

NXQ

Lever Street, Manchester

Drainage Assessment

Revision 01: 27.03.2024



SOLLER



4891 - The NXQ Building, Lever Street
Drainage Assessment
February 2024

Consulting Structural and Civil Engineers

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Contents

1	Introduction	3	8	Maintenance and operation	12
2	Executive Summary	4	8.1	General drainage	12
			8.2	Pumped systems	12
3	The Site	5	8.3	Rainwater Harvesting	13
3.1	Site Location	5	8.4	Attenuation tanks	13
3.2	Site Description	5	Appendices		
3.2.1	Topography	5	1	Topographic survey	
3.2.2	Geology	5	2	Calculations	
3.2.3	Groundwater	6	3	SuDS Option Selection	
3.2.4	Source Protection zones	6	4	Proposed Drawings	
3.3	Development Proposals	6	5	Pre-planning enquiry correspondence with United Utilities	
4	Design Standards	7			
4.1	National Standards	7			
4.2	National Planning Policy Framework (NPPF)	7			
4.3	Local Standards and Guidance	7			
5	Surface water drainage	8			
5.1	Existing scheme	8			
5.1.1	Site Characteristics	8			
5.2	Proposed scheme	9			
5.2.1	Design criteria and modelling results	9			
5.2.2	Water quality and quantity	9			
5.2.3	Exceedance Flows	9			
6	Foul water drainage	10			
6.1	Existing scheme	10			
6.2	Proposed scheme	10			
7	Flood risk assessment requirements	11			

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1 Introduction

AKT II have been commissioned to undertake a foul and surface water drainage strategy in support of the proposed redevelopment of The NXQ Building on behalf of the Client, Soller Nine.

The site area is approximately 1,640 m² of which 390 m² is public footpath and 1,250 m² is building footprint. The site consists of a series of 3-5 storey load bearing masonry buildings with mixed uses as existing, which will be refurbished, part retained, and redeveloped to form new commercial buildings including an 18 storey office development with two levels of basement as proposed.

The site is located at the junctions of Lever Street, Houldsworth Street, Little Lever Street and Great Ancoats Street. The Northings and Eastings for the development land are 398588, 384726 respectively.

The topography of the development land is broadly flat with the eastern corner being at 49.3 m AOD and the western corner being at 48.9 m AOD.

The presence of ground water has not been recorded in adjacent site investigations.

This report is intended to cover only the aspects of drainage design and provide the guidelines and parameters for the detailed design at later/more advanced stages of the project.

This report has been prepared in accordance with the guidance contained in the National Policy Framework (NPPF) and the accompanying Planning Practice guidance and a more detailed explanation of the requirements nationally and locally are laid out in Section 4 of this report.

The following drainage report and strategy will need to be approved by the Lead Local Flood Authority (LLFA) to obtain planning approval and satisfy the relevant consultees (the local Water Authority and the Environment Agency, where relevant)



Figure 1.1 Site - North-east corner



Figure 1.2 Site - North-west corner

2 Executive Summary

SCOPE	
Purpose of the report	To highlight the surface and foul water drainage strategy to inform the preparation of the planning stage for the proposed development.
Current Site status	A series of 3-5 storey load bearing masonry buildings with mixed uses.
Future Site end-use	Primarily office use with a small food and beverage offering at ground floor.
Development Proposal	Refurbishment with part retained and part demolish / rebuild.
SITE INFORMATION	
National Grid Reference	384726 E, 398588 N
Site Area	Approximately 1,640 m² of which 390 m² is public footpath and 1,250 m² is the building footprint.
History	According to the existing records since 1848, it seems the site has been within an urban environment surrounded by buildings.
Geology	Published British Geological Survey maps and BGS borehole records indicate that the soil underlying the site should comprise of a Till / Devensian layer and a bedrock of Chester Formation.
Hydrogeology	<p>The Environment Agency’s new aquifer designation map indicates that the site is underlain by a superficial Till / Devensian layer which is classified as Secondary undifferentiated and Chester Formation bedrock which is classified as a Principal Aquifer.</p> <p>The site is not located within an Environment Agency designated source protection zone (SPZ).</p> <p>The groundwater level will be confirmed during a site-specific ground investigation.</p>
Hydrology	<p>The nearest water features are the Rochdale and Ashton Canals approximately 400m to the south east and the River Irwell approximately 800m to the north west.</p> <p>The Environment Agency maps indicate that the site has a limited potential for groundwater flooding to occur.</p>
Site constraints	Utility maps indicate a large number of in-use utilities running underground around all four edges of the site and the site also has an existing sewer running through it. It is recommended that this sewer is to be surveyed to establish the location, diameter and soffit levels of it. Consultation is required with United Utilities to establish whether this sewer can be divested. United Utilities must approve any ground works in the vicinity of the sewer.
SURFACE WATER DRAINAGE DESIGN	
Conceptual scheme design	Surface water will be disposed of to the public sewer by gravity at greenfield rate using either an existing connection or a new sewer connection if the existing connection locations do not fit with the proposed layout. An attenuation tank will be provided within the basement and may require to be pumped depending upon the outcome of the survey of the existing sewer network.
FOUL WATER DRAINAGE DESIGN	
Conceptual scheme design	Foul water will be disposed of to the public sewer using either an existing connection or a new sewer connection if the existing connection locations do not fit with the proposed layout. It is assumed that all foul water from ground floor level and above will drain by gravity and that the two basement levels may require to be pumped to high level within the basement before also draining by gravity.

3 The Site

3.1 Site Location

The site is located in the Northern Quarter area of Manchester and comprises a number of properties forming a block bounded by Great Ancoats Street to the east, Little Lever Street to the south, Houldsworth Street to the west and Lever Street to the north. Little Ancoats Street runs through the centre of the site.

The Site is located within Flood Zone 1 which has a low probability of flooding and has a very low risk of surface water flooding. The Site is not located within a conservation area and there are no listed buildings in close proximity to the Site.

The nearest postcode is M1 1FJ and the National Grid reference is SJ 84725 98587.

Figures 3.1 to 3.3 below show the location of the proposed development.

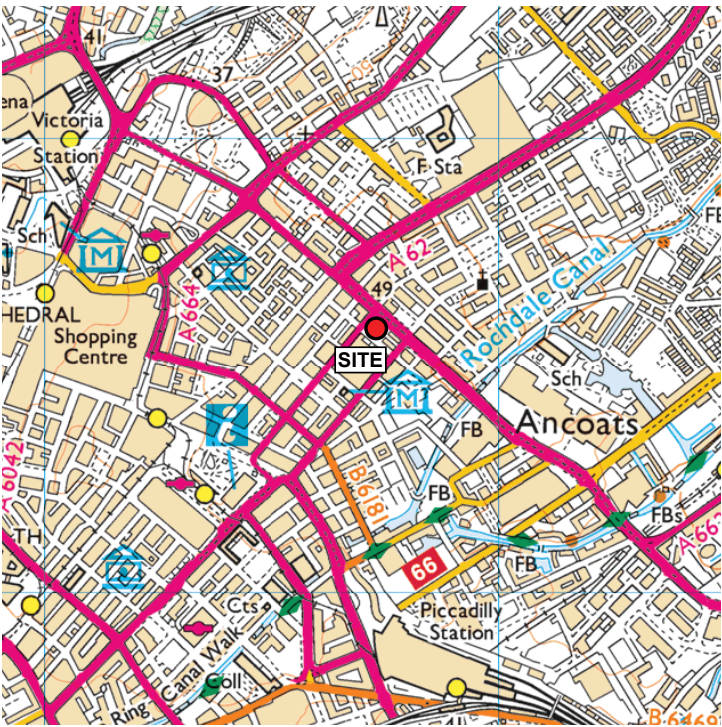


Figure 3.1 Site Location (StreetMap)



Figure 3.2 Site Location (Google Earth)

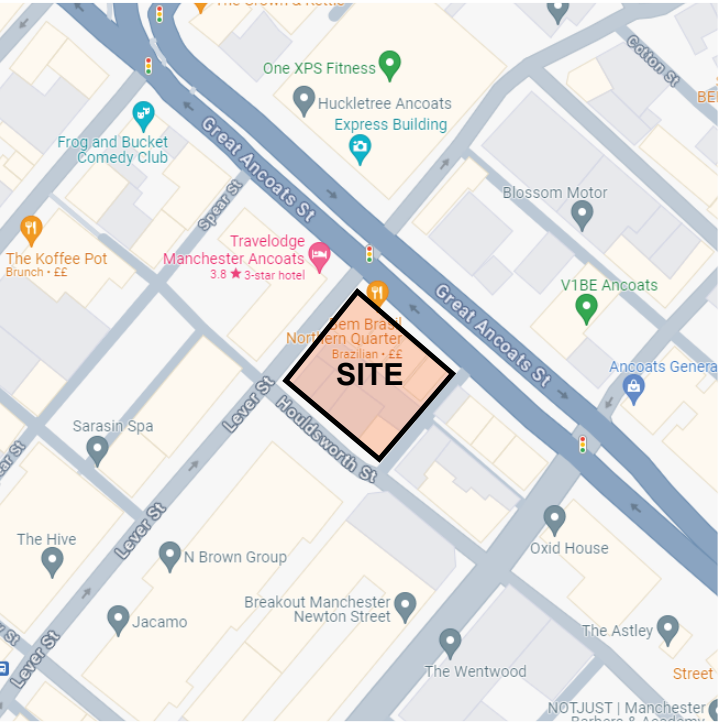


Figure 3.3 Site Location (Google Maps)

3.2 Site Description

There are a number of buildings on the Site as described below:

- **Bem Brasil** - a 3 storey traditional load bearing masonry construction
- **La Rosa** - a 3 storey traditional load bearing masonry construction
- **Golden Tandoori** - 3 storey traditional load bearing masonry construction
- **Houldsworth Street North Units** - typically 5 storey with load bearing masonry facades
- **Little Ancoats Street North Units** - a 4 storey traditional load bearing masonry building with a single storey basement
- **Little Ancoats Street Central units** - one and two storey infill buildings
- **Mob Burgers** - 3 storey traditional load bearing masonry construction with a partial basement below
- **Little Ancoats Street South Units** - a 2 storey traditional load bearing masonry construction

3.2.1 Topography

The site is square with an area of approximately 1,640 m² (40 m x 40 m). Of this, approximately 390 m² is public footpath around the perimeter of the site giving a building footprint of 1,250 m². The site is approximately flat with the eastern corner being at 49.3 m AOD and the western corner being at 48.9 m AOD.

3.2.2 Geology

The following maps related to the Site's geology were received by Envirocheck:

- BGS Superficial Geology Map
- BGS Bedrock Geology Map
- BGS Combined Geology Map (Figure 3.4 below)

The superficial geology map indicates that the Site is underlain by a Till / Devensian layer and the bedrock geology map indicates the Site is underlain by The Chester Formation.

Published British Geological Survey maps and borehole records indicate that the geology underlying the Site comprises Made Ground over stiff Boulder Clay and Red Sandstone.

The superficial Till / Devensian layer is classified as Secondary undifferentiated. Secondary undifferentiated are aquifers where it is not possible to apply either a Secondary A or B definition because of the variable characteristics of the rock type. These have only a minor value.

The Chester Formation bedrock is classified as a Principal Aquifer, as per the Bedrock Aquifer map (Figure 3.5). Principal aquifers are designated by the EA as strategically important rock units that have high permeability and water storage capacity.

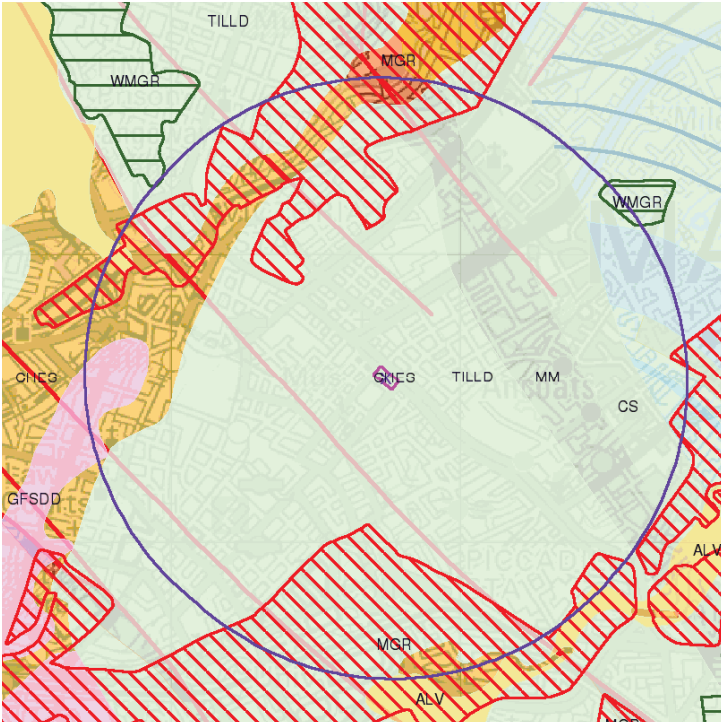


Figure 3.4 BGS Combined Geology Map



Figure 3.5 Bedrock Aquifer Map

3.2.3 Groundwater

Evidence of the likely groundwater level has been obtained from records for adjacent sites where none of the boreholes or trial pits recorded a groundwater strike.

3.2.4 Source Protection zones

The site is not located within a Source Protection Zone (see Figure 3.6).

3.3 Development Proposals

The project involves the Full planning application for part-retention, conversion and extension of existing buildings; selective demolition; the erection of a ground floor + 17 storey building comprising new office space (Use Class E) with ground floor reception area and ancillary uses; supporting infrastructure and ancillary facilities, roof top plant, and landscaping.

- The basement levels will house plant, cycle storage and changing / shower facilities.
- Ground floor to Level 17 will comprise office space.
- Level 18 contains roof plant

The proposed plans are shown in Figures 3.7 to 3.11 for reference.

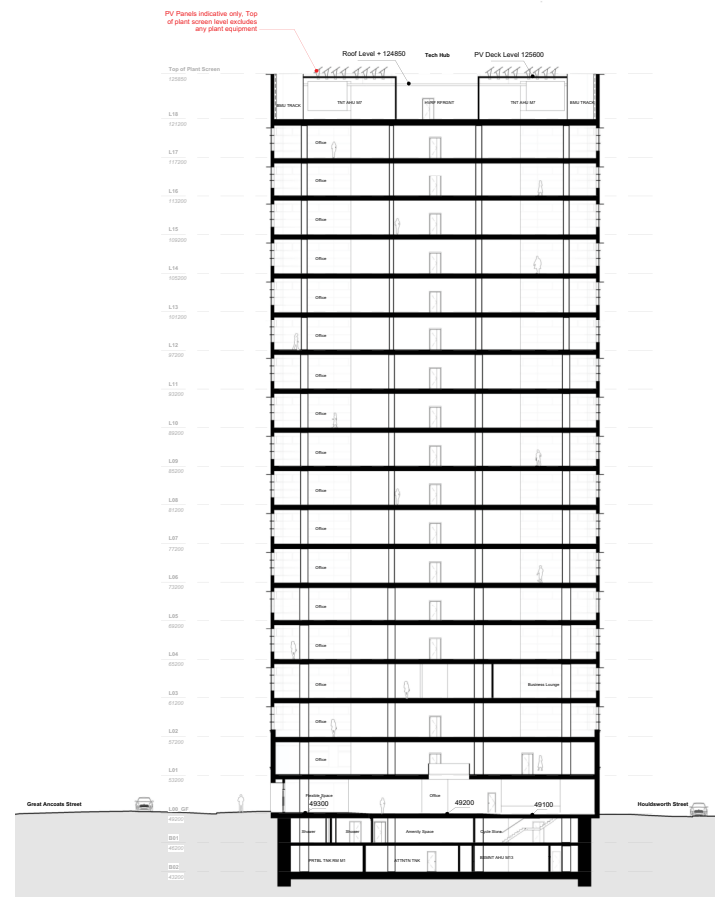


Figure 3.10 Proposed Section (North - South)

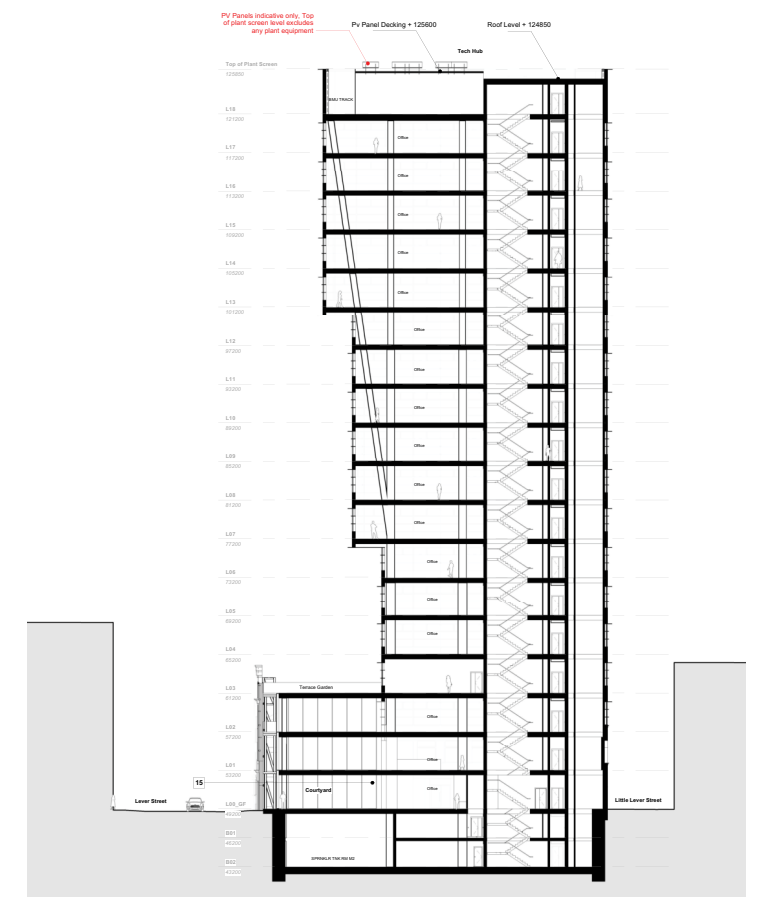


Figure 3.11 Proposed Section (East - West)

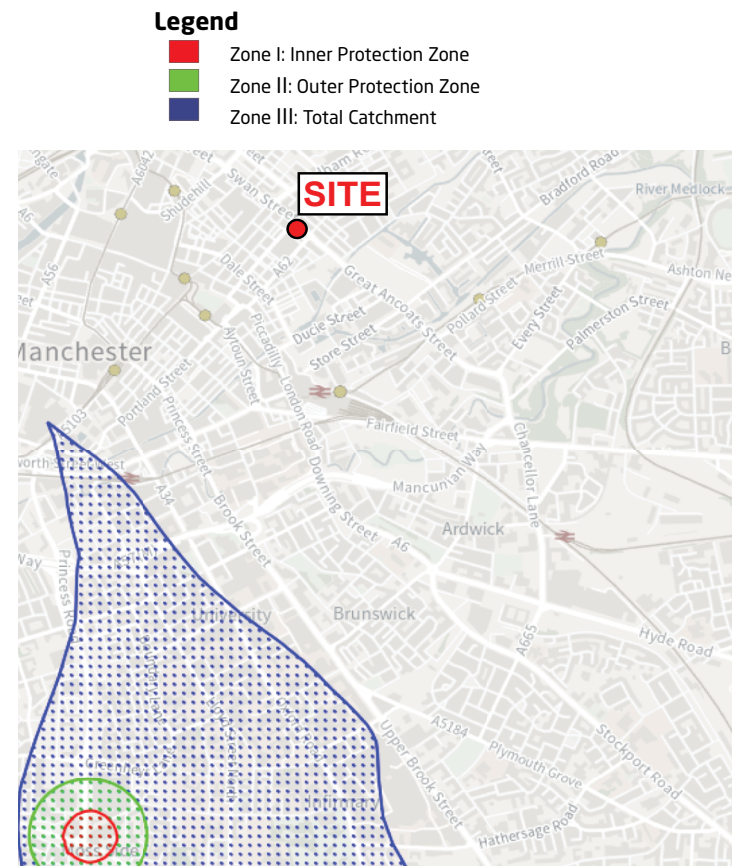


Figure 3.6 Source Protection Zone Map

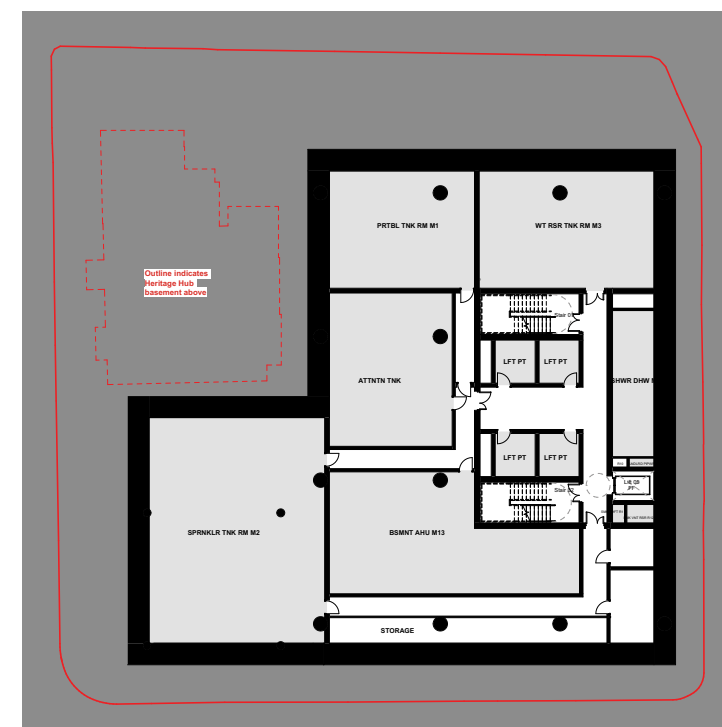


Figure 3.7 Proposed Basement Boz Level

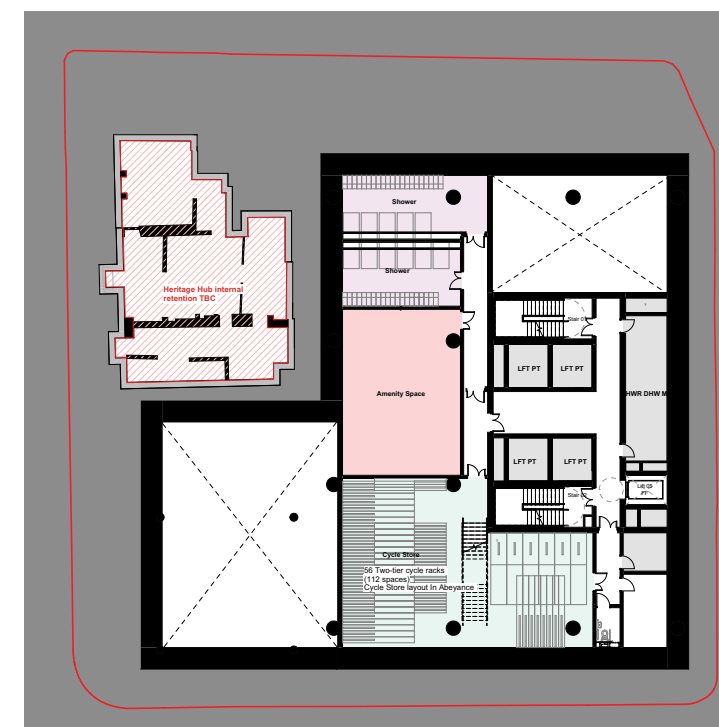


Figure 3.8 Proposed Basement Bo1 Level

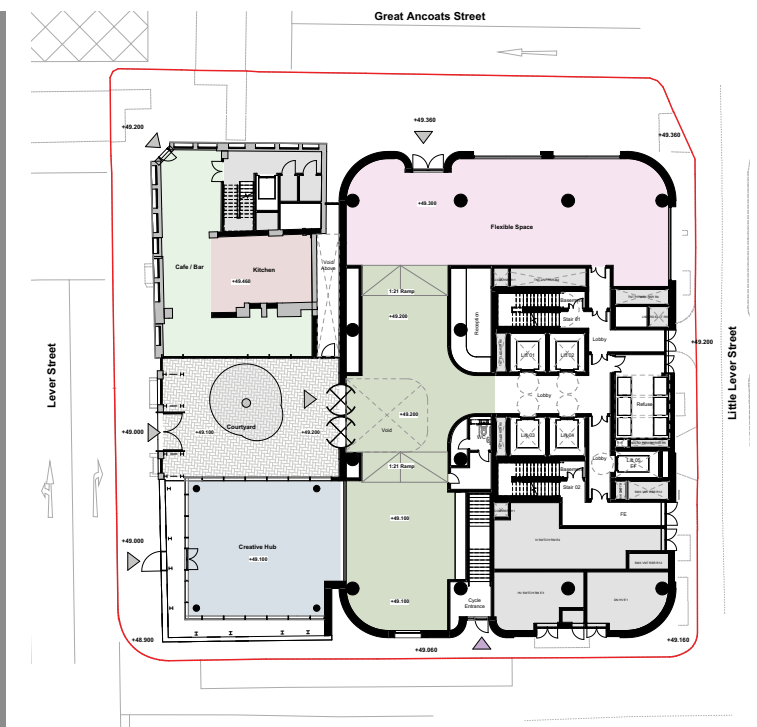


Figure 3.9 Proposed Ground Floor layout

4 Design Standards

4.1 National Standards

The following guides and current British Standards will be used for the design of the drainage elements on this project:

- BS EN 752:2017 Drain and Sewer Systems Outside Buildings. Sewer System Management
- BS EN 12056 Gravity Drainage Systems Inside Buildings: Part 2
- Building Regulations 2010 Part H1 – Foul Water Drainage (2015 Edition)
- Building Regulations 2010 Part H2 – Wastewater Treatment Systems and Cesspools (2015 Edition)
- Building Regulations 2010 Part H3 – Rainwater Drainage (2015 Edition)
- Building Regulations 2010 Part H4 – Building Over Sewers (2015 Edition)
- Building Regulations 2010 Part H5 – Separate Systems of Drainage (2015 Edition)
- Building Regulations 2010 Part H6 – Solid Waste Storage (2015 Edition)
- Environment Agency “Control of Runoff from New Developments Interim Regional Guidance”
- National Planning Policy Framework Planning Practice Guidance

4.2 National Planning Policy Framework (NPPF)

The National Planning Policy Framework (NPPF) superseded Planning Policy Statement 25 “Development and Flood Risk” (PPS 25) although the requirements and goals remain essentially the same:

- The susceptibility of land to flooding is a material planning consideration;
- The Environment Agency has the lead role in providing advice on flood issues, at a strategic level and in relation to planning applications;
- Planning decisions should apply the precautionary principle to the issue of flood risk, using a risk-based search sequence to avoid inappropriate development on undeveloped and undefended flood plains etc;
- Developers should fund flood defences and warning measures required because of the development;
- Planning policies and decisions should recognise that the consideration of flood risk and its management needs to be applied on a whole-catchment basis and not only be restricted to flood plains.

With regard to the NPPF, those proposing particular developments are responsible for:

- Providing an assessment of whether any proposed development is likely to be affected by flooding and whether it will increase flood risk elsewhere and the measures proposed to deal with these effects and risks and;
- Satisfying the local planning authority that any flood risk to the development or additional risk arising from the proposal will be successfully managed with the minimum environmental effect thus ensuring the safe development and secure future occupancy of the site.

After this has been addressed, it is then the local planning authority’s responsibility (advised as necessary by the Environment Agency on fluvial and tidal flood risk, and the LLFA on surface water flood risk) to determine an application for planning permission after taking into account all material considerations, including the issue of flood risk and how it might be managed or mitigated. Local planning authorities are required to adopt a risk-based approach to proposals for development in flood risk areas.

4.3 Local Standards and Guidance

Manchester City Council Lead Local Flood Authority published updated guidance for new developments in 2019. Developers submitting the planning application to Manchester City Council are advised to:

- Assess the suitability of sustainable drainage systems in accordance with: paragraphs 051, 079 and 080 of the revised NPPF Planning Practice Guidance (PPG) for Flood Risk and Coastal Change.
- Design sustainable drainage systems in line with national Non-Statutory Technical Standards for SuDS and local policies DM1, EN08, EN14 and EN17 of the Core Strategy.
- Implement strategy that supports surface water runoff control as near to its source as possible through a sustainable drainage approach to surface water management, preferably through green types of SuDS wherever practicable.
- Maximise use of Green Infrastructure to manage surface water runoff (sustainable drainage systems – SuDS) as feasibility of use of green types of SuDS will be evaluated by LLFA during planning application stage. Even on sites where infiltration is not possible due to the unfavourable soil conditions, utilisation of attenuating types of SuDS should be maximised as part of green infrastructure in order to bring surface water runoff peak and volume reduction but also provide additional environmental and social benefits in line with Policies DM1, EN08, EN14 and EN17 of the Core Strategy. Developers are encouraged to look to CIRIA SuDS Manual 2015 for details of these systems to manage surface water runoff.



Figure 4.1 Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA

- Assess surface water attenuation requirements that offer a reduction in surface water runoff rate in line with the ‘Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA, User Guide, Final, dated May 2010’.
- Submit hydraulic calculations and drawings to support the design along with proposed standards of operation and maintenance in accordance with paragraph 081 of NPF (PPG).
- It will be essential that the type of sustainable drainage system for a site, along with details of its extent/position, is identified at the design stage of the whole scheme. This information will be required for both outline and full applications so it is clearly demonstrated that the SuDS can be accommodated within the development that is proposed. It will no longer be acceptable to leave the design of SuDS to a later stage to be dealt with by planning conditions.
- Evidence that drainage hierarchy has been applied in line with NPPF PPG Paragraph 080. The aim should be to discharge surface run off as high up the following hierarchy of drainage options as reasonably practicable:
 1. into the ground (infiltration);
 2. to a surface water body;
 3. to a surface water sewer, highway drain, or another drainage system;
 4. to a combined sewer.
- Evidence that one viable solution for draining the site is secured and agreed in principle with relevant authority:
- Results of ground investigation carried out under Building Research Establishment Digest 365. If no ground investigations are possible or infiltration is not feasible on site, evidence of alternative surface water disposal routes is required.
- Where surface water is connected to the public sewer, agreement in principle from United Utilities is required that there is adequate spare capacity in the existing system taking future development requirements into account.
- For Brownfield site developments, for all sites located within Conurbation Core Critical Drainage Areas, details of surface water attenuation that offers a reduction in surface water runoff rate in line with the Manchester Trafford and Salford Strategic Flood Risk Assessment, i.e. at least a 50% reduction in runoff rate compared to the existing rates with an aim to reduce to Greenfield runoff rates wherever practicable;
- Evidence that the drainage system has been designed (unless an area is designated to hold and/or convey water as part of the design) so that flooding does not occur during a 1 in 100 year rainfall event in any part of a building; The NPPF suggests that ‘for events with a return-period in excess of 30 years, surface flooding of open spaces such as landscaped areas or car parks is acceptable for short periods, but the layout and landscaping of the site should aim to route water away from any vulnerable property, and avoid creating hazards to access and egress routes. No flooding of property should occur as a result of a one in 100 year storm event (including an appropriate allowance for climate change)’.

5 Surface water drainage

5.1 Existing scheme

The available United Utilities record plans indicate that the closest surface water or combined public sewers to the site are:

- A 1550 mm dia. combined sewer running south east to north west and a 100 mm dia. combined sewer running north west to south east in Great Ancoats Street to the east of the site.
- A 600 mm dia. combined sewer running north east to south west in Lever Street to the north west of the site.
- A 600 mm dia. combined sewer running south east to north west in Houldsworth Street to the south west of the site.
- A 600 mm combined sewer running north east to south west in Little Lever Street to the south east of the site.
- A sewer of unspecified size running north west to south east under Little Ancoats Street within the site.

An extract from the record plans is shown Figure 5.1 for reference.

It is believed that all surface water from the building currently discharges directly to one of these public sewers without any form of attenuation but it is not clear which one and it is therefore recommended that a CCTV survey of the existing site drainage network is undertaken to confirm the location and size of all existing connections from the site.

5.1.1 Site Characteristics

- Total site area = 1,250 m². This excludes the external paving to the public highway.
- Existing impermeable area = 1,250 m²
- Percentage Impermeable (PIMP) = 100%
- M5-60min = 18mm
- Ratio r = 0.45
- IH 124 Soil Type = 4

Detailed calculations are contained in Appendix B for reference. The discharge rates and volumes are summarised below.

Return Period	Existing peak runoff rate ¹	Greenfield Run Off Rate ²	Existing Discharge Volume ³
1 year	15.4 litres/sec	0.63 litres/sec	24.4 m³
30 year	31.9 litres/sec	1.24 litres/sec	52.2 m³
100 year	41.3 litres/sec	1.51 litres/sec	68.3 m³
100 year (+40%)	57.9 litres/sec	2.12 litres/sec	95.6 m³

¹ The existing run off rates were determined by the Wallingford Rational Method with a rainstorm of 15 minutes duration

² Calculated based on EA "Rainfall runoff management for developments"

³ Based on BS 8582 Section 9.8

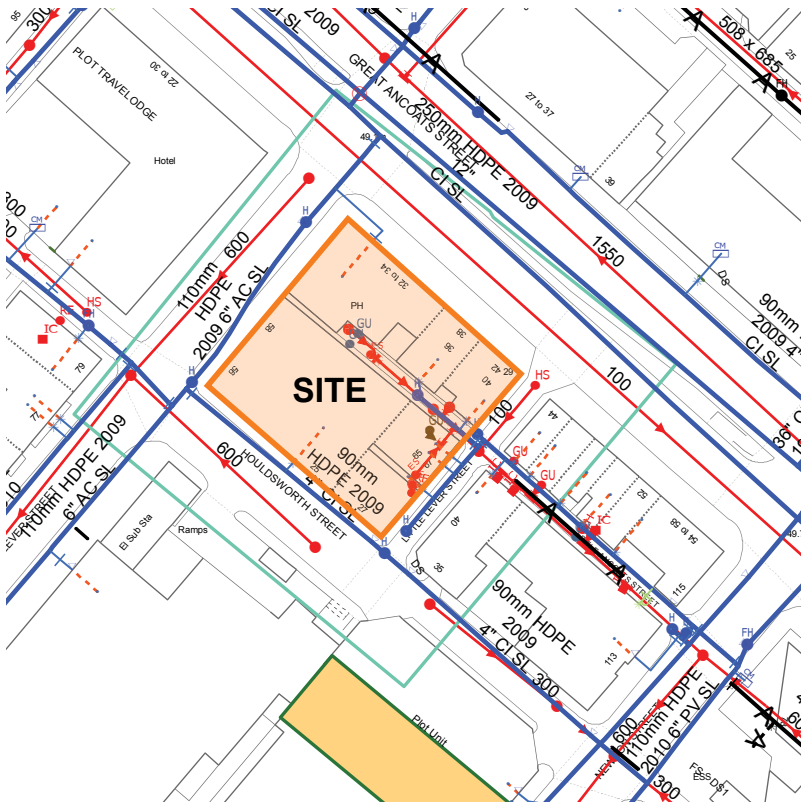


Figure 5.1 United Utilities Sewer Record

5.2 Proposed scheme

The philosophy for the proposed SUDS strategy is to provide an attenuation volume of 65 m³ within the basement which will limit the peak discharge rate to greenfield.

The proposed drainage strategy drawings are contained in Appendix D for reference.

The SuDS selection process undertaken is outlined in detail in Appendix C and the proposed SuDS devices are summarised in Figure 5.2 below.

5.2.1 Design criteria and modelling results

- Proposed impermeable area = 1,250 m²
- Percentage Impermeable (PIMP) = 100%
- Climate change allowance = 40% ("Upper Limit" of the DEFRA guidance (refer to Figure 5.3 below)
- Standard Annual Average Rainfall (SAAR) = 890 mm
- Volumetric runoff coefficient C_v (summer) = 1.00
- Volumetric runoff coefficient C_v (winter) = 1.00
- Hydrological Growth Curve Region = 10 (North West England)

The Microdrainage Simulation and Source Control results demonstrate that all water is contained within the proposed drainage system for every event up to and including the 1 in 100 year plus 40% climate change allowance event.

Detailed calculations for the proposed run off rates and discharge volumes are contained in Appendix B for reference and are summarised below.

Return Period	Existing Peak run off rate¹ (litres/sec)	Proposed scheme unattenuated peak run off rate (litres/sec)	Greenfield run off rate (litres/sec)	Proposed Design Peak run off rate (litres/sec)	Existing Discharge Volume (m³)	Proposed Discharge Volume (m³)
1 in 1 year	15.4	15.4	0.63	0.60	24.4	24.4
1 in 30 years	31.9	31.9	1.24	0.90	52.2	52.2
1 in 100 years	41.3	41.3	1.51	1.00	68.3	68.3
1 in 100 years (+40%)	57.9	57.9	2.12	1.00	95.6	95.6

¹ The existing run off rates were determined by the Wallingford Rational Method with a rainstorm of 15 minutes duration and an assumed runoff of 100% for impermeable areas and 0% for permeable areas

² Calculated based on EA "Rainfall runoff management for developments"

³ calculated using a Microdrainage Source Control model

⁴ Based on BS 8582 Section 9.8

5.2.2 Water quality and quantity

The SuDS features set out in this strategy will help to intercept the majority of the small rainfall events so there should be almost no runoff generated by the first 5mm of rainfall. There will be no runoff from the site for all events up to, and including, the 1 in 100 year plus climate change.

The proposed uses of the site along with the SuDS measures proposed will ensure that water leaving the site through infiltration to the ground or by the piped system to the sewer will be of good quality. Although the site is close to a Source Protection Zone, the necessary measures have been undertaken to ensure that groundwater will be protected from any potential pollution.

The proposed drainage strategy will provide a greater than 97% reduction in the peak flow rate of surface water leaving the site and will therefore reduce the potential risk to the site itself and also the surrounding area from overland flow or sewer flooding.

5.2.3 Exceedance Flows

In the unlikely event that the capacity of the surface water system is exceeded, water will leave the site via overland flow. The exceedance route for any overland flow generated by the site would beto the north and west in line with the flow directions indicated on the EA surface water flooding maps. These routes are shown in Figure 5.4 below.

No exceedance flows will enter the site from external sources. The primary exceedance route for flows generated outside the site is along the eastern channel of Great Ancoats Street and west along Spear Street and Oldham Street.

In addition, the proposed drainage strategy results in the potential for an exceedance event and the exceedance volumes generated by the site being greatly reduced.

Element	Management stage	Water quantity	Water quality	Amenity & biodiversity	Proposed in scheme
Rainwater harvesting	Prevention	✓	✗	✗	✗
Green/brown / blue roof	Source control	✓	✓	✓	✓ / ✗
Raingardens	Source control	✓	✓	✓	✗
Bio-retention	Source control	✓	✓	✓	✗
Permeable surfacing	Source control	✓	✓	✗	✗
Swales	Source control	✓	✓	✓	✗
Detention basin / ponds	Source control	✓	✓	✓	✗
Discharge to tidal river / dock / canals	Site control	✓	✗	✗	✗
Storage tanks / Geocellular storage	Site control	✓	✗	✗	✓
Oversized piping	Site control	✓	✗	✗	✗
Design for exceedance	Site control	✓	✗	✗	✗

Figure 5.2 Summary of proposed SuDS devices

Range	Total potential change anticipated for 2010-2039	Total potential change anticipated for 2040-2059	Total potential change anticipated for 2060-2115
Upper end	10%	20%	40%
Central	5%	10%	20%

Figure 5.3 Peak rainfall intensity climate change allowance

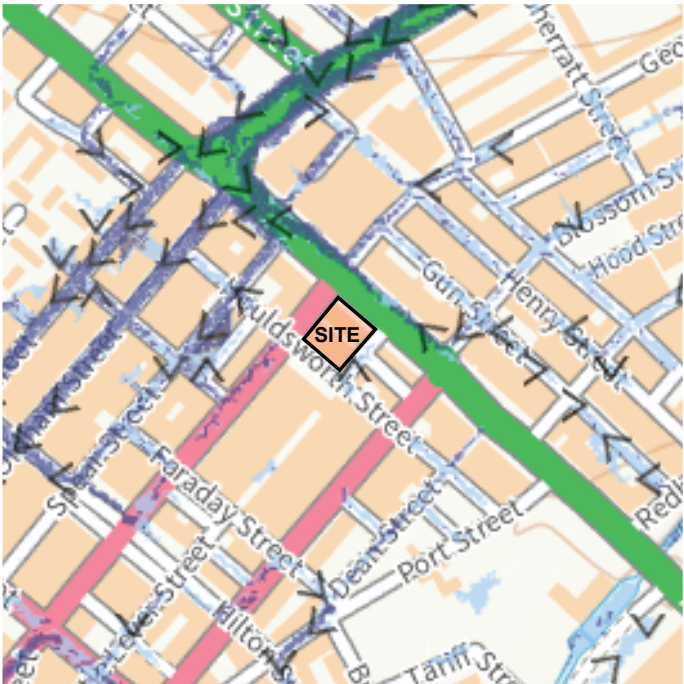


Figure 5.4 Exceedance flow routes

6 Foul water drainage

6.1 Existing scheme

The available United Utilities record plans indicate that the closest foul water or combined public sewers to the site are:

- A 1550 mm dia. combined sewer running south east to north west and a 100 mm dia. combined sewer running north west to south east in Great Ancoats Street to the east of the site.
- A 600 mm dia. combined sewer running north east to south west in Lever Street to the north west of the site.
- A 600 mm dia. combined sewer running south east to north west in Houldsworth Street to the south west of the site.
- A 600 mm combined sewer running north east to south west in Little Lever Street to the south east of the site.
- A sewer of unspecified size running north west to south east under Little Ancoats Street within the site.

An extract from the record plans is shown in Figure 5.1 for reference.

It is believed that all foul water from the building currently discharges directly to one of these public sewers but it is not clear which one and it is therefore recommended that a CCTV survey of the existing site drainage network is undertaken to confirm the location and size of all existing connections from the site.

Surveyed floor plans are available for the existing building which give a partial indication of the number of foul appliances such as sinks and toilets.

The existing foul water discharge rate from the site has been calculated as **4.0 litres/sec** (detailed calculations are contained in Scetion B.1 of Appendix B for reference).

6.2 Proposed scheme

The proposed foul water discharge rate from the site has been calculated as **10 litres/sec** (detailed calculations are contained in Scetion B.2 of Appendix B for reference).

This represents an increase on the existing foul water flow. Any increase will need to be agreed with United Utilities and their written approval of this will be required once a more detailed assessment can be made of the existing and proposed situations.

It is assumed that any foul water drainage from ground floor level and above will be drained by gravity in order to minimise the amount of pumping required. Until the levels of the existing connections and public sewers are confirmed it is unclear whether the basement level will need to be pumped although this appears to be likely. It is therefore recommended that an allowance is made at this stage for pumping foul water from below the basement level slab up to high level in the basement to allow it to discharge by gravity to the public sewer.

As with the surface water drainage, due to the depth of the public sewers it is recommended that, if possible, the existing drainage connection(s) should be reused depending upon the findings of the CCTV survey.

7 Sewer Divestment

The available United Utilities record plans indicate that there is a combined sewer running within the site under Little Ancoats Street (Figure 7.1). It appears that this sewer serves only the development and therefore there should be no issue with removing and divesting this section. However, a CCTV survey of the sewer will be required to confirm all incoming connections and United Utilities approval of the divestment will be required.

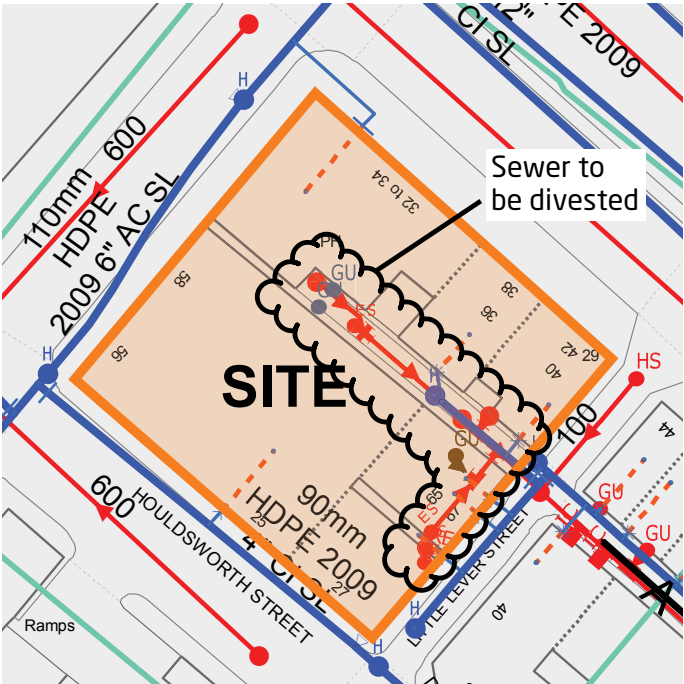


Figure 7.1 Sewer to be divested

8 Flood risk assessment requirements

The Environment Agency's Indicative Floodplain Map (see Figure 8.1) shows that the site lies in Zone 1 - an area with a low probability of flooding. As the site is also less than 1 ha in area, there would be no requirement for a Flood Risk Assessment in accordance with the National Planning Policy Framework and, as there is no increase in the hardstanding area on site, there is no increase in flood risk from the site to other properties.

AKT II have undertaken a site specific flood risk assessment for the redevelopment and the main findings of this assessment are summarised below:

- In accordance with the National Planning Policy Framework, we would categorise the site as lying entirely within Flood Zone 1 - an area assessed as having a less than 1 in 1000 annual probability of river or sea and river flooding (<0.1%).
- In accordance with the NPPF, all proposed land uses are acceptable within Flood Zone 1 and therefore the site passes the Sequential Test.
- The site has been assessed as being at **very low risk** of flooding from rivers or tidal sources.
- The site has been assessed as being at **low risk** from surcharging sewers.
- The site has been assessed as being at **low risk** from groundwater sources.
- The site has been assessed as being at **very low risk** from artificial sources.
- The site has been assessed as being at **very low risk** from surface water flooding sources.
- The proposed redevelopment has an acceptable flood risk within the terms and requirements of the National Planning Policy Framework.

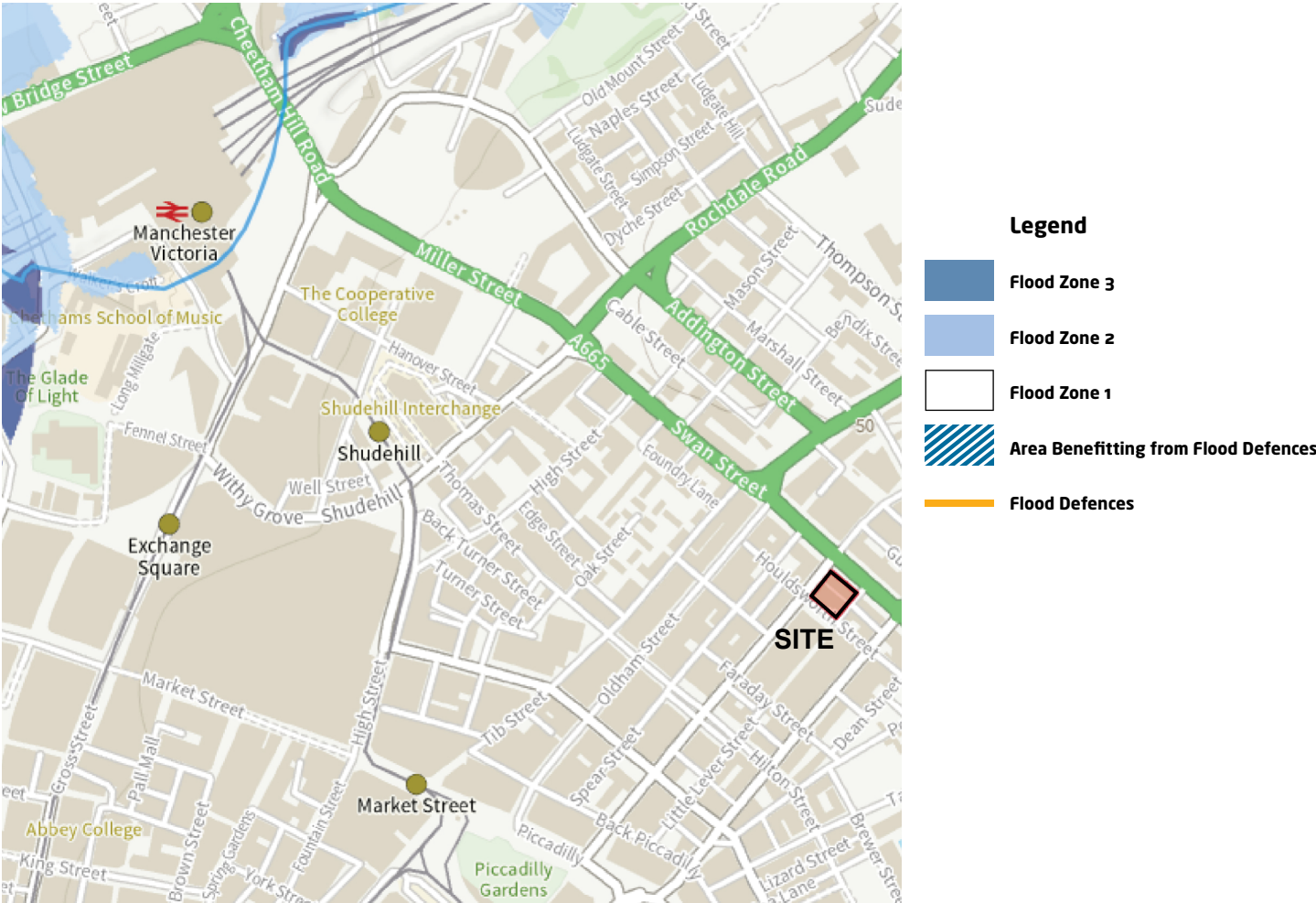


Figure 8.1 Environment Agency indicative flood map

9 Maintenance and operation

Before cleaning, final testing and immediately before handover the Contractor will:

- Lift covers to manholes, inspection chambers and access points. Remove mortar droppings, debris and loose wrappings.
- Thoroughly flush pipelines with water to remove silt and check for blockages. Rod pipelines between access points if there is any indication that they may be obstructed.
- Carry out a CCTV of the pipework to ensure that it is free of silt and blockages.

The End User shall then follow the “Waste Management, The Duty of Care - A Code of Practice (Revised 1996)” and shall ensure that their waste does not escape from their control and is transferred only to a registered waste carrier to be sent for recycling or disposal at a suitably licensed facility.

All waste arising from the maintenance of the drains and sewers shall be handled, stored and disposed of correctly to avoid pollution. Waste may be designated as hazardous / special waste and, as such, the End User shall ensure that they comply with the Hazardous Waste (England and Wales) Regulations 2005.

Reference shall be made to CIRIA publication C753 - The SuDS Manual by the Contractor and the End User. A suitable maintenance schedule must be developed, maintained, followed and updated as required to reflect observed performance. The following items are highlighted for guidance.

9.1 General drainage

The below ground drainage network has been designed in accordance with the requirements of the Building Regulations whilst acknowledging the need to limit the number of inspection chambers within “front of house” areas. To this end, all main runs have rodding eyes, manholes or inspection chambers at the head of the run and at all changes of direction to provide access to rod or jet the main pipework.

Where possible, connections from stacks or gullies have been made directly to these manholes or inspection chambers to allow the connection to be rodded or jetted from the downstream end. Where this is not possible, each stack has been detailed to have an access hatch provided just above floor level (see Figure 9.1) to allow the connection to be rodded or jetted from the upstream end. Similarly, the gullies have a rodding access provided within their body allowing the pipework to be rodded or jetted from the gully downstream.

Gullies and channels have been specified with silt buckets and silt trap manholes have been provided upstream of all tanks and infiltration structures to prevent the ingress of silts into the drainage network and impairing the performance of the system.

Maintenance schedule	Required action	Recorded frequency
Regular maintenance	Inspect and identify areas that are not operating correctly. If required, take remedial action.	Monthly for the first three months then six-monthly
	Remove sediment from pre-treatment structures (e.g. gullies, channels, silt traps).	Six-monthly or as required
Occasional maintenance	Debris removal from catchment surface where this may cause risks to performance.	Monthly
Remedial actions	Repair / rehabilitation of inlets, outlets, overflows and vents.	As required
Monitoring	Inspect all manholes, inspection chambers, inlets, outlets, overflows and vents to ensure they are in good condition and operating as designed.	Annually and after large storms

9.2 Pumped systems

Pumps have been designed as duplex units operating on a duty / standby based on hours run, pump failure and high / high water level. A suitable BMS interface shall be provided monitoring each pump system for the following status points:

- Pump 1 running / Pump 2 running - These statuses shall be provided to the BMS in the form of a volt free contact that is closed when the pump is running.
- Pump 1 failed / Pump 2 failed - These statuses shall be provided to the BMS in the form of a volt free contact that is closed when the pump has deemed to have failed, i.e. failed to run when requested. This shall cause a latched general alarm on the BMS.
- High water level - This status shall be provided to the BMS in the form of a volt free contact that is closed when a high water level is breached. The level shall set at a level that is higher than the normal pump control level switch. This shall cause a latched general alarm on the BMS.
- High / High water level - This status shall be provided to the BMS in the form of a volt free contact that is closed when a high / high water level is breached. The level shall set at a level that is higher than the high water level switch. This shall cause a critical latched alarm on the BMS.
- System not in automatic / not available - This status shall be provided to the BMS in the form of a volt free contact that is open (failsafe) when the system is not available to operate. This shall operate should any event occur that could prevent the system from operating, such as power loss to the control panel, hand / off / auto switches not in Auto, isolators opened. This shall cause a critical latched alarm on the BMS.

The BMS shall be capable of raising the following alarms:

- Excessive Pump Running Alarm - The BMS shall monitor the running status of each pump. Should any pump run for longer than 20 minutes, a general alarm shall be raised on the BMS.
- Excessive Pump Starts Alarm - The BMS shall calculate from the running status the number of starts per hour. Should the number of starts per hour exceed 4, a general alarm shall be raised on the BMS.

A control panel local to each pump station shall be provided to monitor the same status points and alarms as defined for the BMS Interface above.

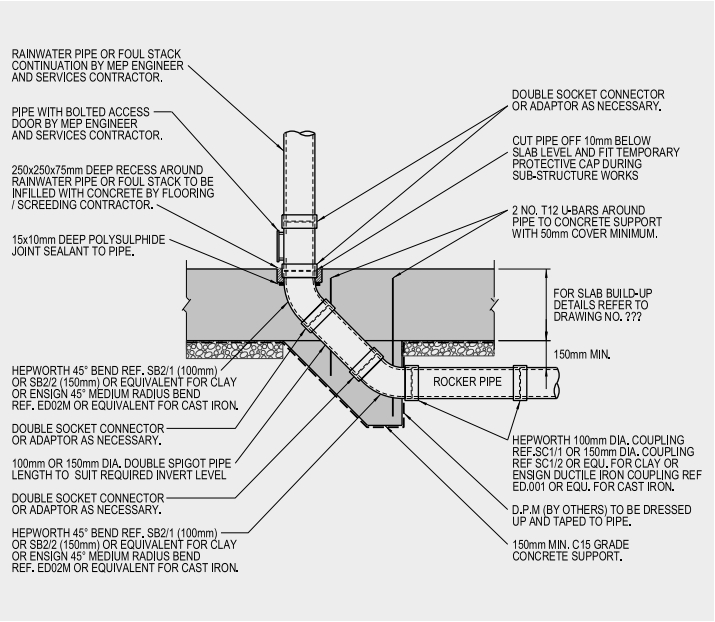


Figure 9.1 Rodding /jetting access detail

9.3 Rainwater Harvesting

Any property with a rainwater harvesting system installed should be provided with appropriate information as to what equipment has been installed, its purpose, its operation and maintenance requirements, the actions needed to address any potential failure and the expected performance of the system. Information on the options for external maintenance support should also be provided.

Most systems require periodic checking and maintenance to ensure troublefree and reliable operation. There are wide differences in the extent of maintenance required for different systems, and manufacturers’ guidelines should always be followed. The table of actions below is not exhaustive and some actions may not always be required.

Maintenance requirements are largely dependent on the runoff source and the runoff use (and thus treatment processes provided). This will range from weekly input through to rare intervention. Routine inspection of the filter system at quarterly annual intervals is advised, even if they do not appear to need specific intervention. Pumps need very little attention, but their design life is generally regarded as only being 10 years. Where automatic provision of potable water occurs (if and when rainwater is either not available or the system has failed), it is useful to have sensor warnings relayed in such a manner as to inform the user of the current status of the system.

Tanks should be accessible for internal inspection, and the cover should preferably be lockable.

The maintenance responsibility for a rainwater harvesting system is usually with the owner of the property, but any communal systems require the participating community to be informed of the system, as detailed, but also be provided with information of who the organisation is that is maintaining the system and any financial commitments and any legally binding maintenance agreement.

Maintenance schedule	Required action	Typical frequency
Regular maintenance	Inspection of the tank for debris and sediment buildup, inlets/outlets/ withdrawal devices, overflow areas, pumps, filters	Annually (and following poor performance)
	Cleaning of tank, inlets, outlets, gutters, withdrawal devices and roof drain filters of silts and other debris	Annually (and following poor performance)
Occasional maintenance	Cleaning and/or replacement of any filters	Three monthly (or as required)
Remedial actions	Repair of overflow erosion damage or damage to tank	As required
	Pump repairs	As required

9.4 Attenuation tanks

CCTV inspection of the tank is recommended at six-monthly intervals and after every major storm. Should the CCTV reveal a build up of silt in the base of the tank jetting of the tank structure should be undertaken to clear the system.

Silt traps prior to inlet pipework should be routinely inspected and cleaned out to minimise debris reaching the tank. It is important to prevent construction silt from entering the attenuation structure.

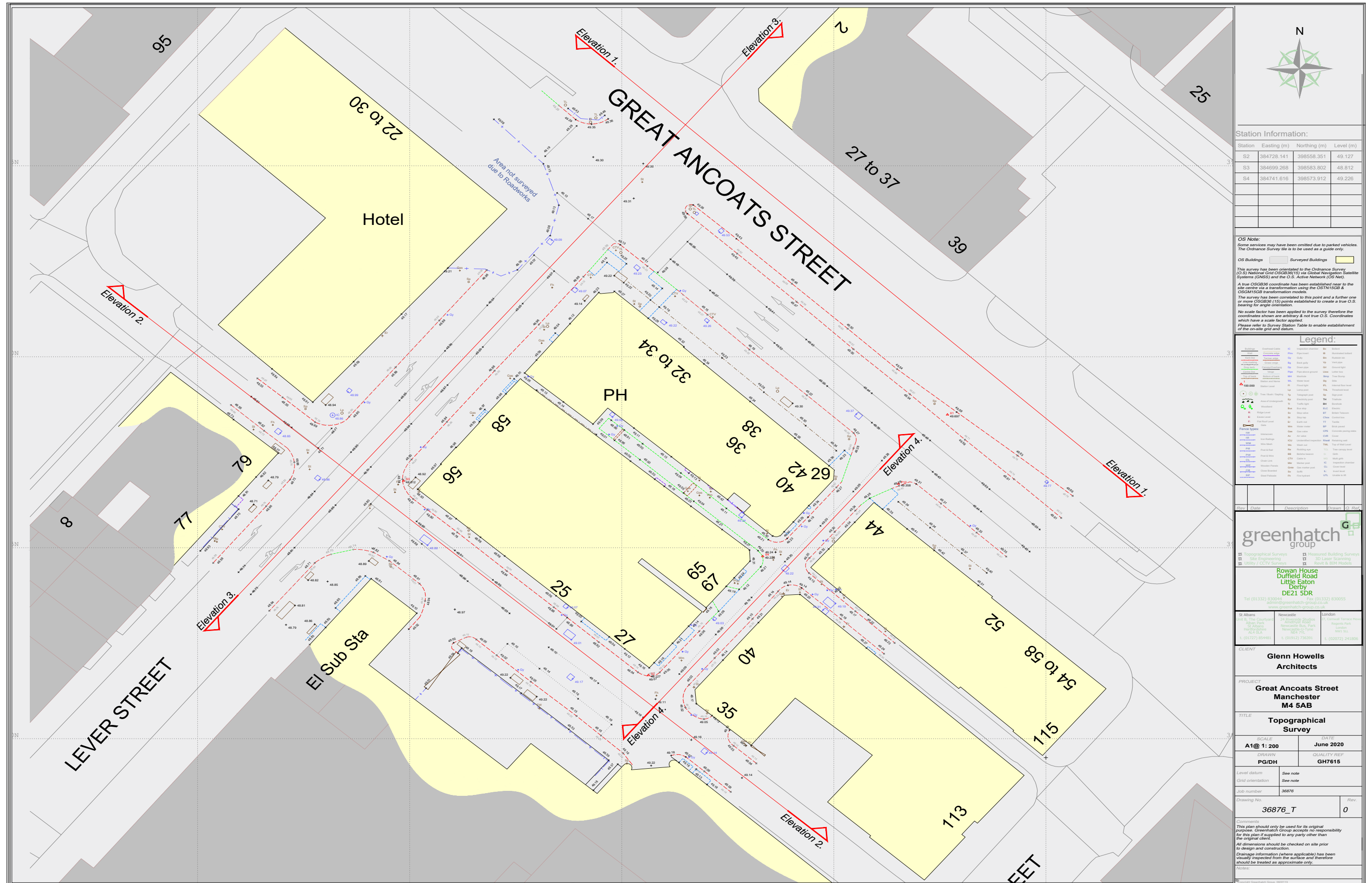
The table below provides guidance on the type of operational and maintenance requirements that may be appropriate. The list of actions is not exhaustive and some actions may not always be required.

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 from 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 blockage by sediment, algae or other matter; remove and replace surface infiltration medium as necessary.	Annually
	Remove sediment from pre-treatment structures and/ or internal forebays	Annually, or as required
Remedial actions	Repair/rehabilitate inlets, outlet, 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

Appendix A

Topographic survey





Appendix B

Calculations



B.1 Climate Change Allowance

In the following calculations, a climate change allowance of 40% has been made which corresponds with the “Upper Limit” of the DEFRA guidance (refer to the table below):

Range	Total potential change anticipated for 2010-2039	Total potential change anticipated for 2040-2059	Total potential change anticipated for 2060-2115
Upper end	10%	20%	40%
Central	5%	10%	20%

B.2 Existing site surface water runoff rates

The development site occupies a total area of 1,250 m² which is currently 100% hardstanding. This excludes the external paving to the public highway.

In accordance with the Modified Rational Method, the peak existing run-off from the site is calculated from the formula:

$$Q = 3.61 \times C_v \times A \times i$$

where C_v is the volumetric runoff coefficient, A is the catchment area in hectares and i is the peak rainfall intensity in mm/hr.

For this site, the following site characteristics have been used in the calculation:

- M5-6min = 18 mm
- Ratio r = 0.45
- Peak run off rates have been based on a 15 minute rainfall event
- Impermeable Area Factor = 1.00
- Permeable Area Factor = 0.00
- $C_v = 1.00$

For the peak return period storm events this gives existing discharge rates from the site of:

$$Q_1 = 3.61 \times 1.00 \times 0.125 \times 34.13 = \mathbf{15.4 \text{ litres/sec}}$$

$$Q_{30} = 3.61 \times 1.00 \times 0.125 \times 70.77 = \mathbf{31.9 \text{ litres/sec}}$$

$$Q_{100} = 3.61 \times 1.00 \times 0.125 \times 91.61 = \mathbf{41.3 \text{ litres/sec}}$$

$$Q_{100+cc} = 3.61 \times 1.00 \times 0.125 \times 128.25 = \mathbf{57.9 \text{ litres/sec}}$$

B.3 Existing Discharge Volumes

The existing discharge volumes have been calculated in accordance with BS 8582 which requires the volume for the 6 hour storm event to be considered. The volume discharged during the storm event is calculated from the formula:

$$Vol_x = A \times i \times 6 \text{ hrs}$$

where A is the existing hardstanding area in hectares and i is the X-year, 6 hour event rainfall intensity in mm/hr.

$$Vol_1 = 0.125 \times 3.25 \times 6 = \mathbf{24.4 \text{ m}^3}$$

$$Vol_{30} = 0.125 \times 6.95 \times 6 = \mathbf{52.2 \text{ m}^3}$$

$$Vol_{100} = 0.125 \times 9.10 \times 6 = \mathbf{68.3 \text{ m}^3}$$

$$Vol_{100+cc} = 0.125 \times 12.74 \times 6 = \mathbf{95.6 \text{ m}^3}$$

B.4 Existing Runoff Summary

The existing runoff values are summarised below:

Return Period	Existing peak runoff rate	Existing Discharge Volume
1 year	15.4 litres/sec	24.4 m³
30 year	31.9 litres/sec	52.2 m³
100 year	41.3 litres/sec	68.3 m³
100 year (+40%)	57.9 litres/sec	95.6 m³

B.5 Greenfield Runoff Rates

In accordance with the method outlined in the Institute of Hydrology Report 124, the Greenfield runoff for the site is calculated from the formula:

$$Q_{BAR} = 0.00108 \times AREA^{0.89} \times SAAR^{1.17} \times SOIL^{2.17}$$

where AREA is the site area in km² (pro rata of 50 ha if the site is less than 50 ha), SAAR is the Standard Average Annual Rainfall in mm and SOIL is the Soil Index. For this site, the following site characteristics have been used in the calculation:

- IH 124 Soil Type = 4, therefore, SOIL = 0.45
- Standard Annual Average Rainfall (SAAR) = 890 mm
- Hydrological Growth Curve Region = 10 (North West England)

For a 50 ha site this gives a greenfield runoff rate of:

$$Q_{BAR} = 0.00108 \times 0.5^{0.89} \times 890^{1.17} \times 0.45^{2.17} = \mathbf{290.9 \text{ litres/sec}}$$

Scaling this for the actual site area gives:

$$Q_{BAR} = (290.9 \times 0.125) \div 50 = \mathbf{0.73 \text{ litres/sec}}$$

Using the Hydrological Growth Curve for north west England, gives:

$$Q_1 = 0.870 \times 0.73 = \mathbf{0.63 \text{ litres/sec}}$$

$$Q_{30} = 1.700 \times 0.73 = \mathbf{1.24 \text{ litres/sec}}$$

$$Q_{100} = 2.080 \times 0.73 = \mathbf{1.51 \text{ litres/sec}}$$

$$Q_{100+cc} = 2.912 \times 0.73 = \mathbf{2.12 \text{ litres/sec}}$$

B.6 Proposed Unattenuated Surface Water Runoff Rates

The proposed architectural drawings development suggest that the site will remain 100% impermeable. Again using the Modified Rational Method, the proposed (unattenuated) peak run-off from the site for the 1-in-1-year return period storm would be:

$$Q_1 = 3.61 \times 1.00 \times 0.125 \times 34.13 = \mathbf{15.4 \text{ litres/sec}}$$

$$Q_{30} = 3.61 \times 1.00 \times 0.125 \times 70.77 = \mathbf{31.9 \text{ litres/sec}}$$

$$Q_{100} = 3.61 \times 1.00 \times 0.125 \times 91.61 = \mathbf{41.3 \text{ litres/sec}}$$

$$Q_{100+cc} = 3.61 \times 1.00 \times 0.125 \times 128.25 = \mathbf{57.9 \text{ litres/sec}}$$

B.7 Proposed Attenuated Surface Water Runoff Rates

In accordance with the Environment Agency’s guidelines, the Building Regulations and the Water Authority’s advice, the preferred means of surface water drainage for any new development is into a suitable soakaway or infiltration drainage system. Sustainable Urban Drainage Systems (SuDS) can reduce the impact of urbanisation on watercourse flows, ensure the protection and enhancement of water quality and encourage recharging of groundwater in a manner which mimics nature.

In addition to this, the National Planning Policy Framework requires that surface water arising from a developed site should, as far as is practicable, be managed in a sustainable manner to mimic surface water flows arising from the site prior to the proposed development, whilst reducing flood risk to the site itself and elsewhere, taking climate change into account.

Therefore, as an absolute minimum, the proposed site discharge under the 1-in-100-year storm plus climate change should be no greater than the existing 1-in-100-year storm discharge (i.e. mitigate the impact of climate change and any increase in the area of hardstanding). In this case, this would mean that, rather than discharging **95.6 litres/sec**, the maximum permissible discharge from the site would be **68.3 litres/sec**.

Further to the above, the London Plan’s Policy SI 13 states that “Development proposals should aim to achieve greenfield run-off rates and ensure that surface water run-off is managed as close to its source as possible”. The Environment Agency (EA) suggests that Developers should aim to achieve greenfield run off from their site.

However, Clause 17 of the DEFRA/EA publication ‘Rainfall runoff management for developments’ states that “A practicable minimum limit on the discharge rate from a flow attenuation device is often a compromise between attenuating to a satisfactorily low flow rate while keeping the risk of blockage to an acceptable level. This limit is set at 5 litres per second, using an appropriate vortex or other flow control device. Where sedimentation could be an issue, the minimum size of orifice for controlling flow from an attenuation device should normally be 150 mm laid at a gradient not flatter than 1 in 150, which meets the requirements of Sewers for Adoption 7th Edition”.

Based on the above considerations, it is proposed to match the post-development runoff rates to the equivalent return period pre-development greenfield runoff rate as summarised below:

$$Q_1 = \mathbf{0.63 \text{ litres/sec}}$$

$$Q_{30} = \mathbf{1.24 \text{ litres/sec}}$$

$$Q_{100} = \mathbf{1.51 \text{ litres/sec}}$$

$$Q_{100+cc} = \mathbf{2.12 \text{ litres/sec}}$$

B.8 Proposed Discharge Volumes

The existing discharge volumes have been calculated in accordance with BS 8582 which requires the volume for the 6 hour storm event to be considered. The volume discharged during the storm event is calculated from the formula:

$$Vol_x = A \times i \times 6 \text{ hrs}$$

where A is the existing hardstanding area in hectares and i is the X-year, 6 hour event rainfall intensity in mm/hr.

$$Vol_1 = 0.125 \times 3.25 \times 6 = \mathbf{24.4 \text{ m}^3}$$

$$Vol_{30} = 0.125 \times 6.95 \times 6 = \mathbf{52.2 \text{ m}^3}$$

$$Vol_{100} = 0.125 \times 9.10 \times 6 = \mathbf{68.3 \text{ m}^3}$$

$$Vol_{100+cc} = 0.125 \times 12.74 \times 6 = \mathbf{95.6 \text{ m}^3}$$

B.9 Network Design

The Microdrainage Simulation and Source Control results contained on the following pages demonstrate that all water is contained within the proposed drainage system for every event up to and including the 1 in 100 year plus 40% climate change allowance event.

B.10 Summary

The proposed and existing runoff values are summarised below:

Return Period	Existing Peak run off rate² (litres/sec)	Proposed scheme unattenuated peak run off rate (litres/sec)	Greenfield run off rate (litres/sec)	Proposed Design Peak run off rate (litres/sec)	Existing Discharge Volume (m³)	Proposed Discharge Volume (m³)
1 in 1 year	15.4	15.4	0.63	0.60	24.4	24.4
1 in 30 years	31.9	31.9	1.24	0.90	52.2	52.2
1 in 100 years	41.3	41.3	1.51	1.00	68.3	68.3
1 in 100 years (+40%)	57.9	57.9	2.12	1.00	95.6	95.6

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Page 1

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Micro Drainage		

Source Control 2020.1

Summary of Results for 1 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	0.136	0.136	0.4	8.8	O K
30 min Summer	0.170	0.170	0.5	11.1	O K
60 min Summer	0.203	0.203	0.5	13.2	O K
120 min Summer	0.230	0.230	0.5	15.0	O K
180 min Summer	0.240	0.240	0.5	15.6	O K
240 min Summer	0.243	0.243	0.5	15.8	O K
360 min Summer	0.246	0.246	0.5	16.0	O K
480 min Summer	0.246	0.246	0.5	16.0	O K
600 min Summer	0.244	0.244	0.5	15.8	O K
720 min Summer	0.240	0.240	0.5	15.6	O K
960 min Summer	0.230	0.230	0.5	14.9	O K
1440 min Summer	0.209	0.209	0.5	13.6	O K
2160 min Summer	0.181	0.181	0.5	11.8	O K
2880 min Summer	0.158	0.158	0.4	10.3	O K
4320 min Summer	0.123	0.123	0.4	8.0	O K
5760 min Summer	0.098	0.098	0.4	6.4	O K
7200 min Summer	0.080	0.080	0.3	5.2	O K
8640 min Summer	0.067	0.067	0.3	4.3	O K
10080 min Summer	0.057	0.057	0.3	3.7	O K
15 min Winter	0.152	0.152	0.4	9.9	O K
30 min Winter	0.191	0.191	0.5	12.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	40.434	0.0	8.9	19
30 min Summer	25.765	0.0	11.4	33
60 min Summer	15.926	0.0	14.2	62
120 min Summer	9.634	0.0	17.2	122
180 min Summer	7.140	0.0	19.2	180
240 min Summer	5.764	0.0	20.7	218
360 min Summer	4.259	0.0	22.9	278
480 min Summer	3.435	0.0	24.6	342
600 min Summer	2.903	0.0	26.0	410
720 min Summer	2.524	0.0	27.1	480
960 min Summer	2.024	0.0	29.0	616
1440 min Summer	1.483	0.0	31.9	892
2160 min Summer	1.086	0.0	35.1	1280
2880 min Summer	0.872	0.0	37.6	1668
4320 min Summer	0.639	0.0	41.3	2416
5760 min Summer	0.513	0.0	44.3	3120
7200 min Summer	0.433	0.0	46.7	3824
8640 min Summer	0.376	0.0	48.7	4504
10080 min Summer	0.334	0.0	50.4	5240
15 min Winter	40.434	0.0	10.0	18
30 min Winter	25.765	0.0	12.8	33

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Page 2

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Micro Drainage

Source Control 2020.1

Summary of Results for 1 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	0.229	0.229	0.5	14.9	O K
120 min Winter	0.261	0.261	0.6	16.9	O K
180 min Winter	0.273	0.273	0.6	17.8	O K
240 min Winter	0.279	0.279	0.6	18.1	O K
360 min Winter	0.280	0.280	0.6	18.2	O K
480 min Winter	0.278	0.278	0.6	18.1	O K
600 min Winter	0.274	0.274	0.6	17.8	O K
720 min Winter	0.267	0.267	0.6	17.4	O K
960 min Winter	0.252	0.252	0.5	16.4	O K
1440 min Winter	0.221	0.221	0.5	14.3	O K
2160 min Winter	0.180	0.180	0.5	11.7	O K
2880 min Winter	0.148	0.148	0.4	9.6	O K
4320 min Winter	0.103	0.103	0.4	6.7	O K
5760 min Winter	0.074	0.074	0.3	4.8	O K
7200 min Winter	0.056	0.056	0.3	3.6	O K
8640 min Winter	0.046	0.046	0.3	3.0	O K
10080 min Winter	0.041	0.041	0.2	2.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	15.926	0.0	16.0	62
120 min Winter	9.634	0.0	19.3	118
180 min Winter	7.140	0.0	21.5	176
240 min Winter	5.764	0.0	23.1	230
360 min Winter	4.259	0.0	25.6	290
480 min Winter	3.435	0.0	27.6	366
600 min Winter	2.903	0.0	29.1	444
720 min Winter	2.524	0.0	30.4	520
960 min Winter	2.024	0.0	32.5	666
1440 min Winter	1.483	0.0	35.7	952
2160 min Winter	1.086	0.0	39.4	1360
2880 min Winter	0.872	0.0	42.1	1756
4320 min Winter	0.639	0.0	46.3	2468
5760 min Winter	0.513	0.0	49.6	3176
7200 min Winter	0.433	0.0	52.3	3888
8640 min Winter	0.376	0.0	54.5	4408
10080 min Winter	0.334	0.0	56.5	5248

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Micro Drainage Source Control 2020.1		
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<div>Model Details</div> <p>Storage is Online Cover Level (m) 3.000</p> <div>Tank or Pond Structure</div> <p>Invert Level (m) 0.000</p> <table><tr><th>Depth (m)</th><th>Area (m²)</th><th>Depth (m)</th><th>Area (m²)</th><th>Depth (m)</th><th>Area (m²)</th></tr><tr><td>0.000</td><td>65.0</td><td>1.000</td><td>65.0</td><td>1.001</td><td>0.0</td></tr></table> <div>Hydro-Brake® Optimum Outflow Control</div> <table><tr><td>Unit Reference</td><td>MD-SCU-0030-1000-1000-1000</td></tr><tr><td>Design Head (m)</td><td>1.000</td></tr><tr><td>Design Flow (l/s)</td><td>1.0</td></tr><tr><td>Flush-Flo™</td><td>Calculated</td></tr><tr><td>Objective</td><td>Linear discharge profile</td></tr><tr><td>Application</td><td>Surface</td></tr><tr><td>Sump Available</td><td>Yes</td></tr><tr><td>Diameter (mm)</td><td>30</td></tr><tr><td>Invert Level (m)</td><td>0.000</td></tr><tr><td>Minimum Outlet Pipe Diameter (mm)</td><td>75</td></tr><tr><td>Suggested Manhole Diameter (mm)</td><td>1200</td></tr></table> <table><tr><th>Control Points</th><th>Head (m)</th><th>Flow (l/s)</th></tr><tr><td>Design Point (Calculated)</td><td>1.000</td><td>1.0</td></tr><tr><td>Flush-Flo™</td><td>0.045</td><td>0.3</td></tr><tr><td>Kick-Flo®</td><td>0.045</td><td>0.3</td></tr><tr><td>Mean Flow over Head Range</td><td>-</td><td>0.7</td></tr></table> <p>The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated</p> <table><tr><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th><th>Depth (m)</th><th>Flow (l/s)</th></tr><tr><td>0.100</td><td>0.4</td><td>1.200</td><td>1.1</td><td>3.000</td><td>1.7</td><td>7.000</td><td>2.4</td></tr><tr><td>0.200</td><td>0.5</td><td>1.400</td><td>1.2</td><td>3.500</td><td>1.8</td><td>7.500</td><td>2.5</td></tr><tr><td>0.300</td><td>0.6</td><td>1.600</td><td>1.2</td><td>4.000</td><td>1.9</td><td>8.000</td><td>2.6</td></tr><tr><td>0.400</td><td>0.7</td><td>1.800</td><td>1.3</td><td>4.500</td><td>2.0</td><td>8.500</td><td>2.7</td></tr><tr><td>0.500</td><td>0.7</td><td>2.000</td><td>1.4</td><td>5.000</td><td>2.1</td><td>9.000</td><td>2.8</td></tr><tr><td>0.600</td><td>0.8</td><td>2.200</td><td>1.4</td><td>5.500</td><td>2.2</td><td>9.500</td><td>2.8</td></tr><tr><td>0.800</td><td>0.9</td><td>2.400</td><td>1.5</td><td>6.000</td><td>2.3</td><td></td><td></td></tr><tr><td>1.000</td><td>1.0</td><td>2.600</td><td>1.5</td><td>6.500</td><td>2.4</td><td></td><td></td></tr></table>			Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)	0.000	65.0	1.000	65.0	1.001	0.0	Unit Reference	MD-SCU-0030-1000-1000-1000	Design Head (m)	1.000	Design Flow (l/s)	1.0	Flush-Flo™	Calculated	Objective	Linear discharge profile	Application	Surface	Sump Available	Yes	Diameter (mm)	30	Invert Level (m)	0.000	Minimum Outlet Pipe Diameter (mm)	75	Suggested Manhole Diameter (mm)	1200	Control Points	Head (m)	Flow (l/s)	Design Point (Calculated)	1.000	1.0	Flush-Flo™	0.045	0.3	Kick-Flo®	0.045	0.3	Mean Flow over Head Range	-	0.7	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	0.100	0.4	1.200	1.1	3.000	1.7	7.000	2.4	0.200	0.5	1.400	1.2	3.500	1.8	7.500	2.5	0.300	0.6	1.600	1.2	4.000	1.9	8.000	2.6	0.400	0.7	1.800	1.3	4.500	2.0	8.500	2.7	0.500	0.7	2.000	1.4	5.000	2.1	9.000	2.8	0.600	0.8	2.200	1.4	5.500	2.2	9.500	2.8	0.800	0.9	2.400	1.5	6.000	2.3			1.000	1.0	2.600	1.5	6.500	2.4		
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Date 08/01/2024 11:26 File 30 Year.SRCX		Designed by MJD Checked by DP				
Micro Drainage		Source Control 2020.1				
<u>Summary of Results for 30 year Return Period (+40%)</u>						
Storm Event		Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer		0.336	0.336	0.6	21.9	O K
30 min Summer		0.425	0.425	0.7	27.6	O K
60 min Summer		0.508	0.508	0.7	33.0	O K
120 min Summer		0.580	0.580	0.8	37.7	O K
180 min Summer		0.611	0.611	0.8	39.7	O K
240 min Summer		0.625	0.625	0.8	40.6	O K
360 min Summer		0.629	0.629	0.8	40.9	O K
480 min Summer		0.623	0.623	0.8	40.5	O K
600 min Summer		0.615	0.615	0.8	40.0	O K
720 min Summer		0.607	0.607	0.8	39.5	O K
960 min Summer		0.589	0.589	0.8	38.3	O K
1440 min Summer		0.549	0.549	0.8	35.7	O K
2160 min Summer		0.492	0.492	0.7	32.0	O K
2880 min Summer		0.442	0.442	0.7	28.7	O K
4320 min Summer		0.361	0.361	0.6	23.5	O K
5760 min Summer		0.300	0.300	0.6	19.5	O K
7200 min Summer		0.253	0.253	0.5	16.4	O K
8640 min Summer		0.215	0.215	0.5	14.0	O K
10080 min Summer		0.185	0.185	0.5	12.0	O K
15 min Winter		0.377	0.377	0.6	24.5	O K
30 min Winter		0.477	0.477	0.7	31.0	O K
Storm Event		Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer		99.072	0.0	22.0	19	
30 min Summer		63.334	0.0	28.1	33	
60 min Summer		38.715	0.0	34.7	64	
120 min Summer		23.015	0.0	41.3	122	
180 min Summer		16.828	0.0	45.3	182	
240 min Summer		13.432	0.0	48.2	242	
360 min Summer		9.736	0.0	52.4	352	
480 min Summer		7.725	0.0	55.5	404	
600 min Summer		6.452	0.0	57.9	464	
720 min Summer		5.571	0.0	60.0	526	
960 min Summer		4.418	0.0	63.4	662	
1440 min Summer		3.183	0.0	68.4	938	
2160 min Summer		2.291	0.0	74.2	1344	
2880 min Summer		1.813	0.0	78.3	1732	
4320 min Summer		1.303	0.0	84.3	2508	
5760 min Summer		1.031	0.0	89.0	3280	
7200 min Summer		0.859	0.0	92.7	3968	
8640 min Summer		0.740	0.0	95.8	4680	
10080 min Summer		0.652	0.0	98.5	5448	
15 min Winter		99.072	0.0	24.6	19	
30 min Winter		63.334	0.0	31.4	33	
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Page 6

White Collar Factory
1 Old Street Yard
London, EC1Y 8AF

Date 08/01/2024 11:26
File 30 Year.SRCX

Lever Street
Manchester

Designed by MJD
Checked by DP

Micro Drainage

Source Control 2020.1

Summary of Results for 30 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	0.572	0.572	0.8	37.2	O K
120 min Winter	0.654	0.654	0.8	42.5	O K
180 min Winter	0.692	0.692	0.8	45.0	O K
240 min Winter	0.710	0.710	0.9	46.2	O K
360 min Winter	0.721	0.721	0.9	46.8	O K
480 min Winter	0.713	0.713	0.9	46.3	O K
600 min Winter	0.701	0.701	0.9	45.6	O K
720 min Winter	0.691	0.691	0.8	44.9	O K
960 min Winter	0.666	0.666	0.8	43.3	O K
1440 min Winter	0.610	0.610	0.8	39.6	O K
2160 min Winter	0.527	0.527	0.8	34.3	O K
2880 min Winter	0.456	0.456	0.7	29.7	O K
4320 min Winter	0.346	0.346	0.6	22.5	O K
5760 min Winter	0.268	0.268	0.6	17.4	O K
7200 min Winter	0.210	0.210	0.5	13.7	O K
8640 min Winter	0.168	0.168	0.5	10.9	O K
10080 min Winter	0.136	0.136	0.4	8.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	38.715	0.0	38.9	62
120 min Winter	23.015	0.0	46.3	120
180 min Winter	16.828	0.0	50.7	178
240 min Winter	13.432	0.0	54.0	236
360 min Winter	9.736	0.0	58.7	346
480 min Winter	7.725	0.0	62.1	450
600 min Winter	6.452	0.0	64.8	486
720 min Winter	5.571	0.0	67.2	558
960 min Winter	4.418	0.0	71.0	712
1440 min Winter	3.183	0.0	76.6	1012
2160 min Winter	2.291	0.0	83.1	1448
2880 min Winter	1.813	0.0	87.7	1848
4320 min Winter	1.303	0.0	94.5	2640
5760 min Winter	1.031	0.0	99.7	3400
7200 min Winter	0.859	0.0	103.9	4112
8640 min Winter	0.740	0.0	107.3	4840
10080 min Winter	0.652	0.0	110.3	5544

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White Collar Factory 1 Old Street Yard London, EC1Y 8AF		Lever Street Manchester				
Date 08/01/2024 11:27 File 100 Year.SRCX		Designed by MJD Checked by DP				
Micro Drainage		Source Control 2020.1				
<div>Summary of Results for 100 year Return Period (+40%)</div>						
Storm Event		Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer		0.437	0.437	0.7	28.4	O K
30 min Summer		0.557	0.557	0.8	36.2	O K
60 min Summer		0.672	0.672	0.8	43.7	O K
120 min Summer		0.773	0.773	0.9	50.2	O K
180 min Summer		0.819	0.819	0.9	53.2	O K
240 min Summer		0.841	0.841	0.9	54.7	O K
360 min Summer		0.852	0.852	0.9	55.4	O K
480 min Summer		0.843	0.843	0.9	54.8	O K
600 min Summer		0.832	0.832	0.9	54.1	O K
720 min Summer		0.822	0.822	0.9	53.4	O K
960 min Summer		0.799	0.799	0.9	51.9	O K
1440 min Summer		0.750	0.750	0.9	48.7	O K
2160 min Summer		0.677	0.677	0.8	44.0	O K
2880 min Summer		0.613	0.613	0.8	39.8	O K
4320 min Summer		0.509	0.509	0.7	33.1	O K
5760 min Summer		0.428	0.428	0.7	27.8	O K
7200 min Summer		0.364	0.364	0.6	23.7	O K
8640 min Summer		0.313	0.313	0.6	20.4	O K
10080 min Summer		0.271	0.271	0.6	17.6	O K
15 min Winter		0.489	0.489	0.7	31.8	O K
30 min Winter		0.625	0.625	0.8	40.7	O K
Storm Event		Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
15 min Summer		128.253	0.0	28.4	19	
30 min Summer		82.677	0.0	36.6	34	
60 min Summer		50.812	0.0	45.6	64	
120 min Summer		30.273	0.0	54.3	122	
180 min Summer		22.125	0.0	59.6	182	
240 min Summer		17.638	0.0	63.3	242	
360 min Summer		12.742	0.0	68.6	360	
480 min Summer		10.074	0.0	72.3	438	
600 min Summer		8.388	0.0	75.2	492	
720 min Summer		7.228	0.0	77.8	554	
960 min Summer		5.712	0.0	81.9	682	
1440 min Summer		4.095	0.0	87.8	954	
2160 min Summer		2.931	0.0	94.9	1364	
2880 min Summer		2.311	0.0	99.7	1760	
4320 min Summer		1.651	0.0	106.8	2552	
5760 min Summer		1.300	0.0	112.3	3296	
7200 min Summer		1.079	0.0	116.5	4040	
8640 min Summer		0.927	0.0	120.0	4760	
10080 min Summer		0.815	0.0	123.0	5456	
15 min Winter		128.253	0.0	31.8	19	
30 min Winter		82.677	0.0	40.8	33	
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White Collar Factory 1 Old Street Yard London, EC1Y 8AF		Lever Street Manchester			
Date 08/01/2024 11:27 File 100 Year.SRCX		Designed by MJD Checked by DP			
Micro Drainage		Source Control 2020.1			
<u>Summary of Results for 100 year Return Period (+40%)</u>					
Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	0.755	0.755	0.9	49.1	O K
120 min Winter	0.871	0.871	0.9	56.6	O K
180 min Winter	0.925	0.925	1.0	60.1	O K
240 min Winter	0.954	0.954	1.0	62.0	O K
360 min Winter	0.973	0.973	1.0	63.3	O K
480 min Winter	0.967	0.967	1.0	62.9	O K
600 min Winter	0.951	0.951	1.0	61.8	O K
720 min Winter	0.936	0.936	1.0	60.9	O K
960 min Winter	0.908	0.908	1.0	59.0	O K
1440 min Winter	0.840	0.840	0.9	54.6	O K
2160 min Winter	0.738	0.738	0.9	48.0	O K
2880 min Winter	0.647	0.647	0.8	42.1	O K
4320 min Winter	0.503	0.503	0.7	32.7	O K
5760 min Winter	0.397	0.397	0.7	25.8	O K
7200 min Winter	0.318	0.318	0.6	20.7	O K
8640 min Winter	0.258	0.258	0.5	16.7	O K
10080 min Winter	0.211	0.211	0.5	13.7	O K
Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)	
60 min Winter	50.812	0.0	51.1	62	
120 min Winter	30.273	0.0	60.9	120	
180 min Winter	22.125	0.0	66.7	178	
240 min Winter	17.638	0.0	70.9	236	
360 min Winter	12.742	0.0	76.8	348	
480 min Winter	10.074	0.0	81.0	458	
600 min Winter	8.388	0.0	84.3	556	
720 min Winter	7.228	0.0	87.1	578	
960 min Winter	5.712	0.0	91.7	730	
1440 min Winter	4.095	0.0	98.0	1036	
2160 min Winter	2.931	0.0	106.3	1472	
2880 min Winter	2.311	0.0	111.7	1900	
4320 min Winter	1.651	0.0	119.7	2684	
5760 min Winter	1.300	0.0	125.7	3464	
7200 min Winter	1.079	0.0	130.5	4184	
8640 min Winter	0.927	0.0	134.4	4928	
10080 min Winter	0.815	0.0	137.8	5648	
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B.11 Foul Water Flows

One method of calculating the peak foul flow is to use the statistical or “discharge unit” method outlined in BS 12056. This method of design is based on probability theory to estimate the maximum peak flows from appliances. It assumes that only a proportion of appliances will be in use at any one time and that appliances on a factory, school, etc. would be used more frequently than in domestic installations. For this reason discharge units (DU) are given higher values in congested and commercial use than they are for domestic.

Alternatively, foul flows can be computed using the “population method”. This method calculates a daily discharge volume from typical figures of discharge per person, per day. It then factors it evenly over the day, to calculate base flow. A peaking factor is then applied (conservatively 6 or a variable factor based upon population such as the Babbitt Formula) to determine peak flow.

Estimates can also be based on internal floor area, which can also be arrived at as a product of an assumed population density and daily discharge per person.

As to which method is the more appropriate, typically, outside the building and at the street and city scale, the population method is considered more accurate and is the preferred method utilised by sewer companies. For sizing pipes inside the building, and in drains near to the point of discharge from the building, the statistical method should be used. Consideration of high peak flows from pump stations is also required.

BS 16933-2 recommends that the statistical method can be used for the design of gravity drains and sewers where the population is less than 1000 and the population method can be used where the population is greater than 500.

B.12 Existing Foul Water

Discharge Unit Method

The available record drawings have been used to estimate the number of foul appliances in the existing building. Using the guidelines for commercial developments given in BS EN 12056-2:2000 - “Gravity Drainage Systems Inside Buildings - Part 2: Sanitary Pipework, layout and calculation”, the proposed foul flow is calculated from the formula:

$$Q = K \times \sqrt{DU}$$

Conservatively, for ‘intermittent use’ (dwellings, offices, etc.) K has a value of 0.5 giving:

Appliance	No.	Discharge units per appliance	Total number of discharge units
Wash hand basin	7	0.5	3.5
Urinals	3	0.5	1.5
Sinks	6	0.8	4.8
Baths	1	0.8	0.8
WC	6	2.0	12.0
Floor drains	5	2.0	10.0
Total discharge units for site =			32.6
Therefore, total flow from site =			2.85 litres/sec

The K value of 0.5 is believed to be conservative as there are restaurants on site which would have a K value of 0.7. This would increase the estimated existing flow to **4.0 litres/sec**.

B.13 Proposed Foul Water

Discharge Unit Method

The draft architectural layouts have been used to estimate the number of foul appliances in the proposed office space. Using the guidelines for commercial developments given in BS EN 12056-2:2000 - “Gravity Drainage Systems Inside Buildings - Part 2: Sanitary Pipework, layout and calculation”, the proposed foul flow is calculated from the formula:

$$Q = K \times \sqrt{DU}$$

For ‘intermittent use’ (representing dwellings, offices, etc.) K has a value of 0.5 giving:

Appliance	No.	Discharge units per appliance	Total number of discharge units
Wash hand basin	123	0.5	61.5
Shower	12	0.6	7.2
Sinks	21	0.8	16.8
WC	123	2.0	246.0
Floor drains	8	2.0	16.0
Total discharge units for site =			347.5
Therefore, total flow from site =			9.32 litres/sec

At this early design stage it is suggested that an allowance is made for **10 litres/sec** to accommodate any design development until more accurate figures are available from the MEP engineer.

Appendix C

SuDS Option Selection



Disposal methods

C.1 SuDS management train

A useful concept used in the development of sustainable drainage systems is the SuDS management train (sometimes referred to as the treatment train). Just as in a natural catchment, drainage techniques can be used in series to change flow and quality characteristics of the runoff in stages. There are a variety of measures that can be implemented to achieve these goals:

C.1.1 Site management / Prevention

Site management procedures are used to limit or prevent runoff and pollution and include:

- Minimising the hardened areas within the site
- Frequent maintenance of impermeable surfaces
- Minimising the use of de-icing products

C.1.2 Source control

Source control techniques will be used where possible as they control runoff at source in smaller catchments. They can also provide effective pollution control and treatment, thereby improving the quality of the effluent discharged to the receiving waters.

C.1.3 Site control

Where source control techniques do not provide adequate protection to the receiving watercourses in terms of flood protection and pollution control, site control may be required.

C.1.4 Regional control

Where large areas of public space are available regional control can be incorporated to provide additional 'communal' storage and treatment to runoff from a number of sites. However, in this case, all storage and treatment will be implemented on site.

C.2 Drainage hierarchy

Based on the above and in line with the London Plan and the Sustainable Drainage Manual published by CIRIA, the following drainage hierarchy will therefore need to be considered when preparing the surface water disposal strategy:

1. Rainwater use as a resource (for example rainwater harvesting, blue roofs for irrigation)
2. Rainwater infiltration to ground at or close to source
3. Rainwater attenuation in green infrastructure features for gradual release (for example green roofs, rain gardens)
4. Rainwater discharge direct to a watercourse (unless not appropriate)
5. Controlled rainwater discharge to a surface water sewer or drain
6. Controlled rainwater discharge to a combined sewer

C.3 Assessment of SuDS techniques

C.3.1 Rainwater harvesting

This involves the capture of rainwater into a tank for re-use (usually non-potable) such as irrigation, toilet flushing or vehicle cleaning. Systems are now available which combine rain water harvesting with tanked attenuation. This means that water is stored during dry periods for re-use but released ahead of predicted storms in order to ensure that the full attenuation capacity remains available when it is needed.

As the project is a new-build, it may be possible to provide a rainwater harvesting system within the development. The roof area available is unlikely to generate sufficient water yield to meet enough of the anticipated non-potable water demand to make the system viable but this will need to be confirmed by the MEP engineer.

C.3.2 Green / brown / blue roofs

These are used on flat or shallow pitched roofs to provide a durable roof covering which also provides thermal insulation, amenity space, biodiversity habitat as well as attenuation of rainwater. Depending on the design, these roofs can attenuate differing volumes of rainwater. The term 'blue roof' is reserved for those roofs designed to maximise water retention. This is a relatively recent area of increased focus and can involve effectively an attenuation tank at roof level which reduces (or avoids) the need for pumping of basement tanks.

As the project is a new-build, green / brown / blue roofs can be considered. However, the extent of roof plant and plant screening means that it is unlikely that any of these roof typologies would be suitable for this development and they have therefore been discounted.

C.3.3 Raingardens

Raingardens are planted areas (usually close to buildings but not immediately adjacent) that allow the diversion of a portion of rainwater from either downpipes or the surrounding paved surfaces. These techniques can be incorporated into the landscaping plans for a site and are most effective where the landscaping regime is designed with the aim of capturing as much rainfall as possible. They can either allow infiltration into the ground or have tanked systems for water retention, depending on the site and soil conditions. There are also a number of vertical raingardens attached to building walls with rainwater downpipes diverted through a stacked series of planters.

Raingardens cannot be incorporated into the scheme as the building fronts directly onto the public footpath meaning there is insufficient external space to accommodate them.

C.3.4 Bio-retention

This refers to a chain of landscaped features, potentially including reed beds, filter drains, etc. designed to hold and treat surface water. They are often used where there is a high risk of low-level pollution, for example from road run-off. However, it does require areas of open space. The design of a bio-retention system can vary widely depending on site conditions and available space. At a small scale this could include flow through planters or tree pits.

Bio-retention cannot be incorporated into the scheme as the building fronts directly onto the public footpath meaning there is insufficient external space to accommodate it.

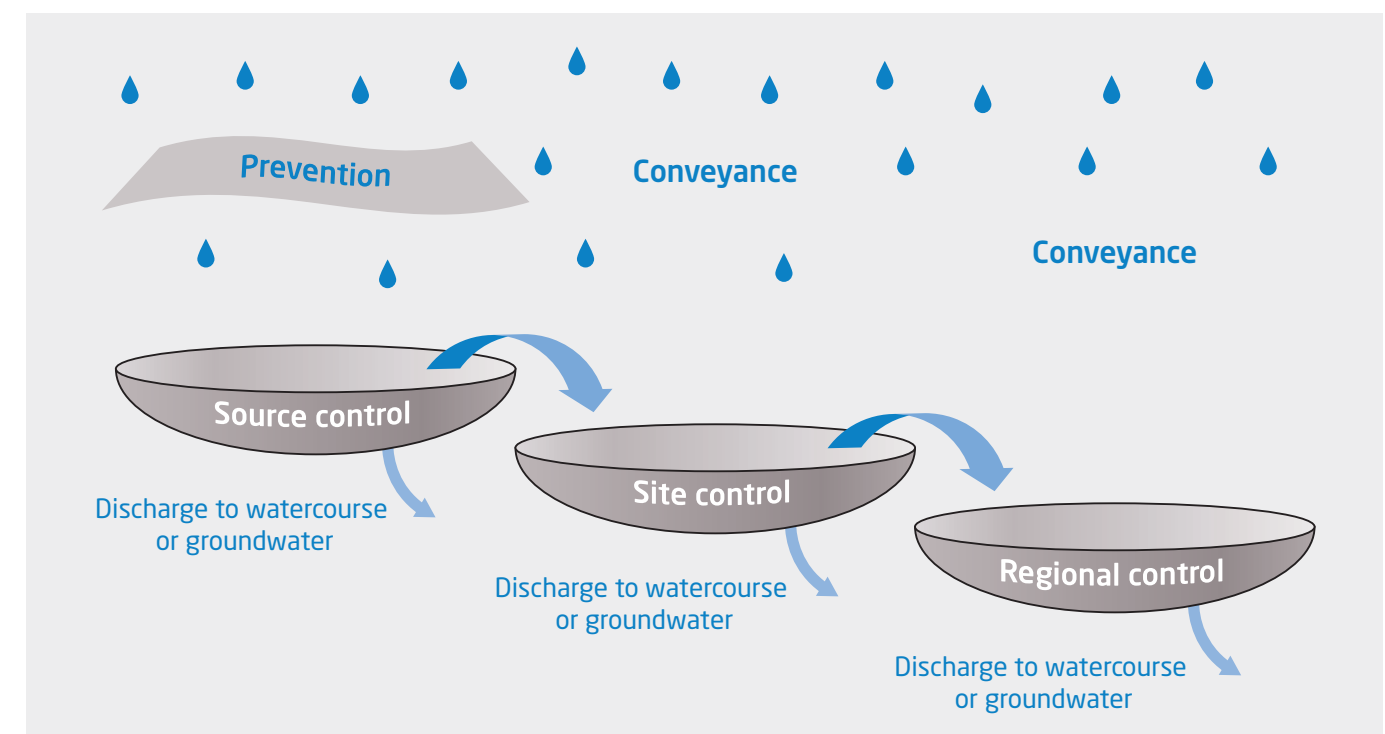


Figure 9.1 SuDS management train

C.3.5 Infiltration

There are many different types of drainage component that can be used to facilitate infiltration. These include soakaways, infiltration trenches, infiltration blankets and infiltration basins.

Infiltration can contribute to reducing runoff rates and volumes while supporting baseflow and groundwater recharge processes. The rate at which water can be infiltrated depends on the infiltration capacity (permeability) of the surrounding soils.

The porosity of the underlying soil is expected to be negligible and so disposal by infiltration will not be possible. The extent of the basement levels also preclude the use of infiltration measures on the site as they cannot be located sufficiently distant from the building to comply with Building Regulations.

C.3.6 Permeable surfacing

Permeable hard surfaces which work in much the same way as traditional impermeable surfaces apart from the ability to allow rainwater to pass through. Permeable blocks are traditionally used but there are now a range of permeable asphalt and resin bound gravel pavings being used increasingly commonly. Permeable surfaces can either allow infiltration into the ground or have tanked systems for water retention, depending on the site and soil conditions. They are suitable in even the most densely built-up development. However, they're not well suited to roads carrying heavy or fast motor traffic.

There is a small area of paving proposed in the entrance courtyard as part of the scheme. However, as this is located above a proposed basement any permeable surfacing would have to be a tanked system for water retention and conveyance only rather than disposal and has therefore been discounted.

C.3.7 Swales

These are dry ditches used as landscape features to allow the storage, carriage and infiltration of rainwater and are often used as linear features alongside roads, footpaths or rail lines. They can also be integrated into the design of many open spaces.

Swales cannot be incorporated into the scheme as the building fronts directly onto the public footpath meaning there is insufficient external space to accommodate them.

C.3.8 Detention basin / ponds

Landscape features designed to store and in some cases infiltrate rainwater. Detentions basins are usually dry, whereas a pond should retain water. These features need areas of open space but can often be combined with other sustainable drainage techniques.

As the site is heavily developed with limited external areas there is insufficient space to provide a basin or pond.

C.3.9 Discharge to tidal river / dock / canals

Discharging clean rainwater directly to tidal rivers, canals or docks isn't normally a sustainable drainage technique. Other more productive techniques should be used first. However, it is generally more sustainable than discharging to the combined or surface drainage systems. Residual surface water can be discharged to tidal/large waterbodies, in some cases with no limitation on volumes. Some storage may be required to allow for outfalls becoming tide locked. Care is needed to prevent scour in the receiving waterbody and potentially to prevent pollution. Consent from the Environment Agency, the asset owner and where applicable the Canal and River Trust is required.

There are no adjacent rivers or ponds and so discharge to a watercourse will not be a viable disposal method.

C.3.10 Storage tanks / geocellular storage

Storage tanks are single GRP units usually located (but not necessarily) below ground level which attenuate rainwater for later slow release back into the drainage system but do not provide the wider benefits of green infrastructure sustainable drainage. They can also have the disadvantage that pumping may be required to empty the tank into the drainage system – especially if the tank is located at or below basement level. Where tanks are designed for large storm events, care is needed to ensure that they still perform a useful sustainable drainage function for low order storms.

Geocellular storage tanks are similar to storage tanks except that the volume is made up from multiple units rather than a single tank meaning they can be more flexible in terms of shape to suit constrained sites.

It is believed that this is the most feasibly disposal option for the site and the table below presents the approximate tank volumes required in order not to exceed the greenfield rates for any return period (including a 40 % climate change allowance):

Return Period	Greenfield Rate	Discharge Rate	Storage volume required
1 year (+40%)	0.63 litres/sec	0.6 litres/sec	18.2 m³
30 year (+40%)	1.24 litres/sec	0.9 litres/sec	46.8 m³
100 year (+40%)	1.51 litres/sec	1.0 litres/sec	63.3 m³

In this case, in order to have a single flow control device, the 1 year event is the critical storm to design for and means that the 100 year discharge rate will actually be less than greenfield.

It is recommended that at this stage that a cost and space allowance is made for a storage volume of 65 m³ (Greenfield rate) as our recent experience United Utilities and Manchester City Council suggests this will be the required discharge rate for Planning.

The attenuation tank should be located at a high enough level so as to allow a connection to be made to the public sewer by gravity - in this case, it is assumed that this would be at relatively high level within the basement but we are still awaiting confirmation of the invert level of the existing sewer. Locating the tank below this level would result in a pumped surface water system which is both unsustainable and uneconomic.

C.3.11 Oversized piping

Using larger than necessary pipework creates more room to store rainwater. Potentially more sustainable than storage tanks/geocellular storage if the pipes drain by gravity and do not require pumping. However, lacks the wider benefits of the green infrastructure based techniques.

Due to the restricted nature of the site the pipework would become impractically large to provide the volume of storage required to achieve the required run-off rate.

C.4 Summary of the proposed SuDS strategy

An attenuation tank of 65 m³ is proposed which will reduce the peak discharge rate from the site to the Greenfield rate.

The outfall from the site will connect to the existing public sewer either with a new connection or, preferably, by reusing one of the existing connections.

Once the CCTV and level survey of the existing network has been undertaken the location and layout of the outfall(s) can be determined but due to the depth of the existing public combined sewers it is recommended that, if possible, the existing drainage connection(s) should be reused to prevent the need for constructing a new, deep connection. This would minimise both the cost of the work and the disruption to the surrounding roads which are busy thoroughfares and consequently require significant traffic management to be provided during the work.

A schematic SuDS layout is contained in Appendix D for reference.

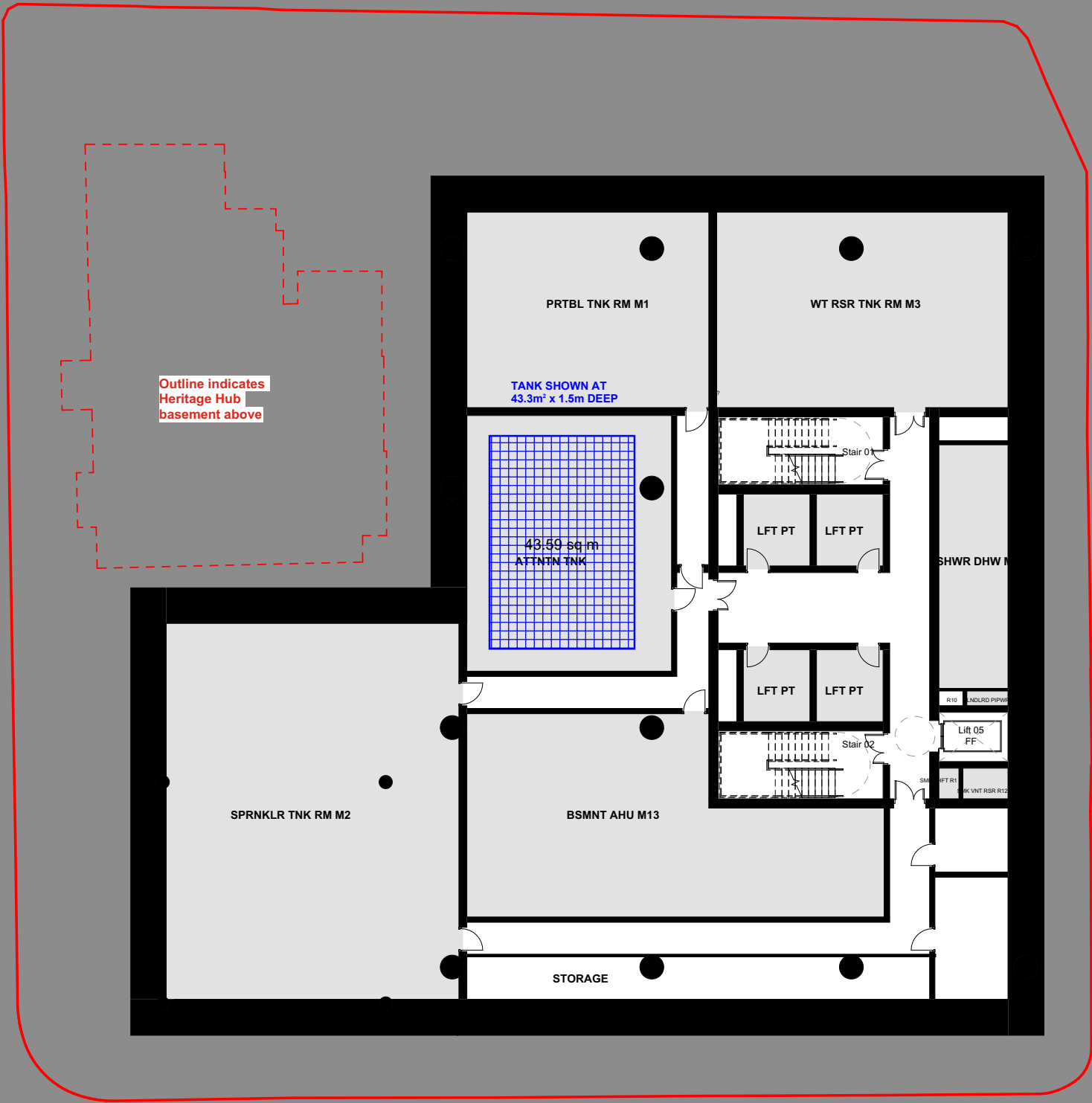
Element	Management stage	Water quantity	Water quality	Amenity & biodiversity	Proposed in scheme
Rainwater harvesting	Prevention	✓	✗	✗	✗
Green / brown / blue roof	Source control	✓	✓	✓	✓ / ✗
Raingardens	Source control	✓	✓	✓	✗
Bio-retention	Source control	✓	✓	✓	✗
Permeable surfacing	Source control	✓	✓	✗	✗
Swales	Source control	✓	✓	✓	✗
Detention basin / ponds	Source control	✓	✓	✓	✗
Discharge to tidal river / dock / canals	Site control	✓	✗	✗	✗
Storage tanks / Geocellular storage	Site control	✓	✗	✗	✓
Oversized piping	Site control	✓	✗	✗	✗
Design for exceedance	Site control	✓	✗	✗	✗

Figure C.1 Summary of proposed SuDS devices

Appendix D

Proposed Drawings





Outline indicates
Heritage Hub
basement above

TANK SHOWN AT
43.3m² x 1.5m DEEP

43.59 sq m
ATTN TNK

SPRNKLR TNK RM M2

BSMNT AHU M13

STORAGE

PRTBL TNK RM M1

WT RSR TNK RM M3

LIFT PT

LIFT PT

LIFT PT

LIFT PT

STAIR 02

STAIR 01

SHWR DHW M

LIFT 05
FF

STAIR 01

STAIR 02

STAIR 03

STAIR 04

STAIR 05

STAIR 06

STAIR 07

STAIR 08

STAIR 09

STAIR 10

STAIR 11

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STAIR 14

STAIR 15

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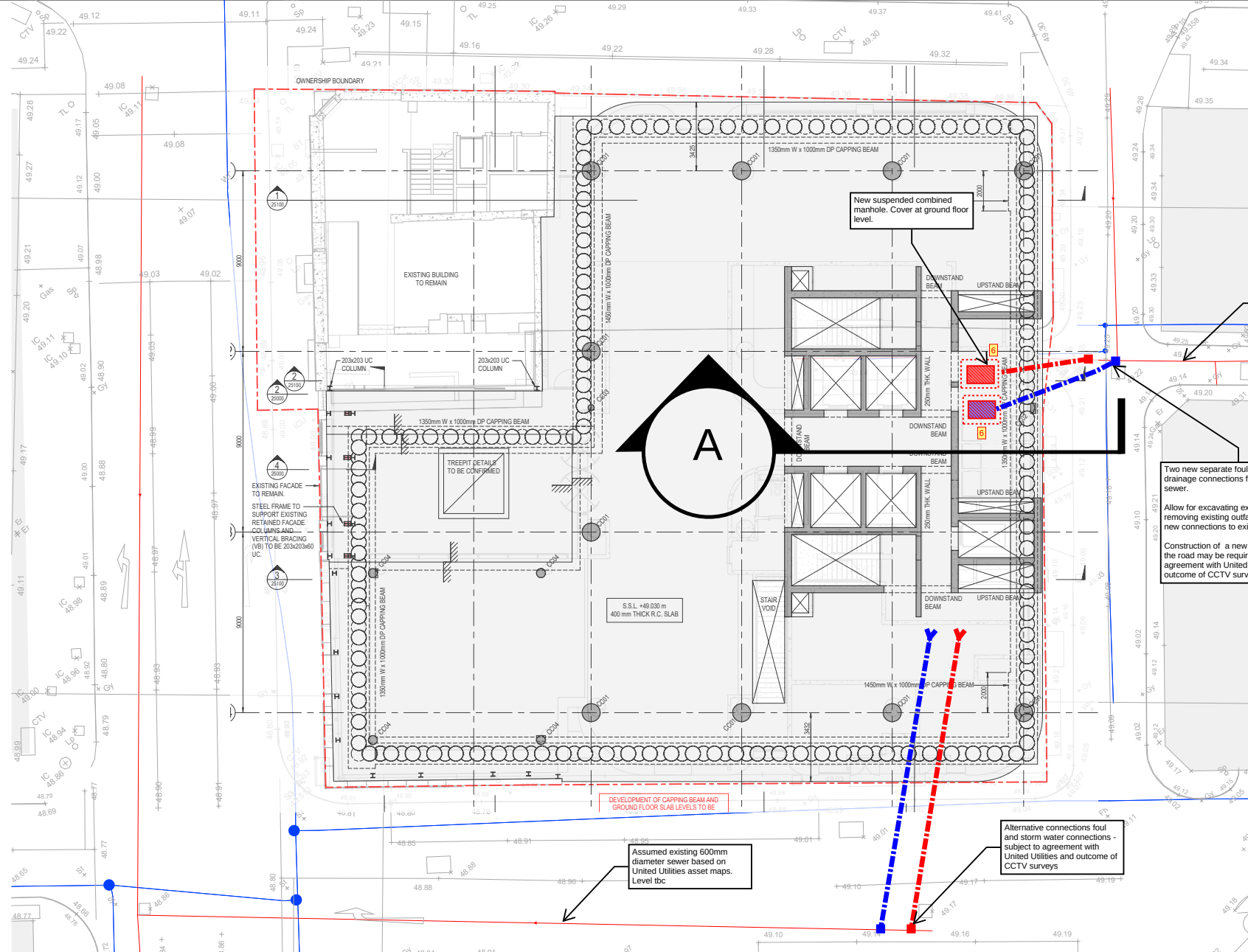
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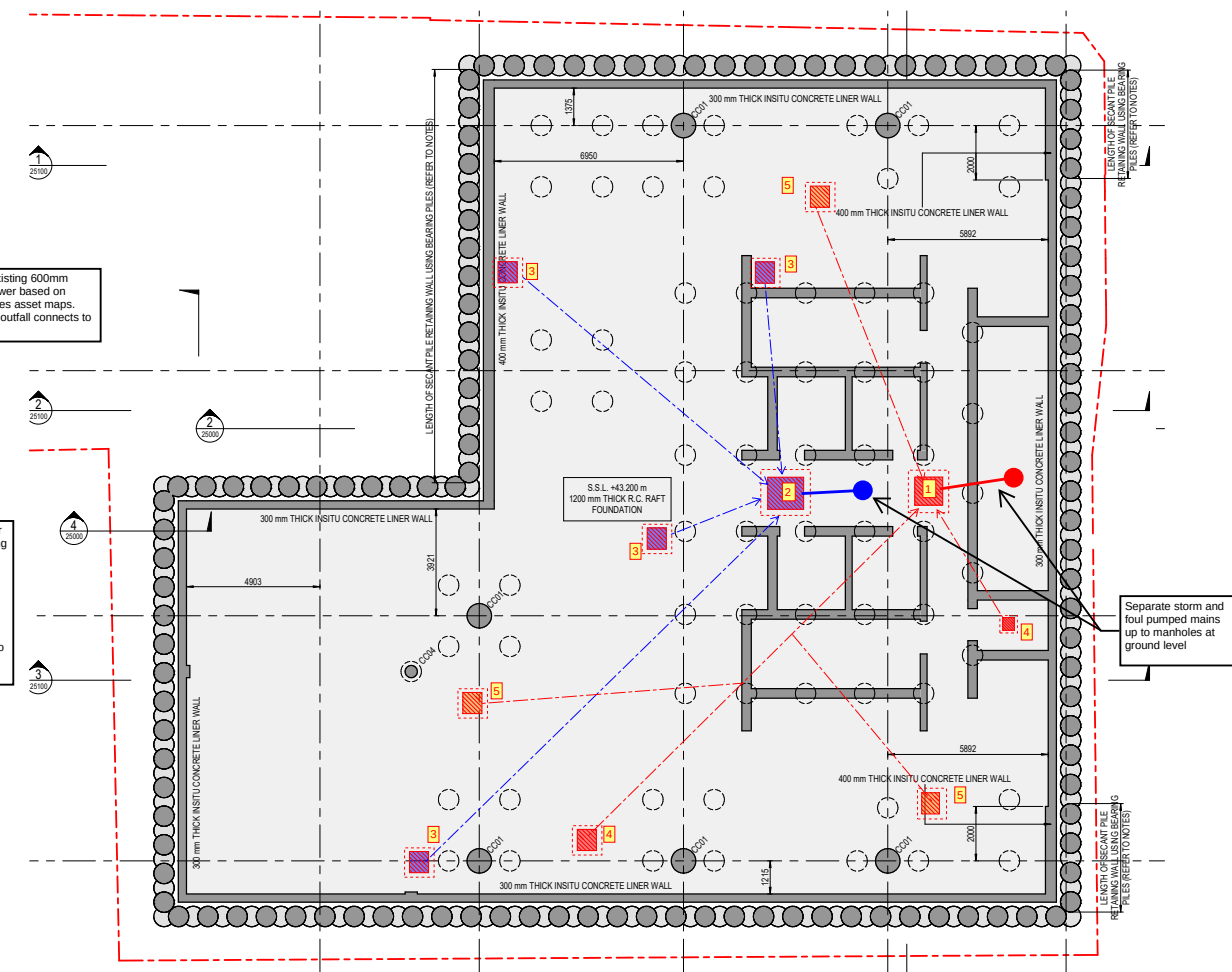
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Ground Floor Plan



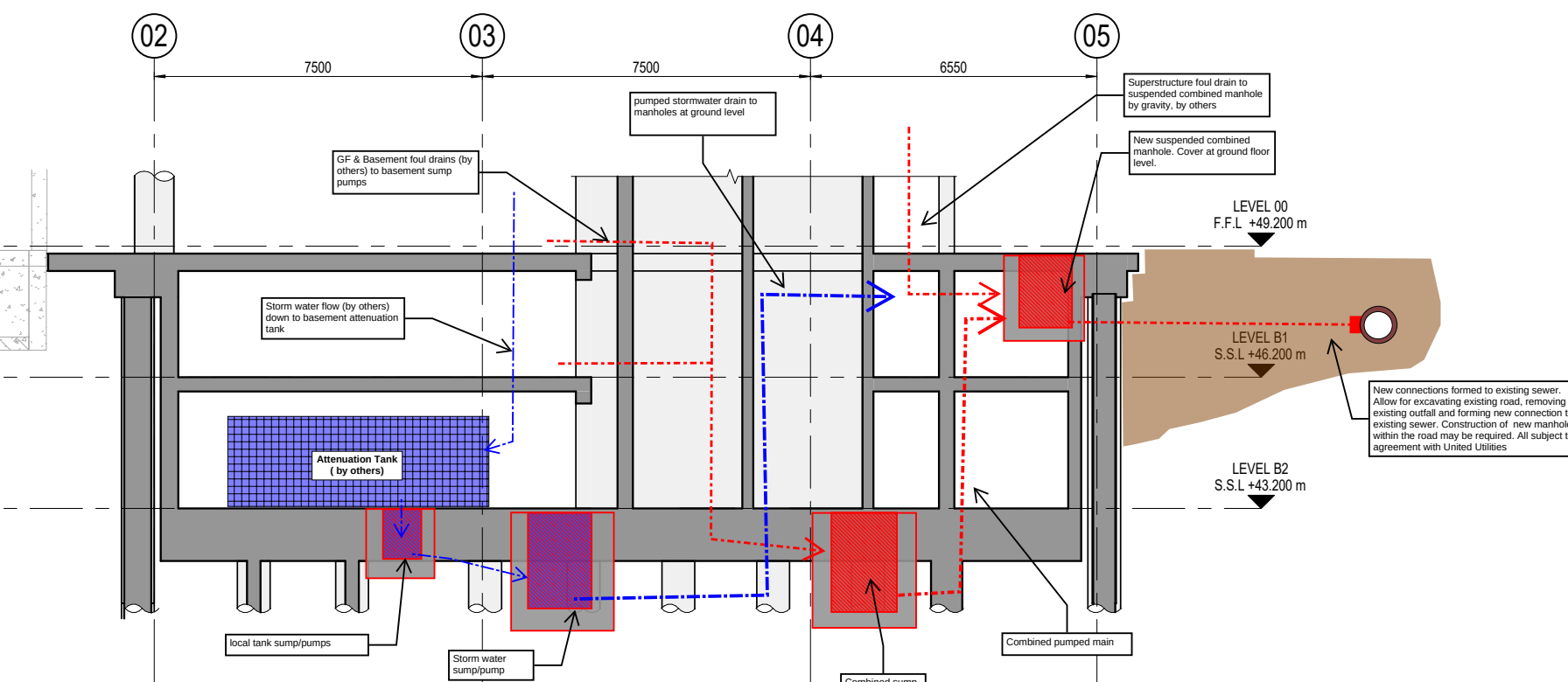
Basement B2 Plan

Key:

- 1 - Combined sump with two pumps, 1.5 - 2m diameter, 2-3m deep
- 2 - Stormwater sump with two pumps 1.5 - 2m diameter, 2-3m deep
- 3 - Local sumps with single pumps for water tank drainage, 1m diameter, 1-2m deep
- 4 - Local foul water sumps with single pumps, 1m diameter, 1-2m deep
- 5 - Cavity drain sumps with single pumps and silt traps - size tbc by Waterproofing Specialist - assume same as sump type 3
- 6 - Storm and Foul suspended manholes, draining via gravity to sewers

Notes:

- For all sumps allow for reinforced concrete surround
- Make allowance for additional manholes, gullies and drainage channels including buried drain runs to sumps
- Scheme indicated assumes surrounding existing sewers can support the additional flow rates outlined in AKTII's drainage strategy report. This is subject to agreement with United Utilities.
- All drainage above B2 Slab SSL within the site to be designed by others
- The below ground drainage scheme shown is indicated and subject to further coordination with the design team during the next design stage. Number of sumps can be reduced following further coordination during Stage 3



Section A-A



PROJECT LEVER STREET		TITLE PRELIMINARY BELOW GROUND DRAINAGE SCHEME	
DATE 15.03.24	SCALE 1:100 @ A3	CAD FILENAME	STATUS
DRAWN DC	CHECKED MD	PROJECT NO. 4891	DRAWING NO. HS - SK126
		REV	P1