_35b.dat	FMP Network	Pannel markers updated to improve conveyance curves

2.2 Impact of Baseline Updates.

The updates made to the baseline model specified above, resulted in some minor changes in flood extent for Wendlebury, with some larger changes seen downstream of Wendlebury and from the Gagle Brook. It is worth noting that the improved conveyance curves resulted in greater conveyance for multiple 1D nodes. The modelled flood extents during the 1.0% AEP event can be seen in Figure 2 and Figure 3.

For the 0.1% AEP runs, two structures continue to be missing, as per the received EA model, this has been reviewed and given their location with respect to the proposed works, their omittance is acceptable and hence has been maintained for stability reasons.

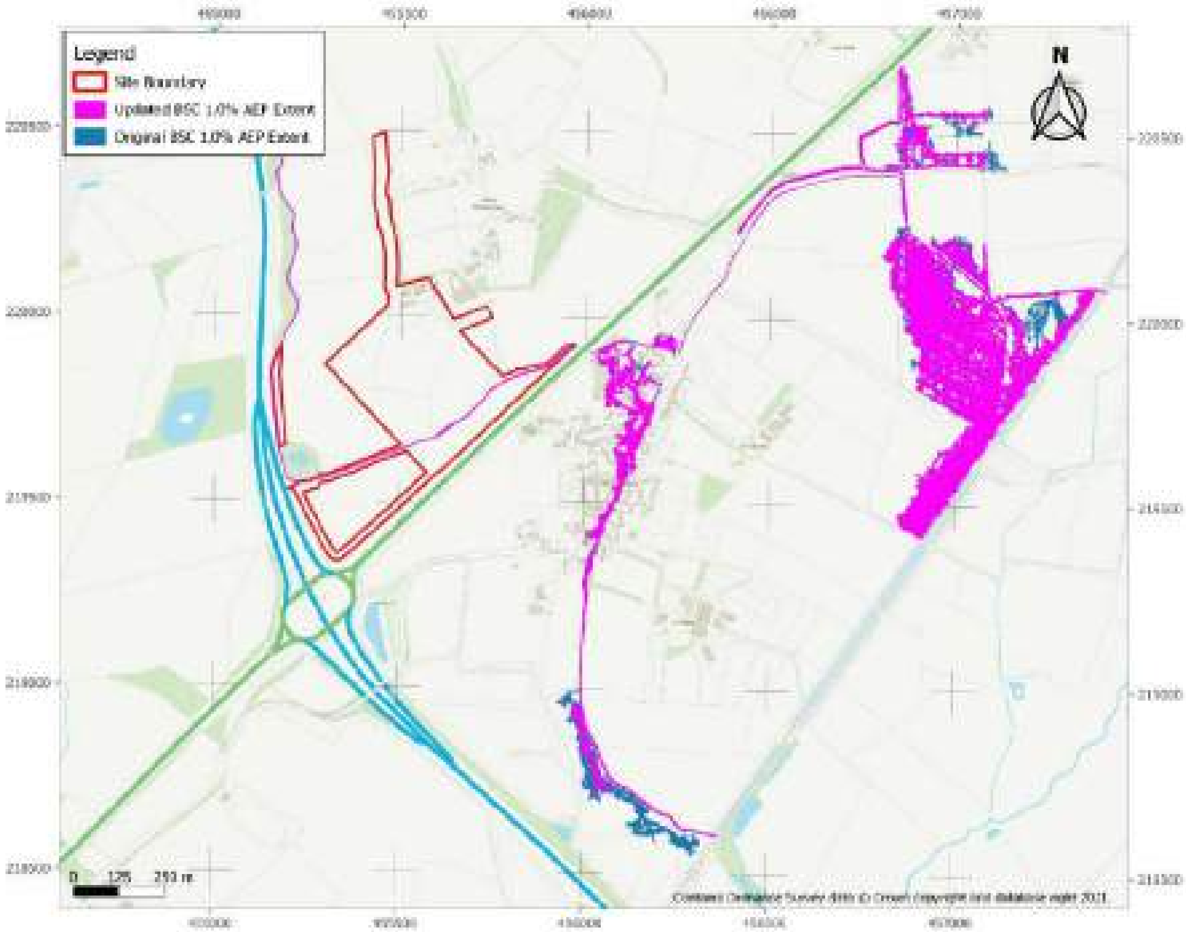


Figure 2 - Original and Updated Baseline Model 1.0% AEP Flood Extents

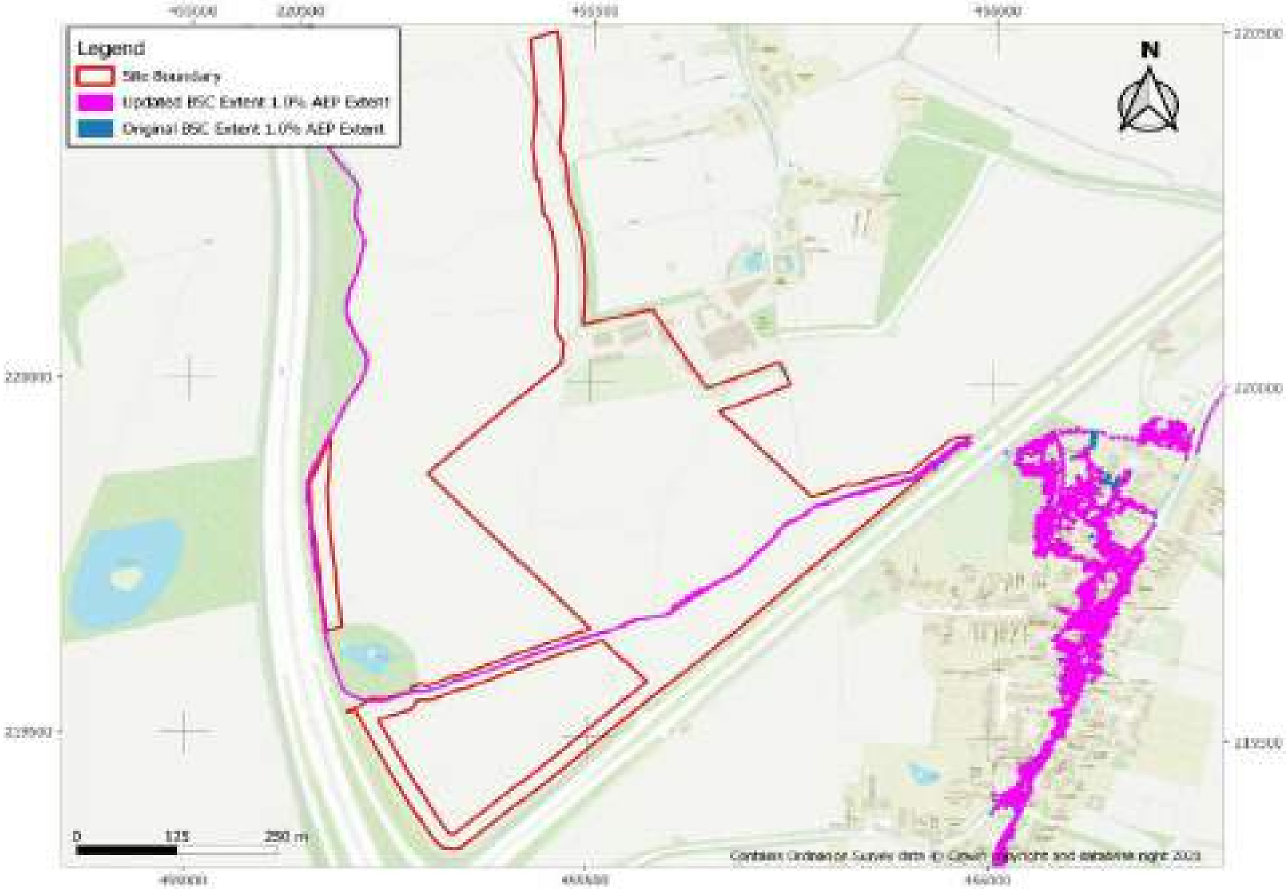


Figure 3 – Original vs Baseline Model 1.0% AEP Extent for the Proposed development site and Wendlebury

3 Baseline Model Results

3.1 1.0% AEP Event

As can be seen in Figure 4, during the 1.0% AEP event there are no significant out of bank flows for the Wendlebury Brook upstream of the A41. As such the site is not at risk of fluvial flooding during the 1.0% AEP baseline event.

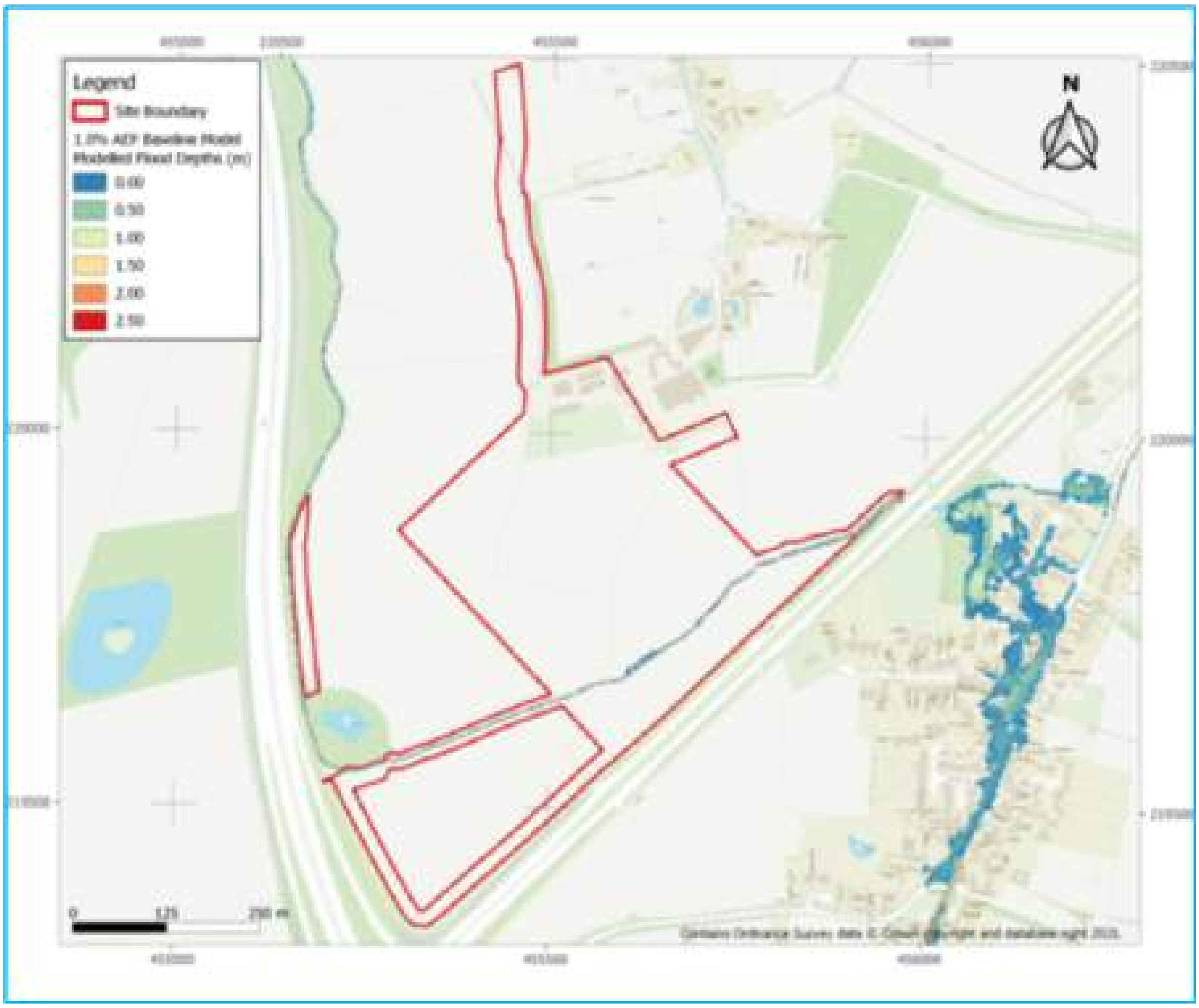


Figure 4 - Baseline 1.0% AEP Modelled Flood Depths

3.2 1.0% AEP + 15% Climate Change (Central) Event

As can be seen in Figure 5, during the 1.0% AEP + 15% climate change event there are no significant out of bank flows within the development site, as in the 1.0% AEP event. As such the site is not at risk of fluvial flooding during the 1.0% AEP + 15% climate change event.

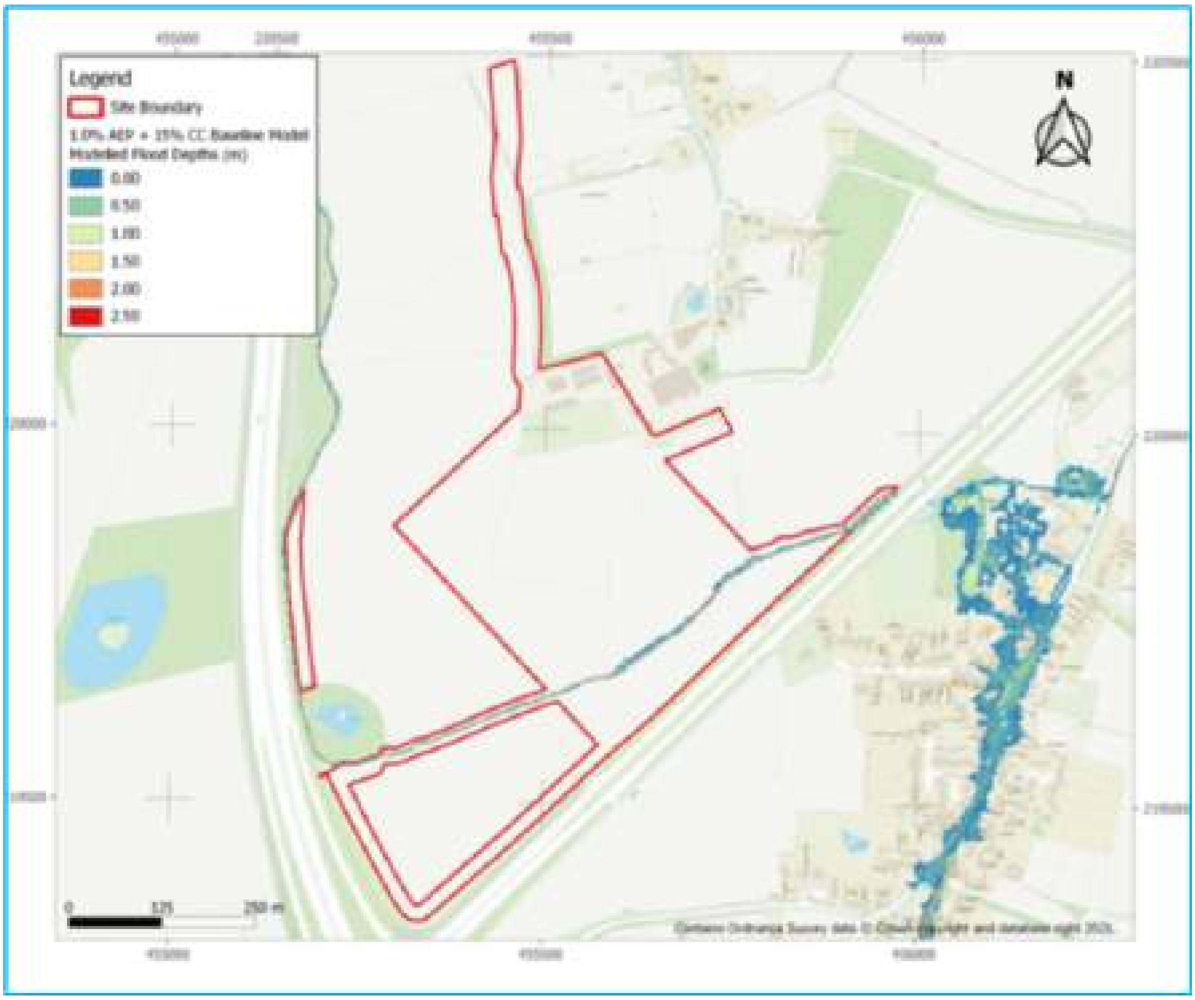


Figure 5 - Baseline 1.0% AEP + 15% Climate Change Modelled Flood Depths

3.3 0.1% AEP Event

As can be seen in Figure 6, during the more extreme 0.1% AEP event some fluvial flooding can be seen within the eastern area of the site. As such some of the site is at risk of fluvial flooding during the 0.1% AEP baseline event.

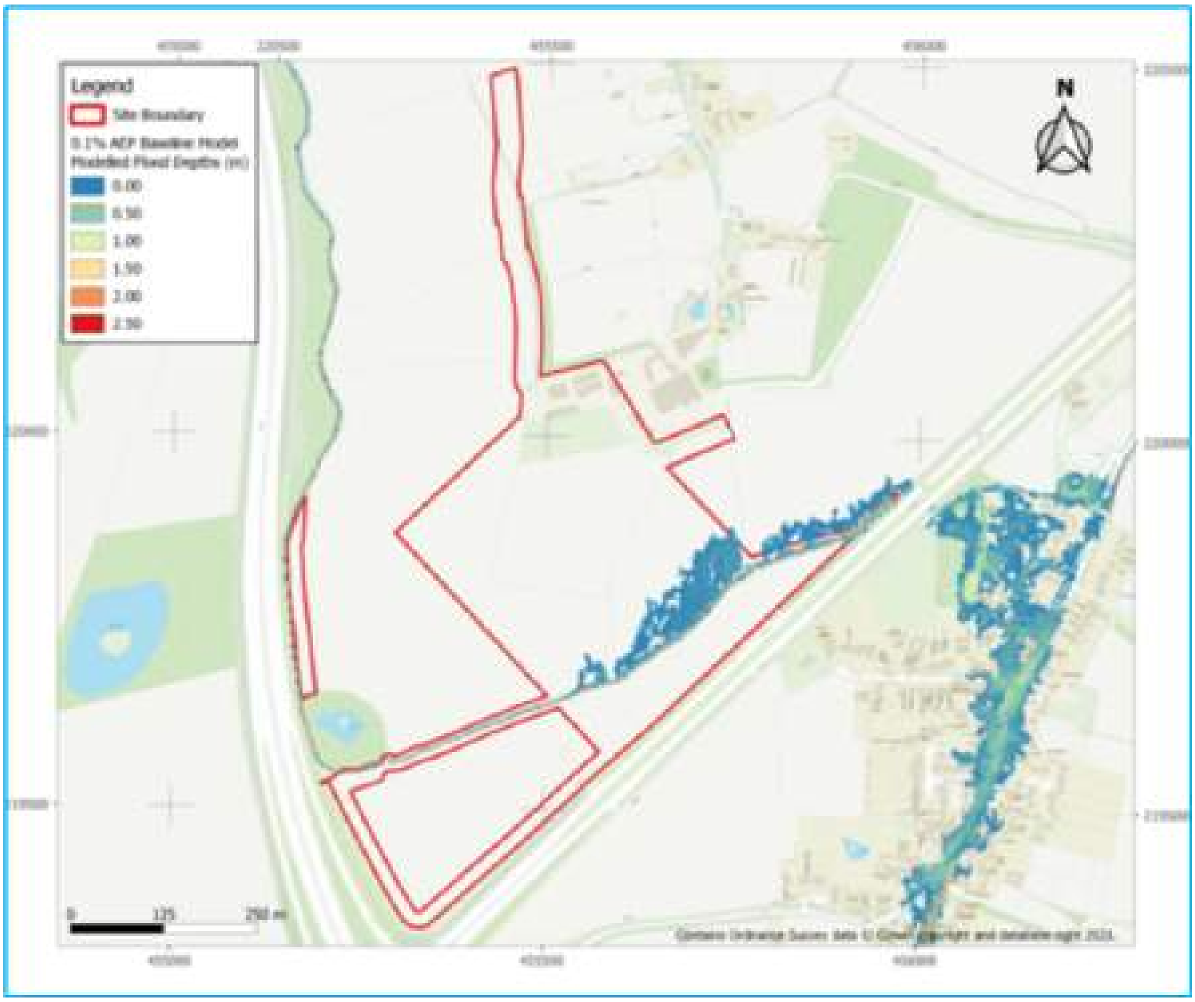


Figure 6 - Baseline 0.1% AEP Modelled Flood Depths

4 Proposed Development

The following sections detail the updates made to the baseline model to create the post-development model. These consisted off:

- Updated 1D river section nodes along the realigned channel
- Updated 1D/2D linkage along the realigned channel
- Updated 2D digital terrain model along the realigned channel
- Updated 2D active and de-active domains along the realigned channel

4.1 FMP Updates

For the development site, it is proposed that an approximate 768m stretch of the Wendlebury Brook, be realigned to the southern boundary of the site. This stretch of the Wendlebury Brook was between model node WB_2733 and WB_1965, the long section of this in the baseline model can be seen in Figure 7 and the georeferenced node locations can be seen in Figure 8.

In the post-development model, the 9 river sections nodes from the 768m stretch of the baseline model between WB_2733 and WB_1965 were replaced with new 8 river section nodes, and 1 interpolate section node with a total chainage for approximately 987m, to represent the realigned Wendlebury Brook. A junction node is used between WB_A-A and WB_A-Ad to connect the inflow from an unnamed tributary, which connects to the Wendlebury Brook within the development site. The realigned channel sections were represented by triangular sections, with the bank and bed levels obtained from the proposed development drawings. In addition to the new river sections, the chainage for WB_2733 was updated to be 33.315m from 133.421m in the baseline model. The CAD for the proposed development can be seen in Appendix 1.

The long section of the realigned channel can be seen in Figure 9, and a typical river cross-section for the realigned channel can be seen in Figure 10, with Table 4 showing the updated river section data.

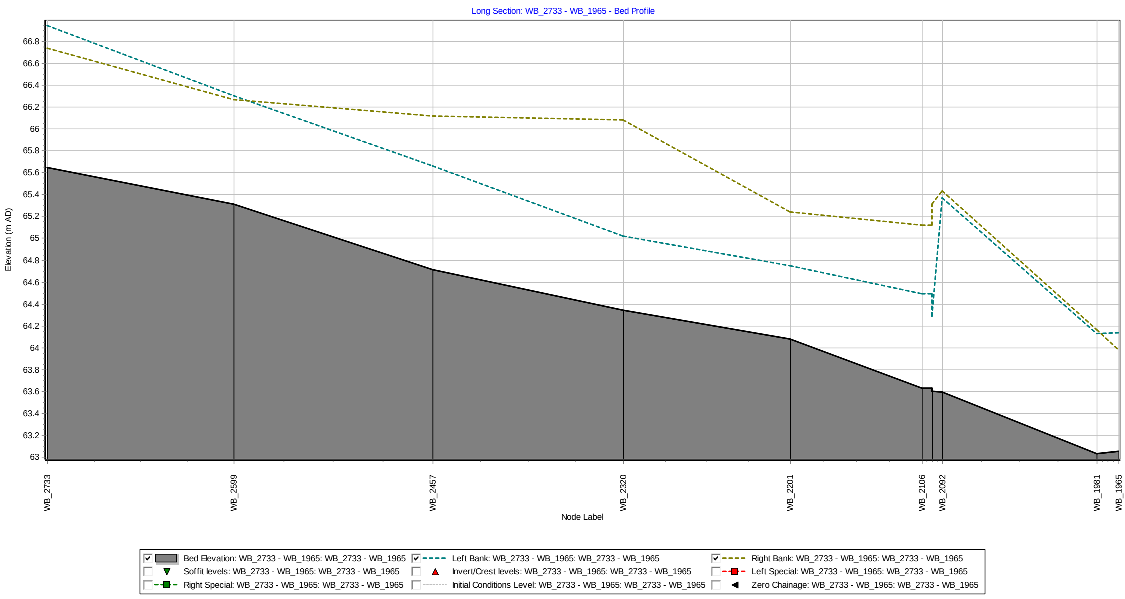


Figure 7 - Baseline Model WB_2733 to WB_1965 Long Section

Figure 8 - Baseline and Post Development models 1D Network and Nodes

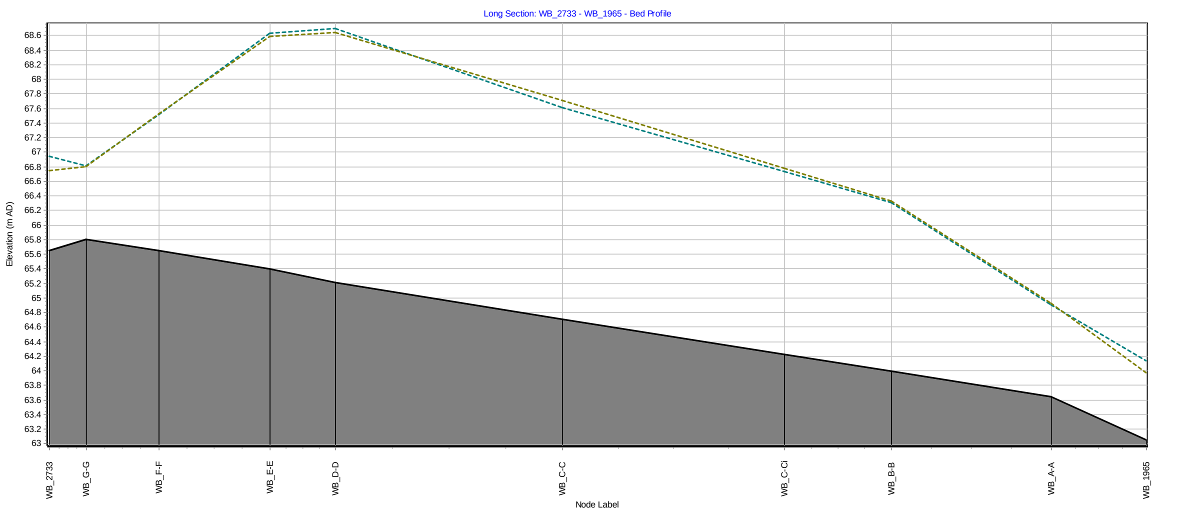


Figure 9 - Post Development Model WB_2733 to WB_1965 Long-section

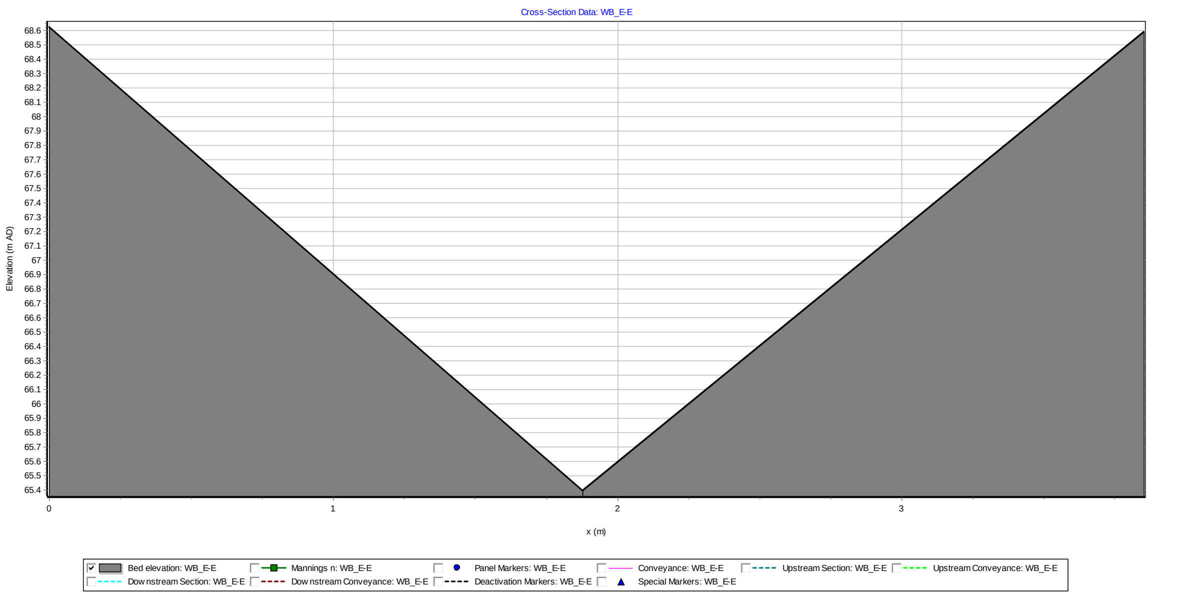


Figure 10 - WB_E-E Cross-Section (Typical Realigned Channel Cross-Section)

Table 4 - Updated Channel Section Data

Node Label	Manning's n	Left Bank Level (mAOD)	Left Bank Chainage (m)	Bed Level (mAOD)	Bed Chainage (m)	Right Bank Level (mAOD)	Right Bank Chainage (m)	Chainage (m)	Lateral Inflow Label
WB_G-G	0.040	66.805	0.000	65.800	1.875	66.800	3.750	64.885	-
WB_F-F	0.040	67.510	0.000	65.644	1.876	67.515	3.752	100.240	WB02a_10
WB_E-E	0.040	68.622	0.000	65.397	1.876	68.588	3.852	59.076	-
WB_D-D	0.040	68.693	0.000	65.203	1.854	68.635	3.730	203.725	-
WB_C-C	0.040	67.610	0.000	64.708	1.957	67.702	3.818	200.000	WB02a_11
WB_C-Ci	Interpolate Section							96.380	WB02a_12
WB_B-B	0.040	66.306	0.000	63.988	2.034	66.324	3.913	143.586	WB02a_13
WB_A-A	0.040	64.900	0.000	63.637	2.168	64.919	3.752	0.000	-
WB-A-Ad	0.040	64.900	0.367	63.637	2.168	64.919	3.752	85.766	-

4.2 TuFLOW Updates

For the post-development model, several updates were made to the TuFLOW model, these consisted of updating the 1D/2D linkage, the digital terrain model (DTM) and the 2D active and de-active domains.

The DTM updates consisted of updating the thick Z shape line files for the bank levels of the Wendlebury Brook upstream of the A41, to follow the bank of the proposed channel. This was done using a zsh line shapefile, with corresponding points file containing elevation levels along the bank of the proposed channel, obtained from the proposed development CAD.

In addition to the DTM updates, the 1D nodes, 1D network and 1D water level lines (WLL) were updated to match the FMP updates. The inactive area in the TuFLOW model was updated to follow the proposed channel alignment in the PDM model, new HX links were added to connect the realigned sections to the TuFLOW model.

A summary of the updates made to the TuFLOW model for the post-development model is in Table 5 and the 2D representation of the baseline and post-development model can be seen in Figure 11.

Table 5 – Post Development Model TuFLOW Changes

Baseline File	Updated Files	Description	Descriptions of Updates
WBs_~s1~_~s2~_e1~_003.tcf			
1D_ND_ISIS_08.shp	*1d_wb_OP3_FMP_Nodes_P_002.shp*	FMP Node Location	Realigned channel nodes added and old channel nodes removed
1d_x1d_ISIS_nwk_WB_04.shp	*1d_x1d_wb_OP3_FMP_nwk_L_002.shp*	FMP channel lines	Updated to connect realigned channel nodes
1d_WLL_03.shp	*1d_WLL_PDM_001.shp*	1D water level lines	Updated to follow realigned channel
WB_21_SLext_with_SP_002.tgc			
2d_code_WB_1d_05.shp	*2d_code_WB_PDM_1d_R_001.shp*	2D inactive area.	Inactive area updated to follow realigned channel extent
2d_zln_bank_levels_Upstream_A41_L.shp | 2d_zln_bank_levels_Upstream_A41_P.shp	*2d_zsh_OP3_Bank_L_001.shp | 2d_zsh_PDM_Bank_P_001.shp*	Bank levels upsteam of A41	Updated to follow realigned channel bank and to use proposed channel bank levels
WB_16SLext.tbc			
2d_bc_hxi_wb_07_L.shp | 2d_bc_hxi_07_P.shp	*2d_bc_hxi_OP3_L_002.shp | 2d_bc_hxi_wb_PDM_P_001.shp*	1D 2D link for the Wendlebury Brook	Updated to follow realigned channel banks

Figure 11 - Baseline and Post Development 2D Representation

5 Mitigation

Following a review of the modelled flooding for the proposed development scenario, it was identified that the proposed design resulted in increased flood depths and extent at the development site. As such mitigation scenario modelling was undertaken to offset these increases in flood risk. The following sections detail the updates made to the proposed development model to create mitigation scenarios.

5.1 Mitigation Scenario 1

For mitigation scenario 1 the bed levels of river section WB_G-G and WB_F-F from the proposed development scenario were lowered, to achieve a more consistent gradient through the proposed channel. The proposed development and mitigation scenario 1 bed levels for sections WB_G-G and WB_F-F can be seen in Table 6 and the updated long-section of the realigned channel can be seen in Figure 12.

Table 6 - Mitigation Scenario 1 Bed Level Updates

Model Node	Proposed Development Bed Level (mAOD)	Mitigation Scenario 1 Bed Levels (mAOD)
WB_G-G	65.800	65.605
WB_F-F	65.644	65.523

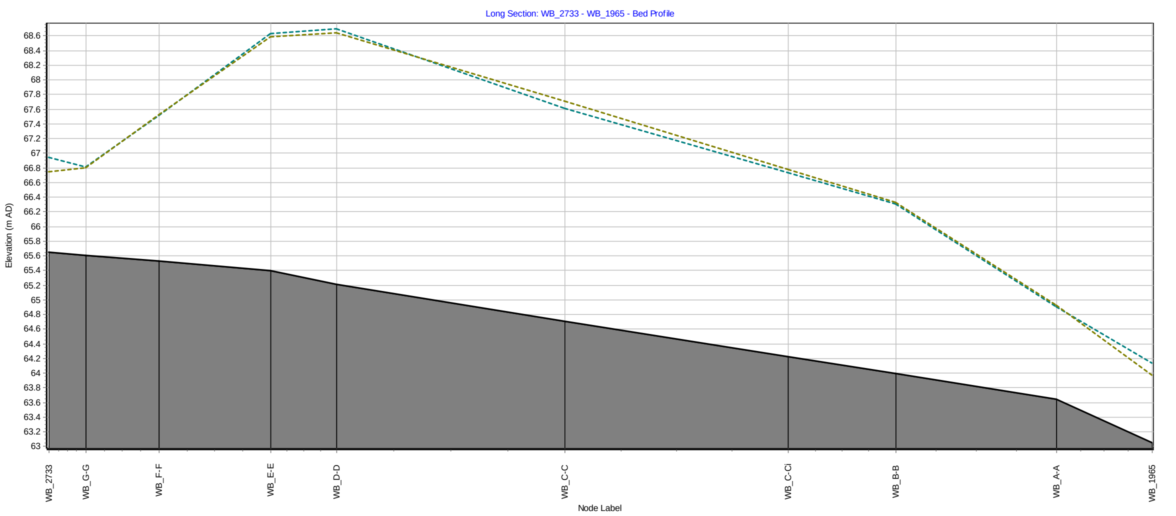


Figure 12 - Mitigation Scenario 1 WB_2733 it WB_1965 Long-section

5.2 Mitigation Scenario 2

This section details the updates made to mitigation scenario 1 to create mitigation scenario 2. The channel cross-sections of the realigned channel were updated to consist of trapezoidal sections, with a bank to bank width of 4.0m and a bed width of 1.5m. The bed and bank levels from mitigation scenario 1 were retained. The typical cross-section for the realigned channel in mitigation scenario 1 can be seen in Figure 13. The details of the updates to the cross-sections of the realigned channel are summarised in Table 7.

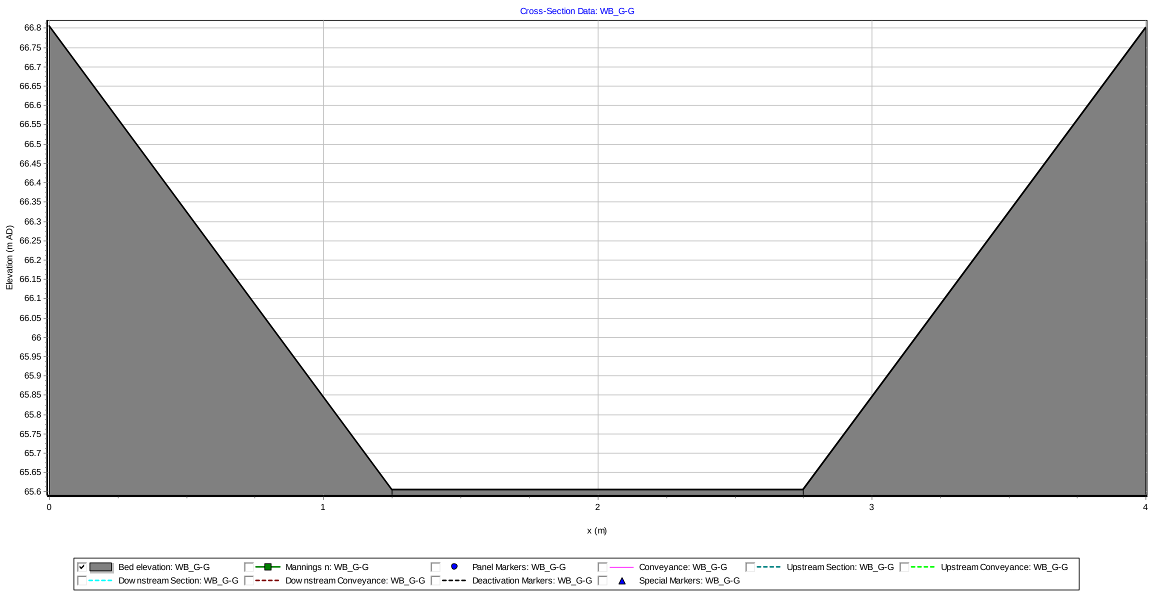


Figure 13 - Mitigation Scenario 2 WB_G-G Cross-Section (Typical realigned channel cross-section)

Table 7 - Scenario 2 Cross-section Width Updates

Model Node	Bank to Bank width Scenario 1 (m)	Change in bank width (Scenario 2 – Scenario 1) (m)
WB_G-G	3.750	0.250
WB_F-F	3.752	0.248
WB_E-E	3.852	0.148
WB_D-D	3.730	0.270
WB_C-C	3.818	0.182
WB_C-Ci	Interpolate Section	
WB_B-B	3.913	0.087
WB_A-A	3.385	0.615
WB_A-Ad	3.385	0.615

5.3 Mitigation Scenario 3

This section details the updates made to mitigation scenario 2 to create mitigation scenario 3. For mitigation scenario 3 the bed levels of the channel were updated from mitigation scenario 2, to ensure the slope at the upstream and downstream extents of the realigned channel are similar to those of the natural channel it replaces at those locations. The realigned channel bed levels for scenarios 2 and 3 can be seen in Table 8. Figure 14 shows the realigned channel long-section for scenario 3.

Table 8 - Mitigation Scenario 3 Updated Bed Levels

Model Node	Scenario 2 Bed Level (mAOD)	Scenario 3 Bed Level (mAOD)	Change in Bed Level (m)
WB_G-G	65.605	65.563	-0.042
WB_F-F	65.523	65.304	-0.219
WB_E-E	65.397	65.049	-0.348
WB_D-D	65.203	64.899	-0.304
WB_C-C	64.708	64.382	-0.326
WB_C-Ci	Interpolate Section		
WB_B-B	63.988	63.630	-0.358
WB_A-A	63.637	63.265	-0.372
WB_A-Ad	63.637	63.265	-0.372

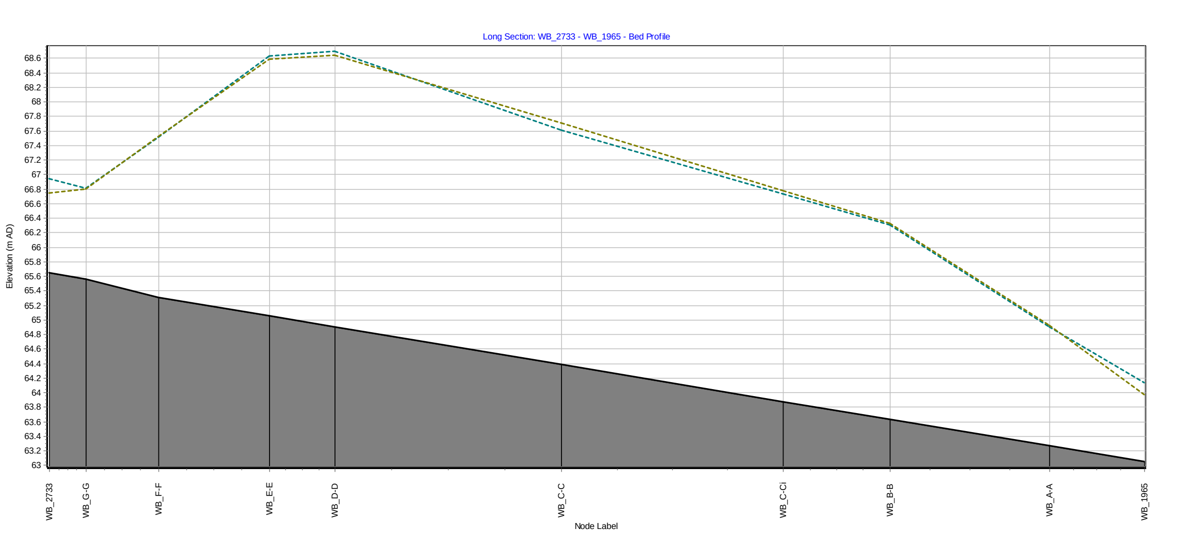


Figure 14 - Mitigation Scenario 3 WB_2733 to WB_1965 Long-section

6 Mitigation Scenario 3 Results

6.1 1.0% AEP Event

The modelled flood depths for the 1.0% AEP flood event can be seen in Figure 15. For mitigation scenario 3 there are no out of bank flows shown for the 1.0% AEP event upstream of the A41 culvert as in the baseline model.

Figure 15 - Mitigation Scenario 3 1.0% AEP Modelled Flood Depths

Figure 16, shows the change in depth plot for mitigation scenario 3 against the baseline. As can be seen below during the 1.0% AEP event there is no modelled impact.

Figure 16 - Mitigation Scenario 3 1.0% AEP Change in Modelled Flood Depth

6.2 1.0% AEP + 15% Climate Change (Central) Event

As can be seen in Figure 17, for the 1.0% AEP + climate change event no out of bank flows are modelled upstream of the A41 culvert, as in the baseline model.

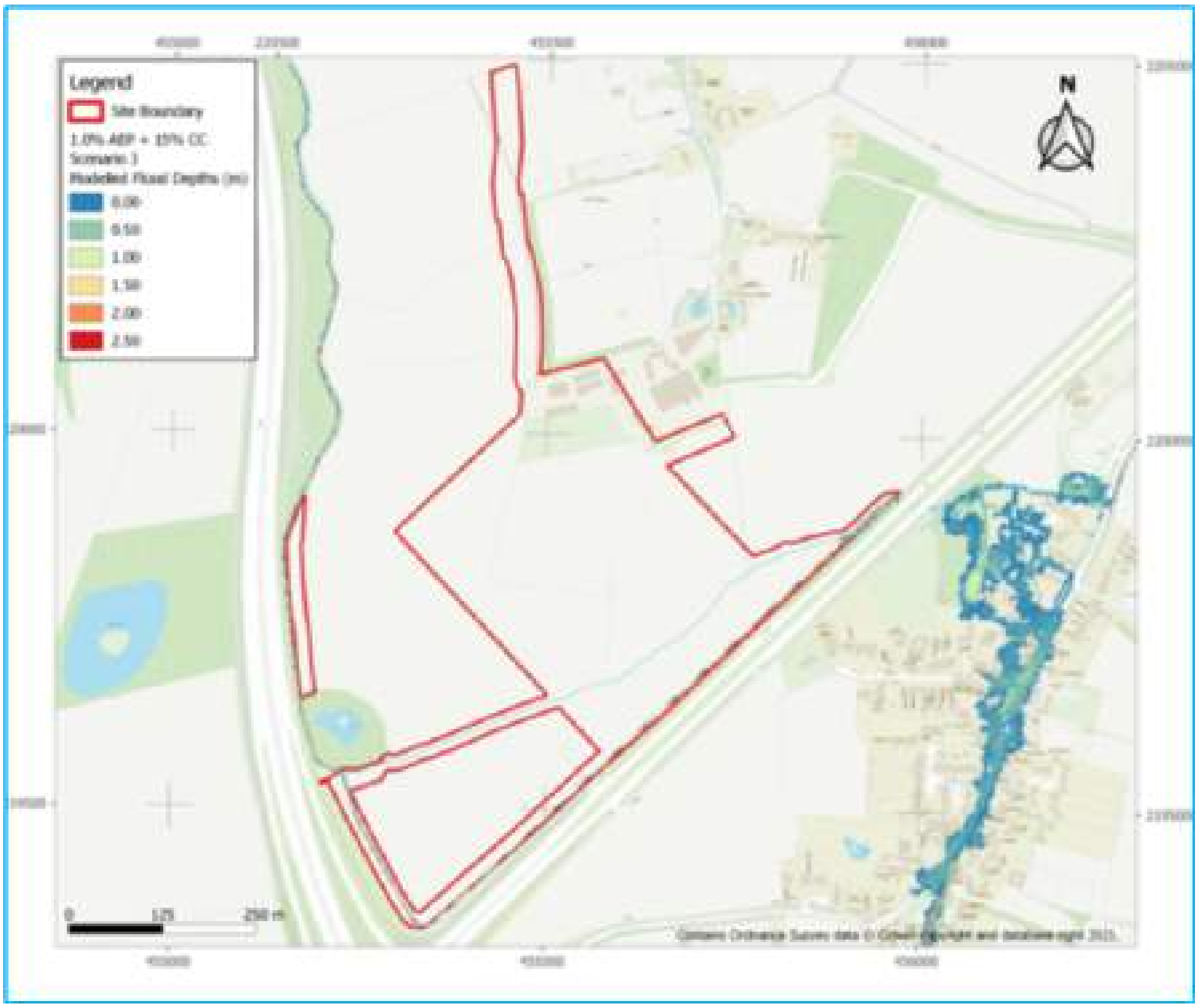


Figure 17 - Mitigation Scenario 3 1.0% AEP + CC Modelled Flood Depths

Figure 18, shows the change in modelled flood depths when compared to the baseline model. As can be seen during the 1.0% AEP + CC there is no modelled impact.

Figure 18 - Mitigation Scenario 3 1.0% AEP + CC Change in Modelled Flood Depth

6.3 0.1% AEP Event

As can be seen in Figure 19, for the 0.1% AEP event a small flood extent just upstream of the A41 culvert can be seen, this is smaller than the extent upstream of the culvert in the baseline model.

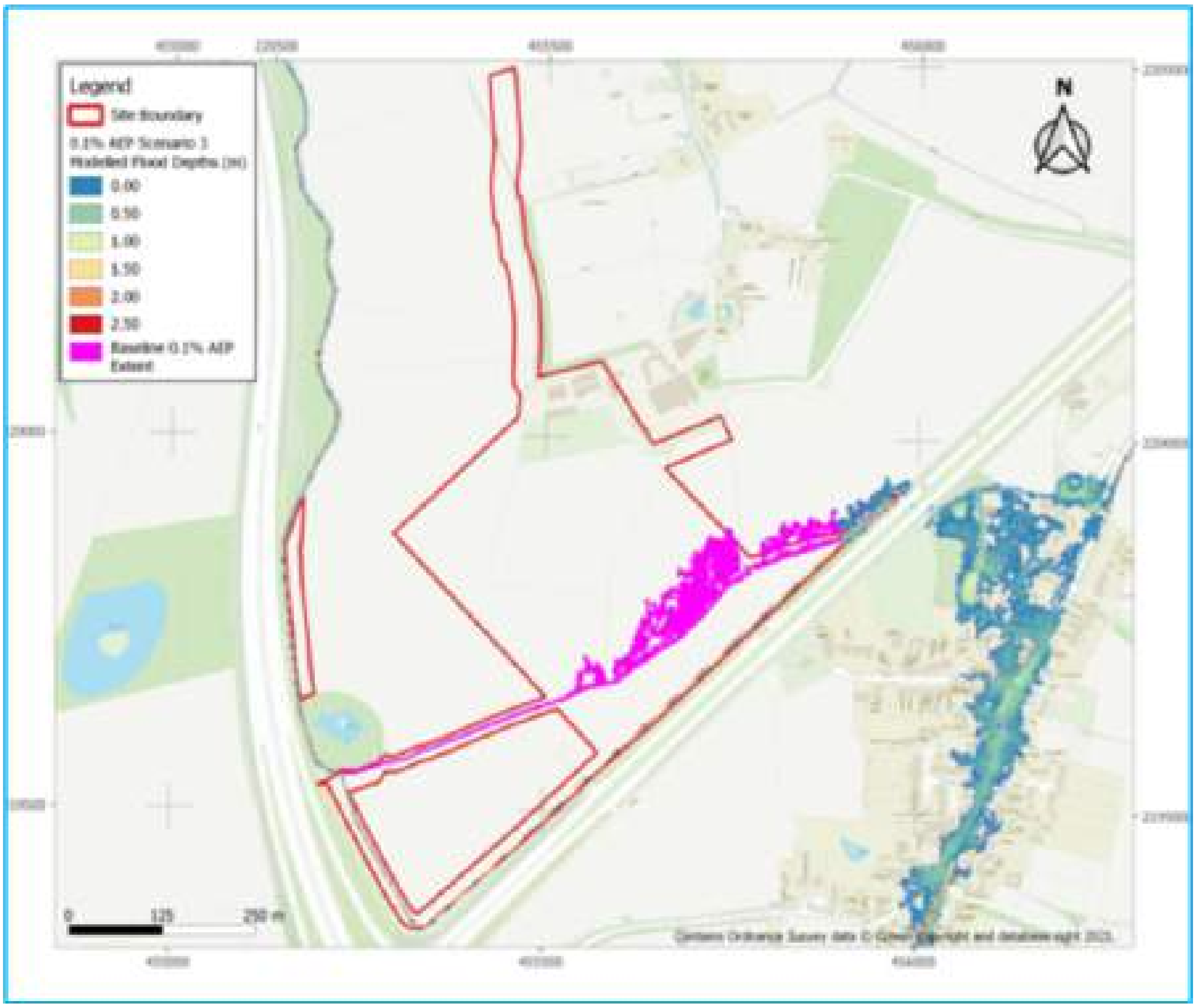


Figure 19 - Mitigation Scenario 3 0.1% AEP Modelled Flood Depths

Figure 20, shows the change in modelled flood depths. As can be seen, at the upstream extremity of the realigned channel, some increases in depth can be seen. However, for the majority of the modelled extent, no impact can be seen.

Upstream of the A41 culvert, the majority of changes in depth lie within the +/- 10mm band, but both some small extents of increases and decreases in depth beyond 10mm can be seen. Downstream of the A41 culvert, some small extents of increased flood depths can be seeb in Wendlebury.

Figure 20 - Mitigation Scenario 3 0.1% AEP Change in Modelled Flood Depth

6.4 Summary of Mitigation Scenario 3 Results

The proposed site is shown to be flood-free during the 1.0% AEP and 1.0% AEP + CC events, with a small amount of flooding shown during the 0.1% AEP event at the eastern site boundary. During the 1.0% AEP and 1.0% AEP + CC events the modelled change in flood depth through Wendlebury shows no impact as a result of the proposed development.

As can be seen in Table 9, mitigation scenario 3 does not result in a significant increase in peak flows through the A41 culvert and therefore Wendlebury. For the 0.1% AEP event there are some isolated cells of flood depth increases beyond 10mm in Wendlebury. Due to the inherited instabilities in the baseline model through Wendlebury, the cause of these increases is thought to be a result of oscillations in the modelling results rather than as a result of the proposed development. In addition to this, there are some small increases in flood depths just upstream of the A41 culvert.

Table 9 - Peak Flows through the A41 Culvert

Event	Baseline A41 Culvert Peak Flow (m^3/s)	Mitigation 3 A41 Culvert Peak Flow (m^3/s)	Change in Peak Flow (m^3/s)
1.0% AEP	1.806	1.800	-0.006
1.0% AEP + 15% CC	2.070	2.073	0.003
0.1% AEP	3.141	3.163	0.022

7 Blockage Scenario

To assess the impact of a blockage of the culvert under the A41 (FMP node WB_1878cu), a blockage scenario was run. Following a review of the risk of a blockage occurring, it was determined that the risk at this location was low, as a result of the large culvert size and lack of potential debris sources. As such a 33% blockage ratio is considered appropriate for this location. The updates made to represent this consisted of; the insertion of blockage unit upstream of the culvert inlet in FMP, with a 33% blockage ratio and updated initial conditions for the model.

7.1 Mitigation Scenario 1.0% AEP + 15% CC Blockage Scenario

Figure 21 below shows the modelled flood depths for the mitigation scenario 3 model with a 33% blockage ratio for the culvert under the A41. As can be seen a small out of bank flood extent can be seen just upstream of the A41 culvert as a result of the blockage scenario. This small increase in extent does not pose any additional risk to the proposed development site and is limited to a small area of agricultural land.

Figure 21 – Mitigation Secenario 3 1.0% AEP + CC Blockage Scenario

8 Conclusions

Modelling of the Wendlebury Brook was undertaken to determine the impact of the proposed development on flood risk. This consisted of the baseline scenario and four different proposed development scenarios. Mitigation scenario 3 has no impact on flood risk to the development site or the town of Wendlebury for the 1.0% AEP and 1.0% AEP + CC events. For the 0.1% AEP event for the majority of the flood extent, no detrimental impact is shown, however, there are some isolated cells showing impact. Due to the inherited instabilities in the baseline model through Wendlebury these impacts are believed to be a result of oscillations in the modelling results rather than the development. Further to this, no significant increase in peak flows through Wendlebury are seen as a result of the development supporting the suggestion that the impacts seen in the 0.1% AEP event are not a result of the proposed development. As the other proposed development scenarios resulted in detrimental impacts on flood risk as a result of the development, mitigation scenario 3 should be incorporated into the final scheme.

Appendix 1 – Proposed Development CAD

Appendix F – Proposed Drainage Layout

STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Storm

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	2	PIMP (%)	100
M5-60 (mm)	20.000	Add Flow / Climate Change (%)	0
Ratio R	0.401	Minimum Backdrop Height (m)	0.200
Maximum Rainfall (mm/hr)	50	Maximum Backdrop Height (m)	1.500
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	1.200
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	400

Designed with Level Soffits

Time Area Diagram for Storm at outfall (pipe 1.009)

Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	1.513	8-12	3.102	16-20	0.752	24-28	0.338
4-8	1.898	12-16	0.833	20-24	0.434	28-32	0.011

Total Area Contributing (ha) = 8.881

Total Pipe Volume (m³) = 7835.605

Time Area Diagram at outfall (pipe 31.006)

Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.149	4-8	0.421	8-12	0.062

Total Area Contributing (ha) = 0.632

Total Pipe Volume (m³) = 766.232

Network Design Table for Storm

« - Indicates pipe capacity < flow

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design

Network Results Table

Network Design Table for Storm

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design
1.000	80.000	0.200	400.0	0.622	4.00	0.0	0.600		o	600	Pipe/Conduit	
2.000	29.115	0.073	400.0	0.526	4.00	0.0	0.600		o	600	Pipe/Conduit	
3.000	24.765	0.062	400.0	0.423	4.00	0.0	0.600		o	600	Pipe/Conduit	
1.001	52.511	0.131	400.8	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
4.000	26.805	0.067	400.0	0.362	4.00	0.0	0.600		o	600	Pipe/Conduit	
1.002	17.749	0.044	403.4	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
5.000	24.998	0.062	400.0	0.362	4.00	0.0	0.600		o	600	Pipe/Conduit	
1.003	63.844	0.160	400.0	0.105	0.00	0.0	0.600		o	600	Pipe/Conduit	
6.000	70.000	0.175	400.0	0.532	4.00	0.0	0.600		o	600	Pipe/Conduit	
7.000	50.000	0.125	400.0	0.783	4.00	0.0	0.600		o	600	Pipe/Conduit	
6.001	22.290	0.075	297.2	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
1.000	50.00	5.10	65.700	0.622	0.0	0.0	0.0	1.21	342.5	84.2
2.000	50.00	4.40	65.700	0.526	0.0	0.0	0.0	1.21	342.5	71.2
3.000	50.00	4.34	65.700	0.423	0.0	0.0	0.0	1.21	342.5	57.3
1.001	50.00	5.82	65.350	1.571	0.0	0.0	0.0	1.21	342.1	212.7
4.000	50.00	4.37	65.700	0.362	0.0	0.0	0.0	1.21	342.5	49.0
1.002	50.00	6.07	65.219	1.933	0.0	0.0	0.0	1.21	341.1	261.8
5.000	50.00	4.34	65.700	0.362	0.0	0.0	0.0	1.21	342.5	49.0
1.003	50.00	6.95	65.175	2.400	0.0	0.0	0.0	1.21	342.5	325.0
6.000	50.00	4.96	65.925	0.532	0.0	0.0	0.0	1.21	342.5	72.0
7.000	50.00	4.69	65.500	0.783	0.0	0.0	0.0	1.21	342.5	106.0
6.001	50.00	5.23	65.375	1.315	0.0	0.0	0.0	1.41	397.9	178.1

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design
8.000	238.000	0.238	1000.0	0.431	4.00	0.0		0.035	→_/→		Swale	
1.004	48.622	0.049	992.3	0.000	0.00	0.0		0.035	→_/		Pond/Tank	
1.005	72.500	0.121	599.2	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
1.006	72.500	0.121	599.2	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
1.007	72.500	0.121	599.2	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
1.008	73.323	0.122	601.0	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
9.000	43.738	0.087	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
9.001	17.570	0.044	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
10.000	45.275	0.091	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
9.002	11.000	0.028	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
11.000	45.275	0.091	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
9.003	11.787	0.029	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
12.000	45.235	0.090	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
13.000	53.275	0.107	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
8.000	40.44	14.78	66.500	0.431	0.0	0.0	0.0	0.37	331.2	47.2
1.004	39.40	15.45	65.000	4.146	0.0	0.0	0.0	1.21	64775.0	442.4
1.005	37.65	16.67	64.951	4.146	0.0	0.0	0.0	0.99	279.2«	442.4
1.006	36.07	17.90	64.830	4.146	0.0	0.0	0.0	0.99	279.2«	442.4
1.007	34.63	19.12	64.709	4.146	0.0	0.0	0.0	0.99	279.2«	442.4
1.008	33.31	20.36	64.588	4.146	0.0	0.0	0.0	0.99	278.8«	442.4
9.000	33.17	20.50	66.700	0.050	0.0	0.0	0.0	0.04	62.6	4.5
9.001	32.88	20.79	65.300	0.050	0.0	0.0	0.0	1.01	160.7	4.5
10.000	33.00	20.67	66.400	0.050	0.0	0.0	0.0	0.05	66.5	4.5
9.002	32.70	20.97	65.256	0.100	0.0	0.0	0.0	1.01	160.7	8.9
11.000	32.18	21.52	66.230	0.050	0.0	0.0	0.0	0.04	50.4	4.4
9.003	31.99	21.71	65.229	0.150	0.0	0.0	0.0	1.01	160.7	13.0
12.000	33.77	19.92	66.000	0.050	0.0	0.0	0.0	0.05	63.9	4.6
13.000	31.07	22.75	66.000	0.050	0.0	0.0	0.0	0.05	63.9	4.2

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design
9.004	17.000	0.043	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
14.000	45.275	0.091	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
15.000	53.275	0.107	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
9.005	10.500	0.026	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
16.000	45.324	0.091	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
17.000	68.618	0.137	500.0	0.050	4.00	0.0		0.075	→|↓|→		Porous Car Park	
9.006	7.738	0.019	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
9.007	108.059	0.108	1000.5	0.000	0.00	0.0		0.035	→_/→		Swale	
9.008	27.944	0.070	399.2	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
9.009	42.814	0.107	400.1	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
18.000	30.448	0.061	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
18.001	28.166	0.094	299.6	0.000	0.00	0.0	0.600		o	300	Pipe/Conduit	
19.000	41.950	0.084	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
9.004	30.83	23.03	65.199	0.250	0.0	0.0	0.0	1.01	160.7	20.9
14.000	35.15	18.66	65.700	0.050	0.0	0.0	0.0	0.05	78.7	4.8
15.000	32.43	21.25	65.700	0.050	0.0	0.0	0.0	0.05	78.7	4.4
9.005	30.68	23.20	65.157	0.350	0.0	0.0	0.0	1.01	160.7	29.1
16.000	37.05	17.12	65.500	0.050	0.0	0.0	0.0	0.06	104.4	5.0
17.000	30.13	23.86	65.500	0.050	0.0	0.0	0.0	0.06	121.6	4.1
9.006	30.03	23.99	65.113	0.450	0.0	0.0	0.0	1.01	160.7	36.6
9.007	27.97	26.79	64.957	0.450	0.0	0.0	0.0	0.64	3080.6	36.6
9.008	27.71	27.18	64.849	0.450	0.0	0.0	0.0	1.21	342.9	36.6
9.009	27.33	27.77	64.779	0.450	0.0	0.0	0.0	1.21	342.5	36.6
18.000	39.02	15.71	66.690	0.120	0.0	0.0	0.0	0.04	51.2	12.7
18.001	38.27	16.23	66.000	0.120	0.0	0.0	0.0	0.90	63.8	12.7
19.000	33.86	19.83	66.700	0.120	0.0	0.0	0.0	0.04	53.7	11.0

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design
19.001	20.991	0.052	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
20.000	58.285	0.117	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
21.000	55.286	0.111	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
18.002	11.500	0.029	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
22.000	48.276	0.097	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
23.000	55.277	0.111	500.0	0.110	4.00	0.0		0.075	→|↓|→		Porous Car Park	
18.003	11.287	0.028	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
24.000	48.290	0.097	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
25.000	55.298	0.111	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
18.004	17.000	0.043	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
26.000	48.290	0.097	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
19.001	33.50	20.18	66.000	0.120	0.0	0.0	0.0	1.01	160.7	11.0
20.000	29.62	24.51	66.400	0.120	0.0	0.0	0.0	0.05	63.9	9.6
21.000	30.47	23.45	66.400	0.120	0.0	0.0	0.0	0.05	63.9	9.9
18.002	29.48	24.70	65.756	0.480	0.0	0.0	0.0	1.01	160.7	38.3
22.000	40.51	14.74	66.000	0.120	0.0	0.0	0.0	0.07	202.3	13.2
23.000	38.17	16.29	66.000	0.110	0.0	0.0	0.0	0.07	202.3	11.4
18.003	29.33	24.88	65.727	0.710	0.0	0.0	0.0	1.01	160.7	56.4
24.000	38.87	15.80	65.900	0.120	0.0	0.0	0.0	0.07	159.5	12.6
25.000	36.54	17.52	65.900	0.120	0.0	0.0	0.0	0.07	159.5	11.9
18.004	29.12	25.17	65.599	0.950	0.0	0.0	0.0	1.01	160.7	74.9
26.000	40.01	15.05	65.600	0.120	0.0	0.0	0.0	0.07	933.8	13.0

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design
27.000	55.298	0.111	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
18.005	3.125	0.008	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
28.000	67.530	0.135	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
29.000	55.298	0.111	500.0	0.120	4.00	0.0		0.075	→|↓|→		Porous Car Park	
28.001	7.275	0.018	400.0	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
9.010	56.335	0.241	233.8	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
9.011	14.784	0.037	399.6	0.000	0.00	0.0	0.600		o	600	Pipe/Conduit	
30.000	90.000	0.225	400.0	0.900	4.00	0.0	0.600		o	750	Pipe/Conduit	
30.001	60.000	0.150	400.0	0.900	0.00	0.0	0.600		o	750	Pipe/Conduit	
30.002	76.000	0.190	400.0	0.900	0.00	0.0	0.600		o	750	Pipe/Conduit	
1.009	14.211	0.047	302.4	0.155	0.00	0.0	0.600		o	375	Pipe/Conduit	
31.000	42.500	0.106	400.0	0.095	4.00	0.0	0.600		o	450	Pipe/Conduit	
32.000	30.000	0.075	400.0	0.095	4.00	0.0	0.600		o	450	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
27.000	37.67	16.66	65.600	0.120	0.0	0.0	0.0	0.07	933.8	12.2
18.005	29.09	25.22	65.324	1.190	0.0	0.0	0.0	1.01	160.7	93.7
28.000	40.58	14.69	65.500	0.120	0.0	0.0	0.0	0.11	710.6	13.2
29.000	44.03	12.76	65.500	0.120	0.0	0.0	0.0	0.11	710.6	14.3
28.001	40.39	14.81	65.000	0.240	0.0	0.0	0.0	1.01	160.7	26.3
9.010	26.95	28.36	64.672	1.880	0.0	0.0	0.0	1.59	449.1	137.2
9.011	26.83	28.56	64.431	1.880	0.0	0.0	0.0	1.21	342.7	137.2
30.000	50.00	5.08	65.000	0.900	0.0	0.0	0.0	1.39	615.4	121.9
30.001	50.00	5.79	64.775	1.800	0.0	0.0	0.0	1.39	615.4	243.7
30.002	50.00	6.70	64.625	2.700	0.0	0.0	0.0	1.39	615.4	365.6
1.009	26.69	28.79	64.394	8.881	0.0	0.0	0.0	1.04	114.5«	641.9
31.000	50.00	4.70	65.500	0.095	0.0	0.0	0.0	1.01	160.7	12.9
32.000	50.00	4.49	65.600	0.095	0.0	0.0	0.0	1.01	160.7	12.9

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	n	HYD SECT	DIA (mm)	Section Type	Auto Design
31.001	9.716	0.024	400.0	0.109	0.00	0.0	0.600		o	450	Pipe/Conduit	
31.002	90.000	0.739	121.8	0.095	0.00	0.0	0.600		o	450	Pipe/Conduit	
31.003	90.000	0.739	121.8	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
31.004	9.940	0.025	397.6	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
31.005	94.345	0.094	1003.7	0.000	0.00	0.0		0.035	→_/→		Swale	
33.000	70.000	0.175	400.0	0.130	4.00	0.0	0.600		o	450	Pipe/Conduit	
34.000	50.000	0.125	400.0	0.108	4.00	0.0	0.600		o	450	Pipe/Conduit	
33.001	11.000	0.028	392.9	0.000	0.00	0.0	0.600		o	450	Pipe/Conduit	
31.006	9.174	0.023	398.9	0.000	0.00	0.0	0.600		o	150	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
31.001	50.00	4.86	65.394	0.299	0.0	0.0	0.0	1.01	160.7	40.5
31.002	50.00	5.68	65.369	0.394	0.0	0.0	0.0	1.84	292.8	53.4
31.003	50.00	6.49	64.630	0.394	0.0	0.0	0.0	1.84	292.8	53.4
31.004	50.00	6.65	63.891	0.394	0.0	0.0	0.0	1.01	161.2	53.4
31.005	50.00	8.77	63.866	0.394	0.0	0.0	0.0	0.74	5528.8	53.4
33.000	50.00	5.15	64.000	0.130	0.0	0.0	0.0	1.01	160.7	17.6
34.000	50.00	4.82	64.000	0.108	0.0	0.0	0.0	1.01	160.7	14.6
33.001	50.00	5.33	63.825	0.238	0.0	0.0	0.0	1.02	162.2	32.2
31.006	50.00	9.08	63.706	0.632	0.0	0.0	0.0	0.50	8.8«	85.6

PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
1.000	o		1	67.565	65.700	1.265	Open Manhole	1500
2.000	o	600	2	67.565	65.700	1.265	Open Manhole	1500
3.000	o	600	3	67.565	65.700	1.265	Open Manhole	1500
1.001	o	600	4	67.155	65.350	1.205	Open Manhole	1500
4.000	o	600	5	67.565	65.700	1.265	Open Manhole	1500
1.002	o	600	6	67.155	65.219	1.336	Open Manhole	1500
5.000	o	600	7	67.565	65.700	1.265	Open Manhole	1500
1.003	o	600	8	67.155	65.175	1.380	Open Manhole	1500
6.000	o	600	9	67.425	65.925	0.900	Open Manhole	1500
7.000	o	600	11	67.130	65.500	1.030	Open Manhole	1500
6.001	o	600	14	67.060	65.375	1.085	Open Manhole	1500

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
1.000	80.000	400.0	4	67.155	65.500	1.055	Open Manhole	1500
2.000	29.115	400.0	4	67.155	65.627	0.928	Open Manhole	1500
3.000	24.765	400.0	4	67.155	65.638	0.917	Open Manhole	1500
1.001	52.511	400.8	6	67.155	65.219	1.336	Open Manhole	1500
4.000	26.805	400.0	6	67.155	65.633	0.922	Open Manhole	1500
1.002	17.749	403.4	8	67.155	65.175	1.380	Open Manhole	1500
5.000	24.998	400.0	8	67.155	65.638	0.917	Open Manhole	1500
1.003	63.844	400.0	16	67.000	65.015	1.385	Open Manhole	1500
6.000	70.000	400.0	14	67.060	65.750	0.710	Open Manhole	1500
7.000	50.000	400.0	14	67.060	65.375	1.085	Open Manhole	1500
6.001	22.290	297.2	16	67.000	65.300	1.100	Open Manhole	1500

PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
8.000	→_/→		15	67.000	66.500	0.000	Open Manhole	1500
1.004	→_/		16	67.000	65.000	0.000	Open Manhole	1500
1.005	o	600	17	67.060	64.951	1.509	Open Manhole	1200
1.006	o	600	15	66.350	64.830	0.920	Open Manhole	1500
1.007	o	600	16	66.350	64.709	1.041	Open Manhole	1500
1.008	o	600	17	66.350	64.588	1.162	Open Manhole	1500
9.000	→|↓|→		18	67.135	66.700	0.300	Open Manhole	1200
9.001	o	450	19	67.135	65.300	1.385	Open Manhole	1200
10.000	→|↓|→		20	66.840	66.400	0.300	Open Manhole	1200
9.002	o	450	21	66.840	65.256	1.134	Open Manhole	1200
11.000	→|↓|→		22	66.660	66.230	0.300	Open Manhole	1200
9.003	o	450	23	66.660	65.229	0.981	Open Manhole	1200
12.000	→|↓|→		24	66.450	66.000	0.300	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
8.000	238.000	1000.0	16	67.000	66.262	0.238	Open Manhole	1500
1.004	48.622	992.3	17	67.060	64.951	0.109	Open Manhole	1200
1.005	72.500	599.2	15	66.350	64.830	0.920	Open Manhole	1500
1.006	72.500	599.2	16	66.350	64.709	1.041	Open Manhole	1500
1.007	72.500	599.2	17	66.350	64.588	1.162	Open Manhole	1500
1.008	73.323	601.0	63	66.350	64.466	1.284	Open Manhole	2100
9.000	43.738	500.0	19	67.135	66.613	0.387	Open Manhole	1200
9.001	17.570	400.0	21	66.840	65.256	1.134	Open Manhole	1200
10.000	45.275	500.0	21	66.840	66.309	0.391	Open Manhole	1200
9.002	11.000	400.0	23	66.660	65.229	0.981	Open Manhole	1200
11.000	45.275	500.0	23	66.660	66.139	0.391	Open Manhole	1200
9.003	11.787	400.0	26	66.450	65.199	0.801	Open Manhole	1200
12.000	45.235	500.0	26	66.450	65.910	0.390	Open Manhole	1200

PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
13.000	→|↓|→		25	66.450	66.000	0.300	Open Manhole	1200
9.004	o	450	26	66.450	65.199	0.801	Open Manhole	1200
14.000	→|↓|→		27	66.170	65.700	0.300	Open Manhole	1200
15.000	→|↓|→		28	66.170	65.700	0.300	Open Manhole	1200
9.005	o	450	29	66.170	65.157	0.563	Open Manhole	1200
16.000	→|↓|→		30	66.000	65.500	0.300	Open Manhole	1200
17.000	→|↓|→		31	66.000	65.500	0.300	Open Manhole	1200
9.006	o	450	33	66.125	65.113	0.562	Open Manhole	3000
9.007	→_/→		33	66.125	64.957	0.000	Open Manhole	3000
9.008	o	600	34	66.125	64.849	0.676	Junction	
9.009	o	600	35	66.125	64.779	0.746	Open Manhole	2100
18.000	→|↓|→		34	67.120	66.690	0.300	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
13.000	53.275	500.0	26	66.450	65.893	0.407	Open Manhole	1200
9.004	17.000	400.0	29	66.170	65.157	0.563	Open Manhole	1200
14.000	45.275	500.0	29	66.170	65.609	0.391	Open Manhole	1200
15.000	53.275	500.0	29	66.170	65.593	0.407	Open Manhole	1200
9.005	10.500	400.0	33	66.125	65.130	0.545	Open Manhole	3000
16.000	45.324	500.0	33	66.125	65.409	0.516	Open Manhole	3000
17.000	68.618	500.0	33	66.125	65.363	0.562	Open Manhole	3000
9.006	7.738	400.0	33	66.125	65.093	0.582	Open Manhole	3000
9.007	108.059	1000.5	34	66.125	64.849	0.108	Junction	
9.008	27.944	399.2	35	66.125	64.779	0.746	Open Manhole	2100
9.009	42.814	400.1	54	66.125	64.672	0.853	Open Manhole	2400
18.000	30.448	500.0	35	67.120	66.629	0.361	Open Manhole	1500

PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
18.001	o	300	35	67.120	66.000	0.820	Open Manhole	1500
19.000	→|↓|→		36	67.135	66.700	0.300	Open Manhole	1200
19.001	o	450	37	67.135	66.000	0.685	Open Manhole	1200
20.000	→|↓|→		38	66.850	66.400	0.300	Open Manhole	1200
21.000	→|↓|→		39	66.850	66.400	0.300	Open Manhole	1200
18.002	o	450	40	68.850	65.756	2.644	Open Manhole	1200
22.000	→|↓|→		41	66.600	66.000	0.300	Open Manhole	1200
23.000	→|↓|→		42	66.600	66.000	0.300	Open Manhole	1200
18.003	o	450	43	66.600	65.727	0.423	Open Manhole	1200
24.000	→|↓|→		44	66.460	65.900	0.300	Open Manhole	1200
25.000	→|↓|→		45	66.460	65.900	0.300	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
18.001	28.166	299.6	40	68.850	65.906	2.644	Open Manhole	1200
19.000	41.950	500.0	37	67.135	66.616	0.384	Open Manhole	1200
19.001	20.991	400.0	40	68.850	65.948	2.452	Open Manhole	1200
20.000	58.285	500.0	40	68.850	66.283	2.417	Open Manhole	1200
21.000	55.286	500.0	40	68.850	66.289	2.411	Open Manhole	1200
18.002	11.500	400.0	43	66.600	65.727	0.423	Open Manhole	1200
22.000	48.276	500.0	43	66.600	65.903	0.397	Open Manhole	1200
23.000	55.277	500.0	43	66.600	65.889	0.411	Open Manhole	1200
18.003	11.287	400.0	46	66.460	65.699	0.311	Open Manhole	1200
24.000	48.290	500.0	46	66.460	65.803	0.397	Open Manhole	1200
25.000	55.298	500.0	46	66.460	65.789	0.411	Open Manhole	1200

PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
18.004	o	450	46	66.460	65.599	0.411	Open Manhole	1200
26.000	→|↓|→		47	66.185	65.600	0.300	Open Manhole	1200
27.000	→|↓|→		48	66.185	65.600	0.300	Open Manhole	1200
18.005	o	450	49	66.185	65.324	0.411	Open Manhole	1200
28.000	→|↓|→		50	66.000	65.500	0.000	Open Manhole	1200
29.000	→|↓|→		51	66.000	65.500	0.000	Open Manhole	1200
28.001	o	450	52	66.000	65.000	0.550	Open Manhole	1350
9.010	o	600	54	66.125	64.672	0.853	Open Manhole	2400
9.011	o	600	56	66.125	64.431	1.094	Open Manhole	2400
30.000	o	750	60	67.320	65.000	1.570	Open Manhole	1800
30.001	o	750	61	67.550	64.775	2.025	Open Manhole	1800
30.002	o	750	62	67.480	64.625	2.105	Open Manhole	1800

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
18.004	17.000	400.0	49	66.185	65.557	0.178	Open Manhole	1200
26.000	48.290	500.0	49	66.185	65.503	0.397	Open Manhole	1200
27.000	55.298	500.0	49	66.185	65.489	0.411	Open Manhole	1200
18.005	3.125	400.0	54	66.125	65.317	0.358	Open Manhole	2400
28.000	67.530	500.0	52	66.000	65.365	0.135	Open Manhole	1350
29.000	55.298	500.0	52	66.000	65.389	0.111	Open Manhole	1350
28.001	7.275	400.0	54	66.125	64.982	0.693	Open Manhole	2400
9.010	56.335	233.8	56	66.125	64.431	1.094	Open Manhole	2400
9.011	14.784	399.6	63	66.350	64.394	1.356	Open Manhole	2100
30.000	90.000	400.0	61	67.550	64.775	2.025	Open Manhole	1800
30.001	60.000	400.0	62	67.480	64.625	2.105	Open Manhole	1800
30.002	76.000	400.0	63	66.350	64.435	1.165	Open Manhole	2100

PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
1.009	o	375	63	66.350	64.394	1.581	Open Manhole	2100
31.000	o	450	62	66.900	65.500	0.950	Open Manhole	1350
32.000	o	450	63	66.900	65.600	0.850	Open Manhole	1350
31.001	o	450	64	66.950	65.394	1.106	Open Manhole	1350
31.002	o	450	65	66.900	65.369	1.081	Open Manhole	1350
31.003	o	450	65	66.030	64.630	0.950	Open Manhole	1350
31.004	o	450	66	66.000	63.891	1.659	Open Manhole	1350
31.005	→_/→		66	66.300	63.866	0.956	Open Manhole	1200
33.000	o	450	67	65.500	64.000	1.050	Open Manhole	3000
34.000	o	450	68	65.930	64.000	1.480	Open Manhole	3000
33.001	o	450	69	65.600	63.825	1.325	Open Manhole	3000
31.006	o	150	70	65.250	63.706	1.394	Open Manhole	3000

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
1.009	14.211	302.4		66.000	64.347	1.278	Open Manhole	0
31.000	42.500	400.0	64	66.950	65.394	1.106	Open Manhole	1350
32.000	30.000	400.0	64	66.950	65.525	0.975	Open Manhole	1350
31.001	9.716	400.0	65	66.900	65.369	1.081	Open Manhole	1350
31.002	90.000	121.8	65	66.030	64.630	0.950	Open Manhole	1350
31.003	90.000	121.8	66	66.000	63.891	1.659	Open Manhole	1350
31.004	9.940	397.6	66	66.300	63.866	1.984	Open Manhole	1200
31.005	94.345	1003.7	70	65.250	63.772	0.000	Open Manhole	3000
33.000	70.000	400.0	69	65.600	63.825	1.325	Open Manhole	3000
34.000	50.000	400.0	69	65.600	63.875	1.275	Open Manhole	3000
33.001	11.000	392.9	70	65.250	63.797	1.003	Open Manhole	3000
31.006	9.174	398.9		65.600	63.683	1.767	Open Manhole	0

Free Flowing Outfall Details for Storm

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
1.009		66.000	64.347	0.000	0	0

Free Flowing Outfall Details for Storm

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
31.006		65.600	63.683	0.000	0	0

Simulation Criteria for Storm

Volumetric Runoff Coeff	0.750	Additional Flow - % of Total Flow	0.000
Areal Reduction Factor	1.000	MADD Factor * 10m³/ha Storage	2.000
Hot Start (mins)	0	Inlet Coeffiecient	0.800
Hot Start Level (mm)	0	Flow per Person per Day (l/per/day)	0.000
Manhole Headloss Coeff (Global)	0.500	Run Time (mins)	60
Foul Sewage per hectare (l/s)	0.000	Output Interval (mins)	1

Number of Input Hydrographs	0	Number of Storage Structures	29
Number of Online Controls	2	Number of Time/Area Diagrams	0
Number of Offline Controls	0	Number of Real Time Controls	0

Synthetic Rainfall Details

Rainfall Model	FSR	Profile Type	Summer
Return Period (years)	2	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Storm Duration (mins)	30
Ratio R	0.401		

Online Controls for Storm

Hydro-Brake® Optimum Manhole: 63, DS/PN: 1.009, Volume (m³): 63.3

Unit Reference	MD-SHE-0194-2050-1500-2050
Design Head (m)	1.500
Design Flow (l/s)	20.5
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	194
Invert Level (m)	64.394
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.500	20.5
Flush-Flo™	0.451	20.5
Kick-Flo®	0.983	16.8
Mean Flow over Head Range	-	17.7

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	6.7	1.200	18.4	3.000	28.6	7.000	43.0
0.200	18.0	1.400	19.8	3.500	30.7	7.500	44.4
0.300	19.9	1.600	21.1	4.000	32.8	8.000	45.8
0.400	20.4	1.800	22.4	4.500	34.7	8.500	47.2
0.500	20.4	2.000	23.5	5.000	36.5	9.000	48.5
0.600	20.2	2.200	24.6	5.500	38.2	9.500	49.8
0.800	19.3	2.400	25.7	6.000	39.9		
1.000	16.9	2.600	26.7	6.500	41.4		

Hydro-Brake® Optimum Manhole: 70, DS/PN: 31.006, Volume (m³): 698.5

Unit Reference	MD-SHE-0061-2000-1500-2000
Design Head (m)	1.500
Design Flow (l/s)	2.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	61
Invert Level (m)	63.706
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Hydro-Brake® Optimum Manhole: 70, DS/PN: 31.006, Volume (m^3): 698.5

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.500	2.0
Flush-Flo™	0.269	1.6
Kick-Flo®	0.545	1.3
Mean Flow over Head Range	-	1.5

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.3	1.200	1.8	3.000	2.7	7.000	4.1
0.200	1.5	1.400	1.9	3.500	3.0	7.500	4.2
0.300	1.6	1.600	2.1	4.000	3.1	8.000	4.3
0.400	1.5	1.800	2.2	4.500	3.3	8.500	4.5
0.500	1.4	2.000	2.3	5.000	3.5	9.000	4.6
0.600	1.3	2.200	2.4	5.500	3.6	9.500	4.7
0.800	1.5	2.400	2.5	6.000	3.8		
1.000	1.7	2.600	2.6	6.500	3.9		

Storage Structures for Storm

Cellular Storage Manhole: 4, DS/PN: 1.001

Invert Level (m) 65.350 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	500.0	500.0	0.601	0.0	553.7
0.600	500.0	553.7			

Swale Pipe: 8.000

Manning's N	0.035	Base Width (m)	0.3
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	238.0
Infiltration Coefficient Side (m/hr)	0.00000	Side Slope (1:X)	3.0
Safety Factor	2.0	Slope (1:X)	1000.0
Porosity	1.00	Cap Volume Depth (m)	0.000
Invert Level (m)	66.500	Cap Infiltration Depth (m)	0.000

Tank or Pond Pipe: 1.004

Manning's N 0.035 Invert Level (m) 65.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	857.0	1.770	1661.0

Porous Car Park Pipe: 9.000

Manning's N	0.075	Width (m)	35.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	43.7
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	425.2	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.700		

Porous Car Park Pipe: 10.000

Manning's N	0.075	Width (m)	35.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	45.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	440.2	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.400		

Porous Car Park Pipe: 11.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	45.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	377.3	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.230		

Porous Car Park Pipe: 12.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	45.2
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	377.0	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.000		

Porous Car Park Pipe: 13.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	53.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	444.0	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.000		

Porous Car Park Pipe: 14.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	45.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	377.3	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.700		

Porous Car Park Pipe: 15.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	53.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	444.0	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.700		

Porous Car Park Pipe: 16.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	45.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	377.7	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.500		

Under Drain Details

Depth above Invert Level (m)	0.000	Number of Pipes	1
Diameter (m)	0.150	Manning's N	0.022

Porous Car Park Pipe: 17.000

Manning's N	0.075	Width (m)	35.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	68.6
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	667.1	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.500		

Under Drain Details

Depth above Invert Level (m)	0.000	Number of Pipes	1
Diameter (m)	0.150	Manning's N	0.022

Swale Pipe: 9.007

Manning's N	0.035	Base Width (m)	0.6
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	108.1
Infiltration Coefficient Side (m/hr)	0.00000	Side Slope (1:X)	3.0
Safety Factor	2.0	Slope (1:X)	1000.5
Porosity	1.00	Cap Volume Depth (m)	0.000
Invert Level (m)	64.957	Cap Infiltration Depth (m)	0.000

Porous Car Park Pipe: 18.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	30.4
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	253.7	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.690		

Under Drain Details

Depth above Invert Level (m)	0.000	Number of Pipes	1
Diameter (m)	0.150	Manning's N	0.025

Porous Car Park Pipe: 19.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	42.0
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	349.6	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.700		

Porous Car Park Pipe: 20.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	58.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	485.7	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.400		

Porous Car Park Pipe: 21.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	55.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	460.7	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.400		

Porous Car Park Pipe: 22.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	48.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	402.3	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.000		

Porous Car Park Pipe: 23.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	55.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	460.6	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	66.000		

Porous Car Park Pipe: 24.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	48.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	402.4	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.900		

Porous Car Park Pipe: 25.000

Manning's N	0.075	Width (m)	30.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	55.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	460.8	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.900		

Porous Car Park Pipe: 26.000

Manning's N	0.075	Width (m)	150.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	48.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	2012.1	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.600		

Porous Car Park Pipe: 27.000

Manning's N	0.075	Width (m)	150.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	55.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	2304.1	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	300
Invert Level (m)	65.600		

Porous Car Park Pipe: 28.000

Manning's N	0.075	Width (m)	45.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	67.5
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	844.1	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	600
Invert Level (m)	65.500		

Porous Car Park Pipe: 29.000

Manning's N	0.075	Width (m)	45.0
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	55.3
Membrane Percolation (mm/hr)	1000	Slope (1:X)	500.0
Max Percolation (l/s)	691.2	Depression Storage (mm)	5
Safety Factor	2.0	Evaporation (mm/day)	3
Porosity	0.30	Membrane Depth (mm)	600
Invert Level (m)	65.500		

Cellular Storage Manhole: 52, DS/PN: 28.001

Invert Level (m)	65.000	Safety Factor	2.0
Infiltration Coefficient Base (m/hr)	0.00000	Porosity	0.95
Infiltration Coefficient Side (m/hr)	0.00000		

Depth (m)	Area (m^2)	Inf. Area (m^2)	Depth (m)	Area (m^2)	Inf. Area (m^2)
0.000	1750.0	1750.0	0.301	0.0	1817.0
0.300	1750.0	1816.9			

Cellular Storage Manhole: 56, DS/PN: 9.011

Invert Level (m)	64.531	Safety Factor	2.0
Infiltration Coefficient Base (m/hr)	0.00000	Porosity	0.95
Infiltration Coefficient Side (m/hr)	0.00000		

Depth (m)	Area (m^2)	Inf. Area (m^2)	Depth (m)	Area (m^2)	Inf. Area (m^2)
0.000	625.0	625.0	0.661	0.0	691.1
0.660	625.0	691.0			

Cellular Storage Manhole: 63, DS/PN: 1.009

Invert Level (m)	64.435	Safety Factor	2.0
Infiltration Coefficient Base (m/hr)	0.00000	Porosity	0.95
Infiltration Coefficient Side (m/hr)	0.00000		

Depth (m)	Area (m^2)	Inf. Area (m^2)	Depth (m)	Area (m^2)	Inf. Area (m^2)
0.000	2000.0	2000.0	1.201	0.0	2214.8
1.200	2000.0	2214.7			

Swale Pipe: 31.005

Manning's N	0.035	Base Width (m)	0.6
Infiltration Coefficient Base (m/hr)	0.00000	Length (m)	94.3
Infiltration Coefficient Side (m/hr)	0.00000	Side Slope (1:X)	3.0
Safety Factor	2.0	Slope (1:X)	1003.7
Porosity	1.00	Cap Volume Depth (m)	0.000
Invert Level (m)	63.866	Cap Infiltration Depth (m)	0.000

Volume Summary (Static)

Length Calculations based on True Length

Pipe Number	USMH Name	Manhole Volume (m³)	Pipe Volume (m³)	Storage Structure Volume (m³)	Total Volume (m³)
1.000	1	3.296	22.195	0.000	25.491
2.000	2	3.296	7.808	0.000	11.104
3.000	3	3.296	6.578	0.000	9.874
1.001	4	3.190	14.423	285.158	302.771
4.000	5	3.296	7.155	0.000	10.451
1.002	6	3.421	4.594	0.000	8.015
5.000	7	3.296	6.644	0.000	9.940
1.003	8	3.499	17.627	0.000	21.126
6.000	9	2.651	19.368	0.000	22.019
7.000	11	2.880	13.713	0.000	16.593
6.001	14	2.978	5.878	0.000	8.856
8.000	15	0.884	438.786	0.000	439.670
1.004	16	3.534	2714.002	0.000	2717.536
1.005	17	2.385	20.117	0.000	22.502
1.006	15	2.686	20.075	0.000	22.761
1.007	16	2.900	20.075	0.000	22.975
1.008	17	3.114	20.223	0.000	23.336
9.000	18	0.492	60.298	0.000	60.790
9.001	19	2.075	2.604	0.000	4.679
10.000	20	0.498	64.790	0.000	65.288
9.002	21	1.791	1.559	0.000	3.350
11.000	22	0.486	51.568	0.000	52.054
9.003	23	1.619	1.684	0.000	3.303
12.000	24	0.509	59.447	0.000	59.956
13.000	25	0.509	70.301	0.000	70.810
9.004	26	1.415	2.513	0.000	3.928
14.000	27	0.532	67.435	0.000	67.966
15.000	28	0.532	79.675	0.000	80.206
9.005	29	1.146	1.336	0.000	2.482
16.000	30	0.565	78.338	0.000	78.903
17.000	31	0.565	140.511	0.000	141.076
9.006	33	7.155	0.754	0.000	7.909
9.007	33	8.256	602.072	0.000	610.328
9.008	34	0.000	7.604	0.000	7.604
9.009	35	4.662	11.469	0.000	16.131
18.000	34	0.486	34.376	0.000	34.862
18.001	35	1.979	1.896	0.000	3.875
19.000	36	0.492	49.511	0.000	50.003
19.001	37	1.284	3.148	0.000	4.431
20.000	38	0.509	77.065	0.000	77.574
21.000	39	0.509	73.016	0.000	73.525
18.002	40	3.499	1.638	0.000	5.137
22.000	41	0.679	127.105	0.000	127.784
23.000	42	0.679	146.008	0.000	146.686
18.003	43	0.987	1.604	0.000	2.591
24.000	44	0.633	110.191	0.000	110.824
25.000	45	0.633	126.589	0.000	127.223
18.004	46	0.973	2.513	0.000	3.486

Volume Summary (Static)

Pipe Number	USMH Name	Manhole Volume (m³)	Pipe Volume (m³)	Storage Structure Volume (m³)	Total Volume (m³)
26.000	47	0.662	603.929	0.000	604.591
27.000	48	0.662	693.807	0.000	694.468
18.005	49	0.973	0.211	0.000	1.184
28.000	50	0.565	447.221	0.000	447.787
29.000	51	0.565	364.655	0.000	365.221
28.001	52	1.431	0.859	499.304	501.594
9.010	54	6.573	15.250	0.000	21.823
9.011	56	7.663	3.544	392.073	403.280
30.000	60	5.904	38.966	0.000	44.869
30.001	61	7.062	25.712	0.000	32.773
30.002	62	7.265	32.714	0.000	39.979
1.009	63	6.775	1.454	2280.633	2288.862
31.000	62	2.004	6.545	0.000	8.549
32.000	63	1.861	4.557	0.000	6.417
31.001	64	2.228	1.331	0.000	3.558
31.002	65	2.191	14.099	0.000	16.291
31.003	65	2.004	14.099	0.000	16.103
31.004	66	3.019	1.378	0.000	4.397
31.005	66	2.753	686.326	0.000	689.079
33.000	67	10.603	10.656	0.000	21.259
34.000	68	13.642	7.475	0.000	21.117
33.001	69	12.547	1.272	0.000	13.819
31.006	70	10.914	0.136	0.000	11.050
Total		206.616	8394.071	3457.169	12057.856

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 29
Number of Online Controls 2 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.401
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0 DVD Status ON
Analysis Timestep Fine Inertia Status ON
DTS Status OFF

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 180, 240, 360, 480, 600, 720, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 40

WARNING: Half Drain Time has not been calculated as the structure is too full.

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)
1.000	1	15 Winter	1	+0%	100/15 Summer				65.922
2.000	2	15 Winter	1	+0%	100/15 Summer				65.916
3.000	3	15 Winter	1	+0%	100/15 Winter				65.896
1.001	4	60 Winter	1	+0%	100/15 Summer				65.551
4.000	5	15 Winter	1	+0%	100/15 Winter				65.880
1.002	6	60 Winter	1	+0%	100/15 Summer				65.458
5.000	7	15 Winter	1	+0%	100/30 Winter				65.881
1.003	8	15 Winter	1	+0%	100/15 Summer				65.424
6.000	9	15 Winter	1	+0%	100/15 Summer				66.130
7.000	11	15 Winter	1	+0%	30/15 Summer				65.791
6.001	14	15 Winter	1	+0%	30/15 Summer				65.711
8.000	15	15 Winter	1	+0%					66.750
1.004	16	120 Winter	1	+0%					65.216
1.005	17	120 Winter	1	+0%	100/15 Winter				65.212
1.006	15	120 Winter	1	+0%	100/30 Summer				65.089
1.007	16	120 Winter	1	+0%	30/360 Winter				64.965
1.008	17	600 Winter	1	+0%	30/180 Winter				64.938

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status	Level Exceeded
1.000	1	-0.378	0.000	0.30			92.8	OK	
2.000	2	-0.384	0.000	0.28			78.3	OK	
3.000	3	-0.404	0.000	0.23			63.1	OK	
1.001	4	-0.399	0.000	0.21		36	62.2	OK	
4.000	5	-0.420	0.000	0.20			53.9	OK	
1.002	6	-0.361	0.000	0.32			75.8	OK	
5.000	7	-0.419	0.000	0.20			54.0	OK	
1.003	8	-0.351	0.000	0.36			111.0	OK	
6.000	9	-0.395	0.000	0.26			79.4	OK	
7.000	11	-0.309	0.000	0.36			107.5	OK	
6.001	14	-0.264	0.000	0.59			182.3	OK	
8.000	15	-0.250	0.000	0.02		13	14.4	FLOOD RISK	
1.004	16	-1.784	0.000	0.00			141.9	OK	
1.005	17	-0.339	0.000	0.36			91.7	OK	
1.006	15	-0.341	0.000	0.36			91.2	OK	
1.007	16	-0.344	0.000	0.36			91.1	OK	
1.008	17	-0.250	0.000	0.21			53.3	OK	

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.
9.000	18	15 Winter	1	+0%				
9.001	19	60 Winter	1	+0%	100/960 Winter			
10.000	20	15 Winter	1	+0%				
9.002	21	60 Winter	1	+0%	100/600 Winter			
11.000	22	15 Winter	1	+0%				
9.003	23	60 Winter	1	+0%	100/480 Winter			
12.000	24	15 Winter	1	+0%				
13.000	25	15 Winter	1	+0%				
9.004	26	60 Winter	1	+0%	100/360 Winter			
14.000	27	15 Winter	1	+0%				
15.000	28	15 Winter	1	+0%				
9.005	29	60 Winter	1	+0%	100/240 Winter			
16.000	30	15 Winter	1	+0%	100/480 Winter			
17.000	31	15 Winter	1	+0%	100/480 Winter			
9.006	33	60 Winter	1	+0%	100/180 Winter			
9.007	33	60 Winter	1	+0%				
9.008	34	60 Winter	1	+0%	100/120 Summer			
9.009	35	720 Winter	1	+0%	100/60 Summer			
18.000	34	15 Winter	1	+0%	100/15 Summer			
18.001	35	30 Winter	1	+0%				
19.000	36	15 Winter	1	+0%	100/15 Summer			
19.001	37	30 Winter	1	+0%				
20.000	38	15 Winter	1	+0%				
21.000	39	15 Winter	1	+0%				
18.002	40	60 Winter	1	+0%				
22.000	41	15 Winter	1	+0%				
23.000	42	15 Winter	1	+0%				
18.003	43	60 Winter	1	+0%				
24.000	44	15 Winter	1	+0%				
25.000	45	15 Winter	1	+0%				
18.004	46	60 Winter	1	+0%				
26.000	47	15 Winter	1	+0%				
27.000	48	15 Winter	1	+0%				
18.005	49	60 Winter	1	+0%	100/1440 Winter			
28.000	50	15 Winter	1	+0%				
29.000	51	15 Winter	1	+0%				
28.001	52	600 Winter	1	+0%	100/120 Summer			
9.010	54	720 Winter	1	+0%	30/360 Winter			
9.011	56	720 Winter	1	+0%	30/60 Summer	100/1440 Winter		
30.000	60	15 Winter	1	+0%	100/15 Summer			
30.001	61	15 Winter	1	+0%	30/15 Winter			
30.002	62	15 Winter	1	+0%	30/15 Summer			
1.009	63	720 Winter	1	+0%	1/120 Winter			
31.000	62	15 Winter	1	+0%	100/15 Summer			
32.000	63	15 Winter	1	+0%				
31.001	64	15 Winter	1	+0%	100/15 Summer			
31.002	65	15 Winter	1	+0%				
31.003	65	15 Winter	1	+0%				

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status
9.000	18	66.730	-0.105	0.000	0.03		6	1.7	OK
9.001	19	65.352	-0.398	0.000	0.02			2.1	OK
10.000	20	66.430	-0.110	0.000	0.02		6	1.6	OK
9.002	21	65.334	-0.372	0.000	0.04			4.2	OK
11.000	22	66.263	-0.097	0.000	0.04		6	1.8	OK
9.003	23	65.320	-0.359	0.000	0.06			6.3	OK
12.000	24	66.033	-0.117	0.000	0.03		6	1.6	OK
13.000	25	66.033	-0.117	0.000	0.02		6	1.5	OK
9.004	26	65.302	-0.347	0.000	0.09			10.2	OK
14.000	27	65.733	-0.137	0.000	0.02		6	1.6	OK
15.000	28	65.733	-0.137	0.000	0.02		6	1.4	OK
9.005	29	65.272	-0.335	0.000	0.14			14.1	OK
16.000	30	65.532	-0.168	0.000	0.01		6	1.5	OK
17.000	31	65.530	-0.170	0.000	0.01		6	0.6	OK
9.006	33	65.239	-0.324	0.000	0.18			16.4	OK
9.007	33	65.088	-1.037	0.000	0.00		42	15.4	OK
9.008	34	64.941	-0.508	0.000	0.04			15.4	OK*
9.009	35	64.926	-0.453	0.000	0.02			5.5	OK
18.000	34	66.745	-0.075	0.000	0.15		7	7.8	OK
18.001	35	66.075	-0.225	0.000	0.14			8.3	OK
19.000	36	66.756	-0.079	0.000	0.10		7	5.5	OK
19.001	37	66.064	-0.386	0.000	0.05			6.6	OK
20.000	38	66.455	-0.095	0.000	0.06		8	3.9	OK
21.000	39	66.455	-0.095	0.000	0.06		8	4.1	OK
18.002	40	65.912	-0.294	0.000	0.20			19.6	OK
22.000	41	66.054	-0.246	0.000	0.02		7	4.0	OK
23.000	42	66.052	-0.248	0.000	0.02		7	3.1	OK
18.003	43	65.892	-0.285	0.000	0.29			28.9	OK
24.000	44	65.955	-0.205	0.000	0.03		7	4.1	OK
25.000	45	65.955	-0.205	0.000	0.02		7	3.7	OK
18.004	46	65.779	-0.271	0.000	0.34			39.0	OK
26.000	47	65.620	-0.265	0.000	0.00		6	2.4	OK
27.000	48	65.620	-0.265	0.000	0.00		6	2.1	OK
18.005	49	65.503	-0.272	0.000	0.33			40.9	OK
28.000	50	65.542	-0.458	0.000	0.00		6	2.1	OK
29.000	51	65.542	-0.458	0.000	0.00		6	2.6	OK
28.001	52	65.021	-0.429	0.000	0.01		530	0.9	OK
9.010	54	64.926	-0.346	0.000	0.05			20.3	OK
9.011	56	64.925	-0.106	0.000	0.03		576	6.1	OK
30.000	60	65.260	-0.490	0.000	0.22			122.6	OK
30.001	61	65.129	-0.396	0.000	0.37			198.5	OK
30.002	62	65.015	-0.360	0.000	0.53			292.9	OK
1.009	63	64.924	0.155	0.000	0.23		672	20.4	SURCHARGED
31.000	62	65.623	-0.327	0.000	0.09			13.1	OK
32.000	63	65.696	-0.354	0.000	0.10			14.2	OK
31.001	64	65.592	-0.252	0.000	0.40			38.4	OK
31.002	65	65.497	-0.322	0.000	0.17			47.9	OK

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status
31.003	65	64.755	-0.325	0.000	0.17			46.4	OK

PN	US/MH Name	Level Exceeded
9.000	18	
9.001	19	
10.000	20	
9.002	21	
11.000	22	
9.003	23	
12.000	24	
13.000	25	
9.004	26	
14.000	27	
15.000	28	
9.005	29	
16.000	30	
17.000	31	
9.006	33	
9.007	33	
9.008	34	
9.009	35	
18.000	34	
18.001	35	
19.000	36	
19.001	37	
20.000	38	
21.000	39	
18.002	40	
22.000	41	
23.000	42	
18.003	43	
24.000	44	
25.000	45	
18.004	46	
26.000	47	
27.000	48	
18.005	49	
28.000	50	
29.000	51	
28.001	52	
9.010	54	
9.011	56	
30.000	60	
30.001	61	

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Level Exceeded
30.002	62	
1.009	63	
31.000	62	
32.000	63	
31.001	64	
31.002	65	
31.003	65	

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)
31.004	66	480 Winter	1	+0%	30/60 Winter				64.202
31.005	66	480 Winter	1	+0%					64.202
33.000	67	480 Winter	1	+0%	30/180 Winter				64.202
34.000	68	480 Winter	1	+0%	30/180 Winter				64.202
33.001	69	480 Winter	1	+0%	30/30 Winter				64.202
31.006	70	480 Winter	1	+0%	1/15 Summer				64.202

PN	US/MH Name	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status	Level Exceeded
31.004	66	-0.139	0.000	0.07			6.6	OK	
31.005	66	-2.098	0.000	0.00			2.3	OK	
33.000	67	-0.248	0.000	0.01			2.1	OK	
34.000	68	-0.248	0.000	0.01			1.7	OK	
33.001	69	-0.073	0.000	0.02			2.2	OK	
31.006	70	0.346	0.000	0.23			1.6	SURCHARGED	

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 29
Number of Online Controls 2 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.401
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0 DVD Status ON
Analysis Timestep Fine Inertia Status ON
DTS Status OFF

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 180, 240, 360, 480, 600, 720, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 40

WARNING: Half Drain Time has not been calculated as the structure is too full.

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)
1.000	1	15 Winter	30	+0%	100/15 Summer				66.080
2.000	2	15 Winter	30	+0%	100/15 Summer				66.068
3.000	3	15 Winter	30	+0%	100/15 Winter				66.026
1.001	4	30 Winter	30	+0%	100/15 Summer				65.774
4.000	5	15 Winter	30	+0%	100/15 Winter				65.994
1.002	6	30 Winter	30	+0%	100/15 Summer				65.694
5.000	7	15 Winter	30	+0%	100/30 Winter				65.997
1.003	8	30 Winter	30	+0%	100/15 Summer				65.639
6.000	9	15 Winter	30	+0%	100/15 Summer				66.270
7.000	11	15 Winter	30	+0%	30/15 Summer				66.147
6.001	14	15 Winter	30	+0%	30/15 Summer				66.057
8.000	15	15 Winter	30	+0%					66.873
1.004	16	600 Winter	30	+0%					65.512
1.005	17	600 Winter	30	+0%	100/15 Winter				65.508
1.006	15	960 Winter	30	+0%	100/30 Summer				65.430
1.007	16	1440 Winter	30	+0%	30/360 Winter				65.372
1.008	17	1440 Winter	30	+0%	30/180 Winter				65.369

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status	Level Exceeded
1.000	1	-0.220	0.000	0.70			219.0	OK	
2.000	2	-0.232	0.000	0.69			192.4	OK	
3.000	3	-0.274	0.000	0.57			154.9	OK	
1.001	4	-0.176	0.000	0.64		28	192.5	OK	
4.000	5	-0.306	0.000	0.48			132.6	OK	
1.002	6	-0.125	0.000	0.99			231.5	OK	
5.000	7	-0.303	0.000	0.49			132.6	OK	
1.003	8	-0.136	0.000	0.95			291.2	OK	
6.000	9	-0.255	0.000	0.59			183.9	OK	
7.000	11	0.047	0.000	0.85			255.9	SURCHARGED	
6.001	14	0.082	0.000	1.40			429.8	SURCHARGED	
8.000	15	-0.127	0.000	0.05		15	43.6	FLOOD RISK	
1.004	16	-1.488	0.000	0.00			126.7	OK	
1.005	17	-0.043	0.000	0.43			109.5	OK	
1.006	15	0.000	0.000	0.29			74.3	OK	
1.007	16	0.063	0.000	0.21			52.8	SURCHARGED	
1.008	17	0.181	0.000	0.20			51.0	SURCHARGED	

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow
9.000	18	15 Winter	30	+0%			
9.001	19	60 Winter	30	+0%	100/960 Winter		
10.000	20	15 Winter	30	+0%			
9.002	21	60 Winter	30	+0%	100/600 Winter		
11.000	22	15 Winter	30	+0%			
9.003	23	60 Winter	30	+0%	100/480 Winter		
12.000	24	15 Winter	30	+0%			
13.000	25	15 Winter	30	+0%			
9.004	26	60 Winter	30	+0%	100/360 Winter		
14.000	27	15 Winter	30	+0%			
15.000	28	15 Winter	30	+0%			
9.005	29	1440 Winter	30	+0%	100/240 Winter		
16.000	30	15 Winter	30	+0%	100/480 Winter		
17.000	31	15 Winter	30	+0%	100/480 Winter		
9.006	33	1440 Winter	30	+0%	100/180 Winter		
9.007	33	1440 Winter	30	+0%			
9.008	34	1440 Winter	30	+0%	100/120 Summer		
9.009	35	1440 Winter	30	+0%	100/60 Summer		
18.000	34	15 Winter	30	+0%	100/15 Summer		
18.001	35	15 Winter	30	+0%			
19.000	36	15 Winter	30	+0%	100/15 Summer		
19.001	37	30 Winter	30	+0%			
20.000	38	15 Winter	30	+0%			
21.000	39	15 Winter	30	+0%			
18.002	40	60 Winter	30	+0%			
22.000	41	15 Winter	30	+0%			
23.000	42	15 Winter	30	+0%			
18.003	43	60 Winter	30	+0%			
24.000	44	15 Winter	30	+0%			
25.000	45	15 Winter	30	+0%			
18.004	46	60 Winter	30	+0%			
26.000	47	15 Winter	30	+0%			
27.000	48	15 Winter	30	+0%			
18.005	49	60 Winter	30	+0%	100/1440 Winter		
28.000	50	15 Winter	30	+0%			
29.000	51	15 Winter	30	+0%			
28.001	52	1440 Winter	30	+0%	100/120 Summer		
9.010	54	1440 Winter	30	+0%	30/360 Winter		
9.011	56	1440 Winter	30	+0%	30/60 Summer	100/1440 Winter	
30.000	60	15 Winter	30	+0%	100/15 Summer		
30.001	61	15 Winter	30	+0%	30/15 Winter		
30.002	62	15 Winter	30	+0%	30/15 Summer		
1.009	63	1440 Winter	30	+0%	1/120 Winter		
31.000	62	15 Winter	30	+0%	100/15 Summer		
32.000	63	15 Winter	30	+0%			
31.001	64	15 Winter	30	+0%	100/15 Summer		
31.002	65	15 Winter	30	+0%			
31.003	65	15 Winter	30	+0%			

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Overflow Act.	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)
9.000	18		66.752	-0.083	0.000	0.08		7	5.1
9.001	19		65.412	-0.338	0.000	0.05			5.8
10.000	20		66.452	-0.088	0.000	0.07		7	4.9
9.002	21		65.405	-0.301	0.000	0.11			11.3
11.000	22		66.287	-0.073	0.000	0.10		7	5.2
9.003	23		65.397	-0.281	0.000	0.17			16.9
12.000	24		66.056	-0.094	0.000	0.08		7	5.0
13.000	25		66.056	-0.094	0.000	0.07		7	4.2
9.004	26		65.385	-0.264	0.000	0.24			27.8
14.000	27		65.756	-0.114	0.000	0.06		7	4.8
15.000	28		65.756	-0.114	0.000	0.05		7	4.1
9.005	29		65.367	-0.240	0.000	0.06			5.5
16.000	30		65.555	-0.145	0.000	0.05		7	4.7
17.000	31		65.550	-0.150	0.000	0.03		7	3.1
9.006	33		65.367	-0.196	0.000	0.08			7.1
9.007	33		65.367	-0.758	0.000	0.00		960	5.5
9.008	34		65.367	-0.082	0.000	0.01			4.8
9.009	35		65.366	-0.013	0.000	0.02			4.5
18.000	34		66.778	-0.042	0.000	0.51		8	26.1
18.001	35		66.145	-0.155	0.000	0.46			26.8
19.000	36		66.789	-0.046	0.000	0.35		8	18.8
19.001	37		66.121	-0.329	0.000	0.16			21.3
20.000	38		66.490	-0.060	0.000	0.19		9	12.5
21.000	39		66.490	-0.060	0.000	0.21		9	13.2
18.002	40		66.018	-0.188	0.000	0.56			56.5
22.000	41		66.093	-0.207	0.000	0.07		8	14.3
23.000	42		66.088	-0.212	0.000	0.05		8	9.4
18.003	43		65.991	-0.186	0.000	0.65			64.8
24.000	44		65.993	-0.167	0.000	0.09		8	14.5
25.000	45		65.993	-0.167	0.000	0.06		8	9.8
18.004	46		65.878	-0.171	0.000	0.70			81.8
26.000	47		65.636	-0.249	0.000	0.01		6	10.2
27.000	48		65.636	-0.249	0.000	0.02		6	14.6
18.005	49		65.561	-0.213	0.000	0.54			66.9
28.000	50		65.571	-0.429	0.000	0.01		8	7.8
29.000	51		65.571	-0.429	0.000	0.01		8	9.9
28.001	52		65.366	-0.084	0.000	0.07		1104	6.5
9.010	54		65.366	0.094	0.000	0.05			20.5
9.011	56		65.366	0.335	0.000	0.05			9.6
30.000	60		65.683	-0.067	0.000	0.52			289.9
30.001	61		65.562	0.037	0.000	0.86			459.6
30.002	62		65.452	0.077	0.000	1.23			670.8
1.009	63		65.366	0.597	0.000	0.23			20.4
31.000	62		65.795	-0.155	0.000	0.22			31.3
32.000	63		65.789	-0.261	0.000	0.24			33.3
31.001	64		65.751	-0.092	0.000	0.98			94.2
31.002	65		65.581	-0.238	0.000	0.43			119.8

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Overflow Act.	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)
31.003	65		64.837	-0.243	0.000	0.42			116.2

PN	US/MH Name	Status	Level Exceeded
9.000	18	OK	
9.001	19	OK	
10.000	20	OK	
9.002	21	OK	
11.000	22	OK	
9.003	23	OK	
12.000	24	OK	
13.000	25	OK	
9.004	26	OK	
14.000	27	OK	
15.000	28	OK	
9.005	29	OK	
16.000	30	OK	
17.000	31	OK	
9.006	33	OK	
9.007	33	OK	
9.008	34	OK*	
9.009	35	OK	
18.000	34	OK	
18.001	35	OK	
19.000	36	OK	
19.001	37	OK	
20.000	38	OK	
21.000	39	OK	
18.002	40	OK	
22.000	41	OK	
23.000	42	OK	
18.003	43	OK	
24.000	44	OK	
25.000	45	OK	
18.004	46	OK	
26.000	47	OK	
27.000	48	OK	
18.005	49	OK	
28.000	50	OK	
29.000	51	OK	
28.001	52	OK	
9.010	54	SURCHARGED	
9.011	56	SURCHARGED	
30.000	60	OK	
30.001	61	SURCHARGED	

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Status	Level Exceeded
30.002	62	SURCHARGED	
1.009	63	SURCHARGED	
31.000	62	OK	
32.000	63	OK	
31.001	64	OK	
31.002	65	OK	
31.003	65	OK	

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)
31.004	66	720 Winter	30	+0%	30/60 Winter				64.547
31.005	66	720 Winter	30	+0%					64.547
33.000	67	720 Winter	30	+0%	30/180 Winter				64.547
34.000	68	720 Winter	30	+0%	30/180 Winter				64.547
33.001	69	720 Winter	30	+0%	30/30 Winter				64.547
31.006	70	720 Winter	30	+0%	1/15 Summer				64.547

PN	US/MH Name	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status	Level Exceeded
31.004	66	0.206	0.000	0.10			10.0	SURCHARGED	
31.005	66	-1.753	0.000	0.00			2.3	OK	
33.000	67	0.097	0.000	0.02			3.2	SURCHARGED	
34.000	68	0.097	0.000	0.02			2.6	SURCHARGED	
33.001	69	0.272	0.000	0.05			4.6	SURCHARGED	
31.006	70	0.691	0.000	0.23			1.6	SURCHARGED	

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 29
Number of Online Controls 2 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.401
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0 DVD Status ON
Analysis Timestep Fine Inertia Status ON
DTS Status OFF

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 180, 240, 360, 480, 600, 720, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 40

WARNING: Half Drain Time has not been calculated as the structure is too full.

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)
1.000	1	30 Winter	100	+40%	100/15 Summer				67.102
2.000	2	30 Winter	100	+40%	100/15 Summer				67.069
3.000	3	30 Winter	100	+40%	100/15 Winter				67.062
1.001	4	30 Winter	100	+40%	100/15 Summer				67.037
4.000	5	30 Winter	100	+40%	100/15 Winter				66.748
1.002	6	30 Winter	100	+40%	100/15 Summer				66.726
5.000	7	30 Winter	100	+40%	100/30 Winter				66.428
1.003	8	30 Winter	100	+40%	100/15 Summer				66.406
6.000	9	15 Winter	100	+40%	100/15 Summer				66.665
7.000	11	15 Winter	100	+40%	30/15 Summer				66.792
6.001	14	15 Winter	100	+40%	30/15 Summer				66.479
8.000	15	15 Winter	100	+40%					66.980
1.004	16	960 Winter	100	+40%					66.112
1.005	17	960 Winter	100	+40%	100/15 Winter				66.100
1.006	15	720 Winter	100	+40%	100/30 Summer				66.191
1.007	16	720 Winter	100	+40%	30/360 Winter				66.177
1.008	17	960 Winter	100	+40%	30/180 Winter				66.063

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status	Level Exceeded
1.000	1	0.802	0.000	0.96			300.4	SURCHARGED	
2.000	2	0.769	0.000	0.94			260.9	SURCHARGED	
3.000	3	0.762	0.000	0.78			209.7	SURCHARGED	
1.001	4	1.087	0.000	1.56			470.1	FLOOD RISK	
4.000	5	0.448	0.000	0.66			179.4	SURCHARGED	
1.002	6	0.907	0.000	2.41			565.6	SURCHARGED	
5.000	7	0.128	0.000	0.66			179.4	SURCHARGED	
1.003	8	0.631	0.000	2.18			670.0	SURCHARGED	
6.000	9	0.140	0.000	1.02			316.6	SURCHARGED	
7.000	11	0.692	0.000	1.54			462.6	SURCHARGED	
6.001	14	0.504	0.000	2.52			773.1	SURCHARGED	
8.000	15	-0.020	0.000	0.10		16	89.4	FLOOD RISK	
1.004	16	-0.888	0.000	0.00			140.8	OK	
1.005	17	0.549	0.000	0.44			111.0	SURCHARGED	
1.006	15	0.761	0.000	0.53			134.0	FLOOD RISK	
1.007	16	0.868	0.000	0.52			132.5	FLOOD RISK	
1.008	17	0.875	0.000	0.42			106.4	FLOOD RISK	

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow
9.000	18	15 Winter	100	+40%			
9.001	19	1440 Winter	100	+40%	100/960 Winter		
10.000	20	15 Winter	100	+40%			
9.002	21	1440 Winter	100	+40%	100/600 Winter		
11.000	22	15 Winter	100	+40%			
9.003	23	1440 Winter	100	+40%	100/480 Winter		
12.000	24	15 Winter	100	+40%			
13.000	25	15 Winter	100	+40%			
9.004	26	1440 Winter	100	+40%	100/360 Winter		
14.000	27	1440 Winter	100	+40%			
15.000	28	1440 Winter	100	+40%			
9.005	29	1440 Winter	100	+40%	100/240 Winter		
16.000	30	1440 Winter	100	+40%	100/480 Winter		
17.000	31	1440 Winter	100	+40%	100/480 Winter		
9.006	33	1440 Winter	100	+40%	100/180 Winter		
9.007	33	1440 Winter	100	+40%			
9.008	34	1440 Winter	100	+40%	100/120 Summer		
9.009	35	960 Winter	100	+40%	100/60 Summer		
18.000	34	15 Winter	100	+40%	100/15 Summer		
18.001	35	15 Winter	100	+40%			
19.000	36	15 Winter	100	+40%	100/15 Summer		
19.001	37	60 Winter	100	+40%			
20.000	38	15 Winter	100	+40%			
21.000	39	15 Winter	100	+40%			
18.002	40	60 Summer	100	+40%			
22.000	41	15 Winter	100	+40%			
23.000	42	15 Winter	100	+40%			
18.003	43	60 Winter	100	+40%			
24.000	44	15 Winter	100	+40%			
25.000	45	15 Winter	100	+40%			
18.004	46	60 Winter	100	+40%			
26.000	47	1440 Winter	100	+40%			
27.000	48	1440 Winter	100	+40%			
18.005	49	1440 Winter	100	+40%	100/1440 Winter		
28.000	50	1440 Winter	100	+40%			
29.000	51	1440 Winter	100	+40%			
28.001	52	1440 Winter	100	+40%	100/120 Summer		
9.010	54	960 Winter	100	+40%	30/360 Winter		
9.011	56	960 Winter	100	+40%	30/60 Summer	100/1440 Winter	
30.000	60	15 Winter	100	+40%	100/15 Summer		
30.001	61	15 Winter	100	+40%	30/15 Winter		
30.002	62	15 Winter	100	+40%	30/15 Summer		
1.009	63	960 Winter	100	+40%	1/120 Winter		
31.000	62	15 Summer	100	+40%	100/15 Summer		
32.000	63	15 Summer	100	+40%			
31.001	64	15 Summer	100	+40%	100/15 Summer		
31.002	65	15 Winter	100	+40%			
31.003	65	15 Winter	100	+40%			

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Overflow Act.	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)
9.000	18		66.773	-0.062	0.000	0.19		8	11.6
9.001	19		65.783	0.033	0.000	0.01			1.4
10.000	20		66.473	-0.067	0.000	0.17		8	11.1
9.002	21		65.783	0.077	0.000	0.03			2.7
11.000	22		66.308	-0.052	0.000	0.24		8	12.0
9.003	23		65.783	0.104	0.000	0.04			4.0
12.000	24		66.079	-0.071	0.000	0.18		8	11.3
13.000	25		66.079	-0.071	0.000	0.14		8	9.3
9.004	26		65.783	0.134	0.000	0.06			6.7
14.000	27		65.781	-0.089	0.000	0.02		744	1.4
15.000	28		65.781	-0.089	0.000	0.02		744	1.4
9.005	29		65.783	0.176	0.000	0.10			9.4
16.000	30		65.782	0.082	0.000	0.01			1.4
17.000	31		65.781	0.081	0.000	0.01			1.2
9.006	33		65.783	0.220	0.000	0.11			10.3
9.007	33		65.783	-0.342	0.000	0.00			6.3
9.008	34		65.787	0.338	0.000	0.01			4.6
9.009	35		65.895	0.516	0.000	0.02			5.2
18.000	34		66.829	0.009	0.000	1.14		8	58.2
18.001	35		66.279	-0.021	0.000	0.98			56.4
19.000	36		66.837	0.002	0.000	0.85		8	45.7
19.001	37		66.229	-0.221	0.000	0.25			33.2
20.000	38		66.523	-0.027	0.000	0.45		8	28.8
21.000	39		66.522	-0.028	0.000	0.48		8	30.6
18.002	40		66.206	0.000	0.000	1.09			109.5
22.000	41		66.133	-0.167	0.000	0.13		9	26.7
23.000	42		66.126	-0.174	0.000	0.09		9	17.3
18.003	43		66.080	-0.097	0.000	0.97			97.0
24.000	44		66.033	-0.127	0.000	0.20		9	32.2
25.000	45		66.033	-0.127	0.000	0.08		9	13.0
18.004	46		65.948	-0.102	0.000	0.96			111.2
26.000	47		65.797	-0.088	0.000	0.01		984	5.6
27.000	48		65.796	-0.089	0.000	0.01		1008	6.5
18.005	49		65.802	0.027	0.000	0.27			33.4
28.000	50		65.780	-0.220	0.000	0.00			3.4
29.000	51		65.781	-0.219	0.000	0.00			3.4
28.001	52		65.787	0.337	0.000	0.06			5.4
9.010	54		65.921	0.649	0.000	0.07			26.0
9.011	56		66.011	0.980	0.000	0.09			17.8
30.000	60		66.928	1.178	0.000	0.88			487.2
30.001	61		66.756	1.231	0.000	1.70			905.8
30.002	62		66.299	0.924	0.000	2.36			1294.4
1.009	63		66.034	1.265	0.000	0.23			20.4
31.000	62		65.970	0.020	0.000	0.40			57.5
32.000	63		65.982	-0.068	0.000	0.42			57.7
31.001	64		65.921	0.077	0.000	1.94			186.9
31.002	65		65.709	-0.110	0.000	0.85			235.0

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Overflow Act.	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)
31.003	65		64.954	-0.126	0.000	0.80			222.1

PN	US/MH Name	Status	Level Exceeded
9.000	18	OK	
9.001	19	SURCHARGED	
10.000	20	OK	
9.002	21	SURCHARGED	
11.000	22	OK	
9.003	23	SURCHARGED	
12.000	24	OK	
13.000	25	OK	
9.004	26	SURCHARGED	
14.000	27	OK	
15.000	28	OK	
9.005	29	SURCHARGED	
16.000	30	FLOOD RISK	
17.000	31	FLOOD RISK	
9.006	33	SURCHARGED	
9.007	33	OK	
9.008	34	SURCHARGED*	
9.009	35	FLOOD RISK	
18.000	34	FLOOD RISK	
18.001	35	OK	
19.000	36	FLOOD RISK	
19.001	37	OK	
20.000	38	OK	
21.000	39	OK	
18.002	40	OK	
22.000	41	OK	
23.000	42	OK	
18.003	43	OK	
24.000	44	OK	
25.000	45	OK	
18.004	46	OK	
26.000	47	OK	
27.000	48	OK	
18.005	49	SURCHARGED	
28.000	50	FLOOD RISK	
29.000	51	FLOOD RISK	
28.001	52	FLOOD RISK	
9.010	54	FLOOD RISK	
9.011	56	FLOOD RISK	
30.000	60	SURCHARGED	
30.001	61	SURCHARGED	

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Status	Level Exceeded
30.002	62	SURCHARGED	
1.009	63	SURCHARGED	
31.000	62	SURCHARGED	
32.000	63	OK	
31.001	64	SURCHARGED	
31.002	65	OK	
31.003	65	OK	

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.	Water Level (m)
31.004	66	1440 Winter	100	+40%	30/60 Winter				64.939
31.005	66	1440 Winter	100	+40%					64.939
33.000	67	1440 Winter	100	+40%	30/180 Winter				64.939
34.000	68	1440 Winter	100	+40%	30/180 Winter				64.939
33.001	69	1440 Winter	100	+40%	30/30 Winter				64.939
31.006	70	1440 Winter	100	+40%	1/15 Summer				64.939

PN	US/MH Name	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Half Drain Time (mins)	Pipe Flow (l/s)	Status	Level Exceeded
31.004	66	0.598	0.000	0.11			10.5	SURCHARGED	
31.005	66	-1.361	0.000	0.00			1.7	OK	
33.000	67	0.489	0.000	0.02			3.4	SURCHARGED	
34.000	68	0.489	0.000	0.02			2.8	SURCHARGED	
33.001	69	0.664	0.000	0.06			6.0	SURCHARGED	
31.006	70	1.083	0.000	0.27			1.8	SURCHARGED	

STORM WATER / SUDs MAINTENANCE PLAN
SYMMETRY PARK, OXFORD NORTH

1. Introduction

This drainage maintenance proposal is to be read in conjunction with the following documents:

- Drainage drawing Ref Nos. T/21/2407 55-01

The considered drainage solution comprises of two attenuated networks with restricted discharge rates via vortex flow control devices to an arterial drainage system prior to discharging into an existing watercourse to the eastern boundary of the proposed development.

The proposed surface water networks are designed to accommodate all storm events up to the 1 in 100 year storm plus an additional 40% to cater for climatic change.

A plan of routine inspection maintenance should be adopted and adhered to in order to prevent failure due to inadequate maintenance. This document describes the drainage systems used and provides a framework for future maintenance procedures.

2. Site Drainage Components

The site drainage network is shown on Tier Consult drawing reference T/21/2407 55-01. The main drainage components are;

1. Roof water from the building is collected into a gravity fed drainage system and routed to the surface water drains. The gutters and downspouts require periodic inspection and de-silting as required.

2. Surface water runoff from external paved areas is discharged into trapped gullies and linear channel. Gullies and channels require periodic inspection and de-silting as required.

3. The pavement areas which consist of service yards, roads and car parks require periodic sweeping as this will remove silt and contaminants directly from the paved surfaces before they become mobilised during rainfall events and transported into the drainage system.

4. Porous paved surfaces receive rainwater run-off from the carpark areas of development. Surface water infiltrates through to the subbase where a perforated collection pipe conveys surface water flows into the wider drainage system. This system requires periodic inspection and cleaning to remove litter, dust and leaf fall.

5. Oil separators remove oil-based pollutants before the flow is discharged from the site. Requires routine de-sludging.

6. The storage tank is formed with a modular crate system wrapped with an impermeable membrane to prevent escape of water and ingress of soil particles. The storage tank requires periodic inspection and possible de-silting if required.

7. The vortex flow control unit limits the discharge of surface water to the receiving drain at a predetermined rate. The chamber housing the control unit requires periodic inspection to check for any siltation and the vortex flow control unit should be checked for any blockages and to ensure it is working correctly.

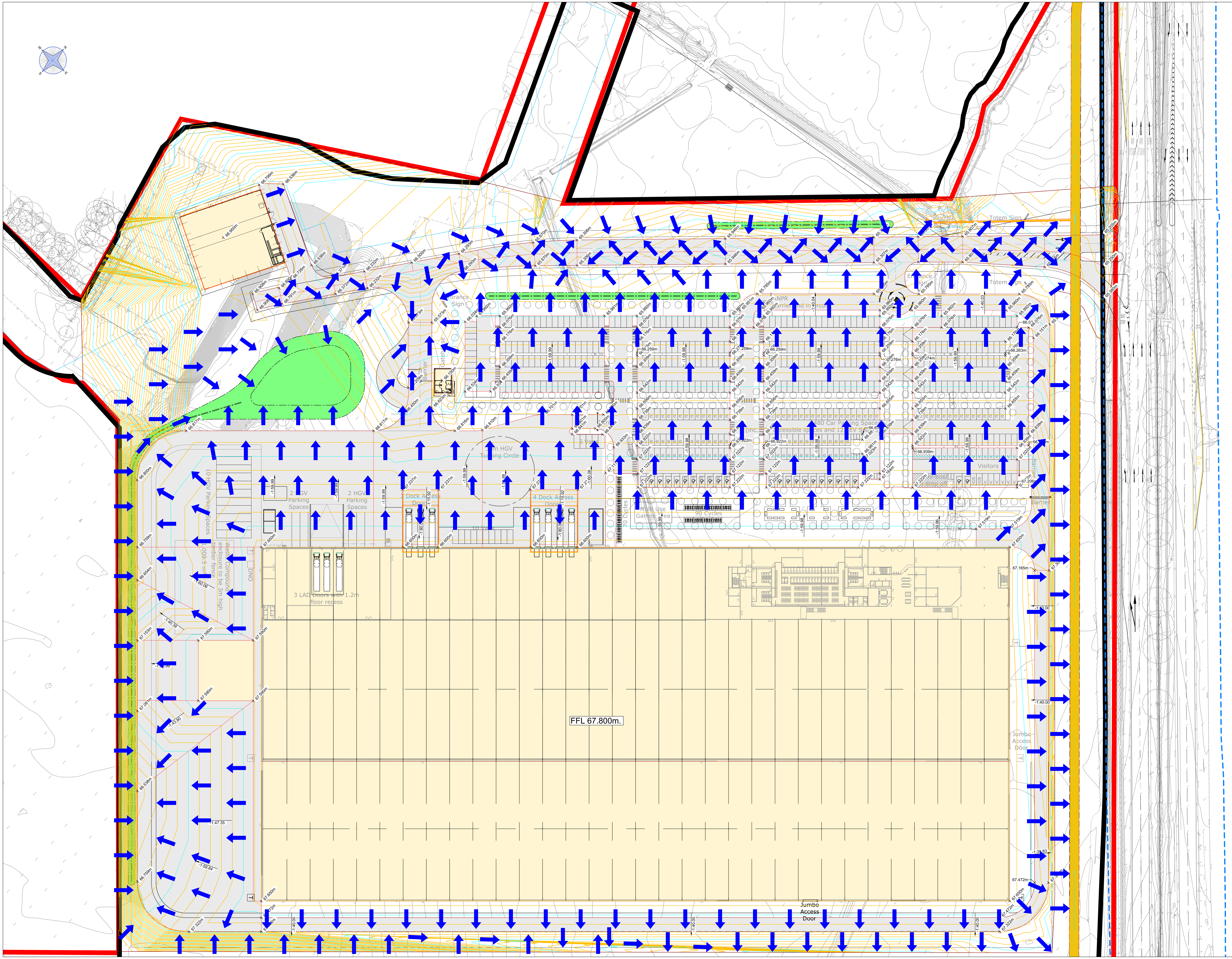
3. Maintenance Schedule

The rate of build up of silt and debris within a drainage system varies from site to site and is dependent upon individual site characteristics. Therefore, the frequency of actions below should be adopted as a minimum standard for a period of 24 months after the completion of the development. This period will be sufficient to assess the system performance over 2 complete seasonal cycles after which the maintenance activity schedule may be reviewed accordingly.

Action	Frequency
Clear external areas of litter including bin and recycling enclosures	Monthly
Clear guttering of leaves and debris.	Twice yearly. Spring and Autumn after leaf fall
Permeable Surfaces	3 times per year to remove debris, dust and leaves. See recommendations above.
Inspect all storage tank access points for sediment.	6 Monthly and after heavy rainfall. Remove debris /silt as required.
Petrol/oil Separator	Inspect bi-annually and also when alerted by the audible/visual alarms. Remove oil and contaminants
Inspect all manholes chambers for siltation and debris	6 monthly and after every major storm event. Remove debris/silt as encountered.
Vortex Flow Control Unit	3 monthly inspect and remove debris.
Catch pits	Minimum 6 monthly and after every major storm event. Remove debris/silt as encountered.
Storage Tank	6 monthly to check for blockages and after every main storm event. The tank can be inspected via the access points and CCTV cameras and high pressure jetting equipment can be deployed if required.

4. Management Company

The majority of the maintenance for the drainage features on each plot will be the responsibility of Tritax Symmetry, Grange Park Court, Roman Way, Northampton Tel; 01604 330630. However, an overall management company will be appointed to manage the maintenance regime for the wider drainage features such as ponds and outfalls. Personnel will be site based and where specialised contractors are required the work will be undertaken through a series of written RAMS.



Notes

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4. ALL LEVELS ARE IN METRES UNO & TO OS DATUM.

Legend

EXPECTED ROUTE OF FLOOD WATER IN EVENT OF SYSTEM FAILURE

Rev	Date	By	Description	Appd
P1	09.11.21	DRL	Issue for Information	DRL

Revisions

Status

PRELIMINARY

Tier Consult Ltd.
Richmond House
Sandpiper Court
Chester Business Park
Chester | CH4 9QZ
t: 01244 684900

Client

TRITAX SYMMETRY

Project

SYMMETRY PARK, OXFORD NORTH

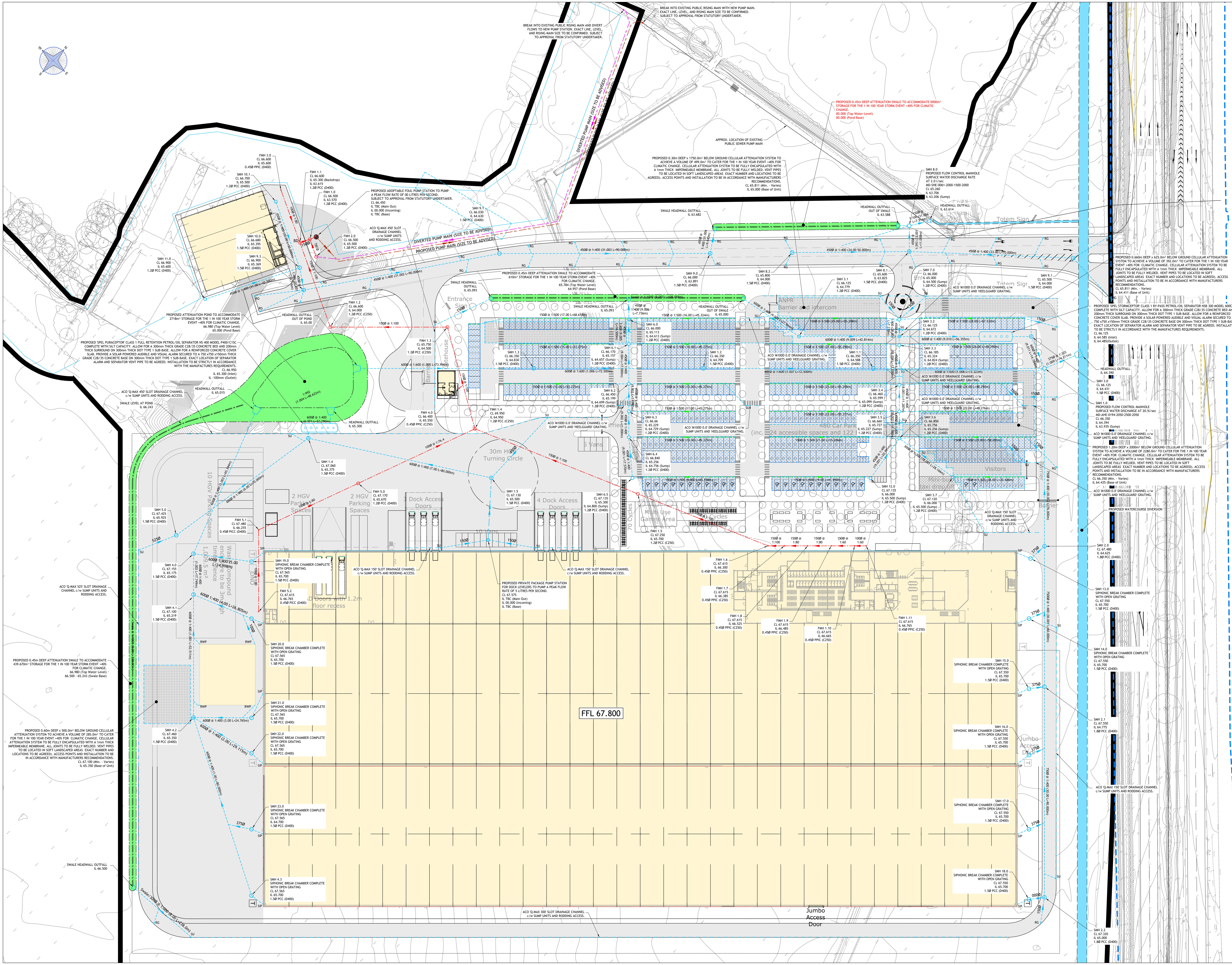
Title

FLOOD EXCEEDANCE ROUTE

Scale 1:500@A0
Drawn DRL
Revision P1
Date Nov'21
Checked PJB

Drawing Ref :
Project no|Originator|Volume|Level|Type|Role|Number

T/20/2407 51-01



Notes

1. THE WORKS SHALL BE IN ACCORDANCE WITH THE NATIONAL BUILDING SPECIFICATION.
2. THE LOCATION LINE & LEVEL OF ALL KNOWN EXISTING DRAINAGE PIPEWORK INDICATED ON THE DRAWINGS ARE APPROXIMATE AND FOR GUIDANCE PURPOSES ONLY.
3. IT IS THE CONTRACTORS RESPONSIBILITY TO DETERMINE THEIR EXACT LINE AND LEVEL, BY WAY OF HAND EXCAVATED TRIAL PITS, PRIOR TO THE COMMENCEMENT OF ANY EXCAVATION WORKS ON SITE. THE CONTRACTOR SHALL TAKE ALL NECESSARY PRECAUTIONS AND MAINTAIN THE STRUCTURAL INTEGRITY OF ALL ABOVE AND BELOW GROUND SERVICE INSTALLATIONS.
4. THE CONTRACTOR SHALL IMPLEMENT AND MAINTAIN THROUGHOUT THE DURATION OF THE CONTRACT A FULL TRAFFIC MANAGEMENT SYSTEM TO ENSURE SAFE PASSAGE OF VEHICLES/PEDESTRIANS IN THE VICINITY OF THE WORKS. ALL TRAFFIC SIGNS, SIGNALS, BARRIERS ETC. SHALL BE IN ACCORDANCE WITH CHAPTER 8 OF THE TRAFFIC SIGNS MANUAL.
5. THE CONTRACTOR SHALL MAKE ADEQUATE PROVISION FOR DEALING WITH AND DISPOSING OF GROUND/SURFACE WATER ENCOUNTERED DURING EXCAVATIONS
6. ALL SOFT / HARD PAVED AREAS AFFECTED BY THE WORKS SHALL BE FULLY REINSTATED FOLLOWING THE INSTALLATION OF ALL DRAINAGE WORKS. ALL SURFACE MARKINGS AFFECTED BY THE WORKS SHALL BE FULLY REINSTATED.
7. ALL SURPLUS EXCAVATED MATERIAL SHALL BE DISPOSED IN A DESIGNATED AREA ON SITE.
8. UPON COMPLETION OF THE WORKS THE CONTRACTOR SHALL PROVIDE AS BUILT SETTING OUT CO-ORDINATE AND LEVEL INFORMATION.
9. PIPEWORK WITH LESS THAN 750mm OF COVER IN TRAFFICKED AREAS TO BE CONCRETE ENCASED. (BEDDING CLASS Z)
10. PIPEWORK WITH MORE THAN 750mm OF COVER TO HAVE CLASS S GRANULAR BED AND SURROUND.
11. THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL OTHER RELEVANT ARCHITECT'S, ENGINEER'S AND SPECIALIST'S DRAWINGS TOGETHER WITH THE APPROPRIATE SPECIFICATION.
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16. ALL LEVELS ARE IN METRES UNO & TO OS DATUM.

Legend

PROPOSED SURFACE WATER DRAINAGE (TYPICALLY 150Ø UNO.)
PROPOSED SURFACE WATER PRECAST CONCRETE MANHOLE.
SD PROPOSED SIPHONIC DOWNSPOUT
RWP PROPOSED RAINWATER DOWNSPOUT
SU PROPOSED DRAINAGE CHANNEL (TYPE AS NOTED) WITH SUMP OUTLET.
RG PROPOSED TRAPPED ROAD GULLY
PROPOSED SURFACE WATER PERMEABLE SURFACEING FULLY TANKED WITH COLLECTING PERFORATED FILTER DRAIN AND 300mm THICK SUB-BASE DfT TYPE 3.
PROPOSED SURFACE WATER CELLULAR ATTENUATION (STORAGE VOLUMES AS NOTED).
PROPOSED ROUTE WATERCOURSE DIVERSION.
PROPOSED SURFACE WATER DRAINAGE SWALES.
PROPOSED FOUL DRAINAGE (TYPICALLY 100Ø UNO.)
PROPOSED FOUL PRECAST CONCRETE MANHOLE.
PROPOSED FOUL POLYPROPYLENE INSPECTION CHAMBERS.
APPROX. ROUTE OF EXISTING PUBLIC SEWER FOUL RISING MAIN
PM PROPOSED DIVERTED PUBLIC SEWER FOUL RISING MAIN
PM PROPOSED FOUL RISING MAIN

NOTES

1. SIPHONIC DOWNSPOUT LOCATIONS AND SIZES ARE SUBJECT TO CONFIRMATION FROM THE ARCHITECT/ SPECIALIST CONSULTANTS.
2. FOUL OUTLET LOCATIONS ARE SUBJECT TO CONFIRMATION FROM THE ARCHITECT/M&E CONSULTANT.

Rev	Date	By	Description	Appd
P3	09.11.21	DRL	Drainage details added.	DRL
P2	14.10.21	DRL	Swale added. Drainage amended.	DRL
P1	05.10.21	DRL	Preliminary issue for information only	DRL

Revisions

Status

PRELIMINARY

Tier Consult Ltd.
Richmond House
Sandpiper Court
Chester Business Park
Chester | CH4 9QZ
t: 01244 684900

Client

TRITAX SYMMETRY

Project

SYMMETRY PARK, OXFORD NORTH

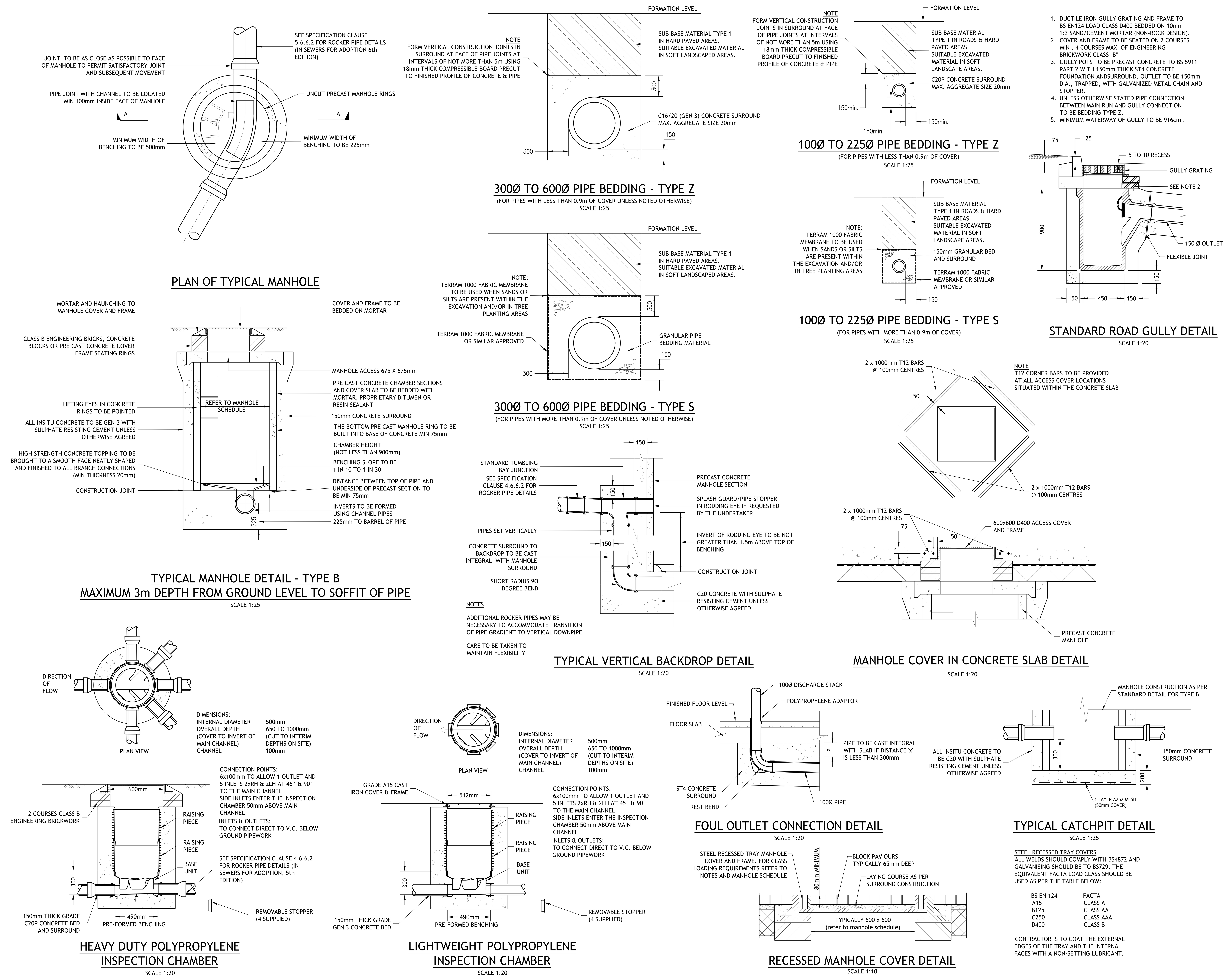
Title

PROPOSED DRAINAGE LAYOUT

Scale 1:500@A0
Drawn DRL
Date Sept'21
Checked PJB
Revision P3

Drawing Ref :
Project no|Originator|Volume|Level|Type|Role|Number

T/20/2407 55-01



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Rev	Date	By	Description	Appd
P1	09-11-21	DRL	Issued for Information	PJB

Revisions

Status

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Tier Consult Ltd.
Richmond House
Sandpiper Court
Chester Business Park
Chester | CH4 9QZ
t: 01244 684900

Client

TRITAX SYMMETRY

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SYMMETRY PARK, OXFORD NORTH

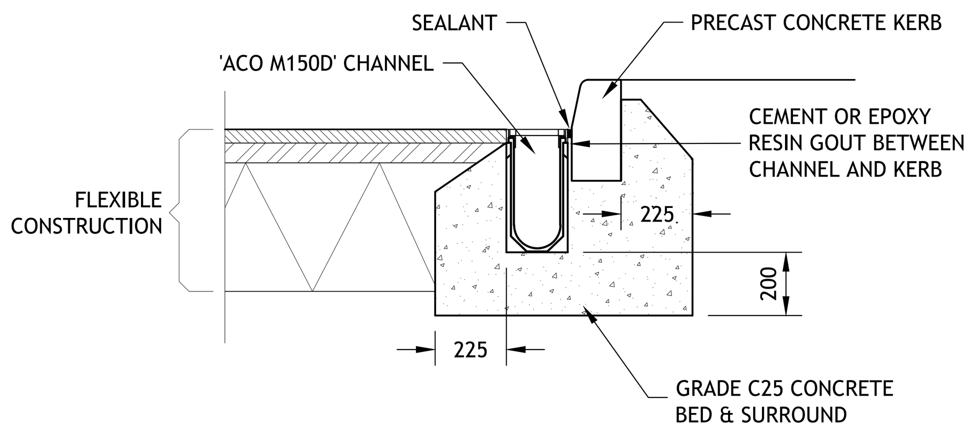
Title

DRAINAGE CONSTRUCTION DETAILS - SHEET 1

Scale	Drawn	Revision
As Shown @A1	DRL	P1
Date: Oct'21	Checked: JH	

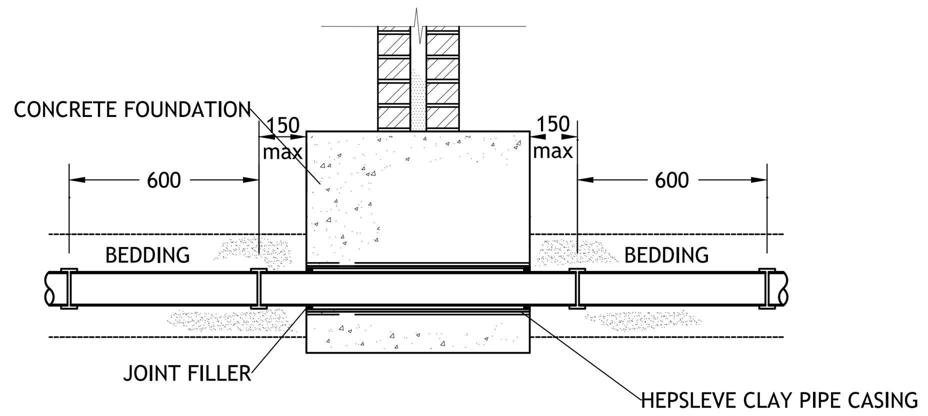
Drawing Ref :
Project no|Originator|Volume|Level|Type|Role|Number

T/21/2407 55-02



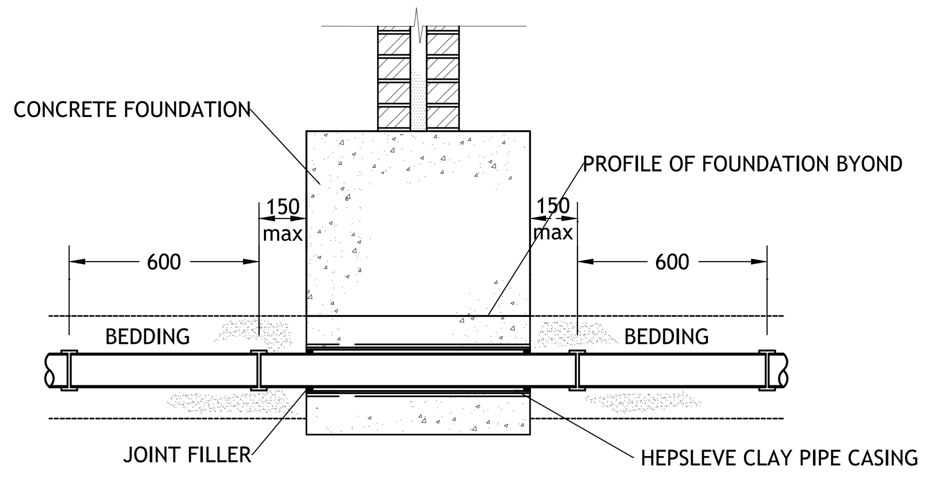
'ACO M150D' CHANNEL DETAIL ADJACENT TO KERB

SCALE 1:20



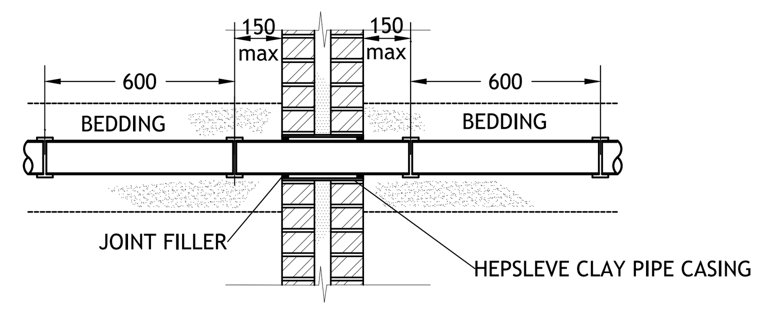
DRAIN PASSING THROUGH FOUNDATION

SCALE 1:20



DRAIN PASSING BELOW FOUNDATION

(Encase in concrete below footing)



DRAIN PASSING THROUGH WALLS

NOTES

1. When foundation is more than 450mm above drain do not encase in concrete.
2. Joint Filler to be plastazote or similarly approved.
3. 12mm diameter chord for 100mm drainage pipes with 150mm diameter casing.
4. 25mm diameter chord filler for 150mm drainage pipes with 225mm diameter casing.
5. Where drainage pipes under buildings have less than 300mm cover they shall have 150mm concrete encasement.

Notes

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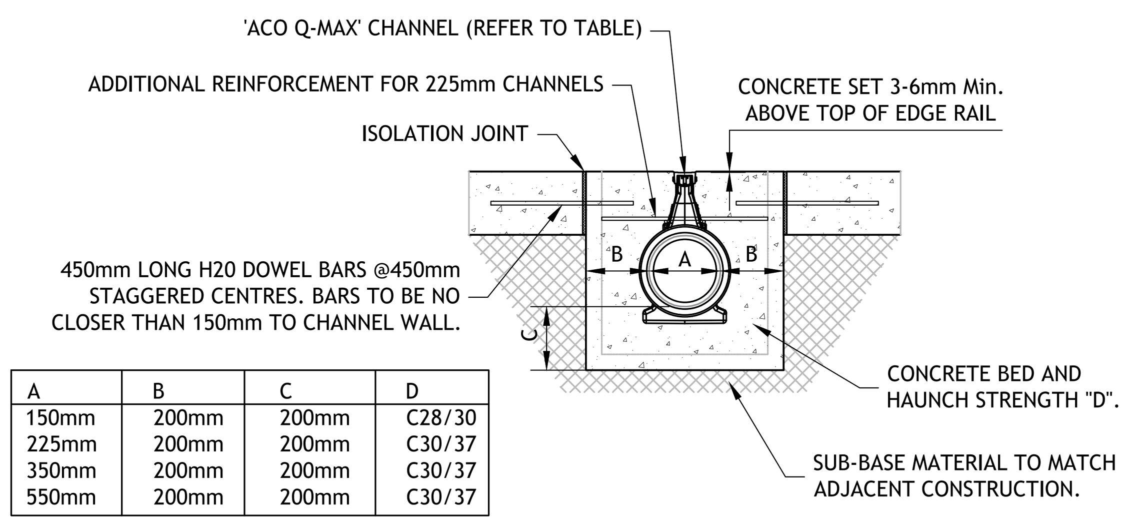
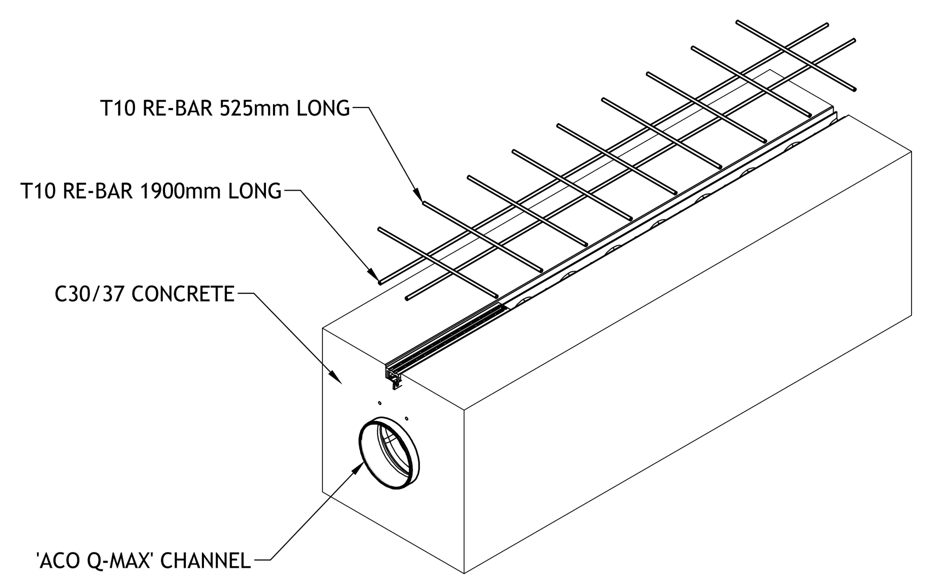


A	B	C	D
150mm	200mm	200mm	C28/30
225mm	200mm	200mm	C30/37
350mm	200mm	200mm	C30/37
550mm	200mm	200mm	C30/37

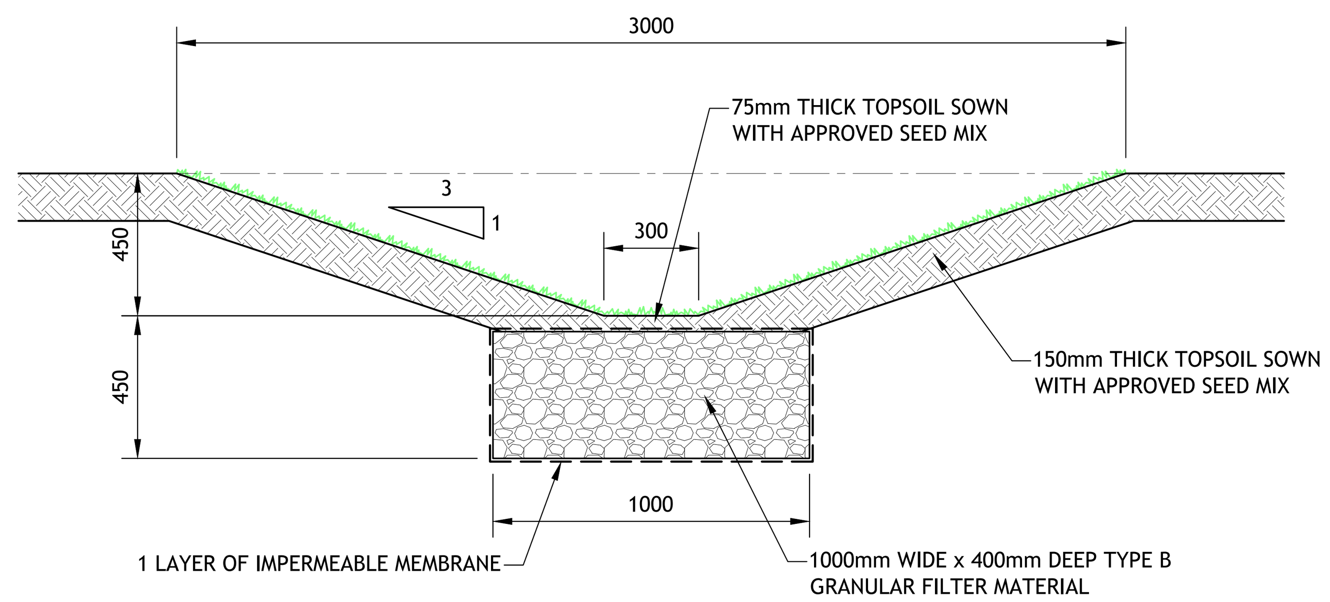
'ACO Q-MAX' SLOTDRAIN DETAIL

SCALE 1:20



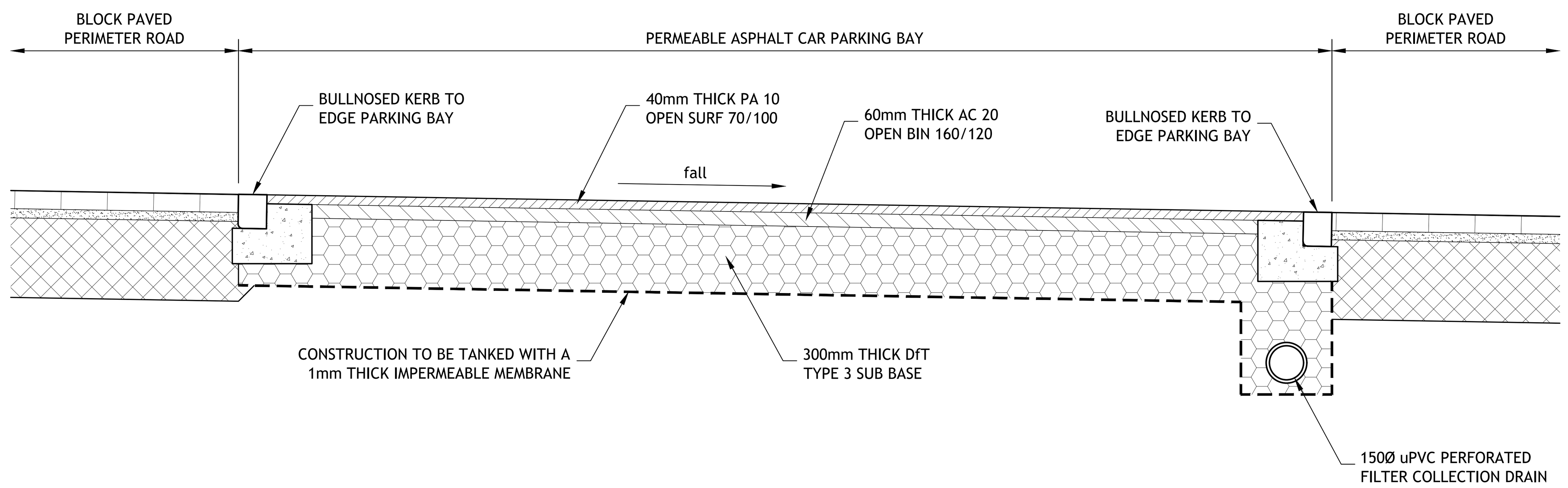
'ACO Q-MAX' ADDITIONAL REINFORCEMENT DETAIL

SCALE 1:10



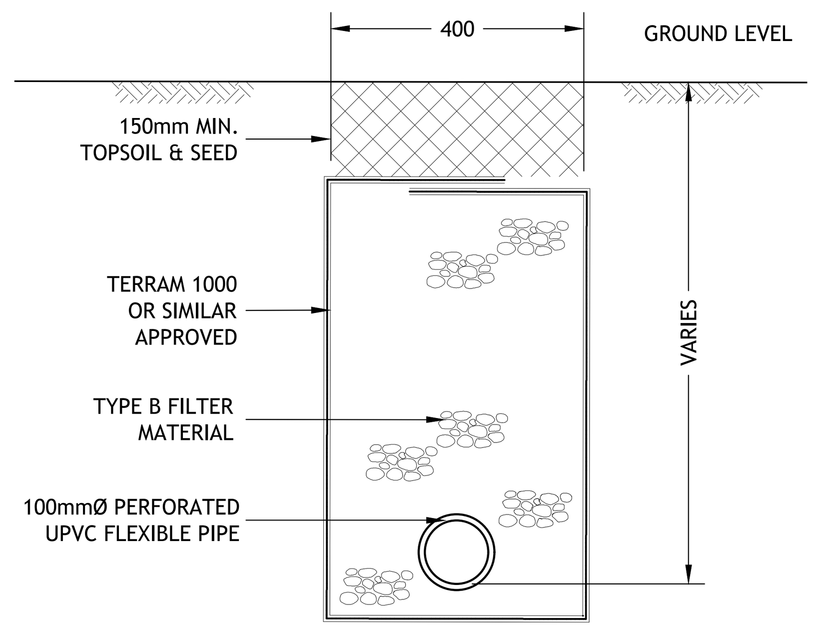
SECTION THROUGH SWALE

SCALE 1:20



TYPICAL SECTION THROUGH PERMEABLE ASPHALT

SCALE 1:20



FILTER DRAIN

SCALE 1:10

Rev	Date	By	Description	Appd
P1	09-11-21	DRL	Issued for Information	PJB

Revisions

Status

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Tier Consult Ltd.
Richmond House
Sandpiper Court
Chester Business Park
Chester | CH4 9QZ
t: 01244 684900

Client

TRITAX SYMMETRY

Project

SYMMETRY PARK, OXFORD NORTH

Title

DRAINAGE CONSTRUCTION DETAILS - SHEET 2

Scale	Drawn	Revision
As Shown @A1	DRL	P1
Date: Oct'21	Checked: JH	

Drawing Ref :
Project no|Originator|Volume|Level|Type|Role|Number

T/21/2407 55-03