



**Rhodes Business Park
Silburn Way
Middleton
Manchester
M24 4NE**

**Flood Risk Assessment
& Drainage Strategy Report**

for

OXENWOOD REAL ESTATE

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1.0 EXECUTIVE SUMMARY

Development comprising the demolition of existing buildings and structures and the erection of a new building for Class B2/B8 (storage / distribution), including ancillary offices, landscaping, yards, alterations to existing access from Silburn Way, car parking, cycle parking and associated engineering works.

The site is located at along Silburn Way within an existing industrial development, Middleton, Manchester, M24 4NE.

A review of the site topographical & GPR survey indicates that the surface water drainage from the existing industrial site currently outfalls unrestricted to the River Irke, with an existing foul water spur serving the site along the northern boundary.

In accordance with the National Planning Policy Framework and the Greater Manchester City Strategic Flood Risk Assessment (SFRA), this report has studied and assessed the flood risk to the site by all sources. A review of the EA Flood Maps shows that the site is located within Flood Zone 3. It is to be noted that the EA flood maps show flood defences along the north bank of the River Irk close to the western boundary of the development site.

A review of the planning portal has shown that the recent application for the residential development, located to the west of the proposed development site, has identified flood defences along the River Irk as part of their strategy. A review of the topographical levels of the defences and the development site suggests the defences continue along the northern bank of the River Irk to the east of the development site. A flood map challenge has been commissioned and undertaken by JBA Consultants, on behalf of the client, and the report is attached in the Appendix B.

The Finish Floor Levels of the proposed development building have been set at 70.900 AOD which is above the EA 1:100 year flood event levels of 70.42 AOD.

The site use is defined as 'less vulnerable' such that the development can be considered 'appropriate' in accordance with NPPF guidelines. As the site is located within Flood Zone 3, the sequential and exception tests may required by the local authority. Further to this, the site is considered to be at 'low-medium risk' from all other sources of flooding; tidal, fluvial, pluvial, sewer, groundwater and artificial sources.

The surface water drainage strategy, as outlined below, is to restrict the proposed outflow rate to satisfy both Greater Manchester Area & the Rochdale SFRA and to ensure that the flood risk to the site and surrounding catchment is not increased by the proposed development. The Sustainable Drainage Systems (SuDS) hierarchy has been considered in the drainage strategy discussed in Section 5. The surface water runoff will be collected from the impermeable areas and directed via the underground network to a geocellular attenuation tank located beneath the main service yard area before connecting to the existing surface water sewer via an existing connection, within the development site, at a restricted rate of 4.5l/s (Q_{BAR}). This overall approach aligns with the Sustainable Drainage Systems (SuDS) policy and provides a significant reduction in the pre-development unrestricted discharge rates. A detailed Drainage layout with hydraulic calculations incorporating a climate change allowance of 40% have been prepared to support this approach and can be found in the Appendices.

There is no residual flood risk from the development site to the surrounding district due to the reduction in storm water flow rates. The development does not therefore increase the risk of flooding to other adjacent neighbourhoods. Out of chamber or gully flooding for the extreme 100 year plus climate change event may occur within the development site and is classed as exceedance flows. Flood water from this event will be contained within the development site and directed away from the building to the service yard and parking areas.

Foul flows will be collected by a new gravity network to the existing spur serving the development plot. The private network discharges into the public sewer, located north of the industrial development along the A576, via a rising main.

2.0 DEVELOPMENT DETAILS

2.1 CONTEXT

This Flood Risk Assessment & Drainage Strategy report has been prepared by Burrows Graham Ltd. on behalf of Oxenwood Real Estate to support the planning application and the erection of a new building for Class B2/B8 (storage / distribution), including ancillary offices, landscaping, yards, alterations to existing access from Silburn Way, car parking, cycle parking and associated engineering works.

2.2 LOCATION

Site Name: Silburn Way, Middleton, Greater Manchester, M24 4NE.

Grid Reference: SD 853052

Site Area: 0.537 Ha

Site Description: The irregular plan shape of the site currently serves as a parking area for business located to the north. The main access to the site is from north of the plot off Silburn Way. A copy of the proposed site plan is shown in Appendix A.

There is woodland located to the east of the property. There are several industrial units located to the north and west of the property, with the River Irk located along the southern boundary.

The site is located within the Rochdale Metropolitan Council.

A review of the topographical information indicates the site to have a fall from circa 70.1m AOD at the eastern boundary, down to circa 69.5m AOD at the western boundary. A copy of the existing topographical plan is shown in Appendix A.



Figure 1: Site Location

3.0 PLANNING POLICY

3.1 NATIONAL PLANNING POLICY FRAMEWORK (NPPF)

Local planning authorities are advised by the National Planning Policy Framework (NPPF 2021) to consult the Environment Agency (EA) and the Lead Local Flood Authority (LLFA) on all developments of an area greater than 1Ha and those that are at an increased risk of flooding.

This report has been prepared to assess the requirements of the NPPF in accordance with the current EA and LLFA policies and will primarily assess the following.

- Determining whether the site is likely to be affected by flooding and whether it would increase flood risk elsewhere.
- Assessing whether the proposed development is appropriate in the suggested location.
- Detailing any measures necessary to mitigate any flood risk identified, to ensure that the proposed development and occupants would be safe, and that flood risk would not be increased elsewhere.
- Determine the current surface water drainage regime and assess any potential increase in surface water runoff as a result of the proposed development.
- Discuss Sustainable Drainage Systems (SuDS) as an option for reducing surface water flood risk.
- Devise an appropriate surface water drainage strategy (including calculation where appropriate) to deal with any potential increase in surface water runoff and include for climate change.
- Include the appropriate climate change allowances for both fluvial and surface water runoff.
- Consider the recommendations of Test Valley Borough Council Flood Risk Assessment (SFRA) and other local policies.

A review of the EA Flood Maps shows that the site is located within Flood Zone 1 (land defined as having less than a 1 in 1000 annual probability of flooding from of river or sea water in any year). The site is defined as 'less vulnerable' such that the development can be considered 'appropriate' in accordance with NPPF guidelines. As the site is within Flood Zone 1, the sequential and exception tests are not required by the local authority.

3.2 STRATEGIC FLOOD RISK ASSESSMENT

The aim of a Strategic Flood Risk Assessment (SFRA) is to assess the risks in a particular area regarding all types of flooding and to determine how much development is permitted in that area. It can assess this by looking at how drainage systems in the area should function and how risks in vulnerable areas can be reduced and/or mitigated. The NPPF states that regional planning bodies (RPB's) or local planning authorities should prepare SFRA's in consultation with the EA.

The SFRA provides a detailed understanding of flood risk across all areas and from all sources. It states that the sequential and exception tests must be provided where applicable and climate change must be taken into consideration over the lifespan of the development.

The City of Manchester has produced a Strategic Flood Risk Assessment to demonstrate and define its understanding and role in managing flood risk across the Greater Manchester Area. A fundamental aim of the strategy is that the Council embraces and supports its role as Lead Local Flood Authority for the area, and this means taking the lead in helping with infrastructure development and re-development and working with other bodies in Manchester with flood risk. The Council wants to encourage and wherever possible enforce current thinking and guidance and be able to provide improvements through working better together. The Strategy is aimed at providing an overview and assessment of local flood risk in the area, setting out objectives and measures for how they will manage and reduce local flood risk. This Strategy builds on existing and developing approaches to flood risk management and promotes the use of a wide range of measures to manage flood risk. Risk should be managed in a co-ordinated way within catchments and balance the needs of communities, the economy and the environment.

The full reports can be obtained from the relevant council's website:

[Microsoft Word - GM_SFRA_Appendix_B_District_Narratives_27-06-2008.doc \(bolton.gov.uk\)](#)

The main objectives of the SFRA:

- To better understand local flood risk and make best use of available information in order to better manage flood risk to people, businesses, property, infrastructure and the natural environment;
- To reduce the potential impact and costs of flooding in the city;
- To ensure resilience of local water bodies and drainage assets;
- To ensure appropriate development in areas of flood risk;
- To develop a collaborative partnership approach to flood risk management, and cooperate with other Risk Management Authorities (RMAs) and key stakeholders working across catchments;
- To assist communities in understanding information on flood risk and supporting themselves;
- To encourage, support and provide flood risk management which seeks to enhance and protect the environment.
- Determine the variations in risk from all sources of flooding across the city;
- Prepare broad policies for the management of flood risk;
- Steer development towards areas of lowest flood risk, through application of the Sequential Test and, where necessary, the Exception Test;
- Consider opportunities to reduce flood risk to existing communities through better management of surface water, provision for conveyance and storage for flood water; and
- Identify the level of detail required for site-specific Flood Risk Assessments.

3.3 LOCAL FLOOD RISK MANAGEMENT STRATEGY

The aim of this document is to set out the council's approach to managing flood risk in the short and long term. The primary aim of the document is to improve the understanding of flood risks within the Borough and provide a source of information which can be utilised to identify ways of managing flood risk. The second objective is to reduce the risk of flooding for people and businesses and put forward key actions to ensure this can be achieved.

3.4 SUSTAINABLE DRAINAGE

The main principles of SuDS integration into the proposed development have been taken from CIRIA C753 and are summarised below:

- That SuDS are used in major developments.
- That SuDS provide wider benefits which include biodiversity, water quality, amenity and attenuation.
- That developments aim to achieve greenfield run-off rates.
- That the minimum standards of operation proposed by the applicant are appropriate.
- Using planning conditions or planning obligations that there are clear arrangements in place for ongoing maintenance of the SuDS scheme over the lifetime of the development (see Appendix E).

The drainage strategy for this development will look to integrate these policies within the design.

4.0 FLOOD RISK

The NPPF guidelines require the developer to assess the impact of the proposed development runoff on the downstream catchment in conjunction with assessing the risk of runoff from the surrounding area on the proposed development layout.

In the following sections the flood risk to the site from all sources will be assessed. As the site will restrict the surface water runoff there is no increased flood risk to the downstream network.

4.1 FLUVIAL & TIDAL FLOODING

The closest identified watercourse is the River Irk, located along the southern boundary of the site.

The National Planning Policy Framework (NPPF) categorises fluvial and tidal flood risk as follows:

- Zone 1 (low probability) – Land assessed as having less than a 1 in 1,000 annual probability of river or sea flooding (<0.1%);
- Zone 2 (medium probability) – Land assessed as having between a 1 in 100 and 1 in 1,000 annual probability of river flooding (1% – 0.1%), or between a 1 in 200 and 1 in 1,000 annual probability of sea flooding (0.5% – 0.1%) in any year; and
- Zone 3a (high probability) - Land assessed as having 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.
- Zone 3b The Functional Floodplain - This zone comprises land where water has to flow or be stored in times of flood. Local planning authorities should identify in their Strategic Flood Risk Assessments areas of functional floodplain and its boundaries accordingly, in agreement with the EA. (Not separately distinguished from Zone 3a on the Flood Map).

Figure 2 below locates the site on the Environment Agency's (EA) indicative flood map for planning. This identifies the site as located within low-risk Flood Zone 3, i.e. as having 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.

The EA Flood Map does identify flood defences along the River Irk northern banks to the west of the development site.

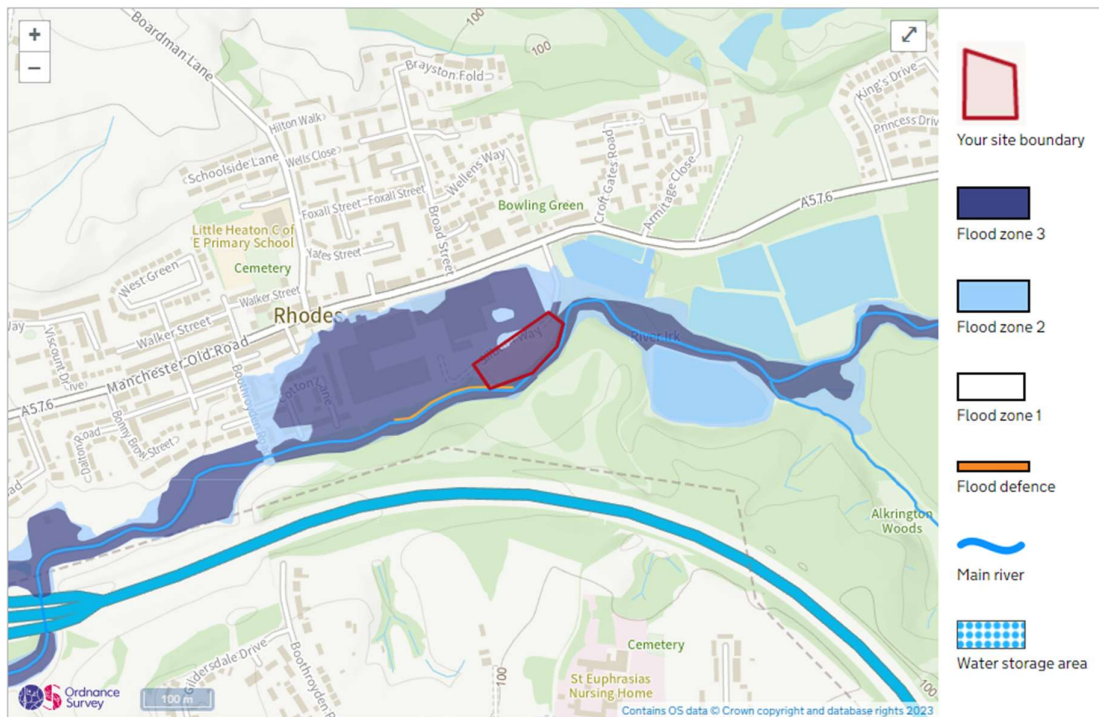


Figure 2: Extract from EA Fluvial Flood Risk Maps

These defences are highlighted in the Campbell Reith FRA document 9940 dated April 2015 (c/o Dolphon Land & Development Consultancy attached in the Appendix), which was submitted for the residential development planning application. This residential development site is located to the west of the proposed development site. An excerpt from drawing SK007 of the Campbell Reith report shows levels of the defences and are reproduced in Figure 2b.

The CBRE Report - 50BCD0443600 - also refers to the defences in the Campbell Reith report and supplement the topographical information along the development site northern banks of the River Irk, see Figure 2a below.

The building FFL has been proposed at 70.900 AOD which at minimum 300mm above the Environment Agency 1:100 flood levels (~70.420 AOD) predicted in the area.



Figure 4 – Comparison of bank levels and flood levels for the 1 in 100 (1%) flood event.
Contains Ordnance Survey data © Crown copyright and database right 2020

Figure 2a: Extract from CBRE Report 50BCD0443600 Flood Risk Review

Figure 2a, show the proposed site development existing topographical levels along the northern bank of the River Irk with river flood levels. The existing topographical levels are higher than the modelled flood levels.

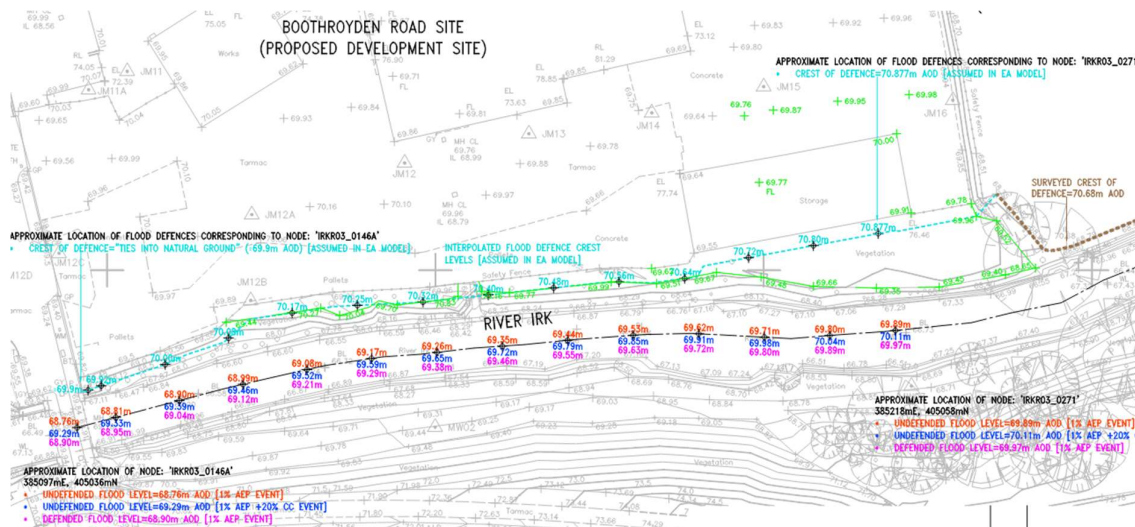
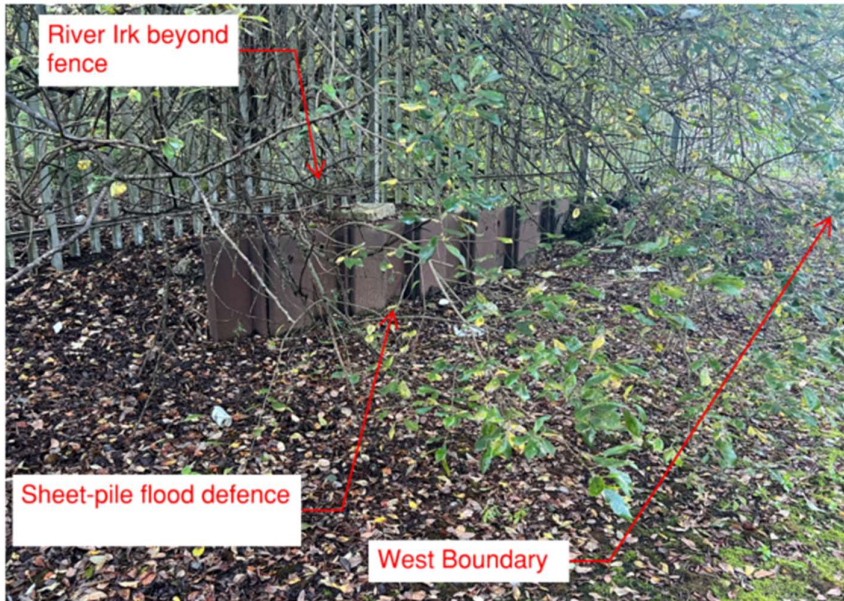


Figure 2b: Extract from CBRE Report 50BCD0443600 Flood Risk Review

Site visits have identified the sheet pile wall defence terminating within the proposed development boundary - see images below:



The developer has appointed JBA Consulting to undertake a flood modelling study of the area with updated topographical data to produce a flood challenge report to be submitted to the EA for review.

4.2 SURFACE WATER FLOODING

The EA descriptions for the High, Medium and Low risk scenarios for surface water flooding are as follows:

- High risk means that each year this area has a chance of flooding of greater than 3.3%.
- Medium risk means that each year this area has a chance of flooding of between 1% and 3.3%.
- Low risk means that each year this area has a chance of flooding of between 0.1% and 1%.
- Very Low risk means that each year this area has a chance of flooding of less than 0.1%.

A review of the EA surface water flood maps (Figure 3 below) indicates that the site is classified as 'Low Risk'. Beyond the site boundary, the overall industrial development is at a low risk of surface water flooding.

The development will mitigate against any surface water flood risk within the site boundary by amending the levels locally with new building and service yards to suit the development intent. All new impermeable areas will be directed into a new underground drainage network designed in line with local and national guidance. Considering these changes surface water will not pose a flood risk to the development.

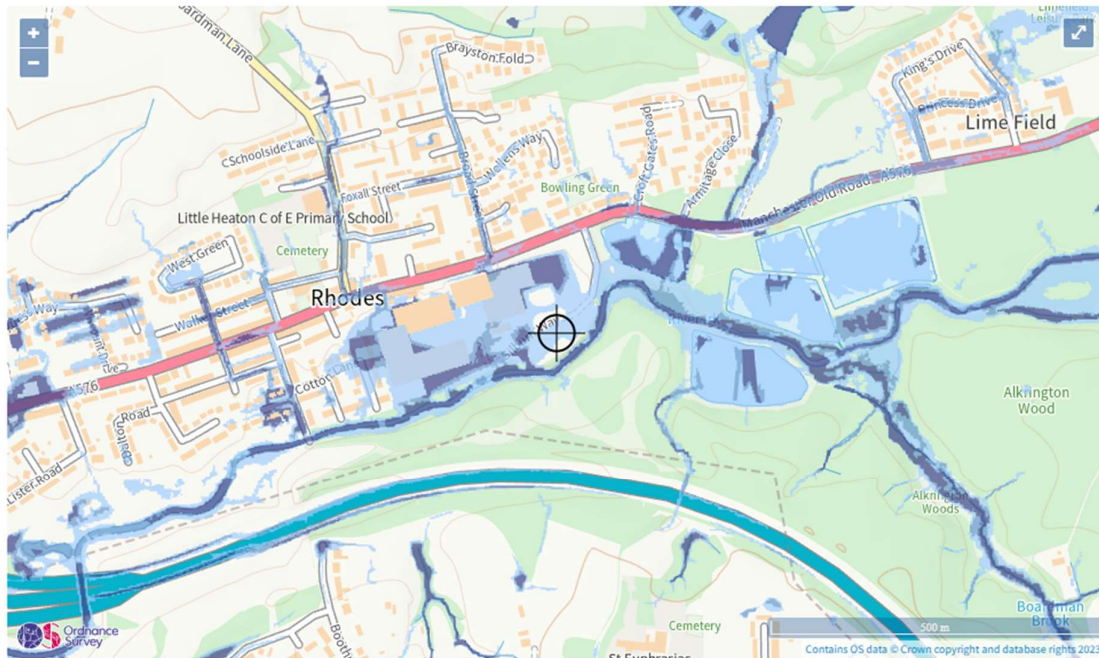
There are existing surface water sewer routes through the site that would need to be maintained. Therefore, based on this assessment, the site can be considered at low risk from surface water flooding.

Flood risk

Extent of flooding

Location

Enter a place or postcode



Extent of flooding from surface water

High Medium Low Very low Location you selected

Figure 3: Extract from EA Surface Water Flood Risk Maps

4.3 GROUNDWATER FLOODING

Groundwater flooding occurs when the water table, the level of water below the ground, rises above the ground surface or into cellars and basements. Groundwater flooding is most likely to occur in low lying areas underlain by permeable rocks (aquifers). These may be extensive, regional aquifers, such as chalk or sandstone, or may be localised sands or river gravels in valley bottoms underlain by less permeable rocks.

The BGS groundwater dataset suggests that there is a low to medium potential for groundwater flooding to occur at the surface of the land in the area.

The proposed development does not include any below ground basement areas and floor levels are at or above existing ground levels. Intrusive site investigations undertaken at the site have not identified any elevated ground water levels. Based on this, the risk of groundwater flooding to the site is considered low.

4.4 SEWER FLOODING

The United Utilities sewer records have been reviewed and confirm that the closest public sewers are a 900mm diameter foul water sewer located in Manchester Road and a 1100mm diameter foul water

sewer along the River Irk. No surface water assets are recorded in the vicinity of the proposed development site. A copy of the United Utilities Asset plan is shown in Appendix C.

All these adopted SW sewers are located outside the site boundary and set down from the proposed finished floor levels of the proposed building. Furthermore, the existing drainage network serving the surrounding urban district ensures that the development footprint is protected from the impact of both upstream and downstream runoff. It is speculated that complete protection may well exist beyond a storm event equivalent to the 30-year statistical event. Beyond this projection, there may be a small degree of peripheral 'Exceedance' flooding within the areas above the sewers. However, this is expected to be localised and restricted to the location of specific manhole covers located outside the development footprint.

A CCTV and GPR survey of the existing site has identified a number of separate private foul and surface water drainage networks that outfall unrestricted to the public sewers and the River Irk respectively. All these private drains will be inspected and jetted. A copy of the GPR survey is shown in Appendix A.

Considering these findings, flood risk to the site from sewers is diminished to acceptable levels and considered to be low.

4.5 ARTIFICIAL FLOODING

A review of the EA online mapping tool shows that the site is located in an area with risk of flooding from reservoirs and other artificial features.

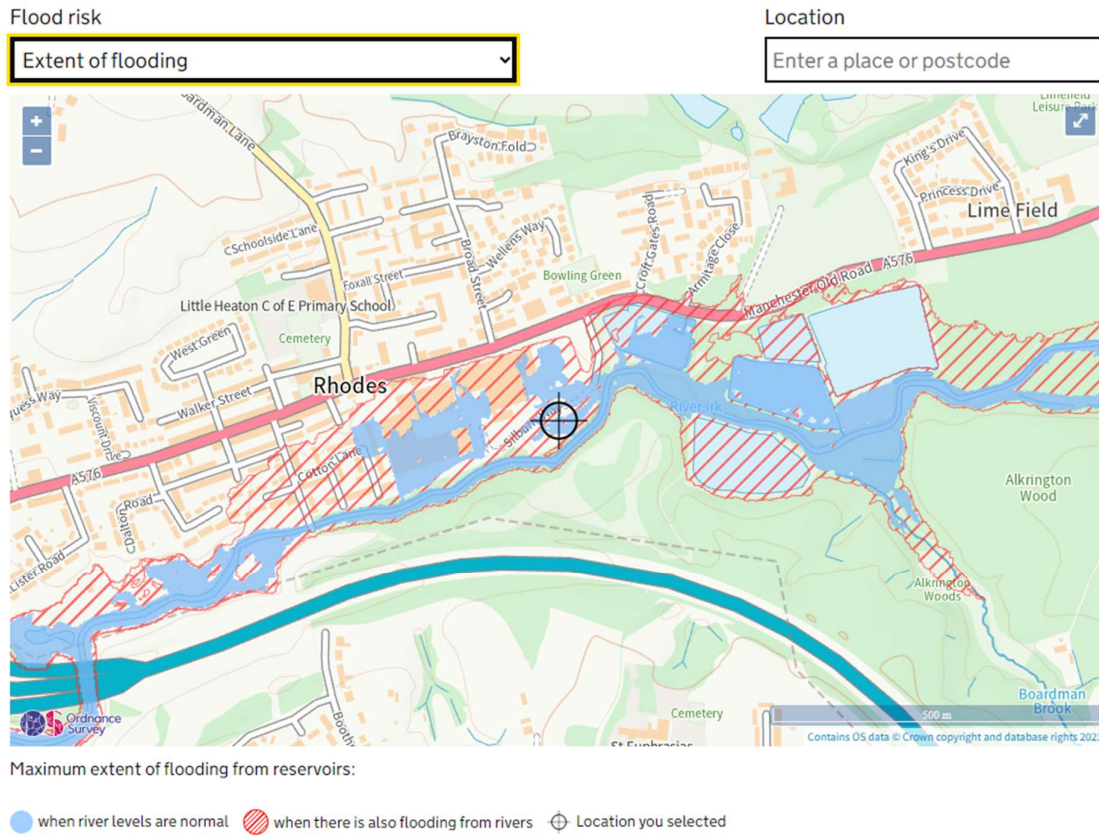


Figure 4: Extract from EA Surface Water Flood Risk Maps

4.6 SUMMARY

Below is a table summary of the flood risks associated with the development site.

Source	Probability of Flood Risk	Impacts	Remarks
Tidal	Low	Low	The development is not located in low tidal zone.
Fluvial (Watercourse)	High	Low	The development is located in high fluvial flood zone 3.
Surface Water (Overland Flood Flow)	Low	Low	The development is located within a very Low Risk Zone. Future Drainage works to remove risk.
Sewers & New Drainage	Low	Low	No adopted sewers within development boundary. New private drainage will serve the units and external areas, all designed to current guidance with any exceedance flooding located away from the units.
Groundwater	Low - Medium	Low	Development does not include basements, no identified elevated ground water levels, and thus risk of considered to be low.
Artificial Sources	Low	Low	Artificial sources in close proximity.

Table 1: Flood Risk Summary

Essential Infrastructure	Essential transport infrastructure (including mass evacuation routes) which has to cross the area at risk Essential utility infrastructure which has to be located in a flood risk area for operational reasons, including electricity generating power stations and grid and primary substations; and water treatment works that need to remain operational in times of flood Wind Turbines
Highly Vulnerable	Police stations, Ambulance stations, Fire stations, Command Centres and telecommunications installations required to be operational during flooding Emergency dispersal points Basement dwellings Caravans, mobile homes and park homes intended for permanent residential use Installations requiring hazardous substances consent
More Vulnerable	Hospitals Residential institutions such as residential care homes, children's homes, social services homes, prisons and hostels

	Buildings used for dwelling houses; student halls of residence, drinking establishments, nightclubs and hotels. Non-residential uses for health services, nurseries and educational establishments Landfill and sites used for waste management facilities for hazardous waste. Sites used for holiday or short-let caravans and camping, subject to a specific warning and evacuation plan.
Less Vulnerable	Police, ambulance and fire stations which are not required to be operational during flooding. Buildings used for shops; financial, professional and other services, restaurants and cafes, hot food takeaways, offices, general industry, storage and distribution, non-residential institutions not included in "more vulnerable", and assembly and leisure. Land and buildings used for agriculture and forestry. Waste treatment (except landfill and hazardous waste facilities). Minerals working and processing (except for sand and gravel working). Water treatment plants and sewage treatment plants (if adequate pollution control measures are in place). Sewage treatment works (if adequate measures to control pollution and manage sewage during flood events are in place).
Water-compatible Development	Flood control infrastructure. Water transmission infrastructure, pumping stations. Sewage transmission infrastructure and pumping stations. Sand and gravel workings. Docks, marinas, wharves Navigation facilities. MOD defence installations. Ship building, repairing and dismantling, dockside fish processing and refrigeration and compatible activities requiring a waterside location. Water-based recreation (excluding sleeping accommodation). Lifeguard and coastguard stations. Amenity open space, nature conservation and biodiversity, outdoor sports and recreation and essential facilities such as changing rooms. Essential ancillary sleeping or residential accommodation for staff required by uses in this category, subject to a specific warning and evacuation plan.
Notes 1 - This classification is based partly on Defra/Environment Agency research on Flood Risks to People (FD2321/TR2)21 and also on the need of some uses to keep functioning during flooding. 2 - Buildings that combine a mixture of uses should be placed into the higher of the relevant classes of flood risk sensitivity. Developments that allow uses to be distributed over the site may fall within several classes of flood risk sensitivity. 3 - The impact of a flood on the particular uses identified within this flood risk vulnerability classification will vary within each vulnerability class. Therefore, the flood risk management infrastructure and other risk mitigation measures needed to ensure the development is safe may differ between uses within a particular vulnerability classification.	

Vulnerability Classification		Essential Infrastructure	Water-compatible	Highly Vulnerable	More Vulnerable	Less Vulnerable
Flood Zone	Zone 1	✓	✓	✓	✓	✓
	Zone 2	✓	✓	Exception Test	✓	✓
	Zone 3a	Exception Test	✓	✗	Exception Test	✓
	Zone 3b	Exception Test	✓	✗	✗	✗
Key ✓ Development is appropriate ✗ Development should not be permitted						

5.0 DRAINAGE STRATEGY

5.1 CONSULTATION

A pre-development enquiry has been submitted to United Utilities to inform on the design for draining the foul and surface water from the site, plus the agreed peak discharge rates. Their formal response has not yet been received.

5.2 EXISTING DRAINAGE AND FLOW RATES

The site area currently encompasses existing transport and storage operations with hard paved areas which all positively drain unrestricted to the River Irk bounding the site to the south. As such the site is considered to be partially brownfield for drainage purposes.

The existing surface water flow from the site has been calculated as approximately 46l/s, with flooding on the surface of 29m³ for the impermeable areas currently draining direct to the existing sewer. A drawing of impermeable area is shown in Appendix A.

Greenfield rates have been defined in Table 2 below.

	Area (ha)	1 year (l/s)	Qbar (l/s)	30 year (l/s)	100 year (l/s)
Greenfield Runoff Rates	0.537	3.4	3.9	6.72	8.22

Table 2: Greenfield Runoff Rates – HR Wallingford

5.3 GEOLOGY

A detailed Phase 2 Site Investigation has been commissioned at the site. This identified made ground comprising a sandy gravelly clay to a maximum depth of up to 3.5m. The made ground was underlain by superficial deposits comprising gravelly clays, silts and horizons of medium dense sands and gravels. Groundwater monitoring recorded standing water at depths of between 1.73m and 4.07m within the eastern area of the site and between 1.15m and 1.77m within the western area of the site. Considering the shallow made ground and elevated groundwater levels, infiltration as a means of draining the site has been discounted.

5.4 SUSTAINABLE DRAINAGE

Sustainable Drainage Systems (SuDS) are utilised in line with the CIRIA 753 guidance and LBB Policy with the aim to minimise the development's impact on the runoff quantity and quality and maximise amenity and biodiversity opportunities.

The CIRIA 753 Management train approach is a fundamental principle of SuDS systems, it aims to primarily reduce pollution, flow rates and volume of runoff from the site. The main objective is to treat the runoff as close to the source as possible and it is imperative that at least two of the following should be included within the drainage strategy.

- **Prevention** – The use of good site design and rainwater re-use measures to prevent runoff.
- **Source Control** – Controlling the runoff as close to source as possible by using green roofs, porous pavements and soakaways.
- **Site Control** – Use of water management techniques within local area, i.e. detention basins and geocellular storage.
- **Regional Control** – Management of water from a number of sites in a specific location, ie. balancing ponds and wetlands.

The drainage strategy for this development will incorporate source control in the form of permeable surfacing to the staff parking area and site control methods in the form of a geocellular attenuation tank to retain the peak flows and allow the water to drain out slowly to the surface water sewer at restricted rates.

Whilst it would be beneficially to include more of these techniques, the space on site means it is not viable to include any additional above ground 'Green' SuDS measures.

5.5 SUDS HIERARCHAL APPROACH

- Based on the SuDS review, plus an assessment of the local site conditions, the SuDS hierarchal approach for discharge of surface water at the development site is considered in greater detail in Table 3 below.
- The site has an existing discharge to the watercourse, we are reducing the release rate to 4.5 L/S to alleviate flow rates to the sewer.

Method	Suitability	Proposed	Suitability for Development
Store rainwater for later use	No	No	Not suitable due to type of development
Use infiltration techniques, such as porous surface in non-clay areas	No	Yes	Based on the BGS data the underlying ground strata may be suitable for infiltration, permeable paving parking area will be used. Water table depth and permeability index to be confirmed by ongoing Soil Investigation works.

Attenuate rainwater in ponds or open water features for gradual release	No	No	Not feasible due to lack of space and site constraints
Attenuate by storing tanks or sealed water features for gradual release	Yes	Yes	Attenuation via underground storage is proposed
Discharge rainwater direct to a water course	Yes	Yes	The site has an existing discharge location to a water course which we are proposing to reuse
Discharge rainwater to a surface water sewer/drain	No	No	The site has an existing discharge location to a water course
Discharge rainwater to the combined sewer	No	No	Separate Surface & Foul Water connections proposed to their respective sewers

Table 3: Drainage Proforma Drainage Hierarchy

5.6 CLIMATE CHANGE

In May 2022, the Environment Agency released updated climate change allowances for peak rainfall intensities which should be applied to new developments. Rather than nation-wide allowances, each area will have its own peak rainfall allowances.

Irwell Management Catchment peak rainfall allowances		
3.3% annual exceedance rainfall event		
Epoch	Central allowance	Upper end allowance
2050s	25%	35%
2070s	30%	40%
1% annual exceedance rainfall event		
Epoch	Central allowance	Upper end allowance
2050s	25%	40%
2070s	30%	45%
*Use '2050s' for development with a lifetime up to 2060 and use the 2070s epoch for development with a lifetime between 2061 and 2125.		

Based on the nature of the development, a lifespan of 50 years is anticipated (BS EN 1990). Therefore, the potential climate change allowance for the 2070's ranges between 25% and 45% for the central and upper end allowance respectively. In line with the EA Guidance for this design life, a central end allowance of 25% for climate change on peak rainfall intensity would be applicable. However, to satisfy local policy guidance, the 40% climate change allowance has been simulated in the hydraulic model to confirm that any exceedance volumes can be retained within the site boundary and appropriately managed such that they won't impact on the proposed buildings or flow off site.

5.7 THE NON-STATUTORY TECHNICAL STANDARDS FOR SUDS

It is best practice to develop drainage strategies to the DEFRA document 'The Non-Statutory Technical Standards for SuDS'.

The DEFRA document advises the following with respect to 'Peak Flow Control' (S3) and 'Volume Control' (S5) for Brownfield sites such as this:

- **S3** For developments which were previously developed, the peak runoff rate from the development to any drain, sewer or surface water body for the 1 in 1 year rainfall event and the 1 in 100 year rainfall event must be as close as reasonably practicable to the greenfield runoff rate from the development for the same rainfall event, but should never exceed the rate of discharge from the development prior to redevelopment for that event.
- **S5** Where reasonably practicable, for developments which have been previously developed, the runoff volume from the development to any highway drain, sewer or surface water body in the 1 in 100 year, 6 hour rainfall event must be constrained to a value as close as is reasonably practicable to the greenfield runoff volume for the same event, but should never exceed the runoff volume from the development site prior to redevelopment for that event.
- **S6** Where it is not reasonably practicable to constrain the volume of runoff to any drain, sewer or surface water body in accordance with **S5** above, the runoff volume must be discharged at a rate that does not adversely affect flood risk.

As noted above, and in line with policy, it is intended that surface water discharge rates will be restricted as close as possible to the Q_{Bar} Greenfield Rate (adopted 2l/s/ha) for all rainfall events in accordance with standard **S3** and to also comply with the peak volume requirements as per **S5** and **S6**.

It should be noted that these rates are indicative only and based on the anticipated measured impermeable areas from the development masterplan. The final discharge rate will be confirmed at detailed design stage and agreed with the LLFA through a reserved matters or condition discharge application.

5.8 SURFACE WATER DRAINAGE STRATEGY

The development plot is currently serviced by an existing private surface water sewer which collects the existing wider developed business park and discharges to the River Irk to the south of the proposed development plot with an existing outfall.

Surface water generated from the roof of the new building will pass into a below ground drainage network that will merge with the runoff from the new service yard & car parking area. The car park area will be made up of permeable pavers allowing for filtration with an overflow pipe leading to attenuation tanks for all storm events. The proposed surface water sewer release rate will be 4.5l/s. As the release rate from the site is restricted by a control device (orifice or similar), the surface water will be attenuated in a geocellular attenuation tank located beneath the external yard areas. Refer to Appendix A for the proposed drainage strategy plan.

This overall approach aligns with the Sustainable Drainage Systems (SuDS) policy and Policy E7 covering Flood Risk Management and covering Sustainable Drainage. Table 5 below defines both the existing and post development flow rates based on the SW restricted flow rate and local policy. As shown in the table, these rates incorporate a reduction in the existing predevelopment discharge rates.

	Q_{BAR}	30 year	100 year
Greenfield Runoff Rates	4.0 l/s	6.7 l/s	8.2 l/s
Post Development Peak Flow Rates	4.5 l/s	4.5 l/s	4.5 l/s (Inc 40% CC)

Table 4: Post Development Runoff Rates

Full details of this strategy can be found on the BGL Drainage, Impermeable Areas & Exceedance drawings in Appendix A and the hydraulic calculations in Appendix B.

The proposed private drainage layout for the new development site will be designed in accordance with BS EN 752: 2008 and Building Regulations part H guidance, i.e. to show no flooding for the 30-year storm return period criterion. Events exceeding this up to and including the 100-year plus climate change allowance have been assessed. Flooding from these events is classed as exceedance flooding and will be directed away from buildings, where it will be stored in the low-lying yard and hard paved areas and drain back into the surface water network as the levels recede.

5.8 WATER QUALITY

Water quality has been considered and the following measures have been included within the surface water drainage proposals:

- Runoff from roofs is considered to be clean and will be discharged directly into the surface water network – CIRIA C753 Table 26.2.
- All drainage from the car parking areas have a low-medium risk and will pass through the permeable paving surface.

- All drainage from the loading dock area, having a medium risk, will pass through the existing Class 1 full retention petrol separator which serves the greater business park.
- Silt will be prevented from entering the system by the use of channels with silt traps and catch pits where required prior to surface water entering the attenuation crates and the outlet control manhole.
- The measures proposed for the development are considered appropriate as per guidance, SuDS water quality treatment indices are shown below:

Land Use (As Defined in the CIRIA SuDS Manual)	Pollution Hazard Level	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Roof	Low	0.2	0.2	0.05
Car Park	Low	0.5	0.4	0.4
Loading Yard	Medium	0.7	0.6	0.7
SuDS Type	Treatment			
Permeable Paving	-	0.7	0.6	0.7
Class 1 Separator	-	0.8	0.6	0.9
Suitable Level of Treatment?	-	Yes	Yes	Yes

Table – Simple Index Approach Summary for the Proposed Development

5.9 FOUL WATER STRATEGY

Foul flows will discharge to the existing private foul water sewer network located to the north of the site via the existing connection. The site will drain by a private gravity network to this sewer and be designed in line with Building Regulations Guidance.

Refer to the drainage strategy plan in Appendix A for details.

6.0 CONCLUSION

In conclusion, this report has studied and assessed the flood risk to the site by all sources with the proposals complying with national, regional and local planning policy & guidance. A review of the EA Flood Maps shows that the site is located within Flood Zone 3, i.e. land defined as having high probability of flooding from river water. Further to this, the report has found the site to be at 'low risk' from all sources of flooding; tidal, fluvial, pluvial, sewer, with medium risk from ground water and reservoir/ artificial sources.

The surface water drainage strategy is to connect to the existing surface water sewer running north south on the site at restricted rates, to ensure that the flood risk to the site and surrounding catchment is reduced post development. The Sustainable Drainage Systems (SuDS) hierarchy has been considered in the drainage strategy with reduced release rates, permeable pavers, additional proprietary existing Class 1 separator and a geocellular attenuation tank located beneath the service yard areas.

There is no residual flood risk from the development site to the surrounding district due to the reduction in storm water flow rates. The development does not therefore increase the risk of flooding to other adjacent neighbourhoods.

APPENDIX A – DRAWINGS

BUILDING GROSS INTERNAL AREA SCHEDULE

UNIT

Warehouse Area	-	1,752 sqm / 18,858 ft ²
Office Ground Floor	-	56 m ² / 602 ft ²
Office First Floor	-	167 m ² / 1,797 ft ²

Total GIA - 1,975 m² / 21,258 ft²

Internal Haunch Height - 9m

VEHICLE PARKING SUMMARY

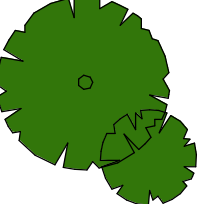
Car Parking = 27 Bays (Including 3 Disabled)
Motorcycle Parking (PTW) = 2 Bays
Cycle Shelters = 10 Cycles Spaces

Level Access = 2 Doors

Site Application Boundary = 0.537 Ha / 1.327 Acres

External Finishes Legend

- Concrete Service Yard**
Concrete slab on sub-base/capping designed to suit ground conditions to Structural Engineer's specification.
- Landscape Areas**
Refer to Landscape Architect's drawings and Arboricultural Survey to details on proposed landscape design.
- Tarmac Finish Access Roads & Car Park Bays**
Tarmac road - construction to Structural Engineer's Specification.
- Block Paving Footpaths**
Permeable block paving footpaths - construction to Structural Engineer's specification.
- Gravel**
To align pedestrian footpaths around perimeter of building. Gravel margins around perimeter of site.
- Standing Fencing**
2.4m High Paladin Security Fence to enclose external service yard area and tie into existing fence line boundary along riverbank. Colour: Dark Green RAL 6005.
- EV Ductwork**
Electric vehicle charging ductwork installed to car parking bays, for future provision for bays to become EV bays.
- WT**
Water Tap
External manual water tap for landscape watering.



Trees and Hedges
All proposed trees are shown indicatively for illustrative purposes. For detailed information on new, retained and removed planting refer to Landscape Architects drawing and specification.



PO6:	Updated indicative position of condensers. Amended red line boundary and updated new proposed and removed trees. Amended tree sizes and add in border fence supported by condensers. Added extent of adjacent footpaths.	06.10.23 DM
PO5:	Updated red line boundary and amended trees to be removed. Update red line area.	05.10.23 DM
PO4:	Updated positions of EV and future EV ducting. Updated area & notes to warehouse. Updated with additional planting & legend. Updated ground level to include updated ground level.	02.10.23 JB
PO3:	Updated building position and size, added M&E equipment and tree locations and RPA details. Issued for Comment. Resubmitted sheet to 101.	26.09.23 DM
PO2:	Updated building position in line with river assessment.	12.09.23 JB
PO1:	First Issue.	04.09.23 JB

Rev	Date/Checked
The Old Rectory Rectory Lane Milton Masor NORTHAMPTON N 7 3 A Q t: +44 (0)1604 859916 f: +44 (0)1604 859123 www.peter-haddon.com	pHp architects

Oxenwood Real Estate
Rhodes Business Park, Middleton
Manchester

Proposed Site Plan

Status **PLANNING**

Drawn JB Checked JL

Date **AUGUST 2023**

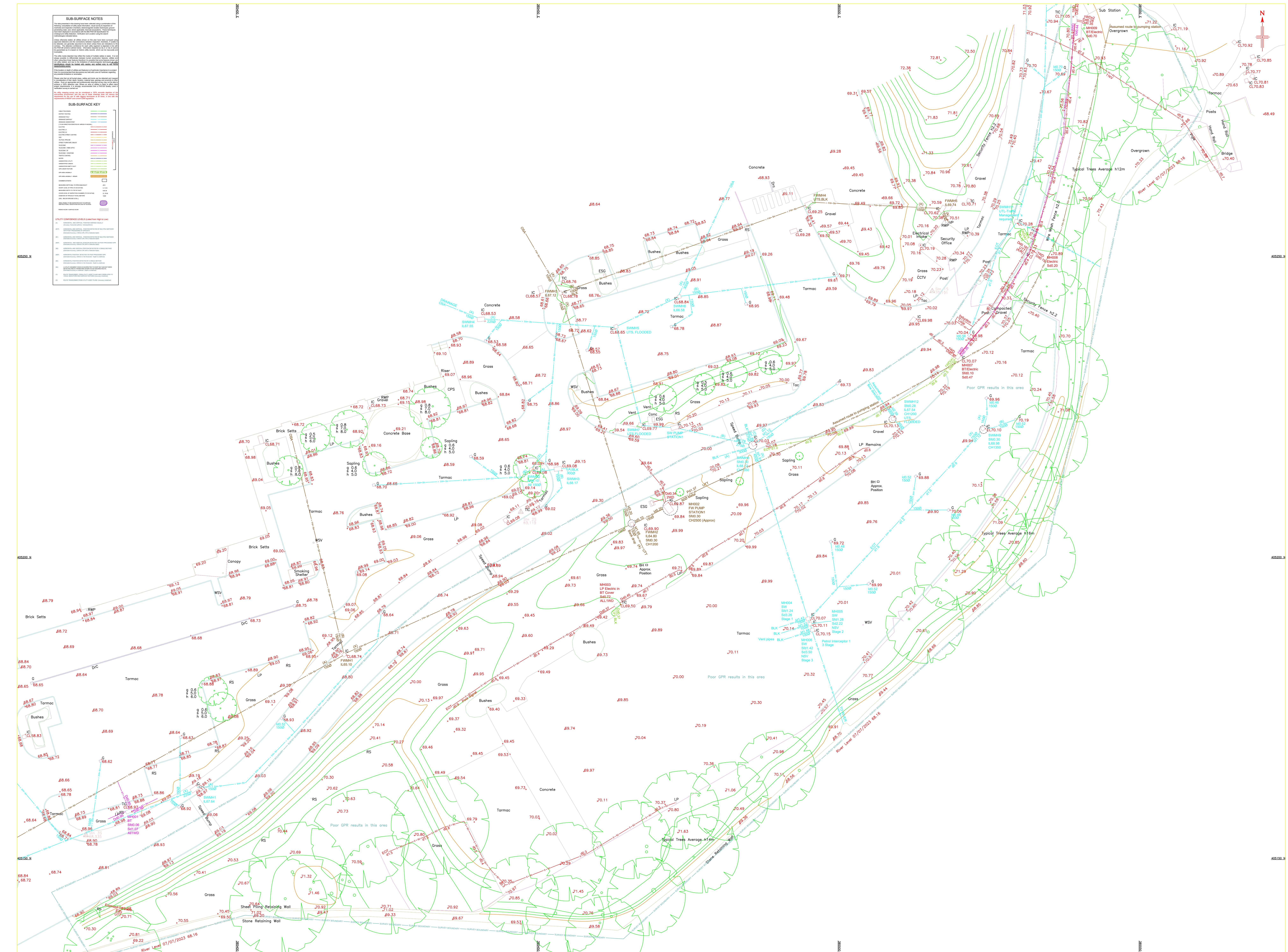
Project Number: 4694

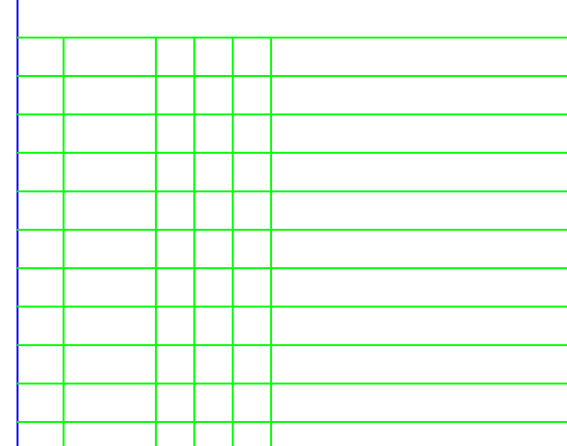
Project Code Originator Zone Level Info Type Rule Obj No Revision

RBP- pHp-XX-00- DR-A- 101- P06

Scale @ A0 As Indicated

Copyright reserved. Dimensions to be checked on site. Discrepancies to be reported before proceeding.





Lane & Frankham 
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SIDCUP OFFICE: Irene House Five Arches Business Park Moldstone Road Sidcup, Kent DA14 5AE T: 020 8399 7777	OXFORD OFFICE: Woodcot Business Park Besselsleigh Road Woodston, Abingdon Oxfordshire OX13 6PD T: 01865 322500
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Client:

Project Title:

Rhodes Business Park

Drawing Title:

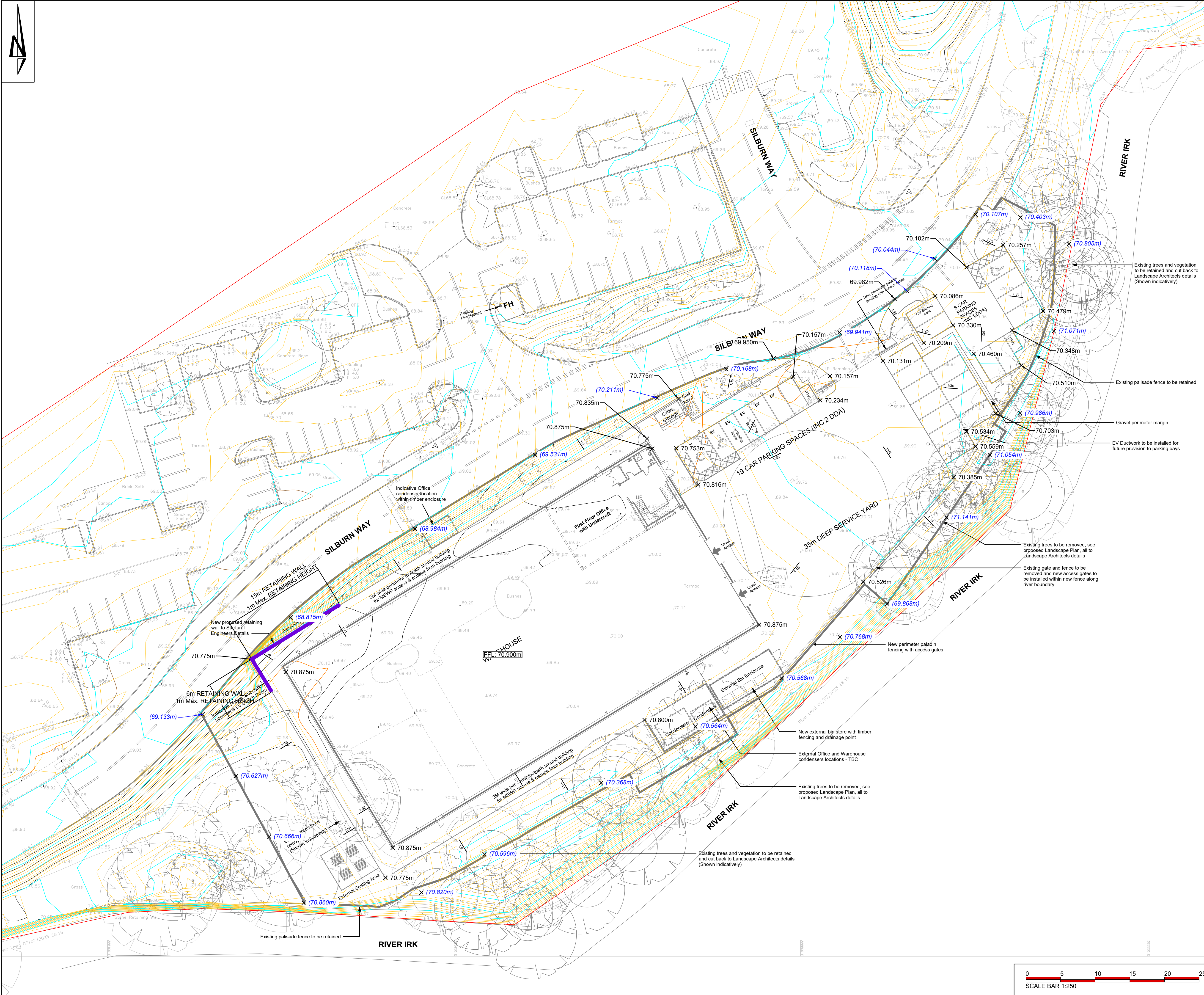
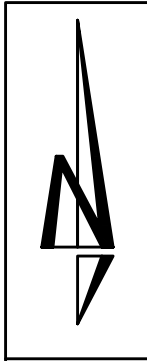
2D Topographical Survey

RECORD

Drawn By:	Designed By:	Checked By:	Approved:
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28/07/2023	1:200		
Job No/ File Ref	Originator	Zone	Level
LE4945	LAE	YY	ST

Type	Discipline	Number	Suitability	Revision
DR	G	1001	S2	R01



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- Key:
- Site Boundary
 - Existing Level
 - Proposed Level
 - Top of Wall Level
 - Proposed Slope
 - 1:40
 - Precast Retaining Wall
 - Existing Retaining Wall
 - Proposed Contour Minor 0.1m
 - Proposed Contour Major 0.5m

P02	10/10/23	DRD	LAYOUT UPDATED	DB
P01	15/09/23	DRD	PRELIMINARY ISSUE	DB
REV	DATE	BY	DESCRIPTION	CHK'D

North Office:
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CLIENT

OXENWOOD REAL ESTATE

PROJECT

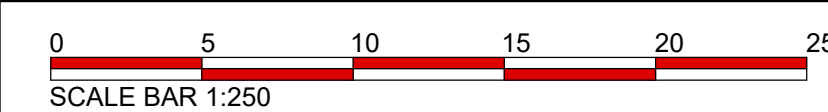
RHODES BUSINESS PARK,
MIDDLETON, MANCHESTER

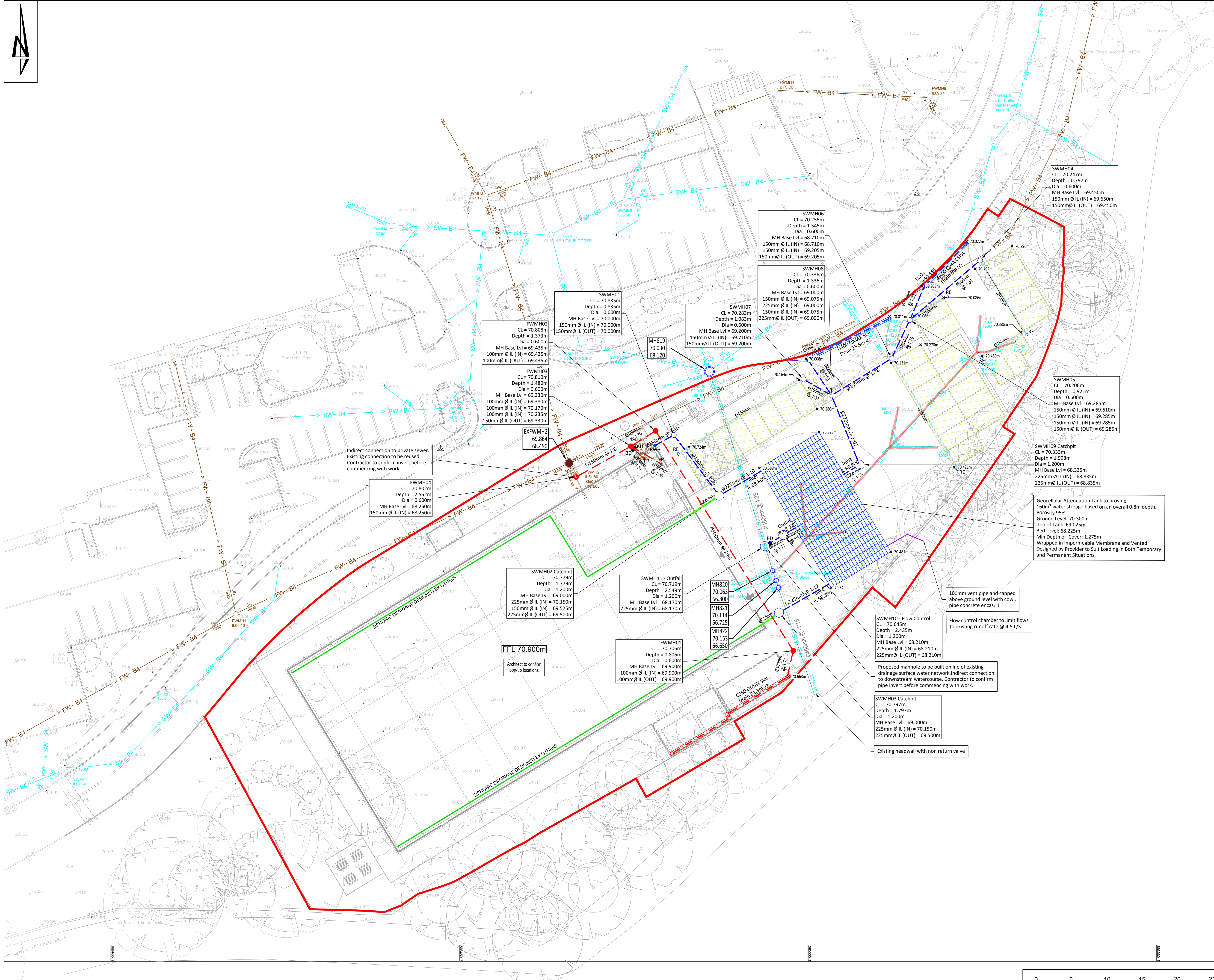
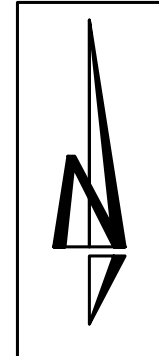
DRAWING TITLE

PROPOSED LEVELS

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SCALE @ A1	DATE	DRAWN BY
1:250	15/09/2023	DRD
		CHECKED BY
		DV

DRAWING NO	REV
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- KEY:
- Site Boundary
 - F.F.L. 77.77m Proposed Building Level
 - SW00 Proposed Surface Water Pipe & Chamber
 - FW00 Proposed Foul Water Pipe & Chamber
 - SU Proposed Slot Drain & Sump Unit
 - SU = Sump Unit
 - AP = Access Point
 - Proposed Attenuation Tank
 - Porous Paving Block Lined Permeable Paving (System C)
 - No Ground Infiltration with Sealed Outlet. Subbase To Be SHW Type 3 Material CL 805 With Perforated Pipe
 - Proposed Vent Pipe
 - Siphonic Drainage (Design By Others)
 - Perforated Pipe
 - RE Rodding Eye
 - FP_O Foul Water Pops Up
 - BD_O Backdrop
 - Existing Surface Water Pipe & Chamber
 - Existing Foul Water Pipe & Chamber
 - Existing Drainage To Be Removed

P02	23/01/24	SMB	Existing Petrol Interceptor Shown	DV
P01	10/10/23	SMB	First Issue	DV
REV	DATE	BY	DESCRIPTION	CHK'D

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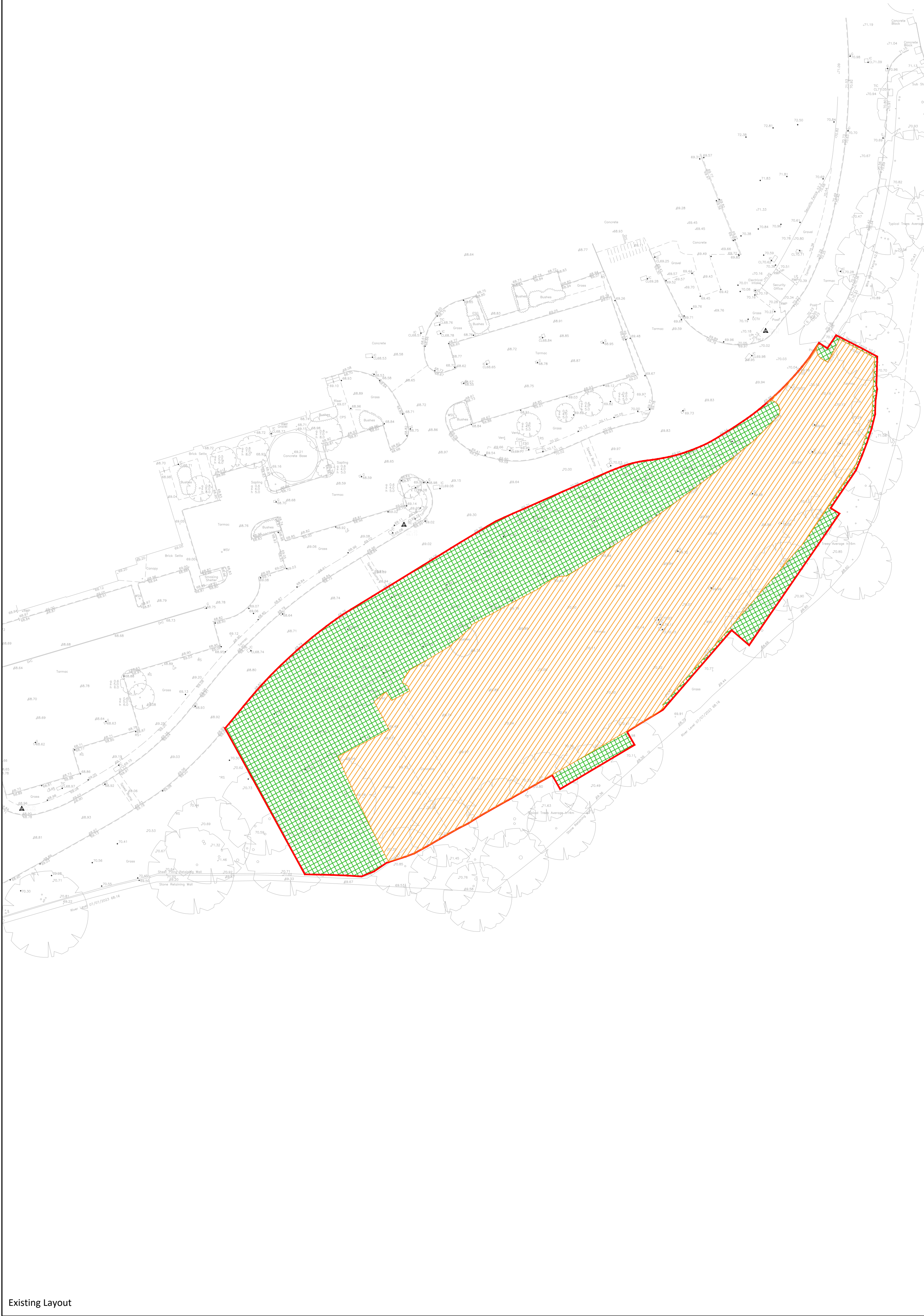
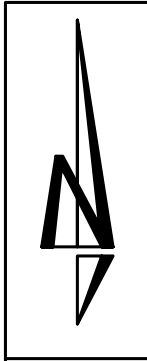
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PROJECT
**RHODES BUSINESS PARK,
MIDDLETON, MANCHESTER**

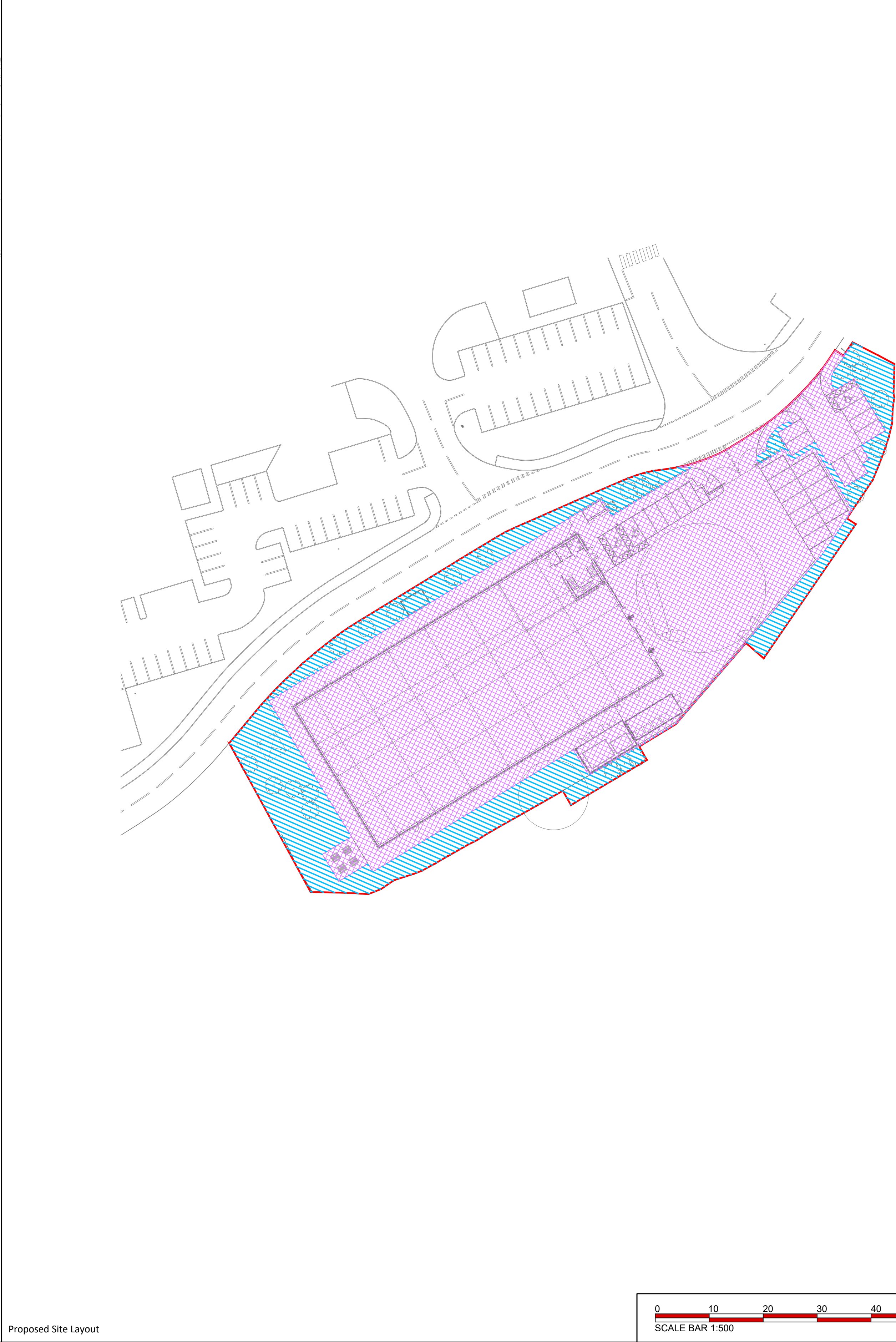
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SCALE @ A1 1:250	DATE 10/10/2023	DRAWN BY SMB
		CHECKED BY DV

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Existing Layout



Proposed Site Layout

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KEY:

SITE BOUNDARY (Total area = 5,375m²)
EXISTING PERMEABLE AREA = 2,165m²
EXISTING IMPERMEABLE AREA = 3,210m²
PROPOSED IMPERMEABLE AREA = 4,190m²
PROPOSED PERMEABLE AREA = 1,185m²

P02	22/01/24	DV	Area Schedule Ammended	DV
P01	10/10/23	SMB	First Issue	DV
REV	DATE	BY	DESCRIPTION	CHK'D

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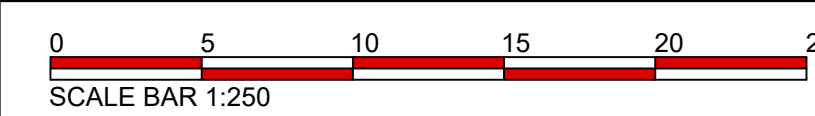
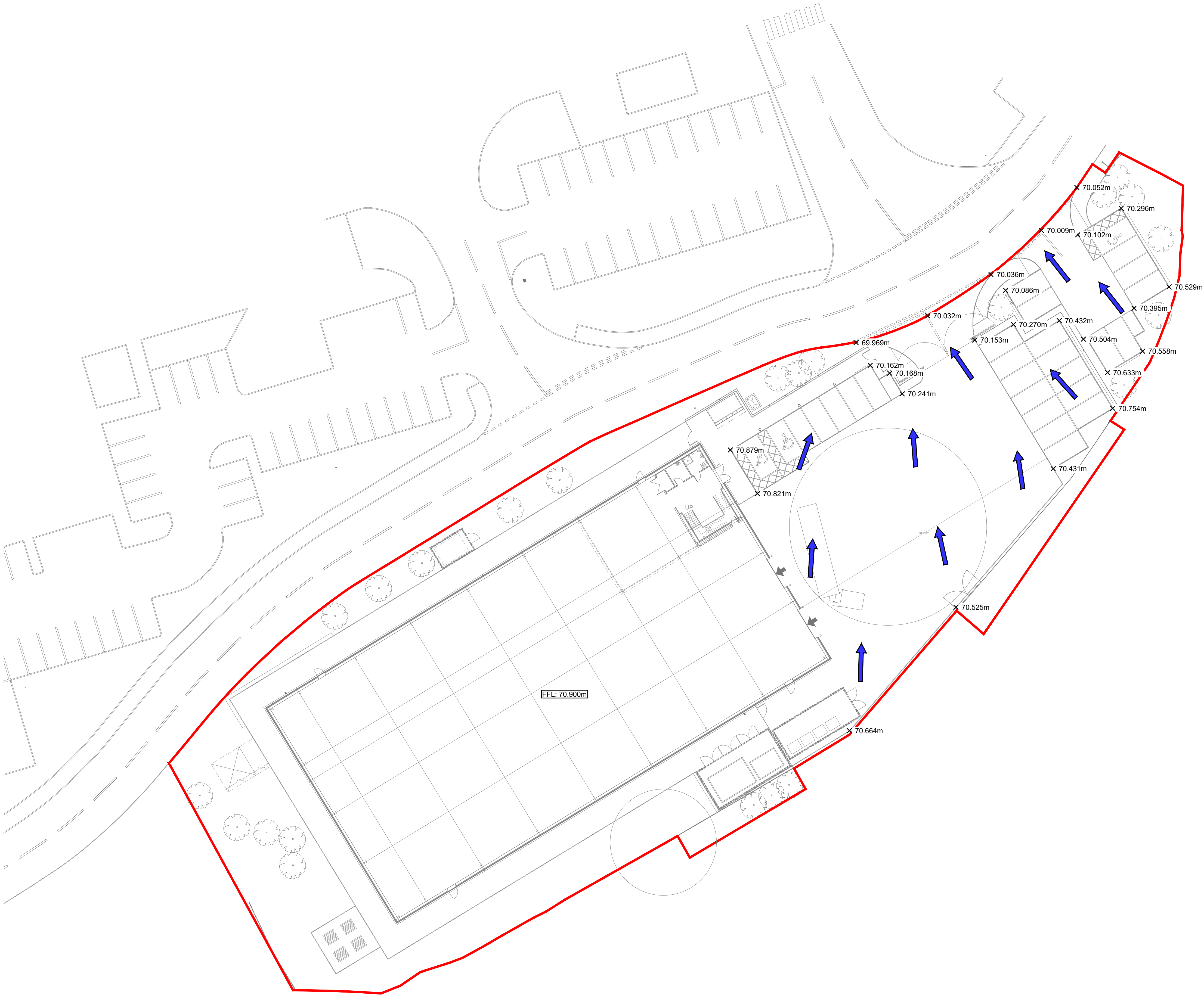
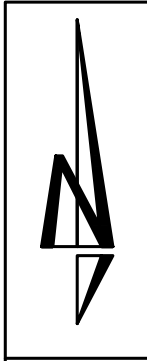
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DRAWING TITLE
EXISTING AND PROPOSED
IMPERMEABLE AREAS

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KEY:

SITE BOUNDARY

100 YEAR + 40% CLIMATE CONTROL FLOW DIRECTION

X 32.900m

PROPOSED SPOT LEVEL

REV	DATE	BY	DESCRIPTION	CHK'D

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	CHECKED BY DV	

DRAWING No
40055-BGL-XX-XX-DR-C-00250

REV
P01

APPENDIX B – HYDRAULIC CALCULATIONS

Design Settings

Rainfall Methodology	FSR	Maximum Time of Concentration (mins)	30.00
Return Period (years)	1	Maximum Rainfall (mm/hr)	50.0
Additional Flow (%)	0	Minimum Velocity (m/s)	1.00
FSR Region	England and Wales	Connection Type	Level Soffits
M5-60 (mm)	17.000	Minimum Backdrop Height (m)	0.200
Ratio-R	0.300	Preferred Cover Depth (m)	1.200
CV	0.850	Include Intermediate Ground	✓
Time of Entry (mins)	5.00	Enforce best practice design rules	x

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Width (mm)	Easting (m)	Northing (m)	Depth (m)
SWMH01	0.018	5.00	70.781	600		385480.741	405209.742	0.531
SWMH02	0.078	5.00	70.800	1200		385487.266	405201.041	1.300
Tank			70.300	1200		385498.156	405196.415	2.075
SWMH03	0.096	5.00	70.827	1200		385495.824	405184.242	1.327
Dummy	0.003	5.00	70.380	450	450	385531.047	405224.590	0.730
SWMH04	0.006	5.00	70.247	600		385524.687	405234.795	0.797
SU01	0.010	5.00	69.985	450	450	385516.694	405231.086	0.540
SWMH05	0.003	5.00	70.240	600		385514.583	405226.230	0.965
SWMH06	0.015	5.00	70.255	600		385511.983	405220.722	1.049
SWMH07	0.025	5.00	70.283	600		385499.352	405217.722	1.083
SU02	0.082	5.00	70.010	450	450	385500.022	405220.592	0.540
SWMH08			70.322	600		385503.361	405215.422	1.322
SWMH09			70.402	600		385510.281	405203.715	1.567
SWMH10 - Control			70.732	1200		385496.442	405195.385	2.522
SWMH11 - Outfall			70.772	1200		385493.806	405193.803	3.587

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
1.001	SWMH01	SWMH02	10.876	0.600	70.250	69.575	0.675	16.1	150	5.07	40.1
1.002	SWMH02	Tank	11.832	0.600	69.500	68.800	0.700	16.9	225	5.13	39.9
2.001	SWMH03	Tank	12.394	0.600	69.500	68.800	0.700	17.7	225	5.07	40.1
3.001	Dummy	SWMH04	12.025	0.600	69.650	69.500	0.150	80.2	100	5.23	39.6
3.002	SWMH04	SWMH05	13.246	0.600	69.450	69.275	0.175	75.7	150	5.42	39.1
3.003	SU01	SWMH05	5.295	0.600	69.445	69.285	0.160	33.1	150	5.05	40.2
3.004	SWMH05	SWMH08	15.580	0.600	69.285	69.075	0.210	74.2	150	5.65	38.4
3.005	SWMH06	SWMH08	10.121	0.600	69.206	69.075	0.131	77.3	150	5.15	39.9

Name	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
1.001	2.522	44.6	2.2	0.381	1.075	0.018	0.0	23	1.310
1.002	3.198	127.1	11.8	1.075	1.275	0.096	0.0	46	2.022
2.001	3.124	124.2	11.8	1.102	1.275	0.096	0.0	47	1.997
3.001	0.860	6.8	0.4	0.630	0.647	0.003	0.0	16	0.456
3.002	1.157	20.4	1.1	0.647	0.815	0.009	0.0	24	0.617
3.003	1.756	31.0	1.2	0.390	0.805	0.010	0.0	20	0.849
3.004	1.168	20.6	2.6	0.805	1.097	0.022	0.0	36	0.802
3.005	1.145	20.2	1.8	0.899	1.097	0.015	0.0	31	0.716

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
4.001	SU02	SWMH08	6.154	0.600	69.470	69.000	0.470	13.1	225	5.03	40.3
4.002	SWMH07	SWMH08	4.622	0.600	69.200	69.075	0.125	37.0	150	5.05	40.2
4.003	SWMH08	SWMH09	13.599	0.600	69.000	68.835	0.165	82.4	225	5.80	38.0
4.004	SWMH09	Tank	14.153	0.600	68.835	68.800	0.035	404.4	225	6.17	37.0
5.001	Tank	SWMH10 - Control	2.000	0.600	68.225	68.210	0.015	133.3	225	6.20	37.0
5.002	SWMH10 - Control	SWMH11 - Outfall	3.074	0.600	68.210	67.185	1.025	3.0	225	6.21	36.9

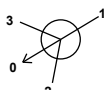
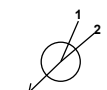
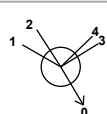
Name	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
4.001	3.635	144.5	10.1	0.315	1.097	0.082	0.0	40	2.112
4.002	1.660	29.3	3.1	0.933	1.097	0.025	0.0	33	1.075
4.003	1.441	57.3	16.8	1.097	1.342	0.144	0.0	83	1.254
4.004	0.644	25.6	16.4	1.342	1.275	0.144	0.0	131	0.681
5.001	1.130	44.9	38.1	1.850	2.297	0.336	0.0	160	1.264
5.002	7.610	302.6	38.1	2.297	3.362	0.336	0.0	53	5.226

Pipeline Schedule

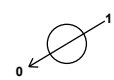
Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
1.001	10.876	16.1	150	Circular	70.781	70.250	0.381	70.800	69.575	1.075
1.002	11.832	16.9	225	Circular	70.800	69.500	1.075	70.300	68.800	1.275
2.001	12.394	17.7	225	Circular	70.827	69.500	1.102	70.300	68.800	1.275
3.001	12.025	80.2	100	Circular	70.380	69.650	0.630	70.247	69.500	0.647
3.002	13.246	75.7	150	Circular	70.247	69.450	0.647	70.240	69.275	0.815
3.003	5.295	33.1	150	Circular	69.985	69.445	0.390	70.240	69.285	0.805
3.004	15.580	74.2	150	Circular	70.240	69.285	0.805	70.322	69.075	1.097
3.005	10.121	77.3	150	Circular	70.255	69.206	0.899	70.322	69.075	1.097
4.001	6.154	13.1	225	Circular	70.010	69.470	0.315	70.322	69.000	1.097
4.002	4.622	37.0	150	Circular	70.283	69.200	0.933	70.322	69.075	1.097
4.003	13.599	82.4	225	Circular	70.322	69.000	1.097	70.402	68.835	1.342
4.004	14.153	404.4	225	Circular	70.402	68.835	1.342	70.300	68.800	1.275
5.001	2.000	133.3	225	Circular	70.300	68.225	1.850	70.732	68.210	2.297
5.002	3.074	3.0	225	Circular	70.732	68.210	2.297	70.772	67.185	3.362

Link	US Node	Dia (mm)	Width (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.001	SWMH01	600		Manhole	Adoptable	SWMH02	1200	Manhole	Adoptable
1.002	SWMH02	1200		Manhole	Adoptable	Tank	1200	Manhole	Adoptable
2.001	SWMH03	1200		Manhole	Adoptable	Tank	1200	Manhole	Adoptable
3.001	Dummy	450	450	Manhole	Adoptable	SWMH04	600	Manhole	Adoptable
3.002	SWMH04	600		Manhole	Adoptable	SWMH05	600	Manhole	Adoptable
3.003	SU01	450	450	Manhole	Adoptable	SWMH05	600	Manhole	Adoptable
3.004	SWMH05	600		Manhole	Adoptable	SWMH08	600	Manhole	Adoptable
3.005	SWMH06	600		Manhole	Adoptable	SWMH08	600	Manhole	Adoptable
4.001	SU02	450	450	Manhole	Adoptable	SWMH08	600	Manhole	Adoptable
4.002	SWMH07	600		Manhole	Adoptable	SWMH08	600	Manhole	Adoptable
4.003	SWMH08	600		Manhole	Adoptable	SWMH09	600	Manhole	Adoptable
4.004	SWMH09	600		Manhole	Adoptable	Tank	1200	Manhole	Adoptable
5.001	Tank	1200		Manhole	Adoptable	SWMH10 - Control	1200	Manhole	Adoptable
5.002	SWMH10 - Control	1200		Manhole	Adoptable	SWMH11 - Outfall	1200	Manhole	Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Width (mm)	Connections	Link	IL (m)	Dia (mm)	
SWMH01	385480.741	405209.742	70.781	0.531	600			0	1.001	70.250	150
SWMH02	385487.266	405201.041	70.800	1.300	1200			1	1.001	69.575	150
								0	1.002	69.500	225
Tank	385498.156	405196.415	70.300	2.075	1200			1	4.004	68.800	225
								2	2.001	68.800	225
								3	1.002	68.800	225
								0	5.001	68.225	225
SWMH03	385495.824	405184.242	70.827	1.327	1200						
								0	2.001	69.500	225
Dummy	385531.047	405224.590	70.380	0.730	450	450					
								0	3.001	69.650	100
SWMH04	385524.687	405234.795	70.247	0.797	600			1	3.001	69.500	100
								0	3.002	69.450	150
SU01	385516.694	405231.086	69.985	0.540	450	450					
								0	3.003	69.445	150
SWMH05	385514.583	405226.230	70.240	0.965	600			1	3.003	69.285	150
								2	3.002	69.275	150
								0	3.004	69.285	150
SWMH06	385511.983	405220.722	70.255	1.049	600						
								0	3.005	69.206	150
SWMH07	385499.352	405217.722	70.283	1.083	600						
								0	4.002	69.200	150
SU02	385500.022	405220.592	70.010	0.540	450	450					
								0	4.001	69.470	225
SWMH08	385503.361	405215.422	70.322	1.322	600			1	4.002	69.075	150
								2	4.001	69.000	225
								3	3.005	69.075	150
								4	3.004	69.075	150
								0	4.003	69.000	225
SWMH09	385510.281	405203.715	70.402	1.567	600			1	4.003	68.835	225
								0	4.004	68.835	225

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Width (mm)	Connections	Link	IL (m)	Dia (mm)	
SWMH10 - Control	385496.442	405195.385	70.732	2.522	1200			1	5.001	68.210	225
								0	5.002	68.210	225
SWMH11 - Outfall	385493.806	405193.803	70.772	3.587	1200			1	5.002	67.185	225

Simulation Settings

Rainfall Methodology	FSR	Analysis Speed	Normal
FSR Region	England and Wales	Skip Steady State	x
M5-60 (mm)	20.000	Drain Down Time (mins)	240
Ratio-R	0.400	Additional Storage (m³/ha)	20.0
Summer CV	0.800	Check Discharge Rate(s)	x
Winter CV	0.900	Check Discharge Volume	x

Storm Durations

15 | 30 | 60 | 120 | 180 | 240 | 360 | 480 | 600 | 720 | 960 | 1440

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
1	0	0	0
30	35	0	0
100	40	0	0

Node SWMH10 - Control Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	68.210	Product Number	CTL-SHE-0087-4500-2000-4500
Design Depth (m)	2.000	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	4.5	Min Node Diameter (mm)	1200

Node Tank Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	68.225
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	200.0	0.0	0.800	200.0	0.0	0.801	0.0	0.0

Node Dummy Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	69.650
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	24

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	29.5	0.0	0.300	29.5	0.0	0.301	0.0	0.0

Node SWMH04 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	69.450
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	104

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	56.0	0.0	0.300	56.0	0.0	0.301	0.0	0.0

Node SWMH05 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	69.275
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	200

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	24.8	0.0	0.300	24.8	0.0	0.301	0.0	0.0

Node SWMH06 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	69.206
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	232

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	150.0	0.0	0.300	150.0	0.0	0.301	0.0	0.0

Node SWMH07 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	69.200
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	240

Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)	Depth (m)	Area (m ²)	Inf Area (m ²)
0.000	103.5	0.0	0.300	103.5	0.0	0.301	0.0	0.0

Results for 1 year Critical Storm Duration. Lowest mass balance: 99.96%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	SWMH01	10	70.276	0.026	2.7	0.0246	0.0000	OK
15 minute winter	SWMH02	10	69.554	0.054	14.5	0.1247	0.0000	OK
180 minute winter	Tank	140	68.398	0.173	12.4	32.9826	0.0000	OK
15 minute winter	SWMH03	10	69.554	0.054	14.5	0.1394	0.0000	OK
15 minute winter	Dummy	13	69.664	0.014	0.5	0.1240	0.0000	OK
30 minute winter	SWMH04	22	69.468	0.018	0.9	0.3136	0.0000	OK
15 minute winter	SU01	10	69.468	0.023	1.5	0.0130	0.0000	OK
15 minute winter	SWMH05	12	69.317	0.042	2.2	0.3266	0.0000	OK
60 minute winter	SWMH06	41	69.226	0.020	1.2	0.8964	0.0000	OK
15 minute winter	SWMH07	13	69.230	0.030	3.8	0.9470	0.0000	OK
15 minute winter	SU02	10	69.514	0.044	12.4	0.1436	0.0000	OK
15 minute winter	SWMH08	11	69.084	0.084	15.7	0.0237	0.0000	OK
15 minute winter	SWMH09	11	68.963	0.128	15.7	0.0361	0.0000	OK
180 minute winter	SWMH10 - Control	140	68.397	0.187	3.4	0.2118	0.0000	OK
15 minute summer	SWMH11 - Outfall	1	67.185	0.000	2.7	0.0000	0.0000	OK

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	SWMH01	1.001	SWMH02	2.7	1.370	0.060	0.0213	
15 minute winter	SWMH02	1.002	Tank	14.4	2.063	0.113	0.0824	
180 minute winter	Tank	5.001	SWMH10 - Control	3.4	0.215	0.075	0.0680	
15 minute winter	SWMH03	2.001	Tank	14.4	2.032	0.116	0.0877	
15 minute winter	Dummy	3.001	SWMH04	0.3	0.414	0.039	0.0076	
30 minute winter	SWMH04	3.002	SWMH05	0.6	0.273	0.031	0.0335	
15 minute winter	SU01	3.003	SWMH05	1.5	0.859	0.048	0.0110	
15 minute winter	SWMH05	3.004	SWMH08	2.0	0.733	0.096	0.0420	
60 minute winter	SWMH06	3.005	SWMH08	0.7	0.536	0.035	0.0136	
15 minute winter	SWMH07	4.002	SWMH08	2.3	0.950	0.077	0.0110	
15 minute winter	SU02	4.001	SWMH08	12.3	1.377	0.085	0.0577	
15 minute winter	SWMH08	4.003	SWMH09	15.7	0.860	0.274	0.2492	
15 minute winter	SWMH09	4.004	Tank	15.5	0.761	0.606	0.2892	
180 minute winter	SWMH10 - Control	Hydro-Brake®	SWMH11 - Outfall	3.4				49.9

Results for 30 year +35% CC Critical Storm Duration. Lowest mass balance: 99.96%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	SWMH01	10	70.298	0.048	9.0	0.0459	0.0000	OK
15 minute winter	SWMH02	10	69.605	0.105	47.9	0.2452	0.0000	OK
240 minute winter	Tank	236	68.948	0.723	31.2	138.2045	0.0000	SURCHARGED
15 minute winter	SWMH03	10	69.606	0.106	48.0	0.2740	0.0000	OK
15 minute winter	Dummy	12	69.679	0.029	1.5	0.2627	0.0000	OK
15 minute winter	SWMH04	13	69.490	0.040	3.9	0.6825	0.0000	OK
15 minute winter	SU01	10	69.487	0.042	5.0	0.0239	0.0000	OK
15 minute winter	SWMH05	12	69.352	0.077	8.9	0.6003	0.0000	OK
15 minute winter	SWMH06	13	69.282	0.076	11.6	3.4620	0.0000	OK
15 minute winter	SWMH07	12	69.293	0.093	12.5	2.9410	0.0000	OK
15 minute winter	SU02	10	69.558	0.088	41.0	0.2862	0.0000	OK
15 minute winter	SWMH08	12	69.282	0.282	51.4	0.0798	0.0000	SURCHARGED
15 minute winter	SWMH09	12	69.134	0.299	46.4	0.0845	0.0000	SURCHARGED
240 minute winter	SWMH10 - Control	240	68.948	0.738	3.8	0.8347	0.0000	SURCHARGED
15 minute summer	SWMH11 - Outfall	1	67.185	0.000	3.6	0.0000	0.0000	OK

Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	SWMH01	1.001	SWMH02	8.9	1.914	0.201	0.0508	
15 minute winter	SWMH02	1.002	Tank	47.6	2.795	0.375	0.2017	
15 minute winter	Tank	5.001	SWMH10 - Control	4.3	0.436	0.096	0.0795	
15 minute winter	SWMH03	2.001	Tank	47.7	2.754	0.384	0.2145	
15 minute winter	Dummy	3.001	SWMH04	1.2	0.638	0.173	0.0220	
15 minute winter	SWMH04	3.002	SWMH05	3.1	0.515	0.153	0.0850	
15 minute winter	SU01	3.003	SWMH05	5.0	1.018	0.160	0.0300	
15 minute winter	SWMH05	3.004	SWMH08	8.5	0.904	0.414	0.1965	
15 minute winter	SWMH06	3.005	SWMH08	8.5	0.973	0.419	0.1344	
15 minute winter	SWMH07	4.002	SWMH08	11.6	1.284	0.397	0.0670	
15 minute winter	SU02	4.001	SWMH08	40.4	1.618	0.280	0.1668	
15 minute winter	SWMH08	4.003	SWMH09	46.4	1.168	0.811	0.5408	
15 minute winter	SWMH09	4.004	Tank	46.5	1.191	1.815	0.5224	
15 minute winter	SWMH10 - Control	Hydro-Brake®	SWMH11 - Outfall	3.6				52.2

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.96%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	SWMH01	10	70.306	0.056	12.1	0.0541	0.0000	OK
360 minute winter	SWMH02	344	69.882	0.382	8.9	0.8912	0.0000	SURCHARGED
360 minute winter	Tank	344	69.883	1.658	30.8	153.9697	0.0000	SURCHARGED
360 minute winter	SWMH03	344	69.882	0.382	8.8	0.9857	0.0000	SURCHARGED
360 minute winter	Dummy	344	69.883	0.233	0.8	2.1284	0.0000	SURCHARGED
360 minute winter	SWMH04	344	69.883	0.433	3.1	5.2358	0.0000	SURCHARGED
360 minute winter	SU01	344	69.883	0.438	0.9	0.2510	0.0000	FLOOD RISK
360 minute winter	SWMH05	344	69.883	0.608	4.1	2.4454	0.0000	SURCHARGED
360 minute winter	SWMH06	344	69.883	0.677	8.4	13.9076	0.0000	SURCHARGED
360 minute winter	SWMH07	344	69.882	0.682	9.8	9.8383	0.0000	SURCHARGED
360 minute winter	SU02	344	69.883	0.413	7.6	1.3374	0.0000	FLOOD RISK
360 minute winter	SWMH08	344	69.883	0.883	15.1	0.2500	0.0000	SURCHARGED
360 minute winter	SWMH09	344	69.882	1.047	13.2	0.2964	0.0000	SURCHARGED
360 minute winter	SWMH10 - Control	344	69.882	1.672	6.0	1.8909	0.0000	SURCHARGED
15 minute summer	SWMH11 - Outfall	1	67.185	0.000	3.6	0.0000	0.0000	OK
Link Event (Outflow)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	SWMH01	1.001	SWMH02	12.0	2.067	0.270	0.0633	
15 minute winter	SWMH02	1.002	Tank	64.1	2.982	0.504	0.2543	
600 minute winter	Tank	5.001	SWMH10 - Control	8.1	0.211	0.179	0.0795	
15 minute winter	SWMH03	2.001	Tank	64.2	2.939	0.517	0.2709	
15 minute winter	Dummy	3.001	SWMH04	1.6	0.701	0.244	0.0282	
15 minute winter	SWMH04	3.002	SWMH05	4.6	0.501	0.225	0.1378	
15 minute winter	SU01	3.003	SWMH05	6.7	1.038	0.215	0.0492	
15 minute summer	SWMH05	3.004	SWMH08	11.4	0.820	0.551	0.2299	
15 minute winter	SWMH06	3.005	SWMH08	14.1	1.103	0.695	0.1752	
15 minute winter	SWMH07	4.002	SWMH08	17.0	1.359	0.579	0.0814	
15 minute winter	SU02	4.001	SWMH08	54.6	1.644	0.377	0.1880	
15 minute winter	SWMH08	4.003	SWMH09	51.7	1.300	0.902	0.5408	
15 minute winter	SWMH09	4.004	Tank	51.7	1.312	2.021	0.5334	
360 minute winter	SWMH10 - Control	Hydro-Brake®	SWMH11 - Outfall	4.2				115.0

Calculated by: Steven Bowron

Site name: Rhodes Business Park

Site location: Rhodes, Manchester

Site Details

Latitude:

Longitude:

Reference: 728753500

Date: Jan 23 2024 15:58

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Runoff estimation approach

IH124

Site characteristics

Total site area (ha): .5375

Methodology

Q_{BAR} estimation method: Calculate from SPR and SAAR

SPR estimation method: Calculate from SOIL type

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

Soil characteristics

	Default	Edited
SOIL type:	0	4
HOST class:	N/A	N/A
SPR/SPRHOST:	0.00	0.47

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

Hydrological characteristics

	Default	Edited
SAAR (mm):	0	1003
Hydrological region:	1	1
Growth curve factor 1 year:	-	0.87
Growth curve factor 30 years:	-	1.7
Growth curve factor 100 years:	-	2.08
Growth curve factor 200 years:	-	2.37

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates	Default	Edited
-------------------------	---------	--------

Q_{BAR} (l/s):	0	3.95
1 in 1 year (l/s):		3.44
1 in 30 years (l/s):		6.72
1 in 100 year (l/s):		8.22
1 in 200 years (l/s):		9.37

This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this tool are estimates of greenfield runoff rates. The use of these results is the responsibility of the users of this tool. No liability will be accepted by HR Wallingford, the Environment Agency, CEH, Hydrosolutions or any other organisation for the use of this data in the design or operational characteristics of any drainage scheme.

Design Settings

Rainfall Methodology	FSR	Maximum Time of Concentration (mins)	30.00
Return Period (years)	1	Maximum Rainfall (mm/hr)	50.0
Additional Flow (%)	0	Minimum Velocity (m/s)	1.00
FSR Region	England and Wales	Connection Type	Level Soffits
M5-60 (mm)	17.000	Minimum Backdrop Height (m)	0.200
Ratio-R	0.300	Preferred Cover Depth (m)	1.200
CV	0.850	Include Intermediate Ground	✓
Time of Entry (mins)	5.00	Enforce best practice design rules	x

Nodes

Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Easting (m)	Northing (m)	Depth (m)
SWMH9	0.330	5.00	70.100	1200	385524.067	405221.607	1.120
MH004			70.070	1200	385495.008	405189.995	3.270

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
1.001	SWMH9	MH004	42.939	0.600	68.980	66.800	2.180	19.7	150	5.31	39.4

Name	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
1.001	2.279	40.3	39.9	0.970	3.120	0.330	0.0	122	2.589

Pipeline Schedule

Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
1.001	42.939	19.7	150	Circular	70.100	68.980	0.970	70.070	66.800	3.120

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.001	SWMH9	1200	Manhole	Adoptable	MH004	1200	Manhole	Adoptable

Manhole Schedule

Node	Easting (m)	Northing (m)	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
SWMH9	385524.067	405221.607	70.100	1.120	1200				
						0	1.001	68.980	150
MH004	385495.008	405189.995	70.070	3.270	1200				
						1	1.001	66.800	150

Simulation Settings

Rainfall Methodology	FSR	Analysis Speed	Normal
FSR Region	England and Wales	Skip Steady State	x
M5-60 (mm)	20.000	Drain Down Time (mins)	240
Ratio-R	0.400	Additional Storage (m³/ha)	20.0
Summer CV	0.800	Check Discharge Rate(s)	x
Winter CV	0.900	Check Discharge Volume	x

Storm Durations

15 | 30 | 60 | 120 | 180 | 240 | 360 | 480 | 600 | 720 | 960 | 1440

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
1	0	0	0
30	0	0	0
100	0	0	0

Node TANK Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	0.000
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	168.0	0.0	0.800	168.0	0.0	0.801	0.0	0.0

Node SWMH03 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	0.000
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Time to half empty (mins)	

Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)	Depth (m)	Area (m²)	Inf Area (m²)
0.000	62.5	0.0	0.300	62.5	0.0	0.301	0.0	0.0

Results for 1 year Critical Storm Duration. Lowest mass balance: 99.63%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	SWMH9	12	69.301	0.321	49.9	2.2513	0.0000	SURCHARGED
15 minute winter	MH004	12	66.952	0.152	41.9	0.0000	0.0000	OK
Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	SWMH9	1.001	MH004	41.9	2.502	1.041	0.7559	22.9

Results for 30 year Critical Storm Duration. Lowest mass balance: 99.63%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
15 minute winter	SWMH9	9	70.100	1.120	122.2	7.8669	17.3826	FLOOD
15 minute summer	MH004	18	66.952	0.152	46.4	0.0000	0.0000	OK
Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
15 minute winter	SWMH9	1.001	MH004	46.4	2.639	1.153	0.7557	41.1

Results for 100 year Critical Storm Duration. Lowest mass balance: 99.63%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	SWMH9	15	70.100	1.120	122.9	7.8669	29.2826	FLOOD
30 minute summer	MH004	14	66.952	0.152	46.4	0.0000	0.0000	OK
Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	SWMH9	1.001	MH004	46.4	2.639	1.153	0.7559	67.6

Hydraulic Modelling Study at Silburn Way, Middleton

Draft Report

S3-P01

November 2023

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This report describes work commissioned by Domenico Venuti on behalf of Belshaw by an instruction dated 17th August 2023. The Client's representative for the work was Domenico Venuti and Daniel Barnes of Burrows Graham. Hannah Evans of JBA Consulting carried out this work.

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Abbreviations

1D	One Dimensional
2D	Two Dimensional
AEP	Annual Exceedance Probability
CC	Climate Change
DTM	Digital Terrain Model
EA	Environment Agency
Flood Modeller	1D Hydraulic Modelling Software
FEH	Flood Estimation Handbook
FFL	Finished Floor Levels
Ha	Hectares
HQ	Head-Flow
JBA	Jeremy Benn Associates
LIDAR	Light Detection and Ranging
m AOD	Metres Above Ordnance Datum
NPPF	National Planning and Policy Framework
TUFLOW	2D Hydraulic Modelling Software

1 Introduction

1.1 Terms of Reference

JBA Consulting (JBA) were commissioned by Belshaw to undertake a hydraulic modelling study of the River Irk which flows along the southern boundary of the proposed development site located along Silburn Way, Middleton.

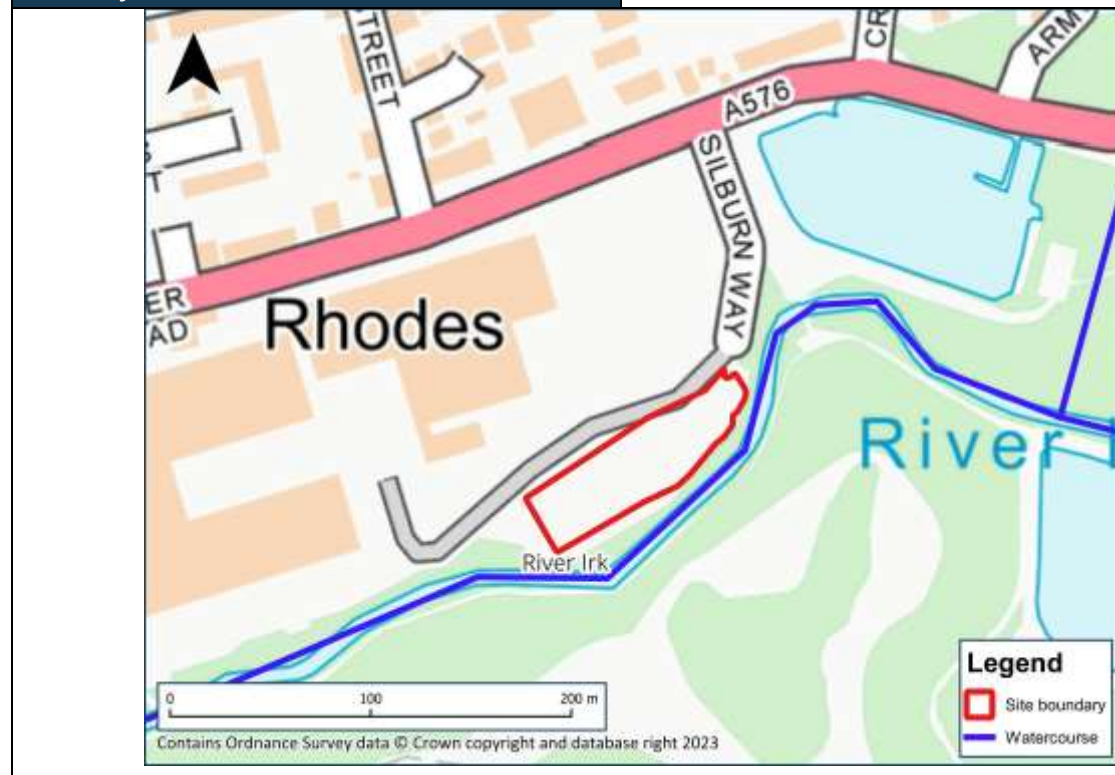
The purpose of this study is to increase the understanding of the fluvial flood risk to the site. To achieve this a 1D-2D ESTRY-TUFLOW hydraulic model was produced to allow accurate representation of flood levels, extents, velocities and hazards within the site boundary.

1.2 Site Description.

The 0.36ha site is located on land off Silburn Way in Middleton. The River Irk flows to the south of the site in a westerly direction.

Table 1-1: Summary of Site Details

Site address	Silburn Way, Middleton
Site area	0.36 hectares (ha)
Existing Land-use	Brownfield Site
OS NGR	SD 85481 05192
Country	England
County	Greater Manchester



1.3 Proposed development

The site is the subject of a proposal for a commercial scheme.

1.4 General approach

A 1D-2D hydraulic model of the River Irk and floodplain in the vicinity of the site was developed in ESTRY-TUFLOW to refine the understanding of flood risk at the site and within the surrounding area to improve upon the EA's Flood Map for Planning

The model incorporates site-specific topographic survey data, channel survey information and the latest publicly available Environment Agency LiDAR data.

Hydrographs were derived for 3.33% (1 in 30-year), 1% (1 in 100-year), 1% with 35% climate change uplift (1 in 100-year plus 35% climate change), and 0.1% (1 in 1,000- year) AEP fluvial events. These hydrographs were applied directly to the model to provide baseline model results.

The model's sensitivity to changes to downstream boundary, roughness and flow values were tested to improve the confidence in the model results within the site boundary and immediate surrounding area.

2 Methodology

2.1 Data availability

Grantham Coates Surveys Ltd (GCS) were commissioned to collect channel survey data from the River Irk in the vicinity of the site in September 2023. In total the survey consisted of 20 cross sections taken at regular intervals along the River Irk and at key hydraulic structures (Appendix B).

A topographic survey was provided by the client for use within the study in October 2023 (Appendix A). The topographic survey covered an area greater than the site as shown in

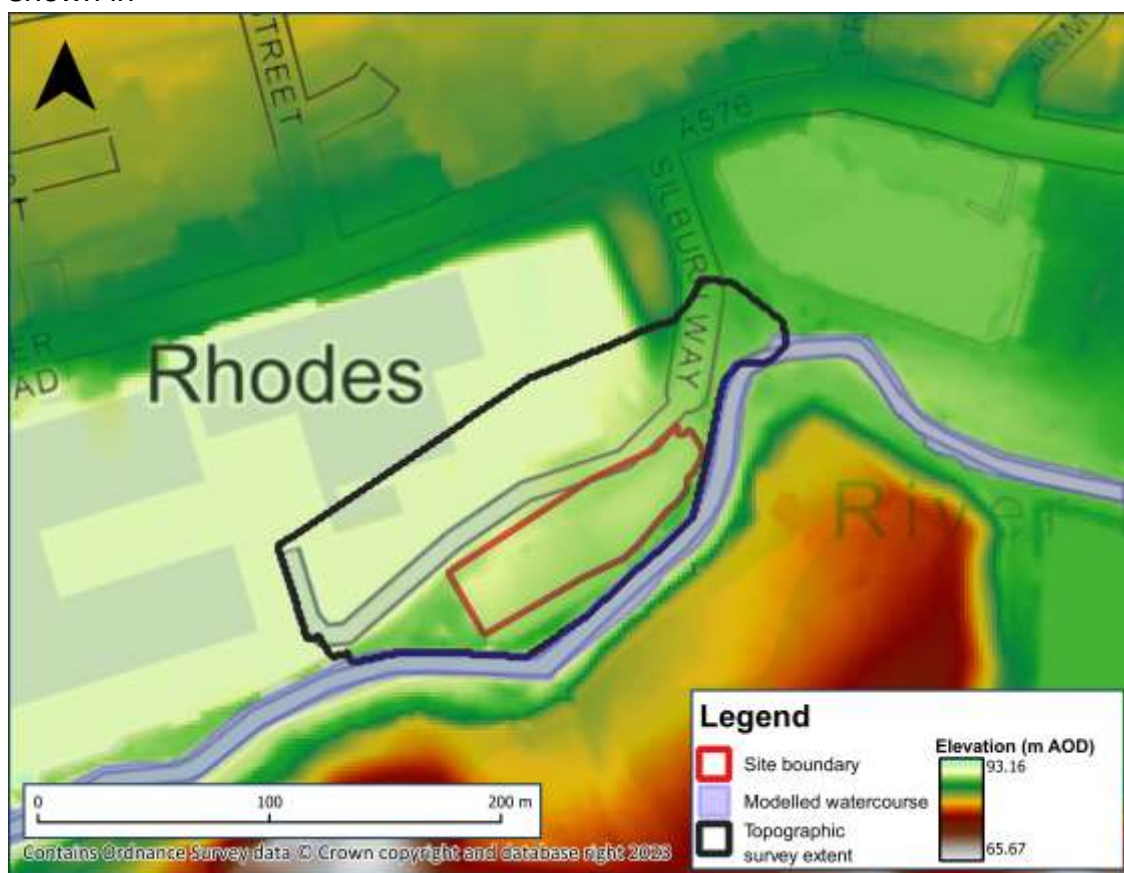


Figure 2-1. Environment Agency LiDAR was obtained from the Open Data website to represent ground levels within the wider floodplain. The latest available LiDAR has a grid resolution of 2m and was flown in 2020.

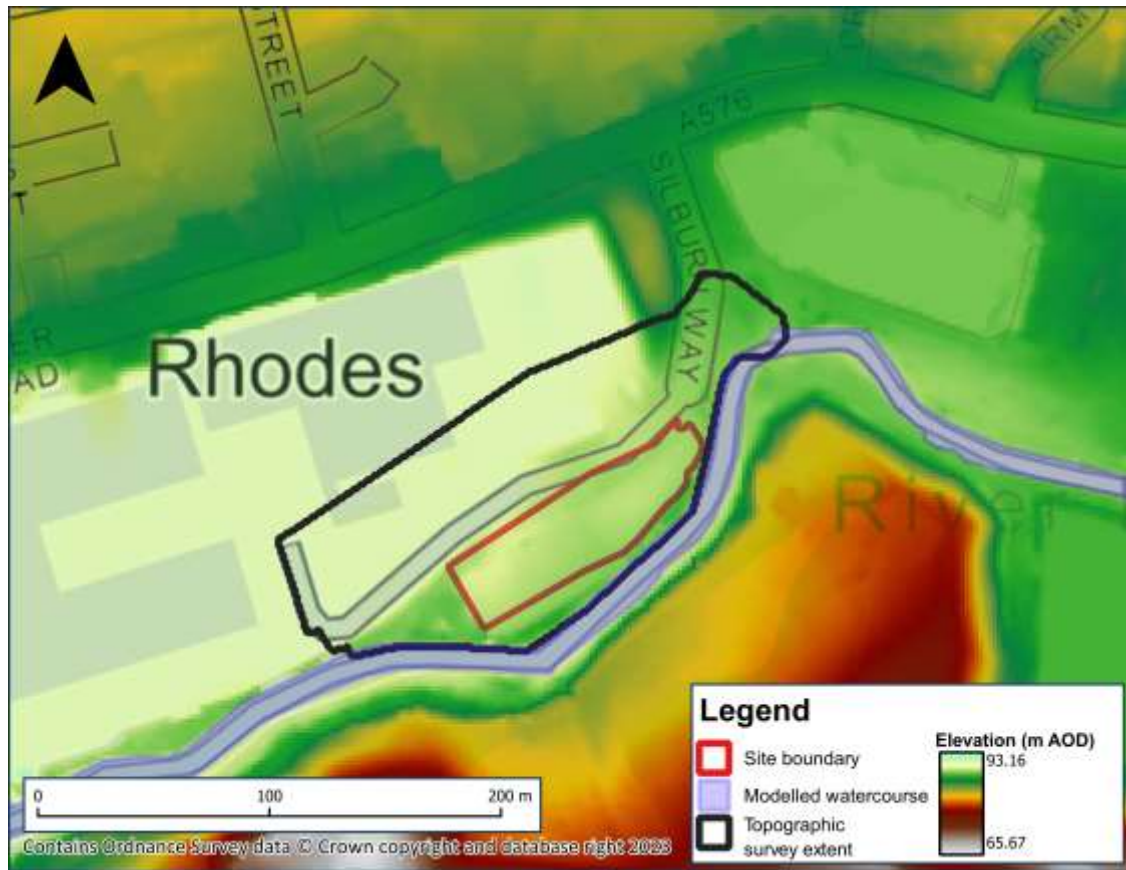


Figure 2-1: Topographic survey extent

2.2 Model extent and build

A 1D-2D ESTRY-TUFLOW model was built to represent the watercourse and floodplain in the vicinity of the proposed development site. The model was run using TUFLOW version 2023-03-AA-IDP-W64.

The River Irk channel was represented within the 1D domain using ESTRY. Model channel cross sections were constructed from channel survey collected by GCS in September 2023. The fluvial 1D ESTRY model, shown in

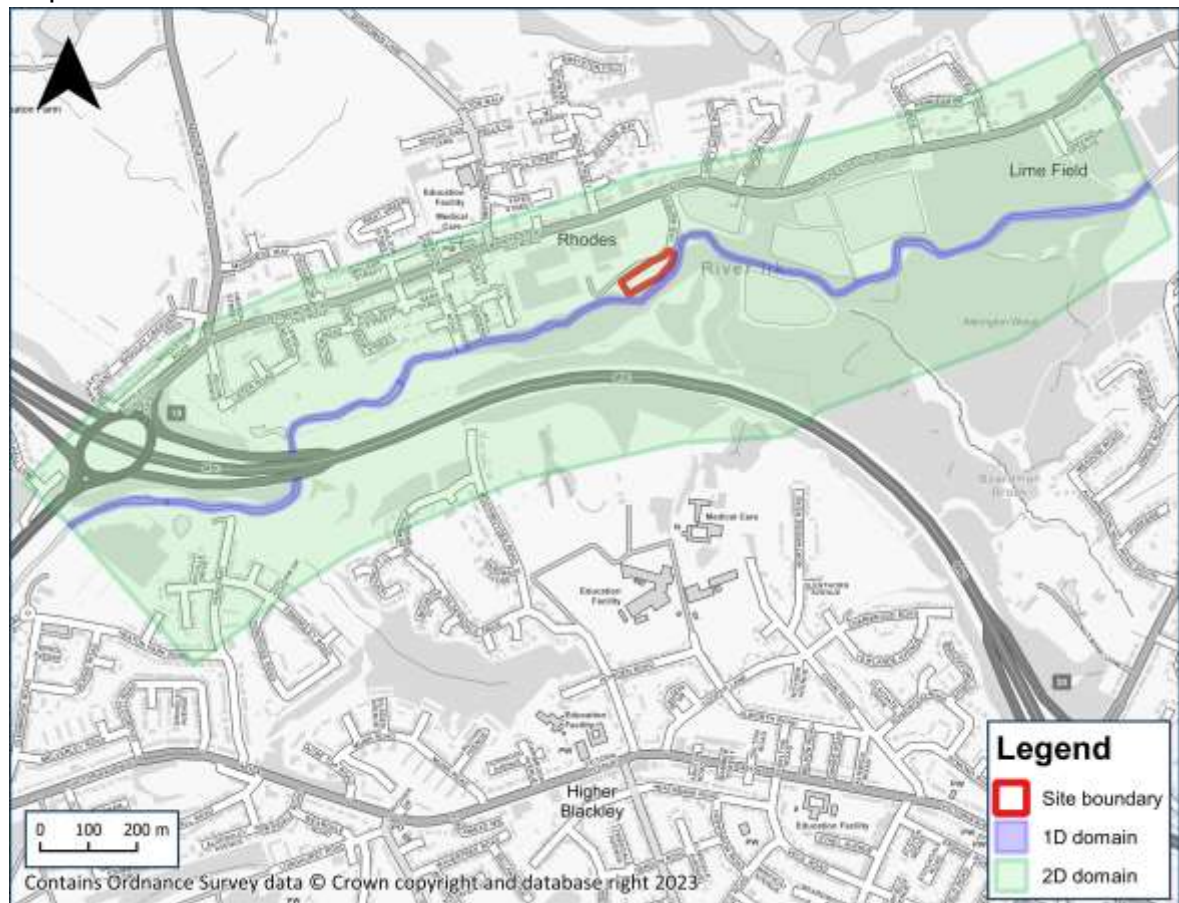


Figure 2-2 covers approximately 2.7km of watercourse within the vicinity of the site. Detailed information on the representation of hydraulic structures within the model are provided as part of Appendix C.

Channel cross sections were trimmed to the top of bank and linked to the 2D domain representing the floodplain.

The 2D TUFLOW model domain has an area of 124 hectares with a 3m grid resolution. Site-specific topographic survey data were used to represent ground levels in the site boundary and floodplain where available (Appendix A) and LiDAR data was used to inform the model topography across the wider floodplain. Bank levels were informed using survey bank levels from the channel survey, site specific topographic survey and LiDAR data.

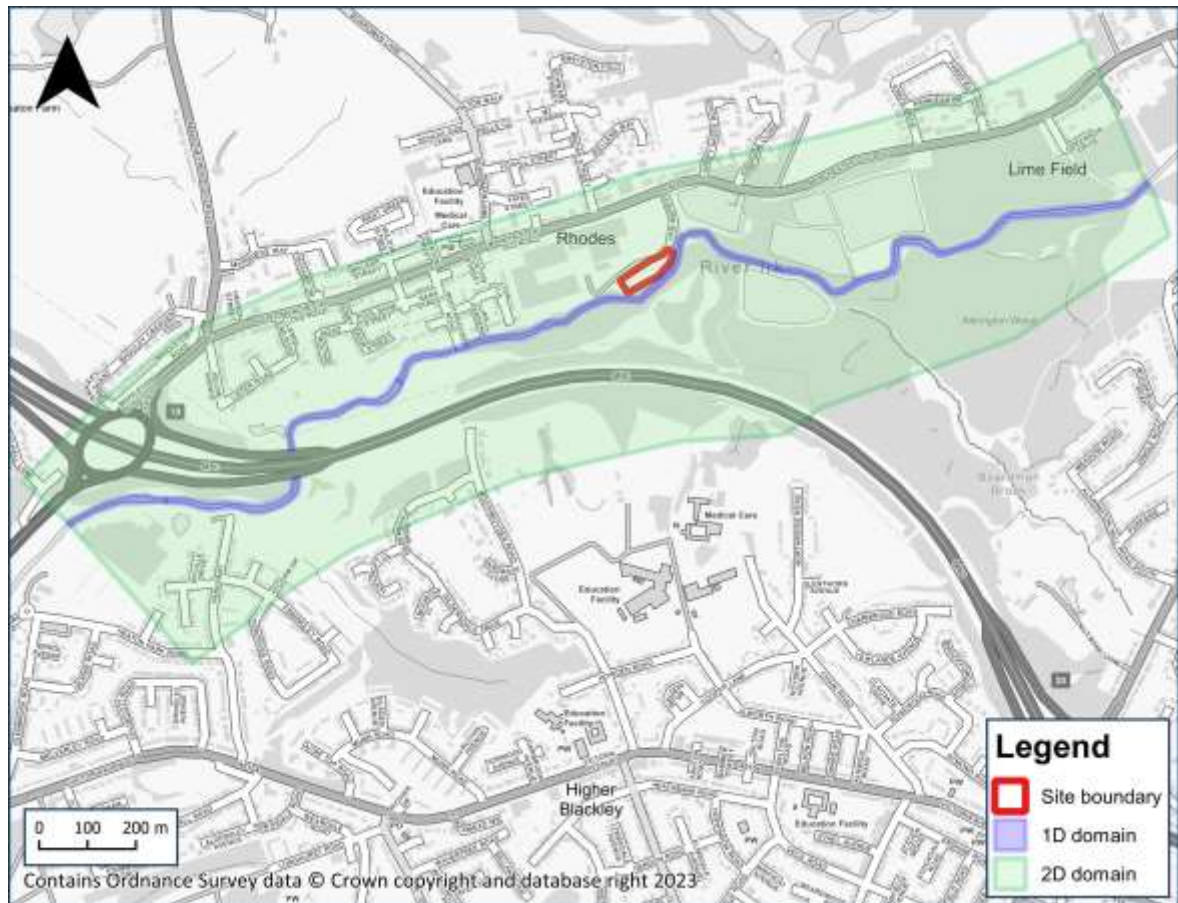


Figure 2-2: Fluvial hydraulic model 1D and 2D domain

2.3 Model roughness

Manning's 'n' values were used to represent the roughness for both the 1D channel and 2D floodplain. The values of the 1D roughness were based on photographs provided by GCS within the channel survey document.

Manning's 'n' values in the floodplain (2D) were set to represent different land uses within the model extent as shown in Table 2-1. Land uses were defined using satellite imagery and OS mapping.

Table 2-1: 2D Manning's 'n'; range within the hydraulic model

Features	Manning's 'n' value assigned
Buildings	0.3
Woodland	0.07
Road and Tracks	0.03
General surface	0.05
Channel roughness	0.04-0.05

2.4 Model boundary conditions

2.4.1 Fluvial model inflows

A hydrological assessment was carried out on the catchment in order to derive the peak flow estimates for the River Irk which flows in a westerly direction along the southern boundary of the site. The full hydrological assessment is included in Appendix D.

Flow-time hydrographs were estimated for the 1 in 30-year (3.3% AEP), 1 in 100-year (1% AEP), 1 in 100-year plus climate change and 1 in 1000-year (0.1% AEP) fluvial flood events. Climate change allowances were applied inline with the latest 2023 Environment Agency Guidance with the 1% AEP flows uplifted by 35% (ref: Central allowances for the 2080's epoch, Irwell Management Catchment).

2.4.2 Downstream boundary conditions

The downstream boundary in the 1D domain was generated using a flow estimator tool to generate a flow-stage rating curve based on the downstream cross section profile. A slope value of 0.001 was used based on the channel slope between the two downstream cross sections. A manning's value of 0.04 was chosen for the channel manning's n based on the channel survey photographs.

In the 2D domain, the downstream boundary is based on the average slope in the floodplain at the downstream extent of the model.

2.5 Model verification

There is no hydrometric information available for the watercourse. Therefore, no hydrometric data specific to the catchment was available to assist with the preparation of design hydrology or model verification/calibration.

The cumulative mass error % does not exceed the preferred +/-1% range and the dvol plot varies smoothly, this suggests that the model is stable. The model has also been validated with a comparison to EA flood maps for planning in Section 4 of this report.

2.6 Sensitivity Analysis

The model's sensitivity to changes in roughness, flow and downstream boundary was checked (see Appendix E).

Appendix E shows that within the site boundary the model results are insensitive to changes in the downstream boundary conditions. The model results are relatively sensitive to the modelled roughness parameters and flow values. However, there are no changes in peak flood extents predicted within the site boundary as a result of the sensitivity testing. The roughness and flow parameters used in the baseline modelling were derived following best practices and are considered to be appropriate.

2.7 Model runs

The following scenarios and events were simulated using the hydraulic model:

- [Baseline scenario] – 1 in 30-year (3.3% AEP) event- existing condition.
- [Baseline scenario] – 1 in 100-year (1% AEP) event- existing condition.
- [Baseline scenario] – 1 in 100-year plus 35% climate change event- existing condition.
- [Baseline scenario] – 1 in 1000-year (0.1% AEP) event- existing condition
- [Sensitivity Analysis] – 1 in 100-year flood event with [+20%] increase in roughness
- [Sensitivity Analysis] – 1 in 100-year flood event with [-20%] increase in roughness
- [Sensitivity Analysis] – 1 in 100-year flood event with [+250mm] increase in peak water level at the downstream model boundary.
- [Sensitivity Analysis] – 1 in 100-year flood event with [+20%] increase in flow.

3 Fluvial Hydraulic modelling Results

3.1 Peak modelled flood extents

The peak modelled flood extent for the 1 in 30-year, 1 in 100-year and 1 in 1000-year fluvial flood events are shown in Figure 3-1.

Figure 3-1 shows that the 30-year and 100-year flood extents do not encroach within the boundary of the site. During the 1,000-year flood event, the entire site is expected to flood.

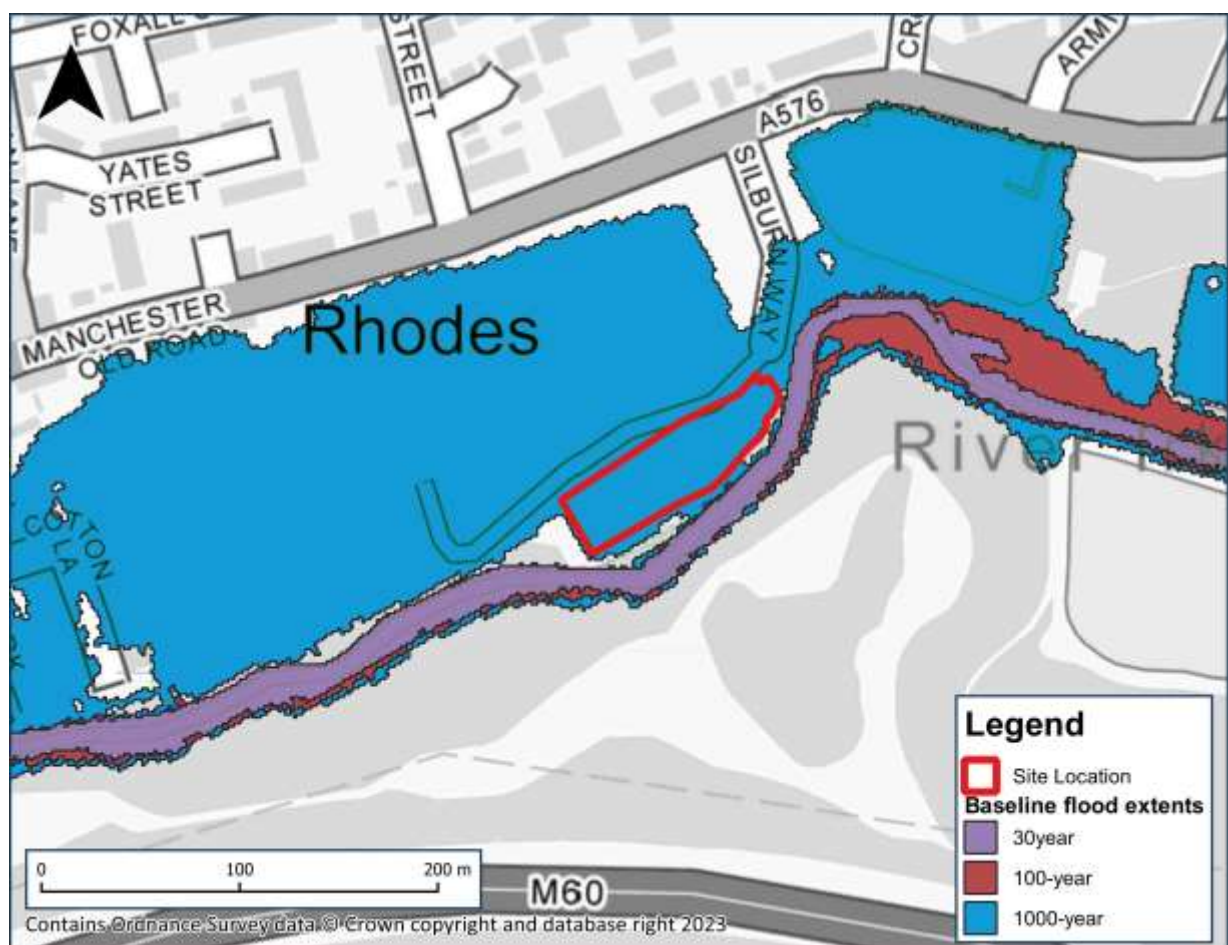


Figure 3-1: Baseline flood extents (30-year, 100-year, 1,000-year)

The 1 in 100-year and 1 in 100-year plus climate change (35%) peak modelled flood extents are shown in Figure 3-2.

Figure 3-2 shows the 100-year plus climate change (35%) flood extent flows into the site along the southern boundary. During the 100-year plus climate change event, the majority of the site is expected to flood.

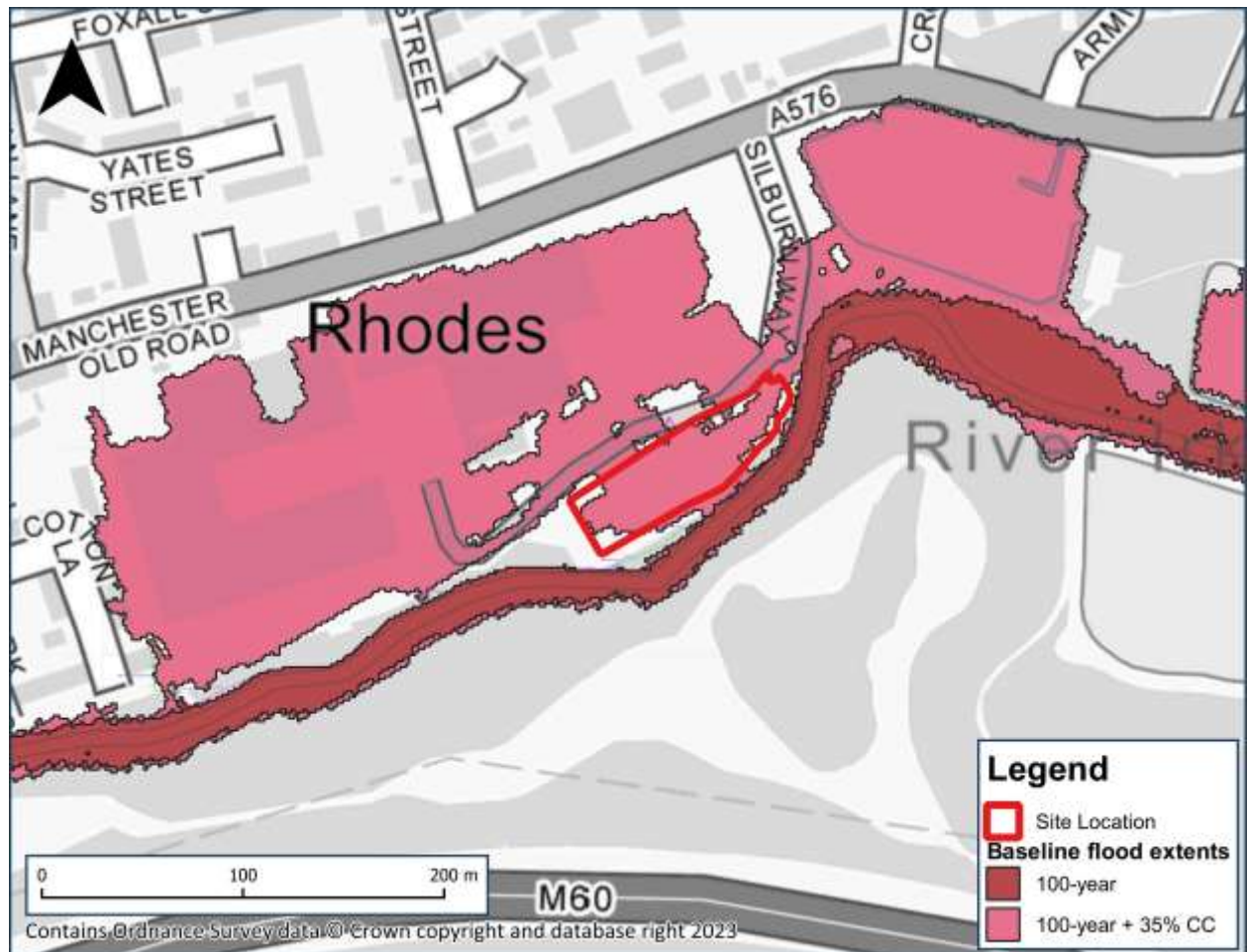


Figure 3-2: Baseline 100-year flood extent with climate change

3.2 Peak modelled flood depths

The peak modelled flood depths during the 100-year plus climate change flood event are shown in

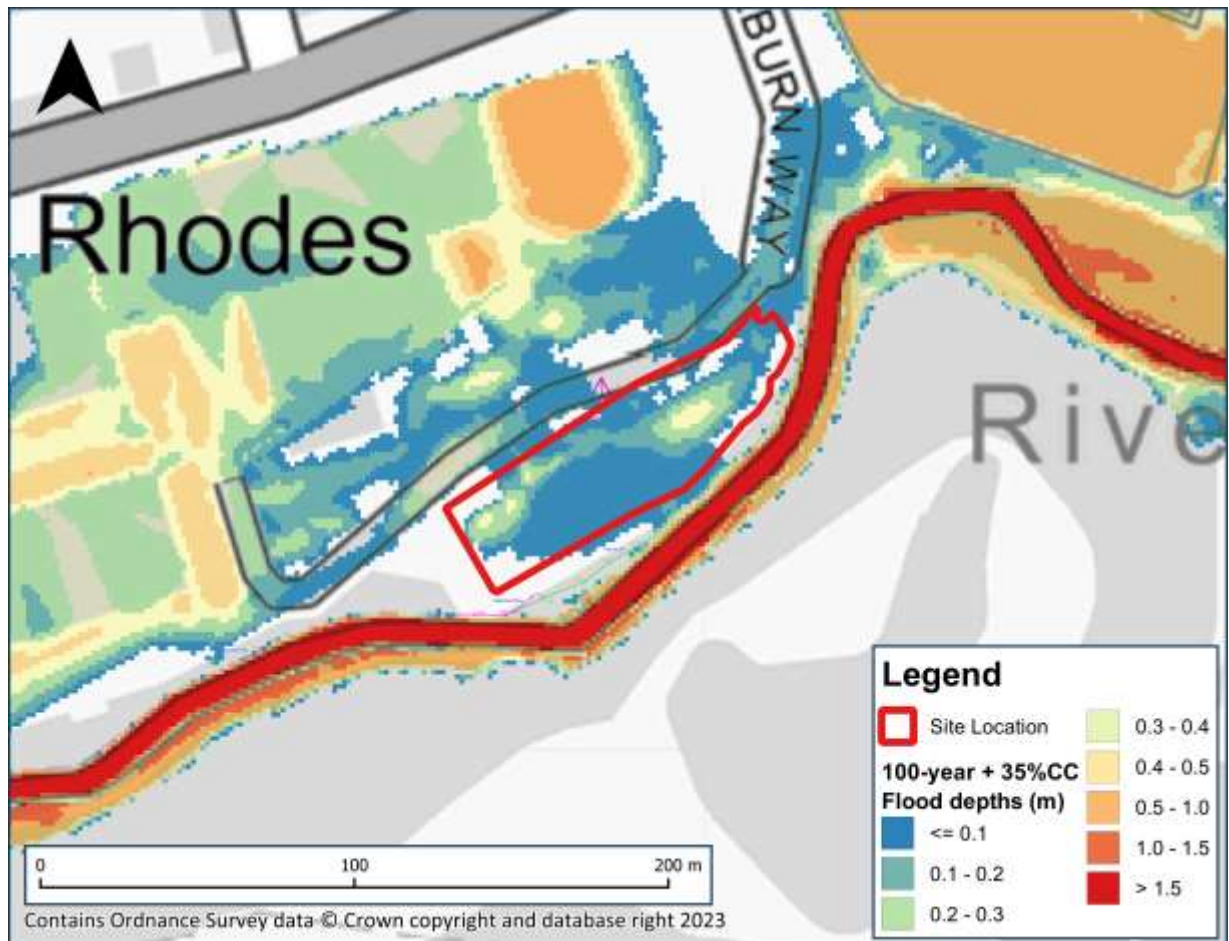


Figure 3-3. Flood depths during this event are typically less than 0.1m. However, peak modelled flood depths reach 0.37m in localised areas.

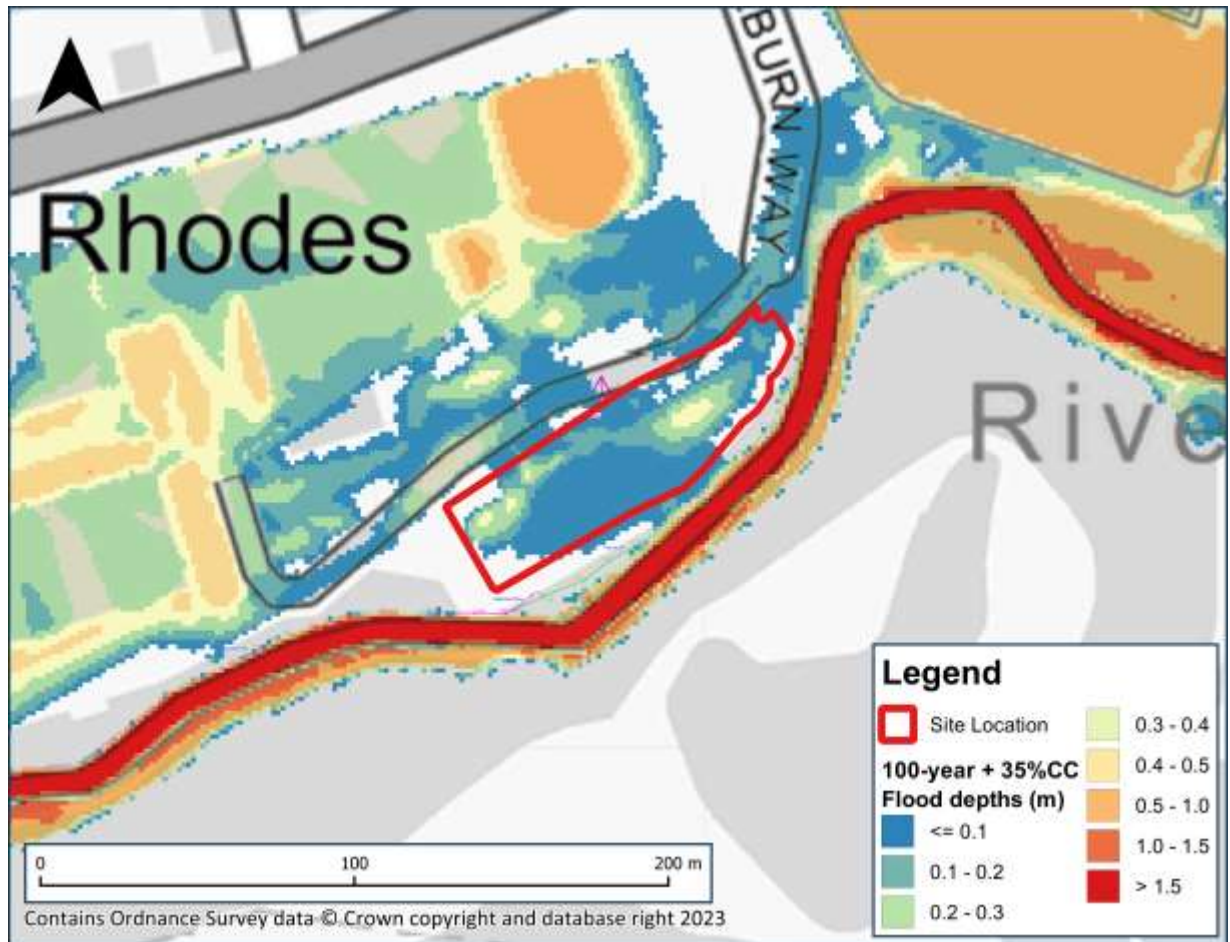


Figure 3-3: The 100-year plus 35% climate change flood depths

3.3 Peak modelled flood levels

The peak modelled flood levels during the 100-year plus climate change flood event are shown in Figure 3-4. Peak modelled flood levels of up to 70.44mAOD are predicted to occur within the site boundary during the 100-year plus 35% climate change fluvial flood event.

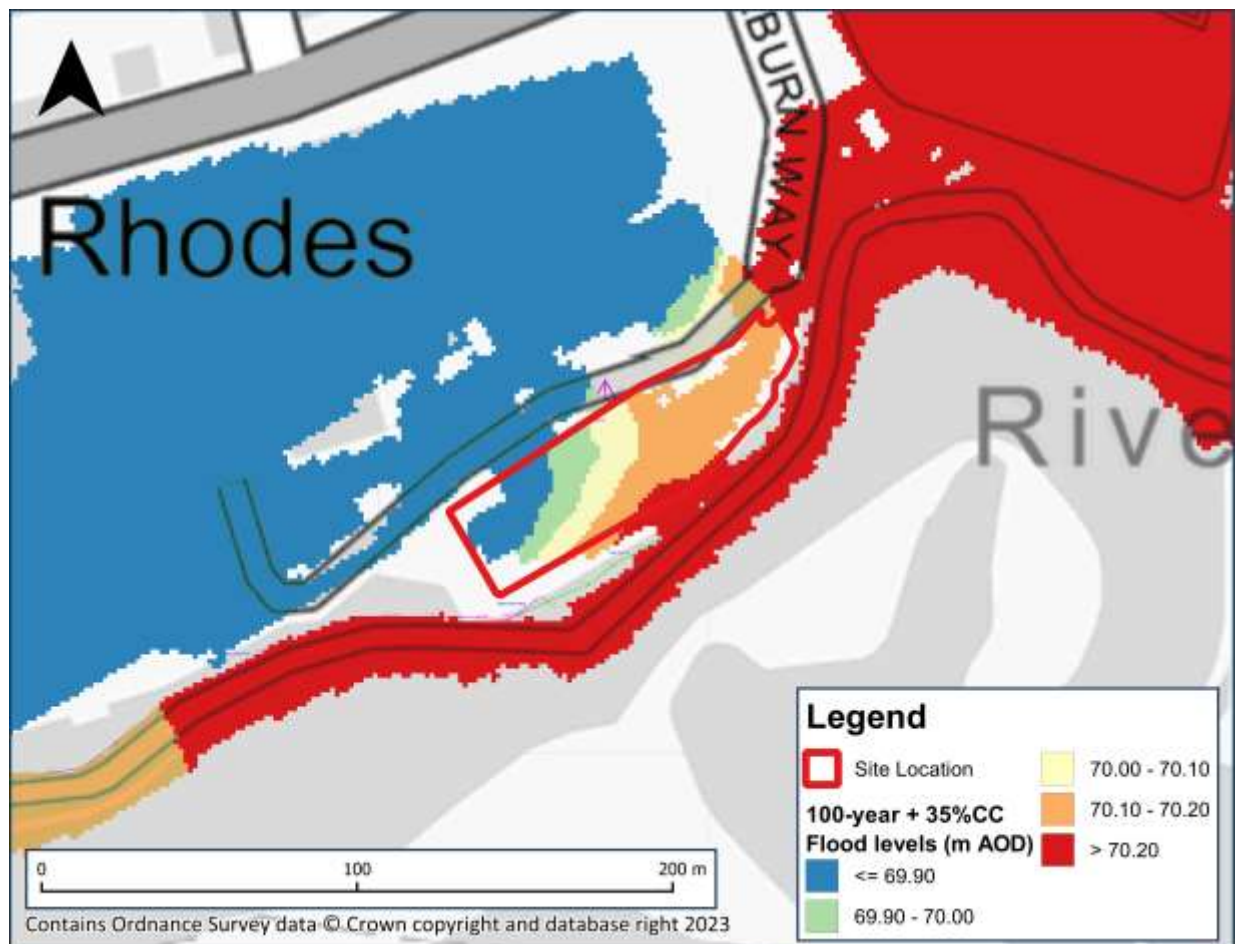


Figure 3-4: The 100-year plus 35% climate change flood levels

4 Model validation

4.1 Model Results and Flood Zone comparison

Figure 4-1 compares the Environment Agency's Flood Zone 2 and the 1000-year flood extent derived from the site specific hydraulic model. There is a good alignment between the 1000-year modelled extents and Flood Zone 2 which improves the confidence in the modelled data.

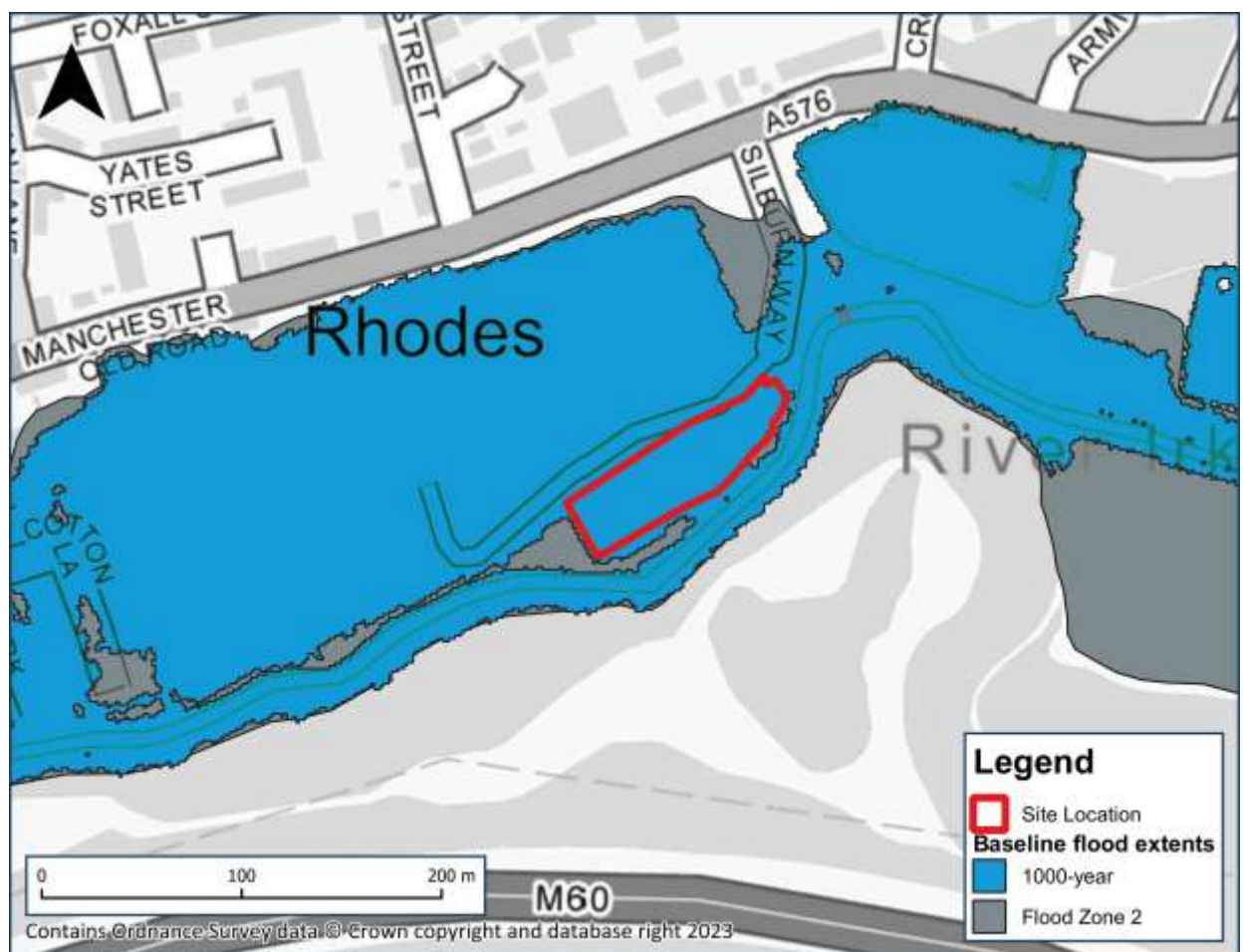


Figure 4-1: Flood Zone 2 and modelled 1000-year comparison

Figure 4-2 compares the Environment Agency's Flood Zone 3 and the 100-year flood extent derived from the site specific hydraulic model. Flood Zone 3 extends across the entire site. However, the modelled 100-year event remains in channel and does not encroach the site. This is due to the hydraulic modelling having a more refined representation of the channel and floodplain topography compared to the generalised modelling that informs the Flood Map for Planning.

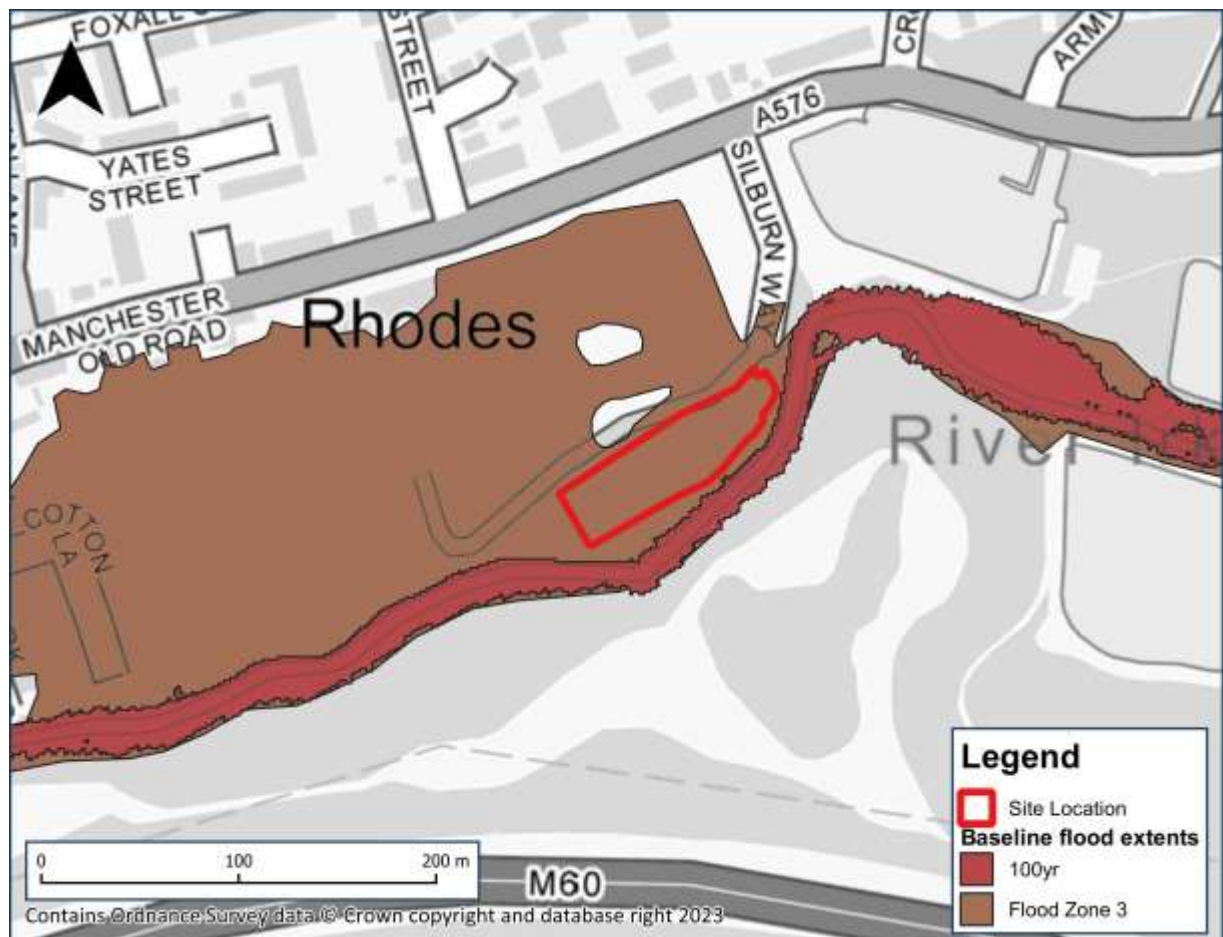


Figure 4-2: Flood Zone 3 and modelled 100-year comparison

5 Conclusion and recommendations

5.1 Conclusions

JBA Consulting was commissioned by Belshaw in August 2023 to undertake a hydraulic modelling study of the River Irk which flows along the southern boundary of the proposed development site located along Silburn Way, Middleton.

An ESTRY-TUFLOW hydraulic model was created to represent flood risk from the River Irk in the vicinity of the site. Channel survey data collected by Grantham Coates Surveys Ltd in September 2023 were used to represent the River Irk channel and structures. Site topographic surveys and LiDAR data flown in 2020 was used to represent ground levels within the floodplain.

Model results indicate that:

- The 30-year and 100-year flood extents do not encroach within the boundary of the site. During the 1,000-year flood event, peak flood extents are predicted to cover the majority of the site.
- The 100-year plus climate change (35%) flood extent flows into the site along the southern boundary. The peak modelled flood event covers the majority of the site during the 100-year plus climate change event.
- Typical peak flood depths of under 0.1m are predicted across the majority of the site during the 100-year plus climate change event. However, peak modelled flood depths reach 0.37m in localised areas within the site.
- Peak modelled flood levels of 68.9m AOD to 70.44mAOD are predicted to occur within the site during the 100-year plus 35% climate change fluvial flood event
- The model results are insensitive to changes in the downstream boundary conditions. The model results are relatively sensitive to the modelled roughness parameters and flow values. However, there are no changes in peak flood extents predicted within the site boundary as a result of the sensitivity testing.

5.2 Recommendations

It is recommended that the hydraulic model results are validated by the Environment Agency, ideally before developing a Masterplan or submitting a planning application.

As the site is predicted to flood during the 100-year plus climate change event, it is recommended that the hydraulic model is further used to assess post-developed flood risk, to ascertain whether there are any third-party flood risk impacts of the proposed development and to identify any requirements for mitigation.

6 Appendices

A Topographic Survey

B

Channel Survey



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Station	Description	Eastings	Northings	Height

Title: WATERCOURSE SURVEY AT
Silburn Way
(Key Plan)

 **Grantham Coates
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Drawing No: GCS/JBA/Silburn Way/1	Rev: -
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File Name: GCS-JBA-Silburn Way-1	Sheet size: A1	Sheet: 1 of 9
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ZERO CHAINAGE
TAKEN AS THE CONFLUENCE
WITH THE RIVER IRWELL

1	
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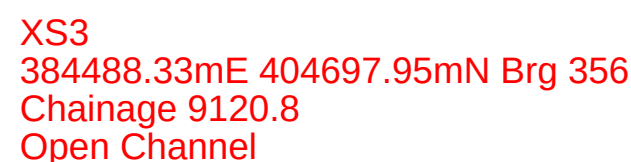
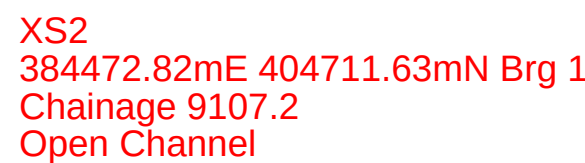
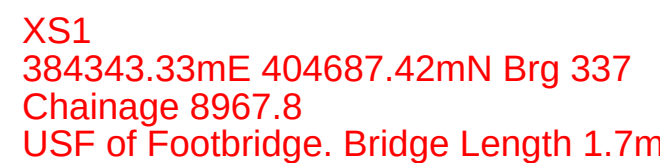
THE SURVEY
ASSOCIATION
FULL MEMBER



chartered
ICS

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File Name: GCS-JBA-Silburn Way-1			Sheet size: A1	Sheet: 2 of 9	

ZERO CHAINAGE
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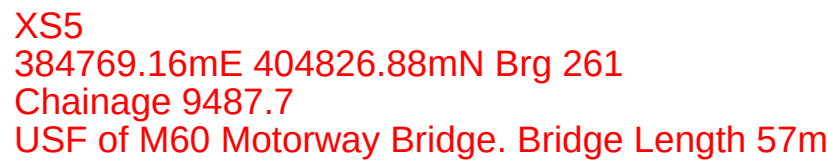
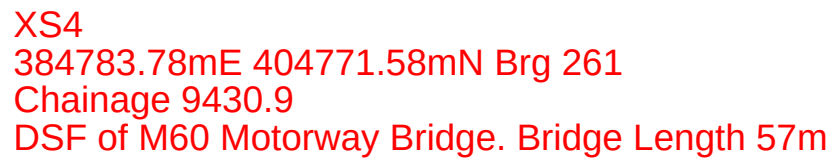
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ARK VALVE	AV	WATER TAP	TAP
BANK	BN	TELEPHONE BENCH MARK	TBM
BONE HOLE	BD	TELEPHONE CANYON	TC
BUS STOP	BS	TELEPHONE POST	TEL
CONCRETE COVERAGE COVER	BT	THRESHOLD D LEVEL	THL
8X 8X	BTB	TRAFFIC LIGHT	TL
CABLE TV INSPCTION COVER	CTK	2X UNABLE TO CL	UTCL
DRAIN	DN	2X UNABLE TO CL	UTCL
DRAIN PIPE	DNP	TELEGRAPHIC POLE	TP
INSULATED KERB	IK	VEHICLE	VP
ELECTRICITY CONTROL BOX	ECB	RAIL WATER	WM
EAVES HEIGHT	EH	RAIL WATER	WM
ELECTRICITY MANHOLE	EM	WATER METER	WE
ELECTRICITY POLE	EP	WASHOUT	WO
ER	ER		
FINISHED FLOOR LEVEL	FFL		
FIRE HYDRANT	FI		
FOUL WATER	FW		
GULLY	GU		
GR	GR		
GS VALVE	GV		
INSULATED BLOCK JOINT	IBJ		
INSPECTION COVER	IC		
INVERT LEVEL	IL		
KERB OUTLET	KO		
LAMPPOST	LP		
LITTER BIN	LB		
MAN	MA		
MAIL POST	MP		
PO	PO		
RODDING EYE	RE		
ROD HEIGHT	RH		
ROAD MANHOLE	RM		
ROAD SIGN	RS		
SE	SE		
SOFFIT LEVEL	SL		
SIGNAL POST	SP		
SIGNAL	SL		
SIGNALS AND TELECOMS BOX	STB		
SIGNALS AND TELECOMS BOX	STB		
SIGNALS AND TELECOMS BOX	STB		
SIGNALS AND TELECOMS BOX	STB		
SIGNALS AND TELECOMS BOX	STB		
SIGNAL POST	SP		
STOP TAP	ST		
STOP VALVE	SV		
STAY	ST		
STORM WATER	SW		

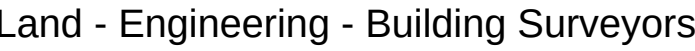
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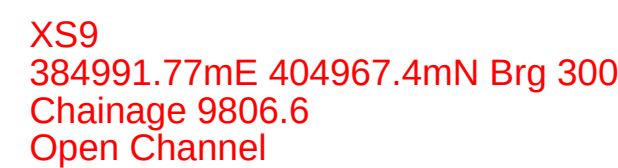
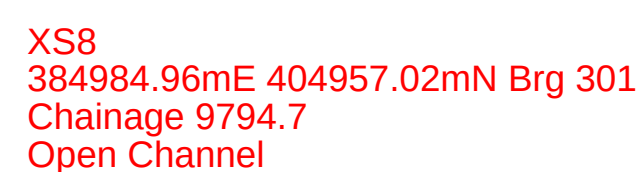
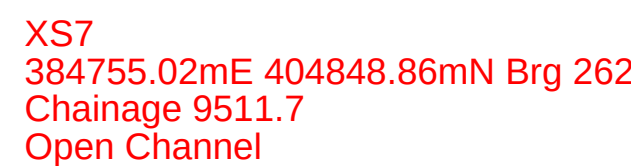


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				4 of 9	



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Legend

AIR VALVE	AV	WATER TAP	TAP
BOLLARD	BH	TEMPORARY BENCH MARK	TBM
BUS STOP	BS	TELEPHONE CAB BOX	TCL
BUS STOP	BS	TELEPHONE POST	TCL
BT INSPECTION COVER	BT	THRESHOLD LEVEL	THL
BT INSPECTION COVER	BT	TRAFFIC LIGHT	TFL
CABLE TV INSPECTION COVER	CTV	TOP OF WALL	TOS
CABLE TV INSPECTION COVER	CTV	TELEGRAPH POLE	TEL
DOWN PIPE	DP	UNABLE TO LIFT	UTL
DROPPED KERB	DK	VENT PIPE	VP
ELECTRICITY CONTROL BOX	ECB	RAIL WELD	WE
EAVES HEIGHT	EH	WATER METER	WM
ELECTRICITY MANHOLE	EM	WASHOUT	WO
ELECTRICITY POLE	EL		
ERATH ROAD	ER		
FINISHED FLOOR LEVEL	FL		
FIRE HYDRANT	FW	SURVEY STATION	△ GCS1
FIRE WATER	FW		
GULLY	GY		
GATE POST	GP	BUSHES	
GAS VALVE	GV		
INSULATED BLOCK JOINT	IBJ		
INSPECTION COVER	IC	HEDGES	
INVERT LEVEL	IL		
KERB OUTLET	KO	TREES	
LAMPPOST	LP		
LITTER BIN	LB		
MARKING POST	MP		
MILE POST	MP		
POST	PO		
RODDING EYE	RE		
RIDGE HEIGHT	RH		
ROAD DEGRADE	RD		
ROAD SIGN	RS		
ROUGH	RS		
SLOTT LEVEL	SL		
SIGNAL POST	SG		
SIGNAL SPLITTER	SP		
SIGNALS AND TELECOMS BOX	STB		
SIGNALS AND TELECOMS BOX	STC		
SIGNALS AND TELECOMS BOX	STC		
SIGNAL POST	SN		
STOP	ST		
STOP VALVE	SV		
STORM WATER	SW		

Control Table

Station	Description	Eastings	Northings	Height

Grid and Datum

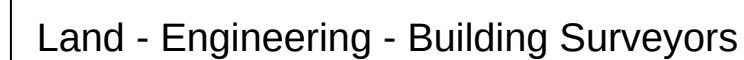
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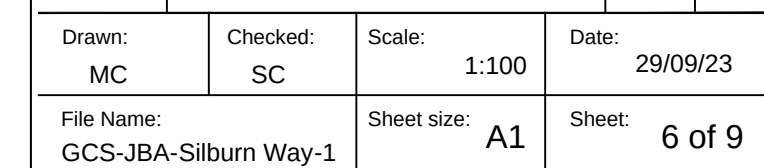
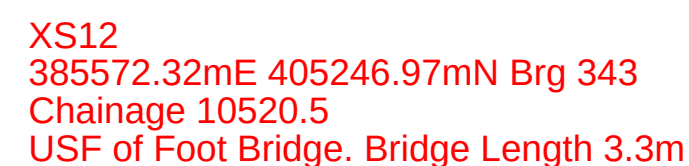
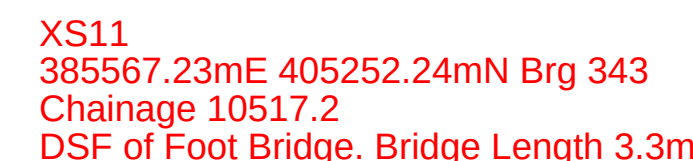
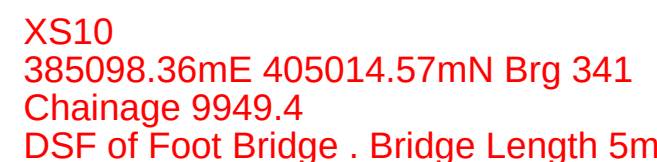
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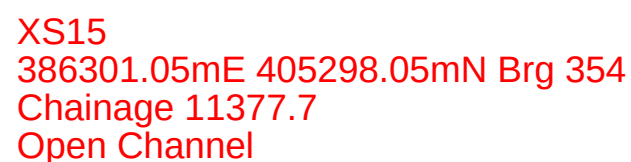
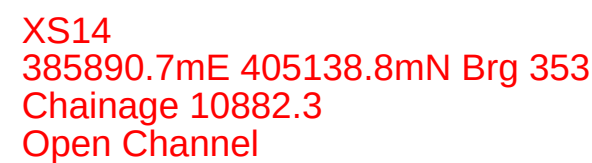


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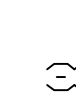
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Legend

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BOLLARD	BB	TEMPORARY BENCH MARK	TBM
BURIED	BHL	TELEPHONE CABLE BOX	TCP
BUS STOP	BT	TELEPHONE POLE	TP
BT INSPECTION COVER	BTB	THRESHOLD LEVEL	TL
BT BOX	BTB	THREE LIGHT	TL
CABLE TV INSPECTION COVER	CT	TOP-OF-CEILING	TC
COVER LEVEL	CL	UNLEIGH POLE	UP
DOWNPIPE	DP	UNABLE TO LIFT	UTL
DRAINAGE KERB	DK	VENT PIPE	VP
ELECTRICITY CONTROL BOX	ECB	WALL WELD	WE
EAVES HEIGHT	EH	WASHOUT	WO
ELECTRICITY MANHOLE	EM		
ELECTRICITY POLE	EP		
EARTH ROD	ER		
FINISHED FLOOR LEVEL	FFL		
FRESH IRRIGANT	FI		
FULL WATER	FW		
GULLY	GY		
GATE POST	GP		
GAS VALVE	GV		
INSULATED BLOCK JOINT	IBJ		
INSPECTION COVER	IC		
INVERT LEVEL	IL		
KERB OUTLET	KO		
LAMPPOST	LP		
LITTER BIN	LB		
MARKER POST	MP		
MALE POST	MK		
POST	PD		
RODDING EYE	RE		
RIDGE HEIGHT	RH		
ROUND NAMEPLATE	RN		
ROAD SIGN	RS		
STUMP	ST		
SOFIT LEVEL	SF		
SIGNAL POST	SIG		
RAIN/SLOPER LEVEL	SL		
SIGNALS AND TELECOMS BOX	STB		
SIGNALS AND TELECOMS CABLE	SC		
SIGNALS AND TELECOMS POST	STP		
SIGN	SN		
SOFT TAP	ST		
STORM WASTE	SW		
STAY	SV		
STORM WATER	SW		

SURVEY STATION



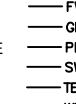
BUSHES



IC HEDGES



TREES



GATE



SECURITY FENCE



OVERHEAD CABLES



FULL WATER



GAS MAIN



UGC POWER LINE



UGC WATER MAIN



TEL



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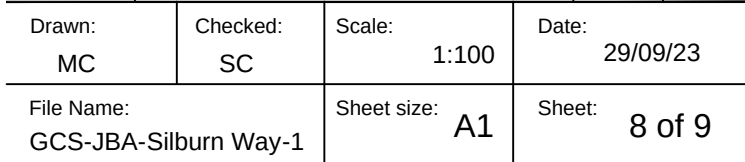
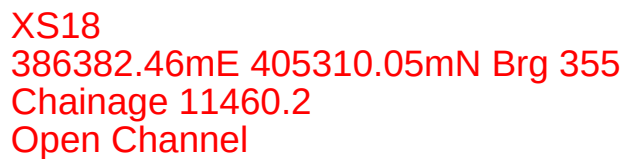
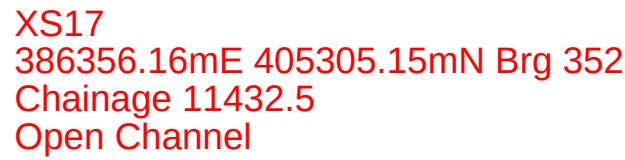
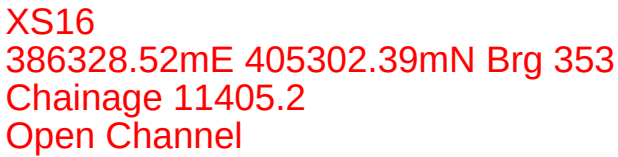
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Legend

[illegible]

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				Sheet: 9 of 9	

C

Hydraulic Structures

Included in model? (state reason if not):	Yes																								
Model label:	IRK_11564us_BB																								
Type:	Bridge																								
How has structure been modelled?	The structure was modelled as a bridge unit using ESTRY. The following modelling parameters were used: <table border="1"> <thead> <tr> <th colspan="2">IRK_11564us_BB</th></tr> <tr> <th>H</th><th>W</th></tr> </thead> <tbody> <tr><td>71.256</td><td>0</td></tr> <tr><td>71.34</td><td>1.83</td></tr> <tr><td>71.39</td><td>2.35</td></tr> <tr><td>71.63</td><td>3.36</td></tr> <tr><td>71.79</td><td>4.99</td></tr> <tr><td>71.97</td><td>5.47</td></tr> <tr><td>71.98</td><td>5.54</td></tr> <tr><td>72.76</td><td>5.1</td></tr> <tr><td>73.57</td><td>4</td></tr> <tr><td>74.43</td><td>0</td></tr> </tbody> </table>	IRK_11564us_BB		H	W	71.256	0	71.34	1.83	71.39	2.35	71.63	3.36	71.79	4.99	71.97	5.47	71.98	5.54	72.76	5.1	73.57	4	74.43	0
IRK_11564us_BB																									
H	W																								
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73.57	4																								
74.43	0																								
Downstream Face, looking Upstream CS: Picture not available	Upstream Face, looking Downstream CS: 																								

Included in model? (state reason if not):	Yes																																								
Model label:	IRK_11487_WW																																								
Type:	Weir																																								
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Model label:	IRK_11460_WW																																												
Type:	Weir																																												
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Downstream Face, looking Upstream CS:	Upstream Face, looking Downstream CS:  																												

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Model label:	IRK_11378_WW																																				
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How has structure been modelled?	The structure was modelled as a weir unit using ESTRY. The following modelling parameters were used: <table> <tr> <th colspan="2">IRK_11378_WW</th></tr> <tr> <th>X</th><th>Z</th></tr> <tr><td>8.567</td><td>72.977</td></tr> <tr><td>9.33</td><td>72.92</td></tr> <tr><td>11.29</td><td>71.97</td></tr> <tr><td>11.53</td><td>71.7</td></tr> <tr><td>12.63</td><td>71.68</td></tr> <tr><td>12.66</td><td>71.6</td></tr> <tr><td>12.68</td><td>71.51</td></tr> <tr><td>12.73</td><td>71.43</td></tr> <tr><td>19.7</td><td>71.43</td></tr> <tr><td>19.75</td><td>71.6</td></tr> <tr><td>19.77</td><td>71.68</td></tr> <tr><td>20.24</td><td>71.66</td></tr> <tr><td>20.76</td><td>72.14</td></tr> <tr><td>21.37</td><td>72.25</td></tr> <tr><td>21.85</td><td>72.88</td></tr> <tr><td>22.52</td><td>73.07</td></tr> </table>	IRK_11378_WW		X	Z	8.567	72.977	9.33	72.92	11.29	71.97	11.53	71.7	12.63	71.68	12.66	71.6	12.68	71.51	12.73	71.43	19.7	71.43	19.75	71.6	19.77	71.68	20.24	71.66	20.76	72.14	21.37	72.25	21.85	72.88	22.52	73.07
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Downstream Face, looking Upstream CS:	Upstream Face, looking Downstream CS:  																																				

Included in model? (state reason if not):	Yes																												
Model label:	IRK_10882_WW																												
Type:	Weir																												
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Included in model? (state reason if not):	Yes																														
Model label:	IRK_10521_BB																														
Type:	Bridge																														
How has structure been modelled?	The structure was modelled as a bridge unit using ESTRY. The following modelling parameters were used: <table border="1"> <thead> <tr> <th colspan="2">IRK_10521_BB</th> </tr> <tr> <th>H</th><th>W</th></tr> </thead> <tbody> <tr><td>66.54</td><td>0</td></tr> <tr><td>66.55</td><td>1.37</td></tr> <tr><td>66.8</td><td>3.28</td></tr> <tr><td>66.87</td><td>3.53</td></tr> <tr><td>67.05</td><td>4.67</td></tr> <tr><td>67.25</td><td>5.05</td></tr> <tr><td>67.26</td><td>5.63</td></tr> <tr><td>68.07</td><td>5.65</td></tr> <tr><td>68.99</td><td>5.69</td></tr> <tr><td>69.22</td><td>5.28</td></tr> <tr><td>69.37</td><td>4.3</td></tr> <tr><td>69.5</td><td>3.2</td></tr> <tr><td>69.64</td><td>0</td></tr> </tbody> </table>	IRK_10521_BB		H	W	66.54	0	66.55	1.37	66.8	3.28	66.87	3.53	67.05	4.67	67.25	5.05	67.26	5.63	68.07	5.65	68.99	5.69	69.22	5.28	69.37	4.3	69.5	3.2	69.64	0
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Included in model? (state reason if not):	Yes																										
Model label:	IRK_9499us_BB																										
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How has structure been modelled?	The structure was modelled as a bridge unit using ESTRY. The following modelling parameters were used: <table border="1"> <thead> <tr> <th colspan="2">IRK_9949us_BB</th> </tr> <tr> <th>X</th><th>Z</th></tr> </thead> <tbody> <tr><td>16.59</td><td>68.59</td></tr> <tr><td>16.653</td><td>66.469</td></tr> <tr><td>17.068</td><td>66.486</td></tr> <tr><td>18.166</td><td>66.412</td></tr> <tr><td>19.733</td><td>66.259</td></tr> <tr><td>21.102</td><td>66.19</td></tr> <tr><td>22.847</td><td>66.234</td></tr> <tr><td>23.694</td><td>66.365</td></tr> <tr><td>23.74</td><td>66.93</td></tr> <tr><td>23.871</td><td>68.588</td></tr> <tr><td>24.141</td><td>68.609</td></tr> </tbody> </table>	IRK_9949us_BB		X	Z	16.59	68.59	16.653	66.469	17.068	66.486	18.166	66.412	19.733	66.259	21.102	66.19	22.847	66.234	23.694	66.365	23.74	66.93	23.871	68.588	24.141	68.609
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Included in model? (state reason if not):	Yes																																		
Model label:	IRK_9807_WW																																		
Type:	Weir																																		
How has structure been modelled?	The structure was modelled as a weir unit using ESTRY. The following modelling parameters were used: <table border="1"> <thead> <tr> <th colspan="2">IRK_9807_WW</th> </tr> <tr> <th>X</th><th>Z</th></tr> </thead> <tbody> <tr><td>12.597</td><td>67.619</td></tr> <tr><td>12.821</td><td>67.54</td></tr> <tr><td>13.19</td><td>67.51</td></tr> <tr><td>13.259</td><td>66.688</td></tr> <tr><td>13.278</td><td>66.51</td></tr> <tr><td>13.343</td><td>66.51</td></tr> <tr><td>16.08</td><td>66.51</td></tr> <tr><td>17.359</td><td>66.525</td></tr> <tr><td>20.169</td><td>66.54</td></tr> <tr><td>22.484</td><td>66.55</td></tr> <tr><td>24.078</td><td>66.55</td></tr> <tr><td>24.14</td><td>66.55</td></tr> <tr><td>24.169</td><td>66.688</td></tr> <tr><td>24.338</td><td>67.467</td></tr> <tr><td>24.82</td><td>67.639</td></tr> </tbody> </table>	IRK_9807_WW		X	Z	12.597	67.619	12.821	67.54	13.19	67.51	13.259	66.688	13.278	66.51	13.343	66.51	16.08	66.51	17.359	66.525	20.169	66.54	22.484	66.55	24.078	66.55	24.14	66.55	24.169	66.688	24.338	67.467	24.82	67.639
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Hydrology Report

Flood estimation report: Silburn Way, Middleton

Introduction

This report template is based on a supporting document to the Environment Agency's flood estimation guidelines. It provides a record of the hydrological context, the method statement, the calculations and decisions made during flood estimation and the results.

Contents

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4	Revitalised flood hydrograph 2 (ReFH2) method	16
5	Discussion and summary of results	19
6	Annex A – Pooling Groups	24
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Approval

	Name and qualifications	Date
Method statement prepared by:	Sam Cogan BSc MSc	30/08/2023
Method statement reviewed by:	Eva Kordomenidi BSc MSc MCIWEM CWEM CSci	26/10/2023
Calculations prepared by:	Sam Cogan BSc MSc	31/08/2023
Calculations reviewed by:	Eva Kordomenidi BSc MSc MCIWEM CWEM CSci	26/10/2023

Revision History

Revision reference	Date issued	Amendments	Issued to

Abbreviations

AMAX.....	Annual Maximum
AREA	Catchment area (km ²)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
FARL.....	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
HOST.....	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR.....	Standard Average Annual Rainfall (mm)
Tp(0).....	Time to peak of the instantaneous unit hydrograph
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH.....	Windows Frequency Analysis Package – used for FEH statistical method

Note on flood probability

This document quotes the probability of a flood magnitude in terms of a return period based on analysis of annual maximum (AMAX) floods. The return period of a flood on the AMAX scale is the average interval between AMAX floods of that magnitude or greater. The inverse of the AMAX return period is the annual exceedance probability (AEP).

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

Return period can also be measured on the peaks-over-threshold (POT) scale as the average interval between floods of that magnitude or greater. The difference between AMAX and POT return periods is only important for short return periods (under 10 years).

The table below is provided to enable quick conversion between these different measures.

AMAX return period (years)	n/a	2	5	10	20	30	50	75	100	200	1,000
AEP (%)	n/a	50	20	10	5	3.33	2	1.33	1	0.5	0.1
POT return period (years)	1	1.5	4.5	9.5	20	30	50	75	100	200	1,000

1 Method statement

1.1 Requirements for flood estimates

Overview

- Purpose of study
- Peak flows or hydrographs?
- Range of return periods and locations

The proposed development site is located adjacent to Silburn Way, Middleton (NGR: SD 85346 05209). Proposals for the site are for the redevelopment of an existing commercial site. The southern site boundary is located adjacent to the River Irk. This is a 'Main River' designated watercourse, falling within the remit of the Environment Agency. According to the Environment Agency's Flood Map for Planning, approximately 90% and 100% of the site is located within Flood Zones 3 and 2, respectively (Figure 1-1).

A detailed review of the Flood Map for Planning maps indicated a number of discrepancies with the topography of the valley. This indicates that the existing EA flood zone extents either originate from an older study and/or originate from broadscale modelling techniques

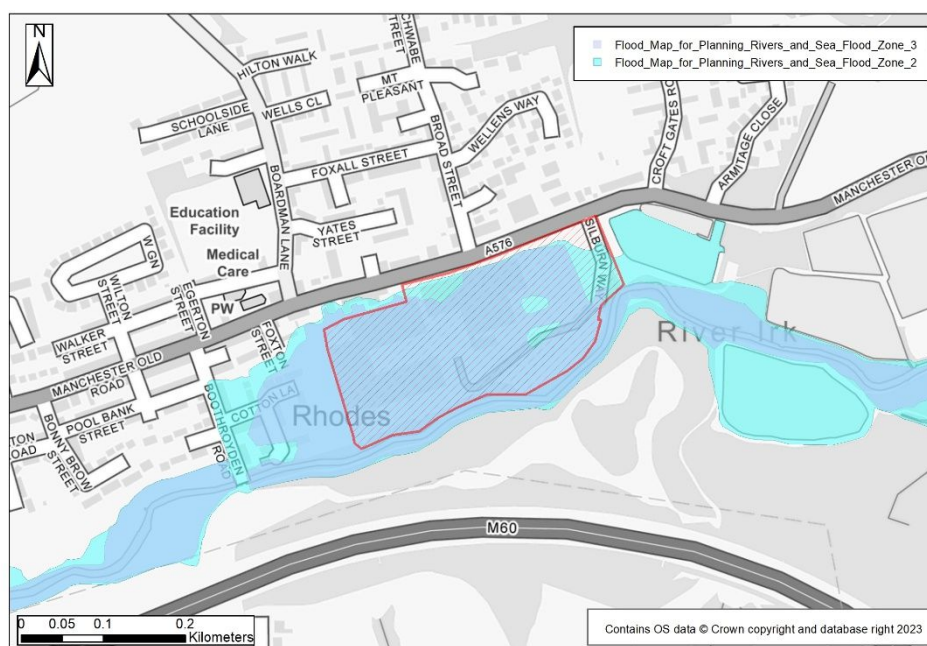


Figure 1.1 - Environment Agency's Flood Map for Planning

JBA Consulting was commissioned by Civils to assess the extent of Flood Zones 2 and 3, using up-to-date fluvial modelling results. Detailed modelling is required to accurately assess the risk of flooding from the River Irk, and evaluate development options for the site. As part of the scope of this project, hydrology estimates are therefore required for input to the hydraulic model of the River Irk. This document details the hydrological assessment undertaken to derive the hydraulic model inflows.

Peak flow estimates and hydrographs are required for three flow estimation points (FEP) for both the River Irk and Boardman Brook. Peak flow estimates and hydrographs have been derived for the following annual exceedance probabilities (AEP) events:

- 50%;
- 3.33%;

- 1%; and
- 0.1%.

Based on July 2021 Environment Agency climate change allowance guidance¹, the proposed development site is located within the Irwell Management Catchment. For the '2080' scenario, the Central allowance climate change uplift is 35%. This has been applied to the 1% AEP event peak flows. Peak flows for the Higher allowance (46% uplift) have also been calculated. Peak flow estimates and hydrographs derived from the hydrological analysis will be incorporated into a 1D-2D hydraulic model capable of simulating flood extents, levels and flows through time in relation to the development site.

1.2 The catchment

Map (Include river network, catchment boundary and gauging stations)

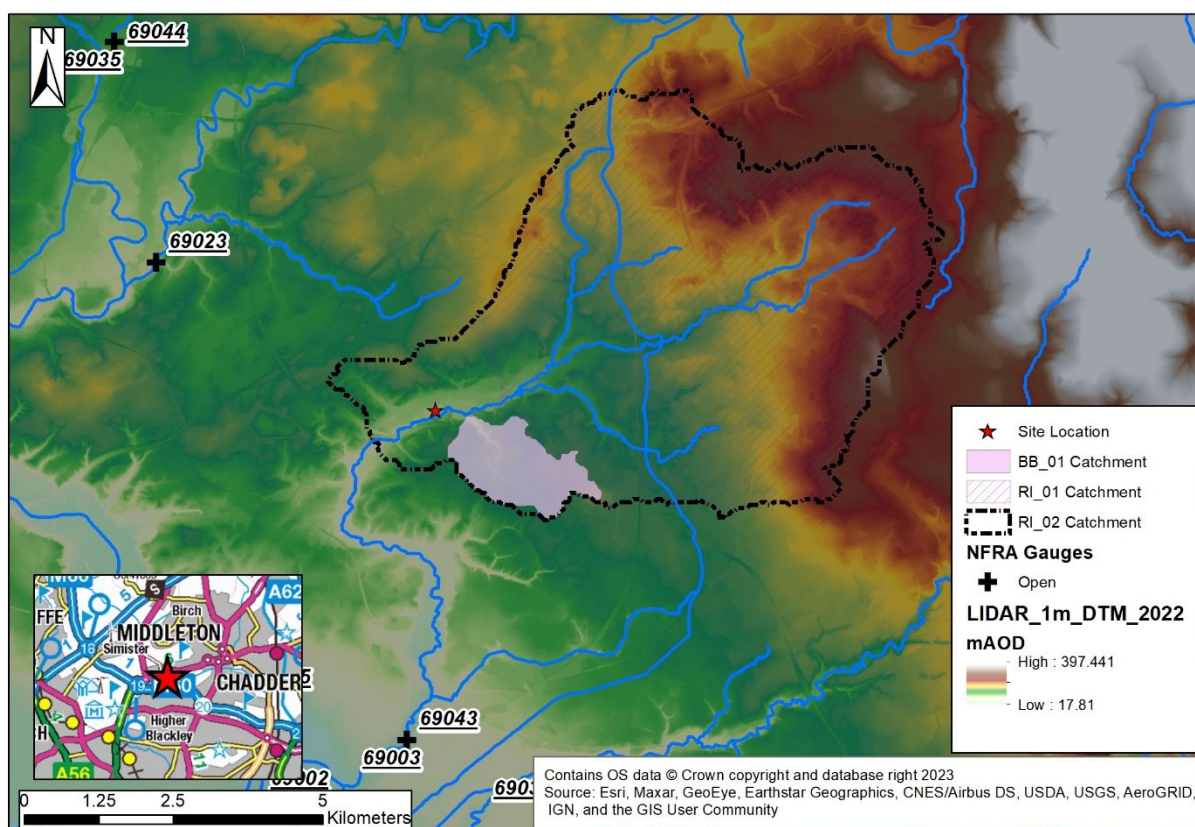


Figure 1.2 – Catchment extent

Description

Include topography, climate, geology, soils, land use and any unusual features that may affect the flood hydrology.

The River Irk, including contributing watercourses, drains an area of approximately 44.9km² to the downstream hydraulic model boundary and includes the contributing catchment of Broadman Brook. The general elevation trend of the larger study catchment is a decline from east to west as land falls from 215mAO to 60mAO.

Underlying bedrock geology of the wider River Irk catchment predominately consists of Pennine lower coal measures formation and

¹ Environment Agency (2023). Guidance: Flood risk assessments: climate change allowances. Accessed 30/08/2023. [Source: <https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances#Select-the-peak-river-flow-allowances-to-use-for-your-assessment>].

south wales lower coal measures formation (undifferentiated) - mudstone, siltstone, sandstone, coal, ironstone and ferricrete with bands of Triassic and Permian rocks (undifferentiated) to the very lower reaches of the catchment. Superficial deposits of the catchment consist of glacial sand and gravel and till.

Typical soils across the study catchment are variable however the catchment is predominately covered by freely draining slightly acid sandy soils with Slowly permeable seasonally wet acid loamy and clayey soils to the northern and southern edges of the catchment, which includes the catchment extent of Boardman Brook².

The underlying bedrock, superficial deposits and soil types would indicate that the fluvial response to rainfall based on soil properties would likely be slow due to the underlying freely draining soils. Generally, the catchment of the River Irk may be considered to be relatively permeable however the catchment of Boardman Brook may be considered to be relatively impermeable.

The catchment is located within the North West England's regional climate. The exposure of NW England to westerly maritime air masses and the presence of extensive areas of high ground mean that the region has some of the wettest places in the UK. The higher parts of the Lake District are particularly wet, with an average of over 3200mm of rain each year. In contrast, the reputedly wet city of Manchester averages only 830mm and the more sheltered areas of Cheshire and the Eden valley in Cumbria are even drier with less than 800mm per year. These areas benefit from the 'rain shadow' effect of the high ground of N Wales and the Lake District respectively³.

Based on the Environment Agency Asset Management website⁴, there are no structural flood defences along the bank of the River Irk, only high ground banks. The Asset ID (65334) has an actual condition of 'adequate' with an upstream crest level of 68.74m and a downstream crest level of 70.21m. The asset maintainer of the embankment is a Private individual, Company or Charity.

The wider catchment is generally comprised of grassland, agricultural land use and urban development associated with the outskirts of Manchester. The urban extent within the catchment may be a limitation to the methodologies applied within this study. In addition, there are a number of significant lakes / reservoirs within the study catchment, specifically within close proximity to the site, however these reservoirs will be included within the hydraulic model so they should not have any impact on the final methodology used within this study.

1.3 Source of flood peak data

Source	National River Flow Archive (NRFA) peak flows dataset, Version 11.1, released March 2023. This contains data up to water year 2020-21.
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² British Geological Survey (2021). UK Soil Observatory Map Viewer. Accessed: 30/08/2023. [Source: mapapps2.bgs.ac.uk/ukso/home.html].

³ https://www.metoffice.gov.uk/binaries/content/assets/metofficegovuk/pdf/weather/learn-about/uk-past-events/regional-climates/north-west-england--isle-of-man_-climate---met-office.pdf

⁴ Environment Agency (2002) Asset Information and Maintenance Programme (data.gov.uk)

1.4 Gauging stations (flow or level)

(at or very near to the sites of flood estimates)

River Irk Gauges are all located downstream of the proposed model extent

Water-course	Station name	Gauging authority number	NRFA number	Catchment area (km ²)	Type (rated / ultrasonic / level...)	Start of record and end if station closed
River Irk	Irk @ Collyhurst Weir	-	69043	72.3	Broad-crested weir	01/1995 to present day
River Irk	Irk @ Scotland Weir	-	69003	72.5	Compound broad-crested weir	01/1949 to 01/2001

1.5 Data available at each flow gauging station in Table 1.4

Station name	Start and end of NRFA flood peak record	Update for this study?	OK for QMED?	OK for pooling?	Data quality check needed?	Other comments on station and flow data quality
Irk @ Collyhurst Weir	01/1995 to present day	N	Y	Y	N	NGR: SJ848996 Curved non-standard round nosed broad-crested weir to replace Scotland Weir (69003). Gauges a catchment with moorland headwaters but urbanised lower catchment. Water year 1994/1995 rejected as the AMAX is from the partial water year at the start of the record. Full period of record peak flow data reviewed and released in August 2021 (WINFAP Files v10).
Irk @ Scotland Weir	01/1949 to 01/2001	N	Y	Y	N	NGR: SJ841991 Non-standard weir affected by siltation, weed growth and debris. Due to siltation, weed growth and debris there is particular doubt on low-flow records before 1976, although data quality in general is not good. Weir damaged by a flood in December 1983. Station closed between February 1991 and October 1995.
Tabulate any updated or revised flood peak series in the Annex.						

Give link/reference to any further data quality checks carried out.

1.6 Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Comments and link to any rating reviews
Irk @ Collyhurst Weir	Two peak flow ratings applied across period of record developed from current meter gaugings since 04/10/1995, with modelled top-end. Algal growth up to 5cm deep can occur on weir crest.	N	No rating updates are currently requested by the scope.
Irk @ Scotland Weir	Original rating based on physical model tests in 1936 and check gaugings at Redbank 1km upstream. Site was re-levelled in 1976 and new rating applied based on gaugings. Third rating, based on gaugings, applied after flood damage to weir crest in 1983. Flood Studies Report grade A1.	N	No rating updates are currently requested by the scope.

1.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings (if planned to review ratings)	N/A	N/A	-	No rating updates are currently requested by the scope.
Historic flood data Include chronology and interpretation of flood history in Annex or separate report.	Yes	Yes	Environment Agency WMS	No historic incidences of flooding recorded at the site or along the proposed model extent.
			BHS Chronology of British Hydrological Events	Based on historical records, approximately 5 flood events have occurred along the River Irk between 1662-1927 however none of these events make mention of Silburn Way but the 1927 event makes mention of Middleton.

			Faber Maunsell (2007) Irwell Areas Benefiting from Defences Study – Phases 2&3	A brief historic flood history for the River Irk was undertaken as part of the Faber Maunsell (2007) Irwell Areas Benefiting from Defences Study – Phases 2&3 study.
Flow or river level data for events	Yes	No	-	Ungauged watercourse
Rainfall data for events	No	No	-	Oldham raingauge is located within the study catchment. No flood event analysis is included within the scope
Potential evaporation data	No	No	-	No flood event analysis in scope.
Results from previous studies			Faber Maunsell (2007) Irwell Areas Benefiting from Defences Study – Phases 2&3	The South Area Northwest region of the Environment Agency appointed Faber Maunsell to undertake an 'Areas Benefiting from Defences' study for the Irwell catchment. The hydrology associated with the model made use of a 4.25hr storm duration for the upper Irk catchment based on a summer storm event profile. Time-to-peak was calculated based on the FEH equation. The final methodology used was FEH rainfall-runoff. A number of FEP's and their corresponding modelled levels and flows, which were centred around the site, was provided within the EA data request as well as 1% and 0.1% AEP level and flow results for a number of ISIS flow nodes along the River Irk.
Other data or information (e.g. groundwater, tides, channel widths, low flow statistics)	-	-	-	-

1.8 Hydrological understanding of catchment

Outline the conceptual model, addressing questions such as: - Where are the main sites of interest? - What is likely to cause flooding at those	The proposed development site is located adjacent to Silburn Way, Middleton (NGR: SD 85346 05209). The southern site boundary is located
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locations? (peak flows, flood volumes, backwater effects, combinations of peaks, groundwater, snowmelt, tides...) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams or NFM features that could collapse?	adjacent to the River Irk. According to the Environment Agency's Flood Map for Planning, approximately 90% and 100% of the site is located within Flood Zones 3 and 2, respectively. The primary flood mechanism is likely to be channel capacity exceedance during times of sustained or heavy rainfall.
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – seek local flow data; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing, extensive floodplain storage – consider choice of method carefully	BFIHOST for the River Irk inflow catchment is around 0.65 due to its underlying freely draining soils and mudstone, siltstone, sandstone, coal, ironstone and ferricrete bedrock. Therefore, the catchment is considered highly permeable which may have an impact on derived flows. In addition, there is considered to be a moderate-high reservoir influence to the lower reaches of the catchment. However the reservoirs will be included within the hydraulic model, therefore the presence of reservoirs is not considered to be a major influence on the hydrological study. In addition, some of the study catchments are considered to be relatively urbanised.

1.9 Initial choice of approach

Is FEH appropriate? (it may not be for extremely heavily urbanised or complex catchments) If not, describe other methods to be used.	Yes. Catchment is somewhat permeable.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	<u>Peak flows</u> Peak flow estimates for inflow for the River Irk and Boardman Brook will be derived from application of FEH methods. Both FEH Statistical and ReH2 will be tested. <u>Hydrographs</u> Hydrographs for the model will be derived using the ReFH2 model. ReFH2 hydrographs will be scaled to the favoured peak flow estimates explored within this study.
Software to be used (with version numbers)	FEH Web Service⁵ / WINFAP v5⁶ / ReFH2.3 calibrated to FEH22

⁵ CEH 2015. The Flood Estimation Handbook (FEH) Online Service, Centre for Ecology & Hydrology, Wallingford, Oxon, UK.

⁶ WINFAP-FEH v4 © Wallingford HydroSolutions Limited 2016.

2 Locations where flood estimates required

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

2.1 Summary of subject sites

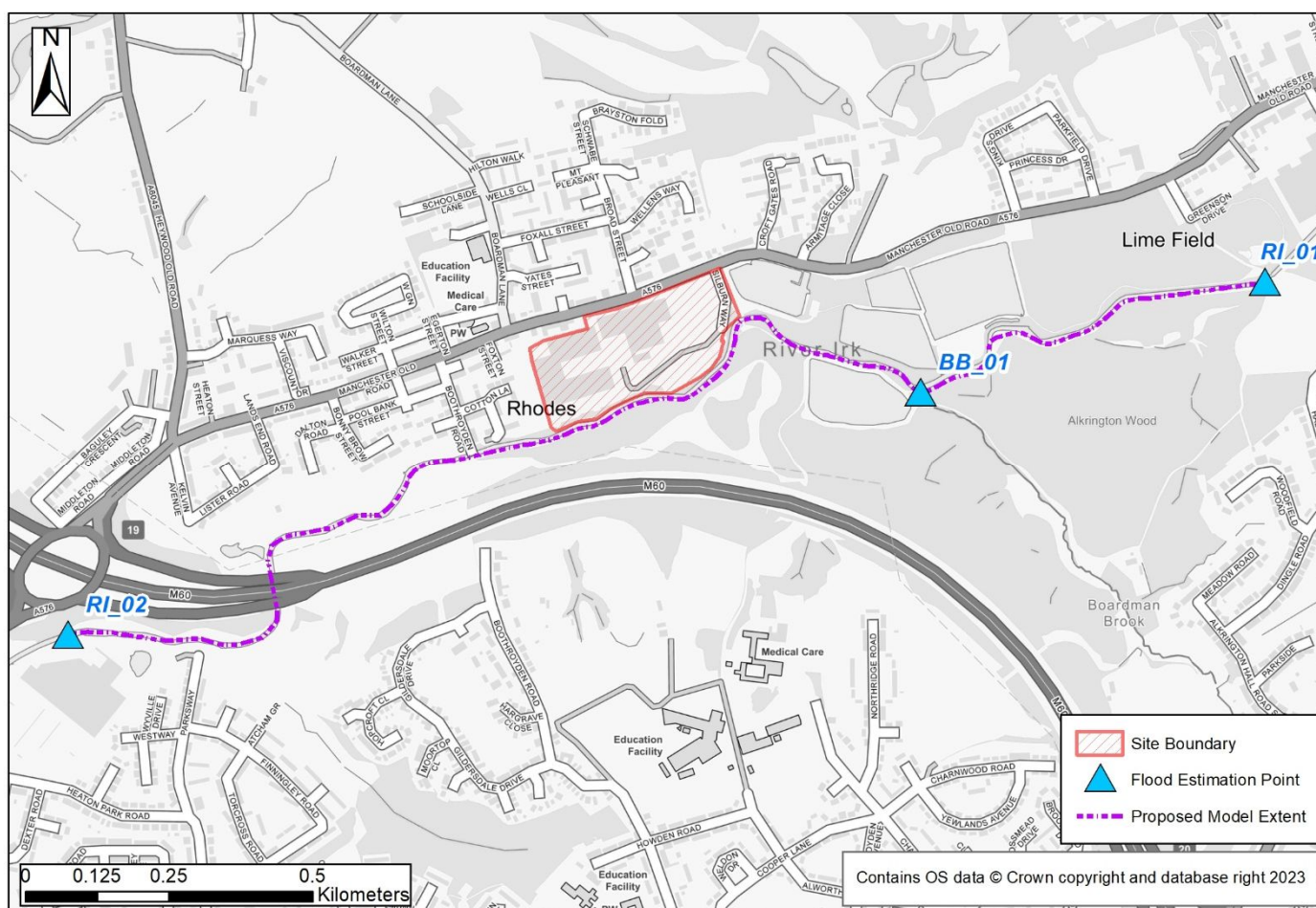


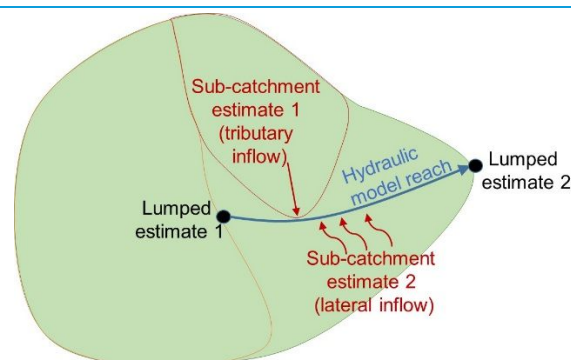
Figure 2.1 – Flood Estimation Point Location

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km ²)	Revised AREA if altered
RI_01	L	River Irk	River Irk - Model inflow location	386473	405337	38.1	38.3
BB_01	L	Boardman Brook	Boardman Brook - Model inflow location	385874	405144	2.7	2.5
RI_02	L	River Irk	River Irk - Model downstream boundary	384390.5	404721.5	44.9	-

Note: Lumped catchments (L) are complete catchments draining to points at which design flows are required.

Sub-catchments (S) are catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system. There is no need to report any design flows for sub-catchments, as they are not relevant: the relevant result is the hydrograph that the sub-catchment is expected to contribute to a design flood event at a point further downstream in the river system. This will be recorded within the hydraulic model output files. However, catchment descriptors and ReFH model parameters should be recorded for sub-catchments so that the results can be reproduced.

The schematic diagram illustrates the distinction between lumped and sub-catchment estimates.



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

Site code	FARL	PROPWET	BFIHOST 19	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000 ⁷	FPEXT
RI_01	0.977	0.57	0.683	7.37	49.9	1072	0.401	0.093
BB_01	0.994	0.57	0.333	1.94	27.4	999	0.377	0.140
RI_02	1.00	0.57	0.644	7.31	50.1	1062	0.380	0.096

2.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (add maps if needed)	The catchment boundaries have been reviewed against Ordnance Survey mapping and 50m contours. 1m LiDAR data and 50m terrain LiDAR data were consulted. Modifications to the natural catchment boundaries were considered necessary to all of the study catchments of the exported catchments to reflect natural topographic elevations.
Record how other catchment descriptors were checked and describe any changes. Include before/after table if necessary.	A qualitative sense check was carried out for BFIHOST and FARL. For RI_02, as the reservoirs within the catchment are to be included within the model, FARL was set to 1.00 to ensure there was no double counting of losses due to reservoir influence. For RI_01 DPLBAR value was modified using the standard modification equation ($\text{Area}^{0.548}$). For BB_01, due to the size of the study catchments the DPLBAR value was modified on an area ratio basis as the standard modification equation is known to produce suspect values for smaller catchments. Catchment descriptors checked against Cranfield University (2021). Cranfield Soil and Agrifood Institute: Soilscales and British Geological Society (2021). Geology of Britain viewer. Underlying bedrock geology of the catchment is generally composed of mudstone, siltstone, sandstone, coal, ironstone and ferricrete. Typical soils across the study catchment are predominately freely draining slightly acid sandy soils⁸. The variable BFIHOST values are considered representative of the variable nature of the catchment.
Version of URBEXT	URBEXT1990 / URBEXT2000
Method for updating of URBEXT	CPRE formula from 2006 CEH report on URBEXT2000

⁷ Updated to current year

⁸ British Geological Society (2021). UK Soil Observatory Map Viewer. Accessed: 16/11/2022. [Source: mapapps2.bgs.ac.uk/ukso/home.html].

Source of BFIHOST	BFIHOST19 was used in the ReFH2 calculations, since the current release (ReFH2.3) was calibrated using BFIHOST19, and also in the FEH Statistical method, since this has been found to improve the results ⁹ .
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⁹ Griffin, A., Young, A. and Stewart, E. (2019). Revising the BFIHOST catchment descriptor to improve UK flood frequency estimates. Hydrology Research.

3 Statistical method

3.1 Overview of estimation of QMED at each subject site

Site code	Initial QMED rural (m ³ /s) (from catchment descriptors)	Final method	Data transfer					Urban adjust-ment factor (UAF)	Final QMED estimate (m ³ /s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d _{ij} (km)	Moderated QMED adjustment factor, (A/B) ^a	If more than one donor			
						Weight	Weighted ave. adjustment		
RI_01	7.1	DT	69043	2.462	1.091	-	-	1.850	14.3
BB_01	1.9	DT	69020	5.855	0.845	-	-	1.365	2.2
RI_02	10.1	DT	69043	1.845	1.101	-	-	1.712	19.1
Are the values of QMED spatially consistent?					Yes, QMED increases sensibly moving downstream.				
Method used for urban adjustment for subject and donor sites					WINFAP v4 ¹⁰				
Parameters used for WINFAP v4 urban adjustment if applicable									
Impervious fraction for built-up areas, IF			Percentage runoff for impervious surfaces, PR _{imp}		Method for calculating fractional urban cover, URBAN				
0.3			70%		From updated URBEXT2000				
Notes									
Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).									
The QMED adjustment factor A/B for each donor site is given in Table 3.2. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is: (A/B) ^a x QMED _{initial} x UAF									
Important note on urban adjustment									
The method used to adjust QMED for urbanisation published in Kjeldsen (2010) ¹¹ in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable.									

¹⁰ Wallingford HydroSolutions (2016). WINFAP 4 Urban adjustment procedures.

¹¹ Kjeldsen, T. R. (2010). Modelling the impact of urbanization on flood frequency relationships in the UK. Hydrol. Res. **41**. 391-405.

3.2 Search for donor sites for QMED (if applicable)

Comment on potential donor sites Include a map if necessary. Note that donor catchments should usually be rural.	A number of potential donor catchments for this study were identified using the NFRA website and the 'Estimating QMED by data transfer' tool which is available in WINFAP-FEH3 as there are no donor catchments available within the study catchments. (There is a gauge located a while downstream on the Irk catchment, which is considered as a donor here). Two were taken forward for further consideration as a potential donor catchment: 69043 - Irk @ Collyhurst Weir 69020 - Medlock @ London Road Both donors were identified by NFRA as potentially suitable due to a mixture of proximity and on catchment descriptors. As the River Irk is gauged downstream of the study catchments (69043 - Irk @ Collyhurst Weir) and considered to be the best donor catchment for the River Irk inflow and downstream checkpoint based on catchment descriptors and on proximity so was used within this study. For BB_01, 69020 - Medlock @ London Road was considered to be the best donor catchment for the site based on catchment descriptors and on proximity so was also used within this study.
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3.3 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing	Method (AM or POT)	Adjustment for climatic variation ?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B) *
69043	Closest hydroclimatological similarities to subject site (RI_01 and RI_02)	AM	No	28.3	15.23	1.154
69020	Closest hydroclimatological similarities to subject site (BB_01)	AM	No	23.3	24.29	0.684
* After application of separate 'de-urbanisation' factors to the observed QMED values.						

3.4 Derivation of pooling groups

Several subject sites may use the same pooling group.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
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BB_01 'Small Catchment' Pooling ¹²	BB_01	No	Removed: 27073 (Brompton Beck @ Snainton Ings) and 49005 (Bolingey Stream @ Bolingey Cocks Bridge): L-moment outliers Included: 73015 (Keer @ High Keer Weir)	L-CV: 0.252, L-SKEW: 0.263.
RI_02 Pooling	RI_02	No 69043 is located significantly downstream of the subject site.	Removed: 47023 (Tamar @ Tamarstone Bridge): Insufficient number of years Included: -	L-CV: 0.233, L-SKEW: 0.198.

Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).

3.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (Error! Reference source not found.)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape after adjustments)	Growth factor for 100-year return period
RI_01	P	RI_02 Pooling	GEV Acceptable fit in WINFAP v5 software	WINFAP-FEHV4 urban adjustment	Location: 1.000 Scale: 0.241 Shape: -0.149	2.54
BB_01	P	BB_01 'Small Catchment' Pooling	GEV Acceptable fit in WINFAP v5 software	WINFAP-FEHV4 urban adjustment	Location: 1.000 Scale: 0.248 Shape: -0.240	2.99
RI_02	P	RI_02 Pooling	GEV Acceptable fit in WINFAP v5 software	WINFAP-FEHV4 urban adjustment	Location: 1.000 Scale: 0.252 Shape: -0.144	2.55

eNotes

Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis

A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.

Urban adjustments are all carried out using the method of Kjeldsen (2010).

Growth curves were derived using the procedures from Science Report SC050050 (2008).

¹² Small Catchment pooling method applied via WINFAP v5

3.6 Flood estimates from the statistical method

Site code	Flood peak (m ³ /s) for the following AEP (%) events					
	50	3.3	1	1 + 35% climate change allowance (Central)	1 + 46% climate change allowance (Higher)	0.1
RI_01	14.3	28.5	36.3	49.0	53.0	55.6
BB_01	2.2	4.9	6.7	9.0	9.8	11.9
RI_02	19.1	38.2	48.6	65.6	70.9	74.0

4 Revitalised flood hydrograph 2 (ReFH2) method

4.1 Parameters for ReFH2 model

In accordance with research findings, all catchments with an URBEXT2000 up to 0.30 should be modelled as if they were rural. Research on flood estimation in small catchments¹³ found that flood frequency estimates on such catchments were more accurate if the catchment was treated as rural. However the study catchments all have an updated URBEXT2000 value over 0.3.

EA Flood Estimation Guidelines LIT 11832 (2022) that the FEH Statistical method or ReFH2 methods can be applied to urbanised catchments with an URBEXT2000 value up to 0.6¹⁴. A winter storm rainfall depth, Cini and initial baseflow values and storm profile should be used for all catchments, unless:

- URBEXT2000 $\geq$ 0.30;
- or URBEXT2000 $\geq$ 0.15 and BFIHOST19 $\geq$ 0.65

Therefore a Summer, Urban Profile was used.

All catchments					Only extremely heavily urbanised catchments
Site code	Method	C _{max} (mm)	T _p _{rural} (hours)	BL (hours)	Area of catchment modelled as urban (km ²)
RI_01	CD	589.25	3.3	49.26	-
BB_01	CD	237.39	1.86	23.82	-
RI_02	CD	532.48	3.28	47.45	-
Link to details of any lag or flood event analysis		Not within scope			
Version of the ReFH2 model applied		ReFH2.3 using the water balance option. This treats BR (baseflow recharge) as a state variable rather than a parameter, setting it automatically in order to conserve volume. The values of BR vary with return period and so are not reported here.			
Parameters for urban runoff model		The impervious fraction of urban areas, IF, was kept at its default of 0.4. The impervious runoff factor, IRF, (which can also be interpreted as the fraction of the impervious surface that is positively drained) was kept at its default of 0.7. The depression storage was kept at its default of 0.5mm. Tp for runoff from areas modelled as positively drained was calculated as 0.75 times Tp _{rural} .			
Methods: OPT: Optimisation from fitting to observed flow data, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)					

4.2 Design events for ReFH2 method: Lumped catchments

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)
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¹³ Stewart, Lisa, Duncan Faulkner, Giuseppe Formetta, Adam Griffin, Tracey Haxton, Ilaria Prosdoci, Gianni Vesuviano and Andy Young (2021). Estimating flood peaks and hydrographs for small catchments (Phase 2). Report – SC090031/R0, Environment Agency.

¹⁴ Environment Agency (2012) Estimating flood peaks and hydrographs for small catchments: Phase 1 and Phase 2. LGH_JBAU_XX_00_RP_HO_0001_S1_P1_FEH calculation record

RI_01	Urban, Summer	10:15:00	-
BB_01	Urban, Summer	05:15:00	-
RI_02	Urban, Summer	10:15:00	-
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?			Storm Duration stated in the table is the Critical Storm Duration for each individual Flood Estimation Point, determined through sensitivity testing of a range of durations within ReFH2 to determine the duration that produced the highest flow. The storm duration is used here to undertake 'worst case' scenario flow comparisons between ReFH2 and FEH Statistical for each individual flow point. Due to the permeability of the catchment, the preferred methodology for this study will be FEH Statistical. For modelling purposes, design ReFH2 hydrographs to a critical storm duration of 10:15:00 (CSD of RI_02) will be taken forward

4.3 Flood estimates from the ReFH2 method

Note: This table is for recording results for lumped catchments. There is no need to record peak flows from sub-catchments or intervening areas that are being used as inputs to a semi-distributed model of the river system.

Peak Flows

Site code	Flood peak (m ³ /s) for the following AEP (%) events					
	50	3.3	1	1 + 35% climate change allowance (Central)	1 + 46% climate change allowance (Higher)	0.1
RI_01	12.9	26.5	34.7	46.9	50.7	55.9
BB_01	2.5	5.3	6.8	9.2	10.0	11.1
RI_02	15.4	31.7	41.6	56.1	60.7	67.5

Model design duration (10:15:00hr)

Site code	Flood peak (m ³ /s) for the following AEP (%) events					
	50	3.3	1	1 + 35% climate change allowance (Central)	1 + 46% climate change allowance (Higher)	0.1
RI_01	12.9	26.5	34.7	46.9	50.7	55.9
BB_01	2.4	4.9	6.5	8.7	9.4	10.6
RI_02	15.4	31.7	41.6	56.1	60.7	67.5

5 Discussion and summary of results

5.1 Comparison of results from different methods

This table compares peak flows from various methods with those from the FEH Statistical method at example sites for two key return periods. Blank cells indicate that results for a particular site were not calculated using that method.

Site code	Ratio of peak flow to FEH Statistical peak	
	Return period 2 years	Return period 100 years
	ReFH2	ReFH2
RI_01	0.91	0.96
IB_01	1.11	1.02
RI_02	0.81	0.86

5.2 Final choice of method

Choice of method and reasons Include reference to type of study, nature of catchment and type of data available.	Flows derived through the application of the ReFH2 model were compared with those derived from the FEH statistical method. Flows derived from the ReFH2 methodology are shown to be up to 11% greater than flows derived from FEH statistical methodology for BB_01. For RI_01 and RI_02 flows derived from the FEH Statistical methodology are shown to be up to 19% greater than flows derived from ReFH2 methodology up to the 0.1% AEP event, where ReFH2 flows are greater at RI_01. LIT 11832 Flood Estimation Guidelines (2023) states that the statistical method is based on a much larger dataset of flood events and has been more directly calibrated to reproduce flood frequency on UK catchments, therefore it is often preferable over design event (rainfall-runoff) approaches. In addition, the EA OI_197_08-Flood Estimation Guidelines, recommends in preference use of FEH Statistical over ReFH2 method for permeable catchments. Therefore the preferred methodology for this study will be FEH Statistical. The Statistical method is valid for up to a 100-year AEP approximately and less certain for rarer events. Therefore, peak flows derived using the FEH Statistical method for the 1000-year event has been adjusted to apply the growth factor derived from ReFH2 methodology (often referred to as the "Hybrid" approach).
How will the flows be applied to a hydraulic model? If relevant. Will model inflows be adjusted to achieve a match with lumped flow estimates, or will the model be allowed to route inflows?	Catchment flows will be distributed along the study reach at locations where flow points have been identified within Figure 2.1. For modelling purposes, hydrographs were obtained using the ReFH2 method to storm duration of 10:15:00hr (CSD for RI_01 and RI_02). During the modelling phase of this study, if it is found that a lateral inflow is required, flows for the lateral area (c. 4km²) will be derived by using an area ratio method from the flows derived for RI_02. The hydrograph adopted for the lateral areas will be the design hydrograph for RI_02.

5.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions are: -The data used for the derivation of local design flow estimates are accurate. -The design peak flows derived are representative of those that would be observed during flood events; -The catchment boundaries and descriptors have been assumed to be representative; -The pooling group used to derive peak flow estimates is representative of the subject catchment; -Design hydrographs generated using the ReFH2 model are representative of those that would be observed during typical flood events. - Peak flows, rather than hydrograph volumes, are assumed to be the main control on flood risk at the site. - Use of GEV distribution in pooling group assessment is assumed to be preferable to the other distributions identified by WINFAP 5 due to its z-value. - There are no significant sewers draining in or out of the catchment. It is not within the scope of this study to define sewer boundaries.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed.	Limitations are generic to the methods used. The main limitation for this study is the absence of flow gauge data for the River Irk within the study catchments which could be used to improve peak flow estimates. As flood generation on urban catchments is sensitive to local circumstances it is not possible to obtain a reliable estimate of the flood frequency curve using generalised methods. There is no substitute for obtaining local data. The scope of the study is also a limitation as it was not possible to assess the surface water sewer network for the purposes of sub-catchment definition or to undertake lag analysis to improve the ReFH2 Tp model parameter. One of the catchments is considered to be small relative to the overall database of gauging station data used for calibration, the use of FEH / ReFH is considered to be a primary limitation of the study. Urbanisation of the catchments is also a limitation. A simplification is also applied with a consistent AEP applied to all inflows, for a given design even model run. A joint probability analysis could be undertaken for the catchment upstream inflows, but applying a constant AEP for the purposes of a Flood Risk assessment gives a conservative result. It is recommended to review the modelling results once the flows are applied to the model. If these are found to over-estimate local flood risk, additional work could be undertaken with a joint probability analysis of the model inflows. However a joint probability analysis is outwith the scope of this project.
Give what information you can on uncertainty in the results, e.g. confidence limits from Kjeldsen (2014).	No published methods of calculating confidence intervals exist for ReFH hydrological models; therefore, the uncertainty for each of these watercourses is difficult to

	quantify. The uncertainty will depend on many factors, for example, how unusual the study catchment is relative to the pooling group and donor catchment, and the uncertainty in flow measurement at other gauges. The Environment Agency Flood Estimation Guidelines present confidence limits for Statistical method design flows at ungauged sites. There are confidence intervals for rural catchments ($URBEXT2000 < 0.03$) and moderately urbanised catchments ($0.03 \leq URBEXT2000 < 0.15$). The 95% confidence limits for a 1% AEP flood estimate without donor adjustment of QMED are: - Rural catchments: 0.45 – 2.23 times the best estimate - Moderately urbanised catchments: 0.33 – 3.01 times the best estimate Confidence intervals for heavily urbanised catchments ($URBEXT2000 > 0.15$) are not provided because there are not enough suitable catchments with flood peak data to provide reliable statistics. It may be assumed that the confidence intervals will be somewhat wider than those for moderately urbanised catchments.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	These flow estimates were derived specifically for this study. They should not be used elsewhere without at least being reviewed for suitability.
Give any other comments on the study, e.g. suggestions for additional work.	Confidence could be improved through installation of hydrometric monitoring and event hydrometric data.

5.4 Checks

Are the results consistent, for example at confluences?	Yes, results are considered sensible in that they increase with downstream distance where applicable.
What do the results imply regarding the return periods of floods during the period of record?	No data is available in regards to flood events applicable to the site. No flow gauges are available within the study catchment, therefore no comparison with previous events can be undertaken at this stage.
What is the range of 100-year growth factors? Is this realistic?	The 1% AEP growth factor for each of the methods are: - FEH Statistical: 2.54-2.99 - ReFH2: 2.68-2.73 The typical range is 2.1 to 4.0 (based on FSR regional growth curves) therefore the 1% AEP growth factor for all FEP's are inside the typical range for both flow estimation methodologies.
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	The 0.1%/ 1% AEP event growth factor for each of the methods are: - FEH Statistical: 1.52-1.77 - ReFH2: 1.61-1.62
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	Flows have been provided within the Faber Maunsell (2007) Irwell Areas Benefiting from Defences Study – Phases 2&3. The report provided as part of the EA data request makes reference to the Final Hydrology Report, however this report was not supplied. A number of flows

for a number of node points are included within the provided report however it is difficult to ascertain exact node locations. None of the design peak flows within the report relates to the FEP's within this study, however there are some nodes where flows and levels from the model have been extracted which have been used for comparison. Flows derived within the 2007 study are significantly greater than the flows derived within this study.

Site code	Flood peak (m ³ /s)	
	100	1000
2023: RI_02 FEH	48.6	78.9
2023: RI_02 ReFH2	41.6	67.5
2007: IRKR02_5773B (without defences)	54.9	101.7

Difference in flows between this study and the 2007 study is likely due to updates in methodologies, software and updates to catchment descriptors. The Revitalised Flood Hydrograph (ReFH1 and ReFH2) method has replaced the original 1999 FEH rainfall-runoff method for most applications. As this study utilises 2023 methodologies and more up to date data, flows derived within this study are the preferred flows.

Are the results compatible with the longer-term flood history?	This will be determined at the modelling stage.
Describe any other checks on the results	-

5.5 Final results

Site code	Flood peak (m ³ /s) for the following AEP (%) events					
	50	3.3	1	1 + 19% climate change allowance (Central)	1 + 30% climate change allowance (Higher)	0.1
RI_01	14.3	28.5	36.3	49.0	53.0	58.4*
BB_01	2.2	4.9	6.7	9.0	9.8	10.9*
RI_02	19.1	38.2	48.6	65.6	70.9	78.9*
* growth factor derived from ReFH2 methodology applied						

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Annex B
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6 Annex A – Pooling Groups

UK Design Flood Estimation

Summary of ESS/Pooled Estimation Analysis using the Flood Estimation Handbook Statistical Method

Date of creation: 30-08-2023 11:53:46
Software: WINFAP Version: 5.0.8181 (30330)
Peak Flow dataset: Peak Flow Dataset 11.1.1
Supplementary data used: No

Site details

Site number: 427445649
Site name: BB_01
Site location: SD 85900 05100
Easting: 385900
Northing: 405100
Catchment area: 2.51 km²
SAAR: 999 mm
BFIHOST19: 0.333
FPEXT: 0.140
FARL: 0.994
URBEXT2000: 0.3770

Analysis settings

At-site data

At-site data present: No

Urbanisation settings

User defined: No
Urban area: 1.48 km²
PRimp: 70.00%
Impervious Factor: 0.300
UAF: 1.36508

Growth curve settings

Distance Measure Method: Small catchment
Pooling group URBEXT2000 Threshold: 0.030
Deurbanise Pooling Group L-moments: Yes

QMED settings

Use at-site data: No
Method: Donor Station(s)

Growth curve data and results

Pooling Group

Station	Distance	Years of data	QMED AM	L-CV Observed	L-CV Deurbanised	L-SKEW Observed	L-SKEW Deurbanised	Discordancy
76011 (Coal Burn @ Coalburn)	0.433	44	1.840	0.168	0.168	0.302	0.302	0.878
45816 (Haddeo @ Upton)	0.962	28	3.352	0.293	0.294	0.424	0.423	1.018
27051 (Crimple @ Burn Bridge)	1.035	49	4.564	0.217	0.218	0.143	0.142	0.584
28033 (Dove @ Hollinsclough)	1.247	46	4.138	0.223	0.223	0.379	0.379	0.974
25019 (Leven @ Easby)	1.515	43	5.677	0.334	0.335	0.373	0.372	0.865
27010 (Hodge Beck @ Bransdale Weir)	1.594	41	9.420	0.224	0.224	0.293	0.293	0.205
47022 (Tory Brook @ Newnham Park)	1.646	26	6.649	0.250	0.252	0.149	0.146	1.173
44008 (South Winterbourne @ Winterbourne Steepleton)	1.650	30	0.496	0.421	0.422	0.289	0.289	2.393
26016 (Gypsy Race @ Kirby Grindalythe)	1.661	24	0.103	0.304	0.304	0.240	0.240	1.072
25011 (Langdon Beck @ Langdon)	1.689	35	15.647	0.226	0.226	0.324	0.324	2.264
72014 (Conder @ Galgate)	1.995	53	16.912	0.231	0.232	0.145	0.144	0.560
18014 (Bannock Burn @ Bannockburn)	2.008	29	16.634	0.224	0.225	0.162	0.160	0.403
73015 (Keer @ High Keer Weir)	2.009	31	12.421	0.208	0.209	0.195	0.195	0.612
Total		479						
Short records	Discordant	No Pooling	No Pooling, no QMED					

Pooling Group Rejected Stations

Station	Distance	Years of data	QMED AM	L-CV Observed	L-CV Deurbanised	L-SKEW Observed	L-SKEW Deurbanised
27073 (Brompton Beck @ Snainton Ings)	1.313	41	0.820	0.212	0.213	0.006	0.005
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	1.475	11	5.777	0.262	0.263	0.207	0.206

Growth curve L-moments

Rural L-CV:	0.252	Urban L-CV:	0.202
Rural L-Skewness:	0.263	Urban L-Skewness:	0.333

Rural fitted parameters

Distribution	Location	Scale	Shape	H	Bound
GL	1.000	0.251	-0.263		0.046
GEV	0.867	0.354	-0.140		-1.666
KAP3	0.932	0.297	-0.208	-0.400	-0.494

Urban fitted parameters

Distribution	Location	Scale	Shape	H	Bound
GL	1.000	0.187	-0.333		0.440
GEV	0.905	0.249	-0.239		-0.134
KAP3	0.950	0.214	-0.292	-0.400	0.218

Goodness of fit

GL: 1.2017 *
GEV: -0.1624 *
P3: -2.1971
GP: -3.6108
KAP3: 0.7506 *

* Distribution gives an acceptable fit (absolute Z value < 1.645)

Heterogeneity

Standardised test value H2: 1.7536

The pooling group is possibly heterogeneous and a review of the pooling group is optional.

Standardised growth curves

Rural

Return period	GL	GEV	KAP3
2	1.000	1.000	1.000
5	1.420	1.458	1.436
10	1.747	1.804	1.773
20	2.116	2.172	2.145
25	2.248	2.297	2.275
30	2.360	2.401	2.385
50	2.703	2.707	2.714
75	3.007	2.964	2.999
100	3.243	3.155	3.217
200	3.887	3.648	3.795
500	4.938	4.377	4.698
1000	5.919	4.993	5.504

Urban

Return period	GL	GEV	KAP3
2	1.000	1.000	1.000
5	1.329	1.354	1.338
10	1.605	1.646	1.623
20	1.934	1.981	1.957
25	2.056	2.100	2.078
30	2.161	2.202	2.183
50	2.490	2.510	2.505
75	2.792	2.782	2.796
100	3.031	2.991	3.024
200	3.710	3.558	3.658
500	4.883	4.465	4.717
1000	6.040	5.296	5.728

QMED data and results

Donor selection criteria

Only sites suitable for QMED: Yes
URBEXT2000: <0.030
Donor adjusted FSE: 1.399
No. of donors: 1

Donor stations

Station	Distance	URBEXT	Use QMED obs deurbanised	QMED obs	QMED deurbanised	QMED CDs urban	QMED CDs rural	Centroid X	Centroid Y	Area	SAAR
*BB_01 @ SD 85900 05100)		0.377						386937	404182	2.510	999
69048 (Tame @ Uppermill)	13.52	0.023	Yes	21.885	21.422	22.430	22.430	399678	408701	40.602	1359

Unused Donor stations

Station	Distance	URBEXT	Use QMED obs deurbanised	QMED obs	QMED deurbanised	QMED CDs urban	QMED CDs rural	Centroid X	Centroid Y	Area	SAAR
69046 (Bradshaw Brook @ Bradshaw Tennis Club)	19.36	0.023	Yes	22.598	22.105	12.256	12.256	372843	417450	36.865	1.0
27088 (Calder @ Mytholmroyd)	25.04	0.023	Yes	82.325	80.632	81.846	81.846	395015	427885	146.868	1.0
69017 (Goyt @ Marple Bridge)	26.72	0.025	Yes	61.047	59.443	53.096	53.096	402591	382528	184.232	1.0
68007 (Wincham Brook @ Lostock Gralam)	30.05	0.020	Yes	20.402	19.888	19.576	19.576	375825	376264	148.340	8.0
68011 (Arley Brook @ Gore Farm)	30.39	0.021	Yes	6.093	5.972	11.089	11.089	366592	381601	33.760	8.0
28043 (Derwent @ Chatsworth)	38.01	0.006	Yes	88.165	87.610	96.834	96.834	418886	383583	344.337	1.0
28023 (Wye @ Ashford)	38.32	0.023	Yes	16.685	16.089	30.120	30.120	411221	374539	152.280	1.0
68044 (Dane @ Hug Bridge)	38.72	0.001	Yes	43.990	43.947	43.891	43.891	398635	367269	72.558	1.0
68018 (Dane @ Congleton Park)	39.43	0.023	Yes	41.750	40.836	52.357	52.357	394496	365488	142.565	1.0

QMED

Rural: 1.909 m³/s
Urban: 2.606 m³/s

Flood Frequency Curve

Rural Flood Frequency Curve

Return period	GL (m³/s)	GEV (m³/s)	KAP3 (m³/s)
2	1.909	1.909	1.909
5	2.711	2.784	2.741
10	3.335	3.444	3.384
20	4.040	4.146	4.095
25	4.291	4.384	4.342
30	4.505	4.583	4.552
50	5.159	5.167	5.180
75	5.740	5.658	5.726
100	6.190	6.024	6.140
200	7.420	6.965	7.245
500	9.427	8.355	8.968
1000	11.298	9.531	10.507

Urban Flood Frequency Curve

Return period	GL (m³/s)	GEV (m³/s)	KAP3 (m³/s)
2	2.606	2.606	2.606
5	3.463	3.527	3.487
10	4.182	4.290	4.228
20	5.041	5.163	5.099
25	5.356	5.473	5.415
30	5.631	5.738	5.687
50	6.487	6.541	6.528
75	7.274	7.250	7.287
100	7.899	7.795	7.881
200	9.668	9.271	9.532
500	12.724	11.636	12.291
1000	15.739	13.802	14.925

UK Design Flood Estimation

Summary of ESS/Pooled Estimation Analysis using the Flood Estimation Handbook Statistical Method

Date of creation: 31-08-2023 09:37:30
Software: WINFAP Version: 5.0.8181 (30330)
Peak Flow dataset: Peak Flow Dataset 11.1.1
Supplementary data used: No

Site details

Site number: 4071048075
Site name: RI_02
Site location: SD 84400 04700
Easting: 384400
Northing: 404700
Catchment area: 44.91 km²
SAAR: 1062 mm
BFIHOST19: 0.644
FPEXT: 0.096
FARL: 1.000
URBEXT2000: 0.3800

Analysis settings

At-site data

At-site data present: No

Urbanisation settings

User defined: No
Urban area: 26.74 km²
PRimp: 70.00%
Impervious Factor: 0.300
UAF: 1.71138

Growth curve settings

Distance Measure Method: Standard
Pooling group URBEXT2000 Threshold: 0.030
Deurbanise Pooling Group L-moments: No

QMED settings

Use at-site data: No
Method: User Defined

Growth curve data and results

Pooling Group

Station	Distance	Years of data	QMED AM	L-CV Observed	L-CV Deurbanised	L-SKEW Observed	L-SKEW Deurbanised	Discordancy
76019 (Roe Beck @ Stockdalewath)	0.529	22	42.495	0.218	0.218	0.311	0.311	1.110
41020 (Bevern Stream @ Clappers Bridge)	0.529	52	13.780	0.201	0.203	0.166	0.164	0.662
73015 (Keer @ High Keer Weir)	0.650	31	12.421	0.208	0.209	0.195	0.195	0.237
48012 (Fal @ Trenowth)	0.676	23	10.210	0.146	0.148	0.279	0.275	2.201
41022 (Lod @ Halfway Bridge)	0.678	51	15.900	0.295	0.297	0.183	0.181	0.599
72014 (Conder @ Galgate)	0.681	53	16.912	0.231	0.232	0.145	0.144	0.266
43029 (Wyllye @ Brixton Deverill)	0.700	38	1.668	0.374	0.375	0.254	0.253	2.803
53017 (Boyd @ Bitton)	0.735	48	13.908	0.240	0.242	0.081	0.079	1.195
76023 (Dacre Beck @ Dacre Bridge)	0.735	21	35.854	0.189	0.189	0.213	0.213	0.834
28041 (Hamps @ Waterhouses)	0.754	36	26.582	0.218	0.218	0.272	0.272	1.645
49002 (Hayle @ St Erth)	0.793	64	4.785	0.240	0.241	0.201	0.199	0.145
49004 (Gannel @ Gwills)	0.798	52	14.510	0.257	0.259	0.114	0.113	0.979
72007 (Brock @ upstream of A6)	0.836	43	29.917	0.195	0.195	0.208	0.208	0.324
Total		534						
Short records	Discordant	No Pooling	No Pooling, no QMED					

Pooling Group Rejected Stations

Station	Distance	Years of data	QMED AM	L-CV Observed	L-CV Deurbanised	L-SKEW Observed	L-SKEW Deurbanised
47023 (Tamar @ Tamarstone Bridge)	0.774	6	18.596	0.154	0.154	0.690	0.690

Growth curve L-moments

Rural L-CV:	0.233	Urban L-CV:	0.186
Rural L-Skewness:	0.198	Urban L-Skewness:	0.265

Rural fitted parameters

Distribution	Location	Scale	Shape	H	Bound
GL	1.000	0.235	-0.198		-0.192
GEV	0.870	0.351	-0.042		-7.384
KAP3	0.935	0.286	-0.127	-0.400	-1.312

Urban fitted parameters

Distribution	Location	Scale	Shape	H	Bound
GL	1.000	0.179	-0.265		0.323
GEV	0.905	0.252	-0.142		-0.867
KAP3	0.951	0.211	-0.210	-0.400	-0.056

Goodness of fit

GL: 3.2886
GEV: 0.9934 *
P3: -0.3868 *
GP: -4.1544
KAP3: 2.4547

* Distribution gives an acceptable fit (absolute Z value < 1.645)

Heterogeneity

Standardised test value H2: 0.4137

The pooling group is acceptably homogeneous and a review of the pooling group is not required.

Standardised growth curves

Rural

Return period	GL	GEV	KAP3
2	1.000	1.000	1.000
5	1.376	1.414	1.392
10	1.648	1.699	1.672
20	1.940	1.981	1.963
25	2.041	2.072	2.061
30	2.126	2.147	2.142
50	2.379	2.359	2.378
75	2.597	2.529	2.576
100	2.763	2.652	2.723
200	3.199	2.953	3.097
500	3.875	3.364	3.644
1000	4.472	3.686	4.101

Urban

Return period	GL	GEV	KAP3
2	1.000	1.000	1.000
5	1.300	1.327	1.311
10	1.534	1.574	1.552
20	1.799	1.837	1.819
25	1.894	1.927	1.912
30	1.974	2.001	1.991
50	2.220	2.221	2.227
75	2.439	2.406	2.432
100	2.609	2.543	2.588
200	3.073	2.898	3.005
500	3.830	3.423	3.656
1000	4.538	3.868	4.238

QMED data and results

QMED

Rural: 18.900 m³/s
Urban: 32.345 m³/s

Flood Frequency Curve

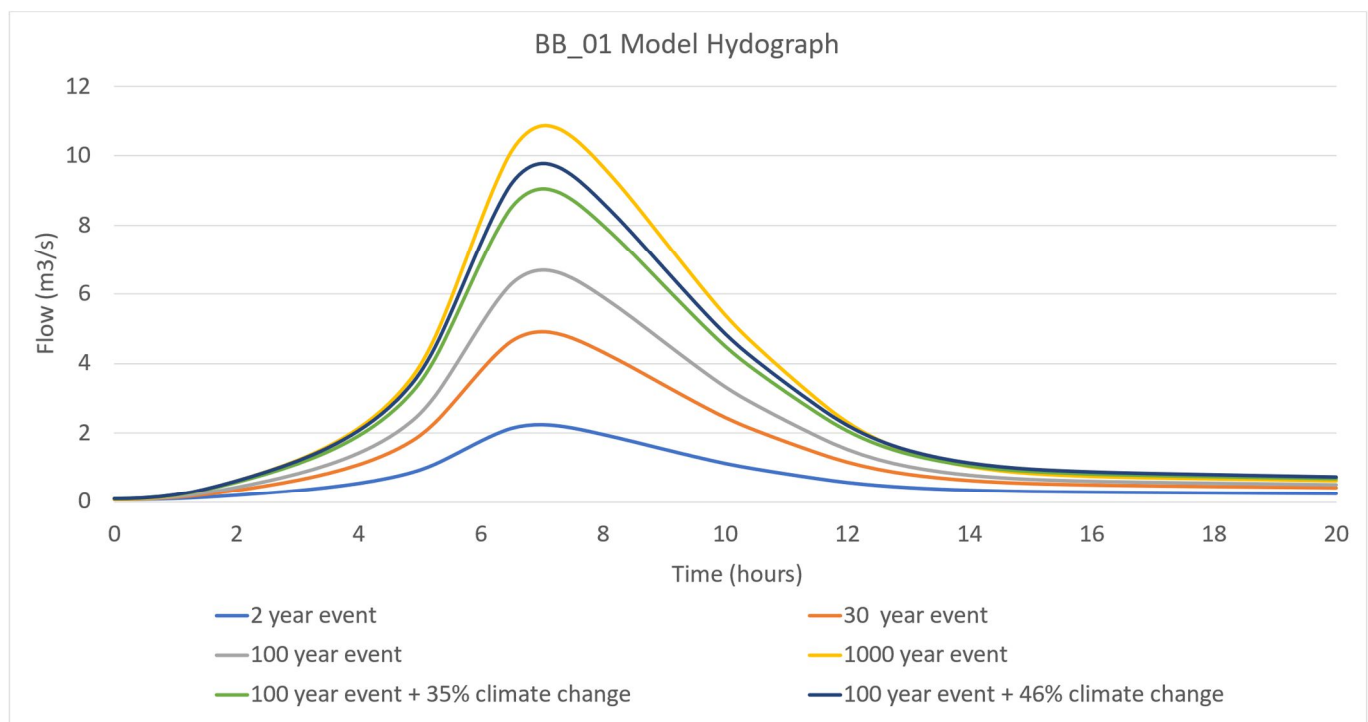
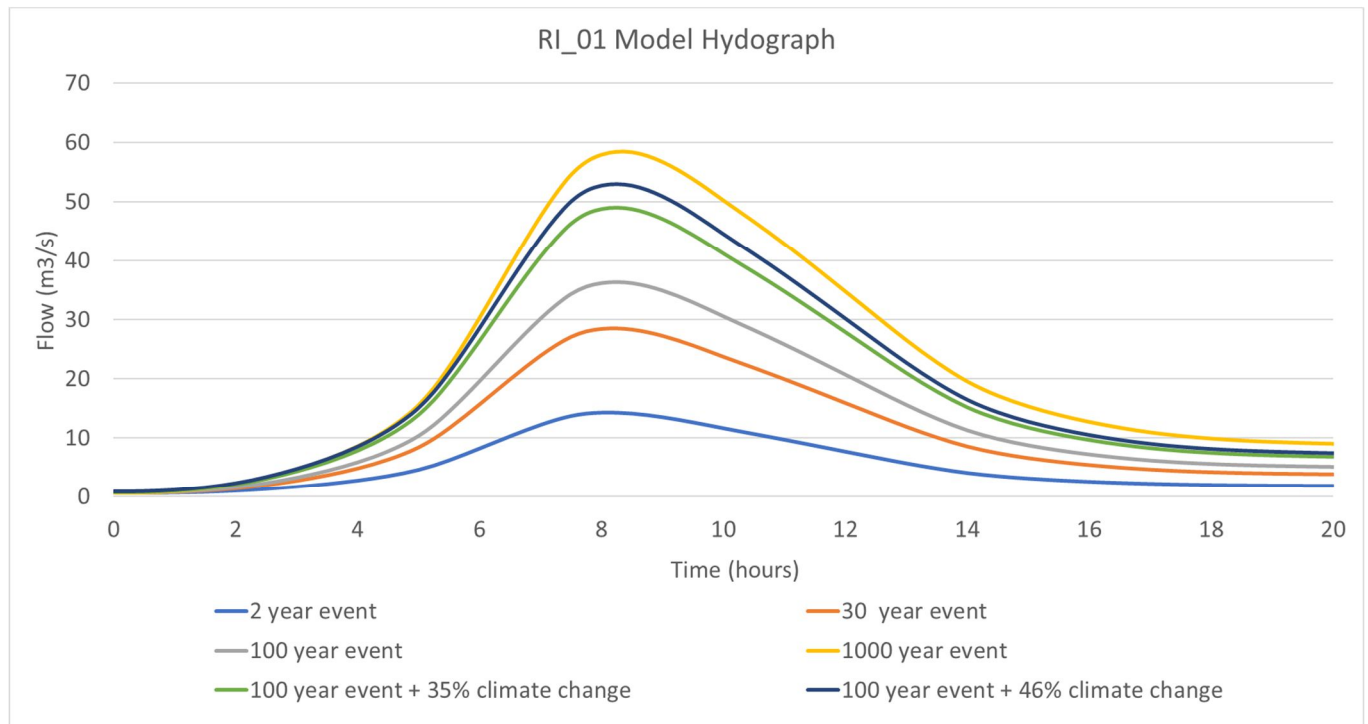
Rural Flood Frequency Curve

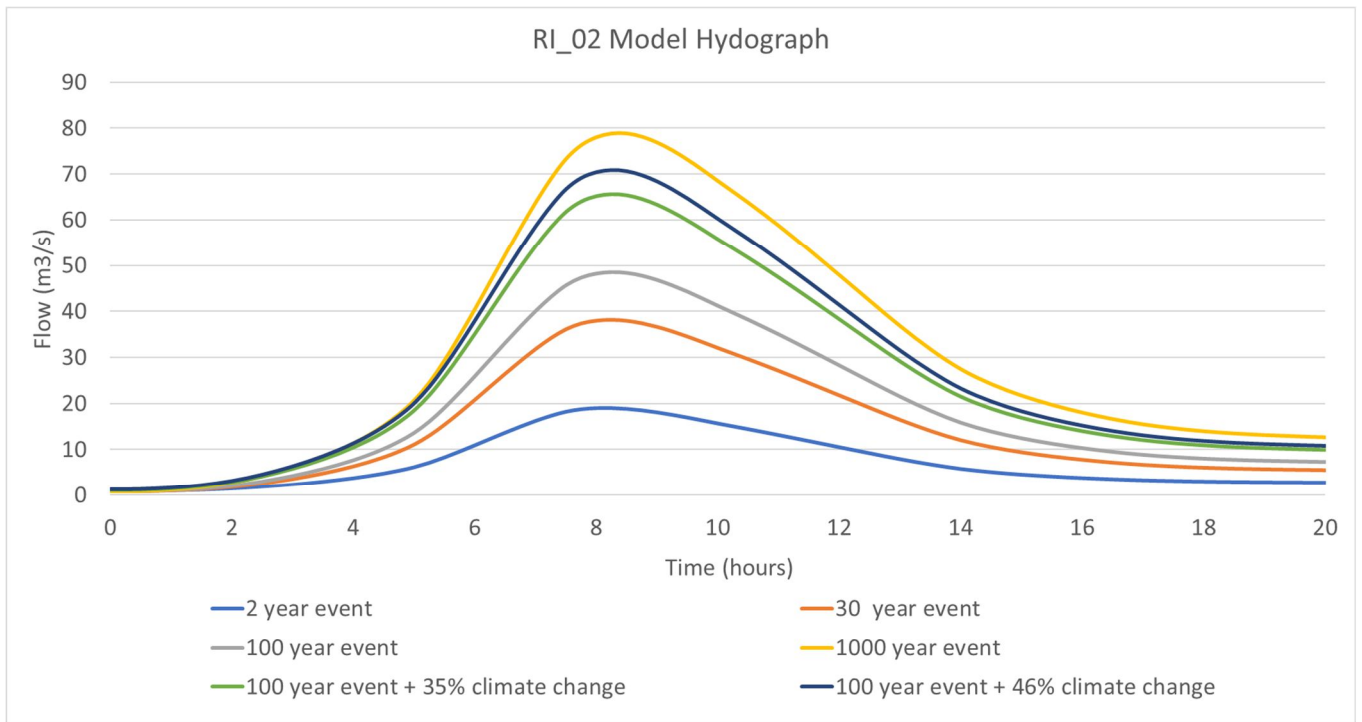
Return period	GL (m³/s)	GEV (m³/s)	KAP3 (m³/s)
2	18.900	18.900	18.900
5	25.997	26.717	26.309
10	31.144	32.103	31.603
20	36.675	37.433	37.102
25	38.578	39.159	38.944
30	40.186	40.575	40.481
50	44.969	44.580	44.953
75	49.092	47.806	48.692
100	52.212	50.123	51.456
200	60.470	55.813	58.529
500	73.233	63.581	68.864
1000	84.529	69.656	77.515

Urban Flood Frequency Curve

Return period	GL (m³/s)	GEV (m³/s)	KAP3 (m³/s)
2	32.345	32.345	32.345
5	42.058	42.914	42.408
10	49.630	50.910	50.210
20	58.202	59.425	58.833
25	61.249	62.314	61.838
30	63.860	64.734	64.389
50	71.817	71.826	72.030
75	78.894	77.809	78.669
100	84.377	82.258	83.722
200	99.387	93.736	97.191
500	123.895	110.726	118.242
1000	146.781	125.123	137.066

7 Annex C – Design Hydrographs





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Limerick
Newcastle upon Tyne
Newport
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Tadcaster
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Wallingford
Warrington

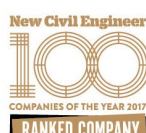
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ISO 45001: 2018



E

Sensitivity Analysis

E.1

Sensitivity to roughness

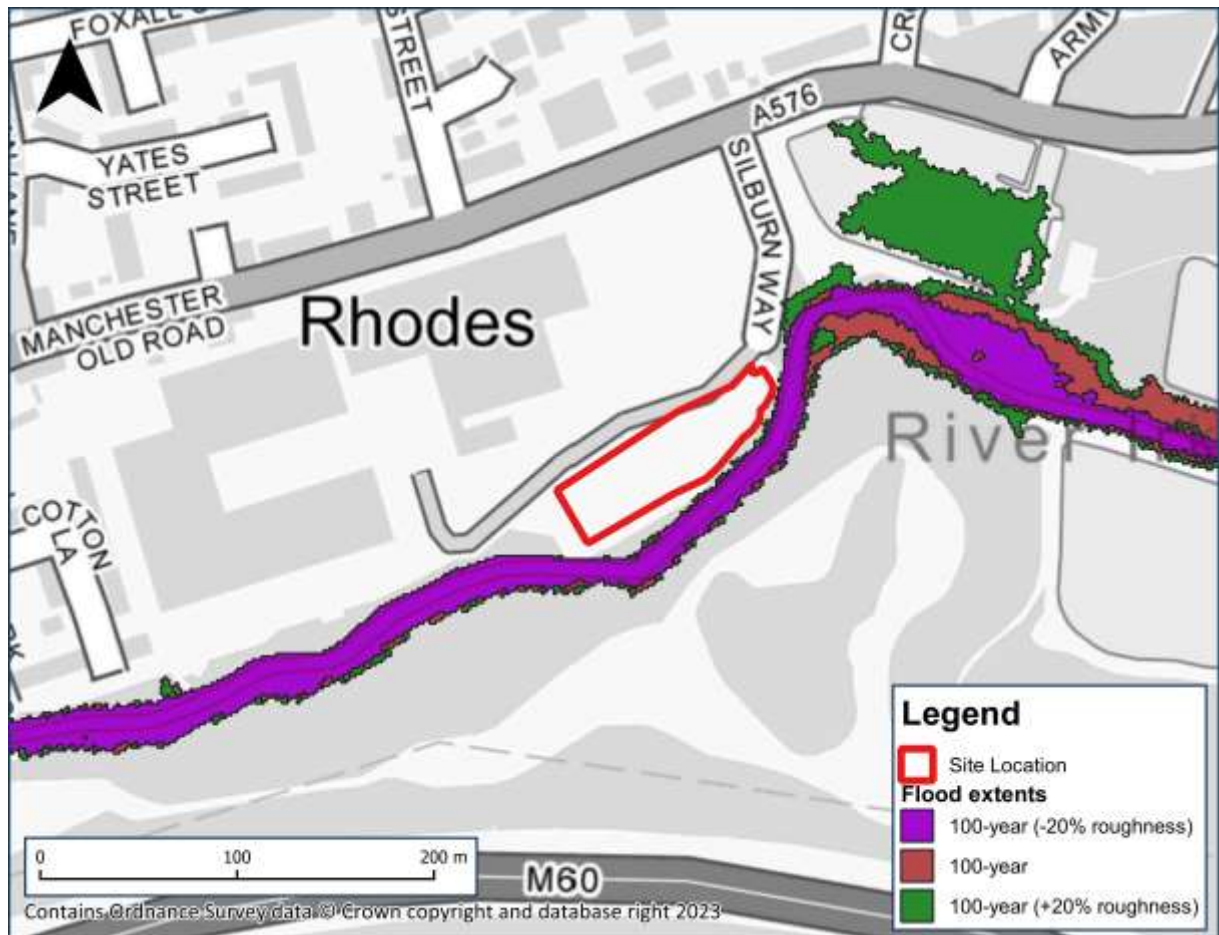


Figure 6-1 shows the changes in peak flood extent compared to the baseline 100-year event when the 1D and 2D roughness is adjusted by +/- 20%.

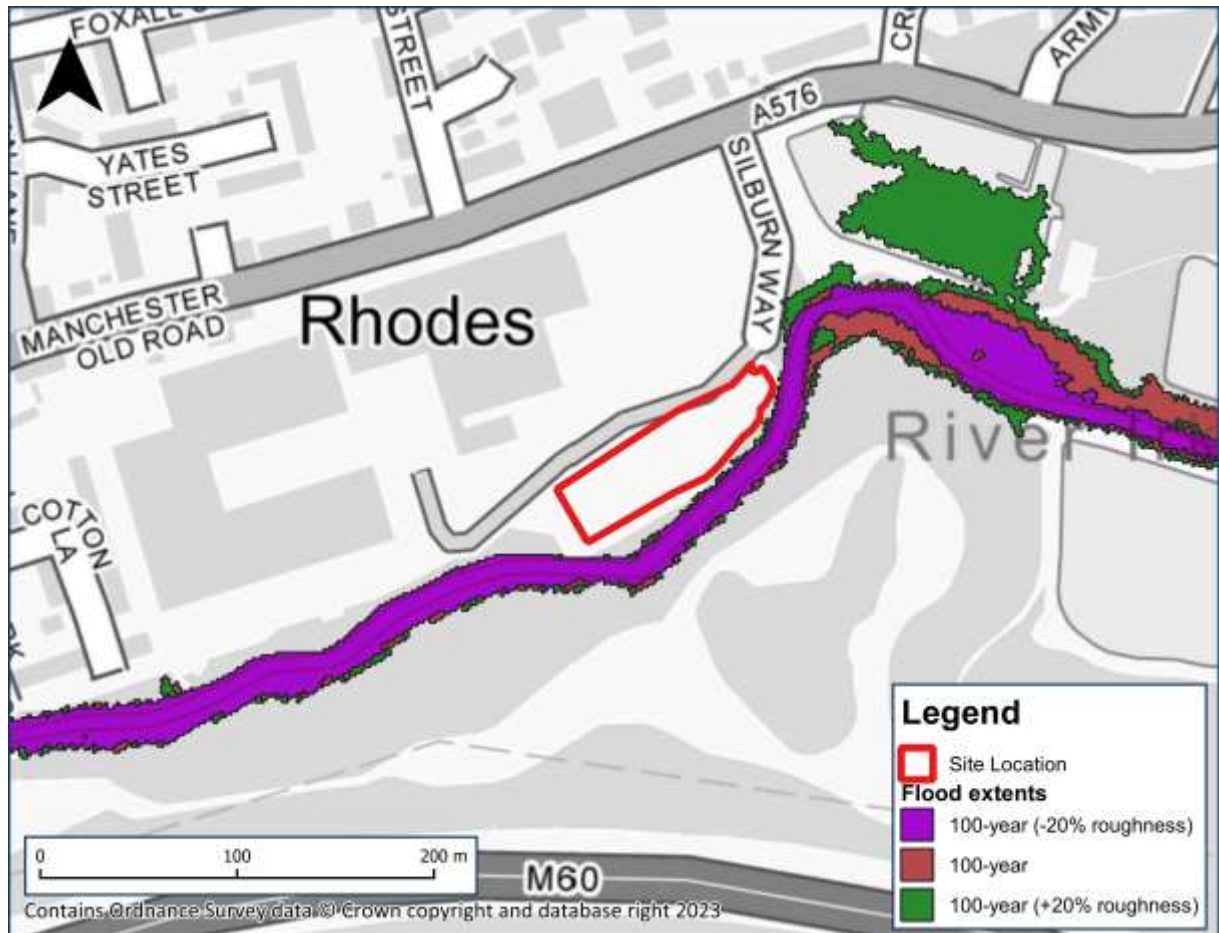


Figure 6-1: Sensitivity analysis +/- 20% roughness

The sensitivity analysis shows that the model is relatively sensitive to the roughness applied to the 1D and 2D domains. However, there are no changes in peak flood extents within the site boundary when roughness is adjusted by +/-20%.

E.2

Sensitivity to downstream boundary

Figure 6-2 shows the changes in peak flood extents compared to the baseline 100-year event when a 250mm increase in gradient to the 1D and 2D downstream boundary.

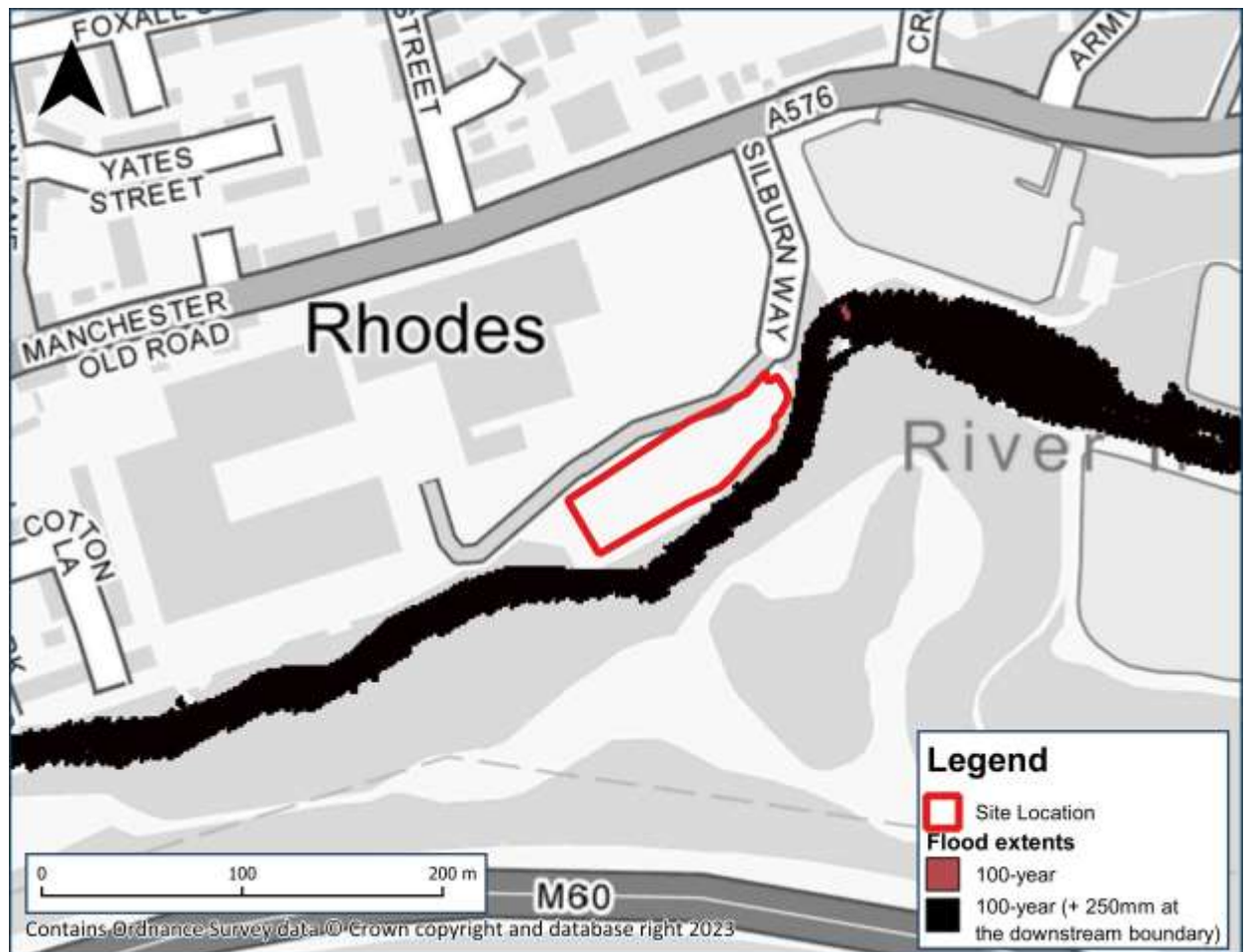


Figure 6-2 Sensitivity analysis for +250mm in gradient at the downstream boundary

Figure 6-2 model results show there is negligible impact on peak flood events within the site boundary when an increase in the downstream boundary is applied, with the downstream boundary far enough downstream of the site to not have a notable impact on the model results within the vicinity of the site.

E.3

Sensitivity to flow

Figure 6-3 shows the changes in peak flood extent compared to the baseline 100-year event when the 1D flow is adjusted by 20%.

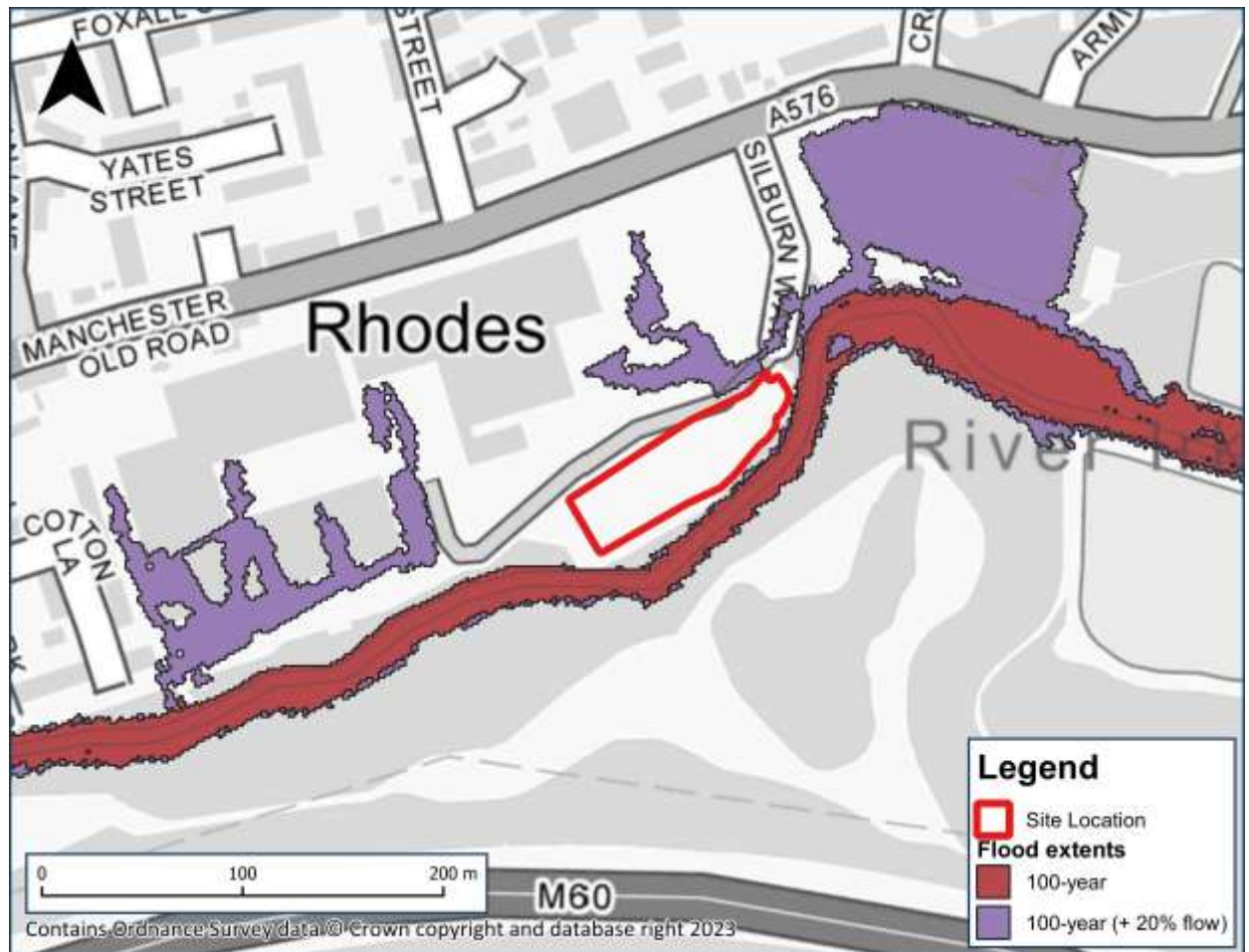


Figure 6-3: Sensitivity analysis for +20% flow

The sensitivity analysis shows that the model is relatively sensitive to flow values. However, there are no changes in peak flood extents within the site boundary when flow is adjusted by +20%.

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United Kingdom

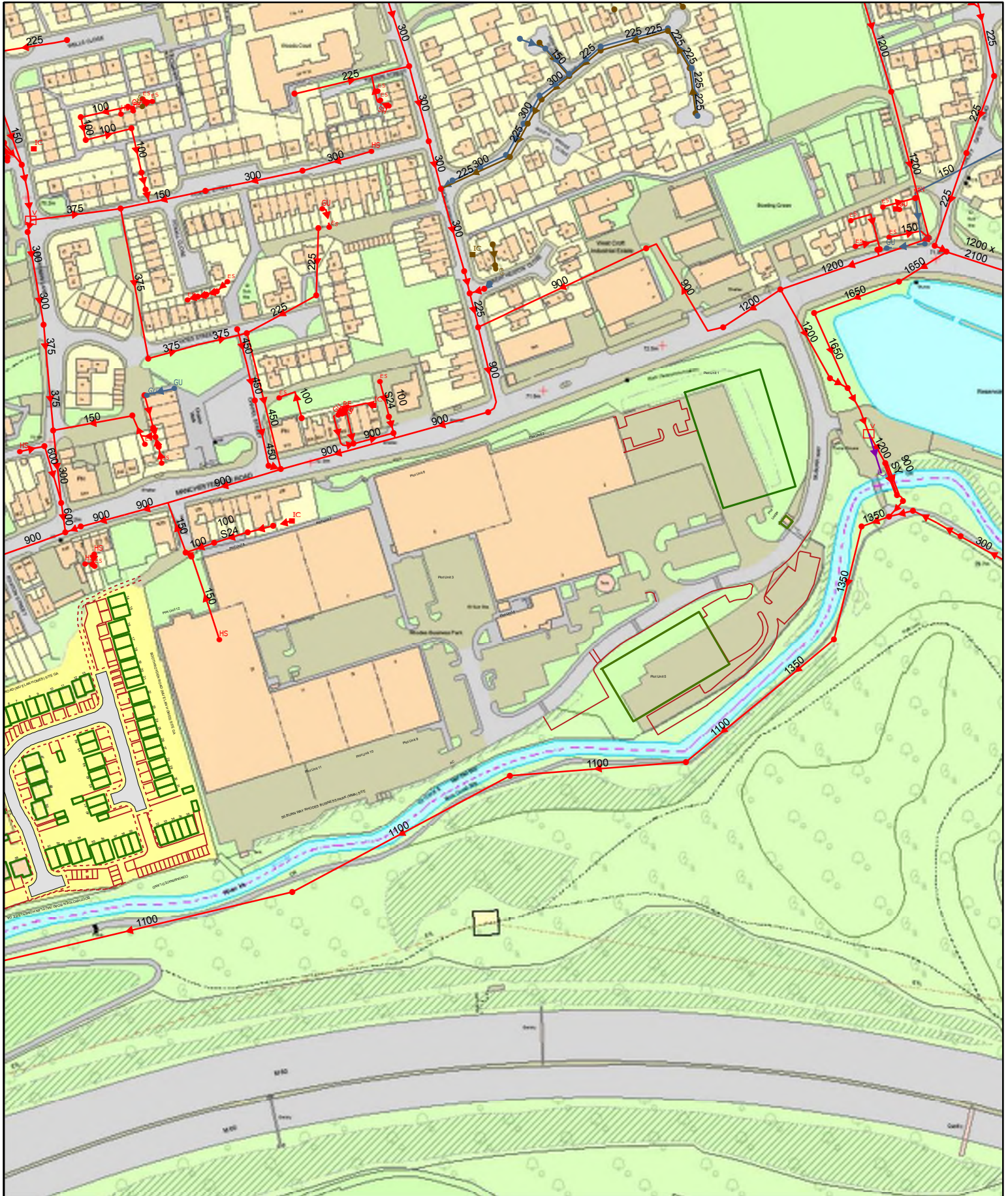
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APPENDIX C – UNITED UTILITIES SEWER MAPS



Date: 08/10/2021

Printed By:
Property Searches

Extract from Map of Public Sewers

Unit 4, Rhodes Business Park Silburn Way, Middleton,
MANCHESTER, M24 4NE



The position of underground apparatus shown on this plan is approximate only and is given in accordance with the best information currently available. The actual positions may be different from those shown on the plan and private pipes, sewers or drains may not be recorded. United Utilities Water PLC will not accept any liability for any damage caused by the actual positions being different from those shown.

Wastewater Symbolology

Abandoned	Foul	Surface Water	Combined	
				Public Sewer
				Private Sewer
				Section 104
				Rising Main
				Sludge Main
				Overflow
				Water Course
				Highway Drain

All point assets follow the standard colour convention: **red** – combined **brown** - foul
blue – surface water **purple** - overflow

Manhole	Side Entry Manhole
Head of System	Outfall
Extent of Survey	Screen Chamber
Rodding Eye	Inspection Chamber
Inlet	Bifurcation Chamber
Discharge Point	Lamp Hole
Vortex	T Junction / Saddle
Penstock	Catchpit
Washout Chamber	Valve Chamber
Valve	Vent Column
Air Valve	Vortex Chamber
Non Return Valve	Penstock Chamber
Soakaway	Network Storage Tank
Gully	Sewer Overflow
Cascade	Ww Treatment Works
Flow Meter	Ww Pumping Station
Hatch Box	Septic Tank
Oil Interceptor	Control Kiosk
Summit	
Drop Shaft	Change of Characteristic
Orifice Plate	

APPENDIX D – LLFA AND EA CORRESPONDENCE

Highways and Engineering Service - Economy and Environment

DRAINAGE CONSULTATION – PLANNING REF. 23/01213/FUL

Consultation Response, on behalf of Head of Highways and Engineering:-

Date of Response: 18th Dec 2023

LOCATION & PROPOSED DEVELOPMENT

Rhodes Business Park, Silburn Way, Middleton, Rochdale, M24 4NE. Erection of 1no. unit (Use Class B2/B8/E(g) with associated service yard, car parking and landscaping.

DOCUMENTATION REVIEWED:

- FRA & Drainage Strategy rpt by Burrows Graham dated 17.10.2023
- Proposed Drainage Plan drg ref. 40055-BGL-XX-XX-DR-C-00250/P1 by Burrows Graham 17.10.2023

DRAINAGE OFFICER COMMENTS

The proposed drainage strategy plan shows that the new development will provide attenuation for the development which increases the impermeable area of the site and connect into the existing pipework that discharges to an existing outfall on the river Irk.

COMMENTS ON ERRORS WITH BURROWS GRAHAM CALCS

On Existing & Proposed Areas drg 40055-BGL-XX-XX-DR-C-00255 dated by Burrows Graham, The following text is provided:

EXISTING PERMEABLE AREA = 2,165m²

EXISTING IMPERMEABLE AREA = 3,210m²

PROPOSED PERMEABLE AREA = 4,190m² (Wrong, should be 1,185m²)

PROPOSED IMPERMEABLE AREA = 1,185m² (Wrong, should be 4,190m²)

SITE BOUNDARY (Total area = 5,375m²)

Drainage Officer note 1: The Proposed Permeable and Impermeable areas are incorrectly labelled; the proposed impermeable area is greater i.e. 4,190m².

Extract from the FRA & Drainage Strategy Doc by Burrows Graham

5.2 EXISTING DRAINAGE AND FLOW RATES

The site area currently encompasses existing transport and storage operations with hard paved areas which all positively drain unrestricted to the existing Anglian Water surface water sewers bounding the site to the south. As such the site is considered to be partially brownfield for drainage purposes.

The existing surface water flow from the site has been calculated as approximately 46l/s, with flooding on the surface of 29m³ for the impermeable areas currently draining direct to the existing sewer. A drawing of impermeable area is shown in Appendix A.

5.8: The proposed surface water sewer release rate will be 4.5l/s (Q_{Bar}).

6.0 CONCLUSION

....The Sustainable Drainage Systems (SuDS) hierarchy has been considered in the drainage strategy with reduced release rates, permeable pavers, additional proprietary Class 1 separator and a geocellular attenuation tank located beneath the service yard areas....

Drainage Officer note 2: The reference to Anglian Water is an incorrect type; this area is United Utilities!

Drainage Officer note 3: Because there is an error in the areas on the drawing, I have no confidence in the calculation of existing SW flow rate of 46 l/s, nor in the proposed flow rate to stream of 4.5 l/s. My SW calc

indicates that the controlled discharge to the River Irk should be ~15 l/s based upon 50% reduction in current brownfield runoff from paved areas plus greenfield runoff from the site for Qbar (1 in 2.3 year return period).

Drainage Officer note 4: Although the Conclusion of the Drainage Strategy mentions a Class 1 (oil) separator, one is not shown on the Proposed Drainage Plan. Guidance on such issues is that an oil separator is only needed for a much larger carpark and a re-fuelling depot. Thus, although an oil separator is probably not required, the documentation submitted should correlate with that shown on drawings, for consistency.

CONCLUSION

Before I can suggest Drainage Condition(s) for the reasons given by the Drainage Officer above, corrections are required to the provided calcs and the Existing & Proposed Areas drg.

Burrows Graham / Agent can contact me directly to discuss.

Consultation by Stephen McCann

Drainage & Flooding Officer

 01706 924606 / 07552 570262

 stephen.mccann@rochdale.gov.uk

APPENDIX E – MAINTENANCE MANUAL

RHODES BUSINESS PARK, MIDDLETON

TECHNICAL NOTE

DRAINAGE MAINTENANCE PLAN

DOCUMENT CONTROL SHEET

Issued by: Burrows Graham Limited
Buckland House,
Dower Mews,
Berkhamsted, Herts
HP4 2BL

Tel: 01442 508402

Client: Oxenwood LLP

Project: Rhodes Business Park, Silburn Way, Middleton, Manchester, M24 4NE

Title: Maintenance Plan

Status : Planning

Date : 09.10.2023

Prepared By: DV

Checked By: DB

Document Revision Record

Version Number	Date	Revision Details
1	09/10/2023	First Issue

Burrows Graham Limited has prepared this report in accordance with the instructions of the above-named Client for their sole and specific use. Any third parties who may use the information contained herein do so at their own risk.

1 INTRODUCTION

The long-term maintenance of the new drainage infrastructure will be with the owner of the proposed commercial unit, or professional maintenance company appointed by the owner, in line with best practice guidance. Periodic inspections will need to be carried out and clearing of drainage channels, gullies, catchpits and petrol interceptors undertaken. A record of inspections should be kept, an inspection checklist template is available within the CIRIA SuDS Manual (C753) which can be used to support an inspection record.

Typically, the maintenance regime will be in line with the Management and Maintenance Plan as set out below and in line with CIRIA C753 guidance.

Should exceedance flooding occur, access and egress to the development shall be from the main site access off Mauretania Road. Refer to Appendix A for Flow exceedance plan.

The maintenance of SuDS appurtenances will help to minimise the flood risk impact to the development and the surrounding sewer network.

Flood risk to people and property can be managed but it can never be completely removed; a residual risk remains after flood management or mitigation measures have been put in place. This relates to a rainfall event beyond what can be fully quantified.

Maintenance schedule has been prepared to provide methodology for the surface water drainage elements proposed for this development. If any discrepancies from manufacturers guidance are encountered, the proprietary system guidance should be adopted.

The surface water drainage works are composed of the following:

- Permeable paving
- Linear drainage channels, connecting pipe network, manholes & inspection chambers.
- Permeable paving.
- Geocellular attenuation tanks.
- Oil separators.
- Flow control device.

Any man-entry into the system and silt removal should be carried out by trained personnel with adequate personal protective equipment. Approved safety procedures must be followed in accordance with the Health and Safety Act.

This drainage system's maintenance will be the responsibility of the project developer or by an appointed maintenance company, and as such an appropriated maintenance regime should be implemented.

Long-term management practices include monthly sweeping of external paved areas. The sweeping program will remove sand and contaminants directly from paved surfaces before they become mobilised during storm events and transported to the drainage system.

During the winter months, drainage features such as gullies and channels should be cleared of ice, snow, debris or litter.

Sediment/material removal should be undertaken in consultation with the environmental regulator to confirm appropriate protocols; especially where run-off is taken from potentially contaminated areas such as the filter drains and the upstream/downstream chambers.

2 MAINTENANCE STRATEGY

2.1 INSPECTION, MANHOLE AND CATCHPIT CHAMBERS

Access points have been located at the head of each run, at a change in direction and at a change of pipe size in accordance with Building Regulations Part H.

The appropriate health and safety equipment must be used when accessing manholes. Confined space certificates must be held by any personnel entering a manhole and the appropriate permits should be obtained from the Maintenance Manager prior to any access.

2.2 PIPES

Pipes are proprietary products and the materials can vary across the site and as such where used the manufacture's recommendations should be followed. Regardless of the product used, the pipes will be fully compliant with National and local certification.

Pipes are intended to be the main conveyance across the development and where oversized they form the attenuation volume required by the limitation of the discharge rate. They are intended to be dry except for during rainfall events. These have been designed to be self-cleaning where possible for smaller diameter pipes, and for larger diameters the risk is reduced due to the overall pipe size.

Access for maintenance is provided through access chambers and manholes.

Regular inspection and maintenance are important to identify areas which may have been obstructed / clogged and may not be drainage correctly thus exposing the development to a greater level of flood risk.

2.3 FLOW CONTROL DEVICES

Regular inspection and maintenance are important to identify any obstructions to flow control devices. Complex control systems, such as Hydrobrake (or similar), are to be maintained in accordance with the manufacturers recommendations. Orifice plates and manholes are to be kept clear of all debris, and the protection grating cleaned thoroughly.

2.4 CHANNELS & GULLIES

Channels and gullies should be inspected and cleaned in accordance with the manufacturer's details. Channel units can be cleaned through the use of a high-pressure hose; this can be fed into the channel system through access units strategically placed along the channel run. The throat section of channel units should be kept clear at all times to ensure uninterrupted flow of surface water into the drainage channel and any debris within the throat should be removed.

Locking bolts should be replaced and sufficiently tightened, taking care that the bolt heads do not stand above the top surface of the cover or grate. If covers are allowed to move within their frame, this may cause damage to the frame or seating

Maintenance schedule	Required action	Typical frequency
Regular Maintenance	Remove manhole and chamber covers to sewers and flow control on network– inspect to ensure water is flowing freely and that the water flow route is unobstructed. Remove debris and silt as required.	Half Yearly Undertake one of these inspections after leaf fall in autumn
Occasional Maintenance	None required	
Remedial Actions	Re-line or replace pipework if unable to clear blockages by jetting.	As required
Monitoring	Inspect chambers for build-up of silt and debris	Half yearly

Table 5 – Operation & maintenance requirements for pipe network, manholes & inspection chambers, drainage channels & gullies.

2.5 ATTENUATION TANKS

Geocellular attenuation tanks are modular storage systems made with plastic units. These units can be assembled to achieved re required volume and usually on multiple layers. Generally, the units have 95% of voids content and are used to create an efficient below ground structure to store surface water.

The geocellular units and geotextile membrane are proprietary products and therefore manufacturer's recommendations should also be taken into consideration.

Access for maintenance should be provided by an access shaft located on the tank and from the manhole chambers downstream of the tank.

Regular inspection and maintenance are important for the effective operation of attenuation tanks as designed. As the feature is buried, a regular inspection regime is very important to ensure the correction functionality of the surface water drainage network.

Maintenance schedule	Required action	Typical frequency
Regular Maintenance	Inspect and identify any areas that are not operating correctly. If required, take remedial action.	Monthly for 3 months, then annually.
	Remove debris form the catchment surface (where it may cause risks to performance).	Monthly.
	For systems where rainfall infiltrates into the tank from above, check surface of filter for blockages by sediment, algae or other matter; remove and replace surface infiltration medium as required.	Annually.
	Remove sediments from pre-treatment structures and/or internal forebays.	Annually, or as required.
Remedial Actions	Repair/rehabilitate inlets, outlets, overflows and vents.	As required.
Monitoring	Inspect/check all inlets, outlets, vents and overflows to ensure that they are in good condition and operating as designed.	Annually
	Survey inside of tank for sediment build-up and remove if necessary.	Every 5 years or as required.

Table 6 – Operation & maintenance requirements for attenuation storage tanks - Based on CIRIA SuDS Manual 2015

2.6 POROUS PAVED AREA

Permeable surfacing enables any water landing on the surface to infiltrate directly to ground at source, thus acting close to a permeable surface. Regular inspection and maintenance are important for the effective operation of the permeable surfacing to allow it to work efficiently through its design life.

Maintenance schedule	Required action	Typical frequency
Regular Maintenance	Brushing and vacuuming (standard cosmetic sweep over whole surface)	Once a year, after autumn leaf fall, or reduced frequency as required, based on site-specific observations of clogging or manufacturer's recommendations – pay particular attention to areas where water runs onto pervious surfaces from adjacent impermeable areas as this is most likely to collect the most sediment.
Occasional Maintenance	Stabilise and mow contributing and adjacent areas.	As required
	Removal of weeds or management using glyphosate applied directly into the weeds by an applicator rather than spraying	As required – once per year on less frequently used pavements.
Remedial Actions	Remediate any landscaping which, through vegetation maintenance or soil slip, has been raised to within 50mm of the level of the surface.	As required
	Remedial works to any depressions, rutting and cracked considered detrimental to the structural performance or a hazard to users, and replace lost jointing material	As required
	Rehabilitation of surface and upper substructure by remedial sweeping.	Every 10 to 15 years or as required (if infiltration performance is reduced due to significant clogging)
Monitoring	Inspect for evidence of poor operation and/or weed growth -if required, take remedial action	Three-monthly, 48 hours after large storms in the first six months
	Inspect silt accumulation rates and establish appropriate brushing frequencies.	Annually
	Monitor inspection chambers	Annually

Table 7 – Operation & Maintenance requirements for Porous Paved Area – CIRIA SuDS Manual 2015

2.7 PROPRIETARY SURFACE WATER TREATMENT SYSTEMS

These are manufactured products to provide a treatment to the surface water by removing specific pollutant, such as petrol interceptors.

Table 8 sets out the recommended maintenance actions and frequency. Manufacture guidance / recommendations should always be used when available.

Maintenance schedule	Required action	Typical frequency
Regular Maintenance	Remove litter and debris and inspect for sediment, oil and grease accumulation.	Monthly for 3 months, then annually.
	Change the filter media.	As recommended by manufacturer.
	Remove sediment, oil grease and floatables.	As necessary – indicated by system inspections or immediately following significant spill.
Remedial Actions	Replace malfunctioning parts or structures.	As required.
Monitoring	Inspect for evidence of poor operation.	Biannually
	Inspect filter media and establish appropriate replacement frequencies.	Biannually
	Inspect sediment accumulation rates and establish removal frequencies.	Monthly during first half year of operation, then every six months.

Table 8 – Operation & maintenance requirements for proprietary surface water treatment systems - CIRIA SuDS Manual 2015

2.8 SPILLAGES – EMERGENCY ACTION

Most spillages on development sites are of compounds that do not pose a serious risk to the environment if they enter the drainage network at a slow and controlled manner allowing time for natural breakdown in the treatment system. The Environment Agency recommends that small spillages should be removed, when possible, using soak mats and residual spillage can be allowed to bio-remediate within the drainage system.

Should a serious spillage occur, either by volume or of unknown/toxic compound, isolate the spillage with soil, turn or fabric and block outlet pipes from chamber(s) downstream of the spillage with bung(s).

The Environment Agency must be contacted immediately.

The owner of the development should be contacted in the event of a concern or failure of a SuDS design Feature.